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key words:
1- base plates, columns
2- research, U of California
3- connections, flexible

May 22, 1989

Mr. Gus Bergsma
Culp & Tanner Inc.
23686 Birtcher Drive
El Toro, CA 92630

author - Astaneh

AISC Fellowship

Dear Gus:

Professor Astaneh has sent a copy of your report on "Moment Rotation Characteristics of a Simple Steel Base Plate Connection." The work is a fundamental start on a problem that has been given much attention for a long time, and your study is most welcome.

We will be looking for your submittal to our Engineering Journal in the near future. We wish you the very best for your future.

Sincerely,

Robert F. Lorenz

Robert F. Lorenz
Director of Education & Training

RFL/aa

cc: G. Haaijer
N. Iwankiw

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CIVIL ENGINEERING BERKELEY, CALIFORNIA 94720

Robert Lorenz,
Director, Education and Training
AISC
400 N. Michigan Avenue
Chicago, IL 60611-4185

May 15,1989

Dear Bob:

Enclosed is a copy of the report prepared by Gus Bergsma on "Moment-Rotation Characteristics of a Simple Steel Base Plate Connection". The work was supported by a fellowship from the AISC Education Foundation in 1988.

I presented some of the results in a poster session at the 1989 Earthquake Engineering Research Institute Annual Meeting and received many excellent comments from the professionals attending the meeting. Currently, one of my Ph. D. graduate students is working on this project to analyze the experimental data obtained by Gus in order to develop design procedures and mathematical models of behavior. Gus and I are planning to submit a paper on his work to the Engineering Journal in the near future.

In closing, I was saddened to read in your newsletter that AISC fellowship program is discontinued. Certainly, in my view the fellowship program was one of the most efficient and best research programs in structural engineering around. After all, where can one obtain this much information (e.g. attached report) for only $5,000? I hope that the fellowship program will be restored.

Thank you for the support.

Sincerely yours,

Abolhassan Astaneh-Asl.

Hassan Astaneh

cc: Gus Bergsma, Culp & Tanner Inc. 23686 Birtcher Dr.,
El Toro, CA 92630

CE 299 REPORT
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STRUCTURAL ENGINEERING,
MECHANICS AND MATERIALS

MOMENT-ROTATION CHARACTERISTICS OF A SIMPLE STEEL BASE PLATE CONNECTION

by
Gus Bergsma

Faculty Superviser: A. ASTANEH

DEC 1988

DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF CALIFORNIA
BERKELEY, CALIFORNIA

MOMENT-ROTATION CHARACTERISTICS OF A SIMPLE STEEL BASE PLATE CONNECTION

by
Gus Bergsma

A. Astaneh-Asl, Supervisor

J. P. Moehle, 2nd Supervisor

Individual Research Project
University of California, Berkeley
Department of Civil Engineering
Division of Structural Engineering, Mechanics and Materials
December 1988

Submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

TABLE OF CONTENTS

ABSTRACT

This paper presents the results of six tests on simple steel column-base plate subassemblies. The problems of designing and current methods in base plate design are discussed first. This is followed by a discussion of the program of research that was to be performed including the design, instrumentation, parameters to be studied and loading of the specimens. Finally, the experimental results are presented followed by a brief summary.

ACKNOWLEDGMENTS

This report is submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering at the University of California, Berkeley.

The author wishes to express his appreciation to Professor Abolhassan Astaneh-Asl for his continued support during this research investigation.

The author would also like to acknowledge the financial support of the American Institute of Steel Construction Fellowship Program.

Many thanks also to Professor Jack P. Moehle.

I INTRODUCTION

1.1 Statement of the problem - Base plates are used in most, if not all of the steel structures built today. Standard design of base plates given in Chapter 3 of the AISC manual[1] for pinned end columns considers the column base plate to be a perfect pin at the foundation. However, there does exist an amount of fixity that should be accounted for in design, particularly with regard to design and behavior of frames. The relationship between applied moment to the connection and rotation occurring at the connection needs to be better understood, so that behavior of frames may be more reliably assessed.

1.2 Objectives - The objectives of the proposed research was to conduct several tests to understand the moment-rotation behavior for simple steel base plate connections under cyclic loading. Through testing, report on the behavior of this connection during cyclic loading and on the types of failure mechanisms. Establish research needs in the area of seismic design and behavior of steel column base plates.

1.3 Scope - To accomplish the stated objectives, six test were performed on six different steel column and base plate subassemblies (Figure 1). Basically only the axial load in the column and the base plate thickness were the parameters chosen to be varied throughout the six test. The main response measured was the moment-rotation at the base. There were several other response quantities measured and they are discussed subsequently.

II REVIEW OF CURRENT METHODS IN PINNED BASE PLATE DESIGN

2.1 Introduction - Aside from chapter three of the AISC manual[1] there are only a few papers discussing the behavior and design of simple and/or fixed column base plate connections. Most recently are References 2,3 and 4. Basically there are two approaches to base plate design: (I) working stress, where the design is based on the loading at service and (II) Ultimate strength, where the design is based on ultimate (factored) loads. Both methods require satisfying equilibrium of vertical forces and summation of moments on the base plate to determine the stresses or forces on the base plate. Although either method gives reasonable results for design, very little is known about the actual moment-rotation behavior of base plates. It has been suggested that[3] method (II) better predicts the strength and more closely models the actual behavior of the base plate at failure. An outline of the method in the AISC manual[1] for the design of base plates is reviewed in appendix A.

2.2 Inaccuracies with the assumption of "pinned" base connection

A) Actual actions that the base plates are subjected to are not considered in design.

B) Design of anchor bolts is not done for combined effects of shear and tension.

C) Because of the improper model that is being used to define base conditions, an unrealistic estimate of frame drift is obtained.

D) Moment actually delivered to the foundation is not considered in the design of the foundation.

III PROGRAM OF RESEARCH

3.1 Introduction - As was mentioned, the information on the behavior of steel column and base plate connections is very limited. The moment-rotation characteristics of this connection were the main focus of the experimental work that was done. By measuring the moment-rotation relation it was hoped that some of the problems discussed in 2.2 could be better understood. Then this information could be used to perform better designs and modeling of base plates and frames. The subassemblies that were tested were thought to represent the actual conditions in field construction (i.e., a footing with anchor bolts and base plate mounted on the bolts, with grout between the footing and base plate.) Six tests were conducted and three different base plate thicknesses used. It was also decided to focus on the two main parameters that govern base plate behavior. These are: 1) base plate thickness, 2) axial load in column.

3.2 Design of specimens - It was decided to test a given base plate thickness for two different axial loads ($.3P_{all}$ and P_{all}). P_{all} is equal to the allowable load that may be carried by the base plate (See Appendix A). The columns (W6X25) were selected based upon past experience. This standard AISC section represents almost a one half scale model of the prototype columns of W14X120 through W14X211 size range. These prototype sections have similar slenderness ratios and radii of gyration of about two times the model W6X25 sections. For these reasons the W6X25 was deemed appropriate for the proposed research. (All section properties of the model and prototypes are given in Appendix B.) In order to eliminate any buckling problems the column lengths were kept short and were selected to be approximately 20 inches. The base plate sizes are shown in Table 1. They are all 9 inches by 12 inches with varying thicknesses as given in the table. The choice of a

9 inch by 12 inch base plate was based on past research experience also and allowed for proper spacing and edge distances of bolts according to standard practice. Based on this column size the base plates were designed following Chapter 3 of the AISC manual[1], which is outlined in the Appendix A. There was no consideration given to the design of the base plate for externally applied moment. Bending moments in the base plates under service loads controlled the plate thicknesses. Calculations containing the design of the base plates are given in Appendix A. For the anchor bolts, standard A307 steel bolts were used. Also standard spacing of bolts and connection details followed recommendations of the AISC manual and are shown in Figures 2 and A.3 in Appendix A. For welding, E7018 electrodes were used. The weld size chosen was based on how large a weld could be made in a single pass with a welding electrode. The welding to the base plate was intended to represent typical "pinned" column base plate connection in a seismic zone. The footings were designed not to be the weak link in the design. It was decided to make them 2'X 2'X 1'- 0" spread type footings. Reinforcement was placed at the top and bottom in the footings to prevent any splitting or concrete failures. These footings as designed were thought to represent actual construction practice in the field. Details of the footing construction are shown in Figures 4 and 5.

3.3 Test set up - The testing arrangement to be used in experimental research depends on the loading conditions and the constraints on the subassembly that have to be reproduced to represent actual loading conditions. The set up that was designed and used is shown in Figure 1. It was decided that the loads were to be applied in a quasi-static manner. Although the lateral loads of wind or earthquake are dynamic in nature, our interest in obtaining moment-rotation data dictated that quasi-static loading would provide accurate measurements and valuable information on that quantity. The actuators. were servovalve controlled which allowed a specific axial load and/or lateral displacement to be dialed in to the system and held constant. The top of the column was designed to be pinned. This was accomplished by using the clevises shown in the test set up in Figure 1. The footings were prestressed to large loading mass blocks which in turn were prestressed to the reaction floor of the laboratory. Consequently, there was no rotation of the footing or movement of the loading block. This test set up was believed to provide a good simulation of the forces which a base plate could be subjected to in a real structure.

3.4 Parameters to be considered in the study of the subassemblies - The main objective of these tests was to determine the moment-rotation characteristics in what is commonly considered a pinned column base connection. Therefore, the most important parameter that was to be measured was the moment applied to the base plate and the corresponding rotation. As was mentioned in 3.3 the effect of footing soil interaction was eliminated from the tests. The footings were also designed to be strong enough to eliminate crushing of the concrete footings that could contribute to the rotation. Also to limit the slip at the anchor bolts from contributing to the rotation, bearing plates were welded to the bottom of the anchor bolts. Strain gages were inserted into the anchor bolt shanks to allow measurement of the bolt elongation and force which is thought to contribute significantly to the response of the subassembly. Another major parameter that was varied in the tests was the thickness of the base plates. It is believed that flexural strength and stiffness of the base plates are significantly affected by plate thickness. Three different thicknesses of base plates were used. The thicknesses were 1/4, 1/2 and 3/4 inches.

3.5 Instrumentation - The instrumentation that was used in the tests is shown in Figure 6 and Table 2. Of particular importance were the LVDT's located off of the W6X25 flanges. These were used to measure the rotation at the base. Through these LVDT's the rotation is calculated from the following equation:

$$\text{ave.} = [(\text{LVDT8} - \text{LVDT10}) + (\text{LVDT9} - \text{LVDT11})]/2(\text{Dist. Between LVDT's})$$

Also the LVDT's located on the edge of the base plate were significant in showing how the base plate lifted off of the grout and where plastic hinges started to form in the plate. Strain gages were placed in half of the anchor bolts that were located diagonally on the base plate, however do due the complexity of attaching these gages, only a limited number of bolt strain gages were operational and limited measurements were made on bolt strain.

3.6 Loading of specimens - The loading sequence was decided to simulate serviceability loading conditions, e.g., wind or moderate earthquake at the beginning of the tests. However, the loading continued in an increasing manner to observe failure limit states. In appendix A the allowable axial loads permitted by the AISC specification[1] are listed. For each base plate thickness an axial load equal to this value of P_{all} was applied for the duration of the test. Also, for each base plate thickness an axial load equal to $(0.3)P_{all}$ was applied to a sister specimen for the duration of that test. Once the axial load was set on the subassembly, the lateral load or displacement was applied. The lateral loading scheme that was used for each subassembly is shown in Figure 7. All specimens were subjected to identical lateral cyclic loading. In summary, the cyclic lateral loading consisted of the following cycles:

DRIFT	CYCLES
1%	2
2%	2
3%	4

4%	2
5%	2
6%	1
7% or more	2+ until failure

This loading schedule was designed to accomplish the requirements for the service and ultimate type of limit states mentioned above. It should be mentioned that drift as defined for these tests was computed by dividing the lateral displacement by the distance from the top of the base plate to the point where the lateral displacement was measured. (See Figure 6)

IV EXPERIMENTAL RESULTS

4.1 Introduction - In this section an account of each test will be discussed. Included will be the behavior related to moment-rotation, column behavior, weld, grout, anchor bolt and base plate effect on the overall response of the specimens.

4.2 Test 1-L-0.50(See Figures 8 and 9) - This was a test of a 1/2" thick base plate with $0.3P_{all}$ (8.25 kips) applied as axial load. As can be seen from the moment-rotation curve, behavior in general was very stable. At one percent drift cycles, the behavior remained essentially linear. There was some observed separation of the plate from the grout at the extreme bounds of testing but after two complete cycles at one percent drift, no permanent separation was noticed when the displacement returned to zero displacement. There was no yielding apparent in any part of the specimen at 1% drift level. As the cyclic loading started into the two percent drift range, noticeable vertical cracking of the grout occurred. These cracks, however did not have any noticeable effect on the response of the specimen. At the end of two cycles of two percent drift, slight yielding of the column flanges was observed. Loading into the three percent drift range caused noticeable yielding in the plate adjacent to the flange weldlines and a permanent separation of the plate from the grout occurred. From the hysteresis curves in figure 8 it can be seen that even after four cycles of three percent drift the response of the assembly is stable and is very close to elastic-plastic response. With further increases in the level of lateral drift, no significant increase in strength was observed. Only more yielding of the plate and column occurred and the moment strength of the connection was generally stable. Failure of the specimen was due to fracture in the heat affected zone of the weld at an extremely large drift of seven percent after 16 cycles.

