**US Army Corps
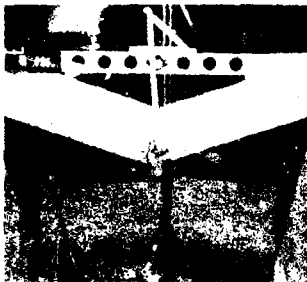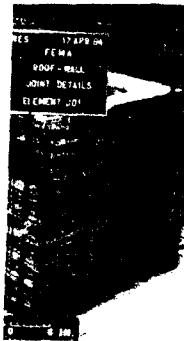of Engineers**

AD-A189 075

EVALUATION OF ALTERNATE ROOF-WALL DETAILS FOR THE KEYWORKER BLAST SHELTER

by

Thomas R. Slawson, Stanley C. Woodson, Aaron L. Harris

Structures Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

November 1987

Final Report

Approved For Public Release. Distribution Unlimited

DTIC
ELECTE
JAN 2 1 1988
E

STRUCTURES LABORATORY

88 1 15 056

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.
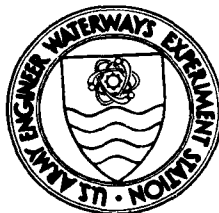
TECHNICAL REPORT SL-87-29

EVALUATION OF ALTERNATE ROOF-WALL DETAILS FOR THE KEYWORKER BLAST SHELTER

by

Thomas R. Slawson, Stanley C. Woodson, Aaron L. Harris

Structures Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

November 1987
Final Report

EVALUATION OF ALTERNATE ROOF-WALL
DETAILS FOR THE KEYWORKER BLAST SHELTER

At the time this study was initiated, several civil defense policy
options were being analyzed for protection of the nation's industrial capabil-
ity and key workers. One option under consideration called for construction
of blast shelters to protect key workers remaining in high-risk areas during
a national crisis. In support of this option, the Federal Emergency Manage-
ment Agency (FEMA) tasked the US Army Engineer Division, Huntsville (HND), to
develop Keyworker shelter designs. The design required an earth-covered
shelter to resist the radiation and blast effects of a 1-MT nuclear detonation
at the 50-psi peak overpressure level. Personnel in the Structures Laboratory
of the US Army Engineer Waterways Experiment Station (WES) supported HND with
design calculations and design verification experiments.

In the construction of a large number of shelters, it is important that
the shelter design provide the required structural capacity at reasonable
costs. The original roof-wall reinforcement detail created a constructibility
problem that increased construction costs. The objective of the experimental
program described in this report was to evaluate alternate roof-wall joint
details for the Keyworker blast shelter in an effort to improve construct-
ibility without reducing structural capacity.

Three 1/4-scale reinforced concrete box-type models were statically
tested under uniform water pressure in the 6-foot-diameter Small Blast Load
Generator (SBLG) at WES. Two of the models (JD1 and JD3) were reinforced
similarly to the original design except for the joint details. The third
model (JD2) was similar to the proposed final shelter design as evaluated in
the prototype tests conducted by WES. Based on test results, detail JD2 is
recommended for use in the final design.

TECHNICAL REPORT SL-87-29

EVALUATION OF ALTERNATE ROOF-WALL DETAILS FOR THE KEYWORKER BLAST SHELTER

by

Thomas R. Slawson, Stanley C. Woodson, Aaron L. Harris

Structures Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

DTIC
COPY
INSPECTED
6

November 1987

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for Federal Emergency Management Agency
Washington, DC 20472

A189 075

REPORT DOCUMENTATION PAGE

1a. REPORT SECURITY CLASSIFICATION	1b. RESTRICTIVE MARKINGS
Unclassified	

2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT
	Approved for public release; distribution unlimited.
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	

4. PERFORMING ORGANIZATION REPORT NUMBER(S)	5. MONITORING ORGANIZATION REPORT NUMBER(S)
Technical Report SL-87-29	

6a. NAME OF PERFORMING ORGANIZATION	6b. OFFICE SYMBOL (If applicable)	7a. NAME OF MONITORING ORGANIZATION
USAEWES Structures Laboratory	WESSS	

6c. ADDRESS (City, State, and ZIP Code)	7b. ADDRESS (City, State, and ZIP Code)
PO Box 631 Vicksburg, MS 39180-0631	

8a. NAME OF FUNDING/SPONSORING ORGANIZATION	8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
Federal Emergency Management Agency		

8c. ADDRESS (City, State, and ZIP Code)	10. SOURCE OF FUNDING NUMBERS			
	PROGRAM ELEMENT NO.	PROJECT NO.	TASK NO.	WORK UNIT ACCESSION NO.
Washington, DC 20472				

11. TITLE (Include Security Classification)

Evaluation of Alternate Roof-Wall Details for the Keyworker Blast Shelter

12. PERSONAL AUTHOR(S)
Slawson, Thomas R., Woodson, Stanley C., and Harris, Aaron L.

13a. TYPE OF REPORT	13b. TIME COVERED		14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM	TO	November 1987	106

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
FIELD	GROUP	SUB-GROUP	Knee joints Reinforcement details
			Load-deformation behavior Static testing
			Reinforced concrete slabs Uniform loading

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

At the time this study was initiated, several civil defense policy options were being analyzed for protection of the nation's industrial capability and key workers. One option under consideration called for construction of blast shelters to protect key workers remaining in high-risk areas during a national crisis. In support of this option, the Federal Emergency Management Agency (FEMA) tasked the US Army Engineer Division, Huntsville (HND), to develop Keyworker shelter designs. The design required an earth-covered shelter to resist the radiation and blast effects of a 1-MT nuclear detonation at the 50-psi peak overpressure level. Personnel in the Structures Laboratory of the US Army Engineer Waterways Experiment Station (WES) supported HND with design calculations and design verification experiments.

In the construction of a large number of shelters, it is important that the shelter design provide the required structural capacity at reasonable costs. The original roof-wall reinforcement detail created a constructibility problem that increased construction

(Continued)

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT	21. ABSTRACT SECURITY CLASSIFICATION
☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT. ☐ DTIC USERS	Unclassified

22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE (Include Area Code)	22c. OFFICE SYMBOL

DD FORM 1473, 84 MAR
83 APR edition may be used until exhausted.
All other editions are obsolete.

19. ABSTRACT (Continued).

costs. The objective of the experimental program described in this report was to evaluate alternate roof-wall joint details for the Keyworker blast shelter in an effort to improve constructibility without reducing structural capacity.

Three 1/4-scale reinforced concrete box-type models were statically tested under uniform water pressure in the 6-foot-diameter Small Blast Load Generator (SBLG) at WES. Two of the models (JD1 and JD3) were reinforced similarly to the original design except for the joint details. The third model (JD2) was similar to the proposed final shelter design as evaluated in the prototype tests conducted by WES. Based on the test results, detail JD2 is recommended for use in the final design.

PREFACE

1

CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

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

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

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
g's (standard free fall)	9.80665	metres per second squared
inches	25.4	millimetres
kilotons (nuclear equivalent of TNT)	4.184	terajoules
megatons (nuclear equivalent of TNT)	4.184	petajoules
pounds (force) per square inch	0.006894757	megapascals
pounds (mass)	0.4535924	kilograms
pound (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2,000 pounds, mass)	907.1847	kilograms

EVALUATION OF ALTERNATE ROOF-WALL DETAILS
FOR THE KEYWORKER BLAST SHELTER

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

At the time this study was initiated, several civil defense policy
options were being analyzed for protection of the nation's industrial
capability and key workers. One option under consideration called for
construction of blast shelters to protect key workers remaining in high-risk
areas during a national crisis. In support of this option the Federal
Emergency Management Agency (FEMA) tasked the U.S. Army Engineer Division,
Huntsville (HND) to develop keyworker shelter designs. The design required an
earth covered shelter to resist the radiation and blast effects of a 1-MT*
nuclear detonation at the 50-psi peak overpressure level. Personnel in the
Structures Laboratory of the U.S. Army Engineer Waterways Experiment Station
(WES) supported the HND with design calculations and design verification
experiments.

A preliminary structural design for the shelter was developed based on
computational procedures developed by Kiger, Slawson and Hyde (1984) in the
Shallow-Buried Structures Research Program at WES. Using a roof slab
thickness of 10 inches, a span of 11.3 feet, and limiting the mid-span
deflection to 7 inches, the minimum required tension and compression
reinforcement ratios were determined to be approximately 0.007. The roof slab
principal reinforcement extended into and 40 bar diameters down the walls.
This reinforcement embedment resulted in the requirement that the roof slab
reinforcement be placed into position prior to placement of concrete in the
walls. This design was experimentally evaluated using scale-model testing.

Six static tests and twelve dynamic tests were performed on approximately
1/4-scale models of the Keyworker blast shelter as reported by Slawson,
et. al. (1985). The effects of backfill type, concrete strength, and depth-
of-burial on structural response were investigated. The roof slab was

---

* A table of factors for converting non-SI units of measurement to SI (metric)
units is presented on page 4.

5

determined to be capable of resisting a 1-Mt weapon at approximately a 150-psi peak overpressure level with moderate damage. Permanent mid-span deflections varied from less than 0.1 inch to collapse in the test series. The failure mode for these models was the classical three-hinge mechanism with roof yield lines at mid-span and at the interior face of the walls. The roof-wall connection performed adequately during this test series. The joint details provided adequate embedment length to prevent principal roof reinforcement pullout and allowed large plastic rotations without premature failure. The joint details used in these test elements will be referred to throughout this report as the "original" joint details for the Keyworker blast shelter.

In the construction of a large number of shelters, it was important that the shelter design provide the required structural capacity at reasonable costs. The original roof-wall reinforcement detail created a constructibility problem that increased construction costs. The research reported herein investigates the modification of the roof-wall connection detail to allow casting of the structure walls without requiring the roof reinforcement to be in place. This facilitates removal of the wall forms from the inside of the shelter through the open roof and allows the placement of a construction joint near the top of the walls. This should decrease construction costs and the amount of time required for casting of the prototype structure.

1.2 OBJECTIVE

The objective of this test program was to evaluate alternate roof-wall joint details for the Keyworker blast shelter in an effort to improve constructibility without reducing structural capacity.

1.3 SCOPE

Three 1/4-scale reinforced-concrete box-type models were statically tested under uniform water pressure in the 6-foot-diameter Small Blast Load Generator (SBLG) at WES. Two of the models (JD1 and JD3) were reinforced similar to the original design except for the joint details. The third model (JD2) was identical to the models studied in the Yield Effects Tests (Slawson, Garner and Woodson, 1986). The principal reinforcement details in JD2 were

developed from tests investigating stirrup details (Woodson, 1985) and principal reinforcement details (Woodson and Garner, 1985). JD2 was constructed after the testing of JD1 and JD3; therefore, the results of the two tests aided in the design of the joint details in JD2. JD2 represented the recommended WES design and was statically tested to: (a) allow a comparison of the model's static response with the original structure's static behavior and (b) validate the structural design (including joint details) recommended for the Keyworker blast shelter.

7

CHAPTER 2

TEST DESCRIPTION

2.1 CONSTRUCTION DETAILS

The three test specimens were one-way, open-ended reinforced-concrete box elements. All models had a roof clear span of 33 inches, a clear height of 30 inches, and wall and floor thicknesses of 2.25 inches. Specimens JD1 and JD3 had roof thicknesses of 2.5 inches and roof effective depths of approximately 1.8 inches at mid-span. JD2 had a roof thickness of 2.25 inches and a roof effective depth of approximately 1.6 inches at mid-span.

The tension and compression steel ratios in the roofs, floors, and walls of JD1 and JD3 were approximately equal to 0.007 (neglecting additional dowel reinforcement at critical sections). Models JD1 and JD3 also contained stirrups in the roofs, floors, and walls. Based on previous test data (Woodson and Garner 1985), approximately 75 percent of the roof and floor principal reinforcement was placed in the tension zones, and 25 percent was placed in the compression zones in JD2. This required the use of reinforcement bent, such that it was placed in the exterior face of the roof or floor slab near the supports and in the interior face at mid-span. This was accomplished by the use of truss bars as shown in Figure 7. Tension and compression steel ratios in the roof and floor of JD2 at mid-span were approximately 0.012 and 0.004, respectively. Stirrups were placed only in the walls of JD2. Steel reinforcement ratios and effective depths for all of the elements are summarized in Table 1.

Construction details for test specimens JD1 and JD3 are presented in Figures 1 through 3. Joint details for both models allow complete construction of the walls prior to placement of roof reinforcement in a prototype structure. The only difference in the two models was that a 180-degree hook was used at the ends of the roof principal steel in JD1, and a 90-degree hook was used in JD3. The 90-degree hook extended only 0.75 inch (3 inches in prototype) into the wall below the bottom face of the roof slab. Additional reinforcement (mid-span dowels) was placed in the tension zone (bottom face) at the roof's mid-span in JD1 and JD3. The purpose of the mid-

8

span dowels was to increase the moment capacity of the roof slab at mid-span, thereby prohibiting premature failure at the roof mid-span and allowing a study of the joint detail behavior.

Construction details for test specimen JD2 are presented in Figures 4 through 7. Midspan dowels were not required in JD2 because the tension reinforcement ratio at mid-span was large enough to insure a ductile response. A 180-degree hook was used at the ends of the roof principal steel. Alternating wall principal reinforcement bars were bent 90 degrees and extended along the top face of the roof a length of about 30 percent of the roof's clear span. The joint also contained dowels between principal steel locations that reinforced the exterior face of the wall and extended into the interior face of the roof. The dowels helped in maintaining a balanced joint detail design by increasing the moment capacity of the wall near the joint to prevent wall failure prior to roof failure.

Photographs taken during construction of the models are shown in Figures 8 through 13.

2.2 STRUCTURAL MATERIAL PROPERTIES

2.2.1 Concrete

The concrete mix was designed to have a 28-day compressive strength of 3,000 psi. The mix consisted of Type I Portland cement obtained from a local commercial supplier, a natural siliceous sand fine aggregate, and a crushed limestone coarse aggregate with a 3/8-inch maximum diameter. Compressive tests on concrete cylinders yielded mean values of 28-day strength for JD1, JD2, and JD3 of 2,520, 2,790, and 2,940 psi, respectively. The mean values for the test day compressive strength of the concrete in models JD1, JD2, and JD3 were 2,680, 3,150, and 2,940 psi, respectively.

2.2.2 Reinforcing Steel

The principal reinforcing steel used in models JD1 and JD3 was ASTM (1984) Grade 60 No. 2 rebar. The principal reinforcing in model JD2 consisted of 0.195-inch diameter (D3) deformed wire that had been heat treated to model

Grade 60 rebar. Stirrups and temperature steel in all models consisted of heat-treated 0.11-inch-diameter (D1) deformed wire. Results of tensile tests on the reinforcement are presented in Table 2.

2.3 LOADING DEVICE

The static tests were performed in the SBLG at WES. The device consists of a Central Firing Station (CFS) (a massive posttensioned concrete reaction structure) and a test chamber. The test chamber consists of two 5-foot 11-3/4-inch inside diameter steel rings stacked on a movable platen to form a cylindrical chamber 6-feet deep. The loading device can be used for both static and dynamic tests. Slowly applied (static) loading can be achieved by sealing the exhaust ports and pumping water into the chamber. A detailed description of the use of the static test device is given by Huff (1969). The three tests discussed in this report were tested statically in the configuration shown in Figure 14.

2.4 INSTRUMENTATION

Each model was instrumented for rebar strain and roof displacement. In addition, water loading pressure and soil-stress measurements were made. The data were recorded on a Sangamo Sabre III FM magnetic tape recorder. The data were digitized and plotted with water pressure as the ordinate. Figures 15 and 16 show the instrumentation layout for the test specimens.

Soil-stress measurements were made in the free field at a depth of 1/2-inch below the roof's surface at a distance of 12 inches from the model's wall. Two Kulite SE Model VQV-080-LR soil-stress gages (SE1 and SE2) having ranges of 200 psi were used in each test.

Reinforcement steel strains were measured on the inside (EI) and outside (EO) of principal steel using Micro-Measurements single-axis, metal film, 350-ohm, temperature-compensated, 1/8-inch strain gages. The gages were model number EA-06-125BZ-350. In addition, dowel strains (ED) were measured in tests JD1 and JD3, and truss bars strains (E) were measured in test JD2.

Two Celesco PT-101 displacement transducers were used in each test, one at mid-span (D1) and one at quarter-span (D2). The transducers had a maximum allowable working range of 10 inches.

Two Kulite Model HKMS-375, 500-psi pressure gages (P1 and P2) were mounted in the test chamber to measure the water pressure applied to the roof's surface.

2.5 TEST PROCEDURE

The 6-foot-diameter SBLG (Figure 14) was used to statically test each model. Prior to placement of the model in the SBLG, sand was placed and compacted to a level of 3 feet 6-1/4 inches below the top of the chamber. The box element was placed on the compacted sand, and instrumentation gages were connected to cables passing through steel endplates and the test chamber's wall to a junction terminal box. The steel plates covered the open ends of the model, and thrust rods were installed between the two plates to keep the plates from bearing against the structure. Silicon caulk filled the gaps between the steel plates and the model.

Sand backfill was compacted in 6-inch lifts to the roof surface level (approximately 8 inches below the top of the test chamber). A 1/4-inch-thick neoprene membrane was placed over the model and clamped to a ring along the inside face of the chamber to waterproof the structure and backfill during the test. The lid was placed on the chamber and the assembly was moved on tracks into the CFS. Water was pumped into the chamber above the structure, pushing the chamber's lid against the CFS and applying a slowly increasing uniform load to the roof's surface. The displacement gage located at mid-span and one of the pressure gages were monitored on an X-Y plotter during the test. The test was terminated when the incipient collapse deflection of the roof slab was reached or when the water pressure suddenly decreased due to rupture of the neoprene membrane.

11

Table 1. Summary of roof reinforcement ratios including dowel
reinforcement at critical sections.

| Model | Effective Depth, d (inches) | | Reinforcement Ratio | | | |
| | Mid-Span | Support | Mid-Span | | Support | |
			Top	Bottom	Top	Bottom
JD1	1.8	2.0	0.0074	0.0140	0.0126	0.0067
JD2	1.6	1.6	0.0040	0.0120	0.0160	0.0080
JD3	1.8	2.0	0.0074	0.0140	0.0126	0.0067

Note:

The reinforcement ratios were calculated for each face of the roof slab
section at the mid-span and at the face of the support by the following:
A/bd, where:

A = the total area of steel in the layer
b = the length of the structural model (33 inches)
d = the distance from the compression face of the slab to the centroid of the
tensile reinforcement

Table 2. Results of tensile tests performed on reinforcing steel.

Specimen	Yield Stress ksi	Ultimate Stress ksi
D1 Deformed Wire	60.4	62.9
(0.11-inch dia)	67.7	71.2
	59.8	63.7
	60.8	63.4
	55.7	58.9
(mean)	(60.9)	(64.0)
D3 Deformed Wire	59.9	66.5
(0.195-inch diam.)	66.8	75.0
	68.2	74.4
	74.0	79.2
	70.4	77.0
(mean)	(67.9)	(74.4)
No. 2 Rebar	73.7	78.7
(0.25-inch diam.)	58.3	63.0
	73.8	78.8
	72.1	77.2
	68.4	73.6
(mean)	(69.3)	(74.3)
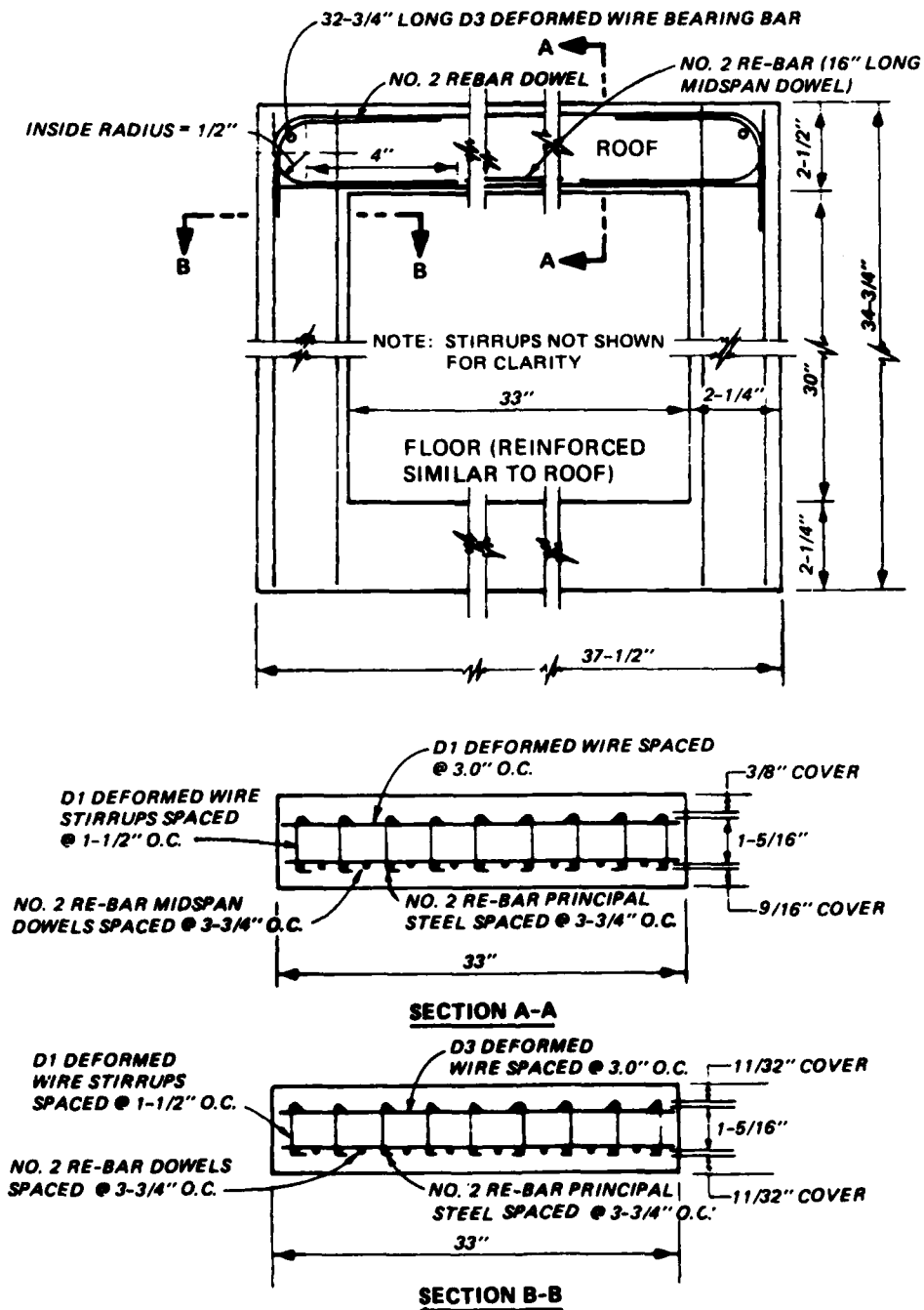
Figure 1. Reinforcement details for JD1.

14

32-3/4" LONG D3 DEFORMED WIRE BEARING BAR

A

NO. 2 RE-BAR DOWEL

NO. 2 RE-BAR (16" LONG) MIDSPAN DOWEL

ROOF

1/2" INSIDE RADIUS

2-1/2"

3-3/4"

B   B

A

NOTE: STIRRUPS NOT SHOWN FOR CLARITY

30"

34-3/4"

33"   2-1/4"

FLOOR (REINFORCED SIMILAR TO ROOF)

2-1/4"

37-1/2"

D1 DEFORMED WIRE SPACED @ 3.0" O.C.

3/8" COVER

D1 DEFORMED WIRE STIRRUPS SPACED @ 1-1/2" O.C.

1-5/16"

NO. 2 RE-BAR PRINCIPAL STEEL SPACED @ 3-3/4" O.C.

9/16" COVER

NO. 2 RE-BAR MIDSPAN DOWELS SPACED @ 3-3/4" O.C.

33"

SECTION A-A

D3 DEFORMED WIRE SPACED @ 3.0" O.C.

11/32" COVER

D1 DEFORMED WIRE STIRRUPS SPACED @ 1-1/2" O.C.

1-5/16"

NO. 2 RE-BAR DOWELS SPACED @ 3-3/4" O.C.

11/32" COVER

NO. 2 RE-BAR PRINCIPAL STEEL SPACED @ 3-3/4" O.C.

33"

SECTION B-B

Figure 2. Reinforcement details for JD3.

15
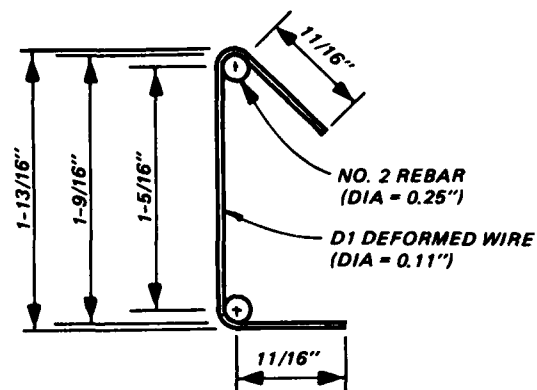
a. ROOF AND WALL SHEAR STIRRUP DETAIL
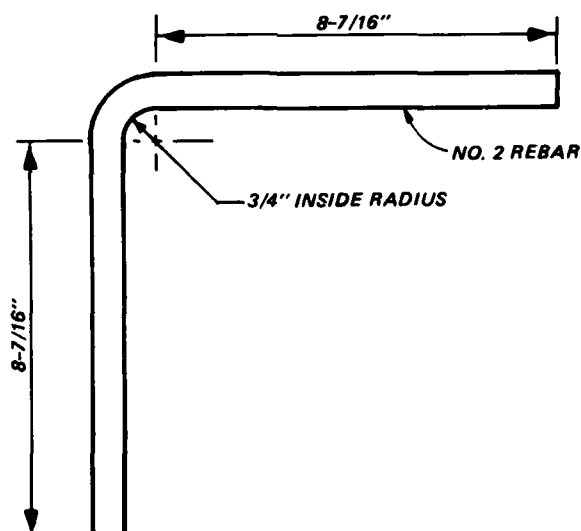
b. ROOF-WALL CONNECTION DOWEL

Figure 3.   Reinforcement fabrication details for JD1 and JD3.

16

D3 DEFORMED WIRE
BEARING BAR

A

ROOF

2-1/4"

B   B   A

33"   2-1/4"

FLOOR
(REINFORCED
SIMILAR TO ROOF)

37-1/2"

D1 DEFORMED WIRE
SPACED @ 3.0" O.C.

1/2" COVER

1"

2 BENT D3 DEFORMED
WIRE BARS SPACED @ 4-7/8" O.C.

1/2" COVER

33"

STRAIGHT D3 DEFORMED WIRE
SPACED @ 4-7/8" O.C.

SECTION A-A

D1 DEFORMED
WIRE STIRRUPS
SPACED @
1-1/2" O.C.

D1 DEFORMED WIRE
SPACED @ 3.0" O.C.

1/2" COVER

1"

D3 DEFORMED WIRE
PRINCIPAL STEEL
SPACED @ 2-7/16" O.C.

1/2" COVER

33"

D3 DEFORMED WIRE
DOWEL SPACED @ 2-7/16" O.C.

SECTION B-B

Figure 4.   Reinforcement details for JD2.

17

D3 BEARING BAR

DOWEL

1/2"

1/2"

1/2"

1-9/32"

1"

1-19/32"

1/2"

9-7/8"

9-7/8"

a. STRAIGHT PRINCIPAL STEEL LOCATION

2-1/4"

9-7/8"

6-19/32"

1/2"

1/2"

D3 BEARING BAR

1-9/32"

1-19/32"

1/2"

DOWEL

1/2"

9-7/8"

1/2"

1/2"

NOTE: BARS NOT DRAWN AT EXACT DEPTHS FOR CLARITY

b. BENT PRINCIPAL STEEL LOCATION
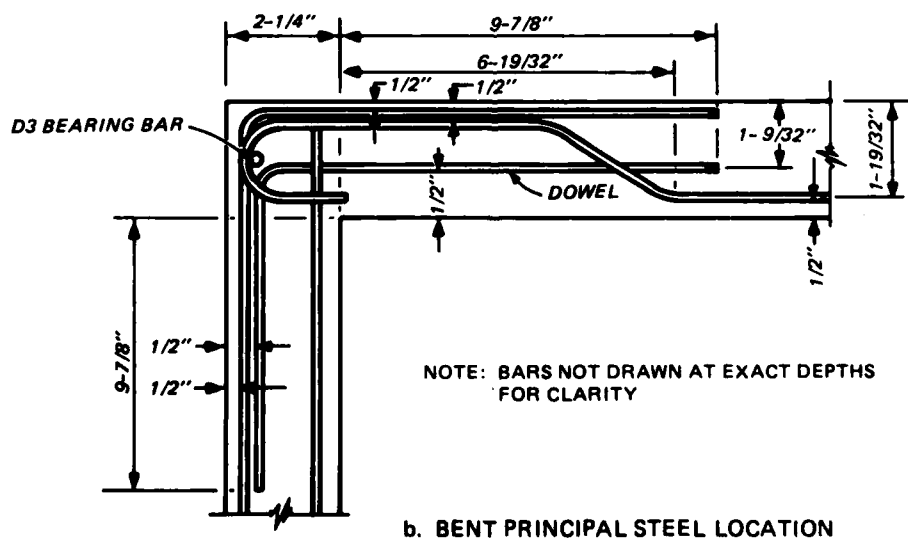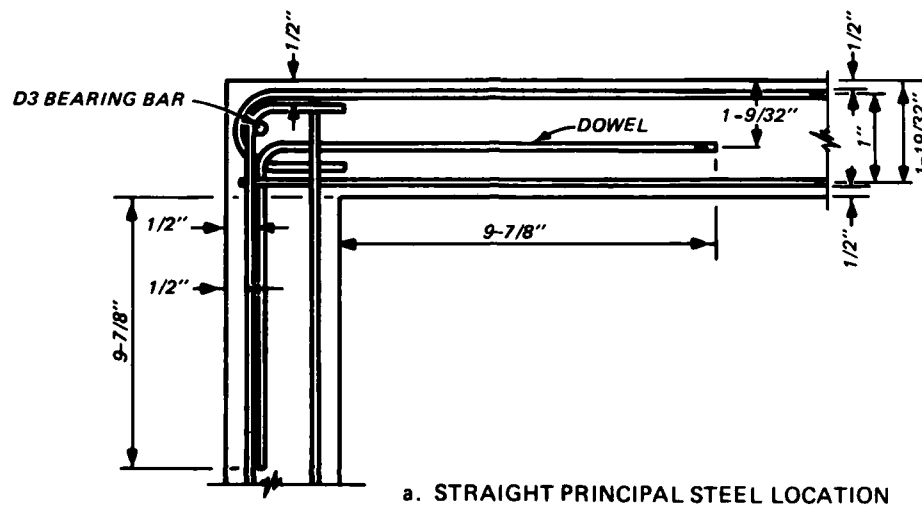
Figure 5. Reinforcement placement details for JD2.
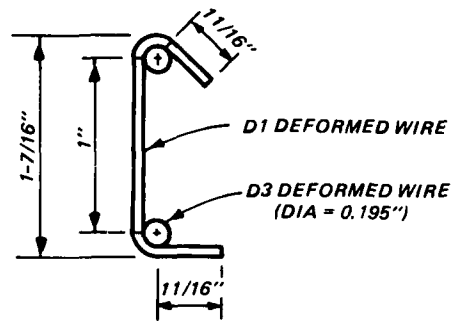
a. WALL SHEAR STIRRUP
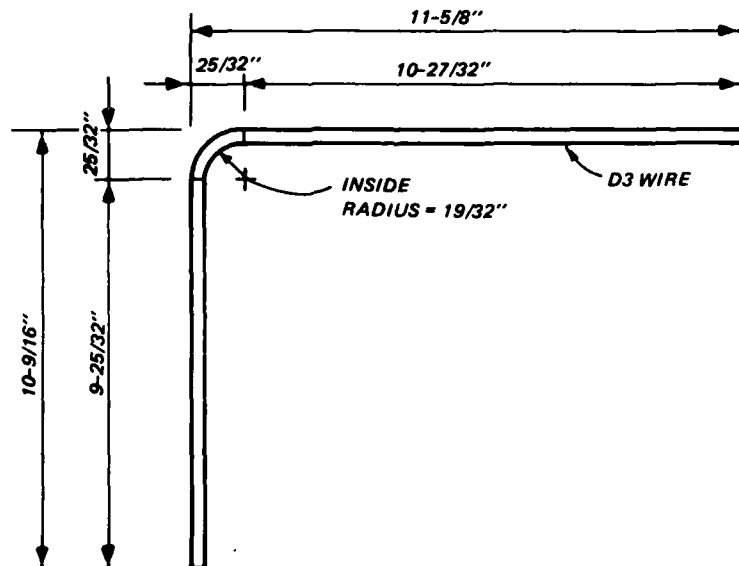
b. ROOF - WALL CONNECTION DOWEL DETAIL

Figure 6.   Reinforcement fabrication details for JD2.

