AIIM

Association for Information and Image Management

1100 Wayne Avenue, Suite 1100
Silver Spring, Maryland 20910

301/587-8202

Centimeter

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 mm

1 2 3 4 5

Inches

MANUFACTURED TO AIIM STANDARDS
BY APPLIED IMAGE, INC.

2

NUREG/CR-6104
LA-12649-MS

Shear Wall Ultimate Drift Limits

Prepared by
T. A. Duffey, A. Goldman, C. R. Farrar

Los Alamos National Laboratory

Prepared for
U.S. Nuclear Regulatory Commission

DISTRIBUTION OF THIS DOCUMENT IS UNLIMITED

AVAILABILITY NOTICE

Availability of Reference Materials Cited in NRC Publications

Most documents cited in NRC publications will be available from one of the following sources:

1. The NRC Public Document Room, 2120 L Street, NW., Lower Level, Washington, DC 20555-0001

2. The Superintendent of Documents, U.S. Government Printing Office, Mail Stop SSOP, Washington, DC 20402-9328

3. The National Technical Information Service, Springfield, VA 22161

Although the listing that follows represents the majority of documents cited in NRC publications, it is not intended to be exhaustive.

Referenced documents available for inspection and copying for a fee from the NRC Public Document Room include NRC correspondence and internal NRC memoranda; NRC bulletins, circulars, information notices, inspection and investigation notices; licensee event reports; vendor reports and correspondence; Commission papers; and applicant and licensee documents and correspondence.

The following documents in the NUREG series are available for purchase from the GPO Sales Program: formal NRC staff and contractor reports, NRC-sponsored conference proceedings, international agreement reports, grant publications, and NRC booklets and brochures. Also available are regulatory guides, NRC regulations in the *Code of Federal Regulations*, and *Nuclear Regulatory Commission Issuances*.

Documents available from the National Technical Information Service include NUREG-series reports and technical reports prepared by other Federal agencies and reports prepared by the Atomic Energy Commission, forerunner agency to the Nuclear Regulatory Commission.

Documents available from public and special technical libraries include all open literature items, such as books, journal articles, and transactions. *Federal Register* notices, Federal and State legislation, and congressional reports can usually be obtained from these libraries.

Documents such as theses, dissertations, foreign reports and translations, and non-NRC conference proceedings are available for purchase from the organization sponsoring the publication cited.

Single copies of NRC draft reports are available free, to the extent of supply, upon written request to the Office of Administration, Distribution and Mail Services Section, U.S. Nuclear Regulatory Commission, Washington, DC 20555-0001.

Copies of industry codes and standards used in a substantive manner in the NRC regulatory process are maintained at the NRC Library, 7920 Norfolk Avenue, Bethesda, Maryland, for use by the public. Codes and standards are usually copyrighted and may be purchased from the originating organization or, if they are American National Standards, from the American National Standards Institute, 1430 Broadway, New York, NY 10018.

NUREG/CR-6104
LA-12649-MS

Shear Wall Ultimate Drift Limits

Manuscript Completed: August 1993
Date Published: April 1994

Prepared by
T. A. Duffey*, A. Goldman†, C. R. Farrar

Los Alamos National Laboratory
Los Alamos, NM 87545

Prepared for
Division of Engineering
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001
NRC FIN L2506

*Consulting Engineer, P. O. Box 1239, Tijeras, NM 87059
†Consulting Statistician, 4723 Sandia, Los Alamos, NM 87544

ABSTRACT

Drift limits for reinforced concrete shear walls are investigated by reviewing the open literature for appropriate experimental data. Drift values at ultimate load are determined for walls with aspect ratios ranging up to a maximum of 3.53 and undergoing different types of lateral loading (cyclic static, monotonic static, and dynamic).

Based on the geometry of actual nuclear power plant structures exclusive of containments and concerns regarding their response during seismic (i.e., cyclic) loading, data are obtained from pertinent references for which the wall aspect ratio is less than or equal to approximately 1, and for which testing is cyclic in nature (typically displacement controlled). In particular, lateral deflections at ultimate load, and at points in the softening region beyond ultimate for which the load has dropped to 90, 80, 70, 60, and 50 percent of its ultimate value, are obtained and converted to drift information.

The statistical nature of the data is also investigated. At ultimate load, the median drift is 0.72 percent, and it increases to 1.84 percent when the load drops to 50 percent of its ultimate value. These data are shown to be lognormally distributed, and an analysis of variance is performed. Median drift limit and statistical parameters are in reasonable agreement with those utilized by Kennedy et al. (1988) The use of these statistics to estimate Probability of Failure for a shear wall structure is illustrated. The fragility estimates obtained with the statistics developed in this study are almost identical to those developed by Kennedy et al. (1988).

Finally, a brief comparison of drift limit results with existing seismic design code requirements is presented.

Acknowledgements

The authors are grateful to Drs. Robert P. Kennedy and John W. Reed and Professor Mete Sozen for reviewing this report and for their numerous helpful suggestions. Professor Sozen also supplied additional drift data not yet available in the open literature.

Contents

Page

Tables

1 Introduction

The Shear Wall Ultimate Drift Limit Program is being carried out at Los Alamos National Laboratory (LANL) under sponsorship of the U. S. Nuclear Regulatory Commission (NRC) Office of Nuclear Regulatory Research. For a shear wall, the ultimate drift limit (also referred to in the literature as drift ratio) is defined as the lateral displacement at the top of the wall relative to its base, which corresponds to some definition of structural failure, normalized by the height of the wall. When performing probabilistic risk assessments (PRAs) and seismic margins assessments (SMAs), the ultimate drift limit is necessary to estimate the inelastic seismic capacity of concrete nuclear power plant structures. In many investigations, loss of equipment function for equipment housed within these structures has been considered to occur when the ultimate drift limits are reached; hence, the ultimate drift limit is a failure parameter in these studies. The analysis procedure used in the PRA of the Diablo Canyon turbine building (Kennedy et al. 1988) is the first to use probabilistically defined ultimate drift limits for predicting probabilities of failure of the structure. This same methodology has been adopted by the Electric Power Research Institute's seismic margins assessment methodology.

The objective of this program is to establish the appropriate values of ultimate drift limit and the associated statistics of this parameter for potential use in the PRAs and SMAs to be done in connection with the Individual Plant Examinations of External Events (IPEEE) for Severe Accident Vulnerabilities (NRC, 1991). However, it is noted that many nuclear power plant shear wall structures, particularly those in the Eastern United States, will not require a detailed evaluation such as given in Kennedy et al. (1988), based on screening criteria given in NRC (1985).

Because the results of this work will be used to assess the inelastic seismic capacity of nuclear power plant structures, attention is focused on lightly reinforced (<1%) shear walls with low aspect ratios (less than or equal to approximately 1), though limited drift information is provided for walls with aspect ratios up to 3.53 and vertical reinforcement ratios up to 2.5%.

Discussions with the NRC staff and engineers familiar with seismic PRAs and SMAs have led to the following specific program tasks:

- Establish a definition of ultimate drift limit and provide technical justification for this definition.
- Review existing experimental studies, screen these data to eliminate results from tests where questionable experimental practices were employed, and form a data base of drift limit values.
- Analyze these data and obtain statistics for the ultimate drift limit that will define this parameter in a probabilistic sense.
- Analyze the sensitivity of ultimate drift limit to various parameters such as amount of reinforcement and types of boundary elements.
- Summarize how the ultimate drift limit parameter enters into a risk calculation to show how the drift limits obtained from this investigation would affect the probability of failure obtained in previous PRAs.

Reinforced concrete shear (or "structural") walls possess characteristics of stiffness, strength, and ductility that are favorable for withstanding lateral seismic loads. Shear walls used in nuclear power plant construction are typically stiff, and therefore, tend to prevent the large deformations that can be a problem for attached nonstructural components. However, under sufficient lateral seismic excitation, shear walls can fail by a variety of mechanisms, resulting in significant lateral displacements and loss in stiffness and strength.

Numerous experimental studies on shear walls laterally loaded beyond the elastic range have been reported over the past 40 years.[1] However, most of these studies report only the ultimate (maximum) load capacity of shear walls. Efforts at recording the load-displacement behavior beyond this point have been more limited. Nonetheless, useful energy-absorption capability may exist beyond the point of maximum load resistance; and displacements may still be sufficiently small that the attached nonstructural components do not fail.

[1] Tomii (1968) has previously tabulated drift as a function of shearing force ratio for 200 shear walls, finding that, at ultimate load, average drift is approximately 0.4 percent. However, details such as aspect ratio, type of loading (e.g., cyclic or monotonic) as well as numerous other parameters are not mentioned. Further, data beyond ultimate load are not presented.

In this report, drift values are determined from numerous references at and beyond the point of ultimate lateral load resistance of the shear wall. Results of the initial data screening are presented in Section 2. Results in Section 2 are restricted to drift at ultimate load only and include shear walls with aspect ratios up to 3.53 subjected to various types of lateral loading (cyclic static, monotonic static, and dynamic).

Section 3 contains drift values for a more restricted set of data from tests on shear walls with aspect ratios less than or equal to approximately 1 and for which the loading was cyclic in nature. In Section 3, results are tabulated at the ultimate load point and at increased deformations corresponding to reduced load resistances of 90, 80, 70, 60, and 50 percent of the ultimate. Sufficient suitable data were found on low-aspect shear walls at each of the above load points to interpret the drift limit at each point to be a random variable amenable to statistical analysis, as presented in Section 4.

Application of the drift limit statistics to probability of failure estimates is illustrated in Section 5. Finally, Section 6 presents a brief summary of drift limits specified in existing seismic codes, and Section 7 is a summary of the results.

2 Preliminary Reviews of Shear Wall References

A review format was established for preliminary screening of the literature (See Appendices A and B). Thirty-nine references were given "full" reviews and placed in this format. Twelve references were given "brief" reviews. References given brief reviews were disqualified immediately from further consideration (they contained no drift data). Table 2.1 gives some basic statistics for the drift limit data presented in Table 2.2. The 39 "full" reviews are presented in Appendix A, and the 12 "brief" reviews are presented in Appendix B. A summary of this preliminary screening is presented in Table 2.2. Results of the drift limits listed in Table 2.2 are based on the horizontal deflection at ultimate strength reported for each shear wall. Note that these data are for shear wall aspect ratios, up to a maximum of 3.53, that have been subjected to various types of loading (cyclic static, monotonic static, or dynamic). In the cyclically loaded specimens, it was generally possible to obtain data points in both the first and third loading quadrants of the load-deflection curve.

When possible, ultimate drift limits for both quadrants are included in Table 2.2. A plot of drift limit (at ultimate load) versus aspect ratio is presented in Figure 2.1. Drift limits appear to increase for higher values of aspect ratio.

The 184 data points presented in Table 2.2 and Figure 2.1 correspond to tests covering a wide range of geometrical, material and loading parameters. Aspect ratio range and different types of loading have already been addressed above. Other ranges in potentially important geometrical and material parameters are

- Wall Thickness Range:
  L/t: 8.00 - 53.8
  H/t: 3.94 - 45.0

- Vertical Steel Reinforcement Range:
  0.0% - 2.50%.

- Reinforcement Yield Strength Range:
  41.5 ksi. - 80.0 ksi.

- Concrete Compressive Strength Range:
  1450 psi. - 7790 psi.

- Boundary elements ranged from none, (i.e., a rectangular shear wall only) to end walls whose width exceeds the length of the shear wall.

Histograms showing the distributions of each of the above parameters are presented in Fig. 2.2.

It is difficult to quantify the influence of the above parameters on ultimate drift limit because of the significant variations between experimental programs. Therefore, the approach taken in this report is to consider the ultimate drift limit as a random variable and to develop the associated statistics without regard to material and geometric parameters. As long as the geometric and material parameters for a shear wall under consideration fall within the rather wide ranges listed above, as they will for most nuclear power plant structures, then the drift limit values developed in this report and their associated statistics will be applicable.

Table 2.1 Drift Limit Statistics for Data in Table 2.2

Number of Samples:	**184**
Sample Mean:	**1.043%**
Sample Median:	**0.820%**
Sample Mean Standard Deviation:	**0.748%**
Range:	**0.16% - 4.78%**

Table 2.2 Drift Limits At Ultimate Load - All Aspect Ratios

No.	Author	Aspect Ratio	Specimen	Quadrant	Drift (%)	Loading (C = Cyclic Static) (M = Monotonic) (D = Dynamic)
1.	Wiradinata (1986)	0.50	Wall 1	1	1.04	C
2.	Wiradinata (1986)	0.25	Wall 2	1	0.50	C
3.	Wiradinata (1986)	0.50	Wall 1	3	1.12	C
4.	Wiradinata (1986)	0.25	Wall 2	3	0.66	C
5.	Saatcioglu (1991)	0.50	Wall 4	1	0.84	C
6.	Saatcioglu (1991)	0.50	Wall 4	3	0.80	C
7.	Saatcioglu (1991)	0.50	Wall 6	1	1.50	C
8.	Saatcioglu (1991)	0.50	Wall 6	3	1.63	C
9.	Saatcioglu (1993)*	0.25	Wall 3	1	1.92	C
10.	Saatcioglu (1993)*	0.25	Wall 3	3	2.30	C
11.	Saatcioglu (1993)*	0.50	Wall 5	1	2.25	C
12.	Saatcioglu (1993)*	0.50	Wall 5	3	0.90	C
13.	Shiga (1976)	0.68	WB-3	1	0.70	C
14.	Shiga (1976)	0.68	WB-17	1	0.70	C
15.	Shiga (1976)	0.68	WB-3	3	0.40	C
16.	Shiga (1976)	0.68	WB-17	3	0.52	C
17.	Shiga (1973)	0.68	WB-1	1	0.40	C
18.	Shiga (1973)	0.68	WB-1	3	0.40	C
19.	Shiga (1973)	0.68	WB-2	1	0.41	C
20.	Shiga (1973)	0.68	WB-2	3	0.38	C
21.	Shiga (1973)	0.68	WB-6	1	0.39	C
22.	Shiga (1973)	0.68	WB-6	3	0.40	C
23.	Shiga (1973)	0.68	WB-7	1	0.39	C
24.	Shiga (1973)	0.68	WB-7	3	0.39	C
25.	Shiga (1973)	0.68	WB-8	1	0.40	C
26.	Shiga (1973)	0.68	WB-8	3	0.40	C
27.	Endo (1982)	1.00	W7101	1	1.25	C
28.	Endo (1982)	1.00	W7101	3	0.50	C
29.	Endo (1982)	1.00	W7102	1	0.43	C
30.	Endo (1982)	1.00	W7102	3	1.00	C
31.	Endo (1982)	1.00	W7103	1	0.69	C
32.	Endo (1982)	1.00	W7103	3	0.45	C
33.	Endo (1982)	1.00	W7104	1	0.70	C
34.	Endo (1982)	1.00	W7104	3	0.75	C
35.	Endo (1982)	1.00	W7402	1	0.48	C
36.	Endo (1982)	1.00	W7402	3	0.95	C
37.	Endo (1982)	1.00	W7404	1	0.95	C
38.	Edno (1982)	1.00	W7404	3	0.90	C
39.	Endo (1982)	1.00	W7504	1	0.95	C
40.	Endo (1982)	1.00	W7504	3	0.88	C
41.	Endo (1982)	1.00	W7505	1	0.48	C
42.	Endo (1982)	1.00	W7505	3	0.35	C
43.	Endo (1982)	1.00	W7506	1	0.48	C
44.	Endo (1982)	1.00	W7506	3	0.48	C
45.	Endo (1982)	1.00	W7606	1	0.45	C

See footnotes at end of table

Table 2 .2 (Continued) Drift Limits At Ultimate Load - All Aspect Ratios

No.	Author	Aspect Ratio	Specimen	Quadrant	Drift (%)	Loading (C = Cyclic Static) (M = Monotonic) (D = Dynamic)
46.	Endo (1982)	1.00	W7606	3	0.43	C
47.	Paulay (1982)	0.50	Wall 1	1	0.67	C
48.	Paulay (1982)	0.50	Wall 3	1	0.39	C
49.	Paulay (1982)	0.50	Wall 2	1	0.59	C
50.	Alexander (1973)	0.75	Panel 4	1	0.81	C
51.	Alexander (1973)	0.75	Panel 4	3	0.65	C
52.	Ogata (1984)	0.94	K1	1	1.00	C
53.	Ogata (1984)	0.94	K1	3	1.00	C
54.	Ogata (1984)	0.94	K2	1	1.00	C
55.	Ogata (1984)	0.94	K2	3	1.00	C
56.	Ogata(1984)	0.94	K3	1	1.00	C
57.	Ogata (1984)	0.94	K3	3	1.00	C
58.	Ogata (1984)	0.94	K4	1	1.00	C
59.	Ogata (1984)	0.94	K4	3	1.00	C
60.	Ogata (1984)	0.94	K5	1	1.00	C
61.	Ogata (1984)	0.94	K5	3	1.00	C
62.	Ogata (1984)	0.94	K6	1	1.00	C
63.	Ogata (1984)	0.94	K6	3	1.00	C
64.	Barda (1972)	0.51	B3-2	1	0.56	C
65.	Barda (1972)	0.51	B3-2	3	0.75	C
66.	Barda (1972)	0.24	B7-5	1	0.85	C
67.	Barda (1972)	0.24	B7-5	3	1.68	C
68.	Barda (1972)	1.07	B8-5	1	0.72	C
69.	Barda (1972)	1.07	B8-5	3	0.40	C
70.	Barda (1972)	0.51	B1-1	1	0.61	M
71.	Barda (1972)	0.51	B2-1	1	0.69	M
72.	Barda (1972)	0.51	B4-3	NA**	0.53	C
73.	Barda (1972)	0.51	B5-4	NA**	0.53	C
74.	Barda (1972)	0.51	B6-4	NA**	0.61	C
75.	Benjamin (1954)	0.58	R-1	1	0.60	M
76.	Benjamin (1954)	0.58	R-2	1	0.49	M
77.	Benjamin (1954)	0.58	R-3	1	0.50	M
78.	Benjamin (1954)	0.58	R-4	1	0.53	M
79.	Benjamin (1954)	0.58	R-5	1	0.71	M
80.	Benjamin (1954)	0.58	1bII-2b	1	0.40	M
81.	Benjamin (1954)	0.32	4bI-4	1	0.28	M
82.	Benjamin (1954)	0.32	4bII-4	1	0.58	M
83.	Benjamin (1954)	0.32	A1-A	1	0.31	M
84.	Benjamin (1954)	0.32	A1-B	1	0.39	M
85.	Benjamin (1954)	0.32	A2-B	1	0.42	M
86.	Benjamin (1955)	0.58	VRR-1	1	0.30	M
87.	Benjamin (1955)	0.58	VRR-2	1	1.03	M
88.	Benjamin (1955)	0.58	VRR-3	1	0.51	M

See footnotes at end of table

Table 2 .2 (Continued) Drift Limits At Ultimate Load - All Aspect Ratios

No.	Author	Aspect Ratio	Specimen	Quadrant	Drift (%)	Loading (C = Cyclic Static) (M = Monotonic) (D = Dynamic)
89.	Benjamin (1955)	0.58	VRR-6	1	0.53	M
90.	Benjamin (1955)	0.58	VRR-7	1	0.99	M
91.	Benjamin (1955)	0.50	NV-1	1	0.43	M
92.	Benjamin (1955)	0.50	NV-2	1	0.61	M
93.	Benjamin (1955)	0.50	NV-4	1	0.75	M
94.	Williams (1952)	0.71	C-1	1	0.78	M
95.	Williams (1952)	0.71	C-3	1	0.16	M
96.	Williams (1952)	0.71	C-5	1	0.64	M
97.	Williams (1953)	0.69	2A-3	1	1.51***	M
98.	Williams (1953)	0.69	2A-4	1	0.73***	M
99.	Williams (1953)	0.71	4BI-2	1	0.62	M
100.	Williams (1953)	0.50	4BI-3	1	0.53	M
101.	Williams (1953)	0.32	4BI-4	1	0.28	M
102.	Williams (1953)	1.25	4BII-1	1	0.44	M
103.	Williams (1953)	0.50	4BII-3	1	0.51	M
104.	Williams (1953)	0.32	4BII-4	1	0.59	M
105.	Williams (1953)	1.25	4BI-1	1	1.30	M
106.	Williams (1953)	0.71	3A2-3	1	0.95	M
107.	Williams (1953)	0.58	3BI-1	1	0.28	M
108.	Williams (1953)	0.58	1BII-2a	1	0.46	M
109.	Williams (1953)	0.58	1BII-2b	1	0.41	M
110.	Williams (1953)	0.58	3BI-3	1	0.45	M
111.	Cervenka (1971)	1.00	W2-1	1	0.93	M
112.	Cervenka (1971)	1.00	W2-2	1	0.93	M
113.	Cervenka (1971)	1.00	W3-2	1	0.73	M
114.	Corley (1981)	3.53	B4	1	4.78	M
115.	Corley (1981)	3.53	B3	1	3.94	C
116.	Corley (1981)	3.53	B3	3	3.33	C
117.	Corley (1981)	3.53	B7	1	2.78	C
118.	Corley (1981)	3.53	B7	3	2.67	C
119.	Corley (1981)	3.53	B9	1	2.97	C
120.	Corley (1981)	3.53	B9	3	2.89	C
121.	Corley (1981)	2.69	F2	1	2.17	C
122.	Corley (1981)	2.69	F2	3	2.22	C
123.	Corley (1981)	3.53	B8	1	2.72	C
124.	Corley (1981)	3.53	B8	3	2.72	C
125.	Corley (1981)	3.53	B1	1	2.19	C
126.	Corley (1981)	3.53	B1	3	2.17	C
127.	Corley (1981)	3.53	B2	1	2.17	C
128.	Corley (1981)	3.53	B2	3	2.17	C
129.	Corley (1981)	3.53	B5	1	2.75	C
130.	Corley (1981)	3.53	B5	3	2.72	C
131.	Corley (1981)	3.53	B6	1	1.61	C

See footnotes at end of table.

Table 2.2 (Continued) Drift Limits At Ultimate Load - All Aspect Ratios

No.	Author	Aspect Ratio	Specimen	Quadrant	Drift (%)	Loading (C = Cyclic Static) (M = Monotonic) (D = Dynamic)
132.	Corley (1981)	3.53	B6	3	1.64	C
133.	Elnashai (1990)	2.00	SW2	1	1.13	C
134.	Elnashai (1990)	2.00	SW2	3	1.17	C
135.	Elnashai (1990)	2.00	SW3	1	0.83	C
136.	Elnashai (1990)	2.00	SW3	3	0.83	C
137.	Elnashai (1990)	2.00	SW4	1	1.00	C
138.	Elnashai (1990)	2.00	SW4	3	0.79	C
139.	Elnashai (1990)	2.00	SW5	1	0.83	C
140.	Elnashai (1990)	2.00	SW5	3	0.67	C
141.	Elnashai (1990)	2.00	SW6	1	1.48	C
142.	Elnashai (1990)	2.00	SW6	3	1.29	C
143.	Elnashai (1990)	2.00	SW7	1	0.98	C
144.	Elnashai (1990)	2.00	SW7	3	1.48	C
145.	Elnashai (1990)	2.00	SW8	1	1.83	C
146.	Elnashai (1990)	2.00	SW8	3	1.83	C
147.	Elnashai (1990)	2.00	SW9	1	1.97	C
148.	Elnashai (1990)	2.00	SW9	3	1.78	C
149.	Fiorato (1976)	2.69	F1	1	2.17	C
150.	Fiorato (1976)	2.69	F1	3	1.61	C
151.	Fiorato (1976)	2.40	R1	1	2.17	C
152.	Fiorato (1976)	2.40	R1	3	2.11	C
153.	Lefas (1990)	1.00	SW11	1	1.07	M
154.	Lefas (1990)	1.00	SW12	1	1.18	M
155.	Lefas (1990)	1.00	SW13	1	1.16	M
156.	Lefas (1990)	1.00	SW14	1	1.47	M
157.	Lefas (1990)	1.00	SW15	1	1.05	M
158.	Lefas (1990)	1.00	SW16	1	0.78	M
159.	Lefas (1990)	1.00	SW17	1	1.44	M
160.	Lefas (1990)	2.00	SW21	1	1.59	M
161.	Lefas (1990)	2.00	SW22	1	1.18	M
162.	Lefas (1990)	2.00	SW23	1	1.02	M
163.	Lefas (1990)	2.00	SW24	1	1.41	M
164.	Lefas (1990)	2.00	SW25	1	0.72	M
165.	Lefas (1990)	2.00	SW26	1	1.62	M
166.	Yamada (1974)	0.44	PW0	1	0.62	M
167.	Yamada (1974)	0.44	PW3	1	0.40	M
168.	Yamada (1974)	0.44	PW6	1	0.53	M
169.	Yamada (1974)	0.44	PW12	1	0 49	M
170.	Yamada (1974)	0.44	PW8T30	1	0.40	M
171.	Yamada (1974)	0.44	PW6T20	1	0.32	M
172.	Yamada (1974)	0.44	PW12T20	1	0.49	M

See footnotes at end of table.