4.3 Test 2-H-0.50(See Figures 10 and 11) - This was a test of a 1/2" thick base plate with the full P_{all} (27.5 kips) applied as axial load. Again, from the moment-rotation curves it can be seen that the behavior of the specimen is very stable. Because of the higher axial load several behavioral items differed from the lighter axially loaded specimen. First, the onset of yielding in both the column and base plate were noticed at earlier stages of loading, e.g., during the one percent drift cycles. However, the moment-rotation curves show the behavior to be essentially linear at that loading stage. Second, after several load cycles, even up to four percent drift, there was no permanent separation of the plate from the grout and third there was more than double the amount of cracking in this test than in test No. 1. The increased amount of cracking in the grout is probably the main reason for the slight pinching that can be seen on the moment-rotation curve for this test. After reaching about two percent drift no significant increase in moment strength was observed. For the higher axially loaded plate there was a slight increase in moment strength when compared to the sister specimen which was only loaded with 3/10 the axial load of test No. 2. This difference amounted to about fifteen percent. Failure of the specimen was by fracture in the weld, as with test No. 1 and at an extremely large drift of seven percent, as in test No. 1. Failures of tests one and two were essentially the same except for extra cracking and spalling of the grout in test No. 2.

4.4 Test 3-L-0.75(See Figures 12 and 13) - This was a test of a 3/4" thick base plate with $0.3P_{all}$ (18.6 kips) applied as axial load. The moment-rotation curve for this test shows a very different behavior from the 1/2" thick plate tests. The most noticeable characteristic is the large increase in moment strength. There is about a thirty percent increase in moment resistance. This shows that the primary source for resistance in the connection is the base plate, not the column. Most of the deformation causing the rotation occurred in the base plate. There is also a decrease in the rotational capacity, about twenty percent, compared to tests one and two. Because of the increased thickness, the base plate was less flexible and this led to formation of other mechanisms of failure. In this test, the grout seemed to be the cause of the pinched nature of the moment-rotation curve. There was considerable cracking and crushing of the grout after the two one percent drift cycles, however, during the one percent drift cycles the behavior was essentially linear, as with tests one and two. What was observed during this test was the base plate-column connection remaining undeformed up to about the third cycle of three percent drift and that the plate being so rigid caused deformations to occur in the grout. The grout would crack and upon reversal of the load the connection would not attain full strength until the cracks closed, this might be the reason for the observed pinched behavior. The final failure occurred by fracture of one of the anchor bolts in tension at about five percent drift.

4.5 Test 4-H-0.75(See Figures 14 and 15) - This was a test of a 3/4" thick base plate with full P_{all} (62.0 kips) applied as axial load. This test started out similar to the previous three tests in that the behavior during the one percent drift cycles remained essentially linear. By comparison with test No. 3 at this loading stage there was only slight differences in the two moment-rotation curves. There was yielding of the column flanges during the one percent drift cycles and a twenty five percent increase in moment strength. This was in part due to the increased axial load from test three to test four. During the first cycle into the two percent drift range noticeable separation of the base plate from the grout and the grout from the footing was observed. This was reflected by the small pinching in the moment-rotation curve of test No. 4. Also during the two percent drift cycles yielding commenced in the base plate at the flanges and in between the bolts and cracking of the grout was first noted. As the loading continued into the three percent drift range, considerably more pinching of the moment-rotation curve was observed. This was mainly the result of plate separation from the grout and cracking of the grout. Yielding of the base plate was continuing to occur at the flanges and between the bolts. At the zero displacement position the base plate formed a convex surface in contact with the grout. This shape of the base plate increased the pinching characteristics of the moment-rotation curve as can be seen. Loading into the four percent drift range caused severe cracking and spalling of the grout and upon loading into the five percent drift range too much grout had spalled to continue the test. In comparison with test No. 3, there was a slight increase in rotational capacity and about twenty five percent increase in moment strength at larger drifts. This was attributable to the increased axial load.

4.6 Test 5-L-0.25(See Figures 16 and 17) - This was a test of a 1/4" thick base plate with $0.3P_{all}$ (2.10 kips) applied as axial load. Of all the tests that were performed, this specimen behaved most similar to a true pinned connection. As can be seen from the moment-rotation curve, there is almost no moment resistance of this connection. Even after the first few seconds of loading, the plate under the tension flange started to yield and separate from the grout. Again, this shows the controlling mechanism of the connection to be the plate. The moment-rotation curve also shows the connection to behave in a stable manner, just as has been seen from the previous other tests. Because of the light axial load the grout was not a factor for this specimen. Although the grout did crack, there was no pinching of the moment-rotation curve, i.e., the cracking was not significant enough to affect the behavior of the specimen. In comparison to the previous four tests, this specimen did not show a significant linear range of behavior. During the first two cycles in the one-percent drift range, the base plate began to yield near the flanges. No yielding was noticed in the column flanges at all. As loading progressed into and past the three-percent drift range, yielding of the plate occurred between the two anchor bolts. Failure of the specimen occurred by weld fracture in the heat-affected zone of the plate where yielding had occurred. This occurred at a drift of five percent.

4.7 Test 6-H-0.25(See Figures 18 and 19) - This was a test of a 1/4" thick base plate with full P_{all} applied as axial load. This specimen, similar to test 5, behaved very much like a true pinned connection. From the moment-rotation curves it is seen that there was very little initial stiffness and negligible moment resistance (about 10% M_p for the W6X25.) The effects of the higher axial load were mainly earlier cracking of the grout. This caused the slight pinching seen in the moment-rotation curve. The effect of grout cracking was discussed earlier. There was no noticeable increase in moment resistance with the increased axial load as was seen with other plate thicknesses. This is probably due in part to the fact that the 1/4" plates are initially very flexible. As loading progressed into the three-percent drift range separation of the plate from the grout was continuous under the column. This separation gap also contributed to the slight pinching seen in the moment-rotation curve along with the grout cracking. This specimen also showed less durability than the previous five tests. This is seen from the fact that failure occurred after the second cycle of four-percent drift. Failure of the specimen was similar to that of test five, i.e., fracture of the plate in the heat-affected zone where yielding had occurred.

V SUMMARY AND GENERAL TRENDS IN BEHAVIOR OF TEST SPECIMENS

5.1 Introduction - Based on the six tests that were performed, some general conclusions can be made. Although these tests are not thoroughly conclusive, there are some trends that should be noted.

5.2 Observations - 1. For each base plate thickness, the tests with the lighter axial compressive loads did not develop as much moment resistance. That is, the larger was the axial compressive force, the larger was the moment resistance of the connection.

2. For each base plate thickness, it was also noted that for lighter axial compressive loads, there was better energy dissipation under the lateral load reversals. That is, there was a more stable response when the axial compressive loads were lighter.

3. Thinner base plates showed the most ductile response. The thinner base plates also showed much more yielding in the plate than in the column and behaved almost as a perfect pin.

4. As the base plate thickness became larger the tension forces in the anchor bolts became much larger and even caused failure of the connection as was observed in Test 3-L-0.75.

5. Except for Test 4-H-0.75, bearing of the base plate against the grout and footing did not seem to be a weak link in the connection, especially for the lighter axial compressive loads and the thinner base plates. However, it should be noted that cracking of the grout, to whatever extent, was the major contributor to the pinching nature of the moment-rotation curves in the tests performed.

6. For all tests except Test 5-1-0.25,(which behaved as the most flexible of all the tests,) response during the one percent drift range was essentially linear. This characteristic may help in assessing the amount of building drift attributed to the column-base plate connection.

7. All specimens reached the two percent drift condition without a structural failure, i.e., no fracture or excessive yielding.

8. For each of the tests there is a plot of the monotonic response. It is interesting to note several trends in this behavior.

(a) The subassemblies did not exhibit any strain hardening strength increases, i.e., the yield strength is very steady.

(b) Thicker base plates are initially more stiff and a larger axial load also shows an initially more stiff connection.

(c) The behavior is very much like that of an elastic-plastic response. This fact could be used to help establish equations that predict connection response.

VI FUTURE RESEARCH NEEDS

6.1 Introduction - These tests provided valuable data on the behavior of the steel column-base plate connection. However, some future research needs were discovered while the tests were performed and during the discussions while writing the report.

6.2 Research needs - The following is a list of some of the research needs in the area of steel column-base plate connections.

1. Tests need to be performed on full scale specimens with pinned base connection details but with plates thinner than those allowed by chapter three of the AISC specifications. This would allow the researcher to investigate if more reliable pinned behavior can be achieved and to see if the bearing stresses on the grout or footing present design problems associated with the thinner base plates.

2. Tests need to be performed on full scale specimens with fixed base connection details. This is needed to evaluate current design methods in fixed base plate connections.

3. For the various base plate connection details discussed above, (pinned and fixed) a method of assessing a realistic value of building drift due to the column-base plate connection needs to be established.

4. Ways to minimize grout cracking during cyclic loading should be investigated, keeping in mind current construction methods that are used to construct the steel column-base plate assembly in the field.

VII REFERENCES

1. AISC, "Manual of Steel Construction," $8^{\text{Th.}}$ Edition, American Institute of Steel Construction, Chicago, Illinois (1980).

2. DeWolf, J.T., "Axially Loaded Column Base Plates," Journal of the Structural Division, ASCE, Vol. 104, No. ST.5, May, 1978

3. DeWolf, J.T., and Sarisley, E.F., "Column Base Plates with Axial Loads and Moments," Journal of the Structural Division, ASCE, Vol. 106, No. ST.11, November, 1980

4. Picard, A., and Beaulieu, D., "Behavior of a simple column base connection," Canadian Journal of Civil Engineering, Vol. 12, 1985

TABLE 1[1]
BASE PLATE SIZES

TEST NUMBER	LENGTH	WIDTH	THICKNESS	NUMBER OF BOLTS
1	1' - 0"	9"	0.50"	4-3/4"
2	1' - 0"	9"	0.50"	4-3/4"
3	1' - 0"	9"	0.75"	4-3/4"
4	1' - 0"	9"	0.75"	4-3/4"
5	1' - 0"	9"	0.25"	4-3/4"
6	1' - 0"	9"	0.25"	4-3/4"

1 See Figure A.3

TABLE 2[2]
INSTRUMENTATION

CHANNEL No.	LOCATION	DESCRIPTION	CALIBRATION FACTOR	COMMENTS
1	LATERAL LOAD	LOAD CELL	15 KIP/V	
2	LATERAL DISP.	COLUMN DISP.	0.60"/V	
3	AXIAL LOAD	LOAD CELL	10 KIP/V	
4	LATERAL TEMP.[3]	CYLINDER DISP.	1.2"/V	
5	AXIAL TEMP.	AXIAL CYL.DISP	0.6"/V	
6	TOP/SOUTH BOLT	BOLT STRIAN	2.00	ONLY ON SOME
7	BOT/NORTH BOLT	BOLT STRAIN	2.00	ONLY ON SOME
8	TOP SOUTH	LVDT/COL. BEND	0.10"/V	
9	BOTTOM SOUTH	LVDT/COL. BEND	0.10"/V	
10	TOP NORTH	LVDT/COL. BEND	0.10"/V	
11	BOTTOM NORTH	LVDT/COL. BEND	0.10"/V	
12	NORTH SHEAR	LVDT/SHEAR COL	0.10"/V	
13	SOUTH PLATE	LVDT/BASE PL.	0.05"/V	ON PLATE EDGE
14	SOUTH PLATE	LVDT/BASE PL.	0.05"/V	1"IN FROM EDGE
15	NORTH PLATE	STRAIN GAGE	2.12	NEVER USED
16	NORTH PLATE	STRAIN GAGE	2.12	NEVER USED

2 This table corresponds with Figure 7
3 TEMP. stands for a temposonic measurement devise

APPENDIX A

AISC MANUAL CHAPTER THREE DESIGN PROCEDURE FOR BASE PLATES

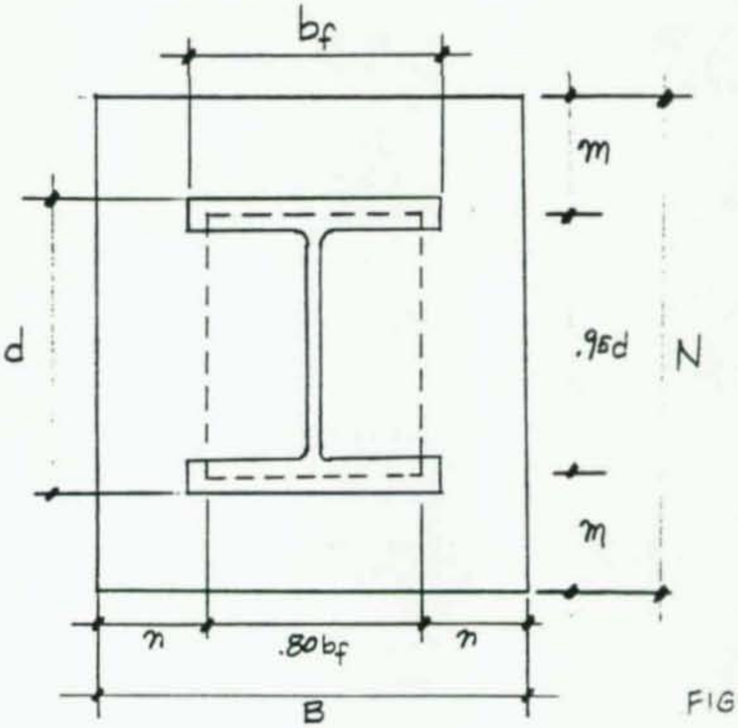


FIGURE A.1

FOR AXIAL LOAD ONLY

LIST OF SYMBOLS:

P = TOTAL COLUMN AXIAL LOAD (SERVICE), KIPS
A_1 = B X N = AREA OF THE BASE PLATE, IN^2
A_2 = FULL AREA OF CONCRETE SUPPORT, IN^2
F_b = THE ALLOWABLE BENDING STRESS FOR THE BASE PLATE (USUALLY $0.75F_Y$), KSI
F_P = THE ALLOWABLE BEARING STRESS ON THE SUPPORTING SURFACE, KSI
F_Y = YIELD STRESS OF THE STEEL, KSI
f_P = THE ACTUAL BEARING STRESS ON THE SUPPORTING SURFACE, KSI
f'_C = CYLINDER COMPRESSIVE STRENGTH , KSI
t_P = THE BASE PLATE THICKNESS, IN
m,n = SEE THE FIGURE ABOVE FOR THESE

The column load is assumed to be uniformly distributed over the concrete area under the base plate. This in turn caused maximum moments to occur at the dashed lines shown in the Figure A.1. From the free body shown in Figure A.2, bending moments and hence base plate thickness may be calculated as follows:

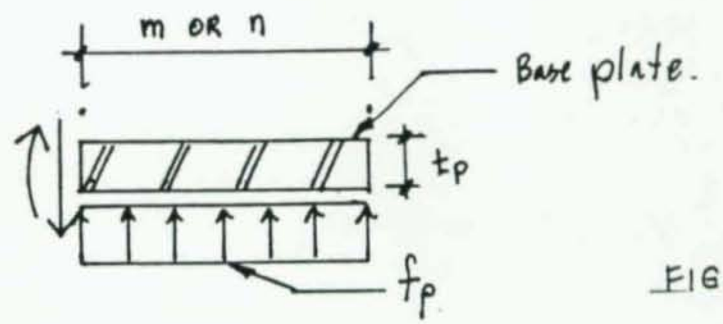


FIGURE A.2

If $F_b = 0.75F_Y$,

$t_p >= 2m\sqrt{f_p/F_Y}$ -OR- $t_p >= 2n\sqrt{f_p/F_Y}$

whichever is larger. The allowable bearing stress on concrete is given as:

$F_p = 0.35f'_c\sqrt{A_2/A_1} =< 0.70f'_c$

The two conditions of base plate bending stress and bearing stress on the concrete must be satisfied.

DESIGN OF TEST SPECIMENS

- Base plate size was set at 9" x t_p x 1'- 0"
- The supporting surface foundation area was the maximum allowed by code, therefore:

$$A_2/A_1 = 4.0 \text{ and } F_p = 0.7f'_c$$

- The base plate thicknesses were chosen to be 1/4", 1/2" and 3/4" for testing purposes.

- Based on these chosen variables the design axial loads (allowable stress) were calculated from the above equations:

$$f_p = P/B \times N$$

$$t_p >= 2m \text{ or } 2n\sqrt{f_p/F_Y}$$

- Therefore: $P =< (t_p(B)(N)F_Y)/(4m^2 \text{ or } 4n^2)$

NOTE: $n = 1/2(9 - 0.80(6.080)) = 2.068"$
$m = 1/2(12 - 0.95(6.38)) = 2.970"$ controls

For the base plate thicknesses noted above:

THE 1/4" PLATE: (ALLOWABLE STRESS OF $0.75f_Y$)

$$P_{all} <= (.25)^2(9)(12)(36)/4(2.97)^2 = 6.9^K$$

THE 1/2" PLATE

$$P_{all} <= (.50)^2(9)(12)(36)/4(2.97)^2 = 27.5^K$$

THE 3/4" PLATE

$$P_{all} <= (.75)^2(9)(12)(36)/4(2.97)^2 = 62.0^K$$

These are the maximum loads allowable from the AISC (chapter 3) code. Details of plate dimensions are shown in Figure A.3.

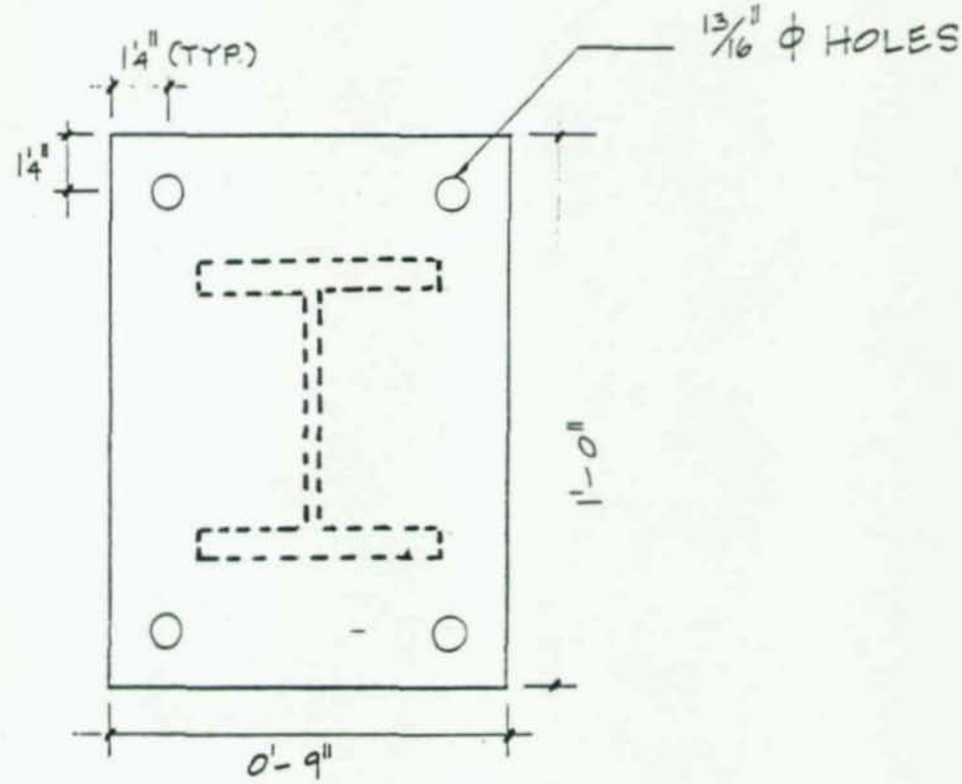


FIGURE A.3

BASE PLATE DIMENTIONS
& DETAILS

APPENDIX B

SECTION PROPERTIES OF STEEL COLUMNS

COLUMN PROPERTIES	W14X120	W14X159	W14X211	W12X120	MODEL W6X25
AREA, A IN^2	35.3	46.7	62.0	35.3	7.34
DEPTH, d IN	14.48	14.98	15.72	13.41	6.38
WEB THICK. t_w IN	0.590	0.745	0.980	0.710	0.320
FLANGE WIDTH b_f IN	14.670	15.565	15.800	12.320	6.080
FLANGE THICK. t_f IN	0.940	1.190	1.560	1.105	0.455
$b_f/2t_f$	7.8	6.5	5.1	5.6	6.7
d/t_w	24.5	20.1	16.0	18.5	19.9
r_T, in	4.04	4.30	4.37	3.38	1.66
d/A_f, 1/in	1.05	0.81	0.64	0.96	2.31
I_{xx}, in^4	1380	1900	2660	1070	53.4
S_{xx}, in^3	190	254	338	163	16.7
r_{xx}, in	6.24	6.38	6.55	5.51	2.7
I_{yy}, in^4	495	748	1030	345	17.1
S_{yy}, in^3	67.5	96.2	130	56.0	5.61
r_{yy}, in	3.74	4.00	4.07	3.13	1.52
P_Y, kips	1270	1680	2230	1270	264
M_P, kip-ft	636	861	1170	558	57

TEST SET UI

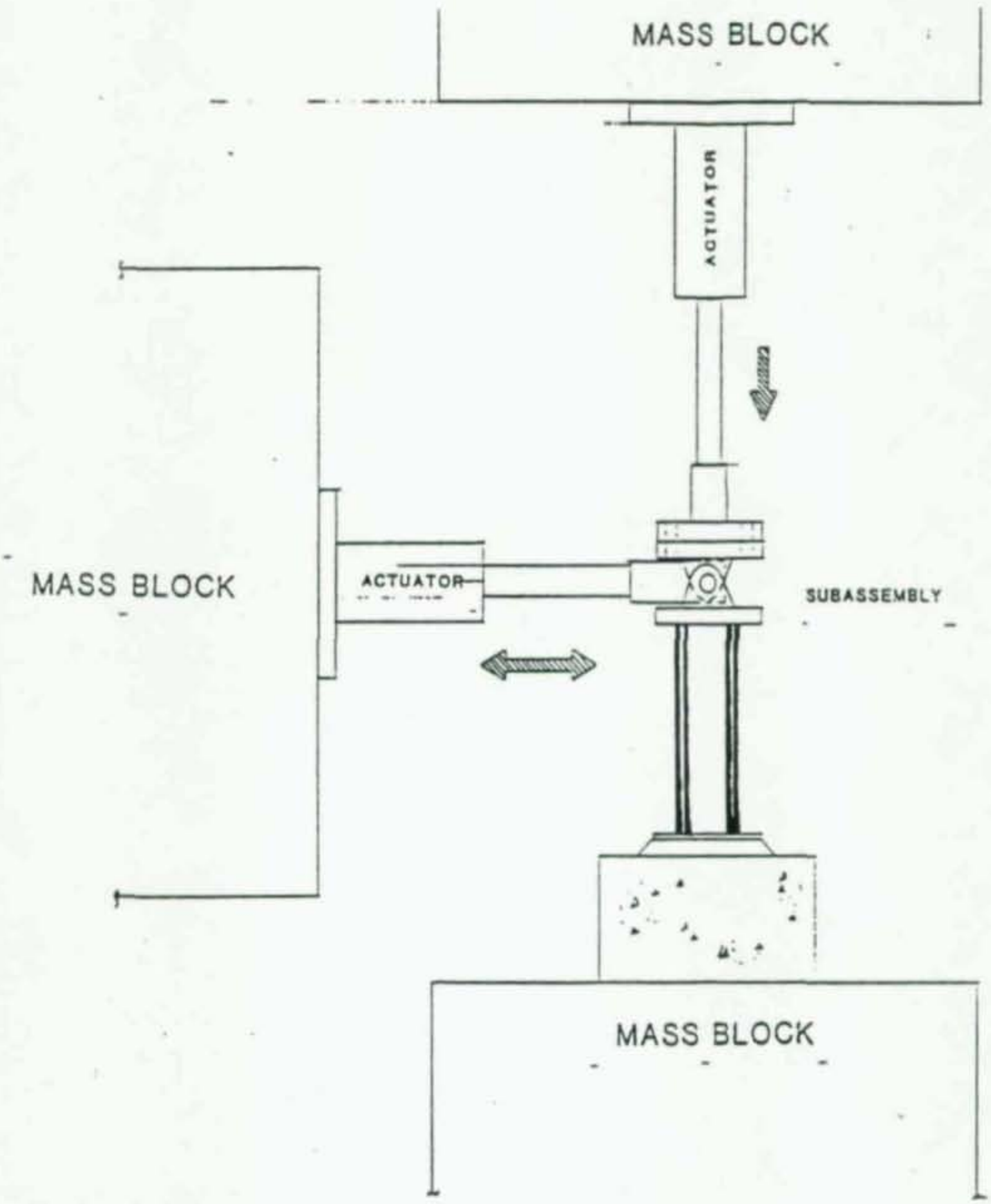
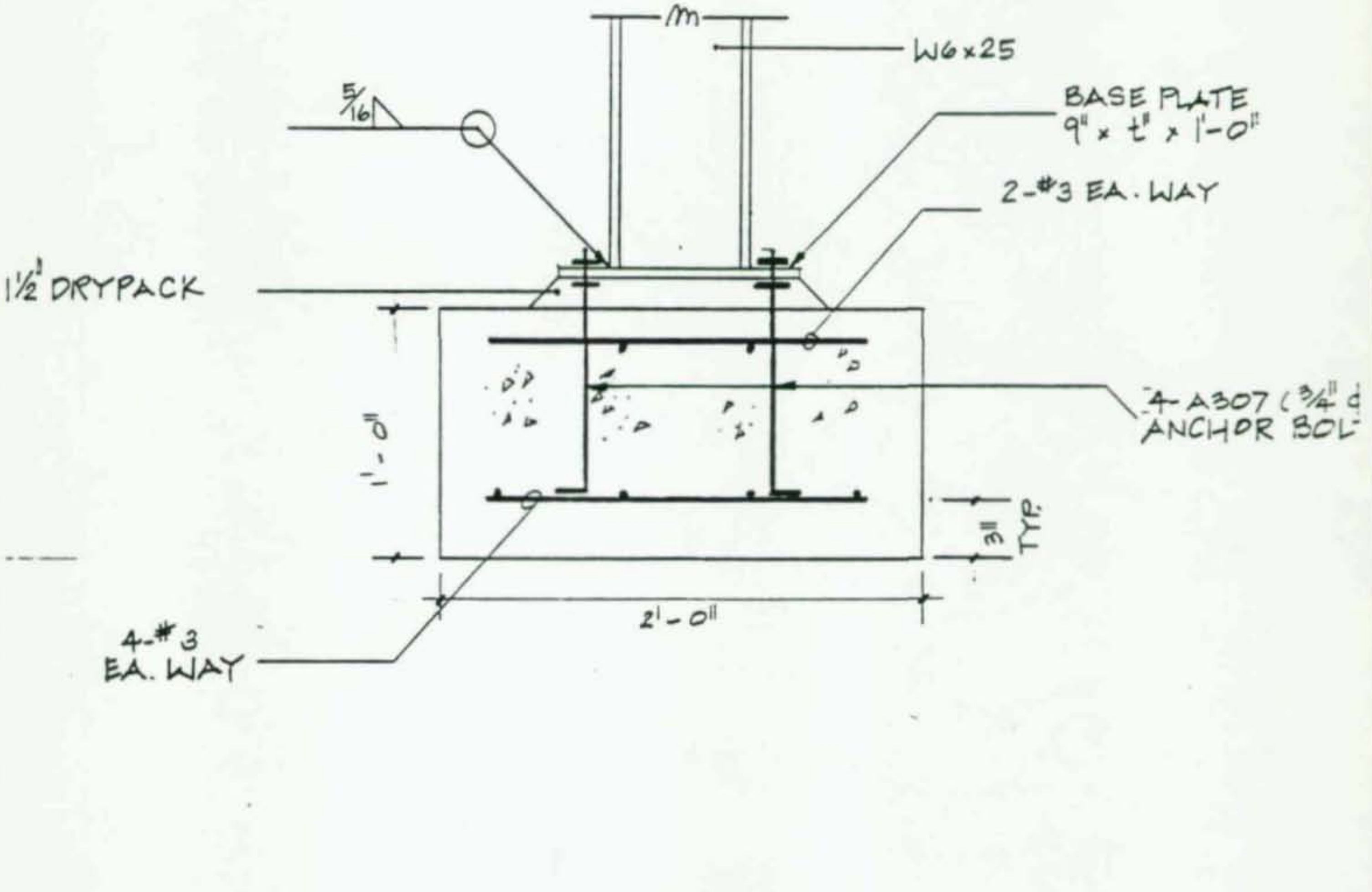
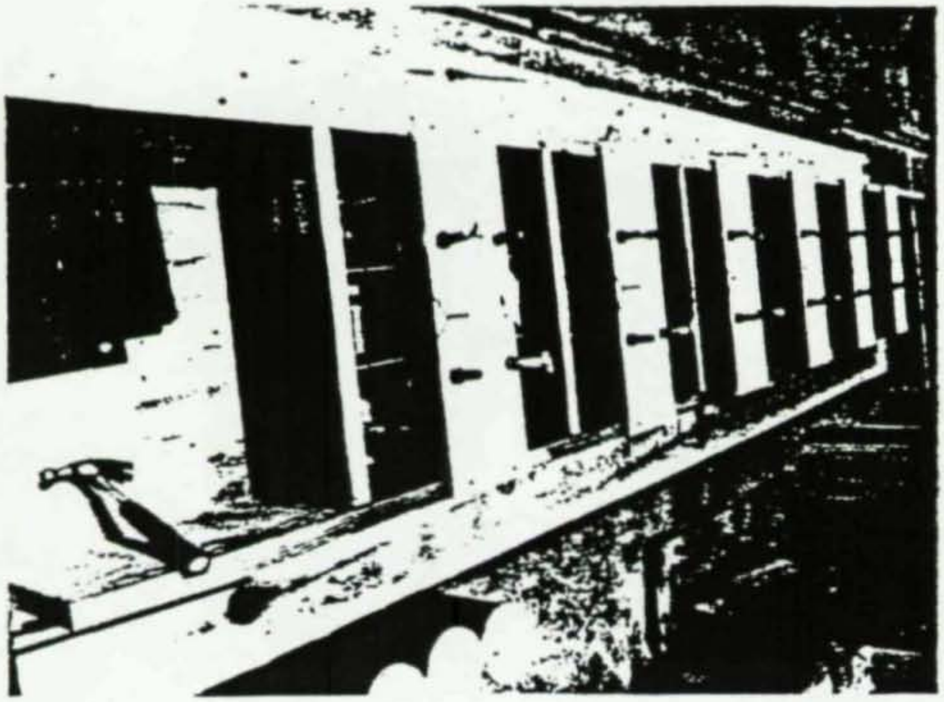