19
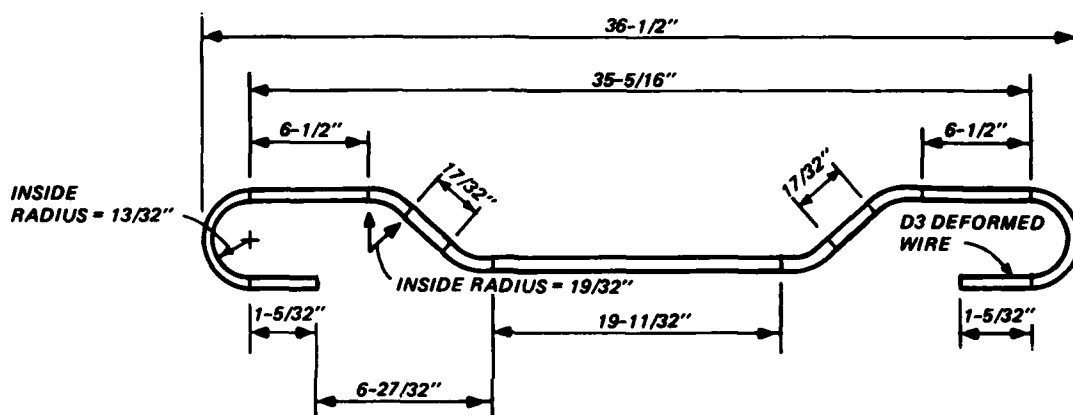
JD2 BENT BAR DETAIL

Figure 7. Truss bar fabrication details for JD2.
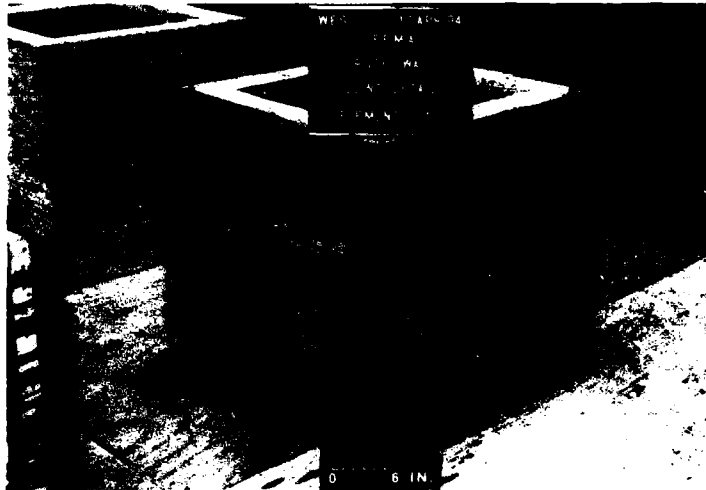
Figure 8. Construction of model JD1.
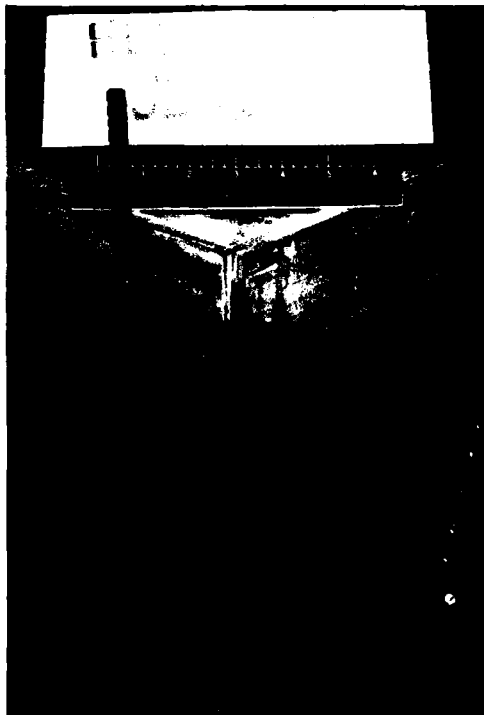
Figure 9. Close-up view of JD1 joint detail.

21
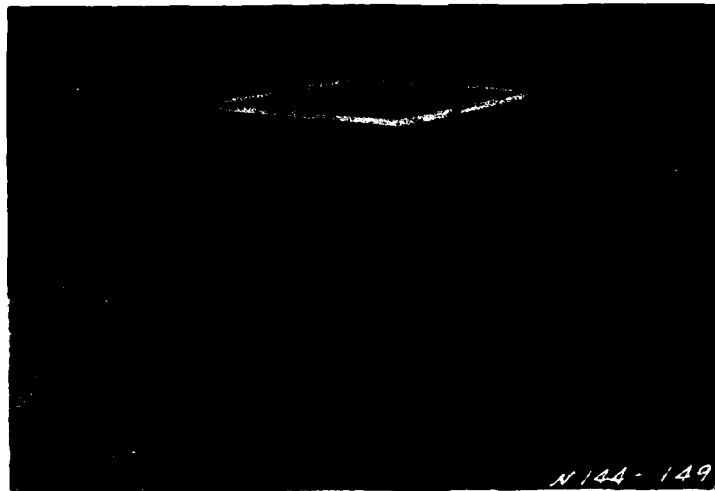
Figure 10.   Construction of model JD2.
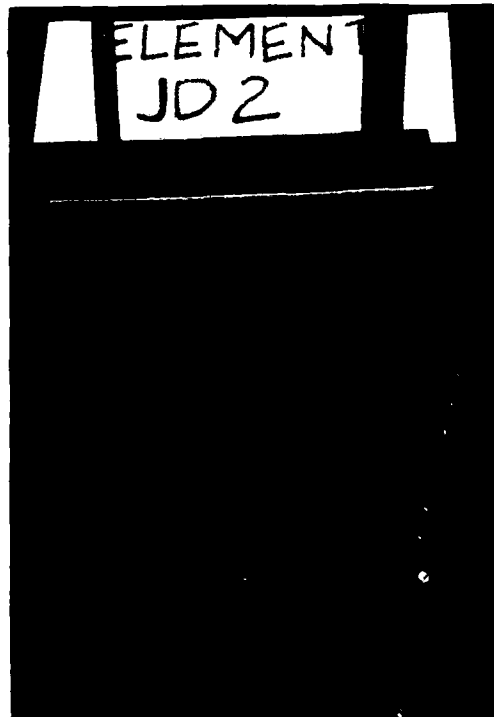
Figure 11.   Close-up view of JD2 joint detail.

22
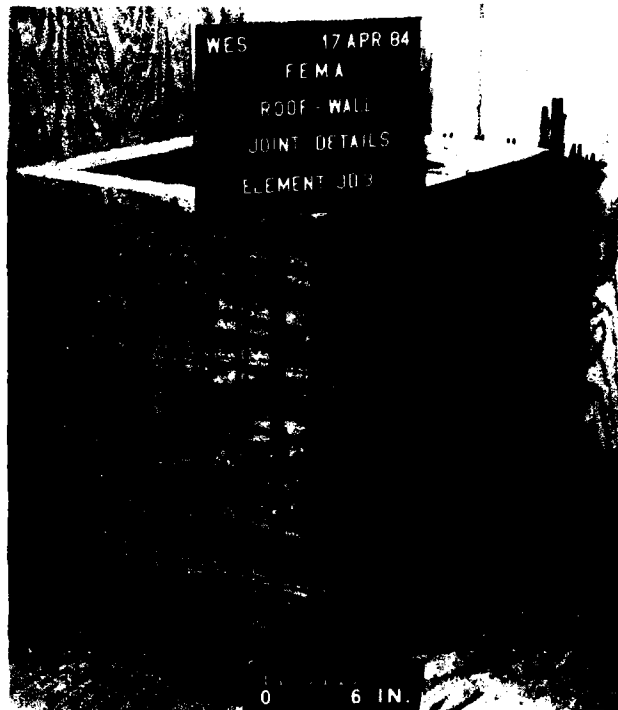
Figure 12. Construction of model JD3.
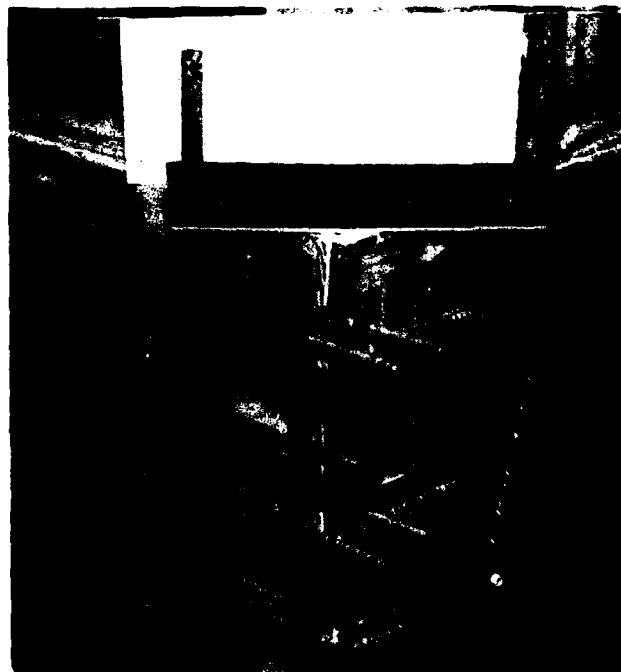
Figure 13. Close-up of JD3 joint detail.

23
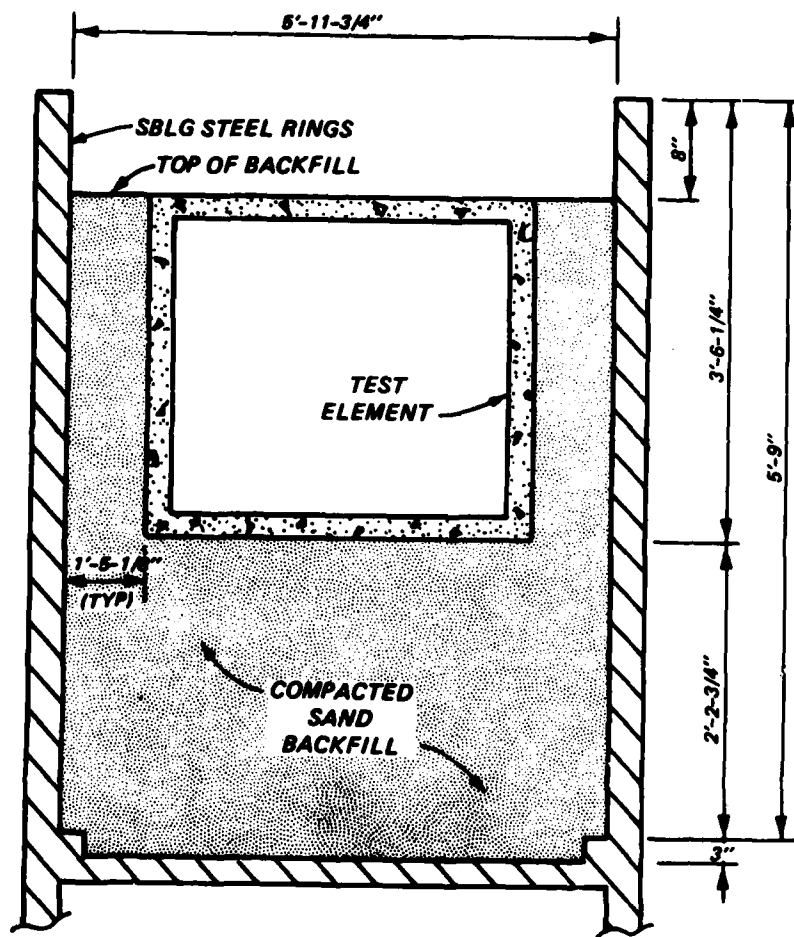
TEST CONFIGURATION

Figure 14.   Test configuration.

24

a. JD1 STRAIN - GAGE LOCATIONS

EO - STRAIN GAGE ON OUTER PRINCIPAL STEEL
EI - STRAIN GAGE ON INNER PRINCIPAL STEEL
ED - STRAIN GAGE ON DOWEL
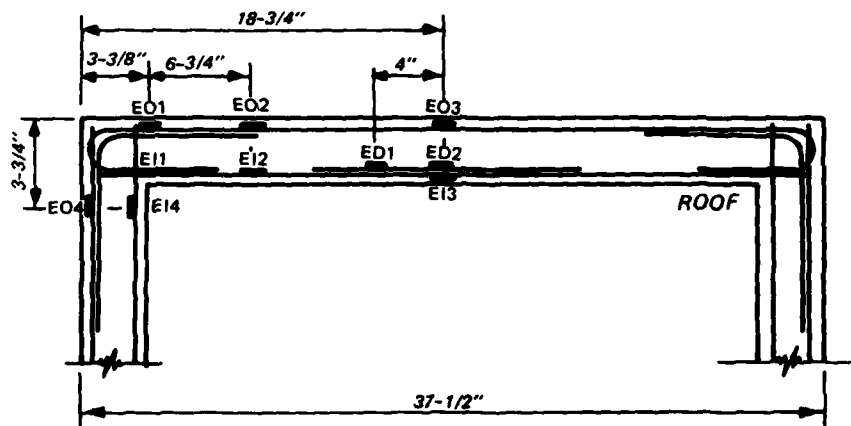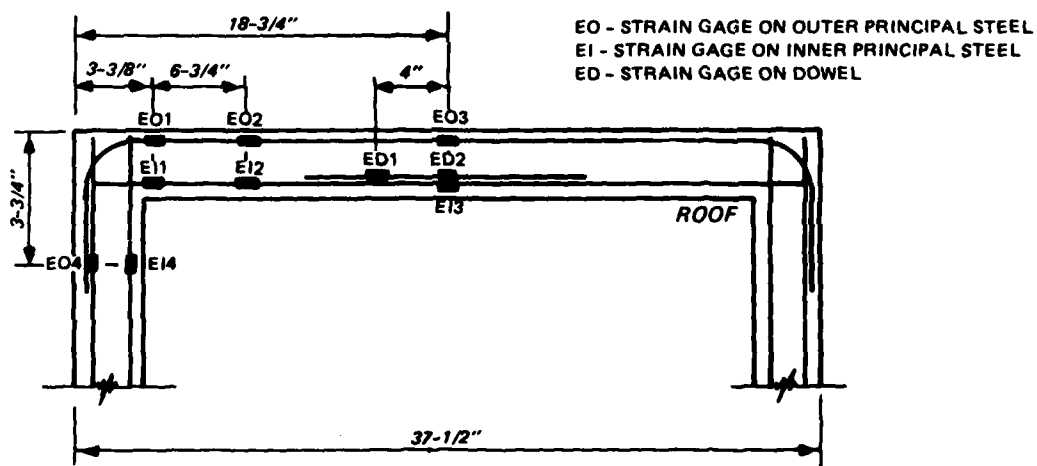
b. JD3 STRAIN - GAGE LOCATIONS

Figure 15. Strain gage locations for JD1 and JD3.

25
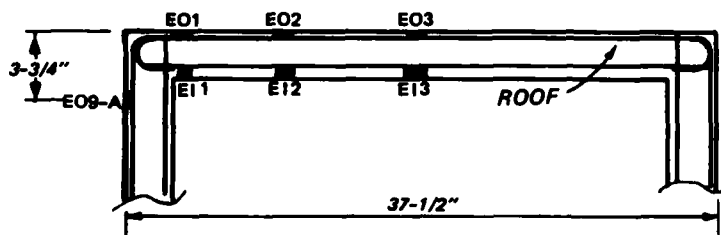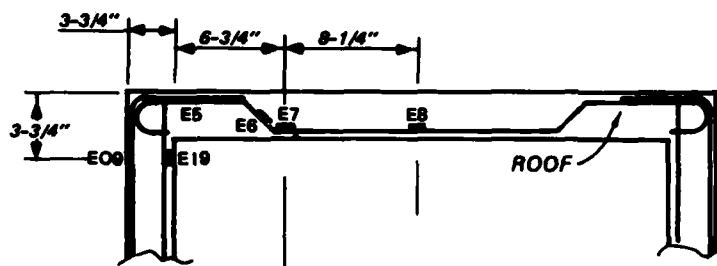
LEGEND

E – STRAIN GAGE ON
     BENT PRINCIPAL STEEL

EO – STRAIN GAGE ON
      OUTER PRINCIPAL STEEL

EI – STRAIN GAGE ON
      INNER PRINCIPAL STEEL

JD2 STRAIN GAGE LOCATIONS
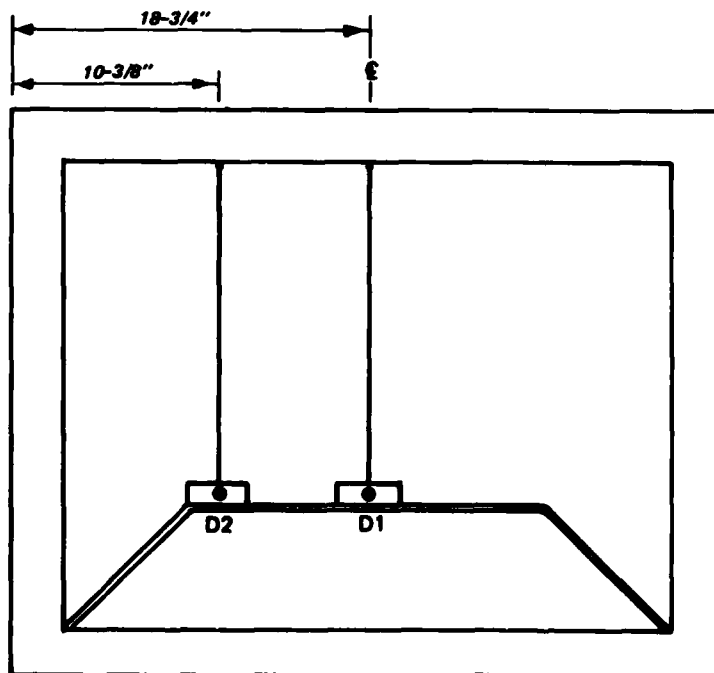
TYPICAL ROOF DEFLECTION GAGE LOCATIONS

Figure 16.   Strain gage locations for JD2 and
typical deflection gage locations.

26

CHAPTER 3

TEST RESULTS

3.1 GENERAL

Test results are presented in this chapter and recovered data are
presented in Appendix A. A general description of the data produced and of
the structural performance of each model is presented herein. Further
discussion and analyses are presented in Chapter 4. A recovered data summary
for the three tests are presented in Table 3.

3.2 LOAD-DEFLECTION BEHAVIOR

The flexural strength of a restrained slab is enhanced by compressive
membrane forces, causing the ultimate capacity to be greater than that
calculated using classical yield-line theory. Compressive membrane behavior
of slabs is documented by Park and Gamble (1980) and Guice (1986a, 1986b).
Figure 17 shows the load-deflection relationship for laterally restrained one-
way slabs. As the load is increased from A to B, with the help of compressive
membrane forces, the slab reaches its enhanced ultimate load at point B. With
an increase in deformation past point B, the compressive membrane enhancement
is reduced by a P-Δ effect until point C is reached. Normally point C
corresponds to the yield-line capacity at a deflection that varies from the
effective depth to the slab thickness for fixed-fixed slabs similar to these
test models. Beyond point C, the tensile reinforcement acts as a catenary to
carry the load with full depth cracking of the concrete at the mid-span region
of the slab.

Ultimate resistances for JD1, JD2, and JD3 were 55, 38, and 63 psi,
respectively. Deflection at ultimate resistance ranged from 0.55 to 0.75 inch
with an average of 0.67 inch. The average ratio of deflection at ultimate
resistance-to-slab thickness for the tests was 0.27.

Enhanced tensile membrane behavior was observed in Test JD2. The maximum
pressure in the tensile membrane region was 34 psi. The test was stopped at
incipient collapse so that the roof failure mechanism could be observed. End-
of-test pressures for Test JD1 and Test JD3 were 36 and 39 psi, respectively.

No tensile membrane enhancement in slab resistance was noted for tests JD1 and JD3. Test JD1 was stopped because of large deflections to the roof slab. The neoprene membrane was ruptured in Test JD3, thus, the sudden decrease in pressure.

Experimental pressure versus deflection at ultimate resistance and at maximum resistance in the tensile membrane region for the tests are given in Table 4.

3.3 STRUCTURAL DAMAGE

The structural models were uncovered and examined immediately after testing. In all three cases, the roofs cracked at the supports and at mid-span on the top surface and at mid-span on the bottom surface. A careful examination of the models after each test provided significant information about overall response. Crack patterns and failures of steel reinforcement were the most significant characteristics that were observed. All visible cracks in the slabs were highlighted with markers. Records were kept of the widths and depths of the crack bands at mid-span in the tension and compression zones, at the supports in the tension zones, and in the wall supports in the tension zones.

Maximum permanent roof mid-span deflection for Test JD1 was 5-1/8 inches, exposing reinforcing at mid-span and at the supports. Cracks at mid-span and at the roof supports ranged in depth from 1/4-inch to the thickness of the roof. Although the band of cracking at mid-span was primarily narrow on the top of the roof slab, the bottom surface revealed additional hairline cracks across the roof span. Wall cracks occurred near the top of both supports and exposed a large percentage of principal reinforcement from the roof-wall connection. See Figures 18 through 21 for posttest photographs of Test JD1.

Test JD2 resulted in a permanent mid-span roof deflection of 5-1/2 inches. Significant cracking occurred at mid-span and at the supports. The crack at mid-span was the depth of the roof across the entire structure. All roof reinforcing at mid-span was exposed and all bottom mid-span bars were broken, except one. Although the mid-span crack was much more extensive than that of Test 1, there were fewer hairline cracks across the top and bottom surfaces of the roof. The width of the crack zone at mid-span

28

ranged from 2-1/2 to 11-1/2 inches on the top surface of the roof. Cracks ranging from 1/2- to 1-1/2-inches deep occurred at the supports. Posttest photographs of Test JD2 are shown in Figures 22 through 25.

The response of the model in Test JD3 was very similar to the model in Test JD1. Permanent roof deflection was 5-1/4 inches. The supports were severely damaged as much cracking and exposure of reinforcement was observed. The mid-span crack zone was much more widespread across the top and bottom surfaces of the roof. Cracks occurred heavily at the top of the walls as well (See Figures 26 through 29). The roof-wall joints in tests JD1 and JD3 failed by diagonal splitting. The joints of test JD2 did not fail during the test.

Table 3. Data Summary for Joint Detail Tests.

Gage	Location	Test JD1	Test JD2	Test JD3
Soil Stress	SE-1			
	SE-2			
Strain	EO-1	+		
	EO-1S	*	*	
	EI-1			
	EI-1S	*	*	
	EO-2			
	EI-2			
	EO-3			
	EO-3S		*	*
	EI-3			
	EI-3S		*	
	EO-4		*	
	EI-4		*	
	EO-9	*		*
	EO-9A	*		*
	EI-9	*		*
	EI-9A	*		*
	ED-1	+	*	
	ED-2		*	
	E-5	*	+	*
	E-5A	*		*
	E-6	*		*
	E-6A	*		*
	E-7	*		*
	E-7A	*		*
	E-8	*		*
	E-8A	*		*
Deflection	D-1			
	D-2			

Notes:

1. * = gage not used
2. + = gage failed during test
3. For Test JD3, the following pairs of records are believed to be reversed: EO-1, EI-1, EO-3, EI-3, EO-4, EI-4.
4. Gage locations ending in an "S" are redundant measurements.
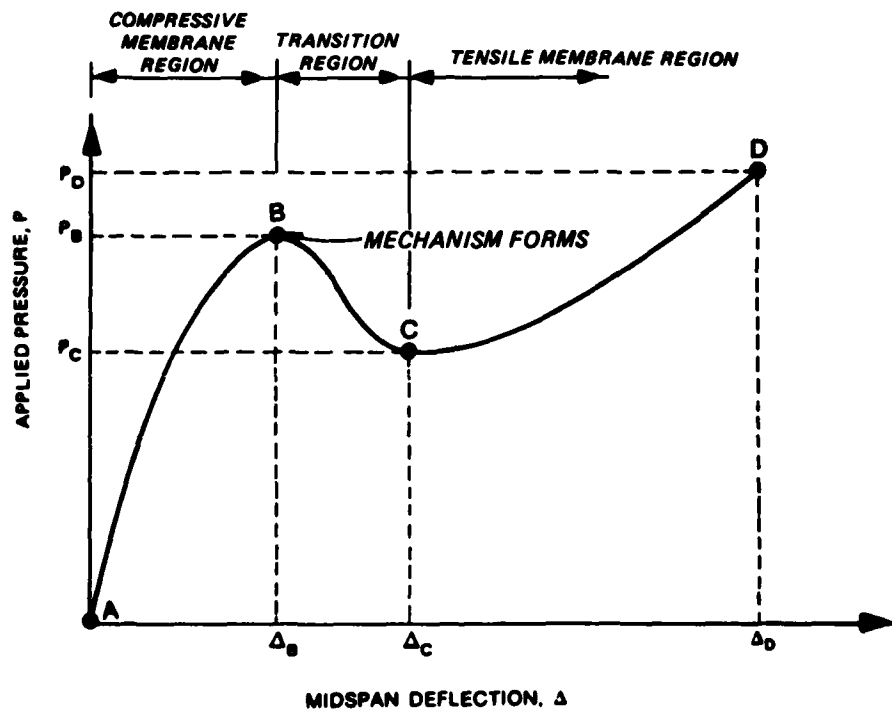
Figure 17. Load-deflection relationship for a one-way slab with restrained ends.
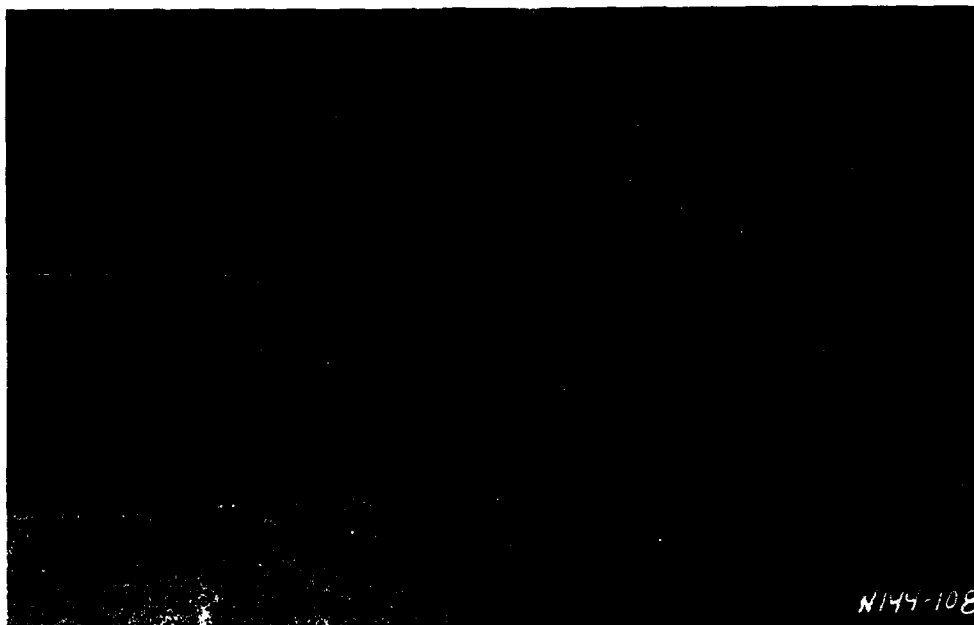
Figure 18. Posttest view of the roof surface of JD1.
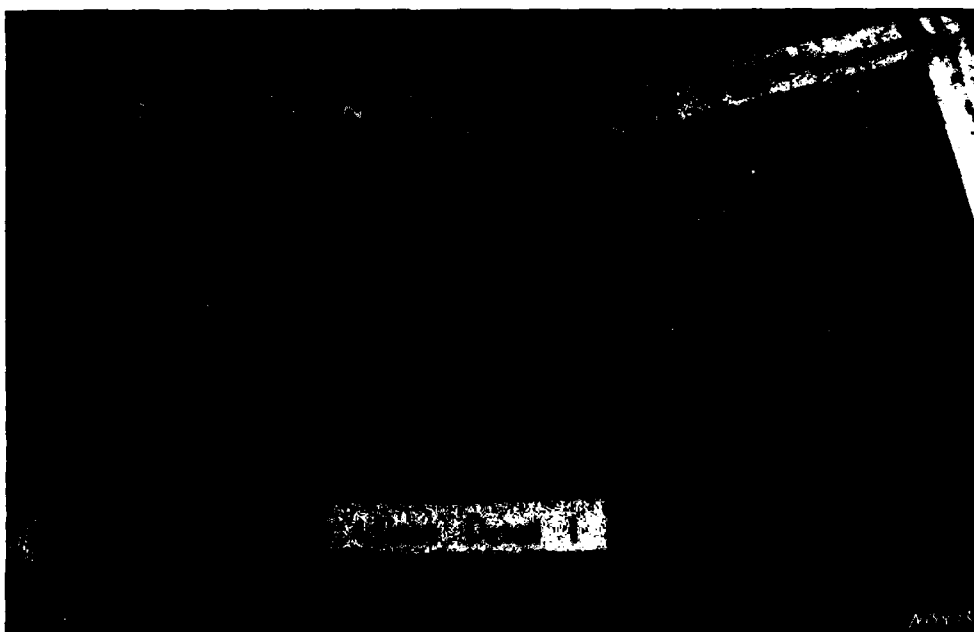
Figure 19. Posttest view of the bottom of the roof slab of JD1.
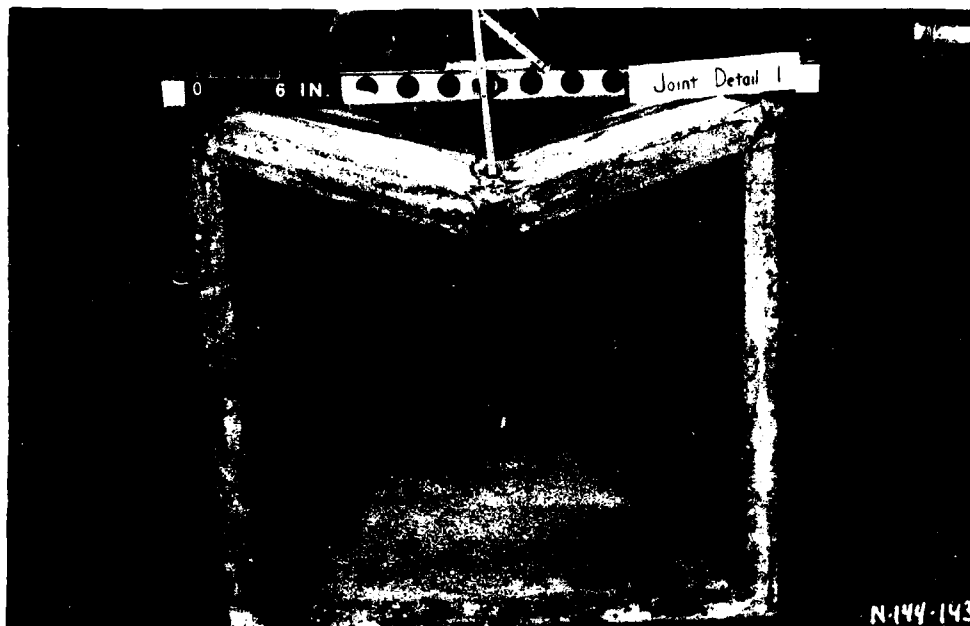
Figure 20. Posttest end view of JD1.
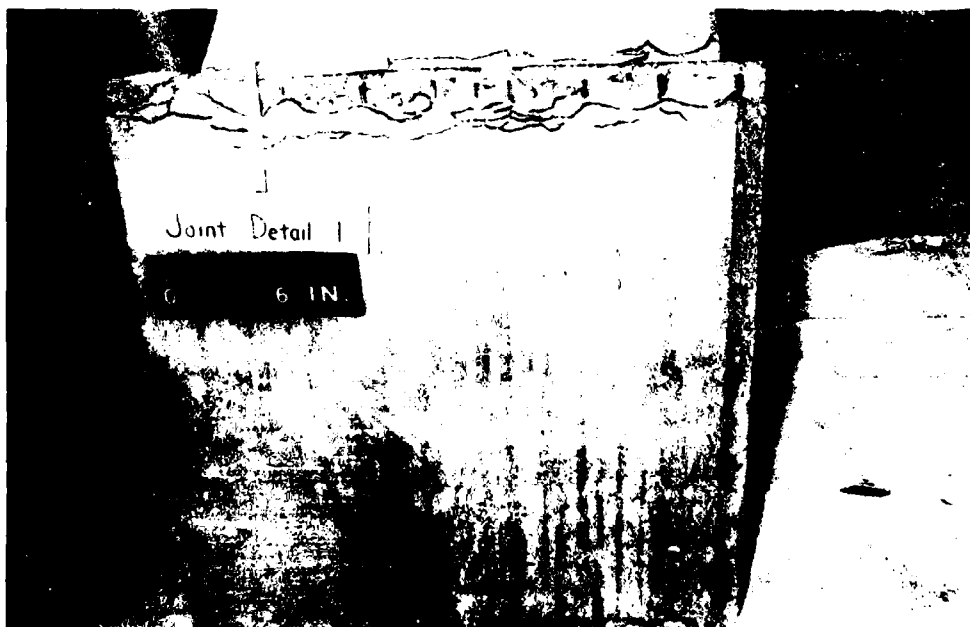
Figure 21. Posttest side view of JD1.

