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Authors

Pan, Austin

Moehle, Jack

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LATERAL DISPLACEMENT DUCTILITY OF REINFORCED CONCRETE FLAT PLATES

BY

AUSTIN PAN

AND

JACK P. MOEHLE

DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF CALIFORNIA
BERKELEY, CALIFORNIA

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REINFORCED CONCRETE FLAT PLATES

by

Austin Pan
Graduate Research Assistant
University of California at Berkeley

and

Jack P. Moehle
Associate Professor of Civil Engineering
University of Calfiornia at Berkeley

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INTRODUCTION

In design of buildings to resist earthquake ground motions, the prevailing ductile design philosophy typically requires that elements of the structural system be capable of deforming into the inelastic range. This requirement often, but not always, extends to elements that are not considered in design as part of the lateral load resisting system. For example, a common form of construction in seismic zones in the United States combines flat-plate frames, to carry gravity loads, and shear walls, to resist the earthquake loads. It is unclear in this form of construction whether the flat-plate connection will possess sufficient lateral displacement capacity to survive the lateral deformations that can reasonably be expected during a strong earthquake.

The objective of this paper is to examine the available data, and therefrom develop an understanding of the major parameters that influence the lateral displacement capacity and ductility of reinforced concrete flat plates. The significant effects of gravity load and biaxial lateral loading are presented. Implications of the findings with regard to seismic design and performance are discussed.

REVIEW OF EXPERIMENTAL OBSERVATIONS

Experimental data from several research investigations reported in the literature were gathered for this study.

Included are data from Ghali et. al. [3], Islam and Park [4], Hanson and Hanson [5], Morrison and Sozen [2], Hawkins et. al. [1], Symonds, et. al.[10], Zee and Moehle [6], and recent tests conducted by the authors (Appendix I). Prerequisites for selection of these data were: (1) specimens represented interior connections, (2) specimens contained no slab shear reinforcement, and (3) specimens were loaded to simulate effects of shears and moments due to vertical and lateral loads. Some of the specimens [Spec. No. 1-11, 15-21] were subjected to reversed cyclic lateral loads simulating severe seismic loading. Others [Spec. No. 12-14, 22-23] were subjected to effectively monotonic lateral loading.

Relevent data of these experiments are tabulated in Table 1. Brief descriptions of the specimens, test procedures, and principal results follow. More detailed information can be found in the original papers [1-6,10]. Appendix I provides details of recent tests by the authors [Spec. No. 16-19] that have not been reported previously in the literature.

Tests Description

All of the test specimens reported herein were similar in that a single interior connection sub-assemblage was isolated from an idealized structure and loads were applied to simulate lateral load effects (Fig. 1). The column in each test specimen extended above and below the slab to "pinned" connections located at a point equivalent to the column midheight of an idealized building (Fig. 1). In the direction parallel to

lateral loading, the slab extended on either side to a point equivalent to midspan of the prototype slab, except for specimens No. 1-6 which extended 0.3 times the slab span. In the transverse direction, the total slab width ranged between half the longitudinal span and the full longitudinal span.

Slab gravity loads were simulated in some of the tests [Spec. No. 1-6, 10-11, 15-17,20-21]. The procedure of adding these vertical loads varied; in some cases subsidiary weights were placed on the slab surface [Spec. No. 10-11, 15-17, 20-21], and in other cases vertical actuators applied the load [Spec. No. 1-6].

In some of the tests [Spec. No. 7-19] lateral load effects were simulated by attaching the slab edge to "roller" supports and then loading laterally at the top of the column (Fig. 1a). In the other tests lateral load effects were simulated by actuators that applied equal and opposite vertical loads to the ends of the slab while the columns were restrained against lateral displacement (Fig. 1b). There is no general agreement as to which of these two testing techniques is more representative of actual conditions. Both probably represent situations to which a redundant, nonlinear building system is exposed.

All tests reported previously in the literature have considered only uniaxial lateral loadings. The tests by the authors [Spec. No. 16-19] considered both uniaxial and biaxial lateral loadings as reported in Appendix I.

Lateral Load-Displacement Relations

For those specimens that were tested by laterally displacing the upper column [Spec. No. 7-19], the lateral load-displacement relation is self-evident from measured lateral loads and displacements at the top of the upper column. For the specimens that were loaded by vertically displacing the slab ends [Spec. No. 1-6, 20-23], the lateral load (reaction at the upper column) was resolved by statics. Lateral displacement of the specimen, D_h, was taken as

$$D_h = D_v(h_c/l_1) \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \quad (1)$$

in which D_v = difference of the vertical displacements at opposite ends of the slab at the point of vertical load application, h_c = total column height, and l_1 = total span of the slab in the direction of lateral loading (Fig. 1). For the specimens No. 1-6 effective lateral displacements, D_h, obtained by equation (1) were further multiplied by a factor 1/0.6 because, as described in [1,10], the slab span of the specimen represents approximately 0.6 times the total span of the slab in the prototype.

Measured lateral load-displacement relations from three of the specimens that were subjected to reversed cyclic lateral loads are plotted in Fig. 2. These relations display the broad range of behaviors observed for the connections included in this study. The data in Fig. 2a reveal a relatively stable hysteretic response to lateral drifts well beyond the range of practical interest in building design. The data in Fig. 2b also reveal a stable hysteretic response, but to significantly less drift than

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obtained for the specimen in Fig. 2a. Failure occurred in a relatively sudden punching shear mode after the slab reinforcement near the column had yielded. Lateral drift at failure in this case can be reasonably expected for a multistory building subjected to a intense earthquake motion [19]. Unless measures are taken to control lateral drift, this connection may not be suitable in a building designed to resist strong earthquakes. Figure 2c presents a response history that is even further curtailed by punching shear failure. The low level of lateral drift, and complete lack of apparent ductility, make this type of specimen clearly unsuitable for almost all types of buildings subject to severe earthquake loading.

It is useful to qualitatively classify responses of the specimens in Table 1 according to the response types depicted in Fig. 2. Column 8 of Table 2 lists the assigned classification, either A, B, or C, signifying behavior similar to that plotted in Fig. 2a, 2b, or 2c, respectively.

Two measures are used in this study to quantify the lateral deformation capacities of the specimens. The first measure is lateral drift D_h at failure, as a percent of specimen height h_c. Lateral drift at failure is tabulated for each specimen in column 6 of Table 2.

The second measure of deformation capacity is the lateral displacement ductility at failure. Lateral displacment ductility cannot be uniquely defined for slab-column connections because the force-displacement relation has no distinct yield point

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(because yield spreads gradually across the slab transverse width [1]). To overcome the uncertainty in defining the yield displacement, an arbitrary procedure was applied. The procedure, illustrated in Fig. 3, was to first construct the envelope relation between lateral displacement and lateral load. The envelope relation was then idealized by an elasto-plastic relation (Fig. 3). The initial slope of the idealized relation is a secant through the measured relation at a load equal to two-thirds of the measured strength. The plastic portion of the idealized relation passes through the maximum load and the maximum deformation at failure. The intersection between these two lines defines an effective yield displacement. Displacement ductility is then calculated as the ratio between the ultimate displacement and yield displacement ($\mu=D_u/D_y$) of the idealized relation (Fig. 3). The computed lateral displacement ductilities (μ) for the test specimens are listed in column 5 of Table 1.

THE EFFECT OF GRAVITY LOAD ON LATERAL DISPLACEMENT DUCTILITY

The level of gravity load carried by the slab is a primary variable affecting the apparent lateral ductility. This phenomenon has been identified in earlier tests by Kanoh and Yoshizaki [13]. The same trend is apparent in the relations plotted in Fig. 2. In that figure, lateral displacement ductility decreases from Fig. 2a to 2c, as gravity load carried by the slab increases (column 7, Table 2).

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In order to generalize the conclusion that gravity load affects lateral displacement capacity of the connection, lateral displacement ductility, (column 5, Table 2), was plotted versus the normalized gravity shear ratio, V_g/V_o (column 7, Table 2), for each connection. The value V_g is the vertical shear acting at failure on the slab critical section defined at a distance $d/2$ from the column face, in which d = the average slab effective depth. The quantity V_o is the theoretical punching shear strength in the absence of moment transfer, as given for these connections by Eq. 2 [14],

$$V_o = 4\sqrt{f'_c}b_o d \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (2)$$

in which V_o is in pounds, f'_c = concrete compressive strength in psi, and b_o = perimeter length of the slab critical section (bo and d have in. units.)