FIGURE 1



m
W6×25
5/16
BASE PLATE
9" × t" × 1'-0"
2-#3 EA. WAY
1½" DRYPACK
4-A307 (¾" d
ANCHOR BOL
1'-0"
3" TYP.
2'-0"
4-#3
EA. WAY

FIGURE 2



4A FOOTING CONSTRUCTION

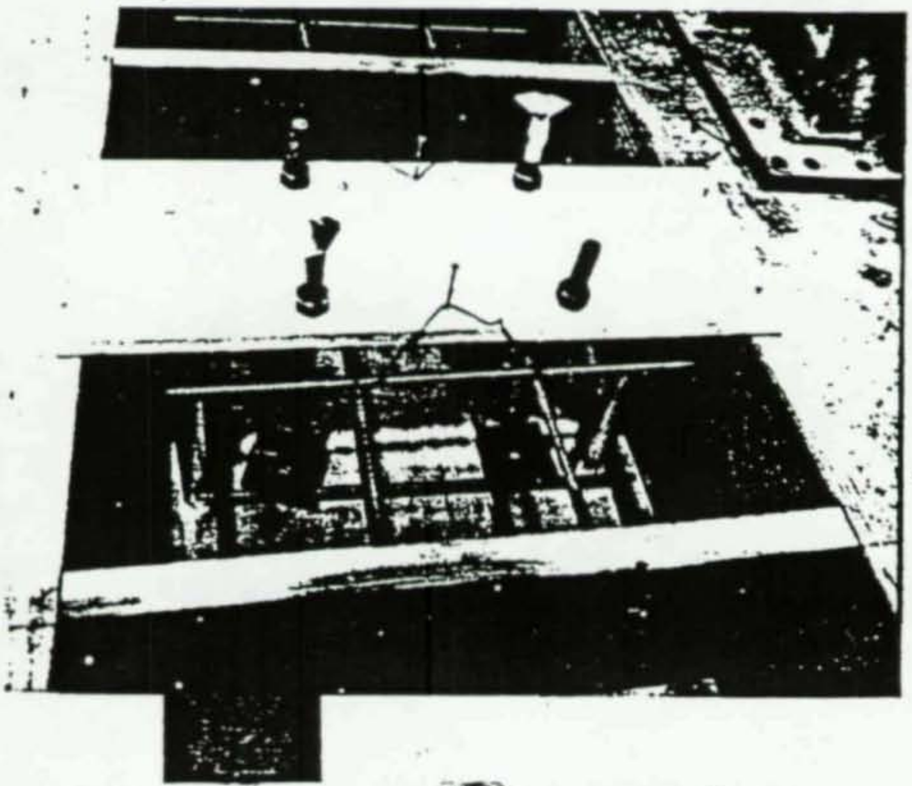


FIGURE 4B

FOOTING BOTTOM STEEL

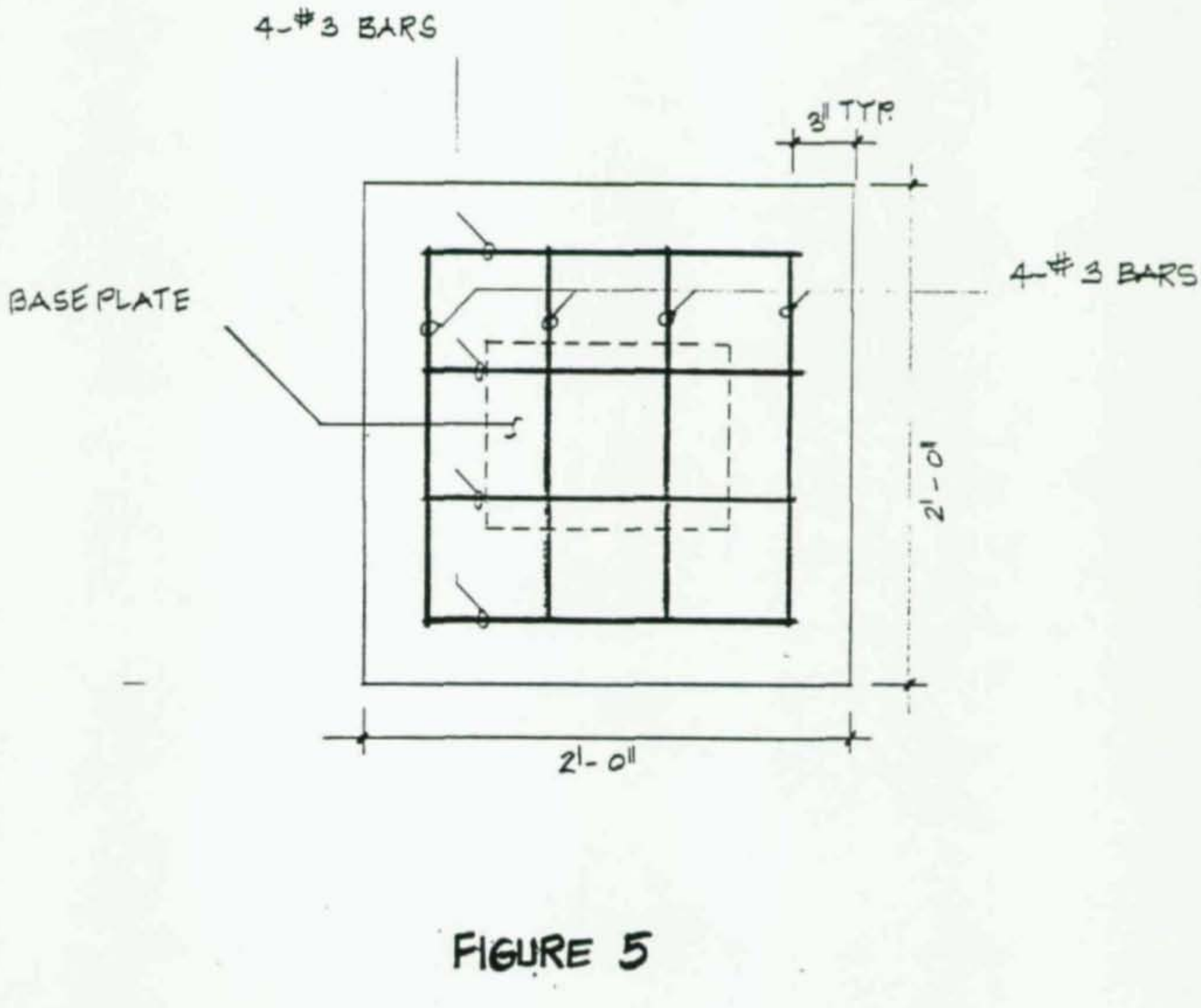


FIGURE 5

INSTRUMENTATION

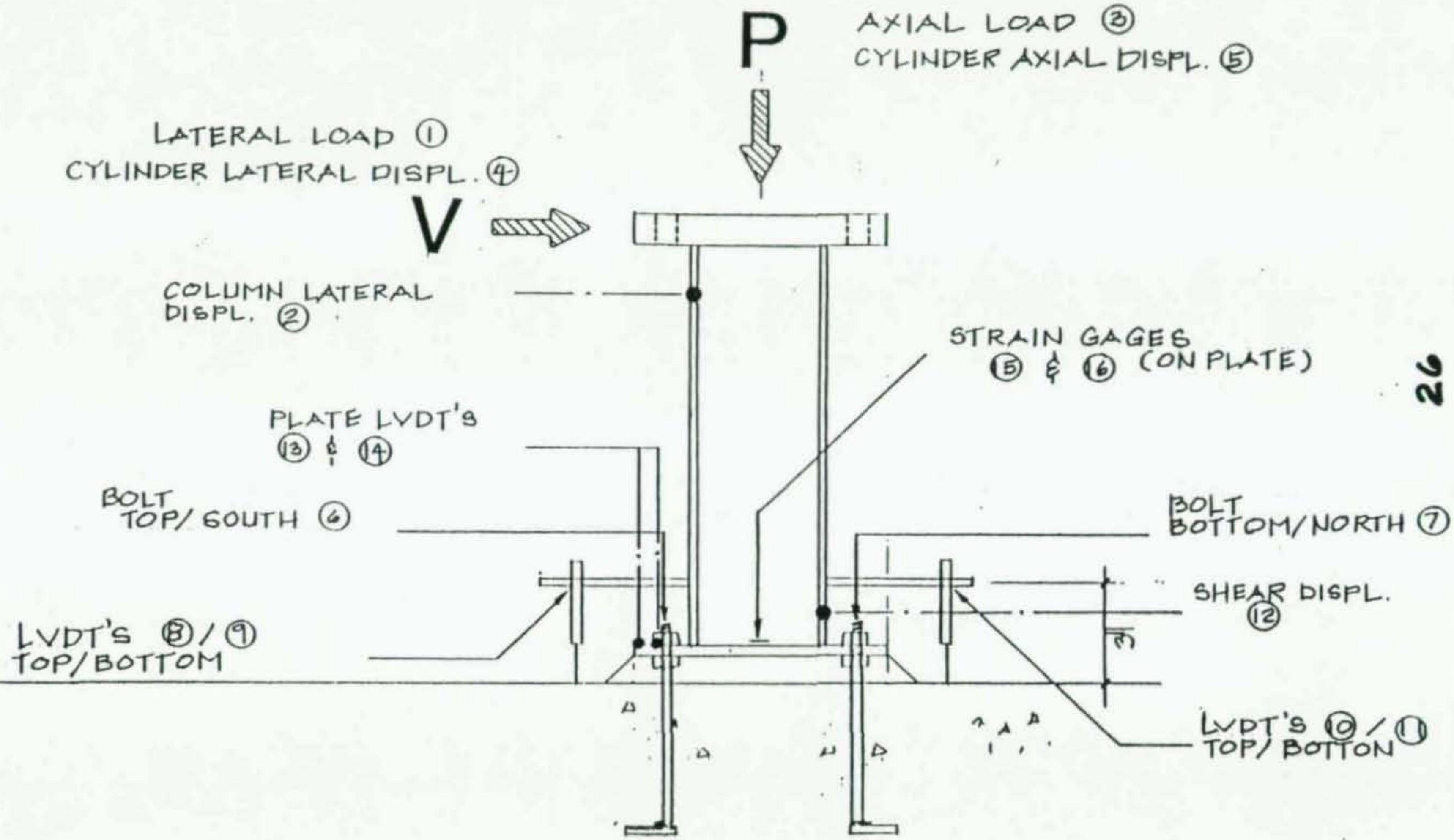


FIGURE 6

LOADING SCHEME

