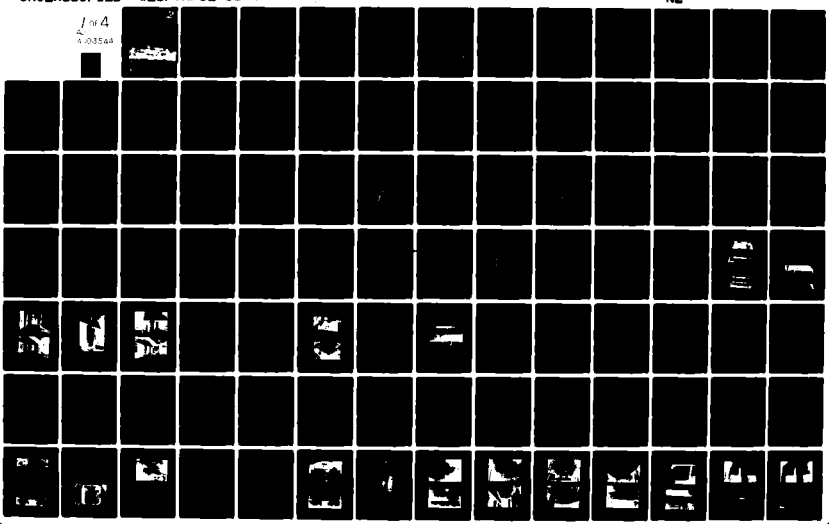
AD-A103 544 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR--ETC(U)
JUL 81 M K MCVAY DCPA01-78-C-0267
UNCLASSIFIED WES/TR/SL-81-4 NL



LEVEL II

12

AD A103544

TECHNICAL REPORT SL-81-4

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS

by

Mark K. McVay

Structures Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

DTIC
ELECTE
SEP 1 1981
S D
D

July 1981

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for Federal Emergency Management Agency
(Formerly the Defense Civil Preparedness Agency)
Washington, D. C. 20472

Under Project Order No. DCPA 01-78-C-0267, Work Unit 1127E

DTIC FILE COPY

81 9 01 024

Destroy this report when no longer needed. Do not return
it to the originator.

The findings in this report are not to be construed as an official
Department of the Army position unless so designated.
by other authorized documents.

The contents of this report are not to be used for
advertising, publication, or promotional purposes.
Citation of trade names does not constitute an
official endorsement or approval of the use of
such commercial products.

TECHNICAL REPORT SL-81-4

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS

by

Mark K. McVay

Structures Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

July 1981

Final Report

Approved For Public Release; Distribution Unlimited

FEMA Review Notice

This report has been reviewed in the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency.

Prepared for Federal Emergency Management Agency
(Formerly the Defense Civil Preparedness Agency)
Washington, D. C. 20472

Under Project Order No. DCPA 01-78-C-0267, Work Unit 1127E

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Technical Report SL-81-4	2. GOVT ACCESSION NO. AD-A103544	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS		5. TYPE OF REPORT & PERIOD COVERED Final report
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Mark K. McVay		8. CONTRACT OR GRANT NUMBER(s) Project Order No. DCPA 01-78-C-0267
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Structures Laboratory P. O. Box 631, Vicksburg, Miss. 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS Work Unit 1127E
11. CONTROLLING OFFICE NAME AND ADDRESS Federal Emergency Management Agency (Formerly the Defense Civil Preparedness Agency) Washington, D. C. 20472		12. REPORT DATE July 1981
		13. NUMBER OF PAGES 317
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Available from National Technical Information Service, Springfield, Va. 22151. This report has been reviewed by the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Buildings
Concrete floors
Dynamic loads
Hardened installations
Heavy duty floors
Reinforced concrete
Surface hardeners (Concrete)

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The problem of upgrading one-way reinforced concrete (R/C) slab floor systems for keyworker shelters was studied in this program. The objective was to develop competent designs for upgrading such systems that would use readily available materials, be easy to construct, and increase the load-carrying capacities of such systems to 50 psi or greater. Two upgrading methods were developed and evaluated: a wooden post method and a steel beam method.

(Continued)

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. ABSTRACT (Continued).

For the wooden post method, several 4- by 4-inch timbers were placed in groups under the midspan of the slabs. These groups acted as units to form columns large enough to accept large loads and provide sufficient bearing area. For the steel beam method, the floor slab panels were supported at midspan with a series of small steel beams held up by steel pipe columns. The components were kept small enough so two or three people could handle them. The existing beams of the floor systems were upgraded with additional posts in both methods.

The increased load capacities resulting from these upgrading methods were verified by conducting dynamic tests on three identical full-scale sections of a typical one-way R/C slab floor system. In order to have a realistic baseline for comparison purposes, a typical slab section without any upgrading was first tested. It was predicted that the nonupgraded slab section would fail when subjected to a peak overpressure of 16 psi. It was tested with average peak overpressures of about 15 and 33 psi. The first test caused cracks to form in the top of the slab along the beams. The second test greatly exceeded the calculated load-carrying capacity of the slab and caused complete collapse. Next, a test series was done on an identical slab section upgraded using the wooden post method. The upgrading system was designed to increase the load capacity of the slab section to about 55 psi, at which pressure it was predicted to fail in punching shear. However, the analysis for punching shear was for concrete columns and static loads. This upgraded slab section was tested five times and resisted average peak overpressures of 16, 24, 38, 73, and 113 psi. During the second, third, and fourth tests, some hairline cracks formed, but no serious damage occurred. During the fifth test, two timbers started punching through the slab and several timbers under the center beam started splitting. The third test series was on the typical slab section upgraded using the steel beam method. This upgrading system was designed to withstand 50 to 60 psi, at which pressure it was predicted to fail in shear. It was loaded with average peak overpressures of 35, 63, and 92 psi. The slab cracked where supported by the steel beams (no negative reinforcement in slab at this location) as a result of the loading for the first test, remained undamaged for the second test, and collapsed under the loading for the third test. Complete data records for the tests are presented in Appendices A, B, and C.

It was concluded that sound upgrading systems can be made of readily available materials that only require simple construction skills. The tests illustrate that the load capacity of a one-way R/C floor system can be increased five to seven times using a proper upgrading system. The design procedures proved to be conservative, resulting in structures much stronger than predicted. Both upgrading methods are excellent techniques for increasing the load capacities of a keyworker shelter above 50 psi.

PREFACE

This study was conducted during the period November 1978 through January 1980 by personnel of the U. S. Army Engineer Waterways Experiment Station (WES) under the sponsorship of the Defense Civil Preparedness Agency (now the Federal Emergency Management Agency) (Project Order No. DCPA 01-78-C-0267, Work Unit 1127E). The work was in the Blast Upgrading Shelter-Experimental Study Area.

This work was conducted under the general supervision of Mr. Bryant Mather, Chief of the Structures Laboratory (SL), and Mr. J. T. Ballard, Chief of the Structural Mechanics Division, SL. Mr. W. L. Huff was the Project Manager of the study. Mr. M. K. McVay planned and supervised the experiments, conducted the analyses, and prepared this report.

Directors of WES during the conduct of this study and the preparation of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director was Mr. Fred R. Brown.

Accession For
NTIS GRA&I ☒
DTIC TAB ☐
Unannounced ☐
Justification
By
Distribution/
Availability Codes
Dist | Avail and/or Special
A

DTIC
ELECTE
SEP 1 1981
S D
D

CONTENTS

	Page
PREFACE	1
CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT	7
CHAPTER 1 INTRODUCTION	8
1.1 BACKGROUND	8
1.2 OBJECTIVE	10
1.3 SCOPE	10
CHAPTER 2 PROPOSED PROCEDURES FOR DESIGNING AN UPGRADING SYSTEM FOR A GIVEN FLOOR	12
2.1 DISCUSSION OF ONE-WAY SLAB FLOOR SYSTEMS IN GENERAL	12
2.2 DETERMINING AND ESTIMATING THE CHARACTERISTICS OF A GIVEN FLOOR SYSTEM	13
2.3 ANALYSIS OF A FLOOR SYSTEM	16
2.4 UPGRADING SYSTEMS IN GENERAL	17
2.5 ESTIMATING THE CHARACTERISTICS OF THE MATERIAL USED FOR UPGRADING	17
2.6 DESIGN OF UPGRADING SYSTEMS	18
CHAPTER 3 TEST STRUCTURES	33
3.1 DESIGN OF THE ONE-WAY R/C FLOOR SYSTEM USED FOR TESTING	33
3.2 DESIGN OF WOODEN POST UPGRADING SYSTEM (TEST STRUCTURE 2)	34
3.3 DESIGN OF STEEL BEAM UPGRADING SYSTEM (TEST STRUCTURE 3)	37
3.4 DESIGN OF THE REACTION STRUCTURE	39
CHAPTER 4 TEST PROCEDURE	58
4.1 PLACEMENT OF TEST STRUCTURES	58
4.2 INSTRUMENTATION	59
4.2.1 Instrumentation of Test Structure 1	59
4.2.2 Instrumentation of Test Structure 2	59
4.2.3 Instrumentation of Test Structure 3	60
4.3 TESTING	61
4.4 POSTSHOT INSPECTION	62
CHAPTER 5 TEST RESULTS AND DISCUSSION	66
5.1 GENERAL	66
5.2 RESULTS AND DISCUSSION OF TESTS ON STRUCTURE 1	66
5.2.1 SLAB 1A	66
5.2.2 SLAB 1B	67
5.3 RESULTS AND DISCUSSION OF TESTS ON STRUCTURE 2	68
5.3.1 SLAB 2A	68
5.3.2 SLAB 2B	69
5.3.3 SLAB 2C	70
5.3.4 SLAB 2D	70
5.3.5 SLAB 2E	71

Page

5.4 RESULTS AND DISCUSSION OF TESTS ON STRUCTURE 3 73
5.4.1 SLAB 3A. 73
5.4.2 SLAB 3B. 74
5.4.3 SLAB 3C. 75

CHAPTER 6 DATA EVALUATION. 106

6.1 TEST SERIES 1. 106
6.2 TEST SERIES 2. 106
6.3 TEST SERIES 3. 108
6.4 COMPARISONS. 110

CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS. 111

7.1 CONCLUSIONS. 111
7.1.1 Increased Load Capacity Verified 111
7.1.2 Readily Available Materials and Simple Construction. . . 111
7.1.3 Design Procedure 111
7.1.4 Modes of Failure 112
7.1.5 Comparison of Upgrading Systems. 112
7.2 RECOMMENDATIONS. 112
7.2.1 Application. 112
7.2.2 Improvements 113
7.2.3 Needed Studies 114

REFERENCES. 115

APPENDIX A: STRUCTURE 1 TEST DATA. 119

APPENDIX B: STRUCTURE 2 TEST DATA. 153

APPENDIX C: STRUCTURE 3 TEST DATA. 257

LIST OF FIGURES

Figure		Page
2.1	Types of one-way R/C floor slabs	27
2.2	Moment capacities of rectangular sections	28
2.3	Comparison of bend and cutoff points in one-way R/C slabs	29
2.4	Comparison of bar details for beams	30
2.5	Approximation of the boundary conditions of a slab panel upgraded under the midspan	31
2.6	Estimating the load applied to a column by an assumed yield-line pattern	32
3.1	Typical beam-girder type floor construction	44
3.2	Dimensions of the "typical" floor section	45
3.3	A sketch of the reinforcing steel layout in the floor section	46
3.4	Composite column upgrading a slab panel	48
3.5	Composite column upgrading a beam	48
3.6	Composite column upgrading a girder	49
3.7	A plan of the wooden column upgrading system	49
3.8	A partial view under the floor section upgraded with wooden posts	50
3.9	Upgrading steel beams in place under midspan of a slab panel	50
3.10	The steel columns supporting the steel upgrading beams	51
3.11	Bolts under the steel columns supporting the upgrading beams	52
3.12	The wooden upgrading columns under the center beam	52
3.13	A plan of the steel beam upgrading system	53
3.14	Plan and profile of the reaction structure	54
3.15	Pictures of the reaction structure and its reinforcing steel	55
3.16	Reaction structure in place inside the LBLG	56
3.17	View of the crawl space under the nonupgraded slab	57
4.1	Instrumentation layout for Test Structure 1	63
4.2	Instrumentation layout for Test Structure 2	64
4.3	Instrumentation layout for Test Structure 3	65
5.1	Indicated pressure from a moderate temperature increase on a XTMS-1-190-25 airblast gage	77
5.2	Picture of the cracks in Test Structure 1 after shot SLAB 1A	78
5.3	A close-up of one of the cracks in the floor section	79
5.4	Top view of Test Structure 1 after shot SLAB 1B	79
5.5	Closeup view of Test Structure 1 after shot SLAB 1B	80
5.6	Weapon fit of the overpressure in shot SLAB 2A	80
5.7	Crack patterns in Test Structure 2 after test 2B	81
5.8	Crack patterns in Test Structure 2 after test 2C	82
5.9	Top view of Test Structure 2 after shot SLAB 2D	83

Figure		Page
5.10	A close-up of a crack in Test Structure 2 after shot 2D.	84
5.11	Cracks in underside of the east slab panel at the north end.	85
5.12	Cracks in the underside of the east slab panel at the center	85
5.13	Cracks in the underside of the east slab panel at the south end.	86
5.14	Cracks in the underside of the east slab panel and center beam.	86
5.15	Cracks in the underside of the west slab panel at the north end.	87
5.16	Cracks in the underside of the west slab panel at the center	87
5.17	Cracks in the underside of the west slab panel at the south end.	88
5.18	Cracks in the underside of the center beam at its center	88
5.19	Wooden column under the center beam after shot SLAB 2D	89
5.20	Close-up showing the amount of permanent deflection in a column under the center beam after shot SLAB 2D.	89
5.21	Picture of the wooden post over load cell L1 punching through the east slab panel.	90
5.22	The concrete lifting up over the wooden column on load cell L1.	90
5.23	A picture of the wooden post, on top of load cell L2, punching through the east slab panel	91
5.24	The concrete lifting up as the wooden post, on top of load cell L2, starts to punch through.	91
5.25	A picture of the damaged columns under the center beam after shot SLAB 2E	92
5.26	A picture of a column under the center beam that split under the load	92
5.27	A picture of another column under the center beam after shot SLAB 2E	93
5.28	A picture of a column that tried to rotate out from under the center beam during shot SLAB 2E.	93
5.29	The cracks in the top of Test Structure 1 after shot SLAB 2E.	94
5.30	Cracks in the underside of the east slab panel at the south end.	95
5.31	Cracks in the underside of the east slab panel at the center after shot SLAB 2E	95
5.32	Cracks in the underside of the west slab panel at the north end after shot SLAB 2E	96
5.33	Cracks in the underside of the west slab panel at the center section after shot SLAB 2E.	96
5.34	Cracks in the underside of the west slab panel at the south end after shot SLAB 2E	97

Figure		Page
5.35	A few wedges crushed under posts in columns upgrading a slab panel, after shot SLAB 2E	97
5.36	Undamaged columns upgrading an outside beam	98
5.37	The permanent deflection of Test Structure 3 between the column rows upgrading the slab panels, after shot SLAB 2E	98
5.38	Weapon fit of shot SLAB 2E	99
5.39	Cracks in the top of Test Structure 3 after shot SLAB 3A	99
5.40	Weapon fit of shot SLAB 3B	100
5.41	The crack pattern in the top of Test Structure 3 after shot SLAB 3B	100
5.42	The permanent deflection in the floor section between the two rows of steel upgrading beams	101
5.43	A column under the center beam after shot SLAB 3B	101
5.44	Crack pattern in the underside of the east slab panel between the center beam and an upgrading beam	102
5.45	Cracks in the underside of the east slab panel, between the outside beam and the upgrading beam	102
5.46	Test Structure 3 after shot SLAB 3C	103
5.47	A close-up of the west slab panel after shot SLAB 3C	104
5.48	Close-up of the edges of the slab panel	105
5.49	A picture of a buckled upgrading beam and broken and bent bolts	105

LIST OF TABLES

Table		Page
2.1	Minimum live loads to be used for floor design according to References 17-20	23
2.2	Material properties of the CRSI handbooks	24
2.3	Moment and shear coefficients in the 1963, 1971, and 1977 ACI codes	25
2.4	Lower bound properties of wood likely to be used for upgrading	26
3.1	Pertinent data on slabs and beams	42
3.2	Yield strengths of the reinforcing steel	43

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic inches	16.38706	cubic centimetres
feet	0.3048	metres
grains per foot	0.2125948	grams per metre
inches	2.54	centimetres
kips per square inch	6894.757	kilopascals
pounds (force)	4.4482	newtons
pounds (force) per square foot	0.04788	kilopascals
pounds (force) per square inch	6.8947	kilopascals
pounds (mass)	0.453592	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square inches	6.4516	square centimetres

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The primary objective of this country's civil defense against a nuclear attack is to save as many lives as possible. The former Defense Civil Preparedness Agency (DCPA), which is now part of the present Federal Emergency Management Agency (FEMA), has the responsibility of maintaining a good civil defense program. Those areas that are most likely to be a target of a nuclear weapon are called high-risk areas. Cities are high-risk areas due to the presence of large populations, hubs of transportation and communication networks, industries, and functions critical to the survival of the Nation (Reference 1). A megaton-size nuclear weapon would most likely be used on a city and detonated at or near the surface. This would produce high blast overpressures, strong winds, and intense radiation over a radius of several miles. Many buildings in such cities have been designated as "fallout shelters." Government survey teams have been locating and designating suitable "fallout shelters," which are listed in the National Fallout Shelter Survey (NFSS), for many years. SRI International (formerly Stanford Research Institute) analyzed some of the NFSS buildings and determined their component collapse overpressures (References 2-4), which were usually less than the overpressures (20 to 50 psi[a]) many of them would receive from a nuclear blast if they were within moderate range of the explosion. There are very few conventionally constructed commercial buildings that are capable of withstanding such loadings. The old civil defense program called for people to take cover in these "fallout shelters" during an attack, but due to the increased size of present weapons and the low resistance to blast of most buildings, "fallout shelters" are no

---

[a] A table of factors for converting inch-pound units of measurement to metric (SI) units of measurement is given on page 7.

longer adequate protection in high-risk areas. The current civil defense program is called Crisis Relocation Planning (CRP). CRP would be used during a time of developing international crisis when a nuclear war is imminent. Basically, CRP would call for the following actions:

a. Moving most of the people out of the high-risk areas into surrounding host areas within 2 to 3 days.

b. The people would move initially to public and private buildings in the host areas and then, if necessary, to shelters (mostly expedient-type shelters).

c. Keyworkers and officials would stay close to the high-risk area in hardened shelters so they could operate key industries and government positions.

This report is related to part c of the CRP. It is estimated that approximately 6 percent of the population in a high-risk area will consist of the keyworkers and officials. Competent, hardened shelters would be needed to protect these key people, as well as others that stay in high-risk areas, from the effects of nuclear weapons. As stated before, the existing "fallout shelters" are no longer adequate protection in their present state. New hardened structures could be built that would be massive and strong enough to resist these large loads, but it would cost billions of dollars to build all of the shelters needed. An alternative would be to upgrade the load capacity of some of the existing "fallout shelters" with additional structural members so they could withstand the large loads. The results of recent analytical and experimental studies show that when proper upgrading methods are used, this is possible. A "fallout shelter" could be rapidly upgraded during a time of crisis with readily available construction materials at a minimum cost. A detailed survey of a statistical sample of 219 NFSS buildings indicated that about 87 percent of the fallout shelters have reinforced concrete (R/C) slab floor systems (Reference 5). The results of previous DCPA-sponsored studies (References 6-8) show that the ultimate load-carrying capacity of common slab systems is much higher than that predicted by conventional analysis procedures. A large number of the R/C floor systems in the survey were one-way. A test program conducted on small, one-way R/C floor slabs at the U. S. Army Engineer

Waterways Experiment Station (WES) indicated that the load capacities of one-way slabs could be increased by factors of 4 to 7 by providing expedient supports of various types at the midspan of the slabs (Reference 9). FEMA has an ongoing contract with Scientific Services Incorporated to prepare a manual for use by the general public to quickly upgrade the blast resistance of buildings. These upgrading techniques need to be easy to understand, easy to perform, well engineered, and proven to work in order for the public to accept them. This study examines the upgrading of entire one-way R/C slab floor systems, examines two good methods to upgrade a sample floor system, and verifies the resulting increased load capacities in full-scale tests. This report will provide a good reference on one-way slabs for the manual.

1.2 OBJECTIVE

The objective of this study was to develop competent upgrading designs that would use readily available materials, be easy to construct, and provide optimum hardening for one-way R/C slab floor systems.

1.3 SCOPE

The general characteristics of one-way R/C slab floor systems were examined thoroughly. The main variables that were considered in the design of an upgrading system were: distribution of moments, punching shear, shear at beam supports, and strengths of the original supports themselves. Various upgrading methods were considered, and the best two were chosen. One method was to support the slab panels under their midspans with groups of wooden posts closely spaced so they formed large square columns and to support the beams with wooden posts under critical points. The other method was to support the slab panels under their midspans with a series of short steel wide-flange beams held up by steel pipes and to support the original beams with wooden posts under critical points. The increased load capacities resulting from these upgrading techniques were verified with dynamic tests on full-scale sections from a typical one-way R/C slab floor system. Three tests were performed: the first was on the typical slab section without any

upgrading, the second was on an identical slab section upgraded by the wooden posts, and the third test was on an identical slab section upgraded by the steel beams and pipe. The tests were conducted in the Large Blast Load Generator (LBLG) at WES, which can produce dynamic loads similar to those from a nuclear weapon in the megaton range (Reference 10). Active measurements of overpressure, deflections, reinforcing steel strains, and support loads were taken during the tests.

CHAPTER 2

PROPOSED PROCEDURES FOR DESIGNING AN UPGRADING SYSTEM FOR A GIVEN FLOOR

2.1 DISCUSSION OF ONE-WAY SLAB FLOOR SYSTEMS IN GENERAL

A one-way slab carries the majority of an applied load in one direction, the direction perpendicular to the support beams. In theory, a one-way slab is similar to a series of beams side-by-side, except there is a slight gain in strength and stiffness due to the resistance against lateral expansion described by Poisson's ratio. There are, however, some differences between the design criteria for one-way slabs and those for beams. For instance, the minimum reinforcement cover, maximum spacing of bars, and requirement of shrinkage and temperature reinforcement are different requirements than those for beams. The criteria which influence the design of one-way slabs most often are the flexural capacities and maximum deflection requirements. Shear, diagonal tension, and bond stresses rarely are critical in normal designs. Due to simplifying assumptions, the small deflection requirements, materials being stronger than design strengths, and conservative regulations on the spacing and amount of flexural steel, one-way slabs are stronger than predicted by conventional analysis procedures.

There are different types of one-way slab floor systems, as shown in Figure 2.1. The slabs may be supported on only two sides with the beams running over column rows, as in Figure 2.1a. On the other hand, the slabs may be supported by a beam-girder system in which an intermediate row of beams runs between column rows, as in Figure 2.1b. The slab is supported on four sides in this setup, but if the length to width ratio of the slab panel is greater than 2, most of the load is carried in the short direction and one-way action is obtained in effect (References 11-13). Still another kind is the one-way ribbed joist system in which a series of small "T" beams lie side by side and span in between girders, as shown in Figure 2.1c. In essence, the "T" beams

act like a rectangular one-way slab with notches cut out of it to lighten the dead load of the structure.

All of these type floor systems are designed using basically the same principles. These principles, along with safety regulations, are incorporated in building codes and design aids. Concrete structures are designed and constructed in accordance with a current "Building Code Requirements for Reinforced Concrete" prepared by the American Concrete Institute (ACI) (References 11-13). Most engineers use design aids to help them quickly design structural components that meet the requirements of the ACI code. ACI issues several different design aids itself. Another design aid that is quite popular is the one issued by the Concrete Reinforcing Steel Institute (CRSI) (References 14-16). The minimum live loads that can be used in the design of most floors are regulated by building codes. The major building codes are "The BOCA Basic Building Code" prepared by the Building Officials Conference of America, Inc.; the "City of New York Building Code"; the National Building Code prepared by the American Insurance Association; the "Southern Standard Building Code" prepared by the Southern Building Code Congress; and the Uniform Building Code prepared by the International Conference of Building Officials. These codes and design aids are rewritten periodically to incorporate new technology and research results.

2.2 DETERMINING AND ESTIMATING THE CHARACTERISTICS OF A GIVEN FLOOR SYSTEM

In order for a group of people to determine the proper upgrading system to apply to a given floor system, they need to know some characteristics of the floor system. These characteristics include the dimensions, strength of the materials, reinforcing steel ratios, bar detailing, etc. The group might be fortunate enough to get a set of construction plans to determine these characteristics, or they may have to measure and estimate them. This section proposes a procedure to use if the characteristics have to be measured and estimated. Such things as the dimensions, type of building, and age can be used to make a conservative estimate of the desired characteristics.

One of the first and easiest things that can be done is measuring the dimensions of the slab panels, beams, and columns. If the ratio of the length to width dimensions of the slab is greater than 2, then the slab was most likely designed as a one-way slab (References 11-13). Once the dimensions of the structural members are known, the dead loads used in the design can be calculated by multiplying the volumes by 150 lb/ft^3 (Appendix J in References 17-20).

The minimum live loads to be used for the design of a floor in a particular type of building are regulated by Section 2304 in the Uniform Building Code (References 17-20). As shown in Table 2.1, hardly any of these minimum live loads have changed over the years. Thus, if the type of building being upgraded is known, the minimum live load the floors would have been designed for can be determined. (Note the minimum design live load is usually used, but it is not unusual for a designer to specify only a few slab designs for an entire floor area of a building, especially for a large multistory building where the first story level may contain concourses, hallways, office, stores, conference rooms, and elevator lobbies. Many different span lengths and several design live load requirements may be involved in such a building, but because of construction practices, it may be more economical to minimize the number of slab designs. Therefore, some slabs would be designed for the maximum span and load requirements in the floor and thus be overdesigned.)

Much information can be estimated based upon the approximate age of the building. The most common strengths of steel and concrete used during the time the building was constructed could be assumed without much error as the strengths actually used. The most common yield strengths of steel were 33 ksi in the early 1900's and 40 ksi in the 1950's and mid-60's. They have been around 60 ksi since then. The most common concrete compressive strengths have ranged from 3000 to 6000 psi for many years. The strengths cited in CRSI handbooks printed since 1963 are shown in Table 2.2 (References 14-16). The building codes and design aids that were most likely used when the building was designed could be used to conservatively estimate hidden characteristics of the

structural members. Before 1963 the Working Stress Design Method was recommended by ACI codes. In the 1963 ACI code the Ultimate Strength Design Method was introduced as an alternate design method (Reference 11), and CRSI subsequently adopted it (Reference 14). In the 1971 ACI code the Ultimate Strength Design Method was adopted as the primary method of design and has been ever since (References 12 and 13). This has been the major change in the codes and design aids over the years; many small changes have also been made. Some of the hidden characteristics of the floor system that could be determined from codes and design aids are: minimum values of the design moments and shears used, the minimum steel reinforcing present, and the approximate bend or cutoff points of the reinforcing.

The minimum design moments and shears can be approximated by using the correct safety factors and moment and shear coefficients given in the ACI code for use in lieu of more accurate analyses. The safety factors have changed over the years. The coefficients in the 1963, 1971, and 1977 ACI codes (References 11-13) are all the same and are given in Table 2.3.

Once the minimum values of the design moments are approximated, the minimum amount of flexural reinforcing steel that should be present can be estimated. This can easily be done from a graph. The correct graph, depending upon the cross section of the structural member (rectangular or "T" shaped) and upon the design method used at the time of design (Working Stress or Ultimate Strength), would have to be chosen. A graph for a rectangular section designed according to the Ultimate Strength Design Method is shown in Figure 2.2 as an example.

The recommended flexural bar details of one-way slabs from several design aids published since 1963 (References 14-16, 21, and 22) are shown in Figure 2.3. They are all quite similar. Selected conservative bend or cutoff points, which meet or exceed all of those recommended, are shown in Figure 2.3e. Notice no reinforcing against shear failure is generally needed in one-way slabs or joists. The recommended flexural bar details of beams from the same references are shown in Figure 2.4. Conservative bend or cutoff points for the beams are shown in

Figure 2.4e. The size and spacing of the stirrups required to resist the minimum amount of shear a beam could have been designed for can be selected from a table similar to those in the CRSI Handbooks.

It would be very feasible to make a series of charts and tables for use in determining the probable characteristics of a floor system being upgraded.

2.3 ANALYSIS OF A FLOOR SYSTEM

Once the probable characteristics of the floor system are determined, it can be analyzed to determine its load capacity, where it needs to be upgraded, and how to upgrade it. Every component of a floor system needs to be examined; these are basically the slab panels, beams, and columns. The analysis should include the ultimate dynamic moment and shear capacities of the slab panels and beams, dynamic punching shear strength of the slab, load capacity of the columns, and contribution of membrane action.

If the upgrading system is being prefabricated for storage in the building until an emergency, some extra time can be spent to do analyses by hand, or on a computer. A good analysis can be done by hand using Bigg's Approximate Design Method, given in Reference 23; however, the analysis is usually conservative. Another good hand analysis for beams and slabs is to determine a resistance function from a static analysis and then use it in a single degree of freedom (SDOF) analysis. Keenan describes a good static flexural analysis in Reference 24. There are many different static and SDOF computer program analyses presently available. Methods for the dyanamic analysis of R/C columns are described in Reference 25. A collection of very good computer programs that seem to be quite accurate was written by SRI International for DCPA to analyze NFSS structures (Reference 26).

If the upgraded system needs to be quickly designed and assembled during a time of crisis, a much quicker and rougher analysis needs to be done. A good, quick approximate analysis procedure that can be used in the field has been developed by SRI International (Reference 27). The analysis is done with charts based on simplified equations. A series

of charts could also be made based upon one of the other analyses techniques mentioned above. An analysis of almost any floor system will show that it needs to be upgraded.

2.4 UPGRADING SYSTEMS IN GENERAL

It is desired to upgrade the floor systems of keyworker shelters to withstand a dynamic pressure of at least 50 psi. About the only easy way to upgrade a floor system is to provide additional supports under it. The main structural members used for supports are bearing walls, beams, and columns. The use of bearing walls for upgrading members is not very ideal, though, due to the excessive amount of work to assemble them, the amount of room they take up, and the obstruction of passages. The effectiveness of bearing walls as upgrading members was not studied in this program, although they would be similar to very stiff beam supports. On the other hand, beams and columns are excellent for upgrading floor systems. A variety of materials can be used for beams and columns such as telephone poles, railroad ties, wood from lumber yards, concrete parking guards, steel "I" beams and columns, steel structural pipes, concrete culvert pipes, and etc. Therefore, materials that can be used for upgrading a floor system are readily available. However, individual components of the upgrading system should be kept to lengths and weights that can be handled by two or three men at the most. In other words, if a large column is required for upgrading a structural member, it can be made up of several small columns strapped together and braced; or if a long slab panel needs to be supported along its midspan, a series of short beams lying end-to-end would work. A minimum amount of floor space will be taken up with these types of upgrading supports.

2.5 ESTIMATING THE CHARACTERISTICS OF THE MATERIAL USED FOR UPGRADING

If the material properties of the beams and columns used for upgrading cannot be determined, they will have to be estimated too. Things made of steel and wood will probably be used most often. Most of

the structural steel used in engineered structures in the United States is a carbon steel designated as A36 by the American Society for Testing and Materials (ASTM) (Reference 28). Over the last decade or so, the use of stronger steels has increased. Since A36 steel has the lowest yield strength (F_y = 36 ksi) of all commonly available structural steel, it would be safe to assume a steel member to be at least as strong as A36. The elastic modulus of steel (E) is usually assumed as 30×10^6 psi. The properties of wood vary greatly, depending upon the kind of wood, how it was dried, and its quality. The Timber Construction Manual (Reference 29) lists the recommended material properties of different woods to be used in design. The lowest values for the material properties of any wood likely to be used for upgrading were chosen from this manual and are shown in Table 2.4.

2.6 DESIGN OF UPGRADING SYSTEMS

The first components of a floor system that should be examined are the slab panels. Most one-way slab floor systems are continuous, have negative steel at the ends of each span, and are cast monolithically with the beams; so they are close to being fixed-fixed. If an overpressure of at least 50 psi causes excessive moments and/or shears in the original slab panels, they can be reduced by decreasing the spans between supports. This can be done either with rows of upgrading beams, in turn supported by columns, or by just upgrading the columns. If an upgrading support is placed under the midspan of a one-way slab (where there is no negative moment resistance), the slab is changed from approximately a fixed-fixed condition to somewhere between two propped cantilevers end-to-end and two fixed-fixed slabs (Figure 2.5). Roughly, this should decrease the maximum moments to somewhere between 2.5 and 37.5 percent of their nonupgraded values. Of course, the actual lengths of the spans after upgrading will be less than half the original lengths due to the width of the upgrading supports. An upgrading support at the midspan should also decrease the maximum applied shear to between 50.0 and 62.5 percent of its nonupgraded value. In most cases, one row of upgrading supports per slab panel will increase the load capacity enough,

but if it does not, two rows of upgrading supports can be used. If two rows are required, it is recommended that they be placed one-fifth of the span length from the original beams to compliment the reinforcing steel placement. (One-third of the positive steel is lost at a distance of one-fourth to one-eighth of the span length from the original beams in most slabs, so the spans between the upgrading supports and original supports need to be shorter than the center spans.)

If beams supported in turn by columns are used for upgrading the slab panels, the design is fairly simple. First, the width of the upgrading beam is assumed. Then each slab section between two beams is analyzed by one of the mentioned methods to determine the load capacity more accurately. Next the loads on the upgrading beams are estimated. Then the upgrading beams are designed and checked to see if their widths are the same as assumed. The upgrading beams can be assumed to be simply supported by the columns for design purposes. Many different kinds of materials can be used as upgrading beams, but it is recommended that steel wide-flange beams be used because they are the strongest for the weight, and the beams will have to be lifted to the ceiling. The SRI simplified analysis procedures (Reference 27) can be used to quickly design the upgrading beams in the field; one of the other analysis procedures (such as the SRI computer code in Reference 26 for steel beams) can be used if the design is performed in the office. For design purposes, the columns used to support the upgrading beams can be assumed to have hinged ends and be loaded with a slight eccentricity. SRI has not yet developed a simplified analysis procedure for quick field design of columns. However, some charts for field design could easily be made. An analysis procedure considering the SDOF analysis and critical buckling loads can be used for design in the office. The stiffness of the column will partially depend upon the moment applied by the load at the eccentricity. It is recommended that a minimum eccentricity of 0.10 times the column width be assumed, as in the ACI code for column design. Many different materials can be used as columns, too. For wood, the Timber Construction Manual specifies multiplying a duration of load factor by the tabulated allowable stress for the wood used. For impact

loads this factor is 2.0. This is essentially the same as assuming a dynamic load factor of 1/2 in lieu of a SDOF analysis for wood. Reference 30 concludes that the dynamic buckling response of a steel column is nearly identical with its static buckling response. Thus, at least for steel columns, the static design considerations for buckling can be applied for dynamic design also. The same will be assumed for wooden columns.

If columns alone are used for upgrading instead of beams, the design is much more complicated. The possibility must be considered that the distribution of the moments and shears in the slab may not follow one-way action after upgrading. Also, the punching strength of the slab at the upgrading columns will have to be considered. It is recommended that the upgrading columns be placed in rows parallel to the original beams in the long direction to take advantage of the principal one-way reinforcing steel (although there is some temperature steel running in the perpendicular direction). This way the slab will primarily be in one-way action between the upgrading column rows and the original beams, even though there will be some local two-way action between the upgrading columns themselves. The dynamic punching shear capacity of the slab should be considered first in the design because the dimensions of the columns may be controlled by it. The ACI codes and several papers relate the punching shear capacity to the perimeter of a critical section at some distance (usually equal to one-half the depth of the slab) from the column in conjunction with some calculated shear strength of the slab under these conditions. How to calculate the shear strength of the slab under these conditions is very controversial. ACI code approximates the static punching shear strength with $4\sqrt{f'_c}$ (where f'_c is the compressive strength of the concrete). Most articles on the subject agree that the punching shear strength is related to the restraining forces at the slab boundaries, the moment present, and dowel action (References 31-33). Criswell studied the punching shear strength of slab-column connections subjected to both static and dynamic loadings (Reference 34). He determined that the failure mechanism and crack patterns of slabs tested with loads of duration similar to those from

nuclear blasts were similar to those resulting from static loadings. In his tests, the punching shear capacity of specimens dynamically loaded was on an average 26 percent greater than that of similar specimens statically loaded. Criswell concluded that for dynamic loads of those durations, the increase in strength can be attributed primarily to the increase of the material strengths accompanying the large strain rates. Thus, one of the methods can be used to predict the static shearing strength and then, after multipling by 1.26, this value can be used in dynamic designs. The ACI value is an easy, conservative value to use. The amount of shear applied to the column is determined by multiplying the overpressure by the contributing area to the column. The contributing area can be determined by a yield-line analysis or a close estimation. An example of a slab panel being upgraded by two square columns under its centerline is shown in Figure 2.6. The next two things to consider are the moment and shear capacities of the slab and what the spacing of the column rows needs to be in order to withstand overpressures. (Remember, the widths of the columns take up some of the clear span length.) This is done with a few dynamic analyses, as described before for the slab panels upgraded by beams. The upgrading columns can be designed using the same analysis procedures as those used to design columns to support the upgrading beams, with the extra stipulation that they support the required perimeter of slab to prevent punching shear. The required perimeter of support is quite large for most slabs. A capital could be used to provide the needed perimeter of support and allow the use of a column with smaller dimensions. Some possible items for use as a capital are manhole covers, steel plates, and water meter covers. Another way to provide large perimeters of support is to use several smaller columns spaced slightly apart to form a composite column. The ACI codes state that pilings spaced closely together can act as a unit, and the same principle applies here. It is recommended that posts be spaced less than two times the thickness of the slab apart to form a composite column.

The next components that need to be analyzed for upgrading are the original beams. The analyses done by SRI International of several

NFSS buildings indicated that a high percentage of the floor systems composed of R/C slabs supported by R/C beams will collapse due to the beams failing first (References 3 and 35). The area contributing load to each beam has to be figured after the slab panels have been upgraded. This can be determined by a yield-line analysis or by a close estimate, as is the case when designing the upgrading for the slab panels. The resulting maximum applied shear and moment are checked to see if they exceed the maximum capacity of the beam. If so, applied moments and shears can be reduced by reducing the free span lengths of the beam with additional upgrading columns propped underneath it.

The last components of the floor system to check for the need of upgrading are the original columns. They usually will not need upgrading, though, because the load has been distributed among the upgrading columns also. The only loads transmitted to the columns are from the original beams and perhaps a slight load from the corners of the slab panels. The original beams will most likely have upgrading columns under them, so the spans of the original beams connecting to the original column will be fairly short. If the original column does need upgrading, it can be done by surrounding it with more columns. The design procedures for these upgrading columns would be the same as described before.

Table 2.1 Minimum live loads to be used for floor design according to References 17-20.

	Load (lb/ft^2)			
	1961	1964	1970	1973
Apartments	40	40	40	40
Armories	150	150	150	150
Auditoriums - fixed seats	50	50	50	50
movable seats	100	100	100	100
Balconies and Galleries - fixed seats	50	50	50	60
movable seats	100	100	100	100
Cornices	60	60	60	60
Corridors, public	100	100	100	100
Dance halls	100	100	100	100
Drill rooms	100	100	100	100
Dwellings	40	40	40	40
Exterior balconies	100	100	100	100
Fire escapes	100	100	100	100
Garages - storage or repair	100	100	100	100
Garages - storage, private pleasure cars	50	50	50	50
Gymnasiums	100	100	100	100
Hospitals - wards and rooms	40	40	40	40
Hotels - guest rooms and private corridors	40	40	40	40
Libraries - reading rooms	60	60	60	60
stack rooms	125	125	125	125
Loft buildings	100	100	100	100
Manufacturing - light	75	75	75	75
heavy	125	125	125	125
Marquees	60	60	60	60
Offices	50	50	50	50
Printing plants - press rooms	150	150	150	150
composing and linotype rooms	100	100	100	100
Public rooms	100	100	100	100
Rest rooms	50	50	50	50
Reviewing stands and bleachers	100	100	100	100
Schools - classrooms	40	40	40	40
Sidewalks	250	250	250	250
Skating rinks	100	100	100	100
Stairways	100	100	100	100
Storage - light	125	125	125	125
heavy (load to be determined from proposed use or occupancy, but never less than)	250	250	250	250
Stores - retail (light merchandise)	75	75	75	75
wholesale (light merchandise)	100	100	100	100

Table 2.2 Material properties of the CRSI handbooks.[a,b]

	Columns		Beams		Slabs		Joists	
	f'_c (psi)	f_y (psi)	f'_c (psi)	f_y (psi)	f'_c (psi)	f_y (psi)	f'_c (psi)	f_y (psi)
1963	3750	60,000	3750	60,000	3750	60,000	3750	60,000
	5000	60,000						
1971	4000	60,000	3750	60,000	3000	60,000	4000	60,000
	5000	60,000						
	6000	60,000						
1977	4000	60,000	4000	60,000	3000	60,000	4000	60,000
	5000	60,000						
	6000	60,000						

[a] References 14-16.
[b] Where f'_c is the compressive crushing strength of the concrete and f_y is the yield stress of the reinforcing steel.

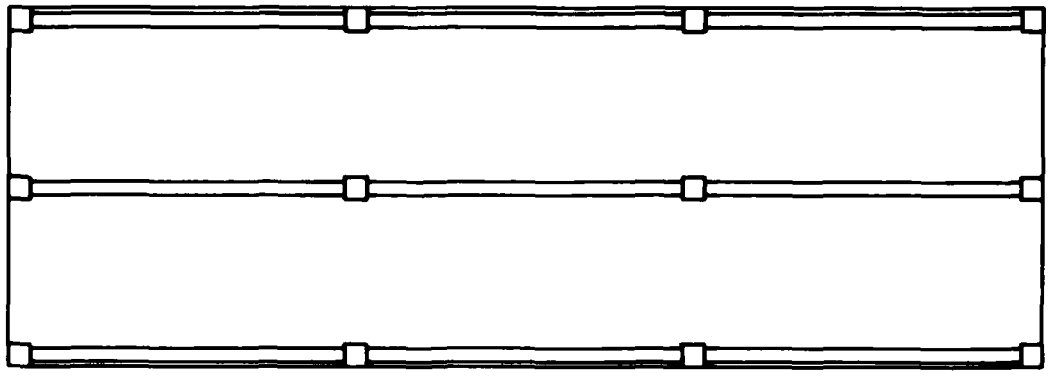
Table 2.3 Moment and shear coefficients in the 1963, 1971, and 1977 ACI codes.[a]

Positive moment-	
End spans:	
If discontinuous end is unrestrained	$\frac{1}{11} w\ell_n^2$
If discontinuous end is integral with the support	$\frac{1}{14} w\ell_n^2$
Interior spans	$\frac{1}{16} w\ell^2$
Negative moment at exterior face of first interior support-	
Two spans	$\frac{1}{9} w\ell_n^2$
More than two spans	$\frac{1}{10} w\ell_n^2$
Negative moment at other faces of interior supports	$\frac{1}{11} w\ell_n^2$
Negative moment at face of all supports for (a) slabs with spans not exceeding 10 ft, and (b) beams and girders where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of the span	$\frac{1}{12} w\ell_n^2$
Negative moment at interior faces of exterior supports for members built integrally with their supports-	
Where the support is a spandrel beam or girder	$\frac{1}{24} w\ell_n^2$
Where the support is a column	$\frac{1}{16} w\ell_n^2$
Shear in end members at first interior support	$1.15 \frac{w\ell_n}{2}$
Shear at all other supports	$\frac{w\ell_n}{2}$

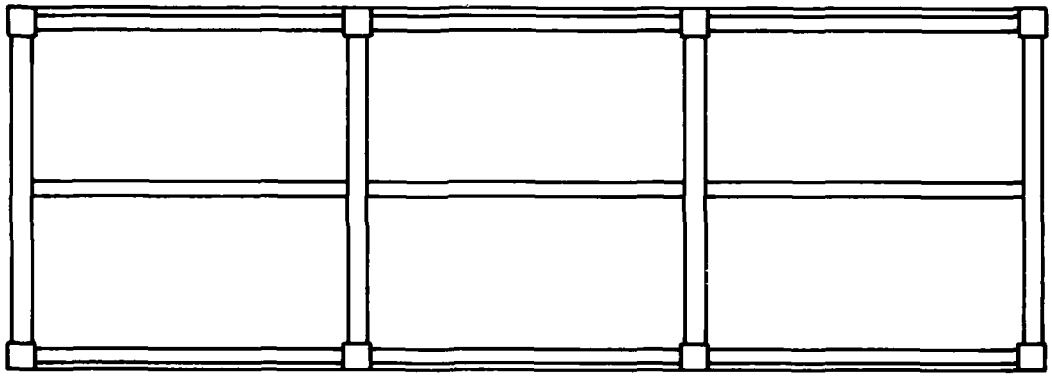
[a] References 11-13.

Table 2.4 Lower bound properties of wood likely to be used for upgrading.

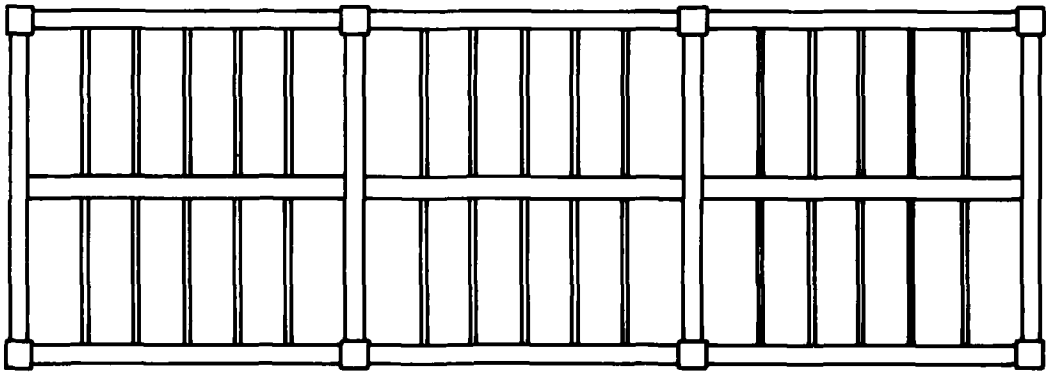
Maximum extreme fiber stress in bending	550 psi
Maximum tensile stress parallel to the grain	375 psi
Maximum horizontal shearing stress	55 psi
Maximum compression perpendicular to the grain	170 psi
Maximum compression parallel to the grain	625 psi
Modulus of elasticity	1×10^6 psi



(a.) SLABS SUPPORTED ON ONLY TWO SIDES - A "BASIC" ONE-WAY FLOOR SYSTEM



(b.) A BEAM-GIRDER TYPE ONE-WAY R/C FLOOR SYSTEM



(c.) A ONE-WAY R/C JOISTS SYSTEM OR "RIBBED" FLOOR SYSTEM

Figure 2.1 Types of one-way R/C floor slabs.

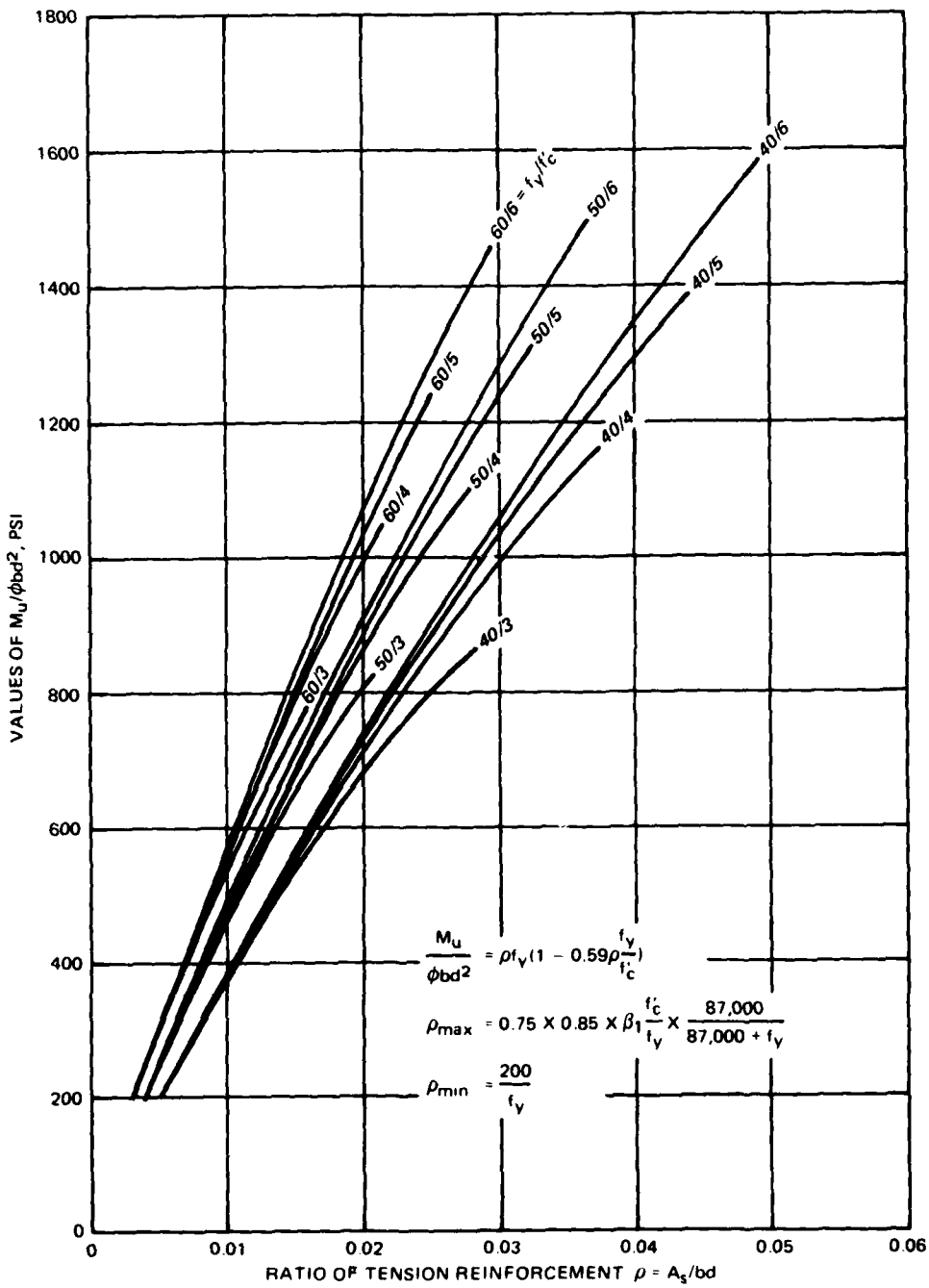
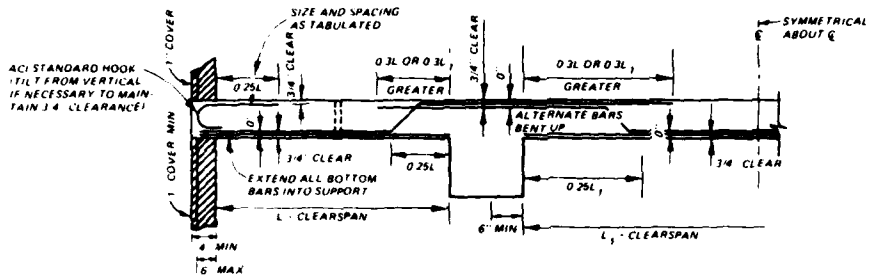
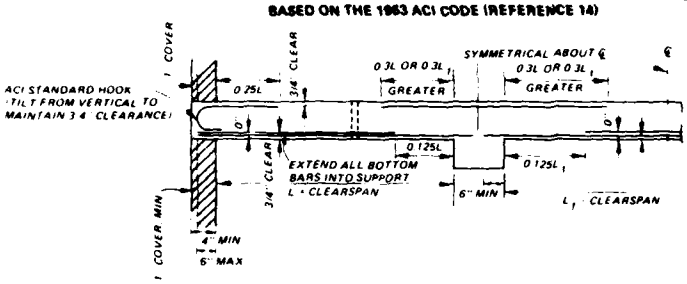


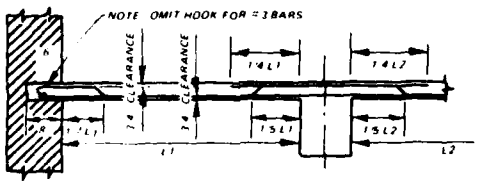
Figure 2.2 Moment capacities of rectangular sections.



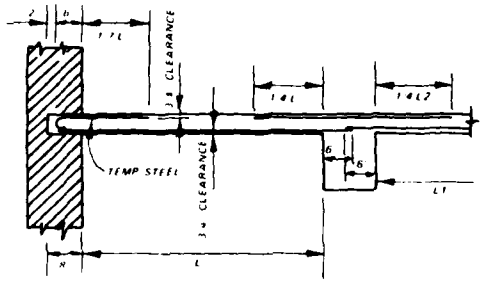
(a) RECOMMENDED BAR DETAILS FOR ONE-WAY SLABS IN CRSI HANDBOOKS BASED ON THE 1963 ACI CODE (REFERENCE 14)



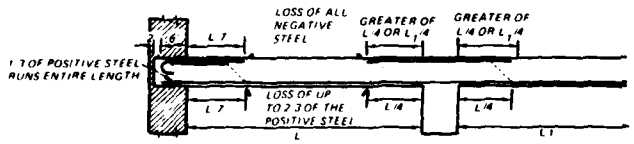
(b) RECOMMENDED BAR DETAILS FOR ONE-WAY SLABS IN CRSI HANDBOOKS BASED ON THE 1971 AND 1977 ACI CODES (REFERENCES 15 & 16)



(c) TYPICAL BAR DETAILS IN REFERENCE 21



(d) TYPICAL BAR DETAILS IN REFERENCE 21



(e) CONSERVATIVE CUTOFF POINTS ONE COULD ASSUME IN MOST SLABS

Figure 2.3 Comparison of bend and cutoff points in one-way R/C slabs.

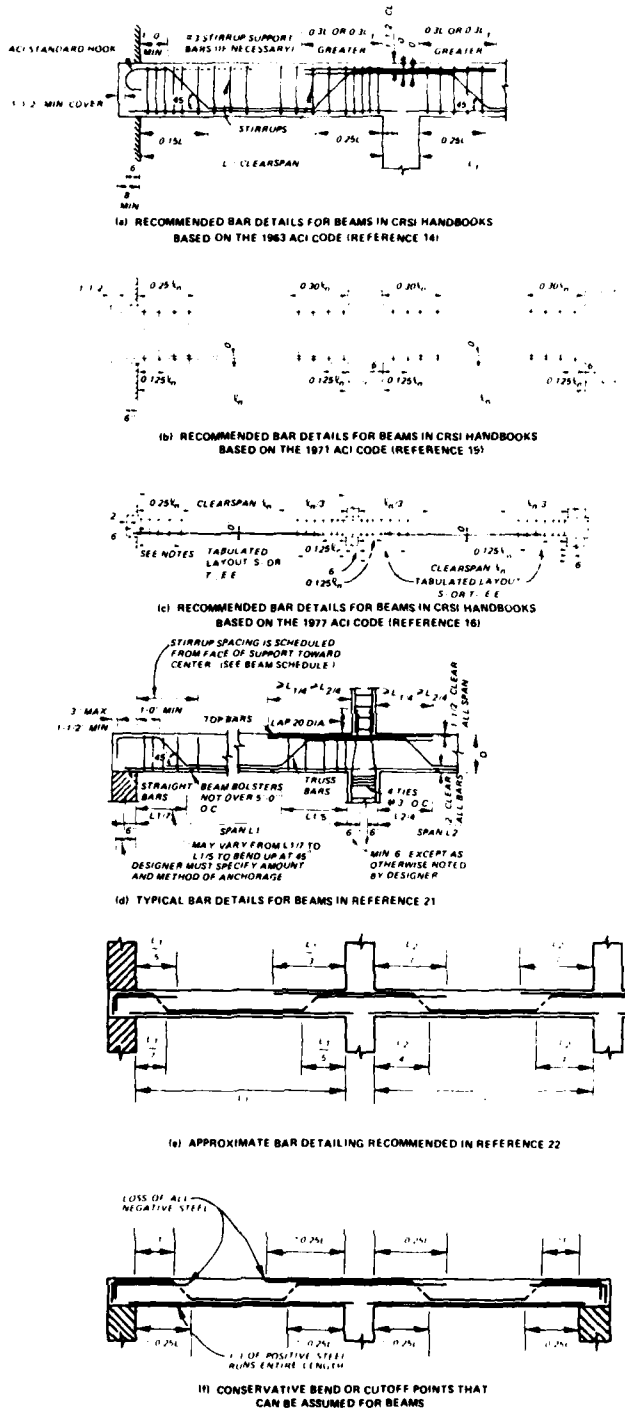


Figure 24. Comparison of bar details for beams.

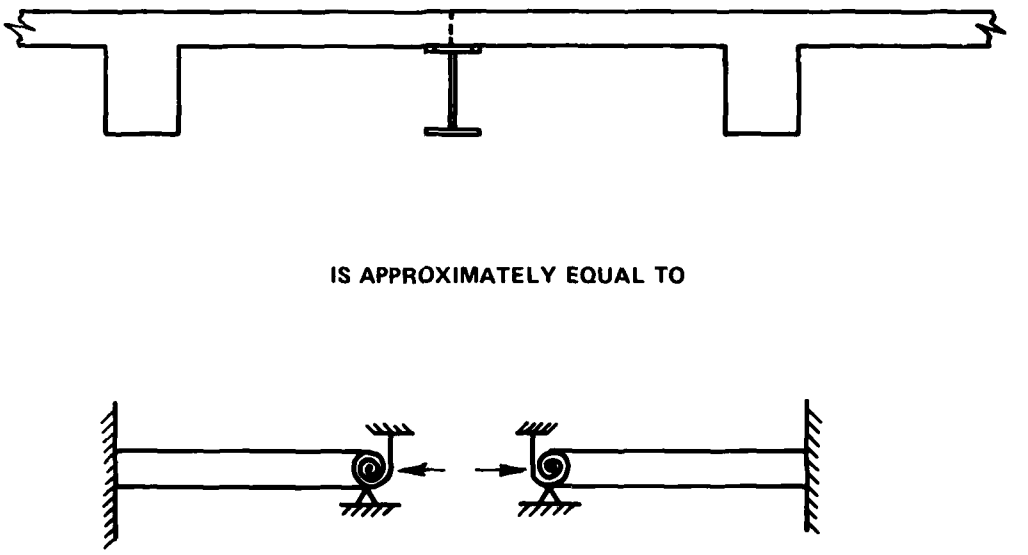
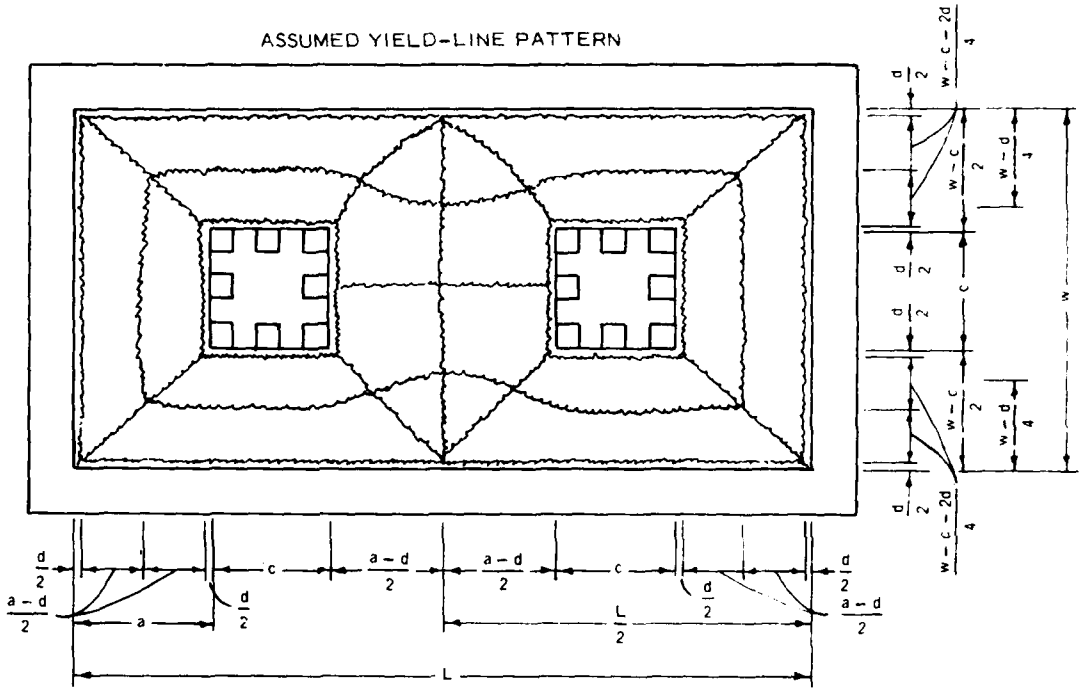
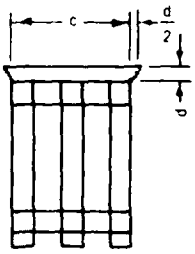


Figure 2.5 Approximation of the boundary conditions of a slab panel upgraded under the midspan.



$$a = \frac{L - 2c}{3} \qquad \frac{a-d}{2} = \frac{\frac{L-2c}{3} - d}{2} = \frac{\frac{L-2c-3d}{3}}{2} = \frac{L-2c-3d}{6}$$



V_{max} = MAXIMUM ALLOWABLE DYNAMIC SHEAR = $1.26\left(4\sqrt{f'_c}\right)bd$

$b = 4c + 4d$

$$V_{max} = 1.26\left(4\sqrt{f'_c}\right)(4c + 4d)\,d$$
$$= 20.16\sqrt{f'_c}\,(cd + d^2)$$

$$V_{on\ column} = \left[(a+c)\left(\frac{w-c+2c}{2}\right) - \left(\frac{w-d}{4} - \frac{w-c}{4}\right)\left(\frac{a}{2}\right)\right] P$$

$$= \left[\left(\frac{L-2c}{3} + c\right)\left(\frac{w+c}{2}\right) - \left(\frac{c-d}{4}\right)\left(\frac{L-2c}{6}\right)\right] P$$

$$= \left(\frac{(L+c)(w+c)}{6} - \frac{(c-d)(L-2c)}{24}\right) P$$

$$= \left(\frac{Lw + Lc + cw + c^2}{6} - \frac{cL - 2c^2 - dL + 2dc}{24}\right) P$$

$$= \left(\frac{4Lw + 4Lc + 4cw + 4c^2 - cL + 2c^2 + dL - 2dc}{24}\right) P$$

$$= \left(\frac{4Lw + 3Lc + 4cw + 6c^2 + dL - 2dc}{24}\right) P = 20.16\sqrt{f'_c}\,(cd + d^2)$$

Figure 2.6 Estimating the load applied to a column by an assumed yield-line pattern.

CHAPTER 3

TEST STRUCTURES

3.1 DESIGN OF THE ONE-WAY R/C FLOOR SYSTEM USED FOR TESTING

A specific one-way R/C floor system was chosen, upgrading methods were designed, and then the increased load capacity of the upgraded floors was verified with dynamic tests. The specific one-way R/C floor system chosen was a beam-girder type, because they are very common. Obviously, a whole floor system would be hard to dynamically test, but a large representative section out of a floor system, placed under the right boundary conditions, could be dynamically tested in a laboratory and thus give representative results. A representative section out of a beam-girder type floor would include everything between four columns, including the columns, as shown in Figure 3.1. The tests were performed in the LBLG test device of WES, which can produce dynamic loads similar to those from a nuclear weapon in the megaton range (Reference 10). The LBLG has an inside diameter of 22 feet 10 inches, so about a 15-foot-square section would be the largest that would fit. A "typical" one-way R/C floor section, within the above restrictions, was chosen from the structures that were listed in the NFSS and analyzed by SRI International. A sketch of the floor section is shown in Figure 3.2. Most of the fallout shelters in the NFSS were built before the 1970's; therefore, they were probably designed according to the Working Stress Method and made of 40,000-psi yield strength reinforcing steel and 3000-psi crushing strength concrete. The typical slab section was similar to a slab section designed for a 150-lb/ft^2 live load. The details of the design are given in Table 3.1. The reinforcing steel was bent at typical locations described in the CRSI handbook (Reference 14). A sketch of the reinforcing layout is shown in Figure 3.3. Note that additional reinforcing bars were placed along the sides of the two outer beams to resist lateral bending caused by a slab being on only one side.

Three of these floor sections were built for testing. Test Structure 1 was a nonupgraded floor section, and Test Structures 2

and 3 were upgraded floor sections. An effort was made to make the three floor sections as identical as possible. All of the reinforcing steel used in the three floor sections came from the same lot to ensure uniformity of strengths. The results of the material properties tests on the reinforcing steel are shown in Table 3.2. Each floor section was cast in one continuous placement of concrete. The concrete was made using the same design mix, at the same ready-mix company, and inspected by the same engineer from WES for each floor section. The average crushing strengths of the concrete in Test Structures 1, 2, and 3 were 4930, 5970, and 3985 psi, respectively.

The floor sections were built according to the given design, but as in all construction practices, the suppliers provided materials that exceeded the design strengths to be on the safe side. Thus, the floor sections were stronger than they were designed for, due to both conservative design assumptions and excessive material strengths, but so are most floors actually constructed.

The floor section was analyzed utilizing three different methods, using the material properties of Test Structure 1. Bigg's approximate design method, given in Reference 23, predicted the floor section would collapse under an overpressure of 9 psi. The second analysis method took the resistance function determined from the static analysis given in Reference 31 and used it in a SDOF analysis. This method predicted a collapse overpressure of 11.3 psi (if 20-degree rotation at the hinges was considered failure). The third analysis used a computer program written by SRI International (Reference 26) which predicted a collapse overpressure of 16 psi. The first two analyses were considered conservative.

3.2 DESIGN OF WOODEN POST UPGRADING SYSTEM (TEST STRUCTURE 2)

Several different upgrading designs were first considered before two were decided upon to test. One of these designs was called the wooden post upgrading system. This upgrading system used composite columns composed of several wooden posts placed under the slab panels in

rows parallel to the existing beams to take advantage of the one-way reinforcing steel. Wooden posts were also used to upgrade the existing beams.

A preliminary check was made to see how many rows were needed. If one row of upgrading columns is placed under the midspan of the slab panels, their load capacities should be increased by roughly 8/3 to 4 times if moments are critical or by 8/5 to 2 times if shears are critical. This is ignoring the reduction in the span length due to the upgrading column widths. Thus, the load capacities of the slab panels in the floor section should be increased from 16 psi to over 42 psi if moments are critical.

Next, the dynamic punching shear capacity of the slabs was analyzed. This capacity determined the perimeter of the column needed to prevent punching. It was calculated that two 24-inch-square columns could support an overpressure of 54.4 psi without punching through. This large of a column reduced the spans considerably. A SDOF analysis using the results of a static analysis as a resistance function (Reference 24) predicted that the upgraded slab panels could resist 199 psi in flexure (assuming that 20-degree rotation at the hinges is failure). The shear in the upgraded slab panels was calculated to reach a critical value when the overpressure is 64.8 psi. Therefore, the upgraded slab panels should withstand greater than 50 psi.

It was decided to build the upgrading columns out of 4- by 4-inch timbers spaced apart to form 24-inch-square columns and braced with 2- by 4-inch boards. These materials are readily available at any lumber company (4- by 4-inch timbers are usually the largest timbers kept readily in stock at most lumber companies). The estimated load on each upgrading column from a 50-psi overpressure is about 12,000 lbf. Using a conservatively low compressive strength of 625 psi for unidentified wood and a recommended dynamic load factor of 1/2 (Reference 29), it was calculated that eight 4- by 4-inch timbers would be needed per column. A picture of a column under a slab is shown in Figure 3.4. The height of the columns in this test was short; however, a buckling

analysis of 4- by 4-inch timbers 10 feet tall indicates they need to be braced at midheight to prevent buckling.

The beams of the floor section were analyzed next and found to be in need of upgrading. Both the shear capacities and moment capacities of the beams were exceeded by the loads. The most critical consideration is the shear in the beams, because there are no stirrups in the center portion of the beams. Each beam needed to be supported at three 24-inch intervals under the center section and at 29-inch spans from each end. The columns that support the beams were made of 4- by 4-inch timbers also. Two timbers were needed per column; they were strapped together with steel bands used in strapping crates commercially. A picture of a column under a beam is shown in Figure 3.5.

The girders had stirrups throughout their entire length since the beams applied such a large shear to the center of the girders in the first place. The load from the upgraded beams would be from only half of the 29-inch spans; load also came from the ends of the slab panels onto the girder itself. The shear is not critical at this time; the moment at the middle of girder is instead. A single column is needed under the middle of the girder. Twelve posts were required to support the load. A picture of a column upgrading a girder is shown in Figure 3.6.

All of the wooden columns used everywhere for upgrading were installed tightly into place by driving wooden wedges underneath them. Two wedges were used under each column and driven with hammers in opposite directions until they would not move. The compressive strength of wood perpendicular to the grain is fairly low (Reference 29), so the wedges will compress a little under moderate loads until the wood compacts in this direction. This "give" in the supports can increase the load capacity of the test structure even more.

A sketch of the upgraded floor section is shown in Figure 3.7. A picture of some of the upgrading system under the typical floor section is shown in Figure 3.8. This is a conservative upgrading design so the slab will most likely support much more than 50 psi.

3.3 DESIGN OF STEEL BEAM UPGRADING SYSTEM (TEST STRUCTURE 3)

This system was to support the slab panels with rows of short beams placed parallel to the original beams to take advantage of the one-way reinforcing steel. The existing beams and columns were upgraded with columns again. The upgrading beams and columns were kept small enough so that two or three men could handle them easily. As calculated before, one row of supports under the midspan of the slab panels will roughly increase their load capacities to 42 psi. Since the width of the upgrading beams will shorten the clear spans of the slab panels even more, one row of upgrading beams should be sufficient to upgrade the slabs to 50 psi.

The design was started by assuming a reasonable width for the upgrading beam, say 10 inches. Next, the slab sections between the upgrading beams and the original beams were analyzed to determine their load capacities. A SDOF analysis that used the results of a static analysis given in Reference 24 as a resistance function predicted that the upgraded slab panels could resist 84.2 psi in flexure. (Failure was assumed when 20-degree rotation at the hinges occurred.) The shear load capacity was calculated to be about 50 to 60 psi with conservative analyses, depending upon the empirical factor multiplied times $\sqrt{f'_c}$ (Reference 36). Thus, the slab panels could be upgraded to withstand 50 psi with one row of steel beams 10 inches wide.

Next, the steel upgrading beams were designed to carry the load and yet be light enough for two or three men to carry. The area of contributing load to the upgrading beams was estimated to be all of the area between positive hinges in a yield-line drawing. Bigg's approximate design method in Reference 23 and the plastic design of steel beams were used to design the steel beams. The design started by assuming a ductility ratio (μ) of 2, the shape of the load function as triangular, the duration of the loading (t_d) as 0.70 seconds, and the natural period of the beams as very short. From Figure 2.24 in Reference 23, the ratio of the maximum resistance (R_m) to the peak load (CF) was around 1.3 for the above values. The maximum applied moment on the

upgrading beams was calculated assuming they were simply supported ($M_{max} = \frac{w\ell^2}{8} \times 1.3 = 655.15$ foot-kips, where w is the distributed load and ℓ is the span length). The minimum required plastic modulus was calculated next ($Z = \frac{M_{max}}{F_y} = 218.4\ in^3$). The lightest wide-flange beam with a large enough Z was found to be W24 by 84 in Reference 37, but this beam would weigh 1106 pounds, which is too heavy. Two short beams would have to be used instead. Two beams, 78 inches long supported by 8-inch-wide steel posts, would work. Their clear spans would be 62 inches. For these beams $M_{max} = 100.88$ foot-kips and $Z = 33.63\ in^3$. The lightest shape that worked was a W14 by 26, but a W12 by 27 would also work. Some W12 by 27 beams were readily available at WES, so they were used. The initial assumption of the natural period (T) was checked and found to be 0.00218 second. This is so small in comparison to the load duration that the load can essentially be assumed static. Thus, $R_m/F = 1.0$ could be used in design. The flange width of W12 by 27 beams is 6.5 inches, which is less than the assumed 10 inches, so the analysis was repeated using new slab span lengths. The decrease in R_m/F and the increase in the spans offset each other, and the W12 by 27 beams were adequate. Other design criteria were checked once the probable beam had been chosen, such as the critical buckling lengths of the compression flange and web and the maximum shear capacity. The shear capacity was found to be slightly low, but the calculation was conservative, and the designed beams were used anyway. A picture of the upgrading beams in position is shown in Figure 3.9.

The columns supporting the upgrading beams were designed next. A SDOF analysis was used in conjunction with an elastic design method for steel columns. (Little is gained in using plastic design methods for columns.) The required length of columns in an actual building is estimated to be about 10 feet. A rough idea of the size of the column was obtained from a static design at first. An 8-inch nominal diameter, standard weight structural steel pipe or a 6-inch nominal diameter, extra strong steel pipe satisfies the static design. The natural period

of these columns would be about 0.0037 second. Since the duration of the load is 0.700 second, it was so much longer than the natural period that the load acted like a static load; therefore, the initial static design was considered adequate. Six-inch, extra strong, structural steel pipe was used because it was readily available at WES. A close-up of some steel columns supporting some upgrading beams is shown in Figure 3.10. One of the columns has a load cell on top of it.

The upgrading beams and columns were secured in place by mounting the columns on top of high strength bolts and nuts, as shown in Figure 3.11, and then tightening the bolts so the beams were thrust tightly against the slabs. The nuts bore against a bearing plate with holes in it to allow the bolts to stick through. The bolts were tightened by placing one wrench on the nut and another on the bolt. The bolts were sized to support the same loads as the columns were designed for.

The upgrading of the original beams, girders, and columns was designed and analyzed using the same methods used for those in Test Structure 2, but the loads were distributed slightly differently. The presence of some stirrups in the beams was taken advantage of to reduce the number of columns needed to upgrade them. The same number of wooden posts were needed, but they were combined in a smaller number of columns. A picture of the upgrading columns under the center beam and one girder is shown in Figure 3.12.

A layout of the upgrading system is shown in Figure 3.13. This upgrading system was conservatively designed, so the load capacity of the structure would most likely exceed 50 psi also.

3.4 DESIGN OF THE REACTION STRUCTURE

The function of the reaction structure was to hold the floor sections under boundary conditions similar to those in a continuous floor system. In order to impose these boundary conditions and survive the high loads used to fail the floor sections, the reaction structure had to be very stiff and strong. The reaction structure was also designed so it could be used for all three tests. The design was such

that a tested floor section could be easily removed and a new one could be installed for the next test. A sketch of the reaction structure is shown in Figure 3.14.

Several boundary conditions had to be matched. The floor sections were supported by the reaction structure at their corners with four short columns of similar cross sections to those that would be used in a continuous floor. The beams and girders along the outer edges of the floor sections were extended beyond the columns, through the walls of the reaction structure, and clamped down with rigid steel brackets so their end conditions at the columns would be similar to those in a continuous floor. Twelve large steel bolts were embedded in the reaction structure at each corner to bolt the steel brackets down with. Since the beams and girders along the outer edges of the floor section have a slab panel on only one side of them, an unbalanced moment will exist that will tend to rotate them. Grout was poured between the walls of the reaction structure and the outside of the beams and girders to prevent them from rotating. The walls of the reaction structure were greased before the grout was placed to keep the grout from sticking to them and allow the beams and girders to vertically deflect. Also, the grout sealed around the floor section to prevent the overpressure from leaking underneath it. The only boundary condition of a section from a continuous floor system that could not be matched was the amount of load the outside beams would receive and, thus, how much they would deflect. This was because the beams and girders along the outer edges did not have a slab on either side of them as they would in a floor system. However, when the slabs are upgraded at the midspans, the resulting smaller slab panels that are on either side of the center beam should have boundary conditions very close to those in an upgraded continuous floor system, since they are away from the edges.

The reaction structure was made of high strength concrete and large amounts of reinforcing steel. The walls of the reaction structure were designed to resist the large lateral pressures on the sides. The floor of the reaction structure had to be designed to withstand the loads from the corner columns, the worst of the different upgrading

methods, and the bending moments from the walls. The columns were designed to have similar cross sections to those in a continuous floor, but were only 20 inches tall. Capitals had to be placed below the columns to prevent them from punching through the floor. Thus, the structure was strong and massive. A picture of the reinforcing steel in the reaction structure is shown in Figure 3.15a, and a picture of the reaction structure itself is shown in Figure 3.15b.

The reaction structure was designed to hold the largest possible floor section inside the LBLG (Figure 3.16). It was 16 feet 5 inches square, with beveled corners. There was only a 1.22-inch clearance between the corners of the reaction structure and the LBLG. The height of the reaction structure was kept to a minimum, but a crawl space of 38 inches under the slab panels was needed to install instruments and upgrading supports (Figure 3.17). An opening was made in one wall of the reaction structure to allow access to the crawl space. The reaction structure was placed on top of 5 feet of sand inside the LBLG so it would hold floor sections 6 inches from the top of the LBLG. Sand was also placed between the sides of the reaction structure and the LBLG rings up to within 6 inches of the top. Thus, there was not much excessive volume left above the surface of the floor section to fill with pressure. An accessory to the reaction structure was a steel entranceway which was placed next to the hole in the wall of the reaction structure to give access to the crawl space from the top. A steel lid was placed over the top of the entranceway and covered with sand before each shot to prevent pressure from leaking under the slab. Some other accessories to the reaction structure were steel plates that isolated the ends of the beams and girders sticking through the walls from any loads.

The next chapter describes in more detail how the floor sections were installed in the reaction structure.

Table 3.1 Pertinent data on slabs and beams.

Slabs

(Designed as if in a continuous floor, with some modification in steel bending at the edges)

Frame type - Reinforced concrete
Support case - One-way-beam partitions
Design live load - 150 lb/ft^2
Clear span in short direction - 74 in (center-to-center length - 84 in)
Clear span in long direction - 158 in (center-to-center span - 168 in)
Thickness of slab - 4 in
Effective depth of steel - 3 in
Ratio of negative steel - 0.00458 (No. 3 bars at 8-in average spacing)
$A_s = 0.165\ in^2/ft$
Ratio of positive steel - 0.0030 (No. 3 bars at 12-in spacing)
$A_s = 0.11\ in^2/ft$
Temperature steel ratio - 0.00204 (No. 3 bars at 18-in spacing)
$A_s = 0.073\ in^2/ft$
Design concrete strength - 3000 psi
Design yield strength of reinforcing - 40,000 psi

3 // Beams

(Designed as if in a continuous floor as T beams, with some modifications at the edges)

Effective flange width (b' in T beam) - 39.5 in
Width of web (b_w) - 10 in
Thickness of beam - 14 in
Effective depth of steel - 11.75 in
Clear span of // beam - 158 in (center-to-center length - 168 in)
Ratio of negative steel - 0.0128 (two No. 6 and two No. 5) $A_s = 1.5\ in^2$
Ratio of positive steel - 0.002 based on b' (three No. 5) $A_s = 0.93\ in^2$
Stirrups - No. 3 stirrups, one at 2 in and seven at 5 in

2 End Beams

(Designed as if in a continuous floor system as T beams)

Effective flange width (b' in T beam) - 39.5 in
Width of web (b_w) - 10 in
Thickness of beam - 14 in
Effective depth of steel - 11.75 in
Clear span between supports - 158 in (center-to-center length - 168 in)
Ratio of negative steel - 0.0204 based on b_w (four No. 7) $A_s = 2.4\ in^2$
Ratio of positive steel - 0.0047 based on b' (two No. 7 and one No. 9)
$A_s = 2.2\ in^2$
Stirrups - No. 3 stirrups, one at 2 in and 44 at 3.5 in

Table 3.2 Yield strengths of the reinforcing steel (provided by supplier).

Size	Grade	Yield Strength (psi)
No. 3	40	60,000
No. 5	40	54,545
No. 6	40	63,636
No. 7	40	62,727

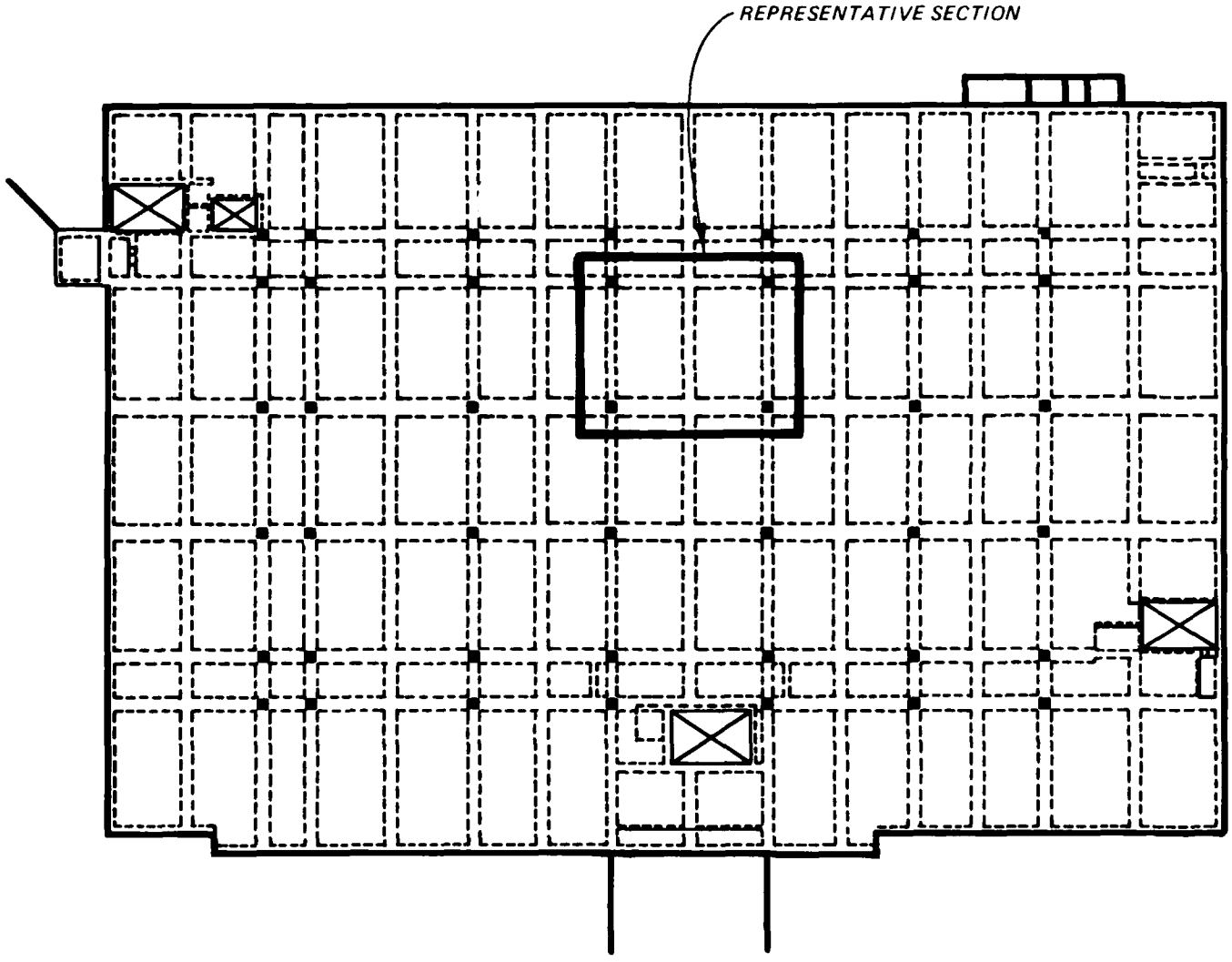


Figure 3.1 Typical beam-girder type floor construction.

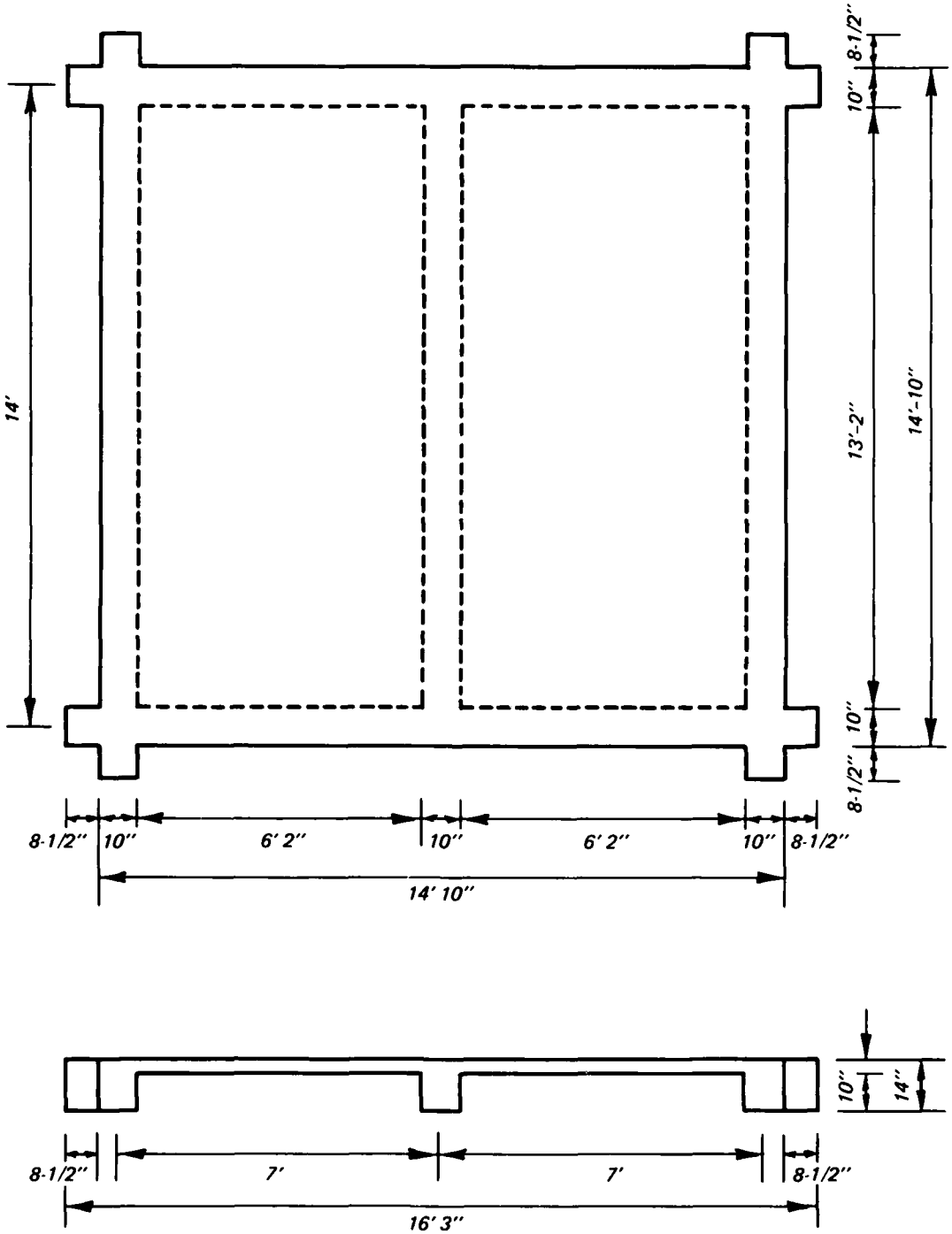
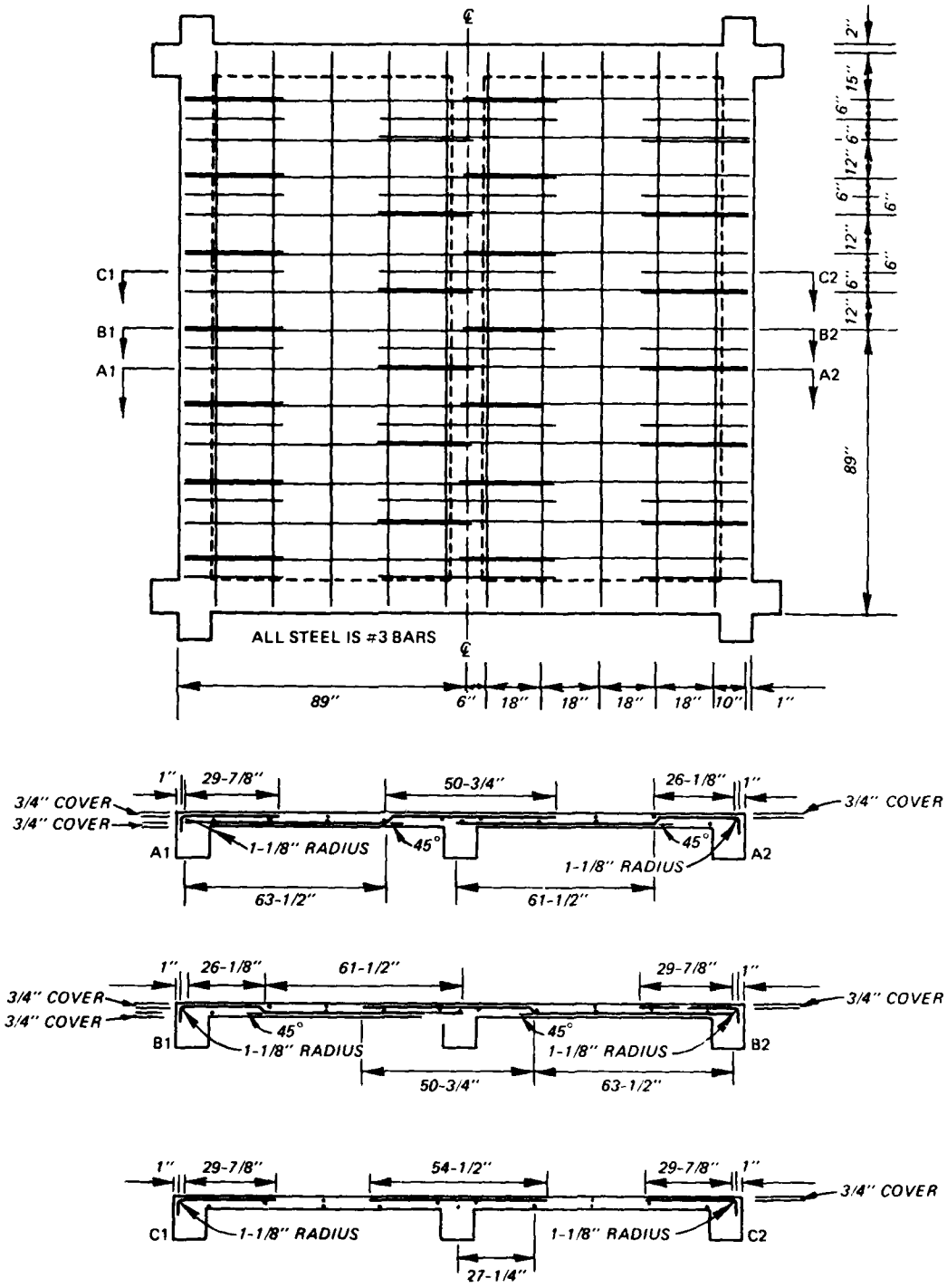
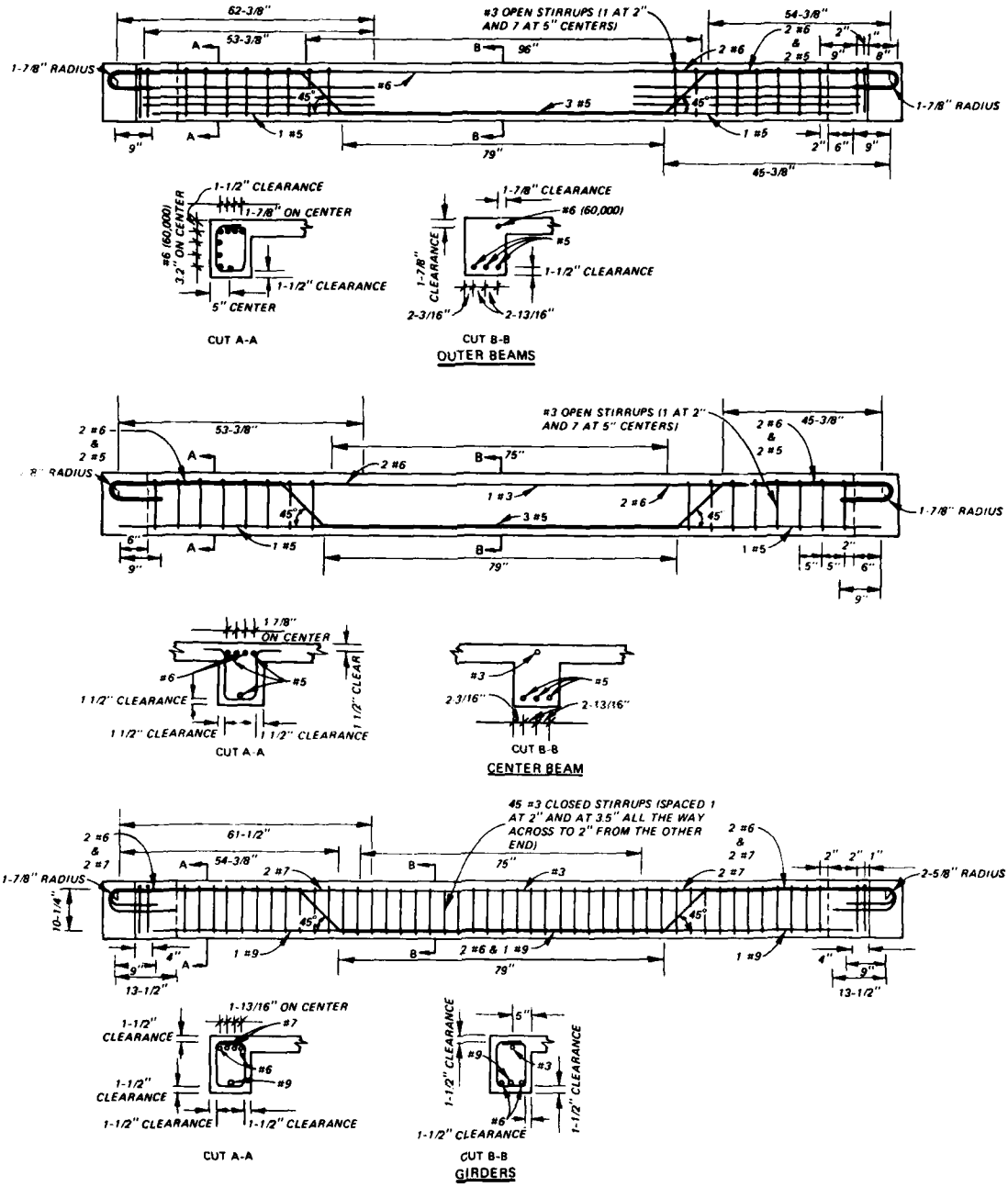


Figure 3.2 Dimensions of the "typical" floor section.



a. Slab steel.

Figure 3.3 A sketch of the reinforcing steel layout in the floor section (Continued).



b. Beam and girder steel.

Figure 3.3 (Concluded).

Figure 3.4 Composite column upgrading a slab panel.

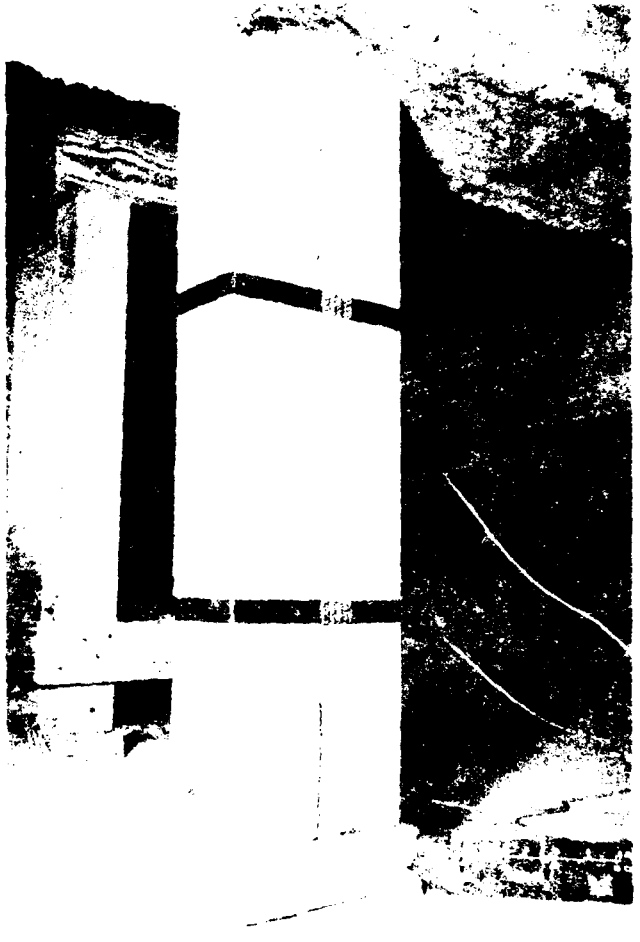


Figure 3.5 Composite [illegible] a beam.

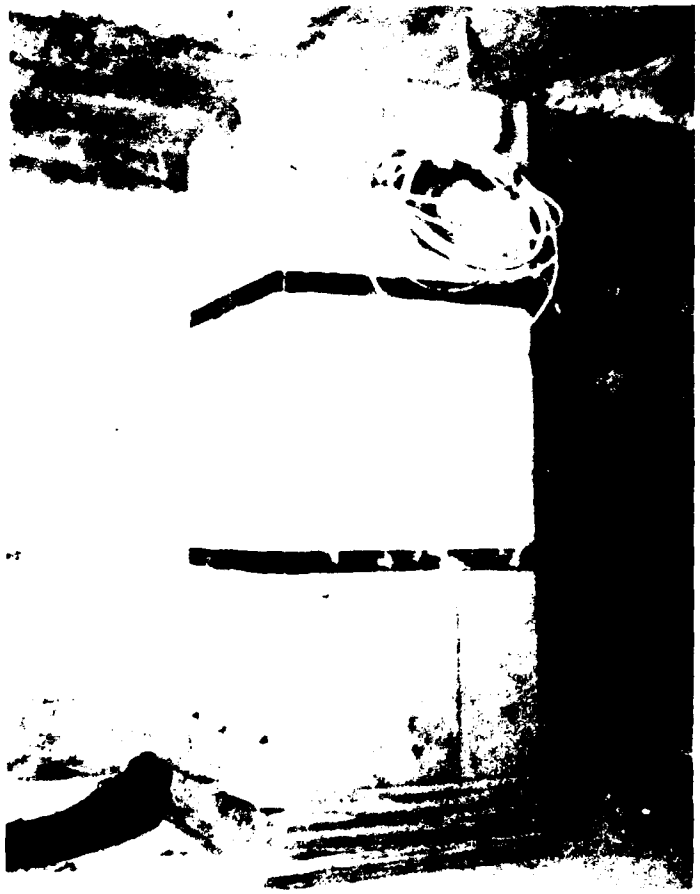


Figure 3.6 Composite column upgrading a girder.

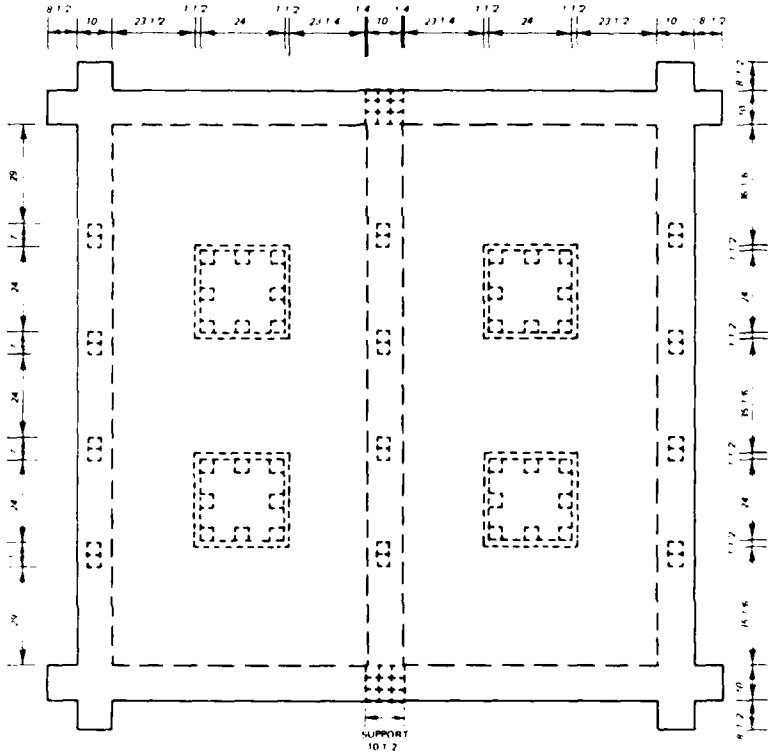


Figure 3.7 A plan of the wooden column upgrading system.

Figure 3.8 A partial view under the floor section upgraded with wooden posts.

Figure 3.9 Upgrading steel beams in place under midspan of a slab panel.

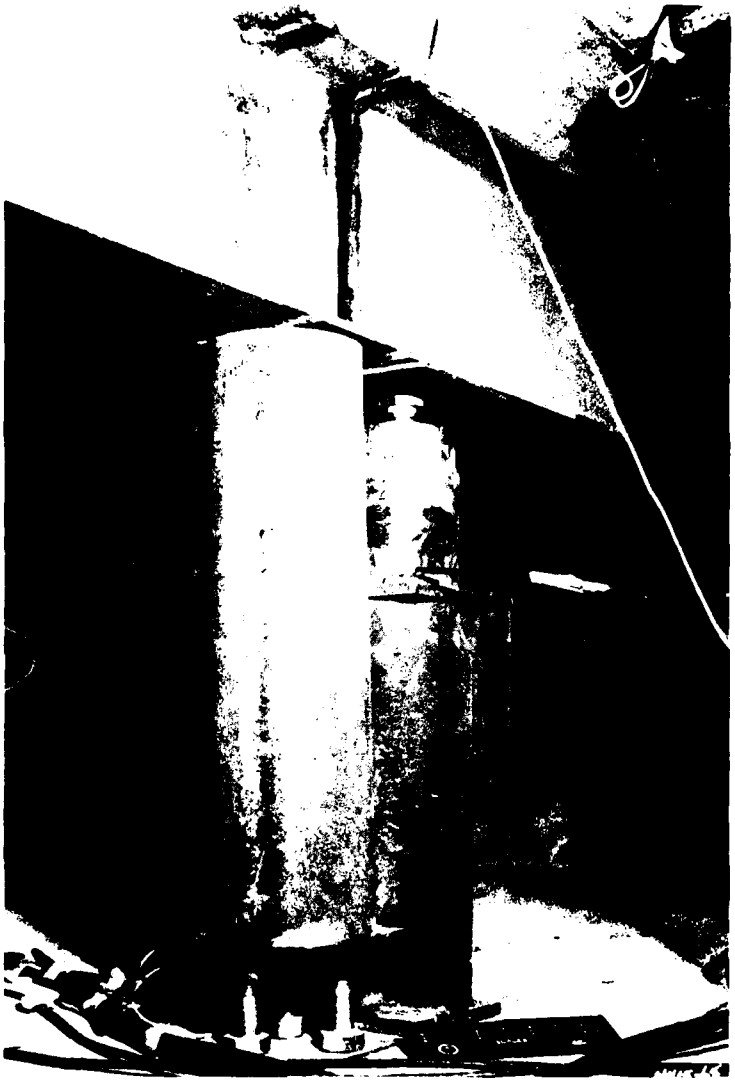


Figure 3.10 The steel columns supporting the steel upgrading beams.

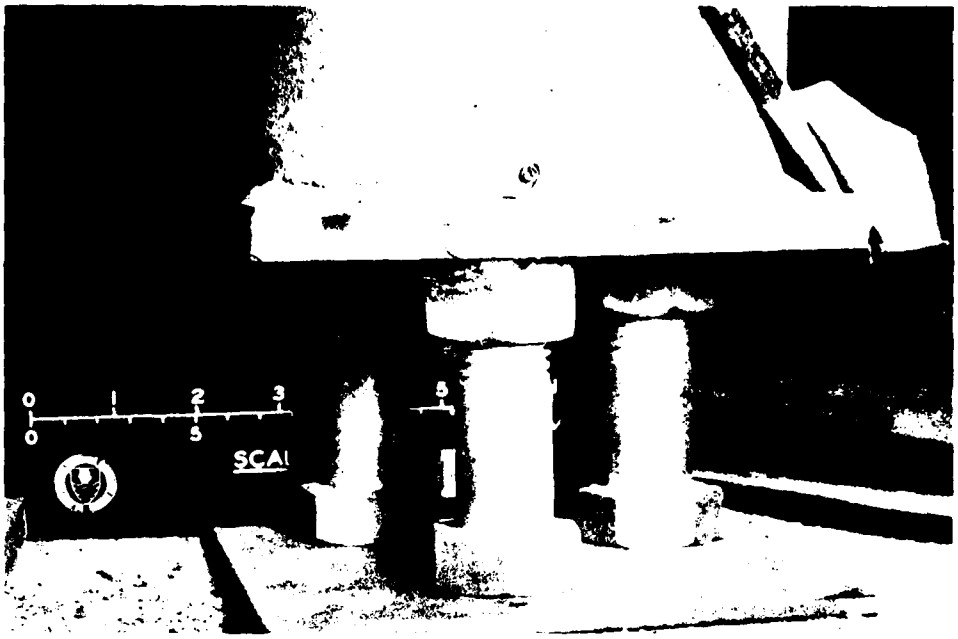


Figure 3.11 Bolts under the steel columns supporting the upgrading beams.

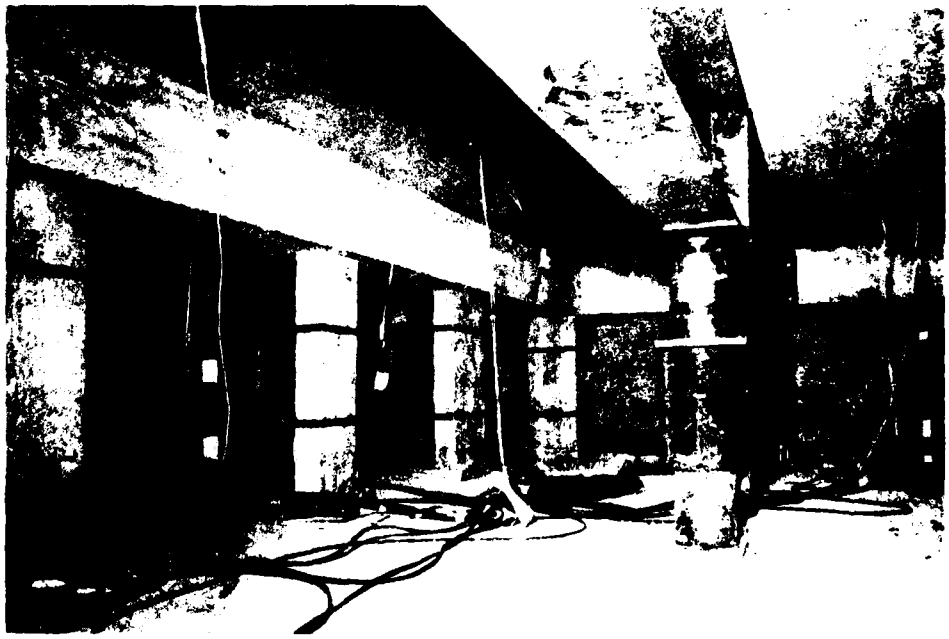


Figure 3.12 The wooden upgrading columns under the center beam.

