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This report describes experimental free vibration tests and results conducted during lateral load tests to determine frequency and damping characteristics of isolated wall specimens. This study is part of an experimental and analytical investigation of structural walls for earthquake-resistant buildings in which large isolated reinforced concrete wall specimens are tested under reversing in-plane lateral loads. Initial tests were conducted on specimens before applying lateral loads, whereas final tests were performed on specimens that had been cycled through large inelastic deformations. The report details test specimens, the test procudure, and test results. Small amplitude free vibration tests of isolated structural walls indicate that frequency and damping characteristics are sensitive to the development of structural cracks in the walls. With increasing damage levels, frequency decreases and damping increases. These observations need to be considered in analyzing the dynamic response of reinforced concrete wall systems.

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FREE VIBRATION TESTS OF STRUCTURAL CONCRETE WALLS

and

ANALYSIS OF FREE VIBRATION TESTS OF STRUCTURAL WALLS

by

R. G. Oesterle and A. E. Fiorato

and

J. D. Aristizabal-Ochoa

Submitted by
CONSTRUCTION TECHNOLOGY LABORATORIES
A Division of Portland Cement Association
5420 Old Orchard Road
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February 1980

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FREE VIBRATION TESTS OF STRUCTURAL CONCRETE WALLS

by

R. G. Oesterle, and A. E. Fiorato*

INTRODUCTION

As part of an experimental and analytical investigation of structural walls for earthquake-resistant buildings, large isolated reinforced concrete wall specimens have been tested under reversing in-plane lateral loads. Free vibration tests were carried out during the lateral load tests to determine the frequency and damping characteristics of isolated wall specimens. These tests were conducted at selected stages as the number and magnitude of the reversed lateral load cycles applied to the specimen were increased.

The objective of the free vibration tests was to evaluate changes in natural frequency and damping that resulted from damage caused by the reversing lateral loads. Initial tests were conducted on specimens before applying the lateral loads. Final tests were conducted on specimens that had been cycled through large inelastic deformations.

The purpose of this paper is to describe the free vibration tests and to present the test results. Analysis of the test results is presented in a companion paper.(1) A detailed description of the experimental program is given elsewhere.(2,3)

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* Respectively, Senior Structural Engineer, Structural Development Department; Manager, Construction Methods Section; Portland Cement Association, Skokie, Illinois

EXPERIMENTAL PROGRAM

This section describes the test specimens and test procedure.

Test Specimens

Test specimens were detailed to represent full-size walls although no particular prototype walls were modeled. Table 1 provides a summary of physical and material properties for the walls subjected to free vibration tests. Controlled variables for these specimens included shape of the wall cross section, amount of main flexural reinforcement, and confinement reinforcement in the boundary elements. One wall was subjected to monotonic loading and one wall was repaired and retested.

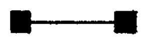
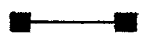
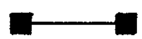
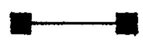
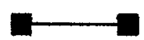
Dimensions of the test specimens are shown in Fig. 1. Flanged, barbell, and rectangular cross sections have been investigated. Nominal cross sectional dimensions are shown in Fig. 2. The types of reinforcement used in the specimens are shown in Fig. 3.

In proportioning the walls, design moment was calculated following procedures in the 1971 ACI Building Code.[4] Strain hardening of the steel was neglected. Horizontal shear reinforcement was provided so that the calculated design moment would be developed. Shear reinforcement was provided to satisfy the ACI Building Code requirements.[4] Design yield stress of the steel was 60,000 psi (414 MPa) and design concrete strength was 6000 psi (41.4 MPa).

Transverse reinforcement around vertical reinforcement in the boundary elements was designed either as ordinary column ties (unconfined) or as special confinement reinforcement (confined). For rectangular sections, the "boundary element" was taken to extend 7.5 in. (190 mm) from each end of the wall.