33
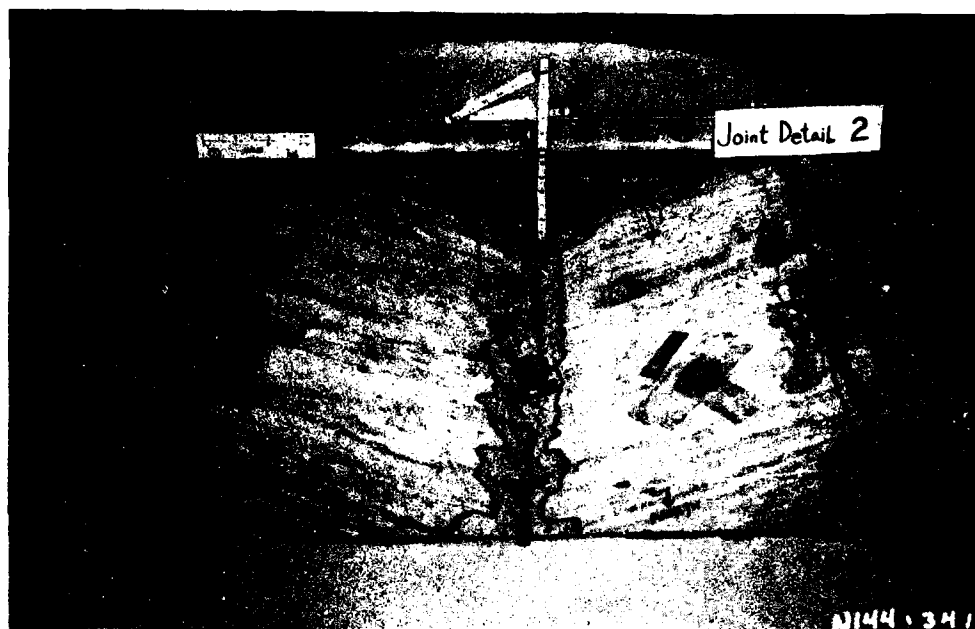
Figure 22. Posttest view of the roof surface of JD2.
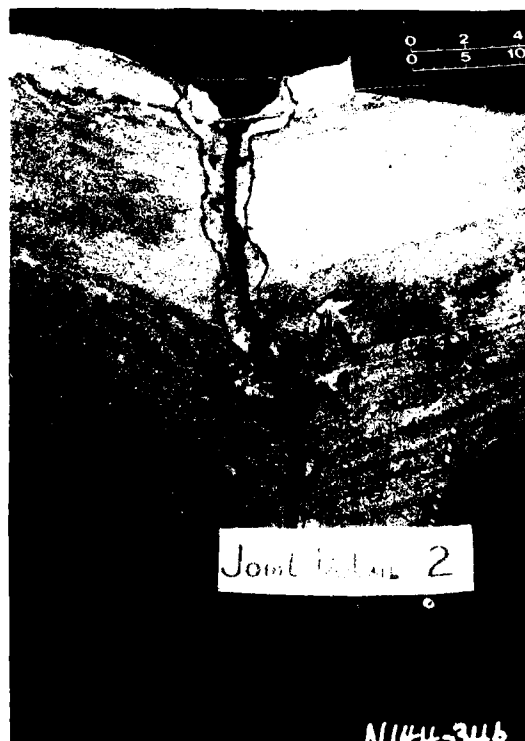
Figure 23. Posttest view of the bottom of the roof slab of JD2.
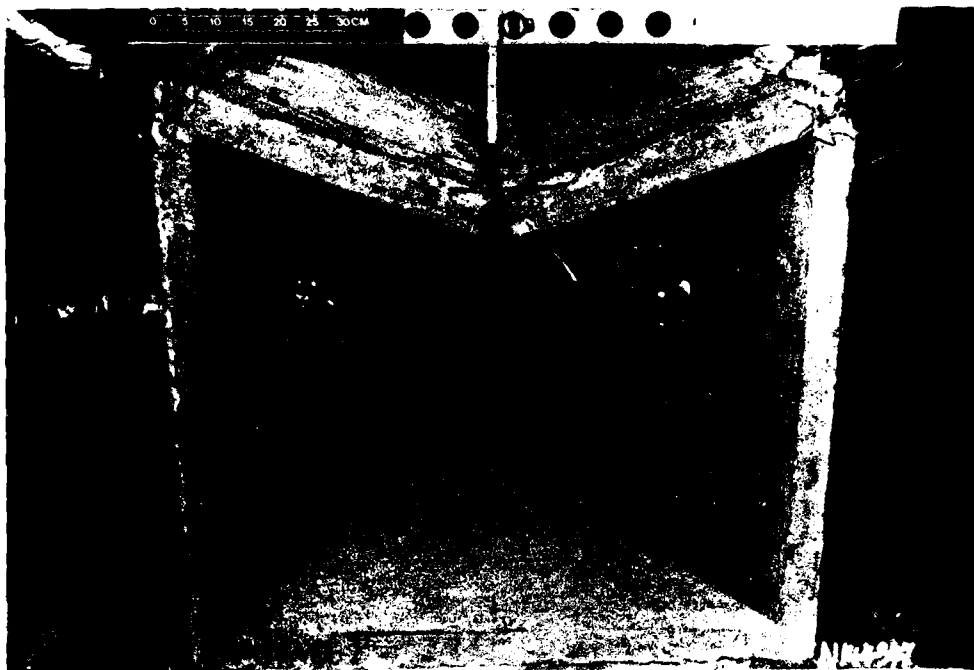
Figure 24. Posttest end view of JD2.
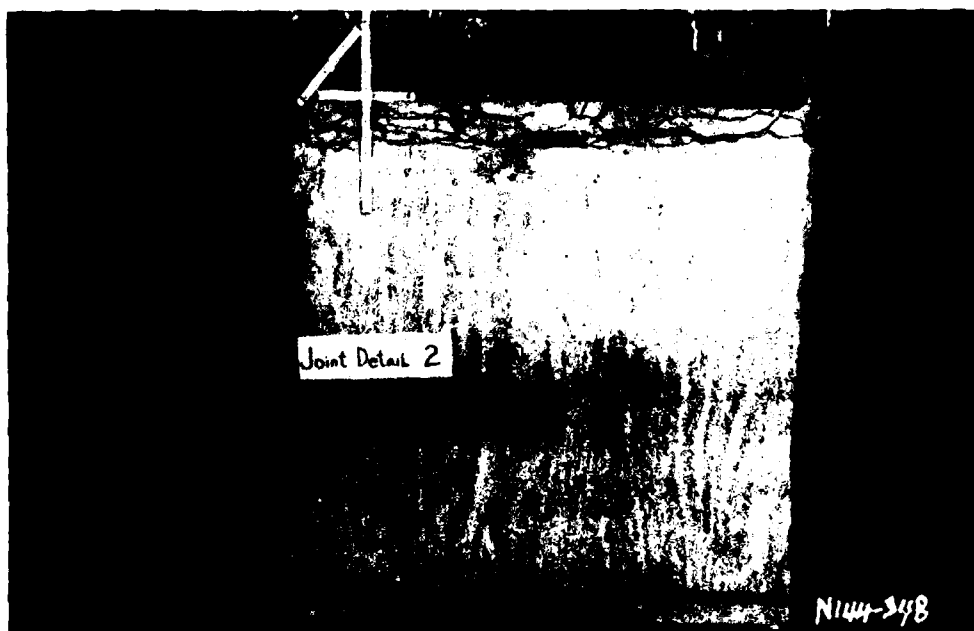
Figure 25. Posttest side view of JD2.

35
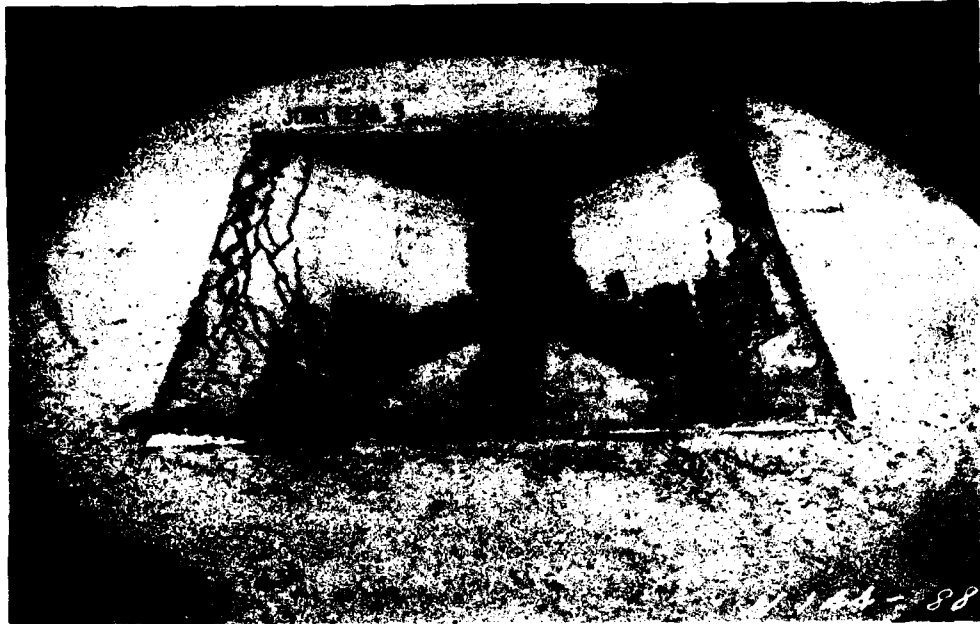
Figure 26. Posttest view of the roof surface of JD3.
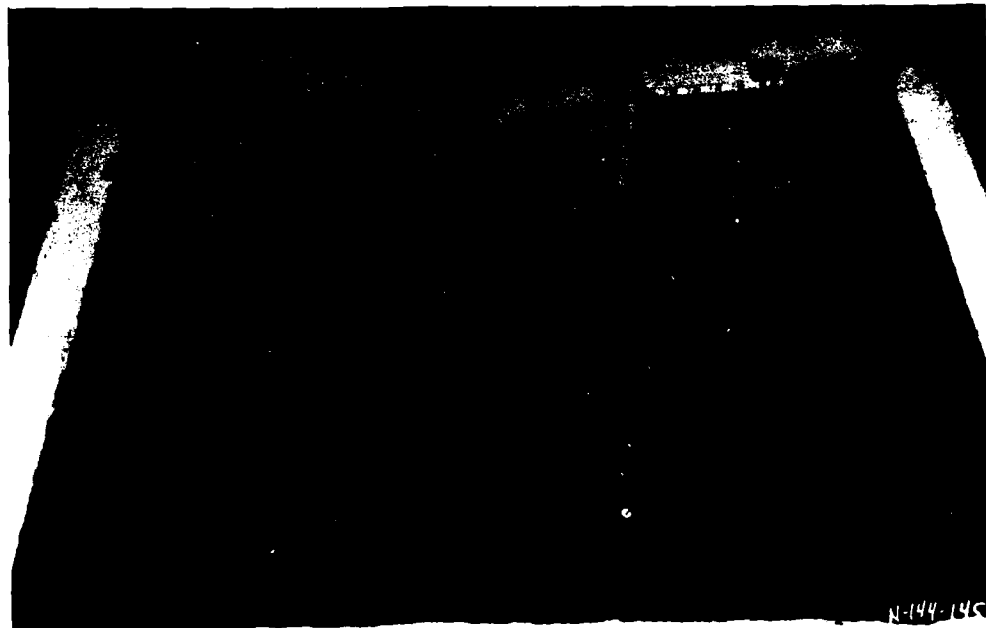
Figure 27. Posttest view of the bottom of the roof slab of JD3.
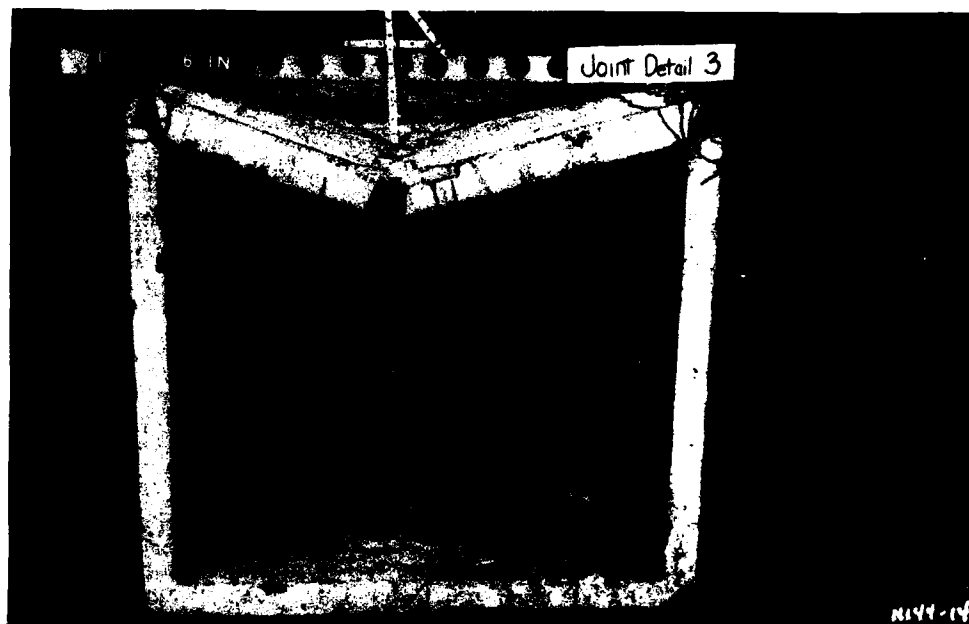
Figure 28.   Posttest end view of JD3.
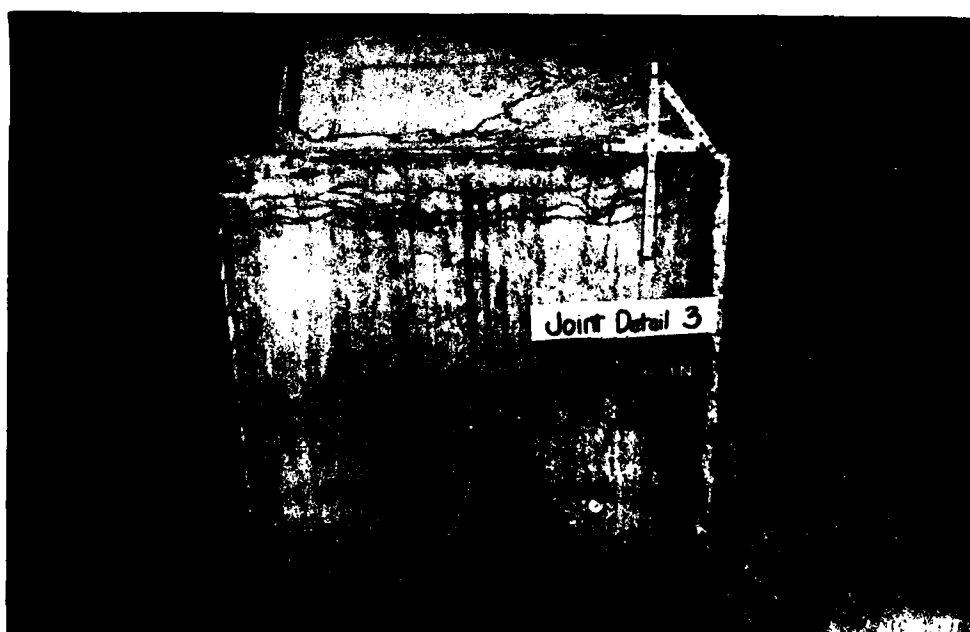
Figure 29.   Posttest side view of JD3.

CHAPTER 4

ANALYSIS

4.1 STRAIN GAGE DATA

In test JD1, the tensile bars at mid-span yielded at a water pressure of 43 psi, while top reinforcing yielded during compressive membrane decay at 54 psi. Dowels at mid-span yielded at 25 psi. The strain gage in the tension zone at the support failed, but the yield lines that were formed indicated yielding in the tensile bars. From observation of wall reinforcing strain gage data (EO-4 and EI-4), the tensile steel yielded after the structure reached its ultimate compressive membrane capacity, indicating that the mid-span yielded first.

Strain gage data in Test JD2 indicated that the straight tensile steel at supports yielded at a water pressure of 37 psi. Bent tensile bars at supports yielded at 15 psi, indicating that the first hinge was formed there. The bent tensile bars yielded at approximately 35 psi at quarter span and at mid-span. Straight tensile bars yielded just prior to ultimate capacity at mid-span, but did not yield at quarter-span. The dowels that were placed at bent bar locations in the wall did not yield. However, the tensile bars in the wall at the support that were placed at straight bar locations yielded approximately at ultimate capacity. The high tensile reinforcement ratio and the use of truss bars did allow the roof to respond with tensile membrane behavior, even though the plastic hinge at the mid-span was concentrated in a narrow band. Yield lines were formed at the faces of the supports rather than through the roof-wall connections as in the model in Tests JD1 and JD3.

Test JD3 indicated that the principal tensile steel at mid-span yielded after ultimate capacity at a water pressure of approximately 62 psi. The mid-span dowels began yielding first at about 44 psi. Top reinforcing bars at mid-span yielded in compression just prior to ultimate capacity. The yield lines at the supports indicated that the straight tensile steel yielded although strain gages did not reflect this. Tensile bars at quarter-span also yielded but at a much larger deflection. Wall tensile steel near the roof-wall connection also showed no sign of yielding until after ultimate capacity was reached.

38

The structural models were loaded to ultimate capacity and in all cases concrete crushing and yielding of reinforcing steel existed. Since the effective depth of JD2 was smaller than that of JD1 and JD3, the ultimate capacity was considerably lower. However, the principal steel configuration used in JD2 allowed enhanced tensile membrane resistance to occur allowing this model to sustain large deflections with a very ductile response mode.

4.2 JOINT BEHAVIOR AND RELATED EFFECTS

The major difference between JD1 and JD3 was the hook detail that was used at the end of the principal steel in the roof. All other parameters were the same for the two models. The higher ultimate resistance of JD1 was not considered as significant (55 psi for JD1 versus 68 psi for JD2), and was partially due to the use of 180-degree hooks at the ends of the principal steel. The hooks that were used for model JD3 were 90-degree hooks. Both models were loaded to large deflections, yet neither offered any increase in resistance beyond ultimate capacity in the tensile membrane region. The roof-wall connections failed in both cases through the potential splitting zone. According to Park and Pauley (1975), the limiting flexural steel content for a knee joint subjected to a closing should be:

$$\rho = \frac{f_t'}{f_y} \text{ , where } f_t' = 6\sqrt{f_c'}$$

where

f_y = tensile yield strength of steel

f_c' = compressive strength of concrete

For values higher than this, a brittle splitting failure can occur causing the ultimate capacity to be reduced. The actual tensile steel content at the supports for JD1 and JD3 was 0.0126. The theoretical maximum value for the two models were 0.0051 and 0.0048, respectively, based on actual concrete and tensile steel strengths. This indicates that the full strength of the roof slab could not be obtained due to premature joint failure and explains why the joints failed diagonally through the roof-wall connection.

39

There were significant differences in the load-deflection behavior of the roof slabs and the roof-wall connection performance in these three tests. The roof-wall connections in Test JD1 and Test JD3 failed diagonally through the joint rather than at the interior face of the wall (as in JD2) due to the applied closing load. Models JD1 and JD3 failed in a three-hinge mechanism with cracking at the supports and additional cracking around the unsupported edges and throughout the tension zones. Few top reinforcing bars were broken at the supports or mid-span, though many of the bars were exposed by the cracks that were formed. The rotation of the slab at the supports due to the splitting of the joint (loss of rotational restraint) allowed the roof to deflect with a reduced plastic rotation of the mid-span hinge. This allowed spreading of the plastic hinge as evidenced by the wider band of mid-span cracking observed in these tests. The use of stirrups in these tests probably aided in the spreading of mid-span cracking due to concrete core confinement and load transfer between the top and bottom principal reinforcement.

The JD2 model contained no stirrups in the roof and had larger reinforcement spacing. The yielding of the roof-wall connections was primarily at the roof-wall interface rather than through the joint resulting in lower rotations allowed at small displacements. Plastic rotations of the mid-span hinge was concentrated in a narrow zone, as a result. Model JD2 failed in a classic three-hinge mechanism, with almost all of the tensile reinforcing broken at mid-span and almost no cracking between hinges. There was a large gap at mid-span where the concrete separated as the reinforcement failed in tension. Much of the tensile reinforcing was broken at the supports as well.

4.3 COMPARISON OF JOINT DETAIL MODELS WITH STRUCTURAL ELEMENT S3

Six structural elements were tested staticly in support of the Keyworker Blast Shelter Program by Slawson, et al, (1985). One type of element tested (S3) was similar to the joint detail models and was tested in a similar configuration with no sand cover (surface flush). The original joint details for the Keyworker shelter were used as the roof-wall connection for this element. The load-deflection behavior of element S3 was indicative of the response of the original design and thus is included for comparison with the

40

modified original design (JD1 and JD3) and the proposed design (JD2). The
actual day-of-test concrete strength for S3 was 5210 psi and the average
tensile strength for the reinforcing steel for the tests was 68.5 ksi.

Element S3 reached an ultimate capacity of 44 psi and the test resulted
in a permanent mid-span deflection of 4.5 inches. The pressure at point D
(end of tensile membrane region) was 21 psi. No tensile membrane capacity
enhancement was observed in this test. A comparison of the load-deflection
curves for the joint detail tests and test S3 is presented in Figure 30.
Although premature joint failure occurred in JD1 and JD3 the overall
performance of the roof slabs was similar to Test S3. The ultimate capacities
of models JD1 and JD3 were 35 and 20 percent greater, respectively, than that
of model S3 due to the additional tension zone reinforcement provided by
dowels. Test JD2 did result in enhanced tensile membrane behavior as shown.
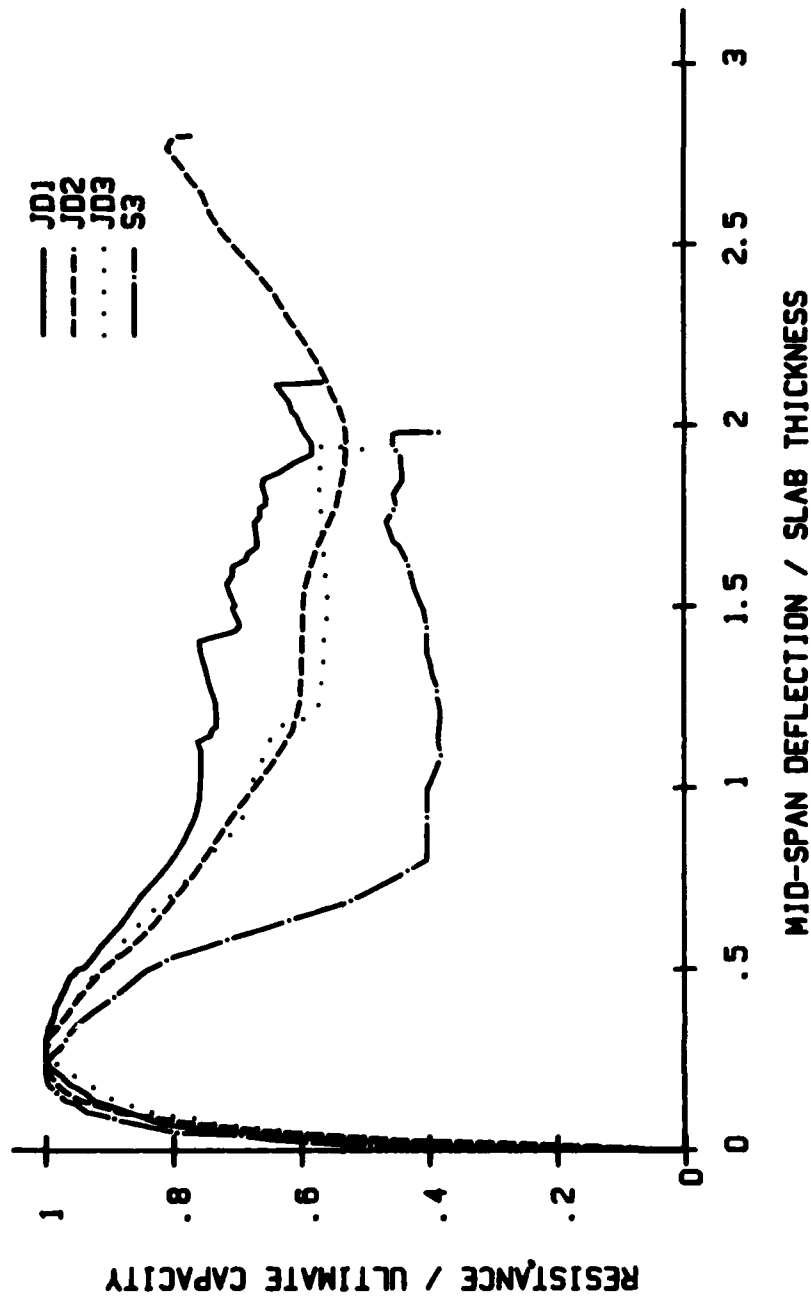
Figure 30. Comparison of load-deflection behavior.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The "original" Keyworker blast shelter reinforcement details (test S3 from Slawson et al., 1985) have been modified to enhance large deformation behavior and reduce construction costs. The tests reported herein investigate design modifications that increase constructibility without degrading structural performance. In particular, the roof principal reinforcement extended 40 bar diameters down into the exterior shelter walls for development which prevented casting the walls without prior placement of the roof reinforcement. The original principal reinforcement detail was modified by the use of 180 (JD1) and 90 (JD3) degree hooks in the roof-wall joint and by the use of dowels and bearing bars. These details performed adequately, but the large deformation behavior of the roof slab was not enhanced. Test JD2 was performed to evaluate the use of 180 degree hooks in conjunction with truss bars for tensile reinforcement to enhance large deformation capacity. The results of test JD2 indicated that enhanced large deformation behavior could be obtained and that constructibility could be improved.

The joint detail used in test JD2 has been incorporated in the shelter design and validated by dynamic tests (Slawson, Garner, and Woodson, 1985; Woodson and Slawson, 1986; and Slawson, 1987).

Tests performed by Guice (1986a, 1986b) show that enhanced tensile membrane behavior can be obtained by using steel ratios of 0.01 in each face. This is an increase of 4 percent as compared with element JD2, but the use of straight bars eases steel placement during construction. The retest of the prototype shelter (Slawson, 1987) resulted in large shear deformations of the roof slab at the exterior and interior walls. This indicates that the use of shear reinforcement may be required near the walls. The current design as tested by Slawson (1987) used truss bars for tensile reinforcement with no shear reinforcement. The shear failure problem is not a concern for the design loading conditions, but should be considered if large deformations are expected as in an overload case.

43

5.2 RECOMMENDATIONS

Based on the results of these static tests and dynamic tests performed in companion Keyworker support programs, the joint details used in test JD2 should be used for the final shelter design.

Since the prototype shelter did suffer significant shear deformation in the overload retest, the use of shear stirrups in the shear failure zone (near the walls) should be considered. In addition, the use of 1 percent straight reinforcement in each slab face should be compared with the current use of truss and straight bars on a cost of materials and construction labor basis. The current design is cheaper based on cost of materials, but this may be outweighed by the cost of manufacturing and installing the truss bars.

REFERENCES

American Society of Testing and Materials, 1984. "Standard Specification Deformed and Plain Billet Steel Bars for Concrete Reinforcement," Designation: A-615-82, 1984 Book of ASTM Standards, Volume 01.04, Philadelphia, PA.

Guice, L.K., 1986a (Feb). "Effects of Edge Restraint on Slab Behavior," Technical Report SL-86-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Guice, L.K., 1986b (Sep). "Behavior of Partially Restrained Reinforced Concrete Slabs," Technical Report SL-86-32, US Army Waterways Experiment Station, Vicksburg, MS.

Huff, W.L., 1969 (Apr). "Test Devices, Blast Load Generator Facility," Miscellaneous Paper N-69-1, US Army Waterways Experiment Station, Vicksburg, MS.

Kiger, S.A., Slawson, T.R., and Hyde, D.W., 1984 (Apr). "A Procedure for Computing the Vulnerability of Shallow-Buried Flat-Roof Structures," Technical Report SL-84-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Park, R., and Pauley, T., 1975. Reinforced Concrete Structures, John Wiley and Sons, New York, NY.

Park, R., and Gamble W.L., 1980. Reinforced Concrete Slabs, John Wiley and Sons, New York, NY.

Slawson, T.R., et al, 1985 (Oct). "Structural Element Tests, Keyworker Blast Shelter Program," Technical Report SL-85-6, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Slawson, T.R., Garner, S.B., and Woodson, S.C., 1986 (Apr). "Yield Effects on the Response of a Buried Shelter," Technical Report SL-86-5, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Slawson, T.R., 1987 (Apr). "Vulnerability Evaluation of the Keyworker Blast Shelter," Technical Report SL-87-10, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Woodson, S.C., and Slawson, T.R., 1986 (Dec). "Demonstration Test of the Keyworker Blast Shelter," Technical Report SL-86-40, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Woodson, S.C., and Garner, S.B., 1985 (Aug). "Effects of Reinforcement on Capacity of Concrete Slabs," Technical Report SL-85-8, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Woodson, S.C., 1985 (Aug). "Effects of Shear Stirrup Details on Ultimate Capacity and Tensile Membrane Behavior of Reinforced Concrete Slabs," Technical Report SL-85-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

APPENDIX A: RECOVERED DATA

Complete data records obtained for the three tests are presented in this appendix. For gage locations refer to Chapter 2 of this report. Tensile strains are positive, and compressive strains are negative. Water pressure is referenced on the ordinate.

For Test JD3 the following pairs of records are believed to be reversed: EO-1, EI-1, EO-3, EI-3, EO-4, and EI-4.

Data plots labels are: J1 for test JD1, JD2 for test JD2, and J3 for test JD3.