The relation between lateral displacement ductility, μ, and the gravity shear ratio, V_g/V_o, is plotted in Fig. 4 Clearly, for values of V_g/V_o exceeding approximately 0.4 there is virtually no lateral displacement ductility ($\mu=1.0$), that is, the connection fails by punching before any yield in the load-displacement relation is detected. As gravity shear decreases, there is an increase in the available ductility.

The effect of gravity shear on lateral drift at failure is presented in Fig. 5. As in Fig. 4, there is a reduction in available drift with increasing gravity shear ratio.

The observation that lateral deformation capacity and gravity shear are related also is not surprising, and has been

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identified previously [1,4]. According to the ACI Building Code [7] shear strength for square interior columns is defined by Eq. 2. This limiting shear strength has been established from test of slab-column connections for which shear failure occurred before widespread yielding of the slab reinforcement [9]. The condition for which this limiting shear stress is defined is consistent with the philosophy of the main body of the ACI Building Code, namely, that design loads are set sufficiently high and inelastic redistribution is sufficiently limited that significant yield at connections will not occur. Under this loading condition, the surrounding slab confines the connection region. This confinement is believed to be the reason why observed nominal shear stresses at failure are larger for slab-column connections than for linear elements such as beams [14]. Some researchers [9] have hypothesized that the confining effect of the slab is diminished when widespread yield of the slab reinforcement occurs in the connection region. Experimental data in support of this hypothesis have been presented previously [9] for connections loaded solely by vertical loads. The data for lateral load tests presented in Fig. 4 augment these vertical load data.

THE EFFECT OF BIAXIAL LATERAL LOADING
ON LATERAL DISPLACEMENT DUCTILITY

During wind or earthquake loading, the slab-column connection resists lateral loads acting in multiple directions.

To investigate the influence of biaxial lateral loading, four specimens were tested. Limited details of the tests are presented for reference in Appendix I. In the test program, two specimens were tested with a uniaxial lateral displacement history, one with "high" gravity load (AP1), and one with relatively "low" gravity load (AP3). Nominally identical specimens (AP2 and AP4) were subsequently subjected to biaxial lateral load histories. The displacement histories are compared in Fig. 6. The biaxial load history followed sequentially the points 1 through 14 indicated in Fig. 6.

Figures 7 and 8 show the measured relations between lateral force and lateral drift. As can be observed in those figures, the biaxially loaded specimens (AP2 and AP4) failed at an earlier stage of testing in comparison with the companion uniaxially-loaded specimens. Both stiffness and strength were less for the biaxially-loaded specimens than for the equivalent uniaxially loaded specimens.

The lower resistance and toughness of the biaxially loaded specimens is explained as follows. Under uniaxial loading, resistance is attributable to torsion on faces AB and CD and shear and moment on faces BC and DA [1] (Fig. 9). If a uniaxially loaded connection is subsequently loaded in the transverse direction, faces AB and CD (which had previouly been loaded in torsion) begin to develop flexure and shear, while faces BC and DA (which had previously been loaded in moment and shear) begin to develop torsion. The interaction between flexure, shear and

torsion [11] are such that the net connection resistance in any given direction is less under biaxial loading than under uniaxial loading. Similarly, more rapid degradation of the concrete occurs under biaxial loading.

Based on the observations from this series of tests, it is concluded that biaxial lateral loading, as might occur during an earthquake or wind loading, reduces the available strength, stiffness, and overall lateral displacement capacity of slab-column connections. Thus, since most of the data included in Fig. 4 and 5 were obtained from uniaxial tests, conclusions drawn therefrom may be on the unconservative side if biaxial loading occurs.

IMPLICATIONS OF THE TEST RESULTS

The data in Fig. 4 and 5 indicate that the available lateral displacement ductility of reinforcement concrete flat-plate connections without shear reinforcement is low by comparison with values often considered marginally acceptable in seismic design [11]. However, the existance of low ductility does not de facto equate with poor performance during a strong earthquake loading. For typical slab-column connections, relatively high flexibility may protect the connection from large displacement ductility demands. For example, Fig. 10 presents idealized load displacement envelopes of a typical slab-column connection [6] and a slender shear wall [23] that might be used to stiffen a flat-plate building. If the wall is sufficiently stiff to

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restrain lateral interstory drifts to approximately 1.5 percent of story height (a value occasionally quoted as a reasonable upper bound for severe seismic loading [20]), the required displacement ductility of the wall will be approximately six. The required displacement ductility of the slab-column connection will be less than two.

According to the data presented in Fig. 4 and 5, a slab-column connection can be expected to possess some minimal ductility, and a drift capacity of at least 1.5 percent of interstory height, only if the vertical shear on the connection is less than or equal to approximately 40 percent of the direct punching shear strength. Expressed in terms of shear stresses, the nominal shear stress due to vertical loads should be limited to approximately $(0.4)(4\sqrt{f'_c}) \sim 1.5\sqrt{f'_c}$ psi on the slab critical section. The vertical load should be taken at least equal to the design gravity load shear. Although it is likely that vertical accelerations during earthquakes (frequently of the same magnitude as the lateral ground accelerations) will further increase the total connection shear, there is no evidence at the present time to prove that this effect should be included when computing the vertical connection shear. Likewise, there is no evidence to confute this possibility.

If gravity loads alone are considered in computing the proposed limiting shear stress, the minimum required column size to satisfy the recommendation is approximately 19x19 in. for a building slab having 20-foot bays, an eight inch slab, 15 psf

superimposed dead load, 40 psf service live load, and 4000-psi concrete normal weight concrete. Supporting calculations are in Appendix II.

For lateral interstory drifts exceeding 1.5 percent of interstory height, adequate performance of the flat-plate cannot be assured. For this reason, a suitable structural system should be provided to limit lateral drifts.

SUMMARY AND CONCLUSIONS

Experimental data from several investigations are analyzed to identify the parameters that influence the lateral displacement ductility of reinforced concrete flat-plate connections subjected to lateral loads. Only interior, conventionally-reinforced slabs without slab shear reinforcement are considered. Implications of the findings are discussed. The following main conclusions are recapitulated:

1. The magnitude of the gravity load shear carried by the slab is a primary variable affecting the lateral displacement capacity and ductility.

2. For a given level of gravity load, biaxial lateral loading reduces lateral load stiffness, strength, and available ductility.

3. The magnitudes of gravity loads and of lateral interstory drifts should be controlled to ensure that integrity of the slab-column connection is maintained under seismic loading. Lateral interstory drifts under an extreme earthquake

loading should not exceed 1.5 percent of interstory height. At this level of deformation, the available data indicate that the flat-plate connection will perform adequately if the gravity level shear stress acting on the slab critical section does not exceed $1.5\sqrt{f'_c}$.

NOTATION

A_c =   cross section area of slab critical section.

b_o =   perimeter of critical section for slab, in.

c_1 =   column dimension in direction of loading, in.

c_2 =   column dimension transverse to direction of loading, in.

D_h =   horizontal displacement at top of column. See Fig. 1a.

D_u =   ultimate displacement at failure.

D_v =   difference of the vertical displacement at opposite ends of the slab. See Fig. 1b.

D_y =   yield displacement.

d   =   distance from extreme compression fiber to centroid of tension reinforcement, in.

d_{avg} =   average d, in.

h   =   overall thickness of slab, in.

h_c =   column height.

f'_c =   concrete compression strength, psi.

f_y =   steel yield strength, psi.

l_1 =   span length in direction of lateral load.

l_2 =   span length transverse to direction of lateral load.

V_g =   shear due to gravity loads including self weight, kips.

V_{all} =   allowable shear stress.

V_o = theorectical punching shear strength Eq. 2.

w_g = dead load.

w_l = live load.

w_{sdl} = superimposed dead load.

w_{slab} = slab self weight.

μ = displacement ductility. = D_u/D_y

APPENDIX I - TESTS WITH BIAXIAL LOADING

To investigate the influence of biaxial loading on connection behavior, four tests were conducted in 1986 by the authors at the University of California at Berkeley. A brief summary of the experimental program follows.

Specimen Description

Four identical interior connections were constructed as shown in Figure (A.1). Slab dimensions were 13 ft by 13 ft with a thickness of 4.8 inches. Column dimensions were 10.8 inches by 10.8 inches in cross section extending 3 ft. above and below the slab. Concrete compression strength was specified at 4,000 psi. Slab reinforcement was all No. 3 Grade 60 reinforcing bars. The column was heavily reinforced to ensure failure of the slab connection. The slabs were similar in design to those tested previously by Moehle and Diebold (21) and by Zee and Moehle (6).