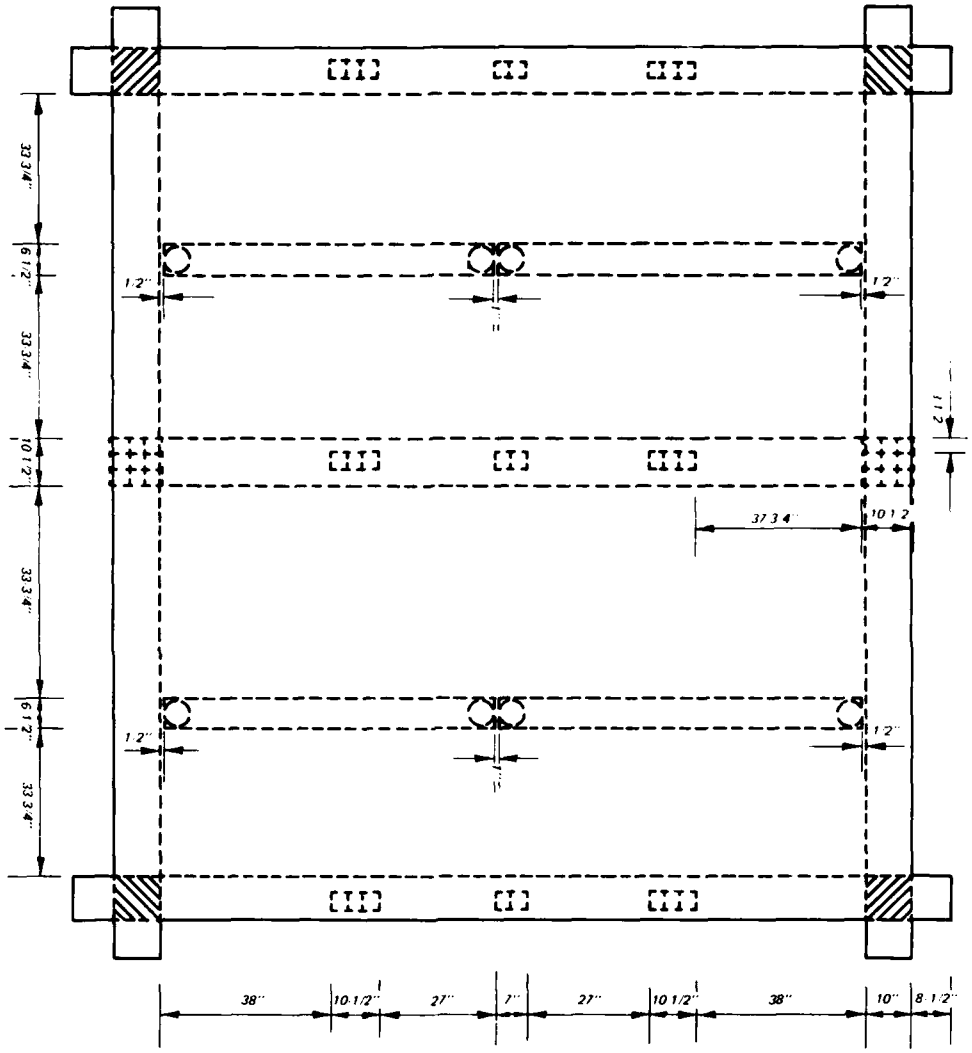


Figure 3.13 A plan of the steel beam upgrading system.

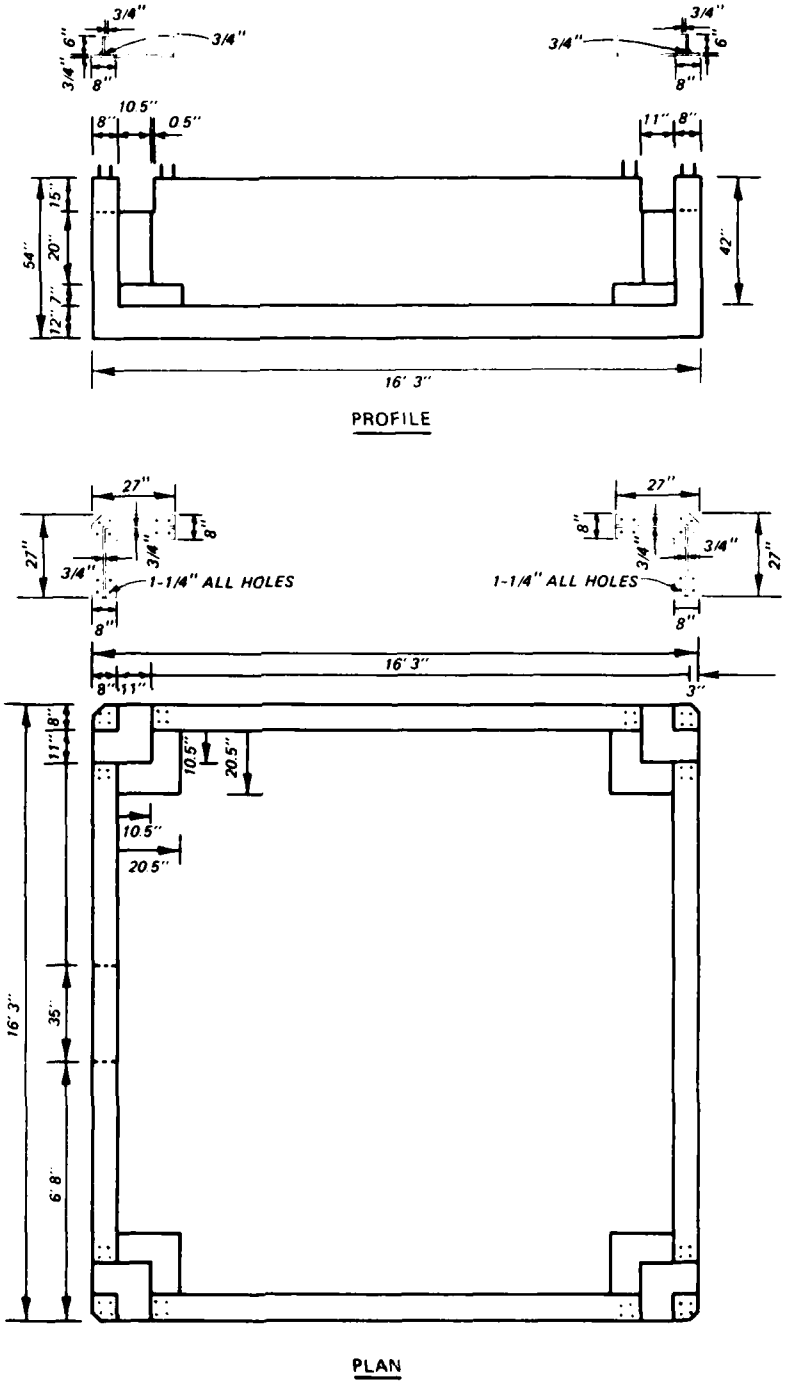
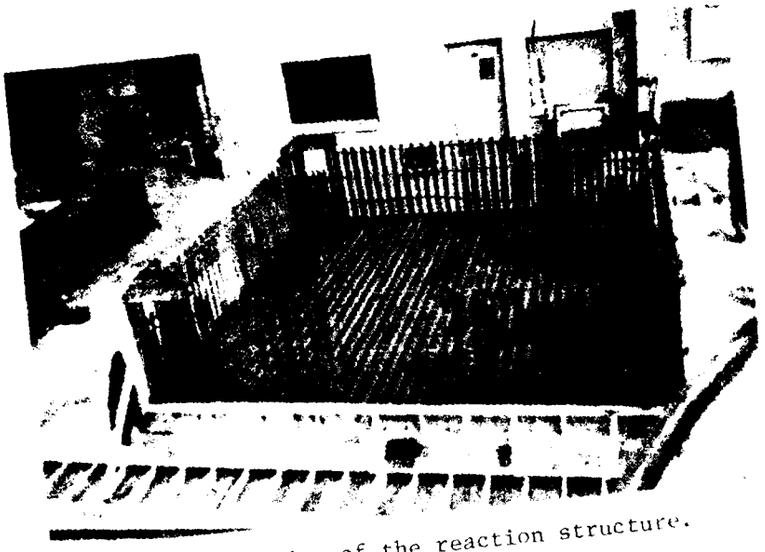
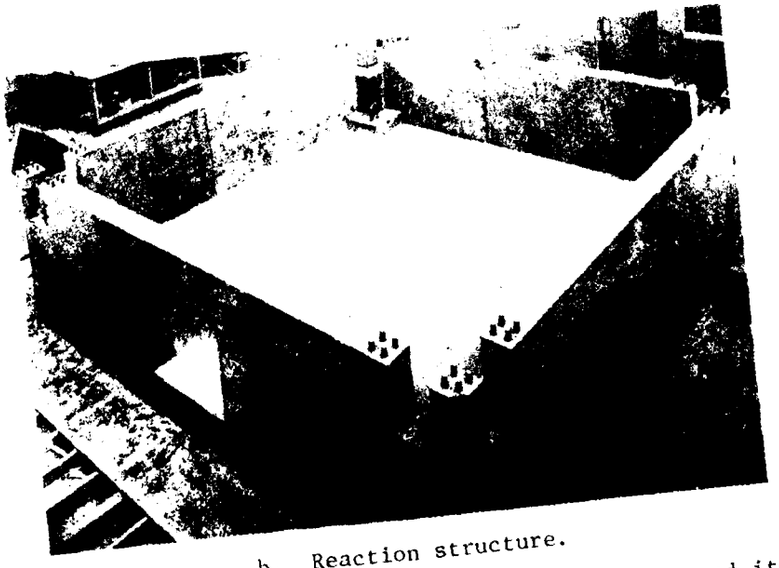


Figure 3.14 Plan and profile of the reaction structure.



a. Reinforcing of the reaction structure.



b. Reaction structure.

Figure 3.15 Pictures of the reaction structure and its reinforcing steel.

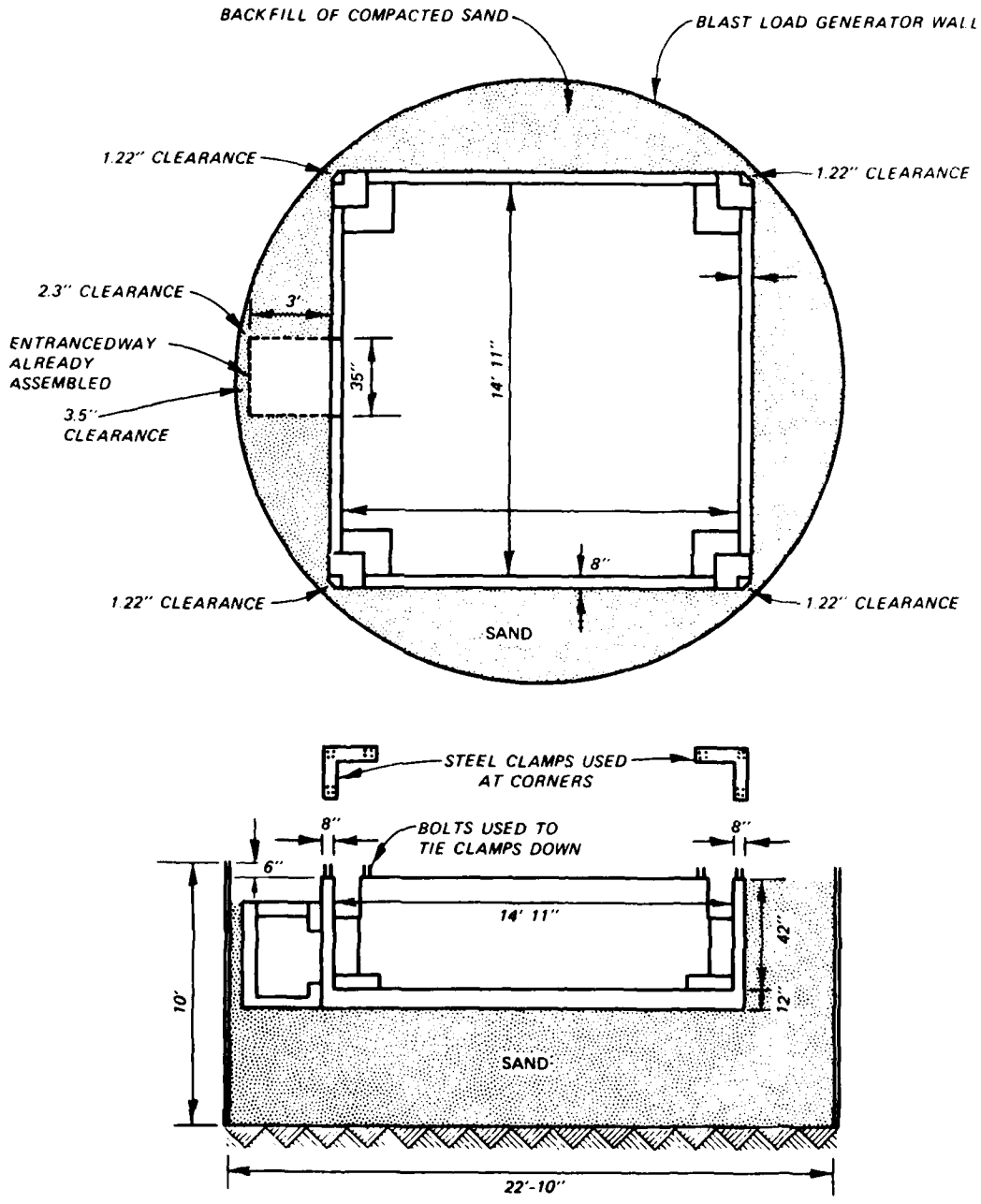


Figure 3.16 Reaction structure in place inside the LBLG.

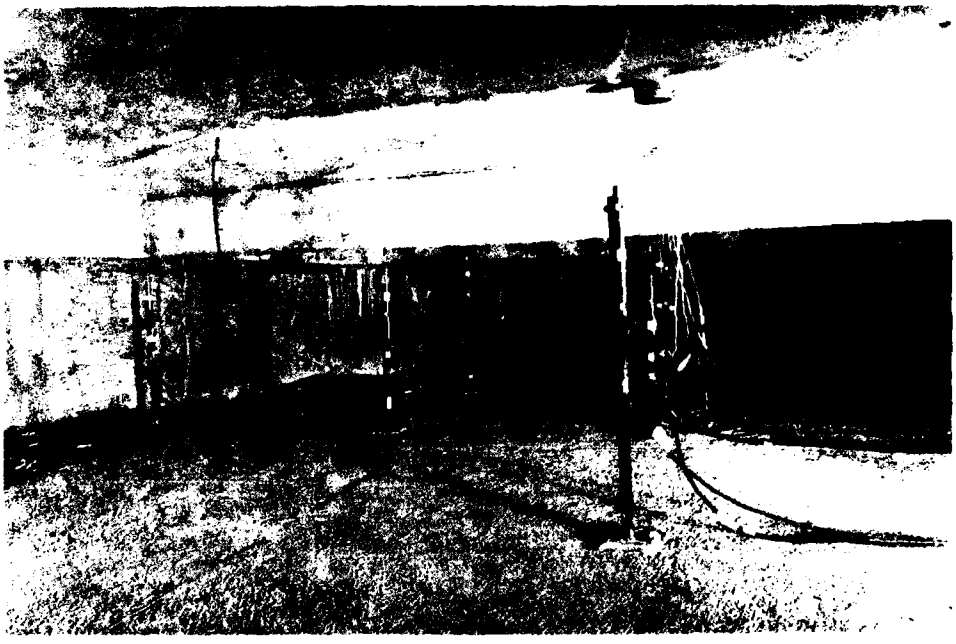


Figure 3.17 View of the crawl space under the nonupgraded slab.

CHAPTER 4

TEST PROCEDURE

4.1 PLACEMENT OF TEST STRUCTURES

The floor sections were tested in the LBLG at WES, which can produce dynamic loads similar to those from large nuclear weapons (Reference 10). The floor sections were tested until they received a large enough load that it caused major damage. The reaction structure was placed inside the LBLG to hold the floor sections under the proper boundary conditions while they were being tested. The following paragraphs describe the procedures for installing the floor sections into the reaction structure and preparing them for a test.

One of the first things done before installing each floor section was to rub grease on the walls of the reaction structure to keep the grout (applied later) from sticking to them. Next, just before a floor section was lowered, a quick-setting, high strength grout was mixed and placed wet on top of the columns to make a sure seal.

Once a floor section was placed inside of the reaction structure, it was clamped down at its corners and sealed around its edges. Quick-setting, high strength grout was also placed wet on top of the extensions of the beams and girders before the rigid steel clamps were set on top of them. Once the grout had dried, the clamps were tightly bolted down. The cracks between the walls of the reaction structure were sealed off by placing forms underneath them and pumping a quick-setting grout into them. This completed the installation of a floor section.

If an upgrading system needed to be installed, this was done next. The upgrading systems were installed by two or three common laborers with common tools just like would be used to install one in a fallout shelter.

The next step was to install the instrumentation, which is described for each floor section in the following section.

4.2 INSTRUMENTATION

Each of the floor sections was extensively instrumented to actively measure such things as accelerations, deflections, overpressures, thrusts in some upgrading columns, and strains of the reinforcing steel at places expected to have large moments. The specific type, number, and placement of the gages used for each floor section are given below.

4.2.1 Instrumentation of Test Structure 1

There were 39 gages installed for Test Structure 1 (the non-upgraded floor section). A sketch of the gage layout is shown in Figure 4.1. An Endevco 2261C-1000 accelerometer was placed in the middle of each slab panel. Several pairs of Micro-Measurements EA06-250-BF-350W strain gages were glued onto the top and bottom reinforcing bars parallel to each other (before the floor section was cast). These pairs of strain gages were located at places of expected large moments to indirectly measure these moments and get some indication of how the floor section fails. Transtek 244 2-inch deflection gages were placed under the beams and girders. The deflection gages under the slab panels were Transtek 246 6-inch deflection gages. Kulite XTMS-1-190-25 airblast gages were placed in the floor section to measure pressures at the quarter and center points of the slab, and two more were placed under the slab to detect pressure leaks. (These types of airblast gages were found to be temperature sensitive after the data were reduced.)

4.2.2 Instrumentation of Test Structure 2

There were 56 gages installed for the first two shots on Test Structure 2 (upgraded with wooden posts). The gage layout is shown in Figure 4.2. Then all of the deflection gages were removed before the third shot, because the floor section was predicted to collapse and they were needed for Test Structure 3. (All of those under Test Structure 1 were lost when it collapsed.) Three additional pressure gages were added for the last test to check the accuracy of the others.

Some of the gages were the same type as those used in Test Structure 1. Endevco 2261C-1000 accelerometers were placed in the middle of each slab panel. Pairs of Micro-Measurements EA06-250-BF-350W strain gages were placed on the reinforcing steel at places predicted to have large moments (different than nonupgraded). Transtek 244 2-inch deflection gages were placed under the beams and girders along the outer edges of the floor section. Transtek 246 6-inch deflection gages were used under the slab panels. Two Kulite XTMS-1-190-25 airblast gages were placed under the floor section to detect pressure leaks.

Some Transtek 245 4-inch deflection gages were placed under the center beam, which deflected more than the others. Since the Kulite XTMS-1-190-25 airblast gages were found to be sensitive to temperature changes, another type of pressure gage was used for Test Structure 2. Kulite XTS-1-190-50 airblast pressure gages were used to measure the overpressure on the slab at five points for those shots with less than 50 psi, and Kulite LQV0808UL soil stress gages and one Precise Sensors 211 Norwood pressure gage were added for the last shot to check the accuracy of the Kulite gages. Two homemade load cells were used to measure the load on single posts that were part of the large columns upgrading the slab panels.

4.2.3 Instrumentation of Test Structure 3

There were 49 channels of data recorded during each shot on Test Structure 3 (upgraded with steel beams). The gage layout for the structure is shown in Figure 4.3. An Endevco 2261C-1000 accelerometer was placed in the middle of each span of the slab panels between the upgrading beams and the original center beam. Pairs of Micro-Measurements EA06-250-BF-350W strain gages were placed on the reinforcing steel at the expected place of large moments. Transtek 244 2-inch deflection gages were used under the beams and girders along the outer edges of the floor section and under the middle of two steel upgrading beams. Transtek 245 4-inch deflection gages were used under the center beam. Transtek 246 6-inch deflection gages were used under the slab panels. Kulite XTS-1-190-100 airblast pressures gages were placed at

the quarter and center points of the slab to measure the overpressure distribution. A Precise Sensors 211 Norwood pressure gage was mounted in the top of the LBLG as a check on the pressure measurements. Two Kulite XTMS-1-190-25 airblast gages were placed under the floor section to detect pressure leaks. Homemade load cells were placed under two of the columns supporting the upgrading beams to measure the load they experienced.

4.3 TESTING

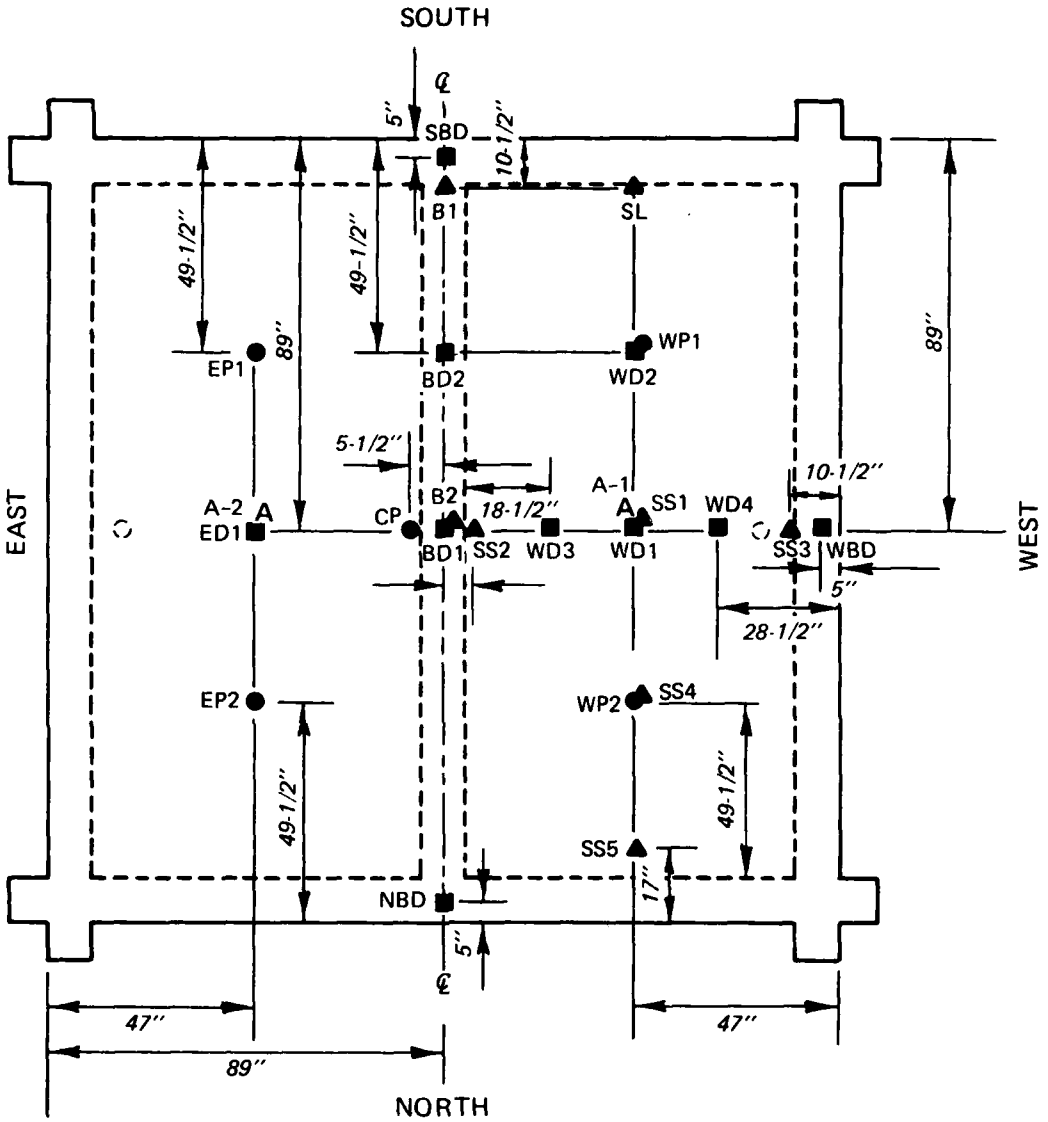
An overpressure is generated inside the LBLG by a combination of igniting a certain amount of explosive in firing tubes and opening pressure release valves at precise times. The explosive was Primacord, which can be obtained in several different sizes. The proper amount of Primacord for the desired overpressure was figured for each shot and then divided among the firing tubes. Each floor section was shot on several times, starting with an overpressure just under that expected to fail it and increasing the overpressure each shot until it was high enough to cause heavy damage to the floor section. The explosive had to be placed in the firing tubes after working hours for safety reasons. Once the explosive had been installed for a shot, the lid of the LBLG was placed on top and the LBLG was rolled under the Central Firing Station (CFS). The lid of the LBLG was jacked against the CFS to seal it, and the structure was ready for testing the next morning.

Measurements were taken during each shot. All data were recorded on magnetic tape using two 32-channel, Sabre III, Sangamo, FM recorders. Also, the zero time and "irig" (which is time in milliseconds) were recorded with the data. The recordings were made at tape speeds of 60 in/s.

The time it takes to perform the shot itself is not long at all. First, all of the electronic equipment is warmed up and the instrumentation checked. Next, a blasting cap is connected to the Primacord. The tape machines are started just before the shot during a short countdown. Once the countdown reaches zero, the explosive is ignited, and the shot is over shortly after that.

4-4 POSTSHOT INSPECTION.

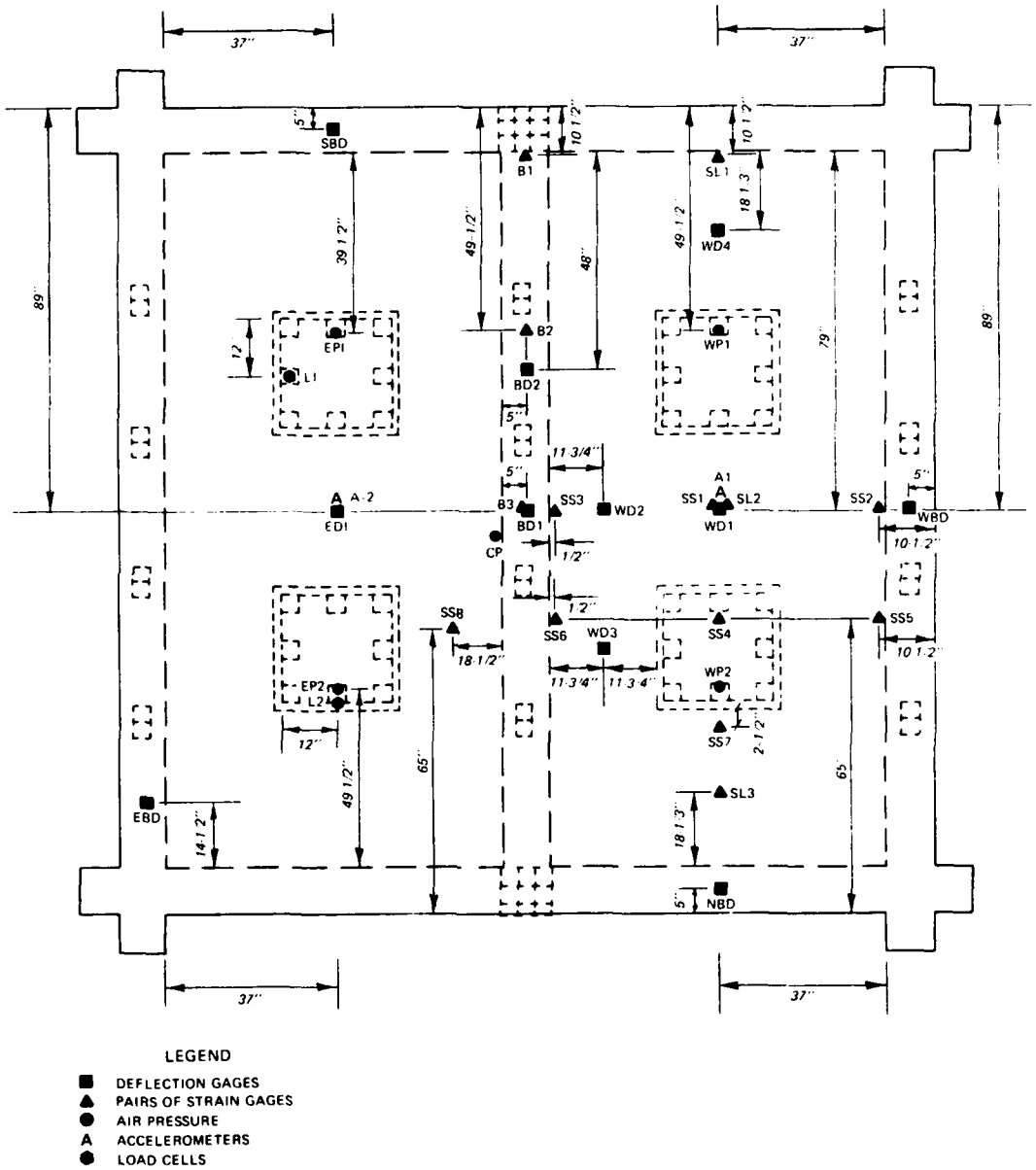
After each shot the LBLG was removed from the CFS and the structure was thoroughly examined. If damage was found, pictures and measurements were taken. If no significant damage occurred, the structure was prepared for another shot. The upgrading supports were checked to see if they were secure each time before testing. The concrete cylinders that were cast during the construction of the floor sections were tested as soon as possible after the floor section failed to determine the compressive strength of the concrete.



LEGEND

- A ACCELEROMETERS
- ▲ PAIRS OF STRAIN GAGES
- ■ DEFLECTION GAGES
- ● AIR PRESSURE
- ◌ INSIDE AIR PRESSURE

Figure 4.1 Instrumentation layout for Test Structure 1.



LEGEND

- ■ DEFLECTION GAGES
- ▲ PAIRS OF STRAIN GAGES
- ● AIR PRESSURE
- A ACCELEROMETERS
- ● LOAD CELLS

Figure 4.2 Instrumentation layout for Test Structure 2.

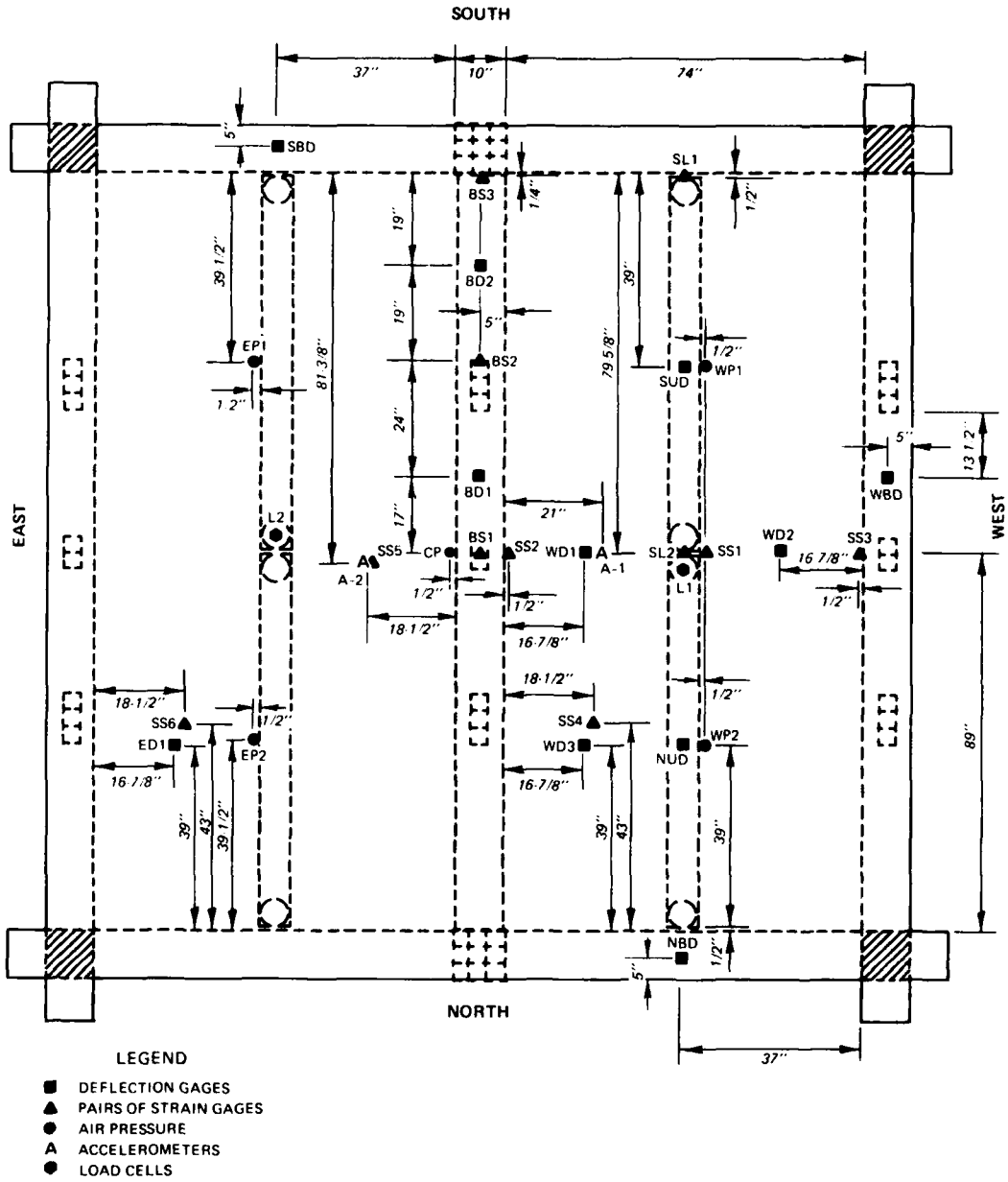


Figure 4.3 Instrumentation layout for Test Structure 3.

CHAPTER 5

TEST RESULTS AND DISCUSSION

5.1 GENERAL

The test results consist of the active measurements by the instrumentation recorded on magnetic tape, hand measurements of any permanent damage, and photographs. The information on the magnetic tapes was digitized and plotted by computer. All of the active measurements were referenced to a common zero time and plotted versus time in milliseconds.

5.2 RESULTS AND DISCUSSION OF TESTS ON STRUCTURE 1

Structure 1 was tested two times. The first test was called SLAB 1A, and the second was called SLAB 1B. The active measurements of these tests are displayed in Appendix A. Notice the unusual shape of the pressure-time histories; this was not the actual applied load on the slab. It was discovered after the data were digitized that Kulite XTMS-1-190-25 airblast pressure gages have a metal diaphragm and are sensitive to temperature changes. A crude experiment was run with a gage to determine its temperature sensitivity. A panel was placed between a gage and a hot air blower. At zero time, the panel was dropped out of the way and the gage was exposed to the hot air. The response of the gage is shown in Figure 5.1. The gage did not respond for the first few milliseconds and gradually indicated a false pressure reading. This particular type of gage was used in both shots. Thus, the overpressure measurements are probably only accurate for the first few milliseconds, after which the heat from the blast affected the measurement.

5.2.1 SLAB 1A

This test applied an average peak pressure of about 15 psi, which was close to the predicted load capacity of 16 psi for the slab panels. Most of the strains were fairly large but still elastic. The

deflection of the middle beam was much greater than that of the outside beams. This was probably due to the center beam receiving twice as much load as the outside beams and some friction existing between the reaction structure walls and the outside beams, in spite of the grease. The slab panels deflected a significant amount themselves, but were influenced somewhat by the deflection of the center beam. Deflections computed from the accelerations matched the measured deflections. The shot caused the slab panels to crack along the outside beams and girders, for a short strip along the west side of the center beam, and down one-half the center of the east panel, as shown in Figure 5.2. A close-up of one of the cracks is shown in Figure 5.3. No cracks were found on the underside of the floor section.

5.2.2 SLAB 1B

This test was to have been subjected to an average overpressure of 16 to 17 psi on the floor system, which was believed would barely fail the slab panels. But the wrong size Primacord was mistakenly used, and the resulting peak overpressure was 25 to 35 psi instead. The 25-g/ft Primacord is orange with one stripe on it and the 40-g/ft Primacord is orange with two stripes on it. The 40 g/ft was mistaken for the 25 g/ft and used instead. The result was a catastrophic failure of the floor system that threw the slab panels and center beam to the floor of the reaction structure as shown in Figure 5.4. The pressure gages lying underneath the floor section started measuring pressure at about 9 ms after zero time. The strain gages all measured very large strains within 10 to 40 ms. The strain gages on negative reinforcing steel in the slabs near the outside beams and girders reached failure faster than the strain gages elsewhere in the slab panels. This was probably due to the slab panels failing in direct shear along the outside beams and girders. The strain gages in the center beam indicate that it experienced a high strain level at the center at about 6 ms and failed at about 35 ms. Examination of the structure in the close-up of Figure 5.5 indicates that these theories are correct. The slab panels appeared to fail in direct shear because the cut at the outside beams and girders was

straight with no appearance of crushed concrete, and the reinforcing bars appeared to have been sheared instead of necked down. The center beam appeared to have pulled out of the girders at each end and rotated, while the middle of the beam failed in flexure.

5.3 RESULTS AND DISCUSSION OF TESTS ON STRUCTURE 2

Structure 2 was tested five times. The tests were called SLAB 2A-SLAB 2E. The data from these tests are contained in Appendix B.

5.3.1 SLAB 2A

This first test used the amount of explosive that was supposed to have been used for SLAB 1B. The average peak overpressure was 16.3 psi, if the peak pressure recorded by gage CP (Figure 4.2) is not used. Gage CP measured a peak load of 25.8 psi, which was about 9 psi greater than the other readings. This may have been due to the presence of a firing line that connected all of the lines of explosive at their centers. This line was not contained in a firing tube and may have projected a thin line of excess pressure directly below it. Thus, the center gage would measure a higher localized pressure. The duration of the entire load was about 700 ms. The nonupgraded slab was predicted to fail under a similar overpressure. The test closely simulated the overpressure from a 88-KT nuclear weapon as shown in the weapon fit in Figure 5.6. The load caused no damage to this upgraded structure. All of the strain gages (Figure 4.2) measured very small strains in the reinforcing steel. The bottom strain gages at SS3 and SS6 measured a small amount of permanent strain. Both gages were at the edge of the center beam. Strain gage pairs SS1, SS3, SS4, SS6, and SS7 all measured positive couples at places of expected negative couples. All of the deflection gages measured very small permanent deflections left in the structure after the loading. The outside beams and girders deflected the least amount, while the center beams deflected the most. Notice that the peak deflections are reached after about 20 ms and the strains histories contain an early large spike in them before they settle down at about 20 ms also. The two load cells indicated that the wooden posts

of the composite columns under the slabs received loads around 11,000 to 13,000 lbf. This is probably deceiving though, because the two posts are sitting on top of the steel load cells, whereas the rest of the posts are sitting on top of wooden wedges, so the posts over the load cells were stiffer and would draw more load. The only visible influence the test had on the structure was that one wooden column under the center beam was compressed enough to be slightly loose after the test.

5.3.2 SLAB 2B

This test used the same amount of explosive used in the SLAB 1B test. The pressure gages measured an average peak pressure of 29.3 psi, if the contribution of the spike on top of the pressure history of gage CP is ignored. Gage CP measured a peak pressure at least 10 psi greater than the rest of the gages again. This was again probably due to a localized line of excess pressure from the firing line, so this effect was ignored. This overpressure caused some small cracks to occur as shown in Figure 5.7. One small crack was detected on the underside of the center beam near one of the upgrading columns. A unique crack pattern forming a large "E" was found on the topside of the west slab panel. The cracks coincided with geometric characteristics of the slab. The back of the "E" ran parallel to the west edge of the center beam, was 1 inch west of it (which coincides with a temperature reinforcing bar), and was 4 feet long. The arms of the "E" were 2 feet long and coincided with rows of reinforcing steel. Two of the arms of the "E" also coincided with the inner edges of the middle two columns upgrading the center beam. All of the strain gages, except one, measured fairly small strains. The bottom strain gage (B3) measured the largest peak strain by far; it was 900 μin/in. Thus, the bending stresses in the structure were fairly small, except directly in the middle of the center beam. All of the deflections were 0.23 inch or less. The floor section deflected the least around the edge, while the center beam and middle portions of the slab panels deflected the most. The wooden columns were not loose after the test. The load cells under the two wooden posts in the slab upgrading columns measured extremely

high loads of 19,000 and 21,000 lbf. These loads would cause stresses in the posts of 1551 and 1714 psi, respectively, which are much greater than the design load. The wood was probably a much better grade wood than the assumed worst condition.

5.3.3 SLAB 2C

This test applied an average peak overpressure of 37.5 psi, if the thin spikes on top of gages CP and WP-1 are ignored. These spikes were again probably due to the firing line not being in a tube. The structure survived the loading with the addition of some small cracks in both girders at their midspans between the upgrading column and the original columns, as shown in Figure 5.8. The strain gages on the tensile steel measured moderate elastic strains. The strain gages on the compression steel still measured small strains. The deflection gages were removed before the test in case the structure prematurely collapsed. (The gages needed to be saved for the third slab). The deflections calculated from accelerations of A1 and A2 were both 0.45 inch. The load cells measured very large loads on the wooden posts. Again, these loads are probably a little larger than what the other posts received because these posts were on top of the steel load cells, whereas the other posts were on softer wooden wedges. The average stress in these posts was around 2,000 to 2,100 psi, which is much more than the assumed design strengths.

5.3.4 SLAB 2D

This test applied an average peak overpressure of 72.6 psi on the floor system. A different method of firing the explosive was used for this test. A firing line was used for every two tubes. Each firing line ran along the top of a tube to the centers and then tied into two lines of explosive. This way the structure was shielded from the direct blast of the firing lines, but half of the LBLG was covered with slightly more explosive than the other half. The peak pressures differed, but the basic load curves were all about the same. The duration of the loads was about 700 ms.

Moderate damage was experienced by the structure. Several additional cracks appeared on the topside of the floor as shown in Figure 5.9. The cracks were highlighted with magic markers so they would show up from a distance. A close-up of one of the cracks before it was highlighted is shown in Figure 5.10. The floor section cracked on the underside, as shown in Figures 5.11-5.18. Some wooden columns received light damage also, as shown in Figures 5.19 and 5.20.

All of the strain gages, except two, measured slightly larger strains than on previous shots, but they were all still elastic, except two. The exceptions were strain gages SL3 Bottom and SS8 Bottom, which measured large peak strains that exceeded the yielding strain and indicated permanent deformation. The load cells measured peak loads of 38,900 and 36,000 lbf. The resulting average peak stresses in the wooden posts were 3175 and 2939 psi, respectively. These stresses are over twice the design value used for the upgrading posts. The deflections calculated from the accelerations of A1 and A2 were 0.8 and 0.6 inch, respectively. These are fairly large deflections considering the short span lengths.

5.3.5 SLAB 2E

This test caused severe damage. The two wooden posts on top of the load cells started punching through the slab, as shown in Figures 5.21-5.24, and the upgrading columns under the center beam failed under the load as shown in Figures 5.25-5.28. Although the floor section was still intact after the shot, it was in bad shape and would not have taken much more load before collapsing. Figure 5.29 shows the cracks in the topside of the floor section. The cracks in the underside of the slab are shown in Figures 5.30-5.34. The cracks were highlighted with a marker so they would show up better in the pictures. The wedges under all of the upgrading supports, everything except the outside beams and girders, started to crush severely. A picture of the wedges under the posts, in a column that upgrades a slab panel, is shown in Figure 5.35. The upgrading columns under the outside beams and girders did not receive as much load and thus did not start to fail, as shown in

Figure 5.36. The portion of the floor section between the two rows of columns upgrading the slab panels deflected a large amount. The permanent deflection left in that portion is shown in Figure 5.37. This portion of the floor section was under conditions very similar to those in a continuous floor.

The average peak overpressure measured by all of the airblast pressure gages in the floor section, except WP-1, was 112.8 psi. WP-1 was excluded from the average because it measured a peak overpressure of about 50 psi greater than the others. The load duration was about 900 ms. Two soil stress gages were placed in the sand around the reaction structure to check the airblast pressure gages on the slab. SE-1 measured a peak overpressure of 130 psi, and SE-2 measured a peak overpressure of 60 psi. The shape of their overpressure curves differed from the overpressure curves of the airblast gages during the first 100 ms. This may have been due to a pressure differential in the LBLG. The soil stress gages were located near the edges of the LBLG, and the airblast gages were located in the center. However, the impulse curve at SE-1 was very similar to those of the airblast gages. SE-2 measured a low peak overpressure and impulse. A Norwood gage was installed in the side of the LBLG near the top as another check on the overpressure measurements. Its overpressure and impulse curves are very similar to those of SE-1. These additional gages confirmed that the airblast gages were fairly accurate.

The overpressure closely simulated the overpressure history of a 690-KT nuclear weapon at a range in which the peak pressure would be 109 psi, as shown in the weapon fit in Figure 5.38.

Several strain gages measured strains that exceeded the yielding strain of the reinforcing steel. The top strain gage at points SL1, SS2, and SS3 measured plastic strains. These gages were all at the edge of an outside beam or girder, both of which are fairly rigid, and where a high negative moment would be present. Both the top and the bottom strain gages at locations SS1, SL2, and SL3 measured plastic strains. These were all in the center of a portion of the slab between supports. The fact that SS1 Top measured a strain greater than the crushing strength

of concrete indicates that the load was near the maximum the structure could take. The bottom strain gage at location SS8, which was in the middle of the slab portion spanning between an upgrading column and the center beam, also measured plastic strains. The bottom gage at location B3, which was at the center span of the center beam between upgrading columns, measured very large plastic strains. This fact, along with the fact that the cracks in the beam at this point were deep, indicates the beam was loaded near its capacity.

The load cells (LOAD 1 and LOAD 2) measured peak loads of 47,000 and 37,800 lbf, respectively. The corresponding average peak stress in each wooden post was 3837 and 3086 psi, respectively. These stresses are greater than twice the design stresses for almost every kind of wood listed in Reference 29. The values given in Reference 29 are conservative, but these stresses are probably not far from the ultimate strength of the wood used.

The maximum deflections calculated from the accelerations measured by A1 and A2 were 1.05 and 1.04 inches, respectively. These are fairly large considering the short slab spans, but the slabs could deflect a little more before they would fail by flexure.

5.4 RESULTS AND DISCUSSION OF TESTS ON STRUCTURE 3

Test Structure 3 was tested three times. The tests were called SLAB 3A-SLAB 3C. The data from these tests are contained in Appendix C.

5.4.1 SLAB 3A

The first test on Structure 3 was the same as SLAB 1B and SLAB 2B. The average peak overpressure was 34.33 psi, ignoring the record of EP-2 (Figure 4.3), which stopped working properly. This overpressure caused the slabs to crack directly over the upgrading steel beams, as shown in Figure 5.39.

The strain gages all measured small elastic strains. The deflection gages under the outside beams and girders all measured peak deflections less than 0.137 inch. The peak deflections under the center

beam were larger than the peak deflections under the other parts of the floor section. This was probably due to the wooden columns compressing a little under the load. The peak deflection at B1 was greater than at B2 because of the presence of negative reinforing steel, stiff columns on each side, and load carried by the girder. The deflection gages NUD and SUD were under the upgrading beams and both measured peaks of 0.13 inch. Deflection gages WD1 and WD3 were both under a slab portion between the center beam and an upgrading beam. They both measured deflections of about 0.30 inch, which is close to what B1 measured under the center beam. Part of the deflections of these slab panels is believed to have been caused by the center beam deflecting. The deflection gages ED1 and WD2 were under the slab portions between the outside beams and upgrading beams and measured peak deflections of about 0.14 inch, which were about half the peaks measured by WD1 and WD3. Of course, the outside beams only had half as much load, so they did not deflect as much.

Load cell LOAD 1 broke during this test. Load cell LOAD 2 measured a peak load of about 30,000 lbf.

5.4.2 SLAB 3B

This test applied an average peak overpressure of 63.0 psi, which simulated an overpressure from a 534-KT weapon as shown in Figure 5.40.

The overpressure caused the floor section to crack in a few places, but no serious damage occurred. A picture of the crack pattern in the top of the floor is shown in Figure 5.41. Figure 5.42 shows that the floor section was permanently deformed between the two upgrading beams. The steel beams did not give much, but the wooden columns under the center beam compressed under the load. In Figure 5.43, a picture of a column under the center beam after the shot shows it was compressed. The crack patterns in the underside of the east slab panel are shown in Figures 5.44 and 5.45.

Four strain gages measured peak strains exceeding the yielding strain of steel; they were SS1 Bottom, SS1 Top, SS4 Bottom, and SS2 Bottom. The remaining strain gages measured moderate elastic strains.

The outside beams and girders and the steel upgrading beams all deflected a maximum of 0.16 inch or less, and they were very stiff. The center beam deflected a maximum of 0.71 inch at BD1 and 0.49 inch at BD2. (Again, BD2 deflected less due to the negative steel stiff columns on each side, and some load carried by the girder.) The peak deflections under the slab portion between the center beam and the upgrading beams were 0.60 inch at WD1 and 0.53 inch at WD3. These were probably affected by the deflection of the center beam. The peak deflections under the slab portions between the outside beams and the upgrading beams were about 0.15 inch at ED1 and 0.17 inch at WD2. These were about one-fourth of the peak deflections at WD1 and WD3.

Load cell LOAD 2 measured a peak load of about 45,000 lbf, which would cause an average stress of 5357 psi in the support column. The structure was essentially not damaged at all.

5.4.3 SLAB 3C

This test applied an average peak overpressure of 92.2 psi. The test structure failed under this load. A picture of the structure after the test is shown in Figure 5.46. Notice that part of a slab panel is still holding on and that the original beams did not fail. Closeup pictures of various portions of the structure are shown in Figures 5.47-5.49. The reinforcing bars are shown pulled out of the concrete in several places. The slab panels appear to have failed by exceeding flexure and bond strengths. The upgrading beams were buckled and some bolts were bent and broken. Most of the measurements indicate that the structure withstood the load for several milliseconds.

The overpressure histories all had a spike at about 85 ms when the structure failed. The pressure gages underneath the floor section started measuring pressure leakage at about 40 ms and continued to measure a rise in the pressure up to a peak measurement of about 15 psi at 79 ms.

All of the strain gages in the slab panels measured very large peak strains exceeding the yielding strain of the reinforcing steel. The strain gages on tensile steel started measuring large strains as

soon as the overpressure peaked, then hesitated at around 20 ms, and then continued to measure even larger strains. Most of the strain gages on the compressive steel only measured moderate strains for the first 25 ms or so, and then they started measuring larger strains and peaked out at anywhere from 30 to 55 ms. All of the strain gages in the center beam, except BS2, measured very small strains for the first 40 ms or more and then started measuring very large strains in quick, jumpy spikes. The extremely large spikes that were followed by nothing are due to the wires being hit and crushed by falling concrete.

The outside beams and girders did not deflect any more than 0.25 inch and remained undamaged. The deflection gages under the center beam were BD1 and BD2. They measured peak deflections of the gages at 1.1 inch and 0.8 inch, respectively. The deflection histories of all the beams and girders peaked at about 20 ms and held at that value for over 80 ms.

The deflection history of gage NUD indicated that the center of one of the upgrading beams started deflecting at about 4 ms, reached a peak deflection of 0.94 inch after 60 ms, and then fell at about 86 ms. The beam over SUD started deflecting at about 4 ms, reached a peak deflection of 0.3 inch at 35 ms, and then something happened to the beam to make the deflection gage read a negative deflection.

All of the deflection gages under the slab panels started measuring deflections at 4 ms. The deflections increased until they peaked at 72 ms, and then the slab panels broke at around 85 to 95 ms. The deflection histories show a slight hesitation around 30 ms and then proceed to increase at a more rapid rate. The peak deflections were the limits to the stroke on the deflection gages.

LOAD 2 measured a peak load of roughly 60,000 lbf. This value exceeds the design shear capacity and critical buckling load of the flange and web of the upgrading beams. The acceleration histories indicated large accelerations to start with, then they died down until about 30 ms, and then they increased again.

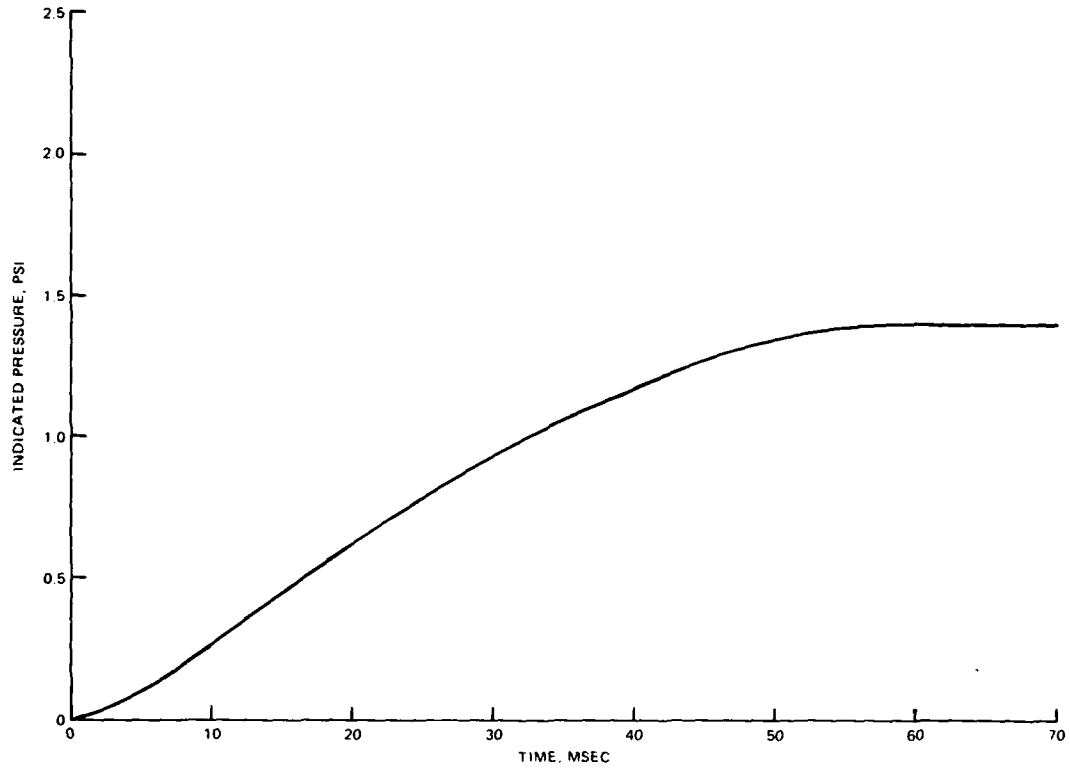


Figure 5.1 Indicated pressure from a moderate temperature increase on a XTMS-1-190-25 airblast gage.

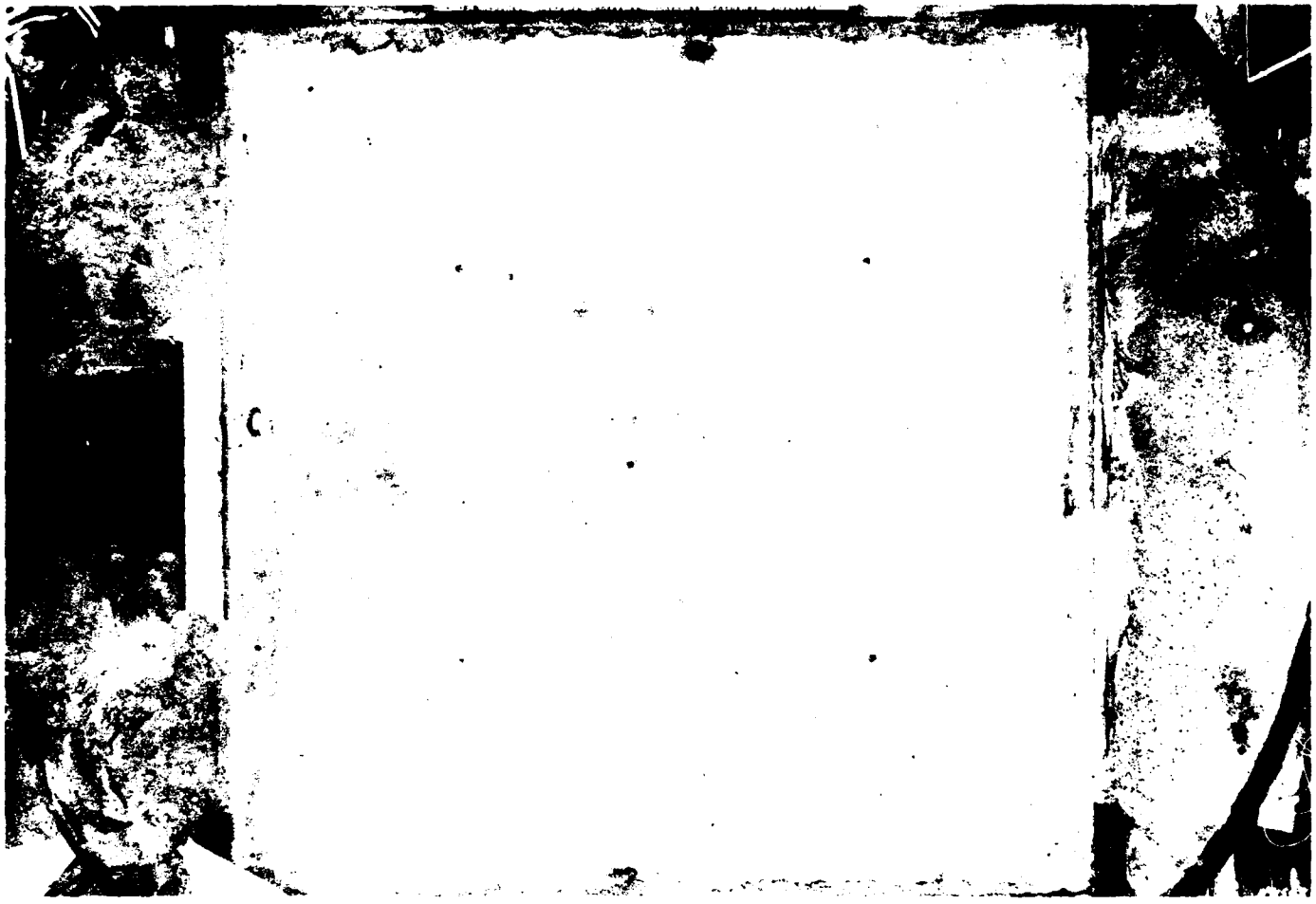


Figure 5.2 Picute of tie cracks in Test Structure 1 after shot SLAB 1A.

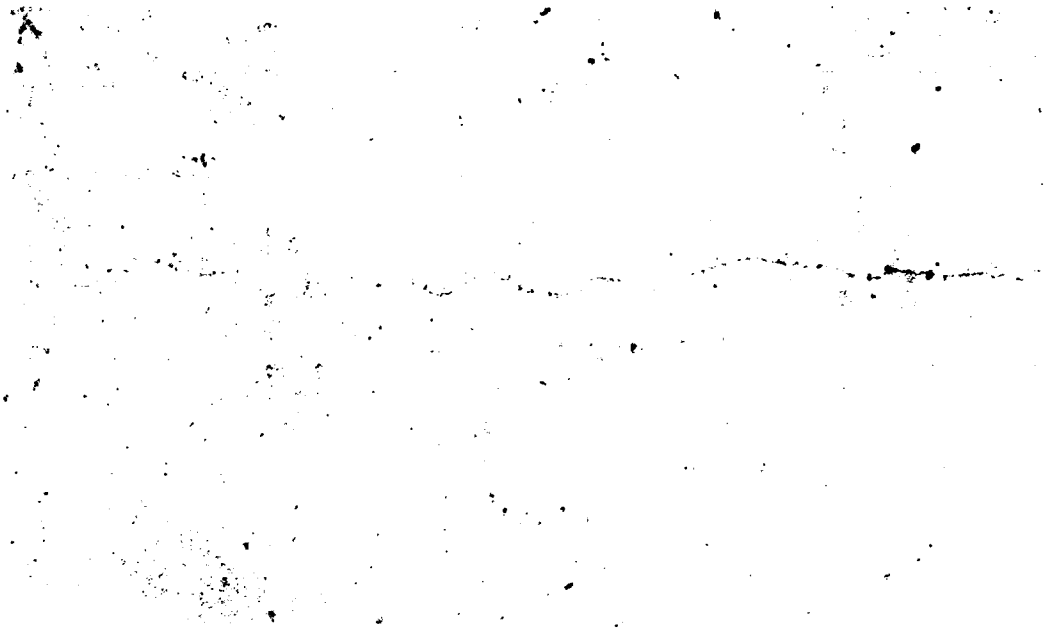


Figure 5.3 A close-up of one of the cracks in the floor section.

Figure 5.4 Top view of Test Structure 1 after shot SLAB 1B.

Figure 5.5 Closeup view of Test Structure 1 after shot SLAB 1B.

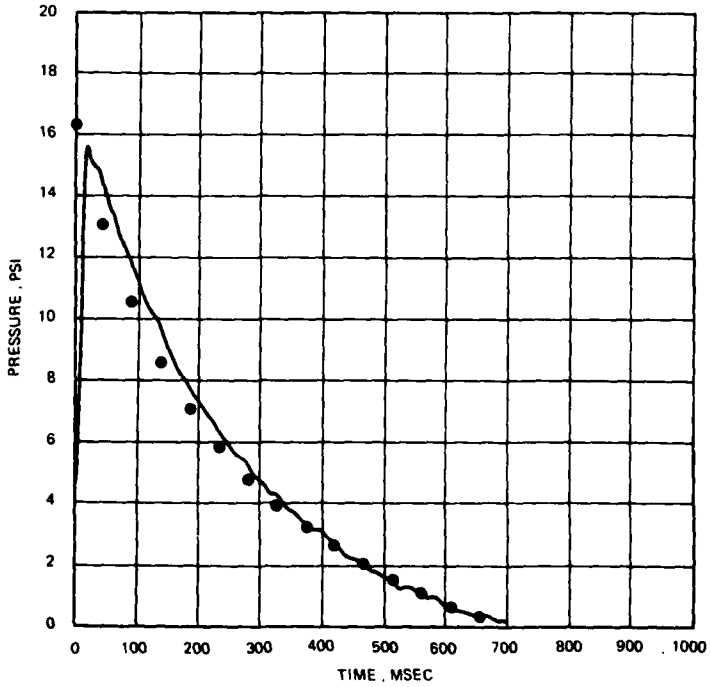


Figure 5.6 Weapon fit of the overpressure in shot SLAB 2A.

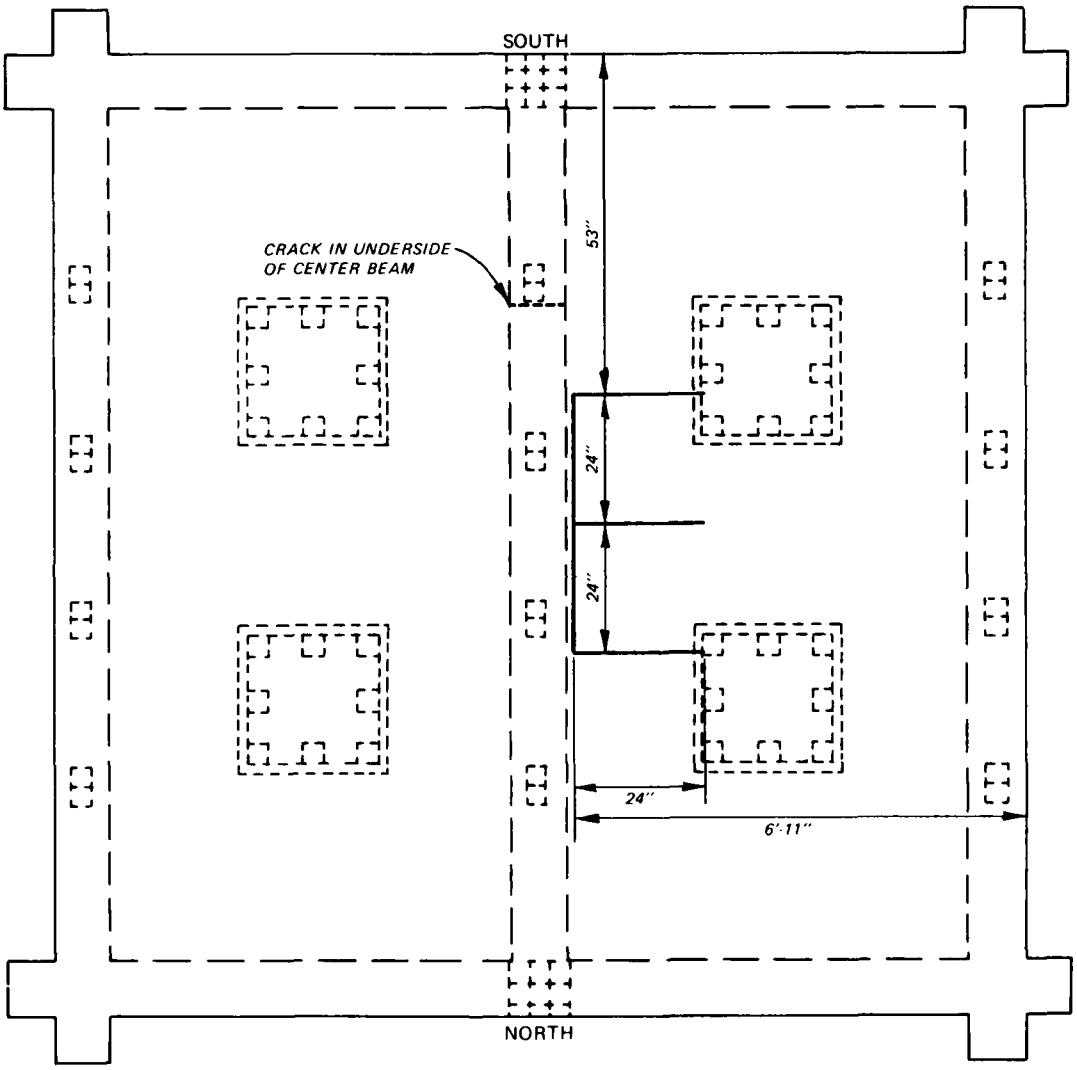


Figure 5.7 Crack patterns in Test Structure 2 after test 2B.

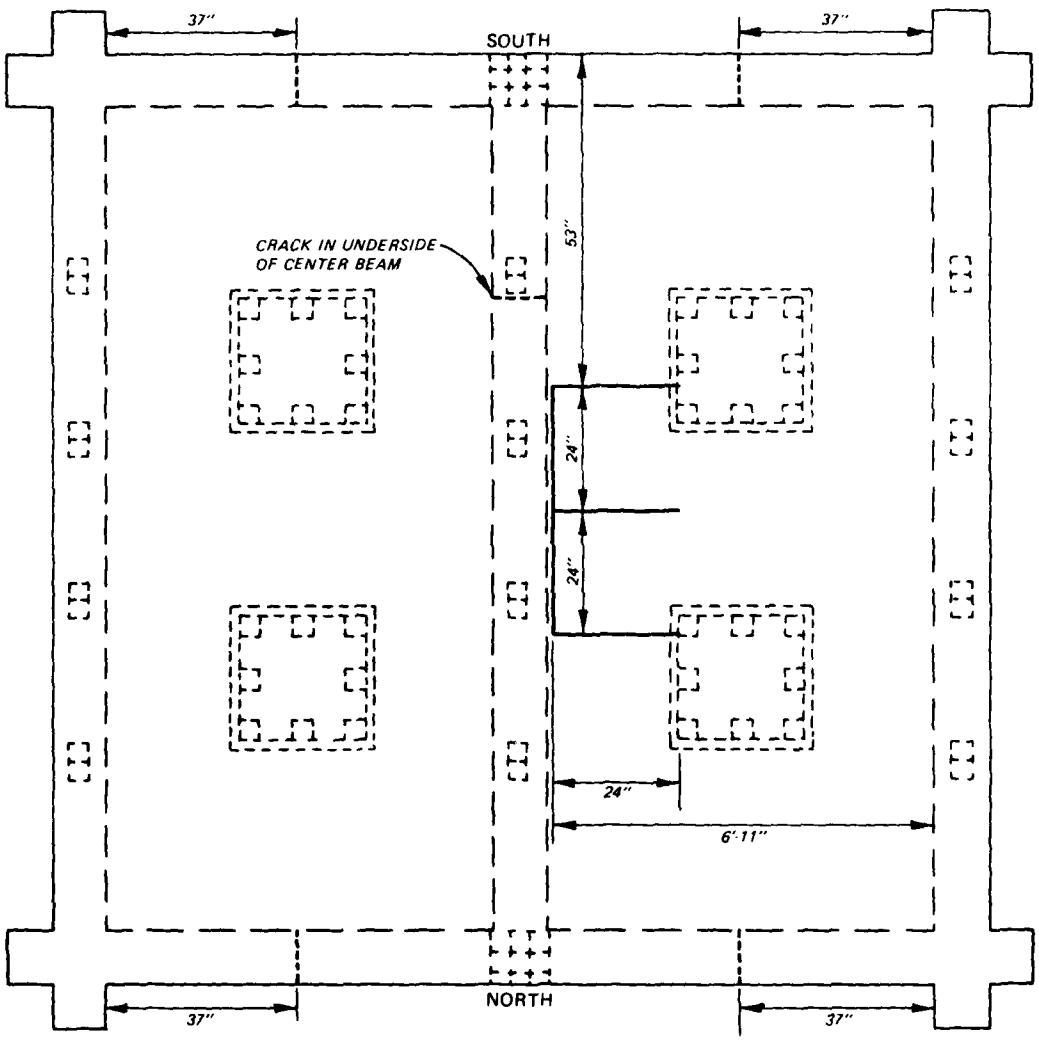


Figure 5.8 Crack patterns in Test Structure 2 after test 2C.

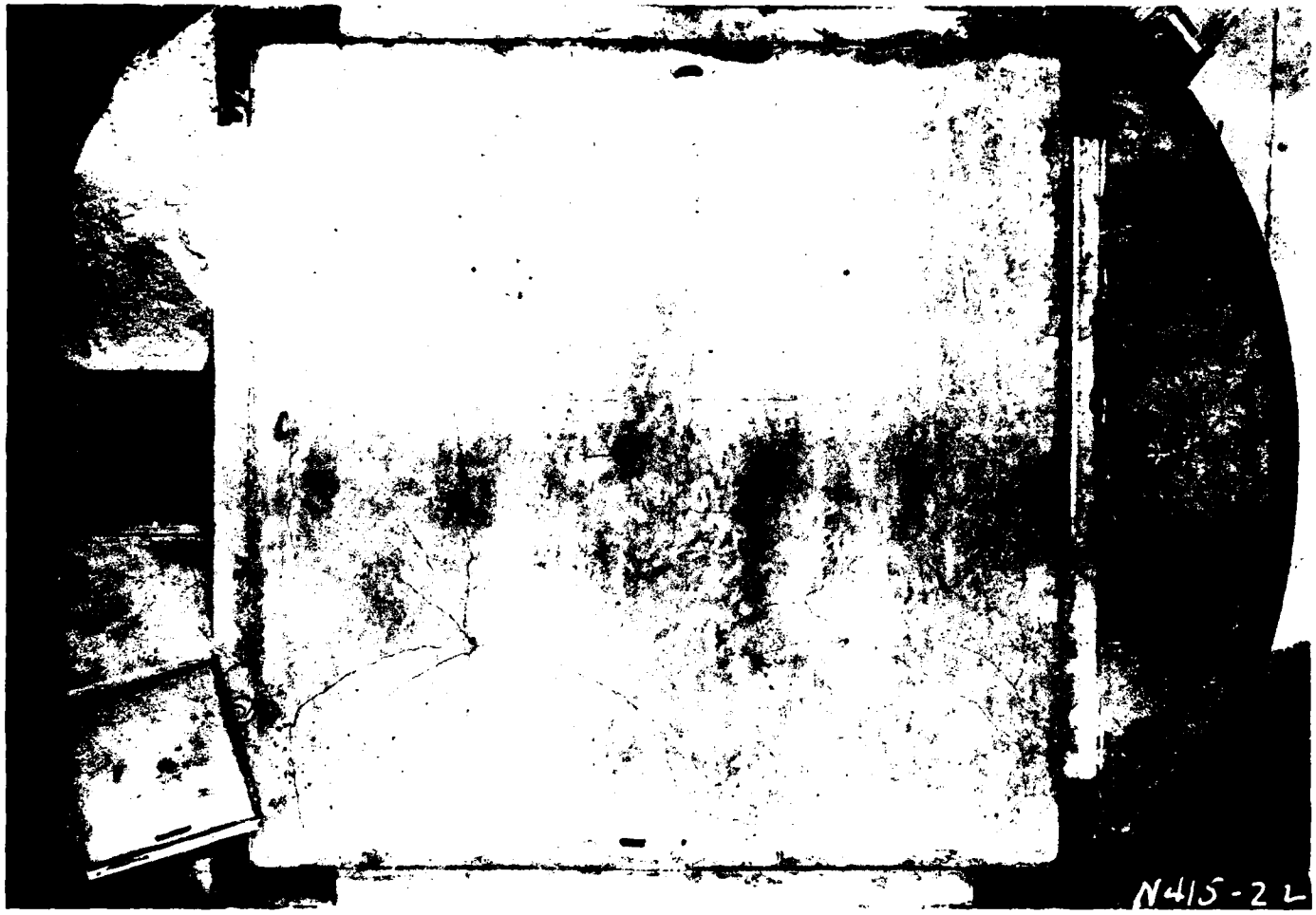


Figure 5.9 Top view of Test Structure 2 after shot SLAB 2D.

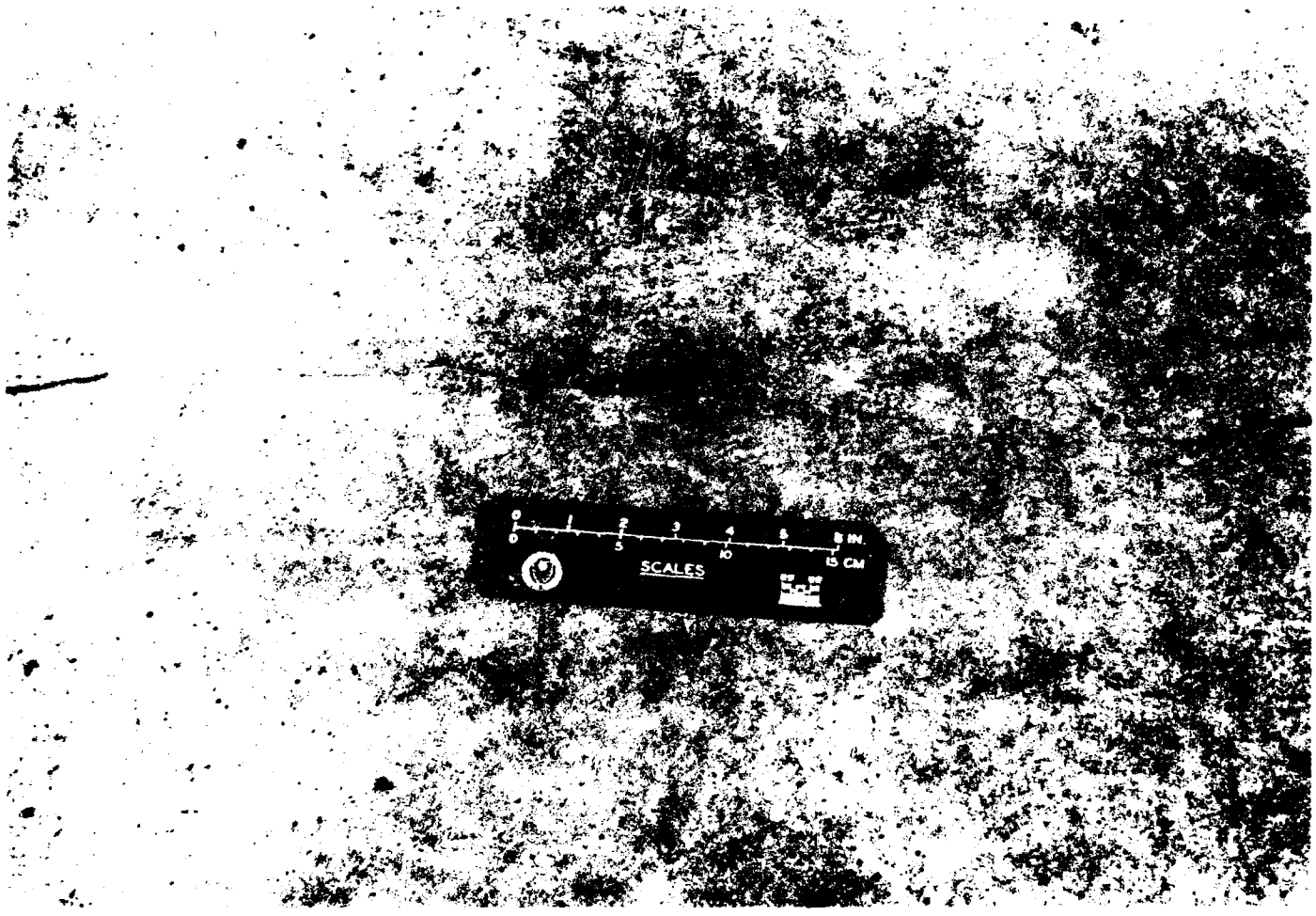


Figure 5.10 A close-up of a crack in Test Structure 2 after shot 2D.

Figure 5.11 Cracks in underside of the east slab panel at the north end.

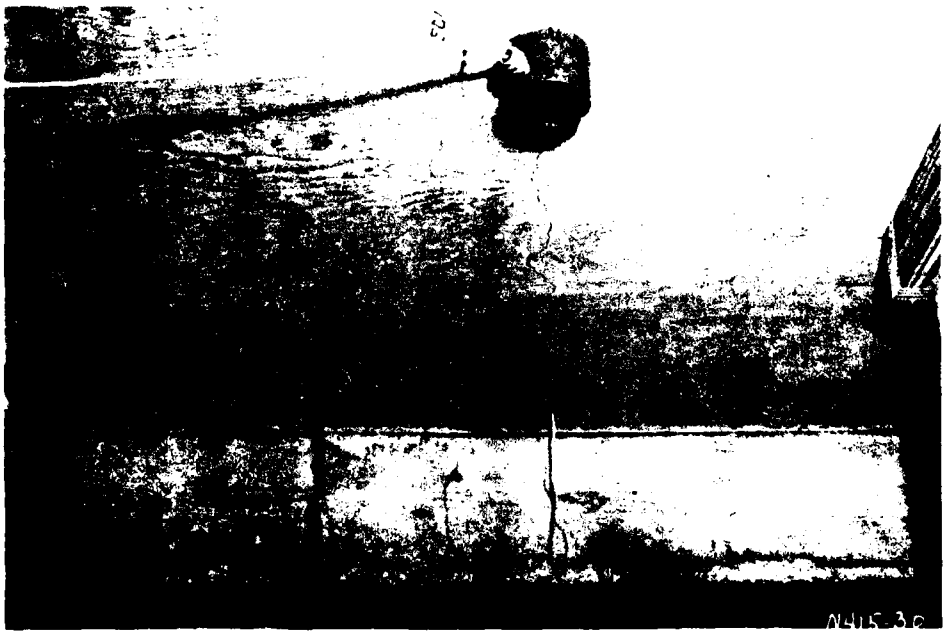


Figure 5.12 Cracks in the underside of the east slab panel at the center.

Figure 5.13 Cracks in the underside of the east slab panel at the south end.

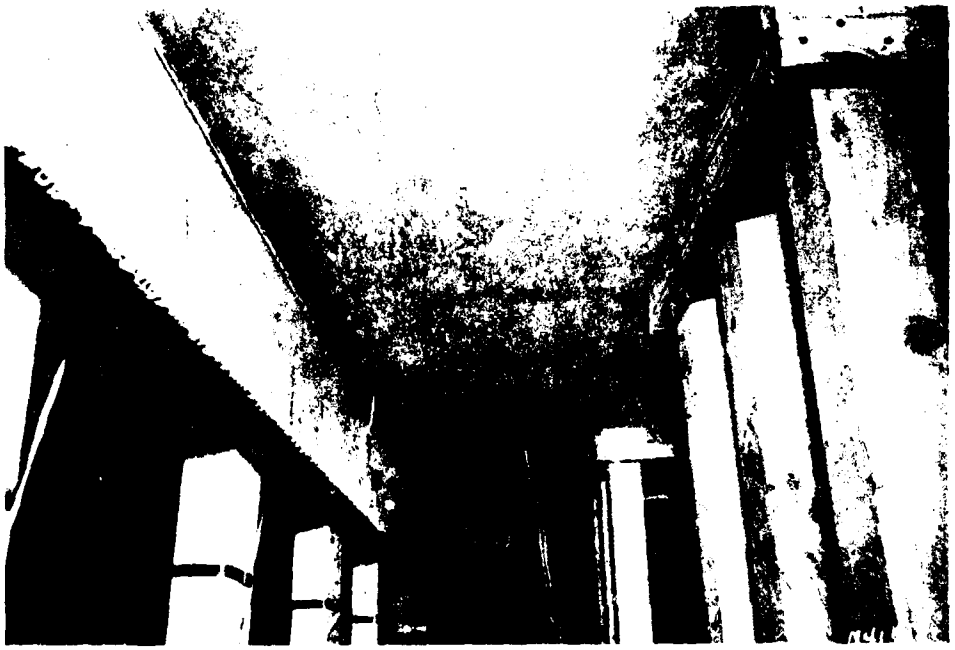


Figure 5.14 Cracks in the underside of the east slab panel and center beam.

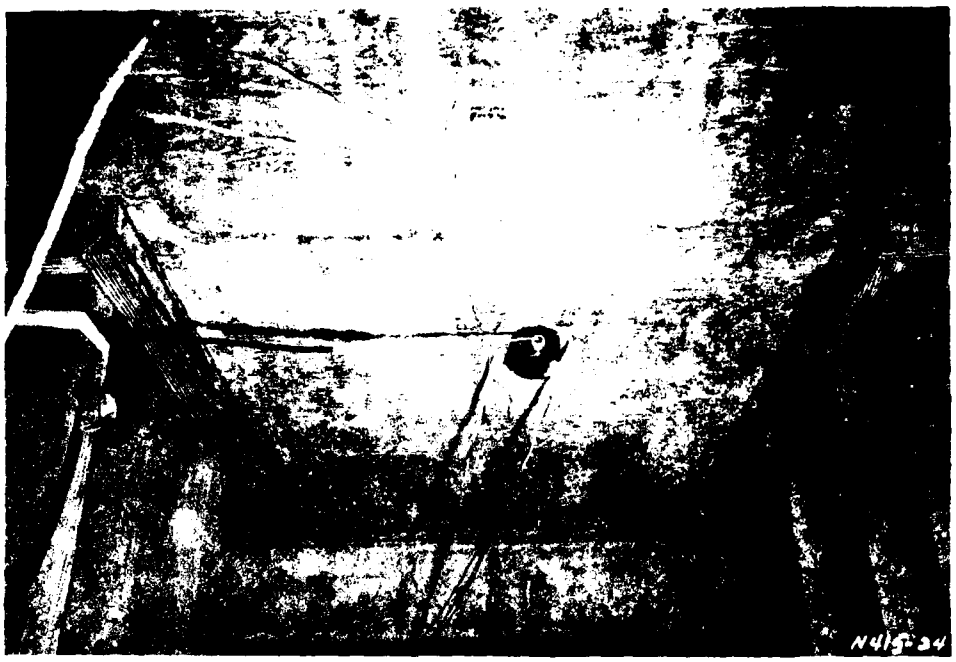


Figure 5.15 Cracks in the underside of the west slab panel at the north end.

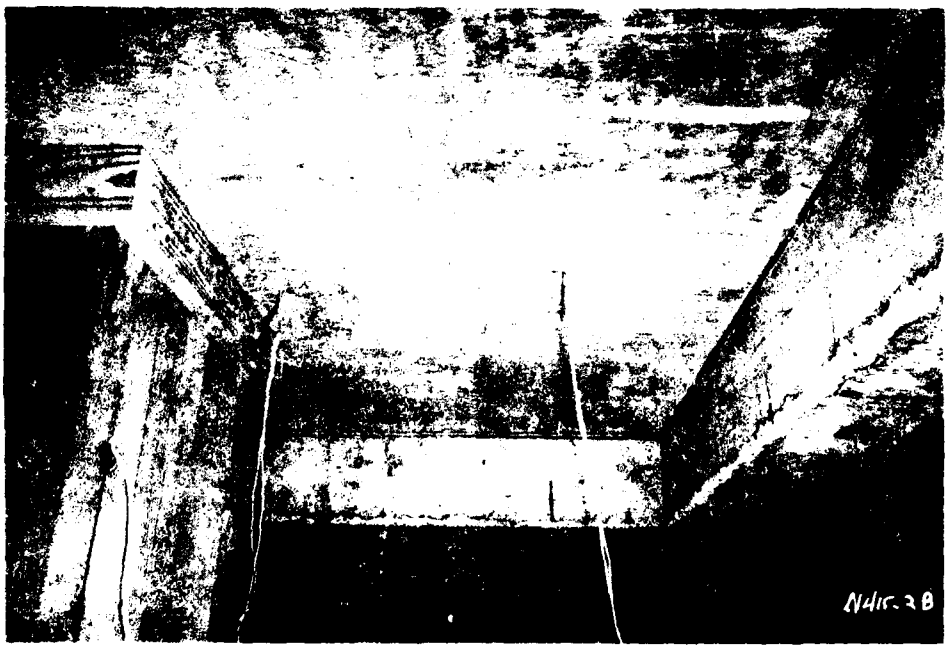


Figure 5.16 Cracks in the underside of the west slab panel at the center.

Figure 5.17 Cracks in the underside of the west slab panel at the south end.

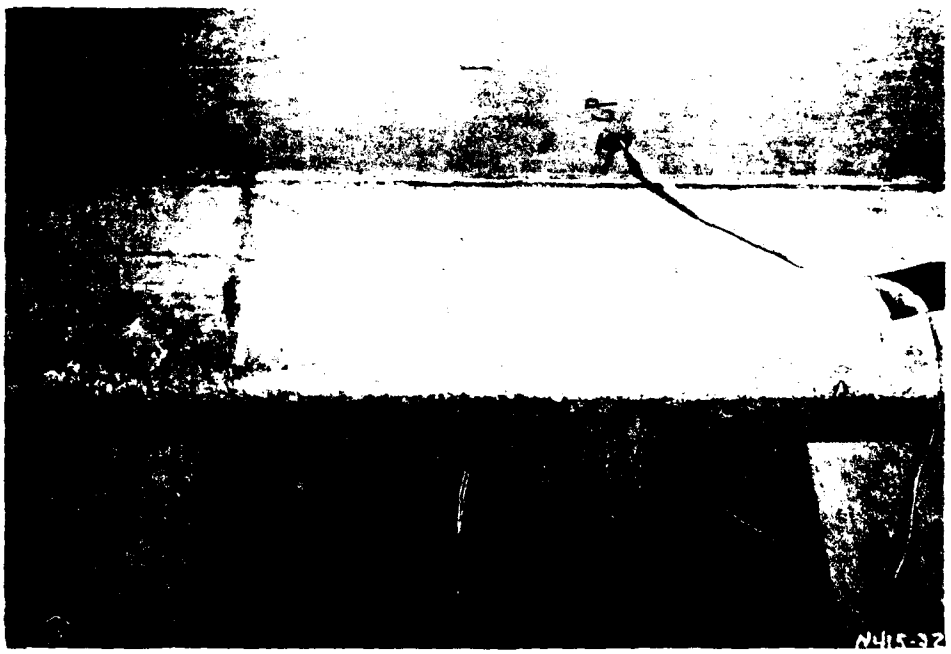


Figure 5.18 Cracks in the underside of the center beam at its center.

Figure 5.19 Wooden column under the center beam after shot SLAB 2D.

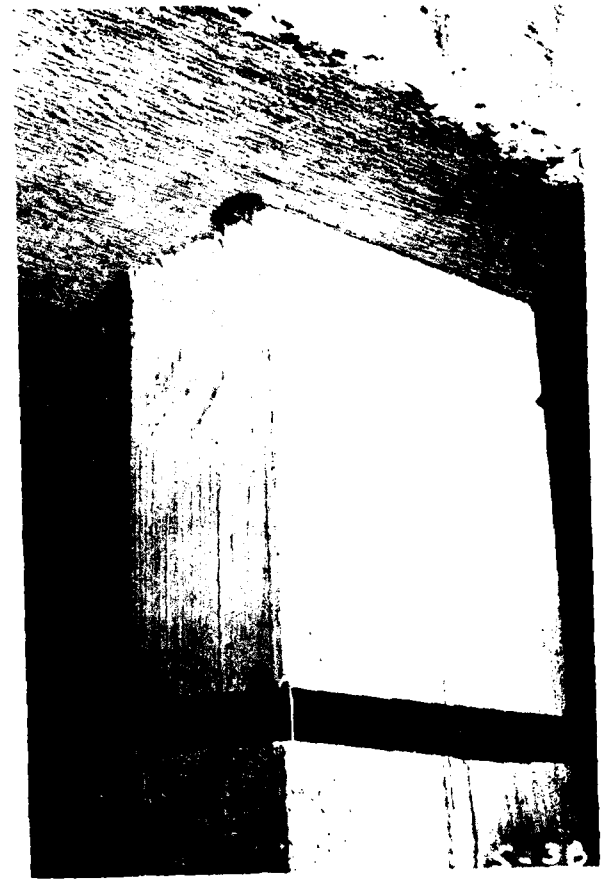


Figure 5.20 Close-up showing the amount of permanent deflection in a column under the center beam after shot SLAB 2D.

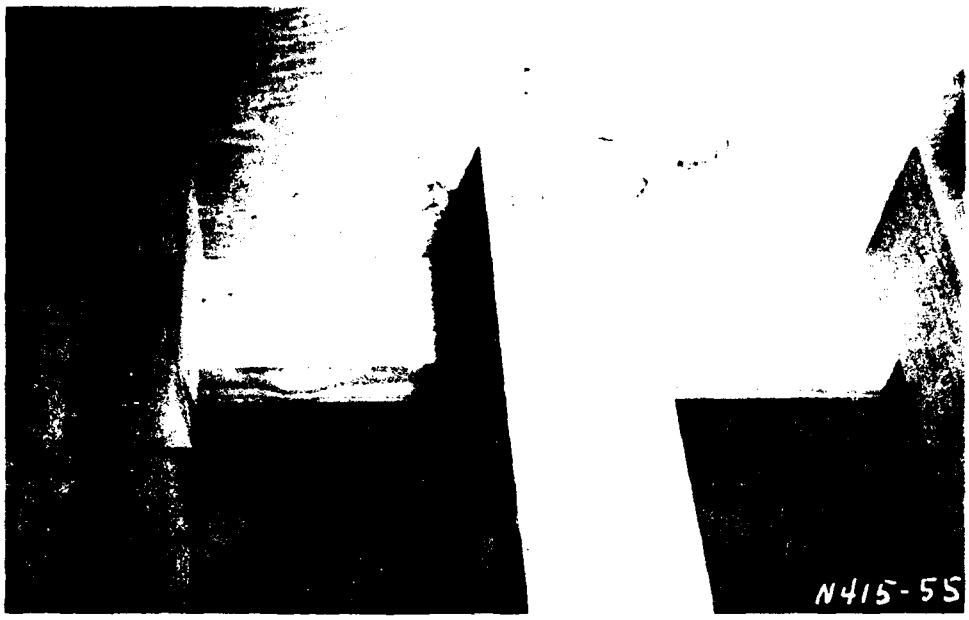


Figure 5.21 Picture of the wooden post over load cell L1 punching through the east slab panel.

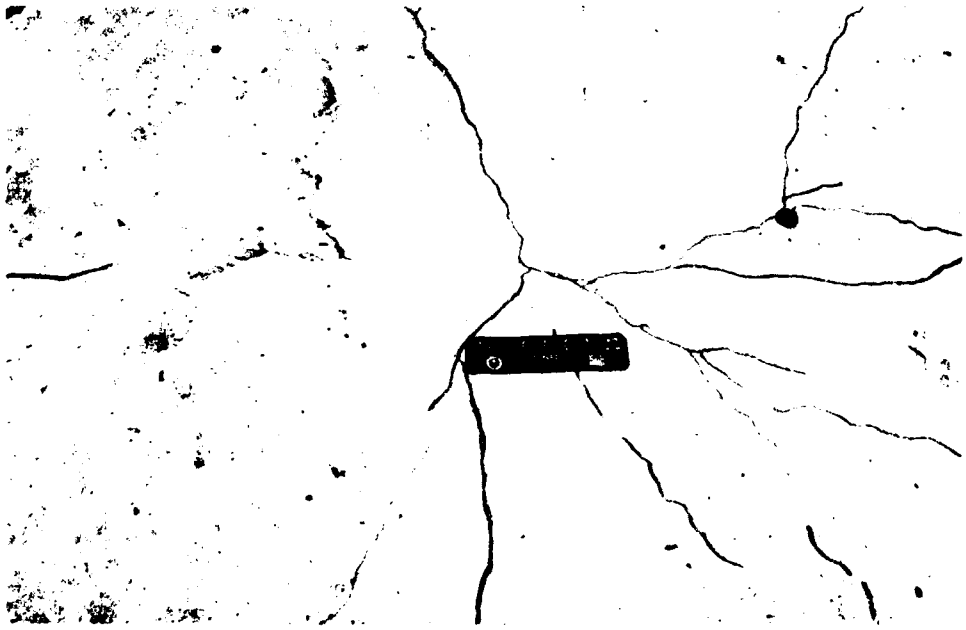


Figure 5.22 The concrete lifting up over the wooden column on load cell L1.

Figure 5.23 A picture of the wooden post, on top of load cell L2, punching through the east slab panel.

Figure 5.24 The concrete lifting up as the wooden post, on top of load cell L2, starts to punch through.

AD-A103 544 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR--ETC(U)
JUL 81 M K MCVAY DCPA01-78-C-0267
UNCLASSIFIED WES/TR/SL-81-4 NL

2 OF 4
AD
A103544

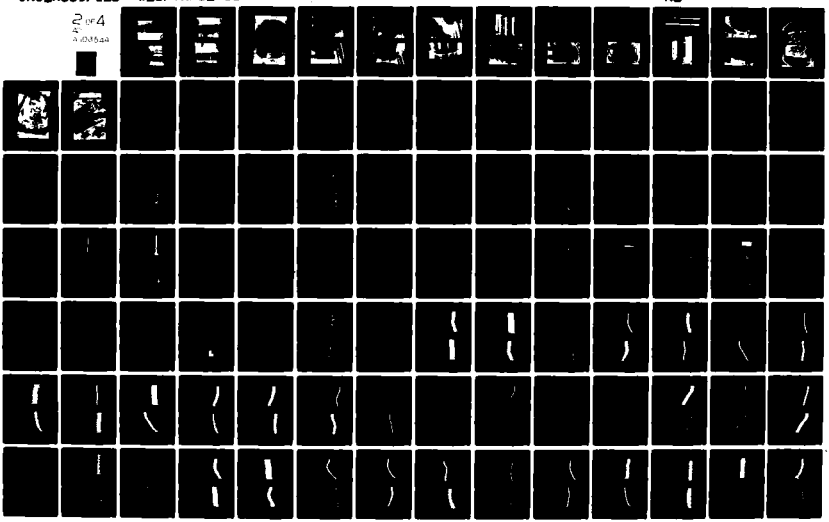


Figure 5.25 A picture of the damaged columns under the center beam after shot SLAB 2E.

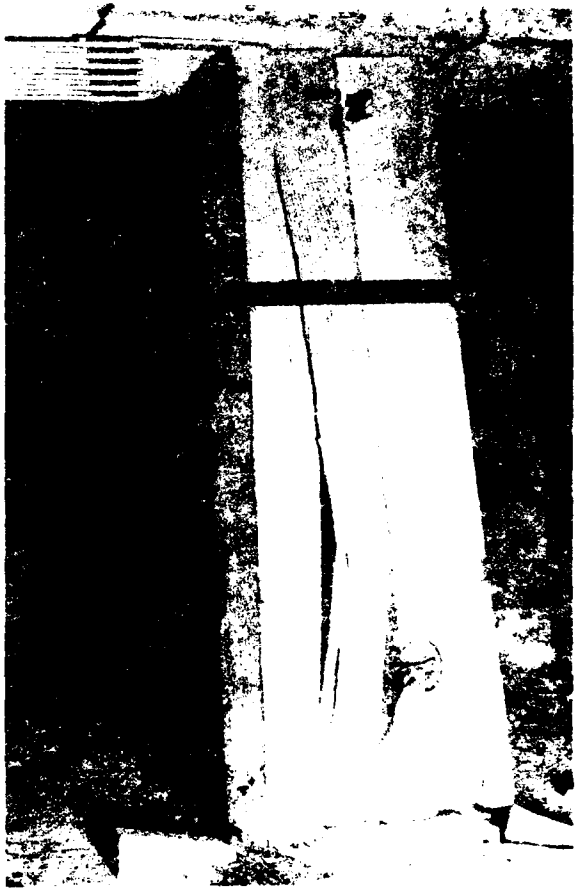


Figure 5.26 A picture of a column under the center beam that split under the load. Also notice the wedges were shattered.

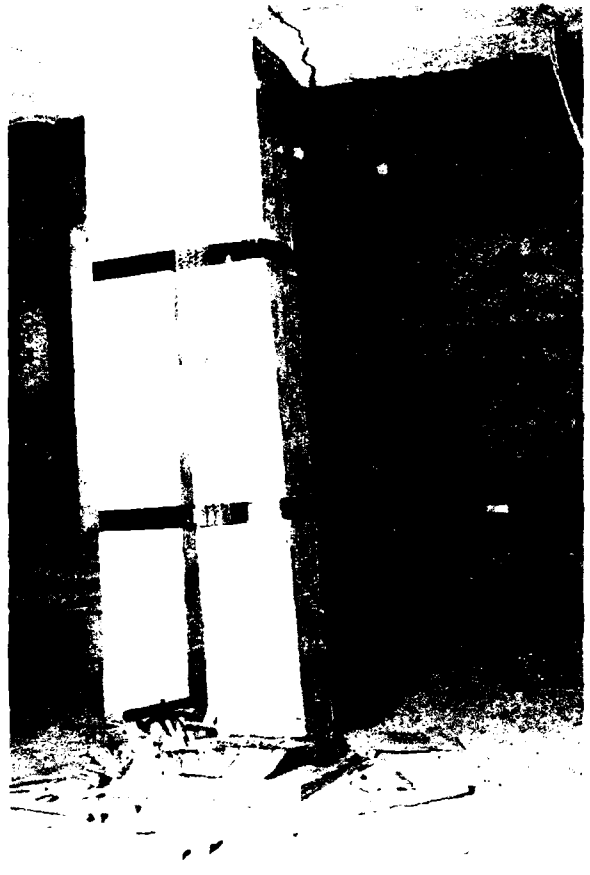


Figure 5.27 A picture of another column under the center beam after shot SLAB 2E.

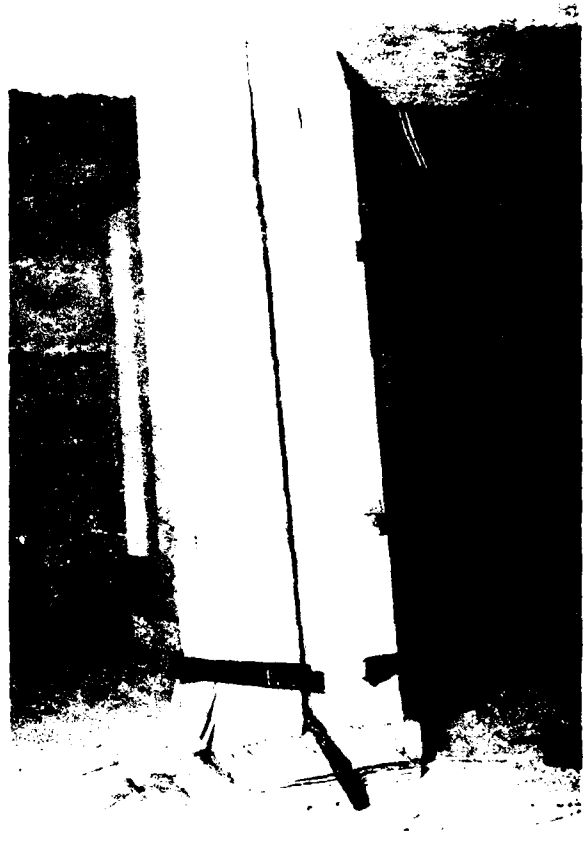


Figure 5.28 A picture of a column that tried to rotate out from under the center beam during shot SLAB 2E.

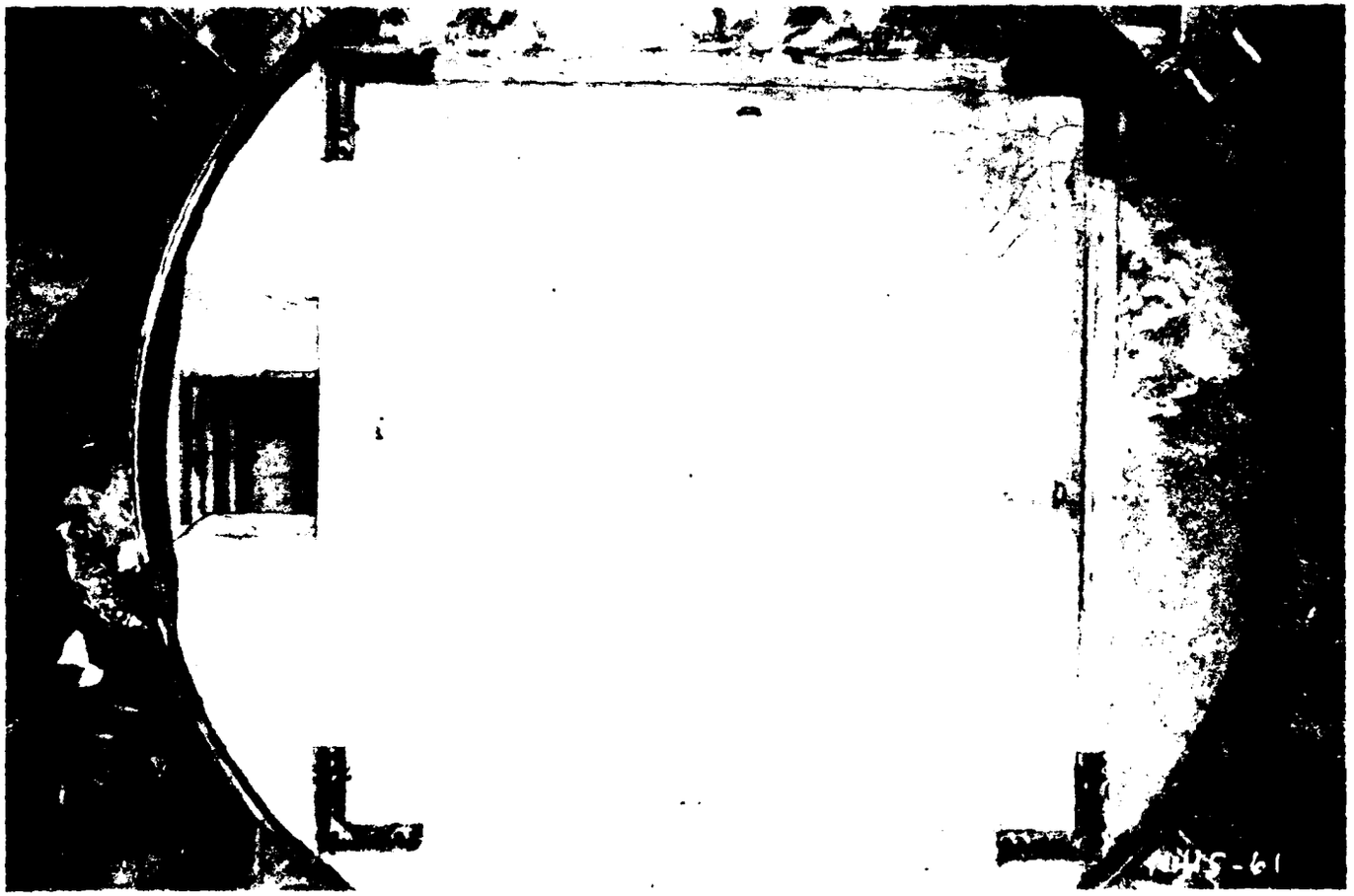


Figure [illegible] The cracks in the top of Test Structure [illegible] after shot SLAB 2[illegible].

Figure 5.30 Cracks in the underside of the east slab panel at the south end.

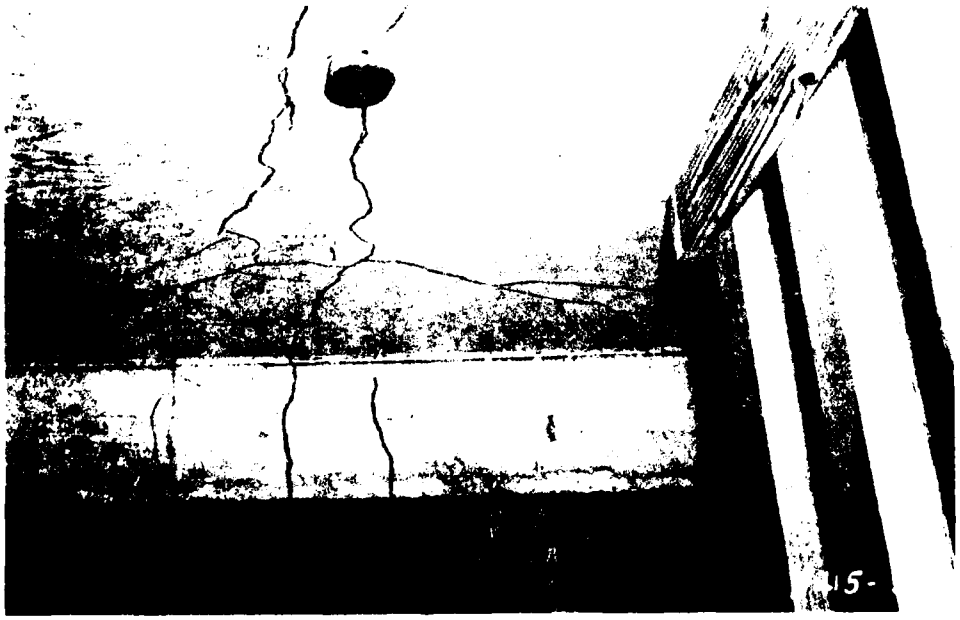


Figure 5.31 Cracks in the underside of the east slab panel at the center, after shot SLAB 2E.

Figure 5.32 Cracks in the underside of the west slab panel at the north end after shot SLAB 2E.

Figure 5.33 Cracks in the underside of the west slab panel at the center section after shot SLAB 2E.

Figure 5.34 Cracks in the underside of the west slab panel at the south end, after shot SLAB 2E.

Figure 5.35 A few wedges crushed under posts in columns upgrading a slab panel, after shot SLAB 2E.

Figure 5.36 Undamaged columns upgrading an outside beam.

Figure 5.37 The permanent deflection of Test Structure 3 between the column rows upgrading the slab panels, after shot SLAB 2E.

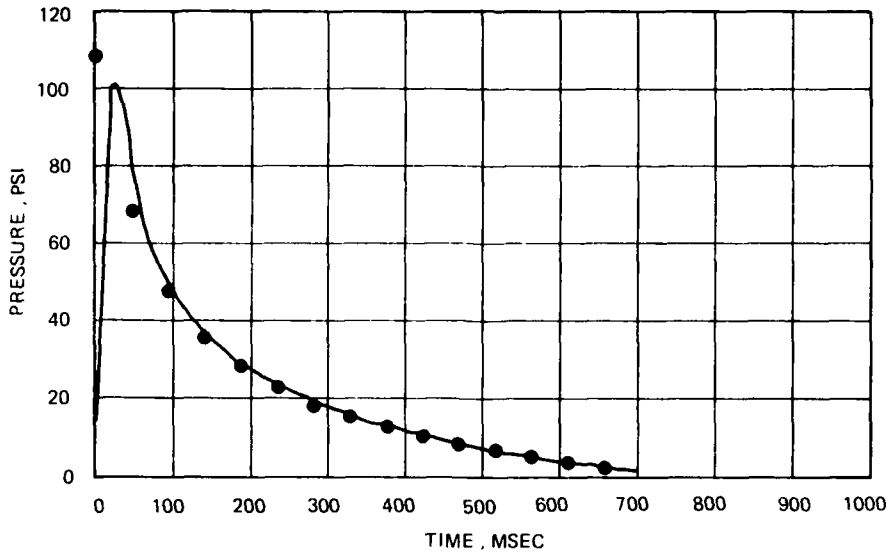


Figure 5.38 Weapon fit of shot SLAB 2E.