FEMA J1
SE-1
MAXIMUM   SIGMA CAL   CAL VAL
55.5856   1.4875   74.2

CHANNEL NO. 15   3513 1
10/10/84   RO450

49
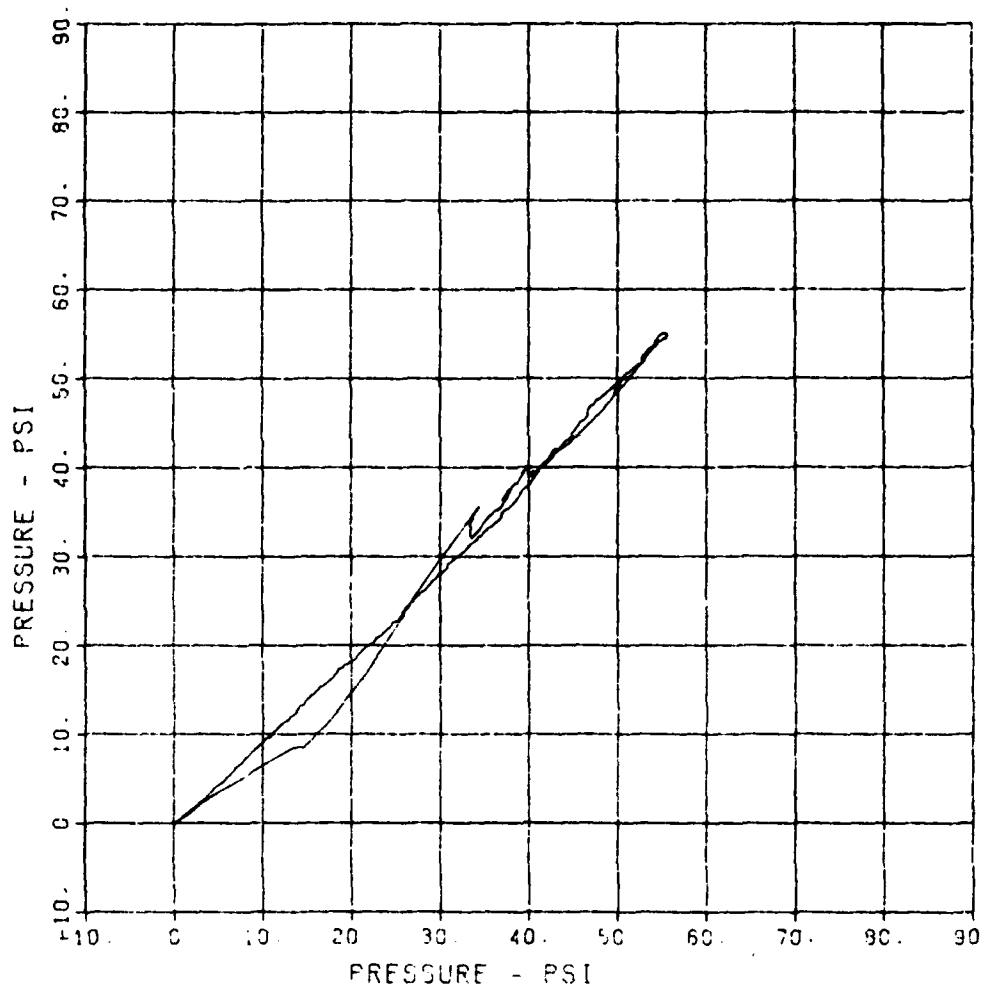
FEMA J1
SE-2

MAXIMUM	SIGMA CAL	CAL VAL
41.1252	1.1729	70.5

CHANNEL NO. 16   3613   1
10/10/84   R0450

50
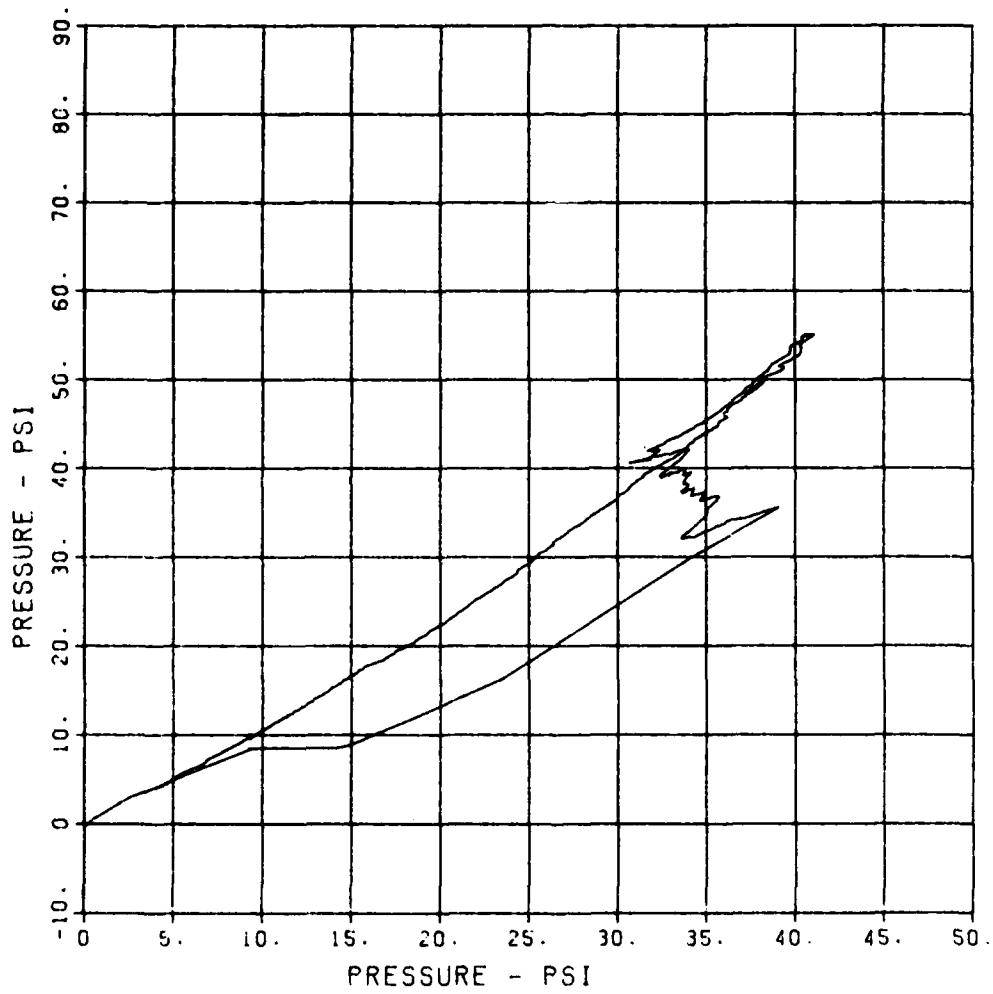
FEM9 J1
EI-1
MAXIMUM   SIGM9 CAL   CAL VAL
  -2350.9333   6.0722   11480.0

CHANNEL NO. 5   3613 1
10/02/84   R0370

51
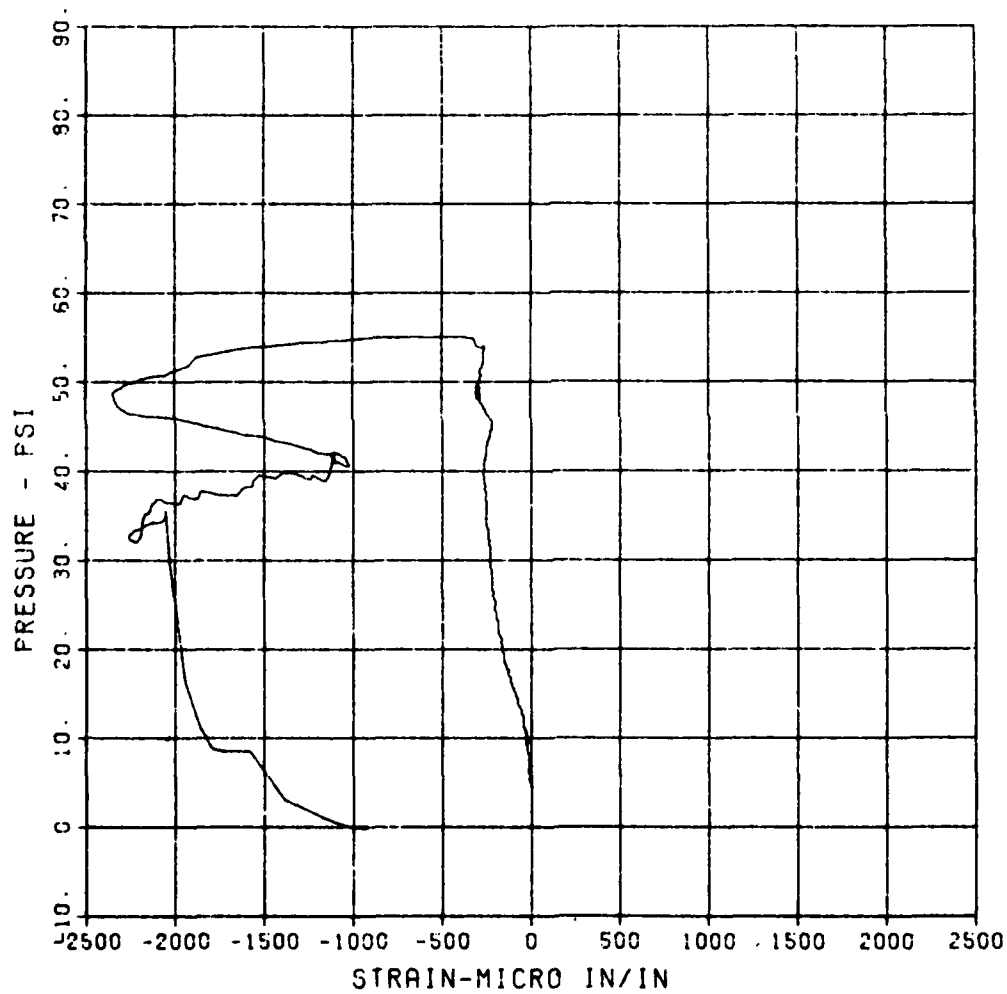
FEMA J1
EQ-2
MAXIMUM   SIGMA CAL   CAL VAL
  626.6663   1.3969   5780.0

CHANNEL NO. 9   3613   1
10/02/94   R0370

52
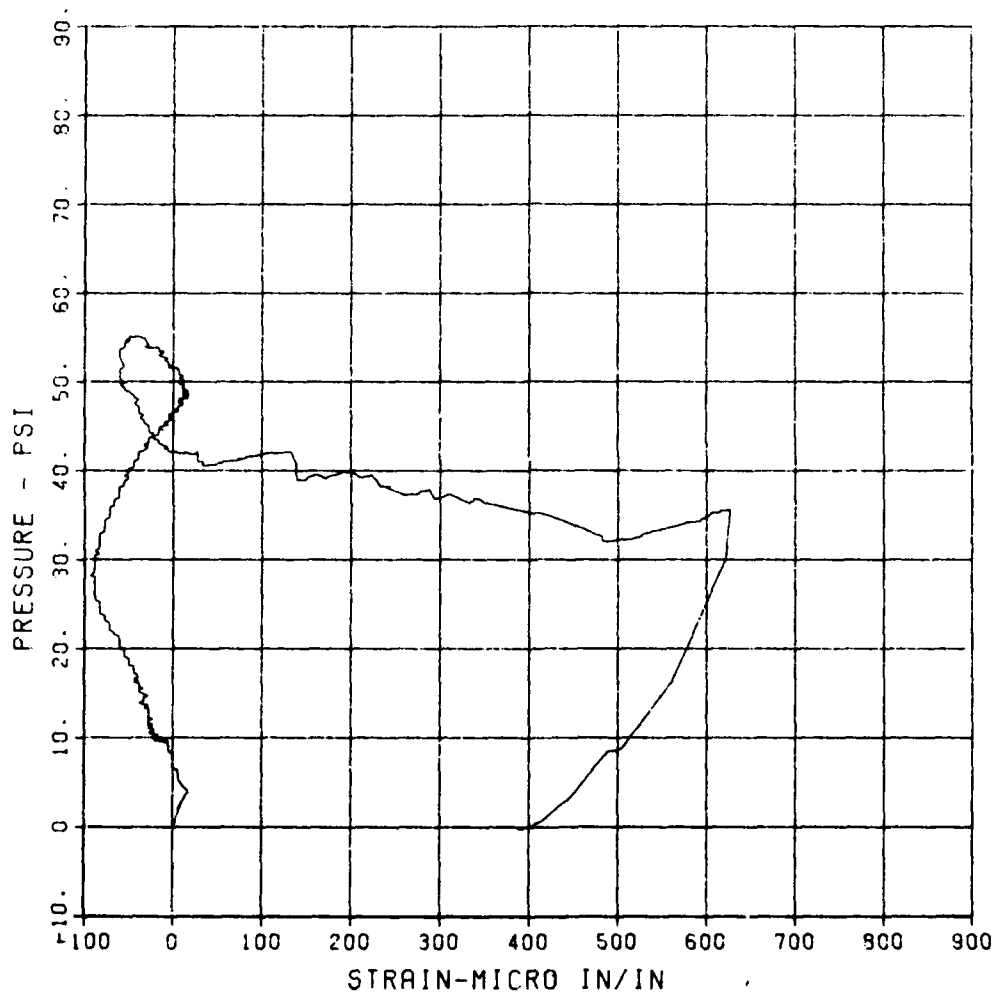
FEMA J1
EI-2
MAXIMUM   SIGMA CAL   CAL VAL
  2032.7857   4.2144   5780.0

CHANNEL NO. 7   3613   1
10/02/84   R0370
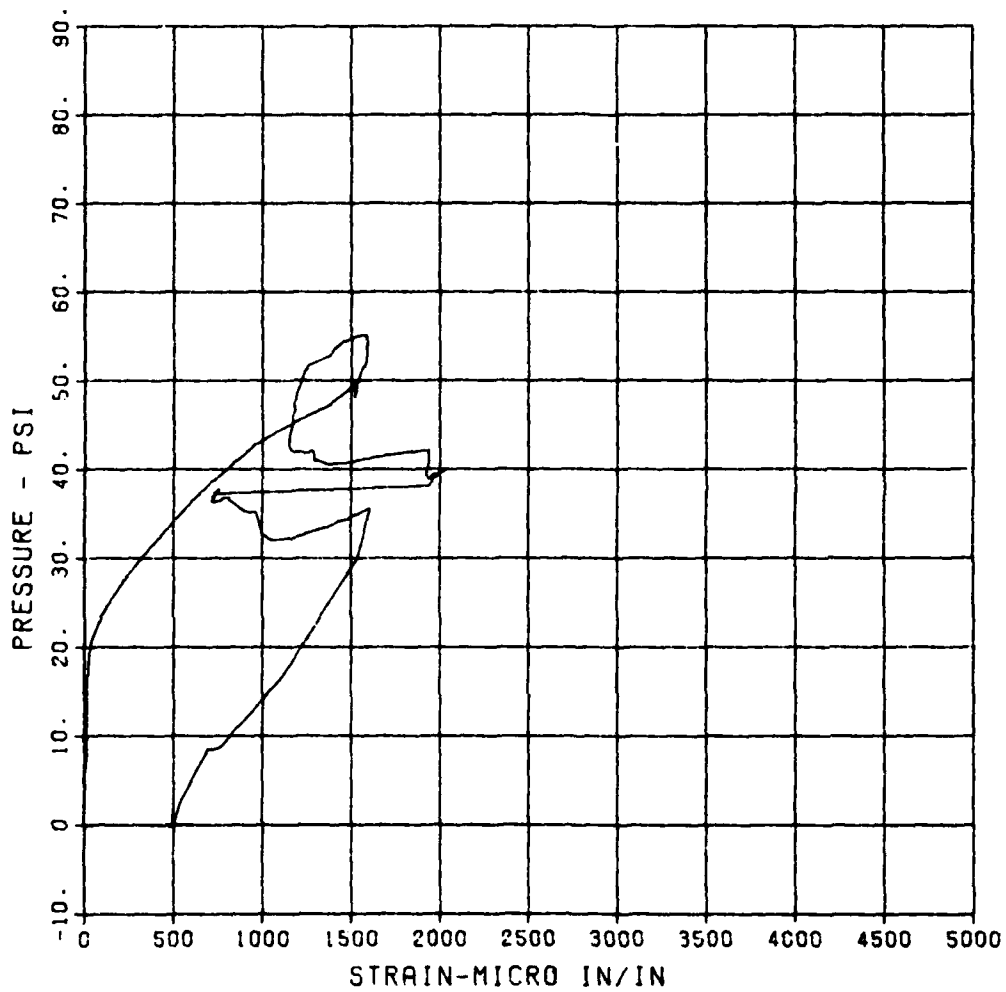
FEMA J1
EO-3
MAXIMUM   SIGMA CAL   CAL VAL
  -4022.9783   6.0518   5780.0

CHANNEL NO. 10   3613   1
10/02/84   R0370
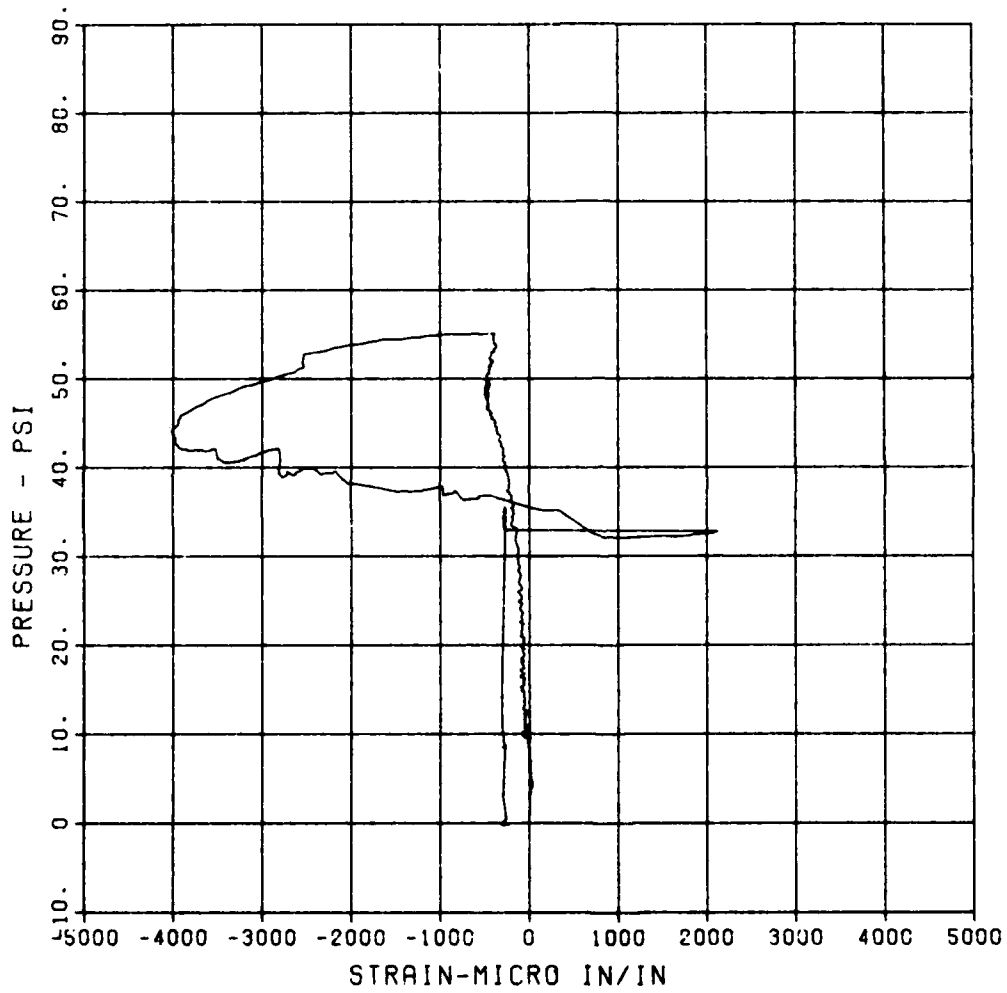
FEMA J1
EI-3
MAXIMUM   SIGMA CAL   CAL VAL
 11927.8564 1.8258   5780.0

CHANNEL NO. 9   3613 1
10/02/84   R0370
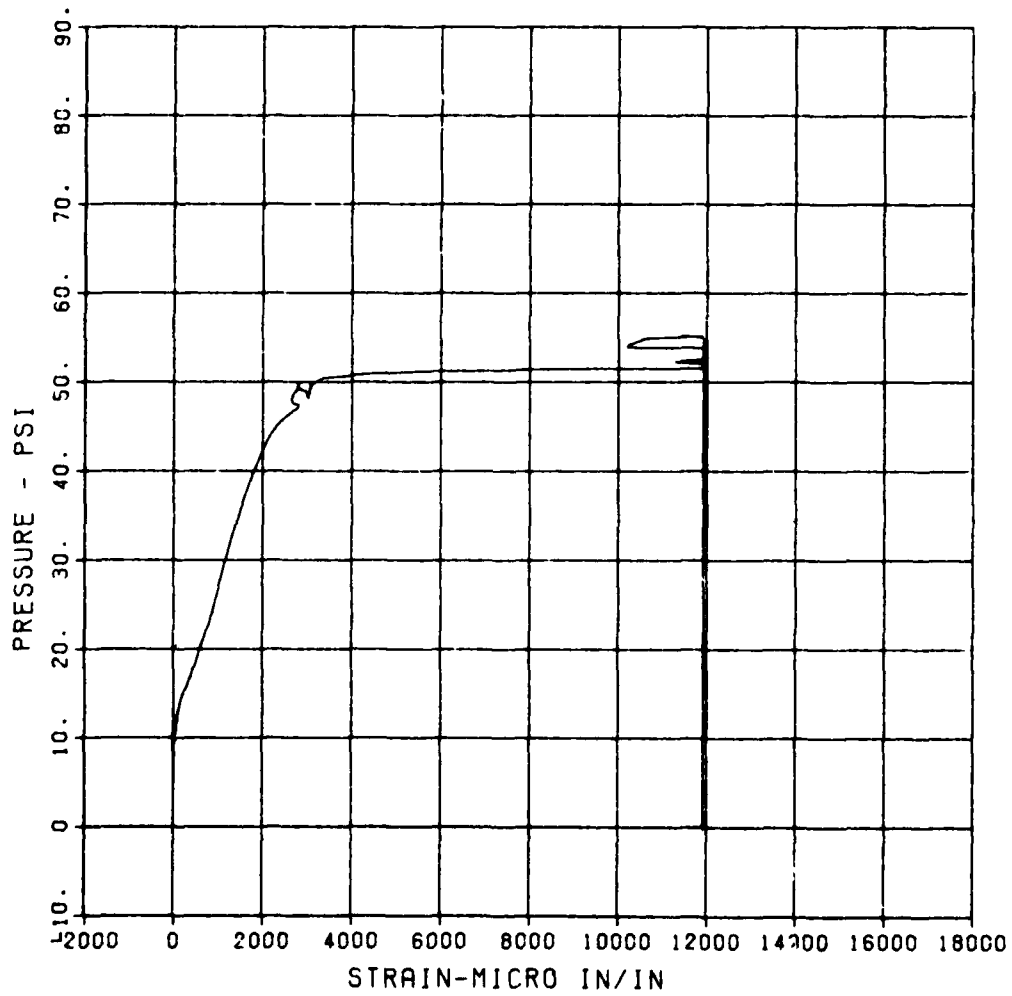
FEMA J1
EO-3S
MAXIMUM   SIGMA CAL   CAL VAL
  -4022.0959   2.6406   5780.0

CHANNEL NO. 19   3613   1
05/28/87   R0395
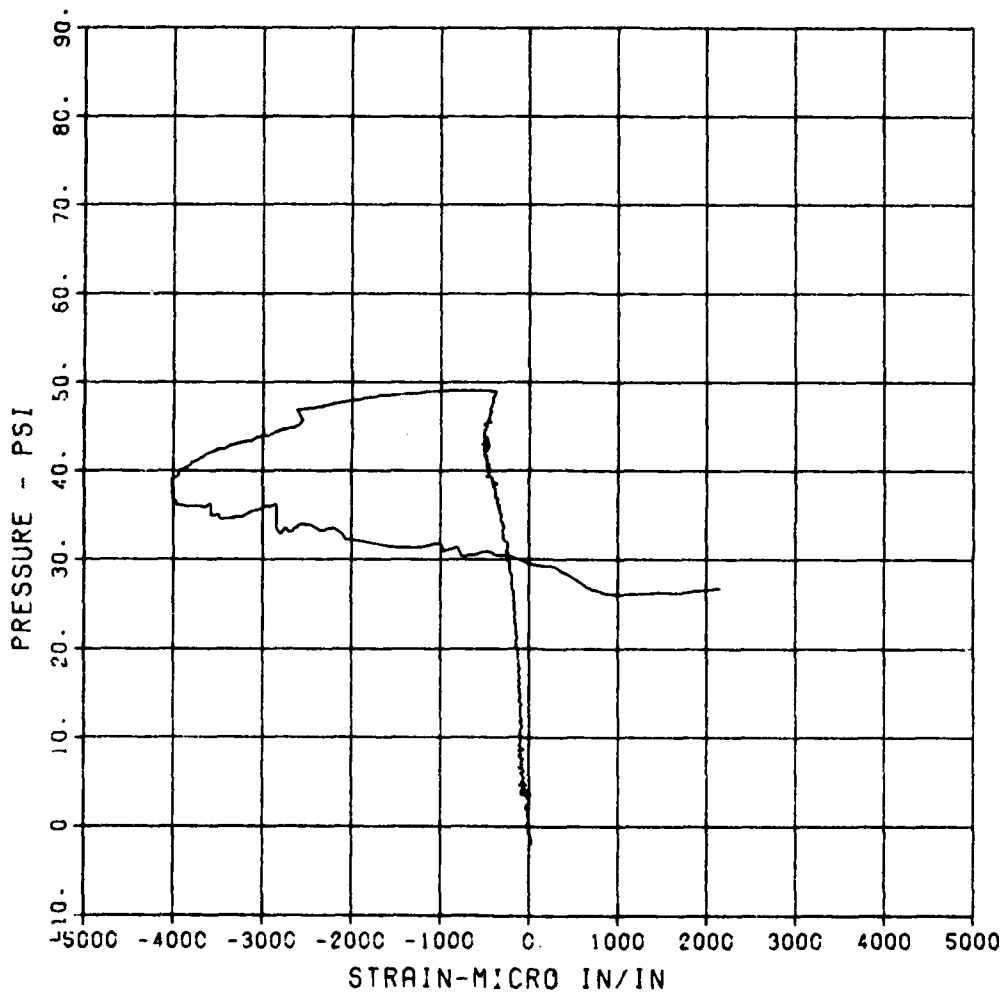
FEMA J1
EI-3S
MAXIMUM   SIGMA CAL   CAL VAL
  17430.6379   3.1369   5780.0

CHANNEL NO. 18   3613   1
05/28/87   R0396

57
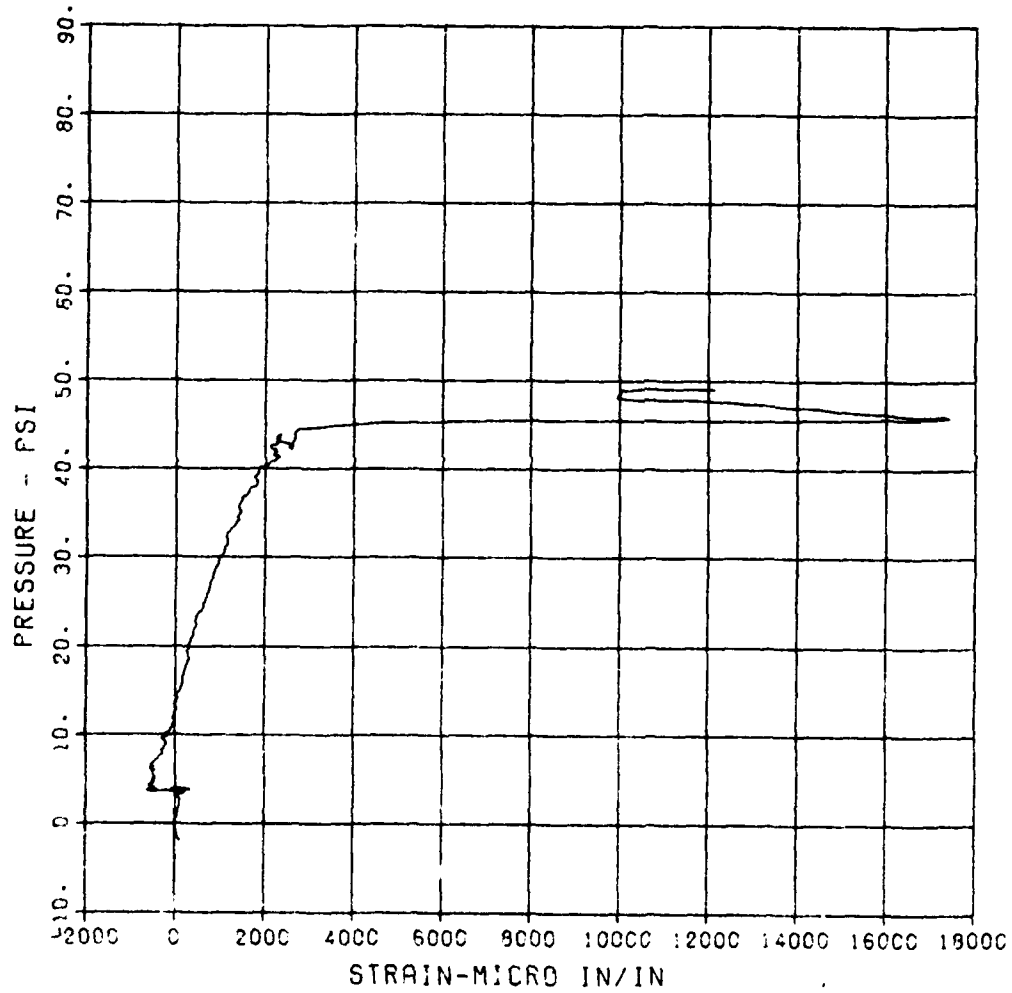
FEMA J1
EO-4
MAXIMUM   SIGMA CAL   CAL VAL
   1699.3239   1.6219   8640.0

CHANNEL NO. 12   3613   1
10/02/84   RO370
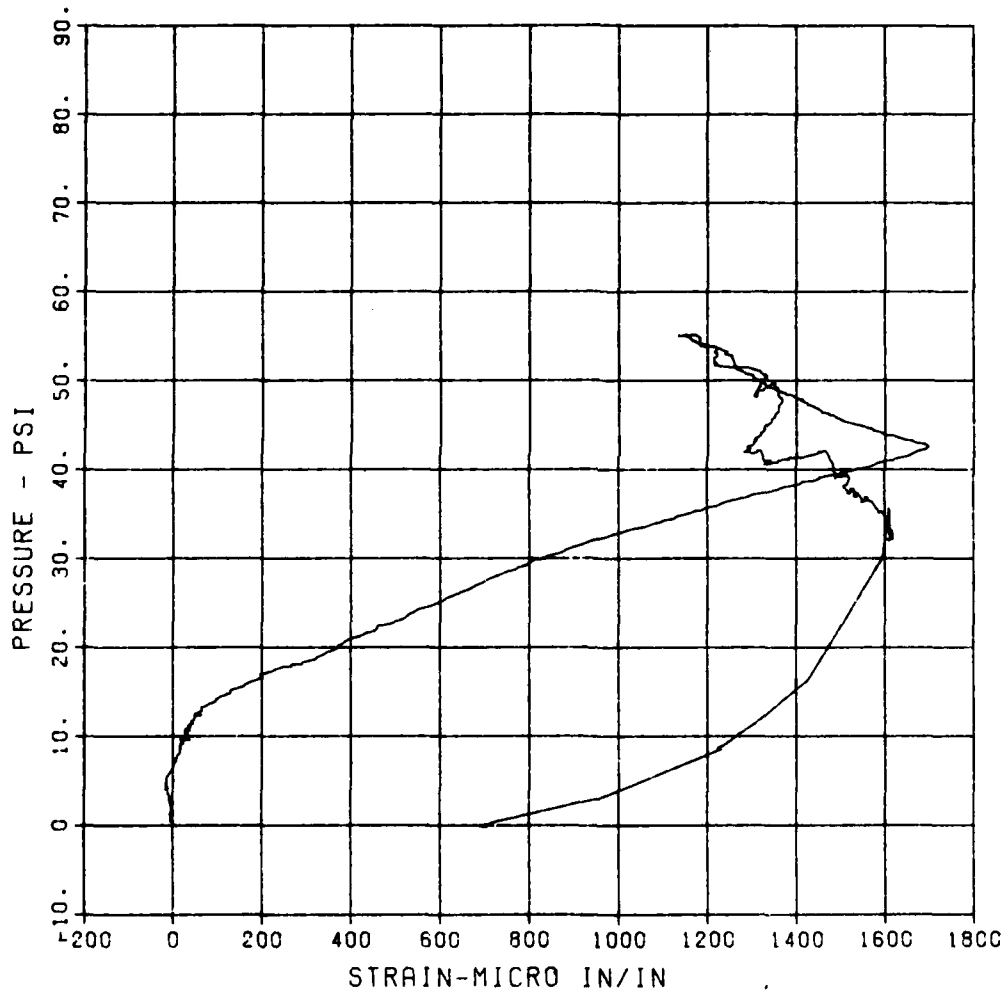
FEMA J1
EI-4
MAXIMUM   SIGMA CAL   CAL VAL
  5162.6968   1.5320   5780.0

CHANNEL NO. 11   3613   1
05/28/87   R0393

59
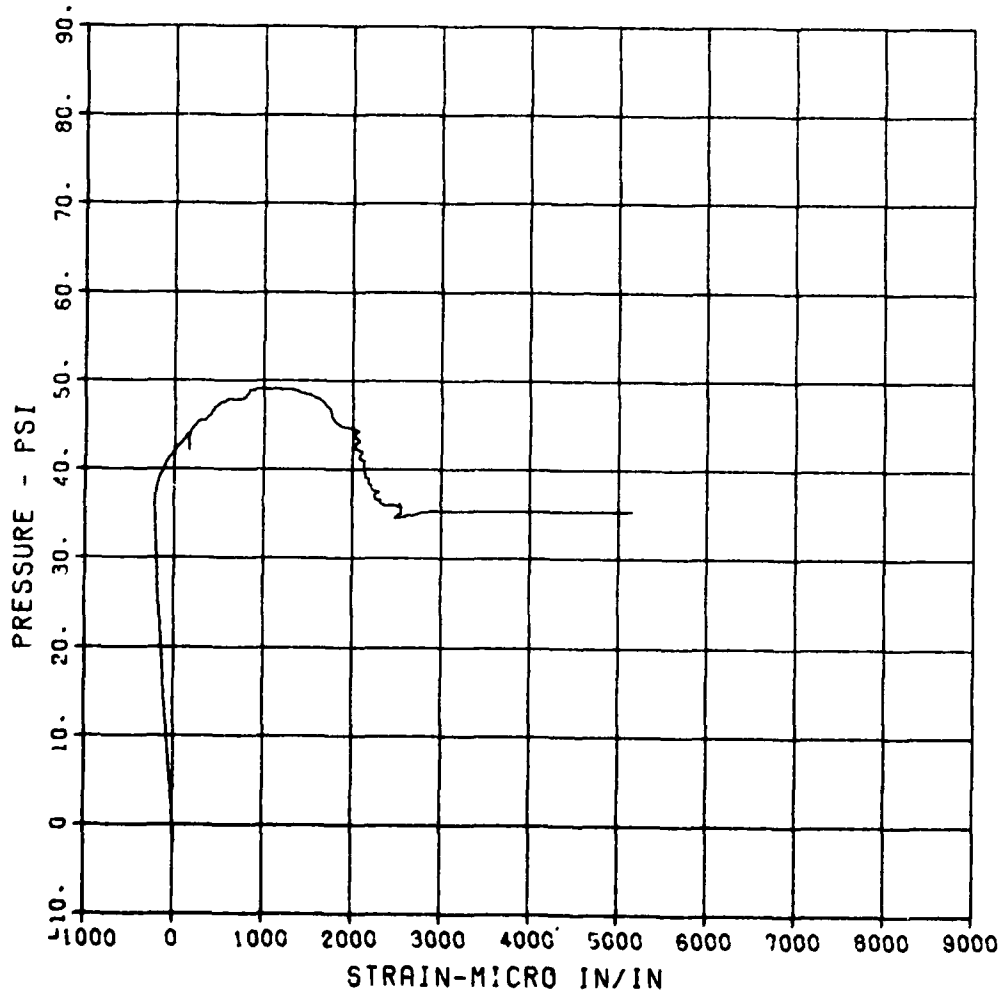
FEMA J1
ED-2
MAXIMUM   SIGMA CAL   CAL VAL
  -8143.9506   2.0048   5780.0