Experimental Setup

To allow for bi-directional movement, the edges of the slab were supported by steel transducer struts fitted with universal bearings at both ends. The top and bottom of the column were also

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fitted with universal bearings (Fig. A.1). At the west end of the slab a torsional restraining frame was attached to reduce in-plane rigid body twisting during testing. Two hydraulic load actuators were connected at the top of the column for lateral loading. Gravity load was simulated and maintained at a constant value by adding lead blocks on the slab and by jacking up the bottom of the column. The specimen was instrumented to measure deflections, slab rotations and profiles, reinforcement strains, and secondary displacements.

Test Sequence

To investigate the influence of gravity load on connection behavior, high gravity load $(0.35V_o)$ was imposed on specimens AP1 and AP2, and low gravity load $(0.18V_o)$ was imposed on specimens AP3 and AP4. Specimens AP1 and AP3 were tested with uniaxial lateral load. Tests of specimens AP2 and AP4 were biaxial. Figure 6 shows the cyclic displacement sequence for the uniaxial tests. The biaxial tests adopted the same sequence but applied in a "clover leaf" pattern from points 1 through 14 of the diagram illustrated in Fig. 6. Plots of lateral force versus drift are shown in figures 7 and 8 for the four specimens.

APPENDIX II - Sample Calculation of Minimum Required Column Size.
Given : $h= 8"$, $l_1=l_2= 20'$, $f'_c= 4000$ psi, $w_{sdl}= 15$ psf, $w_l=40$ psf
Find : Minimum required column size to ensure drift capacity of
        1.5 % and lateral displacement ductility of two.

$d = h-1.0 = 7$ in.

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$w_{slab} = (8/12)X150 = 100$ psf

$w_u = 0.75(1.4D+1.7L+1.7(1.1E))$
$\quad = 0.75(1.4(100+15)+1.7(40)) = 171.75$ psf

$V_g = 171.75X20X20 = 68700$ lb

$v_{all} = 1.5\sqrt{f'_c} = 94.9$ psi

$A_c = V_g/v_{all} = 68700/94.9 = 723.9$ in^2

$b_o = A_c/d = 723.9/7 = 103.4$ in.

$b_o/4 = 25.9$ in.   (Assuming square column)

$c_1 = 25.9-d = 18.9$ in.   Say <u>19 inches</u>

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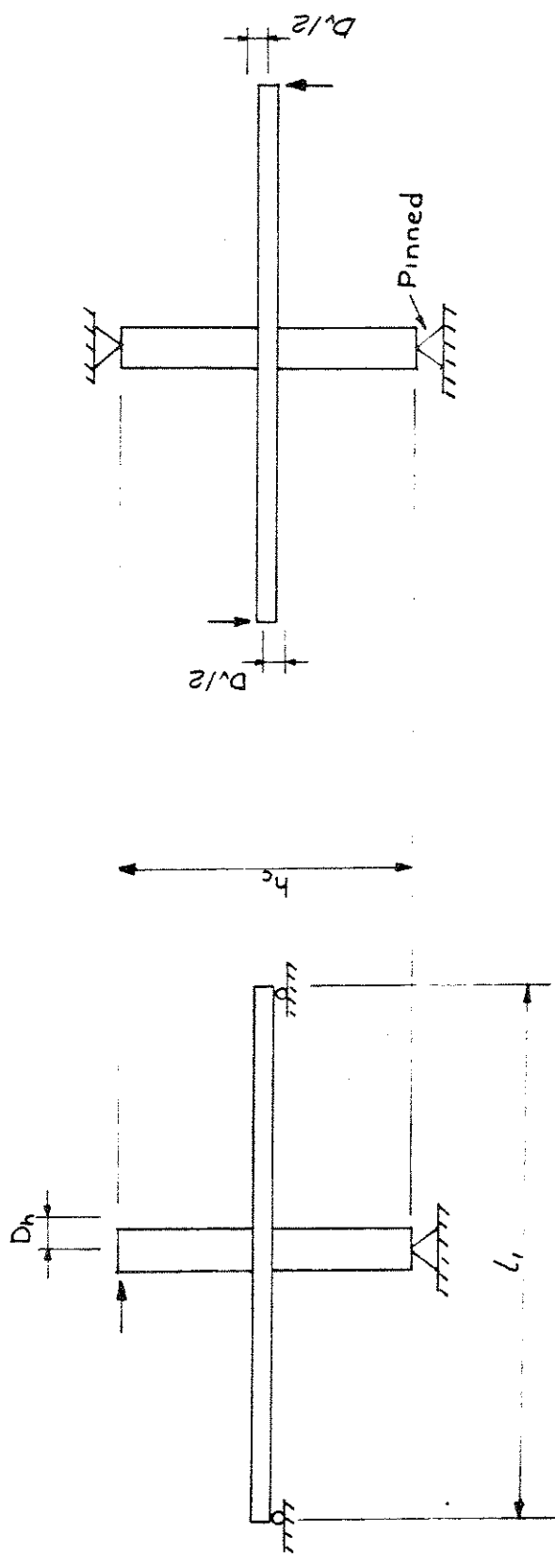
TABLE (1) - DATA OF EXPERIMENTAL SPECIMENS

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Specimen Number	Label *	Researchers	c1 (in.)	c2 (in.)	h (in.)	d/vg (in.)	l1 (in.)	l2 (in.)	lu (in.)	ptc+gh **	ptc+gh ***	f_y (psi)	f'_c (psi)
1	S1	Hawkins et al	12	12	6	4.4	144	84	102	0.012	0.0059	66600	5050
2	S2		12	12	6	4.4	144	84	102	0.0164	0.0049	67100	3400
3	S3		12	12	6	4.1	144	84	102	0.012	0.004	66000	3400
4	S4		12	12	6	4.4	144	84	102	0.012	0.0059	66600	4600
5	S6	Symonds et al	12	12	4	4.4	144	84	102	0.0181	0.0091	66600	3300
6	S7		12	12	4	4.4	144	84	102	0.012	0.0049	67100	3940
7	S1	Morrison and Sozen	12	12	3	2.38	72	72	44	0.0095	0.0045	46500	6640
8	S2		12	12	3	2.38	72	72	44	0.0093	0.0076	47900	5090
9	S3		12	12	3	2.38	72	72	44	0.0131	0.0131	46600	4720
10	S4		12	12	3	2.38	72	72	44	0.0098	0.0098	46000	5000
11	S5		12	12	3	2.38	72	72	44	0.0098	0.0098	47300	5110
12	SM 0.5	Ghali et al.	12	12	6	4.75	72	72	46	0.005	0.00162	69000	5329
13	SM 1.0		12	12	6	4.75	72	72	46	0.01	0.00333	69000	4835
14	SM 1.5		12	12	6	4.75	72	72	46	0.015	0.005	69000	5794
15	Interio	Aoc	5.4	5.4	2.9	1.89	72	72	36	0.008	0.008	53100	5400
16	AP1	Authors	10.8	10.8	4.81	4.07	144	144	72	0.0086	0.0029	70200	4250
17	AP2		11.8	10.8	4.84	4.07	144	144	72	0.0086	0.0029	70200	4400
18	AP3		10.8	10.8	4.84	4.07	144	144	72	0.0086	0.0029	70200	4600
19	AP4		10.8	10.8	4.89	4.07	144	144	72	0.0086	0.0029	70200	4500
20	IPP	Islam & Park	9	9	3.5	2.75	108	90	60	0.0106	0.00534	54300	4630
21	IP3C		9	9	3.5	2.75	108	90	60	0.0106	0.00534	45800	4810
22	B7	Hanson & Hanson	12	4	3	2.03	72	48	63	0.015	0.015	51400	1730
23	C8		6	12	3	2.03	72	48	63	0.015	0.015	54000	4760

* Designation used by original researchers.