Table 2.2 (Continued) Drift Limits At Ultimate Load - All Aspect Ratios

No.	Author	Aspect Ratio	Specimen	Quadrant	Drift (%)	Loading (C = Cyclic Static) (M = Monotonic) (D = Dynamic)
173.	Maier (1985)+	1.22	S1	1	1.88	M++
174.	Maier (1985)+	1.22	S2	1	0.89	M++
175.	Maier (1985)+	1.22	S3	1	1.25	M++
176.	Maier (1985)+	1.22	S4	1	0.98	M++
177.	Maier (1985)+	1.22	S6	1	1.59	M++
178.	Maier (1985)+	1.22	S7	1	0.69	C
179.	Maier (1985)+	1.22	S7	3	0.54	C
180.	Maier (1985)+	1.22	S8	1	0.80	M++
181.	Maier (1985)+	1.22	S8	3	1.67	M++
182.	Maier (1985)+	1.22	S9	1	0.78	M++
183.	Maier (1985)+	1.22	S10	1	1.22	M++
184.	Rothe (1989)+	1.38	T01	3	0.29	D

*These results were transmitted by personal communication from M. Saatcioglu, University of Ottawa, January, 1993.

**NA = Information not available.

***Author states results should be considered as qualitative.

+From Sozen (1991).

++Some reloading, but not cyclic.

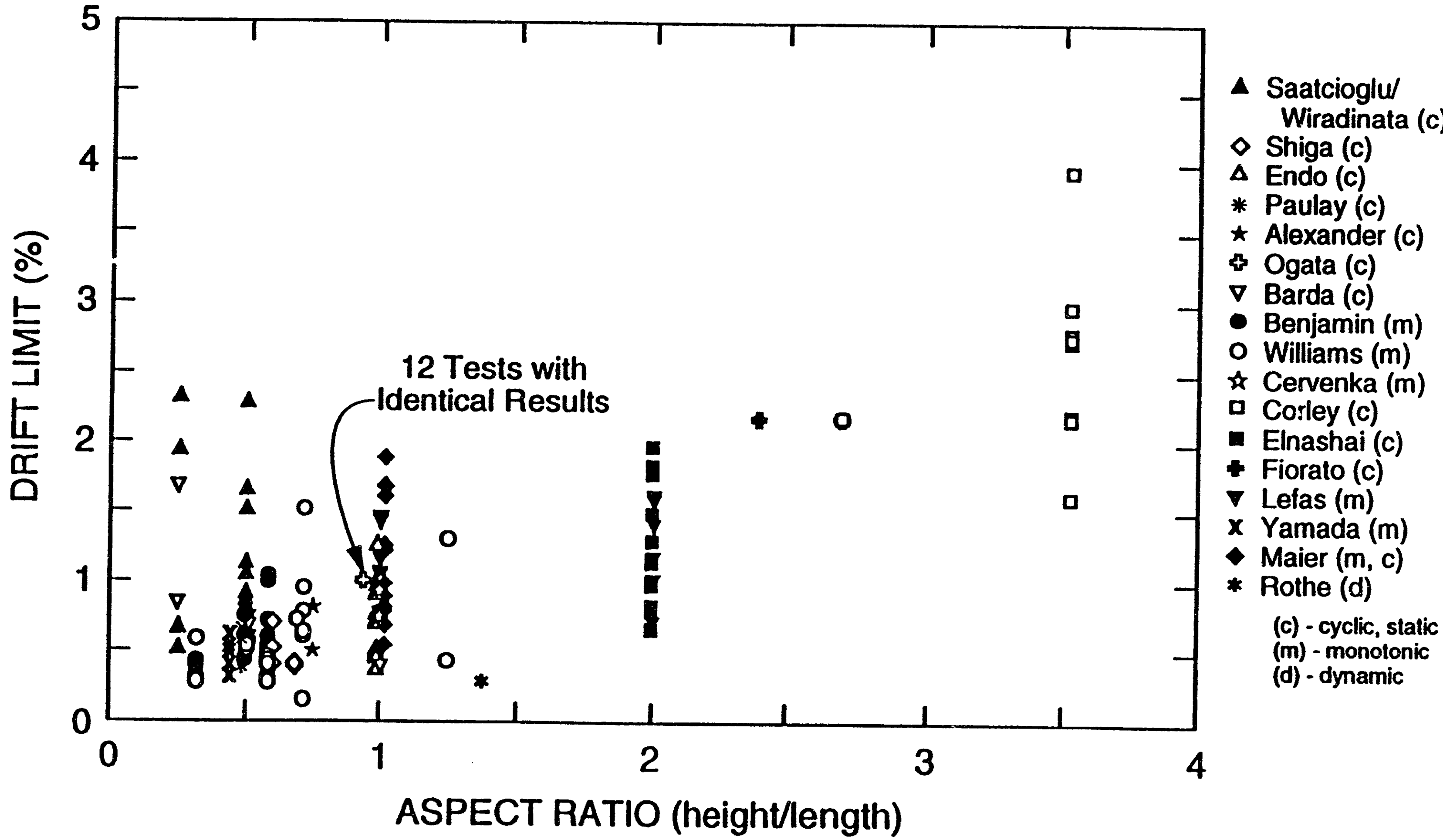


Figure 2.1 Drift Limit at Ultimate Load as a Function of Aspect Ratio from Table 2.2

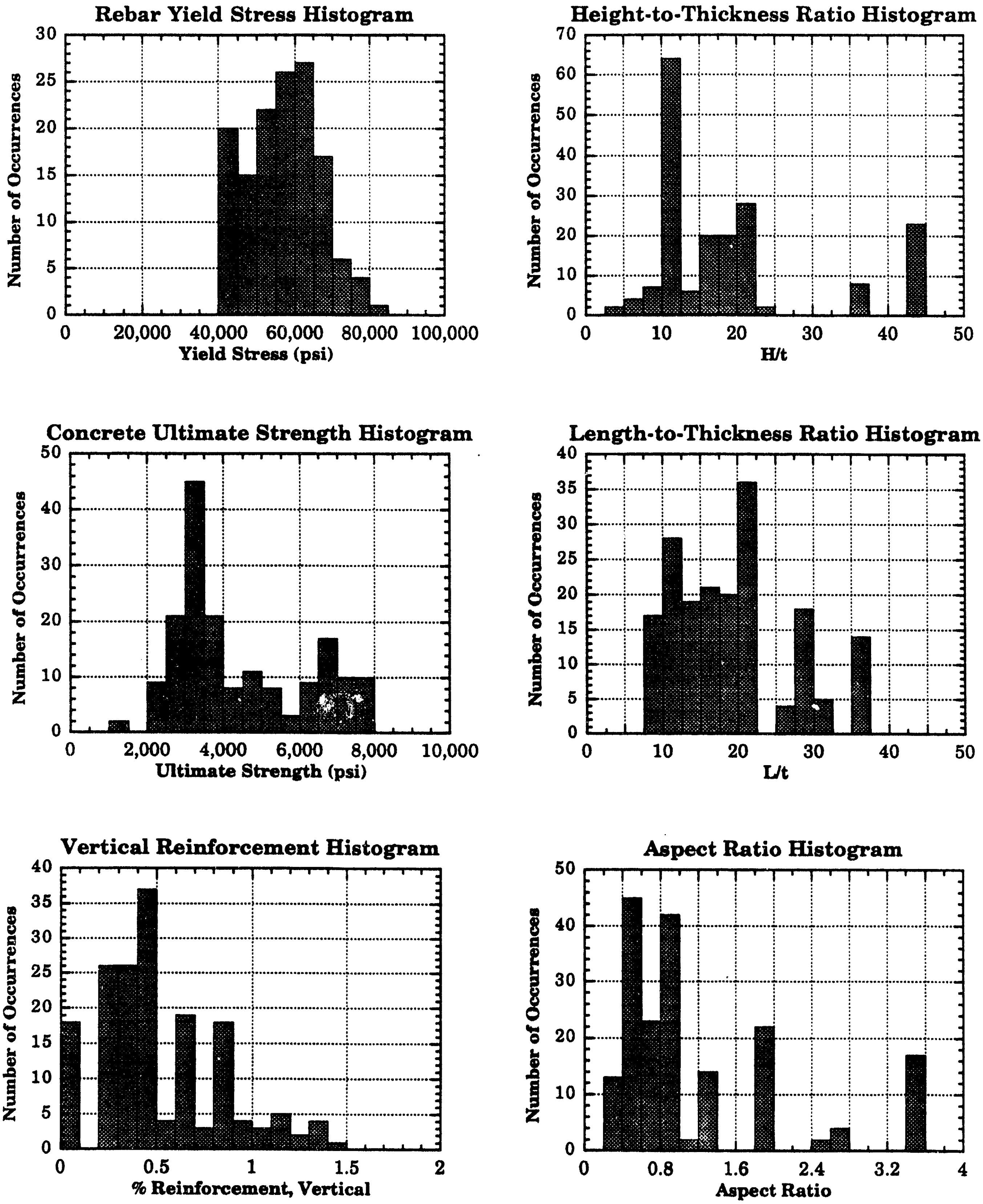


Figure 2.2 Histograms Showing the Distributions of Geometrical and Material Parameters

3 Drift Limit Data Summary

Following the initial screening described in Section 2, those pertinent papers containing drift data at and beyond ultimate load were further studied to extract drift limits at 100, 90, 80, 70, 60, and 50 percent of ultimate load. "Pertinent" papers are taken as those containing data with shear wall aspect ratios of approximately 1 or less undergoing cyclic loading. Loading was displacement controlled in most instances.

So that drift data are obtained from the various references in a consistent manner, a data reduction procedure, which results in a lower-bound estimate for drift at ultimate load, was developed based on a few simple rules. Consider Figure 3.1a, a representative lateral load-deflection curve based on Wall 1 (first quadrant) from Paulay (1982). Note that these loading paths are displacement-controlled. The shear wall is loaded up to a certain controlled horizontal displacement and subjected to a second cycle up to that same displacement. Then the wall is cyclically loaded to a higher controlled horizontal displacement.

Some observations, which typify many of the references examined, are as follows:

1. Loading peaks initially increase in load (a, b, c), up to some peak (ultimate strength) value, followed by a decrease at larger deformation (d, e).

2. Subsequent displacement-controlled cycling at a given lateral displacement generally are at lower loads, (e.g., points f, g).

For this type of load-displacement curve, where the load monotonically increases to a sharp peak until the load is reversed and deflection decreases, the following procedures and assumptions are made:

1. The ultimate load is assumed to occur at the load-deflection point at which the load is at a maximum (pt. c, Figure 3.1a).

2. Straight lines (line c-d-e, Figure 3.1b) are used to connect subsequent loading peaks, as shown in Figure 3.1b. The load-deflection locus is therefore fully defined beyond ultimate load.

3. Only initial loading peaks (i.e., c, d, e) are considered. Subsequent loading peaks at a given displacement when present, (i.e., f, g in Figure 3.1a) are ignored.

4. The load-deflection locus is not extrapolated beyond the last loading peak (i.e., pt. e, Figure 3.1a).

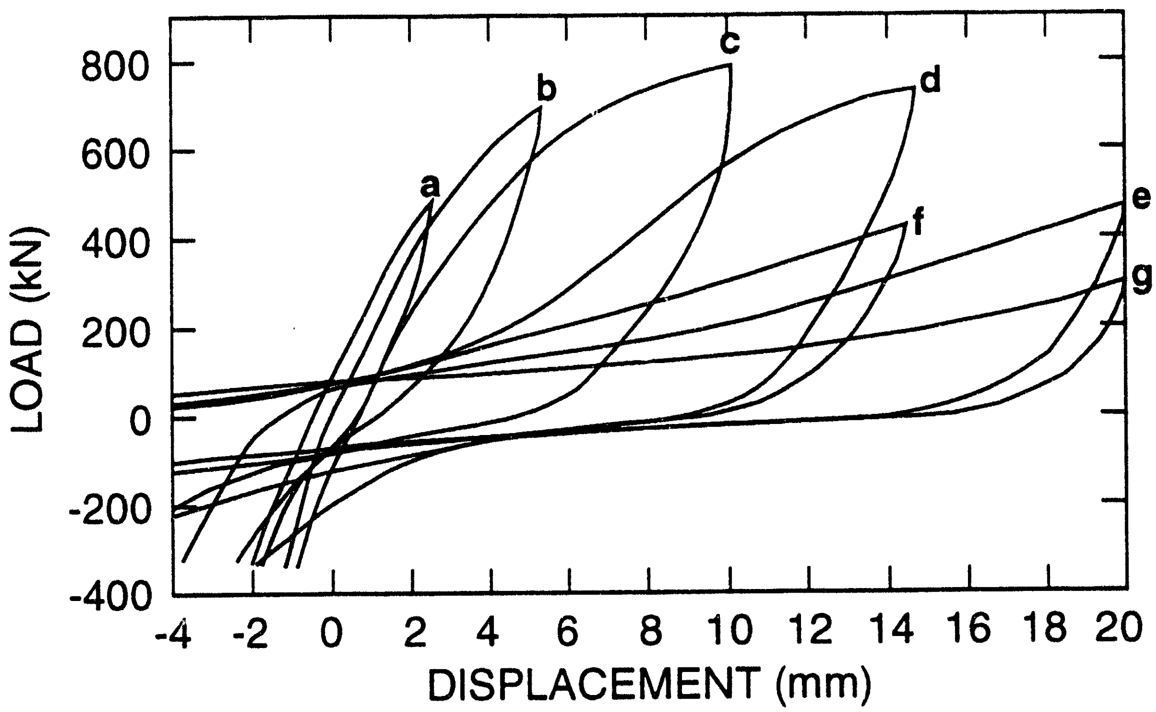
5. Based on the ultimate load value (pt. c, Figure 3.1b) horizontal lines are drawn (not shown) across the curve to determine the displacement at values beyond ultimate) of 90, 80, 70, 60, and 50 percent of ultimate load. Not all values are always available because of (4.) above. As an example, in Figure 3.1b, the displacement at ultimate load is 10 mm; displacement at 80 percent of ultimate load is 16 mm; and displacement at 50 percent cannot be determined for this example.

6. These displacement values are then placed in the form of percent drift. The process is repeated for the third quadrant of the same wall and results are tabulated.

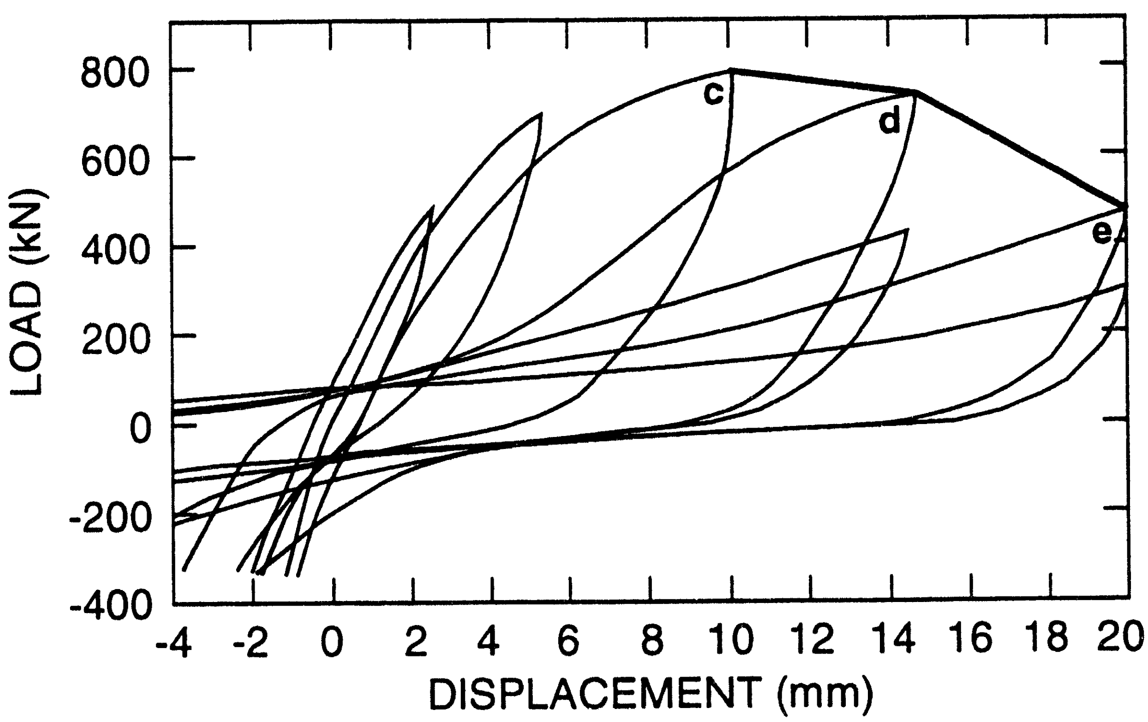
In some cases, the load cycles do not end in distinct, sharp peaks. The situation is shown in Figure 3.2, based upon data from Saatcioglu (1993). In this case, a straight line is connected, as before, between the two peaks in the first quadrant (dashed line). However, following pt. d, the load decreases monotonically on increasing displacement until reaching pt. e. Beyond pt. e, unloading is assumed to occur as both the load and the displacement decrease. The load-deflection locus is therefore taken as the curved path c-d-e. Again, Steps 5 and 6 are then performed to determine the drift value beyond ultimate load. The rule here is that the load-deflection locus is terminated when the slope of the curve becomes less than vertical, indicating unloading.

One additional situation was found to occur in practice, as depicted in the third quadrant of Figure 3.2. In that case, a distinct peak is present on one load cycle (pt. f). On the subsequent load cycle, the peak occurs at a lower displacement (pt. g). In this situation, a vertical line is drawn as shown between pts. f and h, as it would not be valid to connect peaks according to procedure 2 discussed above.

The above process was repeated for each load-deflection curve in all of the "pertinent" papers described at the beginning of this section. Sixty-nine pertinent data sets were obtained, and each set consisted of up to six data points, one point for each of the percentages (100, 90, etc.) of ultimate load. Results for each data set are presented in Table 3.1. The data sets are obtained graphically from ten different references. Note that, in general, each tested



a. Identification of Initial and Subsequent Loading Peaks



b. Straight Lines Added to Define Load-Deflection Locus

Figure 3.1 Representative Lateral Load-Deflection Curve (From Paulay (1982), Wall 1)

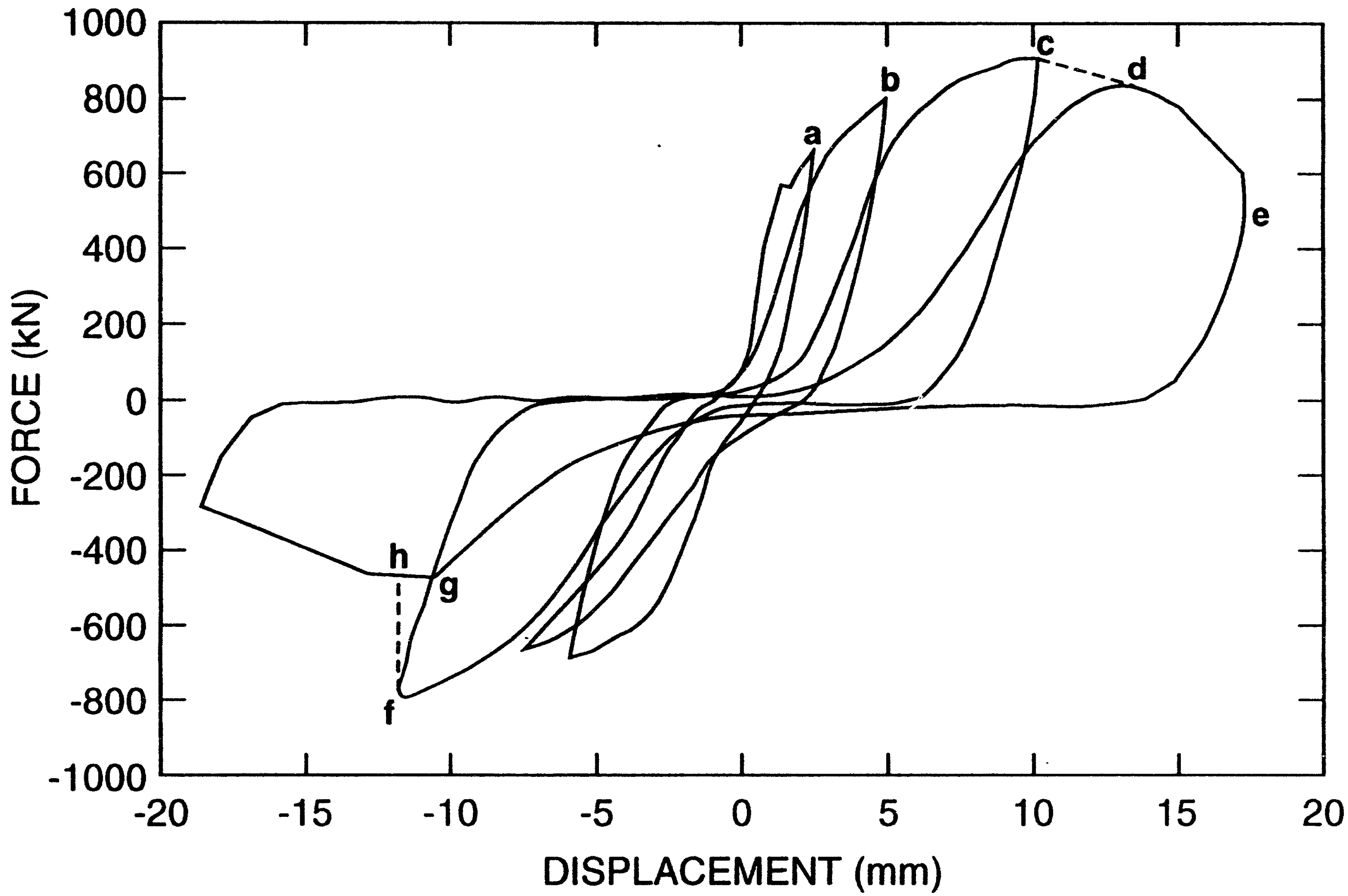


Figure 3.2 Modified Load-Deflecton Curve From Saatcioglu (1993)

Table 3.1 Drift Limits at and Beyond Ultimate Load (In Percent)