CHANNEL NO. 14   3513   1
10/02/84   R0370

60
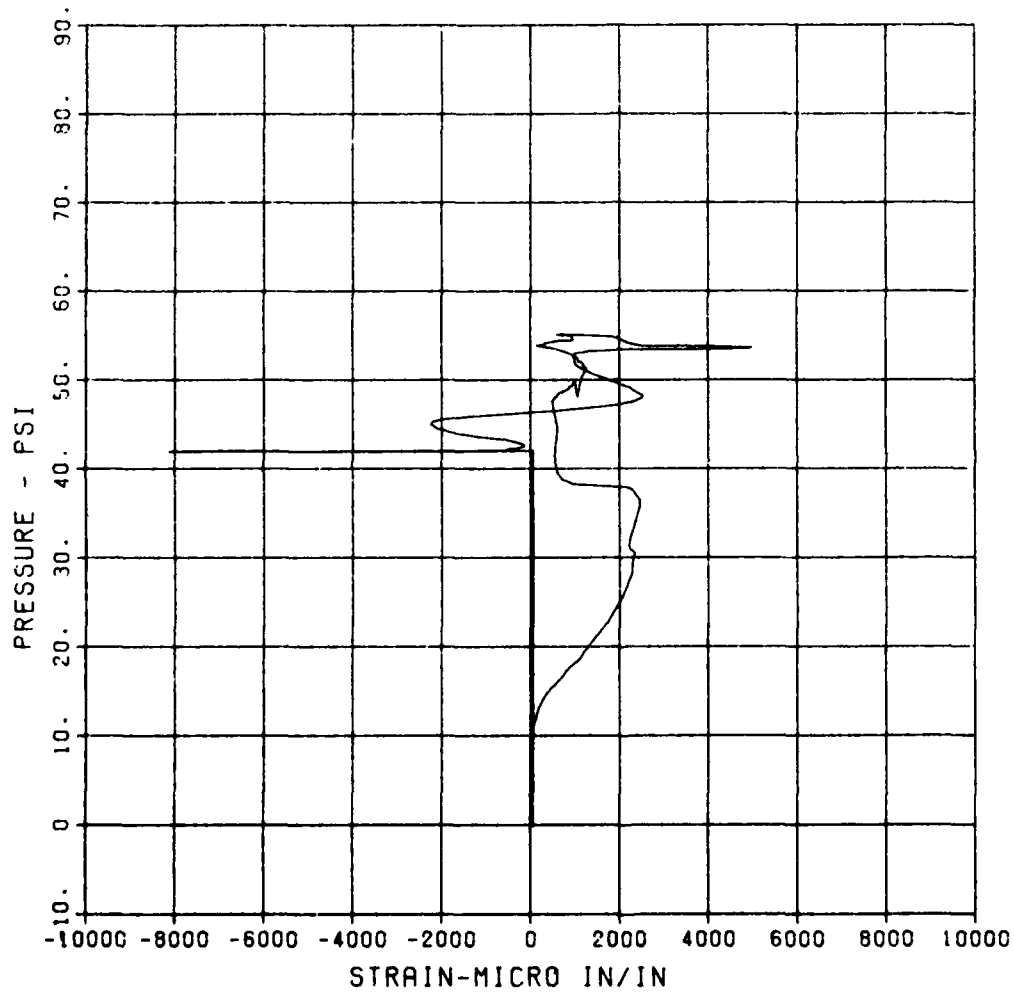
FEMA J1
D-1

MAXIMUM	SIGMA CAL	CAL VAL
5.3232	1.2793	7.0

CHANNEL NO. 3   3613 1

10/02/84   RO370

61
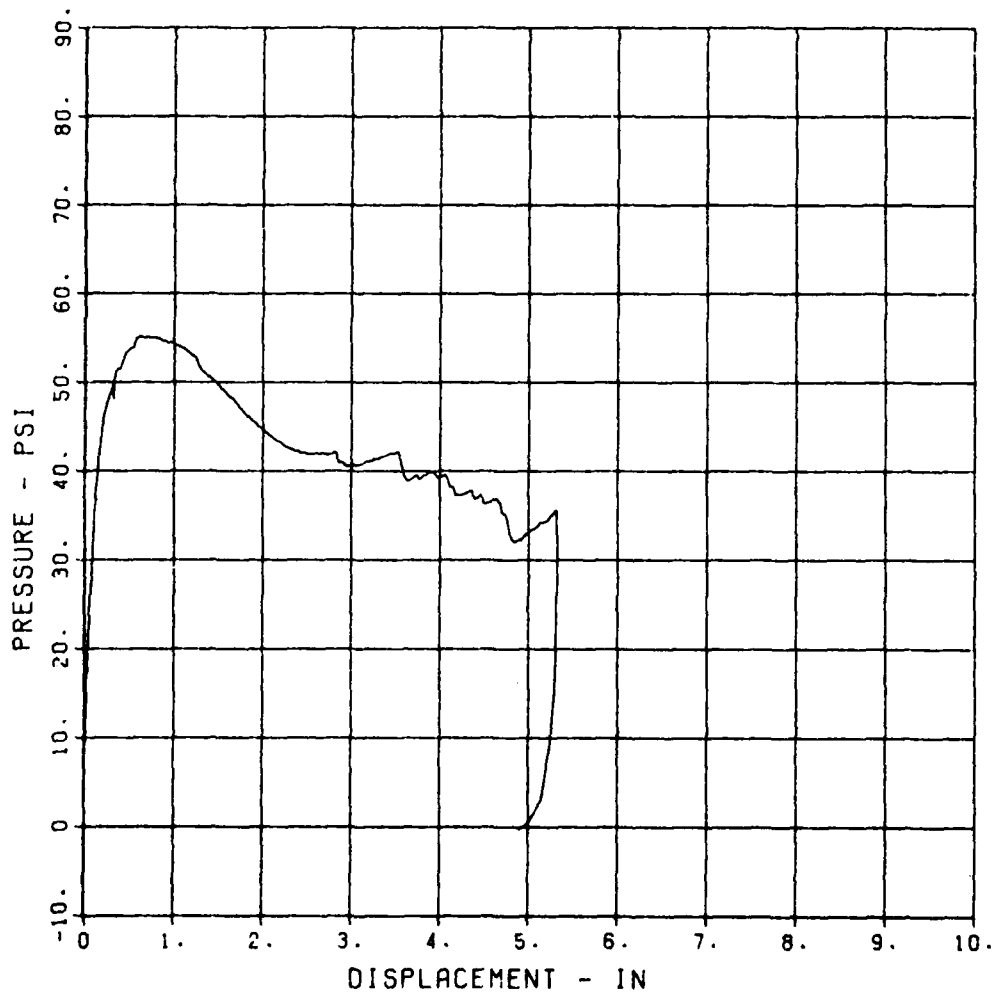
FEMA J1
D-2

MAXIMUM	SIGMA CAL	CAL VAL
3.0943	2.0379	4.3

CHANNEL NO. 4   3613 1
10/02/84   R0370

62
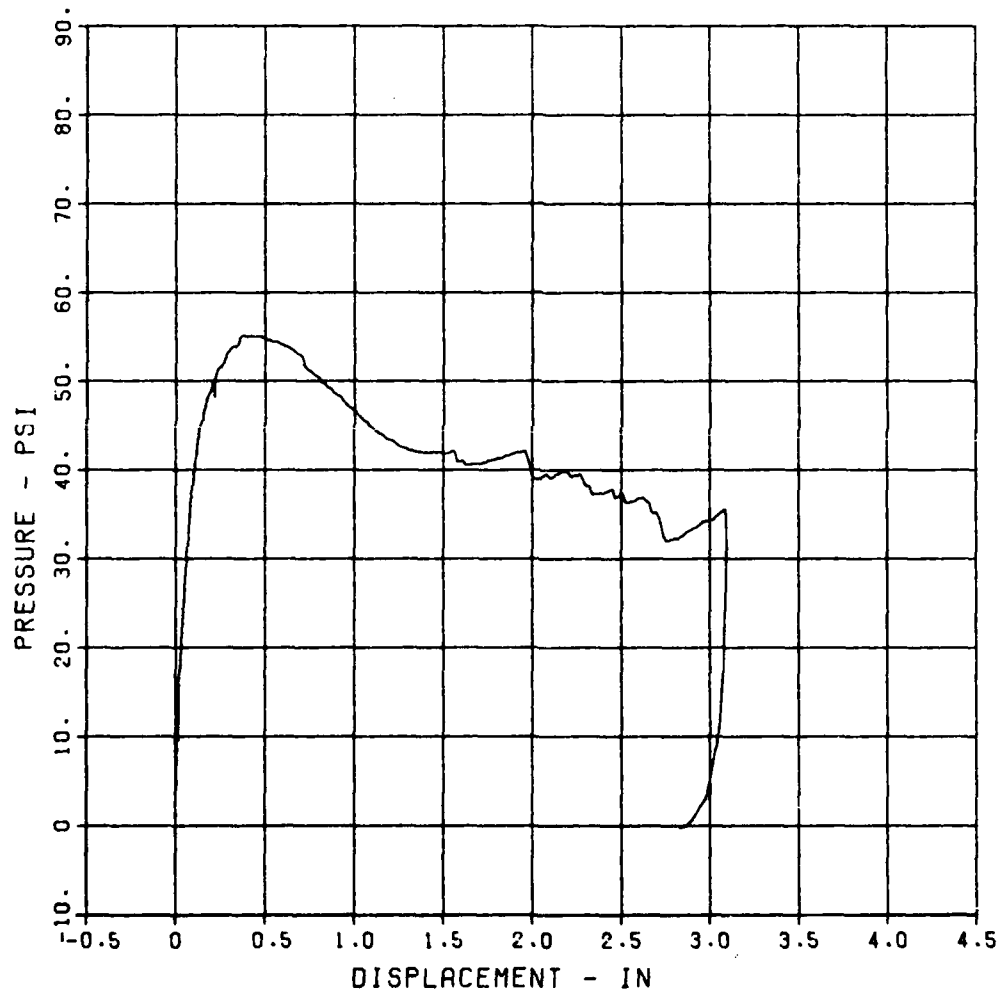
FEMA J02
SE-1
MAXIMUM   SIGMA CAL   CAL VAL
41.7492   3.4865   114.0

CHANNEL NO. 17   15544 1
09/07/95   80607

PRESSURE - PSI (y-axis)

PRESSURE - PSI (x-axis)
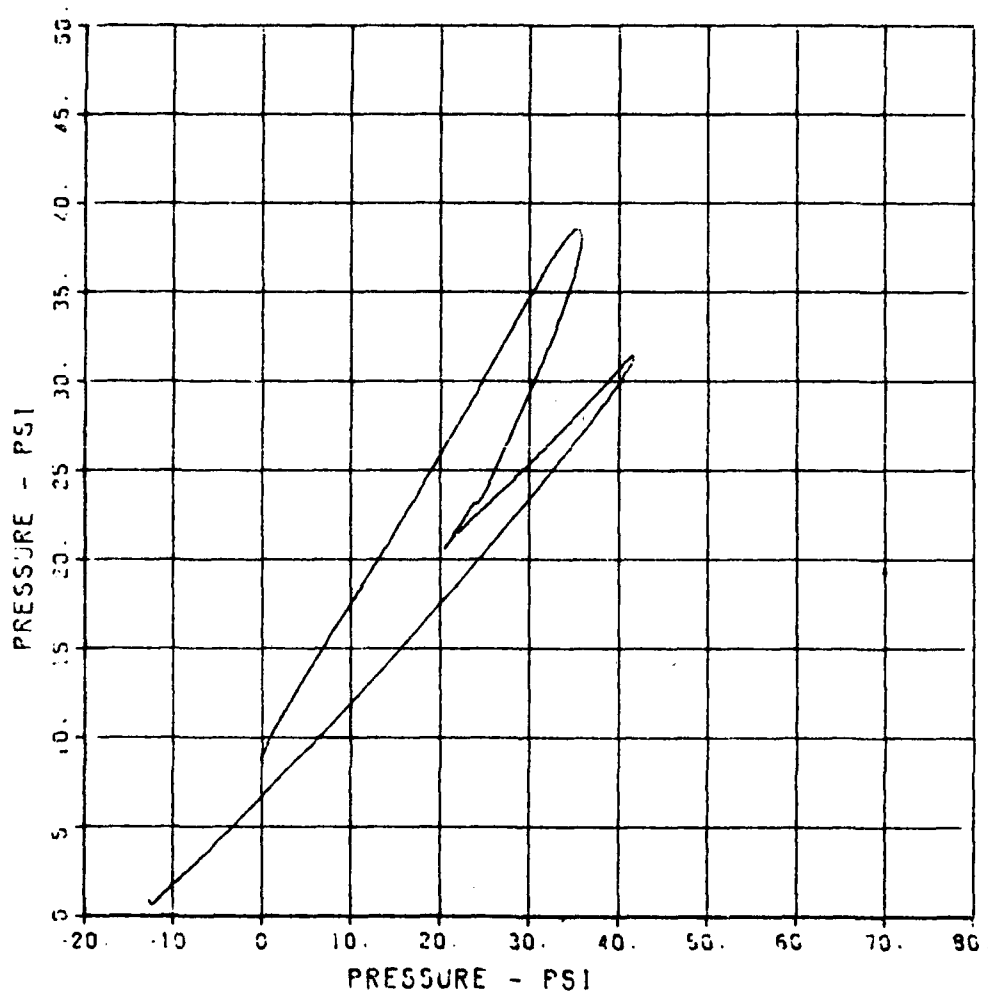
FEMA JD2
SE-2

MAXIMUM	SIGMA CAL	CAL VAL
39.8161	5.0428	119.0

CHANNEL NO. 18   15644 1
09/07/85   R0607

64
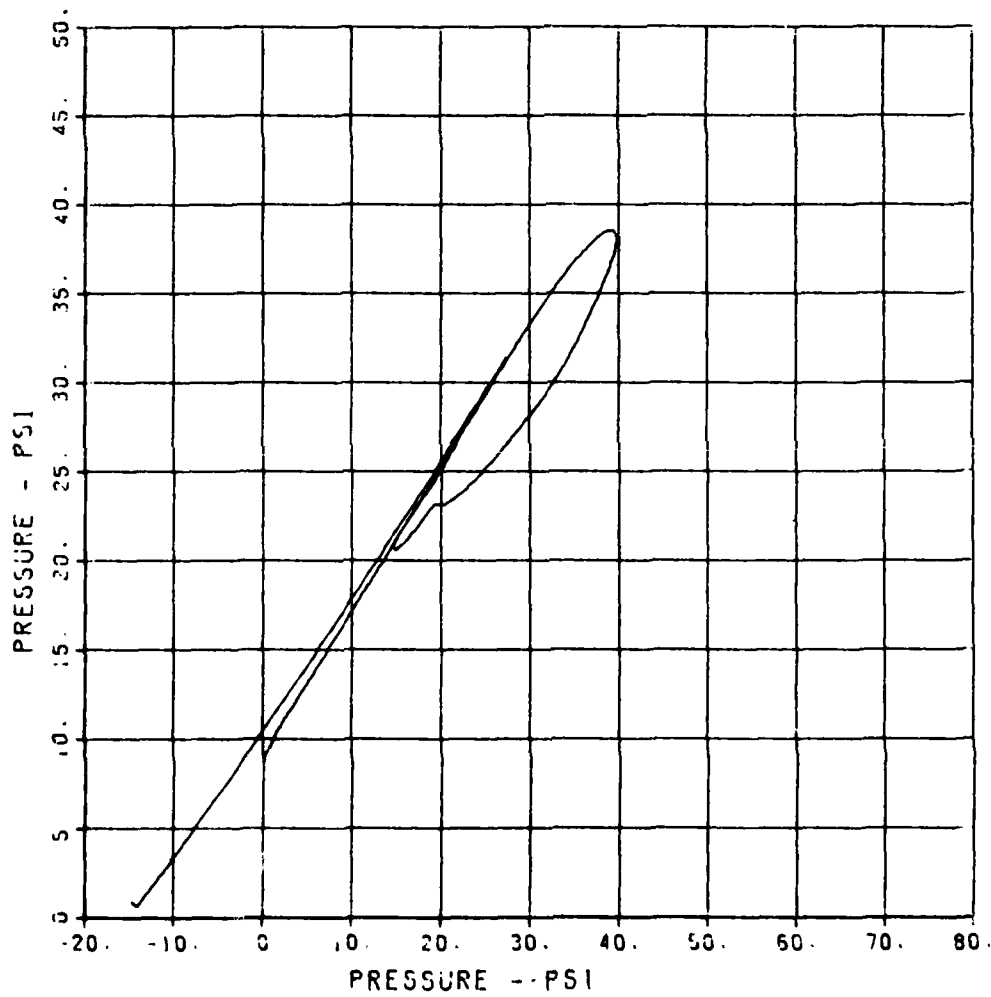
FEMA JD2
EO-1
MAXIMUM   SIGMA CAL   CAL VAL
   3551.8608   2.9601   5785.0

CHANNEL NO. 6   15544   1
03/07/85   P0607

Pressure vs. Strain plot. Y-axis: PRESSURE - PSI (0 to 50). X-axis: STRAIN-MICRO IN/IN (0 to 5000).
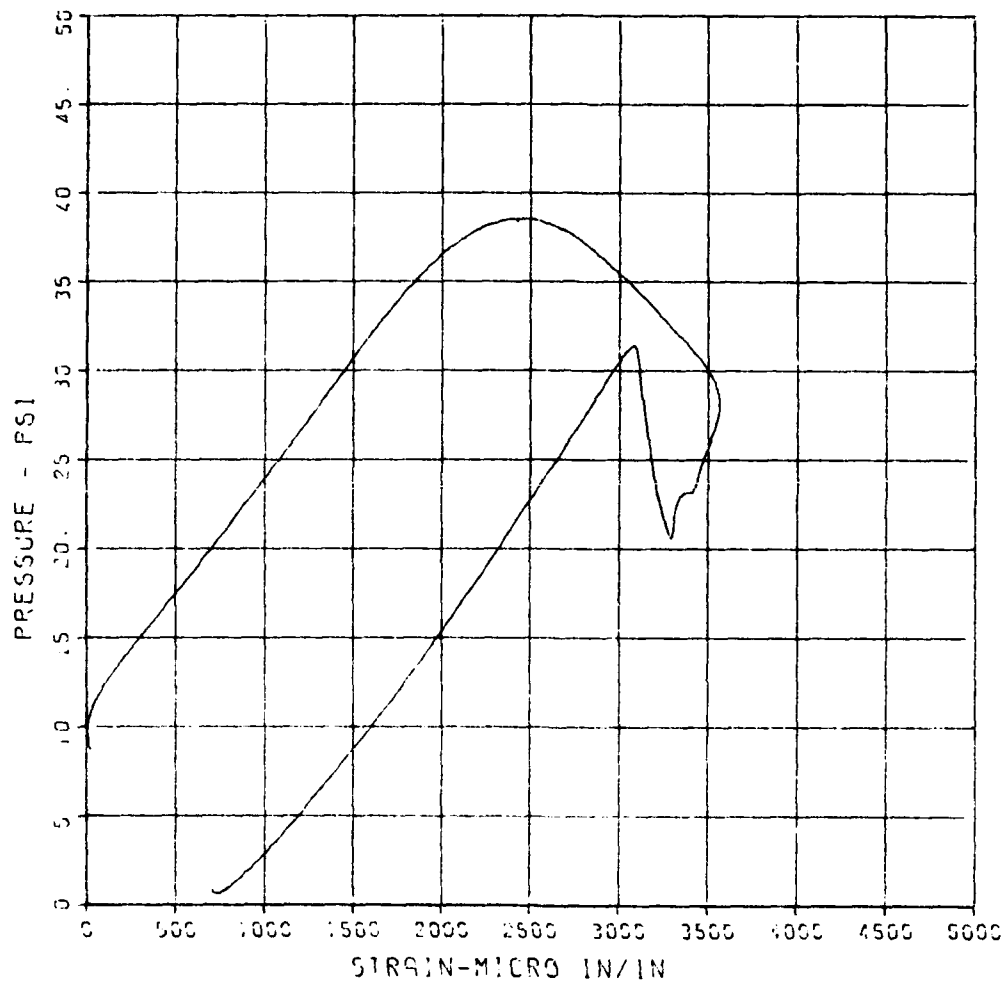
FEMA J02
E!-1
MAXIMUM   SIGMA CAL   CAL VAL
  265.0108   5.1005   2905.0

CHANNEL NO. 5   15644   1
09/07/85   R0607

66
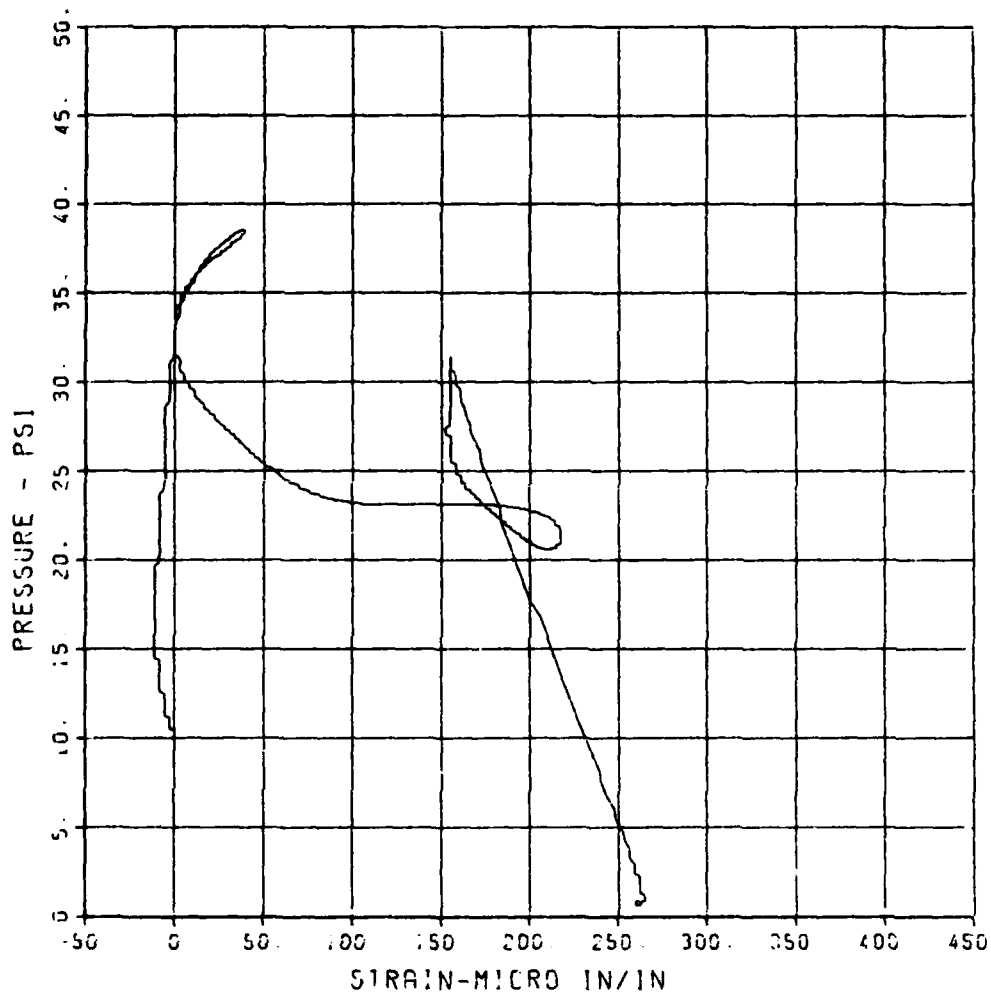
FEMA JC2
EO-2
MAXIMUM   SIGMA CAL   CAL VAL
  296.2508   4.0430   2005.0

CHANNEL NO. 3   15544   1
09/07/85   R0602

67
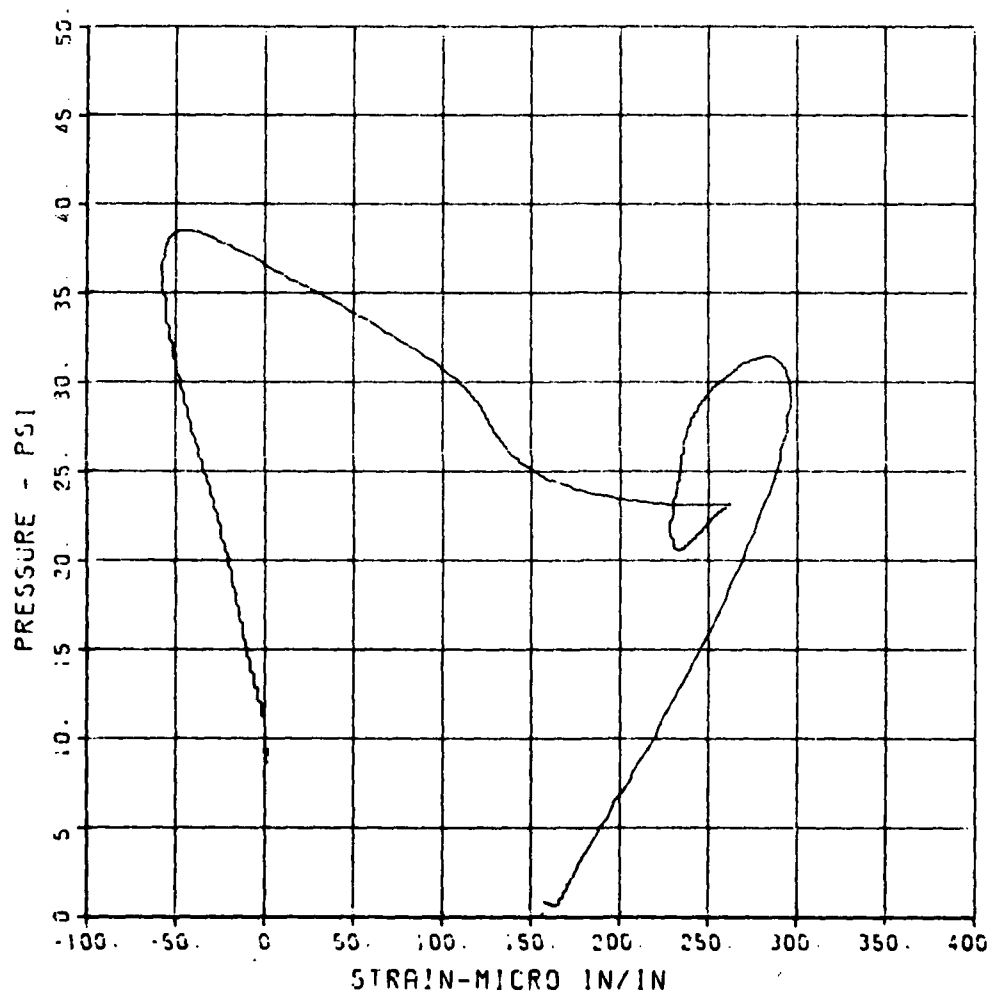
MAXIMUM   SIGMA CAL   CAL VAL
 -543.5921   81.2973   2905 0

CHANNEL NO 7   15544 1
09/07/85   R0607
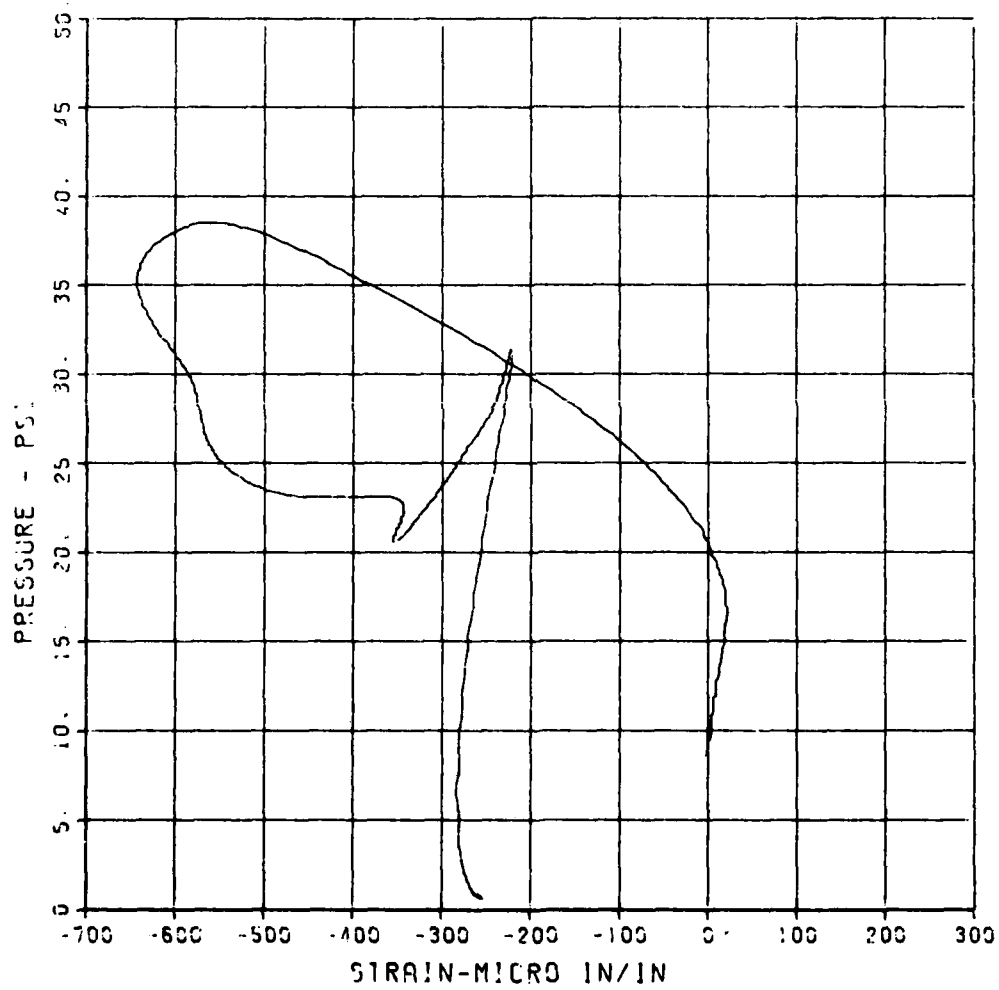
FEMA JC2
LO-3
MAXIMUM   SIGMA CAL   CAL VAL
  6182.3350   3.3135   5785.0

CHANNEL NO. 10   15542
09/07/85   R0607

69
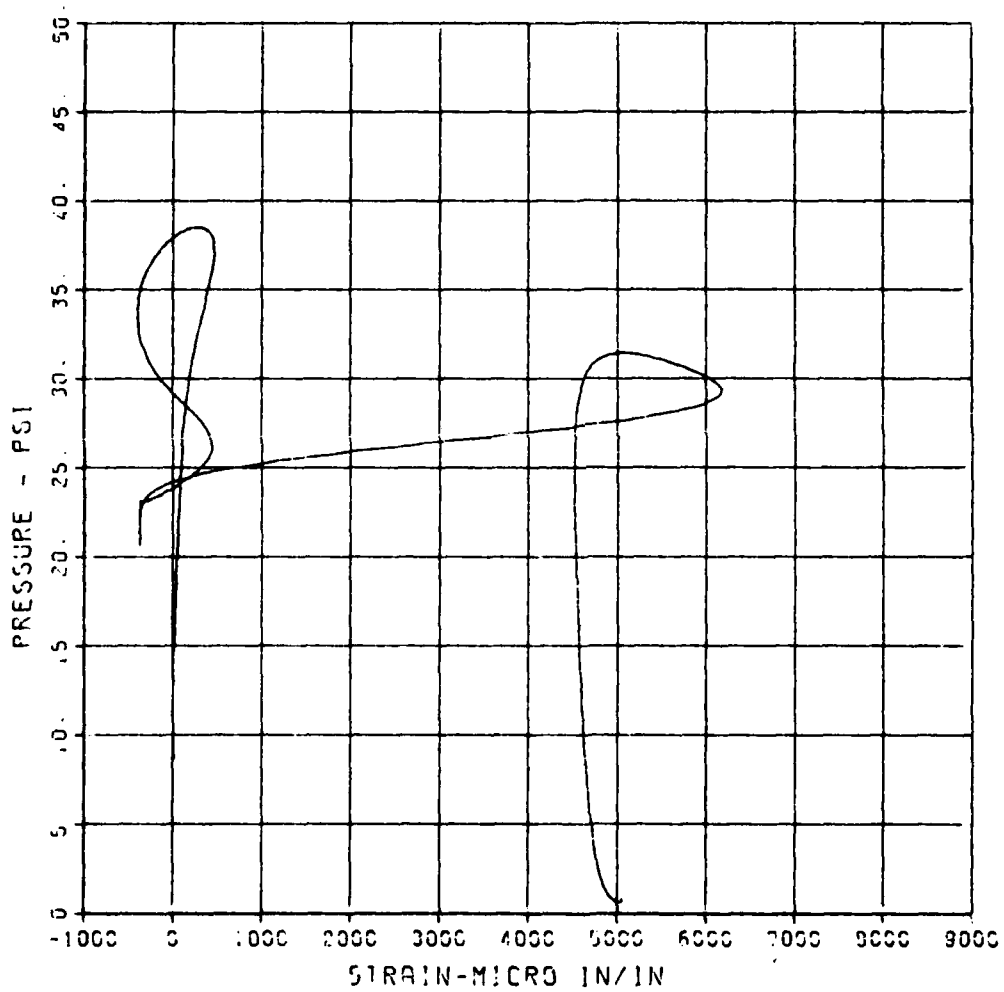
FEMA JD2
EI-3
MAXIMUM   SIGMA CAL   CAL VAL
11739.3824   3.4866   5785.0