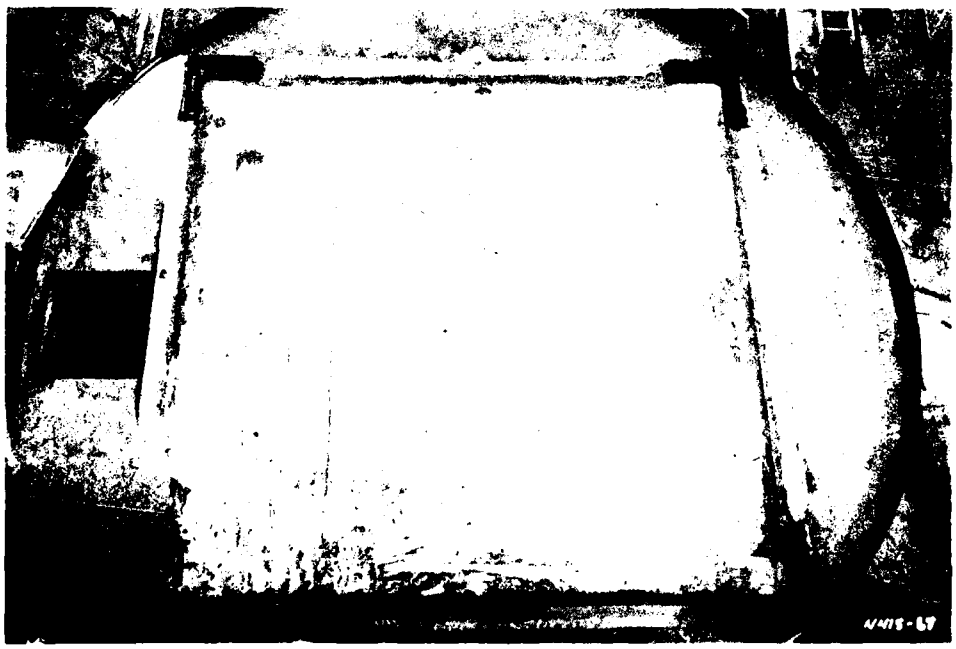


Figure 5.39 Cracks in the top of Test Structure 3 after shot SLAB 3A.

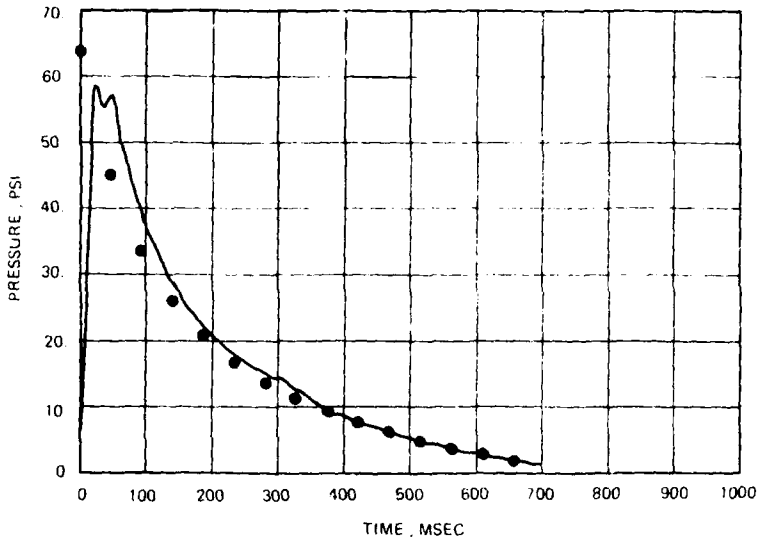


Figure 5.40 Weapon fit of shot SLAB 3B.

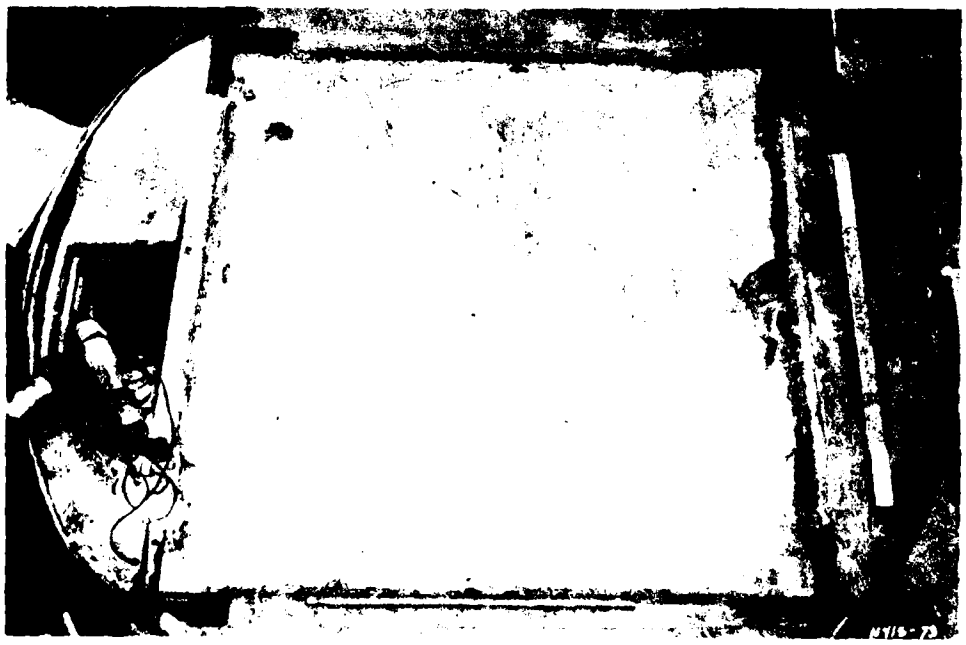


Figure 5.41 The crack pattern in the top of Test Structure 3 after shot SLAB 3B.

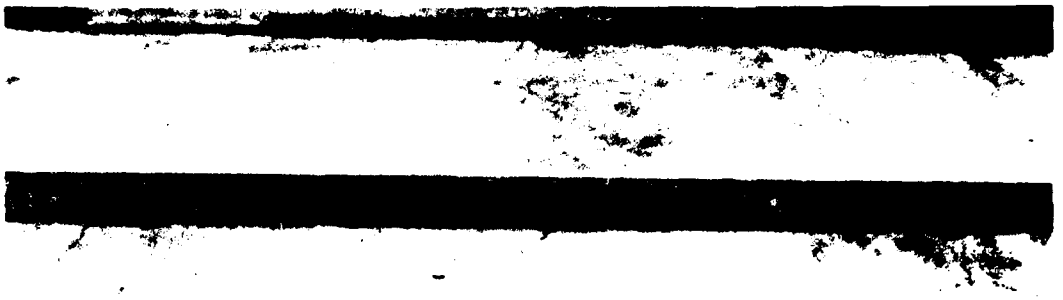


Figure 5.42 The permanent deflection in the floor section between the two rows of steel upgrading beams

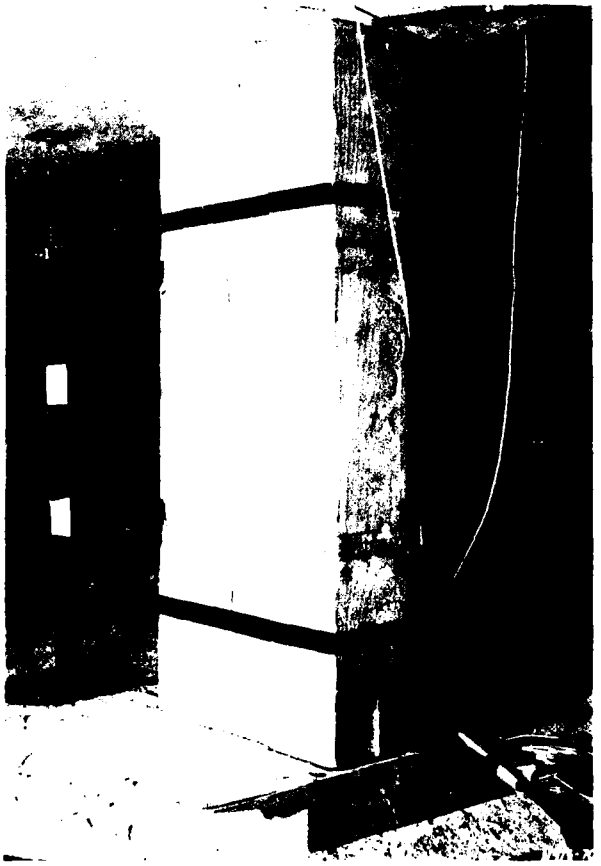


Figure 5.43 A column under the center beam after shot SLAB 3B. (Notice that it was compressed.)

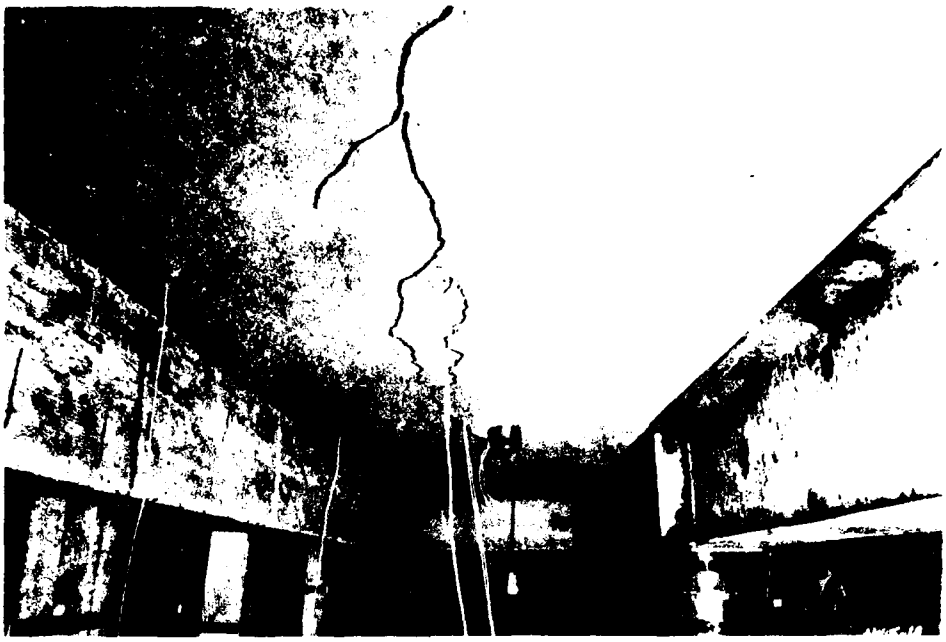


Figure 5.44 Crack pattern in the underside of the east slab panel between the center beam and an upgrading beam.

Figure 5.45 Cracks in the underside of the east slab panel, between the outside beam and the upgrading beam.

Figure 5.46 Test Structure 3 after shot [illegible] 30.

Figure 5.47 A close-up of the west slab panel after shot SLAB 3C. (Notice portions of the slab are still hanging on.)

Figure 5.48 Close-up of the edges of the slab panel.

Figure 5.49 A picture of a buckled upgrading beam and broken and bent bolts.

CHAPTER 6

DATA EVALUATION

6.1 TEST SERIES 1

The results of the test SLAB 1A indicate that it would not have taken a much higher overpressure to fail the slab panels in flexure. This test is further proof that most floor sections are much stronger than predicted by conventional analysis procedures. However, the load capacity of 16 psi predicted by the SRI analysis appears to be just slightly under the actual load capacity.

Unfortunately, test SLAB 1B did not apply the proper load to the floor section. The fact that the structure failed so quickly indicates that the load was much greater than the load capacity. The test did demonstrate how severe the failure of a floor system would be if it received an overpressure much larger than its load capacity. The floor collapsed on almost the entire living space beneath it.

6.2 TEST SERIES 2

Test SLAB 2A put an average peak overpressure of 16.3 psi on Structure 2 upgraded with wooden posts. This overpressure is near the predicted load capacity of the floor system without any upgrading, but it caused no damage to this upgraded structure. The strains and deflections in test SLAB 1A were several times greater than those in test SLAB 2A. The deflection histories indicate that the floor section deflected the least around the edges, while the center beam deflected the most. The floor section was bent in such a manner that the slab panels were under positive moments at the face of the center beam, as strain gage pairs SS3 and SS6 show. Strain gage pairs SS1, SS4, and SS7 indicate that the wooden columns compressed enough to cause the slab to be under extremely small positive moments over them. The wooden posts on top of the wooden wedges were very compressible upgrading supports. The compressible supports caused the load on the floor section to be

distributed proportionally to the stronger members and also acted as shock absorbers under the dynamic load.

In test SLAB 2B, the floor section started bending a little more between the upgrading supports. The floor section still deflected the least around the edges, and the center beam deflected the most again, but this time the slab panels deflected more. The wooden supports seemed to have stiffened somewhat after being compressed and to have caused the slab panels to bend in a complex mode between the supports. The cracks in the slab were not very serious damage. The small strains and deflections indicated that the floor section could withstand a larger overpressure.

Test SLAB 2C applied an overpressure similar to the one that collapsed Structure 1. Although the floor section suffered a few more minor cracks, the floor section suffered no serious damage.

Test SLAB 2D applied an overpressure large enough to cause the floor section to suffer some moderate damage. Most of the crack patterns were in the proximity of the expected yield-line cracks. The floor section bent more like the predicted behavior in design than that in test SLAB 2A. The cracks in the topside of the floor section (except for the "E" shaped pattern) all formed near supports in predicted high negative moment areas. The cracks in the underside of the floor section formed between the supports in regions of predicted high positive moments. Although the floor section was cracked in many places, it was still a sound structure.

Test SLAB 2E applied just the right amount of overpressure on the floor section to start failing it. As described, the two wooden posts on top of the load cells started punching through, and the upgrading columns under the center beam failed. The two wooden posts on top of the load cells were not as compressible as the other wooden posts. The other wooden posts were longer and on top of wooden wedges. Since wood is weakest perpendicular to the grain, the wedges compressed the greatest percentage of the two pieces of wood. Had the load cells not been present, those two particular wooden posts probably would not have tried to punch through. The wooden posts probably would have all acted

as a composite column like those under the other slab panel. The outer beams and girders were upgraded as if they were in a continuous floor but only received half of the load of actual members. Therefore, they did not respond as they would in a continuous floor. However, the portion of the floor section between the midspans of the two slab panels did receive loads similar to those in a continuous floor. The wooden posts under the center beam were loaded above their compressive strengths and started splitting, while the wooden posts under the slab panels had not started splitting. Some extra posts under the center beam would have made the strength of the upgrading system more uniform.

The actual load of 112.8 psi was much greater than the design load capacity of 54.4 psi. This illustrates how conservative some of the assumptions and analysis procedures were. The punching shear was calculated as the critical strength of the floor section at 54.4 psi. However, the formulas for punching strength are based upon work with concrete columns or steel members cast in concrete columns. These columns are much stiffer and are usually cast monolithically with the slabs, along with the proper reinforcing steel. Also, the assumptions of the shear strengths of the concrete are very conservative. The predicted flexural resistance of the slab also seems to be too high.

This wooden post upgrading method increased the load capacity of the given floor section 7.05 times. The upgrading system could be modified so the composite columns under the slab panels were not as big but would still increase the load capacity of the floor section above 50 psi. This indicates that a similar upgrading system can increase the load capacity of most one-way floor systems above 50 psi.

6-3 TEST SERIES 3.

Test SLAB 3A cracked the slab panels directly over the upgrading steel beams. This was no surprise since there was no reinforcing steel in this region of the slab to resist a negative moment. The slab panels did not crack over the wooden columns under a similar overpressure in test SLAB 2B. The difference was the fact that the wooden posts compressed and allowed the slab to deflect and release the

negative moment, whereas the steel beams on top of the steel columns were much more rigid. The wooden posts allowed the center beam to deflect while the steel beams did not deflect very much at all. This caused the floor section to sag between the two rows of steel beams and diverted more load to the steel beams than the center beam. The cracks changed the slab panels into two propped cantilevers tied together end-to-end. Essentially, this load caused no structural damage.

Under the overpressure of test SLAB 3B, the floor section started cracking near the expected yield lines. Two additional cracks were found in the middle of the slab panels perpendicular to the upgrading beams. These were caused by the presence of the rigid steel columns under the center of the slab panels. The slab was stiffest at that point. Wooden posts under the center beam allowed it to deflect even more, which caused the floor section to sag more between the steel beams and thus divert more of the load to the steel beams.

Test SLAB 3C failed the upgraded floor section, but the slow failure indicated that the overpressure did not exceed the load capacity by much. The data did not clearly indicate what caused the structures to fail. The loads on the beams and bolts were well above what they were designed for. The flexural and shear strengths of the slab panels were predicted to be about 84.2 psi and 50 psi, respectively. Either the buckled steel beams, the broken bolts, the flexural and shear strengths of the slab panels, or a combination of the three could have caused the failure. The steel beams and bolts could have been damaged during collapse instead of being the cause of it.

Thus, the steel beam upgrading method increased the load capacity of the given floor section to somewhere between 63.0 and 92.2 psi. It is believed that the load capacity is about 80 to 88 psi since 92.2 psi slowly failed the slab panels. Thus, the load capacity of the slab section was increased by about five times the original strength. This exceeds the design strength of 50 psi significantly, which illustrates that some of the assumptions and analysis procedures were conservative. The predicted flexural strength of the slab panels may not be too far off, but the shear strength analyses are low. The steel

beams and bolts can be redesigned to withstand larger loads to prevent them from being the cause of failure.

6.4 COMPARISONS

The wooden post upgrading method withstood a larger overpressure than the steel beam upgrading method, but this does not necessarily mean it is superior. The span lengths in the wooden post upgrading method were shorter than those in the steel beam upgrading method, so naturally the flexural strength was greater. Also, the concrete strength of Structure 2 was stronger than that of Structure 3. The design of the wooden post upgrading method was much more conservative than that of the steel beam upgrading method, because the physical properties of wood are not uniform and conservatively low values were assumed. Once the strengths of the upgraded floor systems can be analyzed better, the upgrading system can be designed to compliment these strengths better. An advantage of the steel beam upgrading method is that it takes up less floor space in the shelter than the wooden post upgrading method. Advantages of the wooden post upgrading method are that it is more flexible and distributes the load well and that the negative moments above the upgrading posts (usually where there is no negative reinforcing) are relieved when the posts deflect. Either method will work well on actual one-way R/C slab floor systems.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS

These conclusions are based on the results of dynamic tests on sections out of a sample one-way R/C floor system. One section was not upgraded for a baseline comparison, a second section was upgraded with wooden posts, and a third section was upgraded with steel beams and pipes. Each section was tested with increasingly larger overpressures until it collapsed or appreciable damage occurred.

7.1.1 Increased Load Capacity Verified

These tests verified that a properly designed upgrading system can increase the load capacity of a one-way R/C floor system. The load capacity of the sample section from a one-way R/C floor system was increased by about 5.0 to 7.0 times with the steel beam and wooden post upgrading methods, respectively. The load capacities of approximately 80 to 90 psi and 113 psi achieved by the steel beam and wooden post upgrading methods, respectively, were much greater than the desired minimum of 50 psi.

7.1.2 Readily Available Materials and Simple Construction

These tests also illustrate that sound upgrading systems can be made of readily available materials with only simple construction skills. There are several other types of material that could be used that would work just as well as those used in this study. However, the 4- by 4-inch timbers, small steel beams, and steel pipes are some of the more readily available and easy to handle materials. The wooden post upgrading system was the easier of the two to assemble.

7.1.3 Design Procedure

The design procedures described in the text can be used to safely design an upgrading system for a one-way R/C floor system.

However, there were many conservative assumptions and analyses used in the design procedures, so the upgraded systems will be much stronger than predicted. If plans of a given one-way R/C floor system are not available, reasonable values can be determined by using the proposed method for estimating the characteristics of the floor system.

7.1.4 Modes of Failure

The floor section upgraded by the wooden post method failed by punching shear in the slab panels and crushing of the wooden posts under the center beam. However, it is believed that the two wooden posts on top of the load cells were stiffer than the rest of the wooden posts on top of wooden wedges, and this caused an excessive amount of shear stress around these two posts.

It was concluded that the mode of failure for the floor section supported by steel beams could have been caused by buckling of the steel beams, failure of the steel bolts, or flexural failure of the slabs. Whatever the cause, the failure was catastrophic.

7.1.5 Comparison of Upgrading Systems

The wooden post upgrading system was not necessarily better than the steel beam system, even though the former withstood a larger overpressure. The span lengths of the slab panels, concrete strengths, and strengths of upgrading material above design strengths were different in the two structures. The steel beam upgrading method did not take up as much floor space as the wooden post method. The wooden post upgrading system was easier to install and formed a flexible support system that distributed the load to the stiffer members. Both methods represent viable upgrading techniques.

7.2 RECOMMENDATIONS

7.2.1 Application

The two methods used to upgrade the floor sections in this study are recommended for use on any one-way R/C floor system. Although

the upgrading techniques were tested only on a beam-girder type floor system, the other types of one-way R/C floor systems can be safely upgraded using the same design procedures. The wooden post upgrading system is recommended as the easier one to construct, but the steel beam system is recommended if more floor space is needed.

7.2.2 Improvements

A method for estimating the characteristics of a prospective shelter was given in Chapter 2 for use when a set of construction plans is not available. These characteristics include strength of materials, reinforcing steel ratios, bar detailing, etc. A method was also given for estimating the physical properties of the upgrading supports when they are not known. In most cases, construction plans and physical properties of upgrading materials probably will not be available. It is recommended that such methods as these be refined and improved for quick and easy use in designing an upgraded shelter. Methods using flowcharts might be the best.

If more floor space is desired in the wooden post upgraded shelter, the design could most likely be modified to provide the space and still increase the load capacity above 50 psi.

The failure of the steel beam upgraded system was catastrophic. It is not clear whether buckling of the steel beams or bending of the bolts was the cause of failure, but it is recommended that the steel beams and bolts be slightly overdesigned to ensure that they are not the cause of failure.

Another recommended improvement in both upgrading designs, if optimum strength is desired, is to double the size of the upgrading columns under the existing columns. The upgrading columns under the center beams were compressed more than the upgrading supports under the slab panels in both tests series and split in the test of the wooden post upgraded system.

7.2.3 Needed Studies.

The results of this study illustrate that some of the assumptions and analyses that were used were very conservative. The upgrading designs were safe but were much stronger than predicted. It is recommended that studies be conducted on the subjects of dynamic punching shear, punching shear of concrete over wooden columns, dynamic shear strength of concrete, dynamic properties of wood, flexible supports, and flexural strength of short-spanned, one-way slabs.

The other main category of floor system besides a one-way floor system is a two-way floor system. There are unique characteristics of two-way floor systems that should be considered when upgrading them. A study to develop and verify upgrading systems for two-way floor systems is also recommended.

REFERENCES

1. Defense Civil Preparedness Agency; "High Risk Areas for Civil Preparedness Nuclear Defense Planning Purposes"; Technical Report 82, April 1975.

2. C. K. Wiehle, and J. L. Backholt; "Blast Response of Five NFSS Buildings"; October 1971 (AD-738-547); Stanford Research Institute Technical Report for Office of Civil Defense.

3. C. K. Wiehle; "All-Effects Shelter Survey System, Summary of Dynamic Analysis fo 25 NFSS Buildings"; March 1973; Stanford Research Institute Report for Defense Civil Preparedness Agency.

4. _____; "Summary of Dynamic Analysis of Exterior Walls and Floor Systems of 50 NFSS Building"; June 1974; Stanford Research Institute Report for Defense Civil Preparedness Agency.

5. D. F. Tolman, R. O. Lyday, and E. L. Hill; "Statistical Classification Report"; Contract Number DAHC20-72-C-0277, December 1973; Research Triangle Institute for Defense Civil Preparedness Agency.

6. D. R. Denton; "A Dynamic Ultimate Strength Study of Simply Supported Two-Way Reinforced Concrete Slabs"; Technical Report No. 1-789, July 1967; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

7. W. M. Brown, M. S. Black,; "Dynamic Strength Study of Small, Fixed-Edge, Longitudinally Restrained, Two-Way Reinforced Concrete Slabs"; Technical Report N-73-8, December 1973; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

8. M. E. Criswell; "Design and Testing of a Blast-Resistant Reinforced Concrete Slab System"; Technical Report N-72-10, November 1972; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

9. W. L. Huff; "Static and Dynamic Test of Small Upgraded One-Way Slabs" (In Preparation); U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

10. _____; "Test Devices Blast Load Generator Facility"; Miscellaneous Paper N-69-1, April 1969; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

11. American Concrete Institute; Building Code Requirements for Reinforced Concrete; ACI 318-63, 1963; Detroit, MI.

12. _____; Building Code Requirements for Reinforced Concrete; ACI 318-71, 1971; Detroit, MI.

13. _____; Building Code Requirements for Reinforced Concrete; ACI 318-77, 1977; Detroit, MI.

14. Concrete Reinforcing Steel Institute; CRSI Handbook Ultimate Strength Design (1963 ACI Code); 2nd Edition, 1970; Chicago, IL.

15. Concrete Reinforcing Steel Institute; CRSI Handbook Based upon the 1971 ACI Building Code; 1st Edition, 1972; Chicago, IL.

16. _____; CRSI Handbook Based upon the 1977 ACI Building Code; 3rd Edition, 1978; Chicago, IL.

17. International Conference of Building Officials; Uniform Building Code; 1961 Edition; Pasadena, CA.

18. _____; Uniform Building Code; 1964 Edition; Pasadena, CA.

19. _____; Uniform Building Code; 1970 Edition; Pasadena, CA.

20. _____; Uniform Building Code; 1973 Edition; Pasadena, CA.

21. American Concrete Institute; Manual of Standard Practice for Detailing Reinforced Concrete Structures; ACI Standard 315-65, 4th Edition, February 1965; Detroit, MI.

22. G. A. Winter, A. A. Nilson; Design of Concrete Structures; 8th Edition, 1973; McGraw-Hill, NY.

23. J. M. Biggs; Introduction to Structural Dynamics; 1964; McGraw-Hill, NY.

24. W. A. Keenan, "Strength and Behavior of Laced Reinforced Concrete Slabs Under Static and Dynamic Load"; Technical Report R-620, April 1969; Naval Civil Engineering Laboratory for U. S. Department of the Army.

25. C. Y. Yang, S. A. Nuccitelli, and C. N. Tang; "The Dynamic Behavior of Reinforced Concrete Columns"; Contract No. DA-49-146-XZ-015; Massachusetts Institute of Technology, Department of Civil and Sanitary Engineering for Defense Atomic Support Agency.

26. C. K. Wiehle; "Evaluation of Existing Structures"; December 1974; Stanford Research Institute Report for U. S. Defense Civil Preparedness Agency.

27. C. K. Wiehle, J. R. Rempel; "Simplified Blast Prediction Method for NSS Building Elements"; September 1977; Stanford Research Institute Technical Report for U. S. Defense Civil Preparedness Agency.

28. J. C. McCormac; Structural Steel Design; 2nd Edition, 1971; Intext Educational Publishers.

29. American Institute of Timber Construction; Timber Construction Manual; Second Edition, 1974; John Wiley and Sons, NY.

30. L. F. Kahn, R. D. Hanson; "Inelastic Cycles of Axially Loaded Steel Members"; ASCE Structural Division Journal; Vol 102, No. ST5, May 1976; pp 947-959.

31. A. E. Long; "Punching Failure of Slabs--Transfer of Moment and Shear"; American Society of Civil Engineers Journal of the Standard Division; Vol 99, No. ST4, April 1973; pp 665-685.

32. _____; "A Two-Phase Approach to the Prediction of the Punching Strength of Slabs"; American Concrete Institute Journal; February 1975; Title No. 72-5.

33. B. E. Hewitt, B. deV. Batchelor; "Punching Shear Strength of Restrained Slabs"; American Society of Civil Engineers Journal of Structural Division; Vol 101, ST9, September 1975; pp 1837-1853.

34. M. E. Criswell; "Strength and Behavior of Reinforced Concrete Slab-Column Connections Subjected to Static and Dynamic Loadings"; Technical Report N-70-1, December 1970; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

35. A. L. Murphy, C. K. Wiehle, and E. E. Pickering; "Upgrading Basements for Combined Nuclear Weapons Effects: Expedient Options"; Contract No. DCPA01-75-C-0302, May 1976; Stanford Research Institute Report for U. S. Defense Civil Preparedness Agency.

36. R. E. Crawford, C. J. Higgins, and E. H. Bultmann; "The Air Force Manual for Design and Analysis of Hardened Structures"; Technical Report No. AFWL-TR-74-102, October 1974; Air Force Weapons Laboratory, Kirtland Air Force Base, NM.

37. American Institute of Steel Construction; Manual of Steel Construction; 7th Edition, June 1976; NY.

APPENDIX A

STRUCTURE 1 TEST DATA

Data from the tests on the one-way reinforced concrete floor system (Structure 1) are presented in this appendix. Labels on the plots indicate the following:

Line 1: Test name

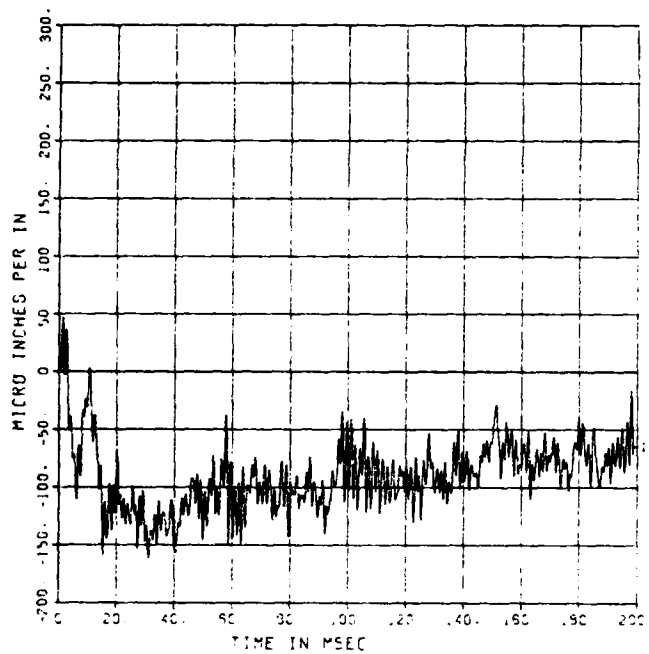
Line 2: Gage symbol and location (see Figure 4.1)

Line 3: Digitizing rate and calibration of gage

Line 4: Filter (if any)

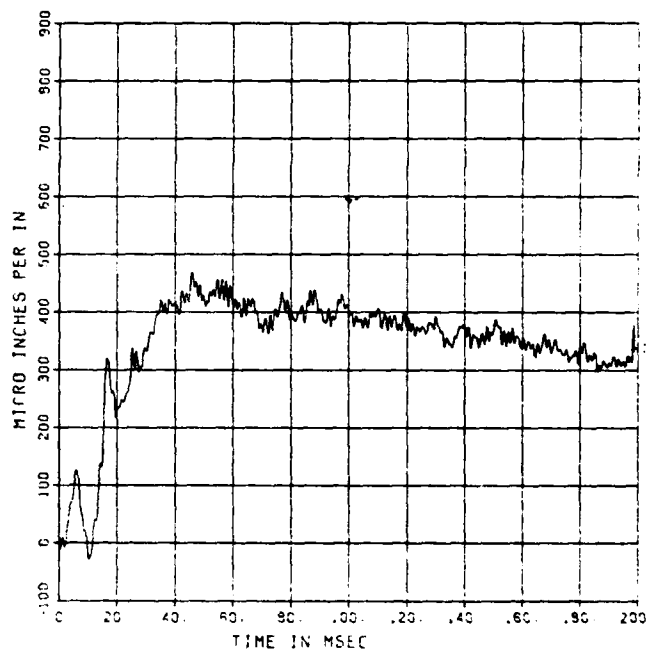
Line 5: Bookkeeping data

SLAB 1-A, TEST 1
SS1 TOP
20000 HZ CAL 1.400
F4
04/17/79 8227 1 R0552



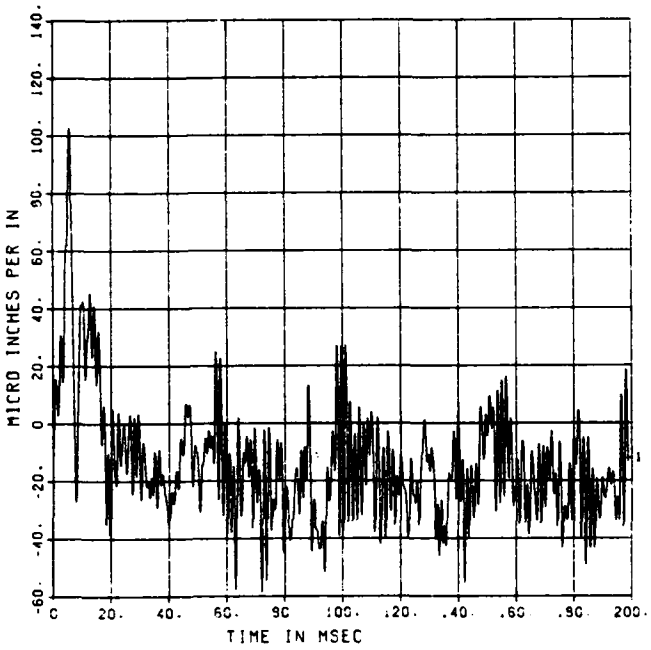
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
SS1 BOTTOM
20000. HZ CAL 1.400
F4
04/17/79 8227 2 R0552



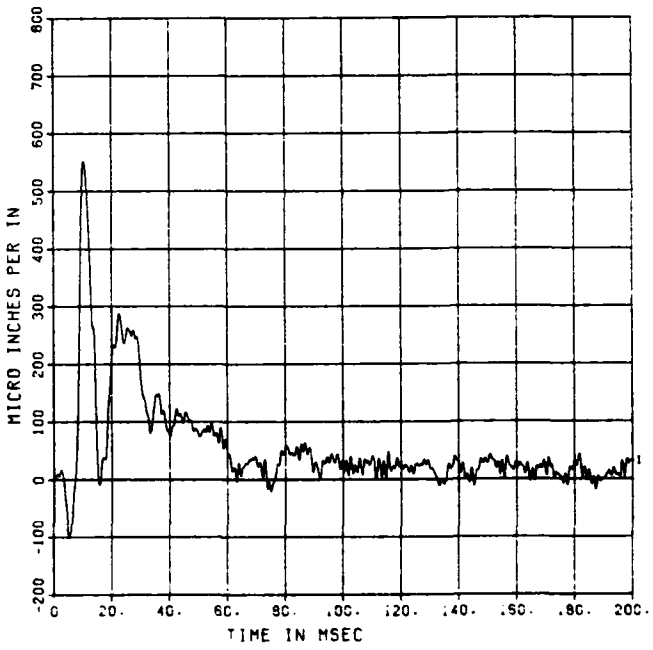
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
SS2 TOP
20000. HZ CAL 11400.
F4
04/17/79 8227 3 R0552



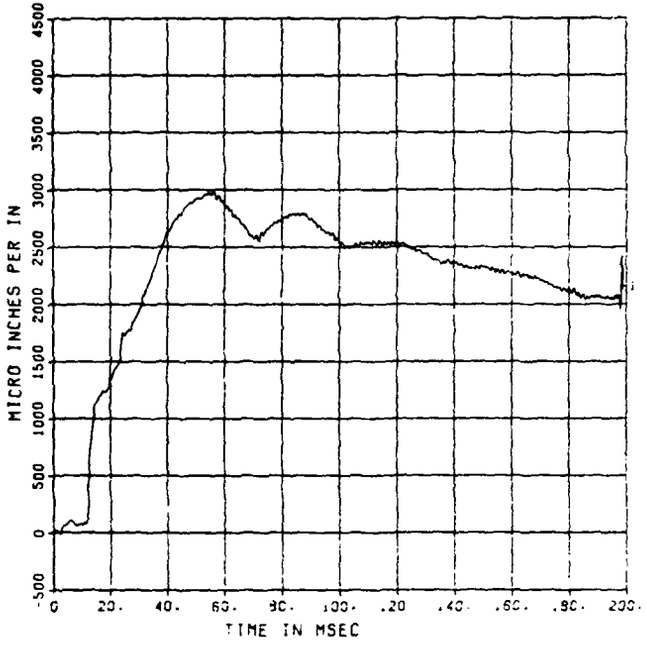
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
SS2 BOTTOM
20000. HZ CAL 11400.
F4
04/17/79 8227 4 R0552

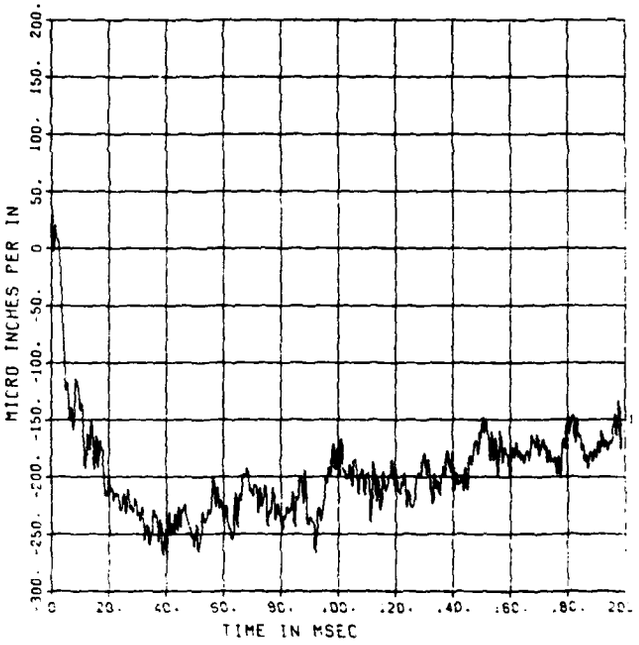


** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
SS3 TOP
20000. HZ CAL 11400.
F4
04/17/78 8227 5 R0552

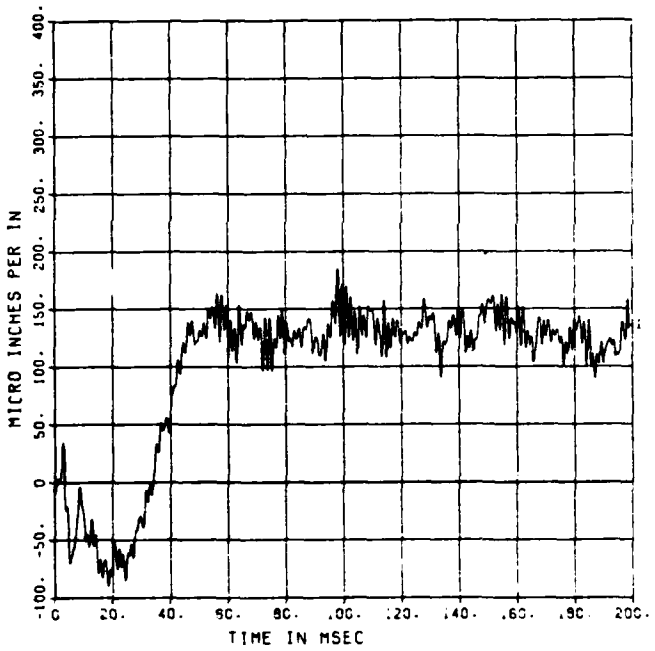


SLAB 1-A. TEST 1
SS3 BOTTOM
20000. HZ CAL 11400.
F4
04/17/78 8227 6 R0552



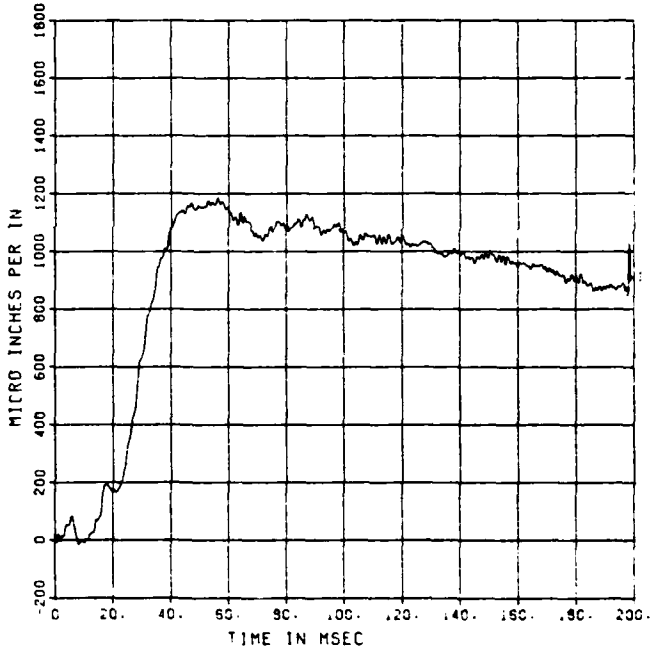
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
SS4 TOP
20000. HZ CAL 11400.
F4
04/17/79 8227 7 R0552



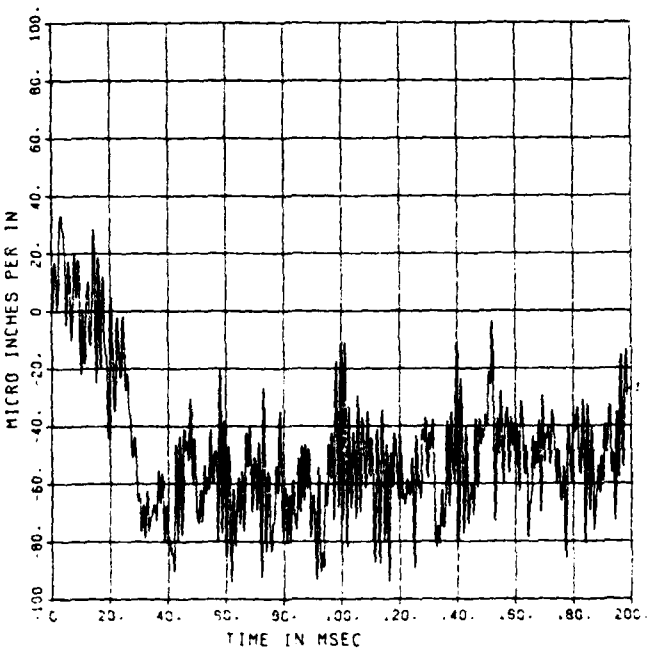
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
SS4 BOTTOM
20000. HZ CAL 11400.
F4
04/17/79 8227 8 R0552



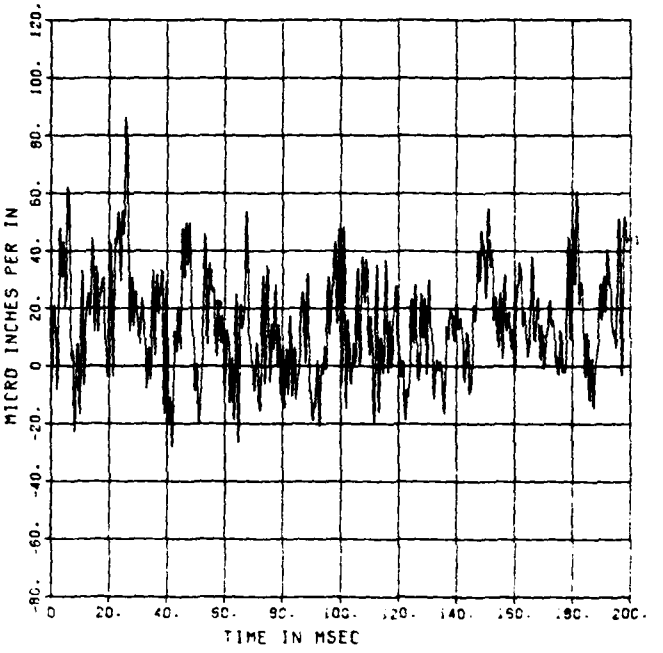
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
SS5 TOP
20000. HZ CAL 11400.
F4
04/17/79 8227 9 R0552



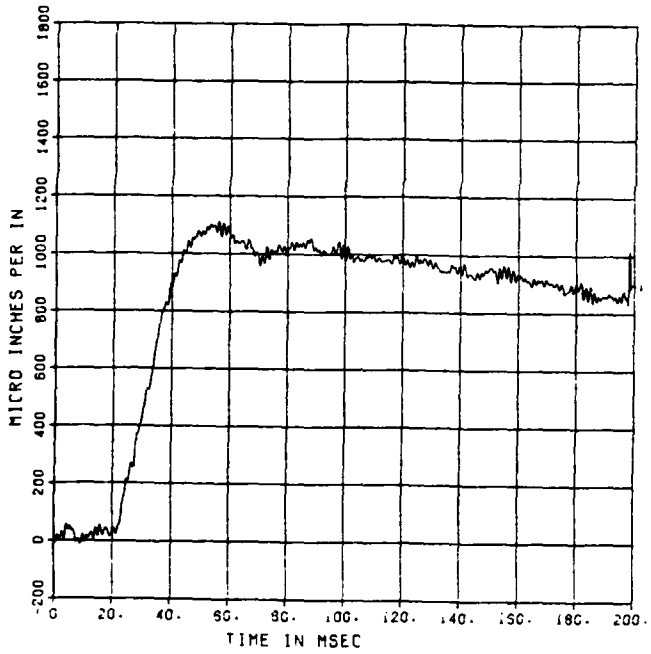
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
SS5 BOTTOM
20000. HZ CAL 11400.
F4
04/17/79 8227 10 R0552



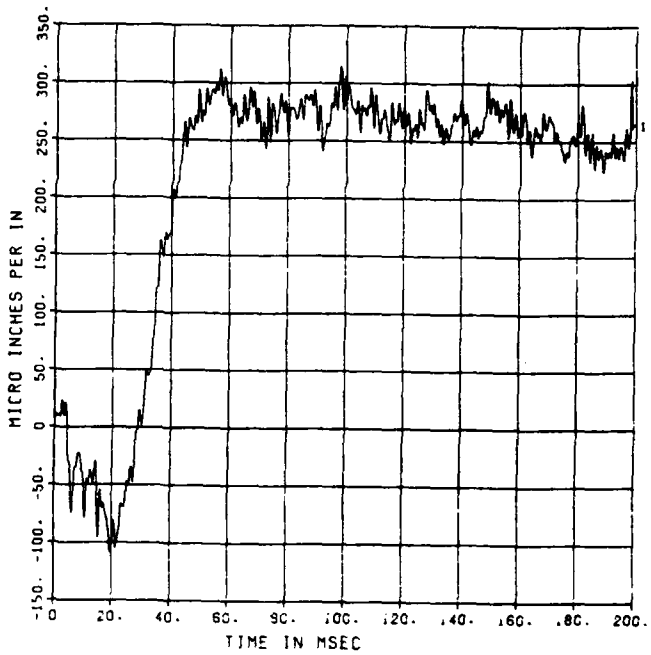
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
SL TOP
20000. HZ CAL 11400.
F4
04/17/79 8227 11 R0552



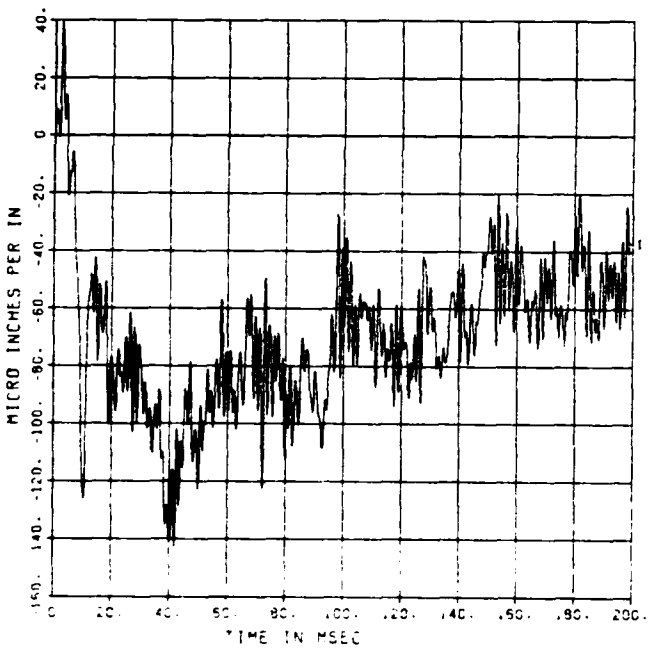
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
SL BOTTOM
20000. HZ CAL 11400.
F4
04/17/79 8227 12 R0552



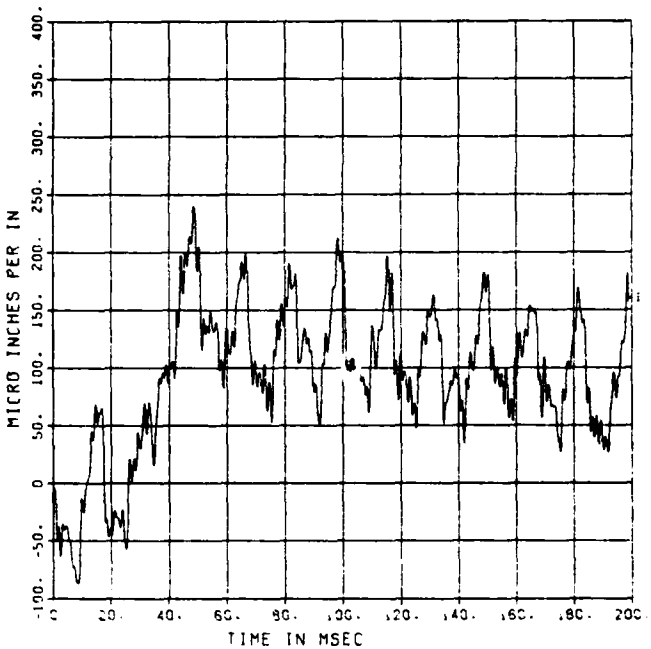
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
B1 TOP
20000. HZ CAL 11400.
F4
04/17/79 8227 13 R0552



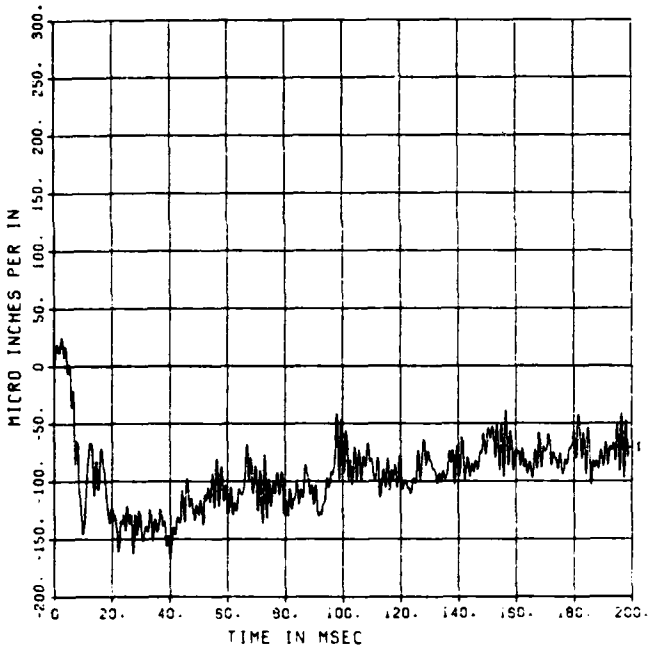
** PEAK VALUE IS 99 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
B1 BOTTOM
20000. HZ CAL 11400.
F4
04/17/79 8227 14 R0552



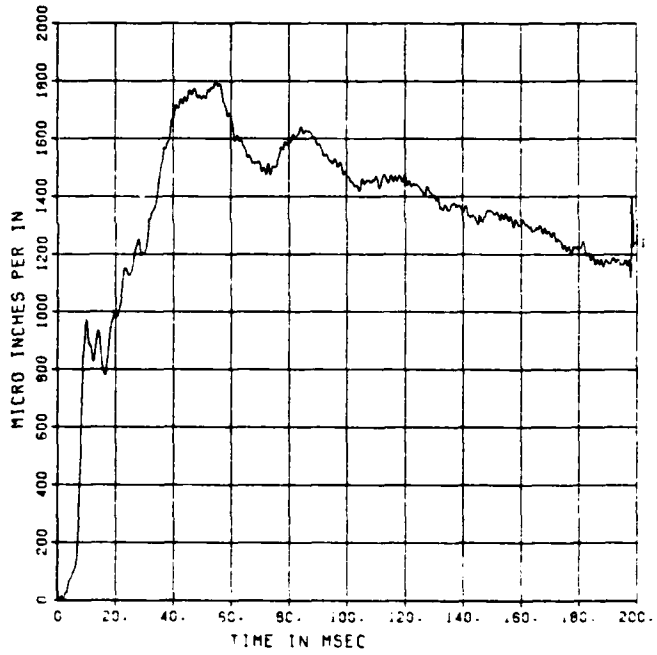
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
B2 TOP
20000. HZ CAL 11400.
F4
04/17/79 8227 15 R0552



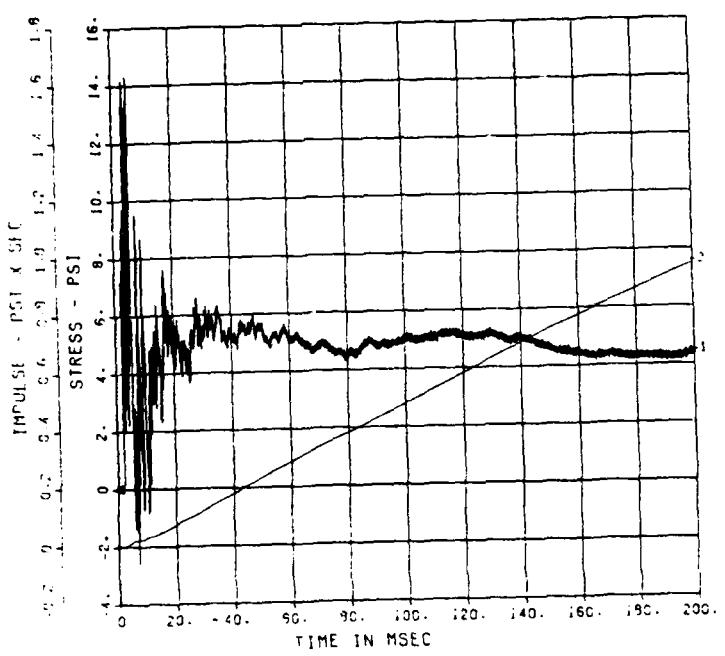
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
B2 BOTTOM
20000. HZ CAL 11400.
F4
04/17/79 8227 16 R0552

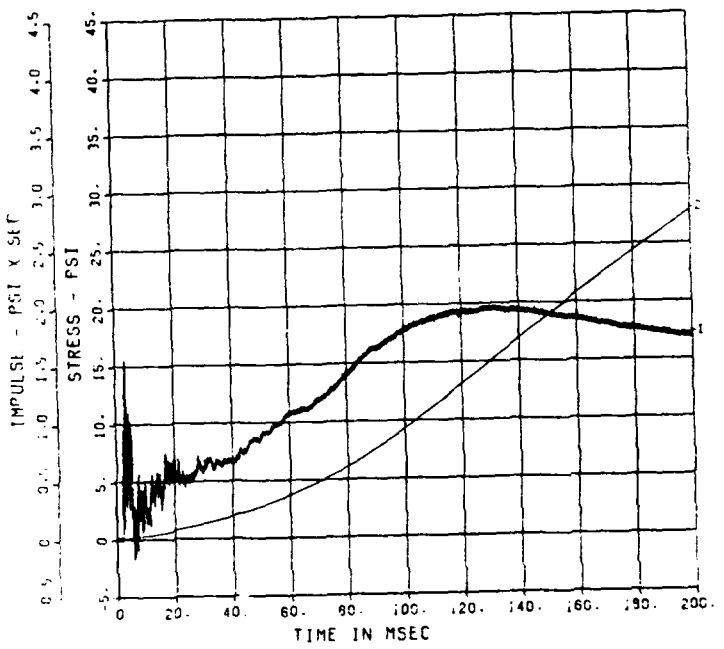


** PEAK VALUE IS -84 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
WP-1
100000. HZ CAL 15.93
04/12/79 8227 17 R0468



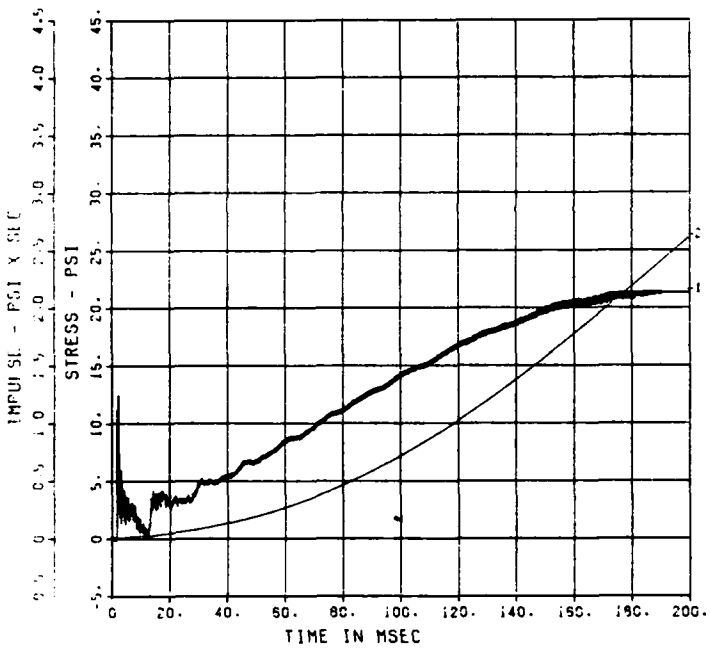
SLAB 1-A, TEST 1
WP-2
100000. HZ CAL 18.29
04/12/79 8227 18 R0468



** PEAK VALUE IS 5 % OVER CALIBRATION **

SLAB 1-A, TEST 1
CP
100000. HZ CAL 14.29

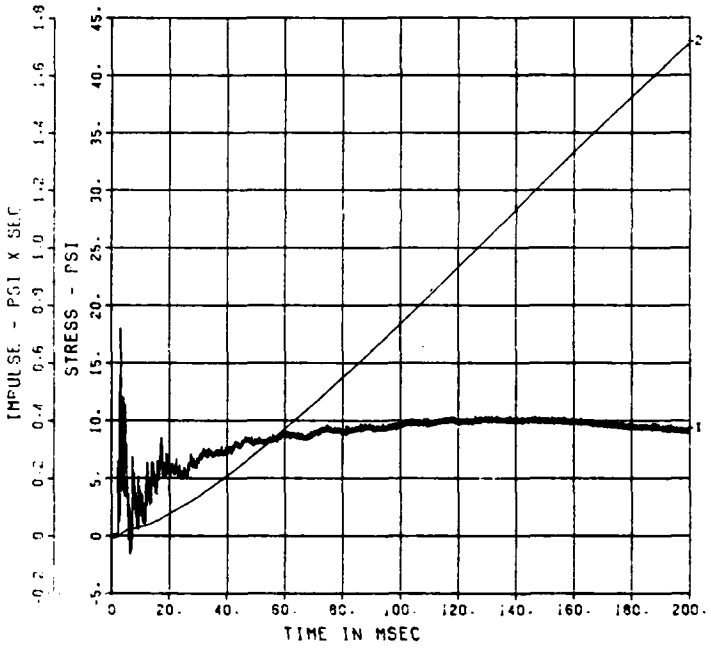
04/12/79 8227 19 R0468



** PEAK VALUE IS 50 % OVER CALIBRATION **

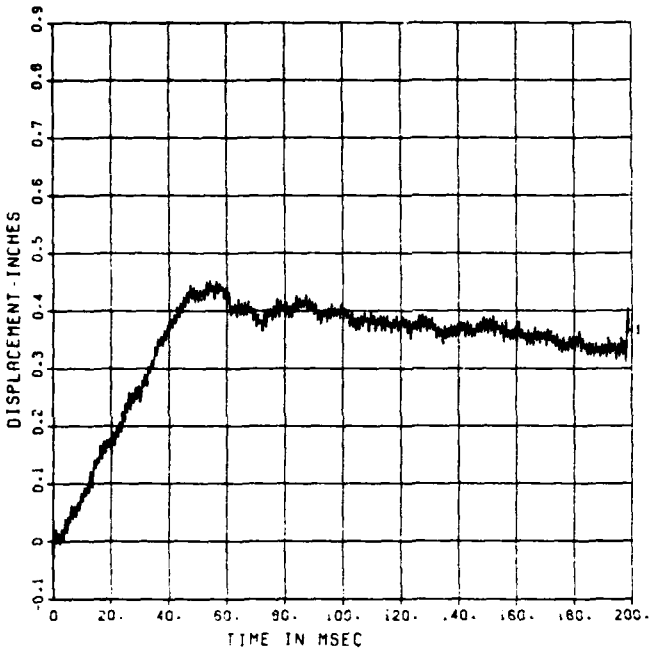
SLAB 1-A, TEST 1
EP-2
100000. HZ CAL 14.44

04/12/79 8227 21 R0468



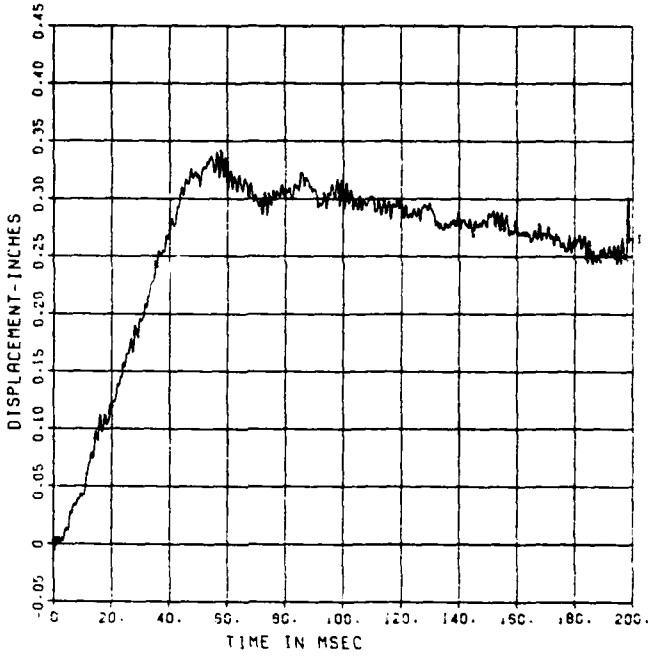
** PEAK VALUE IS 25 % OVER CALIBRATION **

SLAB 1-A. TEST 1
WD-1
20000. HZ CAL 6.127
F4
04/17/79 8227 24 R0552



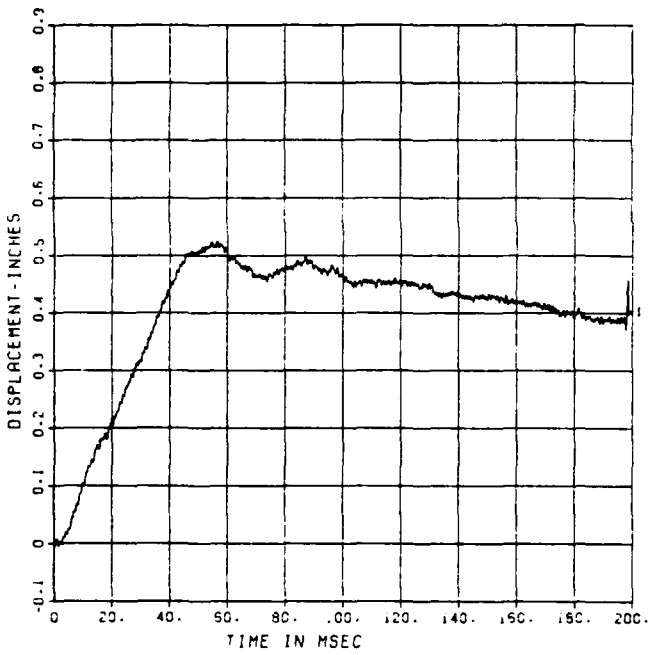
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 1-A. TEST 1
WD-2
20000. HZ CAL 5.548
F4
04/17/79 8227 25 R0552



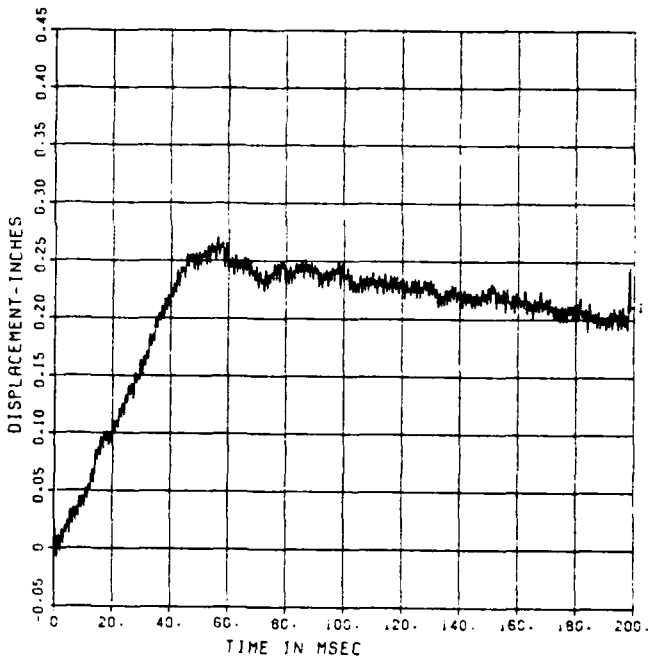
** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
WD-3
20000. HZ CAL 3.685
F4
04/17/79 8227 26 R0552



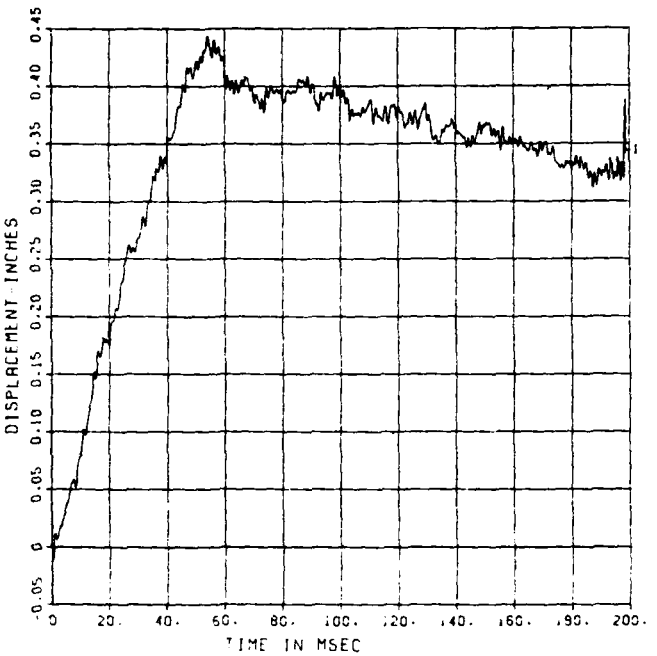
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
WD-4
20000. HZ CAL 3.623
F4
04/17/79 8227 27 R0552



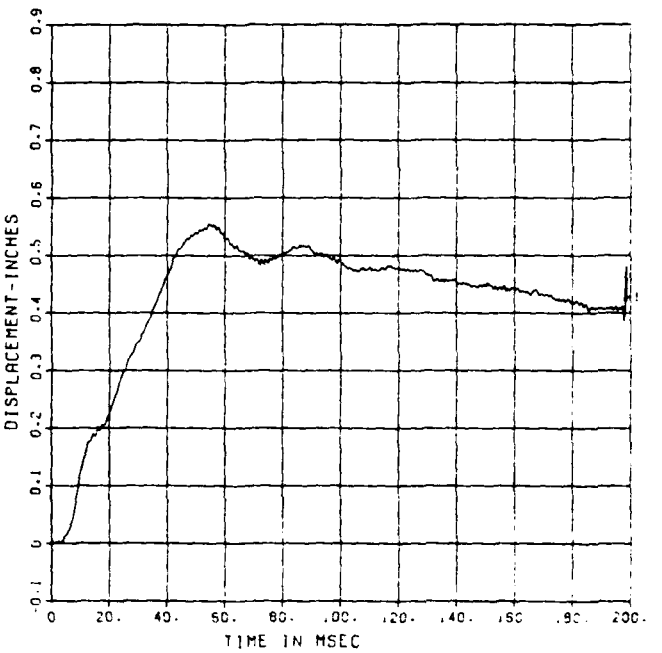
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
ED-1
20000. HZ CAL 6.343
F4
04/17/79 8227 28 R0552

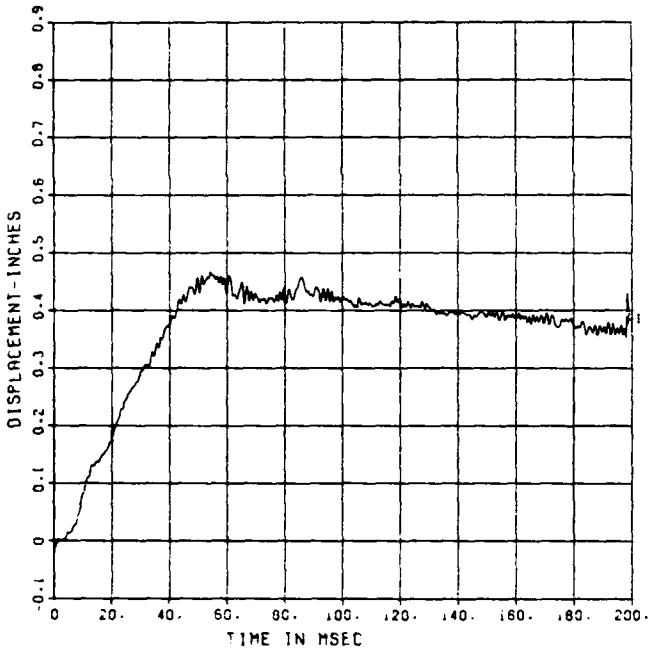


** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 1-A, TEST 1
BD-1
20000. HZ CAL 2.328
F4
04/17/79 8227 29 R0552

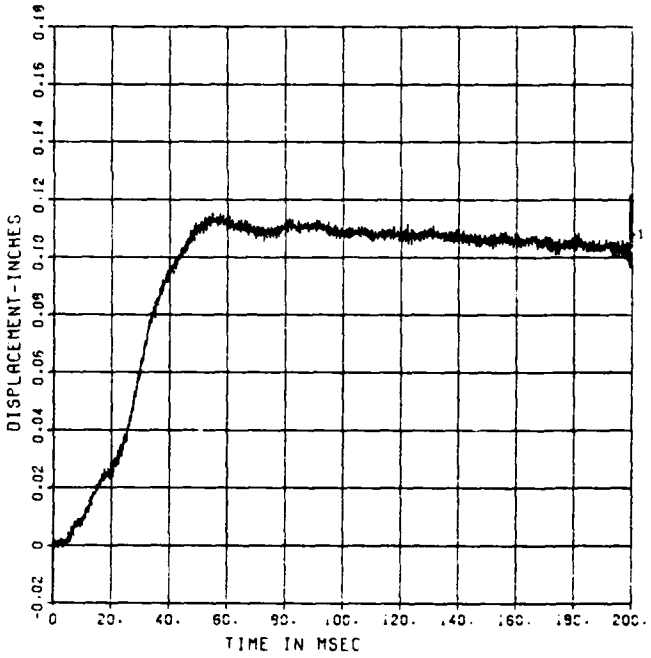


SLAB 1-A. TEST 1
BD-2
20000. HZ CAL 2.342
F4
04/17/79 8227 30 R0552



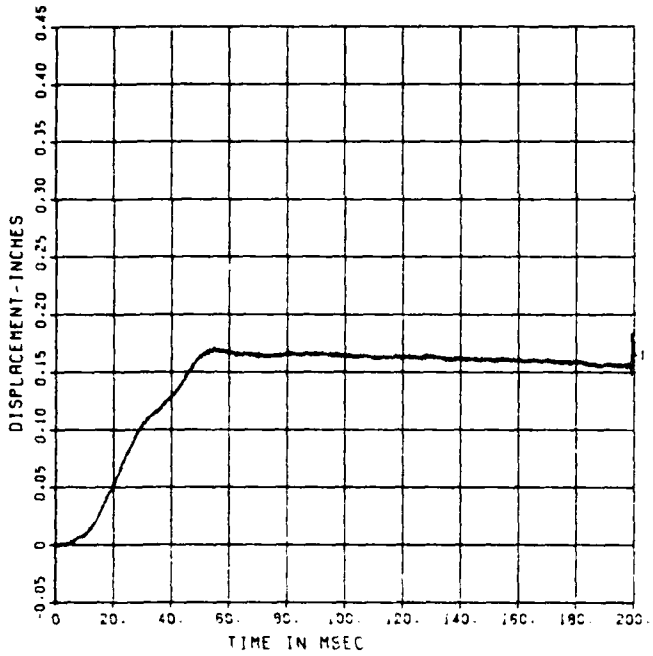
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 1-A, Test 1
NBD
100000. HZ CAL 1.160
F4
04/11/79 9202 1 R0427



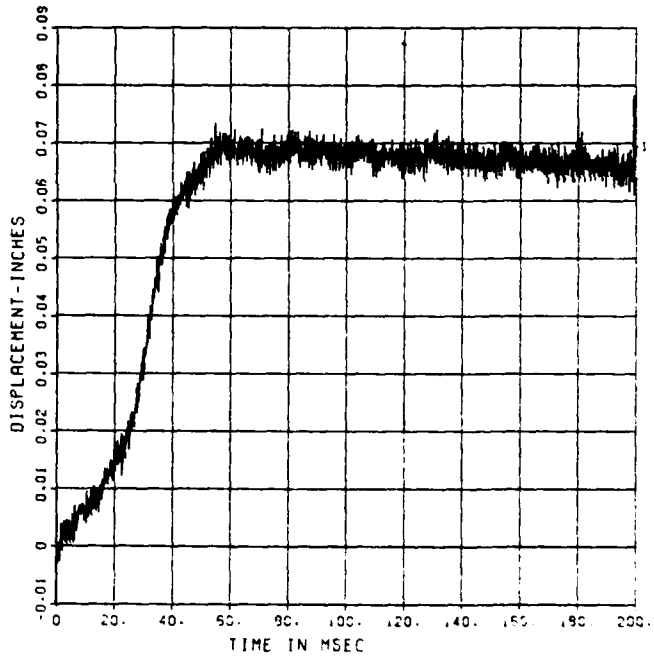
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 1-A, Test 1
SBD
10000C. HZ CAL 1.056
F4
04/11/79 9202 2 R0427



** PEAK VALUE IS -93 % UNDER CALIBRATION **

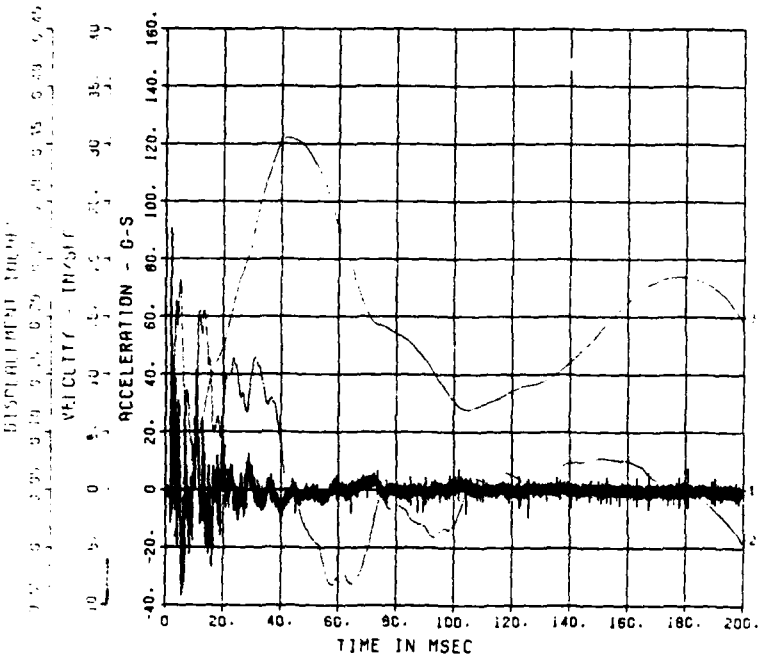
SLAB 1-A, Test 1
WBD
10000C. HZ CAL 1.153
F4
04/11/79 9202 3 R0427



** PEAK VALUE IS -93 % UNDER CALIBRATION **

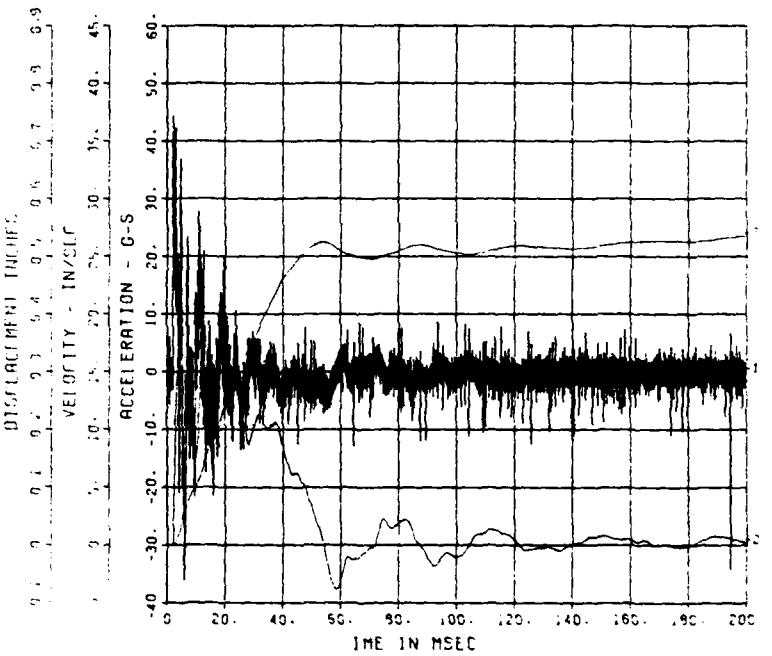
SLAB I-A, TEST 1
A1
100000. HZ LMS CAL 95.2

04/18/79 9202 4 R0613



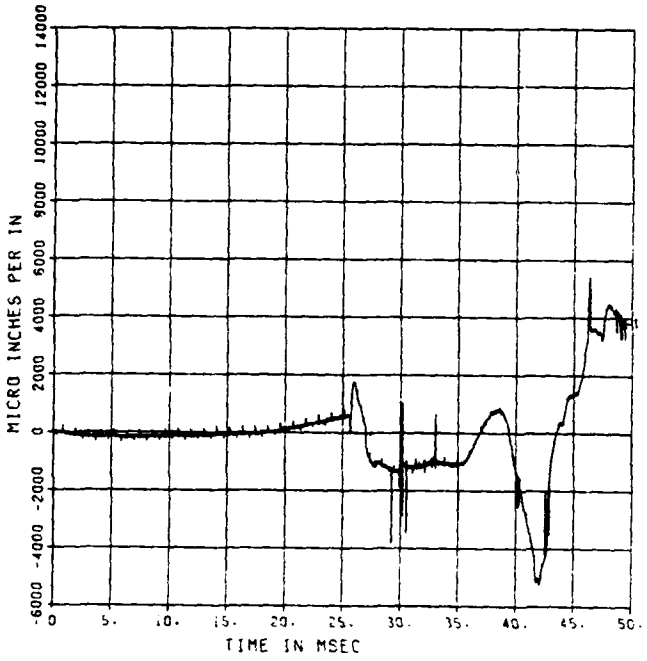
SLAB I-A, TEST 1
A2
100000. HZ CBS CA 91.6

04/13/79 9202 5 R0495



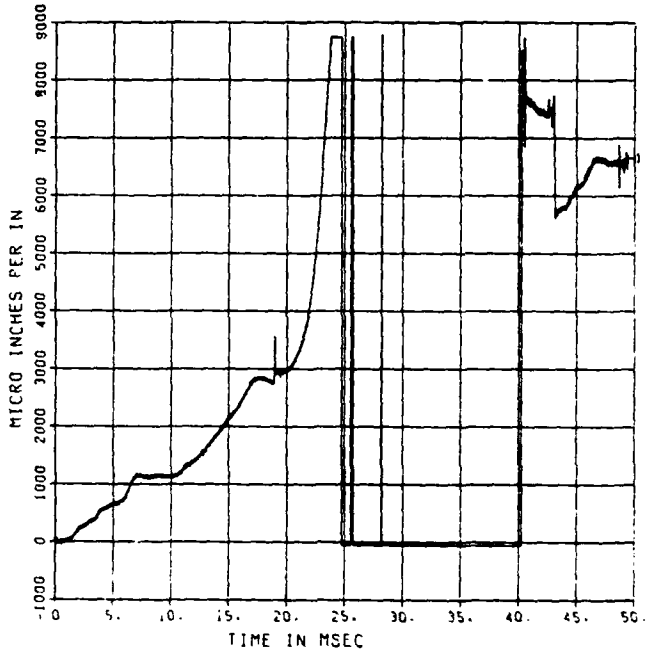
SLAB 1-B. TEST 2
SS1 TOP
100000. HZ CAL 5766.

04/12/79 8227 31 R0468



SLAB 1-B. TEST 2
SS1 BOTTOM
100000. HZ CAL 5766.

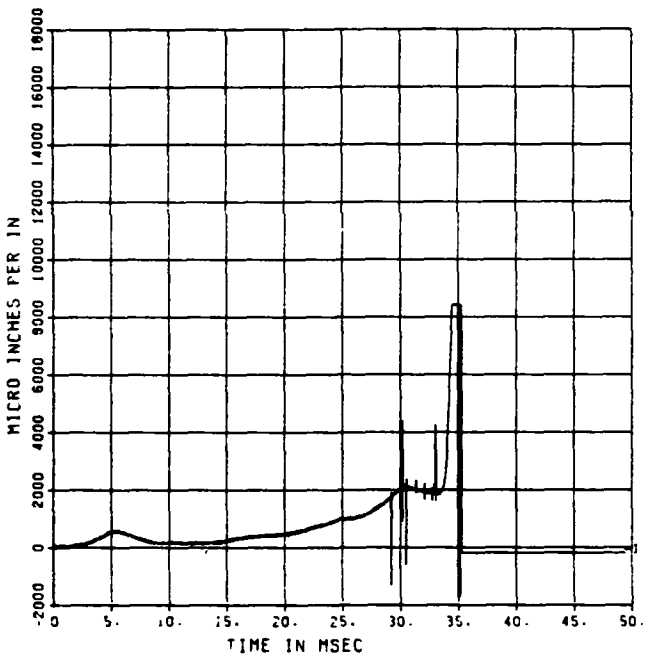
04/12/79 8227 32 R0468



** PEAK VALUE IS 52 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SS2 TOP
100000. HZ CAL 5766.

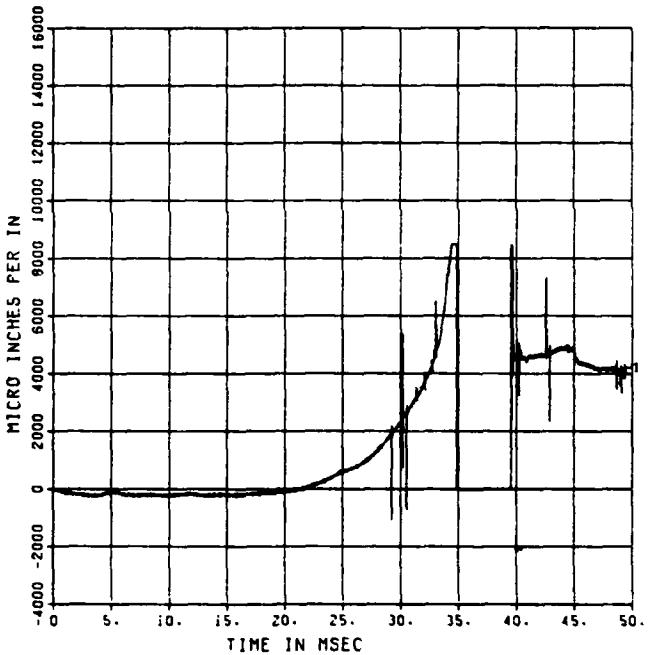
04/12/79 8227 33 R0468



** PEAK VALUE IS 45 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SS2 BOTTOM
100000. HZ CAL 5766.

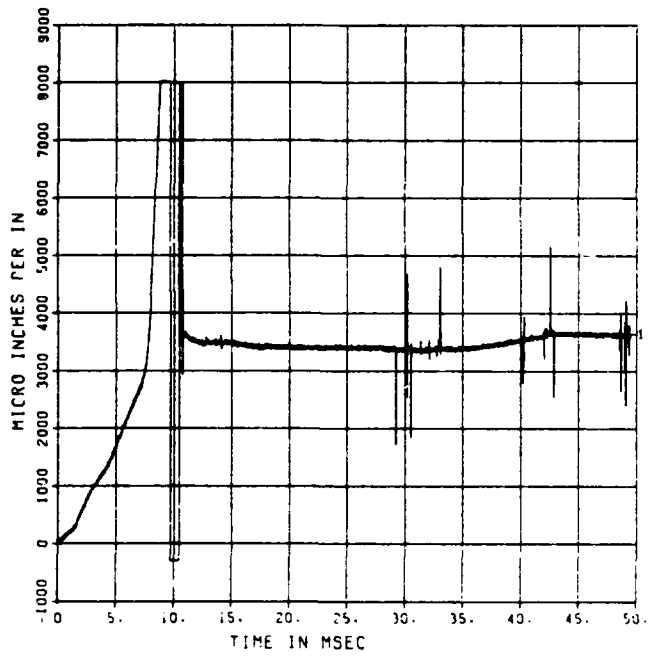
04/12/79 8227 34 R0469



** PEAK VALUE IS 49 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SS3 TOP
100000. HZ CAL 5766.

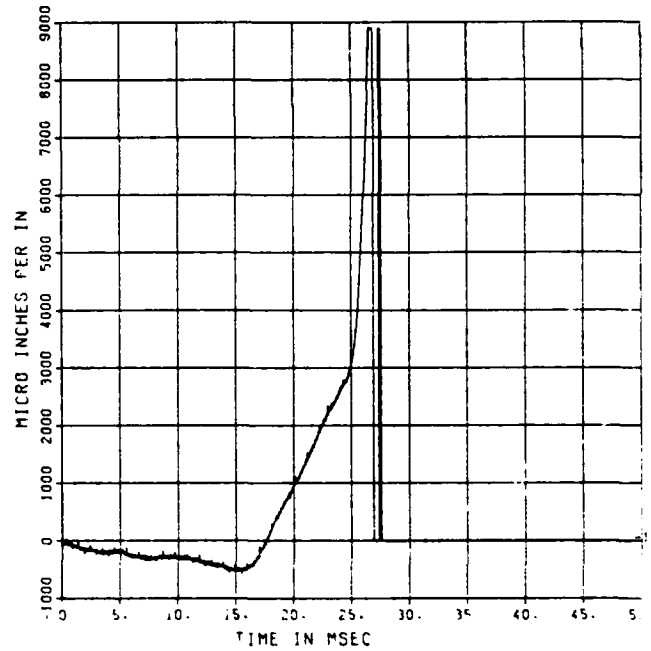
04/12/79 8227 35 R0469



** PEAK VALUE IS 53 % OVER CALIBRATION **

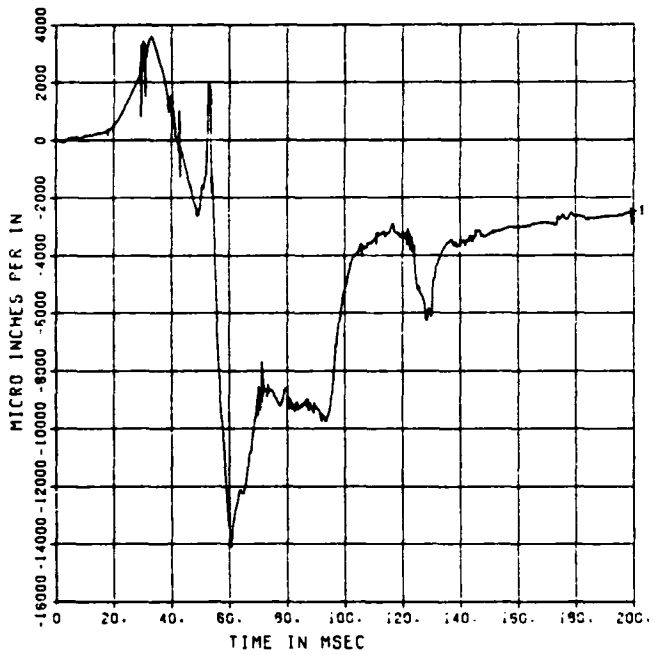
SLAB 1-B. TEST 2
SS3 BOTTOM
100000. HZ CAL 5766.

04/12/79 8227 36 R0469



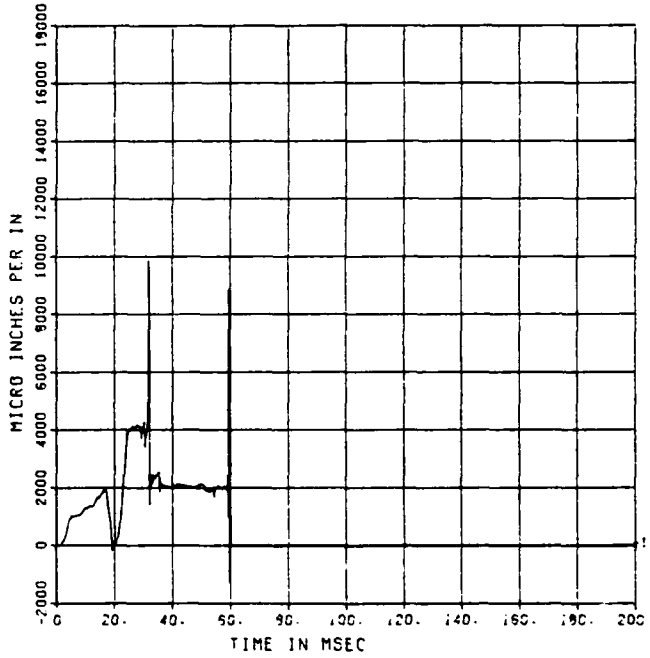
** PEAK VALUE IS 54 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SS4 TOP
100000. HZ CAL 5766.
F4
04/12/79 8227 37 R0469



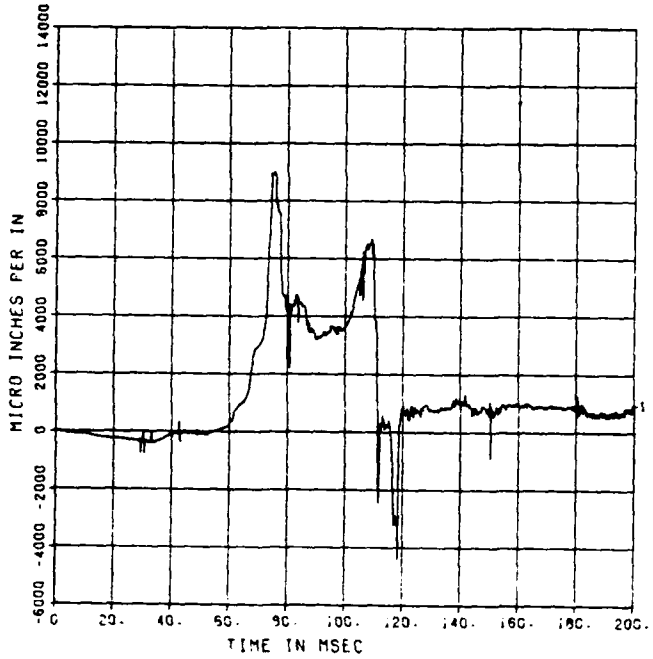
** PEAK VALUE IS 145 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SS4 BOTTOM
100000. HZ CAL 5766.
F4
04/12/79 8227 38 R0468



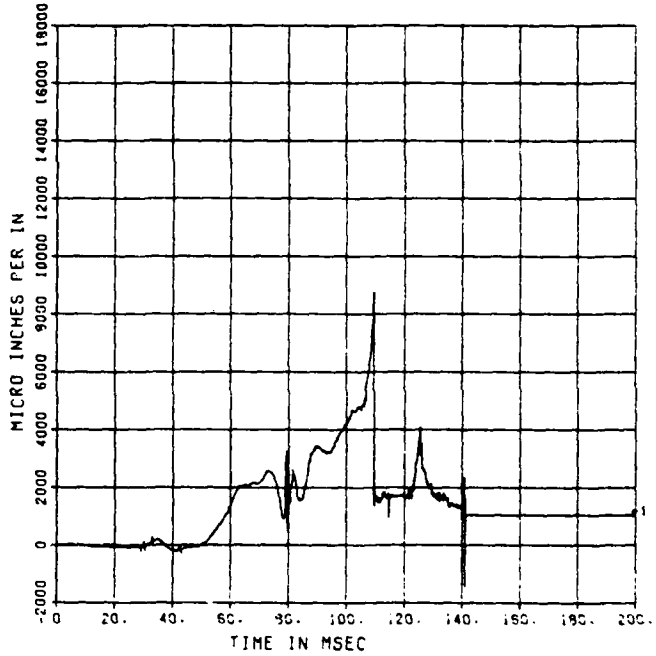
** PEAK VALUE IS 71 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SSS TOP
100000. HZ CAL 5766.
F4
04/12/79 8227 39 R0468



** PEAK VALUE IS 57 % OVER CALIBRATION **

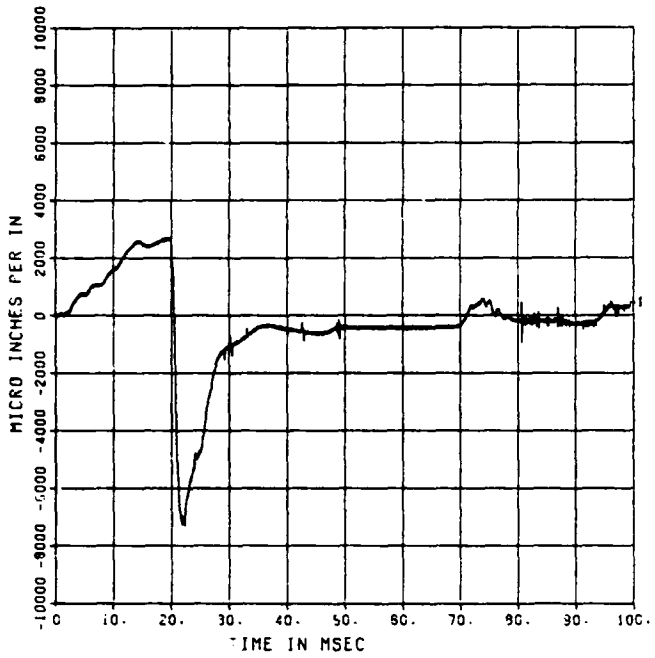
SLAB 1-B. TEST 2
SSS BOTTOM
100000. HZ CAL 5766.
F4
04/12/79 8227 40 R0469



** PEAK VALUE IS 52 % OVER CALIBRATION **

SLAB 1-B. TEST 2
SL TOP
100000. HZ CAL 5766.

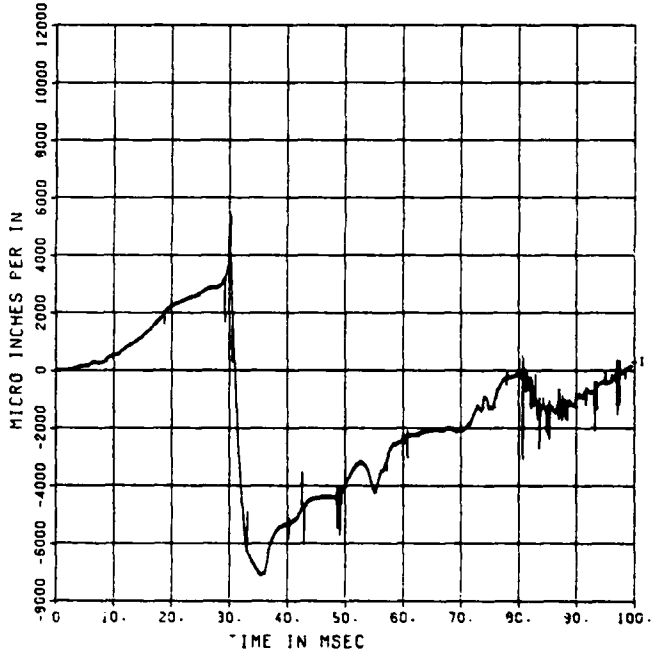
04/12/79 8227 41 R0469



** PEAK VALUE IS 27 % OVER CALIBRATION **

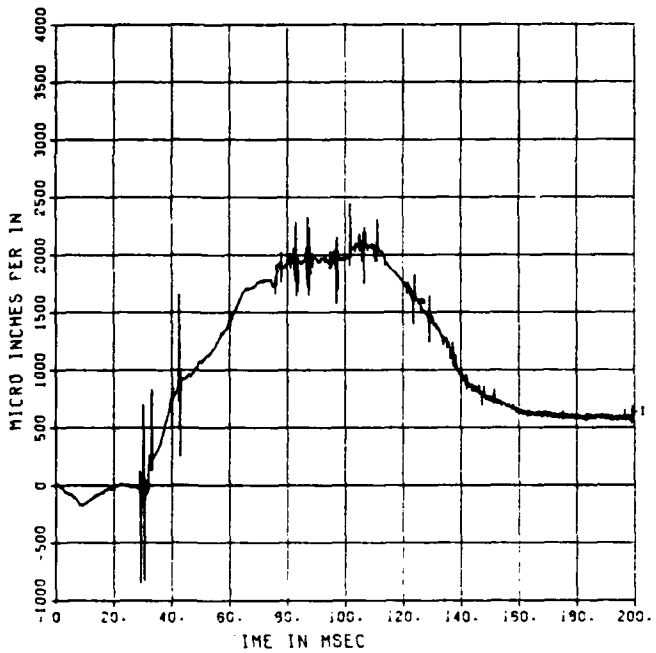
SLAB 1-B. TEST 2
SL BOTTOM
100000. HZ CAL 5766.

04/12/79 8227 42 R0469

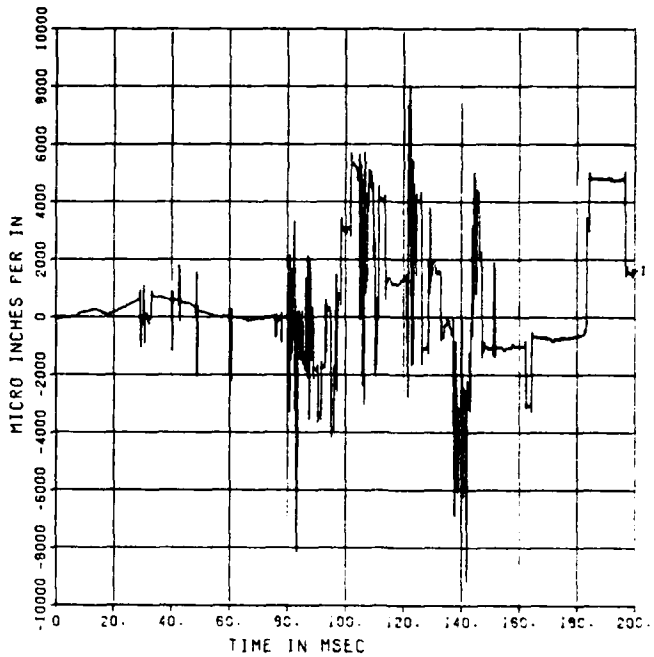


** PEAK VALUE IS 23 % OVER CALIBRATION **

SLAB I-B. TEST 2
B1 TOP
100000. HZ CAL 5766.
F4
04/12/79 8227 43 R0468



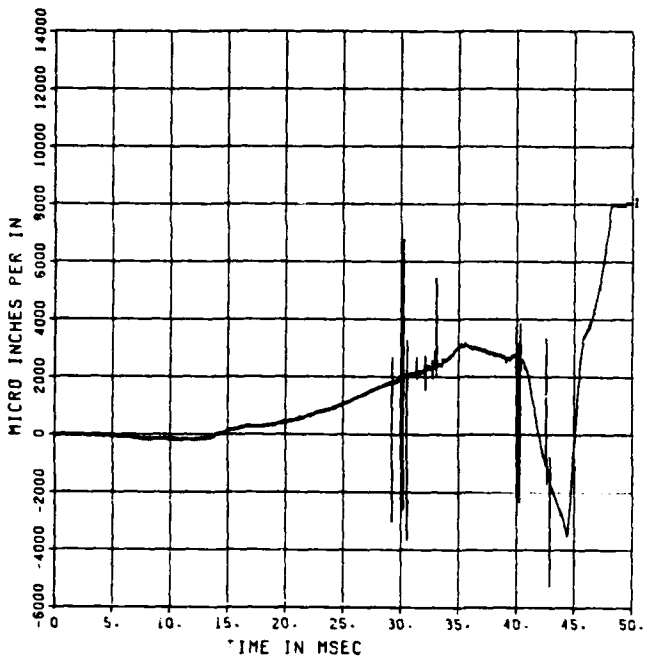
SLAB I-B. TEST 2
B1 BOTTOM
100000. HZ CAL 5768.
F4
04/12/79 8227 44 R0468



** PEAK VALUE IS F8 % OVER CALIBRATION **

SLAB 1-B. TEST 2
B2 TOP
100000. HZ CAL 5766.

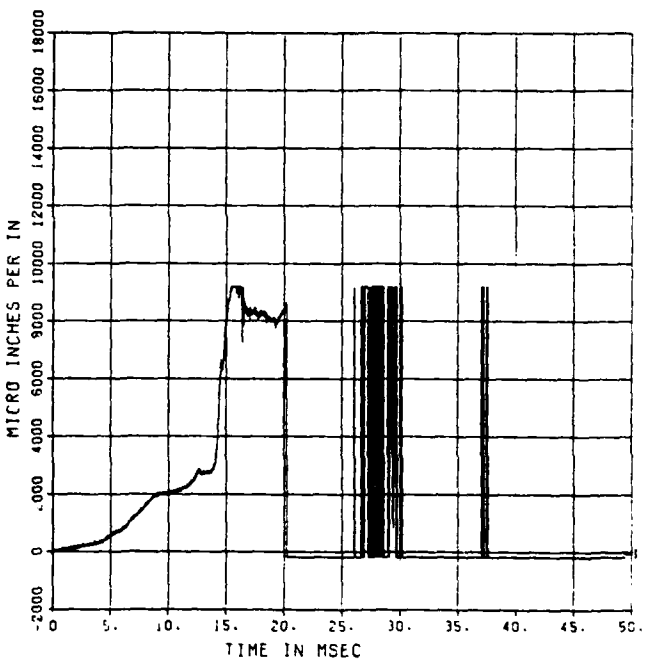
04/12/79 8227 45 R0468



** PEAK VALUE IS 39 % OVER CALIBRATION **

SLAB 1-B. TEST 2
B2 BOTTOM
100000. HZ CAL 5766.