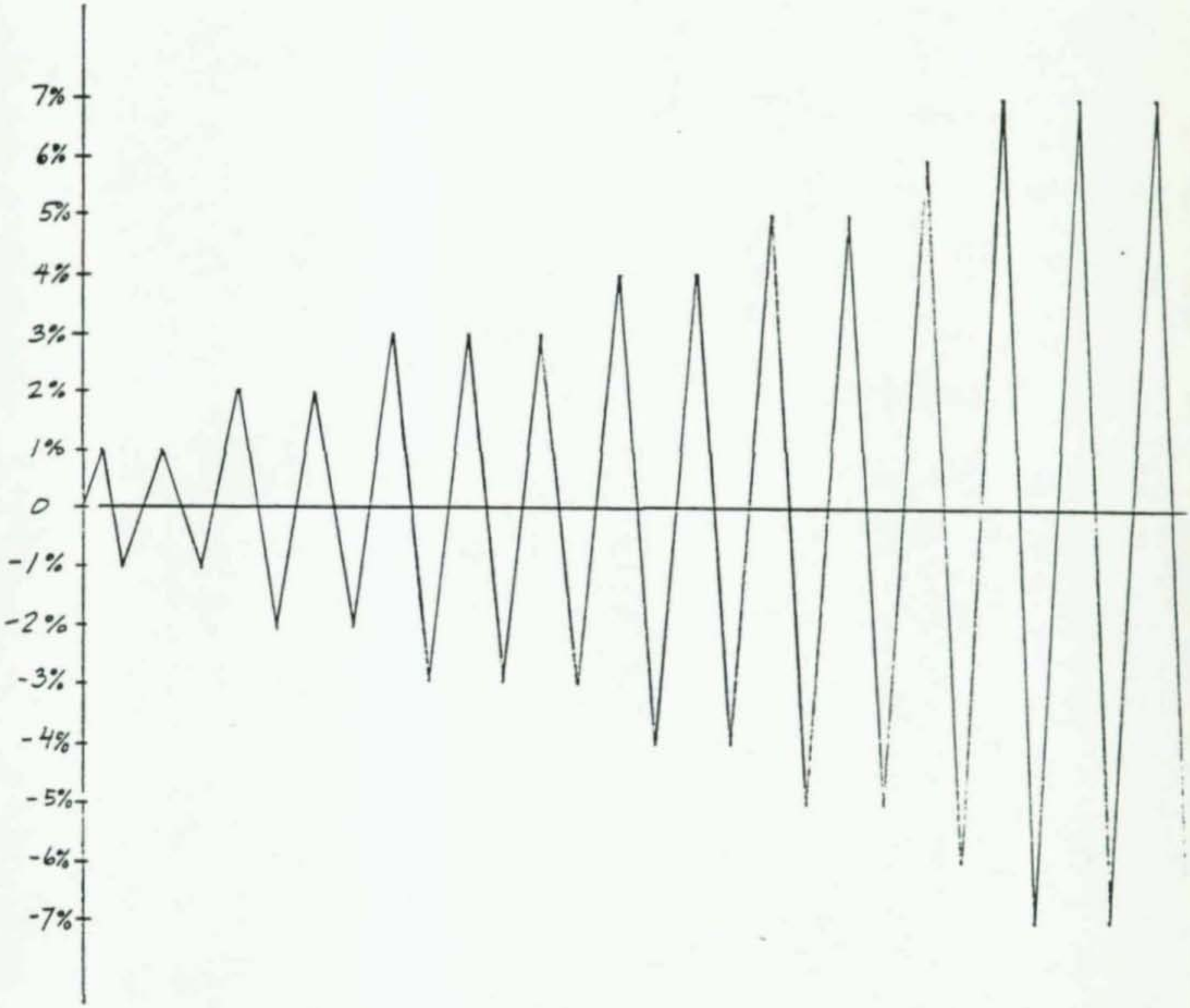


FIGURE 7

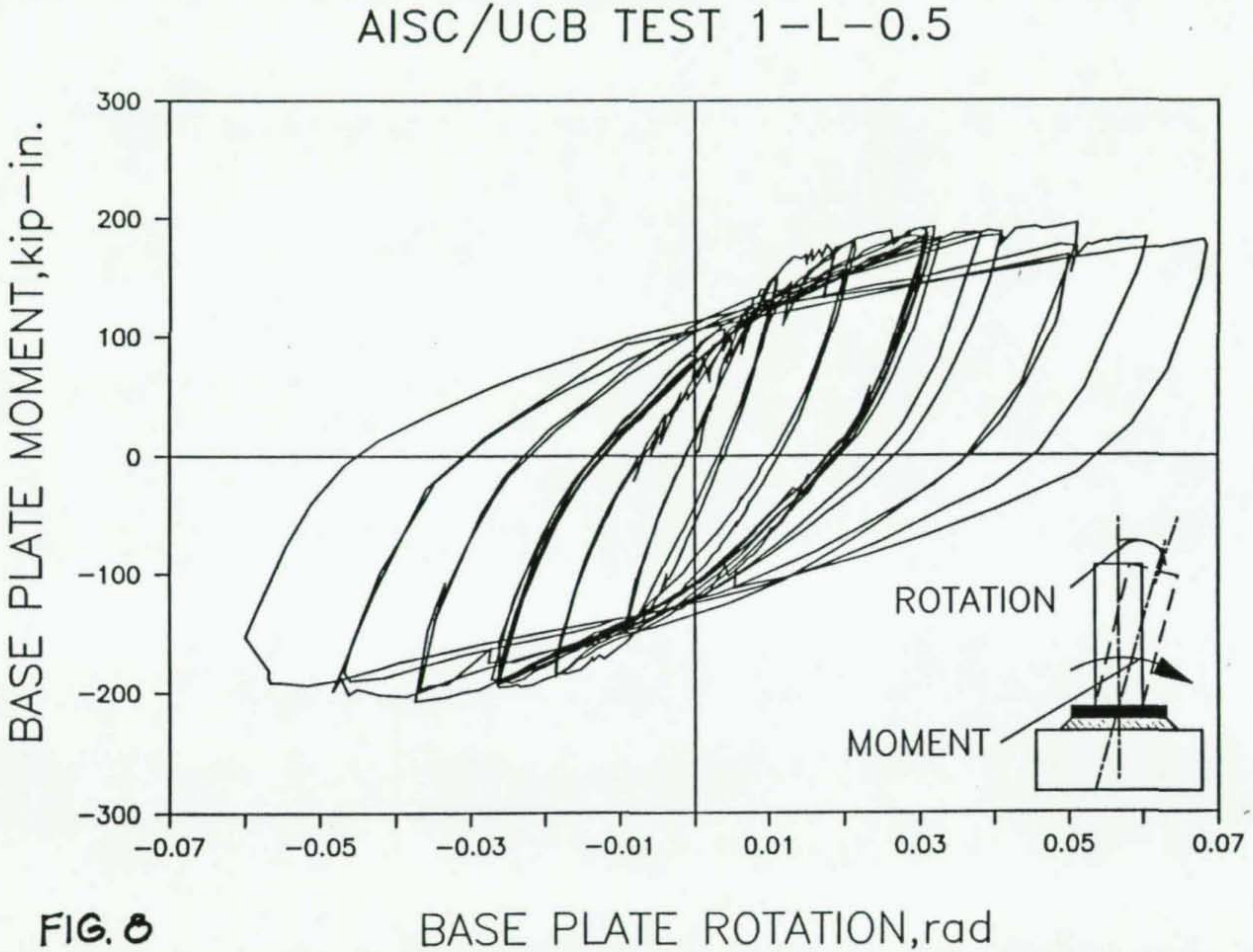

AISC/UCB TEST 1-L-0.5
BASE PLATE MOMENT,kip-in.
300
200
100
0
-100
-200
-300
-0.07
-0.05
-0.03
-0.01
0.01
0.03
0.05
0.07
BASE PLATE ROTATION,rad
ROTATION
MOMENT

FIG. 8

BASE PLATE MOMENT vs. ROTATION

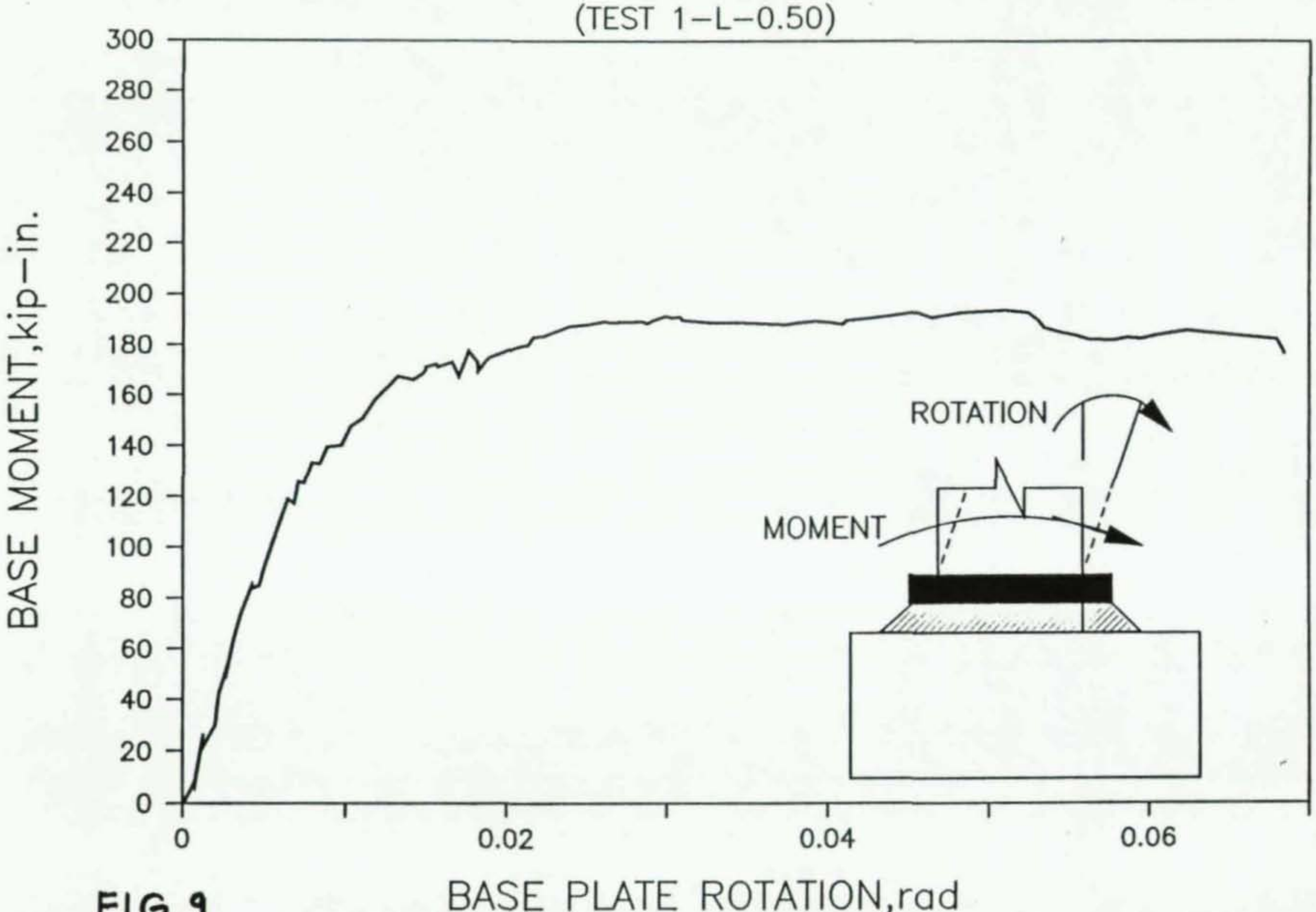


FIG. 9

LATERAL LOAD vs. DISPLACEMENT

(TEST 2–H–0.5)