** Top steel ratio within 3h strip over column.

*** Bottom steel ratio within 3h strip over column.

Note : 1 in. = 25.4 mm

Specimen Number	Label	Researchers	%Vf (type)	Ductility Ratio		SD/Avg	Classification
1	2	3	4	5	6	7	8
1	S1	Hawkins et al		1.40			C
2	S2						C
3	S3						C
4	S4						C
5	S6	Symonds et al					
6	S7						
7	C7	Morrison and Sozen					
8	C8						
9	C8						
10	C4						
11	C5						
12	SM 0.5	Ghali et al					
13	SM 1.0						
14	SM 1.5						
15	Interior	Aso					
16	4P1	Authors					
17	P2						
18	P3						
19	3P4						
20	1P2	Islam & Park					
21	1P3						
22	B7	Hanson & Hanson					
23	C8						

(a) *Lateral Load at Column* (b) *Vertical Load at Slab Edges*

FIG. 1 — LATERAL LOAD SIMULATION

(a) Spec. no. 7

$V_g/V_o = 0.031$

$\mu = 5.2$

FORCE (KIPS)

40
30
20
10
0
-10
-20
-30
-40

-0.06   -0.04   -0.02   0.02   0.04   0.06

DRIFT RATIO

(b) Spec. no. 16

$V_g/V_o = 0.369$

$\mu = 1.7$

FORCE (KIPS)

10
8
6
4
2
-2
-4
-6
-8
-10

-0.06   -0.04   -0.02   0.02   0.04   0.06

DRIFT RATIO
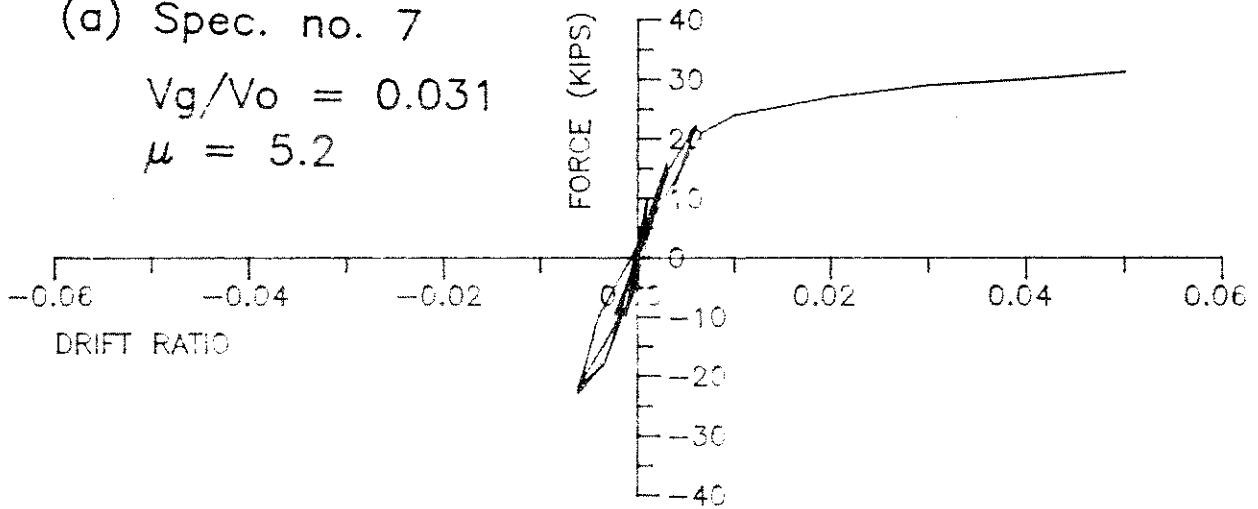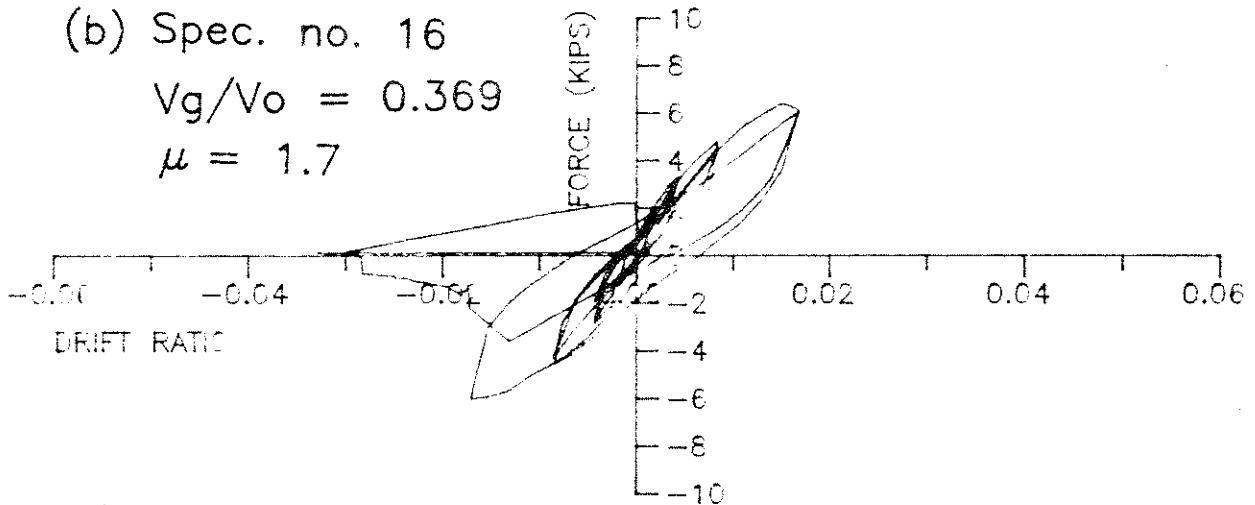
(c) Spec. no. 2

$V_g/V_o = 0.449$

$\mu = 1$

FORCE (KIPS)