					Fraction Of Ultimate Load					
No.	**Author**	**Year**	**Aspect Ratio**	**Quadrant**	**100%**	**90%**	**80%**	**70%**	**60%**	**50%**
1.	Wiradinata	1986	0.5 (Wall 1)	1	1.04	1.29	1.53	1.72	1.89	2.00
2.	Wiradinata	1986	0.25 (Wall 2)	1	0.50	1.20	2.86	3.40	-	-
3.	Wiradinata	1986	0.5 (Wall 1)	3	1.12	2.10	2.10	2.10	2.11	-
4.	Wiradinata	1986	0.25 (Wall 2)	3	0.66	0.96	1.20	2.24	4.00	-
5.	Saatcioglu	1991	0.5 (Wall 4)	1	0.84	2.00	2.45	-	-	-
6.	Saatcioglu	1991	0.5 (Wall 4)	3	0.80	-	-	-	-	-
7.	Saatcioglu	1991	0.5 (Wall 6)	1	1.50	1.75	2.00	2.10	2.18	2.25
8.	Saatcioglu	1991	0.5 (Wall 6)	3	1.63	1.63	1.63	1.63	1.63	2.45
9.	Saatcioglu	1993	0.25 (Wall 3)	1	1.92	2.74	3.08	3.34	3.40	-
10.	Saatcioglu	1993	0.25 (Wall 3)	3	2.30	2.34	2.34	2.34	2.34	3.00
11.	Saatcioglu	1993	0.50 (Wall 5)	1	2.25	2.75	3.20	3.65	4.20	4.50
12.	Saatcioglu	1993	0.50 (Wall 5)	3	0.90	2.30	2.60	3.00	3.40	3.80
13.	Shiga	1976	0.68 (WB-3)	1	0.70	0.90	0.95	1.04	-	-
14.	Shiga	1976	0.68 (WB-17)	1	0.70	0.89	0.93	1.03	1.10	-
15.	Shiga	1976	0.68 (WB-3)	3	0.40	0.60	0.80	1.00	-	-
16.	Shiga	1976	0.68 (WB-17)	3	0.52	-	-	-	-	-
17.	Shiga	1973	0.68 (WB-1)	1	0.40	0.73	0.86	1.04	-	-
18.	Shiga	1973	0.68 (WB-1)	3	0.40	0.51	0.60	0.69	0.74	1.00
19.	Shiga	1973	0.68 (WB-2)	1	0.41	0.78	0.89	1.20	-	-
20.	Shiga	1973	0.68 (WB-2)	3	0.38	0.80	0.88	0.95	1.02	1.09
21.	Shiga	1973	0.68 (WB-6)	1	0.39	0.71	0.93	1.00	1.01	1.01
22.	Shiga	1973	0.68 (WB-6)	3	0.40	0.94	1.07	-	-	-
23.	Shiga	1973	0.68 (WB-7)	1	0.39	0.82	1.00	-	-	-
24.	Shiga	1973	0.68 (WB-7)	3	0.39	0.55	0.68	0.80	0.92	1.03
25.	Shiga	1973	0.68 (WB-8)	1	0.40	0.80	0.87	0.95	1.00	1.07
26.	Shiga	1973	0.68 (WB-8)	3	0.40	0.49	0.58	0.68	0.77	0.85
27.	Endo	1982	1.0 (W7101)	1	1.25	1.33	1.45	2.00	-	-
28.	Endo	1982	1.0 (W7101)	3	0.50	-	-	-	-	-
29.	Endo	1982	1.0 (W7102)	1	0.43	-	-	-	-	-
30.	Endo	1982	1.0 (W7102)	3	1.00	1.20	1.30	1.43	-	-
31.	Endo	1982	1.0 (W7103)	1	0.69	-	-	-	-	-
32.	Endo	1982	1.0 (W7103)	3	0.45	0.58	1.08	-	-	-
33.	Endo	1982	1.0 (W7104)	1	0.70	-	-	-	-	-
34.	Endo	1982	1.0 (W7104)	3	0.75	-	-	-	-	-
35.	Endo	1982	1.0 (W7402)	1	0.48	1.00	1.12	-	-	-
36.	Endo	1982	1.0 (W7402)	3	0.95	-	-	-	-	-
37.	Endo	1982	1.0 (W7404)	1	0.95	1.20	2.50	2.85	3.43	-
38.	Edno	1982	1.0 (W7404)	3	0.90	-	-	-	-	-
39.	Endo	1982	1.0 (W7504)	1	0.95	1.15	1.38	1.60	1.75	2.05
40.	Endo	1982	1.0 (W7504)	3	0.88	1.10	1.35	1.60	1.75	-
41.	Endo	1982	1.0 (W7505)	1	0.48	0.63	0.68	0.78	0.85	0.90
42.	Endo	1982	1.0 (W7505)	3	0.35	0.58	0.68	0.83	1.40	1.88
43.	Endo	1982	1.0 (W7506)	1	0.48	0.58	0.75	0.88	0.93	-
44.	Endo	1982	1.0 (W7506)	3	0.48	0.55	0.63	0.73	0.83	0.90
45.	Endo	1982	1.0 (W7606)	1	0.45	0.93	1.08	1.12	1.13	1.14

TABLE 3.1 (Continued) Drift Limits at and Beyond Ultimate Load (In Percent)

No.	Author	Year	Aspect Ratio	Quadrant	Fraction Of Ultimate Load					
					100%	90%	80%	70%	60%	50%
46.	Endo	1982	1.0 (W7606)	3	0.43	0.80	0.85	0.88	0.90	-
47.	Paulay	1982	0.5 (Wall 1)	1	0.67	0.99	1.11	1.21	1.32	-
48.	Paulay	1982	0.5 (Wall 3)	1	0.39	0.58	0.75	-	-	-
49.	Paulay	1982	0.5 (Wall 2)	1	0.59	1.04	1.27	-	-	-
50.	Alexander	1973	0.75 (Panel 4)	1	0.81	1.00	-	-	-	-
51.	Alexander	1973	0.75 (Panel 4)	3	0.65	0.96	1.04	1.09	-	-
52.	Ogata	1984	0.94 (K1)	1	1.00	1.18	1.37	1.55	1.73	1.92
53.	Ogata	1984	0.94 (K1)	3	1.00	1.22	1.43	1.65	1.87	-
54.	Ogata	1984	0.94 (K2)	1	1.00	1.22	1.44	1.66	1.88	-
55.	Ogata	1984	0.94 (K2)	3	1.00	1.18	1.37	1.55	1.74	1.92
56.	Ogata	1984	0.94 (K3)	1	1.00	1.16	1.32	1.48	1.64	1.80
57.	Ogata	1984	0.94 (K3)	3	1.00	1.16	1.32	1.48	1.64	1.79
58.	Ogata	1984	0.94 (K4)	1	1.00	1.18	1.36	1.55	1.73	1.91
59.	Ogata	1984	0.94 (K4)	3	1.00	1.19	1.39	1.58	1.77	1.96
60.	Ogata	1984	0.94 (K5)	1	1.00	1.18	1.36	1.55	1.73	1.91
61.	Ogata	1984	0.94 (K5)	3	1.00	1.26	1.52	1.78	-	-
62.	Ogata	1984	0.94 (K6)	1	1.00	1.24	1.48	1.73	1.97	-
63.	Ogata	1984	0.94 (K6)	3	1.00	-	-	-	-	-
64.	Barda	1972	0.51 (B3-2)	1	0.56	0.61	0.67	0.74	0.89	-
65.	Barda	1972	0.51 (B3-2)	3	0.75	0.85	0.92	0.99	1.10	-
66.	Barda	1972	0.24 (B7-5)	1	0.85	1.12	1.65	-	-	-
67.	Barda	1972	0.24 (B7-5)	3	1.68	1.87	2.24	-	-	-
68.	Barda	1972	1.07 (B8-5)	1	0.72	0.83	0.93	1.05	1.15	1.28
69.	Barda	1972	1.07 (B8-5)	3	0.40	0.75	0.99	1.31	1.56	1.73

shear wall provided two data sets from a single cyclic load-deflection curve--one set each for first and third loading quadrants.

The ranges in geometrical and material parameters for the shear walls included in Table 3.1 are shown below. They are considerably smaller than those for the data (all aspect ratios) reported in Table 2.2.

- Aspect Ratio Range:
  0.24 - 1.07.

- Wall Thickness Range:
  L/t: 16.8 - 35.0
  H/t: 3.94 - 35.0

- Vertical Steel Reinforcement Range:
  0.0% - 0.86%.

- Reinforcement Yield Strength Range:
  41.8 Ksi - 79.0 Ksi.

- Concrete Compressive Strength Range:
  1450 psi. - 5075 psi.

- Boundary elements ranged from none, i.e., a rectangular shear wall only, to substantial end walls (e.g., bar bell cross section).

Histograms showing the distributions of the above geometrical and material parameters are presented in Fig. 3.3.

The reader is cautioned that care must be exercised in using drift values beyond ultimate load (i.e., in the softening region), because relatively little resistive energy may remain in the structure, although the energy remaining in the earthquake input may still be significant. It should also be mentioned that the limiting factor for lateral drift of shear walls may be damage to attached nonstructural components, such as piping. The drift limit that such nonstructural components are capable of withstanding is design specific, and it appears unlikely that a meaningful general limit could be deduced from consideration of these components themselves. The drift limit at 100 percent of ultimate load therefore appears to be the most appropriate definition of ultimate drift limit when used in conjunction with hysteretic models for nonlinear time-history analysis.

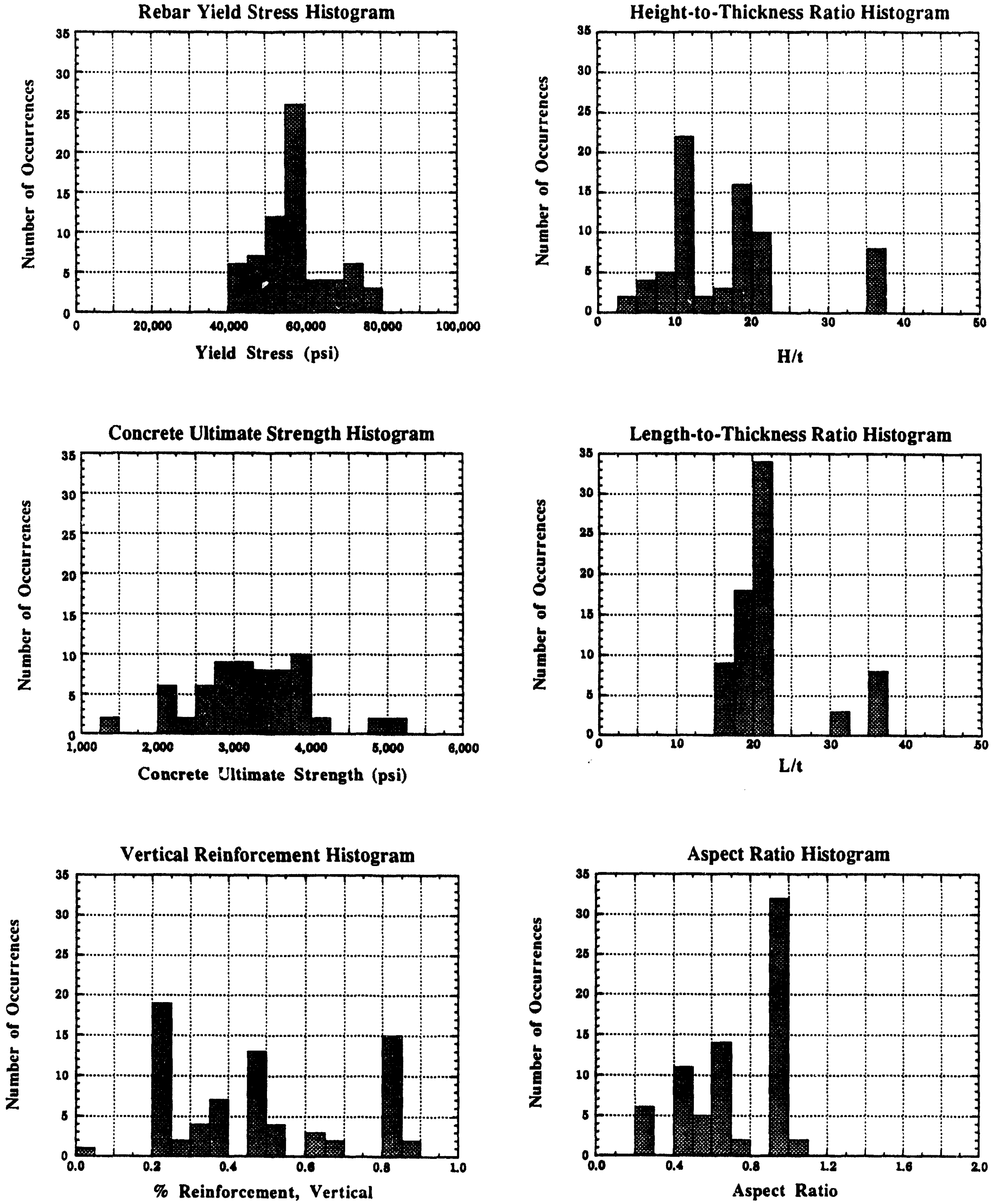


Figure 3.3 Histograms Showing the Distributions of Geometrical and Material Parameters for Low-Aspect-Ratio Shear Walls

4 Statistical Analysis of the Data

Basic statistical results obtained for the data in Table 3.1 are summarized in Table 4.1, where, for instance, the sample median drift limit at ultimate load ("100 percent") is 0.72 percent, increasing beyond ultimate load to 1.84 percent at the point where the load has dropped to 50 percent of ultimate load. Results from Table 4.1 are plotted in Figure 4.1, where the sample mean, median, and range increase monotonically as the lateral load decreases beyond its ultimate value.

4.1 Drift Limit Statistics By Experimenter

More detailed statistical analyses were then performed using the SAS software package (SAS User's Guide, 1985). The first study compares the drift limits obtained by each experimenter for the case of drift at 100 percent of ultimate load. Results presented in Table 4.2 include the mean drift, coefficient of variation, and the approximate lower 95 percent confidence limit for each of the experimenters listed in Table 3.1, as well as the corresponding statistics for the three sets of experiments given in Kennedy et al. (1988)

In Table 4.2, the Coefficient of Variation is the standard deviation divided by the mean, (i.e., a relative standard deviation useful for comparing distributions when there are differences among means). The Lower 95 percent Confidence Limit (one tail) is that value of drift for which one is 95 percent certain that the actual mean is greater.

Note that in constructing Table 4.2, all results of a given experimenter, as taken from Table 3.1, have been combined (e.g., the Shiga data are a combination of data from both his 1973 and 1976 papers. Further, the experiments reported by Wiradinata (1986) and Saatcioglu (1991 and 1993) are related and were therefore combined as if performed by a single experimenter. Finally, as noted on Table 4.2, the Ogata (1984) drift limit at ultimate load was 1.00 percent for all tests (see Table 3.1) because of the displacement-controlled nature of his test procedure. Inspection of Table 4.2 reveals the following observations:

1. With the exception of Ogata (1984), and Wiradinata and Saaticoglu(1986) and Saaticoglu (1991, 1993), the mean and 95 percent confidence limit values are in reasonable agreement.

2. Barda (from Kennedy et al. [1988]), LANL (from Kennedy et al. [1988]), and Alexander (1973) give low coefficients of variation.

3. Notwithstanding the above observations, the drift limit data utilized by Kennedy et al. (1988) and those shown in Table 3.1 are reasonably consistent, based on the measures used in Table 4.2.

4.2 Combined Drift Limit Statistics

The results of an analysis of variance for the data in Table 3.1 are compared with results reported by Kennedy et al. (1988) in Table 4.3. The standard deviations for the data from Table 3.1 are calculated based on a logarithmic transformation of the data, for consistency with Kennedy et al. (1988). Note that direct comparisons of the first two rows is appropriate. However, for completeness, statistics for other fractions of ultimate load are also shown in Table 4.3. Medians are also presented in Table 4.3, because the median is supplied by Kennedy and because the median is a better measure of central tendency for the lognormal distribution than the mean.

In Table 4.3, random standard deviations are a weighted average of those within a given investigation (i.e., "Experimenter," Table 4.2). Weights are assigned based on the number of experiments performed using a procedure described by Graybill (1961). They are a measure of the average variation of the drift limit about the mean of the data for a particular experimenter. Systematic standard deviations are a measure of the average variance of means between experimenters and are assumed to correspond to "uncertainty" reported by Kennedy et al. (1988). When calculating the systematic standard deviation the weighting scheme described by Graybill (1961) was again used. Systematic variation is attributed to bias caused by the inability of experimenters to establish their experimental program and measurement procedures in precisely the same way. It is realized that the random standard deviation as defined above will have a systematic component. However, the statistical models used make the idealization that within a given investigator's set of experiments no systematic error occurs. The systematic error within a given investigation is believed small in comparison to the systematic error between investigators. Composite standard deviations are obtained from the square root of the sum of the squares of random and systematic standard deviations.

Also shown in Table 4.3 are upper and lower 95% confidence limits on the composite log standard deviations. The limits were obtained using theorem 17.1 in Graybill (1961) and using Welch (1956). The limits were computed

Table 4.1 Drift Limit Data Statistics

	100 Pct.	90 Pct.	80 Pct.	70 Pct.	60 Pct.	50 Pct.
Number of Samples	69	59	58	49	40	26
Sample Mean	0.802	1.118	1.342	1.521	1.710	1.813
Sample Median	0.72	1.00	1.24	1.48	1.64	1.84
Sample Standard Deviation	0.422	0.521	0.631	0.723	0.868	0.870
Range	0.35-2.30	0.49-2.75	0.58-3.2	0.68-3.65	0.74-4.20	0.85-4.50

Table 4.2 Drift Limit Statistics By Experimenter For Drift At Ultimate Load

Source	Experimenter	Mean Drift (%)	Coefficient of Variation	95% Confidence Limit (%)
Table 3.1	Shiga (1973, 1975)	0.45	0.25	0.40
Table 3.1	Endo (1982)	0.68	0.40	0.58
Table 3.1	Paulay (1982)	0.55	0.26	0.41
Table 3.1	Alexander (1973)	0.73	0.15	0.60
Table 3.1	Ogata (1984)	1.00	*	*
Table 3.1	Wiradinata (1986) Saatcioglu (1991, 1993)	1.29	0.48	0.99
Table 3.1	Barda (1972)	0.83	0.54	0.53
Kennedy et al. (1988)	Barda (from Kennedy et al. [1988])	0.62	0.16	0.48
Kennedy et al. (1988)	Shiga (from Kennedy et al. [1988])	0.55	0.31	0.33
Kennedy et al. 1988)	LANL (from Kennedy et al. [1988])	0.54	0.11	0.45

*Always reported as 1.00% drift.

Table 4.3 Comparisons Of Median and Associated Logarithmic Standard Deviation: Kennedy et al. (1988) Vs Table 3.1

Source	Fraction of Ultimate Load (%)	Median (%)	Random Log Std. Dev.	Systematic Log Std. Dev.	Composite Log Std. Dev.	Uncertainty in Composite Log Standard Deviation Lower 95%	Uncertainty in Composite Log Standard Deviation Upper 95%
Kennedy (1988)	100	0.70	0.15	0.30	0.335	--	--
Table 3.1	100	0.72	0.35	0.13	0.373	0.330	1.04
Table 3.1	90	1.00	0.28	0.33	0..437	0.286	0.919
Table 3.1	80	1.24	0.29	0.35	0.452	0.297	0.949
Table 3.1	70	1.48	0.28	0.37	0.464	0.304	0.986
Table 3.1	60	1.64	0.30	0.43	0.524	0.370	1.34
Table 3.1	50	1.84	0.24	0.51	0.566	0.332	1.59

as a function of the random and systematic standard deviations as well as tabulated values of the Chi-Square distributions with degrees of freedom given as a function of the number of experimenters and the number of data points.

Observations from Table 4.3 are as follows:

1. The median and the composite logarithmic standard deviations for drift at ultimate load for Kennedy et al. (1988) and the present study (Table 3.1) are nearly identical, notwithstanding substantial differences in the random and systematic log standard deviations.

2. The systematic log standard deviation for drift at ultimate load (100%) for Table 3.1 data appears low in comparison with data at other fractions of ultimate load.

3. Composite log standard deviation monotonically increases as the load fraction decreases beyond ultimate load.

For comparison, combined statistics for the original data from Table 3.1 (i.e., without the log transformation) are shown in Table 4.4. All standard deviations are seen to increase monotonically as the load fraction decreases beyond ultimate load.

No direct comparisons with the results of Kennedy et al. (1988) can be made from Table 4.4. Note that the composite standard deviation is calculated from its random and systematic components and for that reason does not agree precisely with overall sample standard deviation values presented in Table 4.1.

4.3 Investigation of Distribution Type

The Wilk-Shapiro W-Test (Shapiro and Wilk [1965]) was used to determine whether the data (from Table 3.1) were normally or lognormally distributed. Results of the W-Test on the original data are shown in Table 4.5a. Results for the log-transformed data are shown in Table 4.5b.

The acceptance criterion (for normality of the original or the log-transformed data) is taken at the 5 percent level for less than 30 samples and at the 1 percent level for 30 or greater samples. The appropriate results are shown in ***bold italic*** in Tables 4.5a and 4.5b. These results confirm that, with the exception of the 100 percent load point, these data consistently follow the lognormal distribution and that the log transformation should be used (as was done in Kennedy et al. [1988])].

An additional investigation of data distribution for the 100 percent load point was performed. A relative cumulative frequency curve for all data corresponding to the 100 percent load point from Table 3.1 is shown in Figure 4.2 along with the corresponding cumulative lognormal (least squares) fit. These data are reasonably lognormally distributed, except for the very low drift values and near the drift value of 1.0. The deviation at 1.0 is attributed to the Ogata (1984) data which, by the nature of the experiments, was displacement-controlled to a drift at ultimate load of 1.0 without exception. In Figure 4.3, a similar cumulative frequency plot and comparison with the lognormal fit are shown with the 12 Ogata (1984) data points deleted. A significant improvement is seen in the region of 1.0, although deviations from lognormality are increased somewhat at higher values of drift.

An additional comparison of the data for 100 percent load with the lognormal fit is shown in Figure 4.4 (a plot of the drift limit data [in log form] as a function of the number of standard deviations from the mean). The straight line shown in Figure 4.4 corresponds to the lognormal fit in Figure 4.2 (for all data). Again, deviations from the lognormal distribution are observed at the lower tail. The corresponding plot with the Ogata (1984) data removed is shown in Figure 4.5. The conclusion here is that the data appear to be reasonably lognormally distributed, except at the lower tail, and the lognormal assumption appears reasonable. For most seismic zones, the lower tails of the fragility curve will have less than a 5 percent effect on the overall probability of failure estimate (Ravindra et al. 1984). Therefore, the lognormal assumption is reasonable for the 100 percent load data despite the lack of fit at the lower tail.

Because the best fitted points are at upper and middle data points, a truncation of the data is not necessary.

Table 4.4 Mean and Standard Deviations (Without Log Transformation)

Source	Fraction of Ultimate Load(%)	Mean (%)	Random Std. Dev.	Systematic Std. Dev.	Composite Std. Dev.*
Table 3.1	100	0.80	0.33	0.30	0.445
Table 3.1	90	1.12	0.34	0.43	0.547
Table 3.1	80	1.34	0.43	0.52	0.671
Table 3.1	70	1.52	0.48	0.61	0.776
Table 3.1	60	1.71	0.63	0.75	0.975
Table 3.1	50	1.81	0.54	0.96	1.099

*Calculated from random and systematic components. Therefore, composite standard deviation values do not precisely agree with overall sample standard deviation values presented in Table 4.1.