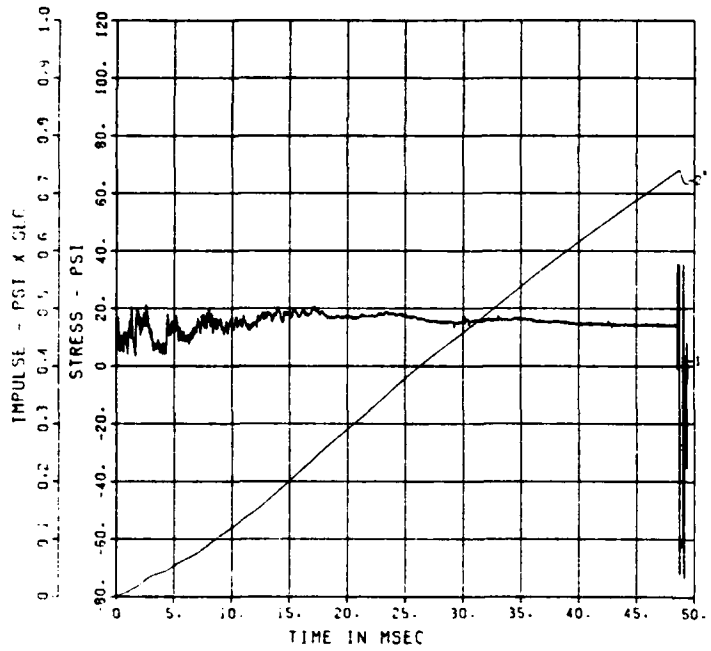
04/12/79 8227 46 R0468



** PEAK VALUE IS 50 % OVER CALIBRATION **

SLAB 1-B. TEST 2
WP-1
100000. HZ CAL 23.768

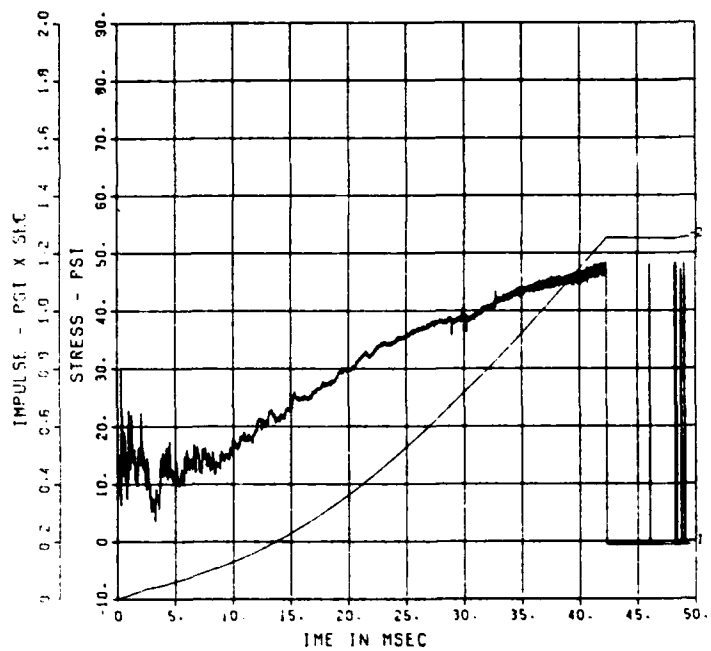
04/12/79 8227 47 R0468



** PEAK VALUE IS 209 % OVER CALIBRATION **

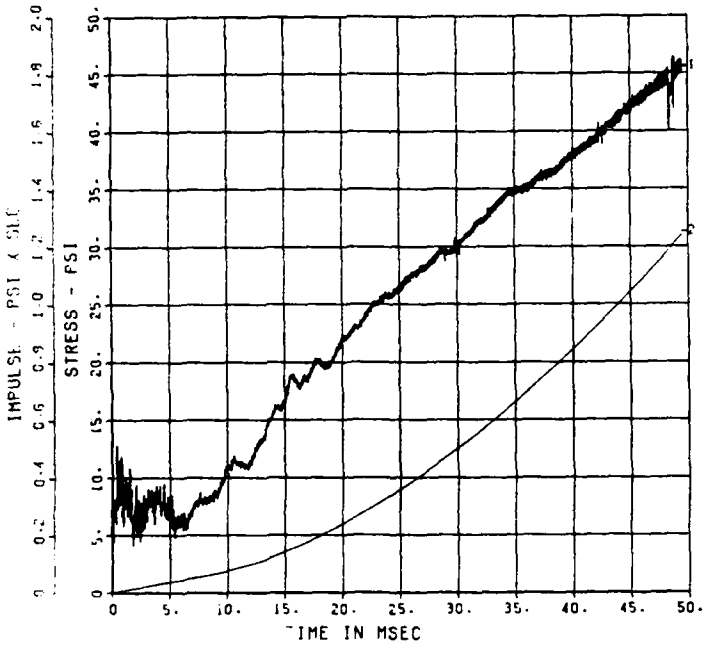
SLAB 1-B. TEST 2
WP-2
100000. HZ CAL 24.321

04/12/79 8227 48 R0468



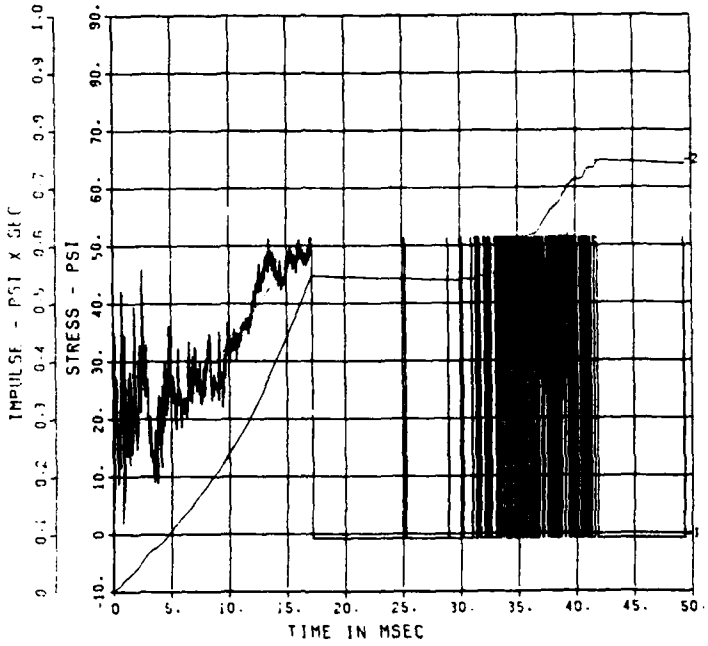
** PEAK VALUE IS 99 % OVER CALIBRATION **

SLAB 1-B. TEST 2
CP
100000. HZ CAL 28.363
04/12/79 8227 49 R0468



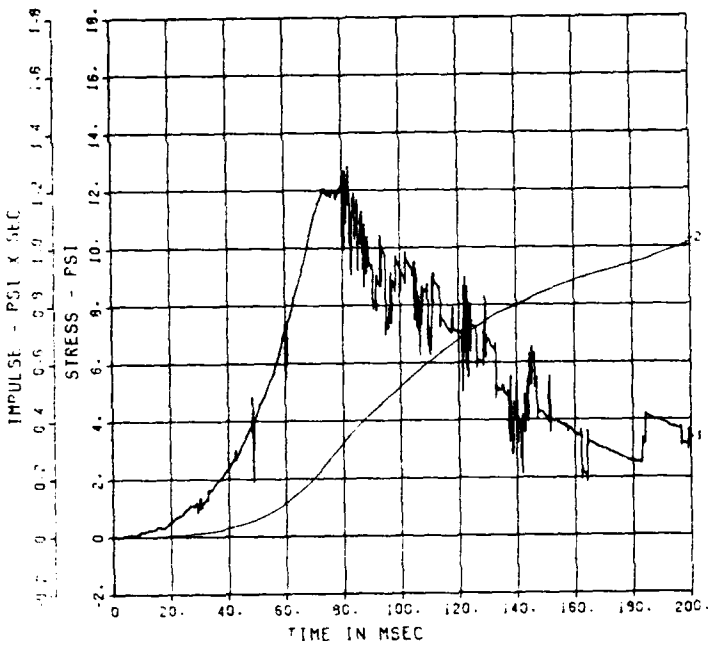
** PEAK VALUE IS 53 % OVER CALIBRATION **

SLAB 1-B. TEST 2
EP-2
100000. HZ CAL 28.721
04/12/79 8227 51 R0468



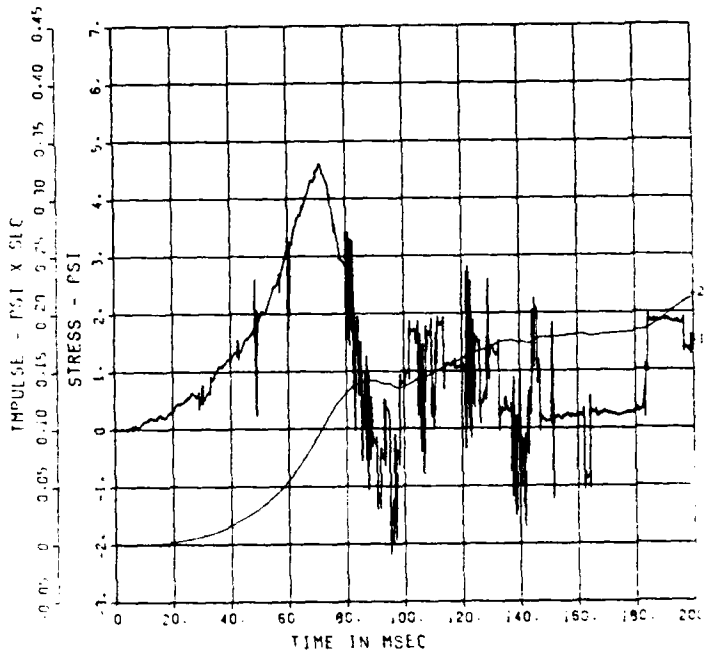
** PEAK VALUE IS 51 % OVER CALIBRATION **

SLAB 1-B. TEST 2
NWC
100000. HZ CAL 8.53
f4
04/12/79 8227 S2 R0468

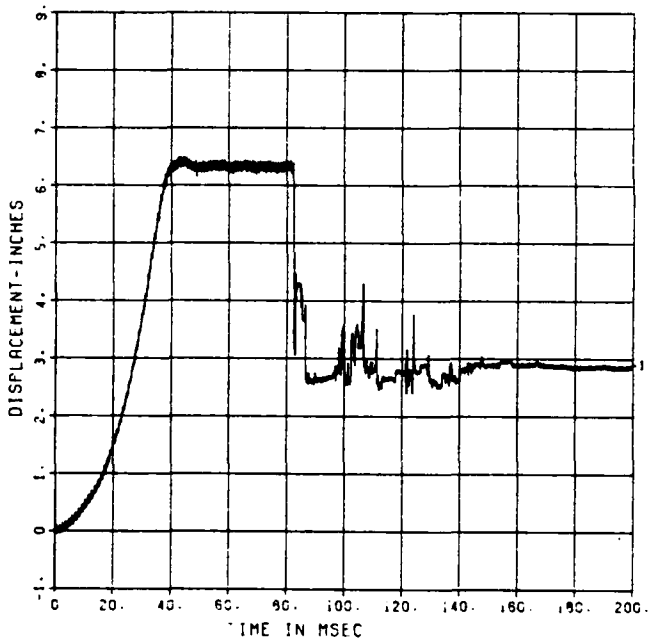


** REGI... 5. % OVER CALIBRATION **

SLAB 1-B. TEST 2
SEC
100000. Z CAL 8.02
f4
04/ 2/79 8227 S3 R0468

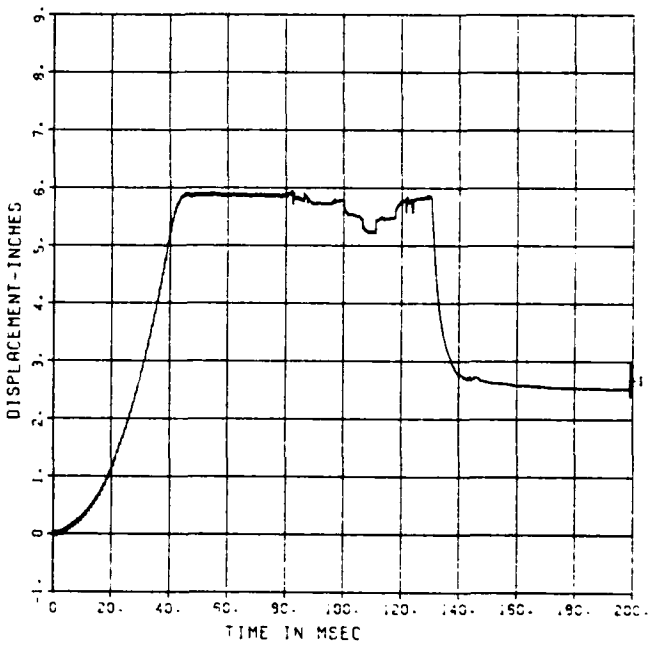


SLAB 1-B, TEST 2
WD-1
100000. HZ CAL 6.127
04/12/79 8227 54 R0468



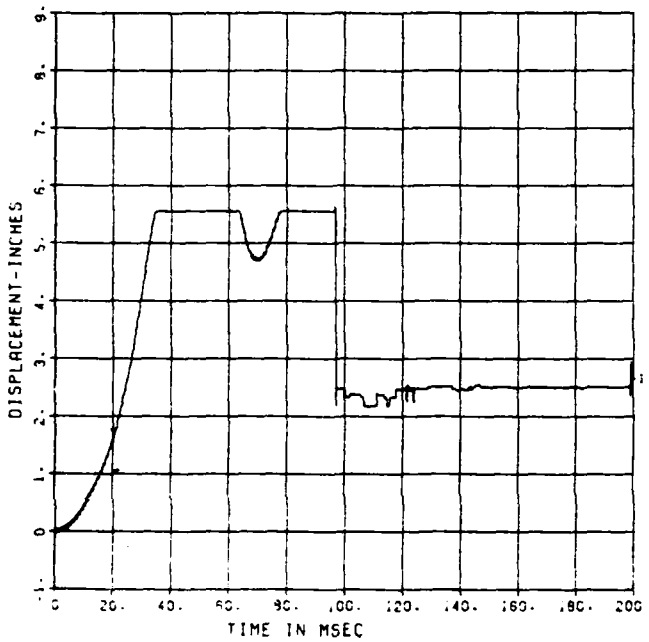
** PEAK VALUE IS 5 % OVER CALIBRATION **

SLAB 1-B, TEST 2
WD-2
100000. HZ CAL 5.548
F4
04/16/79 8227 55 R053.



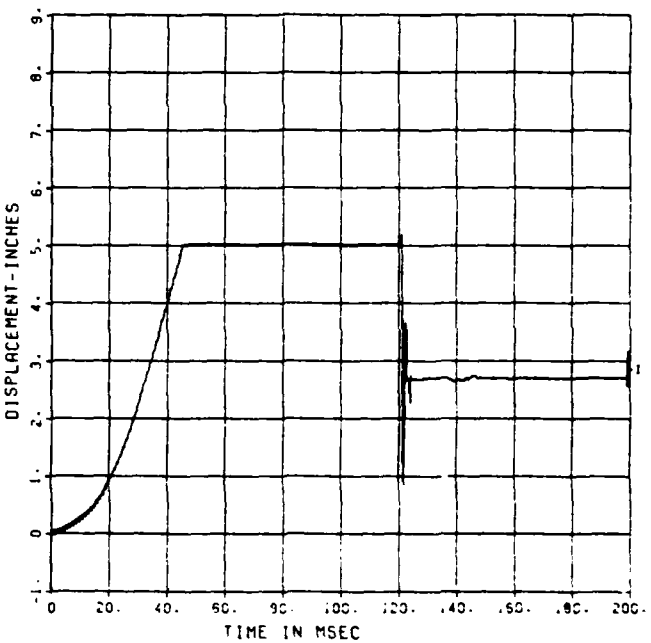
** PEAK VALUE IS [illegible] % OVER CALIBRATION **

SLAB 1-B. TEST 2
WD-3
100000. HZ CAL 3.685
F4
04/16/79 8227 56 R053.



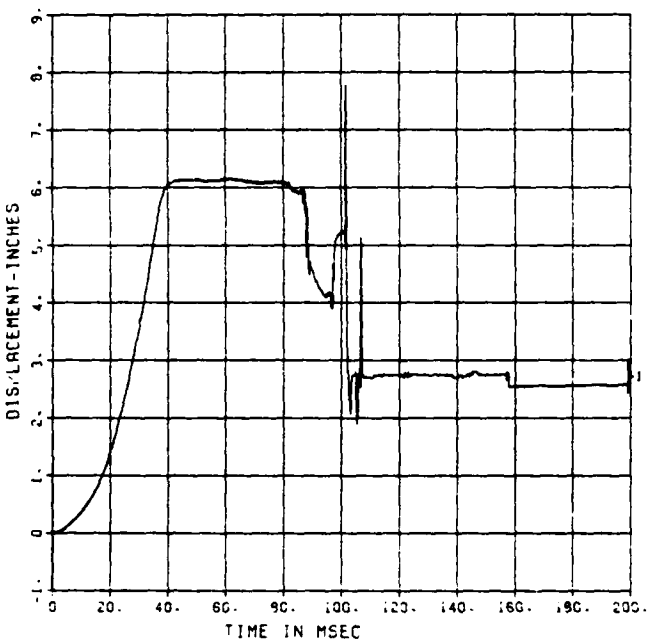
** PEAK VALUE IS [illegible] OVER CALIBRATION **

SLAB 1-B. TEST 2
WD-4
100000. HZ CAL 3.623
F4
04/16/79 8227 57 R053.



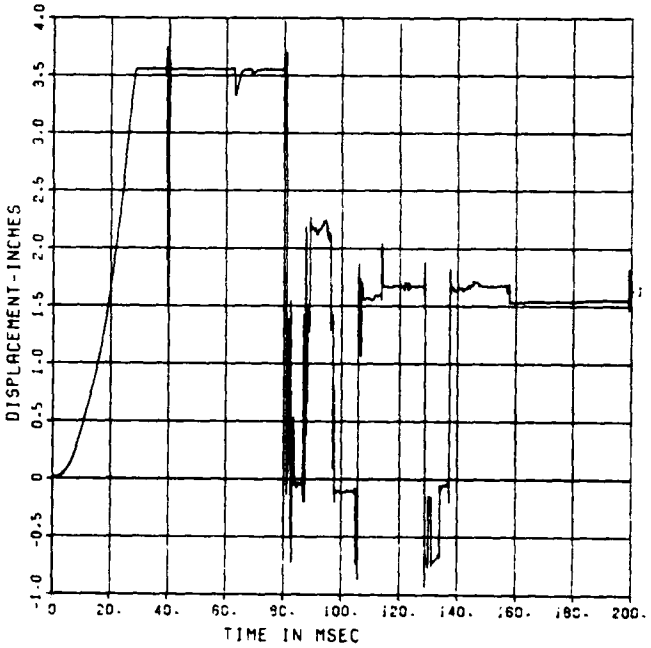
** PEAK VALUE IS 43 OVER CALIBRATION **

SLAB 1-B. TEST 2
ED-1
100000. HZ CAL 6.343
F4
04/16/79 8227 58 R0531



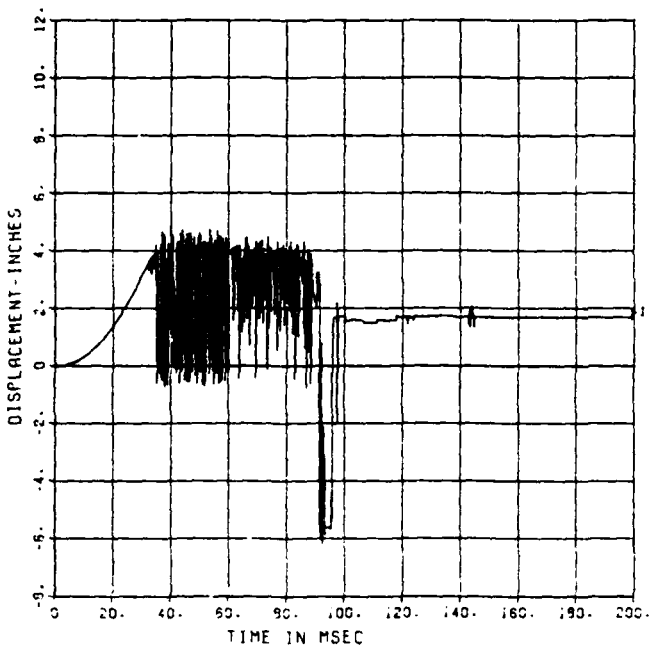
** PEAK VALUE IS 23 * OVER CALIBRATION **

SLAB 1-B. TEST 2
BD-1
100000. HZ CAL 2.328
F4
04/16/79 8227 59 R0531

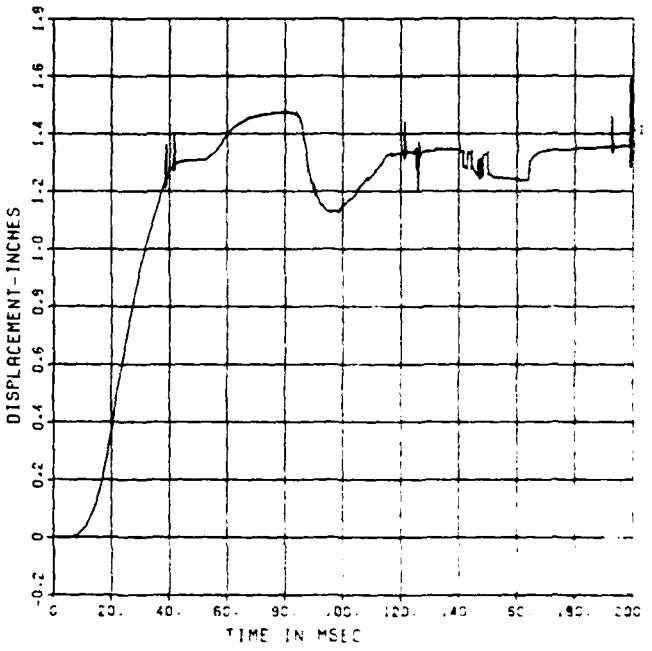


** PEAK VALUE IS 91 * OVER CALIBRATION **

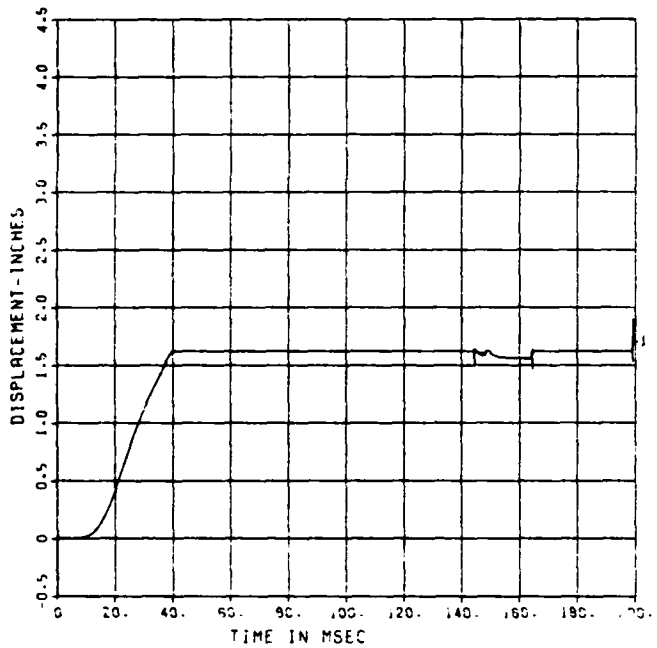
SLAB 1-B. TEST 2
BD-2
100000. HZ CAL 2.342
F4
04/16/79 8227 60 R0531



SLAB 1-B. TEST 2
NBD
100000. HZ CAL 1.160
F4
04/16/79 11152 1 R0532

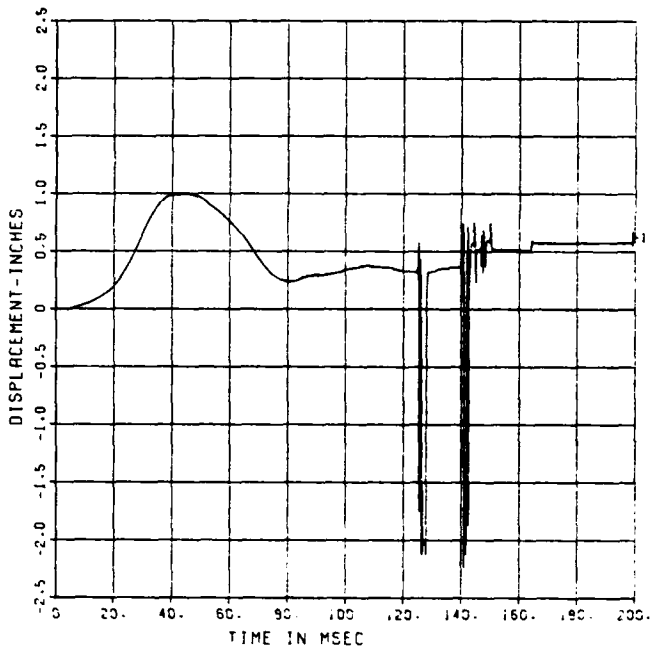


SLAB I-B. TEST 2
SBD
100000. HZ CAL 1.055
F4
04/16/79 11152 2 R0532



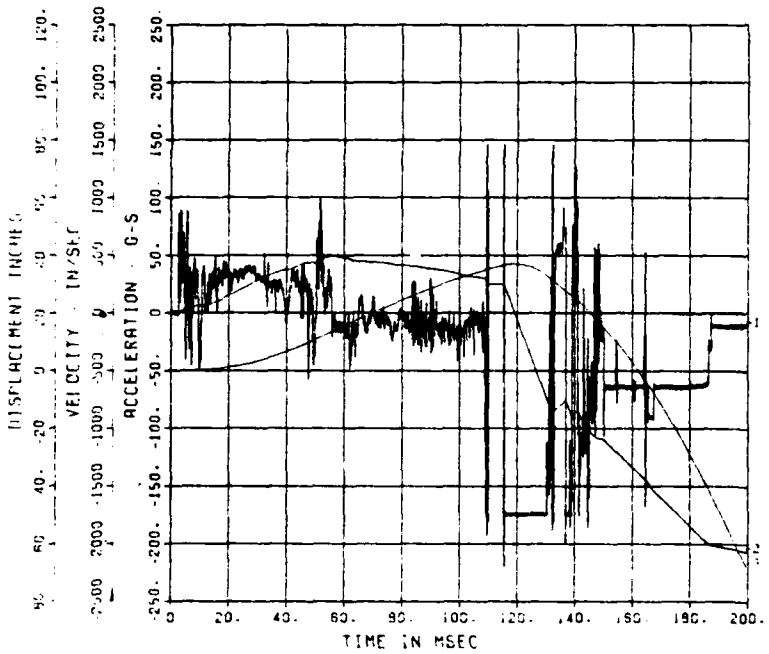
** PEAK VALUE IS 50 % OVER CALIBRATION **

SLAB I-B. TEST 2
WBD
100000. HZ CAL 1.153
F4
04/16/79 11152 3 R0532



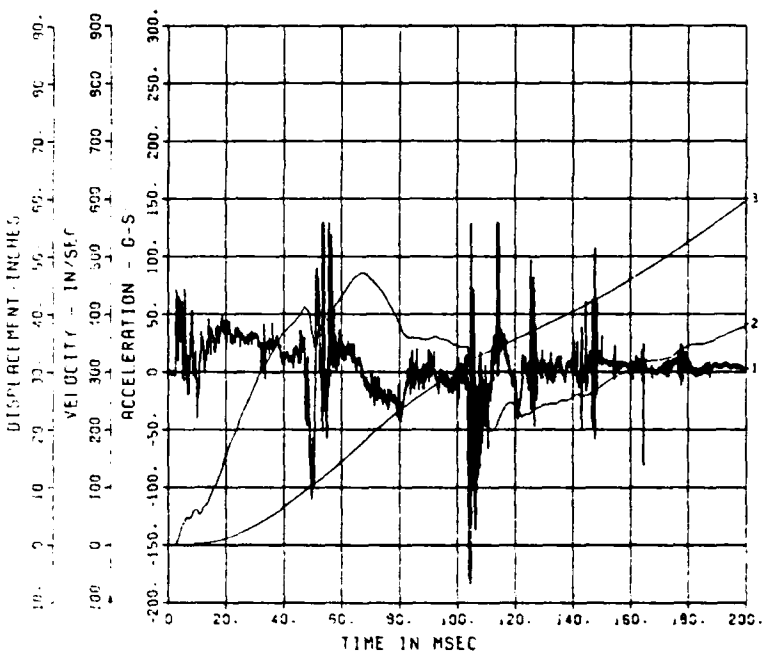
** PEAK VALUE IS 91 % OVER CALIBRATION **

SLAB 1-B. TEST 2
A1
100000. HZ CAL 95.2
04/16/79 11152 4 R0532



** PEAK VALUE IS 131 % OVER CALIBRATION **

SLAB 1-B. TEST 2
A2
100000. HZ CAL 91.6
04/16/79 11152 5 R0532



** PEAK VALUE IS 100 % OVER CALIBRATION **

APPENDIX B

STRUCTURE 2 TEST DATA

Data from the tests on the wooden post upgrading system (Structure 2) are presented in this appendix. Labels on the plots indicate the following:

Line 1: Test name

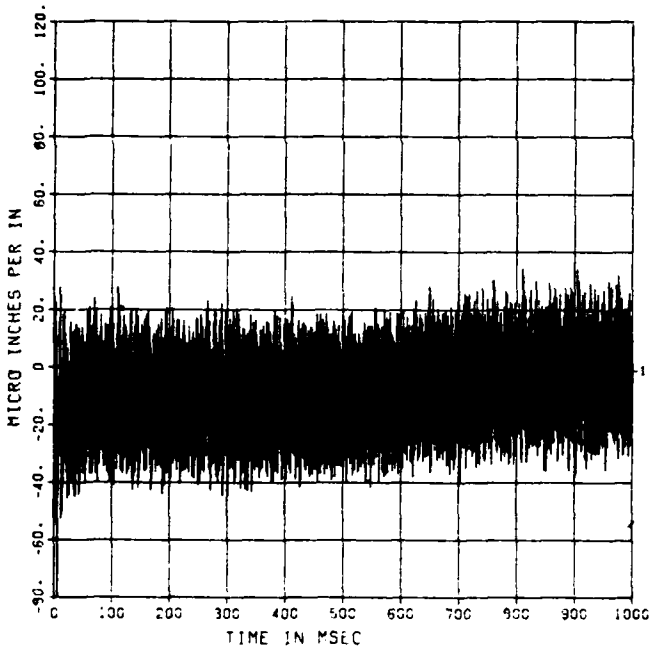
Line 2: Gage symbol and location (see Figure 4.2)

Line 3: Digitizing rate and calibration curve

Line 4: Filter (if any)

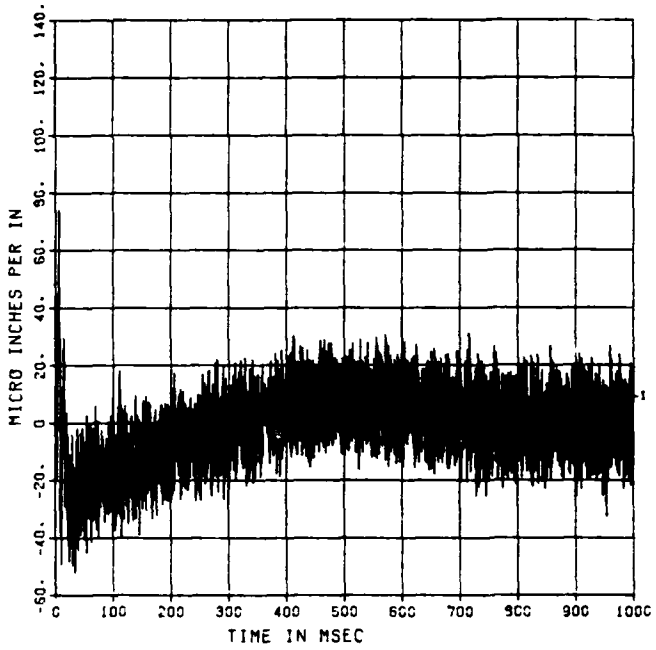
Line 5: Bookkeeping data

SLAB 2A POST UPGRADE
SSI TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 2 R0414



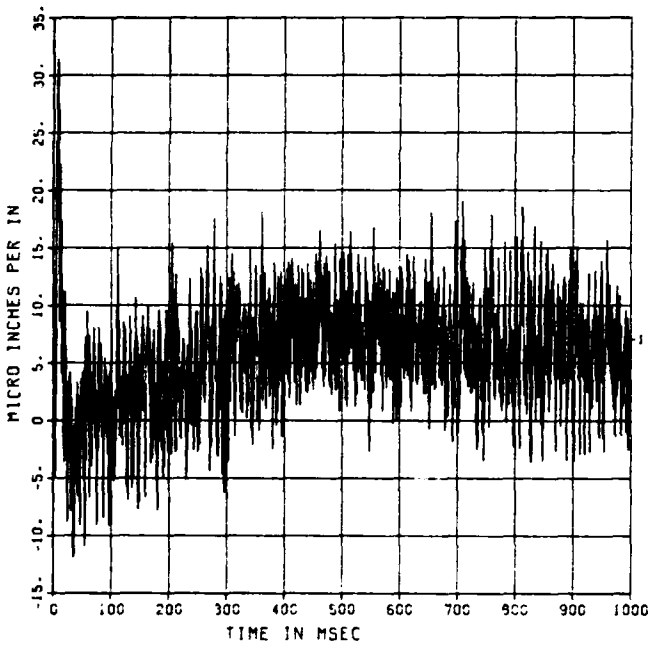
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SSI BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 1 R0414



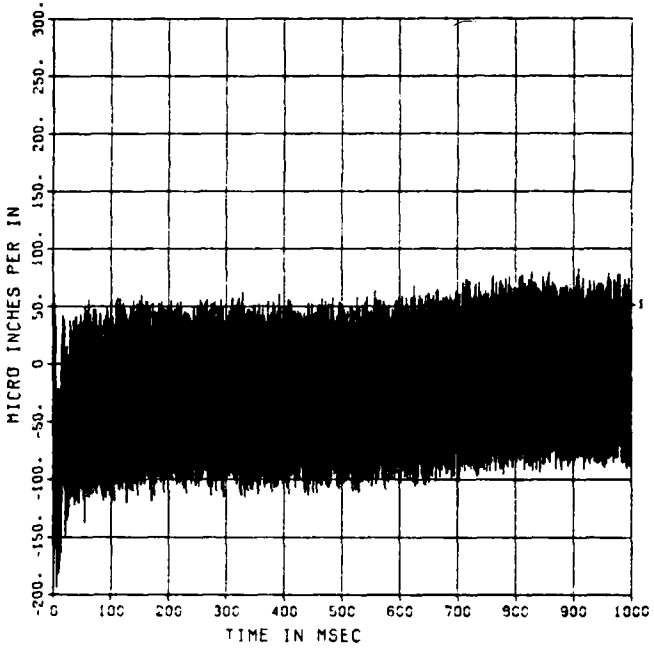
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS2 TOP
25000. HZ CAL 2099.
F4
06/25/79 4467 8 R0414



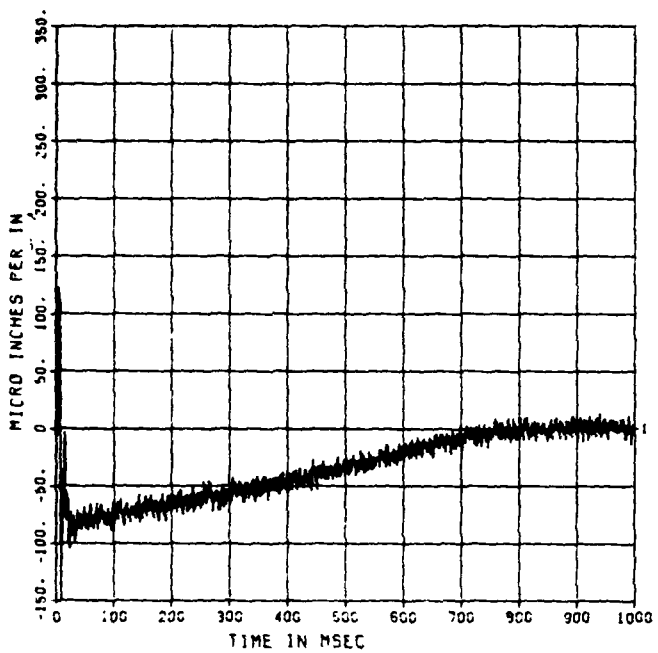
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS2 BOTTOM
25000. HZ CAL 2099.
F4
06/25/79 4467 7 R0414



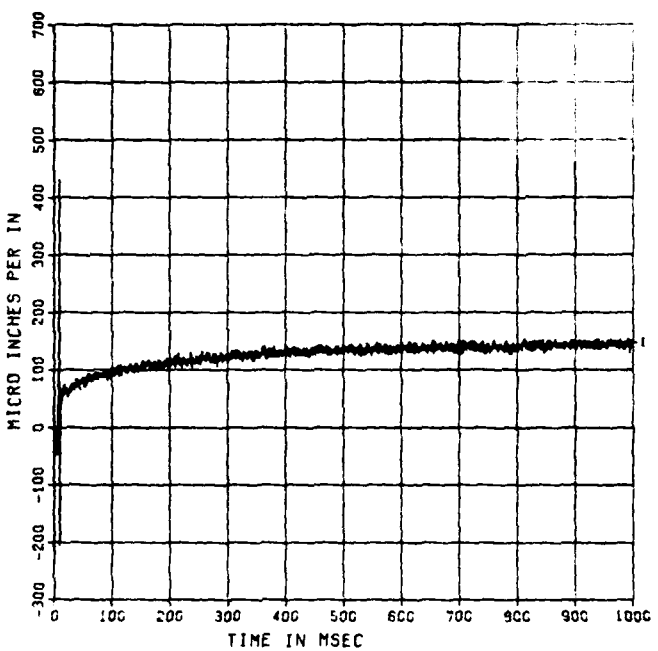
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS3 TOP
25000. HZ CAL 2999.
F4
06/25/79 4467 14 R0414



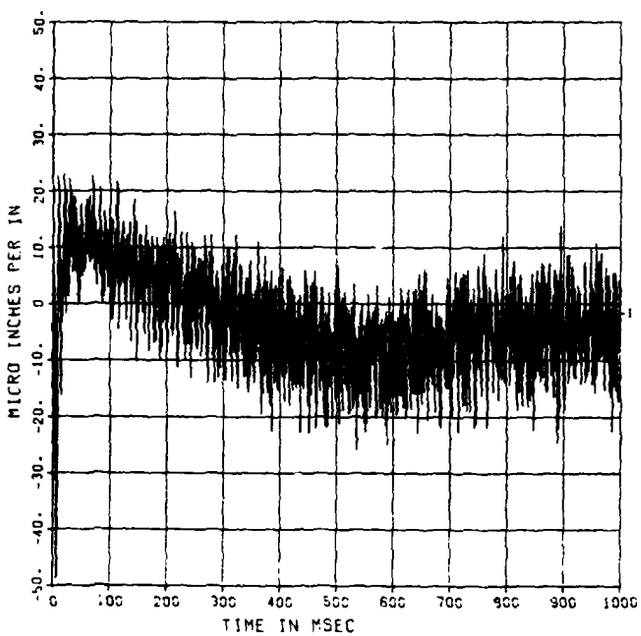
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS3 BOTTOM
25000. HZ CAL 2999.
F4
06/25/79 4467 13 R0414



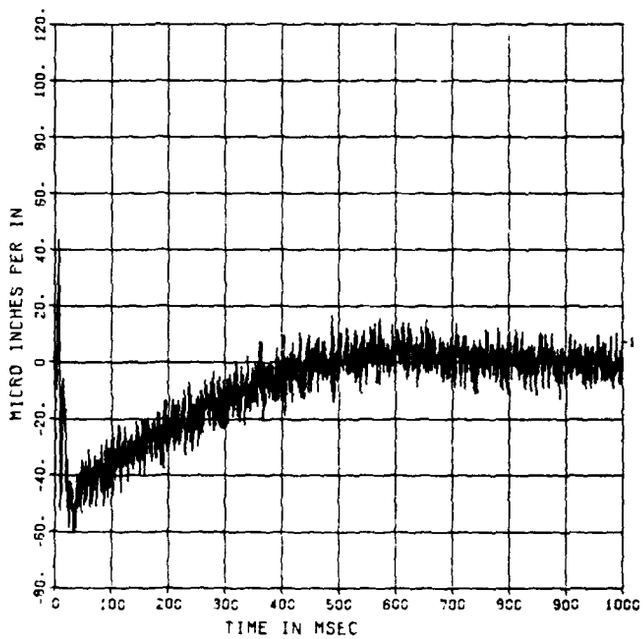
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS4 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 20 R0414



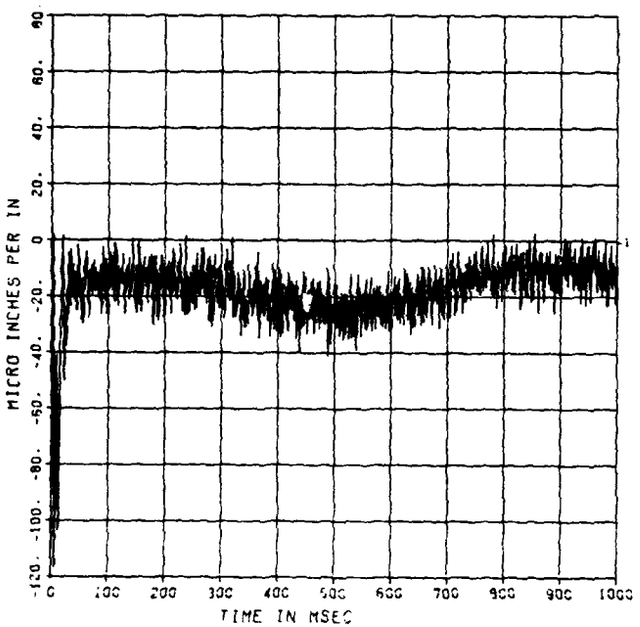
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS4 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 19 R0414



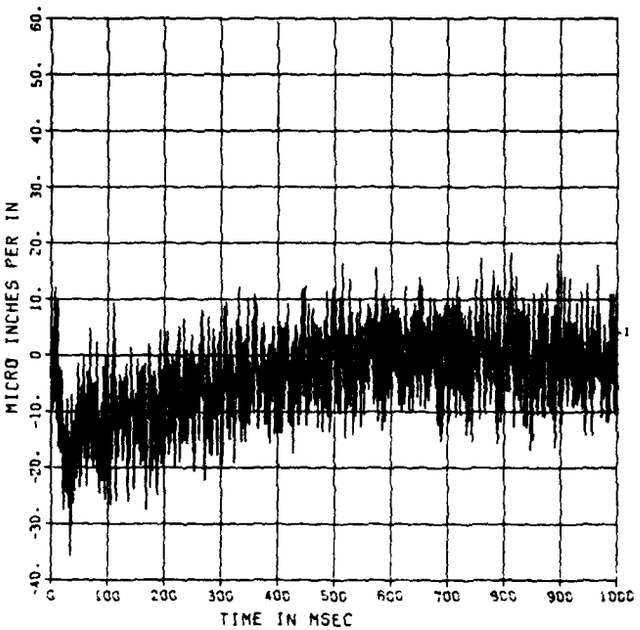
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS5 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 21 R0414



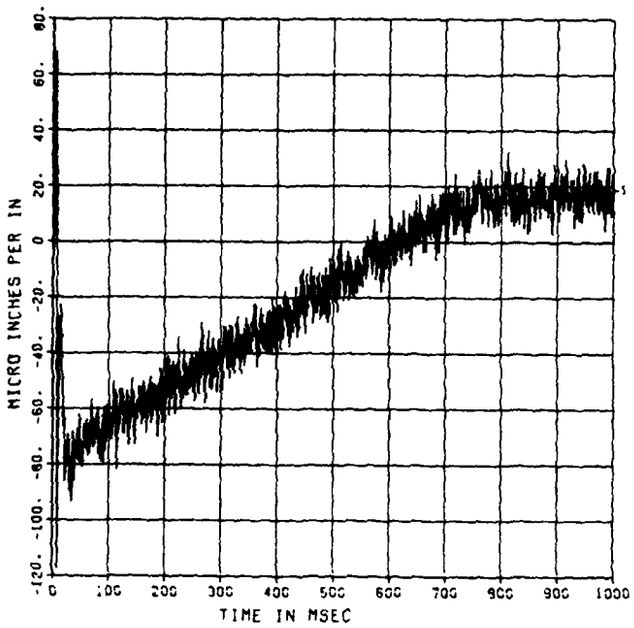
** PEAK VALUE IS 96 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS5 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 22 R0414



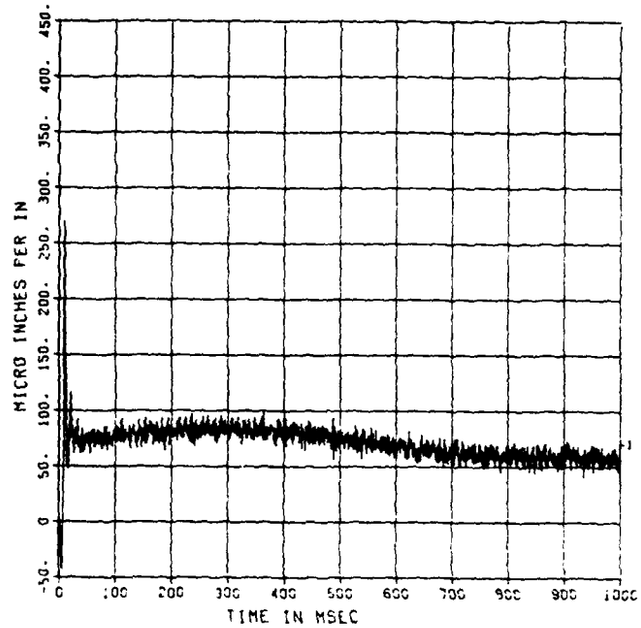
** PEAK VALUE IS 99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS6 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 24 RO414



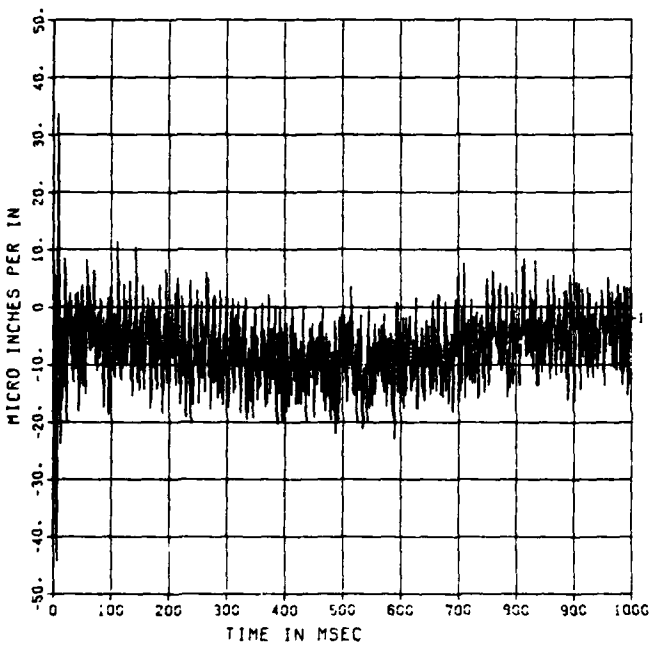
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS6 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 23 RO414



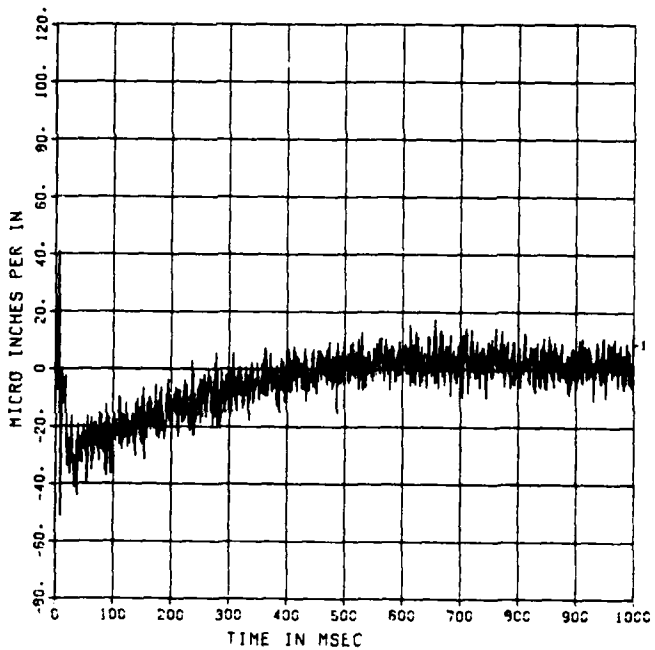
** PEAK VALUE IS -31 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS7 TOP
2500C. HZ CAL 2999.
F4
06/25/79 4467 26 R0414



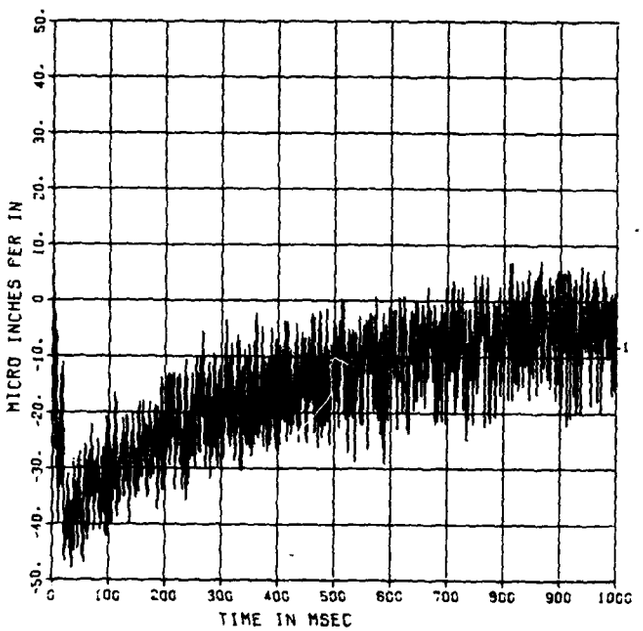
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS7 BOTTOM
2500C. HZ CAL 2899.
F4
06/25/79 4467 26 R0414



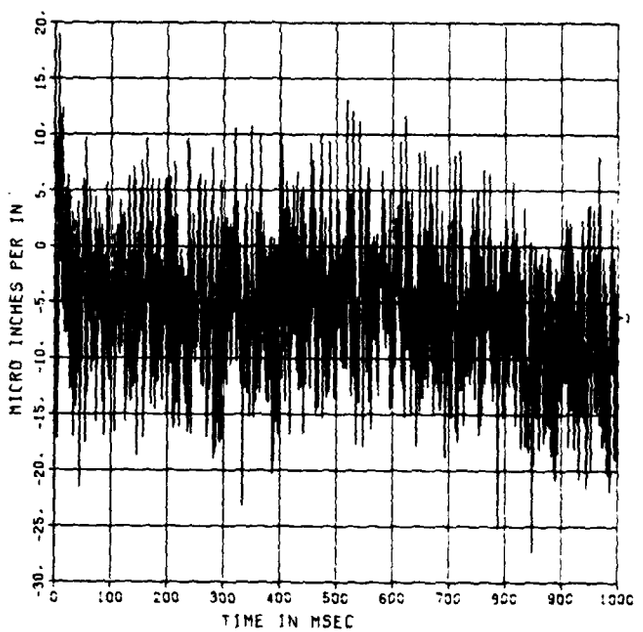
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS8 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 28 R0414



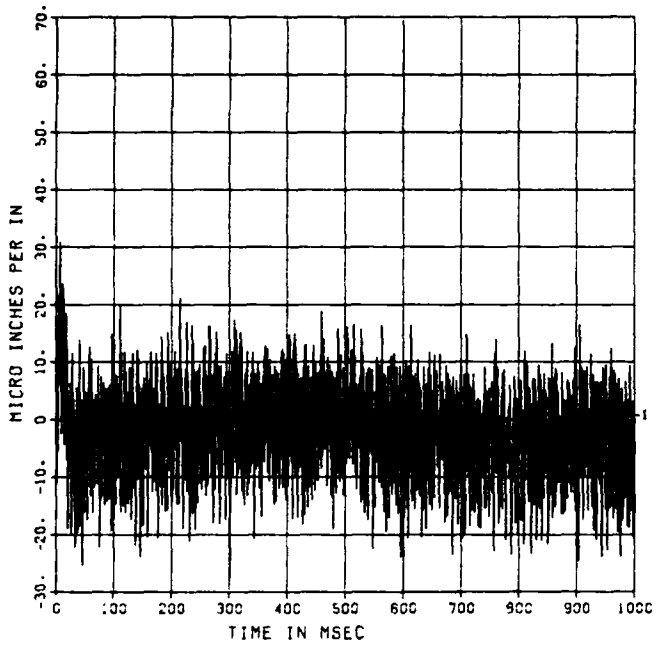
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SS8 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 27 R0414



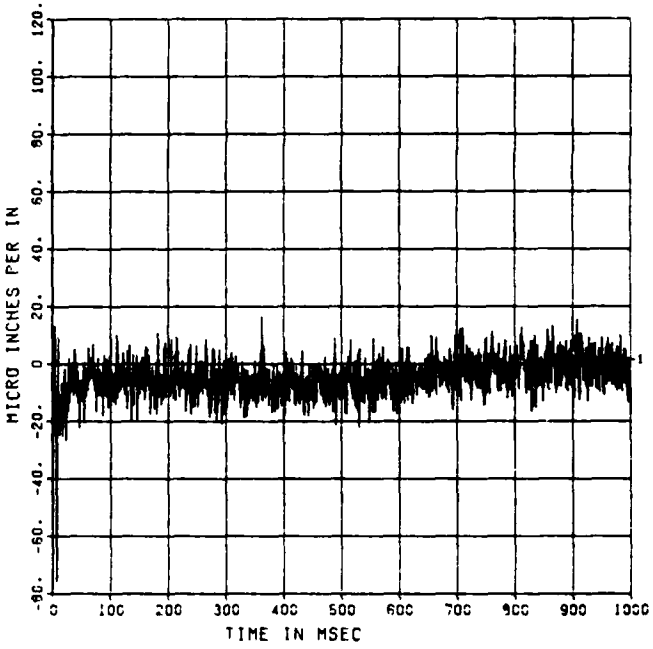
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SL1 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 4 R0414



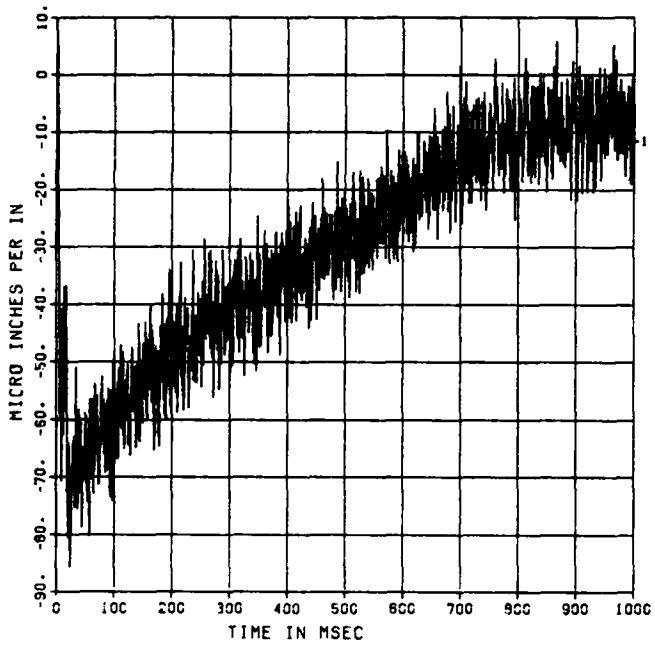
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SL1 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 3 R0414



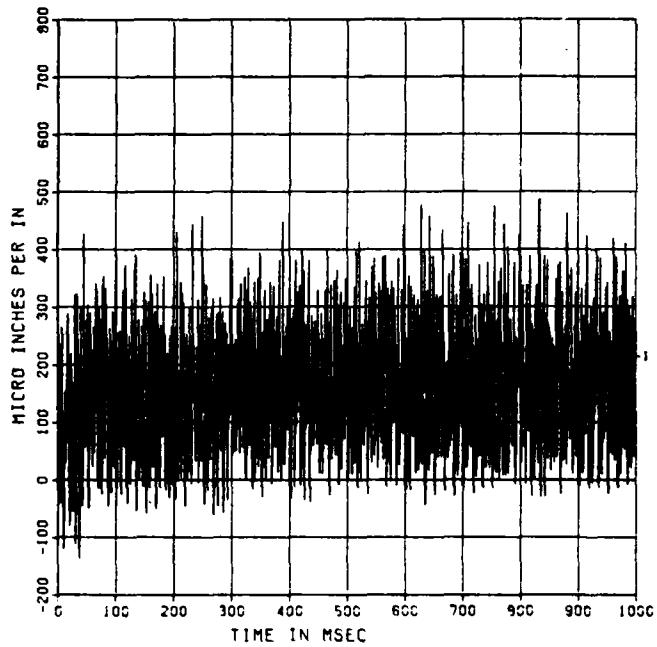
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SL2 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 10 R0414



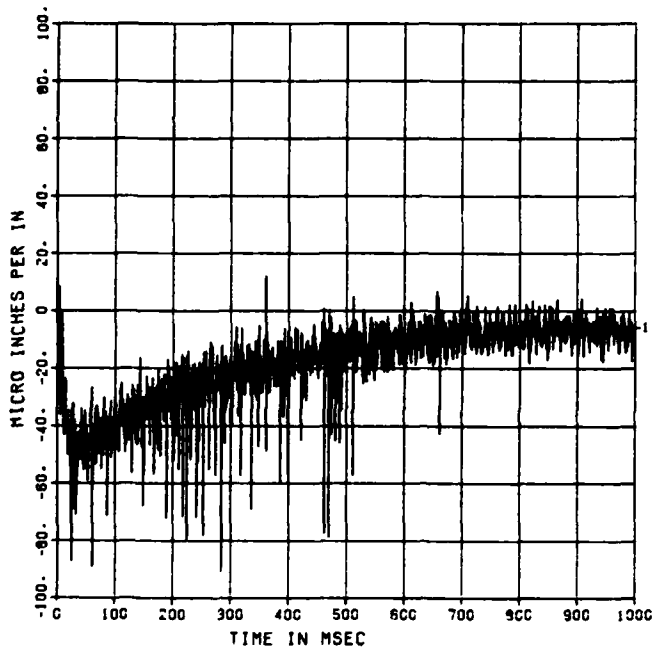
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SL2 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 9 R0414



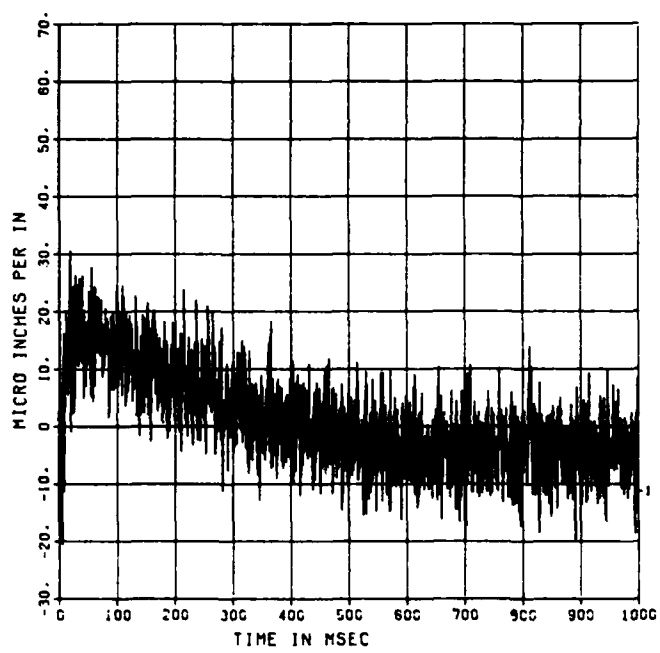
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SL3 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 16 R0414



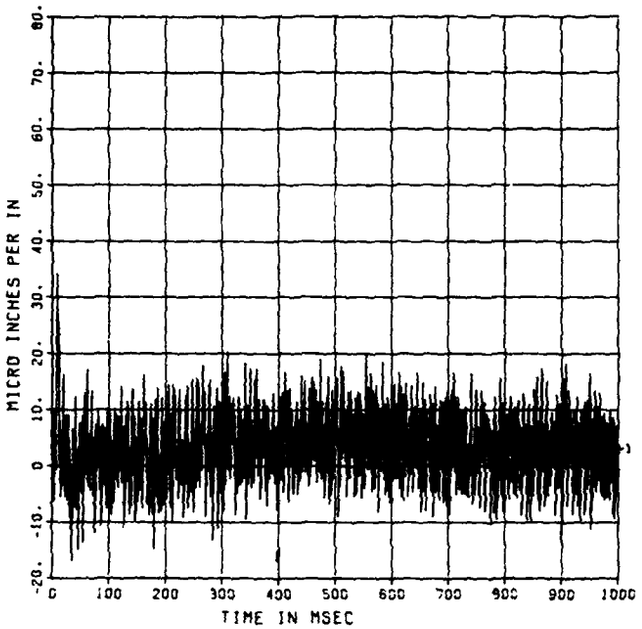
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
SL3 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 15 R0414



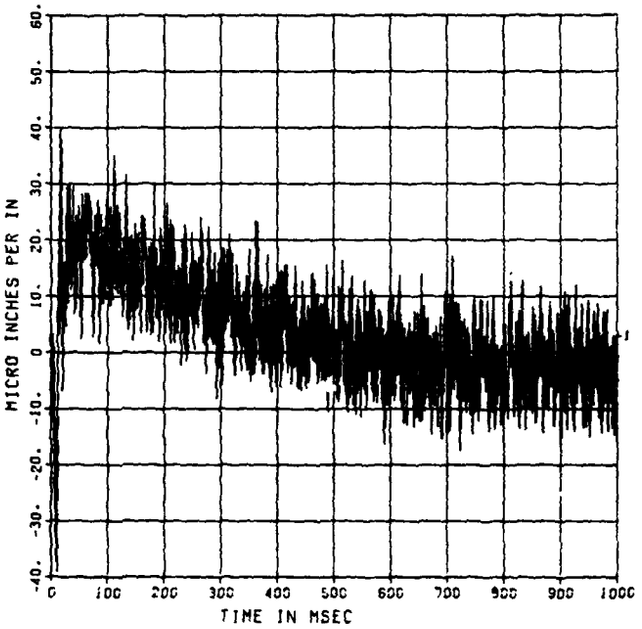
** PEAK VALUE IS 99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
B1 TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 6 R0414



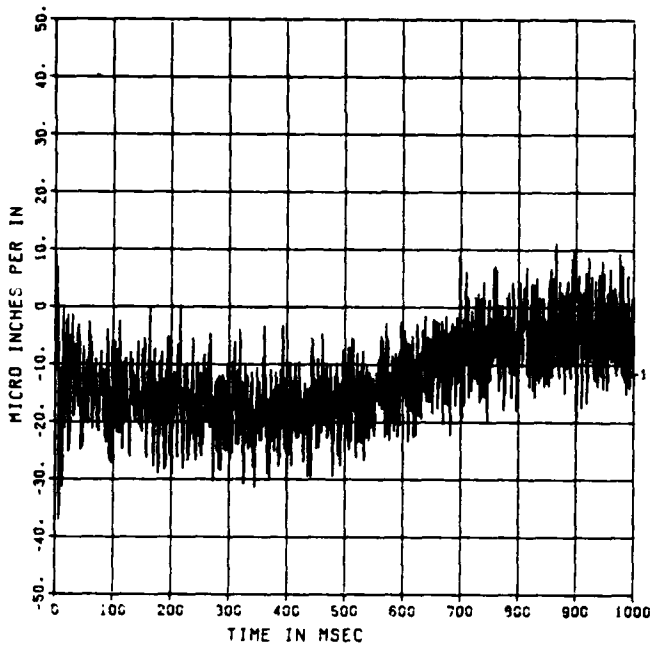
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
B1 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 5 R0414



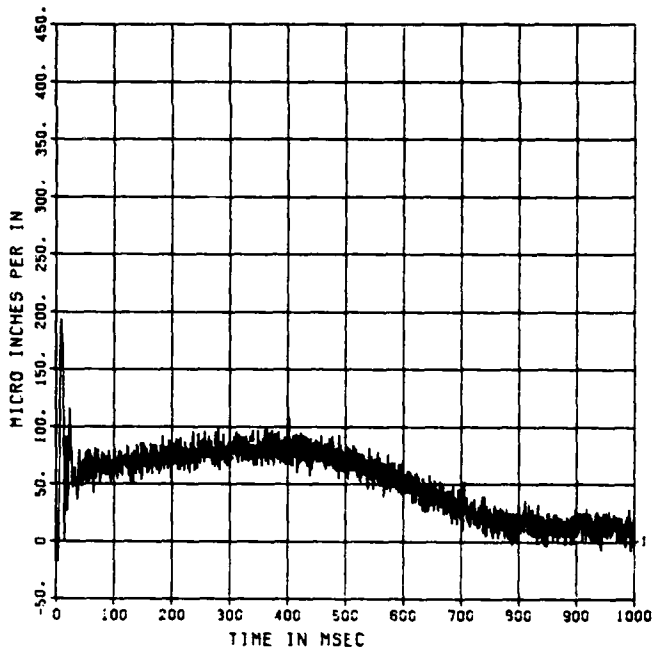
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
B2 TOP
25000. HZ CAL 2999.
F4
06/25/79 4467 12 R0414



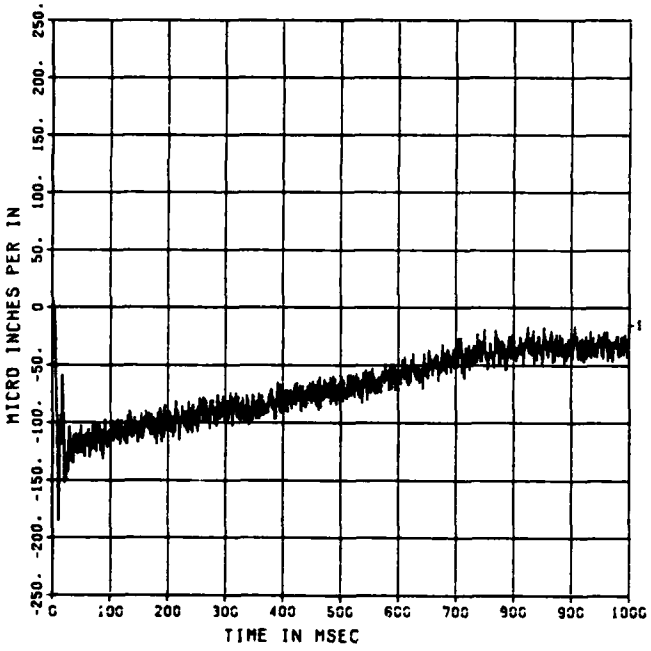
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
B2 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 11 R0414



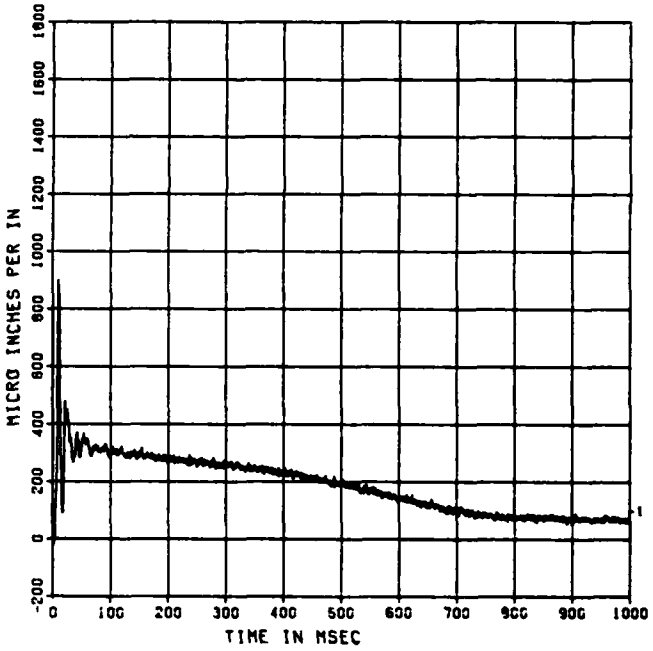
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
B3 TOP
2500C. HZ CAL 2999.
F4
06/25/79 4467 18 R0414

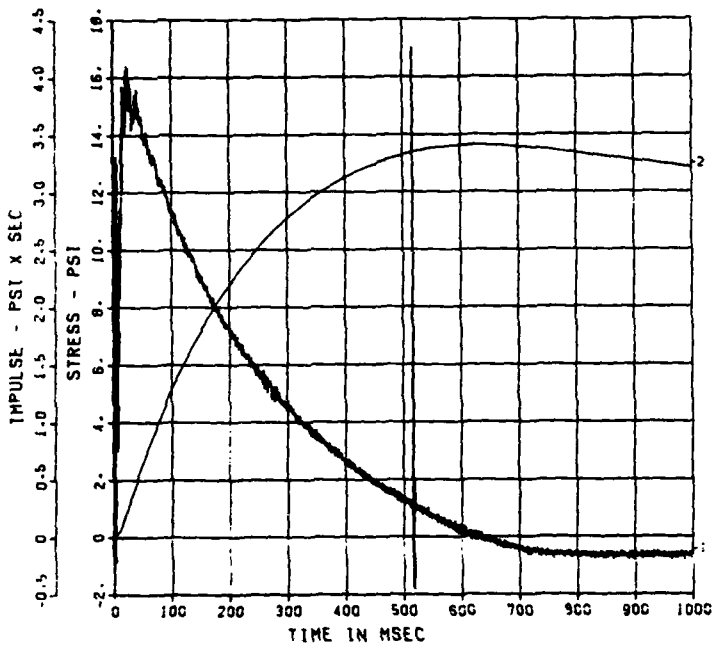


** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
B3 BOTTOM
2500C. HZ CAL 2999.
F4
06/25/79 4467 17 R0414

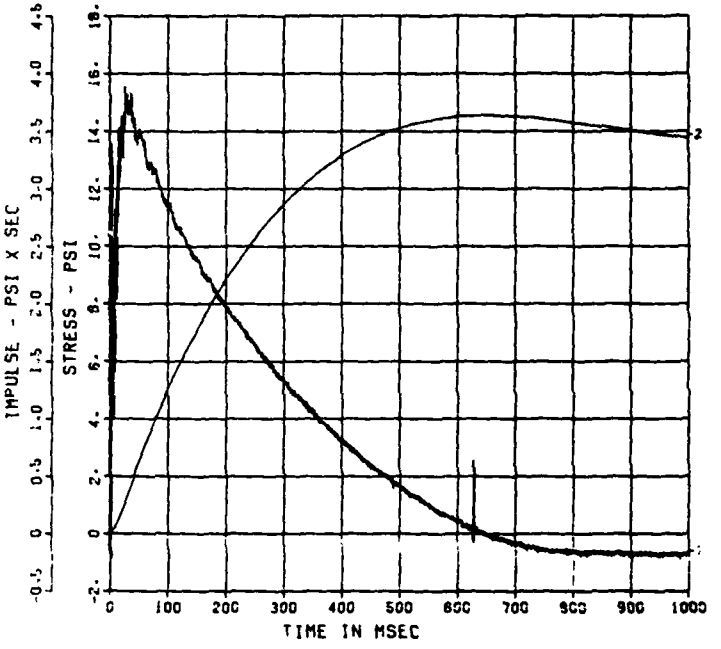


SLAB 2A POST UPGRADE
EP-1
25000. HZ CAL 24.9
F4
06/29/79 12521 33 R0617



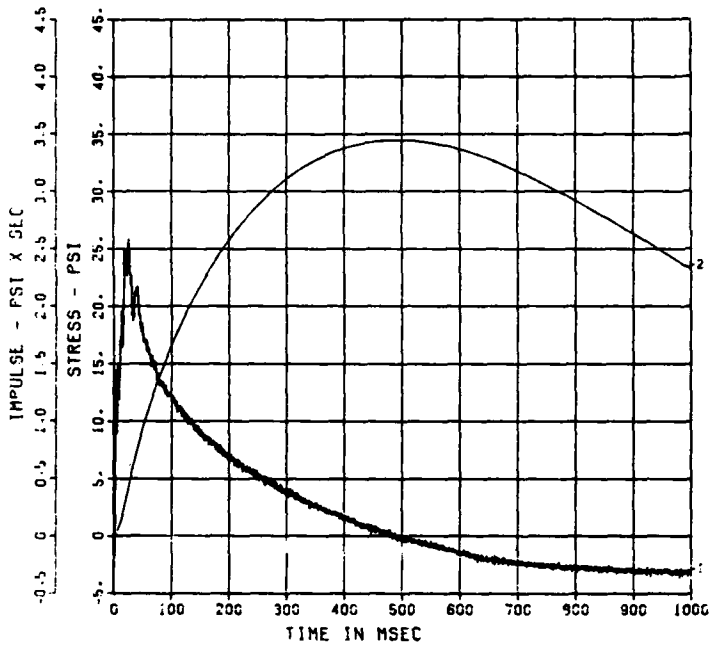
(4.)(.2) = -.8

SLAB 2A POST UPGRADE
EP-2
25000. HZ CAL 19.9
F4
06/29/79 12521 34 R0617



(4.)(.2) = -.8

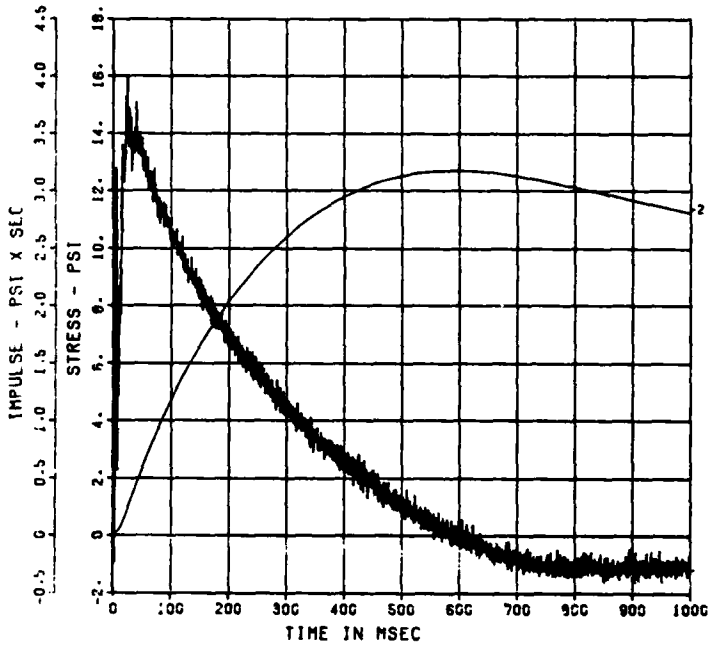
SLAB 2A POST UPGRADE
CP
25000. HZ CAL 20.4
F4
06/25/79 4467 29 R0414



(10.) (.35) = -3.5

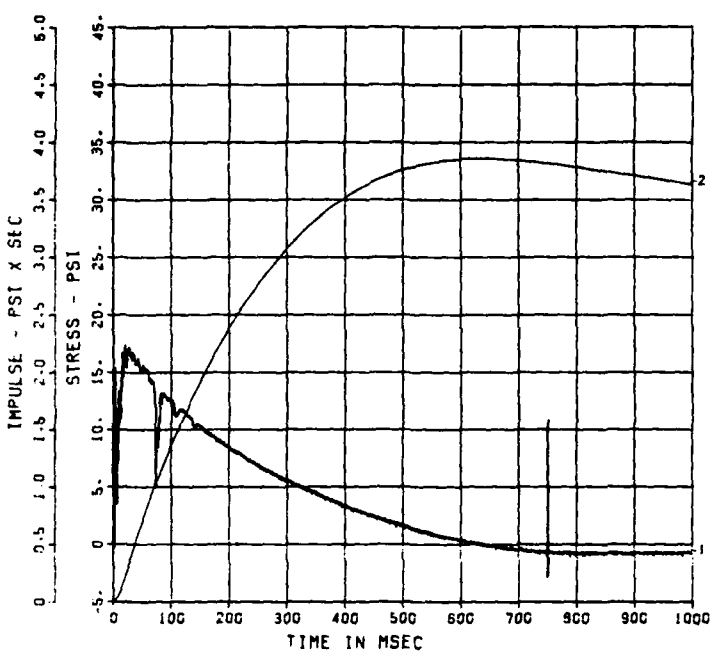
** PEAK VALUE IS 27 % OVER CALIBRATION **

SLAB 2A POST UPGRADE
WP-1
25000. HZ CAL 19.6
F4
06/25/79 4467 30 R0414



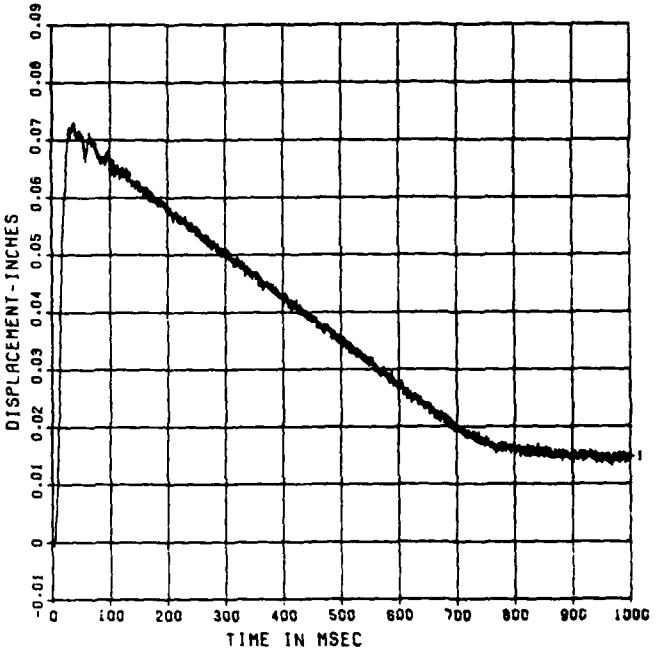
(4.) (.3) = -1.2

SLAB 2A POST UPGRADE
WP-2
25000. HZ CAL 22.0
F4
06/29/79 12521 31 R0617

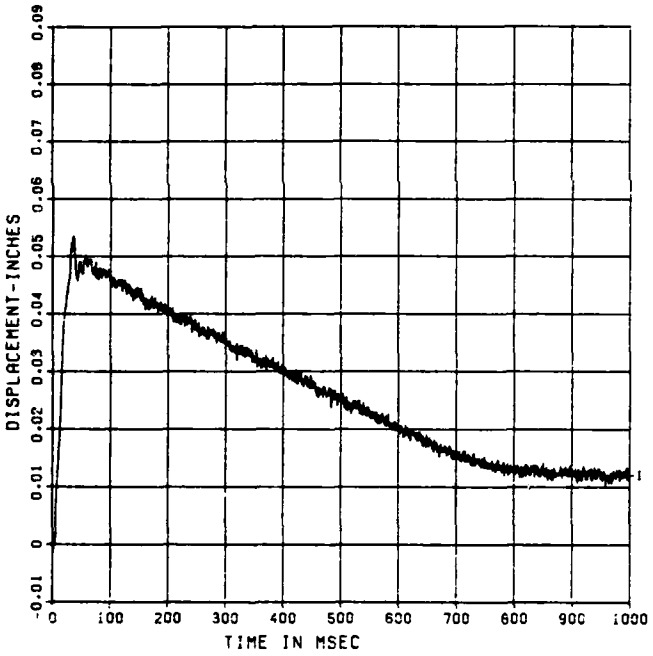


(10.)(.08) = -.8

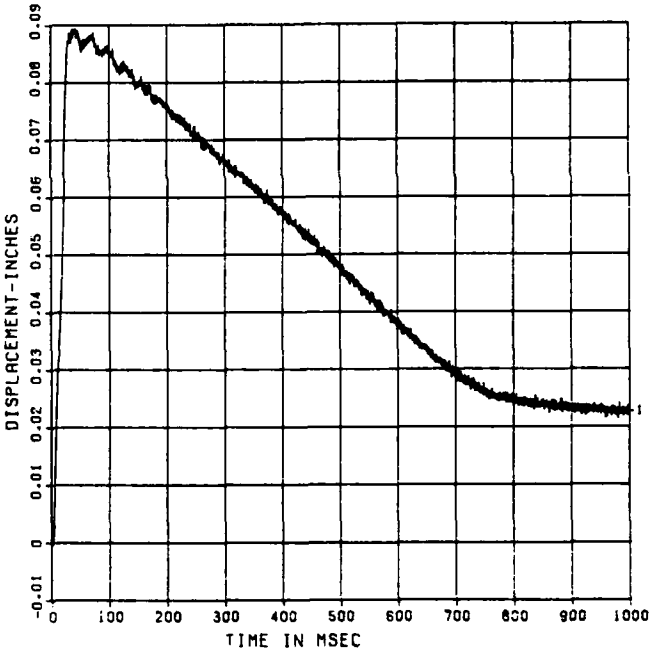
SLAB 2A POST UPGRADE
SBD
25000. HZ CAL 0.455
F4
07/03/79 12521 37 R0727



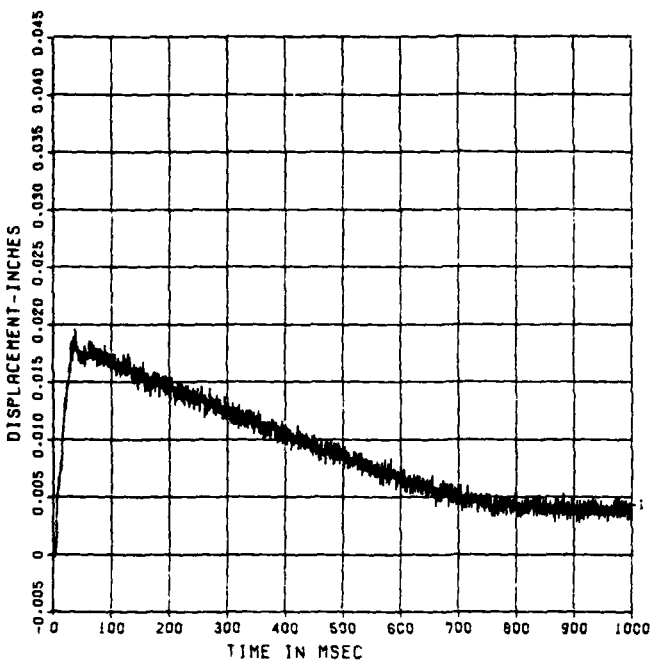
SLAB 2A POST UPGRADE
NBD
25000. HZ CAL 0.465
F4
07/03/79 12521 38 R0727



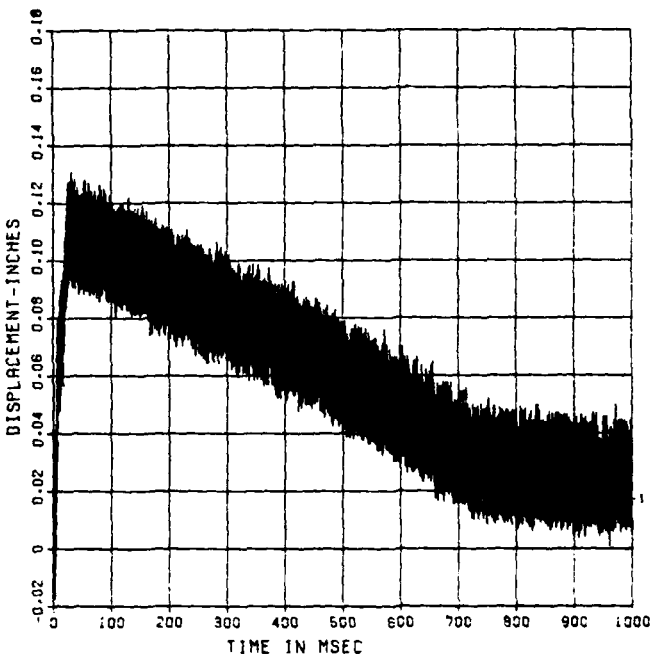
SLAB 2A POST UPGRADE
WBD
25000. HZ CAL 0.347
F4
07/03/79 12521 39 R0727



SLAB 2A POST UPGRADE
EBD
25000. HZ CAL 0.349
F4
07/03/79 12521 40 R0727

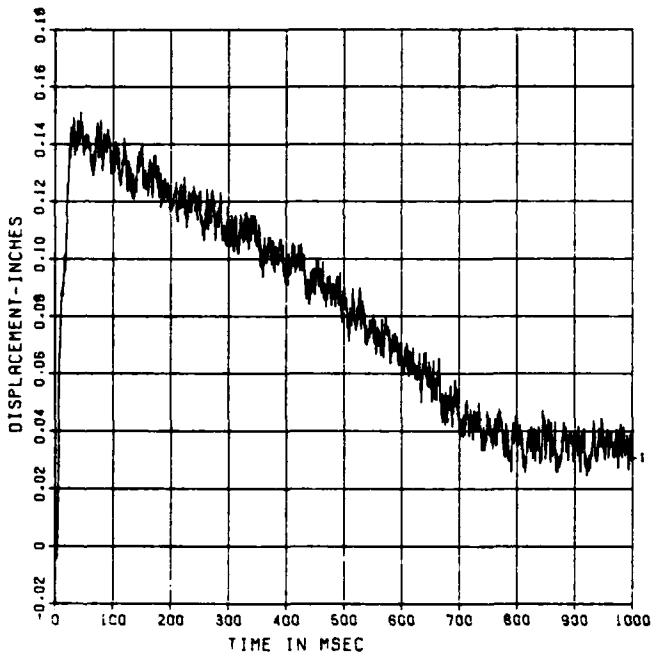


SLAB 2A POST UPGRADE
ED-1
25000. HZ CAL 1.44
F4
07/03/79 12521 41 R0727



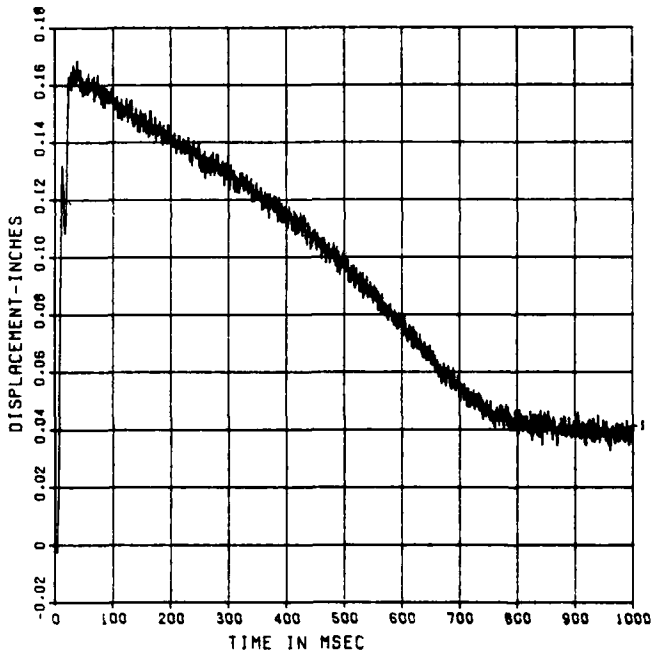
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
WD-1
25000. HZ CAL 1.30
F4
07/03/79 12521 42 R0727



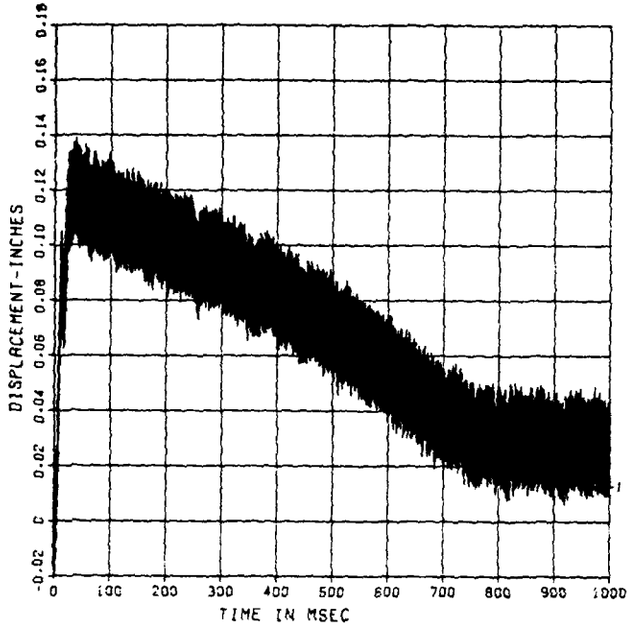
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
WD-2
25000. HZ CAL 1.28
F4
07/03/79 12521 43 R0727



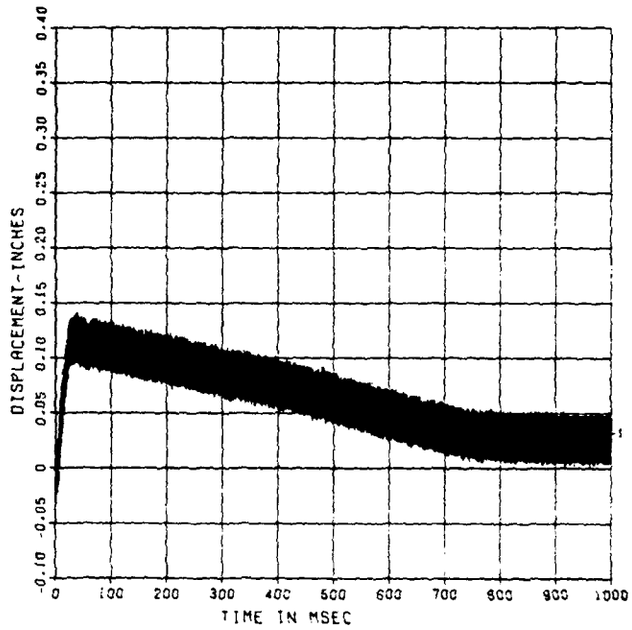
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
WD-3
25000. HZ CAL 1.42
F4
07/03/79 12521 44 R0727



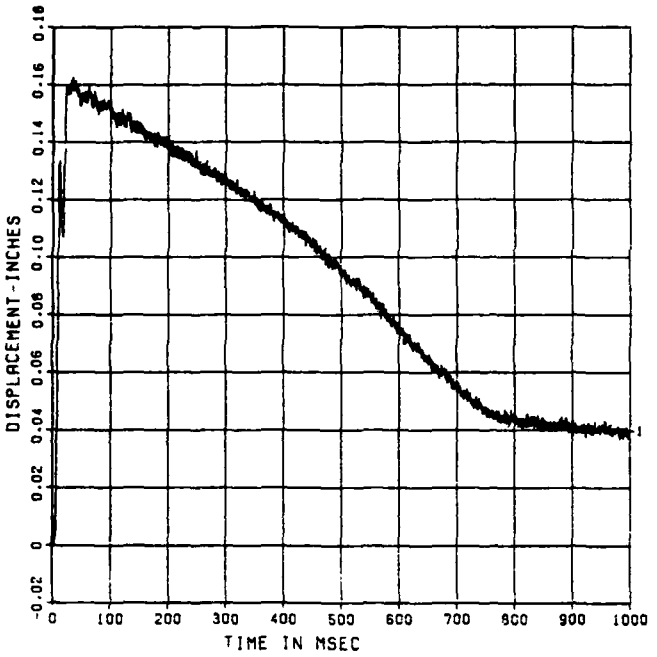
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 2A POST UPGRADE
WD-4
25000. HZ CAL 1.44
F4
07/03/79 12521 45 R0727

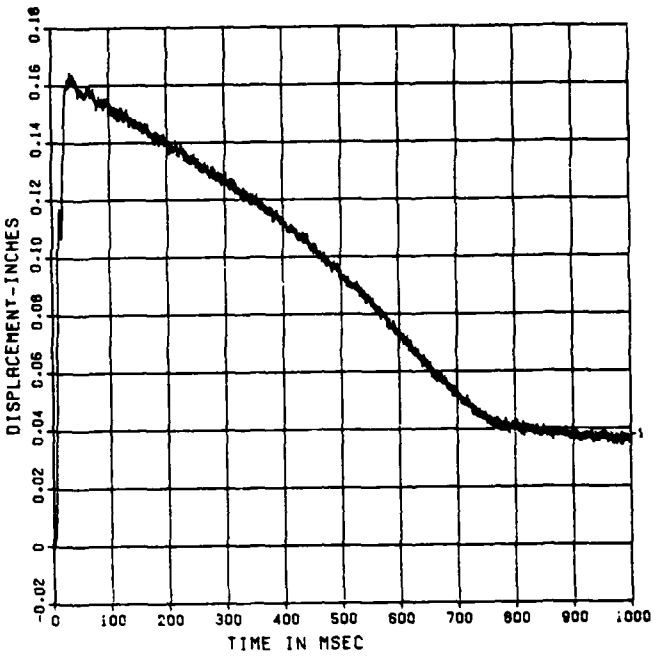


** PEAK VALUE IS -90 % UNDER CALIBRATION **

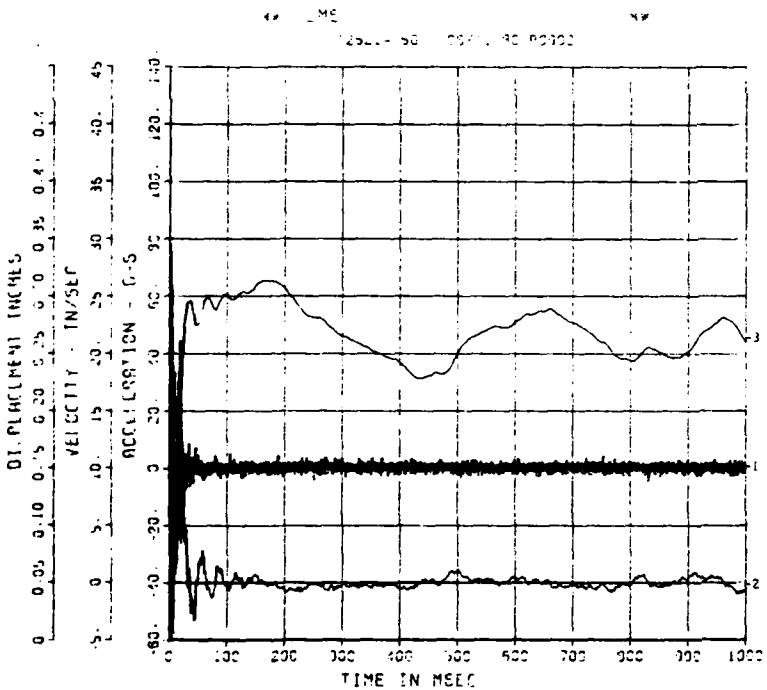
SLAB 2A POST UPGRADE
BD-1
25000. HZ CAL 0.755
F4
07/03/79 12521 46 R0727



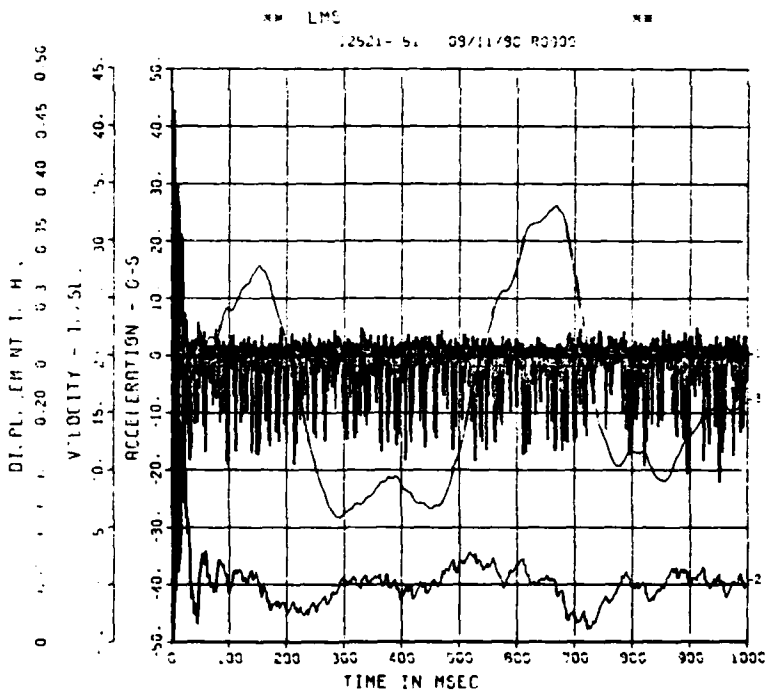
SLAB 2A POST UPGRADE
BD-2
25000. HZ CAL 0.733
F4
07/03/79 12521 47 R0727



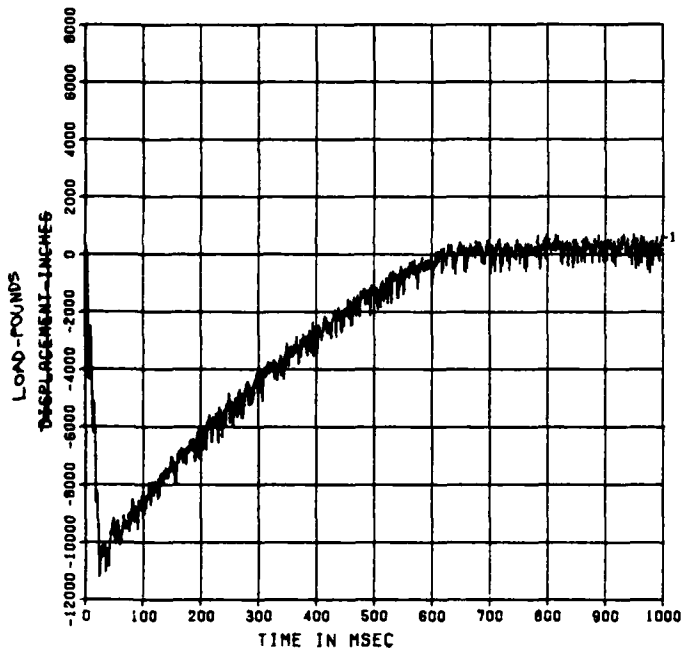
SLAB 2A POST UPGRADE
A1
25000. HZ CAL= 95.21



SLAB 2A POST UPGRADE
A2
25000. HZ CAL= 91.66

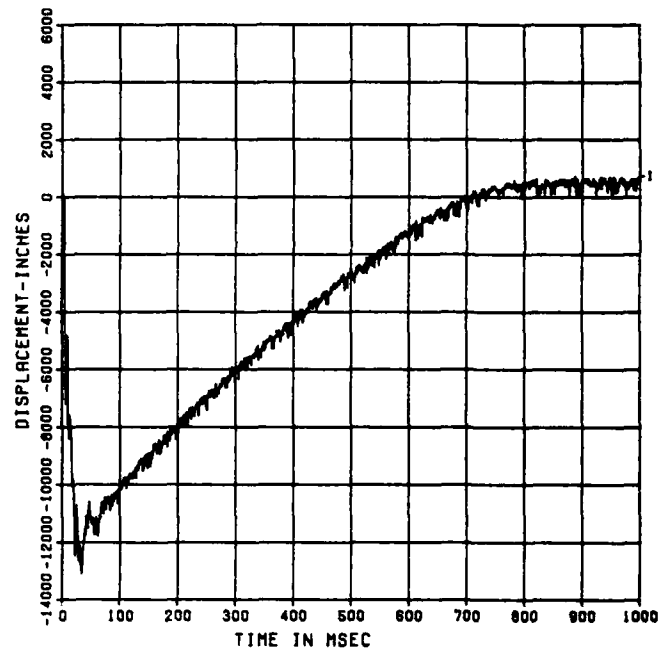


SLAB 2A POST UPGRADE
LOAD 1
25000. HZ CAL 9106.
F4
07/03/79 12521 48 R0727



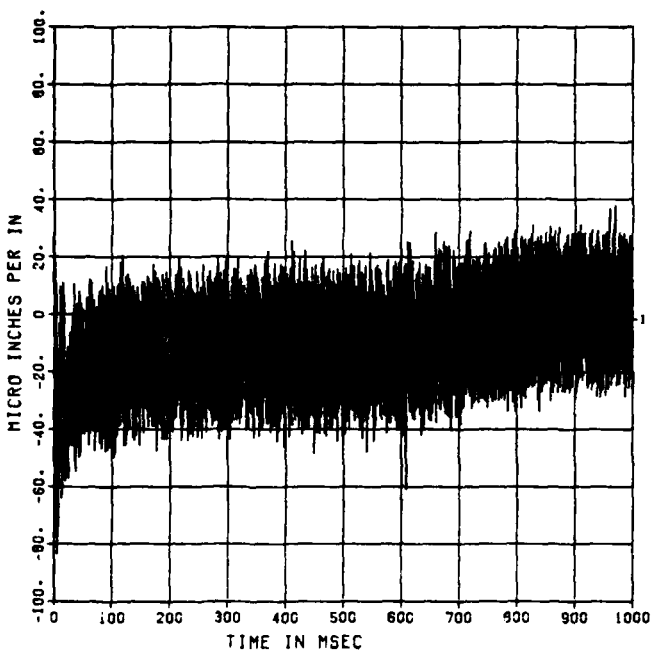
** PEAK VALUE IS 23 % OVER CALIBRATION **

SLAB 2A POST UPGRADE
LOAD 2
25000. HZ CAL 9115.
F4
07/03/79 12521 49 R0727



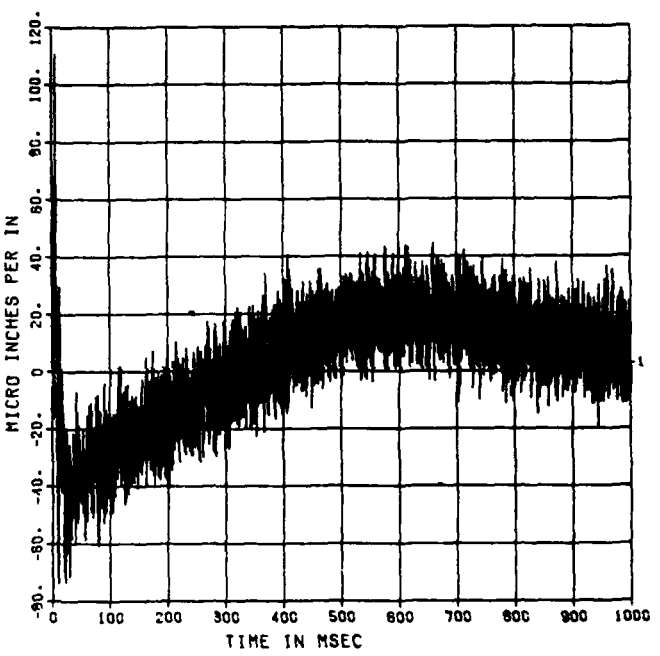
** PEAK VALUE IS 44 % OVER CALIBRATION **

SLAB 2B POST UPGRADE
SSI TOP
25000. HZ CAL 2899.
F4
06/25/79 4467 32 R0414



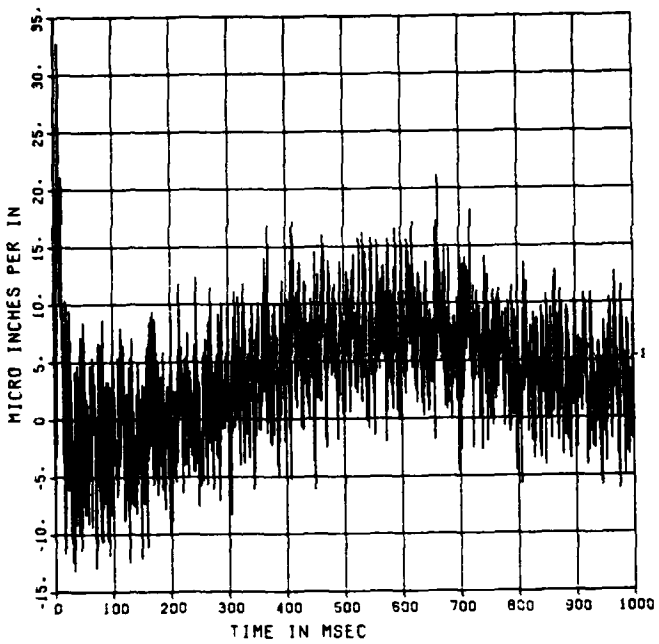
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SSI BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 31 R0414



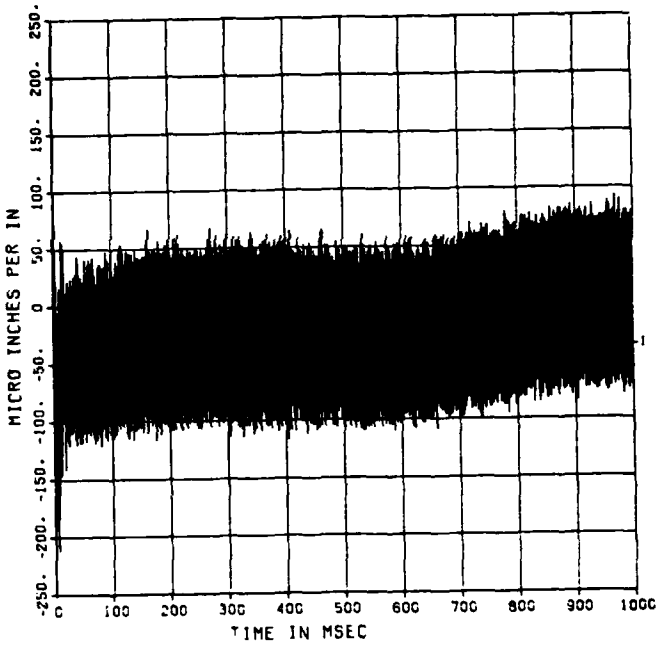
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS2 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 38 R0513



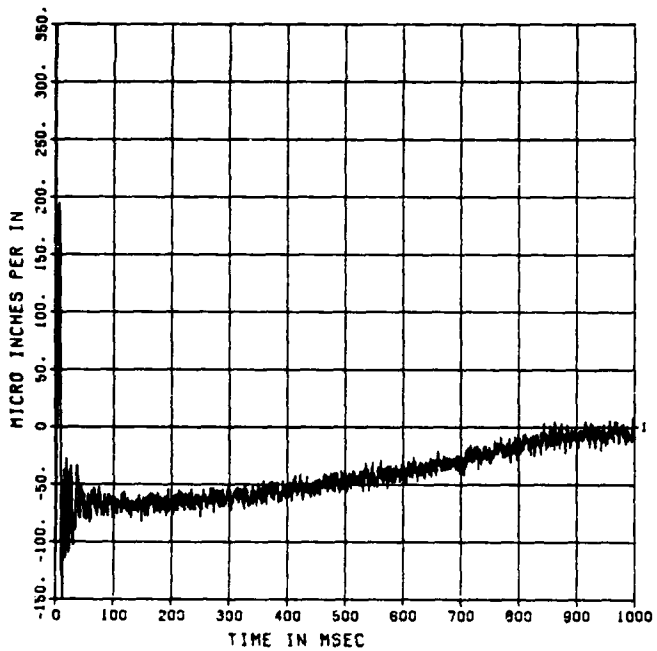
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS2 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 37 R0513



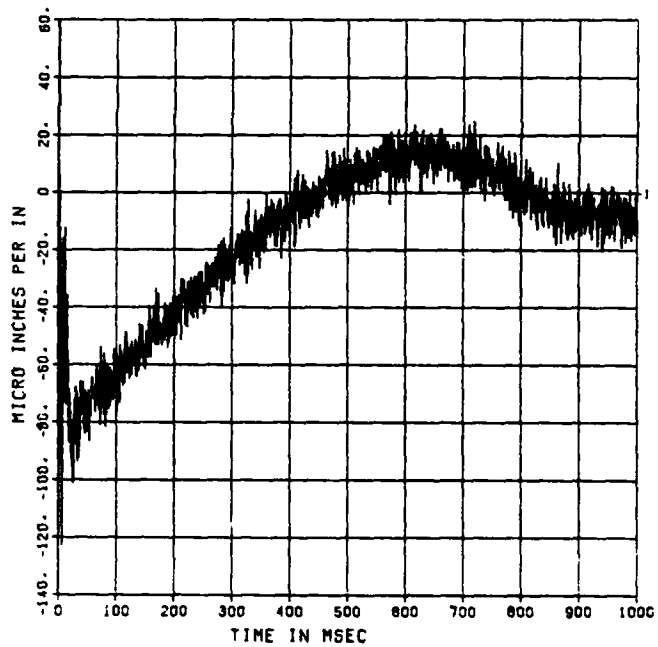
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS3 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 44 R0513



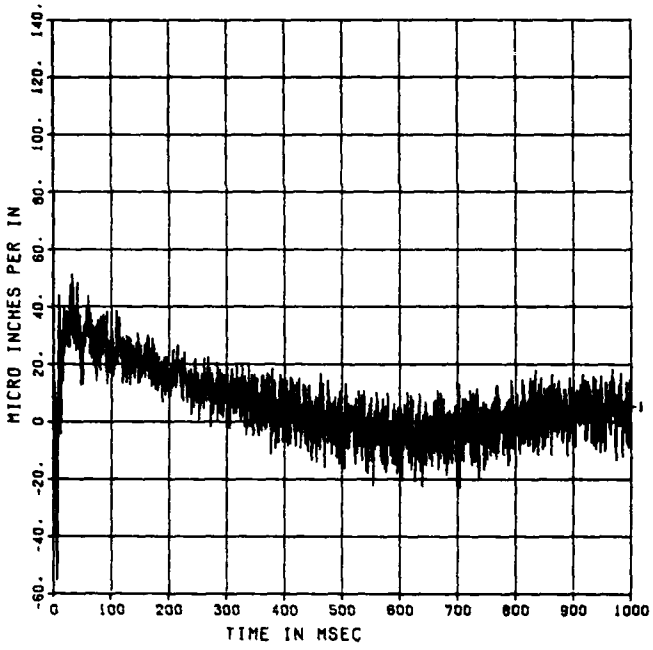
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS3 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 43 R0513



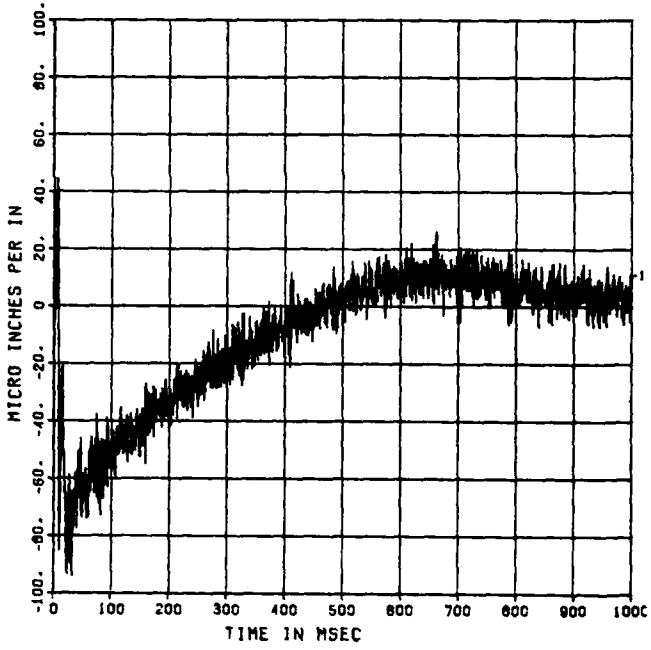
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS4 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 50 R0513



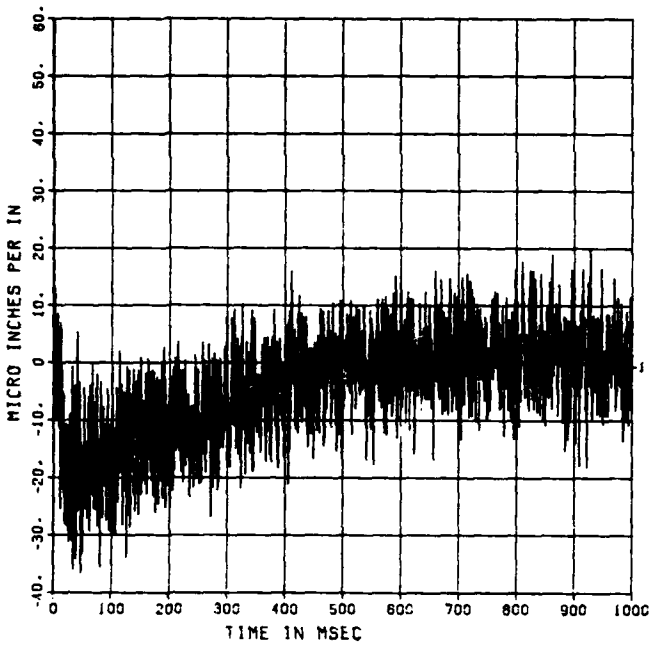
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS4 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 49 R0513