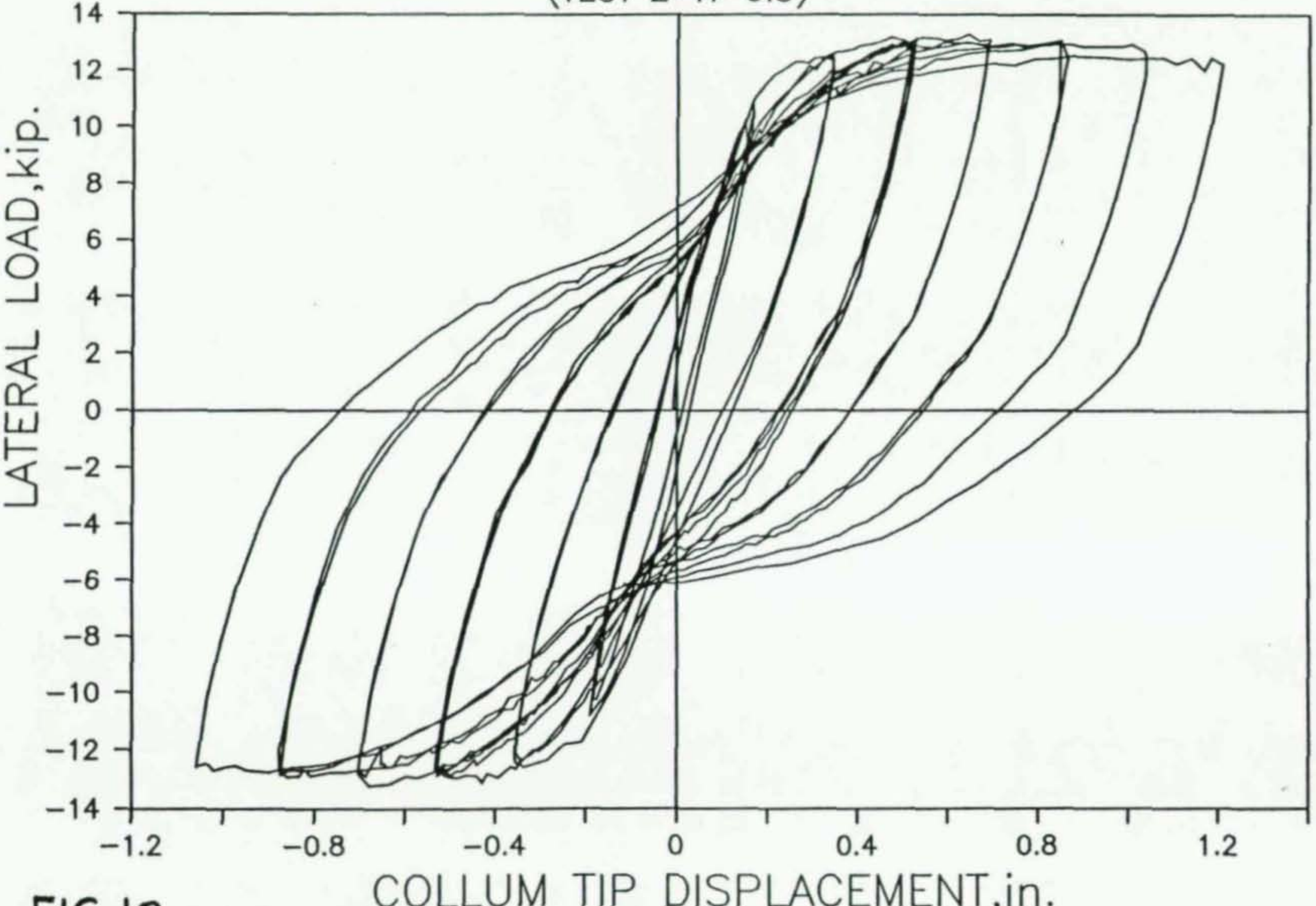


FIG. 10

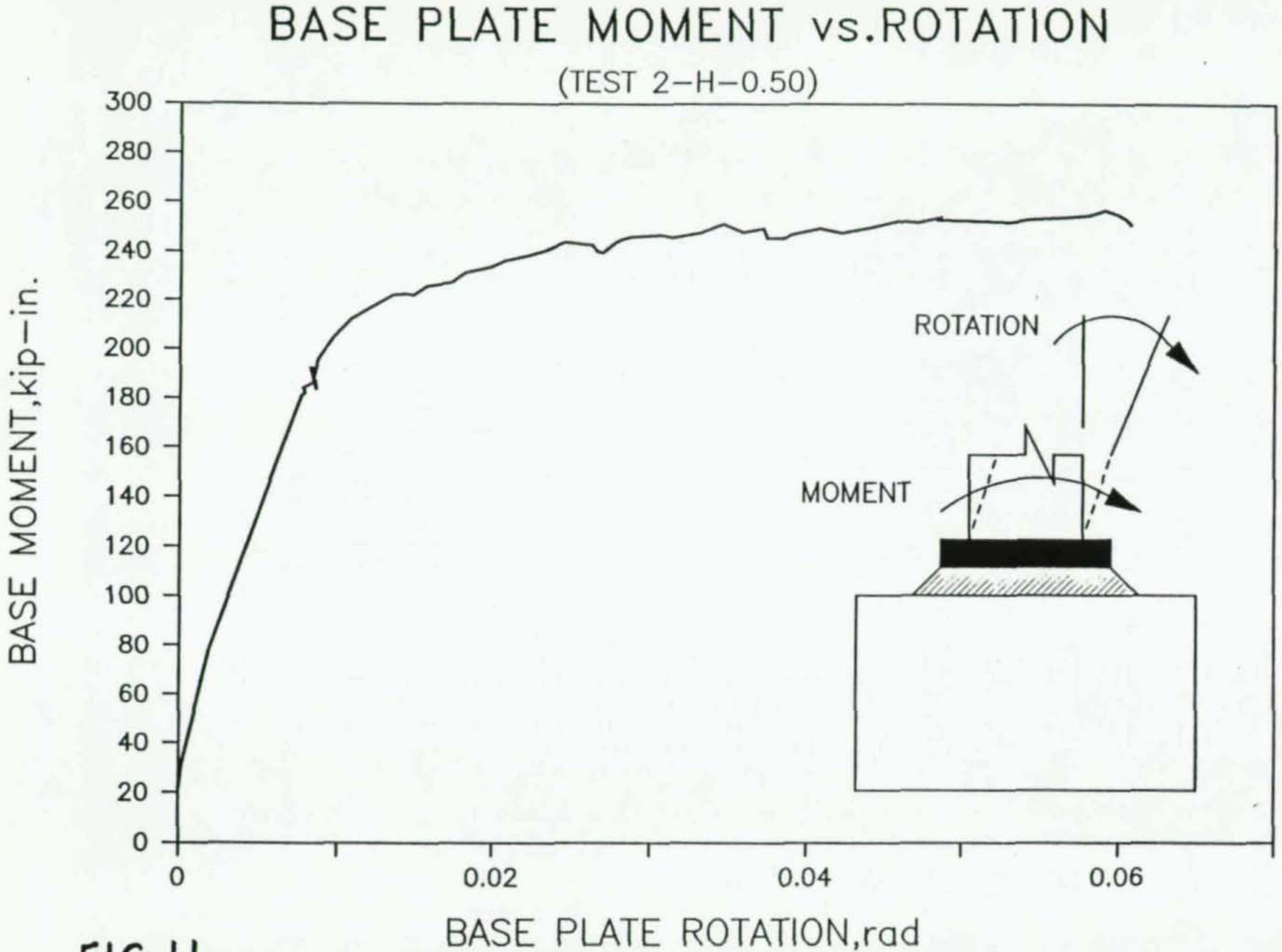

BASE PLATE MOMENT vs.ROTATION
(TEST 2-H-0.50)
BASE MOMENT,kip-in.
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
0
0.02
0.04
0.06
BASE PLATE ROTATION,rad
ROTATION
MOMENT

FIG.11

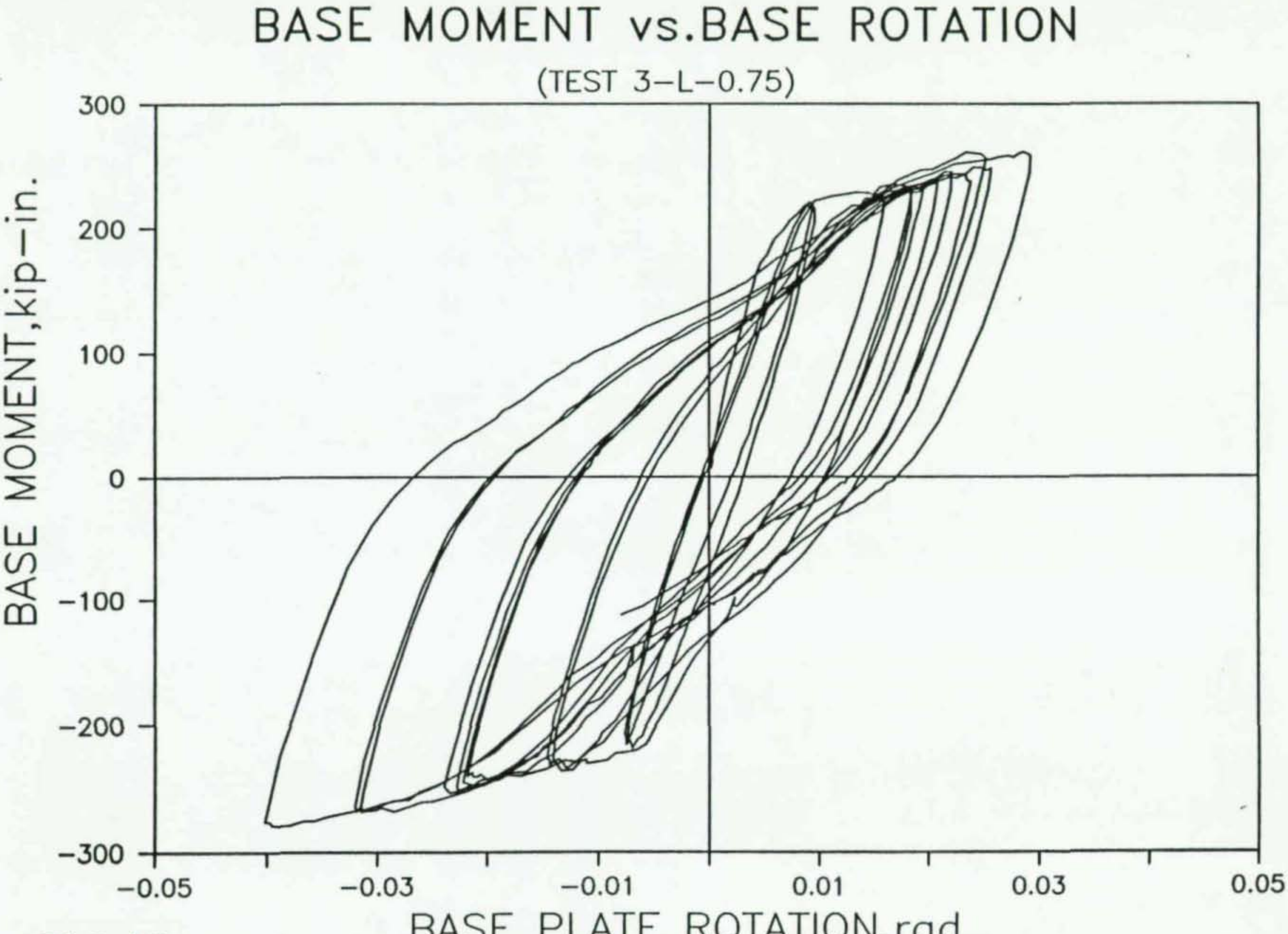

BASE MOMENT vs.BASE ROTATION
(TEST 3-L-0.75)
BASE MOMENT,kip-in.
300
200
100
0
-100
-200
-300
-0.05
-0.03
-0.01
0.01
0.03
0.05
BASE PLATE ROTATION,rad