Table 4.5a Wilk Test for Normality on Original Data

Fraction of Ultimate Load (%)	No. of Samples	Wilk Test Probability Level	Accept or Reject Normality 5 Percent	Accept or Reject Normality 1 Percent
100%	69	<0.0001	Reject	***Reject***
90%	59	<0.0001	Reject	***Reject***
80%	58	<0.0001	Reject	***Reject***
70%	49	<0.0001	Reject	***Reject***
60%	40	<0.0001	Reject	***Reject***
50%	26	0.0007	***Reject***	Close Call

Table 4.5b Wilk Test for Normality on Log Transformed Data

Fraction of Ultimate Load (%)	No. of Samples	Wilk Test Probability Level	Accept or Reject Normality 5 Percent	Accept or Reject Normality 1 Percent
100%	69	<0.0001	Reject	***Reject***
90%	59	0.0234	Reject	***Accept***
80%	58	0.0496	Accept	***Accept***
70%	49	0.0669	Accept	***Accept***
60%	40	0.0326	Reject	***Accept***
50%	26	0.0674	***Accept***	Accept

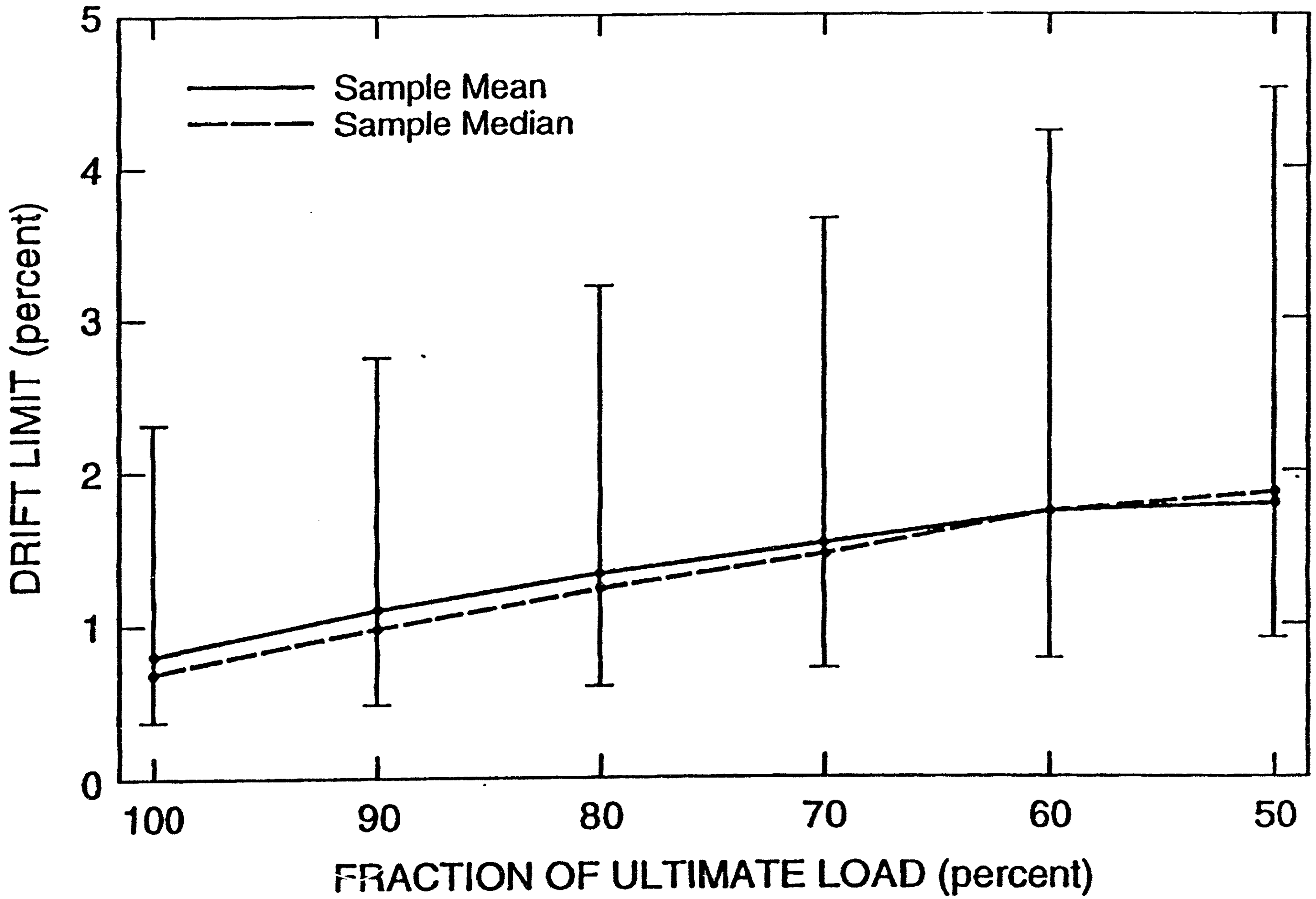


Figure 4.1 Central Tendency and Range of Drift Limit Data

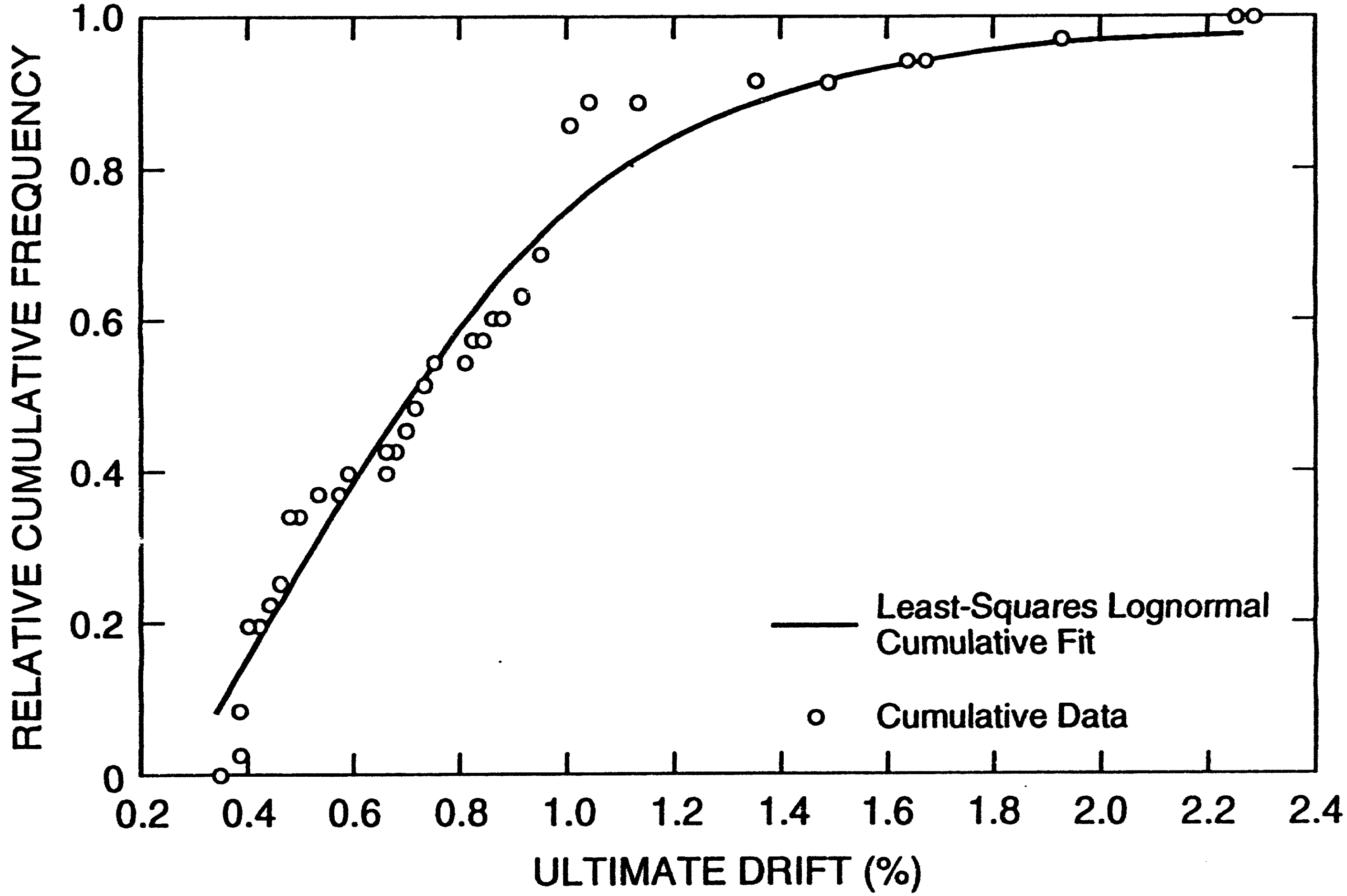


Figure 4.2 Cumulative Frequency and Least-Squares Lognormal Fit (All Data)

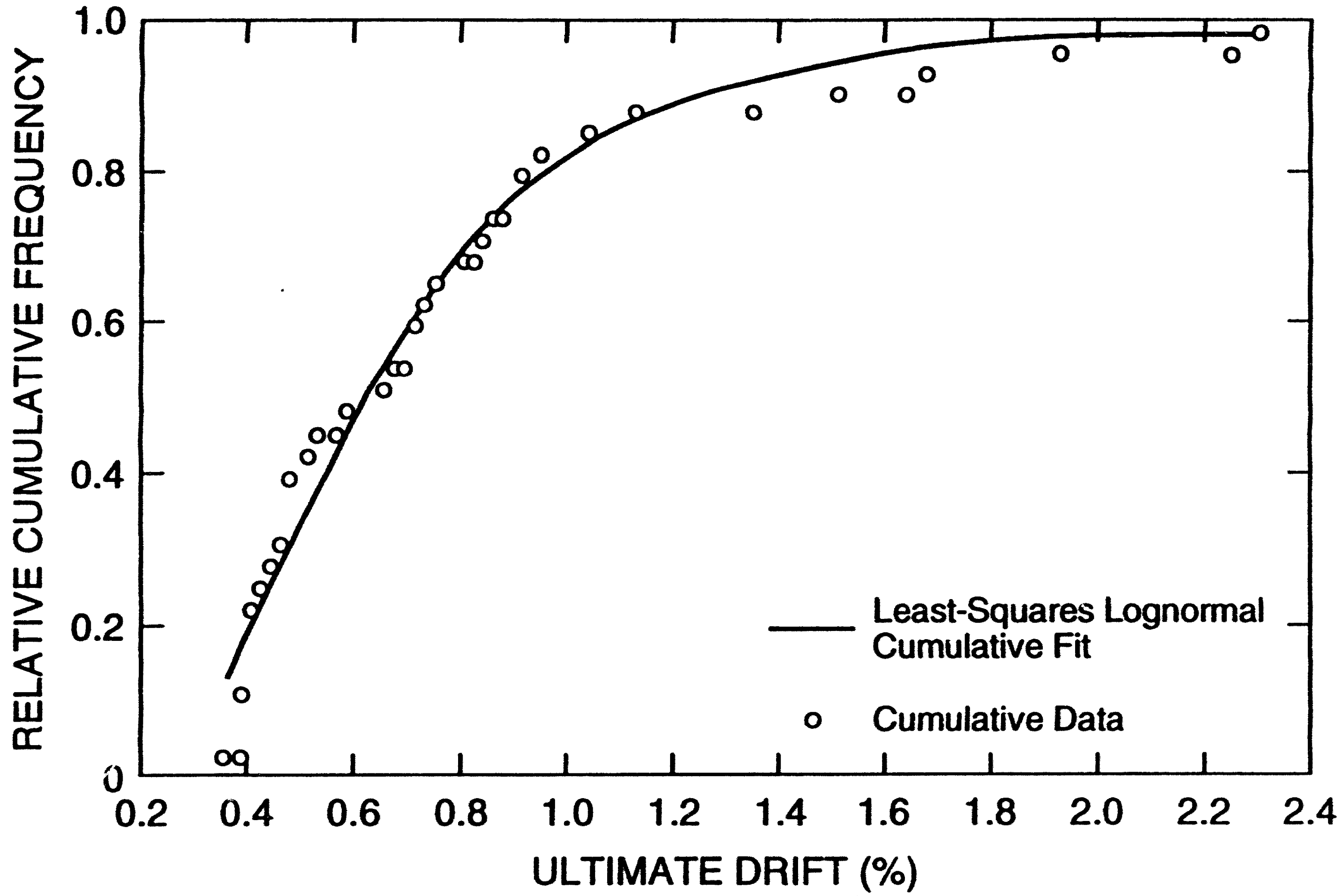


Figure 4.3 Cumulative Frequency and Least Squares Lognormal Fit (Ogata Data Deleted)

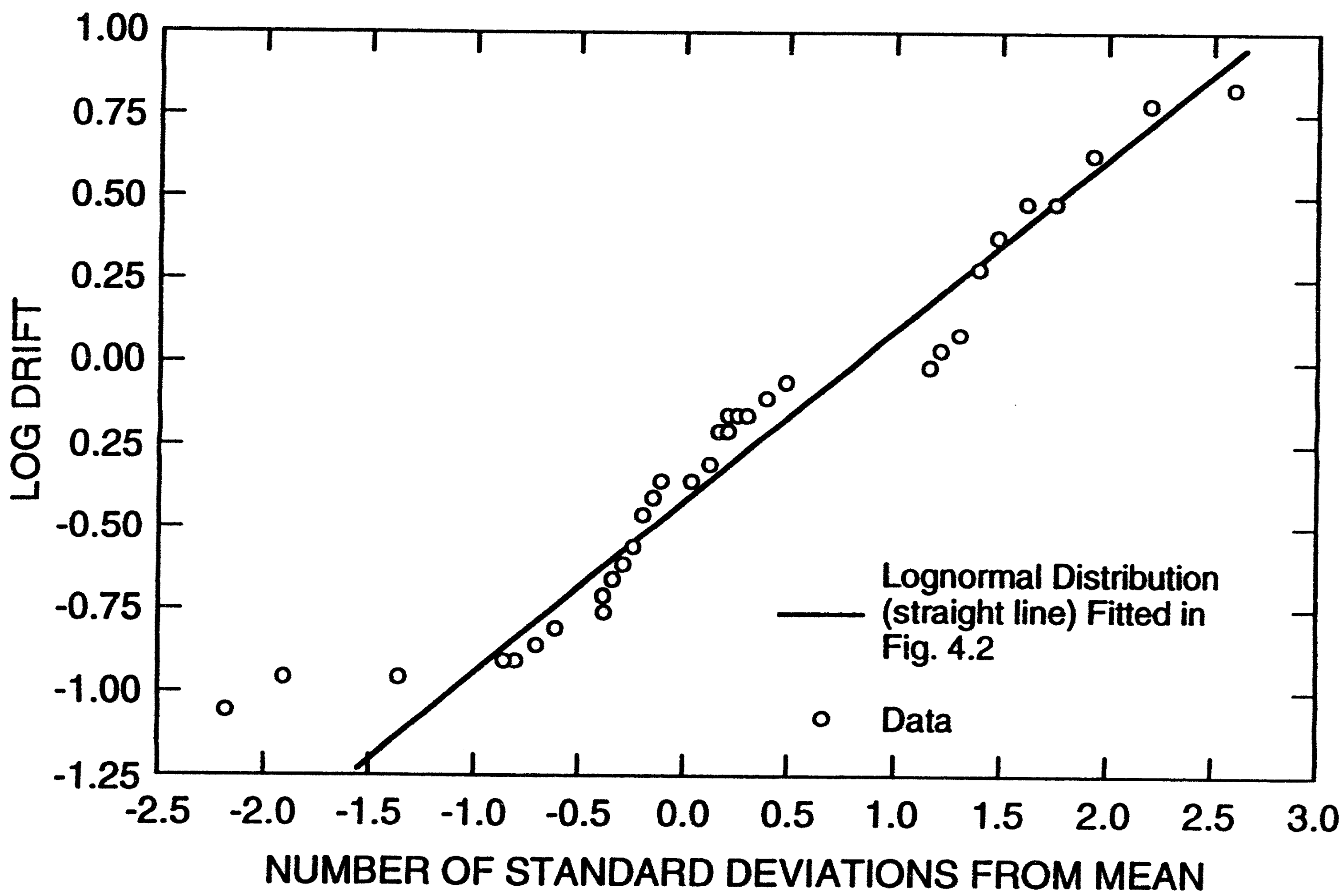


Figure 4.4 Log Frequency Plot (All Data)

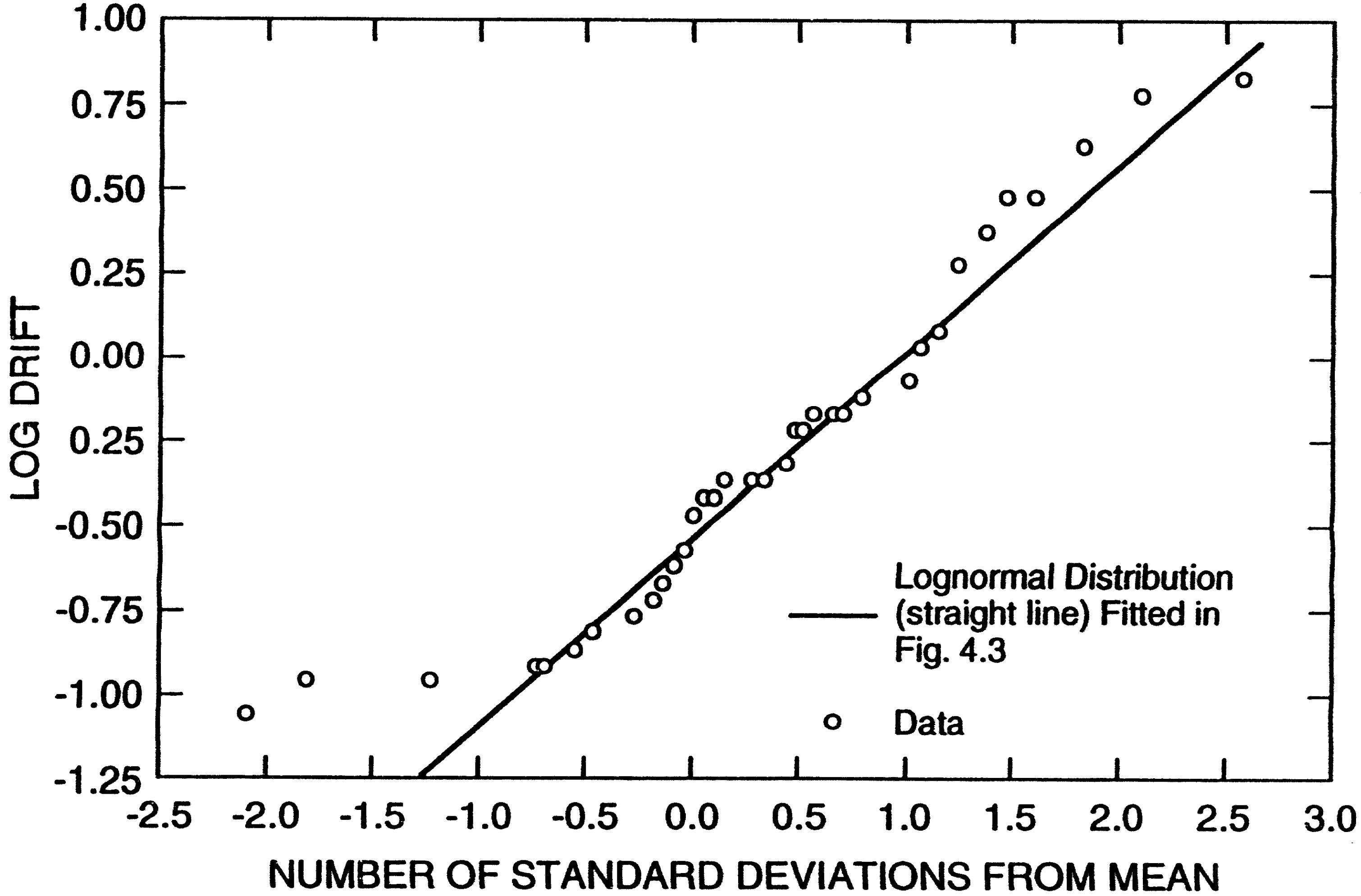


Figure 4.5 Log Frequency Plot (Ogata Data Deleted)

5 Application Of Results To Probability of Failure Estimates

To incorporate the ultimate drift limit data into seismic probabilistic risk assessment or seismic margin assessment, the drift limit statistics must be transformed into a fragility curve. This curve gives the structure's probability of failure as a continuous function of some measure of ground motion level, typically peak spectral acceleration, and is structure specific. In this section, the shear wall ultimate drift limit statistics summarized in Section 4 will be used in conjunction with nonlinear time-history analyses reported in Kennedy et al. (1988) to demonstrate how these statistics are used to develop a fragility curve for a shear wall structure. Kennedy et al. (1988) present a probabilistic evaluation of the seismic capacity of the Diablo Canyon turbine building.

The first step in this procedure is to establish a probability distribution function for failure as a function of maximum story drift. A lognormal distribution was assumed in Kennedy et al. (1988). As discussed in Section 4, statistical analysis of the data summarized in the current study shows that this assumed distribution is valid. Table 4.3 summarizes the statistics used with this distribution, as presented in Kennedy et al. (1988) and those determined in the current study, for various fractions of the ultimate load.

Next, the probability of failure, P_f, is estimated using a logarithmic standard deviation based solely on random variability of the shear wall ultimate drift limit, β_R. Randomness of ground motion is incorporated by using results from Kennedy et al. (1988) for 25 nonlinear, deterministic, time-history analyses, with different inputs scaled to the same average 5 percent damped spectral acceleration, $\bar{S}_a$, in the 3 to 8.5- Hz range. Three different values of $\bar{S}_a$ were used: 2.25 g; 3.0 g; and 6.0 g. All analyses used median structural properties.
To estimate P_{f_i}, the probability of failure for the ith analysis, drift values for all walls were screened to determine the maximum calculated percent story drift. The number of standard deviations that the calculated maximum story drift is from the median ultimate drift limit can be computed as

$$x = \frac{\ln\left(\frac{\delta_c}{\delta_m}\right)}{\beta_R} \tag{5.1}$$

where x = the number of standard deviations from the median, δ_c = the maximum calculated story drift, and δ_m the median ultimate drift limit.

P_{f_i} can now be determined from the cumulative distribution function for a normally distributed random variable as

$$P_{f_i}(x) = \int_{-\infty}^{x} \frac{1}{\sqrt{2\pi}} e^{-0.5t^2} dt \tag{5.2}$$

where t is a variable of integration. Note that for negative values of x, $P_{f_i}(-x) = 1 - P_{f_i}(x)$. The values of P_{f_i} can be found in a standard table of normal probability functions (Beyers, 1981).

The median probability of failure from all analyses corresponding to a particular $\bar{S}_a$ is

$$P_f = \frac{\sum_{i=1}^{n} P_{f_i}}{n} \tag{5.3}$$

where n = the number of analyses performed.

Tables 5.1 through 5.3 estimate the probabilities of failure for the three values of $\bar{S}_a$ considered in Kennedy et al. 1988). Table 5.1 presents results only for the analyses with the three largest percentage drifts because all other analyses with smaller computed ultimate drifts show essentially zero probability of failure. Included in these tables are results determined using the statistics for ultimate drift reported in Kennedy et al. (1988) and the results determined using the corresponding statistics developed in the current study.

A median fragility curve, based on random variability only, can now be developed. First, the assumption is made that the curve is lognormally distributed. A lognormal distribution is fit to the values of P_f determined for the three spectral acceleration levels. This distribution provides a continuous function for the probability of failure versus the peak spectral acceleration. From this fit a median spectral acceleration level that will produce structural failure and a logarithmic standard deviation that considers random variability only can be obtained. Using the drift limit statistics reported in Kennedy et al. (1988), the results of this fit are

Median $\bar{S}_a$ = 4.60 g, corresponding to 50 percent probability of failure and $\beta_R = 0.23$.

When the drift limit statistics developed in the current study are used to predict probabilities of failure, the results of this fit are

Median $\bar{S}_a$ = 4.62 g, corresponding to 50 percent probability of failure), and $\beta_R = 0.26$.

These results show that more than doubling the random variability associated with the median failure drift limit value (as shown in Table 4.3, were random log standard deviations for Kennedy et al. (1988) and the present study are compared) has negligible effect on the median (random variability only) spectral acceleration level that produces failure. Table 5.4 compares the calculated probabilities of failure with those predicted by the lognormal fit to these data based on both sets of drift limit statistics.

The probability of failure based on the composite lognormal standard deviation which considers both randomness and uncertainty is determined in a similar manner as the probability of failure that considers random variability only. The analyses performed in Kennedy et al. (1988) account for uncertainties in structural properties such as shear wall stiffness, strength, and damping. Kennedy et al. (1988) provide results from 50 analyses at four values of $\bar{S}_a$: 2.25 g, 3.0 g, 4.0 g, and 6.0 g. Tables 5.5 through 5.8 summarize the estimates of the probabilities of failure determined from these analyses.

Again, a lognormal fragility curve, this time based on composite variability, is developed by fitting a lognormal distribution to the probability of failure data that is given as a function of $\bar{S}_a$. Using the statistics in Kennedy et al. (1988), the results of this approximation are

Median $\bar{S}_a$ = 4.59 g (corresponding to 50 percent probability of failure), and $\beta_c = 0.37$.

Using the previously determined β_R, the value of the lognormal standard deviation that considers systematic variability only, β_U, can be determined as

$$\beta_U = \sqrt{\beta_c^2 - \beta_R^2} \tag{5.4}$$

For Kennedy et al. (1988) β_R and β_c values, a β_U of 0.29 is determined.

The lognormal fragility curve based on the drift limits statistics developed in the current study has the following fragility statistics:

Median $\bar{S}_a$ = 4.63 g, (corresponding to 50 percent probability of failure), $\beta_c = 0.375$, and $\beta_U = 0.27$.

Table 5.9 compares the calculated probabilities of failure and those predicted by the lognormal fit that considers composite variability for both sets of drift limit statistics.

A spectral acceleration value corresponding to a high confidence of low probability of failure (HCLPF, 95 percent confident that there is less than a five percent probability of failure) can be determined from the assumed lognormal distribution as

$$\text{HCLPF } \bar{S}_a = \text{Median } (\bar{S}_a)\, e^{-1.65\,(\beta_R+\beta_U)}, \tag{5.5}$$

The HCLPF $\bar{S}_a$ determined in Kennedy et al. (1988) was 1.95 g. Based on statistics developed in the current study, an HCLPF $\bar{S}_a$ of 1.93 g is obtained.

Finally, the fragility estimates are revised to reflect other sources of variability related to modeling, directional effects, and incoherence of ground motion not accounted for in the nonlinear analyses. When these sources of variability are accounted for in a manner similar to that presented in Kennedy et al. (1988), the following fragility estimates are obtained for the statistics developed in the current study:

Median $\bar{S}_a$ = 4.91 g,
$\beta r = 0.29$,
$\beta_U = 0.31$,
HCLPF $\bar{S}_a$ = 1.83 g.

For comparison, the corresponding statistics developed by Kennedy et al. (1988) are

Median $\bar{S}_a$ = 4.87 g,
$\beta r = 0.26$,

$\beta_U = 0.33$,

HCLPF $\bar{S}_a = 1.84$ g.

These results show that the fragility curve obtained by Kennedy et al. (1988) for a shear wall structure, based on a very limited amount of shear wall drift limit data, is similar to the fragility curve based on drift limit statistics developed in the current study.