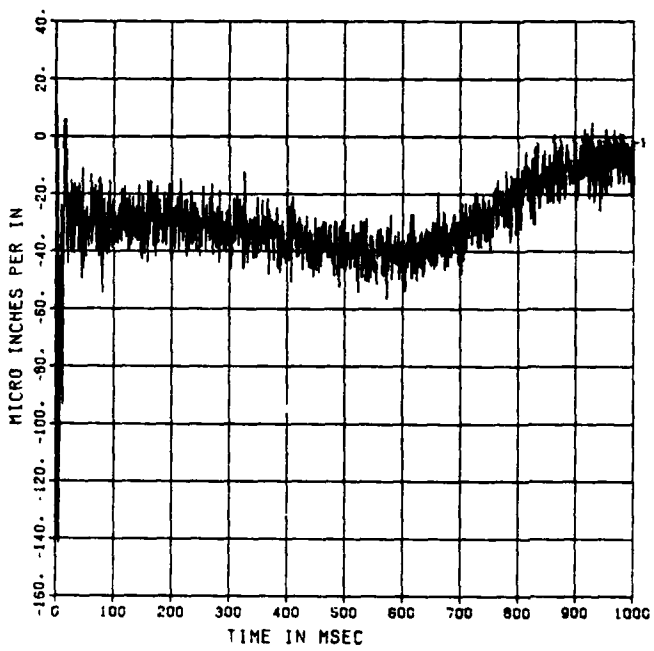
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS5 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 52 R05i3



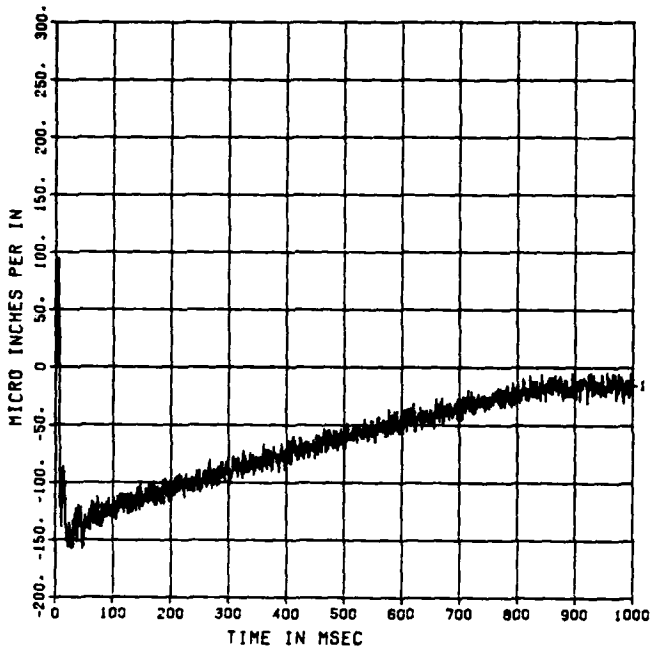
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS5 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4487 5i R05i3



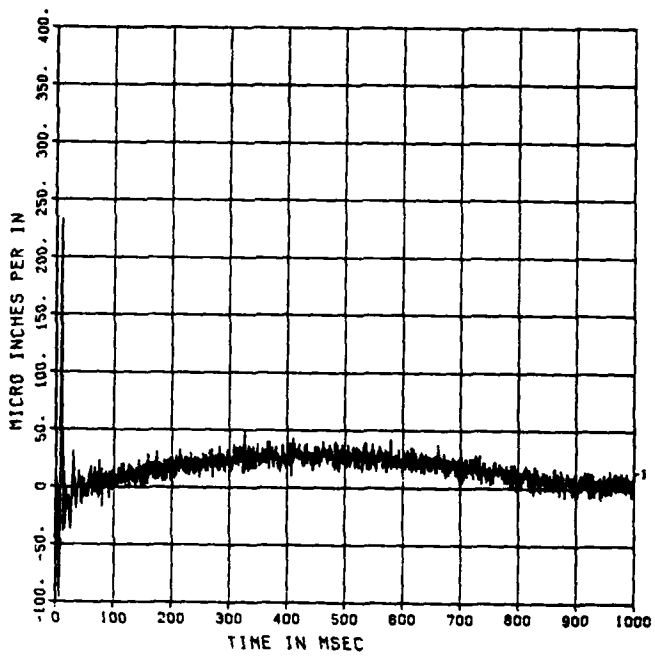
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS6 TOP
25000. HZ CAL 2099.
F4
06/27/79 4467 54 R0513



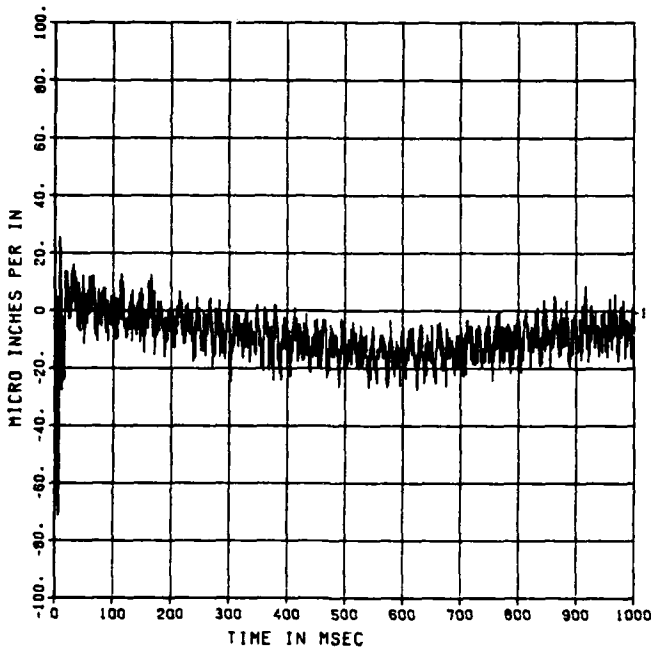
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS6 BOTTOM
25000. HZ CAL 2099.
F4
06/27/79 4467 53 R0513



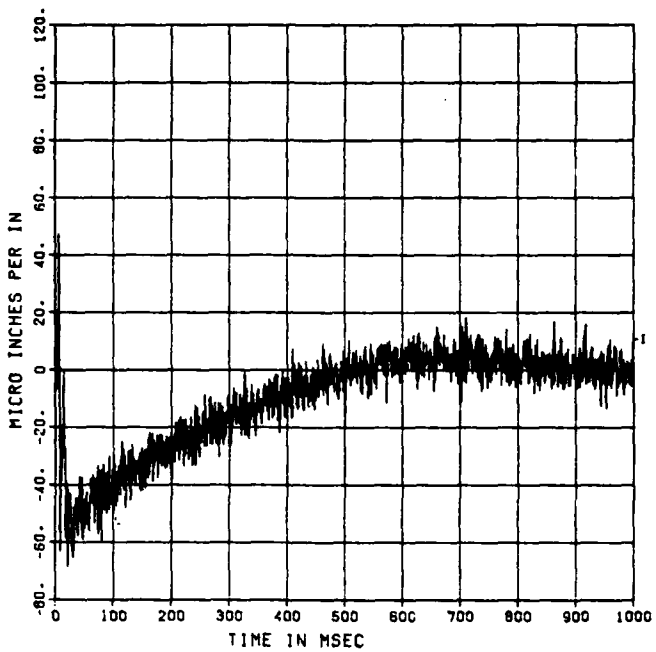
** PEAK VALUE IS -92 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS7 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 58 R0513



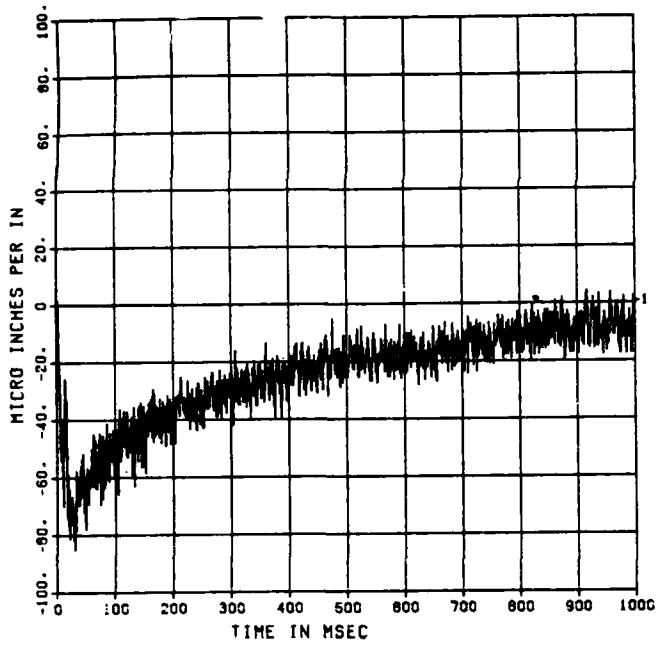
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS7 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 55 R0513



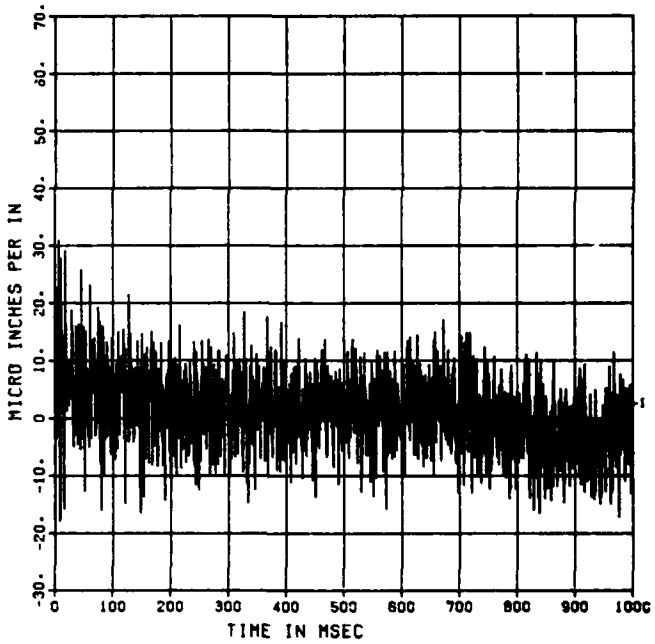
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS8 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 58 R0513



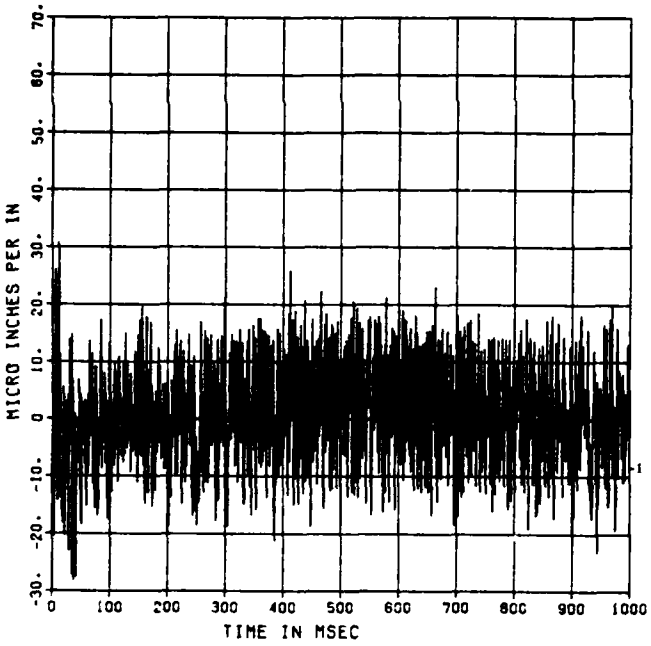
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SS8 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 57 R0513



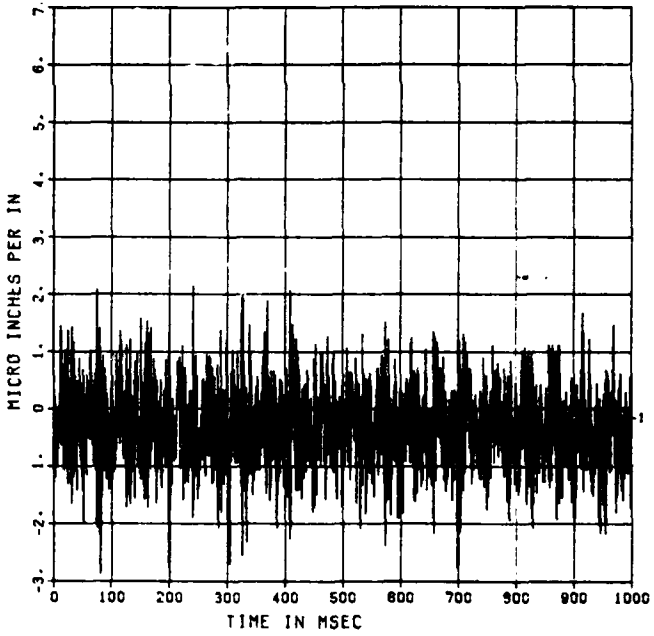
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SL1 TOP
25000. HZ CAL 2699.
F4
06/25/79 4467 34 R0414



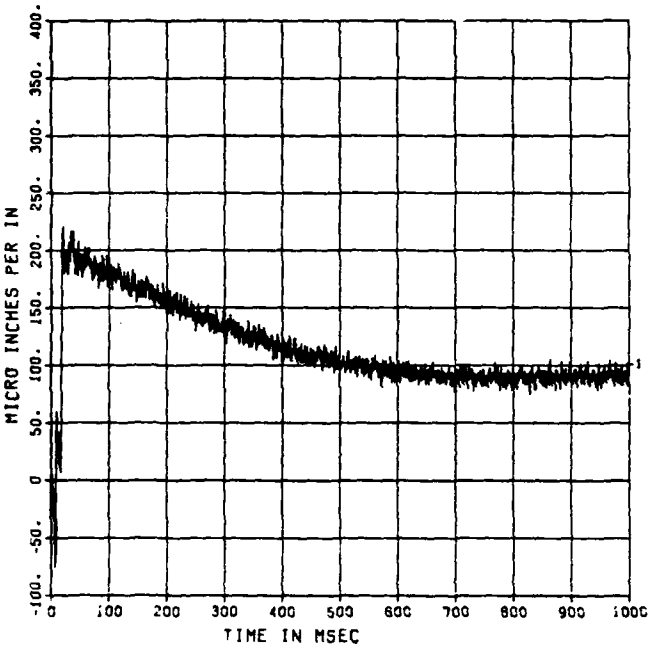
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SL1 BOTTOM
25000. HZ CAL 2699.
F4
06/25/79 4467 33 R0414



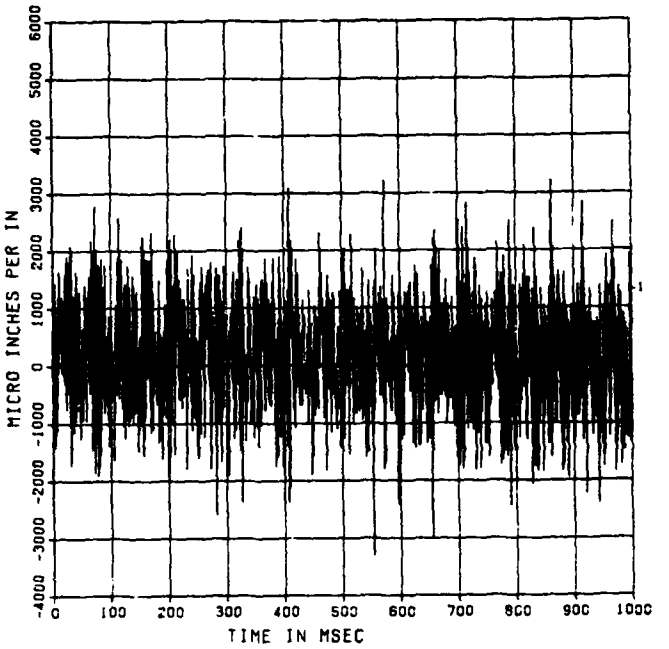
** PEAK VALUE IS -100 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SL2 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 40 R0513



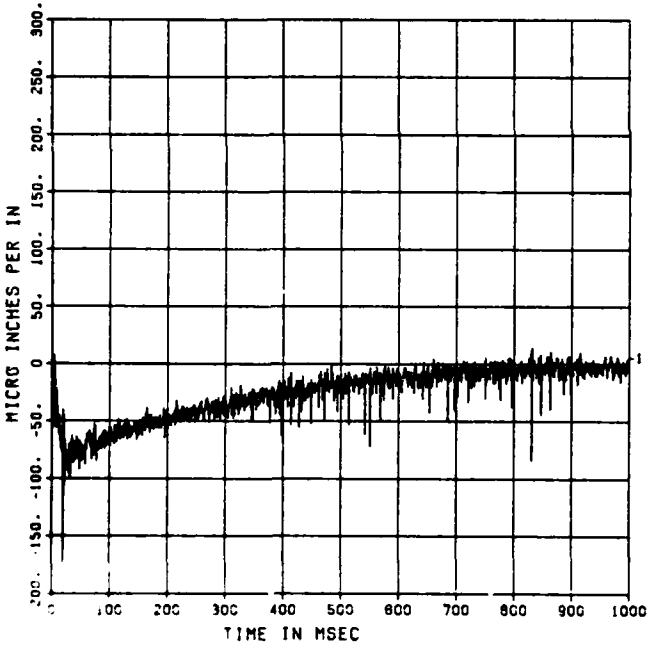
** PEAK VALUE IS -92 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
SL2 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 39 R0513



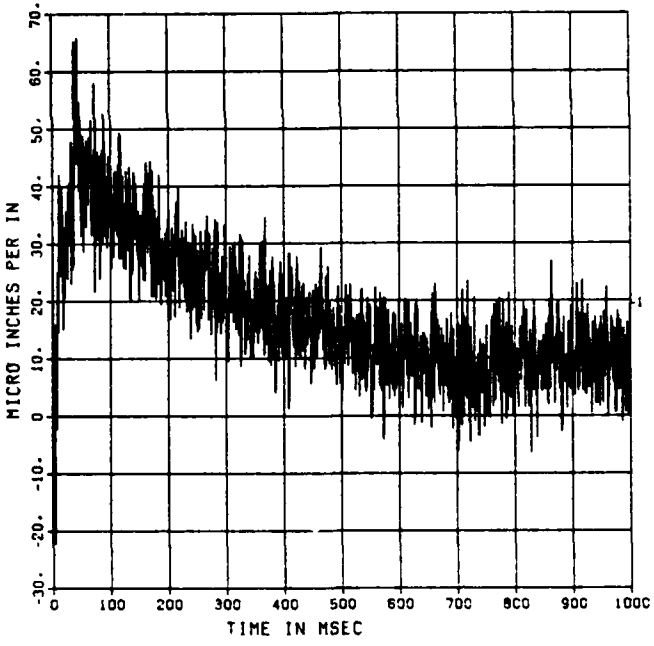
** PEAK VALUE IS 14 % OVER CALIBRATION **

SLAB 2B POST UPGRADE
SL3 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 46 R0513



** PEAK VALUE IS 94 % UNDER CALIBRATION **

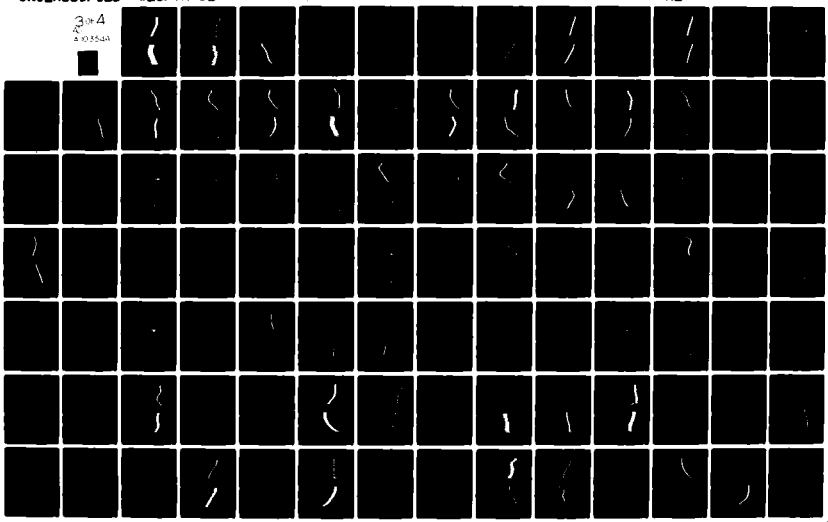
SLAB 2B POST UPGRADE
SL3 BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 45 R0513



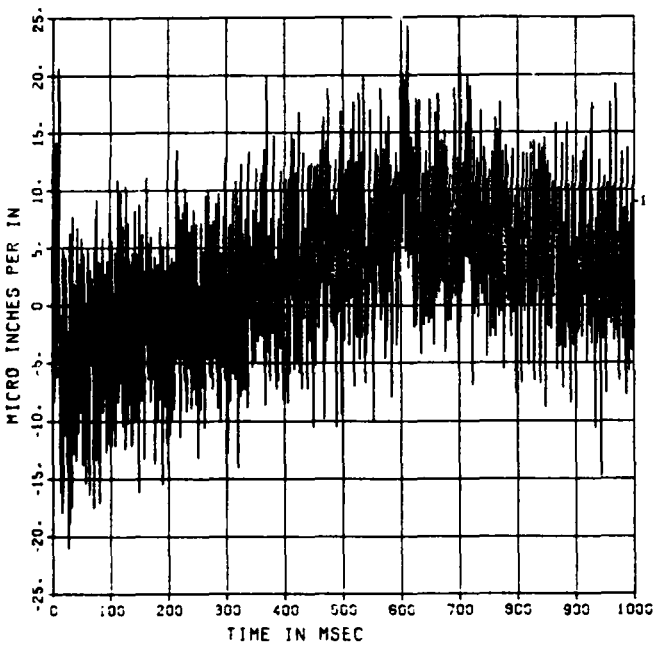
** PEAK VALUE IS -98 % UNDER CALIBRATION **

AD-A103 544 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR--ETC(U)
JUL 81 M K MCVAY DCPA01-78-C-0267
UNCLASSIFIED WES/TR/SL-81-4 NL

3 OF 4
AD
A103544

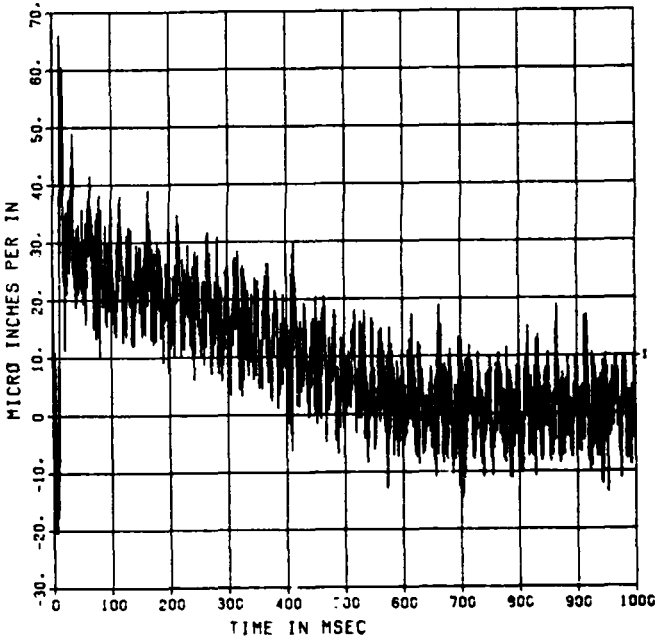


SLAB 2B POST UPGRADE
B1 TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 36 R0513



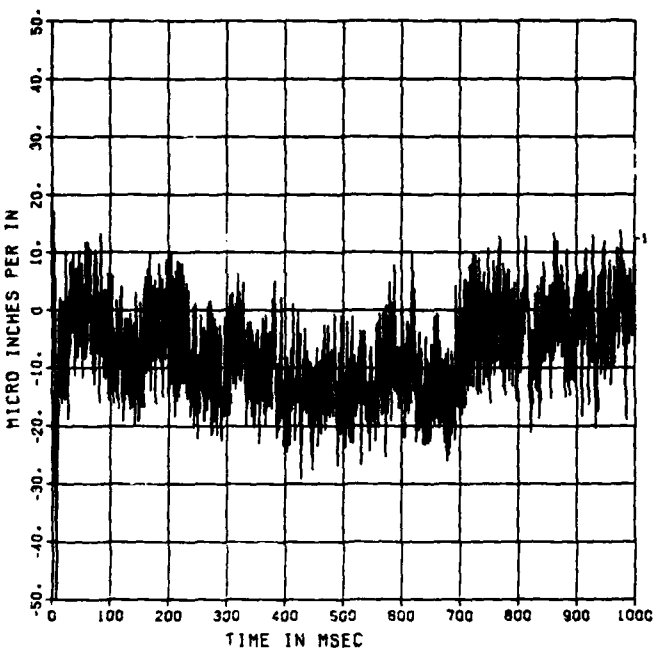
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
B1 BOTTOM
25000. HZ CAL 2899.
F4
06/25/79 4467 35 RU414



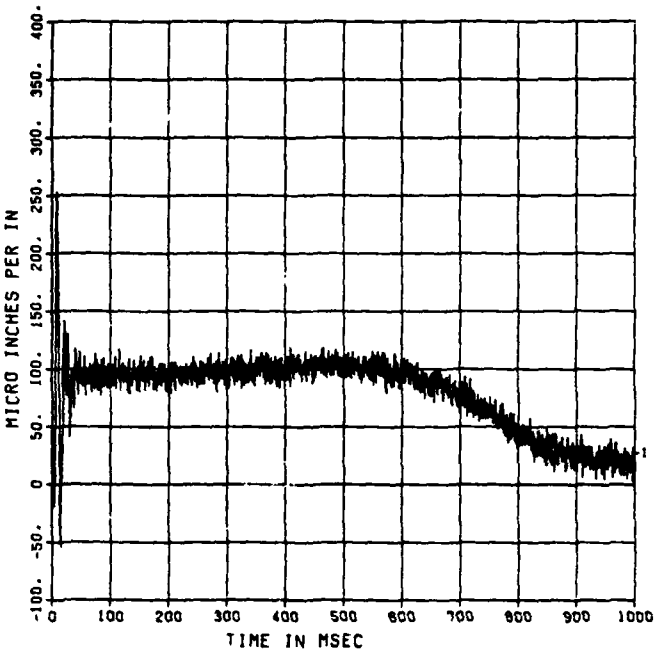
** PEAK VALUE IS 99 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
B2 TOP
25000. HZ CAL 2999.
F4
06/27/79 4467 42 R0513



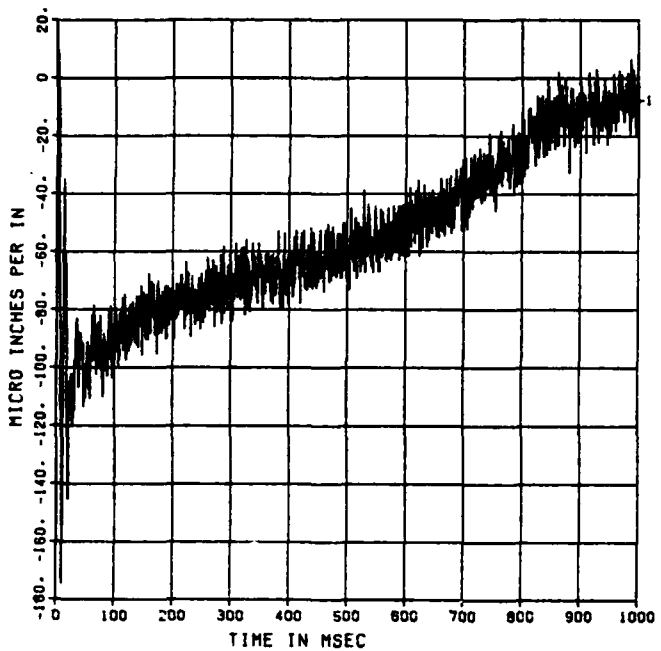
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
B2 BOTTOM
25000. HZ CAL 2999.
F4
06/27/79 4467 41 R0513



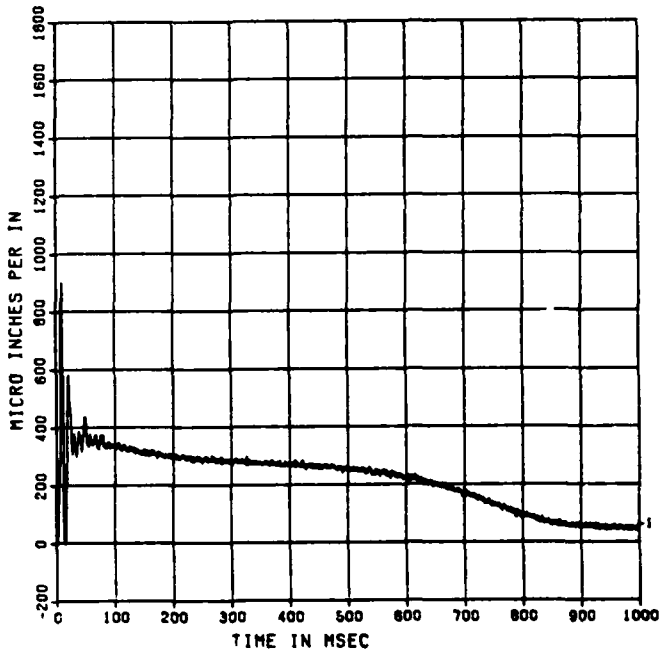
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
B3 TOP
25000. HZ CAL 2699.
F4
06/27/79 4467 48 R0513

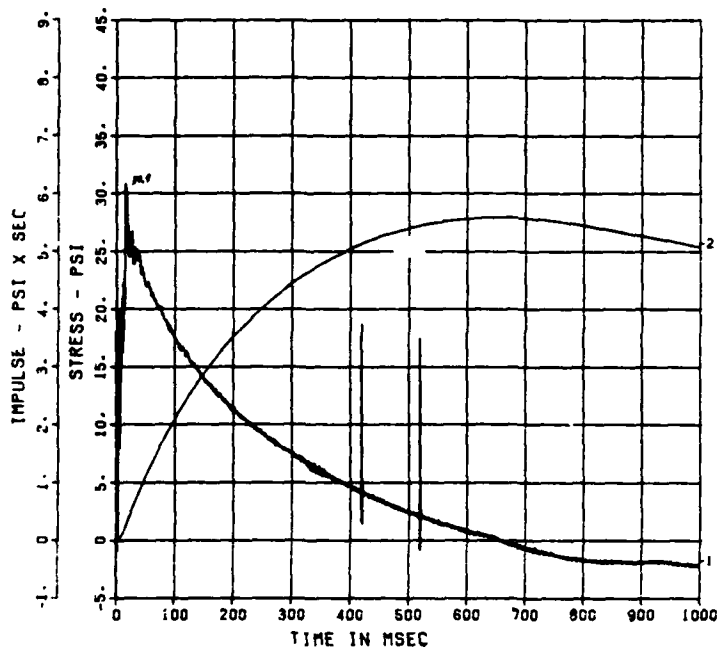


** PEAK VALUE IS -94 % UNDER CALIBRATION **

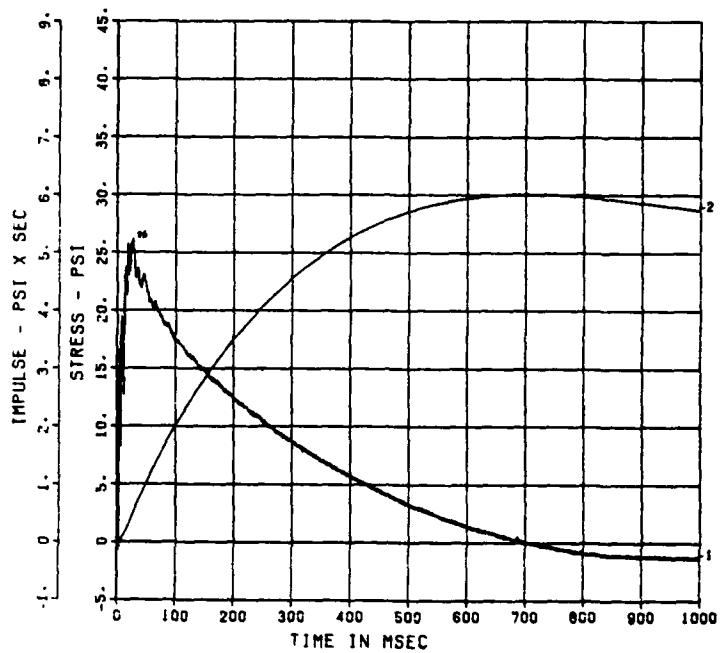
SLAB 2B POST UPGRADE
B3 BOTTOM
25000. HZ CAL 2699.
F4
06/27/79 4467 47 R0513



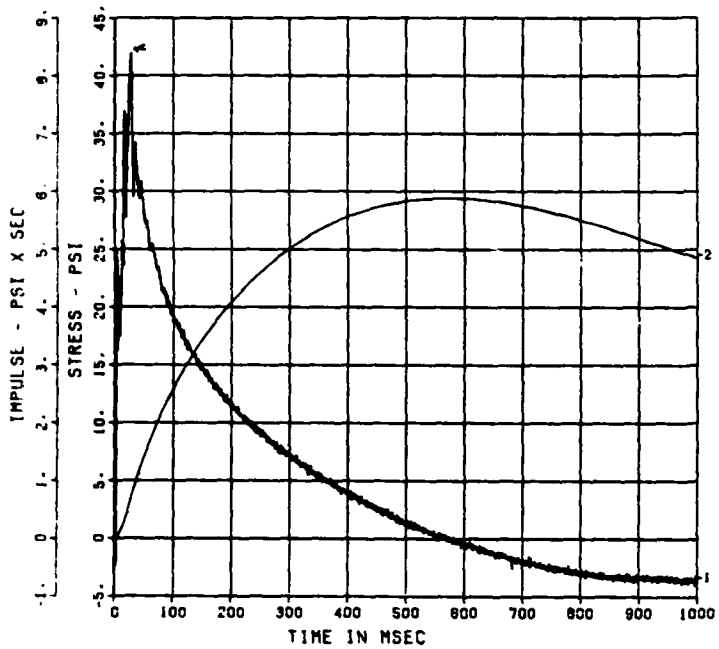
SLAB 2B POST UPGRADE
EP-1
25000. HZ CAL 33.1
F4
07/03/79 12521 57 R0727



SLAB 2B POST UPGRADE
EP-2
25000. HZ CAL 39.6
F4
07/03/79 12521 60 R0727

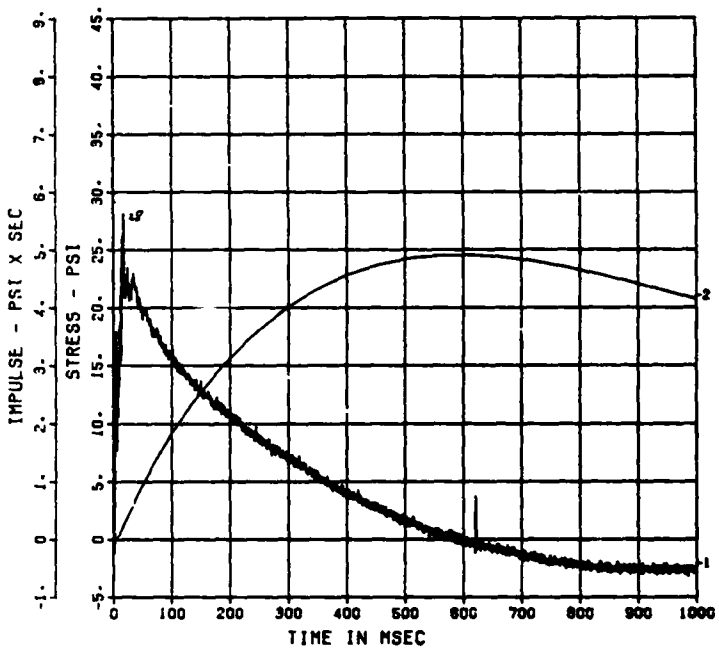


SLAB 2B POST UPGRADE
CP
25000. HZ CAL 40.0
F4
06/27/78 4467 59 R0513

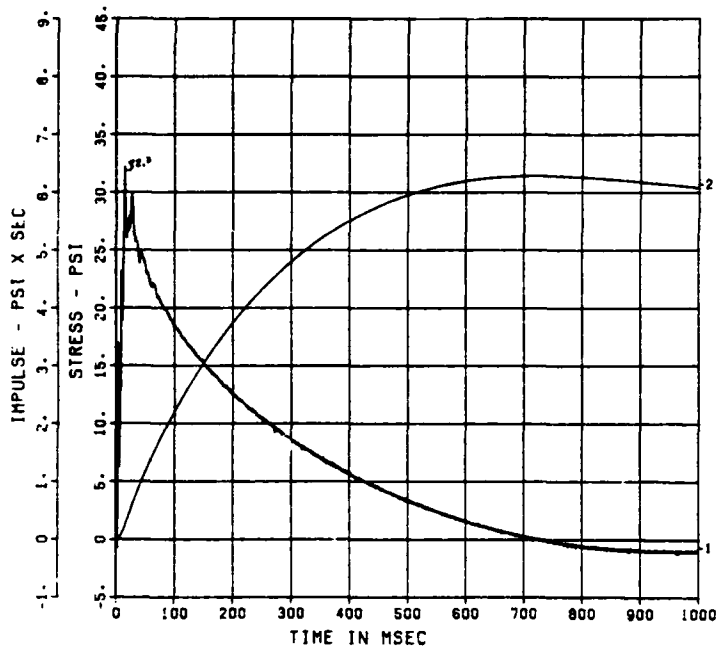


** PEAK VALUE IS 3 % OVER CALIBRATION **

SLAB 2B POST UPGRADE
WP-1
25000. HZ CAL 38.0
F4
06/27/78 4467 60 R0513

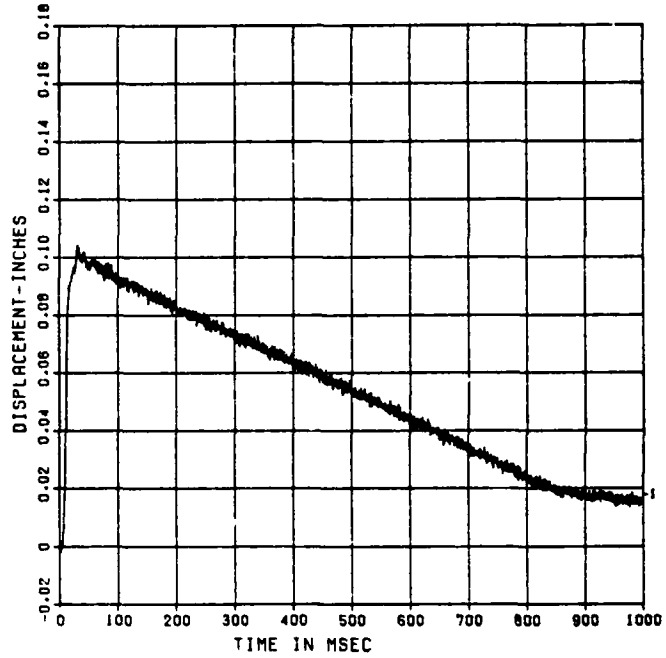


SLAB 2B POST UPGRADE
WP-2
25000. HZ CAL 29.3
F4
07/03/79 12521 55 R0727



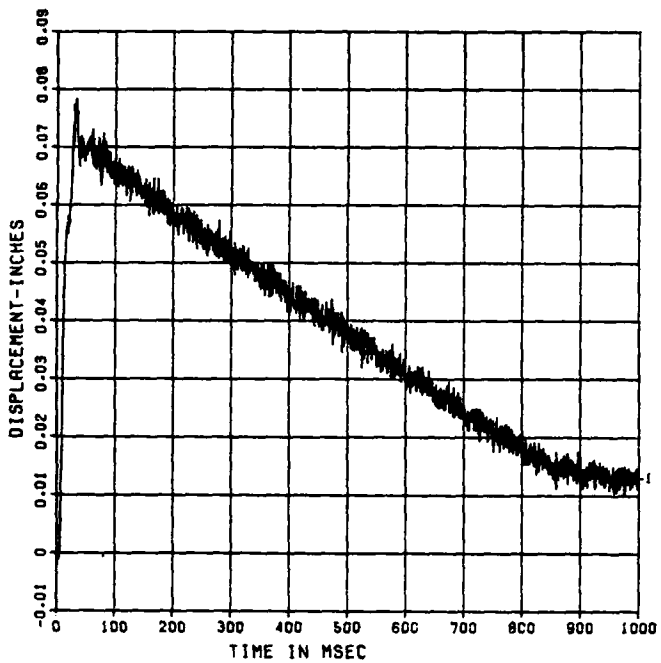
** PEAK VALUE IS 10 % OVER CALIBRATION **

SLAB 2B POST UPGRADE
SBD
25000. HZ CAL 1.05
F4
07/03/79 12521 61 R0727



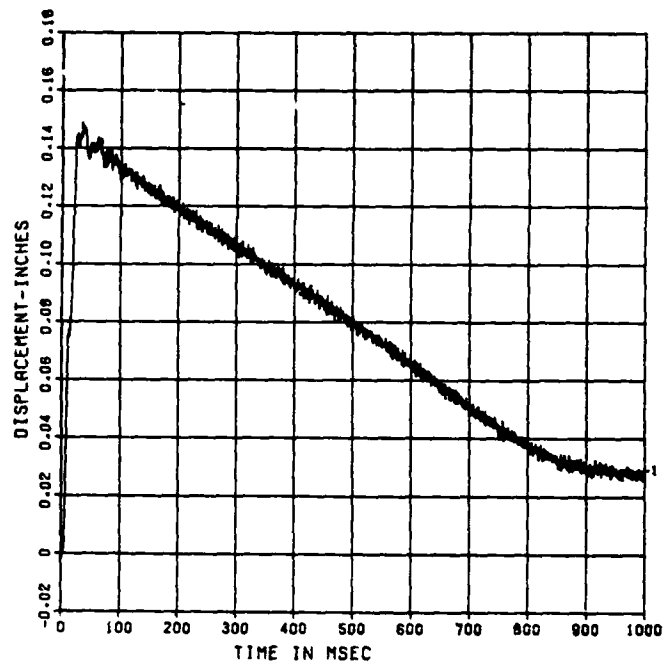
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
NBD
25000. HZ CAL 1.07
F4
07/03/79 12521 62 R0727



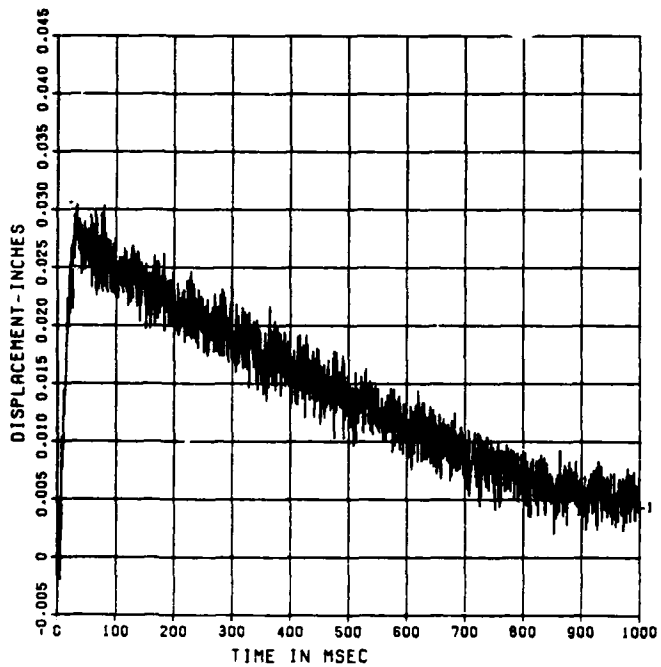
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
NBD
25000. HZ CAL 1.15
F4
07/03/79 12521 63 R0727



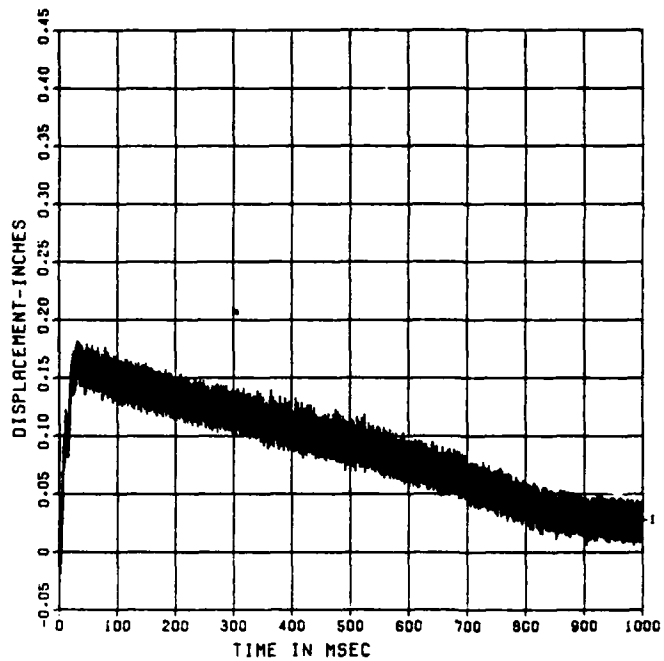
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
EBD
25000. HZ CAL 1.16
F4
07/03/79 12521 64 R0727



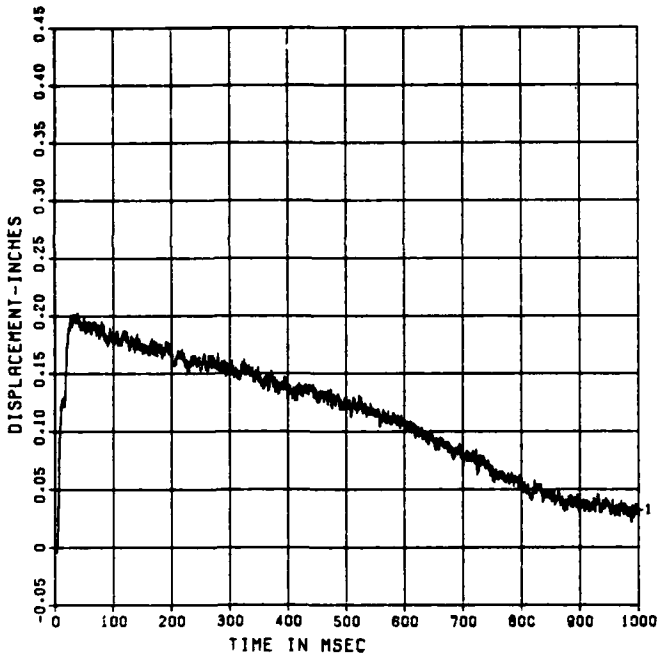
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
ED-1
25000. HZ CAL 2.06
F4
07/03/79 12521 65 R0727



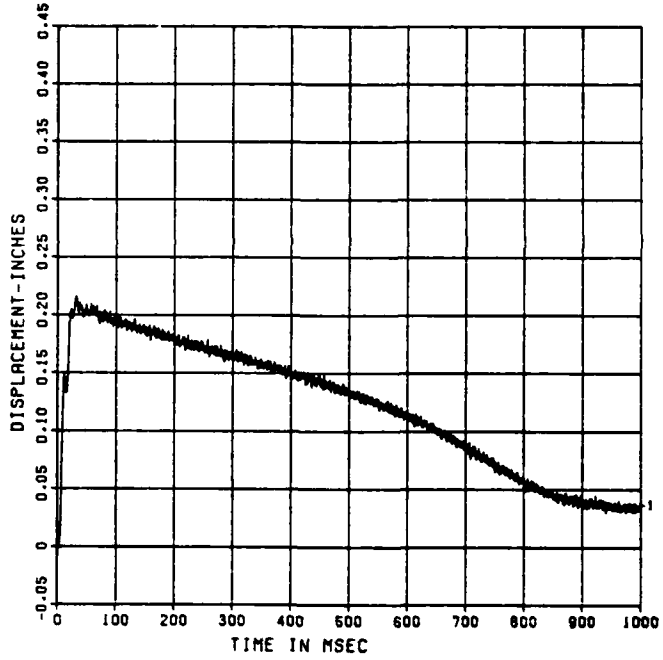
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
WD-1
25000. HZ CAL 1.86
F4
07/03/79 12521 66 R0727



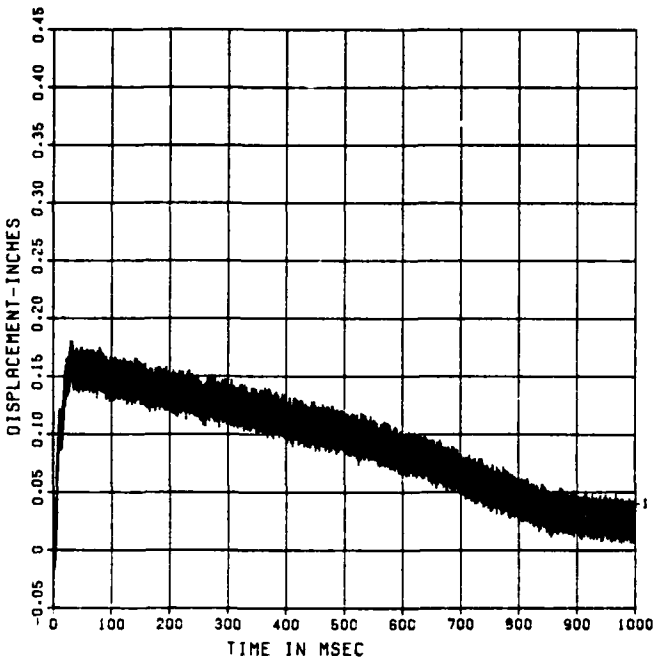
** PEAK VALUE IS -89 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
WD-2
25000. HZ CAL 1.81
F4
07/03/79 12521 67 R0727



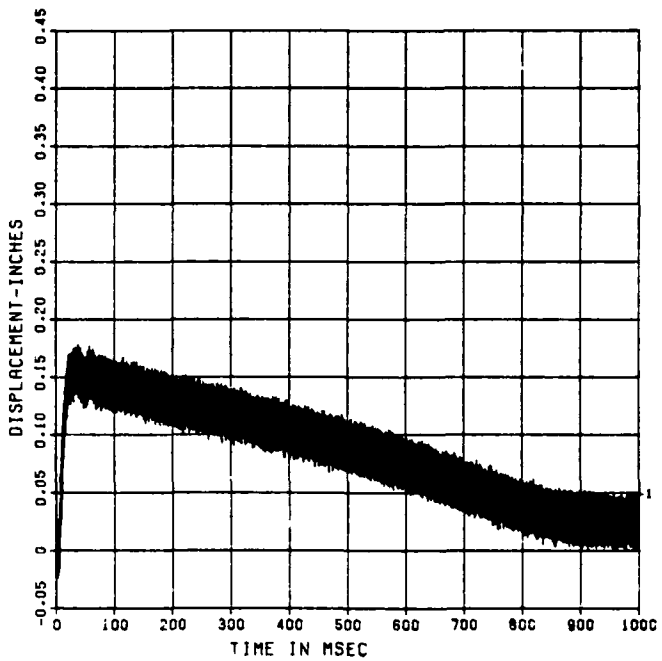
** PEAK VALUE IS -89 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
WD-3
25000. HZ CAL 2.03
F4
07/03/79 12521 68 R0727



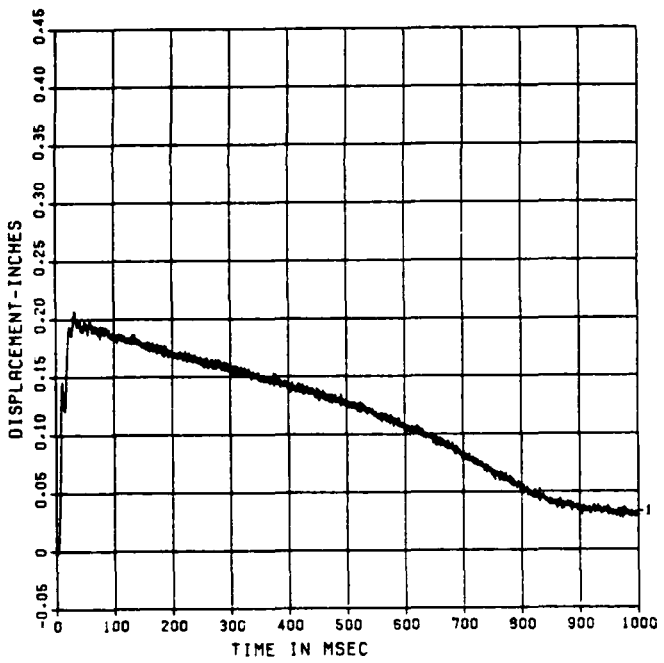
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
WD-4
25000. HZ CAL 2.07
F4
07/03/79 12521 69 R0727



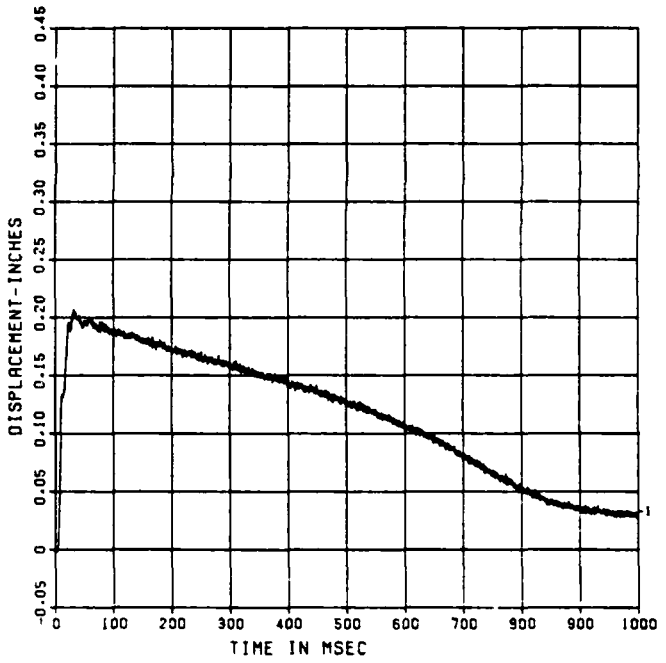
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
BD-1
25000. HZ CAL 1.51
F4
07/11/79 12521 70 R0198



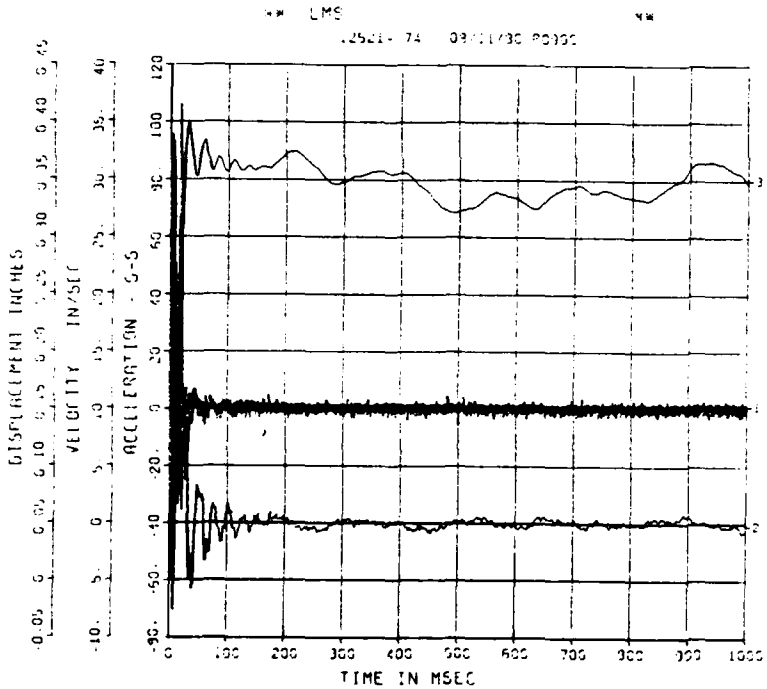
** PEAK VALUE IS -86 % UNDER CALIBRATION **

SLAB 2B POST UPGRADE
BD-2
25000. HZ CAL 1.46
F4
07/11/79 12521 71 R0198

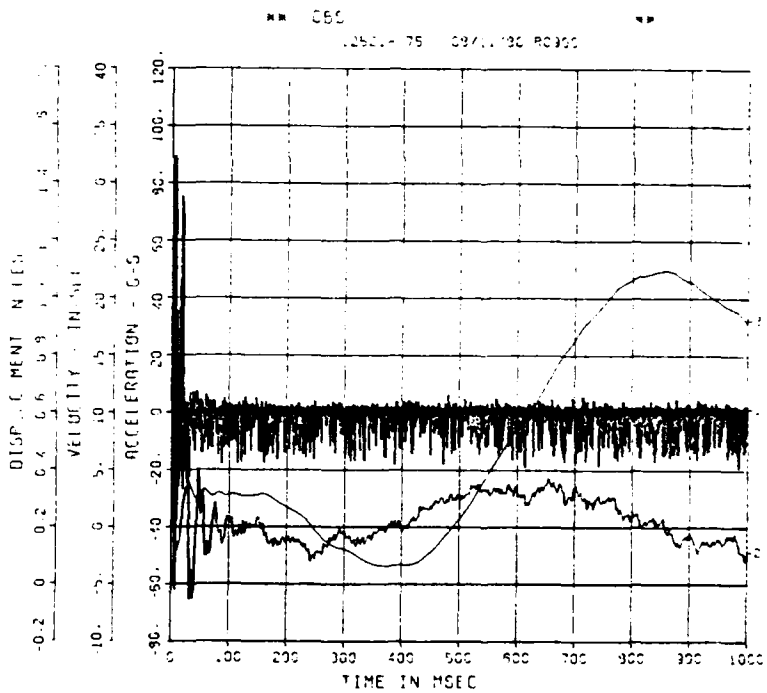


** PEAK VALUE IS -86 % UNDER CALIBRATION **

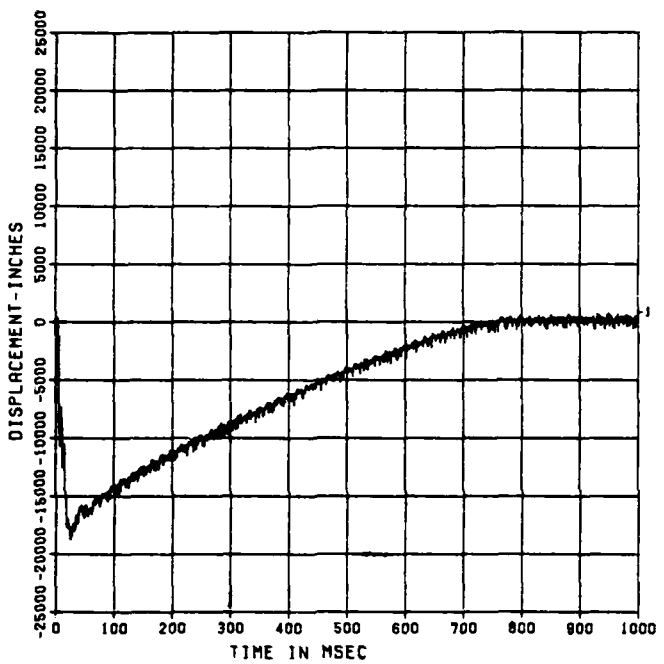
SLAB 2B POST UPGRADE
A1
25000. HZ CAL= 95.21



SLAB 2B POST UPGRADE
A2
25000. HZ CAL= 91.86

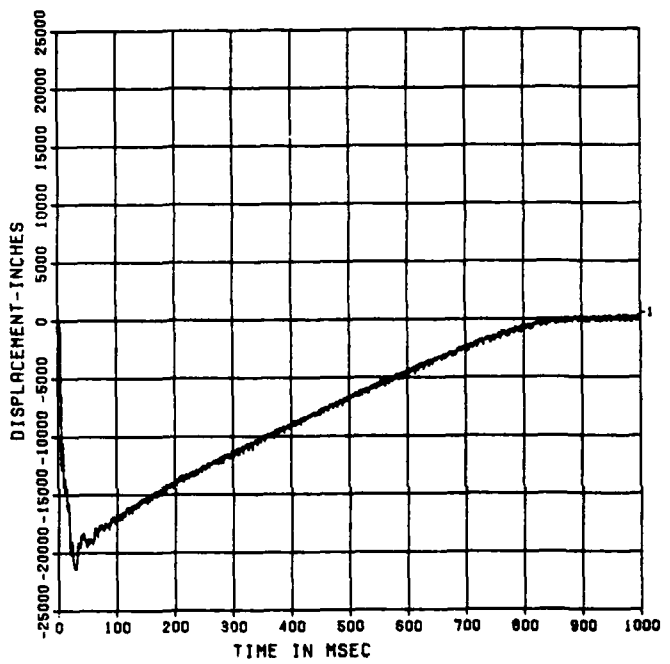


SLAB 2B POST UPGRADE
LOAD 1
25000. HZ CAL 13700.
F4
07/11/79 12521 72 R0198



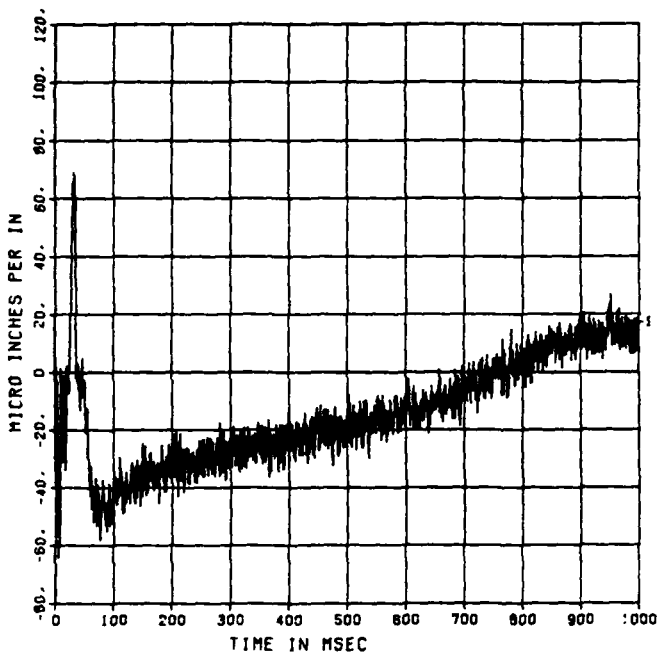
** PEAK VALUE IS 37 % OVER CALIBRATION **

SLAB 2B POST UPGRADE
LOAD 2
25000. HZ CAL 13712.
F4
07/11/79 12521 73 R0198



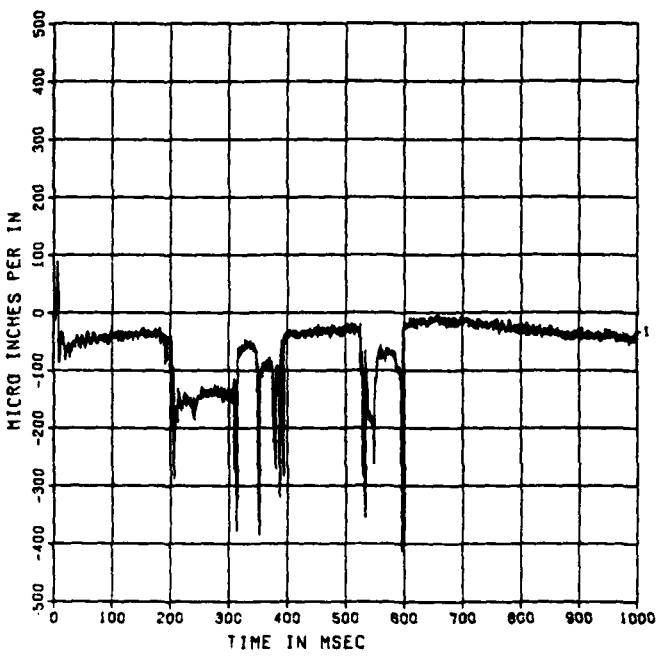
** PEAK VALUE IS 58 % OVER CALIBRATION **

SLAB 2C POST UPGRADE
SSI TOP
25000. HZ CAL 2899.
F4
06/27/79 4467 62 R0513



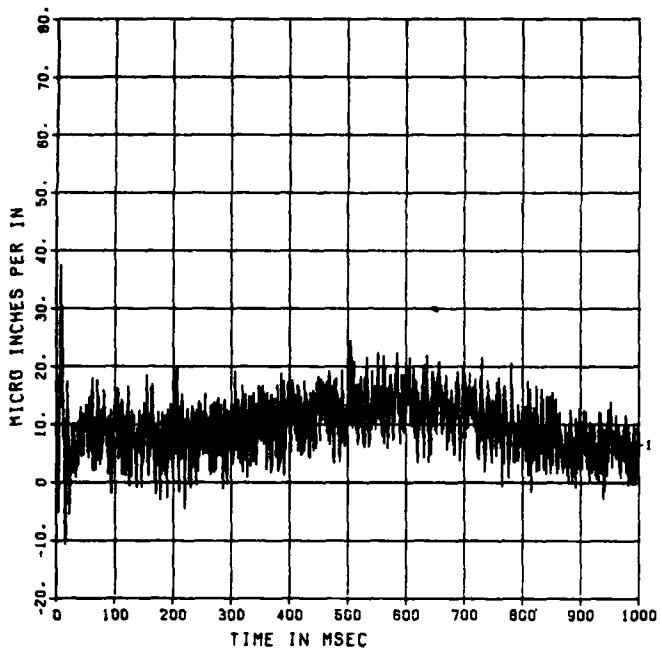
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SSI BOTTOM
25000. HZ CAL 2899.
F4
06/27/79 4467 61 R0513



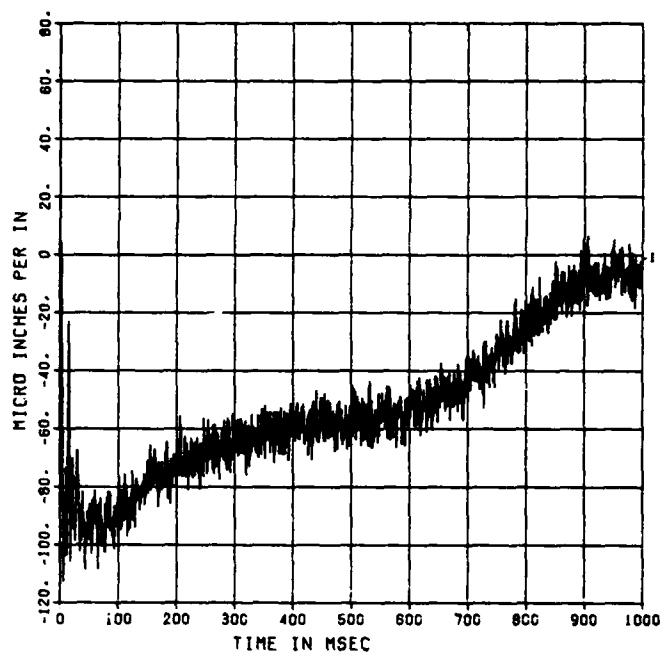
** PEAK VALUE IS -86 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS2 TOP
25000. HZ CAL 2899.
F4
06/29/79 4467 68 R0613



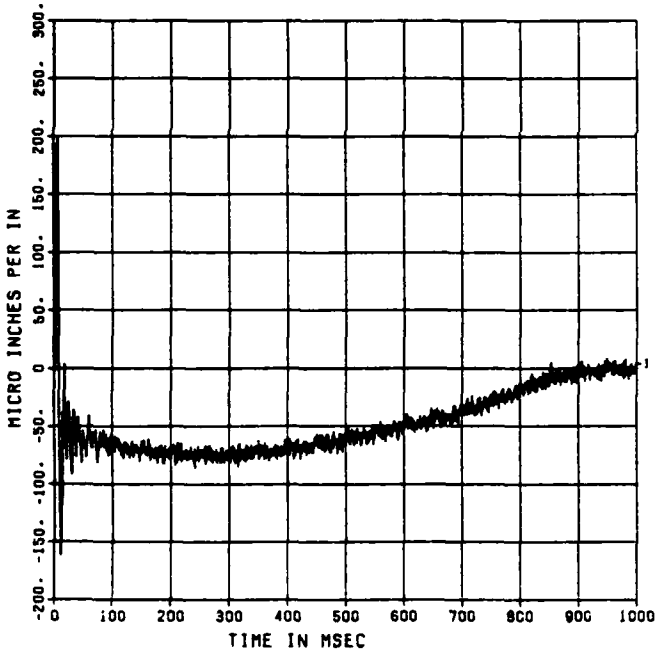
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS2 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 4467 67 R0613



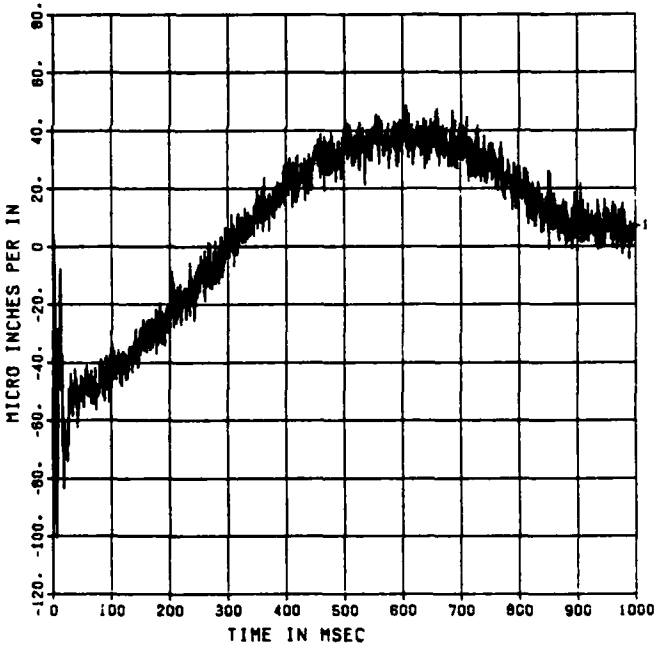
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS3 TOP
25000. HZ CAL 2899.
F4
06/29/79 4467 73 R0613



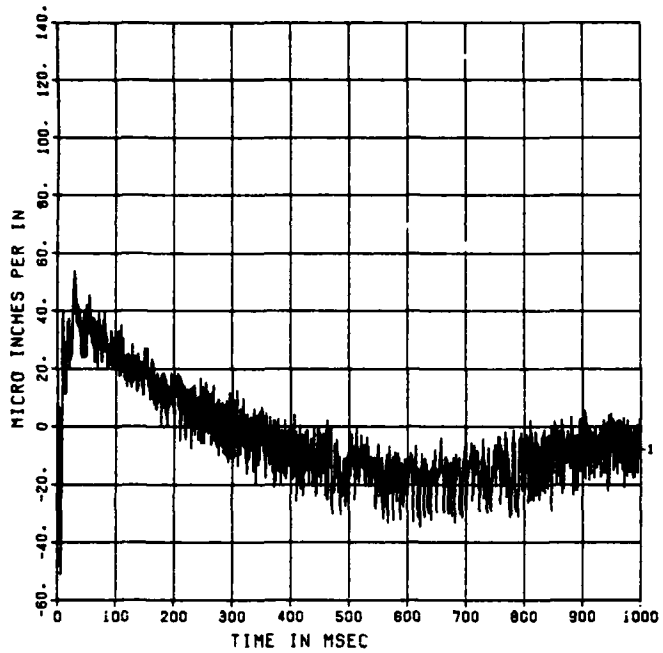
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS3 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 4467 72 R0613



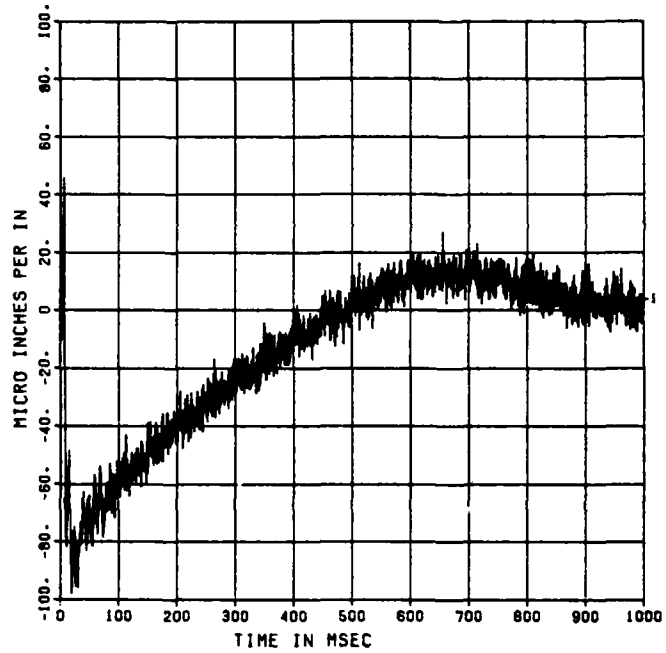
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS4 TOP
25000. HZ CAL 2099.
F4
06/29/79 4467 79 R0613



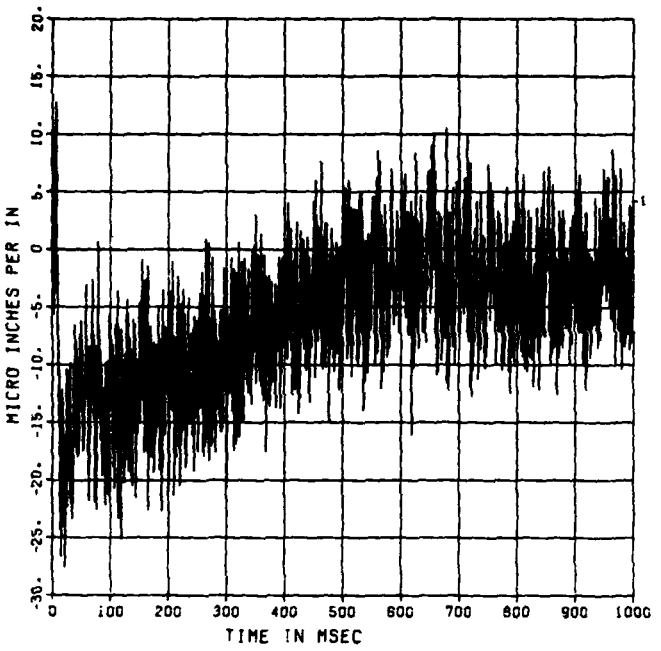
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS4 BOTTOM
25000. HZ CAL 2099.
F4
06/29/79 4467 78 R0613



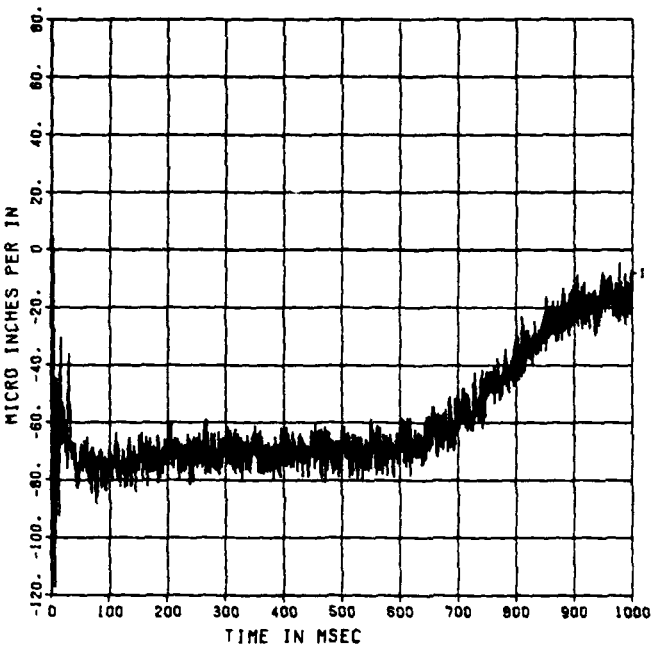
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS5 TOP
25000. HZ CAL 2899.
F4
06/29/79 4467 81 R0613



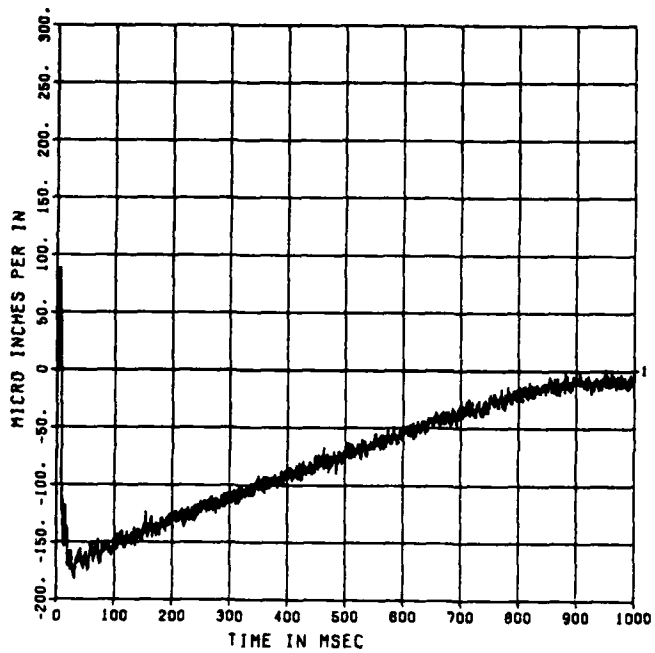
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS5 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 4467 80 R0613



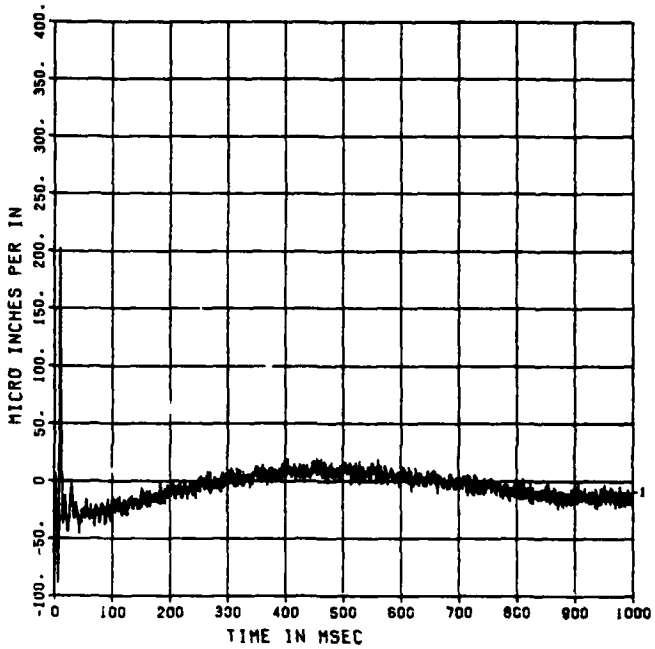
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS6 TOP
25000. HZ CAL 2000.
F4
06/29/79 4467 03 R0613



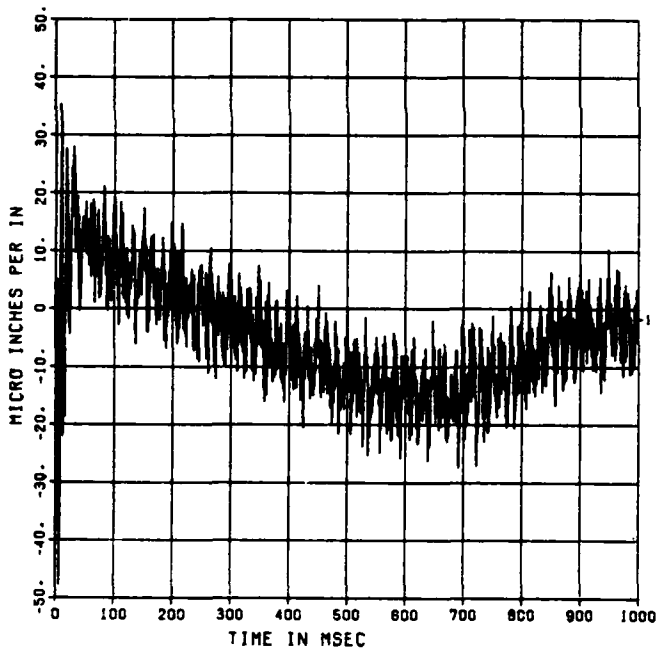
** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS6 BOTTOM
25000. HZ CAL 2000.
F4
06/29/79 4467 02 R0613



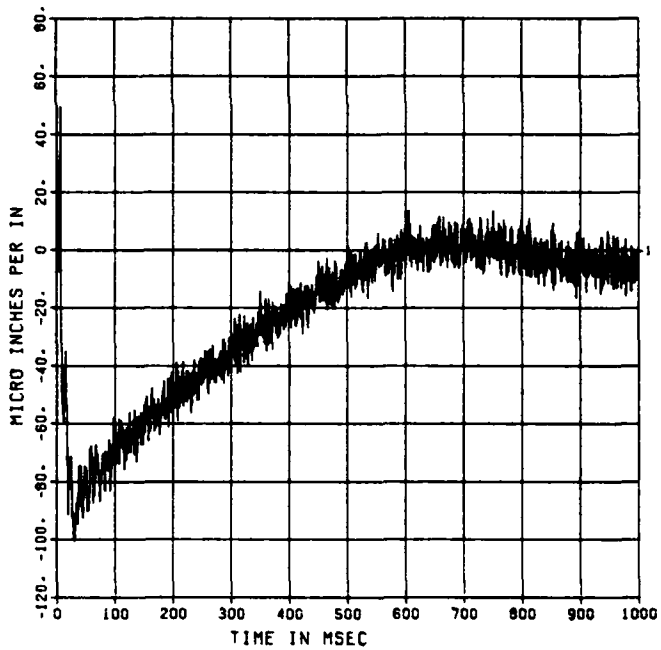
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS7 TOP
25000. HZ CAL 2000.
F4
06/29/79 4467 85 R0613



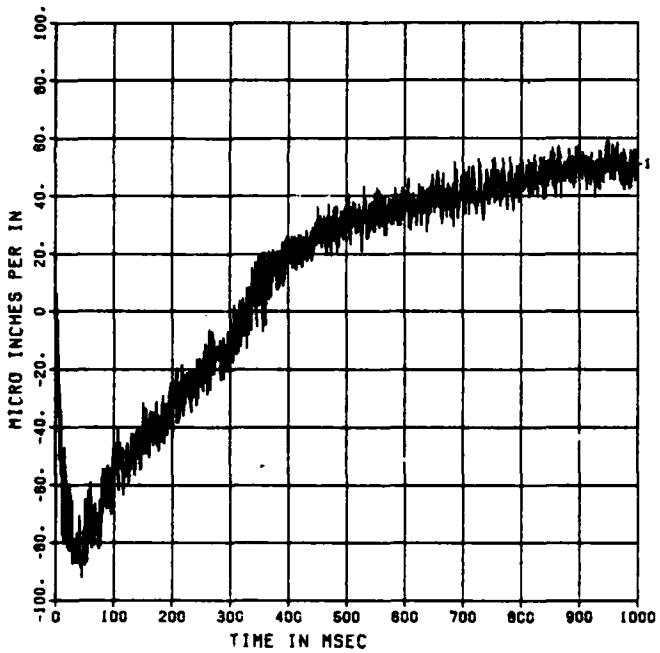
** PEAK VALUE IS -98 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS7 BOTTOM
25000. HZ CAL 2000.
F4
06/29/79 4467 84 R0613



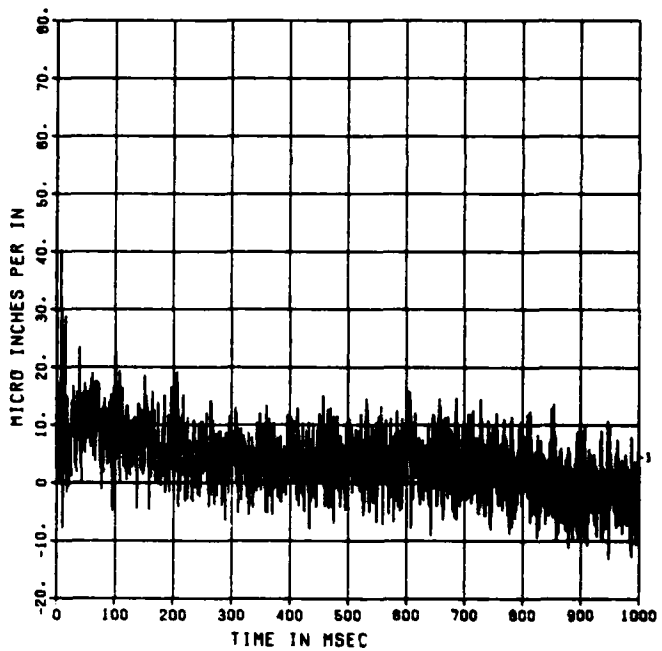
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS8 TOP
25000. HZ CAL 2000.
F4
06/29/79 4467 87 R0613



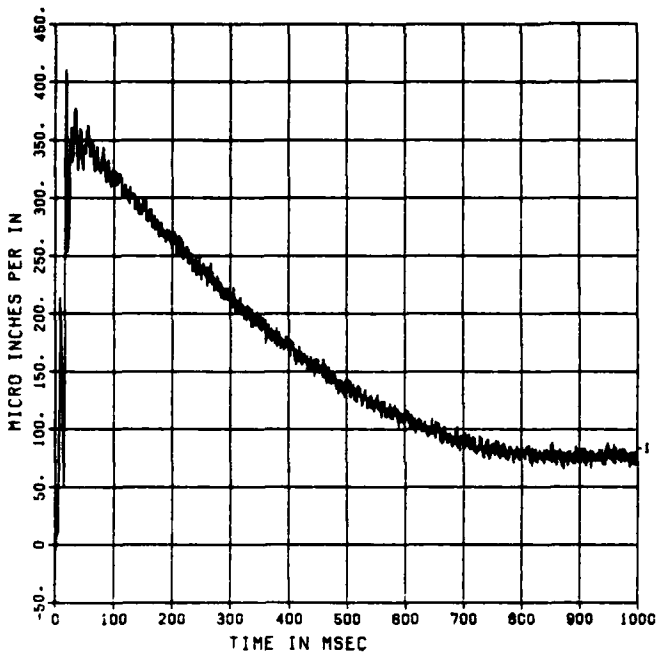
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SS8 BOTTOM
25000. HZ CAL 2000.
F4
06/29/79 4467 86 R0613



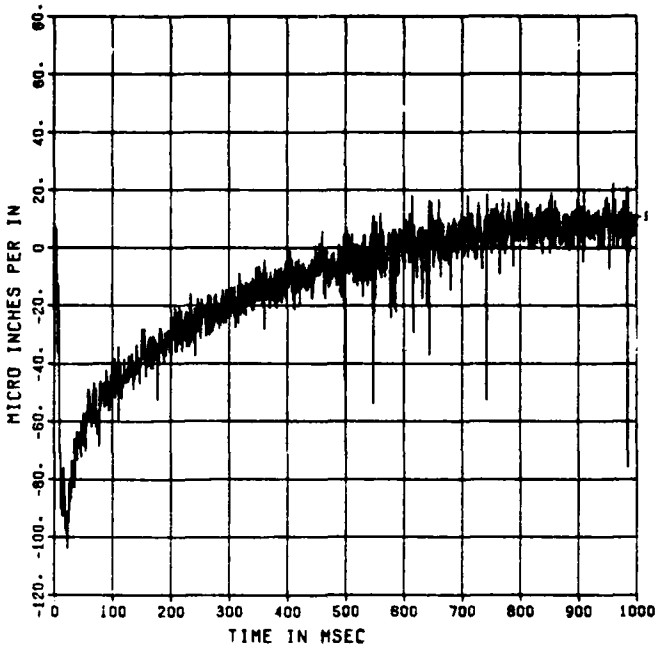
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SL2 TOP
25000. HZ CAL 2499.
F4
06/29/79 4487 89 RD813



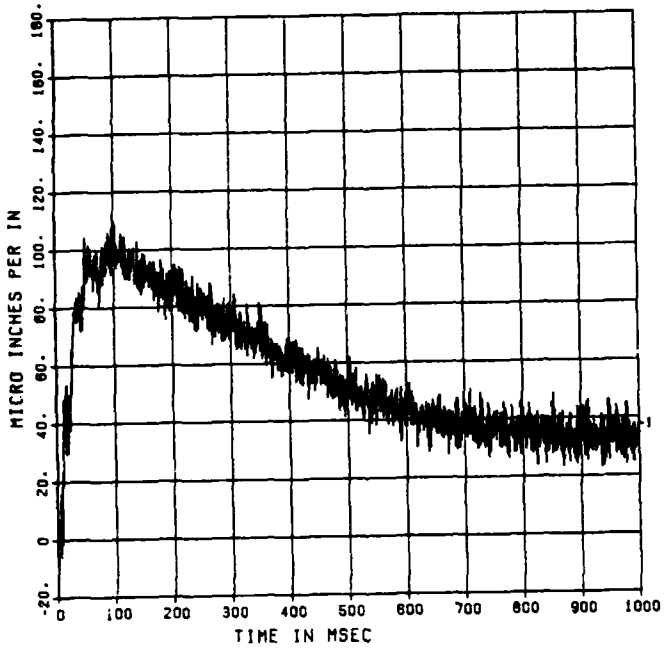
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SL3 TOP
25000. HZ CAL 2499.
F4
06/29/79 4487 76 RD813



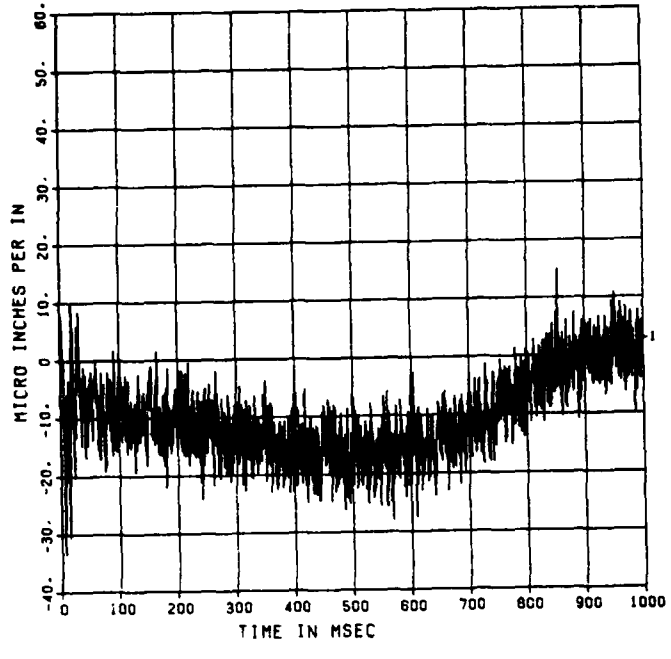
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
SL3 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 4467 74 R0613



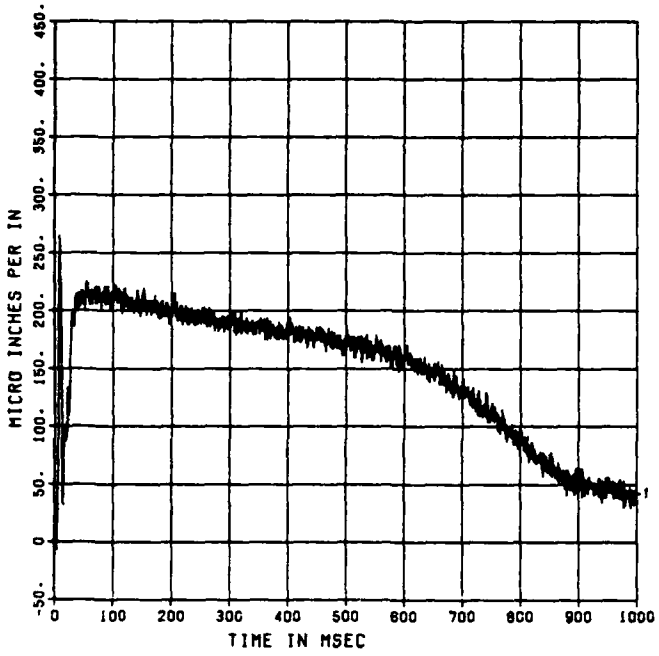
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
B2 TOP
25000. HZ CAL 2899.
F4
06/29/79 4467 71 R0613



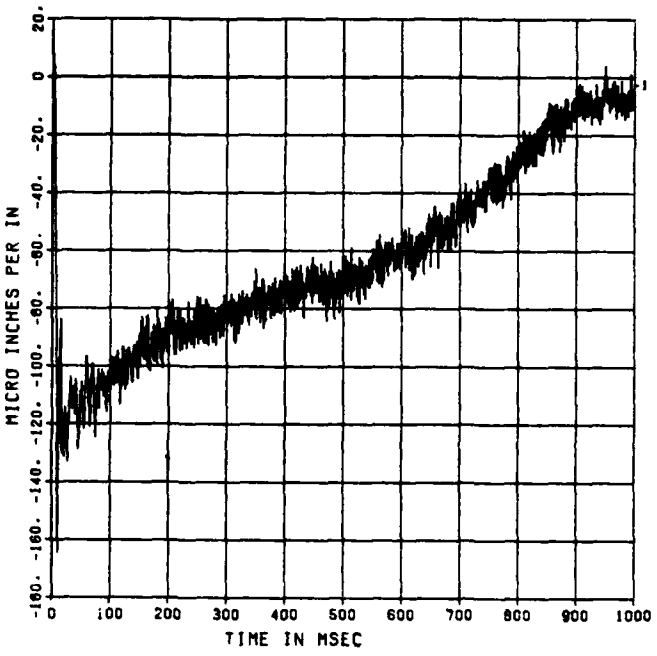
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
B2 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 4467 70 R0613



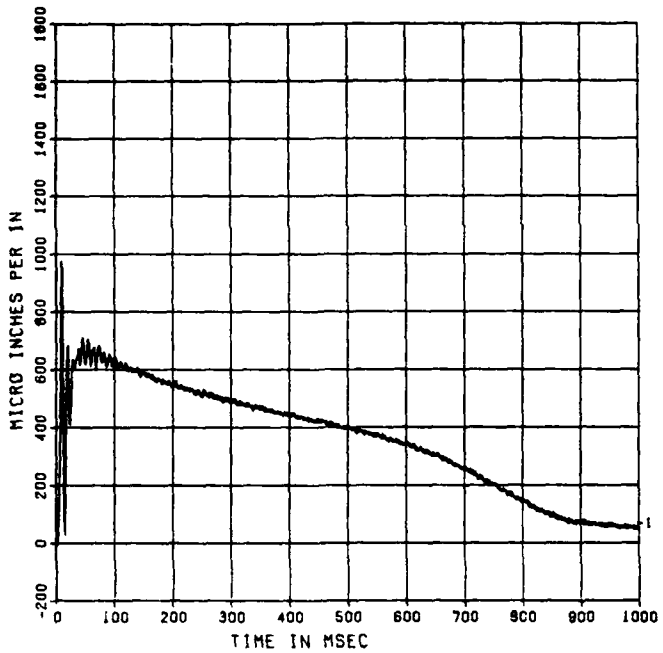
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
B3 TOP
25000. HZ CAL 2899.
F4
06/29/79 4467 77 R0613

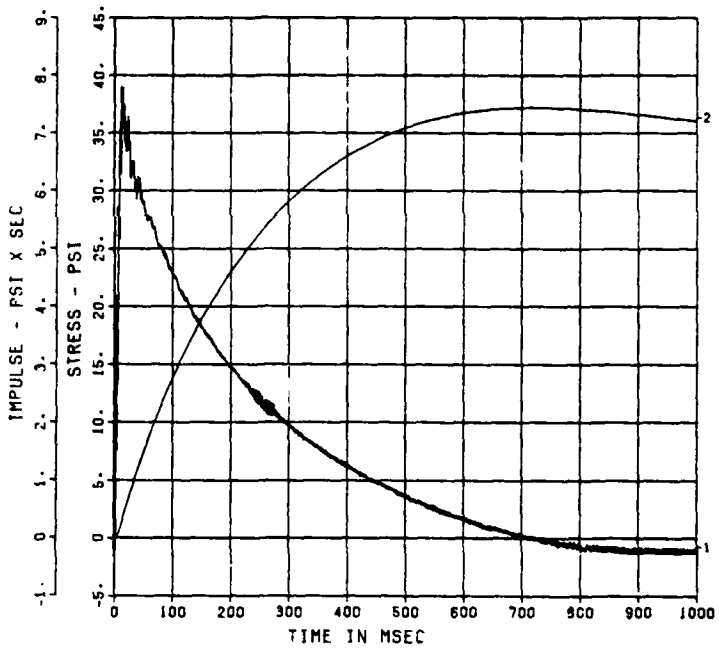


** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 2C POST UPGRADE
B3 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 4467 76 R0613

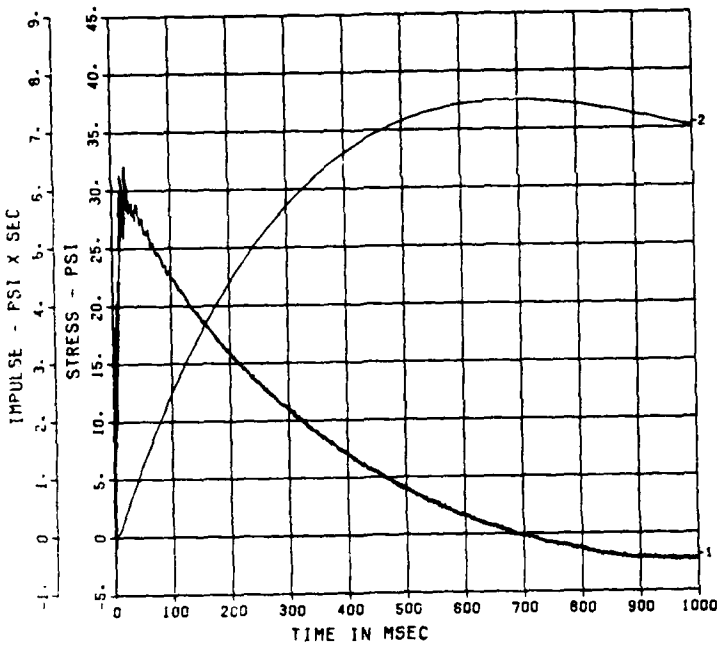


SLAB 2C POST UPGRADE
EP-1
25000. HZ CAL 33.1
F4
07/11/78 12521 01 R0198

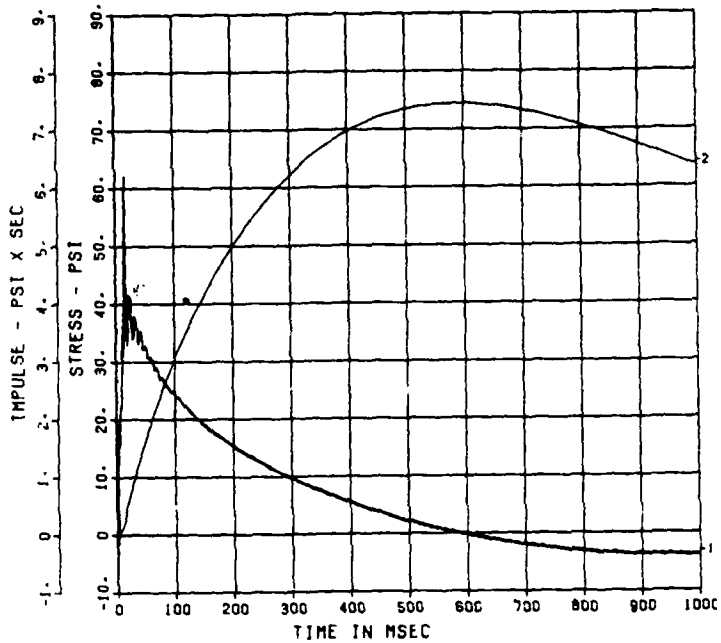


** PEAK VALUE IS 19 % OVER CALIBRATION **

SLAB 2C POST UPGRADE
EP-2
25000. HZ CAL 39.6
F4
07/11/79 12521 82 R0198

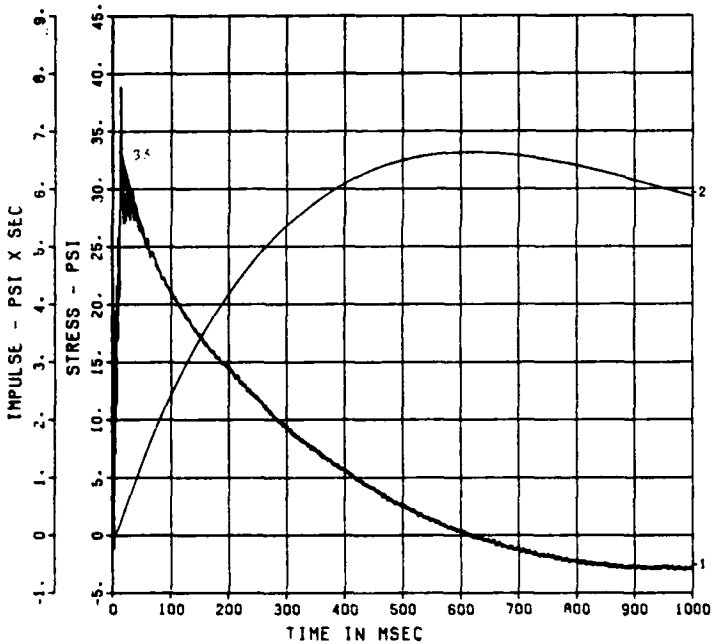


SLAB 2C POST UPGRADE
CP
25000. HZ CAL 40.6
F4
06/29/79 4467 88 R0613

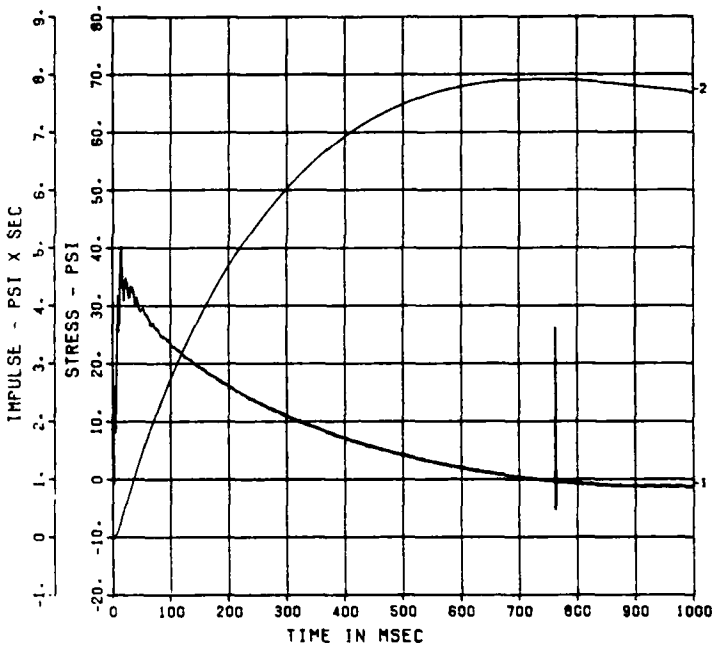


** PEAK VALUE IS 53 % OVER CALIBRATION **

SLAB 2C POST UPGRADE
WP-1
25000. HZ CAL 36.8
F4
06/29/79 4467 89 R0613

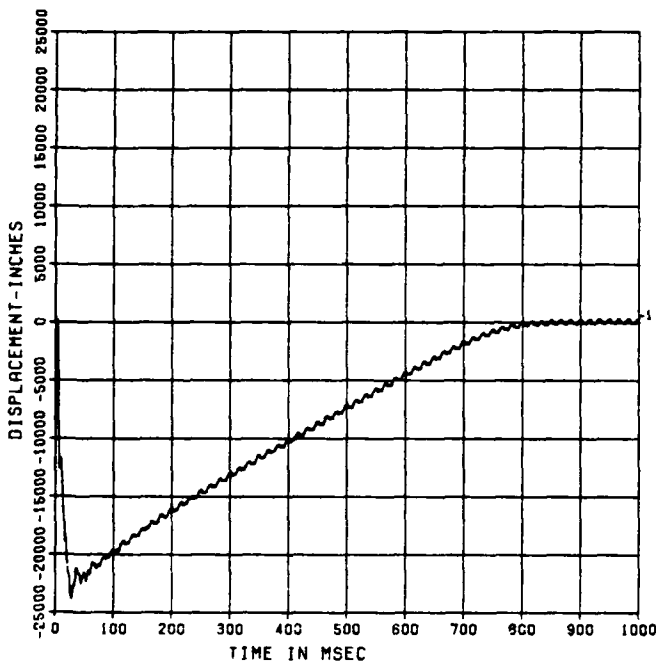


SLAB 2C POST UPGRADE
WP-2
25000. HZ CAL 29.3
F4
07/11/79 12521 79 R0198



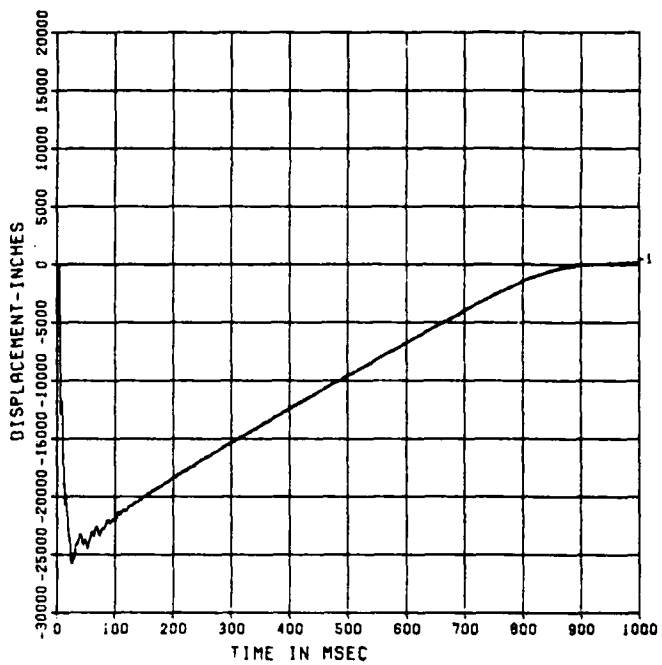
** PEAK VALUE IS 39 % OVER CALIBRATION **

SLAB 2C POST UPGRADE
LOAD 1
25000. HZ CAL 18286.
F4
07/11/79 12521 85 R0198



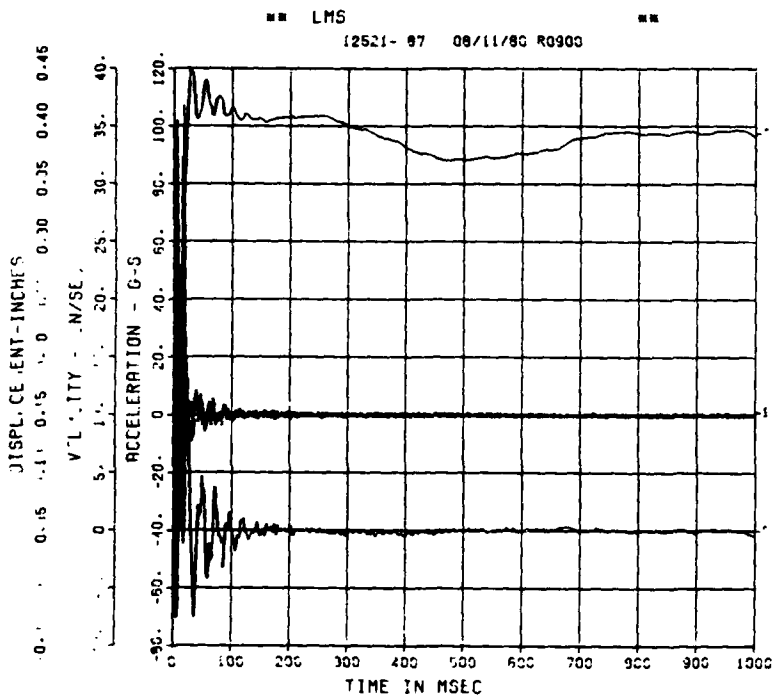
** PEAK VALUE IS 31 % OVER CALIBRATION **