10
8
6
4
2
0
-2
-4
-6
-8
-10

-0.06   -0.04   -0.02   0.00   0.02   0.04   0.06

DRIFT RATIO
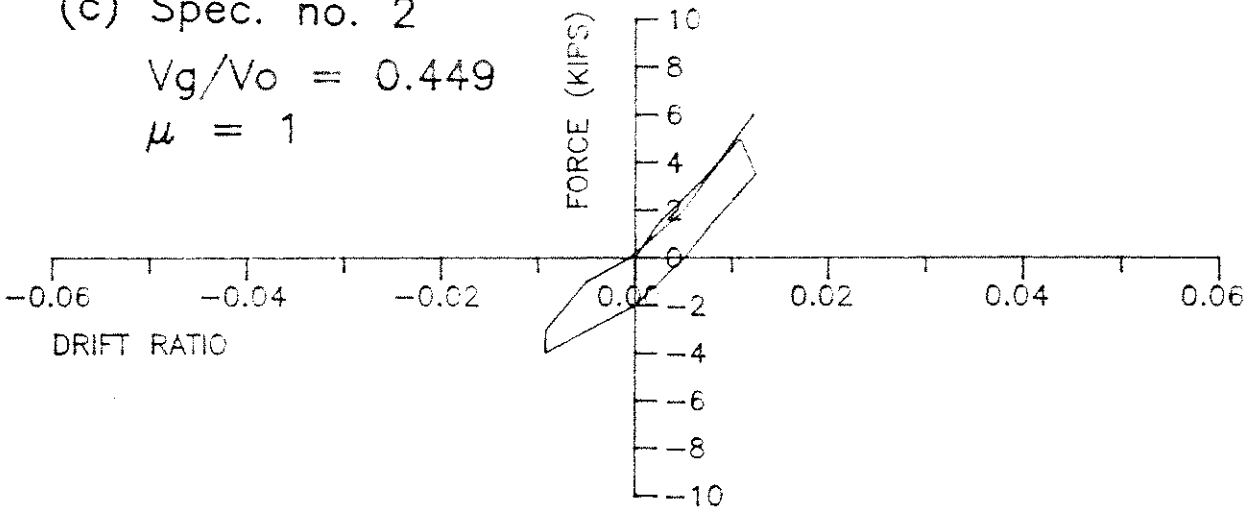
FIG 2 — CLASSIFICATION OF SPECIMEN RESPONSE
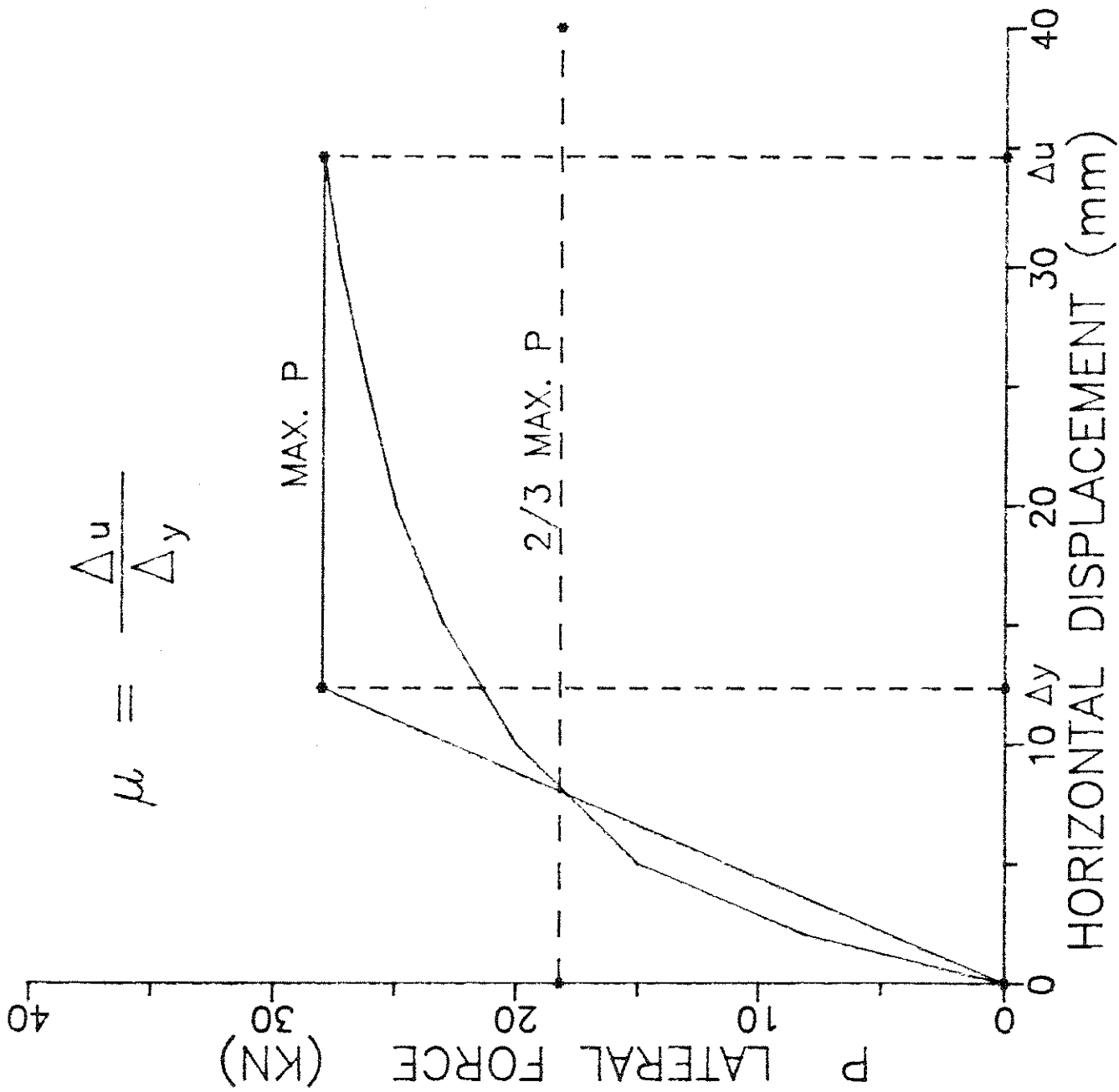
FIG. 3 – DEFINITION OF DISPLACEMENT DUCTILITY
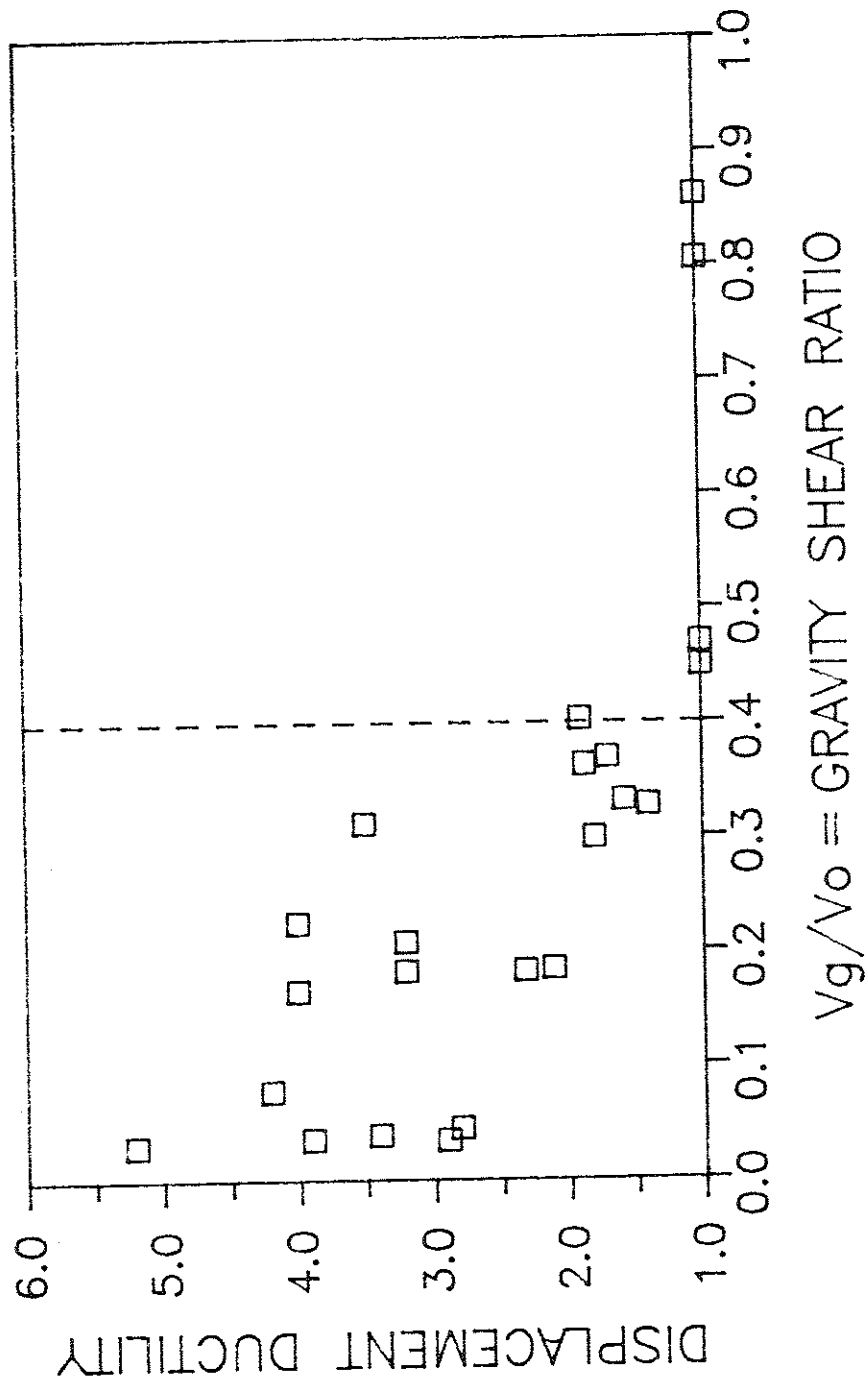
FIG. 4 — EFFECT OF GRAVITY LOAD ON DUCTILITY
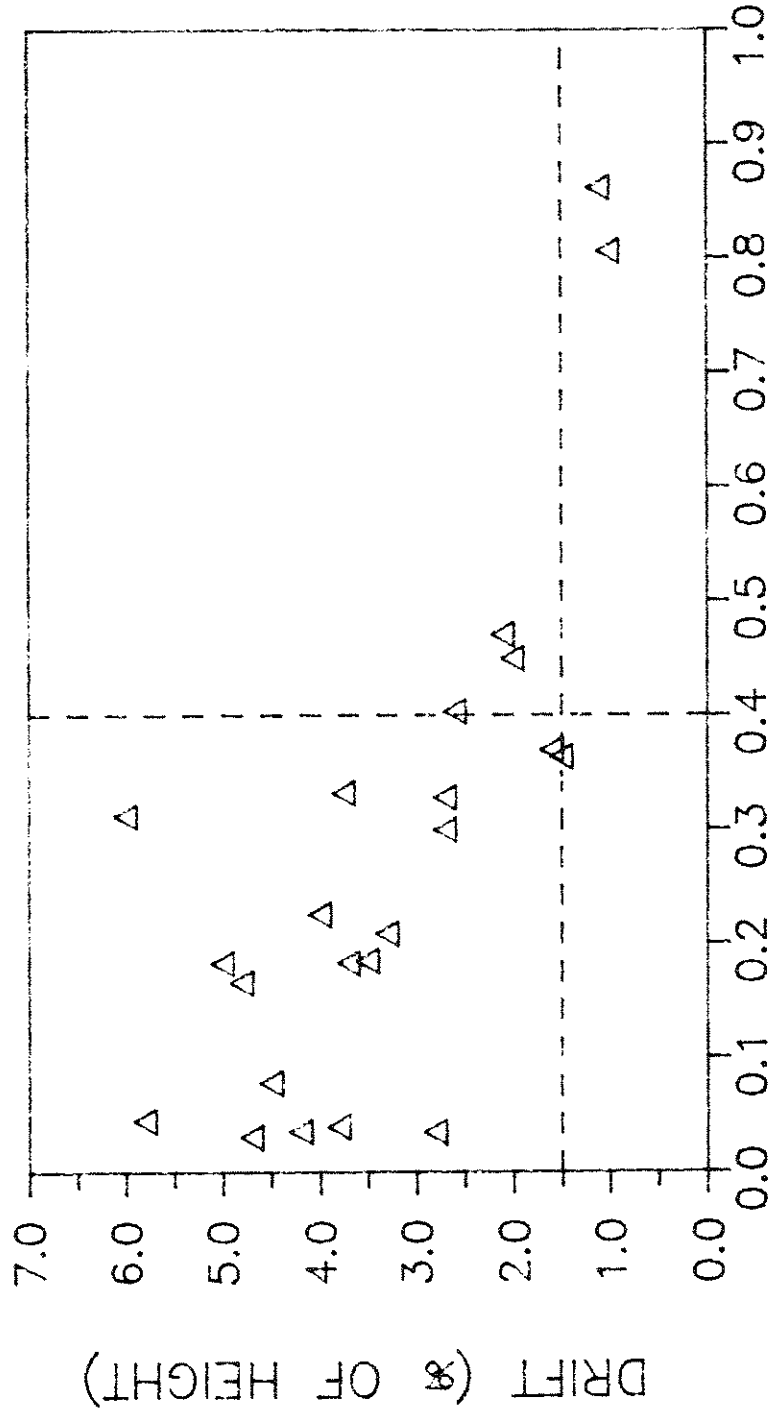
FIG. 5 — EFFECT OF GRAVITY LOAD ON DRIFT