Table 5.1 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Random Variability Only, $\bar{S}_a$ = 2.25 g.

Trial No.**	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
15	0.24	0.26	0.0	0.2
18	0.15	0.15	0.0	0.0
20	0.19	0.24	0.0	0.1
			$P_f = \frac{\sum_{i=1}^{25} P_{f_i}}{25} = 0.\%$	$P_f = \frac{\sum_{i=1}^{25} P_{f_i}}{25} = 0.\%$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_R = 0.15$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_R = 0.35$.

*Kennedy et al.(1988).

**There are a total of 25 trials. All trials not listed have lower story drifts, resulting in essentially zero probabilities of failure.

Table 5.2 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Random Variability Only, $\overline{S_a}$ = 3.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
1	0.18	0.18	0.0	0.0
2	0.35	0.42	0.0	6.2
3	0.09	0.18	0.0	0.0
4	0.04	0.06	0.0	0.0
5	0.17	0.26	0.0	0.2
6	0.26	0.26	0.0	0.2
7	0.04	0.11	0.0	0.0
8	0.04	0.05	0.0	0.0
9	0.30	0.38	0.0	3.4
10	0.20	0.22	0.0	0.0
11	0.16	0.24	0.0	0.1
12	0.10	0.17	0.0	0.0
13	0.18	0.18	0.0	0.0
14	0.06	0.05	0.0	0.0
15	0.43	0.61	17.9	31.8
16	0.35	0.37	0.0	2.9
17	0.20	0.28	0.0	0.3
18	0.53	0.69	46.0	45.2
19	0.04	0.05	0.0	0.0
20	0.51	0.59	12.7	28.5
21	0.03	0.15	0.0	0.0
22	0.11	0.19	0.0	0.0
23	0.21	0.32	0.0	1.0
24	0.43	0.29	0.0	7.0
25	0.04	0.19	0.0	0.0

$$P_f = \frac{\sum_{i=1}^{25} P_{f_i}}{25} = 3.1\% \qquad P_f = \frac{\sum_{i=1}^{25} P_{f_i}}{25} = 5.1\%$$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_R = 0.15$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_R = 0.35$.

*Kennedy et al. (1988).

Table 5.3 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses*Considering Random Variability Only, $\overline{S_a}$ = 6.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
1	0.89	1.46	100	97.8
2	0.97	2.05	100	99.9
3	0.59	0.97	99	80.3
4	0.66	0.90	95	73.8
5	0.84	1.20	100	92.8
6	0.82	1.50	100	98.2
7	0.48	0.65	31	38.5
8	0.43	0.64	27	36.8
9	1.16	1.89	100	99.7
10	0.71	1.13	100	90.1
11	0.48	0.57	9	25.2
12	0.81	1.45	100	97.7
13	0.74	1.41	100	97.3
14	0.73	1.21	100	93.1
15	1.05	2.08	100	99.9
16	1.00	1.67	100	99.2
17	1.09	1.72	100	99.4
18	1.82	2.86	100	100.0
19	0.55	0.95	98	78.6
20	1.23	1.91	100	99.7
21	0.55	0.65	31	38.5
22	0.77	1.33	100	96.0
23	0.82	1.45	100	97.7
24	0.81	1.33	100	96.0
25	0.68	1.28	100	95.0

$$P_f = \frac{\sum_{i=1}^{25} P_{f_i}}{25} = 87.6\% \qquad P_f = \frac{\sum_{i=1}^{25} P_{f_i}}{25} = 84.8\%$$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_R = 0.15$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_R = 0.35$.

*Kennedy et al. (1988).

Table 5.4 Comparison of the Calculated Probabilities of Failure and the Probabilities of Failure Predicted by a Lognormal Fit (Considering Randon Variability Only) to These Data

Spectral Acceleration (g's)	Randomness Only P_f^* (%)	Predicted P_f (%) From Lognormal Fit*	Randomness Only P_f^{**} (%)	Predicted P_f (%) From Lognormal Fit**
2.25	approx. 0	0.1	approx. 0	0.3
3.0	3.1	3.2	5.1	4.8
6.0	87.6	87.8	84.8	84.1

*Based on Kennedy et al. (1988), drift limit statistics.

**Based on drift limit statistics developed in the current study.

Table 5.5 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 2.25 g.

Trial No. **	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
2	0.08	0.15	0.0	0.0
13	0.19	0.22	0.0	0.1
15	0.32	0.41	5.5	6.6
16	0.27	0.33	1.3	1.8
17	0.39	0.56	25.1	25.0
18	0.02	0.03	0.0	0.0
20	0.16	0.16	0.0	0.0
26	0.06	0.14	0.0	0.0
31	0.06	0.16	0.0	0.0
40	0.47	0.61	34.1	32.8
41	0.11	0.11	0.0	0.0
42	0.63	0.94	81.1	76.3
43	0.11	0.22	0.0	0.1
			$P_f = \frac{\sum_{i=1}^{50} P_{f_i}}{50} = 2.9\%$	$P_f = \frac{\sum_{i=1}^{50} P_{f_i}}{50} = 2.9\%$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_C = 0.335$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_C = 0.373$.

*Kennedy et al. (1988).

**There are a total of 50 trials. All trials not listed have lower story drifts, resulting in essentially zero probabilities of failure.

Table 5.6 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 3.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
1	0.19	0.15	0.0	0.0
2	0.45	0.37	9.3	10.4
3	0.19	0.23	0.0	0.1
4	0.04	0.05	0.0	0.0
5	0.02	0.04	0.0	0.0
6	0.19	0.33	1.3	1.8
7	0.03	0.09	0.0	0.0
8	0.02	0.05	0.0	0.0
9	0.06	0.31	0.8	1.2
10	0.03	0.06	0.0	0.0
11	0.22	0.19	0.0	0.1
12	0.14	0.16	0.0	0.0
13	0.42	0.71	51.6	48.5
14	0.09	0.06	0.0	0.0
15	0.72	0.96	82.6	78.0
16	0.45	0.63	37.8	36.0
17	0.51	0.83	69.5	64.8
18	0.45	0.26	9.3	10.4
19	0.02	0.04	0.0	0.0
20	0.40	0.46	0.6	11.5
21	0.24	0.31	0.8	1.2
22	0.33	0.52	18.7	19.2
23	0.01	0.01	0.0	0.0
24	0.05	0.24	0.0	0.2
25	0.31	0.43	7.4	8.3

See Footnotes at end of Table.

Table 5.6 (Continued) Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 3.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
26	0.17	0.41	5.6	6.6
27	0.37	0.17	2.9	3.7
28	0.18	0.20	0.0	0.0
29	0.05	0.18	0.0	0.0
30	0.06	0.11	0.0	0.0
31	0.14	0.45	9.3	10.4
32	0.03	0.12	0.0	0.0
33	0.33	0.30	1.3	1.8
34	0.08	0.09	0.0	0.0
35	0.12	0.15	0.0	0.0
36	0.19	0.23	0.0	0.1
37	0.16	0.28	0.3	0.6
38	0.25	0.36	2.4	3.2
39	0.04	0.05	0.0	0.0
40	0.75	1.10	91.1	87.3
41	0.31	0.43	7.4	8.3
42	0.67	0.98	84.1	76.6
43	0.41	0.67	44.8	42.4
44	0.08	0.06	0.0	0.0
45	0.11	0.13	0.0	0.0
46	0.06	0.15	0.0	0.0
47	0.35	0.25	1.9	2.7
48	0.15	0.36	2.4	3.2
49	0.27	0.29	0.4	0.7
50	0.11	0.22	0.0	0.1

$$P_f = \frac{\sum_{i=1}^{50} p_{f_i}}{50} = 11.1\% \qquad P_f = \frac{\sum_{i=1}^{50} p_{f_i}}{50} = 10.8\%$$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_C = 0.335$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_C = 0.373$.

*Kennedy et al (1988).

Table 5.7 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 4.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
1	0.29	0.64	39.4	37.6
2	1.10	0.65	91.1	87.2
3	0.40	0.46	10.6	11.5
4	0.18	0.25	0.0	0.2
5	0.06	0.11	0.0	0.0
6	0.47	0.52	18.7	19.2
7	0.16	0.29	0.4	0.7
8	0.05	0.09	0.0	0.0
9	0.24	0.40	4.7	5.8
10	0.06	0.20	0.0	0.0
11	0.75	0.35	58.3	54.4
12	0.29	0.44	8.2	9.3
13	0.67	1.24	95.6	92.7
14	0.28	0.40	4.7	5.8
15	1.18	1.84	99.8	99.4
16	0.87	0.99	84.8	80.3
17	0.73	0.96	82.6	78.0
18	0.38	0.79	63.7	59.8
19	0.20	0.17	0.0	0.0
20	0.45	0.70	50.0	47.0
21	0.45	0.45	9.3	10.4
22	0.58	0.74	56.8	52.9
23	0.02	0.03	0.0	0.0
24	0.32	0.36	2.3	3.2
25	0.49	0.82	68.1	63.6

See Footnotes at End of Table.

Table 5.7 (Continued) Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 4.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+ (%)	Probability of Failure++ (%)
26	0.69	0.85	71.9	67.2
27	0.30	0.40	4.7	5.8
28	0.43	0.46	10.6	11.5
29	0.06	0.13	0.0	0.0
30	0.25	0.27	0.2	0.4
31	0.79	1.37	97.7	95.8
32	0.17	0.29	0.4	0.7
33	0.40	0.51	17.1	17.8
34	0.35	0.40	4.7	5.8
35	0.27	0.39	4.0	5.0
36	0.30	0.35	2.0	2.7
37	0.51	0.90	77.3	72.5
38	0.51	0.99	84.8	80.3
39	0.13	0.27	0.2	0.4
40	1.20	1.70	99.6	98.9
41	0.53	0.90	77.3	72.5
42	1.50	1.72	99.6	99.0
43	0.74	1.20	94.6	91.5
44	0.14	0.20	0.0	0.0
45	0.22	0.39	4.0	5.0
46	0.22	0.32	1.0	1.5
47	0.80	0.92	79.4	74.5
48	0.51	0.79	63.7	59.8
49	0.58	0.51	28.8	28.1
50	0.43	1.04	88.1	83.8

$$P_f = \frac{\sum_{i=1}^{50} P_{fi}}{50} = 37.2\%$$

$$P_f = \frac{\sum_{i=1}^{50} P_{fi}}{50} = 36.0\%$$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_C = 0.337$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_C = 0.373$.

*Kennedy et al. (1988).

Table 5.8 Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses* Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 6.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
1	0.94	1.27	96.2	93.6
2	1.76	1.62	99.7	99.2
3	1.23	0.99	95.4	92.4
4	0.63	0.86	72.9	68.3
5	0.37	0.60	67.4	1.3
6	1.68	0.92	99.5	98.8
7	0.42	0.66	57.1	40.8
8	0.27	0.44	8.2	9.3
9	1.15	1.09	93.1	89.5
10	0.47	0.66	42.9	40.8
11	0.72	0.86	72.9	68.3
12	0.53	0.97	83.4	78.8
13	1.20	1.97	99.9	99.7
14	0.53	0.90	77.3	72.5
15	1.97	2.81	100.0	100.0
16	1.44	1.51	98.9	97.6
17	1.11	1.42	98.3	96.6
18	1.46	1.15	98.6	97.1
19	0.45	0.71	51.6	48.5
20	0.80	1.21	94.8	91.8
21	0.69	1.02	86.9	82.5
22	0.84	1.25	95.8	93.0
23	0.24	0.35	2.0	2.7
24	0.86	0.78	72.9	68.3
25	0.71	1.15	93.1	89.5

See Footnotes at End of Table.

Table 5.8 (Continued) Probability of Failure Estimates From Diablo Canyon Turbine Building Nonlinear Analyses[*] Considering Composite Variability and Uncertainties in Structural Properties, $\overline{S_a}$ = 6.0 g.

Trial No.	Wall 19 Max. Story Drift (%)	Wall 31 Max. Story Drift (%)	Probability of Failure+(%)	Probability of Failure++(%)
26	1.36	1.76	99.7	99.2
27	0.57	0.76	59.9	55.8
28	0.79	1.12	65.5	88.2
29	0.35	0.43	7.4	8.3
30	0.87	1.07	89.8	85.6
31	1.71	2.54	100.0	100.0
32	0.41	0.62	35.9	34.4
33	0.50	0.66	42.9	40.8
34	0.80	1.11	91.6	87.7
35	0.70	0.67	50.0	47.0
36	0.51	0.61	34.1	33.3
37	1.31	1.99	99.9	99.7
38	2.11	1.90	100.0	99.8
39	0.91	1.17	93.7	90.3
40	1.70	2.20	100.0	99.9
41	1.03	1.41	98.2	96.4
42	2.86	3.14	100.0	100.0
43	1.69	2.30	100.0	99.9
44	0.35	0.46	10.6	11.5
45	0.51	0.67	44.8	42.4
46	0.57	0.76	59.9	55.8
47	1.23	1.42	98.3	96.6
48	0.59	1.15	93.1	89.5
49	1.14	1.73	99.7	99.1
50	1.49	2.00	100.0	99.7
			$P_f = \frac{\sum_{i=1}^{50} P_{fi}}{50} = 76.7\%$	$P_f = \frac{\sum_{i=1}^{50} P_{fi}}{50} = 74.2\%$

+Using statistics reported in Kennedy et al. (1988), $\delta_m = 0.7$, $\beta_C = 0.335$.

++Using statistics reported in this study, $\delta_m = 0.72$, $\beta_C = 0.373$.

[*]Kennedy et al. (1988).

Table 5.9 Comparison of the Calculated Probablities of Failure and the Probabilities of Failure Predicted by a Lognormal Fit (Considering Composite Variability) to These Data

Spectral Acceleration (g)	Composite P_{f_i}* (%)	Predicted P_f (%) From Lognormal Fit+	Composite Only P_f++ (%)	Predicted P_f (%) From Lognormal Fit++
2.25	2.9	2.7	2.9	2.7
3.0	11.1	12.5	10.8	12.4
4.0	37.2	35.5	36.0	34.8
6.0	76.7	76.5	74.2	75.5

+ Based on Kennedy et al. (1988) drift limit statistics.

++ Based on drift limit statistics developed in the current study.

6 Drift Limits in Existing Earthquake Design Codes

Bertero et al. (1991) reviewed existing earthquake-resistant building design codes for a variety of different countries. Drift limits specified are in terms of Interstory Drift Index (IDI), which, for a single story structure, reduces to the definition of Drift Limit used in this report.

Results are presented by Bertero et al. (1991) for two limit states: Serviceability Level and Safety (Collapse) Level. At the Serviceability Level, Berteró et al. (1991) indicates that present seismic codes give maximum IDI's in the range 0.06 percent - 0.6 percent. As this limit state is not of particular interest in the present study, the reader is referred to Bertero et al. (1991) for details on the codes reviewed and the results for the various codes.

Results in Bertero et al.(1991) at the Safety Level are, however, of particular interest. Maximum acceptable IDI values at ultimate limit states (collapse) are obtained from Bertero et al. (1991) as listed in Table 6.1. Bertero et al. (1991) further states that the usual variation of IDI's for present seismic codes are in the range of 1 to 3 percent, varying with the type of structure and its function.

It is interesting to compare the results summarized in Table 6.1 with the drift limits found for shear walls in this study, although it is acknowledged that code drift limits are probably more appropriate to medium- or high-rise structures than to a low aspect-ratio shear wall structure. The codes typically do not distinguish between low, medium, or high rise structures. From Table 2.1 (Aspect ratios in the range 0.24-3.53), the mean and median drifts at ultimate load are

Mean: 1.043%
Median: 0.820%.

For the squat walls in Table 3.1 (Aspect ratios less than or approximately equal to 1) the drift limits are determined at and beyond ultimate load. The results at ultimate load are

Mean: 0.802%
Median: 0.72%.

Corresponding results (well into the softening region) for the case when the load has dropped to 50 percent of the ultimate value are

Mean: 1.813%
Median: 1.84%.

It is clear that code drift limits reported by Bertero et al. (1991) are generally unconservative for low aspect ratio shear walls.

Table 6.1 Code Drift Limits at Ultimate State

Country	Code	Year	Maximum I.D.I. (%)	Comment
USA	UBC	1988	1.5	Implicit for buildings with short period (<65 ft. high).
			1.125	Implicit for buildings > 65 ft. high.
USA	ATC 3-06	1978	1.00	Recommended value for "Essential Facilities" (SHEG III)
			1.50	Recommended value for SHEG I and SHEG II.
			2.00	Recommended value for SHEG I ("Structures of Ordinary Importance") when height is less than 3 stories and no brittle-type finishes.
Mexico	Mexico DF	1987	0.60 or 1.20	Value depends on whether or not the nonstructuural components can be damaged.
Japan	BSL	N/A	1.00	In practice. No code limit specified.
New Zealand	NZS	1984	1.00	Value from Fig. 5.2, Bertero et al. (1991).
Europe	CEB	1987	2.50	Value from Fig. 5.2, Bertero et al. (1991).

7 Summary and Recommendations

7.1 Summary

1. A review of shear wall references from the open literature resulted in a set of 190 drift limits at ultimate load for shear walls with aspect ratios up to 3.53 subjected to different types of loading (monotonic static, cyclic static, and dynamic). Drift at ultimate load did appear higher at the higher aspect ratios. However, it was found to be infeasible to quantify the influence of various geometrical and material parameters on ultimate drift limit because of significant simultaneous variations in numerous parameters between experimental programs.

2. Further screening of references containing shear wall experimental data led to the selection of ten references containing relevant data for shear walls with aspect ratios of approximately 1 or less undergoing cyclic loading. These low aspect ratios are common for shear walls used in Nuclear Power Plant structures. Further, cyclic loading is relevant to seismic response.

3. A total of 69 data sets were obtained graphically from these papers using a set of rules established in Section 3. The rules provide lower-bound estimates for drift at ultimate load. Drift limits at ultimate load and beyond (at 90, 80, 70, 60 and 50 percent of ultimate load) were determined and their statistical properties were examined.

4. The data were found to be lognormally distributed, and an analysis of variance was performed. Median drift limit and statistical parameters were found to be in reasonable agreement with those used by Kennedy et al. (1988).

5. The shear wall ultimate drift limit statistics were used in conjunction with nonlinear time-history analysis results reported by Kennedy, et al (1988) to demonstrate how these statistics are used to estimate the probability of failure for a shear wall structure. Almost identical fragility estimates are obtained with the statistics developed in the present study as with those developed by Kennedy et al. (1988).

6. Ultimate drift limit results were compared with building design codes (as summarized by Bertero, et al. (1991). Code drift limits were found to be generally unconservative for the low-aspect-ratio shear walls investigated herein, although such code drift limits are likely more directed toward high-rise structures than squat shear walls.

7.2 Recommendations

The following recommendations pertain to the appropriate values of ultimate drift limits to be used in seismic probabilistic risk assessments and seismic margin assessments done in conjunction with IPEEE (NRC, 1991):

1. The most appropriate definition of ultimate drift limit is the drift limit at 100 percent of ultimate load. (See discussion in Section 3.)

2. For reinforced concrete shear walls with aspect ratios of approximately 1 or less undergoing cyclic loading, the recommended ultimate drift limit is 0.72 percent (Median value). Corresponding recommended values in the softening region beyond ultimate load (where their use can be justified) are:

Fraction of Ultimate Load (%)	Median Drift Limit (%)
90	1.00
80	1.24
70	1.48
60	1.64
50	1.84

3. Code drift limits were found to be generally unconservative for the low-aspect-ratio shear walls investigated herein. Such code drift limits appear more appropriate to high-rise structures than to the squat shear walls investigated herein, however.

8 References

Alexander, C., A. Heidebrecht, and W.Tso, "Cyclic Load Tests on Shear Wall Panels," Proceedings of the Fifth World Conference on Earthquake Engineering, 1973.

Antebi, J., "Model Analysis of the Response of Shear Walls to Dynamic Loads," Thesis, Massachusetts Institute of Technology, 1961.

Barda, F., "Shear Strength of Low-Rise Walls with boundary Elements," Ph.D. Dissertation, Lehigh University, Bethlehem, PA, 1972.

Barda, F., J. M. Hanson, and W. G. Corley, "Shear Strength of Low-Rise Walls With Boundary Elements," In: Reinforced Concrete Structures in Seismic Zones, SP-53, American Concrete Institute, Detroit, MI, 1976.

Benjamin, J. R., and H. A. Williams, "Investigation of Shear Walls, Part 6." Department of Civil Engineering, Stanford University, Stanford, CA, Technical Report No. 4, August 1, 1954.

Benjamin, J. R., and H. A. Williams, "Investigation of Shear Walls, Part 9," Department of Civil Engineering, Stanford University, Stanford, CA, September 1955.

Benjamin, J. R., and H. A. Williams, "The Behavior of One-Story Reinforced Concrete Shear Walls," Journal of the Structural Division, ASCE, Vol. 83, Paper 1254, pp. 1-49, May 1957. (Also appears as Paper No. 2998, ASCE Transactions, Vol. 124, pp. 669-708, 1959).

Benjamin, J. R., and H. A. Williams, "Reinforced Concrete Shear Wall Assemblies," Journal of the Structural Division, ASCE, Vol. 86, pp. 1-32, August 1960.

Bertero, V. V., The Response of Shear Walls Subjected to Dynamic Loads," Dissertation, Massachusetts Institute of Technology, Cambridge, MA, June 1957.

Bertero, V. V., J. C. Anderson, H. Krawinkler, and E. Miranda, "Design Guidelines for Ductility and Drift Limits," Report No. UCB/EERC-91/15, University of California at Berkeley, Berkeley, CA, July 1991.

Beyer, W. H., CRC Standard Mathematical Tables, 26th Edition, CRC Press, Inc., Boca Raton, Florida, 1981.

Cardenas, A. E., H. G. Russell, and W. G. Corley, Strength of Low-Rise Structural Walls," SP-63, American Concrete Institute, 1980.

Cervenka, V., and K. Gerstle, "Inelastic Analysis of Reinforced Concrete Panels: Experimental Verification and Application," Journal of the International Association of Bridge and Structural Engineering, Vol. 32-II, 1971.

Corley, W. G., A. E. Fiorato, and R. G. Oesterle, "Structural Walls." ACI Journal, SP-72-4, American Concrete Institute, pp. 77-131, 1981.

Elnashai, A. S., K. Pilakoutas, and N. N. Ambraseys, "Experimental Behavior of Reinforced Concrete Walls Under Earthquake Loading," Earthquake Engineering and Structural Dynamics, Vol. 19, pp. 389-407, 1990.

Endo, T., "Hysteretic Behavior of Reinforced Concrete Shear Walls," Memoirs of Faculty of Tech. Tokyo Metropolitan University, No. 32, pp. 3195-3206, 1982.

Esaki, F., M. Tomii, M. Setoguchi, and Y. Matsuishi, "Statistical Investigation of Angular Shear Distortion of Framed Shear Walls in Which the First Shear Crack Occurred in Panel Walls," Transactions of the Japan Concrete Institute, Vol. 3, pp. 273-280, 1981.

Farrar, C. R., J. G. Bennett, W. E. Dunwoody, and W. E. Baker, "Static-Load Cycle Testing of a Low-Aspect-Ratio Six-Inch Wall TRG-Type Structure TRG-4-6" (NUREG/CR-5222), LA-11422-MS, Los Alamos National Laboratory, Los Alamos, NM, 1989.

Farrar, C. R., J. G. Bennett, W. E. Dunwoody, and W. E. Baker, "Static Load Cycle Testing of a Low-Aspect-Ratio Four-Inch Wall, TRG-Type Structure TRG-5-4," NUREG/CR-5487, LA-11739-MS, Los Alamos National Laboratory, Los Alamos, NM, 1990.

J. M. Ferritto, "An Economic Analysis of Earthquake Design Levels," TN No: N-1640, Naval Civil Engineering Laboratory, Port Hueneme, CA, July, 1982.

Ferritto, J. M., "An Economic Analysis of Earthquake Design Levels for New Concrete Construction," TN No: N-1671, Naval Civil Engineering Laboratory, Port Hueneme, CA, July 1983.

References

Fiorato, A. E., R. G. Oesterle, and J. E. Carpenter, "Reversing Load Tests of Five Isolated Structural Walls." International Symposium on Earthquake Structural Engineering, St. Louis, MO, August 1976.

Fiorato, A. E., R. G. Oesterle, P. H. Kaar, G. B. Barney, B. G. Rabbat, J. E. Carpenter, H. G. Russell, and W. G. Corley, "Highlights of an Experimental Investigation of the Seismic Performance of Structural Walls." Proceedings of the ASCE/EMD Specialty Conference, Dynamic Response of Structures, University of California, Los Angeles, March 30, 1976.