SLAB 2C POST UPGRADE
LOAD 2
25000. HZ CAL 18283.
F4
07/11/79 12521 86 R0198

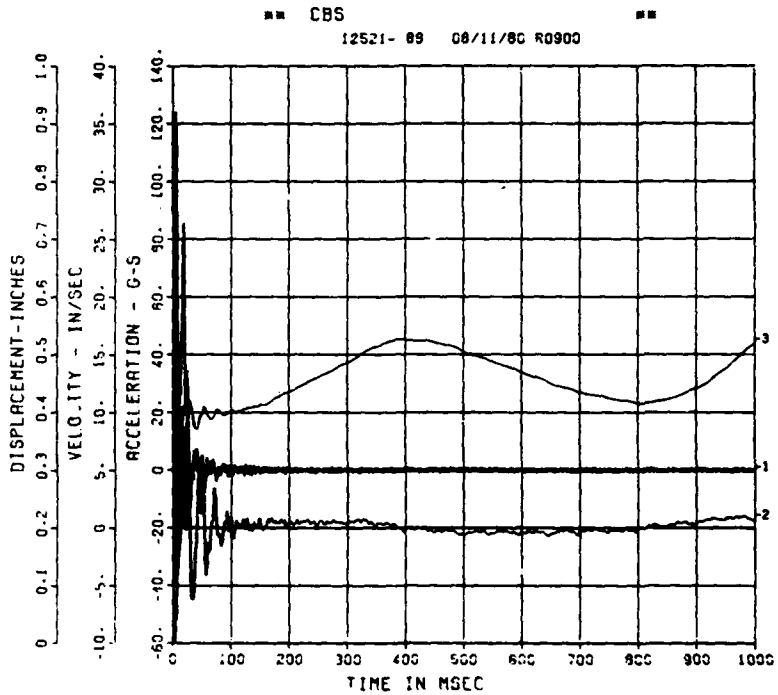


** PEAK VALUE IS 41 % OVER CALIBRATION **

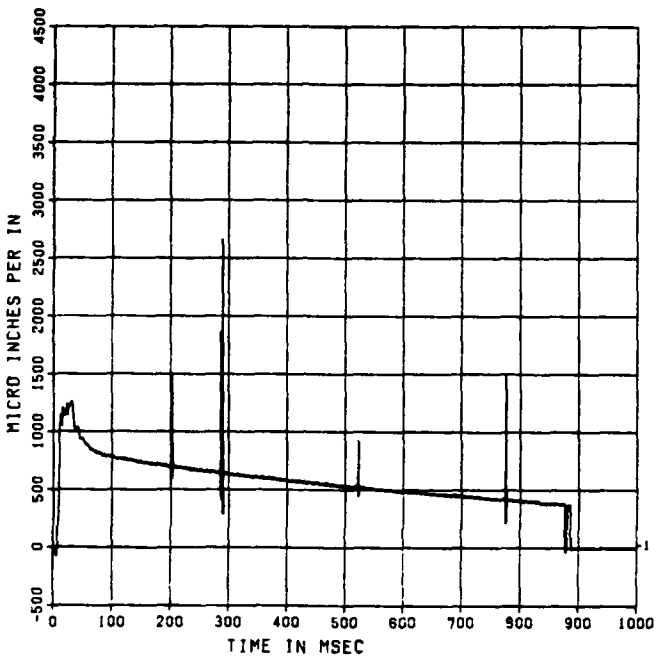
SLAB 2C POST UPGRADE
A1
25000. HZ CAL= 95.21



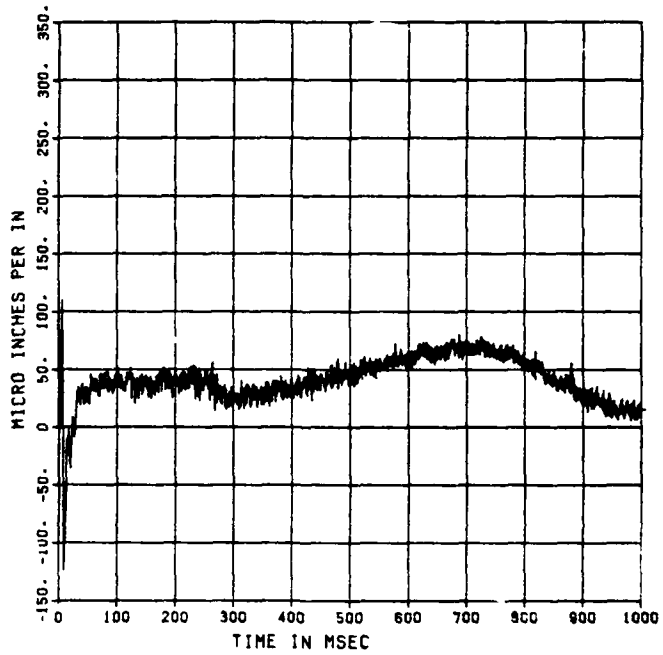
SLAB 2C POST UPGRADE
A2
25000. HZ CAL= 91.66



SLAB 2D POST UPGRADE
SS1 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 2 R0617

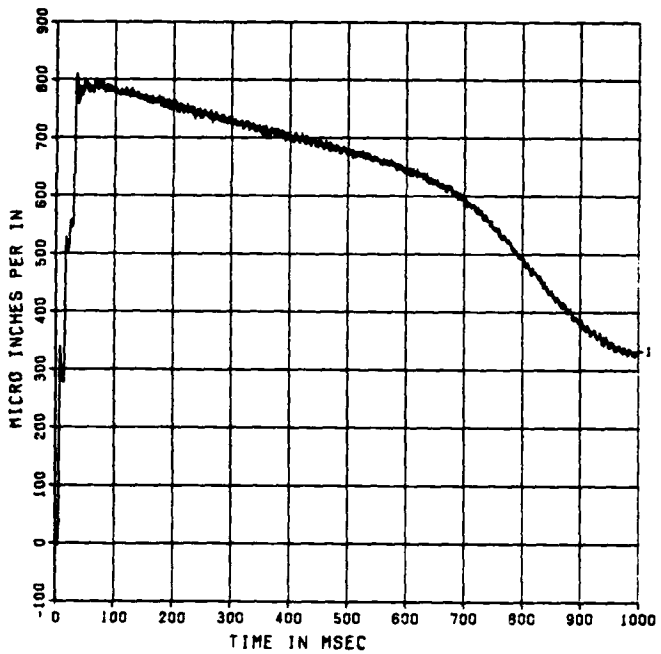


SLAB 2D POST UPGRADE
SS1 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 1 R0617

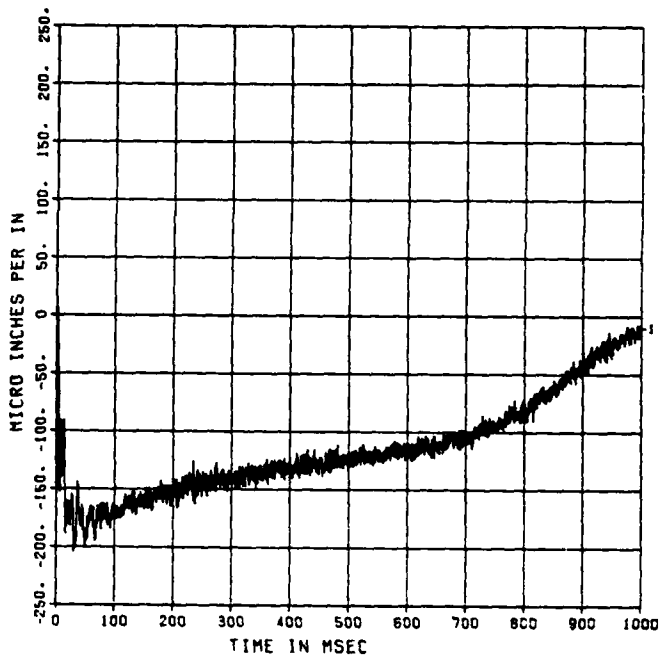


** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS2 TOP
25000. HZ CAL 2099.
F4
06/29/79 12521 8 R0617

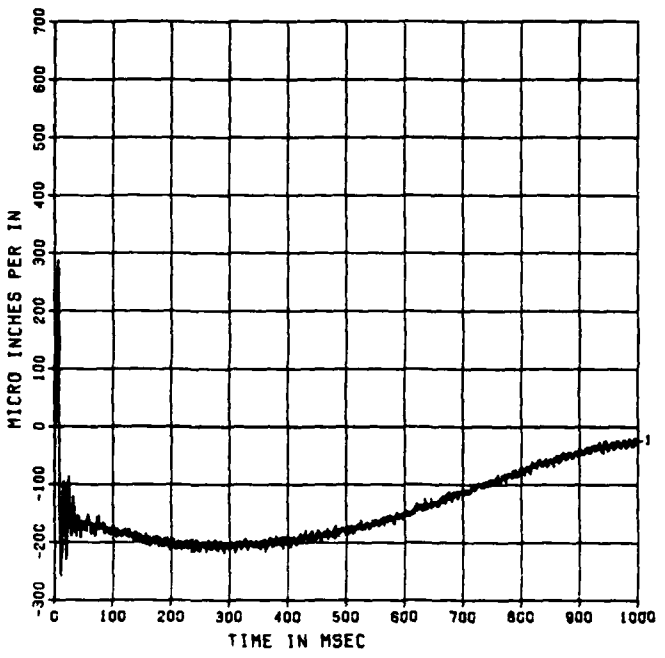


SLAB 2D POST UPGRADE
SS2 BOTTOM
25000. HZ CAL 2099.
F4
06/29/79 12521 7 R0617



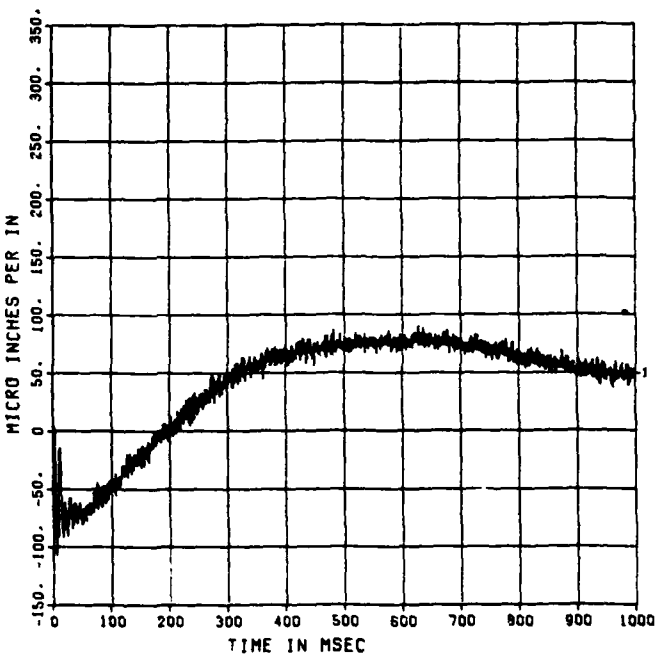
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS3 TOP
25000. HZ CAL 2000.
F4
06/29/79 12521 13 R0617



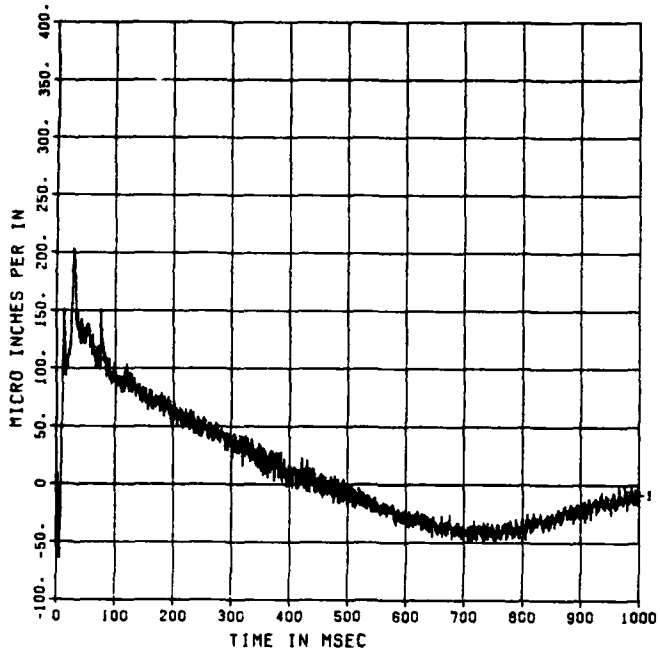
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS3 BOTTOM
25000. HZ CAL 2000.
F4
06/29/79 12521 12 R0617



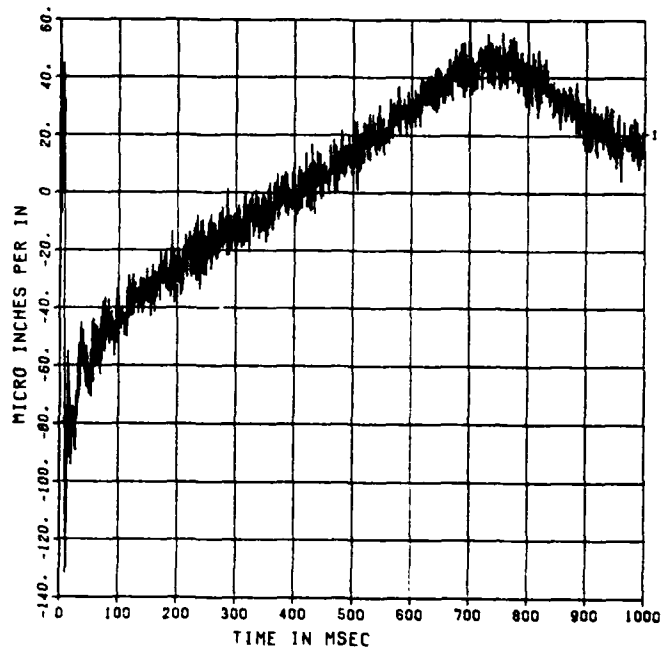
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS4 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 19 R0617



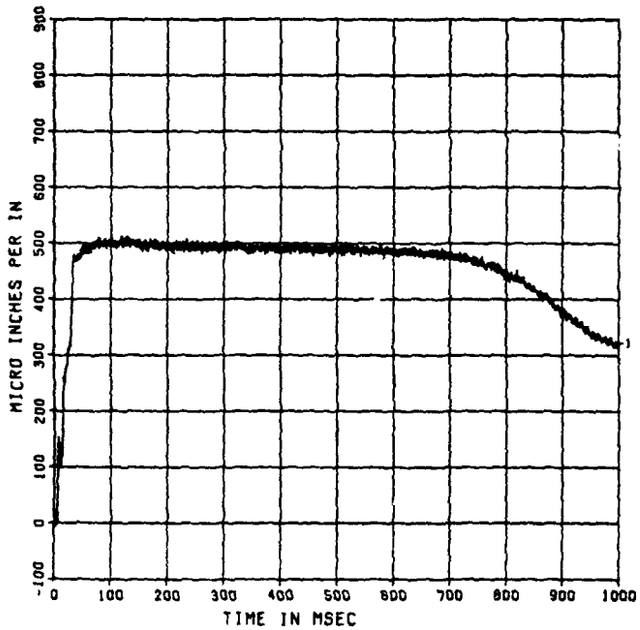
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS4 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 18 R0617



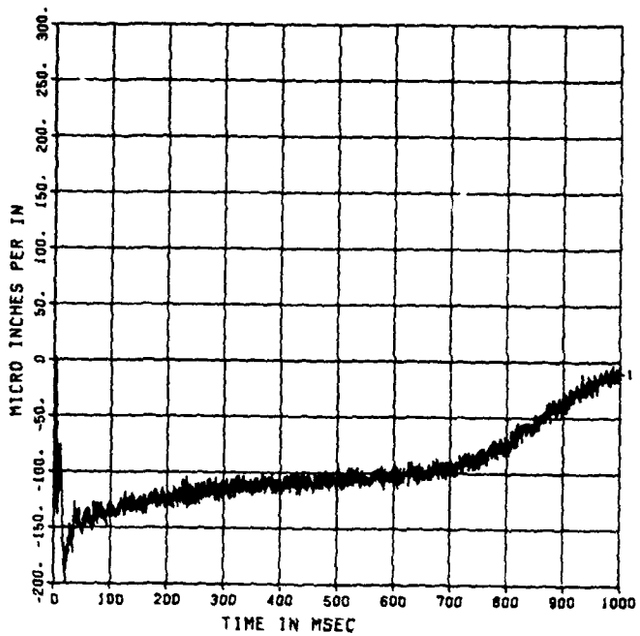
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS5 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 21 R0617



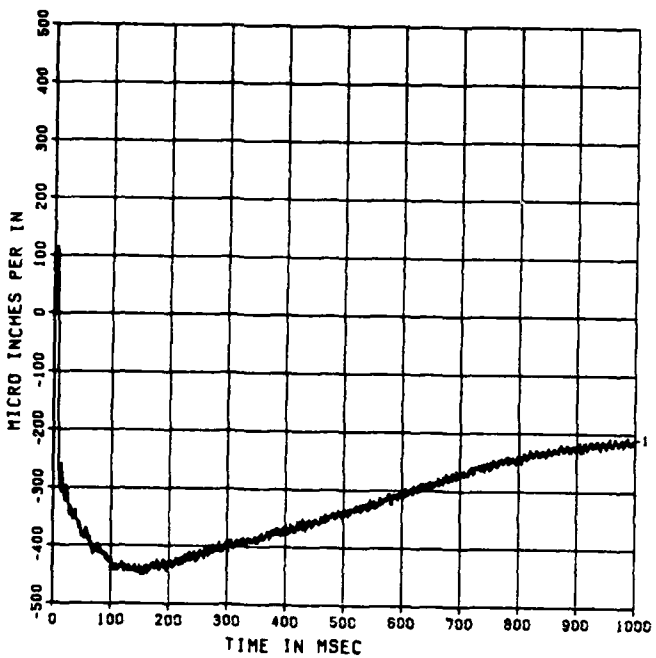
** PEAK VALUE IS -82 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS5 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 20 R0617



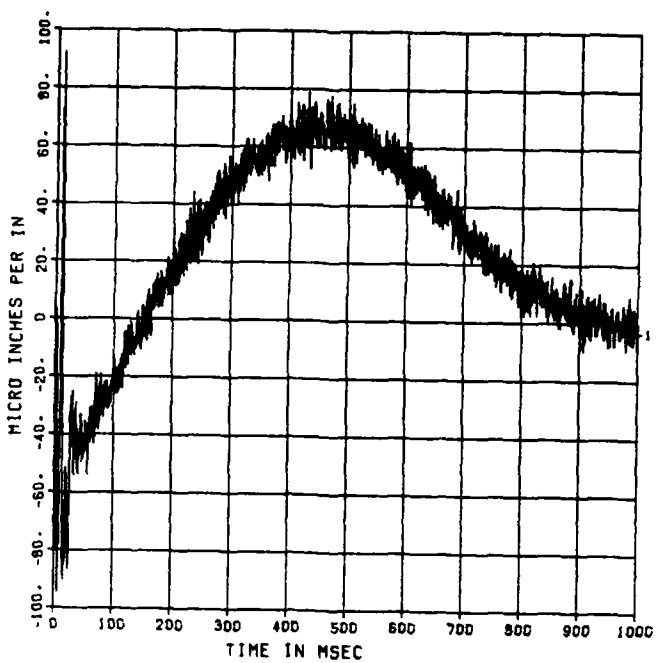
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS6 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 23 R0617



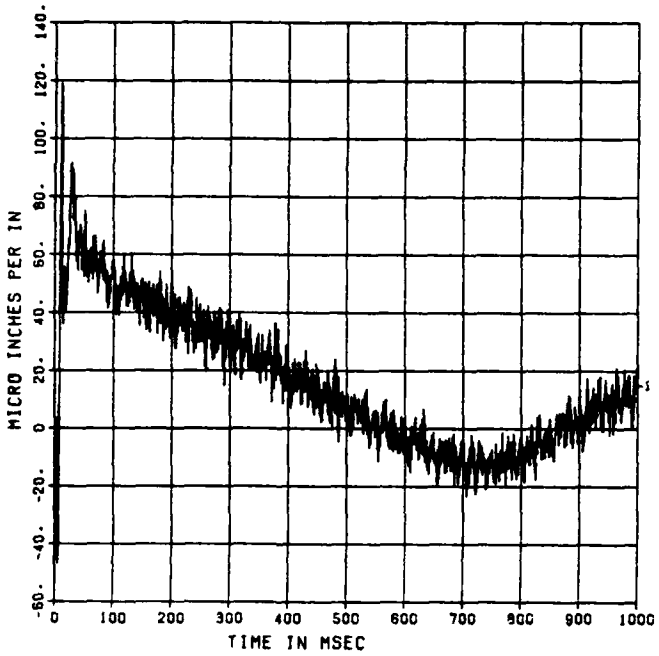
** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS6 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 22 R0617



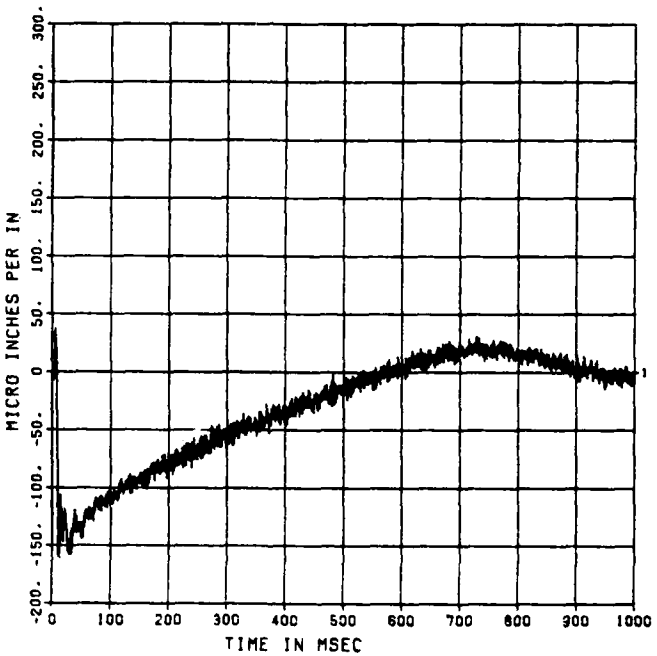
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS7 TOP
25000. HZ CAL 2999.
F4
06/29/79 12521 25 R0617



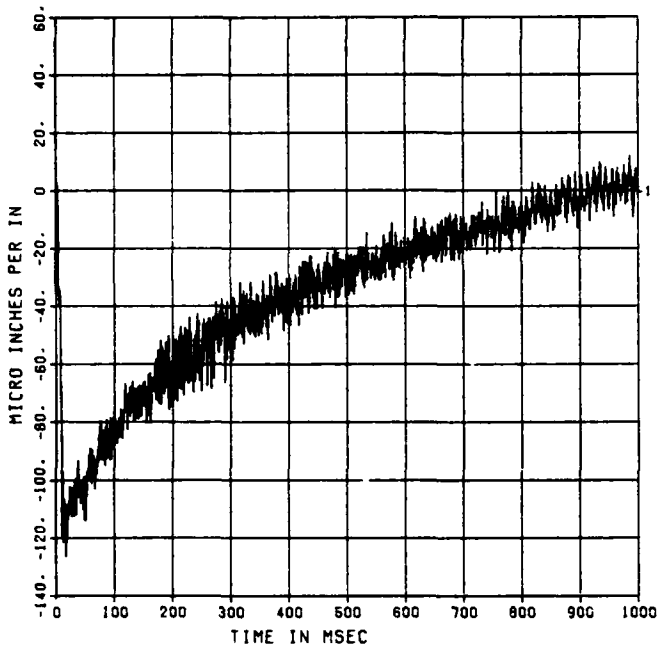
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS7 BOTTOM
25000. HZ CAL 2999.
F4
06/29/79 12521 24 R0617



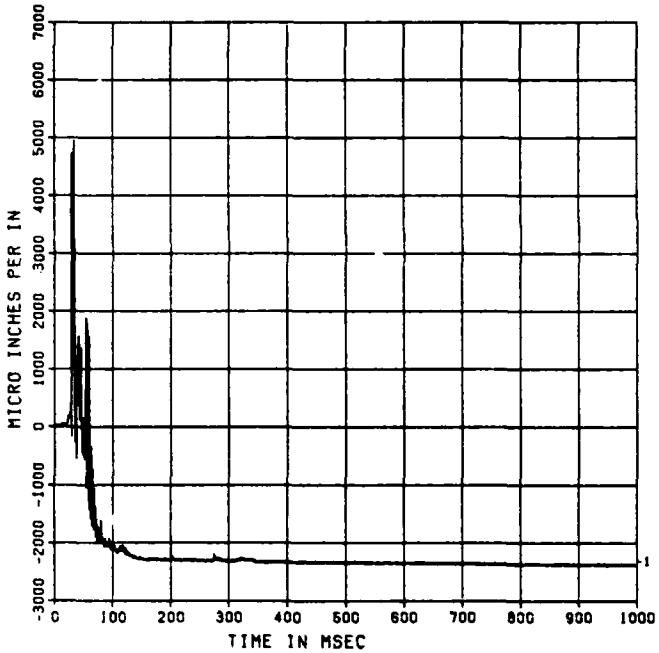
** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS8 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 27 R0617



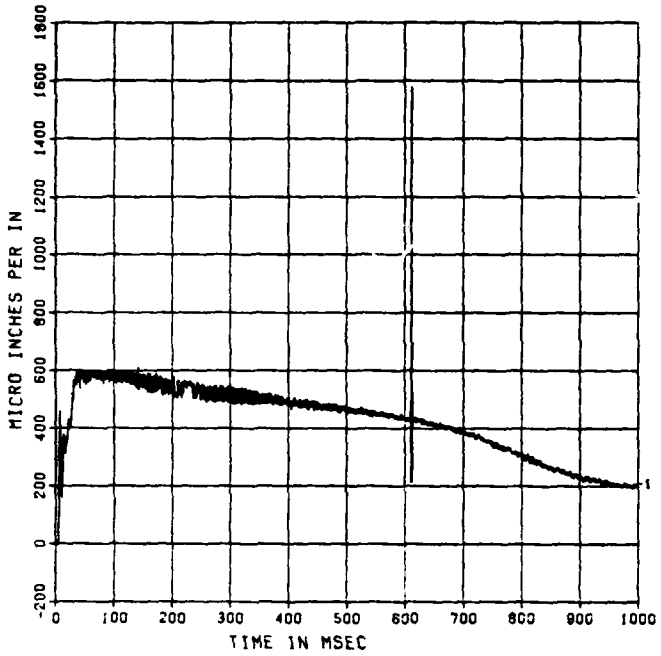
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SS8 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 26 R0617

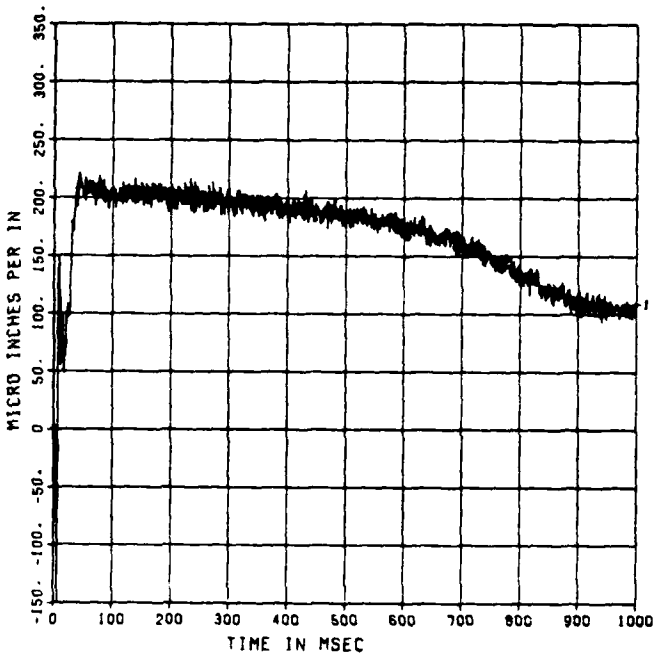


** PEAK VALUE IS 71 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
SL1 TOP
25000. HZ CAL 2000.
F4
06/29/79 12521 4 R0617

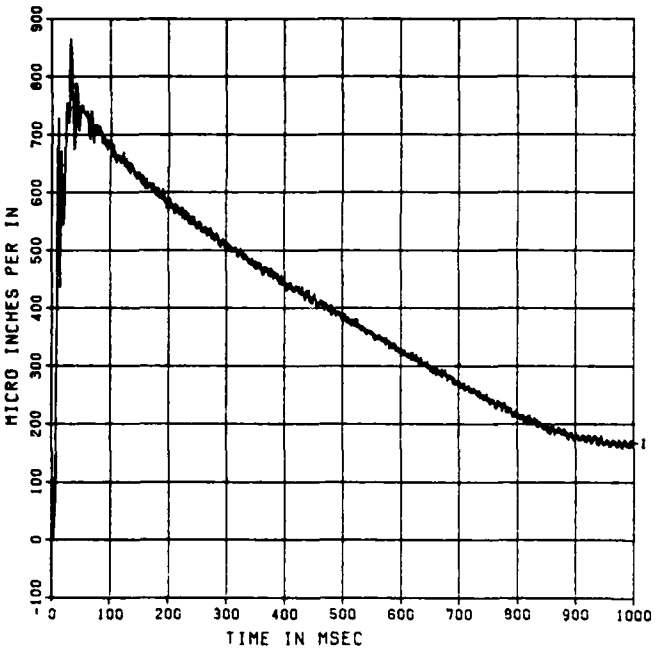


SLAB 2D POST UPGRADE
SL1 BOTTOM
25000. HZ CAL 2000.
F4
06/29/79 12521 3 R0617

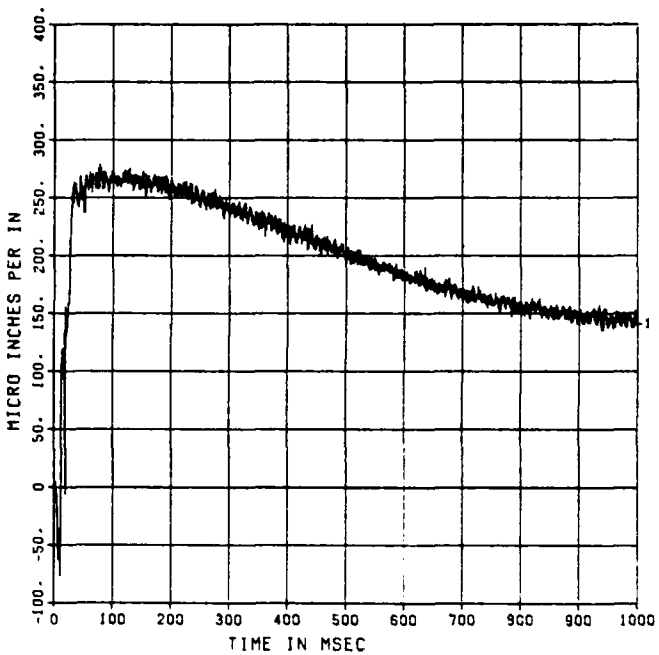


** PEAK VALUE IS -92 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SL2 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 9 R0617

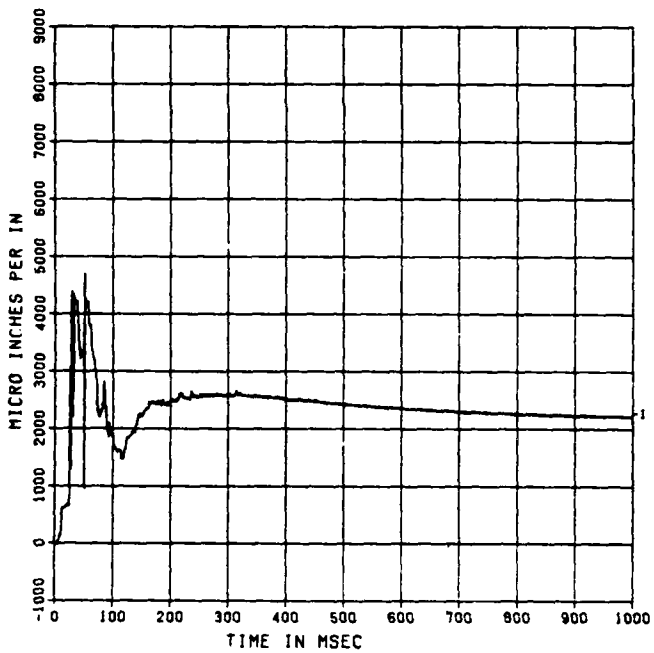


SLAB 2D POST UPGRADE
SL3 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 15 R0617



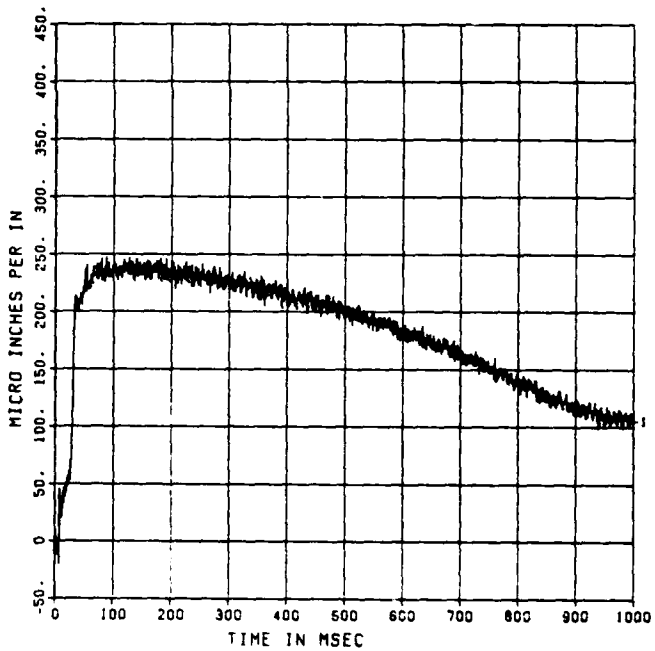
** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
SL3 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 14 R0617



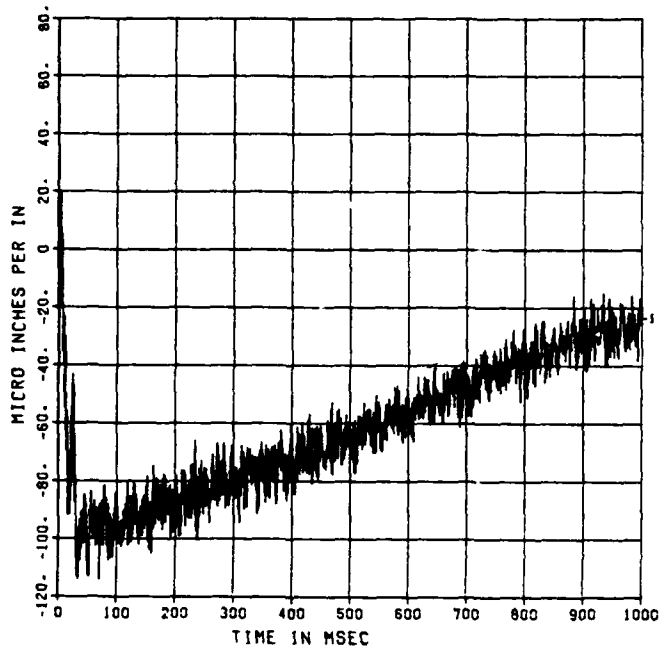
** PEAK VALUE IS 63 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
B1 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 6 R0617



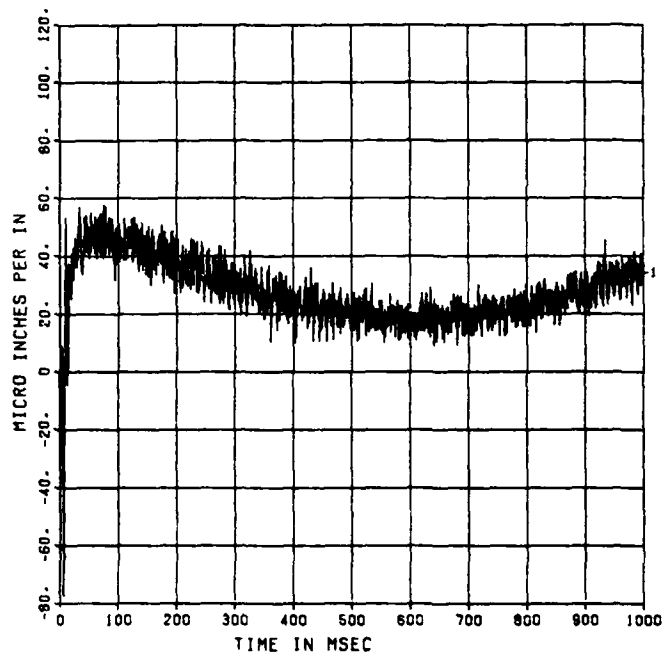
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
B1 BOTTOM
25000. HZ CAL 2899.
F4
06/29/79 12521 5 R0617



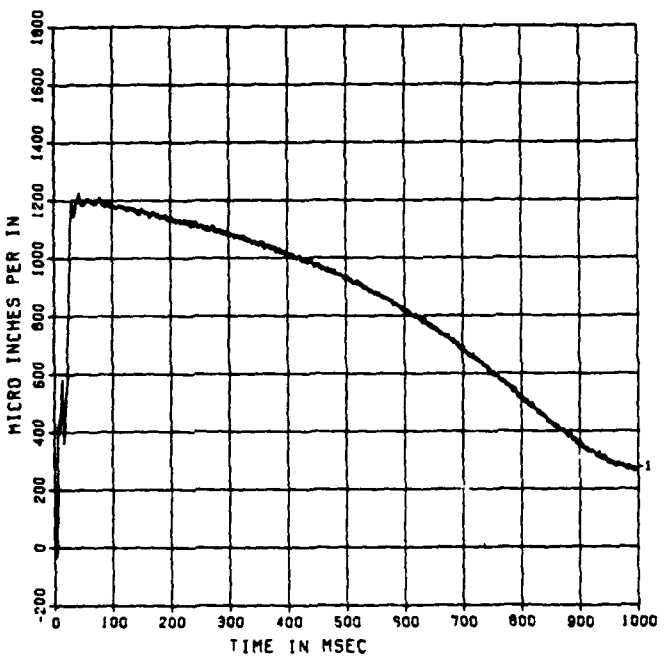
** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
B2 TOP
25000. HZ CAL 2899
F4
06/29/79 12521 11 R0617

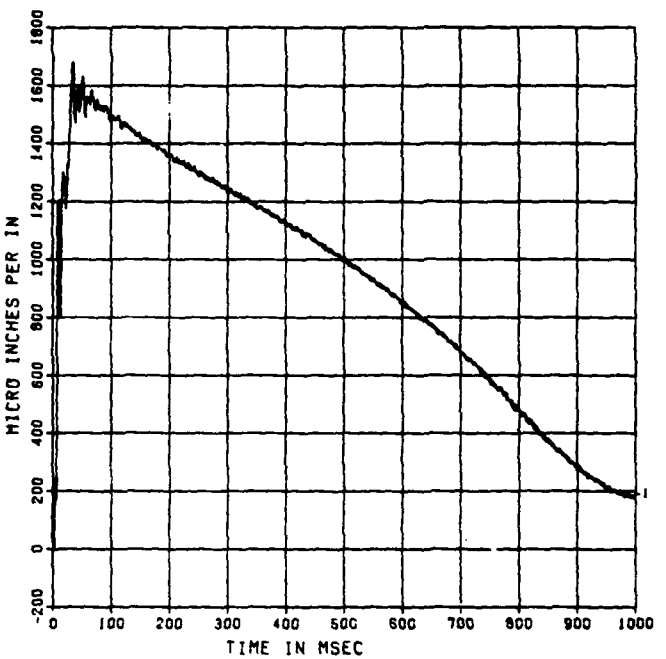


** PEAK VALUE IS -97 % UNDER CALIBRATION **

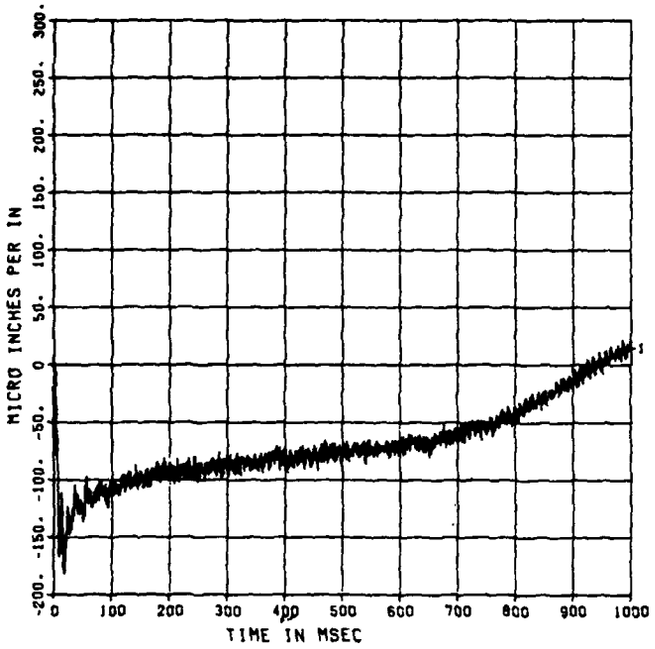
SLAB 2D POST UPGRADE
B2 BOTTOM
25000. HZ CAL 2899.
F4
08/29/79 12521 16 R0617



SLAB 2D POST UPGRADE
B3 BOTTOM
25000. HZ CAL 2899.
F4
08/29/79 12521 16 R0617

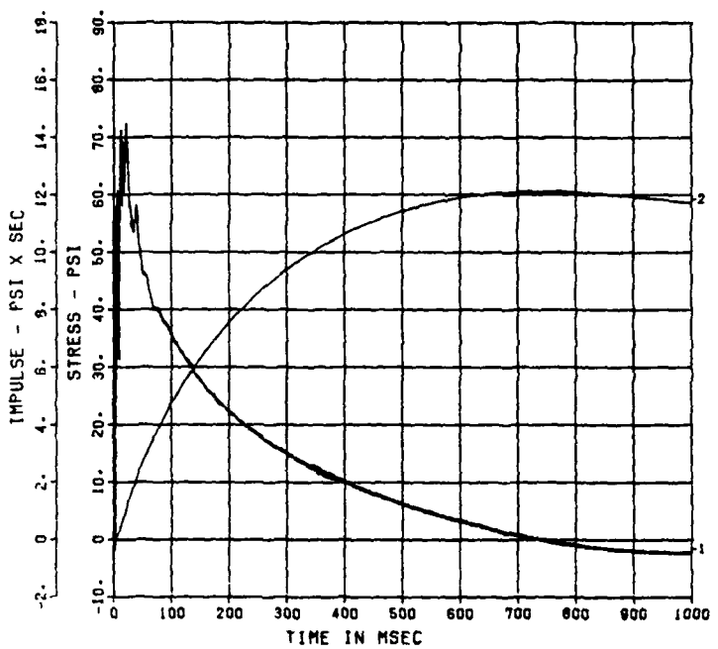


SLAB 2D POST UPGRADE
B3 TOP
25000. HZ CAL 2899.
F4
06/29/79 12521 17 R0617



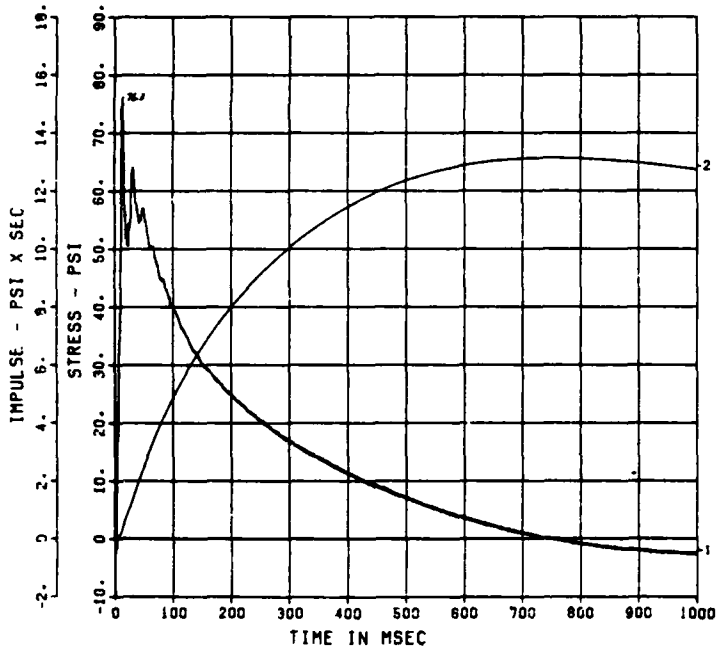
** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 2D POST UPGRADE
EP-1
25000. HZ CAL 33.1
F4
07/11/79 12521 94 R0198



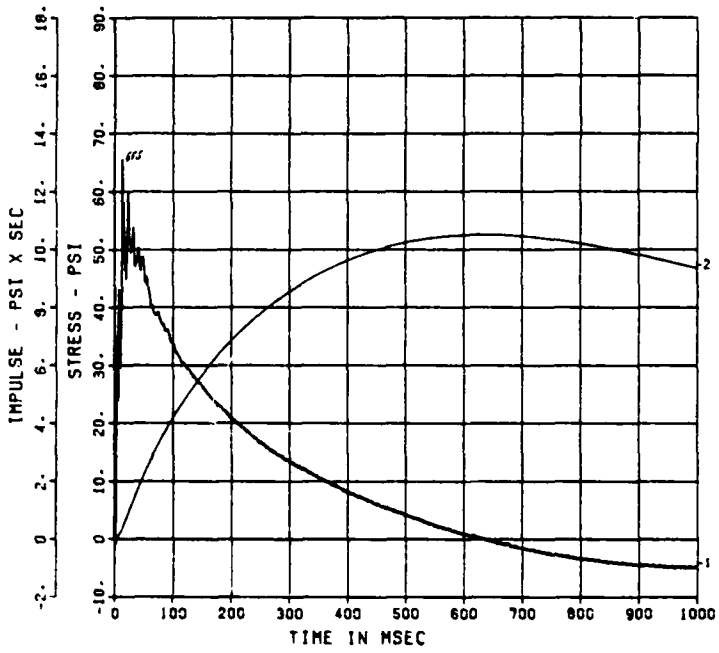
** PEAK VALUE IS 119 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
EP-2
25000. HZ CAL 39.6
F4
07/11/79 12521 95 R0198



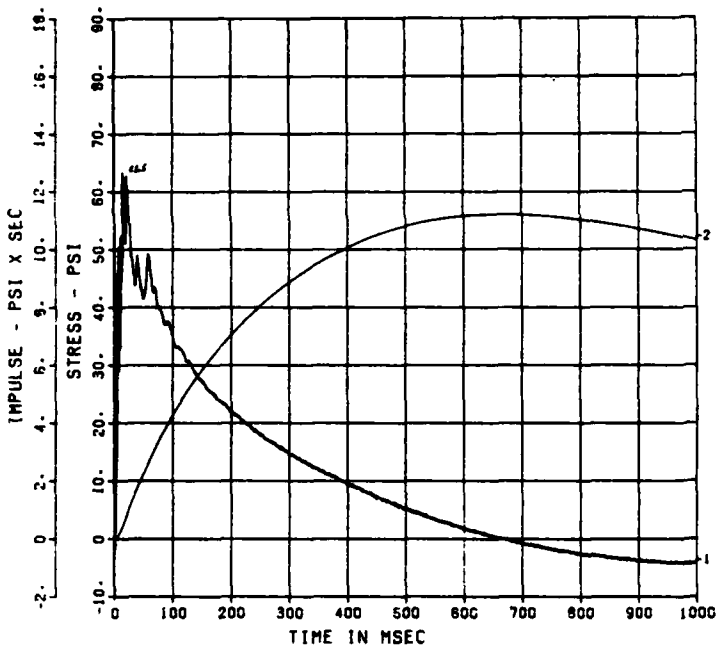
** PEAK VALUE IS 92 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
CP
25000. HZ CAL 40.6
F4
06/29/79 12521 29 R0617



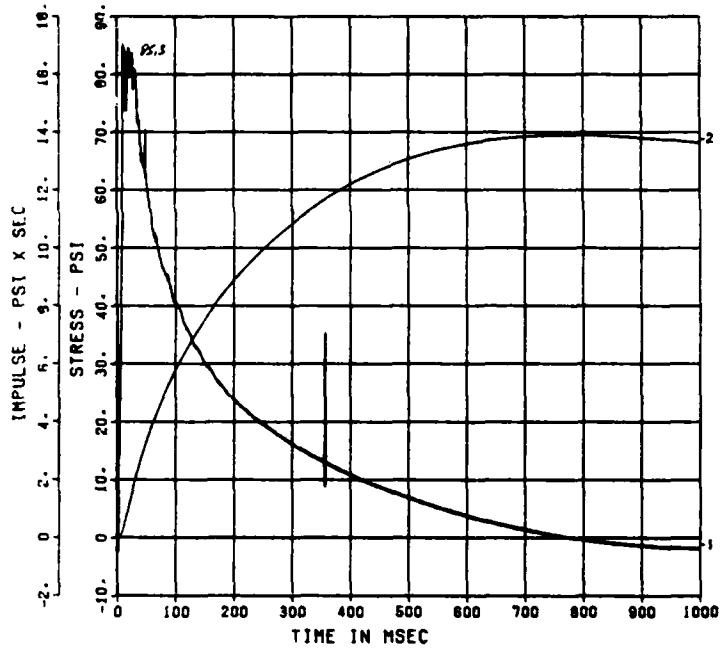
** PEAK VALUE IS 61 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
WP-1
25000. HZ CAL 30.0
F4
08/29/79 12521 28 R0617



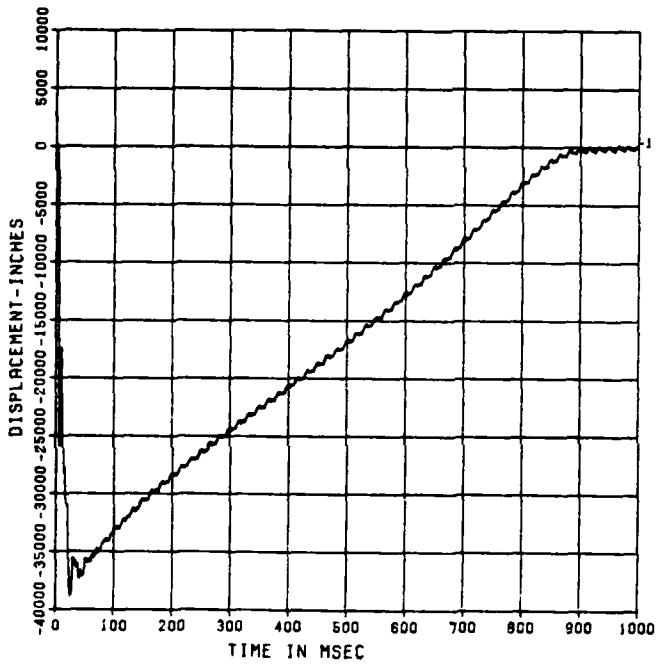
== PEAK VALUE IS 63 % OVER CALIBRATION ==

SLAB 2D POST UPGRADE
WP-2
25000. HZ CAL 29.3
F4
07/11/79 12521 92 R0198



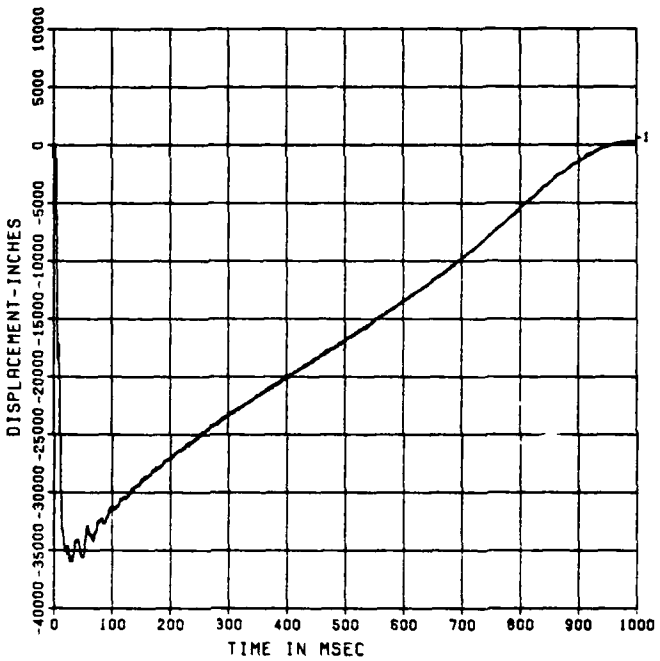
== PEAK VALUE IS 191 % OVER CALIBRATION ==

SLAB 2D POST UPGRADE
LOAD 1
25000. HZ CAL 27162.
F4
07/11/79 12521 98 R0198



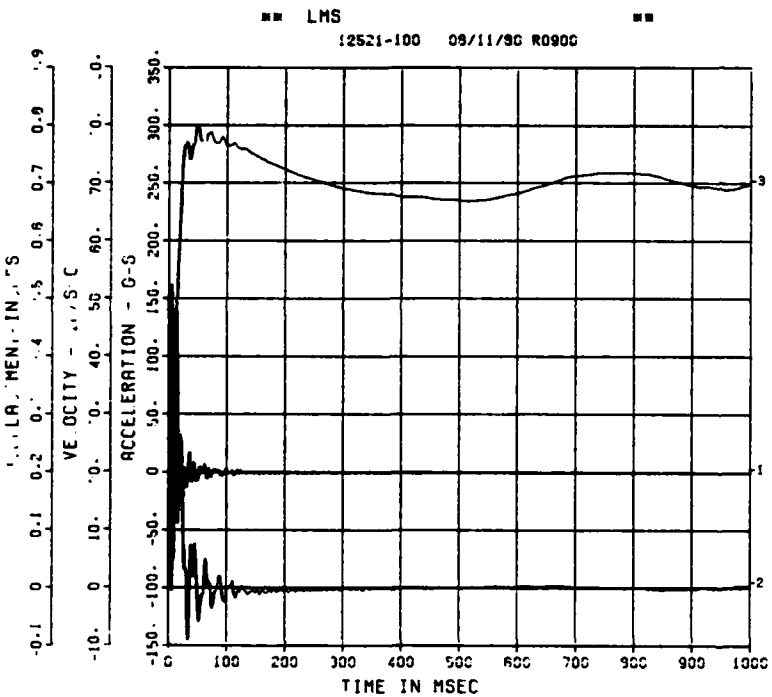
** PEAK VALUE IS 43 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
LOAD 2
25000. HZ CAL 27187.
F4
07/11/79 12521 99 R0198

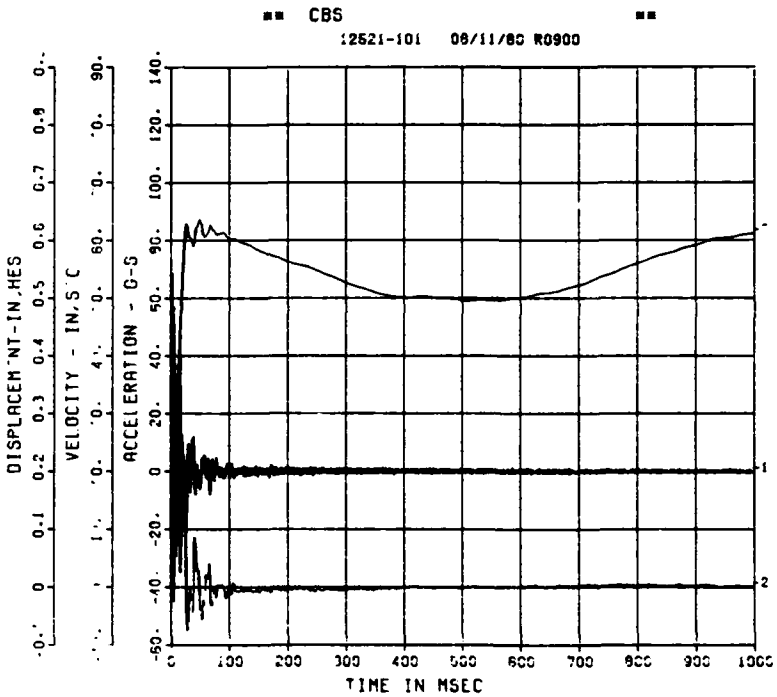


** PEAK VALUE IS 33 % OVER CALIBRATION **

SLAB 2D POST UPGRADE
A1
25000. HZ CAL= 95.21

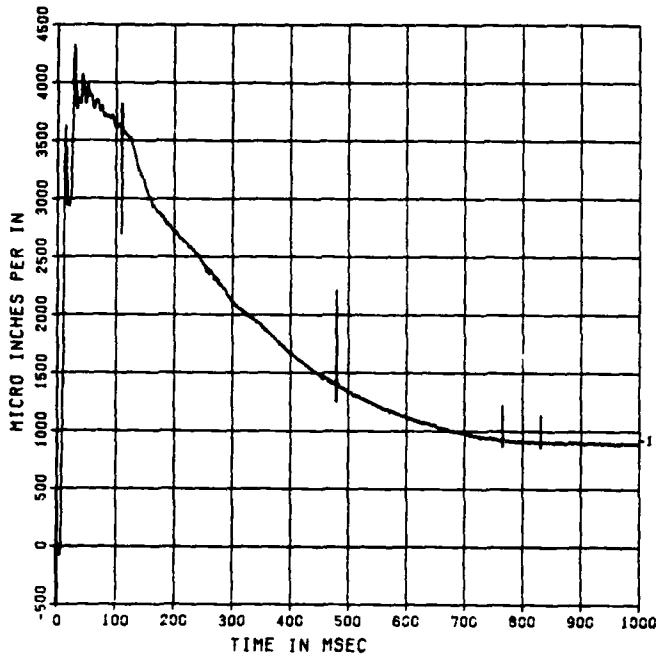


SLAB 2D POST UPGRADE
A2
25000. HZ CAL= 91.66



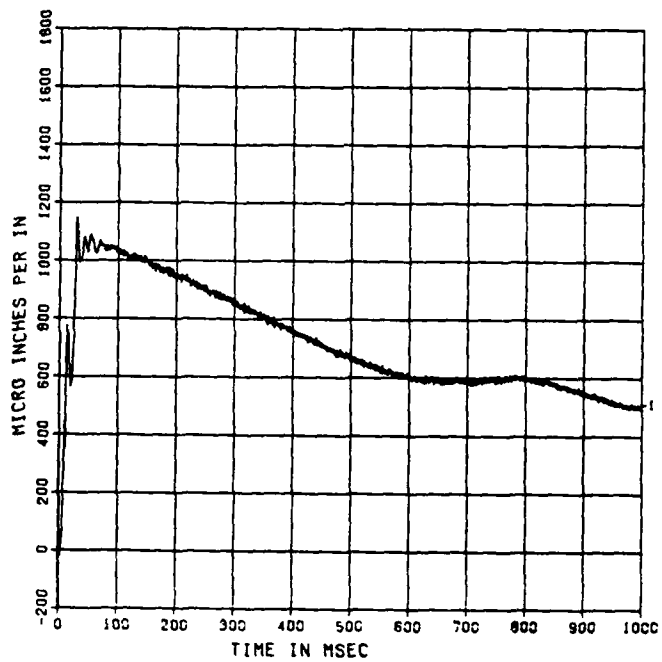
** PEAK VALUE IS 90 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
SS1 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 2 R0209

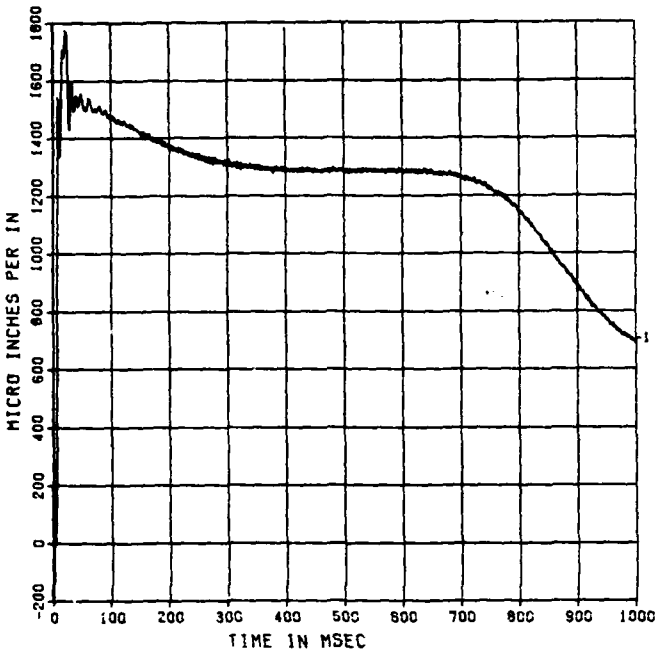


** PEAK VALUE IS 49 % OVER CALIBRATION **

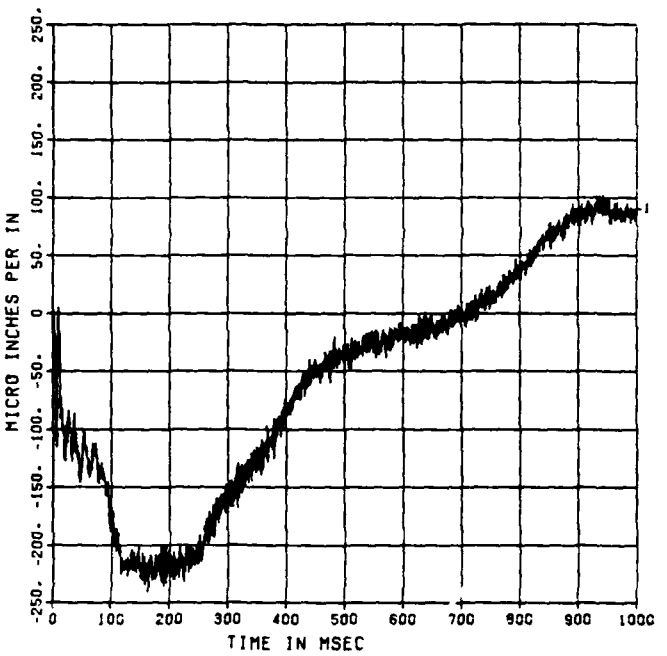
SLAB 2E POST UPGRADE
SS1 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 1 R0209



SLAB 2E POST UPGRADE
SS2 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 8 R0209

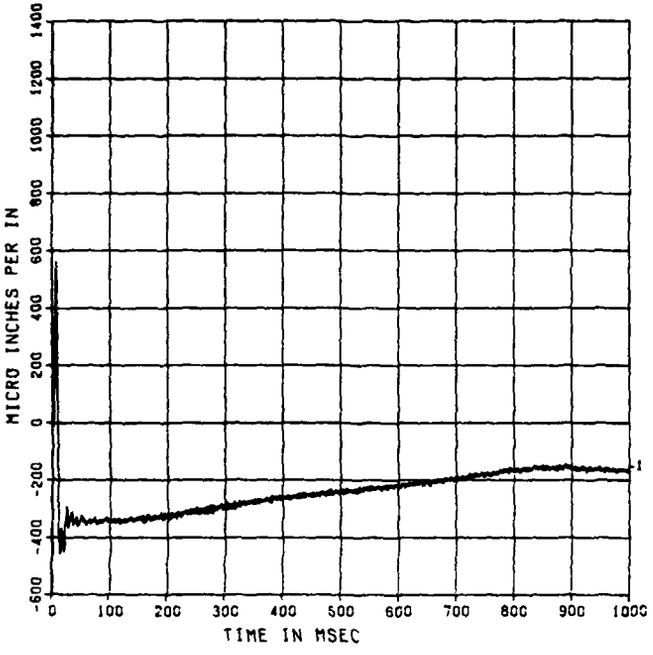


SLAB 2E POST UPGRADE
SS2 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 7 R0209



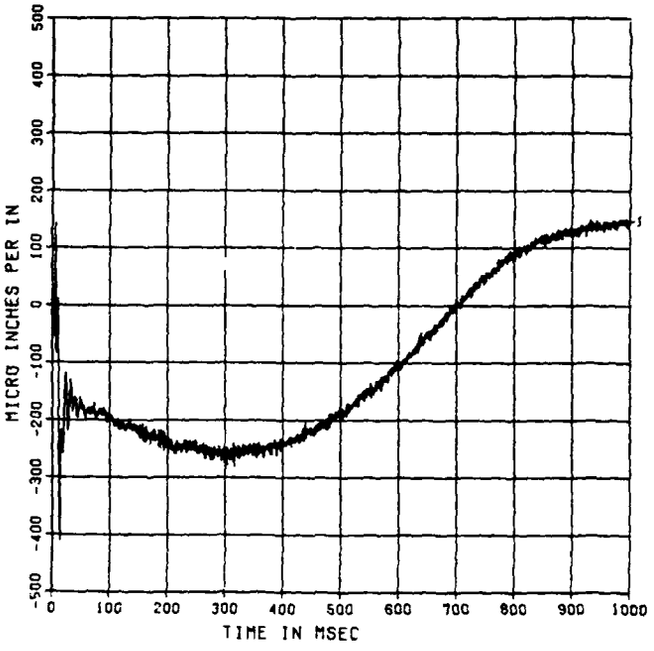
** PEAK VALUE IS -92 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS3 TOP
25000. HZ CAL 2899.
F4
08/21/79 4344 14 R0209



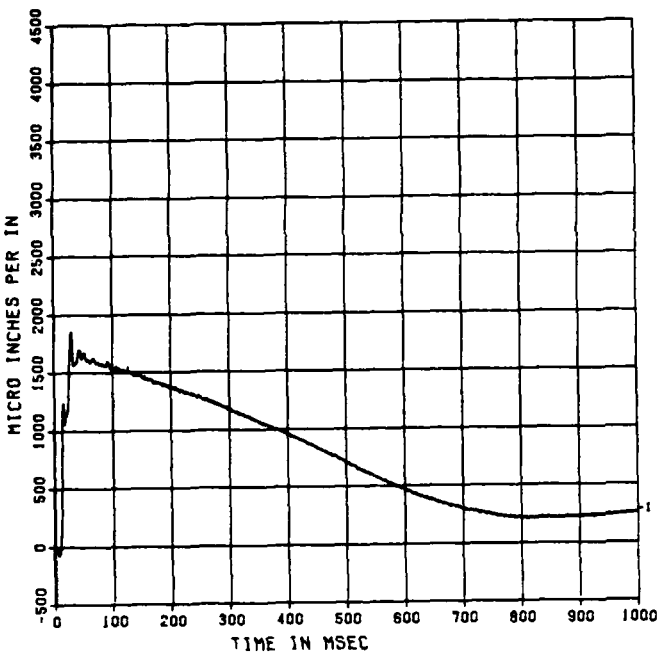
** PEAK VALUE IS -81 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS3 BOTTOM
25000. HZ CAL 2898.
F4
08/21/79 4344 13 R0209

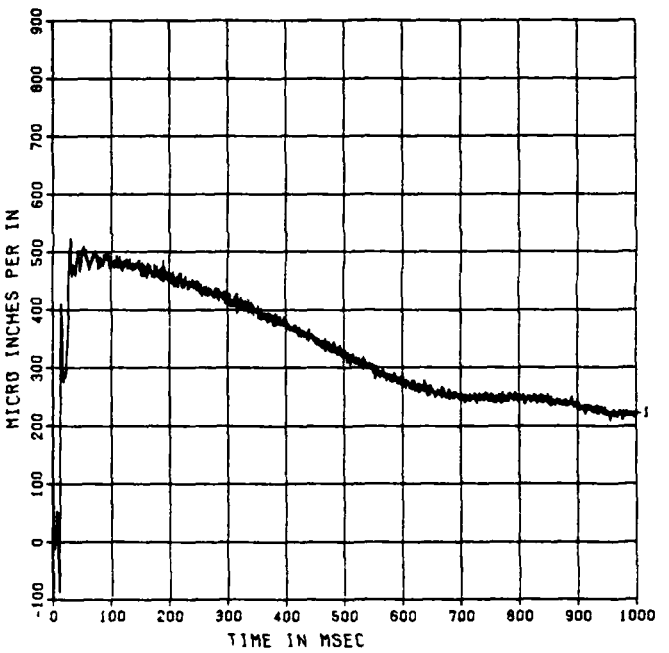


** PEAK VALUE IS -96 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS4 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 20 R0209

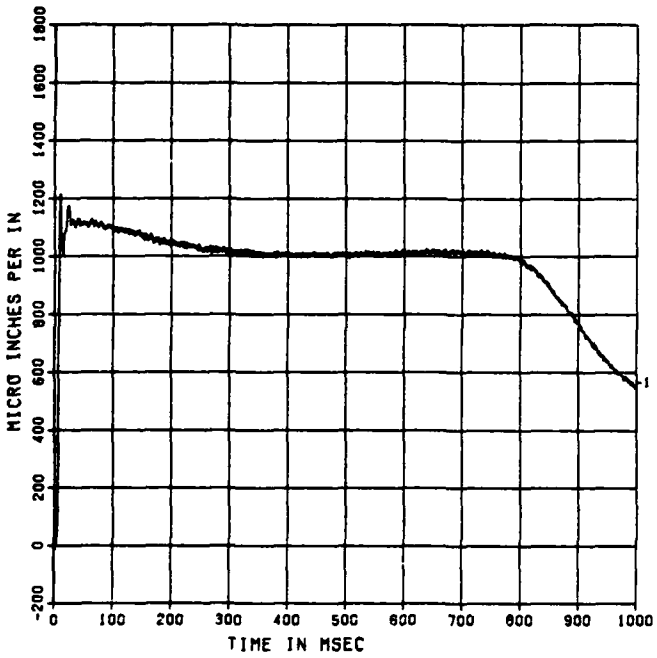


SLAB 2E POST UPGRADE
SS4 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 19 R0209

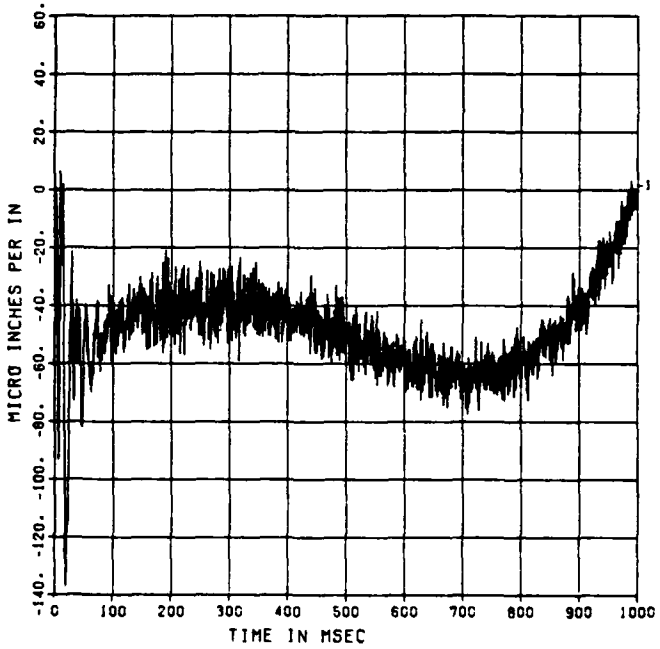


** PEAK VALUE IS -92 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS5 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 22 R0209

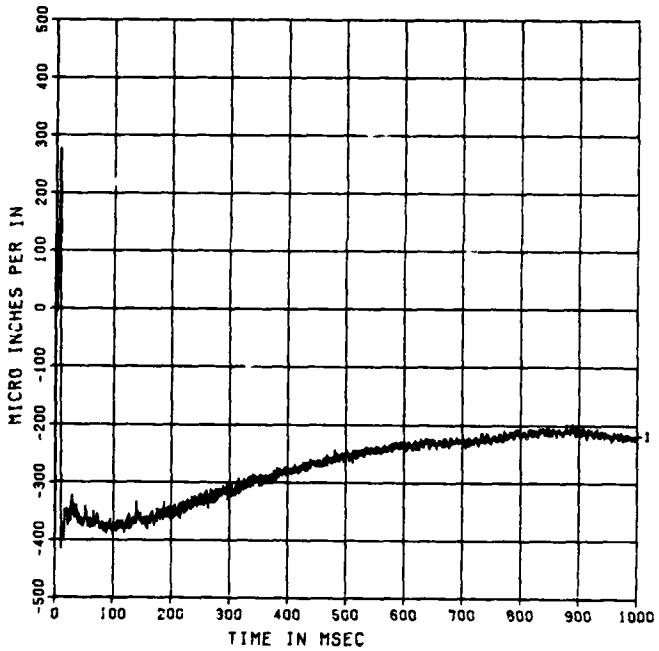


SLAB 2E POST UPGRADE
SS5 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 21 R0209



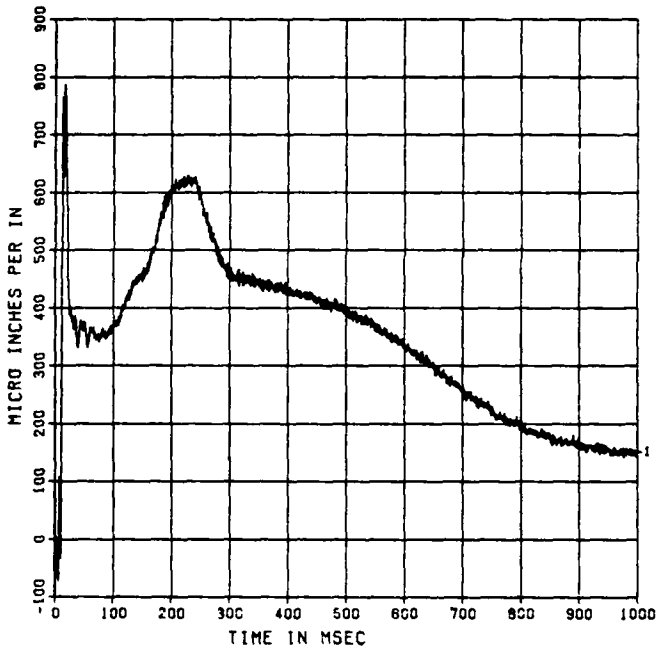
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS6 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 24 R0209

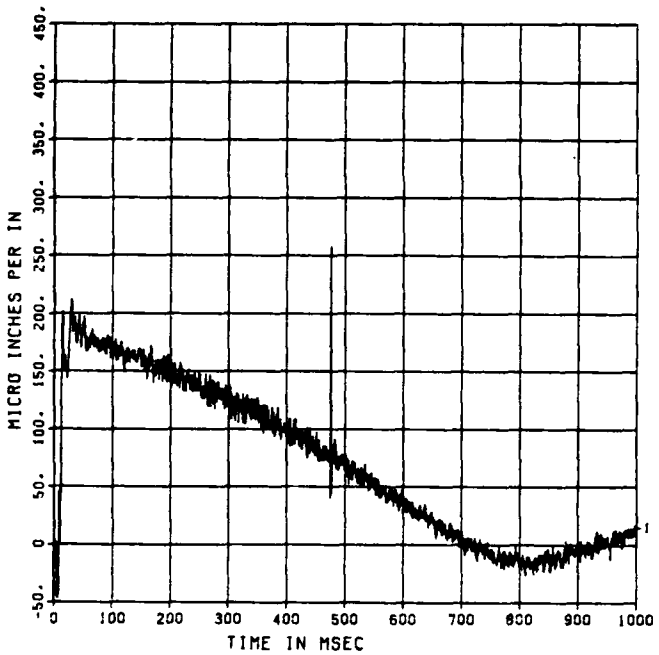


** PEAK VALUE IS -86 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS6 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 23 R0209

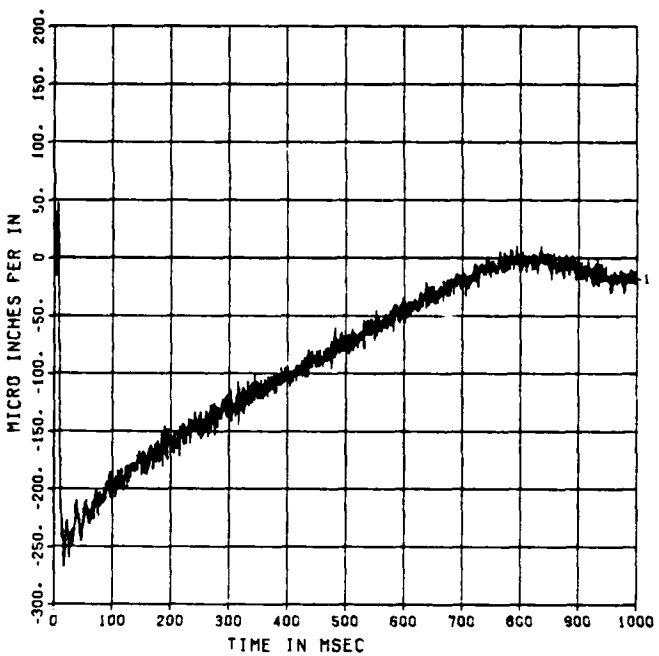


SLAB 2E POST UPGRADE
SS7 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 26 R0209



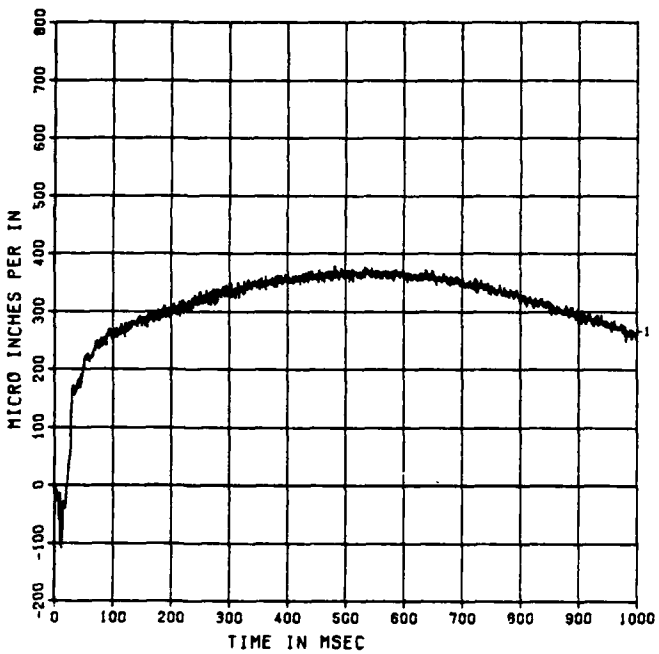
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SS7 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 25 R0209



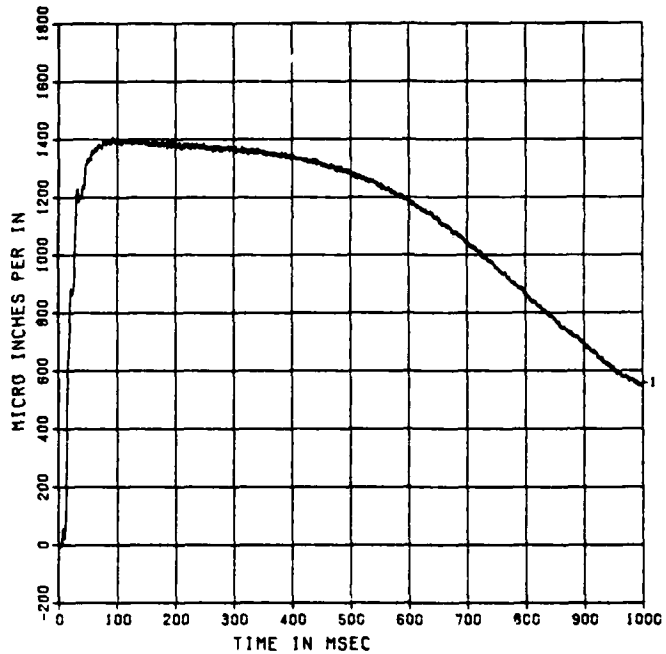
** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SSB TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 28 R0209

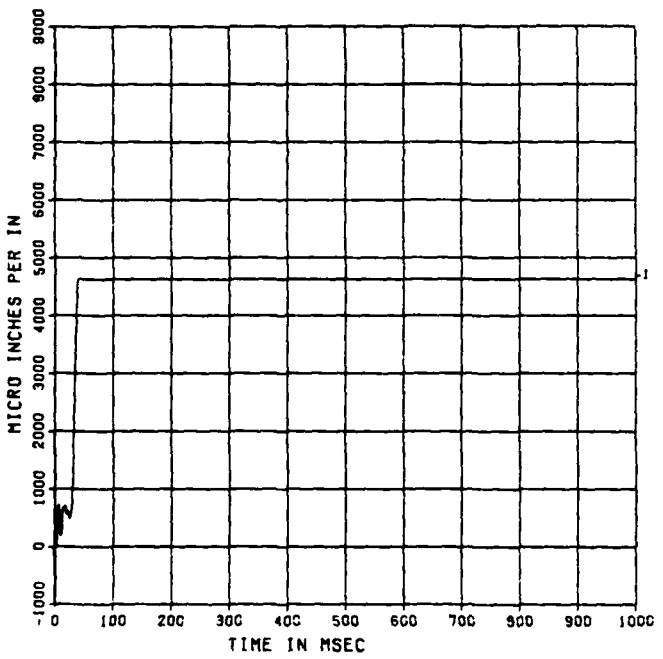


** PEAK VALUE IS -87 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
SSB BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 27 R0209

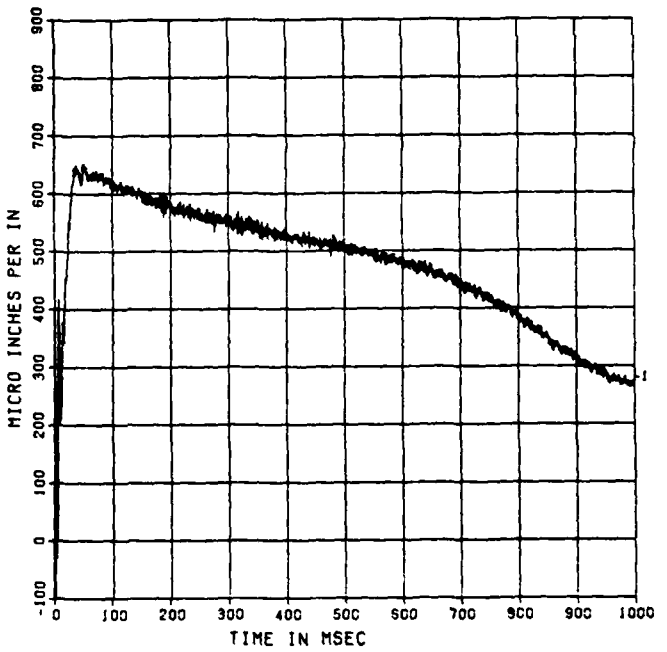


SLAB 2E POST UPGRADE
SL1 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 4 R0209

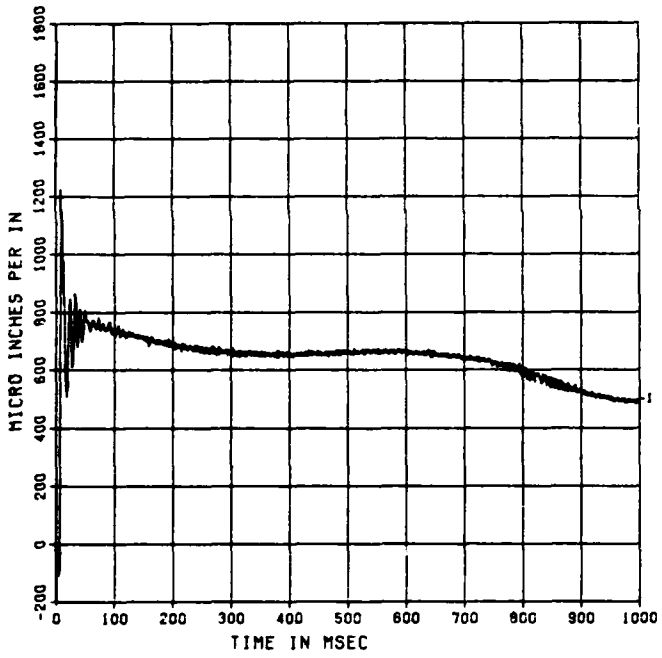


** PEAK VALUE IS 60 % OVER CALIBRATION **

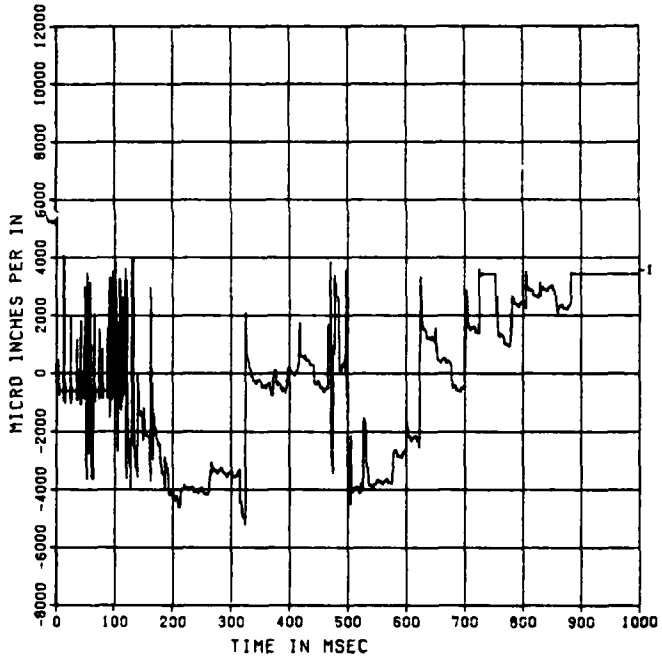
SLAB 2E POST UPGRADE
SL1 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 3 R0209



SLAB 2E POST UPGRADE
SL2 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 10 R0209

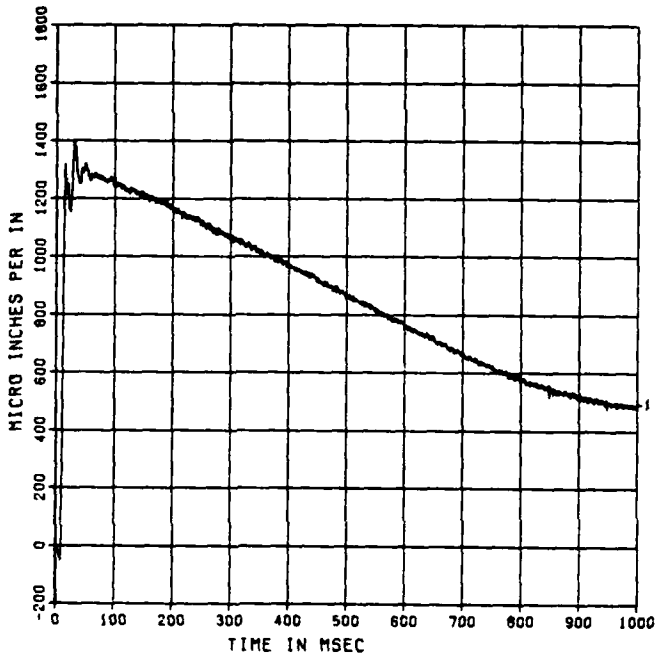


SLAB 2E POST UPGRADE
SL2 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 9 R0209

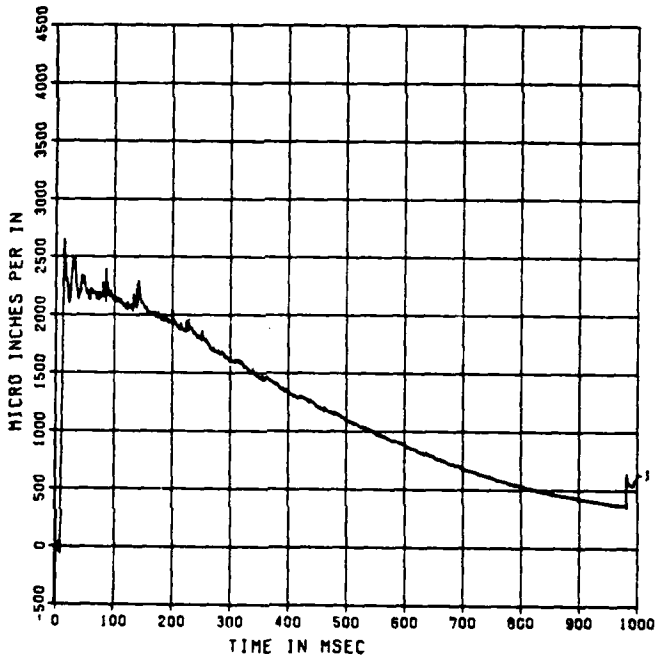


** PEAK VALUE IS 90 % OVER CALIBRATION **

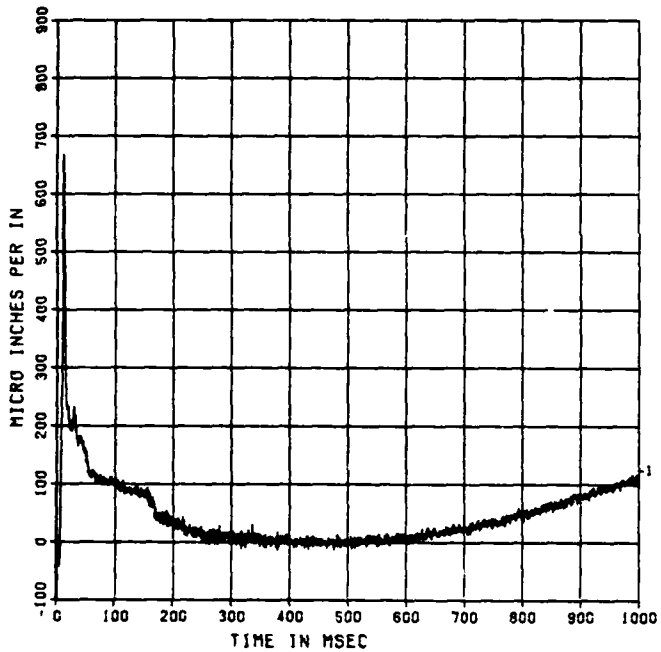
SLAB 2E POST UPGRADE
SL3 TOP
25000. HZ CAL 2899.
F4
08/21/79 4344 16 R0209



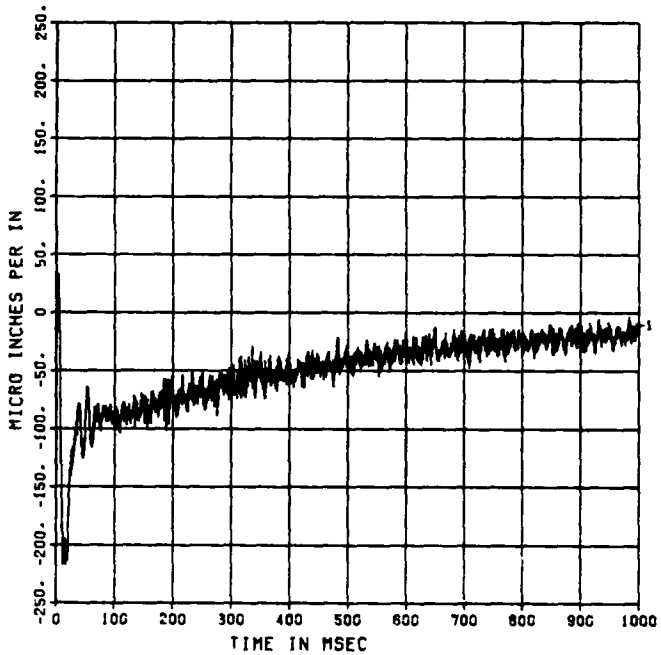
SLAB 2E POST UPGRADE
SL3 BOTTOM
25000. HZ CAL 2899.
F4
08/21/79 4344 15 R0209



SLAB 2E POST UPGRADE
B1 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 6 R0209

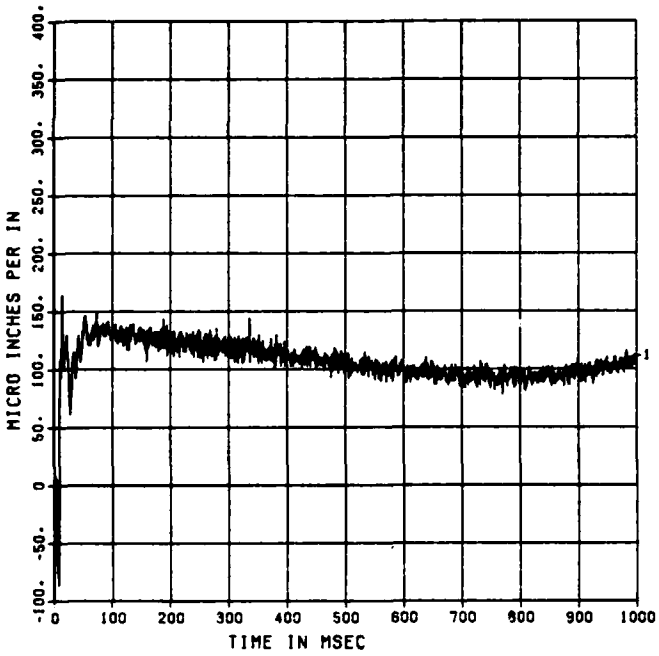


SLAB 2E POST UPGRADE
B1 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 5 R0209



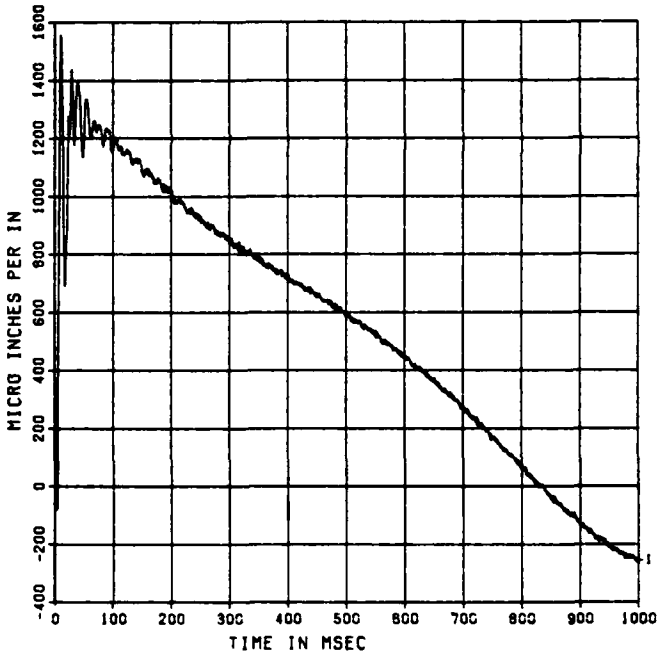
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 2E POST UPGRADE
B2 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 12 R0209

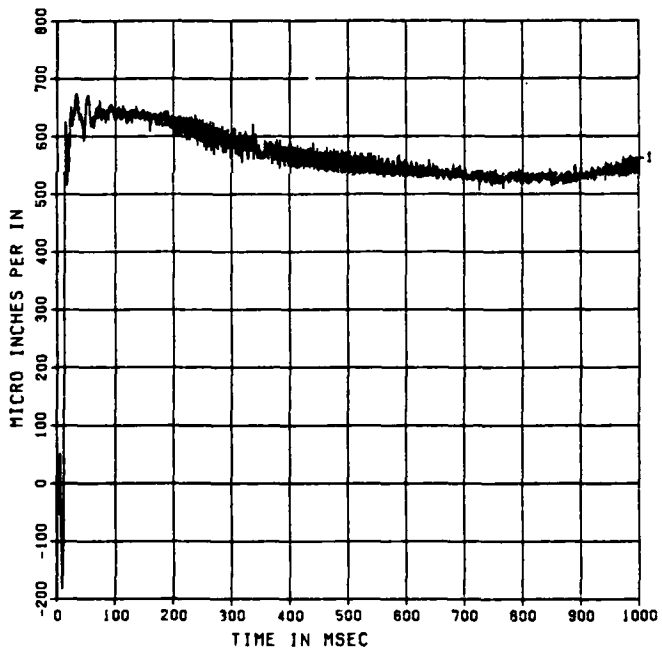


** PEAK VALUE IS -94 % UNDER CALIBRATION **

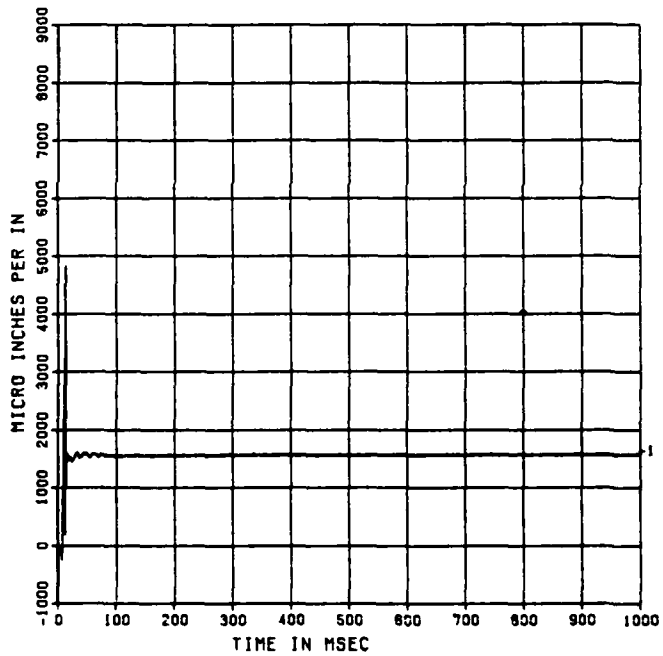
SLAB 2E POST UPGRADE
B2 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 11 R0209



SLAB 2E POST UPGRADE
B3 TOP
25000. HZ CAL 2899.
F4
06/21/79 4344 18 R0209

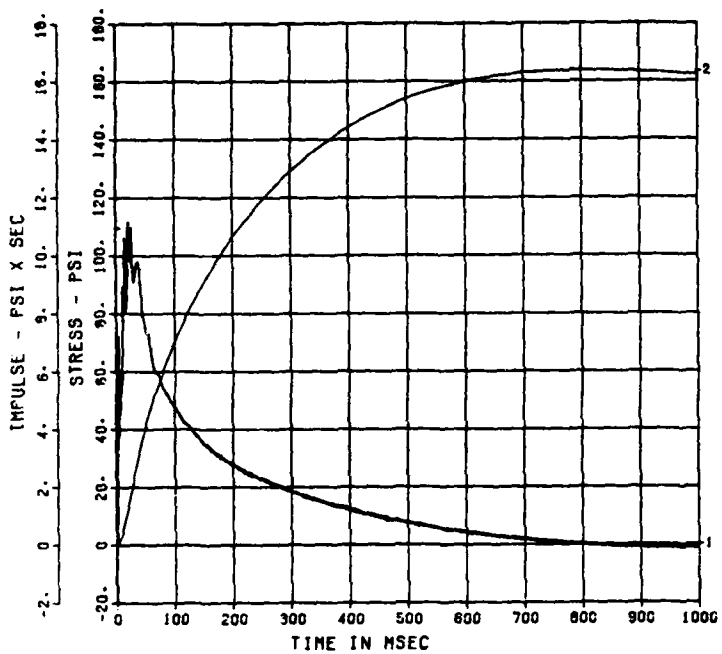


SLAB 2E POST UPGRADE
B3 BOTTOM
25000. HZ CAL 2899.
F4
06/21/79 4344 17 R0209

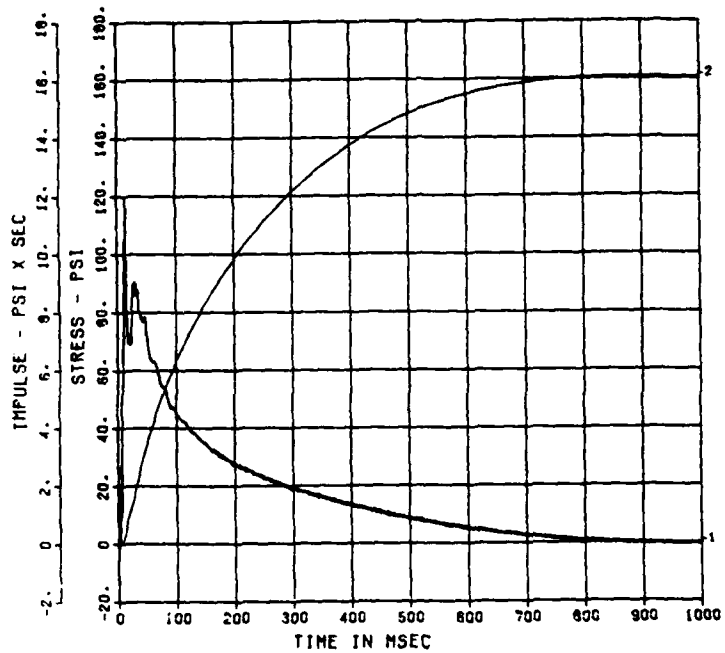


** PEAK VALUE IS 66 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
EP-1
25000. HZ MSB CAL 110.68
F4
06/25/79 4344 61 R0390

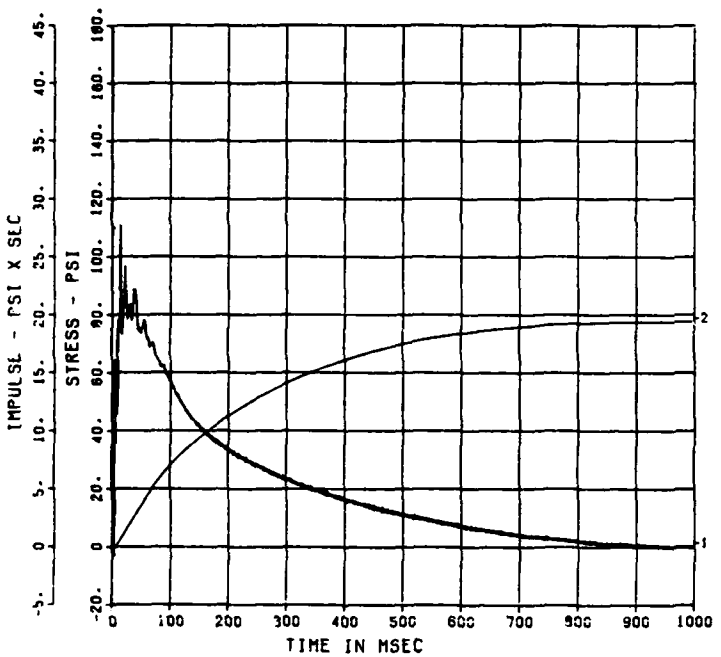


SLAB 2E POST UPGRADE
EP-2
25000. HZ MSB CAL 106.03
F4
06/25/79 4344 62 R0390

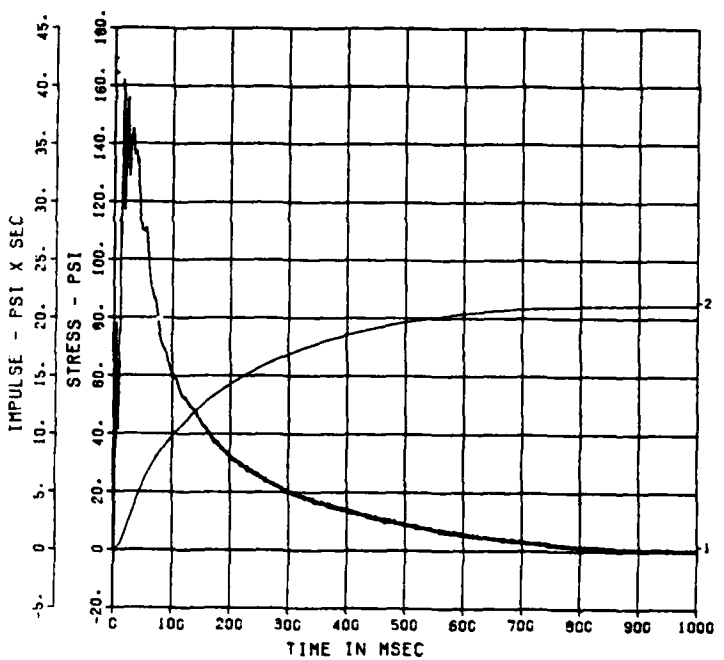


** PEAK VALUE IS 19 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
CP
25000. HZ MSB CAL 121.70
F4
06/25/79 4344 60 R0390

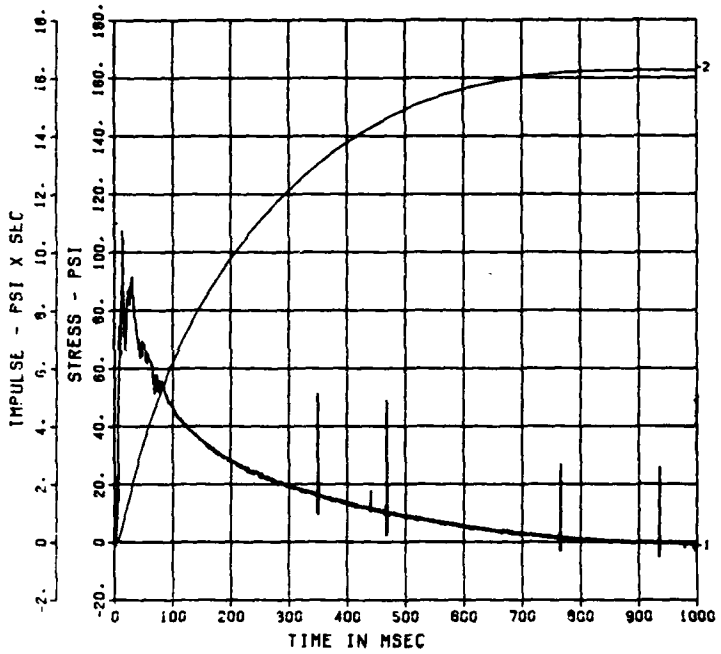


SLAB 2E POST UPGRADE
WP-1
25000. HZ MSB CAL 129.39
F4
06/25/79 4344 58 R0390

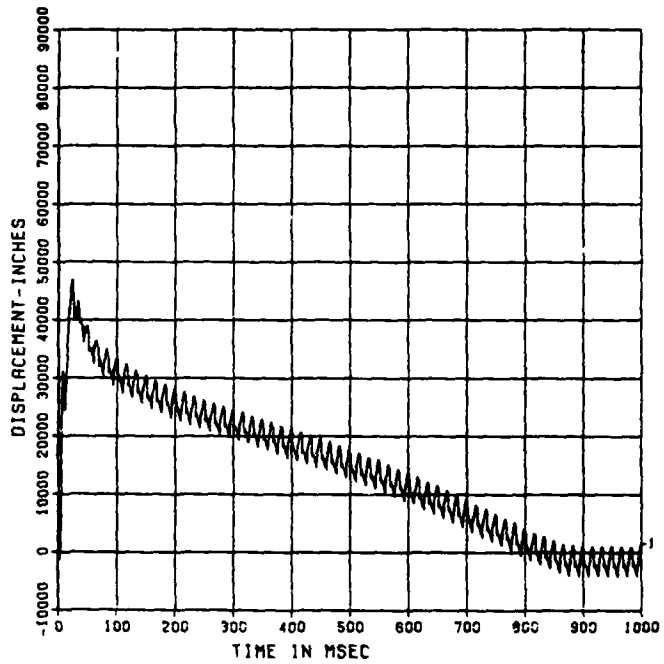


** PEAK VALUE IS 31 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
WP-2
25000. HZ MSB CAL 116.84
F4
08/25/79 4344 59 R0390