UNI-DIRECTIONAL

* * * *
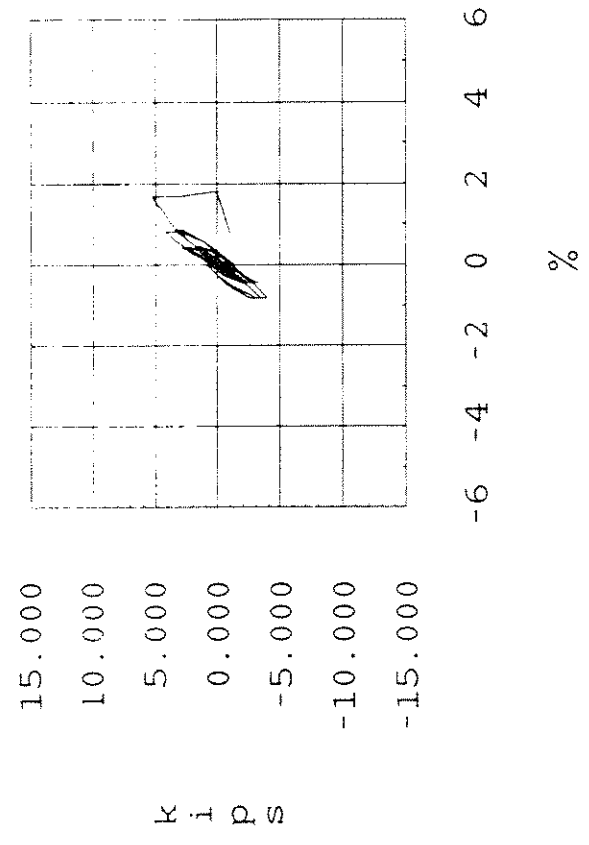
BI-DIRECTIONAL

```
        3           2      7        6
        
        4    11  1           10 5
        
        12      14          9
        
        13      8
```

DISPLACEMENT (INCHES)

3.0 2.0 1.0 0.0 -1.0 -2.0 -3.0

TIME
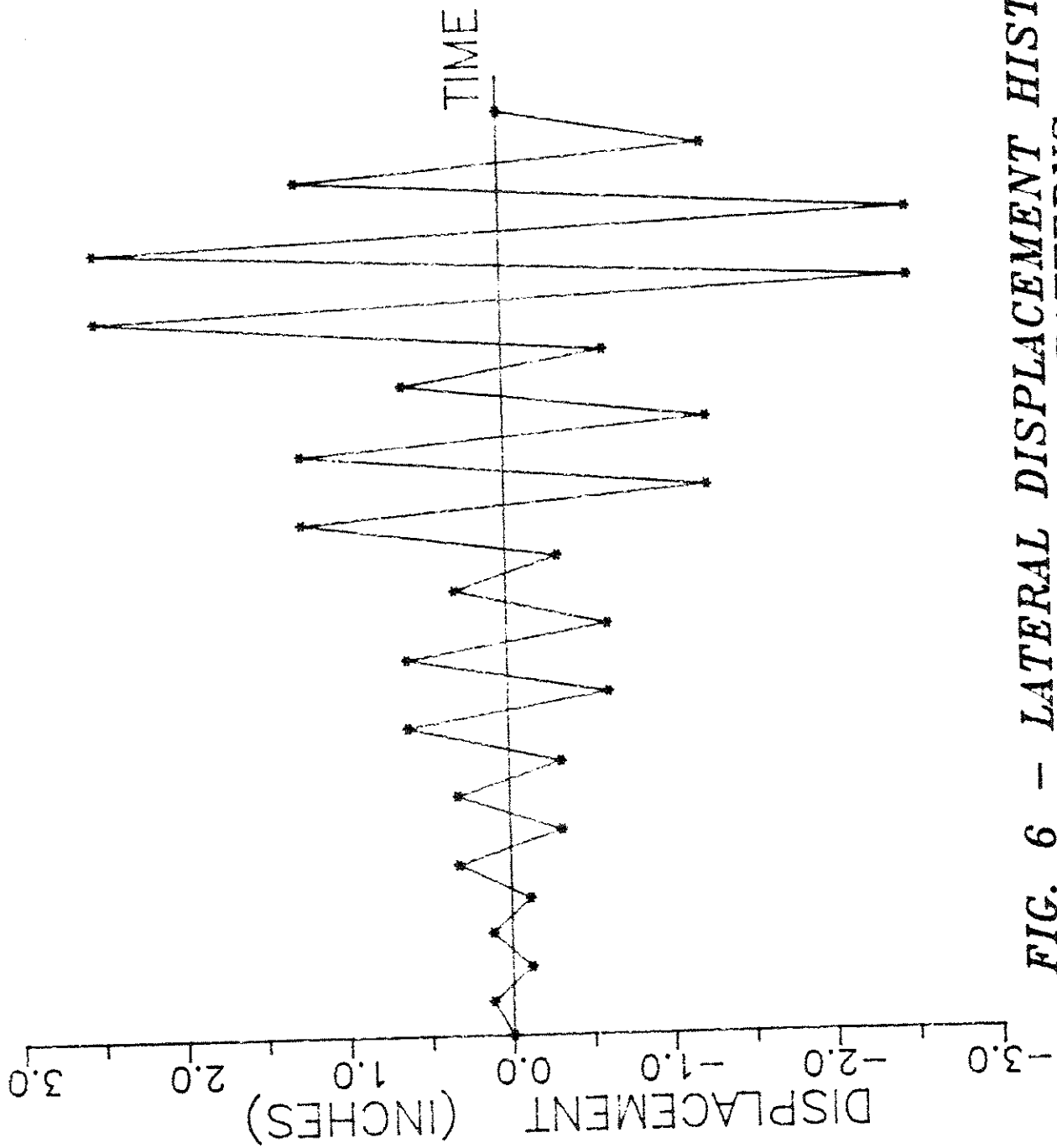
FIG. 6 — LATERAL DISPLACEMENT HISTORY
AND LOADING PATTERNS