Fiorato, A. E., and W. G. Corley, "Laboratory Tests of Earthquake-Resistant Structural Wall Systems and Elements." Proceedings of Earthquake-Resistant Reinforced Concrete Building Construction, Berkeley, CA, June 1978.

Galletley, G. D., "An Experimental and Analytical Investigation of Reinforced Concrete Shear Panels," Ph.D. Dissertation, Massachusetts Institute of Technology, 1952.

Gantenbein, F., J. Dalbera, C. Duretz, J. C. Queval, and A. Epstein, "Experimental Study on Concrete Shear Wall Behavior Under Seismic Loading," Proceedings of the Eleventh SMIRT Conference, Volume K, Tokyo, Japan, 1991.

Graham, C., "Tests on Short Columns and Structural Walls Under Cyclic Actions," Darmstadt Concrete, Vol. 2, pp. 89-102, 1987.

Graybill, F. A., An Introduction to Linear Statistical Models, Volume 1, McGraw-Hill, New York, 1961.

Kennedy, R. P., D. A. Wesley, and W. A. Tong, "Probabilistic Evaluation of the Diablo Canyon Turbine Building Seismic Capacity Using Nonlinear Time History Analyses." NTS Report Number 1643.01, 1988.

Lefas, I. D., M. D. Kotsovos, and N. N. Ambraseys, "Behavior of Reinforced Concrete Structural Walls: Strength, Deformation Characteristics, and Failure Mechanism," ACI Structural Journal, pp. 23-31, January-February 1990.

Lefas, I. D., and M. D. Kotsovos, "Strength and Deformation Characteristics of Reinforced Concrete Walls Under Load Reversals," ACI Structural Journal, pp. 716-726, November-December 1990.

Moehle, J. P., M. A. Sozen, and H. T. Tang, "Concrete Wall Stiffness: Calculation vs. Measurement," Proceedings of the Third Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment and Piping, Orlando, FL, December 1990.

Nuclear Regulatory Commission, "Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities - 10 CFR 50.54(f), Generic Letter No. 88-20, Supplement 4, June 28, 1991.

Oersterle, R. G., A. E. Fiorato, and J. D. Aristizabal-Ochoa, "Free Vibration Tests of Structural Concrete Walls and Analysis of Free Vibration Tests of Structural Walls," NSF/RA-800043, Portland Cement Association, Skokie, IL, 1980.

Oersterle, R. G., A. E. Fiorato, J. D. Aristizabal-Ochoa, and W. G. Corley, "Hysteretic Response of Concrete Structural Walls," SP 63, American Concrete Institute, 1980.

Ogata, K., and T. Kabeyasawa, "Experimental Study on the Hysteretic Behavior of Reinforced Concrete Shear Walls Under the Loading of Different Moment-to-Shear Ratios," Transactions of the Japan Concrete Institute, Vol. 6, pp. 717-724, 1984.

Paulay, T., "Some Aspects of Shear Wall Design," Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 5, No. 3, pp. 89-100, 1972.

Paulay, T., "Earthquake Resistant Structural Walls," Proceedings of "Earthquake-Resistant Reinforced Concrete Building Construction," Berkeley, CA, 1978.

Paulay, T., M.J.N. Priestly, and A. J. Synge, "Ductility in Earthquake Resisting Shear Walls," Journal of the American Concrete Institute, pp. 257-269, July-August, 1982.

Ravindra, M. K., H. Banon, R. H. Sues, and R. D. Thrasher, "Sensitivity Studies of Seismic Risk Models", NP-3562, Electric Power Research Institute, June 1984.

Saatcioglu, M., "Hysteretic Shear Response of Low-Rise Walls," Concrete Shear in Earthquake, Proceedings of the International Workshop on Concrete Shear in Earthquake, University of Houston, Houston, Texas, Elsevier, 1991.

SAS User's Guide: Statistics, Version 5 Edition, SAS Institute, Inc., Carey, NC, 1985.

Shapiro, S.S., and M. B. Wilk, "An Analysis of Variance Tests for Normality (Complete Samples)," Biometrika, Vol. 52, pp. 591-611, 1965.

Shiga, T., A. Shibata, and J. Takahashi, "Experimental Study on Dynamic Properties of Reinforced Concrete Shear Walls," Proceedings of the Fifth World Conference on Earthquake Engineering, pp. 1157-1166, 1973.

Shiga, T., A. Shibata, and J. Takahashi, "Hysteretic Behavior of Reinforced Concrete Shear Walls," Proceedings of the Review Meeting U. S.-Japan Cooperative Research Program in Earthquake Engineering, August 18-20, 1975, Honolulu, HI, pp. 107-117, 1976.

Simeonov, B., "Experimental Investigation of Strength and Deformation of Reinforced Concrete Structural Walls," Proceedings of the Eighth World Conference on Earthquake Engineering, pp. 387-394, 1984.

Sozen, M. A., and J. P. Moehle, "Stiffness of Reinforced Concrete Walls Resisting In-Plane Shear," EPRI Research Project 3094-1, Electric Power Research Institute, Palo Alto, CA, 1991.

Stivers, R. H., Stresses and Deflections in Reinforced Concrete Shear Walls Containing Rectangular Openings," Ph.D. Dissertation, Stanford University, February 1955.

Tanaka, H., K. Imoto, S. Yoshizaki, K. Emori, Y. Inada, and H. Nanba, "An Evaluation Method for Restoring Force Characteristics of R/C Shear Walls of Reactor Buildings," Proceedings of Ninth World Conference on Earthquake Engineering, August 2-9, 1988, Tokyo-Kyoto, Japan, Vol. VI, pp. 747-752, 1988.

Tomii, M., and M. Takeuchi, "The Relations Between the Deformed Angle and the Shearing Force Ratio (0.80 ~ 1.00) with Regard to 200 Shear Walls," Trans. of AIJ, No. 153, November 1968.

Umemura, H., H. Aoyama, M. Ito, and Y. Hosokawa, "Aseismic Characteristics of R. C. Box and Cylinder Walls," Proceedings of the Sixth World Conference on Earthquake Engineering, New Delhi, India, pp. 3144-3149, 1976.

Umemura, H., H. Aoyama, and Y. Hosokawa, "Restoring Force Characteristics of R. C. Walls with Openings and Reinforcing Methods," Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, pp. 209-216, 1980.

USNRC, "An Approach to the Quantification of Seismic Margins in Nuclear Power Plants," U. S. Nuclear Regulatory Commission, Washington, DC, August 1985.

Wang, F., F. Gantenbein, J. Dalbera, and C. Duretz, "Seismic Behavior of Reinforced Concrete Shear Walls," SMIRT 89, Anaheim, CA, 1989.

Welch, B. L., "On Linear Combinations of Several Variances," Journal of the American Statistical Association, Vol. 51, pp. 132-148, 1956.

Williams, H. A., and J. R. Benjamin, "Investigation of Shear Walls, Part 1," Technical Report No. 1, Part 1, Department of Civil Engineering, Stanford University, Stanford, CA, 1952.

Williams, H. A., and J. R. Benjamin, "Investigation of Shear Walls, Part 3," Department of Civil Engineering, Stanford University, Stanford, CA, 1953.

Wiradinata, S., "Behavior of Squat Walls Subjected to Load Reversals," Thesis, University of Toronto, Canada, 1989.

Wiradinata, S., and M. Saatcioglu, "Tests of Squat Shear Wall Under Lateral Load Reversals," Proceedings of the Third U. S. National Conference on Earthquake Engineering, 1986.

Yamada, M., H. Kawamura, and K. Katagihara, "Reinforced Concrete Shear Walls Without Openings; Test and Analysis," ACI Publication SP-42, Paper No. 24, American Concrete Institute, Detroit, MI, 1974.

Yamada, M. H. Kawamura, and K. Katagihara, "Reinforced Concrete Shear Walls with Openings; Test and Analysis," ACI Publication SP-42, Paper No. 25, American Concrete Institute, Detroit, MI, 1974.

Appendix A

Full Reviews

AUTHOR C. Alexander, A. Heidebrecht, W. Tso (1973).
TITLE "Cyclic Load Tests on Shear Wall Panels."
SOURCE Proceedings of the 5th World Conference on Earthquake Engineering.
TEST STRUCTURE DESCRIPTION AND SCALE Five single story isolated shear walls with top beam and foundation beam. 2-D structures.
WALL THICKNESS, ASPECT RATIO t = 4.0 in. (10.2 cm); AR = 0.5, 0.75, and 1.5.
BOUNDARY CONDITIONS (initial stresses) Structure was bolted to test facility floor. No details are given about methods to avoid initial stresses.
TYPE OF LOADING (monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Static-cyclic loads (displacement controlled) applied to the reinforcement in the top beam. Normal load varied (0 - 278 psi).
REINFORCEMENT (type, amount) 0.125 in. (3.2 mm) bars, 0.3% reinforcement, no column steel in ends of the walls.
CONCRETE (properties, aggregate size, curing, etc.) No information given.
INSTRUMENTATION No details given.
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Very few details concerning the experimental portions of this study.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) Panel 4 (AR = 0.75): 0.65% - 0.81%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 2.

AUTHOR Felix Barda (1972).
TITLE "Shear Strength of Low-Rise Walls with Boundary Elements."
SOURCE Ph.D. Dissertation, Lehigh University, Bethlehem, PA.
TEST STRUCTURE DESCRIPTION AND SCALE Eight low-rise shear walls with boundary elements. Scale ~ 1/3.
WALL THICKNESS, ASPECT RATIO AR = 0.25 to 1. t = 4 in.
BOUNDARY CONDITIONS (initial stresses) Base was prestressed to the laboratory floor.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Two specimens subjected to loads in one direction. Six specimens subjected to load reversals. Displacement - controlled loading.
REINFORCEMENT (type, amount) Grade 60 reinforcement (60,000 psi yield). Vertical wall reinforcement: 0.0 to 0.5%.
CONCRETE (properties, aggregate size, curing, etc.) 3000 psi design compressive strength. Maximum aggregate size = 3/4 in. Mix details given.
INSTRUMENTATION Potentiometers used to measure slip at construction joints. Load cells used to measure applied load. Strain gages attached to reinforcement. Lateral deflections were measured with electrical resistance potentiometers and one DCDT. LVDT's used to measure rotation of the top slab.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Quantitative evaluation of the effect of vertical and horizontal shear reinforcement in walls with low aspect ratios; also, the effect of reversals on loading.
COMMENTS Contains deflection information beyond ultimate load.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 0.61% - 0.69% (Monotonic loading) AR = 0.5. 0.53% - 0.75% (cyclic) AR = 0.5. 0.85% - 1.68% (cyclic) AR = 0.25. 0.40% - 0.61% (cyclic) AR = 1.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 1: Very Useful.

AUTHOR
F. Barda, J. M. Hanson, and W. G. Corley (1976).

TITLE
"Shear Strength of Low-Rise Walls with Boundary Elements."

SOURCE
In Reinforced Concrete Structures in Seismic Zones, SP-53, American Concrete Institute, Detroit, MI.

TEST STRUCTURE DESCRIPTION AND SCALE
Shear wall with massive base, two vertical boundary elements and top slab. Scaled, but scale factor unknown.

WALL THICKNESS, ASPECT RATIO
AR = 0.25 to 1.0.
t = 4 in.

BOUNDARY CONDITIONS (initial stresses)
Massive base bolted to floor. Initial stresses probably minimal.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Some static, 2 specimens subjected to cyclic loading. Tests displacement - controlled using two hydraulic rams. Systematic pattern of increasing alternating force or deflection used for the load program.

REINFORCEMENT (type, amount)
Horizontal and vertical wall reinforcement was varied from 0% to 0.5%.
Grade 60 deformed bars: 60,000 psi design yield stress.

CONCRETE (properties, aggregate size, curing, etc.)
"normal-weight" concrete with 3,000 psi compressive strength. (Measured strength was 2,400 to 4,200 psi).
Maximum size of coarse aggregate was 3/4 in.

INSTRUMENTATION
Strain gages were applied to reinforcing bars. Lateral deflections were measured using electrical resistance potentiometer, a dial gage and theodolite sighting.

DEFINITION OF ULTIMATE DRIFT LIMIT
Loading was continued until a deflection of 3" was achieved.

PURPOSE OF TEST (hysteretic model, strength)
To determine the effect of load reversals.

COMMENTS
Shear strength increased significantly with added vertical wall reinforcement. Specimens subjected to load reversals had a shear strength about 10% less than similar specimens subjected to loading in one direction. The load-carrying capacity beyond maximum load was found to depend primarily on the ability of the boundary elements to act as a frame. The frame action provided a gradual, rather than sudden, failure mode.

ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION)
Monotonic: 4.6% (Arbitrarily carried out to 3 in. deflection). Cyclic: 0.6%.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
1: Very useful.

AUTHOR J. R. Benjamin and H. A. Williams (1954).
TITLE "Investigation of Shear Walls, Part 6."
SOURCE Department of Civil Engineering, Stanford University, Stanford, CA, Technical Report No. 4, August 1, 1954.
TEST STRUCTURE DESCRIPTION AND SCALE Four types of specimens loaded in two types of fixtures.
WALL THICKNESS, ASPECT RATIO AR = 0.32 and 0.58. t = 1.75 - 2.00 in.
BOUNDARY CONDITIONS (initial stresses) Two test fixtures.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Monotonic, static loading. Force control initially, then effective displacement control.
REINFORCEMENT (type, amount) Yield point in the range 42,000 psi - 52,000 psi. 1.0 to 1.5% steel ratio.
CONCRETE (properties, aggregate size, curing, etc.) Mix details are provided. Compressive strength in the range 2800 psi - 3800 psi.
INSTRUMENTATION Ames dial for lateral displacement measurement. Strain gages were attached to panel and reinforcement.
DEFINITION OF ULTIMATE DRIFT LIMIT None given.
PURPOSE OF TEST (hysteretic model, strength) Influence of reinforcement on wall strength. (Earlier work used steel ratios of 0, 0.25, and 0.50 percent. The range of steel ratios is extended to 1.0 and 1.5 percent in this report).
COMMENTS Shows influence of steel ratio on UD.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) (AR = 0.32) 0.28% - 0.58%. (AR = 0.58) 0.40% - 0.71%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Monotonic loading only. However, shows influence of steel ratio.

AUTHOR J. R. Benjamin and H. A. Williams (1957).
TITLE "The Behavior of One-Story Reinforced Concrete Shear Walls."
SOURCE Journal of the Structural Division, ASCE, Vol. 83, Paper 1254, pp. 1-49, May 1957. (Also appears as paper No. 2998, ASCE Transactions, Vol. 124, pp. 669-708, 1959).
TEST STRUCTURE DESCRIPTION AND SCALE One-Story plain and reinforced concrete shear walls without openings. Scale: 1/8 to 3/8.
WALL THICKNESS, ASPECT RATIO t = 1 in - 3 in (scaling study). AR = 0.667 (scaling study) t = 2 in. (AR study) AR = 0.32 - 1.25 (AR study)
BOUNDARY CONDITIONS (initial stresses) Two different fixtures used.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Two methods of monotonic, static loading. Load-controlled tests, then combination of load/displacement control following cracking of concrete.
REINFORCEMENT (type, amount) Structural or intermediate grade bars.
CONCRETE (properties, aggregate size, curing, etc.) Type I Portland cement with compressive strengths in the range 2300 psi - 3800 psi.
INSTRUMENTATION Not given.
DEFINITION OF ULTIMATE DRIFT LIMIT None given.
PURPOSE OF TEST (hysteretic model, strength) Investigate aspect ratio and reinforcing; and boundary elements and reinforcing.
COMMENTS No scale effect observed in the range 1/8 to 3/8 scale. Contains the effect of scale and aspect ratio on ductility, at least for monotonic loading.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) (AR = 0.50) 0.43% - 0.75%. (AR = 0.58) 0.30% - 1.03%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 2-3: (Good aspect ratio, but monotonic loading only).

AUTHOR V. V. Bertero (1957)
TITLE "The Response of Shear Walls Subjected to Dynamic Loads."
SOURCE Dissertation, M.I.T., June 1957.
TEST STRUCTURE DESCRIPTION AND SCALE Four specimens identical in every respect except the strength of concrete were tested. Scale 1/4.
WALL THICKNESS, ASPECT RATIO AR = 0.74. t = 2 in.
BOUNDARY CONDITIONS (initial stresses) Load cells at base are used for support,. Deflections are significant.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) 1 wall: Static loading. 3 other walls: Force-time pulse applied (non-cyclic).
REINFORCEMENT (type, amount) Yield point = 39,300 psi. Ultimate strength = 52,000 psi. Reinforcement details given.
CONCRETE (properties, aggregate size, curing, etc.) 3000 psi - 6300 psi concrete strength. Full details of concrete mix are given.
INSTRUMENTATION Load cells for loading and reactions. Deflections measured by LVDT's. (Ames dial deflection gages used on static test.)
DEFINITION OF ULTIMATE DRIFT LIMIT None given.
PURPOSE OF TEST (hysteretic model, strength) Shear wall behavior under dynamic (single-pulse) loads.
COMMENTS
ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION) 0.32% - 1.36%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 4: Non-cyclic loading.

AUTHOR V.V. Bertero et al. (July 1991). All authors are listed on others.
TITLE "Design Guidelines for Ductility and Drift Limits."
SOURCE Report No. UCB/EERC-91/15, University of California at Berkeley, Berkeley, CA.
TEST STRUCTURE DESCRIPTION AND SCALE No experiments.
WALL THICKNESS, ASPECT RATIO No experiments.
BOUNDARY CONDITIONS (initial stresses) No experiments.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) No experiments.
REINFORCEMENT (type, amount) No experiments.
CONCRETE (properties, aggregate size, curing, etc.) No experiments.
INSTRUMENTATION No experiments.
DEFINITION OF ULTIMATE DRIFT LIMIT Seismic-Code-Based.
PURPOSE OF TEST (hysteretic model, strength) No experiments.
COMMENTS Reviews and comments upon drift and interstorey drift index.
ULTIMATE DRIFT RESULTS Interstorey drift index: Acceptable maximum to control damage lies in the range 1-3% (based upon current seismic codes in various countries for ultimate limit states). Mexico DF Code: 0.6% - 1.2%. UBC: 1.125% - 1.5%. BSL: In practice 1%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 4: Contains no experimental data; only seismic code requirements.

AUTHOR
A. E. Cardenas, H. G. Russell, and W. G. Corley (1980).

TITLE
"Strength of Low-Rise Structural Walls."

SOURCE
SP 63-10, American Concrete Institute.

TEST STRUCTURE DESCRIPTION AND SCALE
Seven large specimens. No boundary elements.

WALL THICKNESS, ASPECT RATIO
AR = 1.0.
t = 3 in.

BOUNDARY CONDITIONS (initial stresses)
Thick base block post tensioned to laboratory floor. Rotations near the base of the wall were measured.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Static in-plane horizontal loads. (One specimen subjected to ten cycles of load reversals). No vertical load applied.

REINFORCEMENT (type, amount)
Vertical reinforcement: 0 - 0.5%.
Grade 60 bars. (60,000 psi - 67,500 psi).

CONCRETE (properties, aggregate size, curing, etc.)
Compressive design strength = 6000 psi. (Measured: 5540 psi - 6300 psi). Other details not given.

INSTRUMENTATION
Strain gages placed on reinforcement and concrete. Rotations near the base of the wall were measured with LVDT's.
Load cells were used to monitor loads.

DEFINITION OF ULTIMATE DRIFT LIMIT
None given.

PURPOSE OF TEST (hysteretic model, strength)
Effect of vertical and horizontal reinforcement on strength and deformation.

COMMENTS
Load reversals had very little effect on the load-deflection relationship.

ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION)
1.86% (SW-8). 2.07% (SW-9). 2.27% (SW-12).

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
3: Specimens barely taken to ultimate load.

AUTHOR V. Cervenka and K. Gerstle (1971).
TITLE "Inelastic Analysis of Reinforced Concrete Panels: Experimental Verification and Application."
SOURCE Journal of the International Association of Bridge and Structural Engineering, Vol. 32-II.
TEST STRUCTURE DESCRIPTION AND SCALE Two panels were combined during testing, though panels act independently.
WALL THICKNESS, ASPECT RATIO t = 2 or 3 in. AR = 1.0
BOUNDARY CONDITIONS (initial stresses) Details not given.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Monotonic and cyclic loading histories.
REINFORCEMENT (type, amount) 0.92% to 1.22% reinforcement.
CONCRETE (properties, aggregate size, curing, etc.) Not given.
INSTRUMENTATION Not given.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Load-displacement response; crack patterns and crack propagation; failure mechanisms.
COMMENTS
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 0.73% - 0.93%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Monotonic loading only. Details are limited.

AUTHOR
W. G. Corley, A. E. Fiorato, and R. G. Oesterle (1981).

TITLE
"Structural Walls."

SOURCE
In: Significant Developments in Engineering Practice and Research, SP-72-4, American Concrete Institute, pp. 77-131.

TEST STRUCTURE DESCRIPTION AND SCALE
Flanged, barbell and rectangular cross sections, 15' x 6'.

WALL THICKNESS, ASPECT RATIO
t = 4.0 in (102 mm). AR = 2.40.

BOUNDARY CONDITIONS (initial stresses)
Base block used. Connection of test specimen to base block not given, but looks like specimen may be bolted.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Force applied laterally with hydraulic ram through top slab. Vertical load applied to top slab by hydraulic ram. Three lateral loading histories used (Monotonic and reversed).

REINFORCEMENT (type, amount)
Vertical: 0.20% - 0.31%.

CONCRETE (properties, aggregate size, curing, etc.)
Not given.

INSTRUMENTATION
Not given.

DEFINITION OF ULTIMATE DRIFT LIMIT
Not given.

PURPOSE OF TEST (hysteretic model, strength)
Strength and deformation capacity.

COMMENTS
Paper relates damage to displacements. No correction for Rigid Body Rotations.

ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD)
1.61% - 4.78%.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
3: Aspect ratio of 2.4 puts wall in bending regime.

AUTHOR A. S. Elnashai, K. Pilakoutas, and N. N. Ambraseys (1990).	
TITLE "Experimental Behavior of Reinforced Concrete Walls Under Earthquake Loading."	
SOURCE Earthquake Engineering and Structural Dynamics, Vol. 19, pp. 389-407.	
TEST STRUCTURE DESCRIPTION AND SCALE Nine isolated "flexural" walls. Scale: 1/5, 1/2.5.	
WALL THICKNESS, ASPECT RATIO	t = 32 mm, 60 mm. AR = 2
BOUNDARY CONDITIONS (initial stresses) Base rotations of concrete beam and test rig are shown to be insignificant. Platform rotations are believed to be insignificant.	
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Static cyclic and shake-table loading.	
REINFORCEMENT (type, amount) All reinforcement details given. Walls were designed in pairs, each pair having equal flexural reinforcement but different shear reinforcement.	
CONCRETE (properties, aggregate size, curing, etc.) Same concrete mix is used throughout experiments. Specially manufactured model reinforcement is utilized. Maximum aggregate size = 10 mm. 28-day cube strength = 46N/mm^2.	
INSTRUMENTATION Accelerometers; displacement transducers for measurement of vertical and horizontal accelerations; displacements at top beam; and accelerations at bottom beam. Strain gages at bottom main reinforcement bars. Frequency analyzer for dominant response frequency.	
DEFINITION OF ULTIMATE DRIFT LIMIT	None given.
PURPOSE OF TEST (hysteretic model, strength)	Strength.
COMMENTS Contains comparison of shake-table and static results. Contains literature review, but does not emphasize "squat" walls. Shake table: Shows reduction in natural frequency before and after testing.	
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 0.67% - 1.97%.	
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Aspect ratio a little high, but good data.	

AUTHOR T. Endo (1982).
TITLE "Hysteretic Behavior of Reinforced Concrete Shear Walls."
SOURCE Memoirs of Faculty of Tech. Tokyo Metropolitan University, No. 32, pp. 3195-3206.
TEST STRUCTURE DESCRIPTION AND SCALE Single and multi-story models. Scale 3:1. Boundary elements and thick beams interior to shear walls are included.
WALL THICKNESS, ASPECT RATIO t = 5 cm - 10 cm. AR = 0.5 - 1.0.
BOUNDARY CONDITIONS (initial stresses) Initial stresses possible because of bolting of specimen base to test frame.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Monotonic or cyclic horizontal force, constant vertical force.
REINFORCEMENT (type, amount) 0.23% - 0.70% in wall panel.
CONCRETE (properties, aggregate size, curing, etc.) Not given.
INSTRUMENTATION Strain gages on reinforcing and concrete.
DEFINITION OF ULTIMATE DRIFT LIMIT "Limit Deflection" was defined as the top horizontal deflection at the load step where the horizontal load dropped suddenly or decreased lower than 75% of its maximum.
PURPOSE OF TEST (hysteretic model, strength) Obtain shear force-displacement curves and ductilities of shear walls.
COMMENTS Load-displacement curves are different between monotonic and cyclic loading. Thick beams are contained inside continuous shear walls. Some multi-story models.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) For AR = 0.5: 0.35% - 1.25%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 2: Thick beams are contained inside continuous shear walls.