CHANNEL NO. 9   15544 1
09/07/85   R0607

FEMA JO2
EO-9
MAXIMUM   SIGMA CAL   CAL VAL
-1724.7103   4.5099   2905 0

CHANNEL NO. .6   15644 1
09/07/85   R0607

PRESSURE - PSI

STRAIN-MICRO IN/IN

FEMG JD2
ED-9A
MAXIMUM   SIGMA CAL   CAL VAL
  5120.2596   3.5303   5795.0

CHANNEL NO. 30   15544   I
03/07/85   R0607
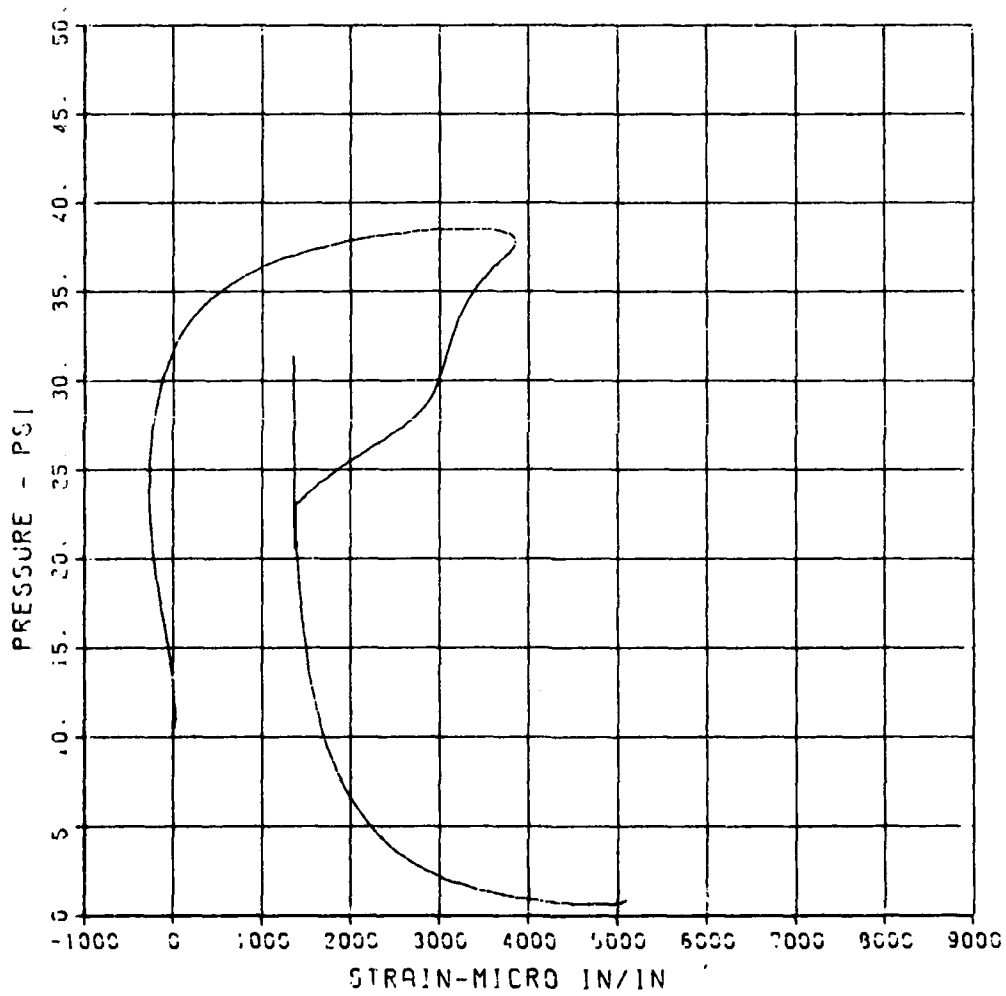
72

FEMA JD2
EI-9
MAXIMUM   SIGMS CAL   CAL VAL
  -576.5706   4.7358   2905 0

CHANNEL NO. 15   15644   1
03/07/85   R0607

PRESSURE - PSI

STRAIN-MICRO IN/IN
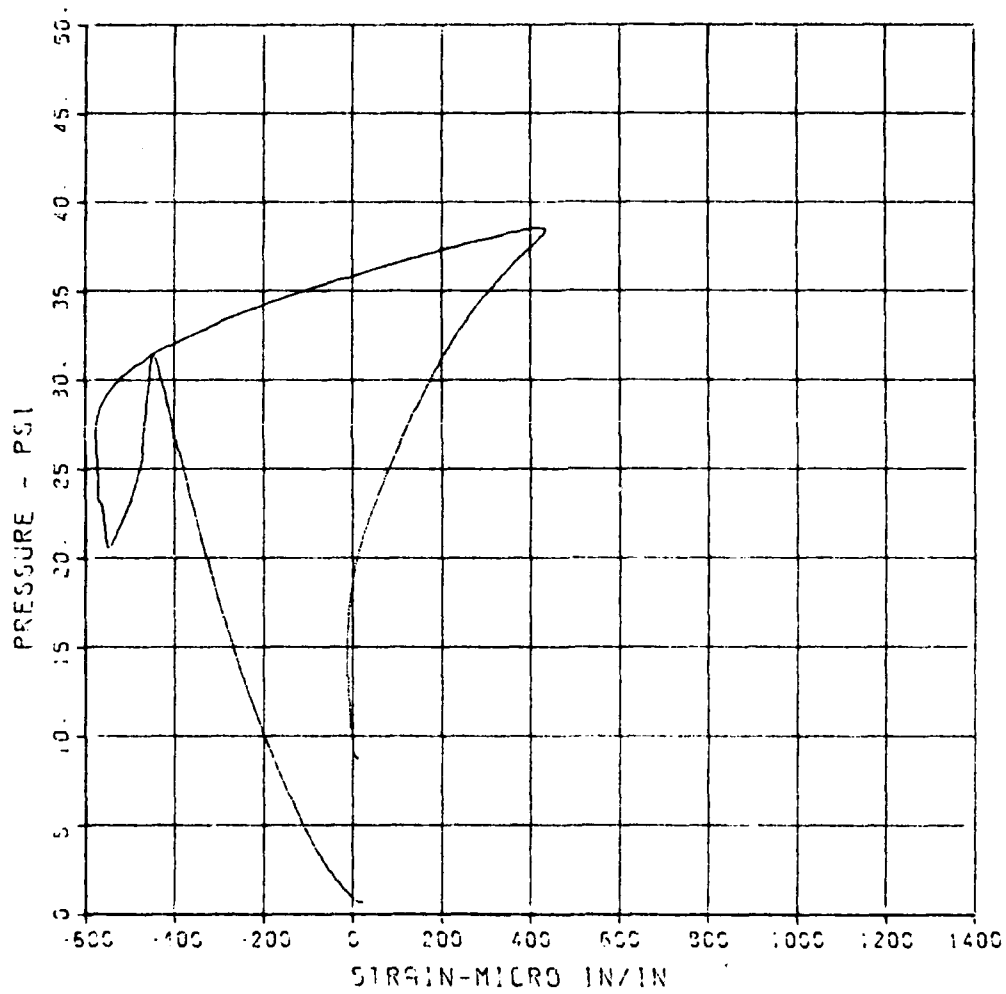
FEMA J02
E1-9A

MAXIMUM	SIGMA CAL	CAL VAL
1392.5900	3.3703	5785.0

CHANNEL NO. 29   15544   1
08/07/85   R0607
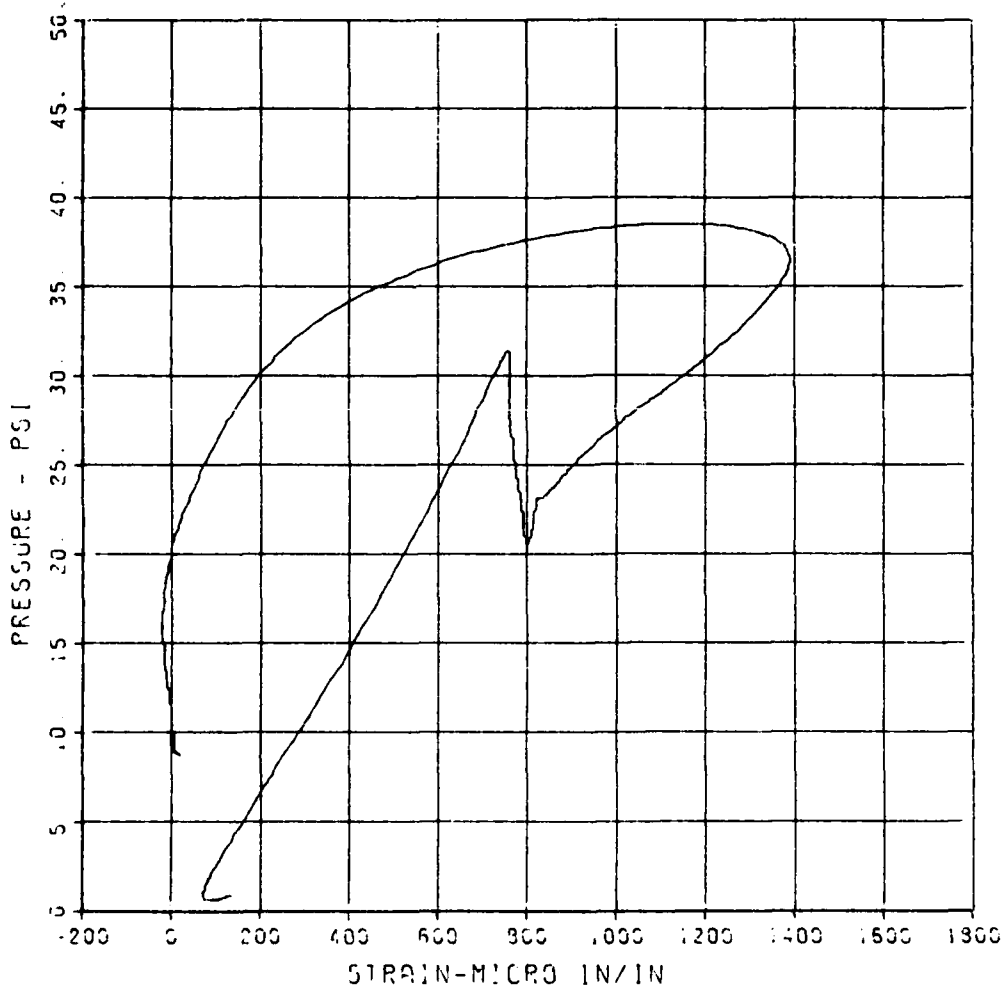
FEMS J32
F-5A
MAXIMUM   SIGMA CAL   CAL CAL
  22038.5893   3.0153   1143010

CHANNEL NO   25   15544   1
09/07/35   R0657
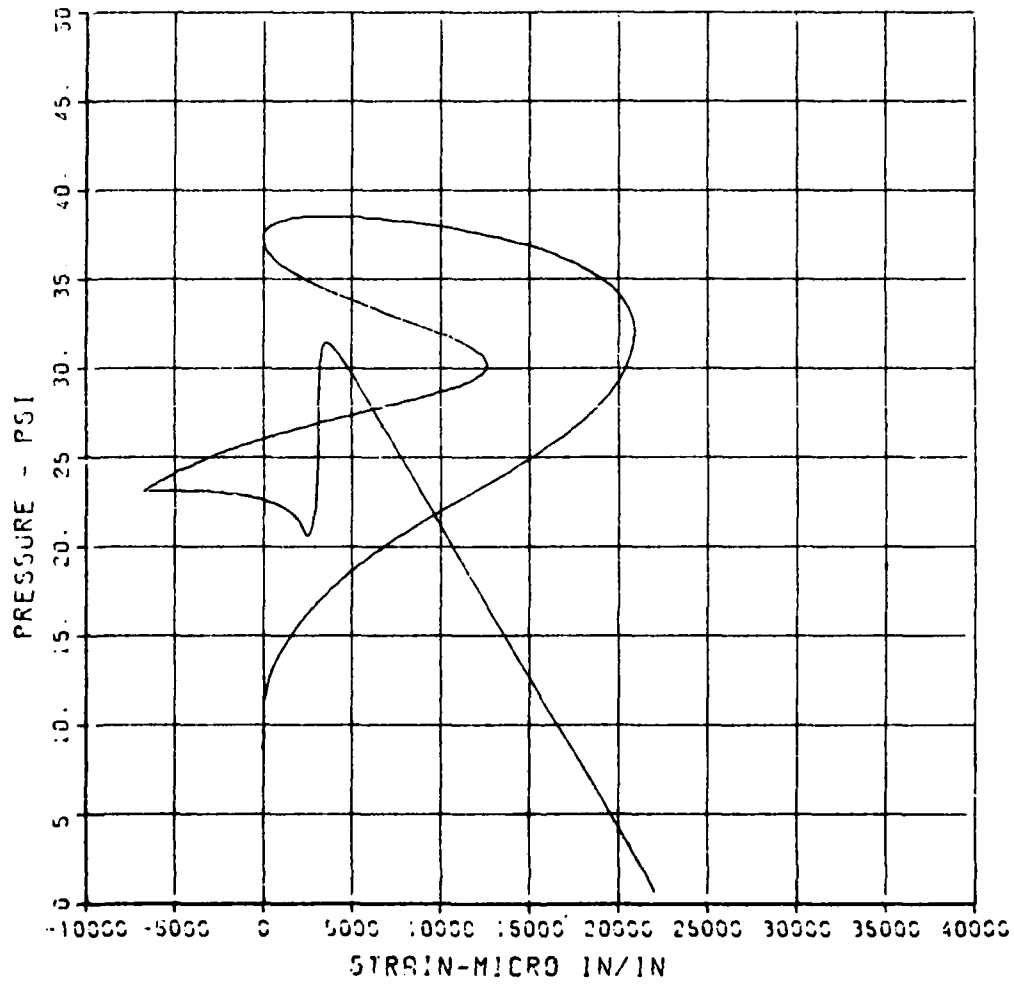
PRESSURE - PSI

STRAIN-MICRO IN/IN

75

PRESSURE - PSI

STRAIN-MICRO IN/IN

MAXIMUM   SIGMA CAL   CAL VAL
  314.2739   2.8527   9545.0

CHANNEL NO. 26   15644   1
03/07/85   R0607

PRESSURE - PSI (vertical axis, 0 to 50)

STRAIN-MICRO IN/IN (horizontal axis, 0 to 1000)
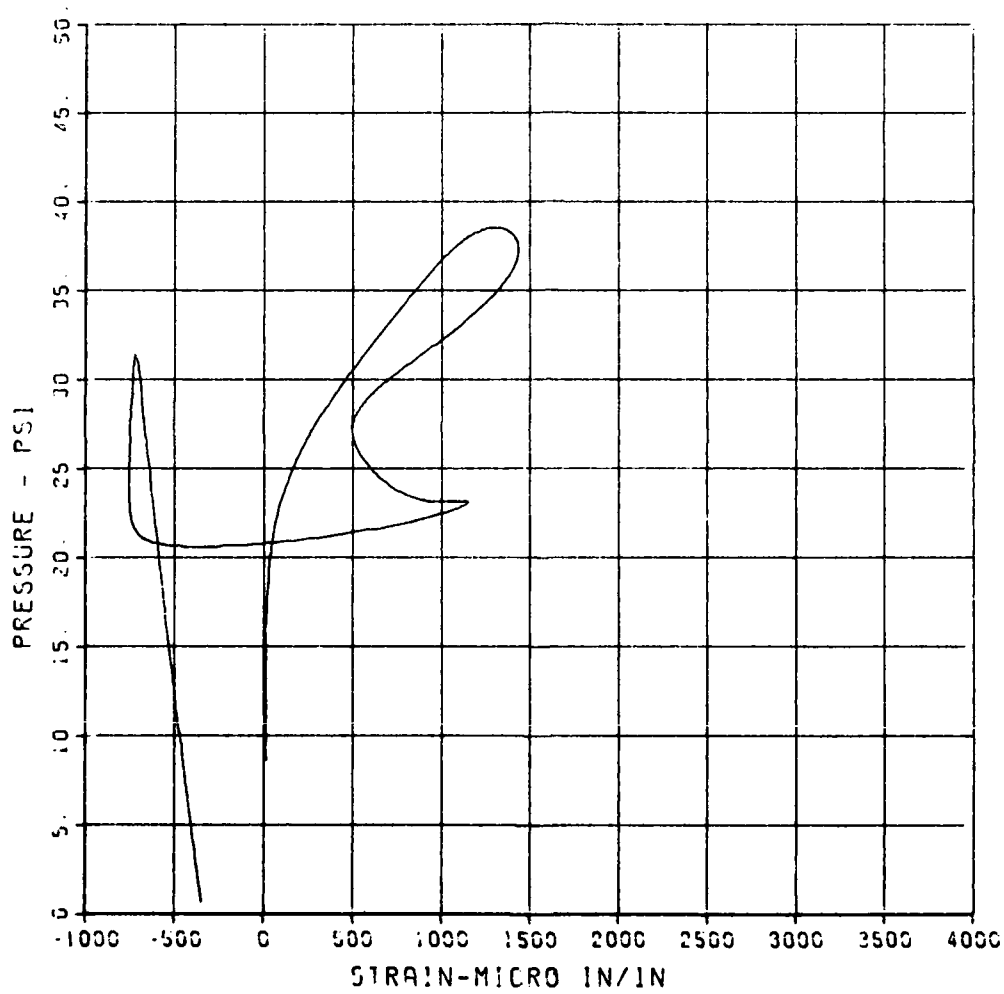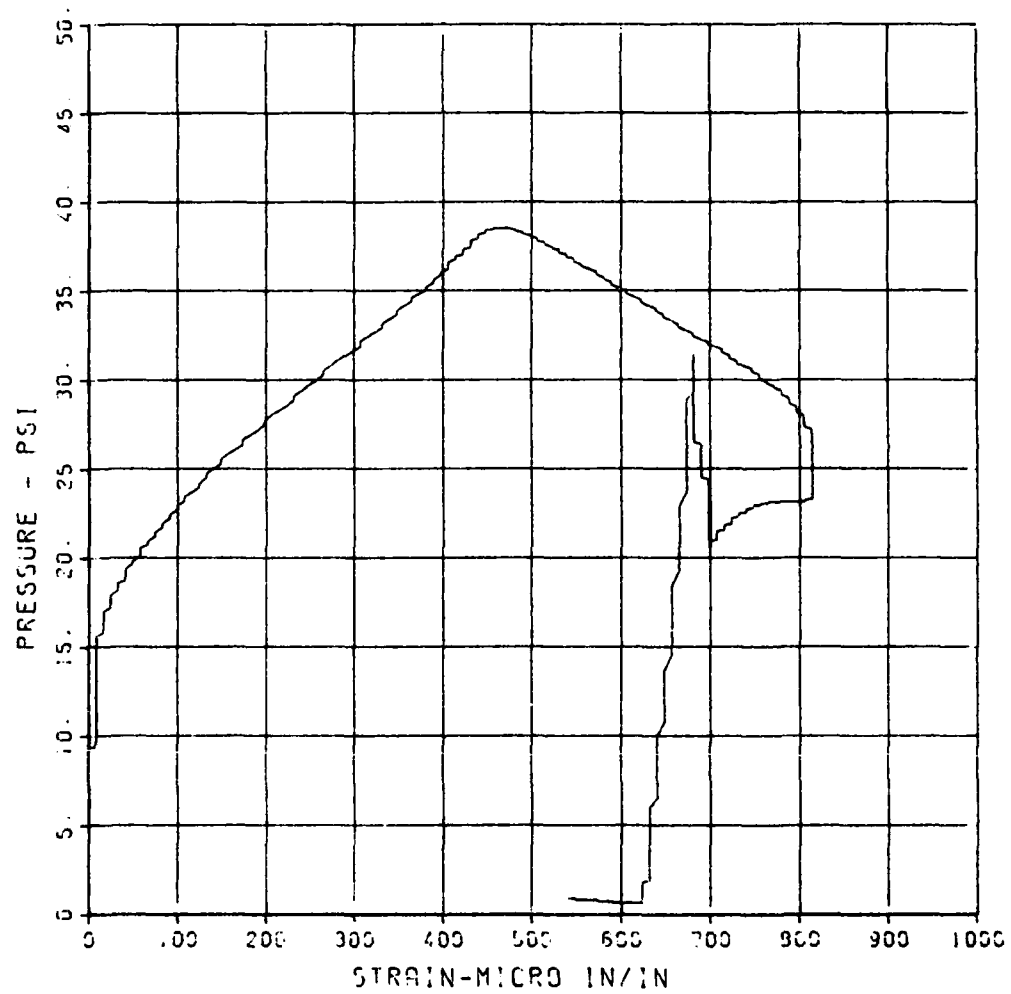
FEMA JD2
E-7
MAXIMUM   SIGMA CAL   CAL VAL
  -4635.3340 4.1221   2305.0

CHANNEL NO. 13   15644 1
08/07/85   R0607
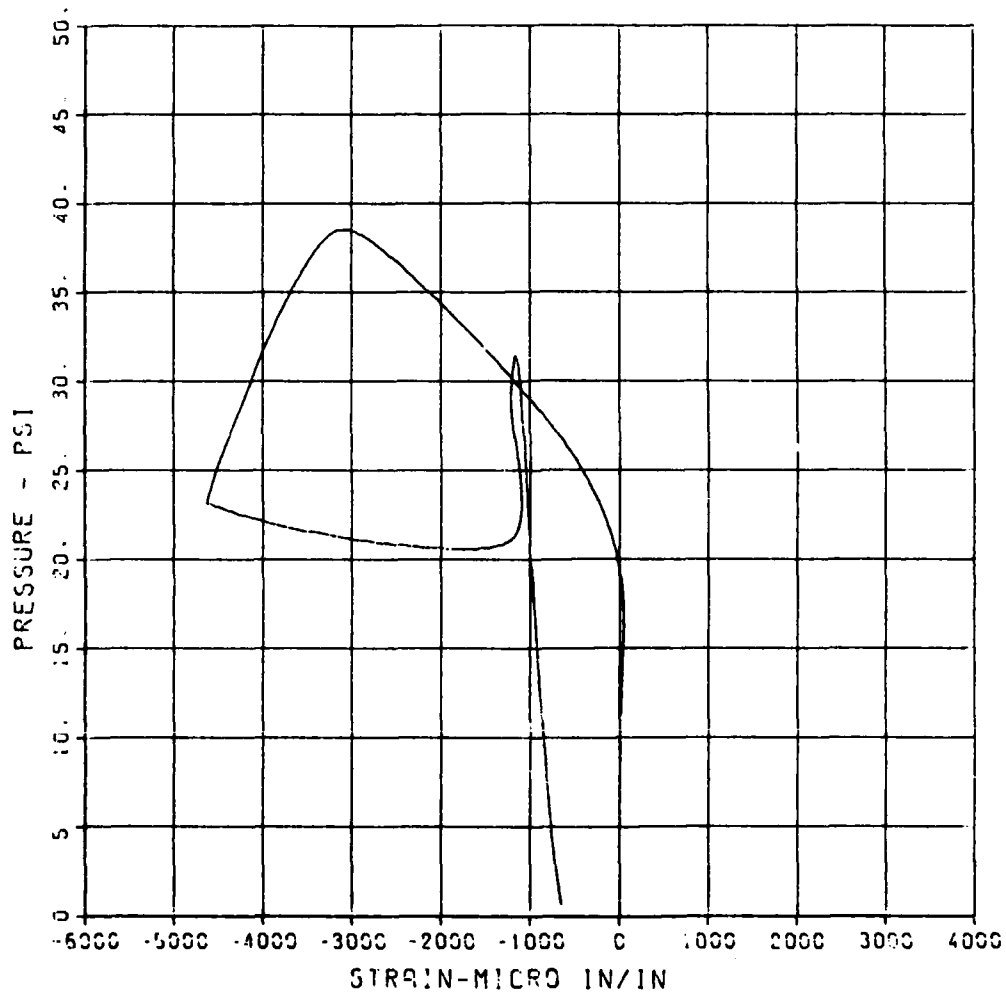
FEMA J02
L-7A
MAXIMUM   SIGMA CAL   CAL CAL
-3435.8628   3.4066   5786.0

CHANNEL NO. 27   15644   1
06/07/85   R0607
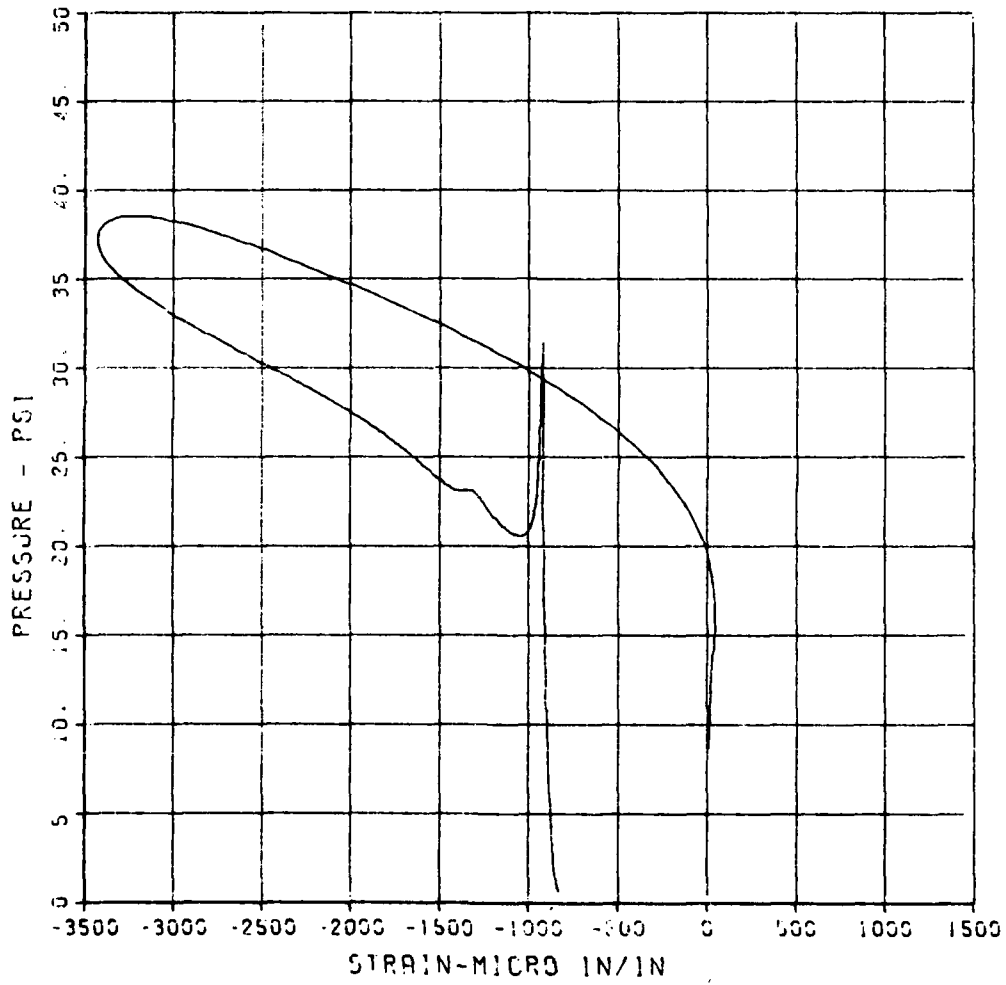
PRESSURE - PSI

STRAIN-MICRO IN/IN

79
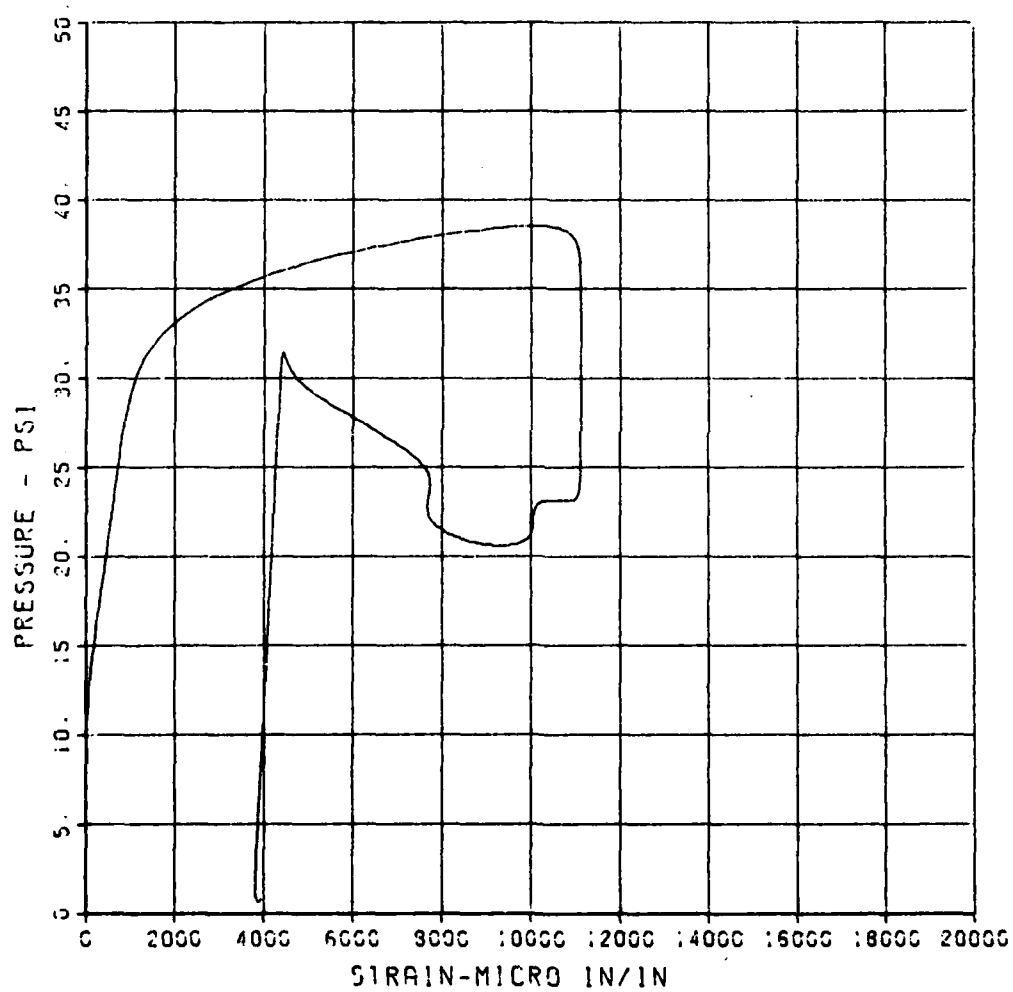
FEMA JD2
E-8A
MAXIMUM   SIGMA CAL   CAL VAL
  23099.7595   2.5535   71480.0

CHANNEL NO. 28   15644   1
08/07/85   R0607
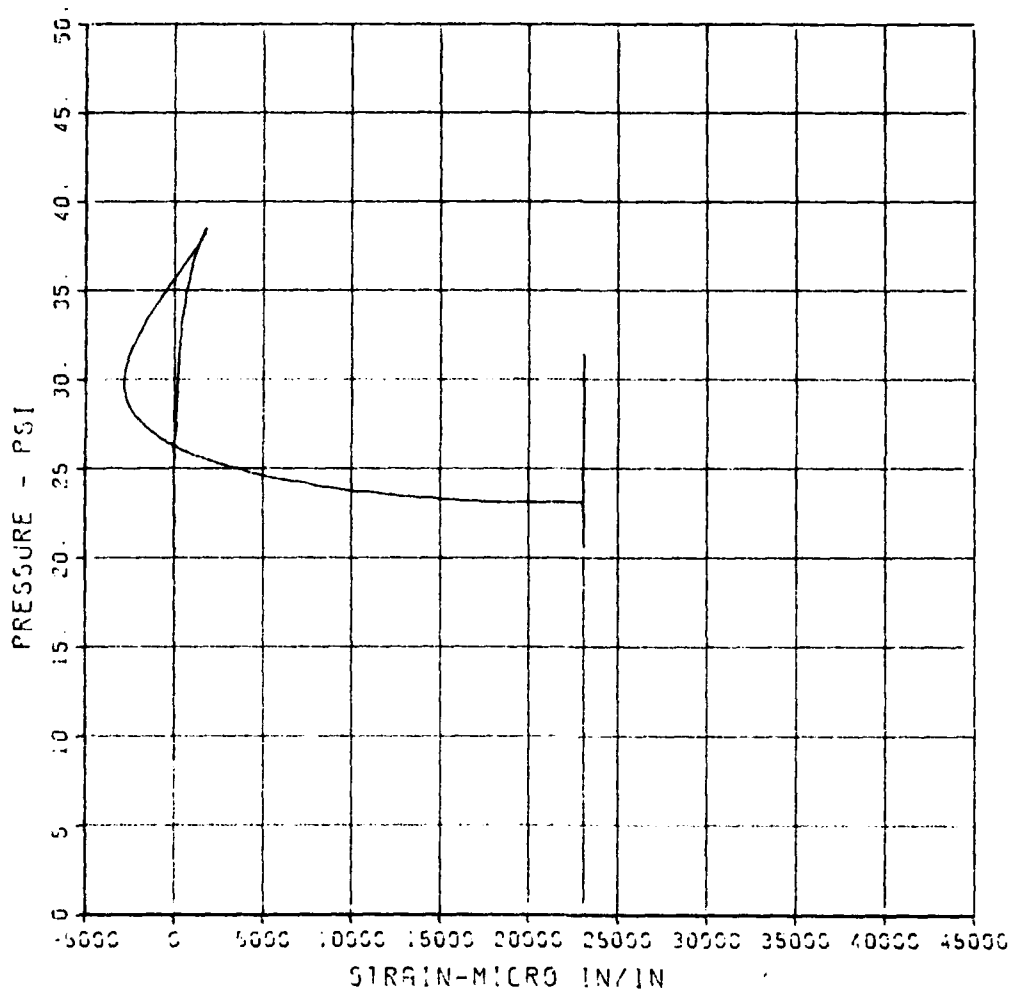
81