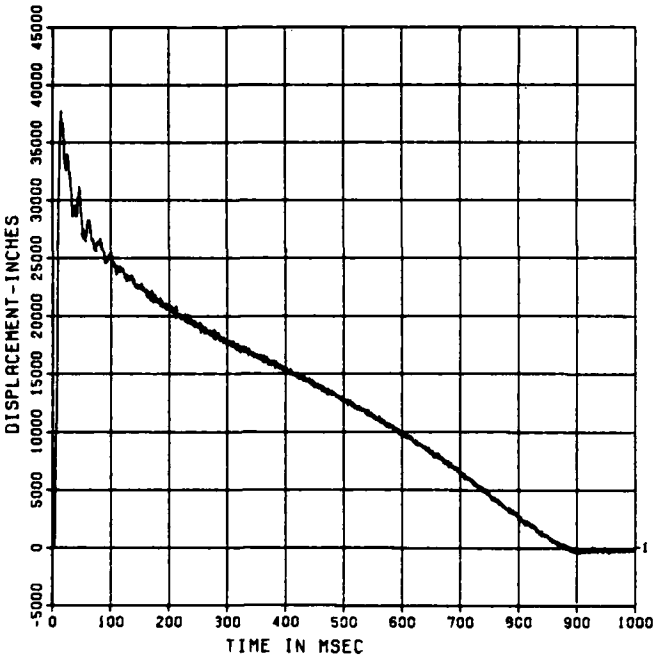


SLAB 2E POST UPGRADE
LOAD 1
25000. HZ CAL 40382.
F4
08/21/79 4344 65 R0209

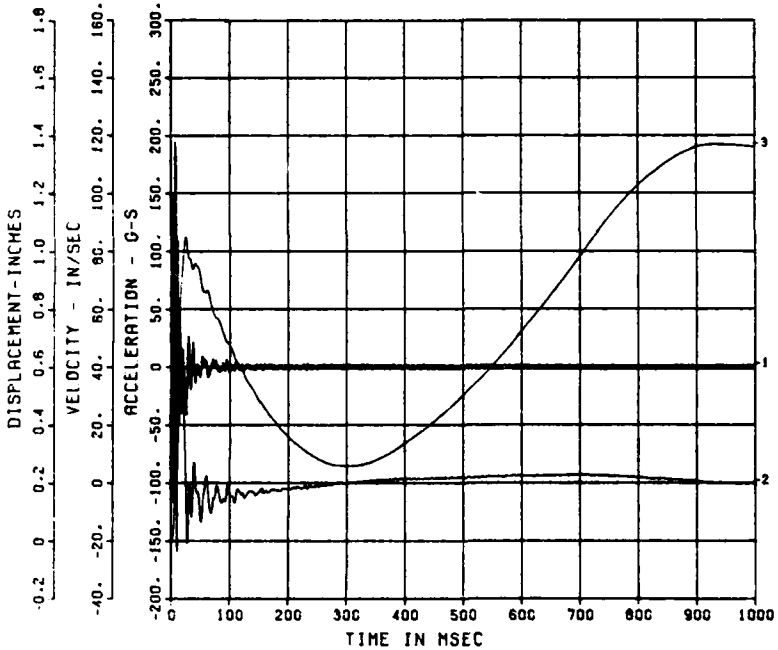


** PEAK VALUE IS 16 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
LOAD 2
25000. HZ CAL 39623.
F4
06/21/79 4344 66 R0209

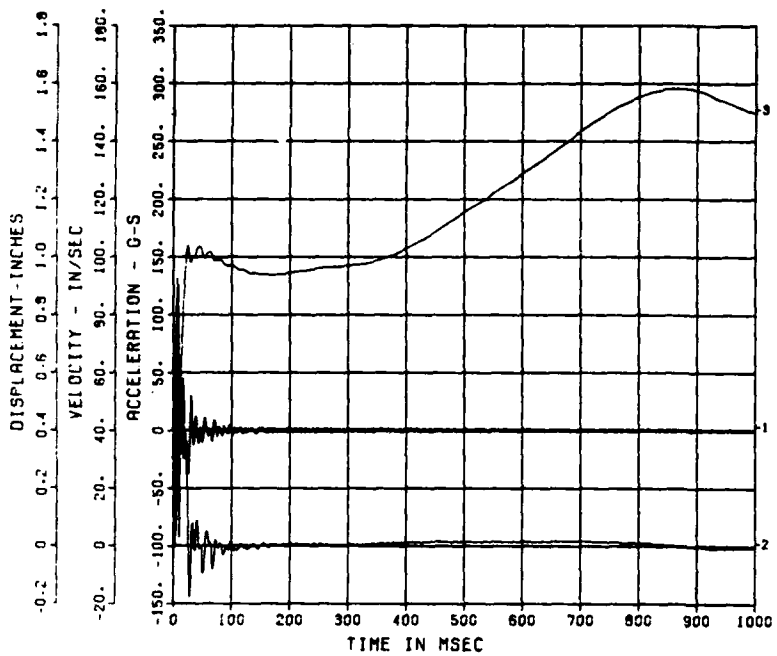


SLAB 2E POST UPGRADE
A1
25000. HZ CBS CAL 95.21
06/25/79 4344 67 R0390



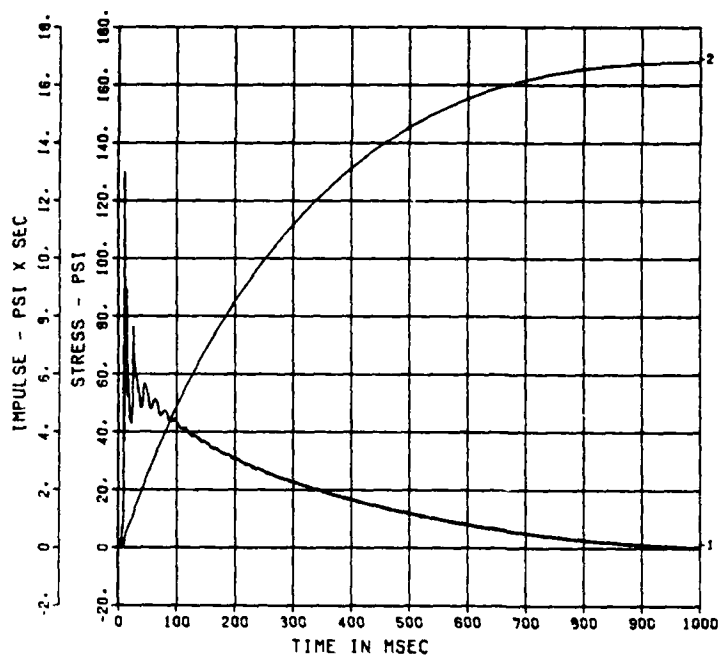
** PEAK VALUE IS 67 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
A2
25000. HZ CBS CAL 91.88
06/25/79 4344 88 R0390



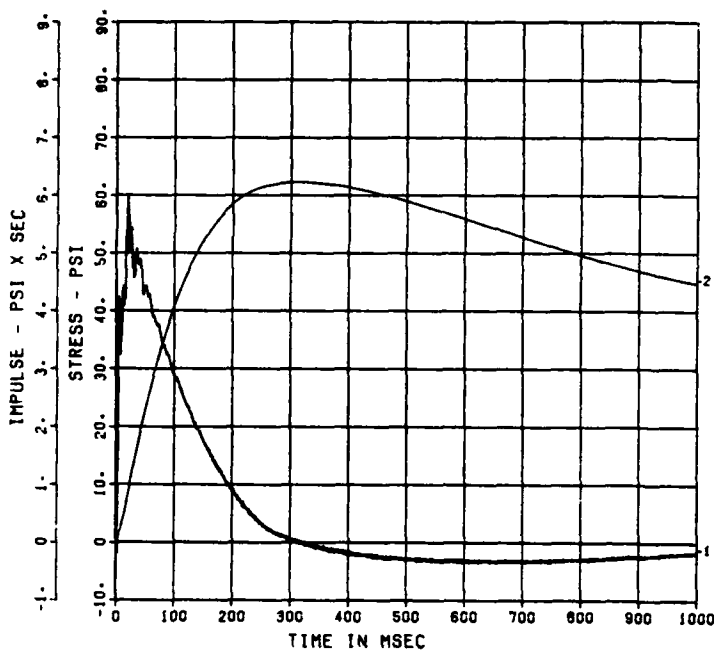
** PEAK VALUE IS 19 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
SE-1
25000. HZ CAL 79.91
F4
06/21/79 4344 71 R0209

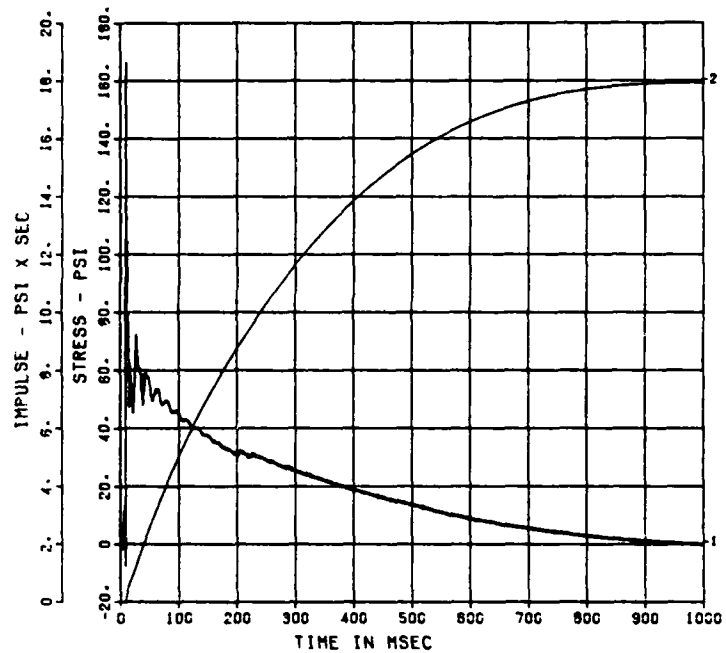


** PEAK VALUE IS 63 % OVER CALIBRATION **

SLAB 2E POST UPGRADE
SE-2
25000. HZ CAL 72.74
F4
06/21/79 4344 72 R0209



SLAB 2E POST UPGRADE
NORWOOD
25000. HZ MSB CAL 100.78
F4
06/25/79 4344 73 R0390



** PEAK VALUE IS 65 % OVER CALIBRATION **

APPENDIX C

STRUCTURE 3 TEST DATA

Data from the tests on the steel beam upgrading system (Structure 3) are presented in this appendix. Labels on the plots indicate the following:

Line 1: Test name

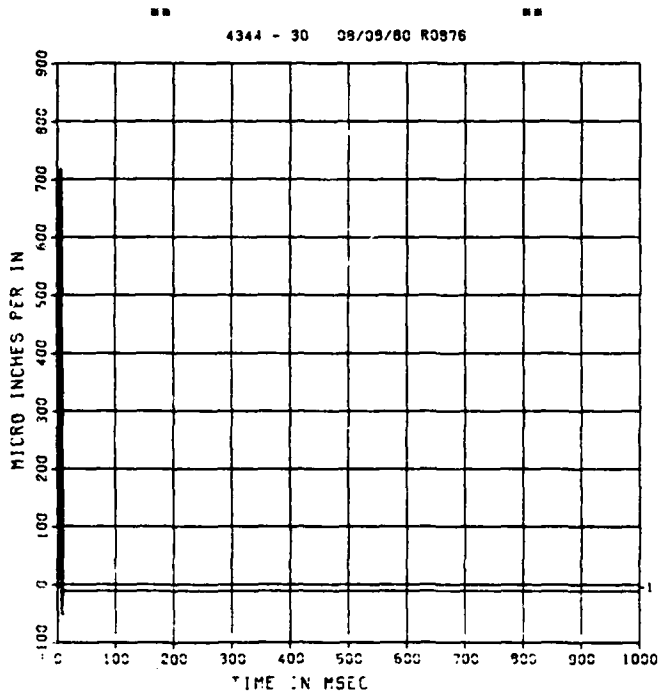
Line 2: Gage symbol and location (see Figure 4.3)

Line 3: Digitizing rate and calibration of gage

Line 4: Filter (if any)

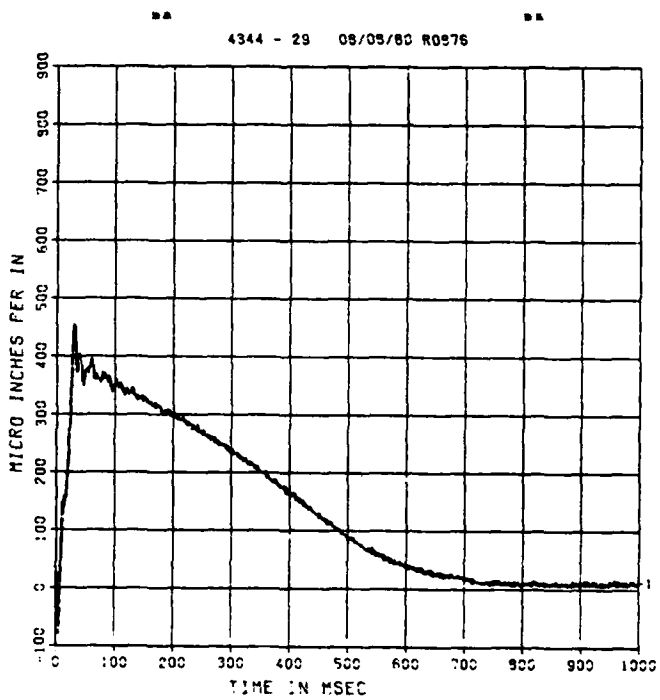
Line 5: Bookkeeping data

SLAB 3A BEAM UPGRADE
SS1 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



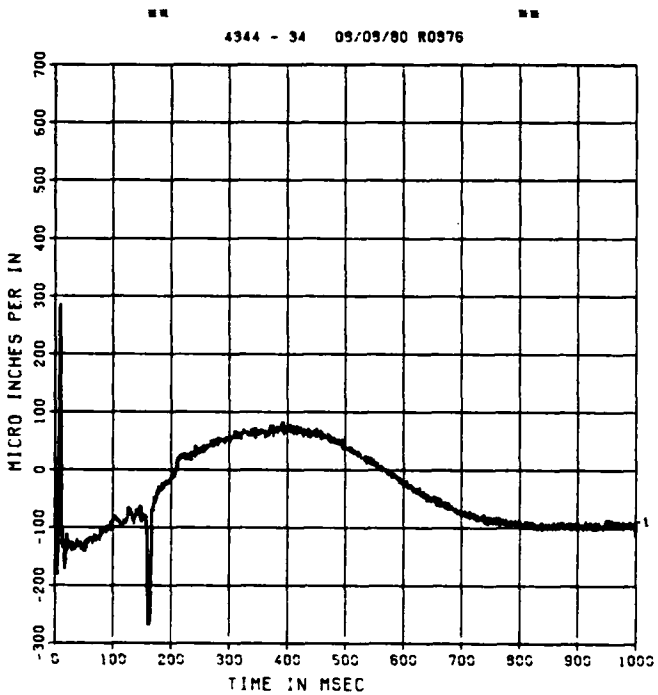
** PEAK VALUE IS 105 % OVER CALIBRATION **

SLAB 3A BEAM UPGRADE
SS1 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

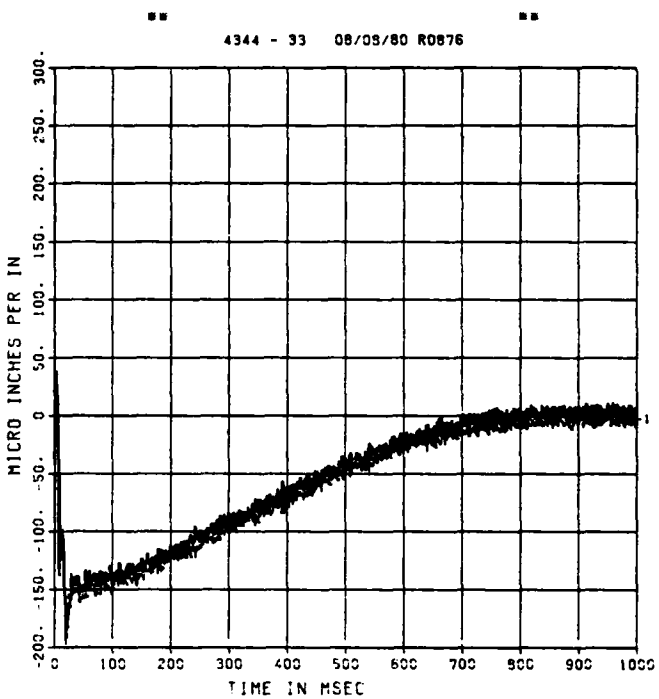


** PEAK VALUE IS 30 % OVER CALIBRATION **

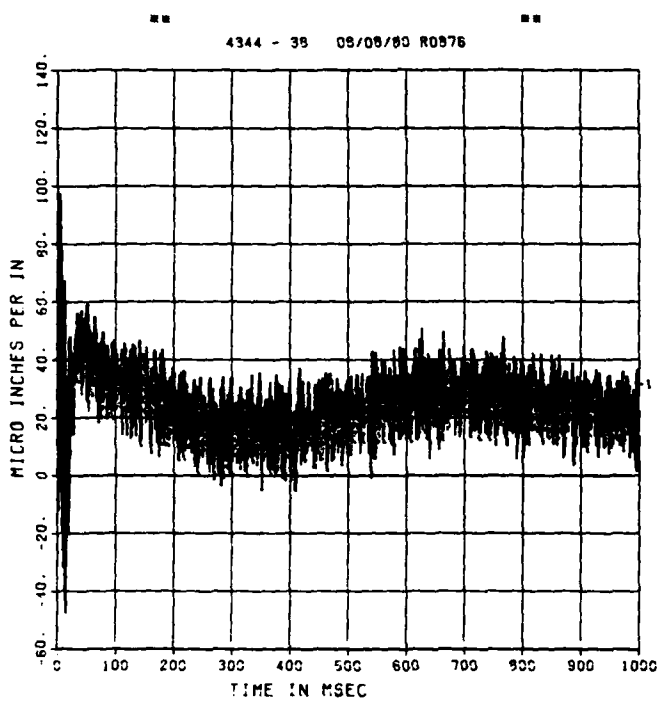
SLAB 3A BEAM UPGRADE
SS2 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



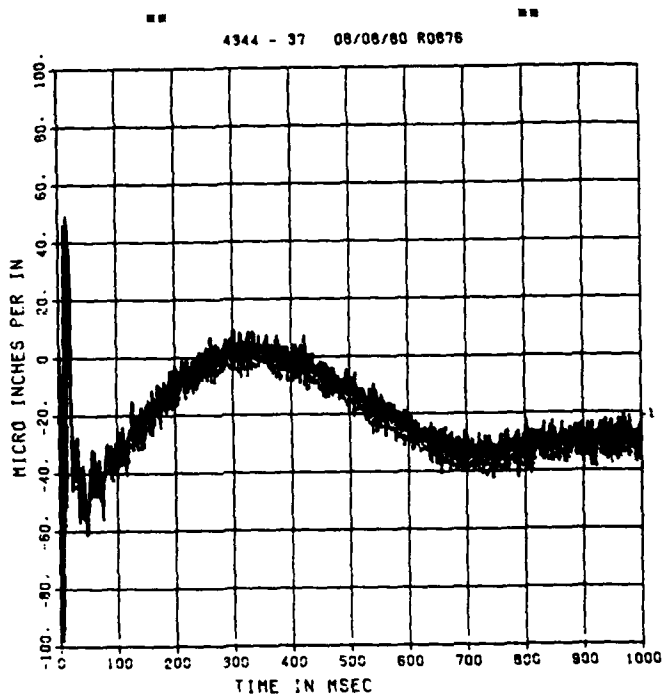
SLAB 3A BEAM UPGRADE
SS2 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



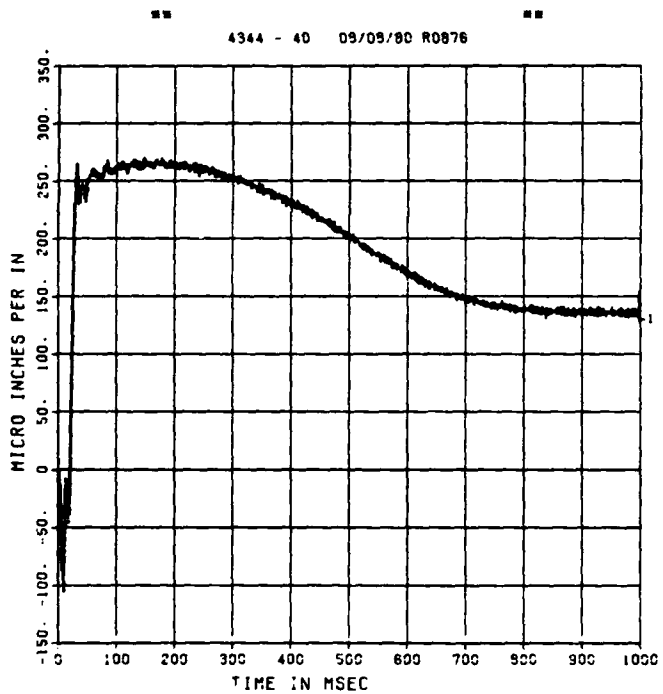
SLAB 3A BEAM UPGRADE
SS3 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



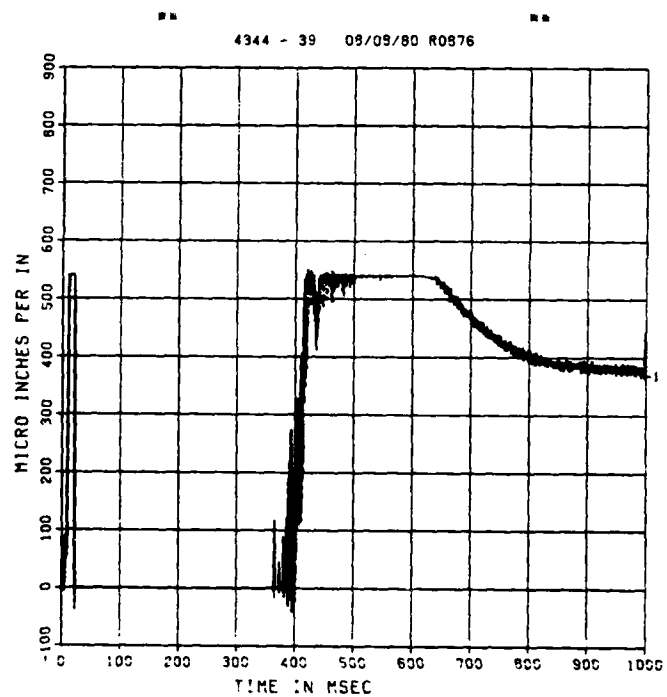
SLAB 3A BEAM UPGRADE
SS3 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



SLAB 3A BEAM UPGRADE
SS4 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

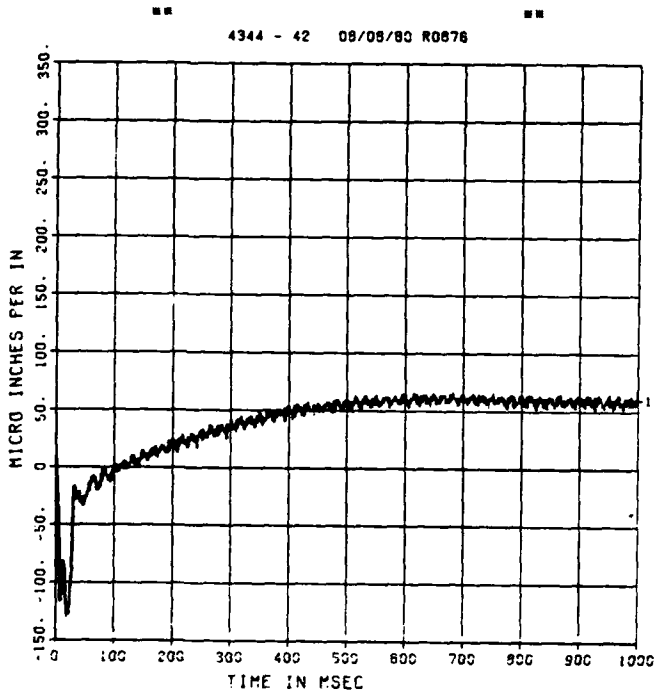


SLAB 3A BEAM UPGRADE
SS4 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125 HZ

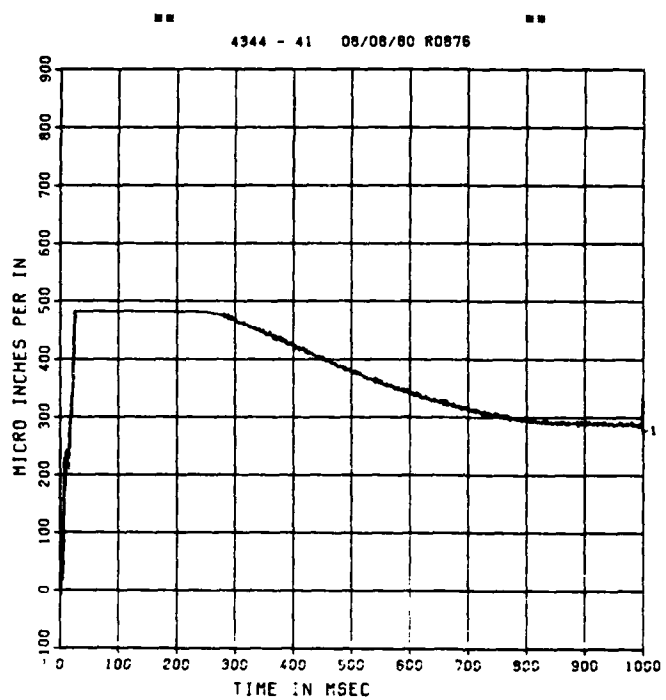


** PEAK VALUE IS 55 % OVER CALIBRATION **

SLAB 3A BEAM UPGRADE
SS5 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

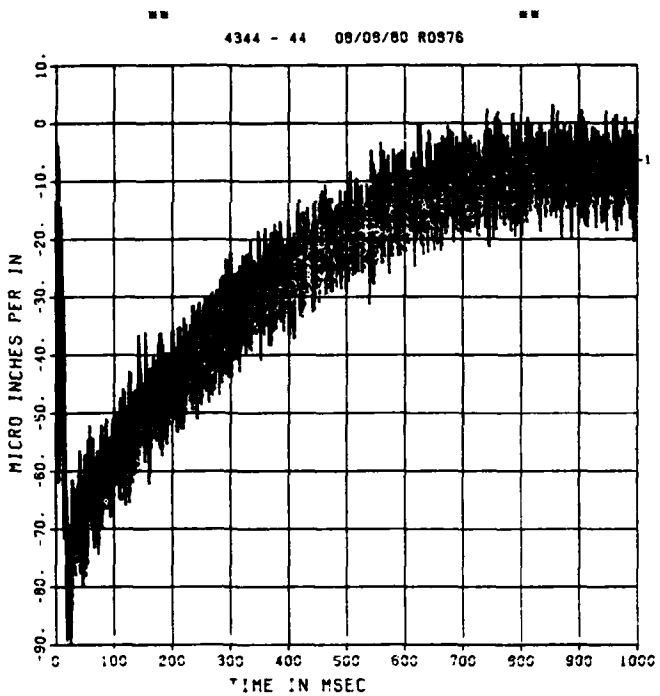


SLAB 3A BEAM UPGRADE
SS5 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

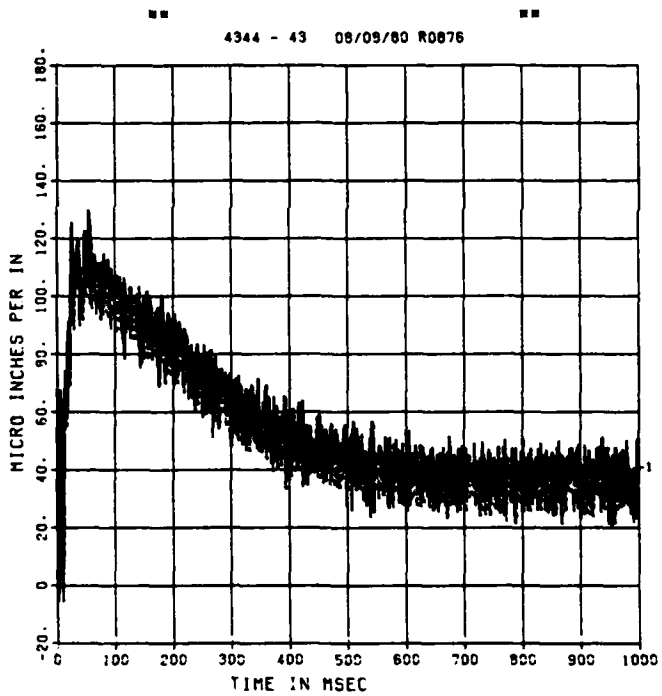


** PEAK VALUE IS 39 % OVER CALIBRATION **

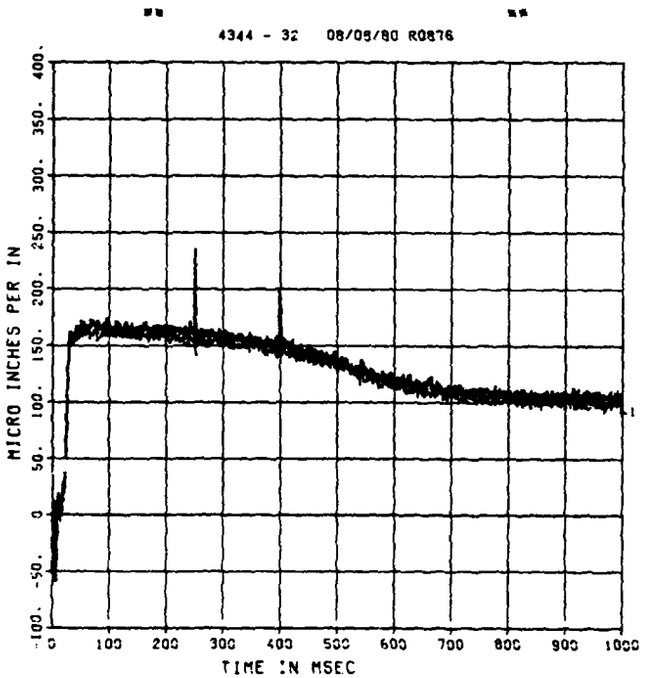
SLAB 3A BEAM UPGRADE
SS6 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



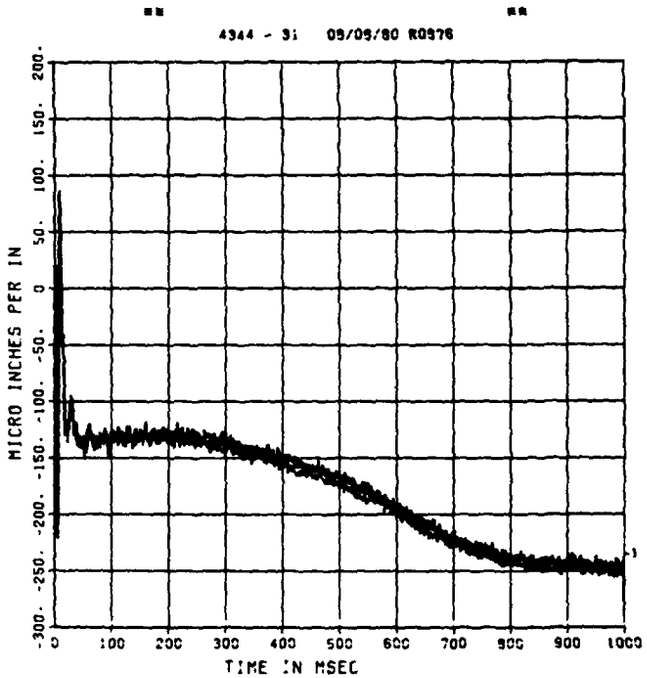
SLAB 3A BEAM UPGRADE
SS6 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



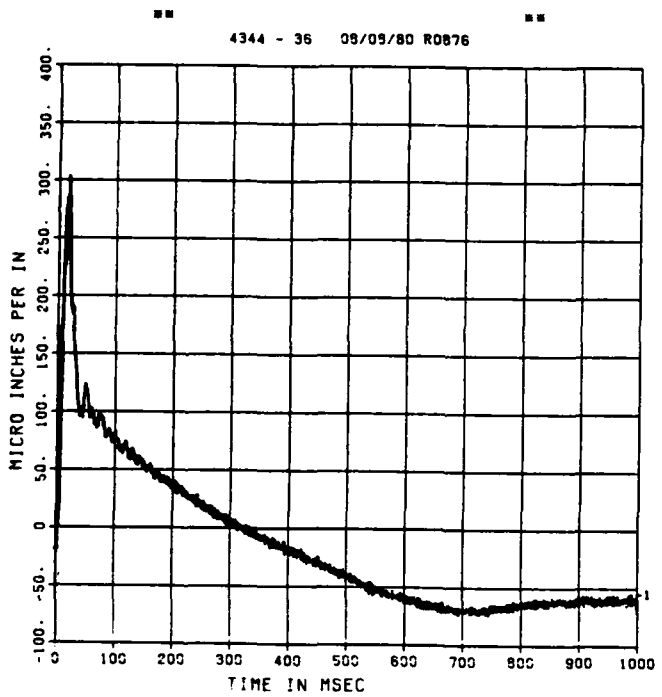
SLAB 3A BEAM UPGRADE
SL1 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



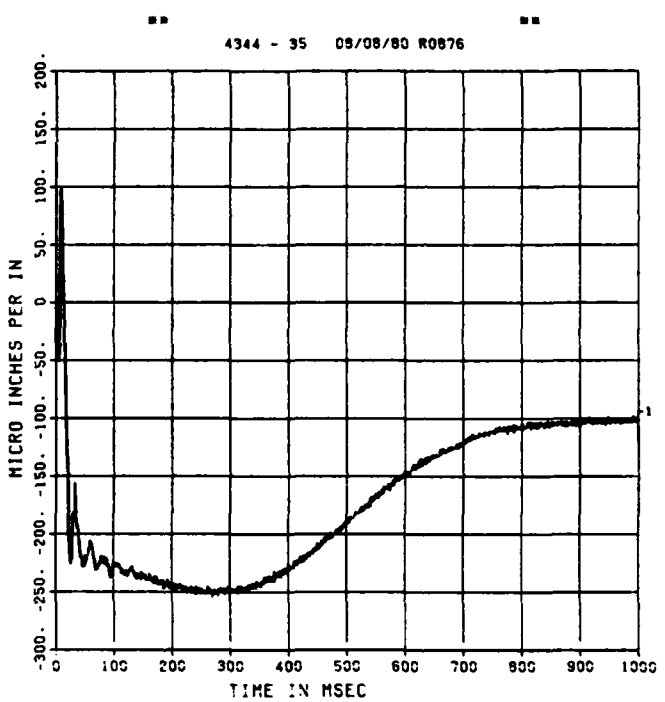
SLAB 3A BEAM UPGRADE
SL1 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



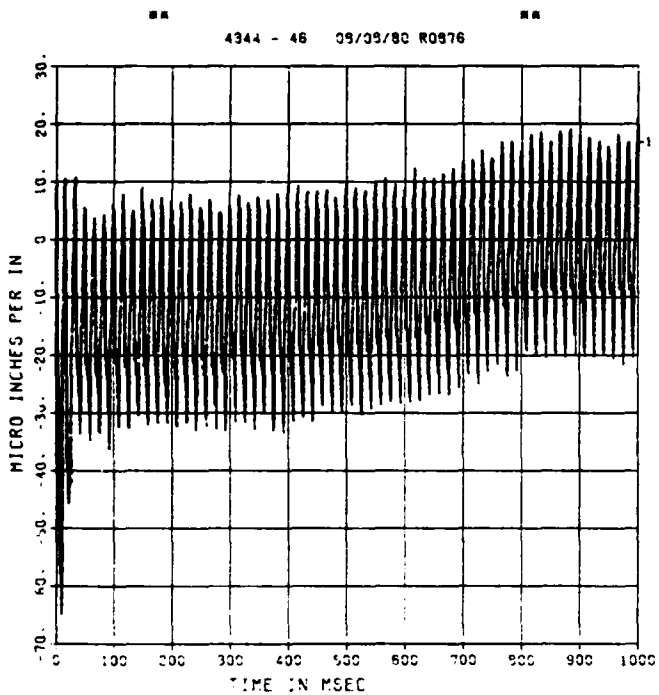
SLAB 3A BEAM UPGRADE
SL2 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



SLAB 3A BEAM UPGRADE
SL2 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

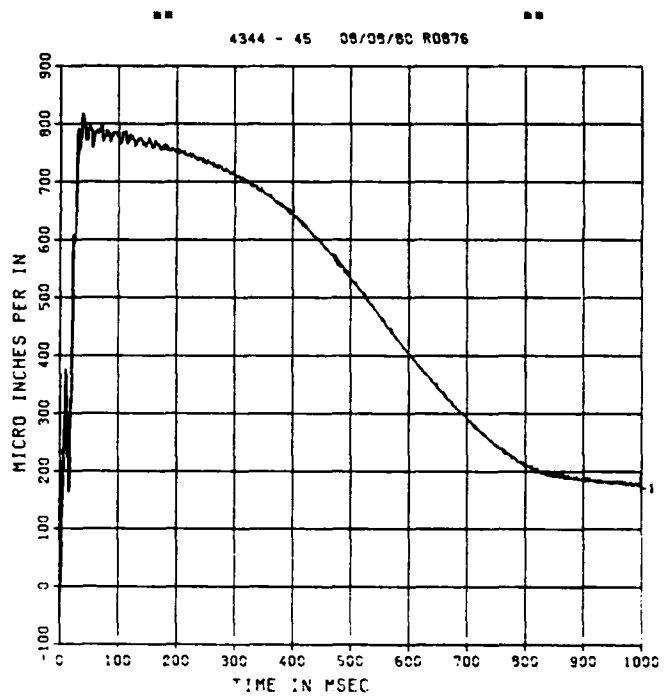


SLAB 3A BEAM UPGRADE
BS1 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



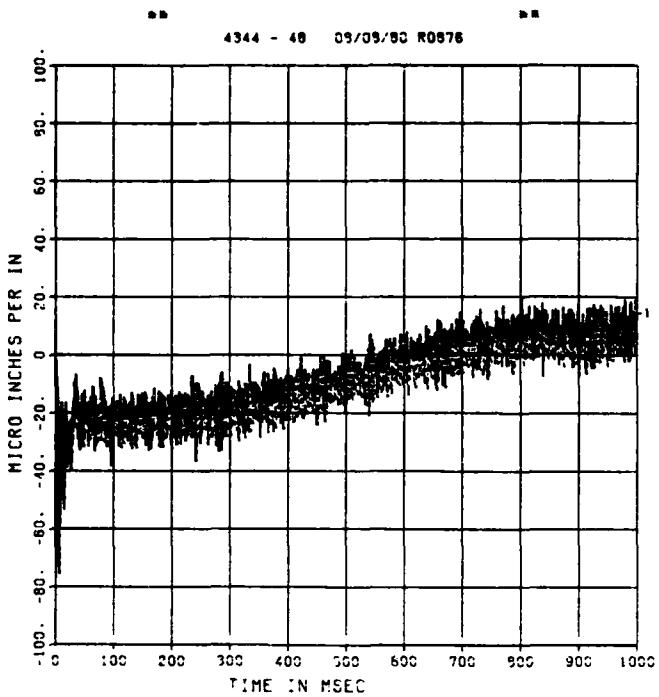
** PEAK VALUE IS 91 % UNDER CALIBRATION **

SLAB 3A BEAM UPGRADE
BS1 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

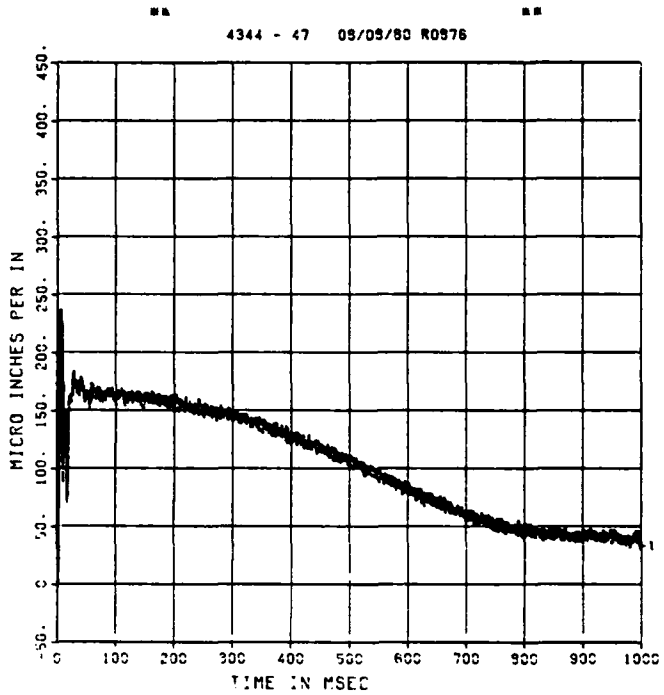


** PEAK VALUE IS 134 % OVER CALIBRATION **

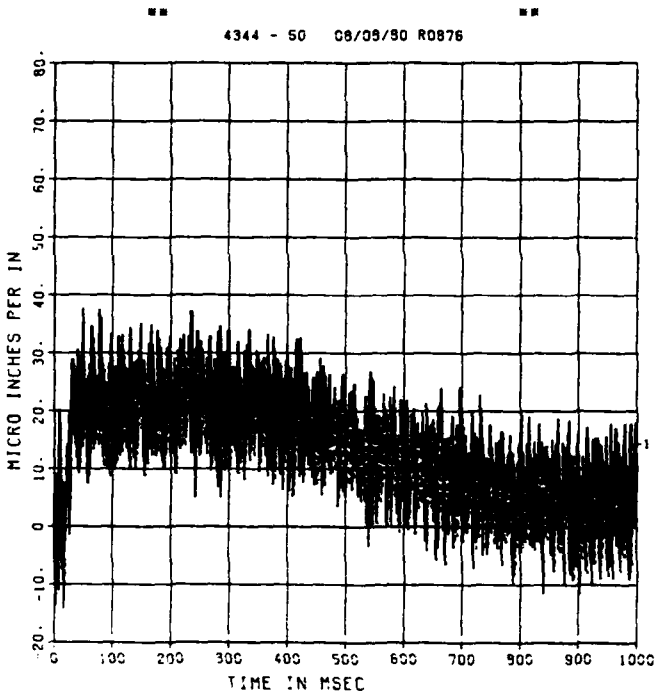
SLAB 3A BEAM UPGRADE
BS2 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



SLAB 3A BEAM UPGRADE
BS2 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

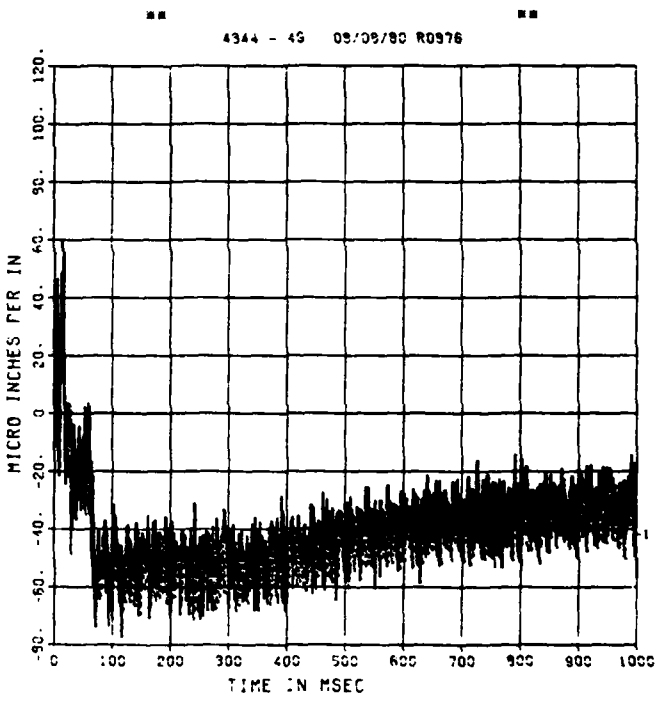


SLAB 3A BEAM UPGRADE
BS3 TOP
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ

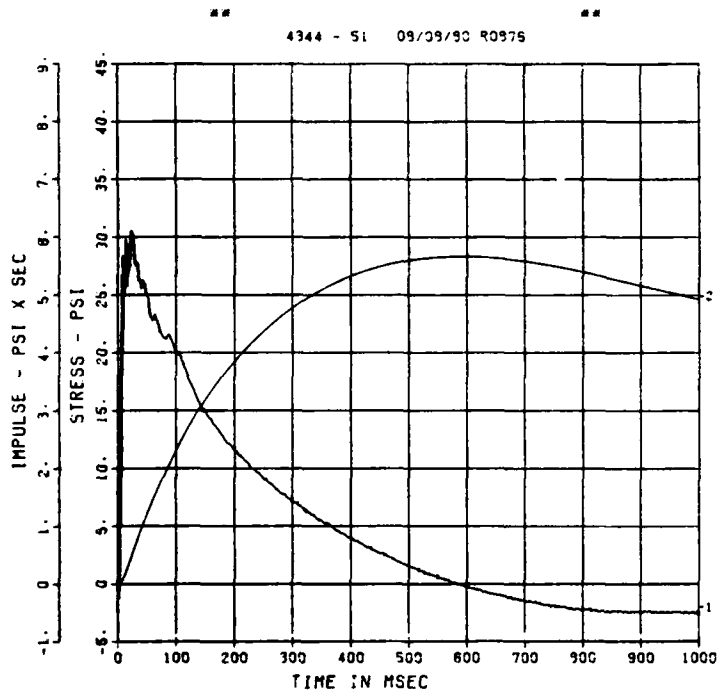


** PEAK VALUE IS 99 % UNDER CALIBRATION **

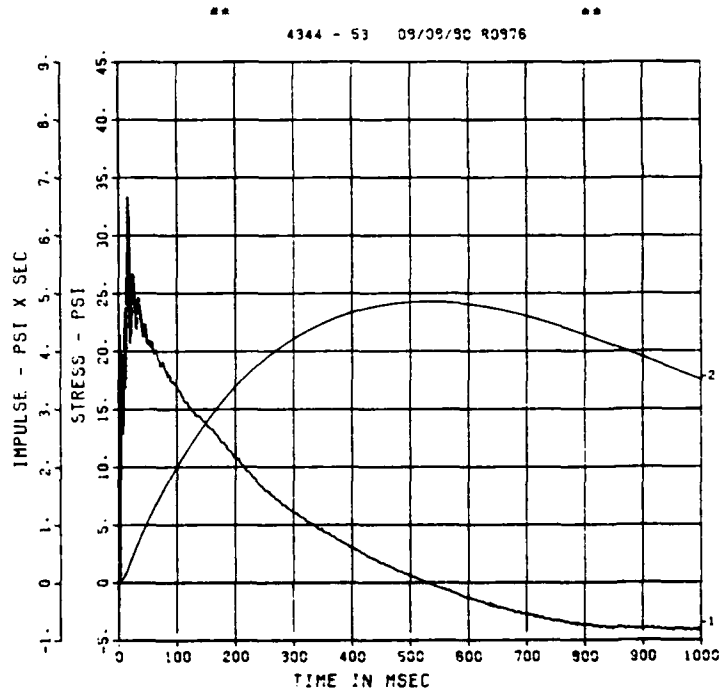
SLAB 3A BEAM UPGRADE
BS3 BOTTOM
25000. HZ CAL= 349.7
LP4 70% CUTOFF= 1125. HZ



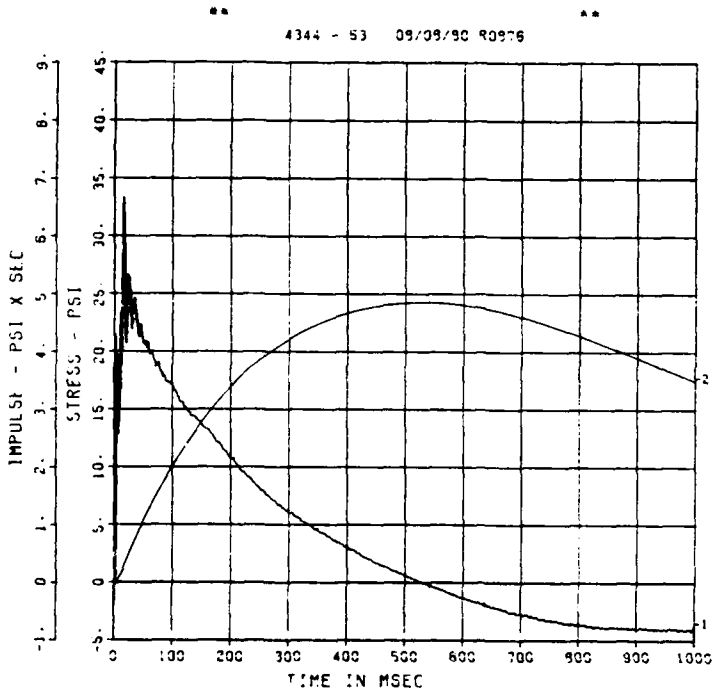
SLAB 3A BEAM UPGRADE
WP-1
25000. HZ CAL= 46.91
LP4 70% CUTOFF= 1125. HZ



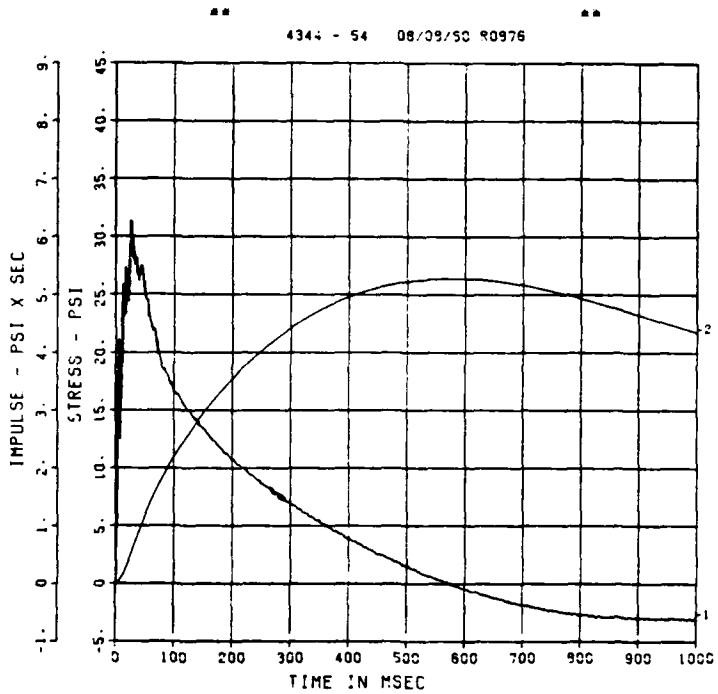
SLAB 3A BEAM UPGRADE
CP
25000. HZ CAL= 44.42
LP4 70% CUTOFF= 1125. HZ



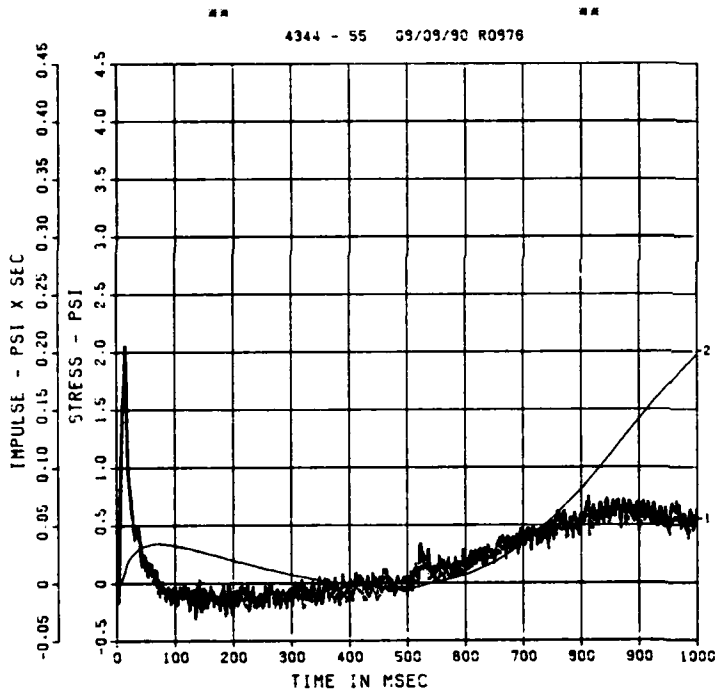
SLAB 3A BEAM UPGRADE
CP
25000. HZ CAL= 44.42
LP4 70% CUTOFF= 1125. HZ



SLAB 3A BEAM UPGRADE
EP-1
25000. HZ CAL= 40.31
LP4 70% CUTOFF= 1125. HZ

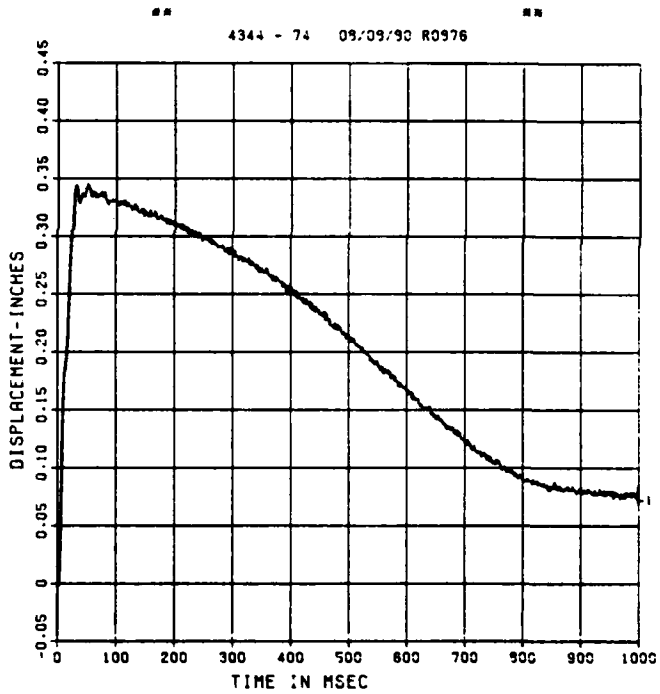


SLAB 3A BEAM UPGRADE
EP-2
25000. HZ CAL= 42.08
LP4 70% CUTOFF= 1125. HZ

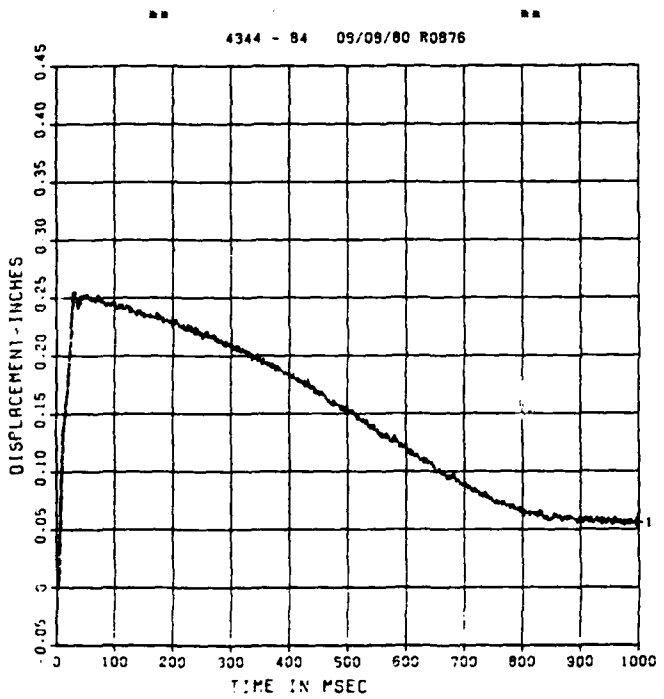


** PEAK VALUE IS 95 % UNDER CALIBRATION **

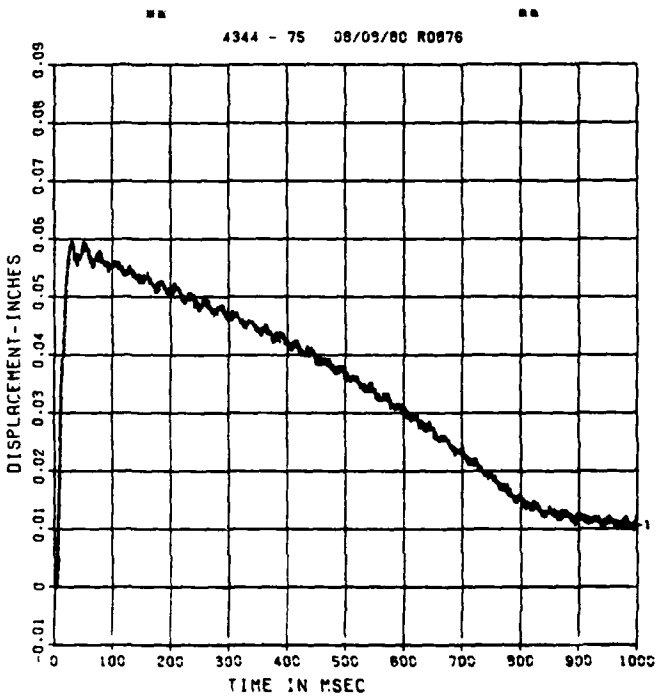
SLAB 3A BEAM UPGRADE
BD1
25000. HZ CAL= 0.755
LP4 70% CUTOFF= 1125. HZ



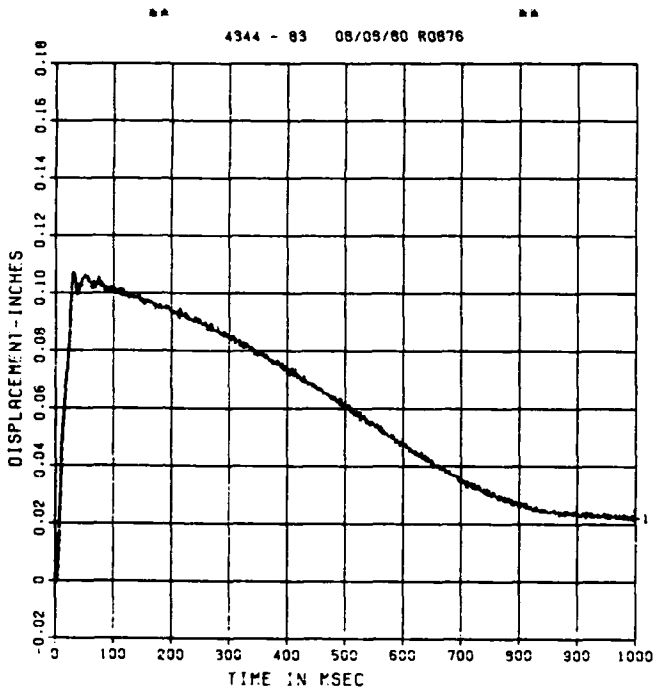
SLAB 3A BEAM UPGRADE
BD2
25000. HZ CAL= 0.735
LP4 70% CUTOFF= 1125. HZ



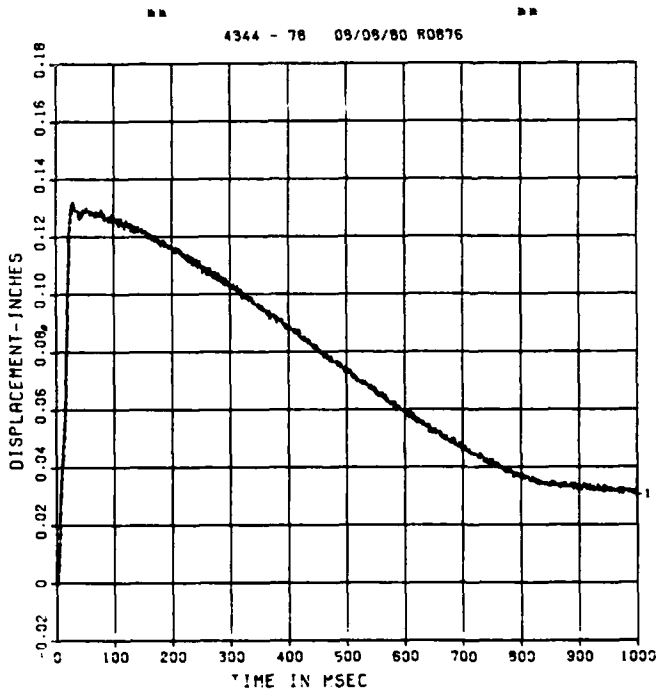
SLAB 3A BEAM UPGRADE
NBD
25000. HZ CAL= 0.318
LP4 70% CUTOFF= 1125. HZ



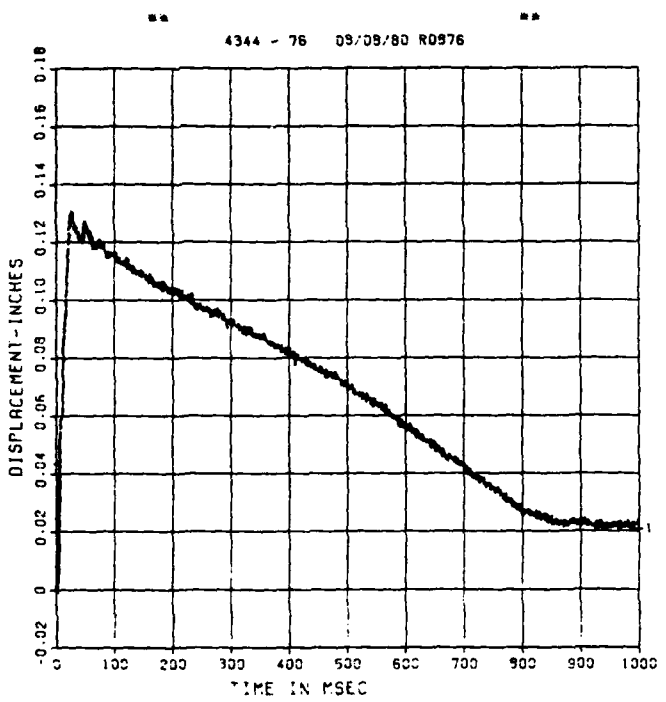
SLAB 3A BEAM UPGRADE
SBD
25000. HZ CAL= 0.349
LP4 70% CUTOFF= 1125. HZ



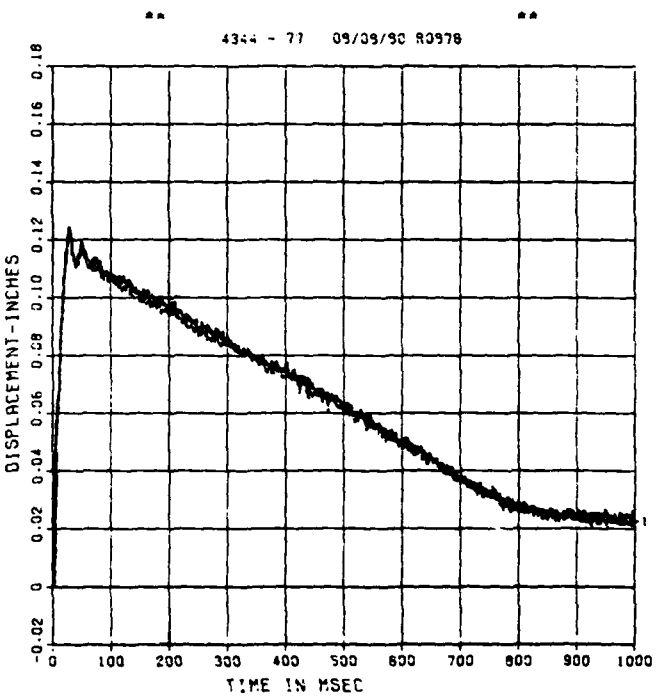
SLAB 3A BEAM UPGRADE
WBD
25000. HZ CAL= 0.314
LP4 70% CUTOFF= 1125. HZ



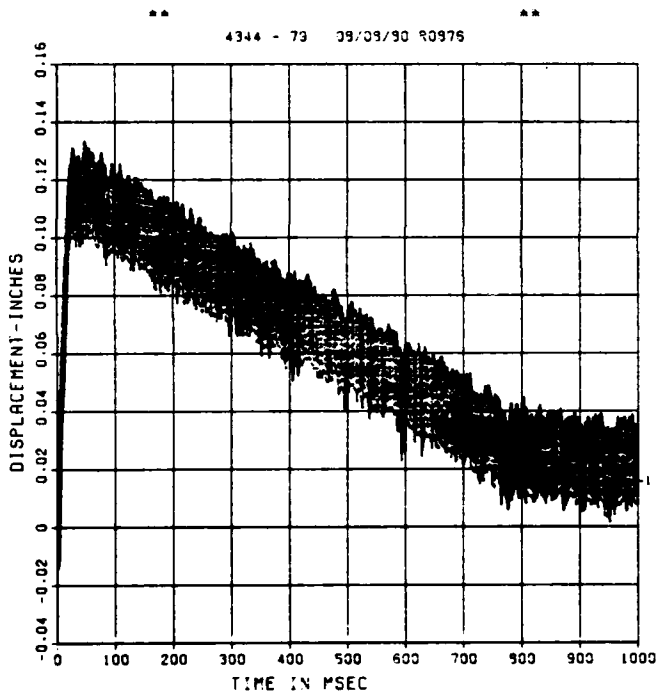
SLAB 3A BEAM UPGRADE
NUD
25000. HZ CAL= 0.695
LP4 70% CUTOFF= 1125. HZ



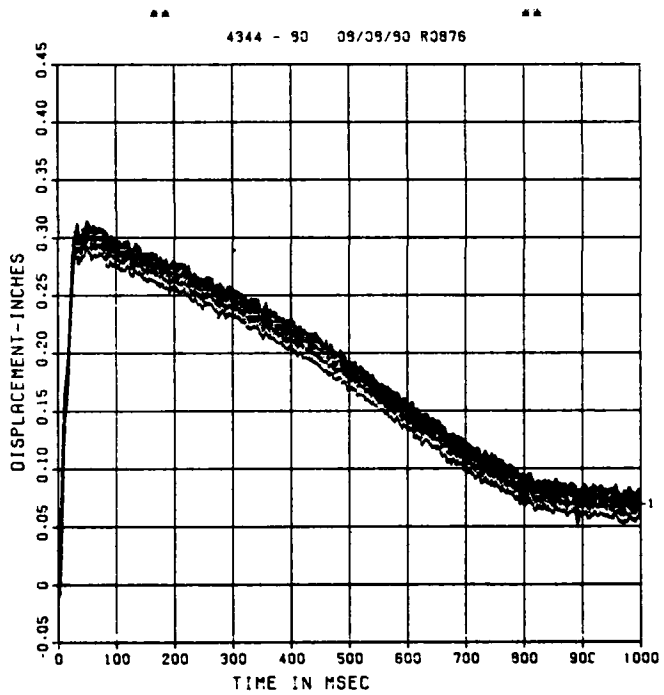
SLAB 3A BEAM UPGRADE
SUD
25000. HZ CAL= 0.650
LP4 70% CUTOFF= 1125. HZ



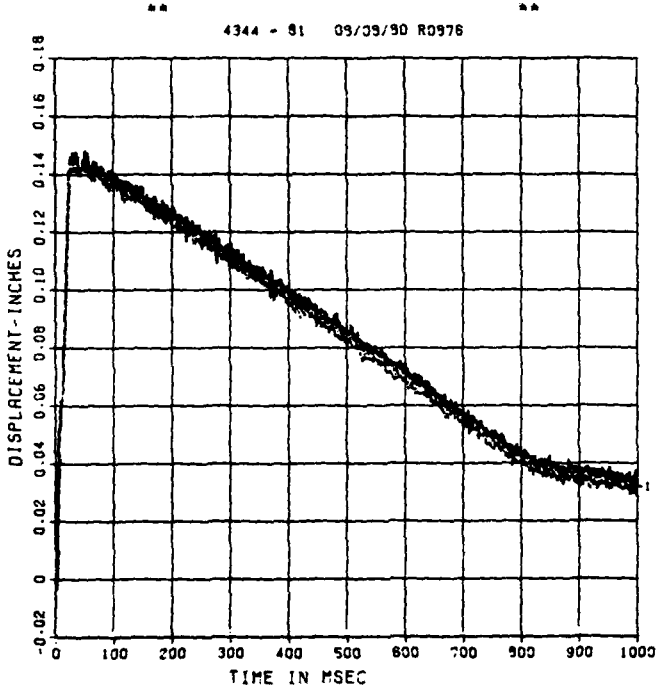
SLAB 3A BEAM UPGRADE
ED1
25000. HZ CAL= 0.710
LP4 70% CUTOFF= 1125. HZ



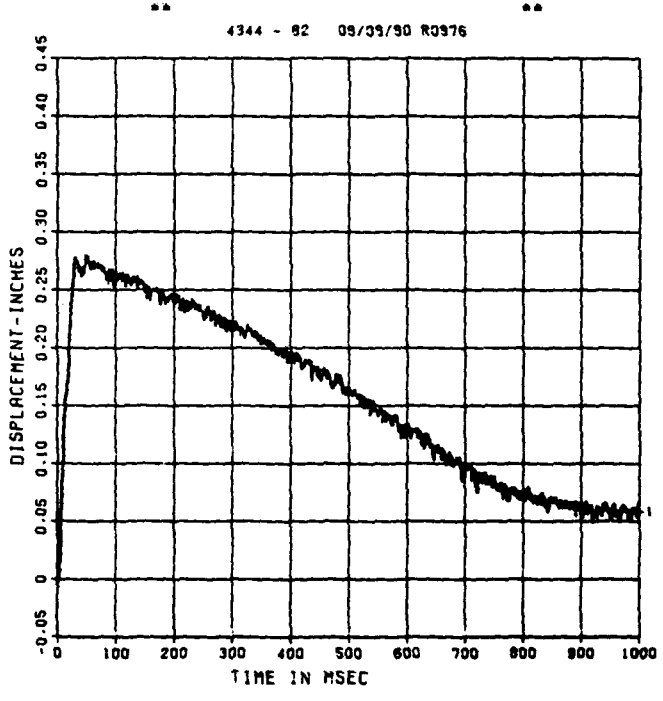
SLAB 3A BEAM UPGRADE
WD1
25000. HZ CAL= 0.722
LP4 70% CUTOFF= 1125. HZ



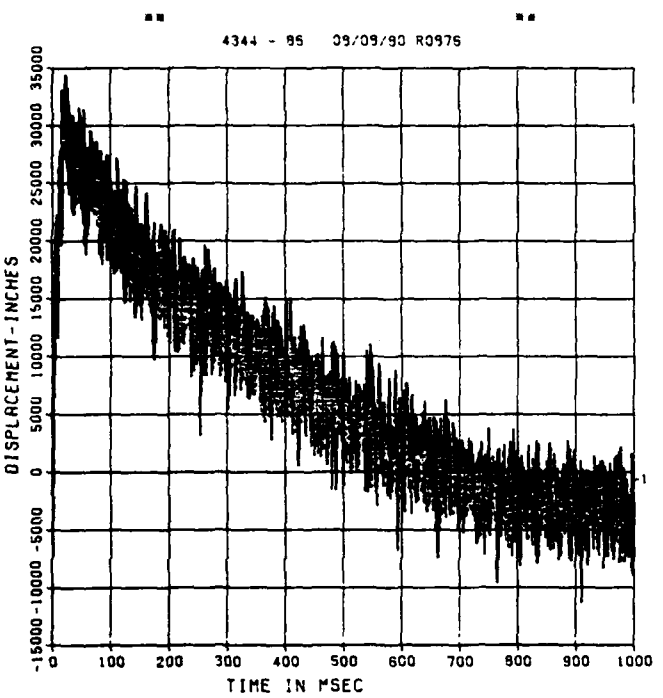
SLAB 3A BEAM UPGRADE
.WD2
25000. HZ CAL= 0.634
LP4 70% CUTOFF= 1125. HZ



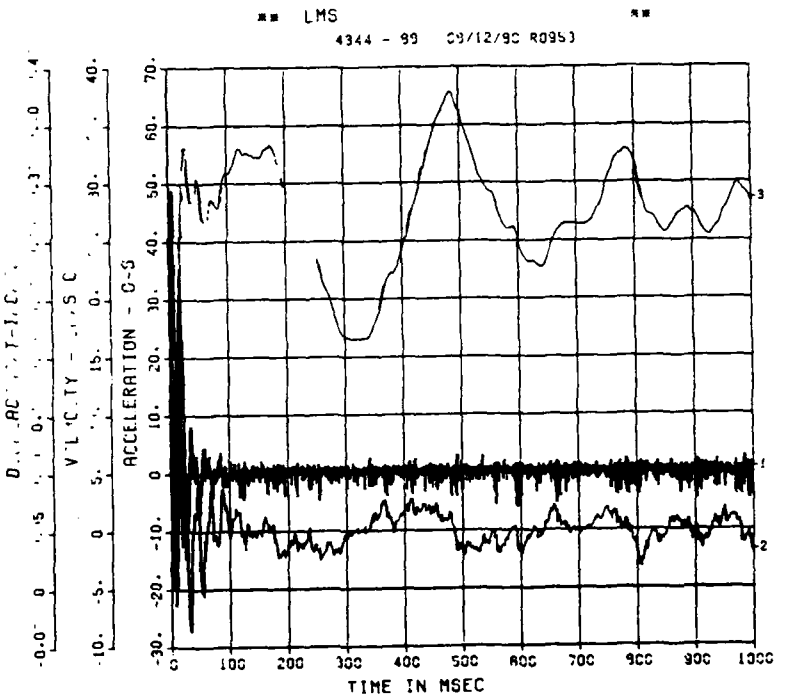
SLAB 3A BEAM UPGRADE
WD3
25000. HZ CAL= 0.650
LP4 70% CUTOFF= 1125. HZ



SLAB 3A BEAM UPGRADE
LOAD 2
25000. HZ CAL= 77180.
LP4 70% CUTOFF= 1125. HZ

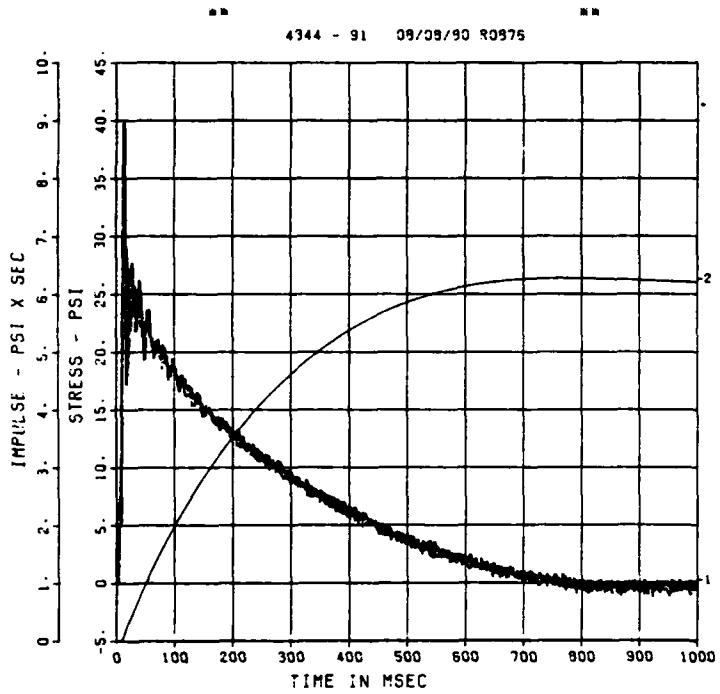


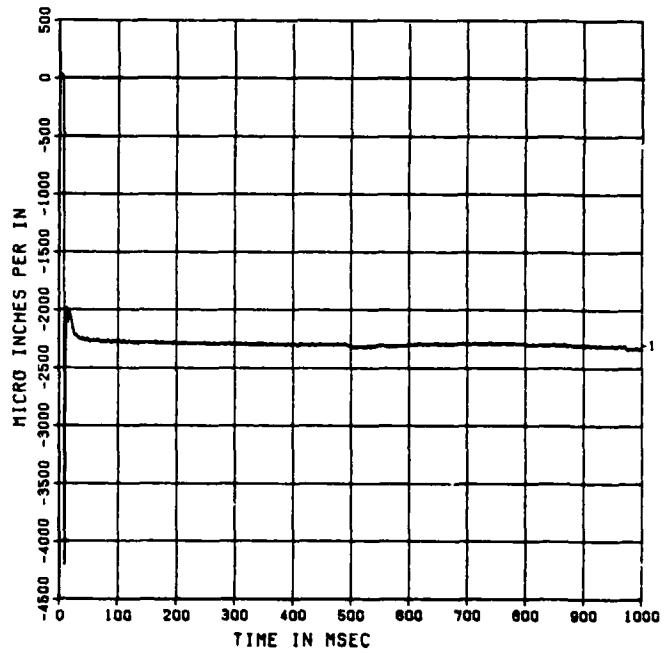
SLAB 3A BEAM UPGRADE
A2
25000. HZ CAL= 91.60
LP4 70% CUTOFF= 1125. HZ



SLAB 3A BEAM UPGRADE
NORWOOD
25000. HZ CAL= 50.44
LP4 70% CUTOFF= 1125. HZ

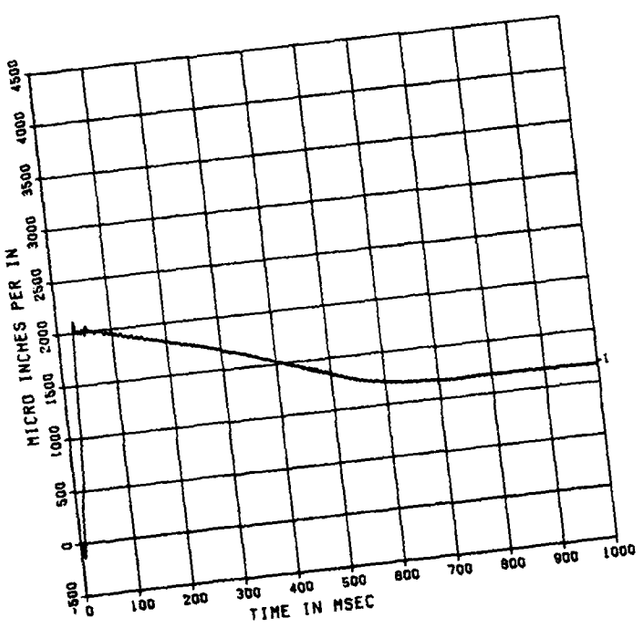
SLAB 3B BEAM UPGRADE
SSI TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 2 R0815





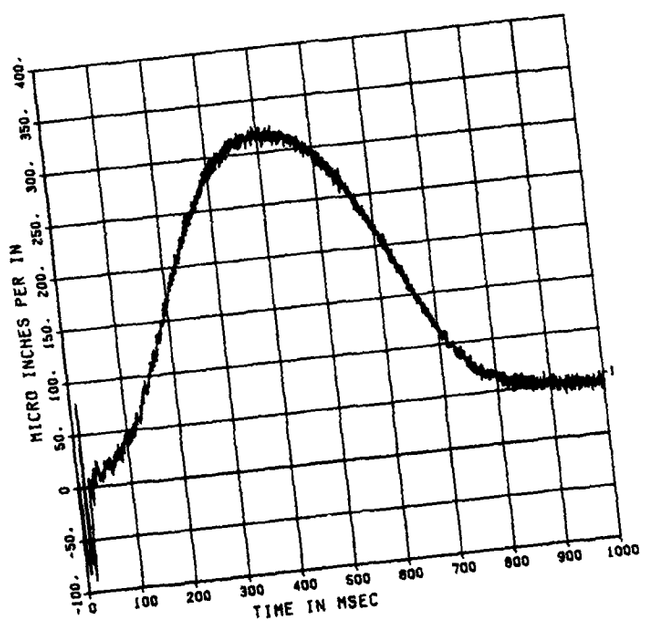
** PEAK VALUE IS 141 % OVER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS1 BOTTOM CAL 1743.
25000. HZ
F4
08/29/79 12285 1 R0615



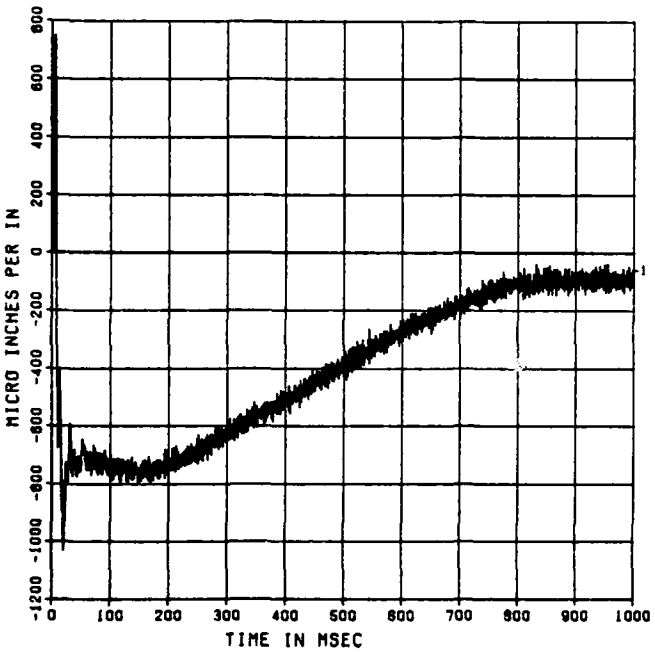
** PEAK VALUE IS 21 % OVER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS2 TOP CAL 1743.
25000. HZ
F4
08/29/79 12285 6 R0615

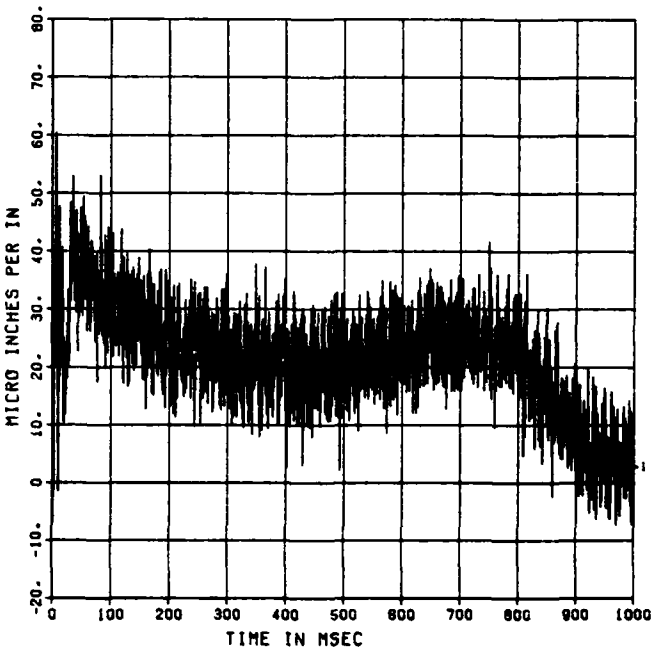


** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS2 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 5 R0615

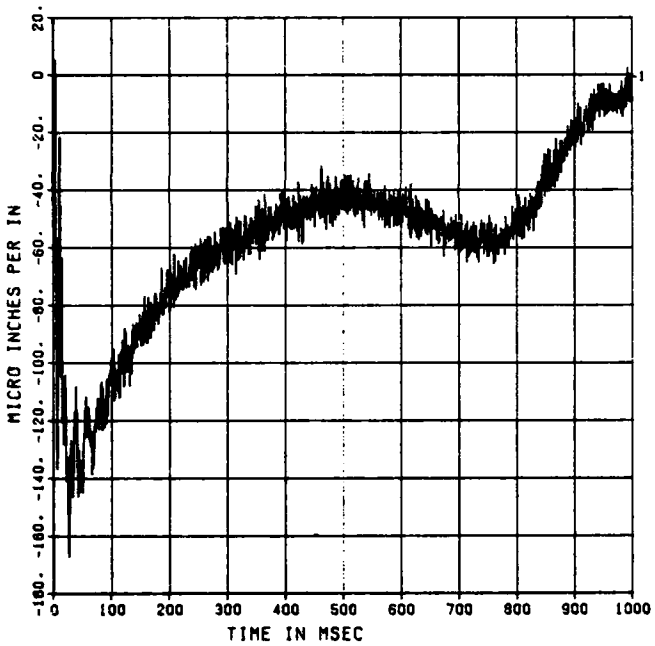


SLAB 3B BEAM UPGRADE
SS3 TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 10 R0615



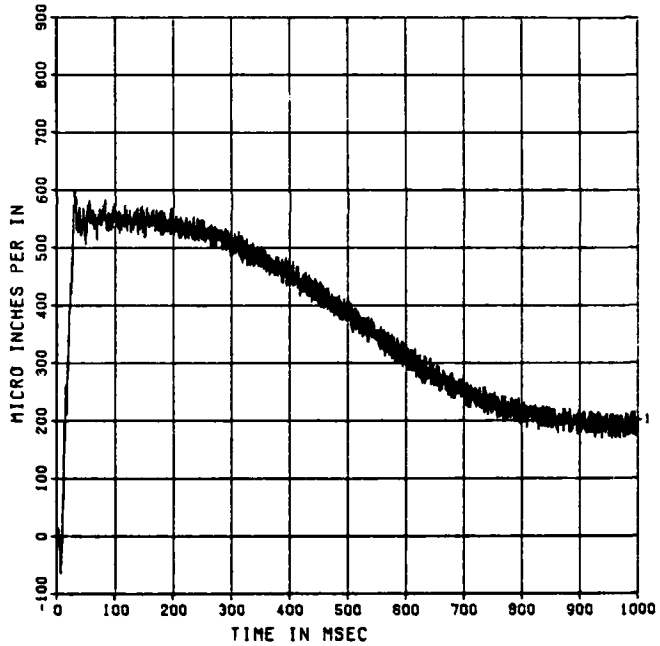
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS3 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 9 R0615

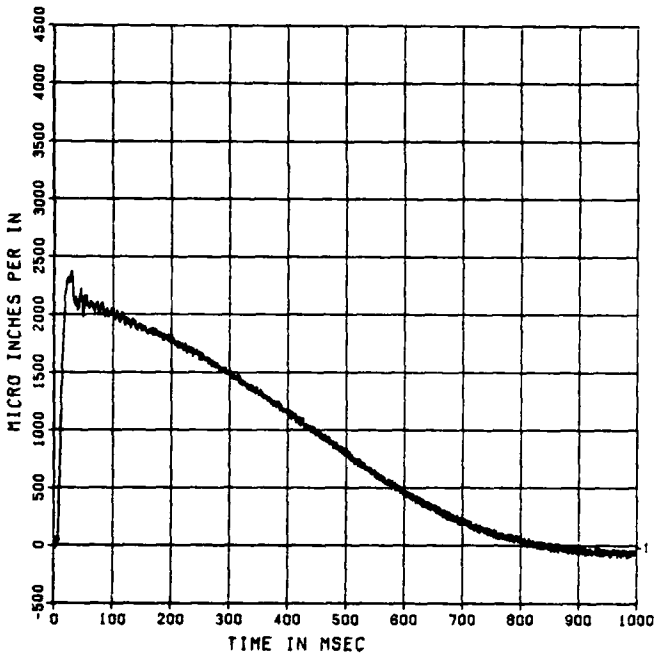


** PEAK VALUE IS -90 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS4 TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 12 R0615

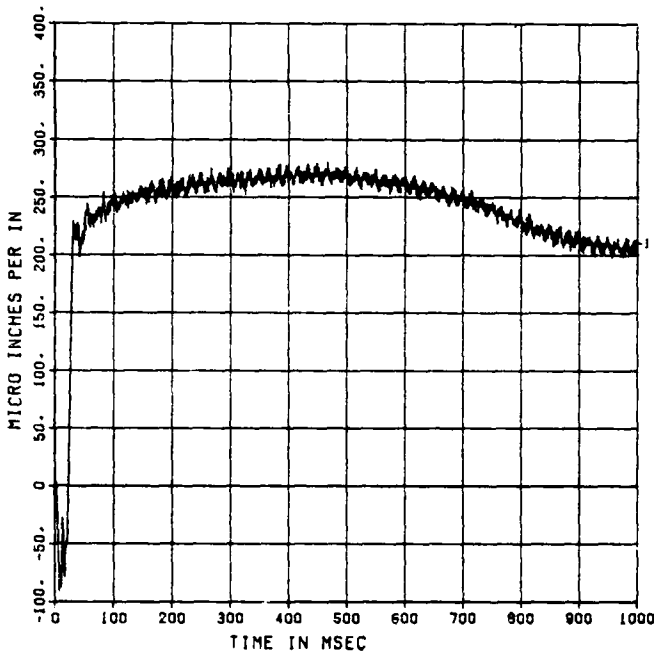


SLAB 3B BEAM UPGRADE
SS4 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12285 11 R0815



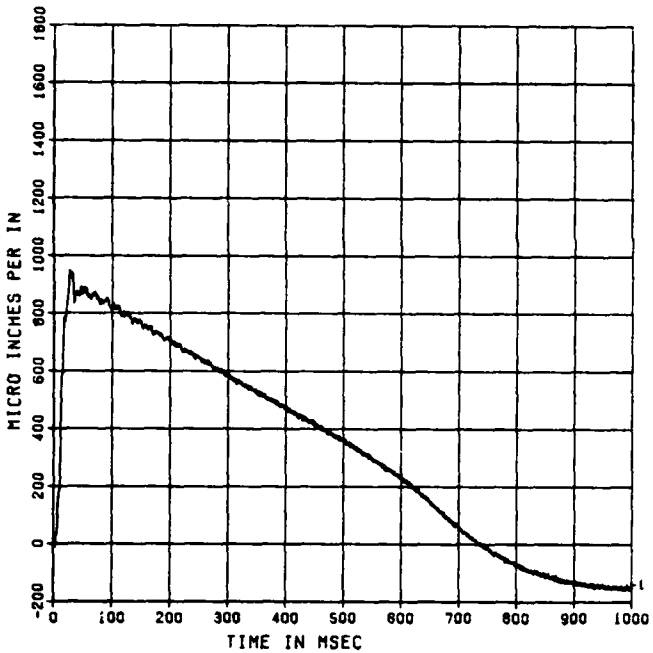
** PEAK VALUE IS 36 % OVER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS5 TOP
25000. HZ CAL 1743.
F4
06/29/79 12285 14 R0815

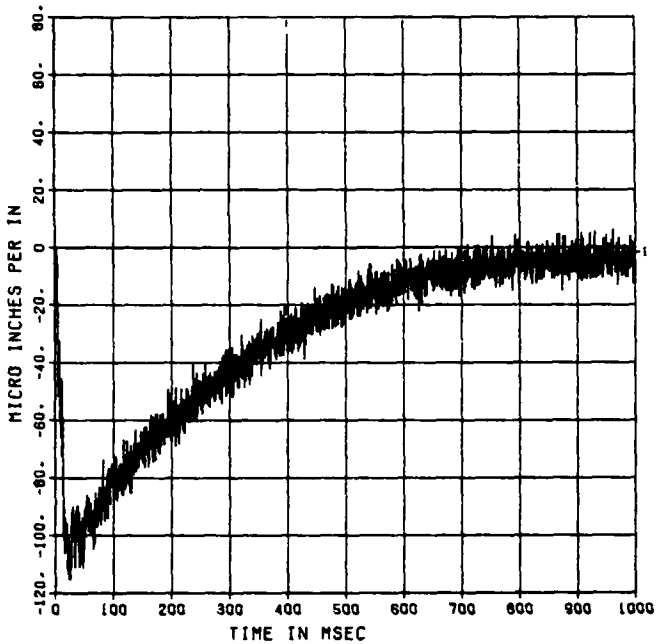


** PEAK VALUE IS -94 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS5 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 13 R0615

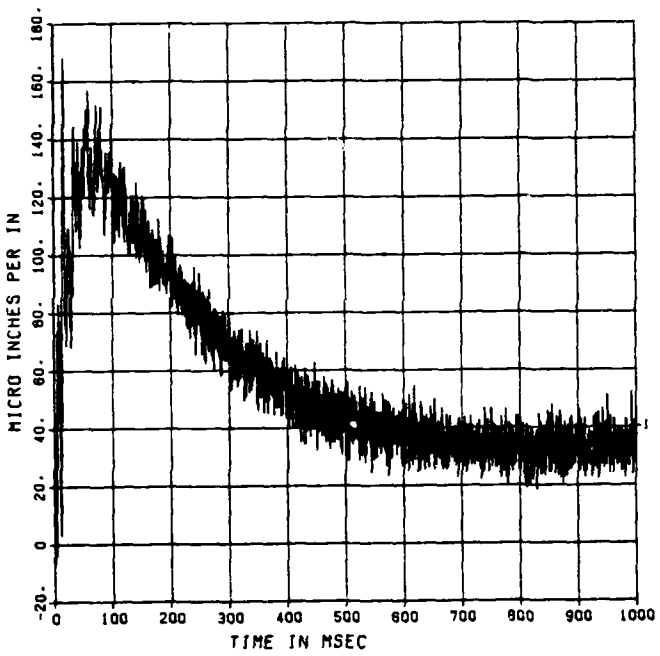


SLAB 3B BEAM UPGRADE
SS6 TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 16 R0615



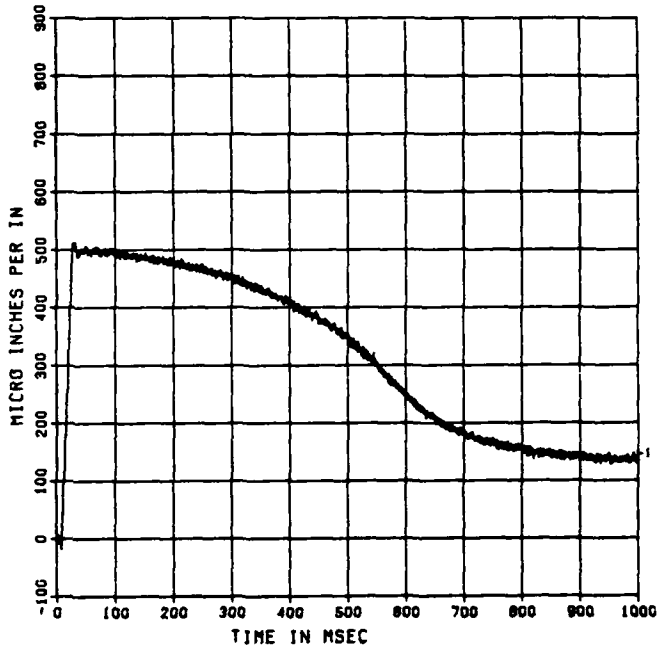
** PEAK VALUE IS -93 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
SS6 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 15 R0615

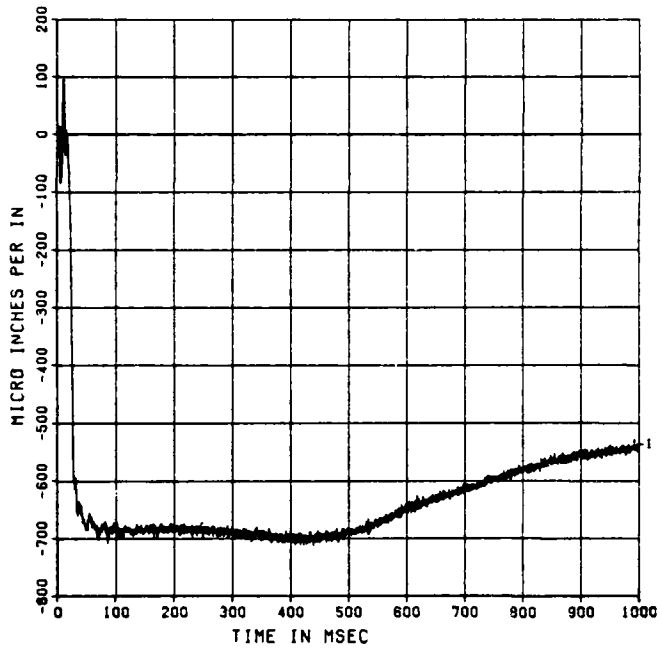


** PEAK VALUE IS -90 % UNDER CALIBRATION **

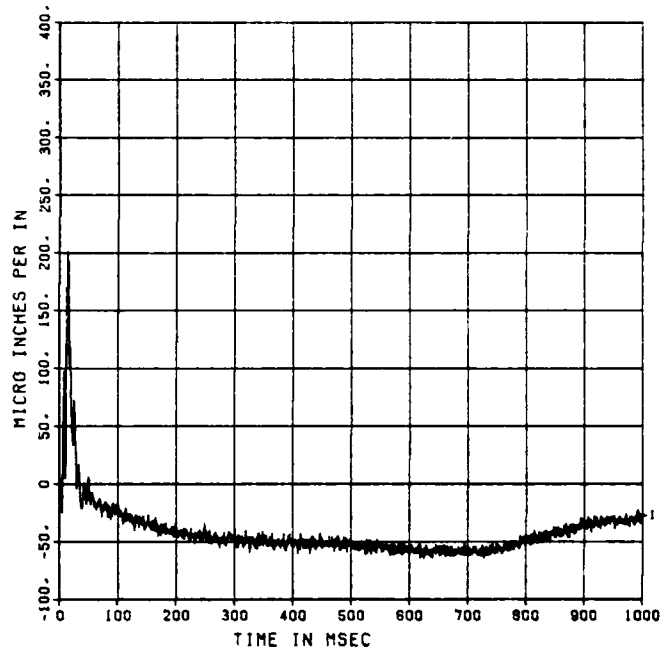
SLAB 3B BEAM UPGRADE
SL1 TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 4 R0615



SLAB 3B BEAM UPGRADE
SL1 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 3 R0615

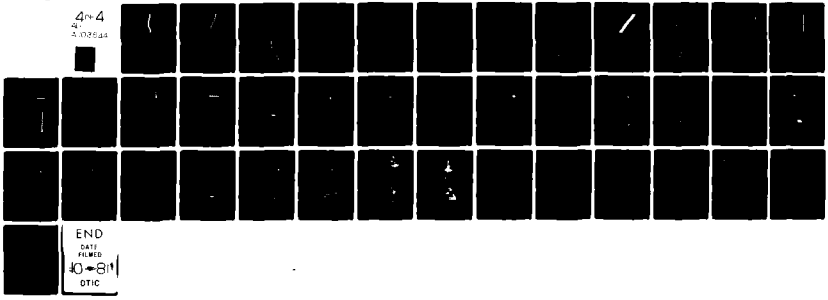


SLAB 3B BEAM UPGRADE
SL2 TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 8 R0615

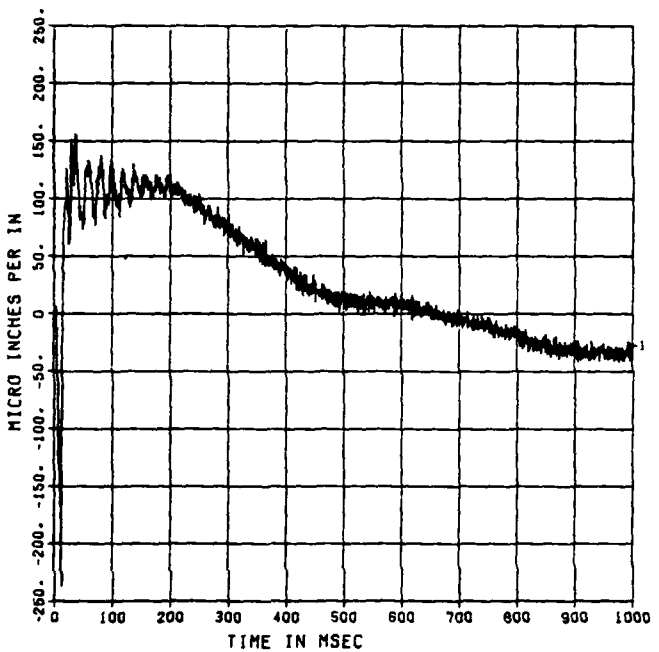


** PEAK VALUE IS -99 % UNDER CALIBRATION **

AD-A103 544 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/13
TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR--ETC(U)
JUL 81 H K MCVAY DCPA01-78-C-0267
UNCLASSIFIED WES/TR/SL-81-4 NL

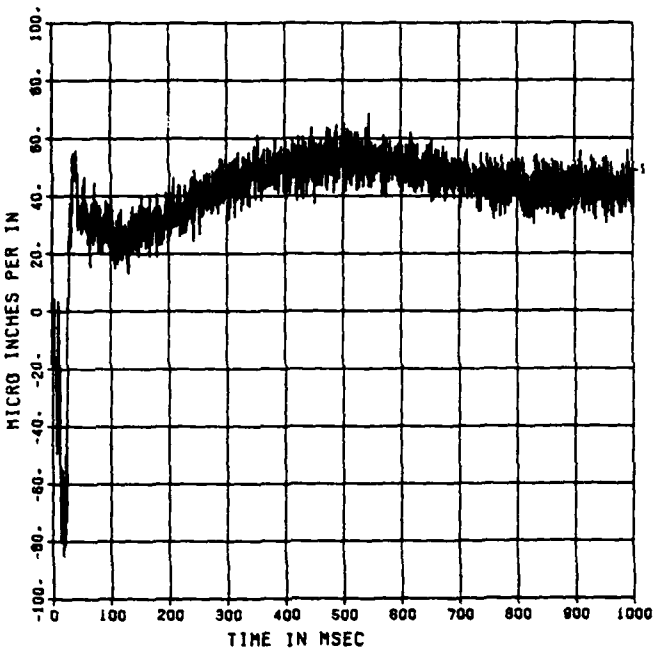


SLAB 3B BEAM UPGRADE
SL2 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 7 R0615



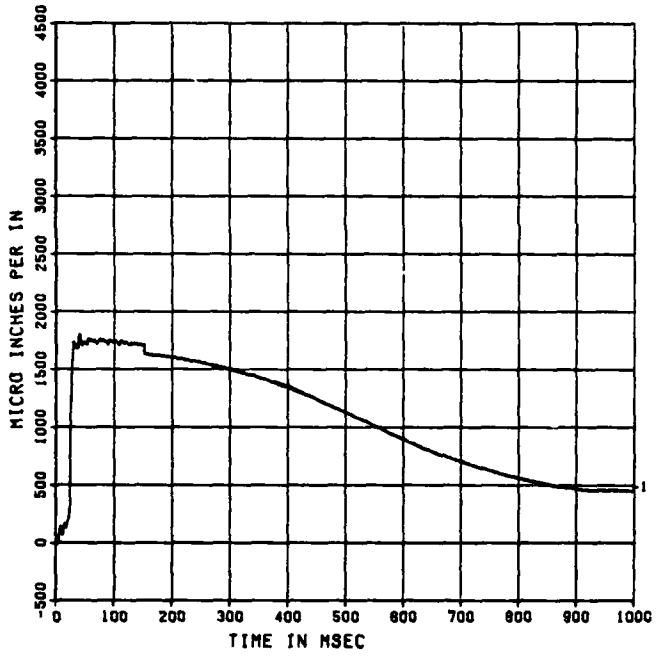
** PEAK VALUE IS -86 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
BS2 TOP
25000. HZ CAL 1743.
F4
06/29/79 12265 19 R0615



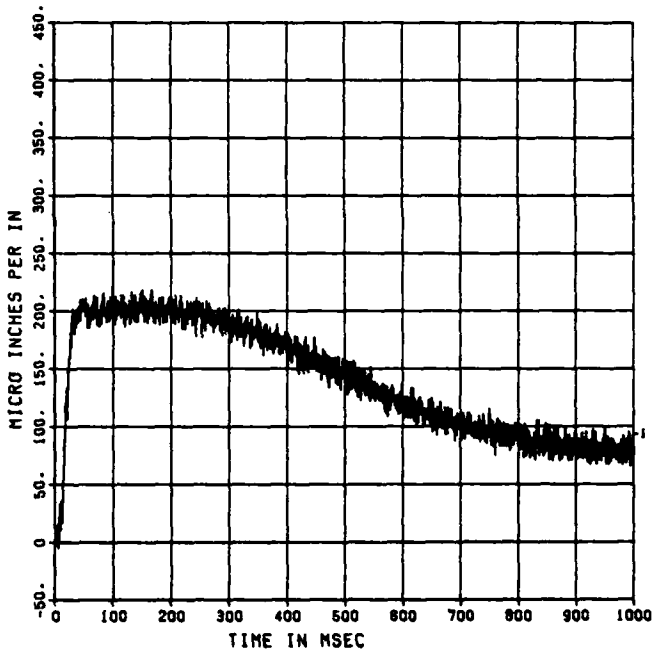
** PEAK VALUE IS -95 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
BS2 BOTTOM
25000. HZ CAL 1743.
F4
08/29/79 12265 18 R0615



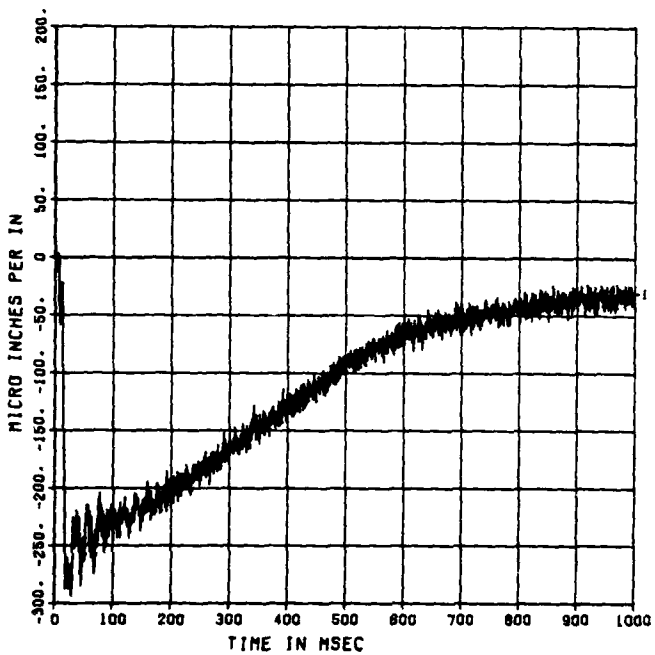
** PEAK VALUE IS 4 % OVER CALIBRATION **

SLAB 3B BEAM UPGRADE
BS3 TOP
25000. HZ CAL 1743.
F4
08/29/79 12265 21 R0615



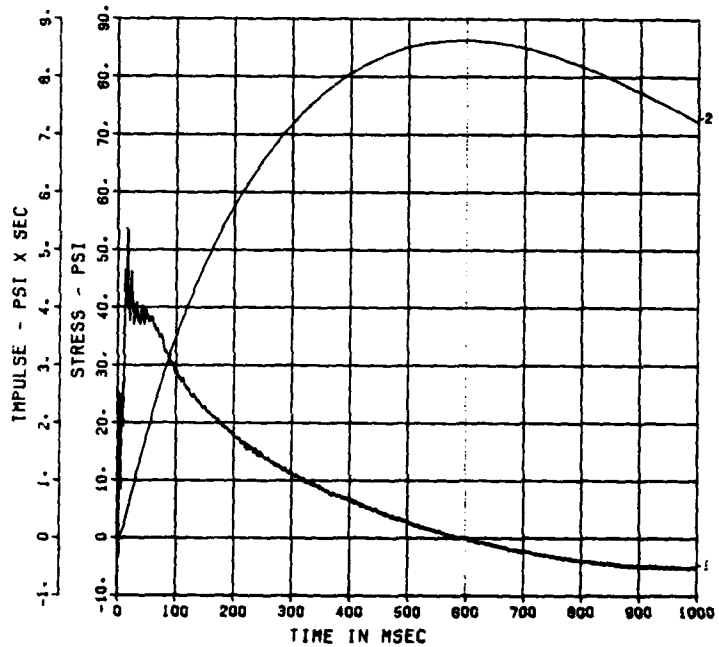
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
BS3 BOTTOM
25000. HZ CAL 1743.
F4
06/29/79 12265 20 R0615