AUTHOR
F. Esaki, M. Tomii, M. Setoguchi, and Y. Matsuishi (1981).

TITLE
"Statistical Investigation of Angular Shear Distortion of Framed Shear Walls in Which the First Shear Crack Occurred in Panel Walls."

SOURCE
Transactions of the Japan Concrete Institute, Vol. 3, pp. 273-280.

TEST STRUCTURE DESCRIPTION AND SCALE
No tests performed. Data are summarized from numerous other references on drift of shear walls at first shear cracking. All are 1-story, 1-bay, framed shear walls.

WALL THICKNESS, ASPECT RATIO
Not given.

BOUNDARY CONDITIONS (initial stresses)
Not given.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Static shear force.

REINFORCEMENT (type, amount)
Not given.

CONCRETE (properties, aggregate size, curing, etc.)
Not given.

INSTRUMENTATION
Not given.

DEFINITION OF ULTIMATE DRIFT LIMIT
Paper addresses drift at which first shear cracking occurs.

PURPOSE OF TEST (hysteretic model, strength)
Statistical investigation of drift of shear walls at the first shear crack.

COMMENTS
Paper only considers drift at first shear cracking.

ULTIMATE DRIFT RESULTS (AT FIRST SHEAR CRACKING)
Considering data from numerous references probabilistically, the drift at the point of shear cracking has a maximum probability density at .022% for concrete shear walls.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
5: paper only considers crack initiation, not ultimate drift.

AUTHOR C. R. Farrar, J. G. Bennett, W. E. Dunwoody, and W. E. Baker (1989).
TITLE "Static-Load Cycle Testing of a Low-Aspect-Ratio Six-Inch Wall TRG-Type Structure TRG-4-6."
SOURCE NUREG/CR-5222, LA-11422-MS, Los Alamos National Laboratory, Los Alamos, NM.
TEST STRUCTURE DESCRIPTION AND SCALE Reinforced concrete shear wall - TRG structure. "Prototype" structure.
WALL THICKNESS, ASPECT RATIO t = 6 in. AR = 1.0.
BOUNDARY CONDITIONS (initial stresses) Model constructed in place on the base of the load frame.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Static cyclic. Loading by hydraulic actuator. Force-controlled.
REINFORCEMENT (type, amount) 0.25% in each direction.
CONCRETE (properties, aggregate size, curing, etc.) Properties and mix details given. Compressive strength was 3936 psi - 4562 psi.
INSTRUMENTATION Strain gages attached to reinforcement. Relative displacement gages. Fixed reference displacement gages. Force input monitored by load cell.
DEFINITION OF ULTIMATE DRIFT LIMIT Cyclic loading history (force controlled) applied and continued until the structure would no longer hold the applied load.
PURPOSE OF TEST (hysteretic model, strength) Stiffness reduction determination during static cyclic testing.
COMMENTS Relative displacement readings were independent of any rigid body rotation or translation. During the failure cycle, the deformation shown by the external gages is substantially greater than that shown by the internal gages.
ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION) 0.38% - 0.44% UD.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Limited data to failure.

AUTHOR
C. R. Farrar, J. G. Bennett, W. E. Dunwoody, and W. E. Baker (1990).

TITLE
"Static Load Cycle Testing of a Low-Aspect-Ratio Four-Inch Wall, TRG-Type Structure TRG-5-4."

SOURCE
NUREG/CR-5487, LA-11739-MS, Los Alamos National Laboratory, Los Alamos, NM.

TEST STRUCTURE DESCRIPTION AND SCALE
Reinforced concrete shear wall - TRG structure. "Prototype" structure.

WALL THICKNESS, ASPECT RATIO
t = 4 in.
AR = 1.0.

BOUNDARY CONDITIONS (initial stresses)
Structure was bolted to the load frame base in an attempt to obtain a fixed boundary condition.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Static cyclic testing. Loading was by a hydraulic actuator - force controlled.

REINFORCEMENT (type, amount)
See TRG-4 review.

CONCRETE (properties, aggregate size, curing, etc.)
See TRG-4 review.

INSTRUMENTATION
Ono-Sokki displacement transducers to provide relative displacement readings. Load cell used to measure force input. Strain gages used on rebar.

DEFINITION OF ULTIMATE DRIFT LIMIT
None given.

PURPOSE OF TEST (hysteretic model, strength)
Determination of stiffness reduction during static-cycle testing.

COMMENTS
Relative displacement measurements were independent of rigid-body rotation and translation. See TRG-4 review.

ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION)
0.339% - 0.484%. (Based on internal and external gages.)

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
3: Limited failure information.

AUTHOR J. M. Ferritto (July 1982).
TITLE "An Economic Analysis of Earthquake Design Levels."
SOURCE TN No: N-1640, Naval Civil Engineering Laboratory, Port Hueneme, CA.
TEST STRUCTURE DESCRIPTION AND SCALE Shear wall system in which the walls are combined with a steel frame which carries the vertical load. Analysis only, no testing.
WALL THICKNESS, ASPECT RATIO t = 14 in. AR = 0.37. Analysis Only
BOUNDARY CONDITIONS (initial stresses) Analysis only.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Base cyclic acceleration.
REINFORCEMENT (type, amount) Not given as a percentage.
CONCRETE (properties, aggregate size, curing, etc.) Not given - Analysis only.
INSTRUMENTATION Analysis only.
DEFINITION OF ULTIMATE DRIFT LIMIT Provides damage ratios as a function of shear wall interstory drift: A drift of 7% corresponds to complete destruction. 50% destruction occurs at about 2.6% U.D. (See Table 1). This is related to a "cost of repair" based definition of U.D.
PURPOSE OF TEST (hysteretic model, strength) Analysis only.
COMMENTS Analysis only. Details and assumptions of analysis are sketchy. See Ferritto (1983).
ULTIMATE DRIFT RESULTS ("COST OF REPAIR" DEFINITION) 7% - Complete destruction. 2.6% - 50% destruction.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Analysis only -- no testing. Gives drift guidelines. "Damage ratio" not defined.

AUTHOR J. M. Ferritto (July 1983).
TITLE "An Economic Analysis of Earthquake Design Levels for New Concrete Construction."
SOURCE TN No: N-1671, Naval Civil Engineering Laboratory, Port Hueneme, CA.
TEST STRUCTURE DESCRIPTION AND SCALE Shear wall stiffened RC structure. Analysis only, no testing.
WALL THICKNESS, ASPECT RATIO t = not given. A.R. = 0.43.
BOUNDARY CONDITIONS (initial stresses) Analysis only.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Cyclic seismic - Analysis only.
REINFORCEMENT (type, amount) No information given. Analysis only.
CONCRETE (properties, aggregate size, curing, etc.) No information given. Analysis only.
INSTRUMENTATION Analysis only.
DEFINITION OF ULTIMATE DRIFT LIMIT Provides damage ratios as a function of shear wall interstorey drift: A drift of 7% corresponds to complete destruction. 50% destruction occurs at about 2.6% U.D.
PURPOSE OF TEST (hysteretic model, strength) Analysis only.
COMMENTS See Ferritto (1982). Details of analysis sketchy. Analysis only.
ULTIMATE DRIFT RESULTS ("COST OF REPAIR" DEFINITION) 7% - Complete destruction. 2.6% - 50% destruction.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Analysis only. No testing. Gives drift guidelines. "Damage Ratio" not defined.

AUTHOR
A. E. Fiorato, R. G. Oesterle, and J. E. Carpenter (1976).

TITLE
"Reversing Load Tests of Five Isolated Structural Walls."

SOURCE
International Symposium on Earthquake Structural Engineering, St. Louis, MO, August 1976.

TEST STRUCTURE DESCRIPTION AND SCALE
Five structural walls with three cross-section shapes (flanged, bar bell, and rectangular) and different reinforcement. Scale = 1/3.

WALL THICKNESS, ASPECT RATIO
t = 4 in.
AR = 2.40.

BOUNDARY CONDITIONS (initial stresses)
Large base block used to post-tension specimens to test floor using bolts.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
In-plane horizontal reversing loads using hydraulic rams. Load control to yielding; then displacement control.

REINFORCEMENT (type, amount)
Grade 60. Yield strength = 69.4 - 77.2 ksi.
Vertical reinforcement = 0.25 - 0.30%.
Deformed wire was used to represent smaller bar sizes.

CONCRETE (properties, aggregate size, curing, etc.)
Concrete compressive strength = 5575 psi - 7775 psi. Maximum aggregate size of 3/8 in. Mix details are given.

INSTRUMENTATION
Applied loads were measured by load cells. Linear potentiometers and DCDT displacement gages measured horizontal, vertical and diagonal displacements. Strain gages were placed on reinforcement. Dial gages measured relative slip at construction joints.

DEFINITION OF ULTIMATE DRIFT LIMIT
None given.

PURPOSE OF TEST (hysteretic model, strength)
Information on ductility, energy dissipation, and strength.

COMMENTS
Reference planes for displacement measurements were themselves checked for displacements. Rotations in lower portions of specimens were recorded.

ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD)
1.61% - 3.89%.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
3: Aspect ratio high, but good data.

AUTHOR A. E. Fiorato, R. G. Oesterle et al.(1976). Most others are listed.
TITLE "Highlights of an Experimental Investigation of the Seismic Performance of Structural Walls."
SOURCE Proceedings of the ASCE/EMD Specialty Conference, Dynamic Response of Structures, University of California, Los Angeles, CA, March 30, 1976.
TEST STRUCTURE DESCRIPTION AND SCALE Isolated walls (part I). Scale = 1/3.
WALL THICKNESS, ASPECT RATIO AR = 2.5 t = 4 in.
BOUNDARY CONDITIONS (initial stresses) Massive base block, bolted to frame or laboratory floor.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) In-plane lateral reversing loads. Several walls were subjected to monotonic loading.
REINFORCEMENT (type, amount) No details given.
CONCRETE (properties, aggregate size, curing, etc.) No details given.
INSTRUMENTATION No details given.
DEFINITION OF ULTIMATE DRIFT LIMIT No details given.
PURPOSE OF TEST (hysteretic model, strength) Ductility, energy dissipation capacity, and strength.
COMMENTS
ULTIMATE DRIFT RESULTS Paper contains no drift data.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: No drift data.

AUTHOR M. Yamada, H. Kawamura, and K. Katagihara (1974).
TITLE "Reinforced Concrete Shear Walls with Openings; Test and Analysis."
SOURCE SP 42-25, American Concrete Institute, Detroit, MI.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Paper not reviewed because wall contains opening. Companion paper without openings included elsewhere.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR
A. E. Fiorato and W. G. Corley (June 1978).

TITLE
"Laboratory Tests of Earthquake-Resistant Structural Wall Systems and Elements."

SOURCE
Proceedings of "Earthquake-Resistant Reinforced Concrete Building Construction," Berkeley, CA.

TEST STRUCTURE DESCRIPTION AND SCALE
Oesterle (1976): Shear wall with "barbell" cross section.
UC (Bertero): 3 storey; 1/3 - scale.

WALL THICKNESS, ASPECT RATIO
Oesterle (1976): t = 4 in (102 mm); AR = 2.39.
U.C. (Bertero): t = 4 in (102 mm); AR = 1.7 (overall).

BOUNDARY CONDITIONS (initial stresses)
Oesterle (1976): Base block clamped to test floor.
U.C. (Bertero): Base block, connection not given.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Oesterle (1976): Cyclic loading. U.C. (Bertero): Cyclic loading.

REINFORCEMENT (type, amount)
Oesterle (1976): No information given. U.C. (Bertero): No information given.

CONCRETE (properties, aggregate size, curing, etc.)
Oesterle (1976): No information given. U.C. (Bertero): No information given.

INSTRUMENTATION
Oesterle (1976): No information given. U.C. (Bertero): No information given.

DEFINITION OF ULTIMATE DRIFT LIMIT
Oesterle (1976): No information given. U.C. (Bertero): No information given.

PURPOSE OF TEST (Hysteretic model, strength)
Oesterle (1976): Hysteretic and strength. U.C. (Bertero): Hysteretic and strength.

COMMENTS
Provides good summary of types of shear wall tests with advantages/disadvantages.
Oesterle (1976): PCA Tests. No comment on Rigid body rotations.
U.C. (Bertero): 3 storey. No comment on Rigid body rotations.

ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION)
Oesterle (1976): 2.77 - 4.44%.
U.C. Bertero):
Wall specimen 2 2.50%.
Wall specimen 3 5.92%.
Wall specimen 4 3.00%.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
Oesterle (1976): 3: Aspect ratio high.

AUTHOR
G. D. Galletley (1952).

TITLE
"An Experimental and Analytical Investigation of Reinforced Concrete Shear Panels."

SOURCE
M.I.T. Ph.D. Dissertation.

TEST STRUCTURE DESCRIPTION AND SCALE
Twin concrete frames encasing single-story shear walls. 2-D structure.

WALL THICKNESS, ASPECT RATIO
1.75 in. (4.445 cm), 0.72.

BOUNDARY CONDITIONS (initial stresses)
Test setup prevented the introduction of initial stresses from mounting in the test frame.

TYPE OF LOADING (monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Static, monotonic.

REINFORCEMENT (type, amount)
3/8 in. dia. (10 mm), amount varies (see attached sheet), yield strength varies from 43-53.5 ksi (296-367 MPa), no ductility information.

CONCRETE (properties, aggregate size, curing, etc.)
Modulus of elasticity, ultimate compressive strength, and modulus of rupture are tabulated for each specimen on attached sheet. No details of aggregate size, placement, or curing.

INSTRUMENTATION
A single Ames dial gage measured top beam lateral deflection.

DEFINITION OF ULTIMATE DRIFT LIMIT

PURPOSE OF TEST (hysteretic model, strength)

COMMENTS
Ultimate load is not well defined in the plots; data appear to have been stopped just prior to the displacement softening region. Single (nonsymmetric) point loading. Single displacement reading.

ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION)

A-8 = 0.69%	B-8 = 0.75%	C-8 = 0.65%
A-4 = 0.69%	B-4 = 0.82%	C-4 = 0.63%
A-2 = 0.58%	B-2 = 0.98%	C-2 = 1.10%

Specimens A had more frame beam steel than B and C.
Specimens C had smaller frame beam than A and B.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
4.

AUTHOR F. Gantenbein, J. Dalbera, C. Duretz, J. C. Queval, and A. Epstein (1991).
TITLE "Experimental Study on Concrete Shear Wall Behavior Under Seismic Loading."
SOURCE Proceedings of the 11th SMIRT Conference, Tokyo, Japan.
TEST STRUCTURE DESCRIPTION AND SCALE 13 squat shear walls with and without openings were tested up to collapse. Seven were without openings, three with a centered opening, and three with a corner opening.
WALL THICKNESS, ASPECT RATIO t = 0.05 m. AR = 0.5.
BOUNDARY CONDITIONS (initial stresses) Lower beam supporting shear wall "fixed firmly to floor."
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Constant compressive loading. Dynamic loading is applied horizontally using an actuator. Sinusoidal force with linearly increasing envelope was applied. Different frequencies were applied to different walls. One wall was statically loaded.
REINFORCEMENT (type, amount) Welded lattice, leading to a 0.28% steel ratio.
CONCRETE (properties, aggregate size, curing, etc.) Details not given.
INSTRUMENTATION Force and displacement sensors and accelerometers.
DEFINITION OF ULTIMATE DRIFT LIMIT None given.
PURPOSE OF TEST (hysteretic model, strength) Stiffness of walls with and without openings.
COMMENTS
ULTIMATE DRIFT RESULTS None given, except for wall with opening.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not Useful.

AUTHOR
Carlos Graham (1987).

TITLE
"Tests on Short Columns and Structural Walls Under Cyclic Actions."

SOURCE
Darmstadt Concrete, Vol. 2, pp. 89-102.

TEST STRUCTURE DESCRIPTION AND SCALE
Dynamic (earthquake simulator) and cyclic-static tests on a shear wall. Rectangular and "barbell" cross sections. 11 walls.

WALL THICKNESS, ASPECT RATIO
t = 8 cm. AR = 1.5.

BOUNDARY CONDITIONS (initial stresses)
"Specimens are mounted to the base of the steel frame."

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Dynamic Test: Hydraulic system using 'El Centro' loading history. Then harmonic sinusoidal loading to maximum capacity. Cyclic-static: Displacement controlled using hydraulic jack. Forces recorded with load cells. Horizontal and vertical displacements measured at top of wall.

REINFORCEMENT (type, amount)
Horizontal and vertical reinforcement shown, but percentage not given. Horizontal and vertical reinforcement was varied between tests.

CONCRETE (properties, aggregate size, curing, etc.)
Not given.

INSTRUMENTATION
Not given.

DEFINITION OF ULTIMATE DRIFT LIMIT
Not given. Tests were taken to "maximum capacity" (maximum shear force).

PURPOSE OF TEST (hysteretic model, strength)
Hysteretic and strength.

COMMENTS
Relates static-cyclic and dynamic (seismic) loading results. No mention of correction for Rigid body rotations.

ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION)
Wall loaded to maximum capacity (Maximum shear force).
Test TO 6 UD = 1.50%; Test TO 7 UD = 1.42%.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
3: Aspect ratio too high (1.5).

AUTHOR
I. D. Lefas, M. D. Kotsovos, and N. N. Ambraseys (1990) (1).

TITLE
"Behavior of Reinforced Concrete Structural Walls: Strength, Deformation Characteristics, and Failure Mechanism."

SOURCE
ACI Structural Journal, pp. 23-31; January-February 1990.

TEST STRUCTURE DESCRIPTION AND SCALE
Thirteen large scale wall models.

WALL THICKNESS, ASPECT RATIO
AR varies from 1 to 2.
t = 76-70 mm.

BOUNDARY CONDITIONS (initial stresses)
Base rotation values during testing were negligible.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Combined action of a constant axial load and a horizontal load monotonically increasing to failure.

REINFORCEMENT (type, amount)
2.4% - 2.5% vertical web reinforcement. Properties of reinforcement bars given.

CONCRETE (properties, aggregate size, curing, etc.)
Concrete mix given.

INSTRUMENTATION
LVDT's used for deformation response. Strain gages were used to measure steel strains.

DEFINITION OF ULTIMATE DRIFT LIMIT
Drift at ultimate load.

PURPOSE OF TEST (hysteretic model, strength)
Investigate the effects of aspect ratio, axial load, concrete strength and reinforcement on wall behavior.

COMMENTS
Differences in concrete strength as high as 35% resulted in almost negligible variation in wall strength, suggesting strength and deformational characteristics of walls are not significantly affected by variability in concrete strength. Axial load appears to reduce recorded values of horizontal displacement at the ultimate state.

ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD)
0.72% - 1.62% (AR = 2).
0.78% - 1.47% (AR = 1).

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
1: Very Useful.

AUTHOR
I. D. Lefas, M. D. Kotsovos, and N. N. Ambraseys (1990) (1).

TITLE
"Behavior of Reinforced Concrete Structural Walls: Strength, Deformation Characteristics, and Failure Mechanism."

SOURCE
ACI Structural Journal, pp. 23-31, January-February 1990.

TEST STRUCTURE DESCRIPTION AND SCALE
Thirteen large scale wall models.

WALL THICKNESS, ASPECT RATIO
AR varies from 1 to 2.
t = 76-70 mm.

BOUNDARY CONDITIONS (initial stresses)
Base rotation values during testing were negligible.

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Combined action of a constant axial load and a horizontal load monotonically increasing to failure.

REINFORCEMENT (type, amount)
2.4% - 2.5% vertical web reinforcement. Properties of reinforcement bars given.

CONCRETE (properties, aggregate size, curing, etc.)
Concrete mix given.

INSTRUMENTATION
LVDT's used for deformation response. Strain gages were used to measure steel strains.

DEFINITION OF ULTIMATE DRIFT LIMIT
Drift at ultimate load.

PURPOSE OF TEST (hysteretic model, strength)
Investigate the effects of aspect ratio, axial load, concrete strength and reinforcement on wall behavior.

COMMENTS
Differences in concrete strength as high as 35% resulted in almost negligible variation in wall strength, suggesting strength and deformational characteristics of walls are not significantly affected by variability in concrete strength. Axial load appears to reduce recorded values of horizontal displacement at the ultimate state.

ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD)
0.72% - 1.62% (AR = 2).
0.78% - 1.47% (AR = 1).

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
1: Very Useful.

AUTHOR K. Ogata and T. Kabeyasawa (1984).
TITLE "Experimental Study on the Hysteretic Behavior of Reinforced Concrete Shear Walls Under the Loading of Different Moment-to-Shear Ratios."
SOURCE Transactions of the Japan Concrete Institute, Vol. 6, pp. 717-724.
TEST STRUCTURE DESCRIPTION AND SCALE Six shear wall assemblies with boundary columns, in which the ratio and the detail of reinforcement were varied; 2/5 scale.
WALL THICKNESS, ASPECT RATIO t = 8 cm. AR = 0.94.
BOUNDARY CONDITIONS (initial stresses) Base plate (massive) bolted to fixture. Possibility of initial stresses.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Cyclic horizontal loading with actuator; Constant vertical load. Variable moment/shear loading applied by using the two vertical oil jacks to apply bending moment according to shear force level. Tests appear to be displacement-controlled.
REINFORCEMENT (type, amount) Wall panel shear reinforcement varies from .27% to .83%.
CONCRETE (properties, aggregate size, curing, etc.) Concrete compressive strength = ~ 200 Kg/cm^2. Tensile strength = ~ 16 Kg/cm^2.
INSTRUMENTATION Strain gages on reinforcing bars; Displacement gages at various locations for measuring displacements relative to the base and local deformations.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Hysteresis and energy dissipation capabilities of shear walls with different flexural and shear reinforcements and loading with different moment-to-shear ratio.
COMMENTS First paper that independently controls moment and shear loading of shear wall.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 1% in all cases (displacement-controlled).
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 1: Very Useful.

AUTHOR T. Paulay (1972).
TITLE "Some Aspects of Shear Wall Design."
SOURCE Bulletin of N.Z. Society for Earthquake Engineering
TEST STRUCTURE DESCRIPTION AND SCALE Three single-story isolated shear walls. 2-D structures.
WALL THICKNESS, ASPECT RATIO 6.0 in. (15.2 cm), 1.0.
BOUNDARY CONDITIONS (initial stresses) Loads are applied in a manner such that initial stresses are not introduced.
TYPE OF LOADING (monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Static-cyclic loads (displacement controlled) applied to the reinforcement in the top beam.
REINFORCEMENT (type, amount) 0.20 - 0.31% reinforcement.
CONCRETE (properties, aggregate size, curing, etc.) No information given.
INSTRUMENTATION No details given.
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Very few details concerning the experimental portions of this study.
ULTIMATE DRIFT RESULTS AT MAXIMUM REPORTED DEFLECTION) Wall 201 = 0.95% Wall 202 = 1.24% Wall 203 = 1.24%
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3.

AUTHOR T. Paulay, M. J. N. Priestley, and A. J. Synge (1982).
TITLE "Ductility in Earthquake Resisting Squat Shearwalls."
SOURCE ACI Journal
TEST STRUCTURE DESCRIPTION AND SCALE Four single-story shear walls with top beam and foundation beam. Two of the structures were built with flange walls at either end of the shear wall. Construction joint at the top of the foundation beam. 2-D structure.
WALL THICKNESS, ASPECT RATIO 3.94 in. (10 cm), 0.5.
BOUNDARY CONDITIONS (initial stresses) Structures appear to have been bolted to a load frame or strong-floor, but details of tests setup are not given.
TYPE OF LOADING (monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Static-cyclic point load (displacement controlled) applied to the top beam.
REINFORCEMENT (type, amount) Two structures had diagonal reinforcement. Range of yield strengths are given.
CONCRETE (properties, aggregate size, curing, etc.) Ultimate compressive strength given.
INSTRUMENTATION No details given.
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Very few details concerning the experimental portions of this study.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) Wall 1 = 0.67. Wall 2 = 0.59 (diagonal reinforcement). Wall 3 = 0.39 (flanged).
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 1.