MAXIMUM   SIGMA CAL   CAL VAL
  5.2948   3.0230   10.8

CHANNEL NO. 11   15644   1
03/07/85   R0607

PRESSURE - PSI

DISPLACEMENT - IN

FEMA J02
D-2
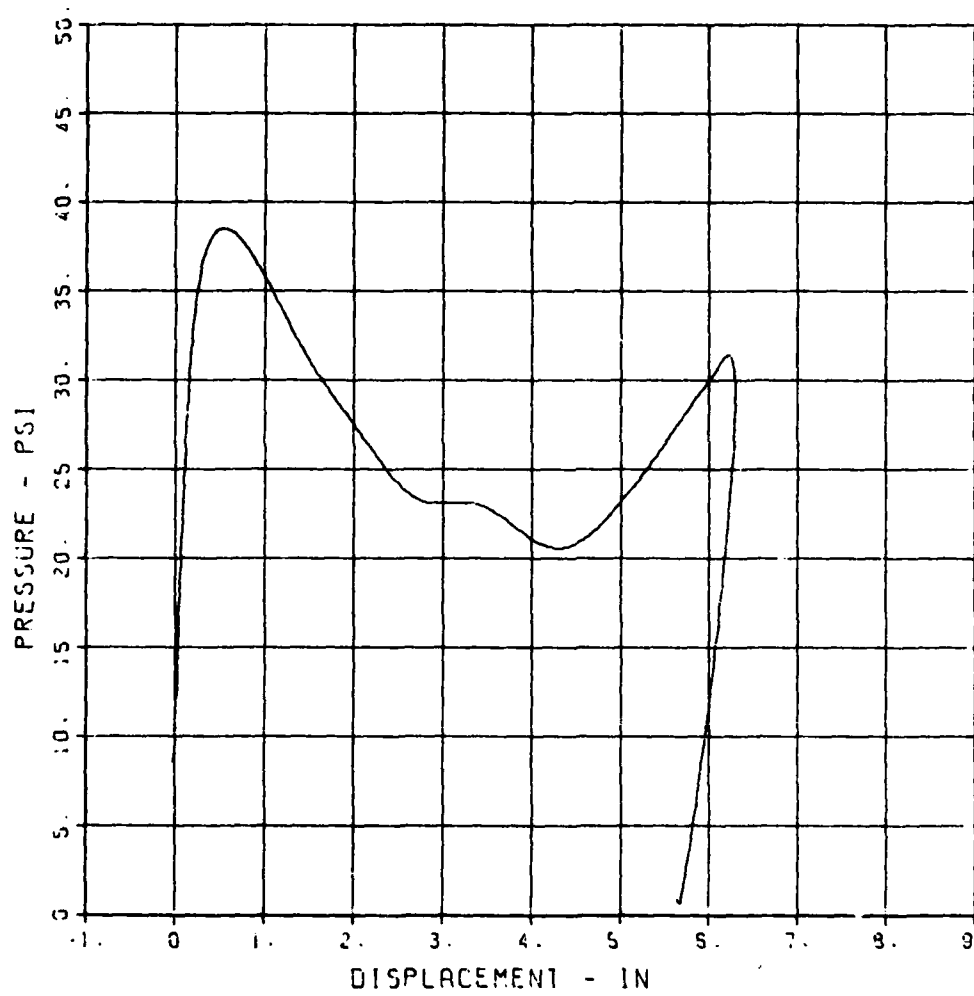
MAXIMUM	SIGMA CAL	CAL VAL
3 1315	2 7195	4 3

CHANNEL NO 4   15544 1
03/07/85   R0602
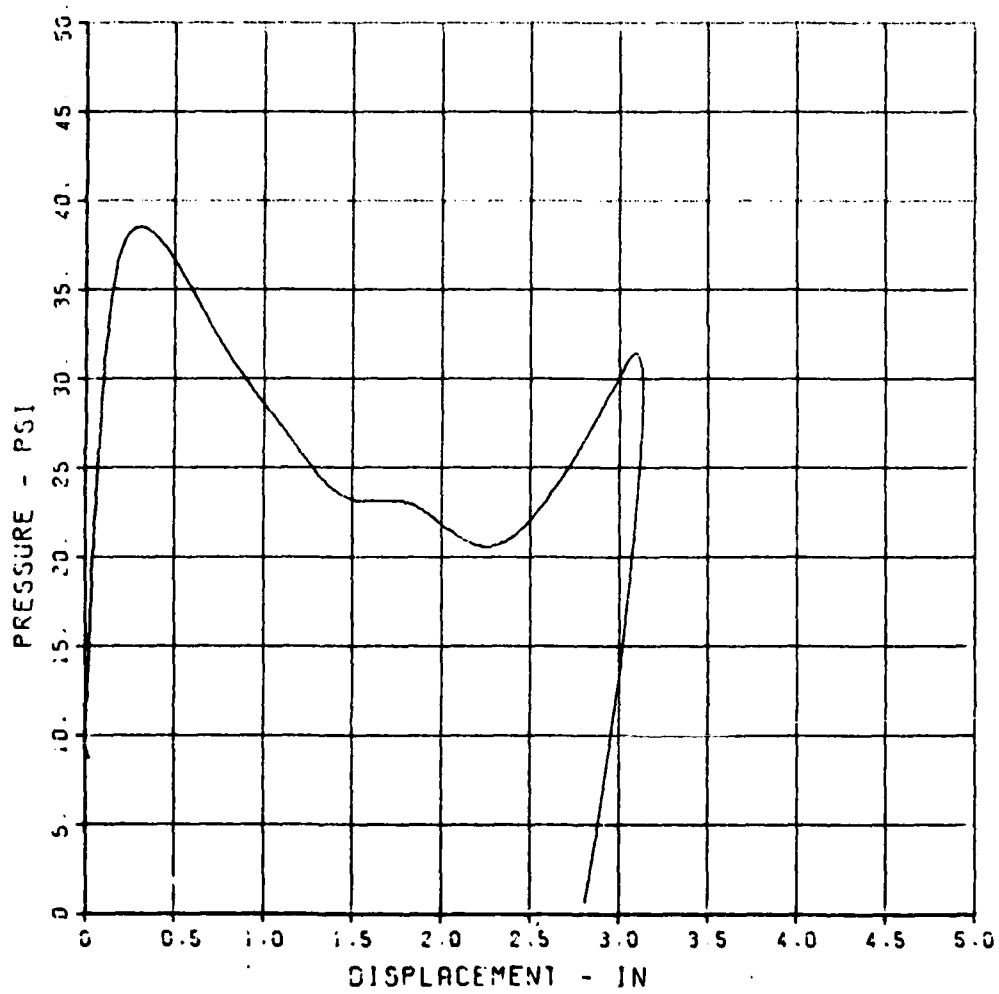
FEMA J3
SE-1

MAXIMUM	SIGMA CAL	CAL VAL
73.3341	1.5574	74.2

CHANNEL NO. 15   15051 1
06/13/84   R0423
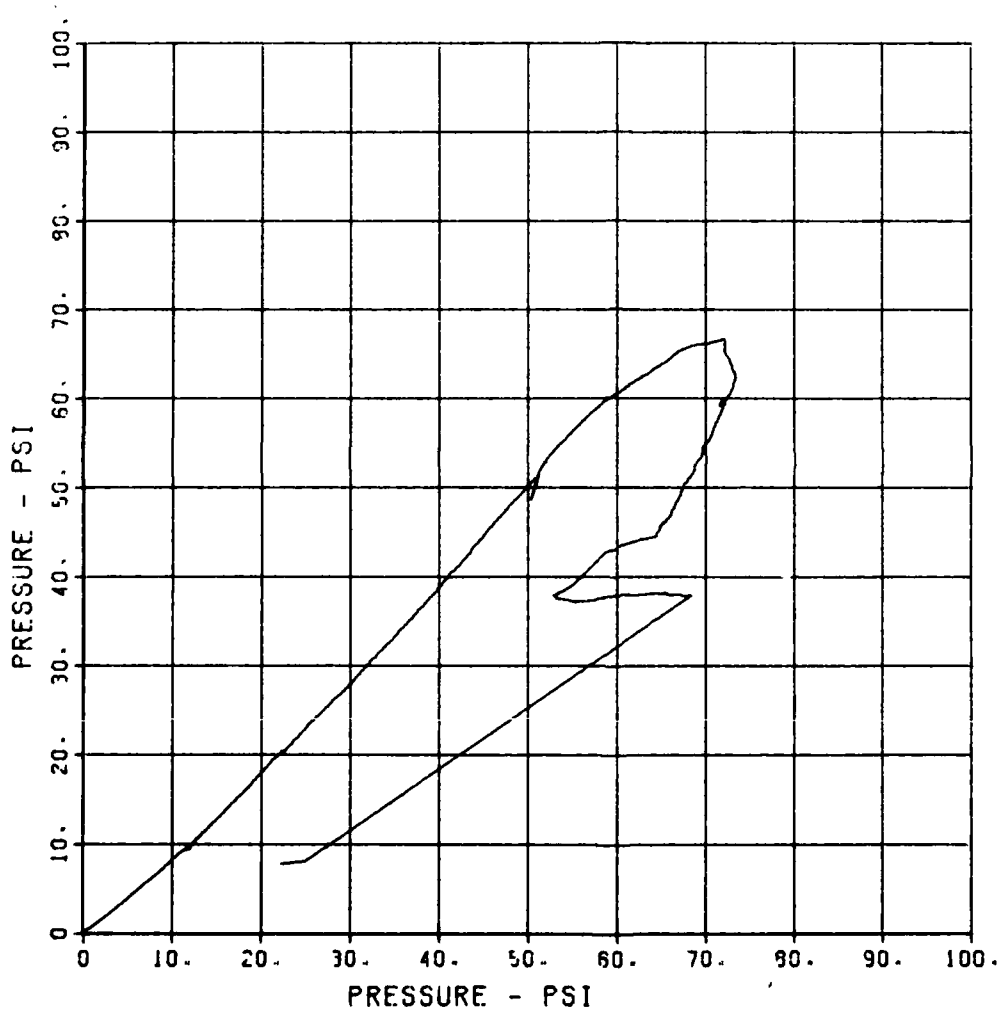
FEMA J3
SE-2
MAXIMUM   SIGMA CAL   CAL VAL
  63.9990   1.2448   70.5

CHANNEL NO. 16   15051   1
06/13/94   R0423

PRESSURE - PSI

PRESSURE - PSI

85
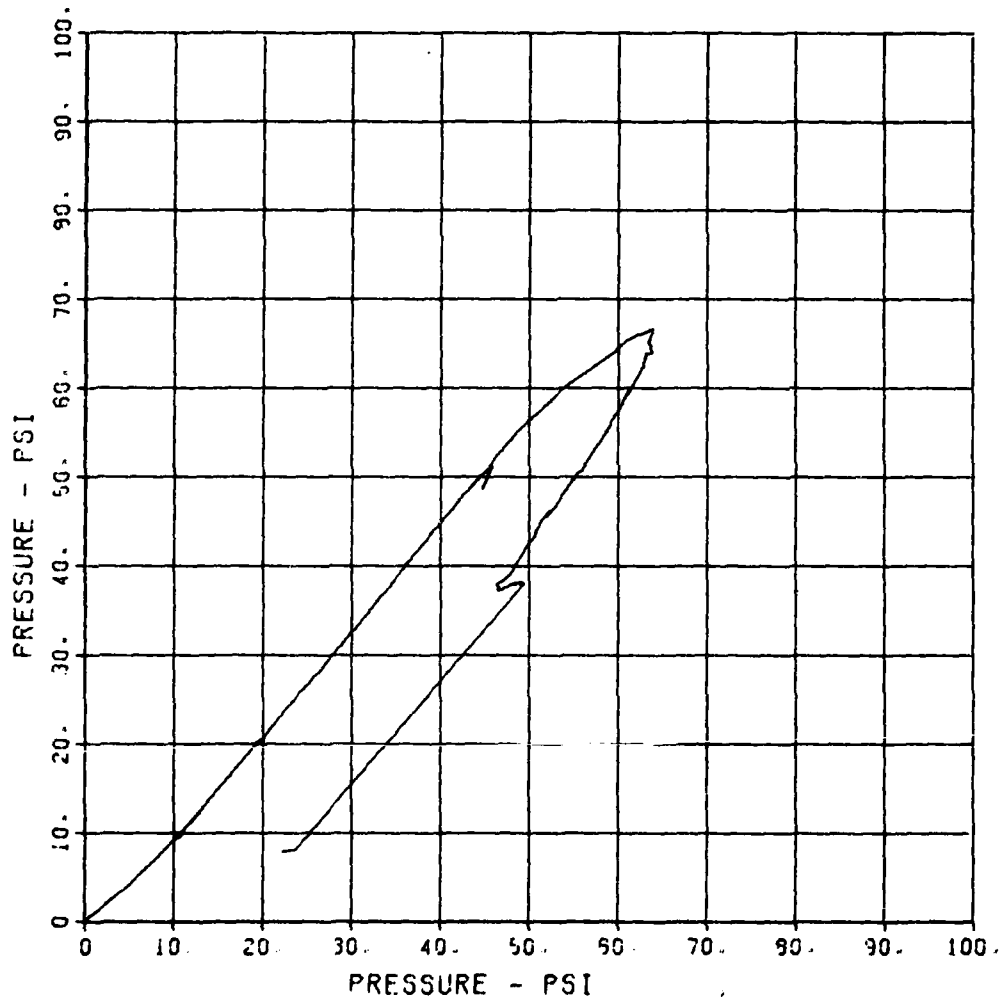
FEMA J3
EO-1
MAXIMUM   SIGMA CAL   CAL VAL
   -1541.3578   1.1315   5780.0

CHANNEL NO. 5   15051   i
06/13/84   RO423
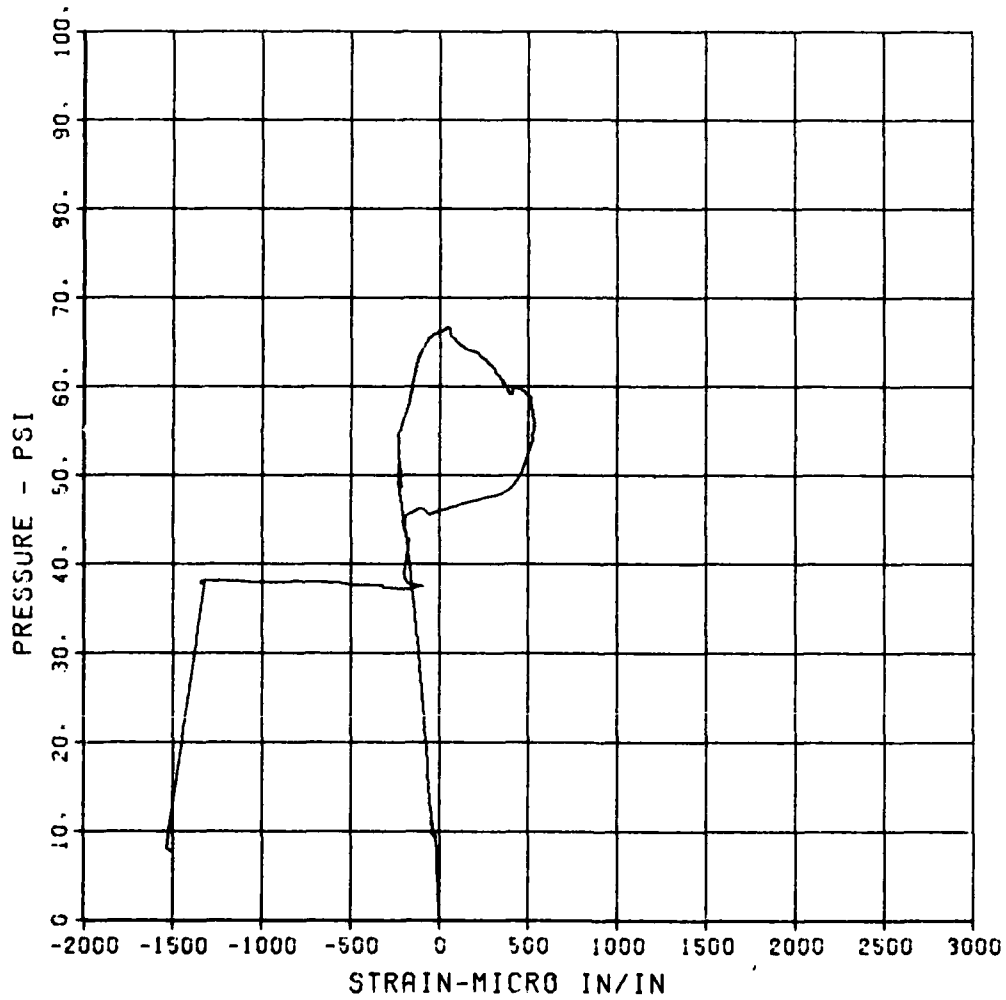
FEMA J3
EO-1S
MAXIMUM   SIGMA CAL   CAL VAL
  -1557.5425   1.5955   5790.0

CHANNEL NO. 17   15051 i
05/13/94   R0423

87
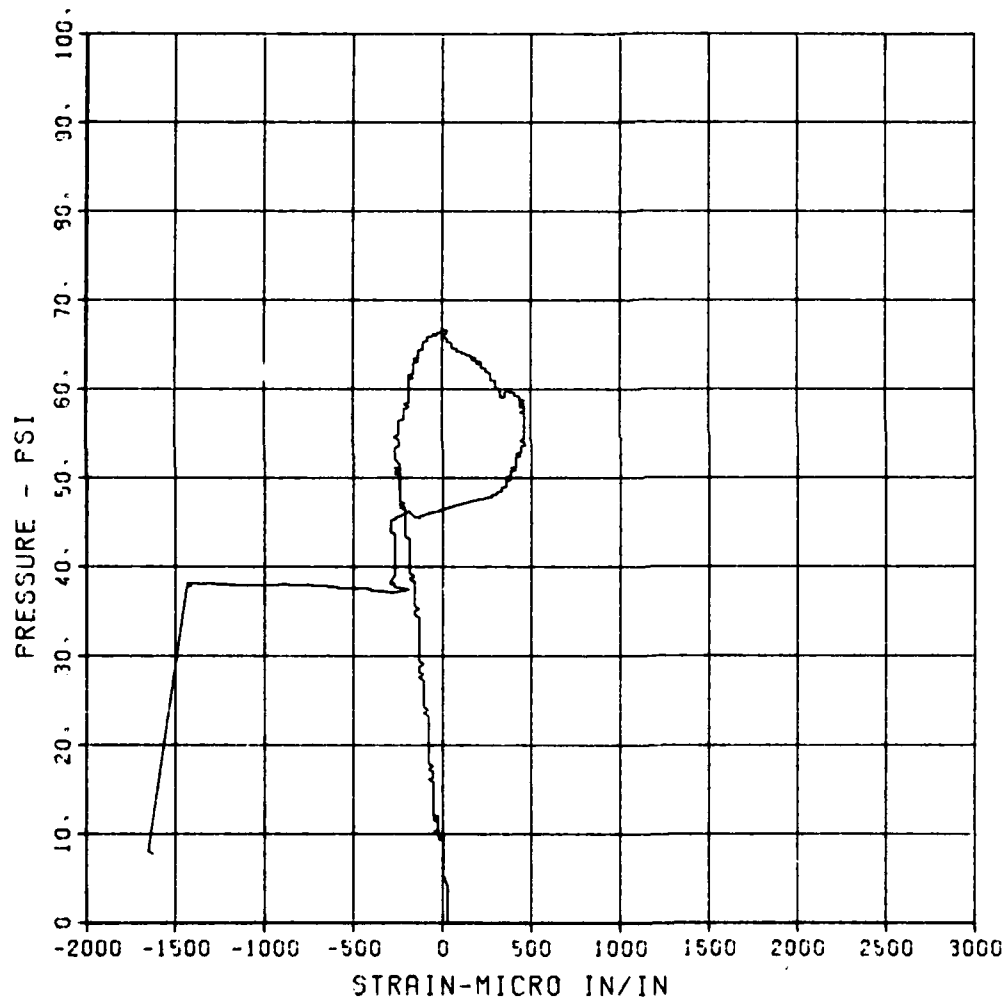
FEMA J3
EI-1
MAXIMUM   SIGMA CAL   CAL VAL
   5202.1517   1.2894   11490.0

CHANNEL NO. 5   15051 i
06/13/94   R0423

88
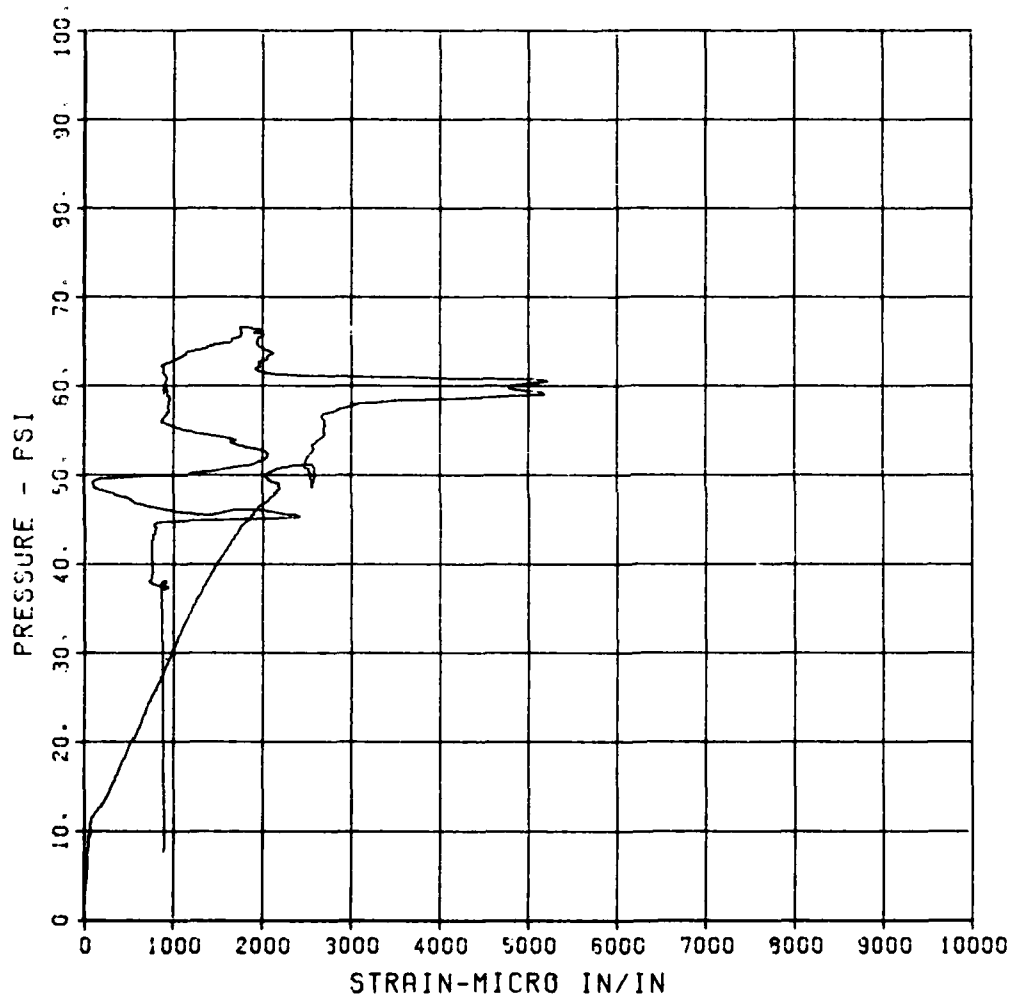
FEMA J3
EO-2
MAXIMUM   SIGMA CAL   CAL VAL
2933.9131   1.2065   9540.0

CHANNEL NO. 7   1505i i
05/13/94   R0423

89
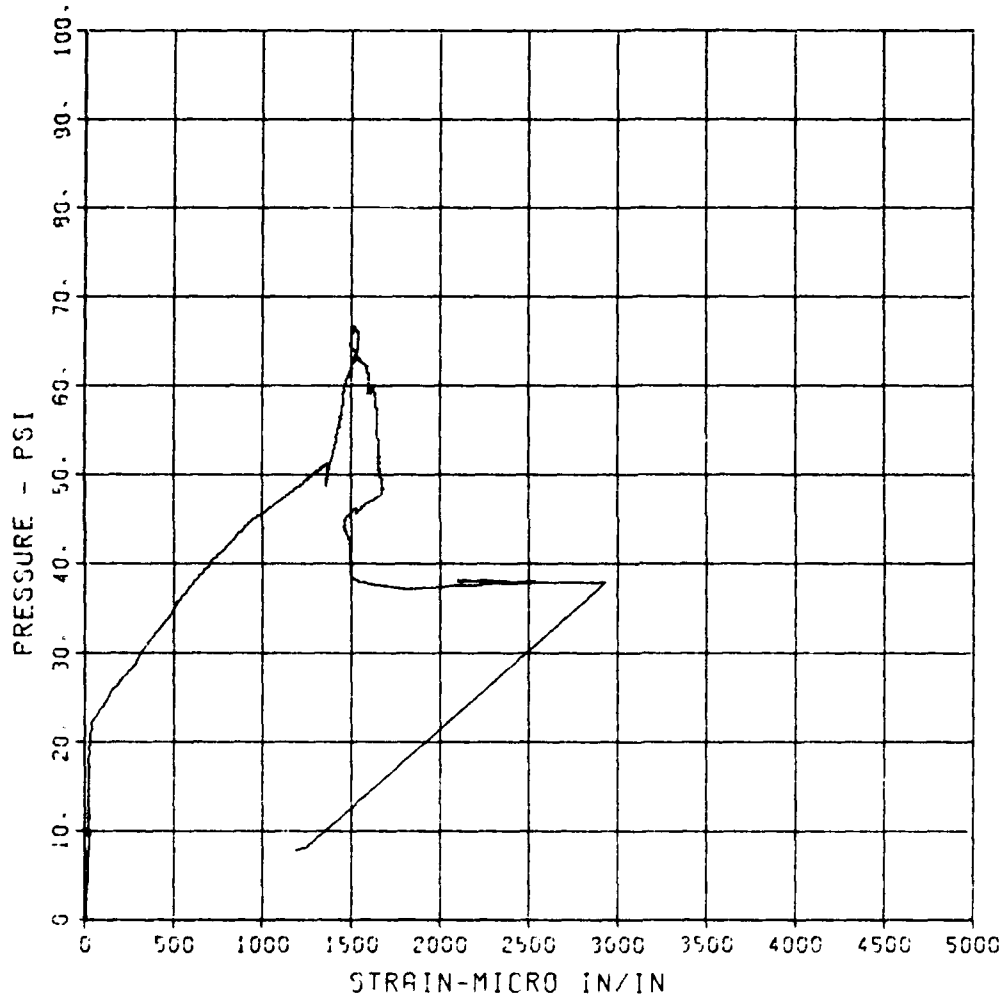
FEMA J3
EI-2

MAXIMUM   SIGMA CAL   CAL VAL
1420.7836   1.4711   9640.0

CHANNEL NO. 9   15051   i
06/13/84   R0423

90
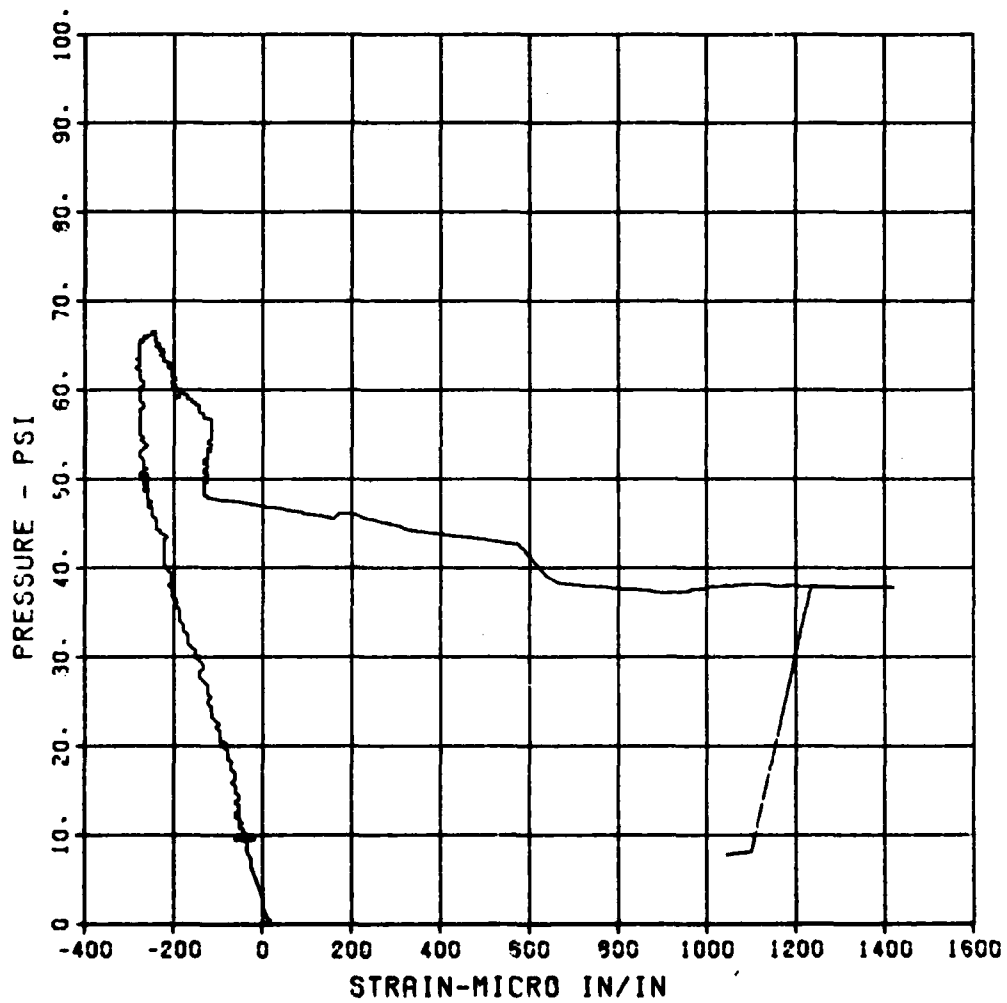
FEMA J3
EO-3
MAXIMUM   SIGMA CAL   CAL VAL
  15942.3322   1.6345   8640.0

CHANNEL NO. 9   15051 i
06/13/94   RO423

91
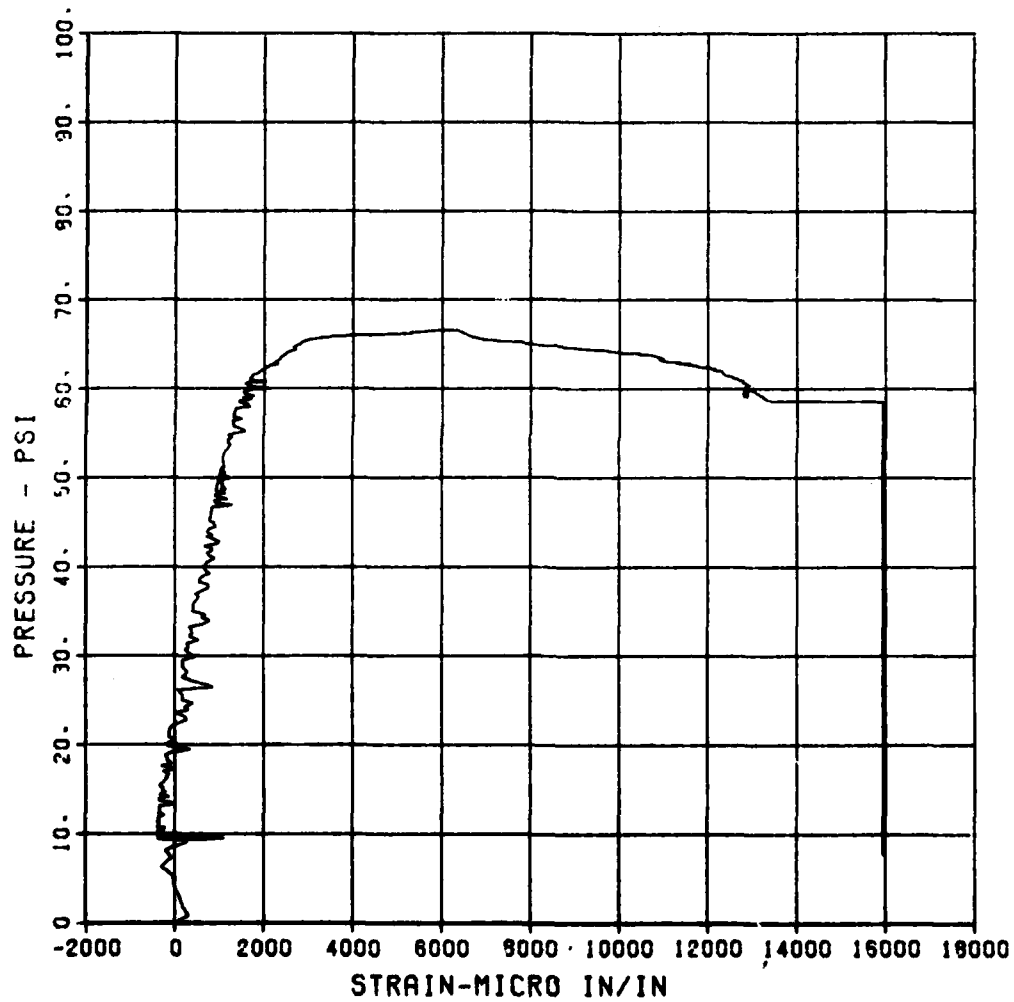
MAXIMUM   SIGMA CAL   CAL VAL
 -2217.7941   9.0765   5790.0