FIG. 12

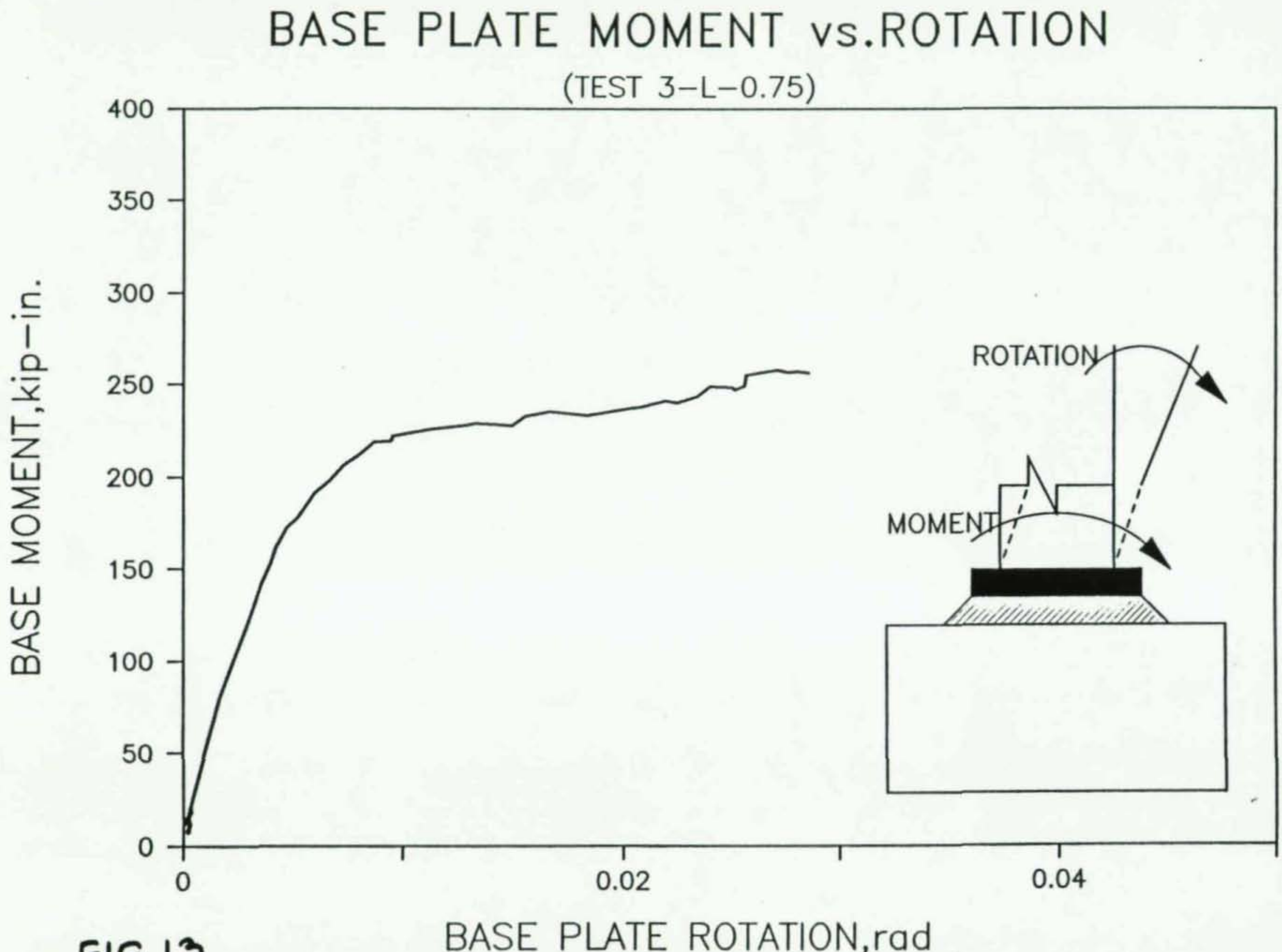


FIG.13

this is to start printer

33

00744

BASE MOMENT vs.ROTATION

(TEST 4-H-0.75)

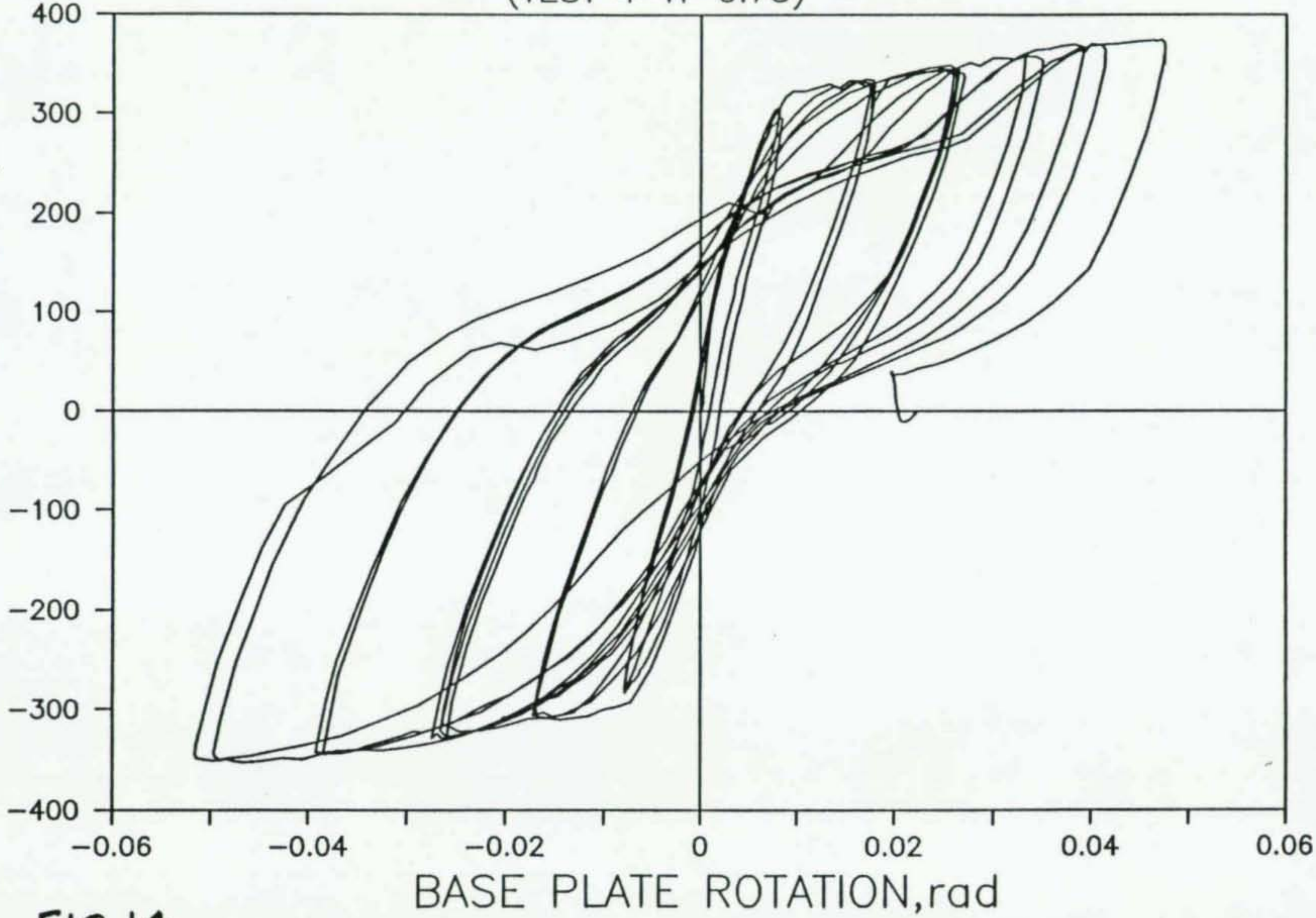


FIG. 14

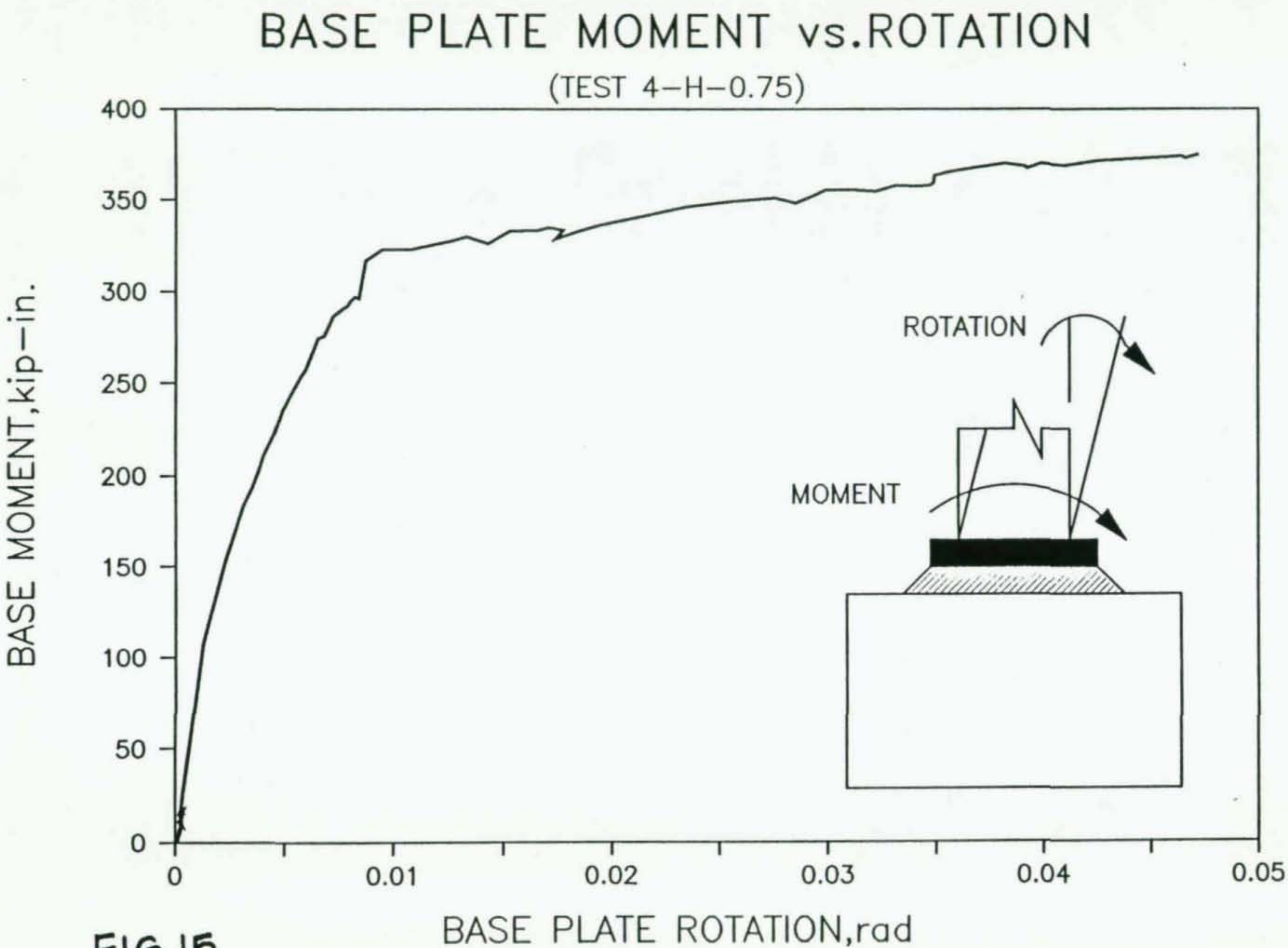

BASE PLATE MOMENT vs.ROTATION
(TEST 4-H-0.75)
BASE MOMENT,kip-in.
400
350
300
250
200
150
100
50
0
0
0.01
0.02
0.03
0.04
0.05
BASE PLATE ROTATION,rad
ROTATION
MOMENT

FIG. 15

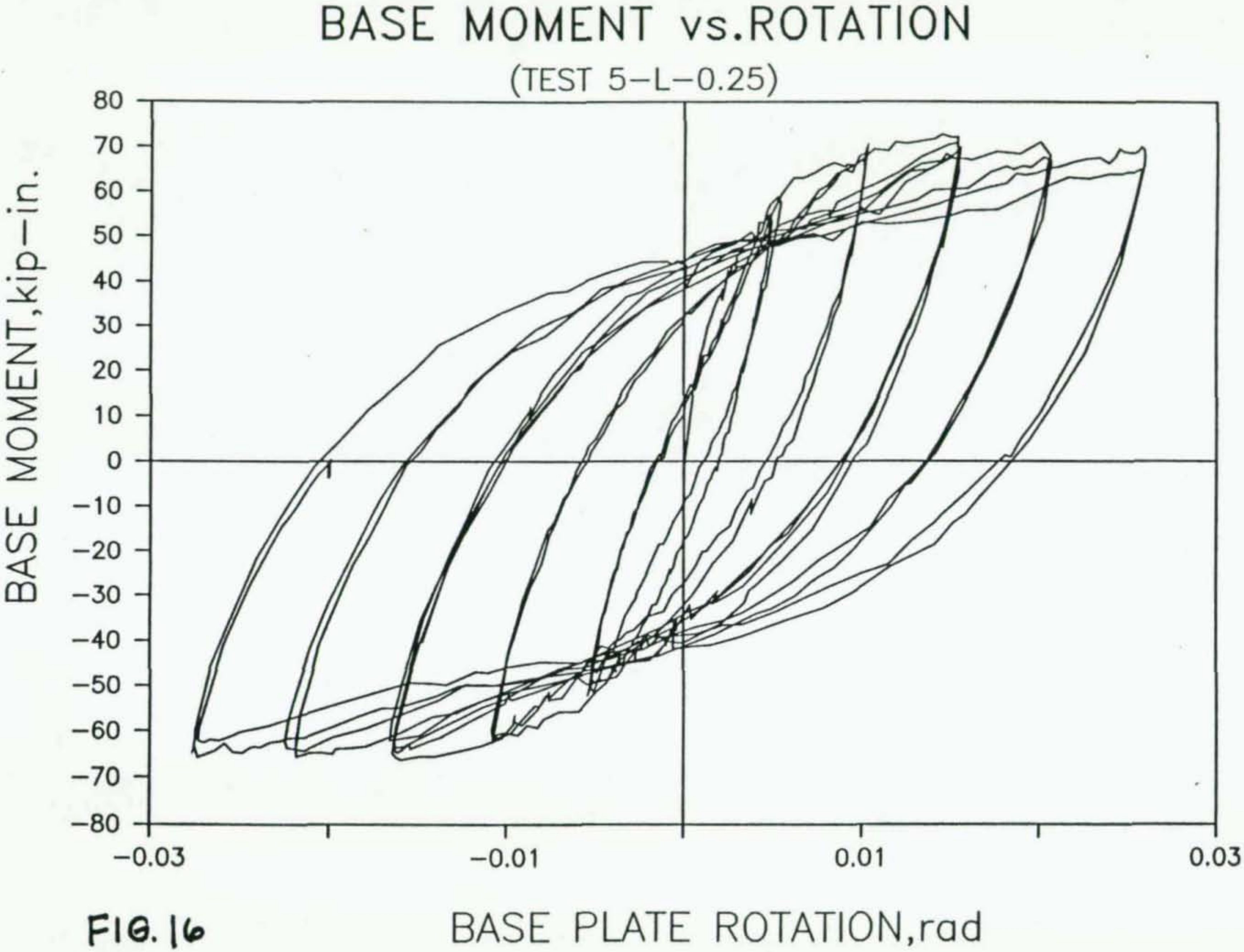

BASE MOMENT vs.ROTATION
(TEST 5-L-0.25)
BASE MOMENT,kip-in.
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-0.03
-0.01
0.01
0.03
BASE PLATE ROTATION,rad