AUTHOR Murat Saatcioglu (1991).
TITLE Hysteretic Shear Response of Low-Rise Walls.
SOURCE Concrete Shear in Earthquake, Proceedings of the International Workshop on Concrete Shear in Earthquake, University of Houston, Houston, Texas, Elsevier, 1991.
TEST STRUCTURE DESCRIPTION AND SCALE Large scale, low rise shear walls. 3 specimens tested. (Wall 1 was previously reported - See Wiradinata [1985]).
WALL THICKNESS, ASPECT RATIO AR = 0.5 t = 100 mm
BOUNDARY CONDITIONS (initial stresses) Foundation beam was bolted to the laboratory strong floor.
TYPE OF LOADING (monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Slowly applied lateral load reversals. Displacement-controlled. Load applied by a hydraulic jack, supported by a reaction frame.
REINFORCEMENT (type, amount) Vertical and horizontal reinforcement both 0.8%.
CONCRETE (properties, aggregate size, curing, etc.) f_c^1 = 33-35 MPa
INSTRUMENTATION LVDT's and strain gages were used to measure deformations caused by shear, flexure, bar extension and sliding shear.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Shear response of walls, particularly with respect to different modes of shear behavior. Also, characteristics of hysteretic shear response.
COMMENTS Continuation of work by Wiradinata (1985). Wall 6 had special sliding shear reinforcement.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) Wall 4 - Large portion of lateral deflection due to shear sliding. 0.80% - 1.63%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 1 - Very Useful.

AUTHOR
T. Shiga, A. Shibata, and J. Takahashi (1973).

TITLE
"Experimental Study on Dynamic Properties of Reinforced Concrete Shear Walls"

SOURCE
Proc. 5th World Conference on Earthquake Engineering.

TEST STRUCTURE DESCRIPTION AND SCALE
Eight single-story isolated shear walls with boundary column, top beam and deep foundation beam. 2-D structures.

WALL THICKNESS, ASPECT RATIO
1.97 in. (5 cm.), 0.68.

BOUNDARY CONDITIONS (initial stresses)
Structure was bolted to the test facility floor. No details are given about methods to avoid initial stresses.

TYPE OF LOADING (monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
Static-cyclic loads (displacement controlled) applied to the reinforcement in the top beam. Axial load applied to boundary columns.

REINFORCEMENT (type, amount)
0.125 in. (3.2 mm) bars in walls, 0.25% and 0.5% reinforcement, yield and ultimate strength given.

CONCRETE (properties, aggregate size, curing, etc.)
3/8 in. 10 mm aggregate; average ultimate compressive strength given.

INSTRUMENTATION
Mechanical dial gages with a resolution of 0.01 mm.

DEFINITION OF ULTIMATE DRIFT LIMIT

PURPOSE OF TEST (hysteretic model, strength)

COMMENTS
Very few details concerning the experimental portions of this study.

ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD)
WB-1 (0.25% reinforcement, no normal force) = 0.40%.
WB-2 (0.25% reinforcement, no normal force) = 0.38% - 0.41%.
WB-6 (0.50% reinforcement, no normal force) = 0.39% - 0.40%.
WB-7 (0.50% reinforcement, normal force = 20 tons) = 0.39%.
WB-8 (0.50% reinforcement, normal force = 40 tons) = 0.40%.

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
1. Very Useful.

AUTHOR T. Shiga, A. Shibata, and J. Takahashi (1976).
TITLE "Hysteretic Behavior of Reinforced Concrete Shear Walls."
SOURCE Proceedings of the Review Meeting U.S.-Japan Cooperative Research Program in Earthquake Engineering, August 18-20, 1975, Honolulu, pp. 107-117.
TEST STRUCTURE DESCRIPTION AND SCALE 17 "Medium-size" shear walls subjected to static loads representing gravity loads and earthquake forces (Cyclic loading tests).
WALL THICKNESS, ASPECT RATIO t = 50 mm.   A.R. = 0.6.
BOUNDARY CONDITIONS (initial stresses) Concrete base block "firmly fastened" to rigid testing floor. Wall cast into base block. Should be no initial stresses.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Cyclic loading. Oil jacks used for loading. Different cyclic loading programs used. Displacement-controlled tests.
REINFORCEMENT (type, amount) 0.25% - 0.50% (Same in vertical and horizontal directions).
CONCRETE (properties, aggregate size, curing, etc.) Not given.
INSTRUMENTATION Load cell, mechanical dial gage.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Hysteretic behavior when subjected to various loading histories.
COMMENTS Total axial load was varied in tests.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 0.52% - 0.70%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 2: Good aspect ratio. Might give influence of axial load.

AUTHOR Boris Simeonov (1984).
TITLE "Experimental Investigation of Strength and Deformation of Reinforced Concrete Structural Walls."
SOURCE Proceedings of the Eighth World Conference on Earthquake Engineering, San Francisco, pp. 387-394.
TEST STRUCTURE DESCRIPTION AND SCALE Scale: 1 to 3. Lower 3 stories of a 9-storey building. Flanged and rectangular wall cross sections. Loading by hydraulic jacks.
WALL THICKNESS, ASPECT RATIO t = 7 cm. AR = 1.63 (total of 3 shear walls).
BOUNDARY CONDITIONS (initial stresses) Details not given.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Cyclic horizontal load with constant vertical load and moment.
REINFORCEMENT (type, amount) 0.16 - 0.28% vertical reinforcement.
CONCRETE (properties, aggregate size, curing, etc.) Not given.
INSTRUMENTATION Not given.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Hysteretic model and strength.
COMMENTS Paper difficult to read. No discussion of Rigid body rotations.
ULTIMATE DRIFT RESULTS (AT MAXIMUM REPORTED DEFLECTION) DFM1E2 = 0.82%. DFM1E3 = 1.18%. DFM1E4 = 0.99%. DFM3E1 = 0.87%. DFM2E2 = 0.84%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 4: Aspect ratio too high. Also, 3 stories.

AUTHOR R. H. Stivers (1955).
TITLE "Stresses and Deflections in Reinforced Concrete Shear Walls Containing Rectangular Openings.
SOURCE Ph.D. Dissertation, Stanford University.
TEST STRUCTURE DESCRIPTION AND SCALE Eleven walls tested in shear fixture.
WALL THICKNESS, ASPECT RATIO t = 2.0 in. AR = 1.73.
BOUNDARY CONDITIONS (initial stresses) Not given.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Monotonic, displacement controlled.
REINFORCEMENT (type, amount) 0.5% reinforcing in all walls.
CONCRETE (properties, aggregate size, curing, etc.) 3000 psi concrete (all tests) using Portland Cement plus Tricosol admixture. Mix and property details in paper.
INSTRUMENTATION Ames dial gage for lateral movement.
DEFINITION OF ULTIMATE DRIFT LIMIT Based on deflection at ultimate (peak) load.
PURPOSE OF TEST (hysteretic model, strength) Strength.
COMMENTS Monotonic loading only. Samples had rectangular cutouts.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) H-3: 0.45%. H-4: 0.95%. H-5: 0.55%
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Because monotonic loading and test samples had cutouts.

AUTHOR H. Tanaka, K. Imoto, S. Yoshizaki, K. Emori, Y. Inada, and H. Nanba (1988).
TITLE "An Evaluation Method for Restoring Force Characteristics of R/C Shear Walls of Reactor Buildings."
SOURCE Proceedings of Ninth World Conference on Earthquake Engineering, August 2-9, 1988, Tokyo-Kyoto, Japan, Vol. VI, pp. 747-752.
TEST STRUCTURE DESCRIPTION AND SCALE Summarizes data from other papers. Includes box walls, cylindrical walls, truncated conical walls and I-shaped section walls. Papers reviewed were 1/10 to 1/30 scale.
WALL THICKNESS, ASPECT RATIO t = 5 to 10 cm.
BOUNDARY CONDITIONS (initial stresses) Not given.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Not given.
REINFORCEMENT (type, amount) Most less than 1.2%.
CONCRETE (properties, aggregate size, curing, etc.) Not given.
INSTRUMENTATION Not given.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given for I-shaped section shear walls.
PURPOSE OF TEST (hysteretic model, strength) Provide a method of determining restoring-force characteristics of shear walls for nonlinear dynamic response analysis.
COMMENTS Reviews existing shear wall experimental results.
ULTIMATE DRIFT RESULTS Not given.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Does not discuss UD.

AUTHOR M. Tomii and M. Takeuchi (1968).
TITLE "The Relations Between the Deformed Angle and the Shearing Force Ratio (0.80 ~ 1.00) with Regard to 200 Shear Walls.
SOURCE Trans. of A.I.J., No. 153, Nov. 1968.
TEST STRUCTURE DESCRIPTION AND SCALE 200 statically tested shear walls without openings.
WALL THICKNESS, ASPECT RATIO Varies - Not given.
BOUNDARY CONDITIONS (initial stresses) Vary - Not given.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Does not include dynamic load tests. Unclear whether cyclic data are included.
REINFORCEMENT (type, amount) Varies - Not given.
CONCRETE (properties, aggregate size, curing, etc.) Varies - Not given.
INSTRUMENTATION Varies - Not given.
DEFINITION OF ULTIMATE DRIFT LIMIT UD occurs when ultimate (maximum) load is reached.
PURPOSE OF TEST (hysteretic model, strength) Relation between drift and shearing force ratio for a large number of tests reported by others.
COMMENTS Statistical investigation of relation between drift and shearing force ratio.
ULTIMATE DRIFT RESULTS Results presented probabilistically. Mean UD for concrete shear walls at ultimate load is ~ 0.4%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 1: Very useful for comparison with results. Provides a possible statistical approach for representing the data.

AUTHOR H. A. Williams and J. R. Benjamin (1952).
TITLE "Investigation of Shear Walls, Part 1."
SOURCE Technical Report No. 1, Part 1, Department of Civil Engineering, Stanford University, Stanford, CA.
TEST STRUCTURE DESCRIPTION AND SCALE Single shear walls enclosed by frames. Scale: Not given.
WALL THICKNESS, ASPECT RATIO AR = 0.71. t = 1.75 in.
BOUNDARY CONDITIONS (initial stresses) Initial stresses minimized by using a symmetric loading fixture.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Static loading, monotonic.
REINFORCEMENT (type, amount) Yield point = 41,000 psi - 61,000 psi.
CONCRETE (properties, aggregate size, curing, etc.) Compressive strength 3230 psi - 5100 psi. Aggregate, mix, and curing details given in Appendix A.
INSTRUMENTATION Lateral deflections determined using Ames dial gages. Strain gages used on reinforcing bars. Some strain gages were cemented to concrete panels.
DEFINITION OF ULTIMATE DRIFT LIMIT None given.
PURPOSE OF TEST (hysteretic model, strength) Strength-deflection characteristics of shear walls.
COMMENTS
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 0.16% - 0.78%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Monotonic loading only.

AUTHOR H. A. Williams and J. R. Benjamin (1953).
TITLE "Investigation of Shear Walls, Part 3."
SOURCE Department of Civil Engineering, Stanford University, Stanford, CA.
TEST STRUCTURE DESCRIPTION AND SCALE Two types of test structures used. Two models were 3/8 scale.
WALL THICKNESS, ASPECT RATIO AR = 0.32 - 1.25. t = 1 in., 2 in.
BOUNDARY CONDITIONS (initial stresses) See Part 1.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Monotonic, static, load controlled and displacement controlled. Vertical load is applied in some cases.
REINFORCEMENT (type, amount) Intermediate grade bars (45,300 psi-49,500 psi yield). 0 - 0.5% reinforcement.
CONCRETE (properties, aggregate size, curing, etc.) Properties, mix details given on pg. 9. Nominal 3000 psi concrete used.
INSTRUMENTATION Ames dials for measuring lateral deflections of specimens. Strain gages were attached to the wall panel and reinforcing steel.
DEFINITION OF ULTIMATE DRIFT LIMIT Not given.
PURPOSE OF TEST (hysteretic model, strength) Investigation of certain definite items influencing shear wall behavior (Aspect ratio, reinforcement, normal and shear loads).
COMMENTS Contains drift information as a function of aspect ratio and amount of reinforcement. The scale effect is isolated experimentally. Effect of vertical load is not significant. Gives empirical formula for displacement past ultimate load.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) (AR = 1.25) 0.44% - 1.30%. (AR = 0.71) 0.16% - 0.95%. (AR = 0.69) 0.73% - 1.51%. (AR = 0.58) 0.28% - 0.46%. (AR = 0.50) 0.51% - 0.53%. (AR = 0.32) 0.28% - 0.59%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 2: In spite of the fact that loading is monotonic, the role of reinforcement and, particularly, aspect ratio can be isolated from the data. Also, the influence of the frame can be isolated from the data.

2 of 2

AUTHOR S. Wiradinata and M. Saatcioglu (1986).
TITLE "Tests of Squat Shear Wall Under Lateral Load Reversals."
SOURCE Proceedings of the Third U. S. National Conference on Earthquake Engineering.
TEST STRUCTURE DESCRIPTION AND SCALE Two squat walls with rectangular cross-sections. Massive beams on top and bottom of wall. Large Scale.
WALL THICKNESS, ASPECT RATIO t = 100 mm. AR = 0.5, 0.25.
BOUNDARY CONDITIONS (initial stresses) Specimens post-tensioned to the laboratory strong floor.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Cyclic, applied uniformly through top beam. Horizontal load applied using hydraulic jacks. Displacement-controlled horizontal load cycles. Complicated load cycling program.
REINFORCEMENT (type, amount) 0.8% vertical and 0.25% horizontal reinforcement. Yield strength of reinforcement: Vertical: 63,000 psi; Horizontal: 61,600 psi.
CONCRETE (properties, aggregate size, curing, etc.) 3200 psi-3600 psi concrete strength.
INSTRUMENTATION Vertical and horizontal deflections measured by LVDT's and strain gages placed on selective reinforcement. A "Zurich" gage was used for shear deformations on the concrete surface.
DEFINITION OF ULTIMATE DRIFT LIMIT Magnitude of deformations were increased until a significant drop was observed in the load resistance of the specimen.
PURPOSE OF TEST (hysteretic model, strength) Hysteretic, failure modes.
COMMENTS On one specimen, sliding of the wall with respect to the foundation beam was significant. Failure mode of a squat-wall subjected to reversed cyclic loading is affected by Aspect Ratio. Walls with low AR's may fail in sliding shear prior to flexure or diagonal tension or compression failures.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 1.04% - 1.12% (AR = 0.5). 0.50% - 0.66% (AR = 0.25).
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 1: Very Useful.

AUTHOR M. Yamada, H. Kawamura, and K. Katagihara (1974).
TITLE "Reinforced Concrete Shear Walls Without Openings; Test and Analysis."
SOURCE SP 42-24, American Concrete Institute.
TEST STRUCTURE DESCRIPTION AND SCALE RC shear walls in a rectangular reinforced concrete rigid loading frame. Scale models: 1/5.
WALL THICKNESS, ASPECT RATIO AR = 0.44. t = 20, 30, 40 mm.
BOUNDARY CONDITIONS (initial stresses) None, by design of load frame.
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.) Monotonic static loading to failure. Loading by hydraulic jack oriented diagonally. Constant vertical loading also applied.
REINFORCEMENT (type, amount) 4 mm dia steel bars. Stress-strain properties given. Reinforcement ratios of 0%, 0.31%, 0.63%, 1.26% were tested. Yield stress: 2920 kg/cm^2 - 3360 kg/cm^2.
CONCRETE (properties, aggregate size, curing, etc.) High quality Portland cement and aggregates with a maximum size of 15 mm, a mix proportion of 1:2.55:3.34 by weight, with water-cement ratio of 60%. Mechanical properties given. fc^1: 307 kg/cm^2 - 363 kg/cm^2.
INSTRUMENTATION Dial gages used to measure lateral deflections.
DEFINITION OF ULTIMATE DRIFT LIMIT None given.
PURPOSE OF TEST (hysteretic model, strength) To determine effects of web reinforcement ratio and panel thickness on deformation.
COMMENTS Paper shows influence of web reinforcement ratio on maximum resistance and drift at maximum resistance.
ULTIMATE DRIFT RESULTS (AT ULTIMATE LOAD) 0.32% - 0.62%.
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 3: Because not cyclic loading.

Appendix B

Brief Reviews

AUTHOR
J. Antebi (1961).

TITLE
"Model Analysis of the Response of Shear Walls to Dynamic Loads."

SOURCE
Thesis, Massachusetts Institute of Technology.

TEST STRUCTURE DESCRIPTION AND SCALE
Same as Bertero (1957). Scale modeling study performed.

WALL THICKNESS, ASPECT RATIO

BOUNDARY CONDITIONS (initial stresses)

TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)

REINFORCEMENT (type, amount)

CONCRETE (properties, aggregate size, curing, etc.)

INSTRUMENTATION

DEFINITION OF ULTIMATE DRIFT LIMIT

PURPOSE OF TEST (hysteretic model, strength)
Develop and verify modeling techniques for reduced scale structures under dynamic loads.

COMMENTS
Paper is based on load frame and some of the tests reported by Bertero (1957). Since single-pulse loading, detailed review not performed.

ULTIMATE DRIFT RESULTS

PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful)
5: Not Useful.

AUTHOR J. R. Benjamin and H. A. Williams (1955).
TITLE "Investigation of Shear Walls, Part 9."
SOURCE Department of Civil Engineering, Stanford University, Stanford, CA, September 1955.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Report focuses on reinforcement around openings in shear walls, variations in panel reinforcing, and combined normal and shear loading. Only combined loading would be of interest here. Because no drift data are presented for combined loading, this report is not reviewed.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR J. R. Benjamin and H. A. Williams (1960).
TITLE "Reinforced Concrete Shear Wall Assemblies."
SOURCE Journal of the Structural Division, ASCE, Vol. 86, pp. 1-32, August 1960.
TEST STRUCTURE DESCRIPTION AND SCALE One-storey and two-storey shear wall assemblies. Four one-storey models had parallel shear walls (2 or 3). Three individual shear walls were first tested as controls. Scale = 1/4.
WALL THICKNESS, ASPECT RATIO t = 2 in. AR = 0.875 (Approx.).
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Paper not reviewed because displacements not taken to ultimate load.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not Useful.

AUTHOR J. P. Moehle, M. A. Sozen, and H. T. Tang (1990).
TITLE "Concrete Wall Stiffness: Calculation vs. Measurement."
SOURCE Proceedings of the Third Symposium on Current Issues Related to Nuclear Power Plant Structures, Equipment and Piping, Orlando, FL, December 1990 (Ed: A. K. Gupta).
TEST STRUCTURE DESCRIPTION AND SCALE Low rise reinforced concrete shear walls.
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Paper focuses on a comparison of lateral load stiffnesses measured by others with calculated stiffness values. Paper focuses on working stress levels and contains little drift limit information. Therefore, paper not reviewed.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not Useful.

AUTHOR R. G. Oesterle, A. E. Fiorato, and J. D. Aristizabal-Ochoa (1980).
TITLE "Free Vibration Tests of Structural Concrete Walls and Analysis of Free Vibration Tests of Structural Walls."
SOURCE NSF/RA - 800043, Portland Cement Association, Skokie, IL.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Free vibration testing of shear walls only. No drift limit information given, so paper not reviewed.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR R. G. Oesterle, A. E. Fiorato, J. D. Aristizabal-Ochoa, and W. G. Corley (1980).
TITLE "Hysteretic Response of Conrete Structural Walls."
SOURCE ACI, SP 63, 1980.
TEST STRUCTURE DESCRIPTION AND SCALE 16 large structural walls. Flanged, bar bell, and rectangular cross sections. Scale = 1/3.
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Paper not useful to present study as results are not presented in terms of drift. Only shear/bending distortions presented.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR Thomas Paulay (1978)
TITLE "Earthquake Resistant Structural Walls."
SOURCE Proceedings of "Earthquake-Resistant Reinforced Concrete Building Construction," Berkely, CA, June 1978.
TEST STRUCTURE DESCRIPTION AND SCALE No testing - review paper only.
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (yysteretic model, strength)
COMMENTS
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Contains only incomplete data reported elsewhere.

AUTHOR H. Umemura, H. Aoyama, and Y. Hosokawa (1980).
TITLE "Restoring Force Characteristics of RC Walls with Openings and Reinforcing Methods."
SOURCE Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, pp. 209-216.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Paper not reviewed, because structure was box shape -- no conventional shear walls.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR H. Umemura, H. Aoyama, and Y. Hosokawa (1980).
TITLE "Restoring Force Characteristics of RC Walls with Openings and Reinforcing Methods."
SOURCE Proceedings of the Seventh World Conference on Earthquake Engineering, Istanbul, pp. 209-216.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Paper not reviewed, because structure was box shape -- no conventional shear walls.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR F. Wang, F. Gantenbein, J. Dalbera, and C. Duretz (1989)
TITLE "Seismic Behavior of Reinforced Concrete Shear Walls."
SOURCE SMIRT 89, Anaheim, CA.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Analysis only. Contains no relevant experimental data.
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not useful.

AUTHOR S. Wiradinata (1985).
TITLE "Behavior of Squat Walls Subjected to Load Reversals."
SOURCE Thesis, University of Toronto, Canada.
TEST STRUCTURE DESCRIPTION AND SCALE
WALL THICKNESS, ASPECT RATIO
BOUNDARY CONDITIONS (initial stresses)
TYPE OF LOADING (Monotonic, cyclic, static, dynamic, force or displacement controlled, etc.)
REINFORCEMENT (type, amount)
CONCRETE (properties, aggregate size, curing, etc.)
INSTRUMENTATION
DEFINITION OF ULTIMATE DRIFT LIMIT
PURPOSE OF TEST (hysteretic model, strength)
COMMENTS Contains further details of the shear wall study reported in S. Wiradinata and M. Saatcioglu (1986).
ULTIMATE DRIFT RESULTS
PAPER USEFULNESS TO OUR STUDY (1 = Very Useful; 5 = Not Useful) 5: Not Useful.

NRC FORM 335 (2-89) NRCM 1102, 3201, 3202

U.S. NUCLEAR REGULATORY COMMISSION

BIBLIOGRAPHIC DATA SHEET

(See instructions on the reverse)

1. REPORT NUMBER (Assigned by NRC. Add Vol., Supp., Rev., and Addendum Numbers, if any.)

NUREG/CR-6104
LA-12649-MS

2. TITLE AND SUBTITLE

Shear Wall Ultimate Drift Limits

3. DATE REPORT PUBLISHED

MONTH	YEAR
April	1994

4. FIN OR GRANT NUMBER

L2506

5. AUTHOR(S)

T.A. Duffey[1], A. Goldman[2], and C.R. Farrar

6. TYPE OF REPORT

Technical

7. PERIOD COVERED (Inclusive Dates)

8. PERFORMING ORGANIZATION – NAME AND ADDRESS (If NRC, provide Division, Office or Region, U.S. Nuclear Regulatory Commission, and mailing address; if contractor, provide name and mailing address.)

Los Alamos National Laboratory
Los Alamos, NM 87545

[1]Consulting Engineer, P.O. Box 1239
Tijeras, NM 87059
[2]Consulting Statistician, 4723 Sandia
Los Alamos, NM 87544

9. SPONSORING ORGANIZATION – NAME AND ADDRESS (If NRC, type "Same as above"; if contractor, provide NRC Division, Office or Region, U.S. Nuclear Regulatory Commission, and mailing address.)

Division of Engineering
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

10. SUPPLEMENTARY NOTES

11. ABSTRACT (200 words or less)

Drift limits for reinforced-concrete shear walls are investigated by reviewing the open literature for appropriate experimental data. Drift values at ultimate load are determined for walls with aspect ratios ranging up to a maximum of 3.53 and undergoing different types of lateral loading (cyclic static, monotonic static, and dynamic).

Based on the geometry of actual nuclear power plant structures exclusive of containments and concerns regarding their response during seismic (i.e.,cyclic) loading, data are obtained from pertinent references for which the wall aspect ratio is less than or equal to approximately 1, and for which testing is cyclic in nature (typically displacement controlled). In particular, lateral deflections at ultimate load, and at points in the softening region beyond ultimate for which the load has dropped to 90, 80, 70, 60, and 50 percent of its ultimate value, are obtained and converted to drift information.

The statistical nature of the data is also investigated. These data are shown to be lognormally distributed, and an analysis of variance is performed. The use of these statistics to estimate Probability of Failure for a shear wall structure is illustrated.

12. KEY WORDS/DESCRIPTORS (List words or phrases that will assist researchers in locating the report.)

Shear Wall, Drift Limit, Seismic,
Probabilistic Risk Assessment

13. AVAILABILITY STATEMENT

Unlimited

14. SECURITY CLASSIFICATION

(This Page)

Unclassified

(This Report)

Unclassified

15. NUMBER OF PAGES

16. PRICE