Specimens F1, B1, B2, and R1 had ordinary ties as required by Section 7.12 of the 1971 ACI Building Code.[4] All other specimens had rectangular hoop and supplementary cross-tie reinforcement proportioned to meet requirements of Appendix A of the 1971 ACI Building Code.[4] This design resulted in a

TABLE 1 - SUMMARY OF TEST SPECIMENS

Specimen	Shape	Reinforcement (%)				f'_c (psi)	E_c (ksi)	f_y for ρ_f (ksi)
		ρ_f	ρ_h	ρ_n	ρ_s			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
F1		3.89	0.71	0.30	--	5580	3690	64.5
B1		1.11	0.31	0.29	--	7690	4080	65.2
B2		3.67	0.63	0.29	--	7780	4200	59.5
B3		1.11	0.31	0.29	1.28	6860	3960	63.5
B4[(a)]		1.11	0.31	0.29	1.28	6530	4100	65.3
B5		3.67	0.63	0.29	1.35	6570	3970	64.4
B5R[(b)]		1.11	0.31	0.29	1.28	6200	4010	--
R1		1.47	0.31	0.25	--	6490	4030	74.2
R2		4.00	0.31	0.25	2.07	6740	3890	65.3

(a) Monotonic loading
(b) Repaired specimen
(c) 1000 psi = 1.0 ksi = 6.895 MPa

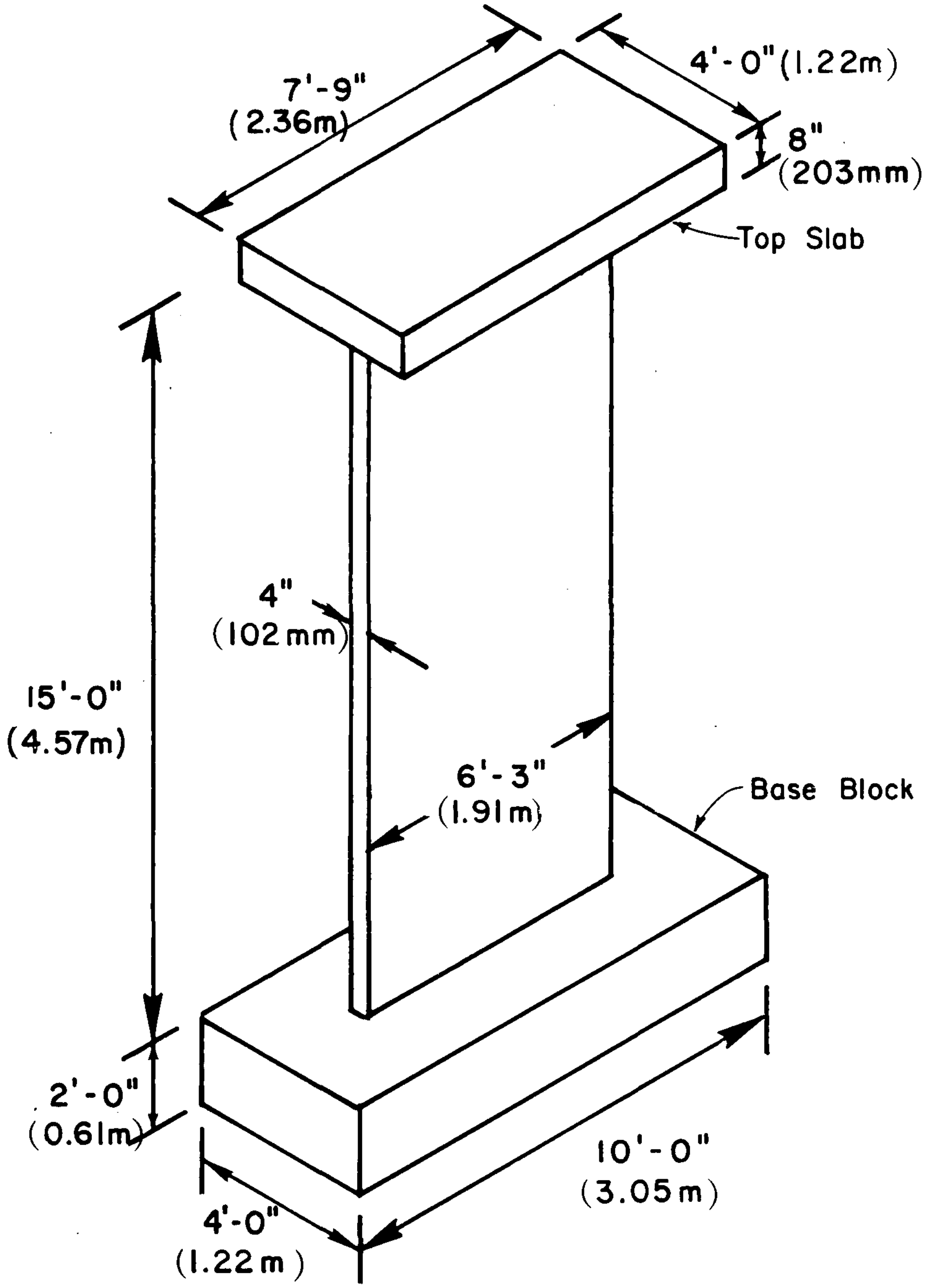


Fig. 1 Nominal Dimensions of Test Specimen
with Rectangular Cross Section

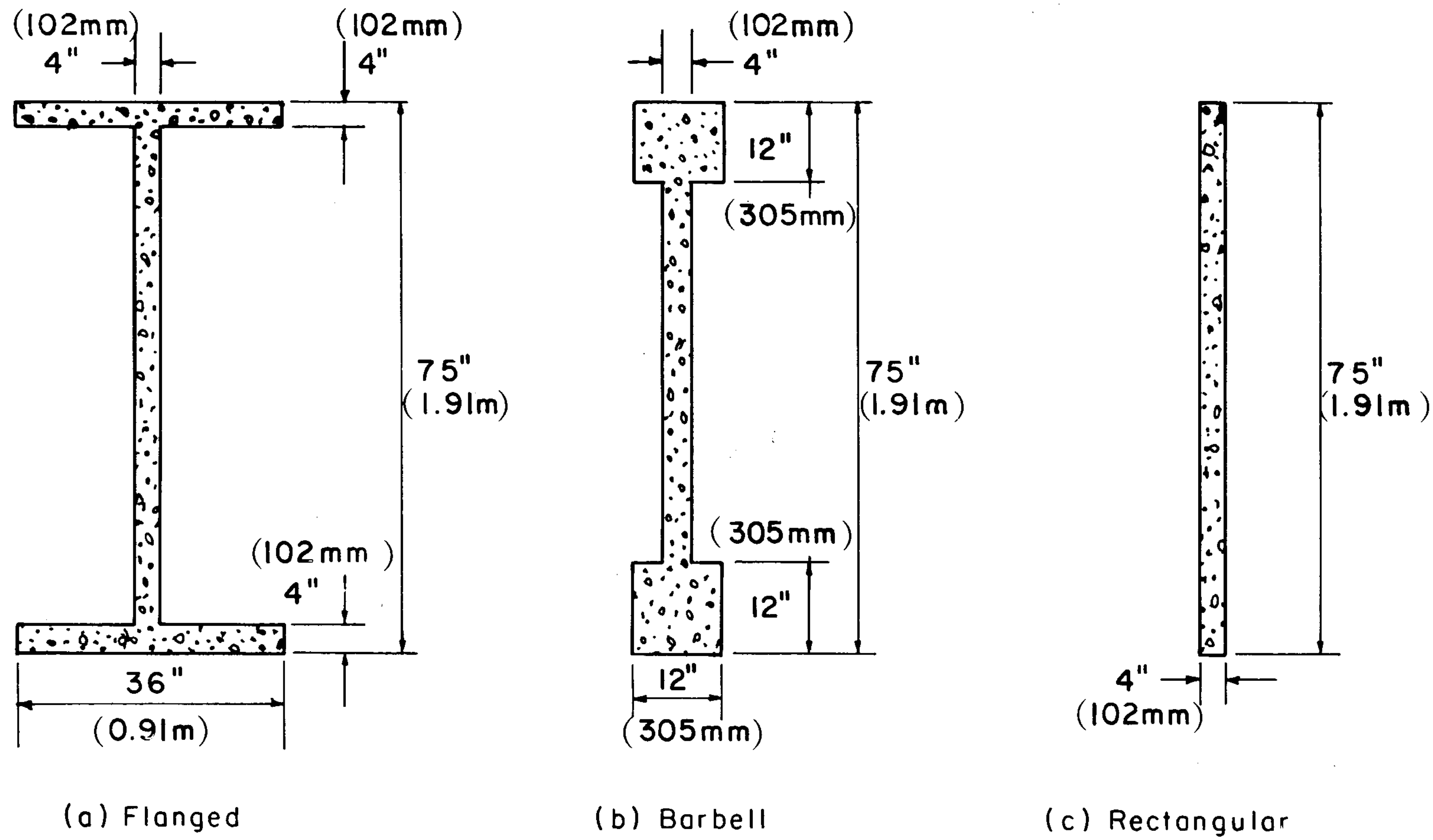


Fig. 2 Nominal Cross-Sectional Dimensions of Test Specimens

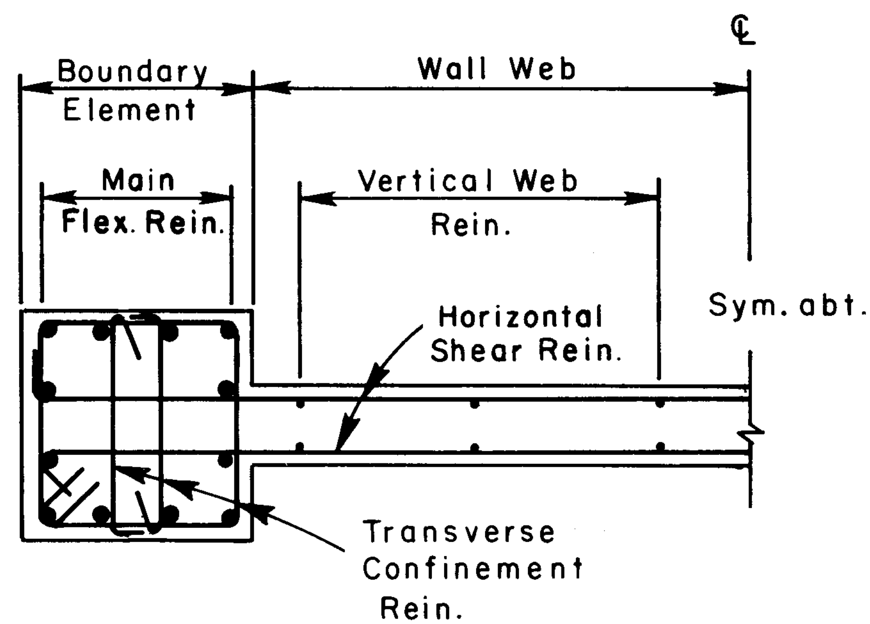


Fig. 3 Types of Wall Reinforcement

hoop spacing of 1.33 in. (34 mm). Confinement reinforcement was used only over the first 6 ft (1.83 m) above the base of the wall. Ordinary column ties were used over the remaining height.

Specimen B5R was a retest of repaired Specimen B5. Following the test of B5, the wall was returned to its original undeflected position. Then, the damaged web concrete was removed up to a height of about 9 ft (2.74 m). New web concrete was cast in three lifts. The boundary elements were given a cosmetic repair by hand rubbing the surfaces with neat cement paste.