FIG. 16

BASE PLATE MOMENT vs. ROTATION

(TEST 5–0.25)

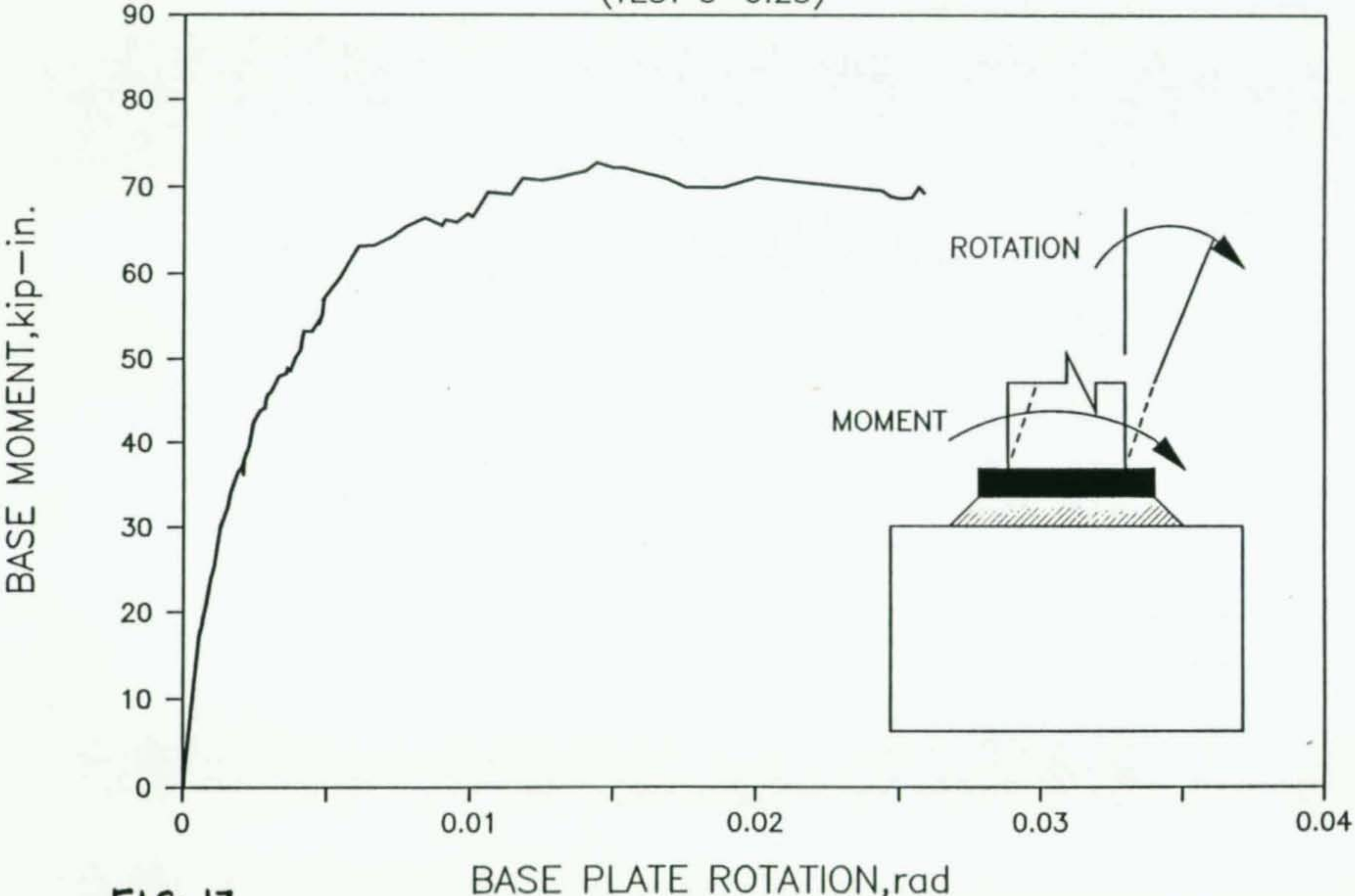
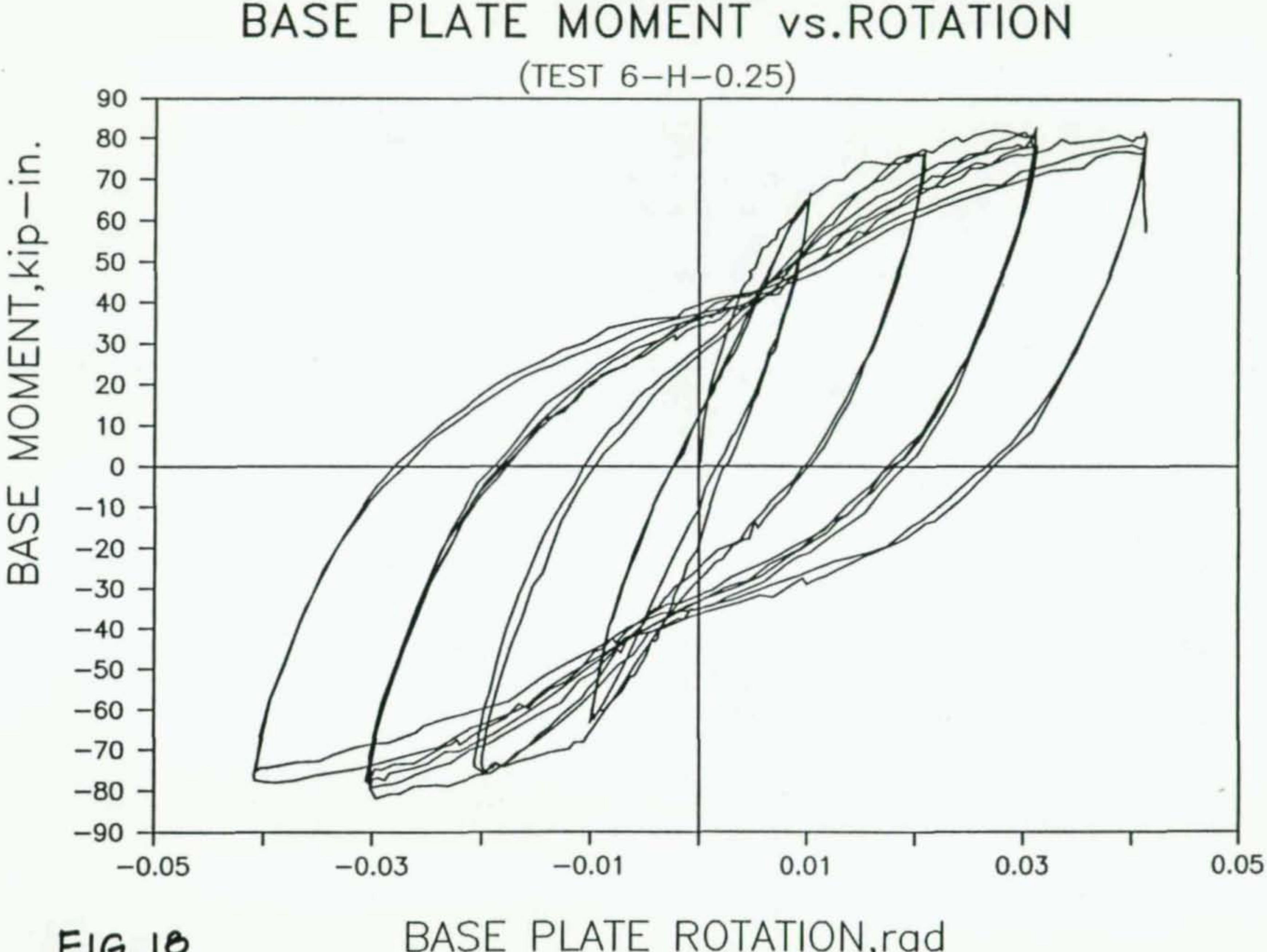


FIG. 17



BASE PLATE MOMENT vs.ROTATION
(TEST 6-H-0.25)
BASE MOMENT,kip-in.
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
-50
-60
-70
-80
-90
-0.05
-0.03
-0.01
0.01
0.03
0.05
BASE PLATE ROTATION,rad

FIG. 18

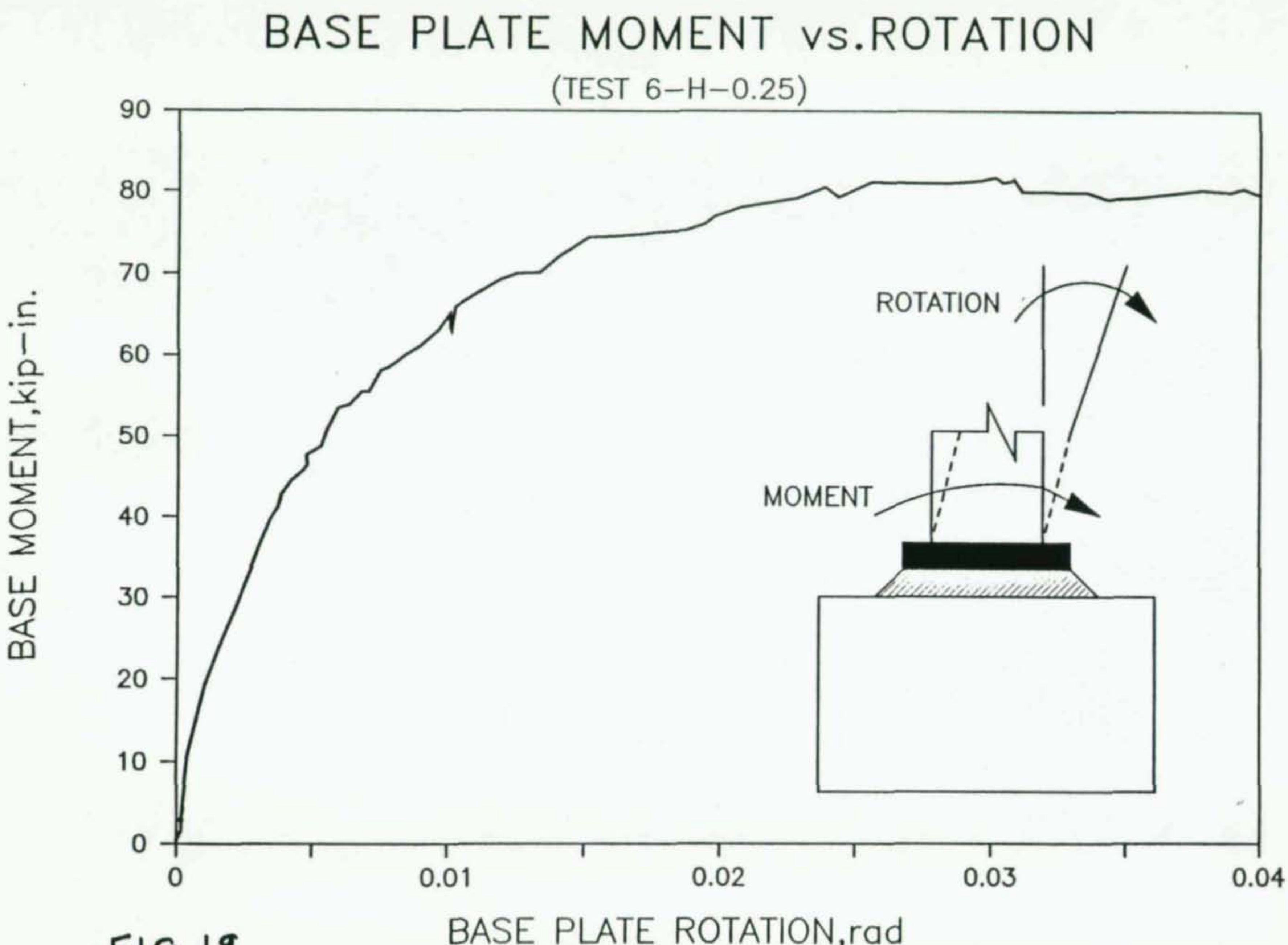

BASE PLATE MOMENT vs.ROTATION
(TEST 6-H-0.25)
BASE MOMENT,kip-in.
0
10
20
30
40
50
60
70
80
90
0
0.01
0.02
0.03
0.04
BASE PLATE ROTATION,rad
ROTATION
MOMENT

FIG. 19