SPECIMEN AP1 EW LATERAL FORCE – DRIFT

SPECIMEN AP2 NS FORCE – DRIFT

15.000
10.000
5.000
0.000
-5.000
-10.000
-15.000

k
i
p
s

-6   -4   -2   0   2   4   6

%

SPECIMEN AP2 EW FORCE – DRIFT

15.000
10.000
5.000
0.000
-5.000
-10.000
-15.000

k
i
p
s

-6   -4   -2   0   2   4   6

%

15.000
10.000
5.000
0.000
-5.000
-10.000
-15.000

k
i
p
s

-6   -4   -2   0   2   4   6

%
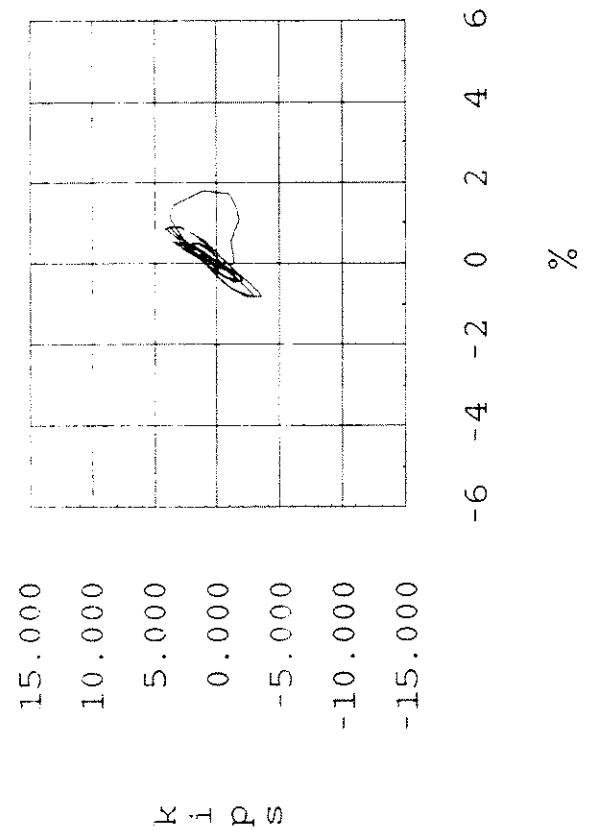
FIG. 7 – UNIAXIAL AND BIAXIAL TESTS (HIGH GRAVITY LOAD)
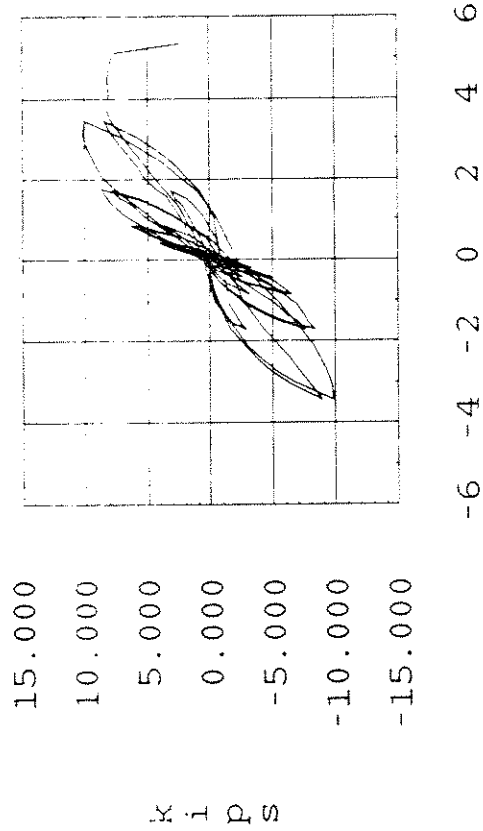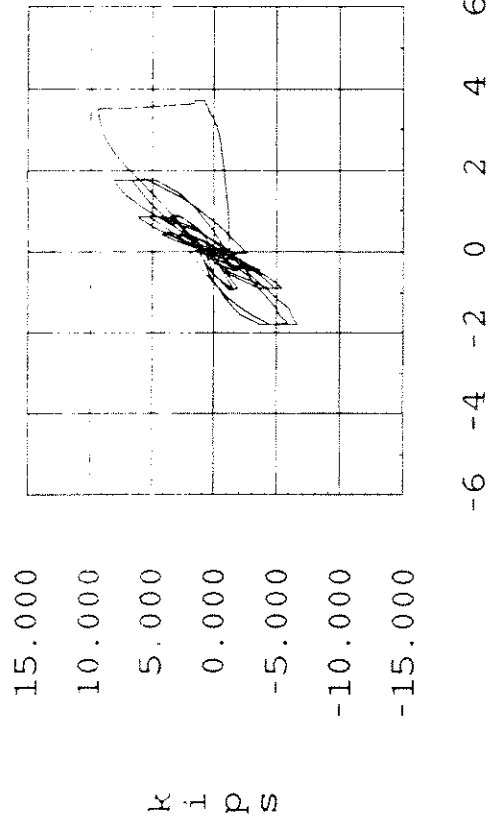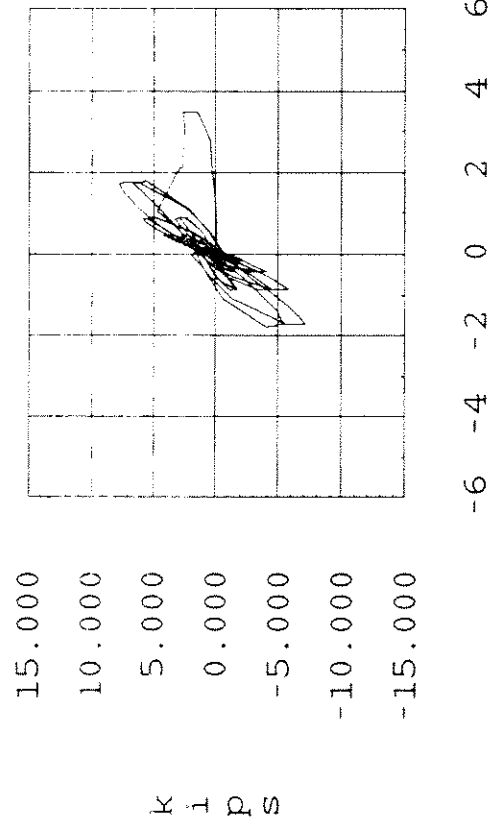
FIG. 8 — UNIAXIAL AND BIAXIAL TESTS (LOW GRAVITY LOAD)
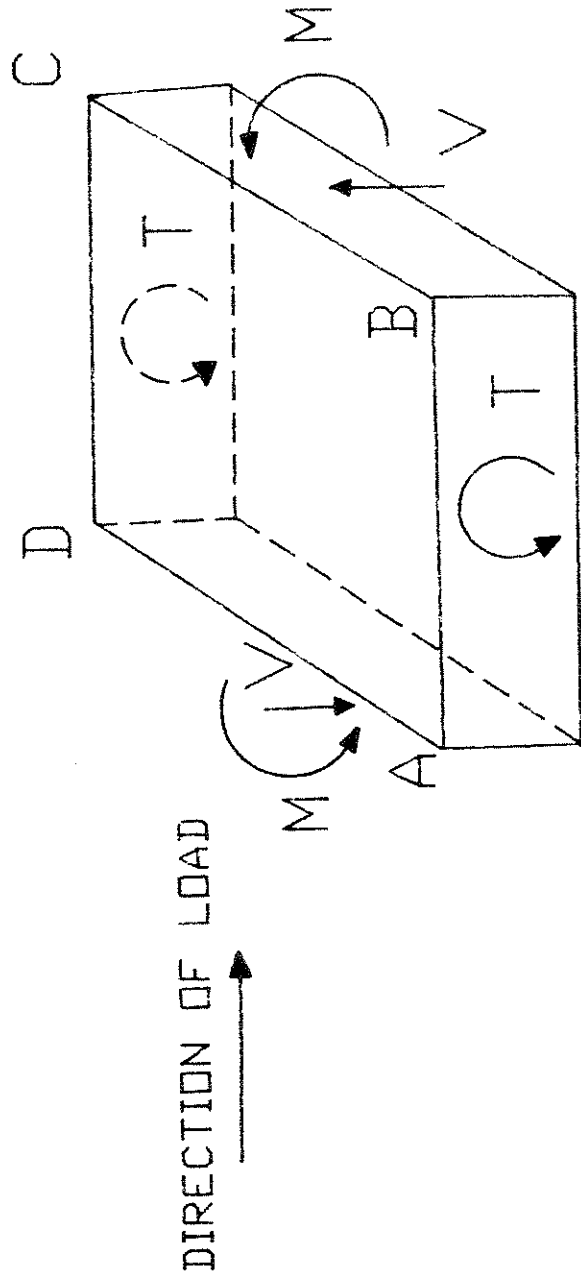
FIG. 9 — FORCES ON CONNECTION UNDER UNIAXIAL LOADING