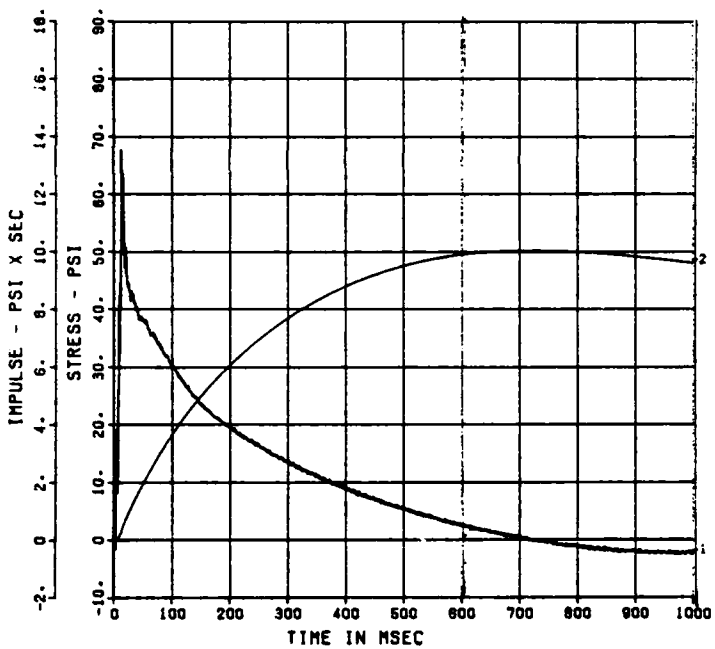


** PEAK VALUE IS -93 % UNDER CALIBRATION **

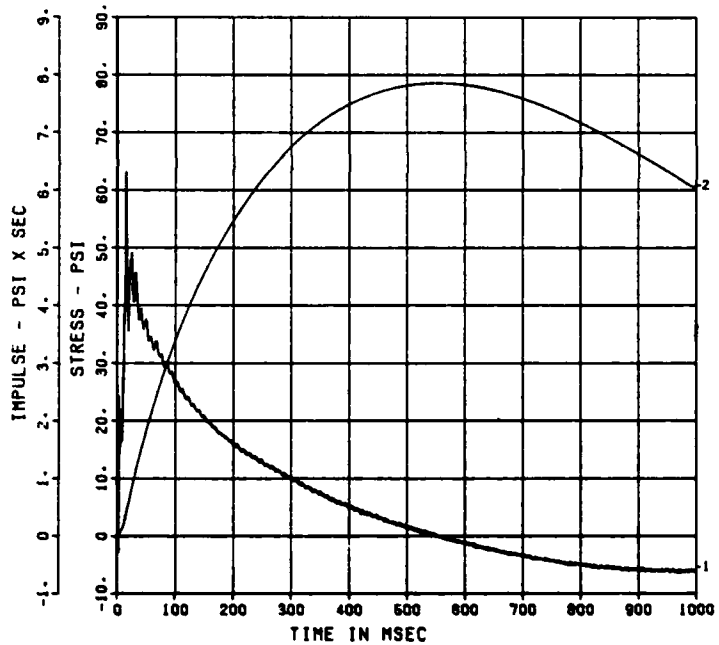
SLAB 3B BEAM UPGRADE
EP-I
25000. HZ CAL 79.9
F4
06/29/79 12265 25 R0615



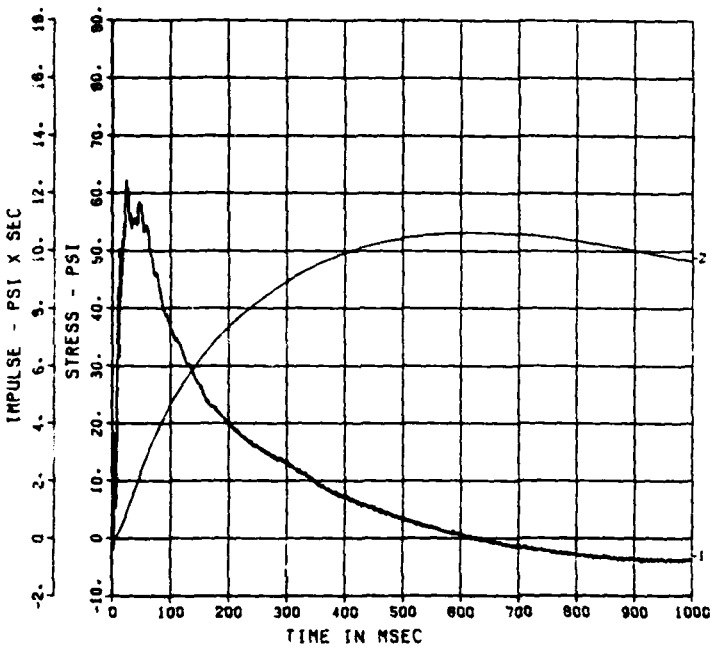
SLAB 3B BEAM UPGRADE
EP-2
25000. HZ CAL 98.
F4
06/29/79 12265 26 R0615



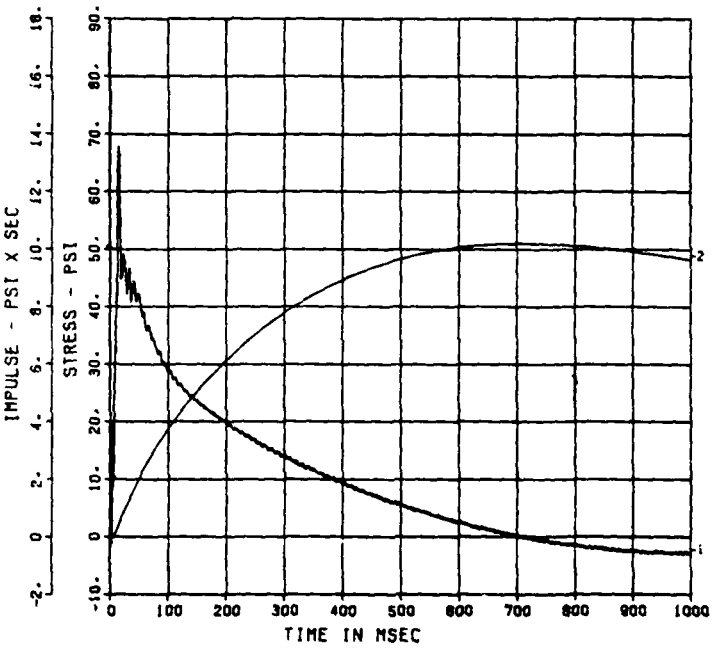
SLAB 3B BEAM UPGRADE
CP
25000. HZ CAL 88.1
F4
06/29/79 12265 24 R0615



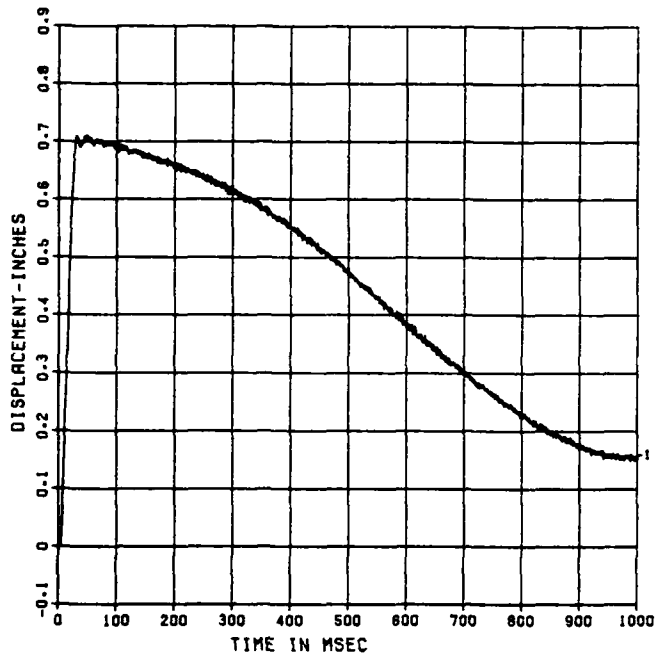
SLAB 3B BEAM UPGRADE
WP-1
25000. HZ CAL 93.
F4
06/29/79 12285 22 R0815



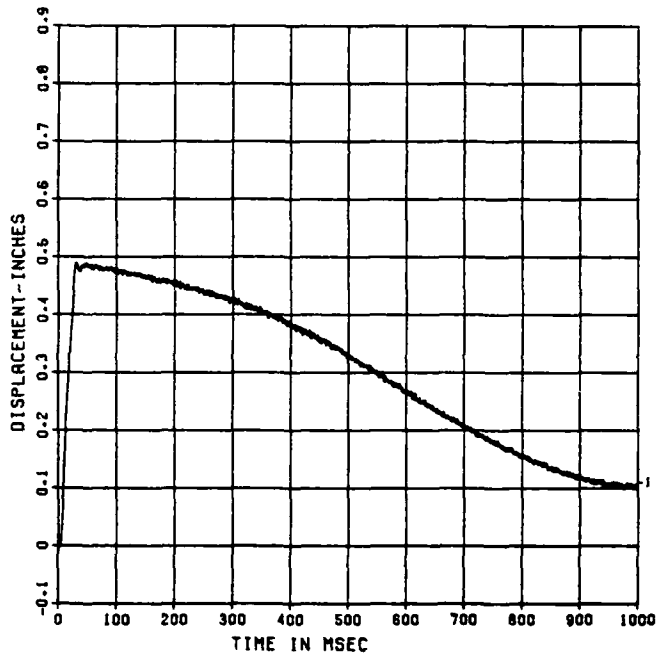
SLAB 3B BEAM UPGRADE
WP-2
25000. HZ CAL 91.7
F4
06/29/79 12285 23 R0815



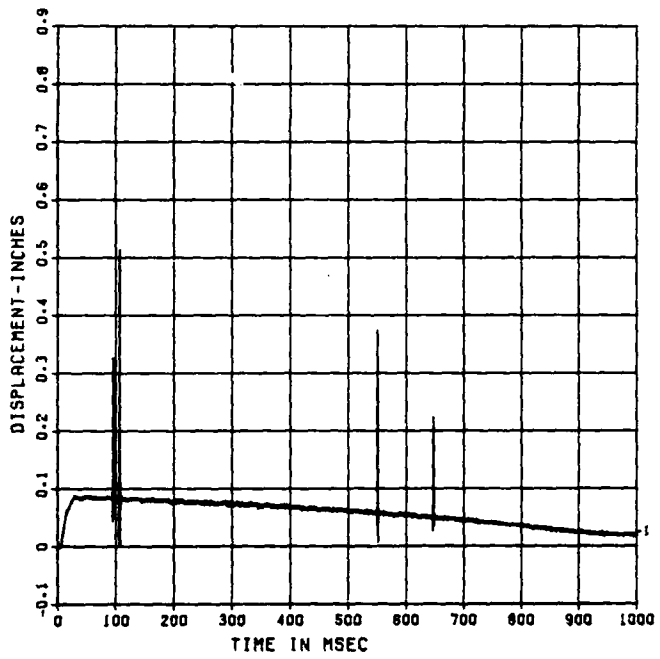
SLAB 3B BEAM UPGRADE
BD1
25000. HZ CAL 1.92
F4
07/11/79 12265 57 R0199



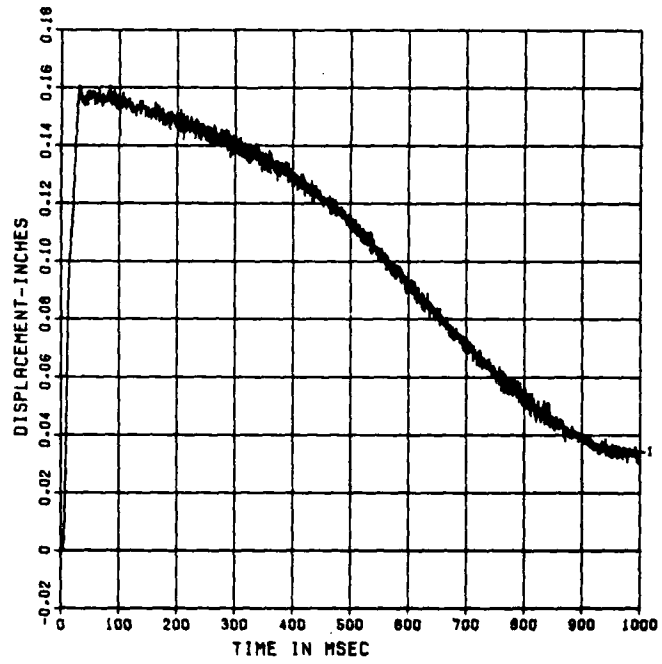
SLAB 3B BEAM UPGRADE
BD2
25000. HZ CAL 1.87
F4
07/11/79 12265 67 R0199



SLAB 3B BEAM UPGRADE
NBD
25000. HZ CAL 1.27
F4
07/11/79 12265 58 R0199

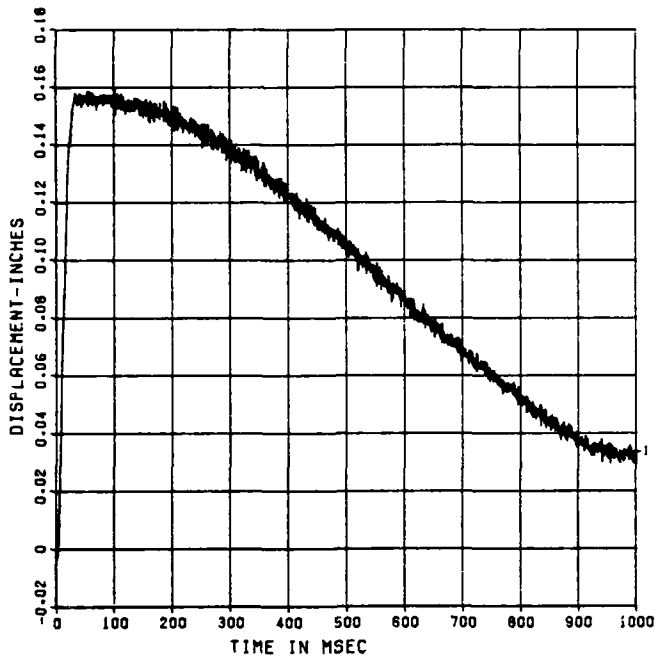


SLAB 3B BEAM UPGRADE
SBD
25000. HZ CAL 1.16
F4
07/11/79 12265 66 R0199



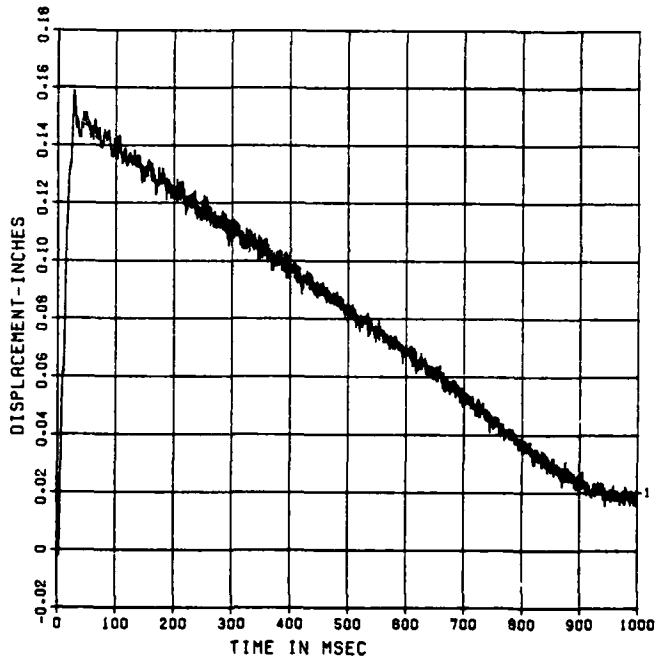
** PEAK VALUE IS -66 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
WBD
25000. HZ CAL 1.25
F4
07/11/79 12265 61 R0199



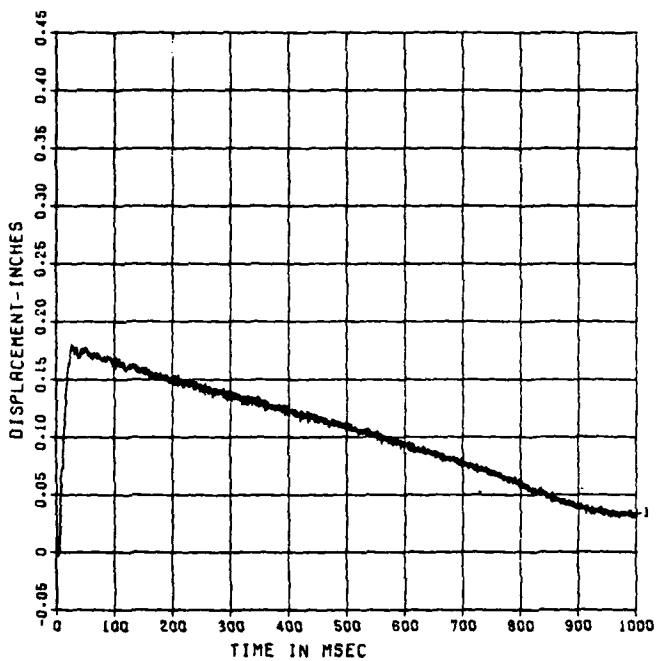
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
SUD
25000. HZ CAL 1.29
F4
07/11/79 12265 60 R0199



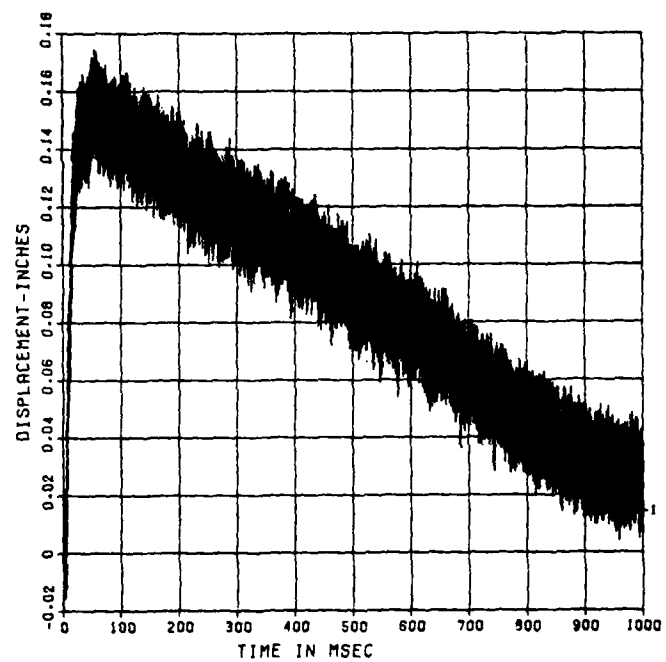
** PEAK VALUE IS -99 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
NUD
25000. HZ CAL 1.38
F4
07/11/79 12265 59 R0199



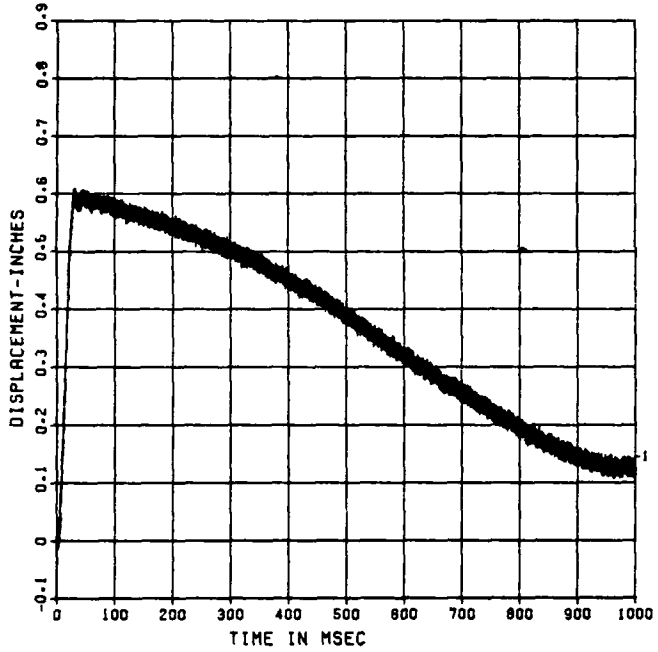
** PEAK VALUE IS -97 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
ED1
25000. HZ CAL 2.03
F4
07/11/79 12265 62 R0199

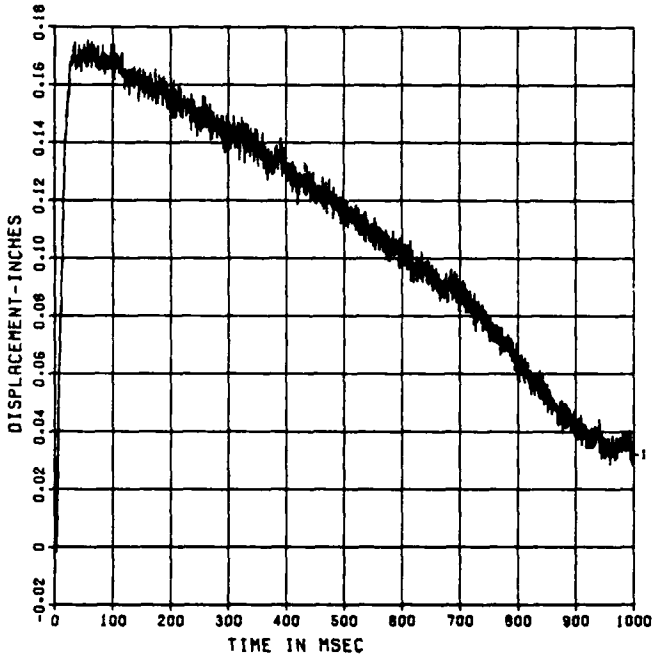


** PEAK VALUE IS -91 % UNDER CALIBRATION **

SLAB 3B BEAM UPGRADE
WD1
25000. HZ CAL 2.06
F4
07/11/79 12265 63 R0199

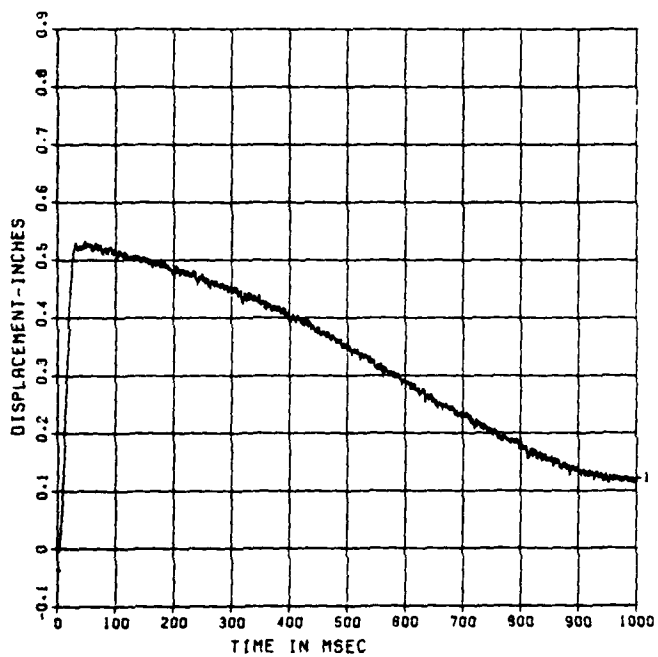


SLAB 3B BEAM UPGRADE
WD2
25000. HZ CAL 1.81
F4
07/11/79 12265 64 R0199

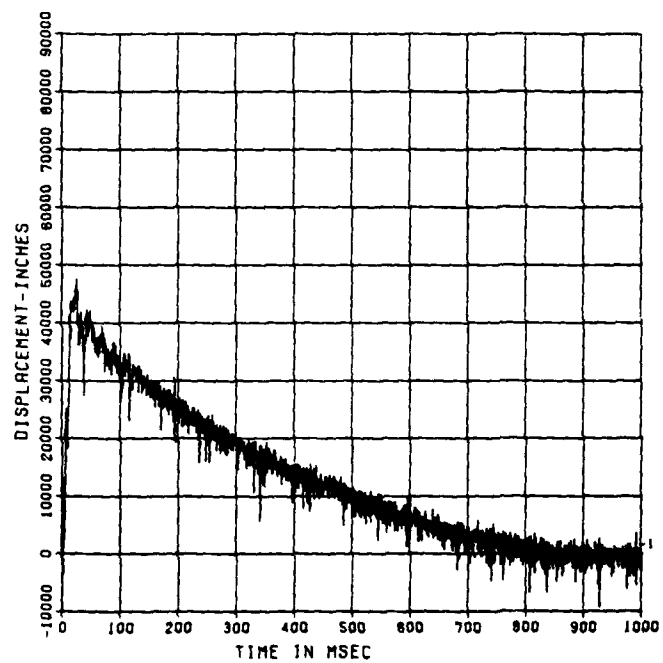


** PEAK VALUE IS -90 % UNDER CALIBRATION **

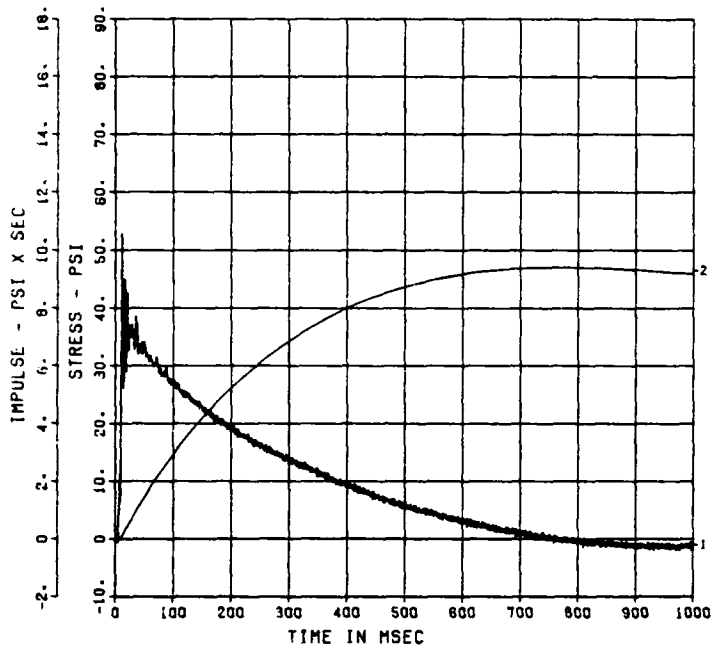
SLAB 3B BEAM UPGRADE
WD3
25000. HZ CAL 1.86
F4
07/11/79 12285 65 R0199



SLAB 3B BEAM UPGRADE
LOAD 2
25000. HZ CAL 96433.
F4
07/11/79 12265 68 R0199

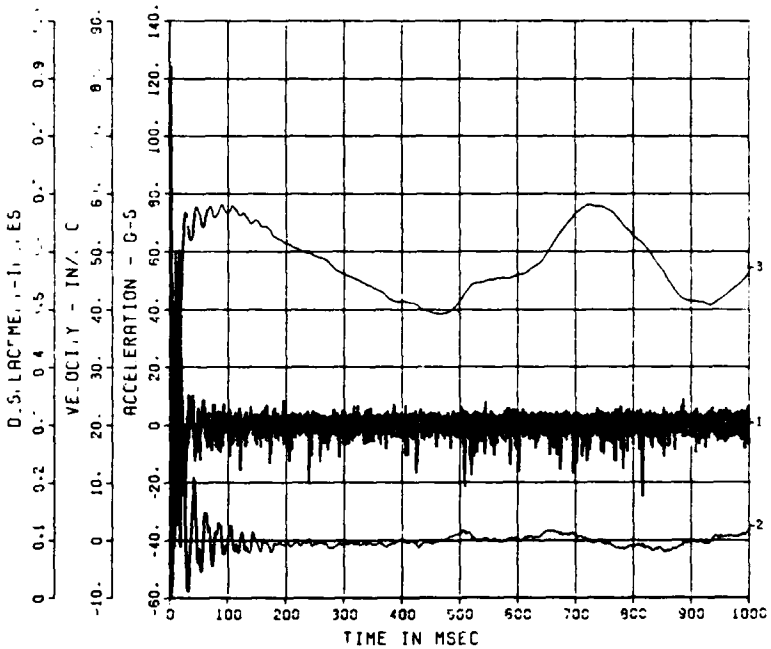


SLAB 3B BEAM UPGRADE
NORWOOD
25000. HZ CAL 80.6
F4
07/11/79 12265 73 R0199



SLAB 3B BEAM UPGRADE
A1
25000. HZ CAL= 95.21

** LMS **
12265- 69 08/11/80 R0999

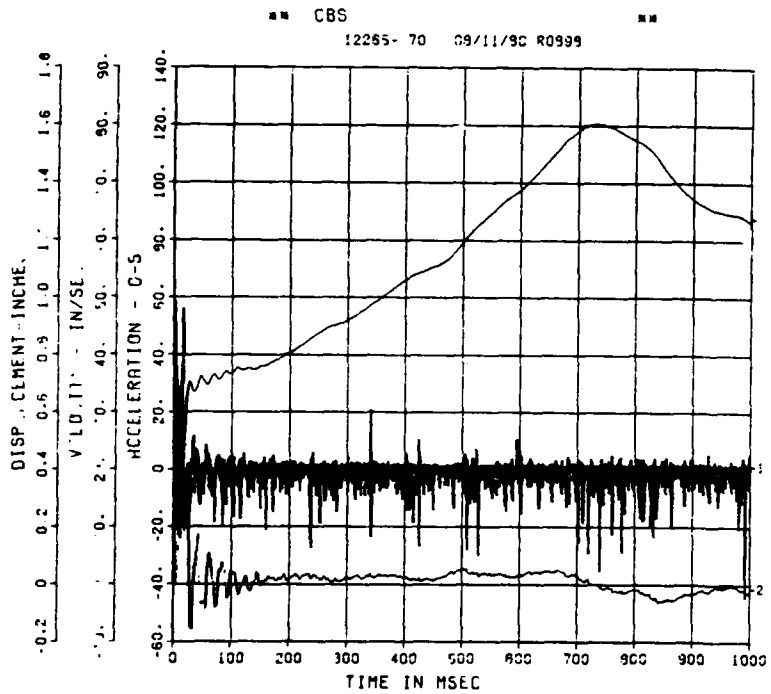


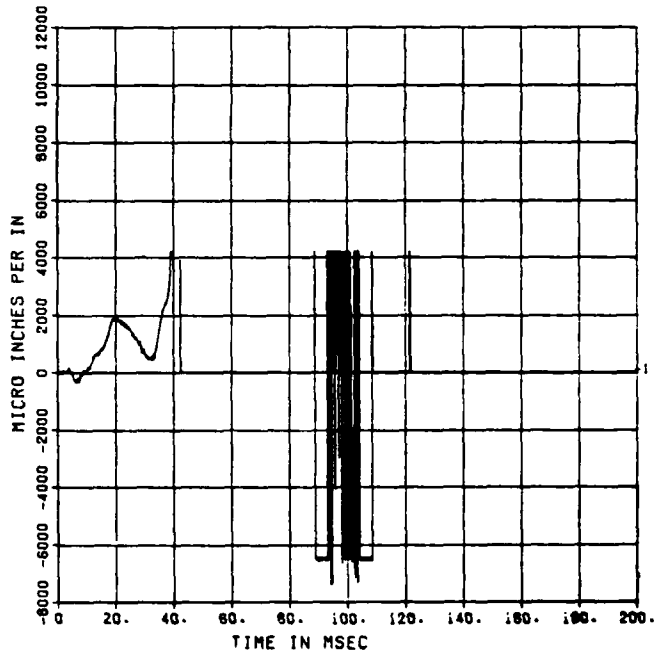
** PEAK VALUE IS 30 % OVER CALIBRATION **

SLAB 3B BEAM UPGRADE
A2
25000. HZ CAL= 91.60

SLAB 3C BEAM UPGRADE
SS1 BOTTOM
25000. HZ CAL 2899.

07/03/79 12265 29 R0726

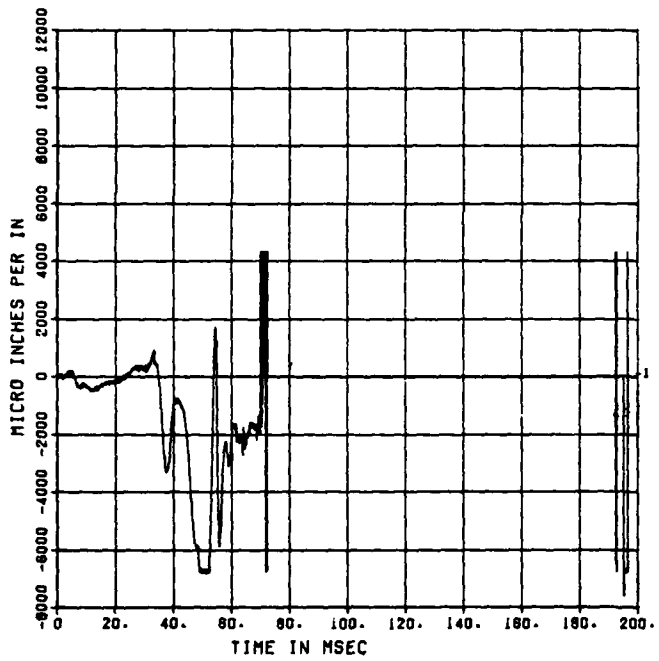




** PEAK VALUE IS 165 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS1 TOP
25000. HZ CAL 2999.

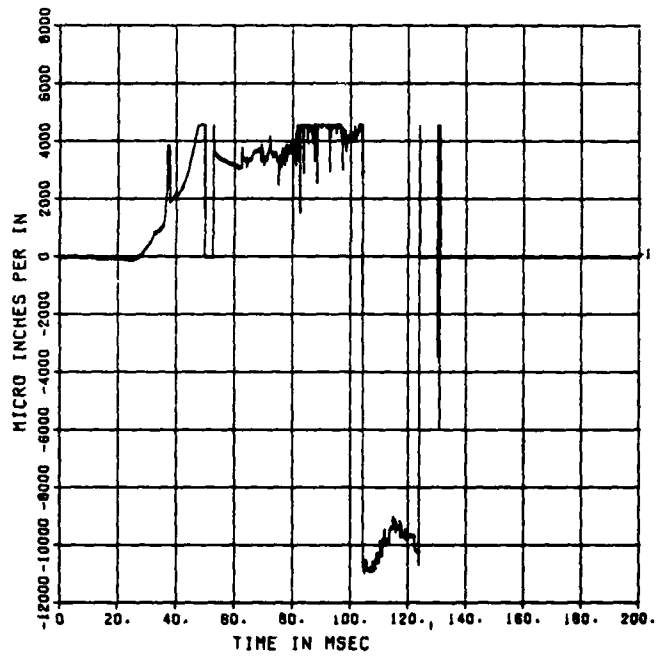
07/03/79 12265 30 R0726



** PEAK VALUE IS 162 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS2 TOP
25000. HZ CAL 2899.

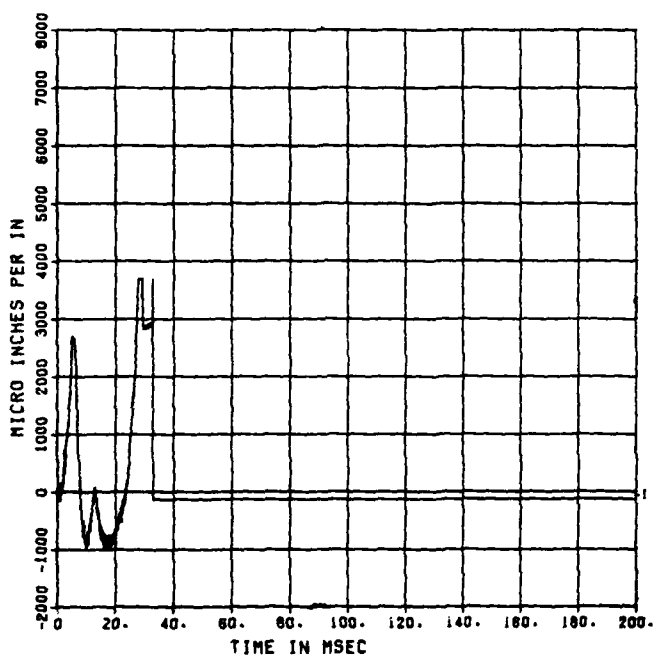
07/03/79 12265 34 R0726



** PEAK VALUE IS 278 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS2 BOTTOM
25000. HZ CAL 2899.

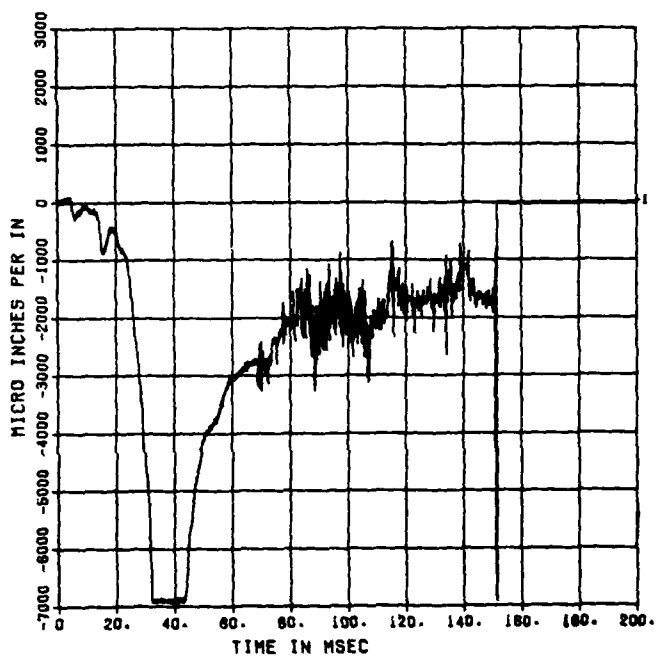
07/03/79 12265 33 R0726



** PEAK VALUE IS 28 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS3 TOP
25000. HZ CAL 2899.

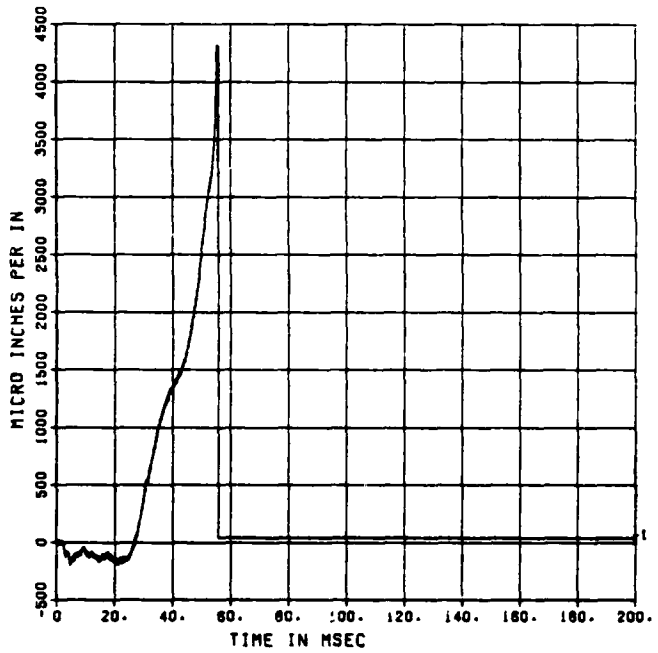
07/03/79 12265 38 R0726



** PEAK VALUE IS 139 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS3 BOTTOM
25000. HZ CAL 2899.

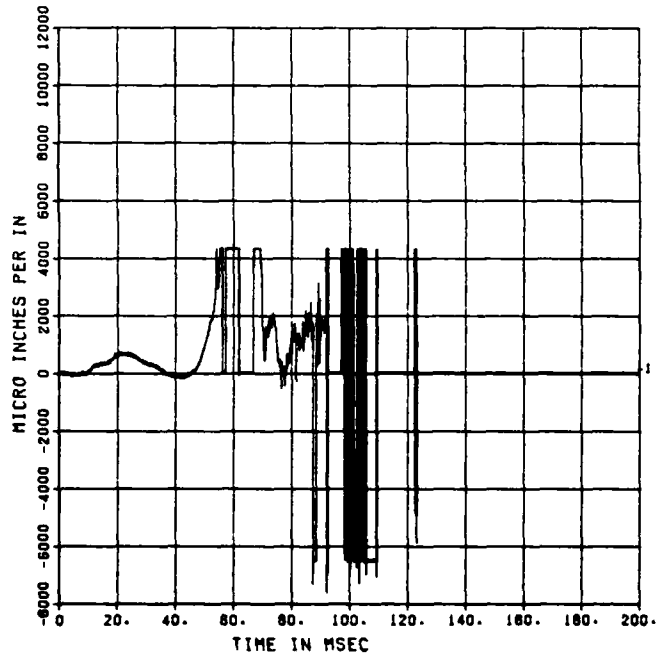
07/03/79 12265 37 R0726



** PEAK VALUE IS 43 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS4 TOP
25000. HZ CAL 2899.

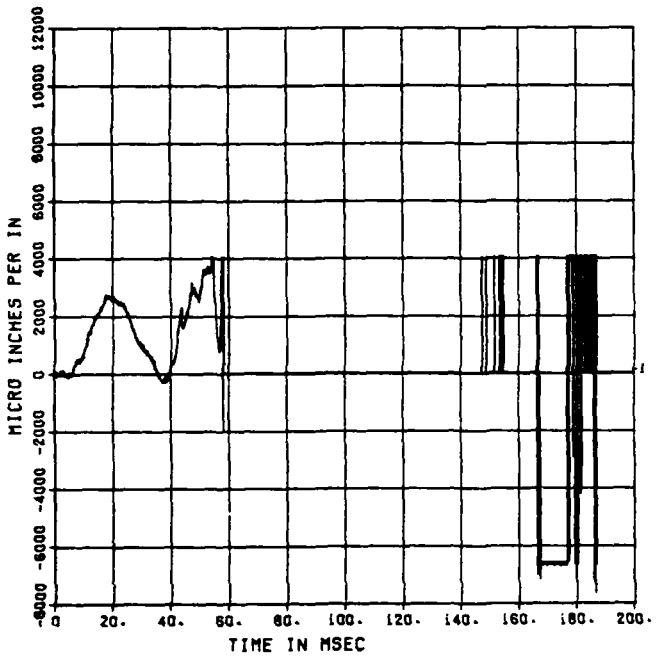
07/03/79 12265 40 R0726



** PEAK VALUE IS 162 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS4 BOTTOM
25000. HZ CAL 2899.

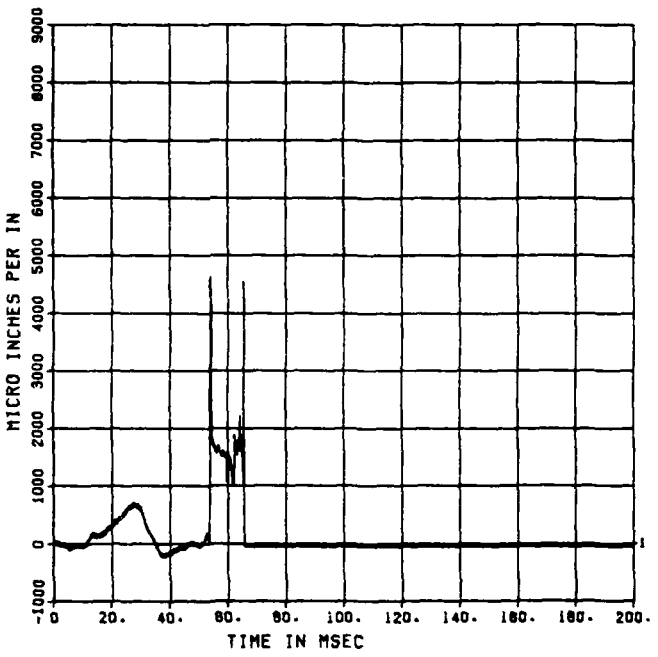
07/03/79 12265 39 R0726



** PEAK VALUE IS 184 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS5 TOP
25000. HZ CAL 2899.

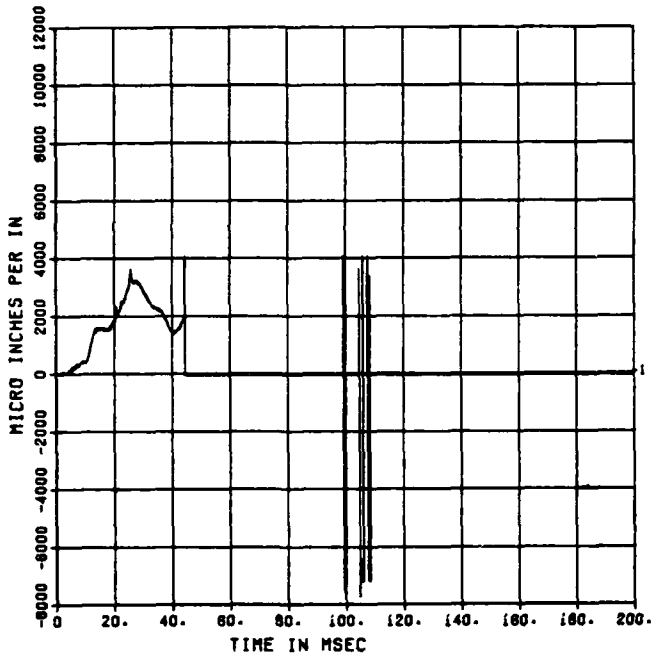
07/03/79 12265 42 R0726



** PEAK VALUE IS 60 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS5 BOTTOM
25000. HZ CAL 2899.

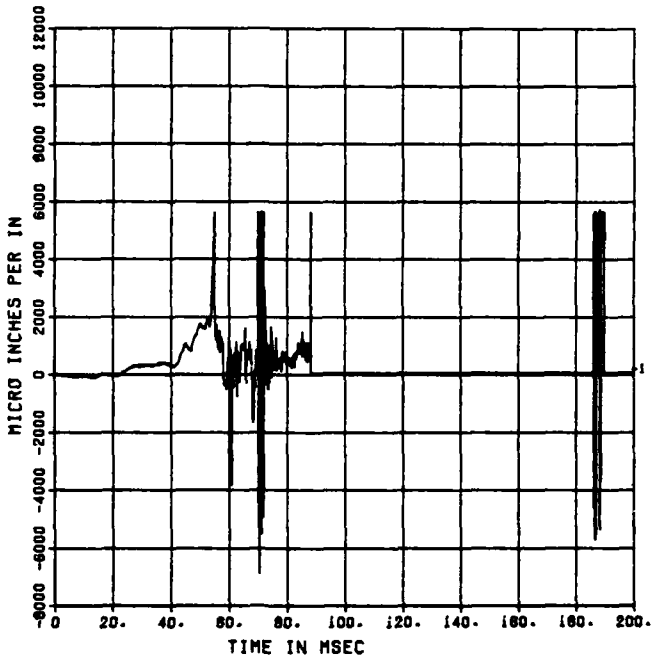
07/03/79 12285 41 R0726



** PEAK VALUE IS 166 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS6 TOP
25000. HZ CAL 2899.

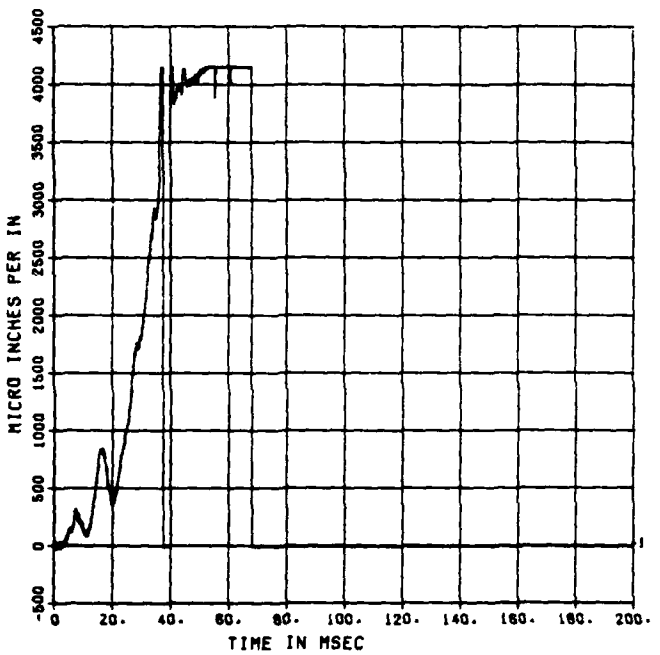
07/03/79 12285 44 R0726



** PEAK VALUE IS 137 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SS6 BOTTOM
25000. HZ CAL 2899.

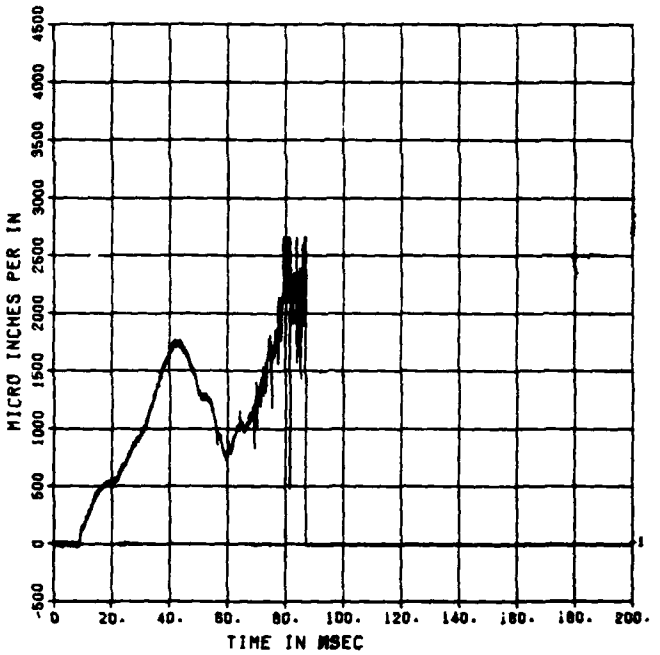
07/03/79 12265 43 R0728



** PEAK VALUE IS 43 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SL1 TOP
25000. HZ CAL 1743.

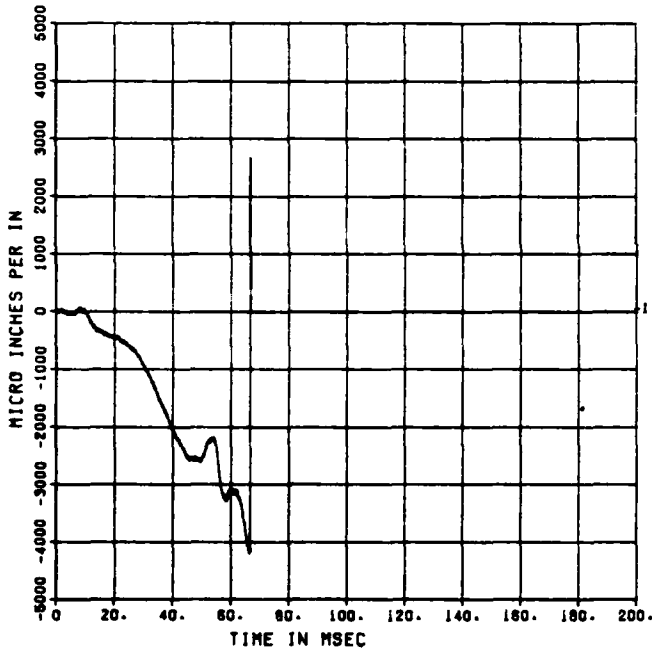
07/03/79 12265 32 R0728



** PEAK VALUE IS 53 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SL1 BOTTOM
25000. HZ CAL 1743.

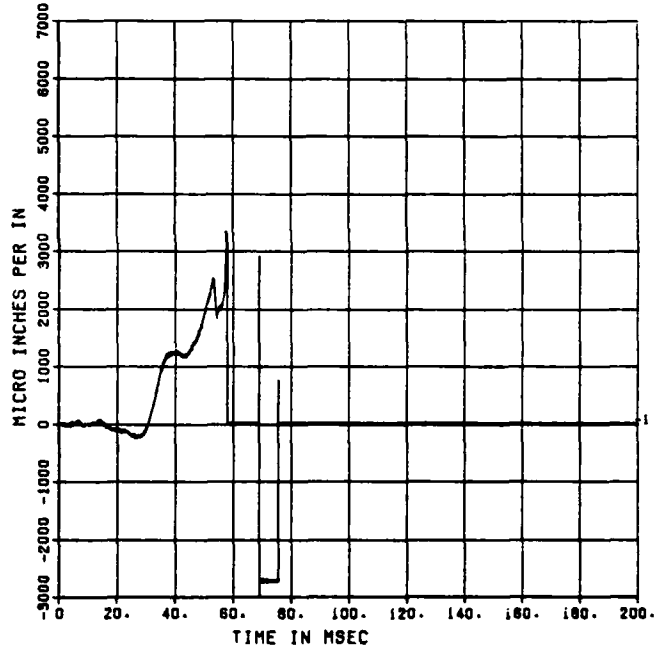
07/03/79 12265 31 R0726



** PEAK VALUE IS 141 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SL2 TOP
25000. HZ CAL 1743.

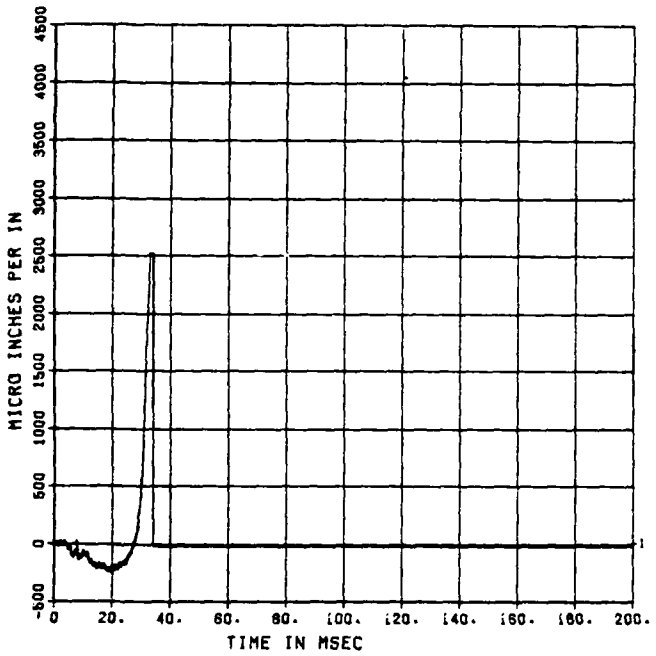
07/03/79 12265 38 R0726



** PEAK VALUE IS 91 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SL2 BOTTOM
25000. HZ CAL 1743.

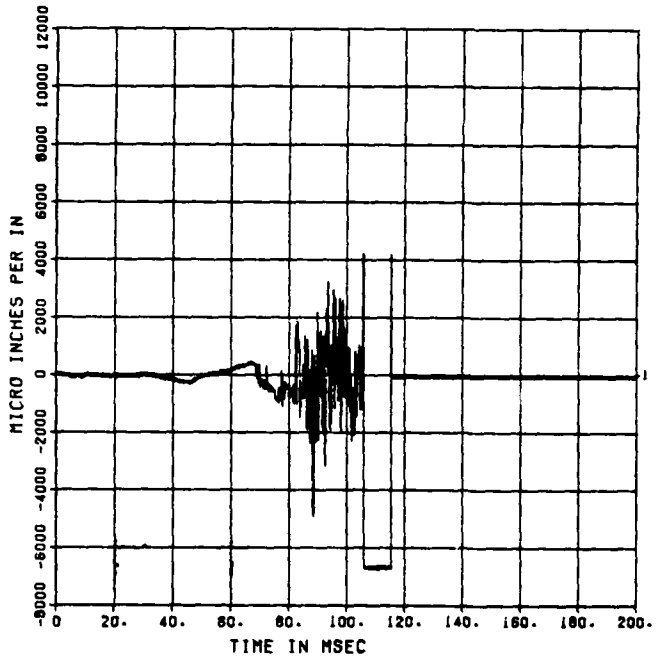
07/03/79 12285 35 R0726



** PEAK VALUE IS 45 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BS1 TOP
25000. HZ CAL 2899.

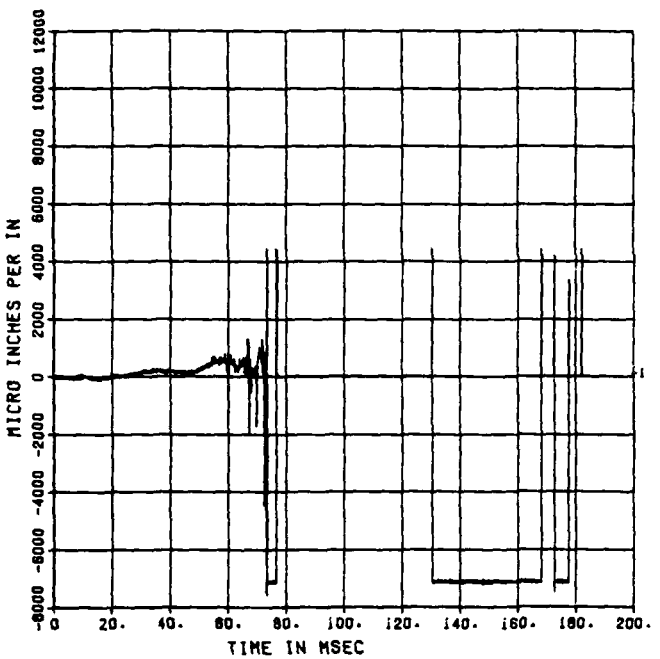
07/03/79 12285 45 R0726



** PEAK VALUE IS 131 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BS2 TOP
25000. HZ CAL 2899.

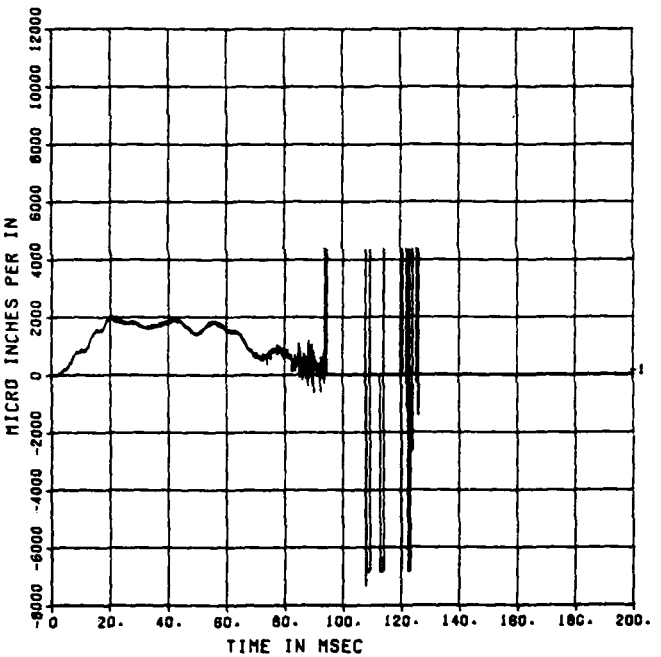
07/03/79 12265 47 R0726



** PEAK VALUE IS 182 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BS2 BOTTOM
25000. HZ CAL 2899.

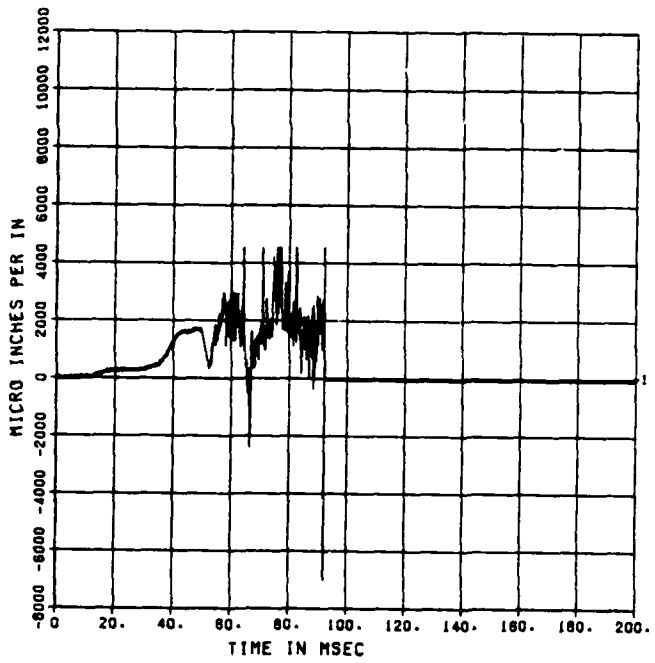
07/03/79 12265 46 R0726



** PEAK VALUE IS 162 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BS3 TOP
25000. HZ CAL 2899.

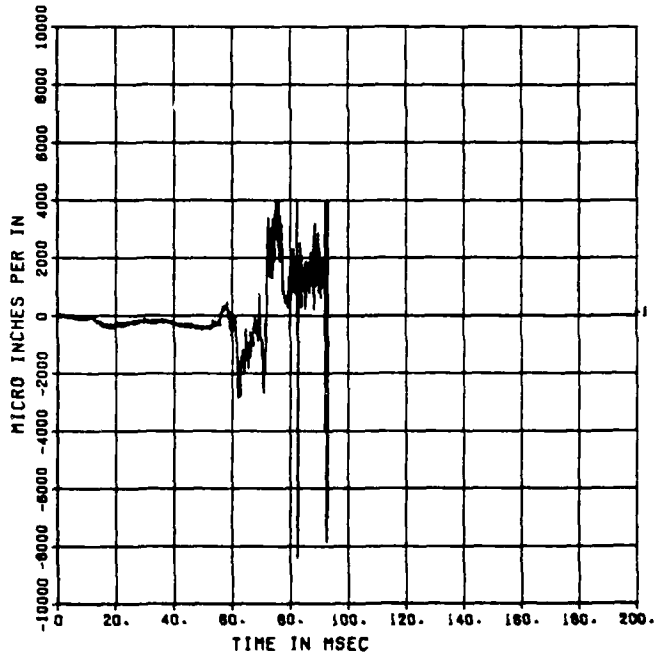
07/03/79 12265 49 R0726



** PEAK VALUE IS 142 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BS3 BOTTOM
25000. HZ CAL 2899.

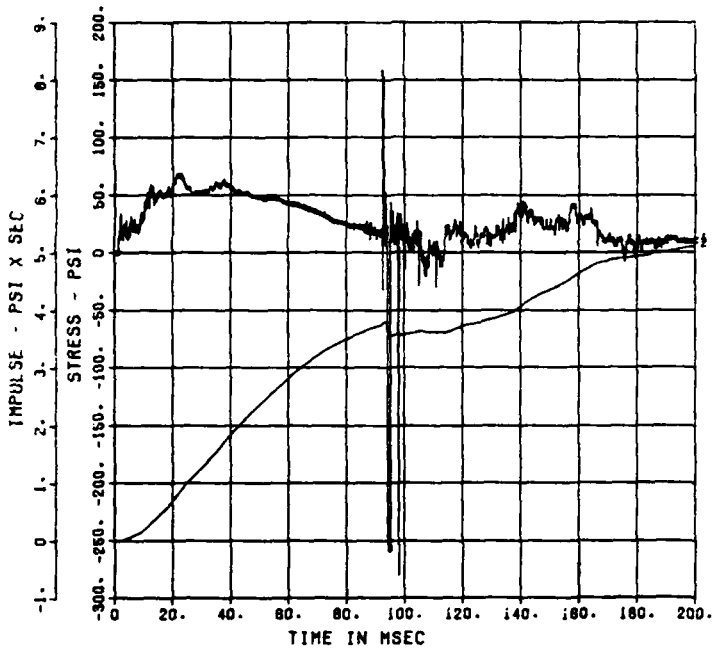
07/03/79 12265 48 R0726



** PEAK VALUE IS 190 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
EP-1
25000. HZ CAL 106.

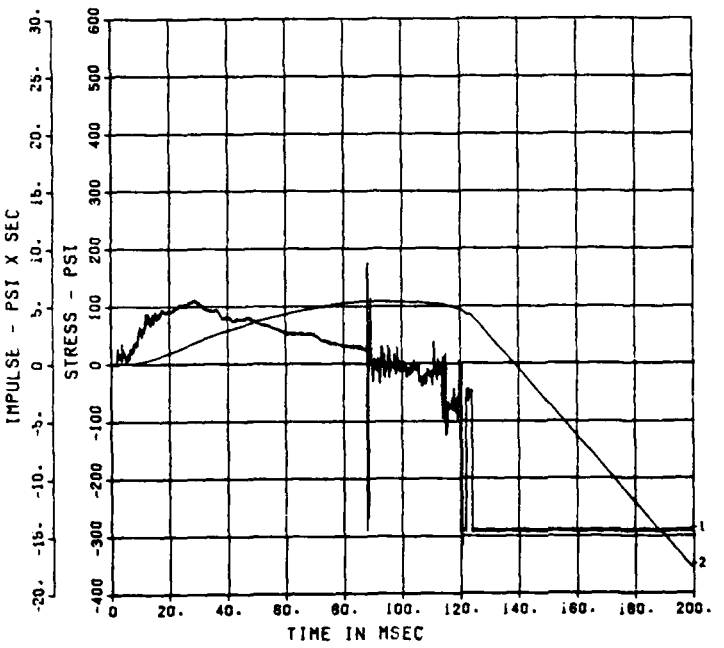
07/03/79 12265 53 R0726



** PEAK VALUE IS 164 % OVER CALIBRATION **

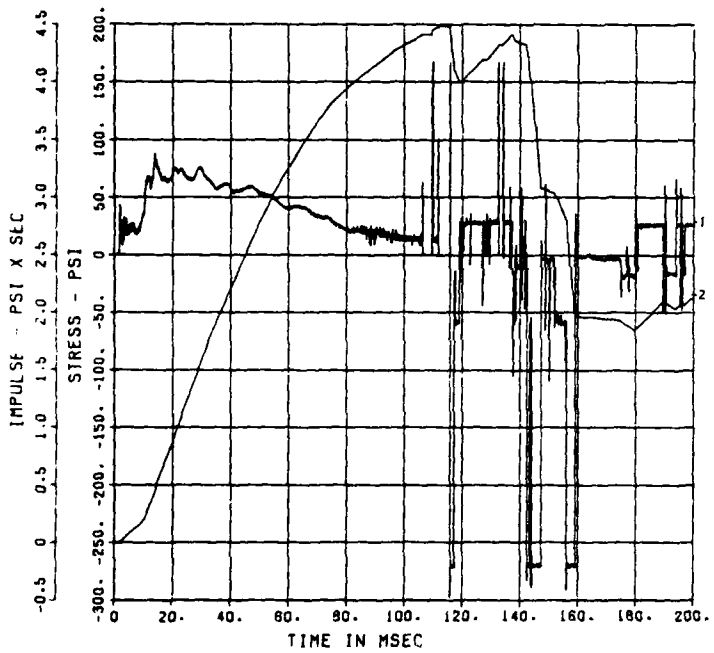
SLAB 3C BEAM UPGRADE
EP-2
25000. HZ CAL 116.7

07/03/79 12265 54 R0726



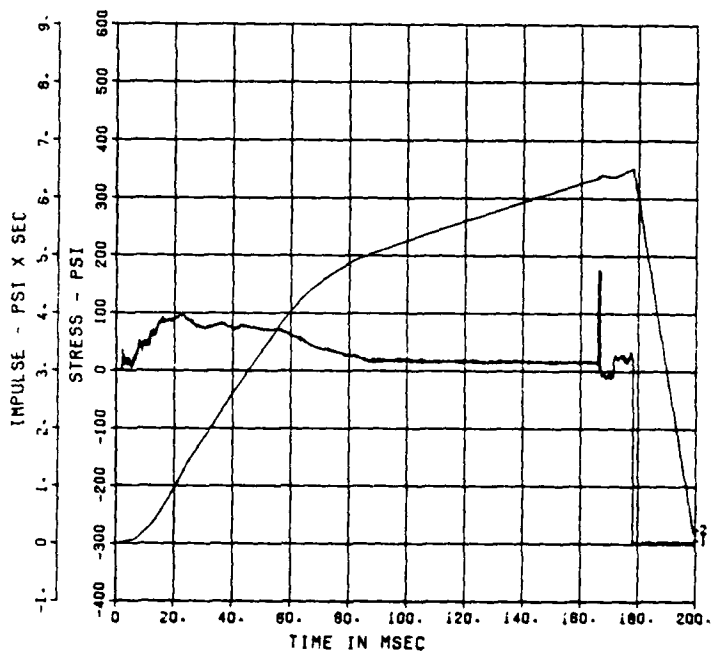
** PEAK VALUE IS 171 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
CP
25000. HZ CAL 116.8
07/03/79 12265 52 R0726



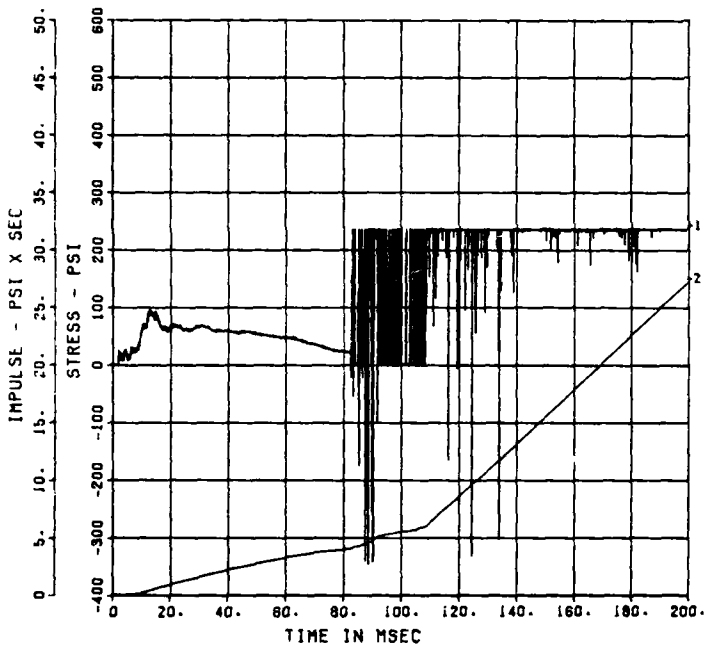
** PEAK VALUE IS 155 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
WP-1
25000. HZ CAL 123.3
07/03/79 12265 50 R0726



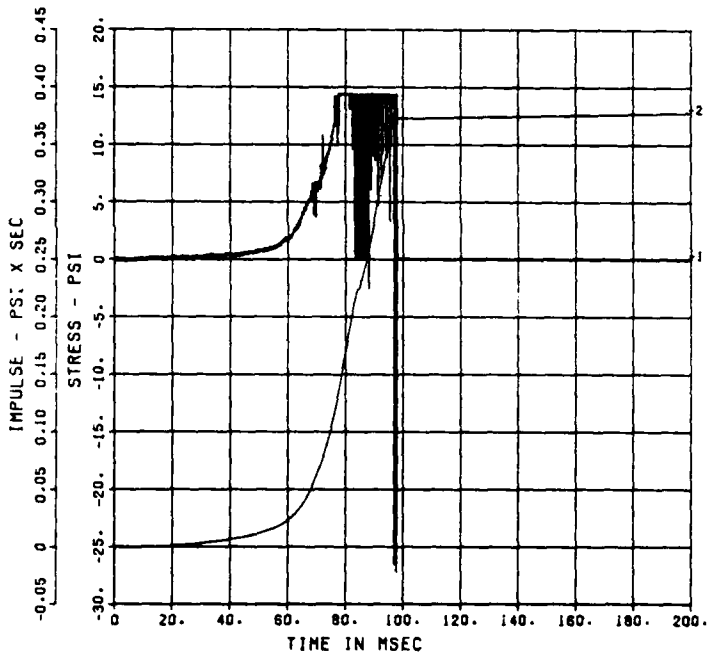
** PEAK VALUE IS 143 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
WP-2
25000. HZ CAL 121.7
07/03/79 12265 S1 R0726



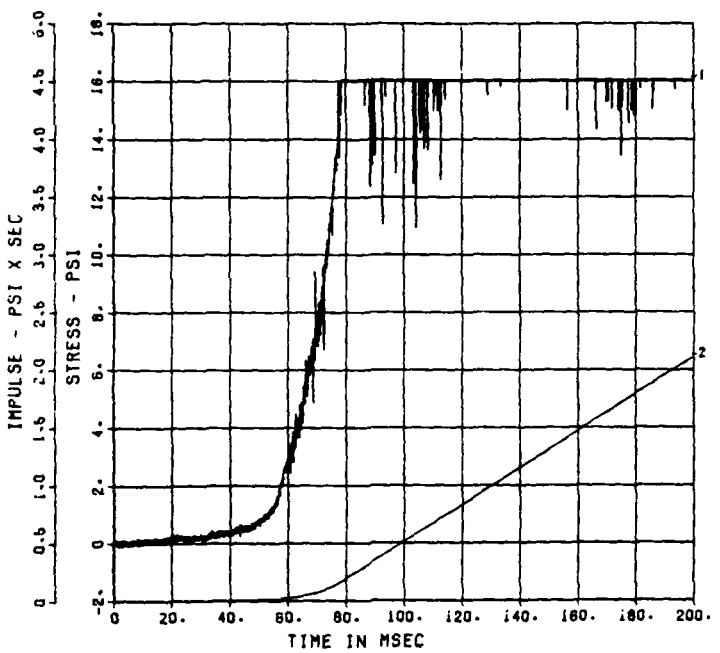
** PEAK VALUE IS 183 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
SP
25000. HZ CAL 9.55
07/03/79 12265 S5 R0726



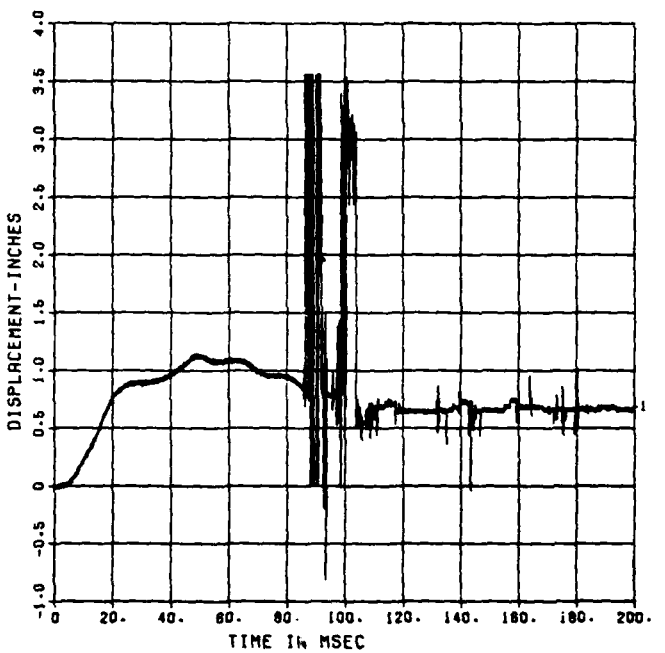
** PEAK VALUE IS 185 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
NP
25000. HZ CAL 11.30
07/03/79 12265 56 R0726



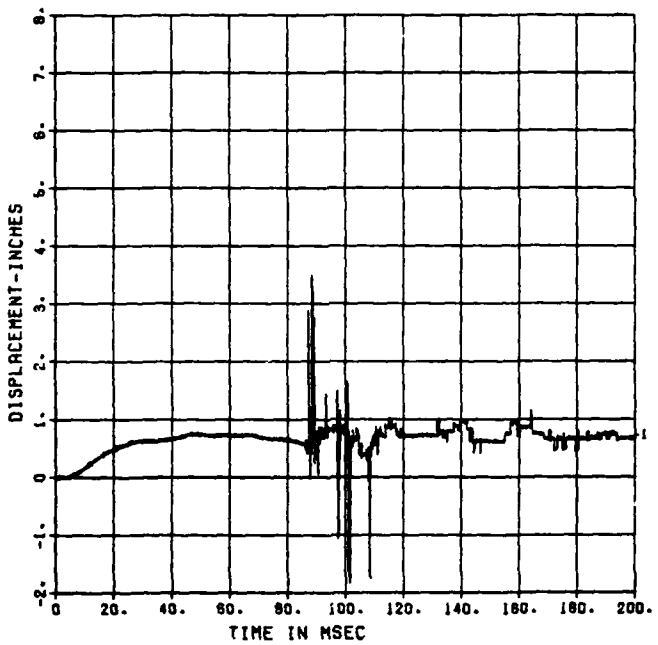
** PEAK VALUE IS 35 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BD1
25000. HZ CAL 2.51
07/11/79 12265 74 R0199



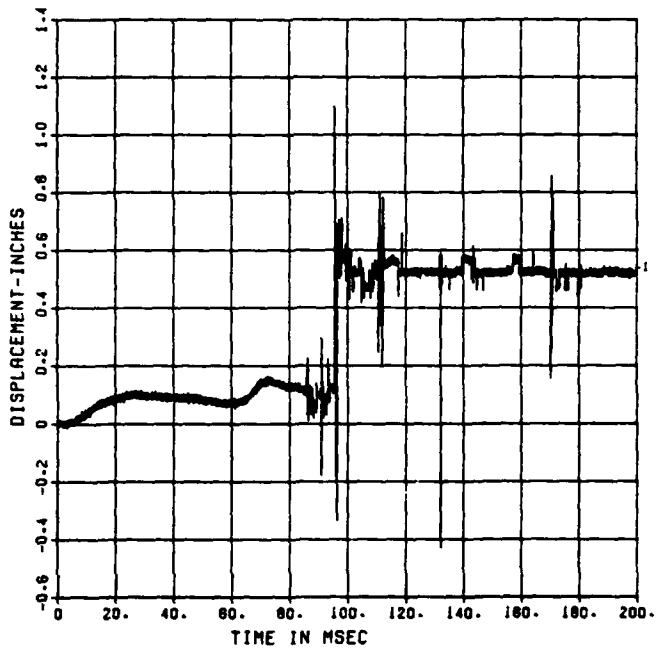
** PEAK VALUE IS 42 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
BD2
25000. HZ CAL 2.44
07/11/79 12265 84 R0199

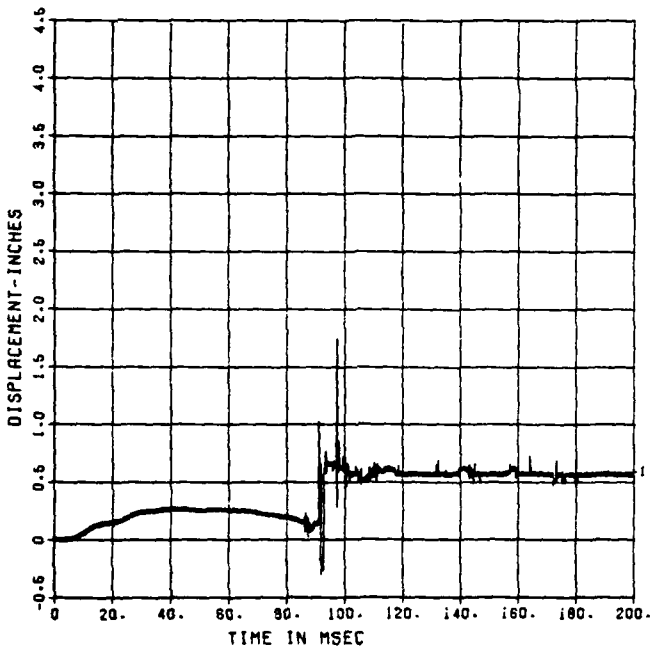


** PEAK VALUE IS 43 % OVER CALIBRATION **

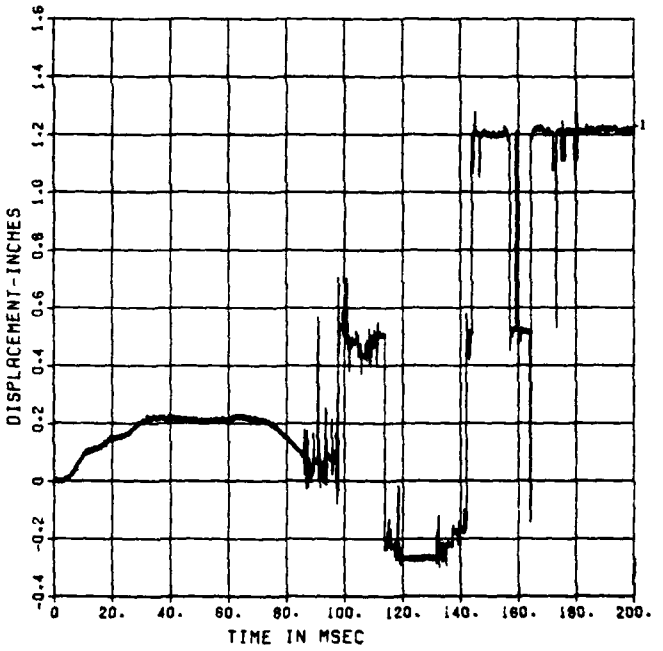
SLAB 3C BEAM UPGRADE
NBD
25000. HZ CAL 1.59
07/11/79 12265 75 R0199



SLAB 3C BEAM UPGRADE
SBD
25000. HZ CAL 1.74
07/11/79 12265 93 RD199

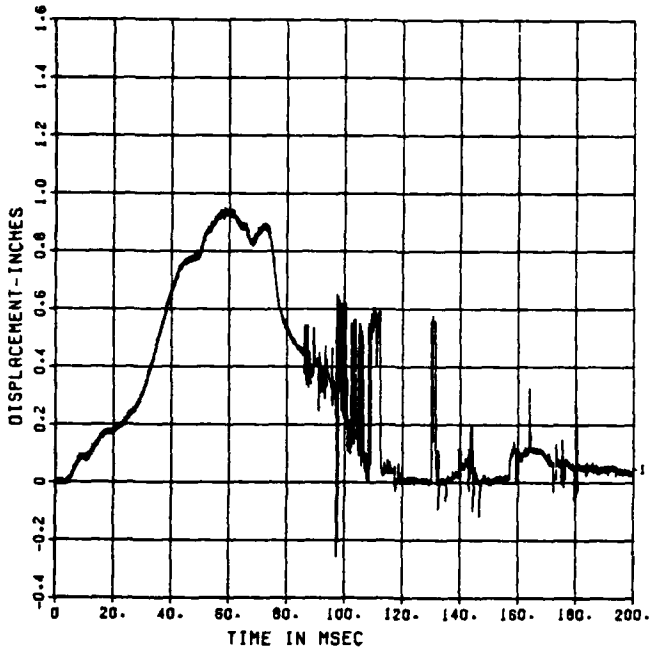


SLAB 3C BEAM UPGRADE
WBD
25000. HZ CAL 1.57
07/11/79 12265 78 RD199



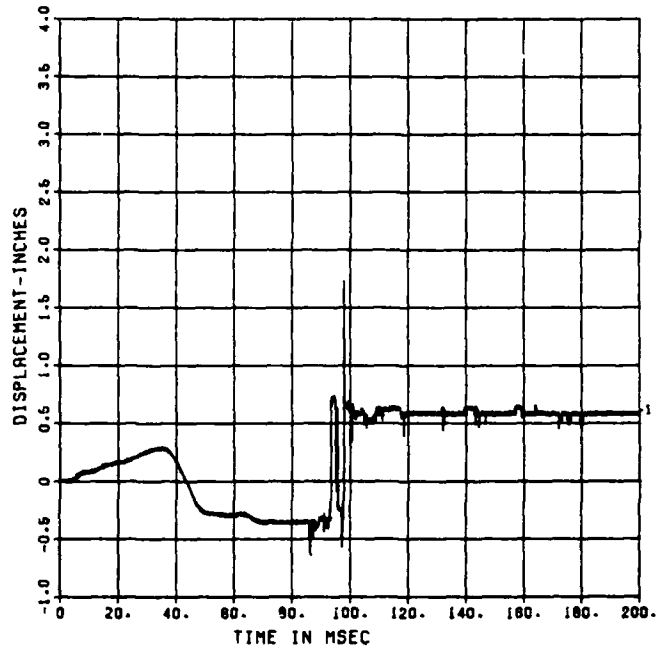
SLAB 3C BEAM UPGRADE
NUD
25000. HZ CAL 1.73

07/11/79 12265 76 R0199



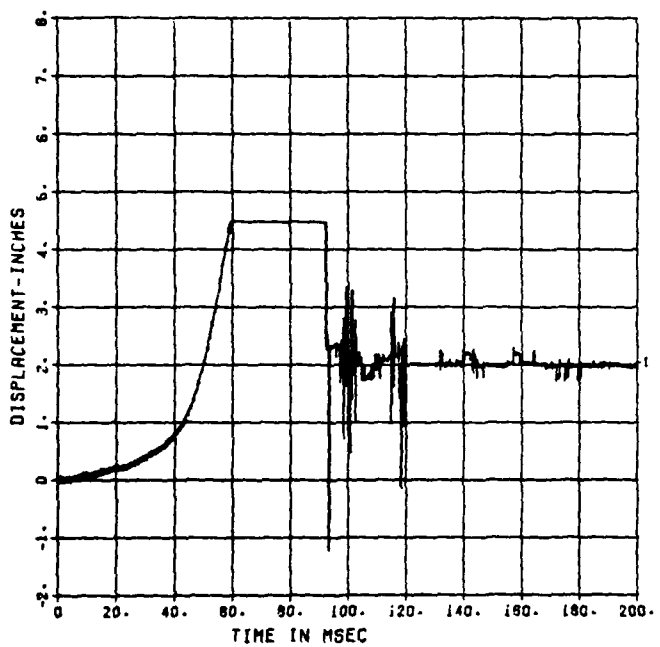
SLAB 3C BEAM UPGRADE
SUD
25000. HZ CAL 1.62

07/11/79 12265 77 R0199



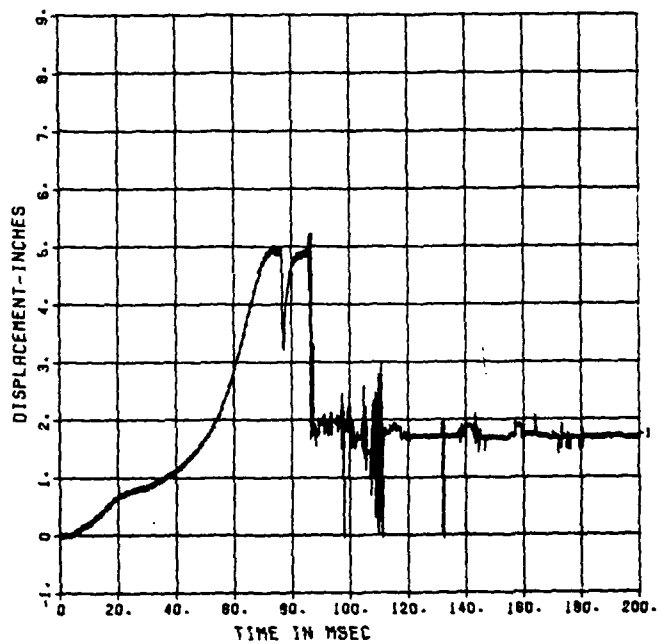
** PEAK VALUE IS 7 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
ED1
25000. HZ CAL 2.84
07/11/79 12285 79 R0199



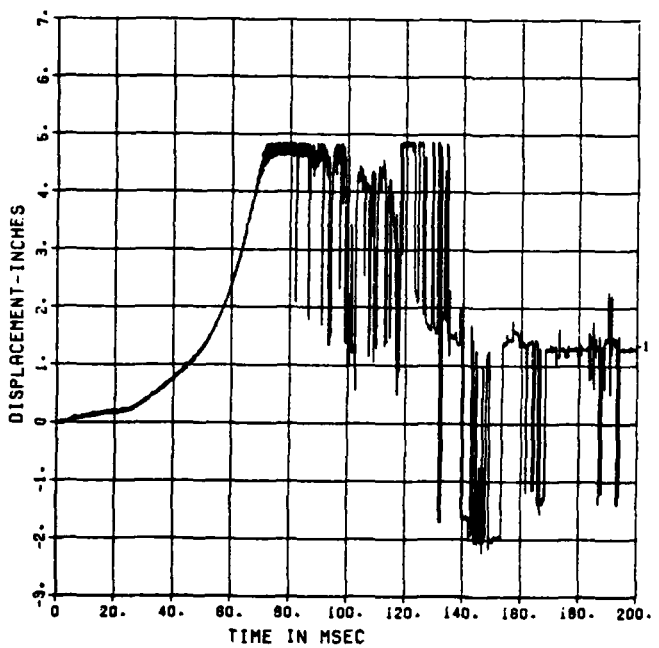
** PEAK VALUE IS 59 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
WD1
25000. HZ CAL 3.60
07/11/79 12285 80 R0199



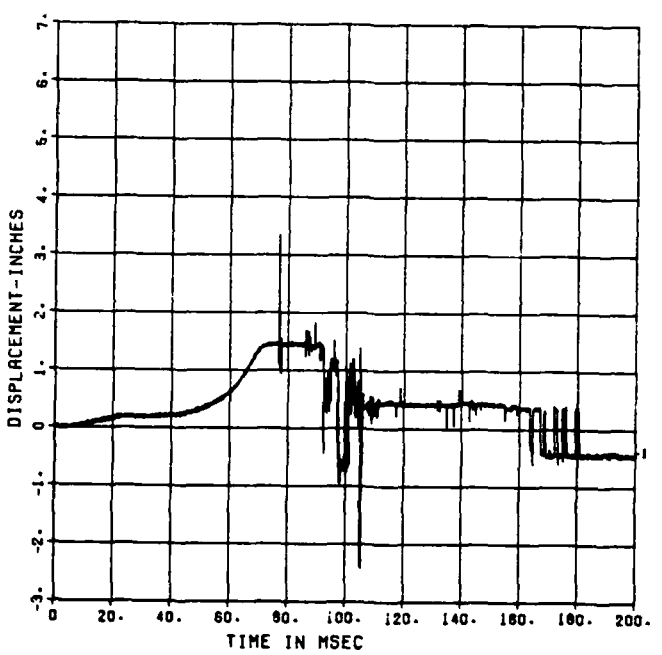
** PEAK VALUE IS 42 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
WD2
25000. HZ CAL 2.54
07/11/79 12265 81 R0199



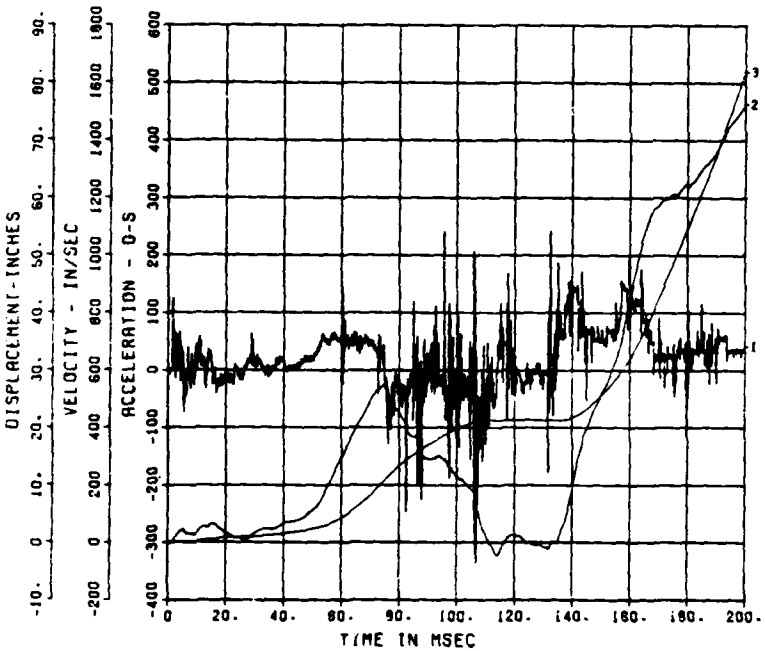
** PEAK VALUE IS 91 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
WD3
25000. HZ CAL 3.31
07/11/79 12265 82 R0199



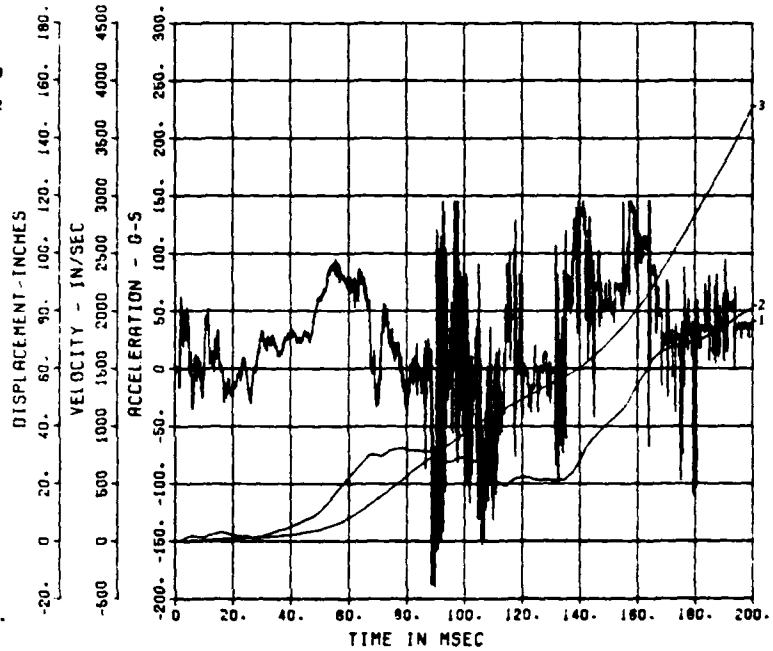
** PEAK VALUE IS 1 % OVER CALIBRATION **

SLAB 3C BEAM UPGRADE
A1
25000. HZ CAL 95.21
07/11/79 12285 86 R0199



** PEAK VALUE IS 253 % OVER CALIBRATION **

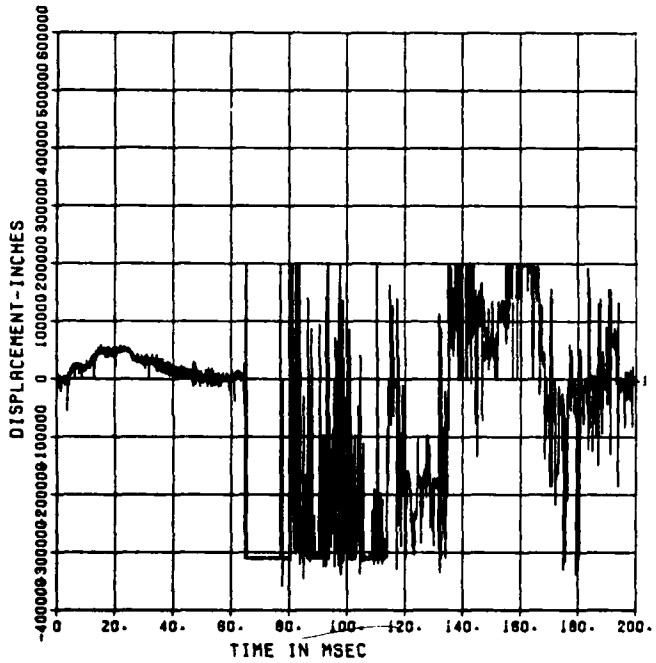
SLAB 3C BEAM UPGRADE
A2
25000. HZ CAL 91.80
07/11/79 12285 87 R0199



** PEAK VALUE IS 107 % OVER CALIBRATION **

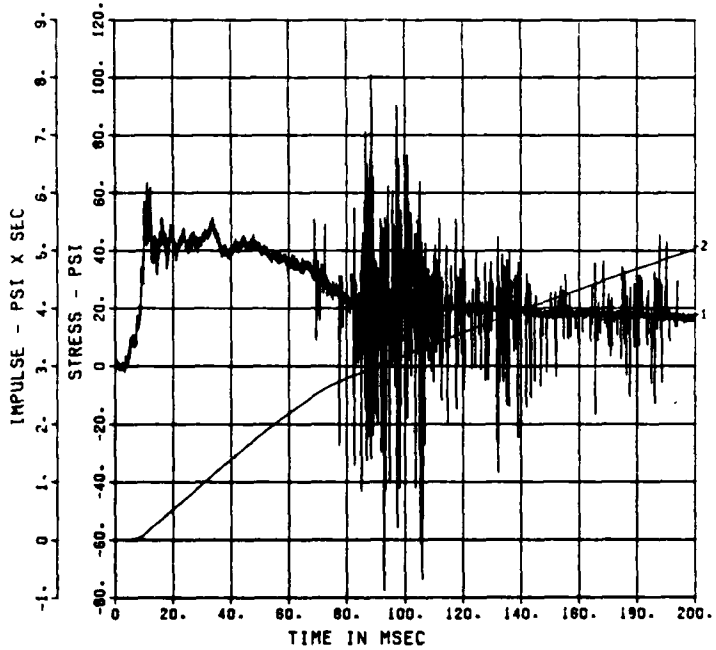
SLAB 3C BEAM UPGRADE
LOAD 2
25000. HZ CAL 128484.

07/11/79 12265 85 R0199



SLAB 3C BEAM UPGRAD
NORWOOD
25000. HZ CAL 134.2

07/11/79 12265 90 R0199



** PEAK VALUE IS 179 % OVER CALIBRATION **

FEMA DISTRIBUTION LIST

Federal Emergency Management Agency
Mitigation and Research
60 cy ATTN: Administrative Office
Mr. Robert C. Crawford
Mr. David W. Benson
Mr. Donald A Bettge
1725 I Street
Washington, D. C. 20472

Director, Defense Nuclear Agency
ATTN: Mr. Tom Kennedy
STTL/Technical Library
Washington, D. C. 20305

Defense Intelligence Agency
ATTN: Mr. Carl Wiehle
CKW DB-4C2
Washington, D. C. 20301

Assistant Secretary of the Army (R&D)
ATTN: Assistant for Research
Washington, D. C. 20301

Chief of Engineers
Department of the Army
ATTN: ENGEME-RD
Washington, D. C. 20314

U. S. Army Engineer Waterways
Experiment Station
2 cy ATTN: Mr. Mark McVay
2 cy Technical Library
Mr. Bill Flathau
Mr. Bill Huff
Mr. Stan Woodson
P. O. Box 631
Vicksburg, Mississippi 39180

Defense Technical Information Center
12 cy ATTN: TC/Mr. Myer B. Kahn
Cameron Station
Alexandria, Virginia 22314

Director, Army Materials and Mechanics
Research Center
ATTN: Technical Library
Watertown, Massachusetts 02172

Associate Director, Natural Resources and
Commercial Services
Office of Science and Technology Policy
ATTN: Mr. Phillip M. Smith
Executive Office Bldg.
Washington, D. C. 20500

AFWL/Civil Engineering Division
ATTN: Technical Library
Kirtland Air Force Base
Albuquerque, New Mexico 87117

Civil Engineering Center/AF/PRECET
ATTN: Technical Library
Wright-Patterson Air Force Base
Dayton, Ohio 45433

Chief of Naval Research
Washington D. C. 20306

Commanding Officer
U. S. Naval Civil Engineering Laboratory
ATTN: Document Library
Port Hueneme, California 93041

Command and Control Technical Center
Department of Defense
Room 2E312 Pentagon
Washington, D. C. 20301

National Bureau of Standards
ATTN: Mr. Samuel Kramer
Dr. Lewis V. Spencer
Washington, D. C. 20234

Department of Energy
Headquarters Library, G-49
Washington, D. C. 20545

Oak Ridge National Laboratory
Emergency Technology Division
ATTN: Library
P. O. Box X
Oak Ridge, Tennessee 37830

ITT Research Institute
ATTN: Mr. Anatole Longinow
10 West 35th Street
Chicago, Illinois 60616

H. L. Murphy Associates
Box 1727
San Mateo, California 94401

James E. Beck and Associates
4216 Los Palos Avenue
Palo Alto, California 94306

Research Triangle Institute
ATTN: Mr. Edward L. Hill
P. O. Box 12194
Research Triangle Park, North Carolina 27709

Los Alamos Scientific Laboratory
ATTN: Document Library
Los Alamos, New Mexico 87544

Agbabian Associates
250 North Nash Street
El Segunda, California 90245

Sandia Corporation
ATTN: Dr. Clarence R. Mehl
Dept. 5230
Box 5800, Sandia Base
Albuquerque, New Mexico 87115

Bell Telephone Laboratories
ATTN: Mr. E. Witt
Mr. R. May
Whippany Road
Whippany, New Jersey 07981

Human Science Research
ATTN: Dr. William Chenault
Westgate Industrial Park
P. O. Box 370
McLean, Virginia 22010

Scientific Services, Inc.
ATTN: Mr. C. Wilton
517 East Bayshore Drive
Redwood City, California 94060

University of Florida
Civil Defense Technical Services Center
College of Engineering
Dept. of Engineering
Gainesville, Florida 32601

The RAND Corporation
ATTN: Document Library
1700 Main Street
Santa Monica, California 90401

GARD, Inc.
7449 North Natchez Avenue
Niles, Illinois 60648

Director, Ballistic Research Laboratory
ATTN: Mr. George Coulter
Document Library
Aberdeen Proving Ground, Maryland 21005

Director, Office of Administration
Program Planning and Control
Department of Housing and Urban Development
ATTN: Mr. Bert Greenglass
Washington, D. C. 20410

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS, Unclassified, U. S. Army Engineer Waterways Experiment Station, July 1981, 317 pp, Project Order No. DCPA 01-78-C-0267, Work Unit 1127E.

The problem of upgrading one-way reinforced concrete (R/C) slab floor systems for keyworker shelters was studied in this program. The objective was to develop competent designs for upgrading such systems that would use readily available materials, be easy to construct, and increase the load-carrying capacities of such systems to 50 psi or greater. Two upgrading methods were developed and evaluated: a wooden post method and a steel beam method.

It was concluded that sound upgrading systems can be made of readily available materials that only require simple construction skills. The tests illustrate that the load capacity of a one-way R/C floor system can be increased five to seven times using a proper upgrading system. The design procedures proved to be conservative, resulting in structures much stronger than predicted. Both upgrading methods are excellent techniques for increasing the load capacities of a keyworker shelter above 50 psi.

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS, Unclassified, U. S. Army Engineer Waterways Experiment Station, July 1981, 317 pp, Project Order No. DCPA 01-78-C-0267, Work Unit 1127E.

The problem of upgrading one-way reinforced concrete (R/C) slab floor systems for keyworker shelters was studied in this program. The objective was to develop competent designs for upgrading such systems that would use readily available materials, be easy to construct, and increase the load-carrying capacities of such systems to 50 psi or greater. Two upgrading methods were developed and evaluated: a wooden post method and a steel beam method.

It was concluded that sound upgrading systems can be made of readily available materials that only require simple construction skills. The tests illustrate that the load capacity of a one-way R/C floor system can be increased five to seven times using a proper upgrading system. The design procedures proved to be conservative, resulting in structures much stronger than predicted. Both upgrading methods are excellent techniques for increasing the load capacities of a keyworker shelter above 50 psi.

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS, Unclassified, U. S. Army Engineer Waterways Experiment Station, July 1981, 317 pp, Project Order No. DCPA 01-78-C-0267, Work Unit 1127E.

The problem of upgrading one-way reinforced concrete (R/C) slab floor systems for keyworker shelters was studied in this program. The objective was to develop competent designs for upgrading such systems that would use readily available materials, be easy to construct, and increase the load-carrying capacities of such systems to 50 psi or greater. Two upgrading methods were developed and evaluated: a wooden post method and a steel beam method.

It was concluded that sound upgrading systems can be made of readily available materials that only require simple construction skills. The tests illustrate that the load capacity of a one-way R/C floor system can be increased five to seven times using a proper upgrading system. The design procedures proved to be conservative, resulting in structures much stronger than predicted. Both upgrading methods are excellent techniques for increasing the load capacities of a keyworker shelter above 50 psi.

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS, Unclassified, U. S. Army Engineer Waterways Experiment Station, July 1981, 317 pp, Project Order No. DCPA 01-78-C-0267, Work Unit 1127E.

The problem of upgrading one-way reinforced concrete (R/C) slab floor systems for keyworker shelters was studied in this program. The objective was to develop competent designs for upgrading such systems that would use readily available materials, be easy to construct, and increase the load-carrying capacities of such systems to 50 psi or greater. Two upgrading methods were developed and evaluated: a wooden post method and a steel beam method.

It was concluded that sound upgrading systems can be made of readily available materials that only require simple construction skills. The tests illustrate that the load capacity of a one-way R/C floor system can be increased five to seven times using a proper upgrading system. The design procedures proved to be conservative, resulting in structures much stronger than predicted. Both upgrading methods are excellent techniques for increasing the load capacities of a keyworker shelter above 50 psi.

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

McVay, Mark K.
Test and analysis of upgraded one-way reinforced concrete floor slabs : final report / by Mark K. McVay (Structures Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station, [1981].
319 p. : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; SL-81-4)
Cover title.
"July 1981."
"Prepared for Federal Emergency Management Agency (Formerly the Defense Civil Preparedness Agency) under Project Order No. DCPA 01-78-C-0267, Work Unit 1127E."
"This report has been reviewed in the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency."

McVay, Mark K.
Test and analysis of upgraded one-way reinforced : ... 1981.
(Card 2)

1. Concrete--testing. 2. Reinforced concrete. 3. Floors, concrete. 4. Materials--Dynamic testing. I. United States. Federal Emergency Management Agency. II. U.S. Army Engineer Waterways Experiment Station. Structures Laboratory. III. Title IV. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; SL-81-4.
TA7.W34 no.SL-81-4

TECHNICAL REPORT SL-81-4

TEST AND ANALYSIS OF UPGRADED ONE-WAY REINFORCED CONCRETE FLOOR SLABS

by

Mark K. McVay

Structures Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

July 1981
Final Report

Approved For Public Release; Distribution Unlimited

FEMA Review Notice

This report has been reviewed in the Federal Emergency Management Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Emergency Management Agency.

Prepared for Federal Emergency Management Agency
(Formerly the Defense Civil Preparedness Agency)
Washington, D. C. 20472

Under Project Order No. DCPA 01-78-C-0267, Work Unit 1127E

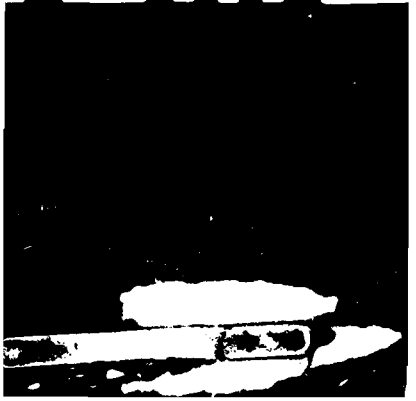
SUMMARY

The problem of upgrading one-way reinforced concrete (R/C) slab floor systems for keyworker shelters was studied in this program. The objective was to develop competent designs for upgrading such systems that would use readily available materials, be easy to construct, and increase the load-carrying capacities of such systems to 50 psi or greater. Two upgrading methods were developed and evaluated: a wooden post method and a steel beam method.

For the wooden post method, several 4- by 4-inch timbers were placed in groups under the midspan of the slabs. These groups acted as units to form columns large enough to accept large loads and provide sufficient bearing area. For the steel beam method, the floor slab panels were supported at midspan with a series of small steel beams held up by steel pipe columns. The components were kept small enough so two or three people could handle them. The existing beams of the floor systems were upgraded with additional posts in both methods.

The increased load capacities resulting from these upgrading methods were verified by conducting dynamic load tests on three identical full-scale sections of a typical one-way R/C slab floor system. In order to have a realistic baseline for comparison purposes, a typical slab section without any upgrading was first tested. It was predicted that the nonupgraded slab section would fail when subjected to a peak overpressure of 16 psi. It was tested with average peak overpressures of about 15 and 33 psi. The first test caused cracks to form in the top of the slab along the beams. The second test greatly exceeded the calculated load-carrying capacity of the slab and caused complete collapse. Next, a test series was done on an identical slab section upgraded using the wooden post method. The upgrading system was designed to increase the load capacity of the slab section to about 55 psi, at which pressure it was predicted to fail in punching shear. However, this analysis for punching shear was for concrete columns and static loads. This upgraded slab section was tested five times and resisted average peak overpressures of 16, 24, 38, 73, and 113 psi. During the second, third, and fourth tests,

some hairline cracks formed, but no serious damage occurred. During the fifth test, two timbers started punching through the slab and several timbers under the center beam starting splitting. The third test series was on the typical slab section upgraded using the steel beam method. This upgrading system was designed to withstand 50 to 60 psi, at which pressure it was predicted to fail in shear. It was loaded with average peak overpessures of 35, 63, and 92 psi. The slab cracked where supported by the steel beams (no negative reinforcement in slabs in this location) as a result of the loading for the first test, remained undamaged for the second test, and collapsed under the loading for the third test. Complete data records for the tests are presented in Appendices A, B, and C.

It was concluded that sound upgrading systems can be made of readily available materials that only require simple construction skills. The tests illustrate that the load capacity of a one-way R/C floor system can be increased five to seven times using a proper upgrading system. The design procedures proved to be conservative, resulting in structures much stronger than predicted. Both upgrading methods are excellent techniques for increasing the load capacities of a keyworker shelter above 50 psi.

ILME