Test Procedure

The apparatus for lateral load testing of the walls is shown in Fig. 4. Each specimen was loaded as a vertical cantilever with forces applied through the top slab. Walls were loaded in a series of increments except for Specimen B4. Each increment consisted of three complete reversed cycles. About three increments of force were applied prior to initial yielding. Subsequent to initial yielding, loading was controlled by deflections in 1-in. (25 mm) increments. Specimen B4 was subjected to a monotonically increasing load.

Free vibration tests were conducted at selected stages as the number and magnitude of lateral loading increments applied to the specimen increased. The first vibration test was run prior to the application of lateral loading. At this stage, the specimen could have contained cracks as a result of shrinkage and handling.

The second vibration test for Specimens F1, B5, B5R and R1 was conducted after cracking of the wall sections but before the lateral loading increment in which the flexural reinforcement yielded. For all specimens, vibration tests were run after the lateral loading cycles closest to the yield level had been applied. Additional tests were run at later loading stages depending on the physical condition of the test specimen.

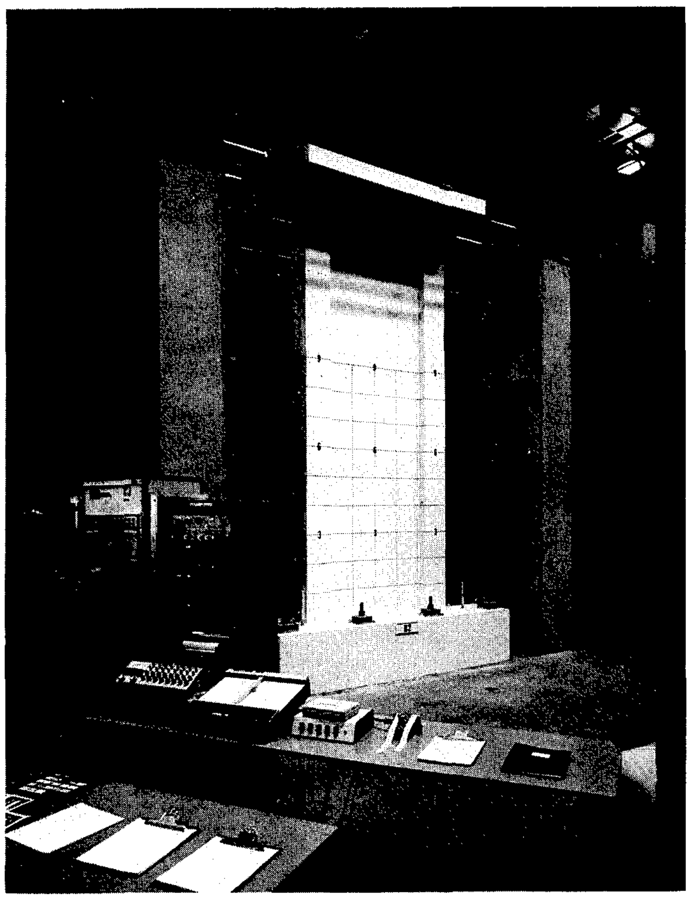


Fig. 4 Lateral Load Test Setup

Free vibration tests were performed using two methods. Both are considered small amplitude tests. They were run with the specimen disconnected from the hydraulic rams that applied lateral loads.

In the first method, vibrations were initiated using an "Initial Displacement-Sudden Release" system. A 1/4-in. (6.4 mm) diameter prestressing wire was attached to a bracket on the top slab of the wall. The wire was pulled to a predetermined force and then cut. The force used to displace the wall was lower than the calculated cracking load. This method is illustrated in Fig. 5.

In the second method, smaller amplitude tests were performed using the impact force of a 8-lb. (3.63 kg) hammer to initiate vibrations. These are termed "Hammer Impact Tests."