CHANNEL NO. 10   15051 1
06/13/84   R0423

STRAIN-MICRO IN/IN
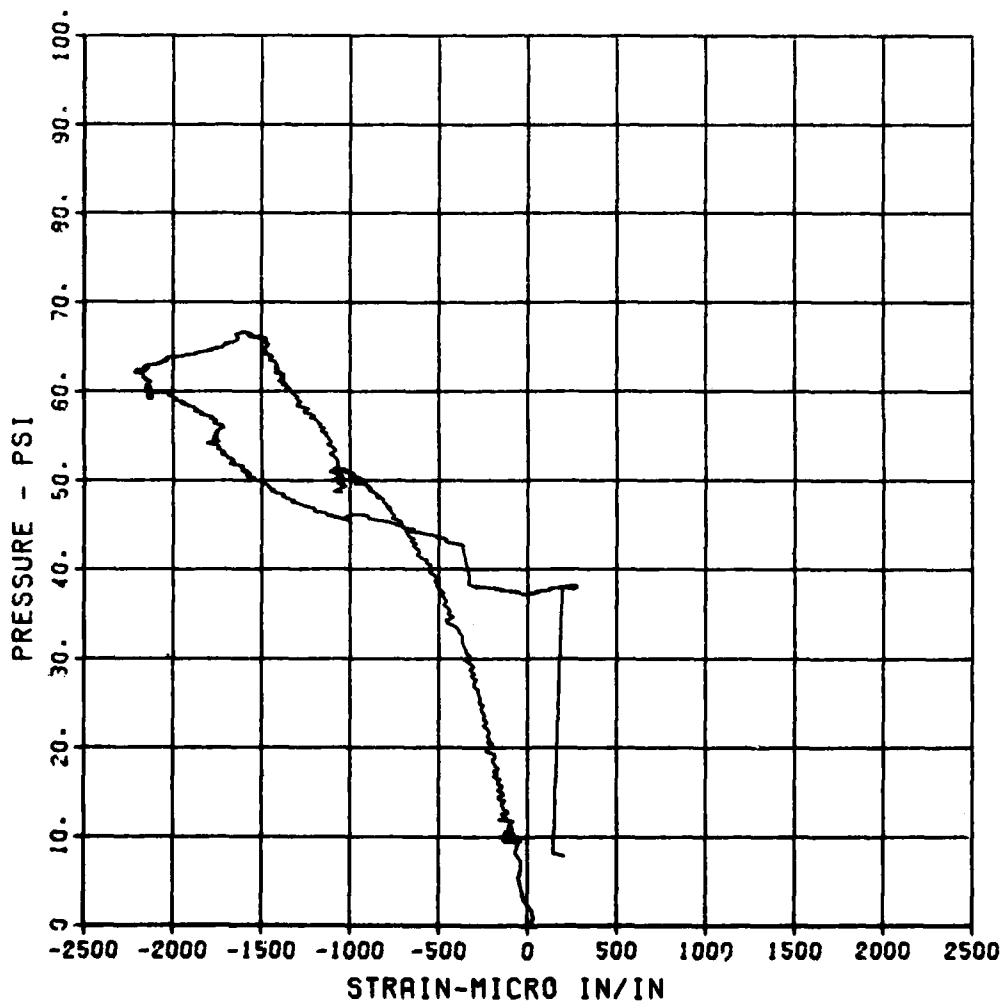
MAXIMUM   SIGMA CAL   CAL VAL
  -2594.0605   1.9908   5780.0

CHANNEL NO. 18   15051 i
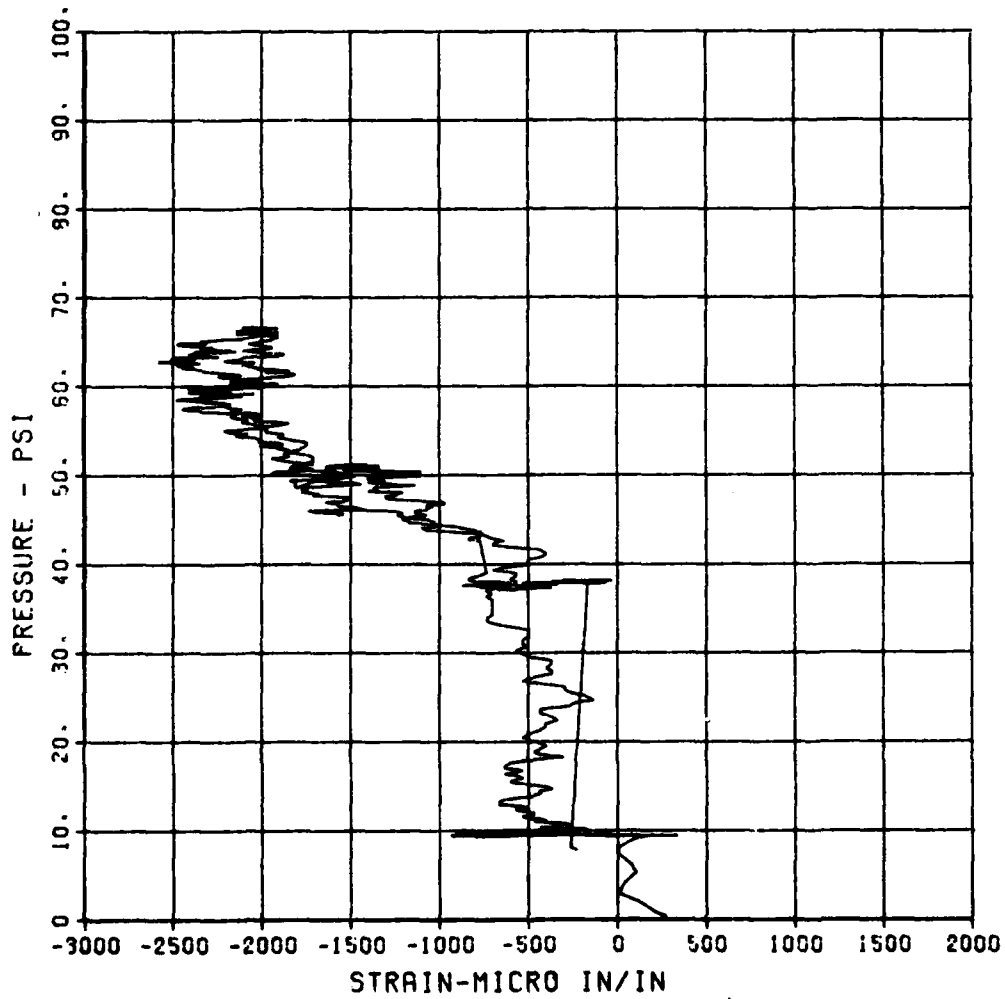
06/13/84   R0423

FEMA J3
EO-4

| MAXIMUM | SIGMA CAL | CAL VAL |
| -4372.0574 | 1.6399 | 5780.0 |

CHANNEL NO. 11   15051   1
06/13/84   RO423

94
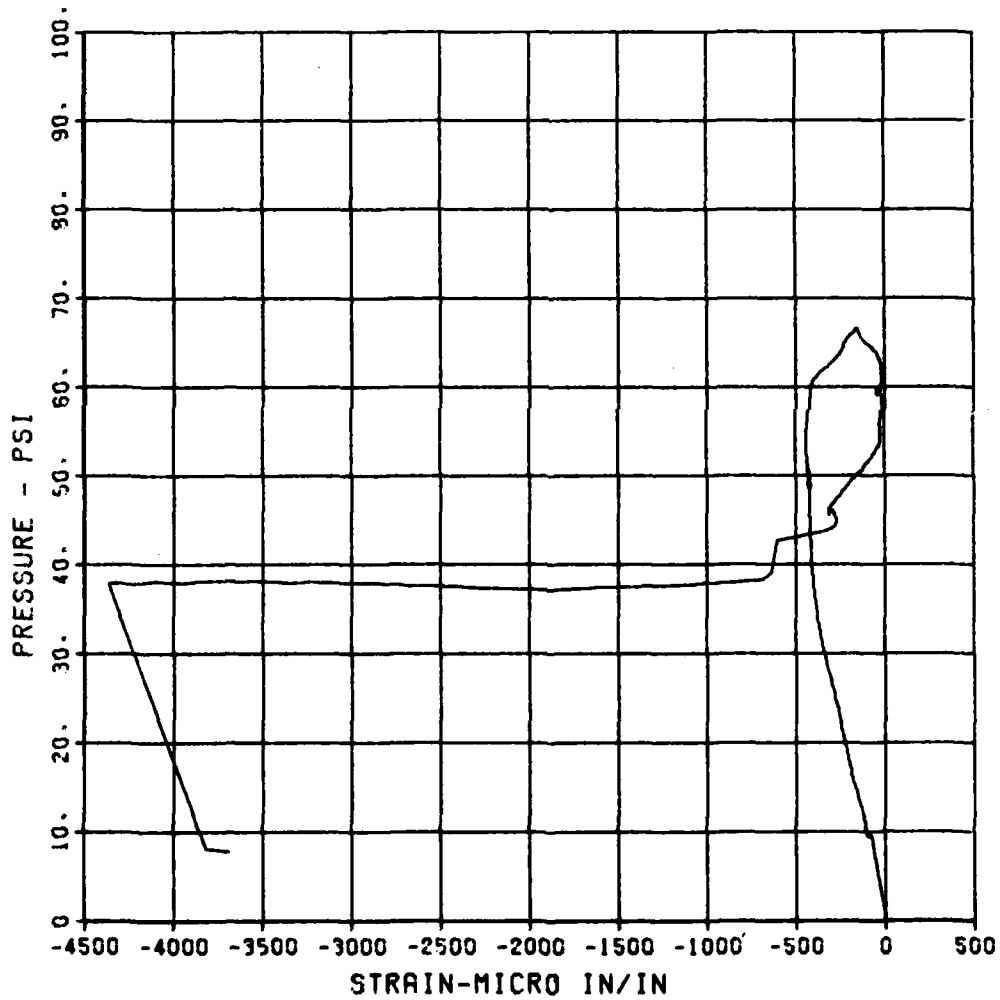
FEMA J3
EI-4
MAXIMUM   SIGMA CAL   CAL VAL
   1294.0896   1.5145   5790.0

CHANNEL NO. 12   1505i i
06/13/84   R0423
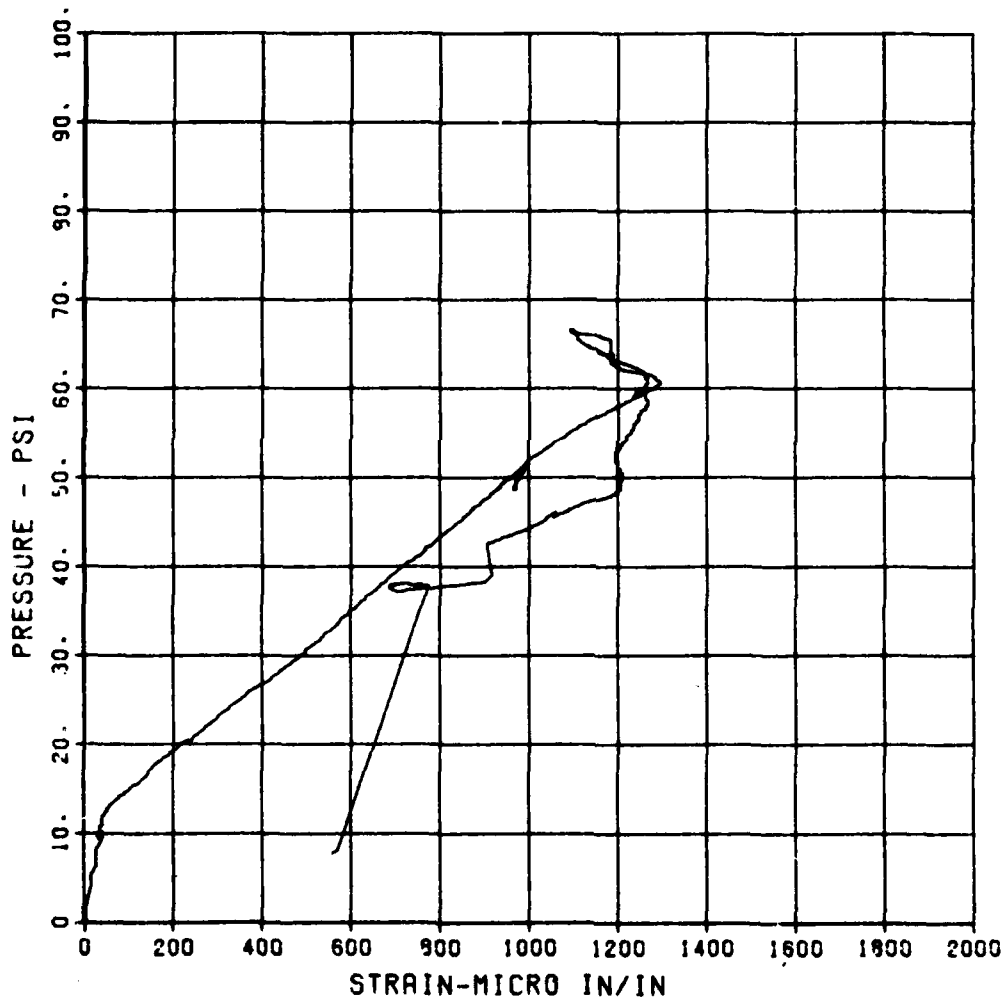
FEMA J3
ED-1
MAXIMUM   SIGMA CAL   CAL VAL
2043.5796   1.6731   5790.0

CHANNEL NO. 13   15051   i
06/13/94   R0423
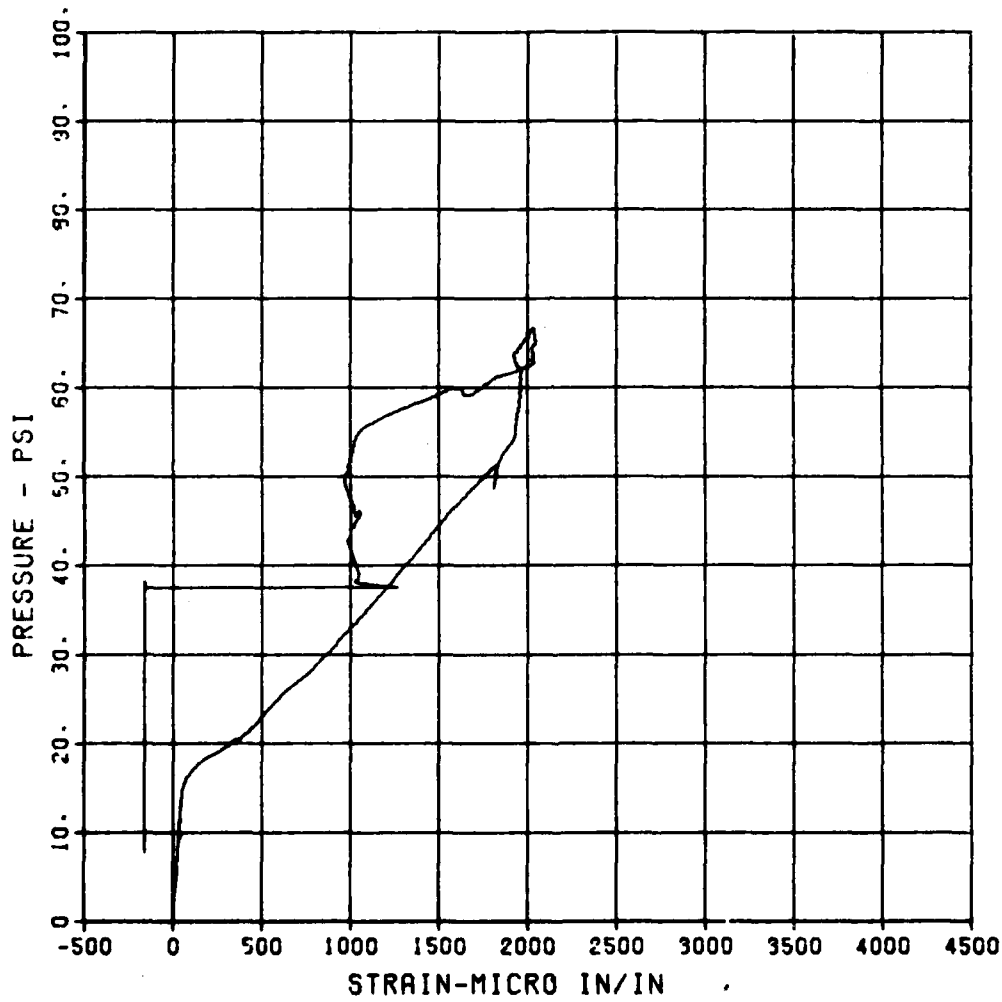
FEMA J3
ED-2
MAXIMUM   SIGMA CAL   CAL VAL
  -11945.7603 1.6750   5780.0

CHANNEL NO. 14   15051 i
06/13/84   R0423
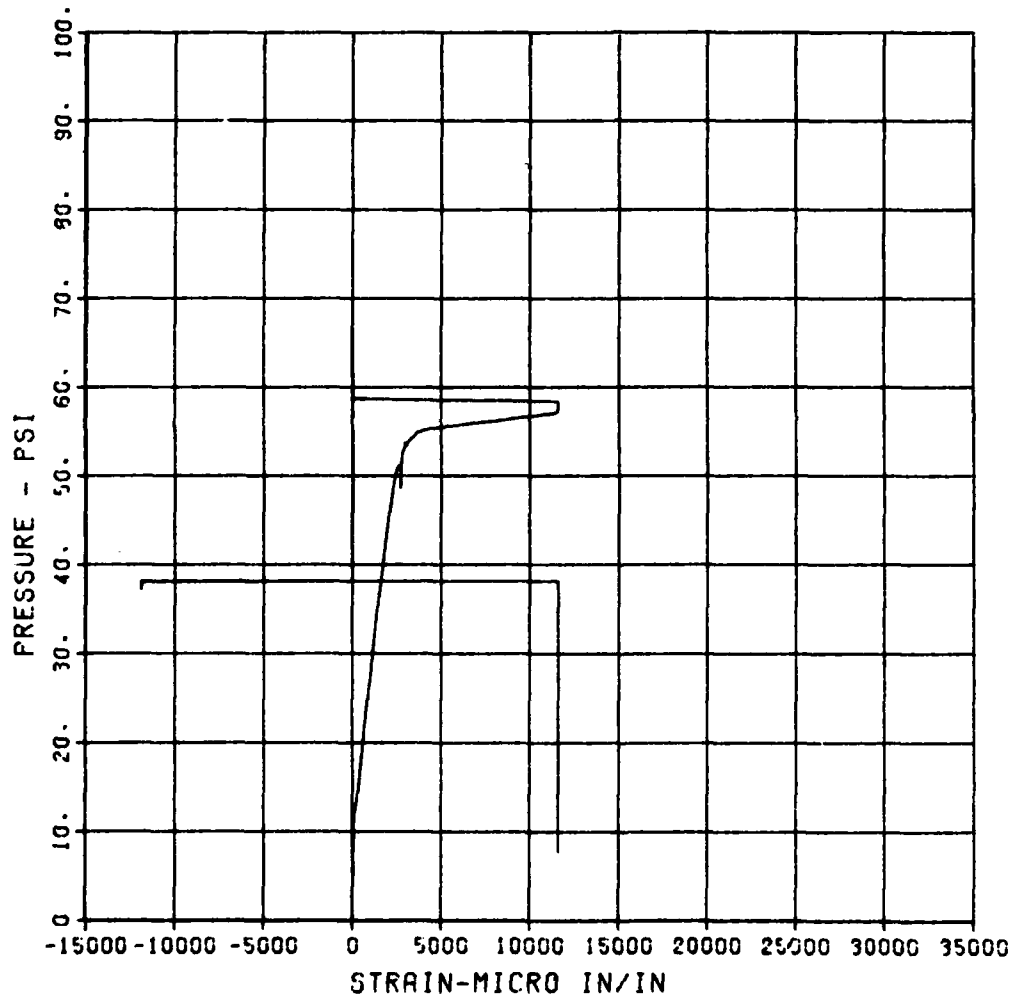
FEMA J3
D-1
MAXIMUM   SIGMA CAL   CAL VAL
4.8434   1.2698   7.0

CHANNEL NO. 3   1505i i
06/13/84   R0423

PRESSURE - PSI vs DISPLACEMENT - IN

98
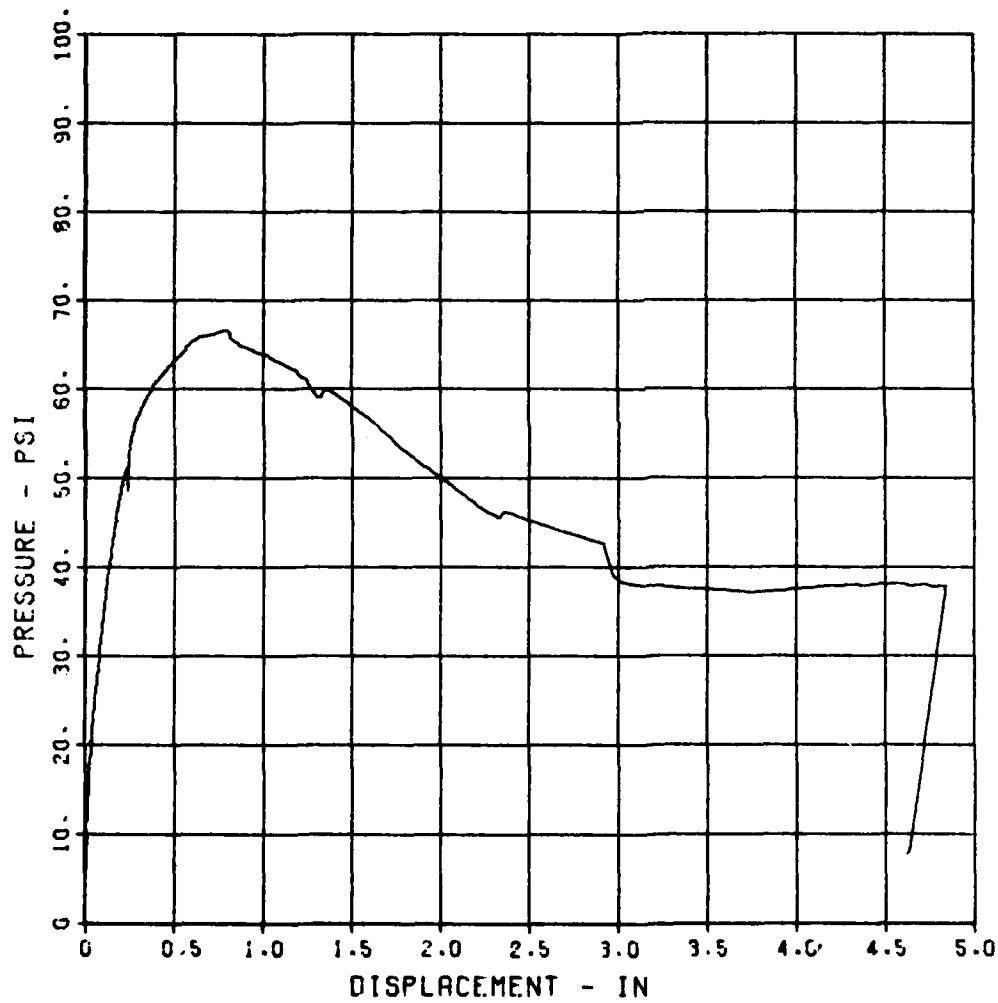
FEMA J3
D-2

MAXIMUM	SIGMA CAL	CAL VAL
3.3202	1.9691	4.3

CHANNEL NO. 4   15051 i
06/13/94   R0423
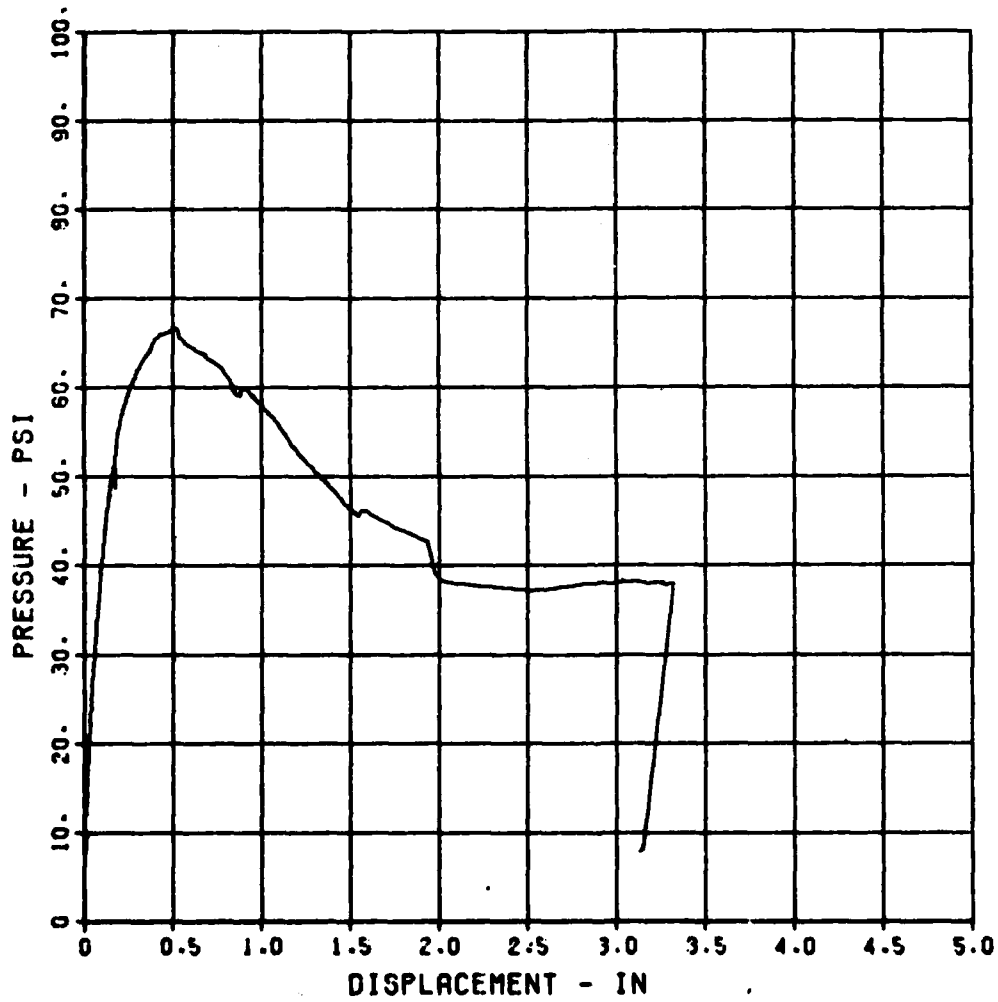
Director, Federal Emergency Management Agency
    ATTN: Mr. Tom Provenzano (6 Cys)
500 C St. SW
Washington, DC 20472

Commander, US Army Engineer District, Wilmington
    ATTN: Pat Burns/Library
PO Box 1890
Wilmington, N. C. 28402

Headquarters, Department of Energy
    ATTN: Library/G-049 MA-232.2/GTN
Washington, DC 20545

National Bureau of Standards
    ATTN: Mr. Samuel Kramer
        Dr. Lewis V. Spencer
Washington, DC 20234

Associate Director, Natural Resources and
  Commercial Services
Office of Science & Technology
    ATTN: Mr. Phillip M. Smith
Executive Office Building
Washington, DC 20500

Director, Office of Administration
Program Planning and Control
Department of Housing and Urban Development
    ATTN: Mr. Bert Greenglass
Washington, DC 20410

Director, Defense Nuclear Agency
    ATTN: SPTD/Mr. Tom Kennedy
        STTL/Technical Library
Washington, DC 20305

Director, Defense Intelligence Agency
    ATTN: Mr. Carl Wiehle (DB-4C2)
Washington, DC 20301

Assistant Secretary of the Army (R&D)
    ATTN: Assistant for Research
Washington, DC 20301

Office, Chief of Engineers,
  Department of the Army
    ATTN: DAEN-RDZ-A
        DAEN-ECE-D
Washington, DC 20314

Sandia Corporation
    ATTN: Dr. Clarence R. Mehl
Dept. 5230
Box 5800, Sandia Base
Albuquerque, N. Mex. 87115

Director, US Army Engineer Waterways
  Experiment Station
    ATTN: Dr. S. A. Kiger
        Mr. Bill Huff
        Mr. Stan Woodson
        Mr. Tom Slawson
5 cys   Library
PO Box 631
Vicksburg, Miss. 39180

Defense Technical Information Center
12 cy ATTN: (DTIC-DDAB/Mr. Myer B. Kahn)
Cameron Station
Alexandria, Va. 22314

Commander, US Army Materials and Mechanics
  Research Center
    ATTN: Technical Library
Watertown, Mass. 02172

Command and Control Technical Center
Department of Defense
Room 2E312 Pentagon
Washington, DC 20301

Oak Ridge National Laboratory
    ATTN: Greg Zimmerman
        Conrad Chester
Oak Ridge, Tenn. 37831

Los Alamos Scientific Laboratory
    ATTN: Report Library MS-364
PO Box 1663
Los Alamos, N. Mex. 87544

Director, Ballistic Research Laboratory
    ATTN: (DRXBR-TBD/Mr. George Coulter)
Aberdeen Proving Ground, Md. 21005

Commanding Officer, Office of Naval Research
Department of the Navy
Washington, DC 20390

Commanding Officer
US Naval Civil Engineering Laboratory
Naval Construction Battalion Center
    ATTN: Library (Code L08A)
Port Hueneme, Calif. 93043

Commander, Air Force Weapons Laboratory/SUL
    ATTN: Technical Library
Kirtland Air Force Base, N. Mex. 87117

Civil Engineering Center
AF/PRECET
Tyndall AFB, Fla. 32403

University of Florida
Civil Defense Technical Services
College of Engineering
Department of Engineering
Gainesville, Fla. 32601

Technical Reports Library
Kurt F. Wendt Library
College of Engineering
University of Wisconsin
Madison, Wisc. 53706

Agbabian Associates
250 N. Nash Street
El Segundo, Calif. 90245

AT&T Bell Laboratories
    ATTN: Mr. E. Witt
Whippany, N. J. 07981

James E. Beck & Associates
4216 Los Palos Avenue
Palo Alto, Calif. 94306

Chamberlain Manufacturing Corp.
GARD, Inc.
7449 N. Natchez Avenue
Niles, Ill. 60648

ITT Research Institute
    ATTN: Mr. A. Longinow
10 West 35th Street
Chicago, Ill. 60616

H. L. Murphy Associates
Box 1727
San Mateo, Calif. 94401

RAND Corporation
    ATTN: Document Library
1700 Main Street
Santa Monica, Calif. 90401

Research Triangle Institute
    ATTN: Mr. Edward L. Hill
PO Box 12194
Research Triangle Park, N. C. 27709

Scientific Services, Inc.
517 East Bayshore Drive
Redwood City, Calif. 94060

US Army Engineer Division, Huntsville
    ATTN: Mr. Paul Lahoud (10 Cys)
PO Box 1600, West Station
Huntsville, Ala. 35807

EVALUATION OF ALTERNATE ROOF-WALL DETAILS FOR THE KEYWORKER BLAST SHELTER, Unclassified, US Army Engineer Waterways Experiment Station, October 1987, 106 pp.

At the time this study was initiated, several civil defense policy options were being analyzed for protection of the Nation's industrial capability and keyworkers. One option under consideration called for construction of blast shelters to protect keyworkers remaining in high-risk areas during a National crisis. In support of this option, the Federal Emergency Management Agency (FEMA) tasked the US Army Engineer Division, Huntsville (HND), to develop Keyworker shelter designs. The design required an earth-covered shelter to resist the radiation and blast effects of a 1-MT nuclear detonation at the 50-psi peak overpressure level. Personnel in the Structures Laboratory of the US Army Engineer Waterways Experiment Station (WES) supported HND with design calculations and design verification experiments.

In the construction of a large number of shelters, it is important that the shelter design provide the required structural capacity at reasonable costs. The original roof-wall reinforcement detail created a constructibility problem that increased construction costs. The objective of the experimental program described in this report was to evaluate alternate roof-wall joint details for the Keyworker blast shelter in an effort to improve constructibility without reducing structural capacity.

Three 1/4-scale reinforced concrete box-type models were statically tested under uniform water pressure in the 6-foot-diameter Small Blast Load Generator at WES. Two of the models (JD1 and JD3) were reinforced similarly to the original design except for the joint details. The third model (JD2) was similar to the proposed final shelter design as evaluated in the prototype tests conducted by WES. Based on test results, detail JD2 is recommended for use in the final design.