SHEAR WALL   (Ref. 23)

SLAB-COLUMN   (Ref. 6)

DRIFT (%)

LATERAL LOAD / MAX. LATERAL LOAD
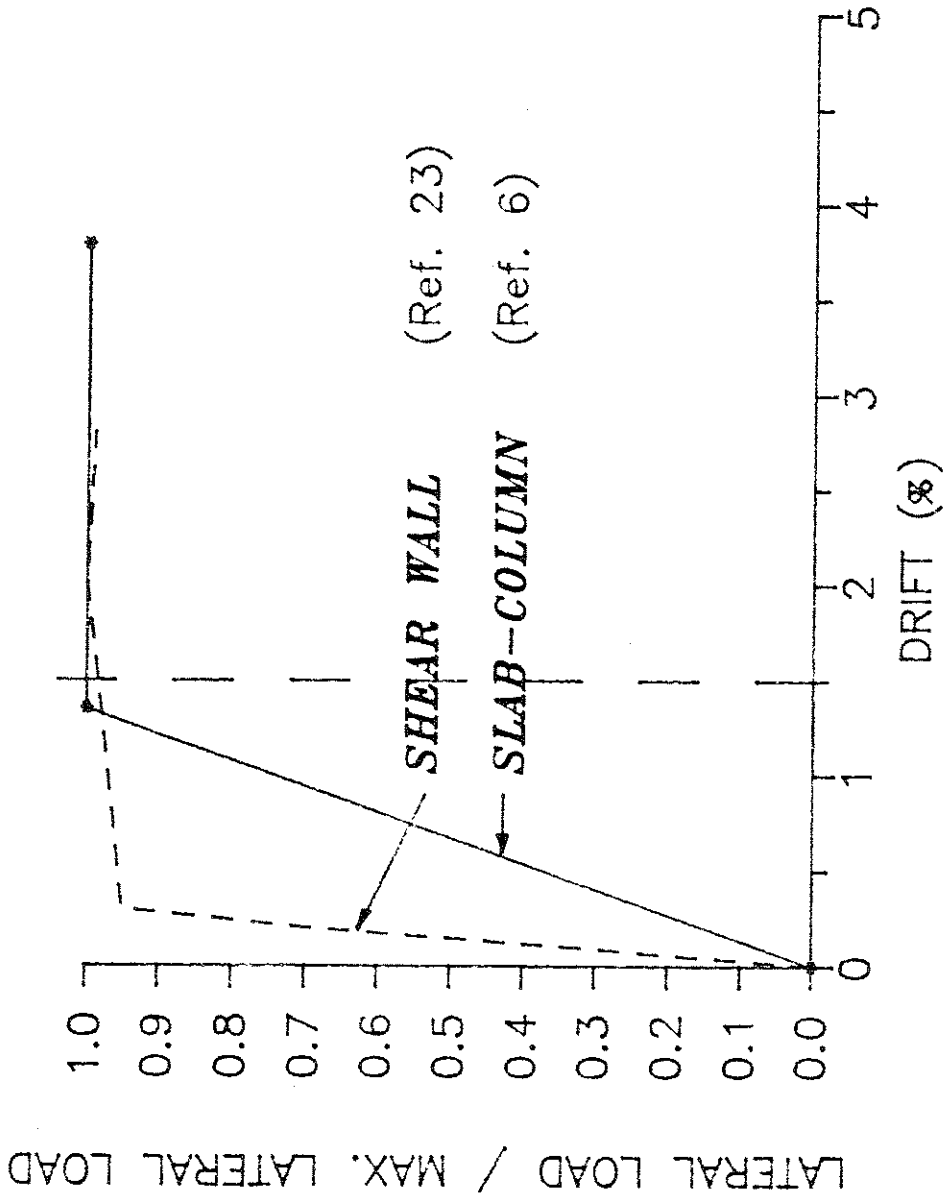
FIG. 10 – COMPARISON OF DUCTILITY REQUIREMENTS AT 1.5% DRIFT

ADDED WEIGHT

ACTUATOR

RESTRAINING FRAME

N

13'

13'

ACTUATOR

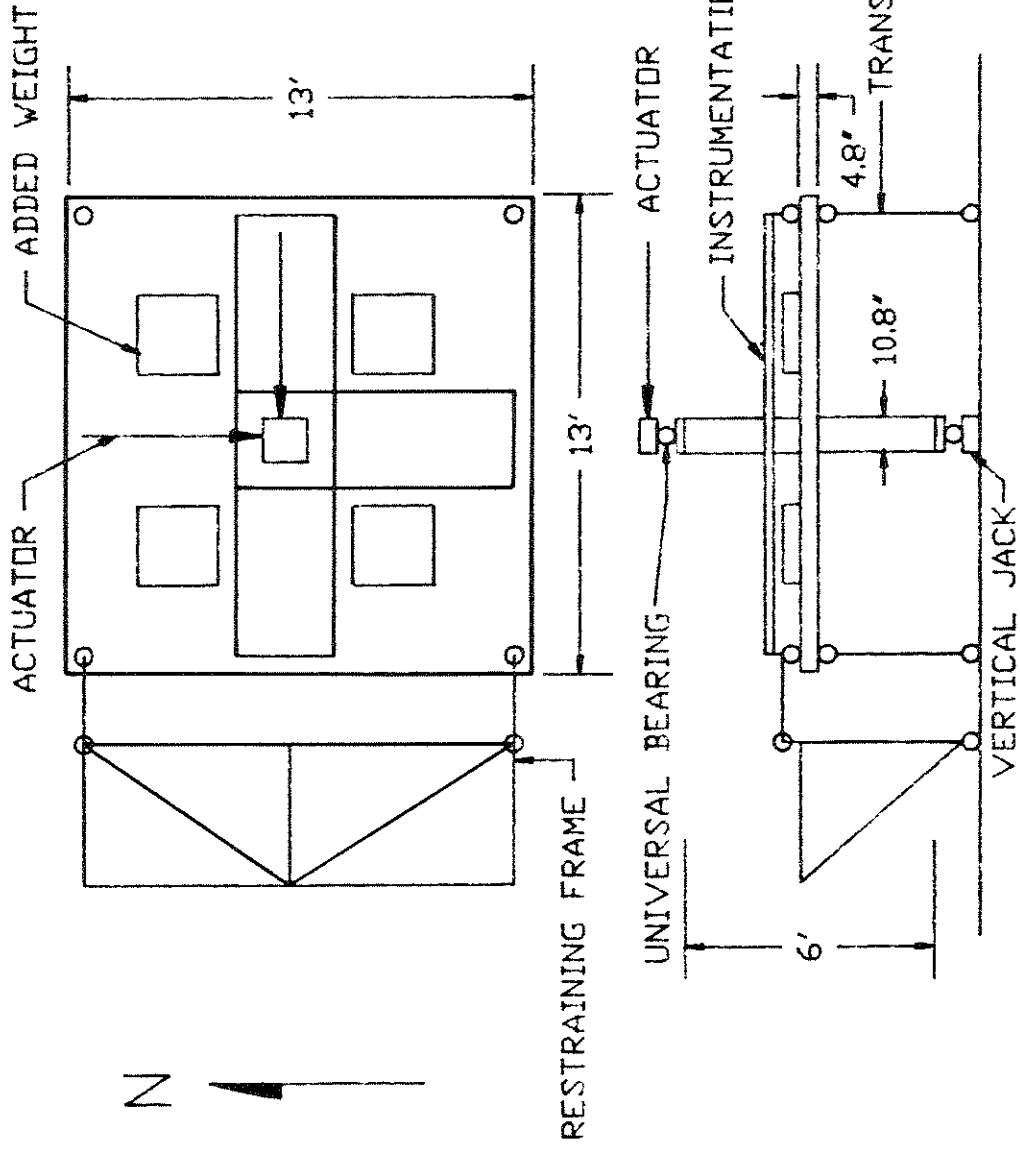INSTRUMENTATION FRAME

TRANSDUCER

UNIVERSAL BEARING

VERTICAL JACK

4.8"

10.8"

6'

FIG. A.1 — TEST SETUP FOR BIAXIAL LOADING