In both methods, the top lateral deflection of the test specimen was plotted versus time using an oscillographic recorder. An example of the time-displacement relationships for Specimen B1 is shown in Fig. 6.

Each free vibration test consisted of two separate excitations resulting in two time-displacement plots. The natural frequencies and damping coefficients reported for each test are the averages of four separate values determined from the positive and negative half of the two plots. The damping coefficient was calculated as equivalent viscous damping from the logarithmic decrement taken over several of the initial cycles.

TEST RESULTS

The results of the tests are presented in Table 2. Unless otherwise indicated, all results are from "Initial Displacement - Sudden Release" tests. Damping is presented as a percentage of the critical viscous damping.

The following observations are made on the results of the free vibrating tests.

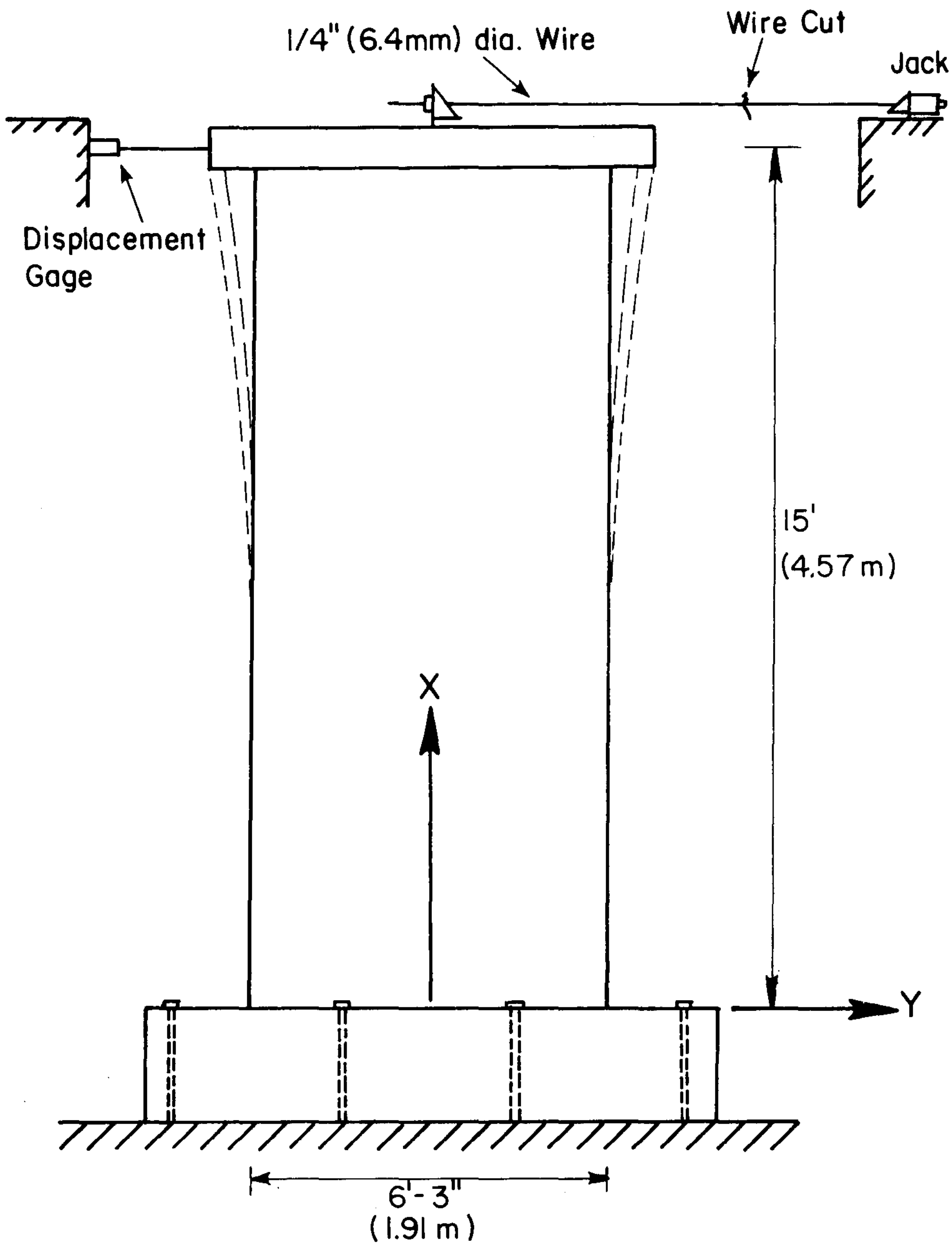
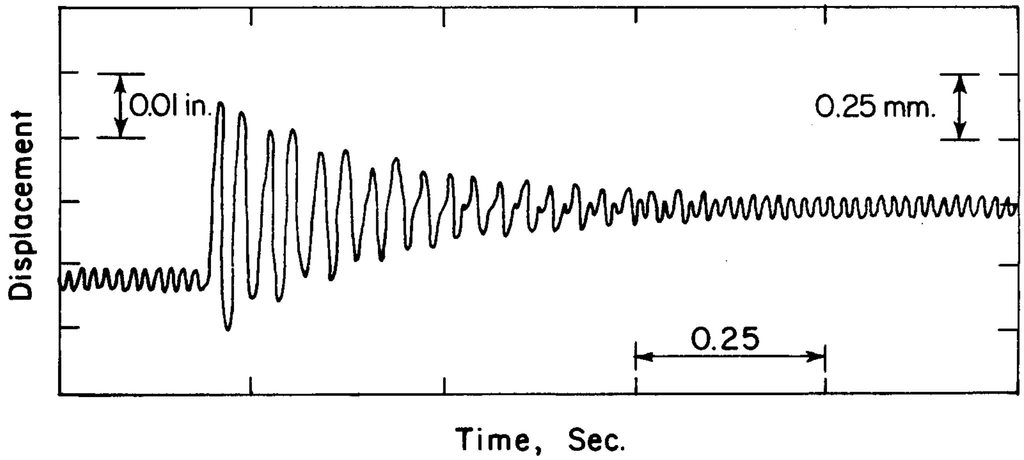
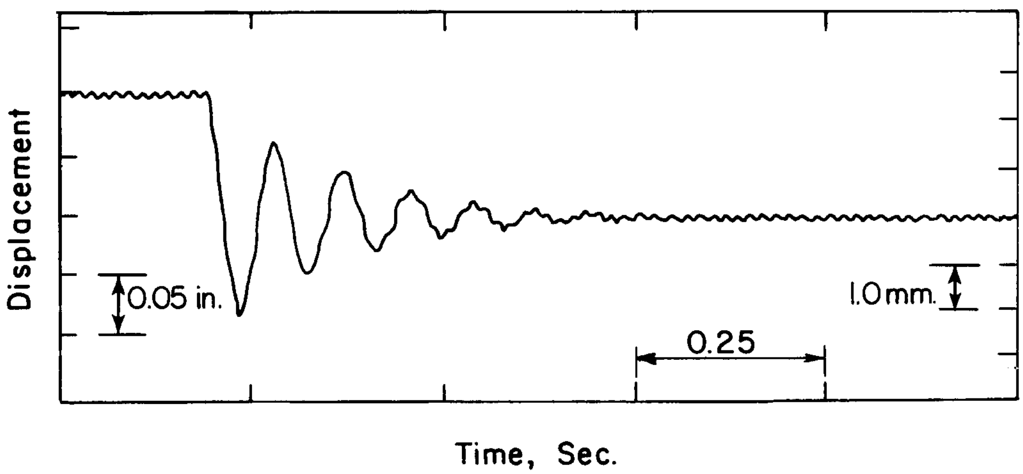


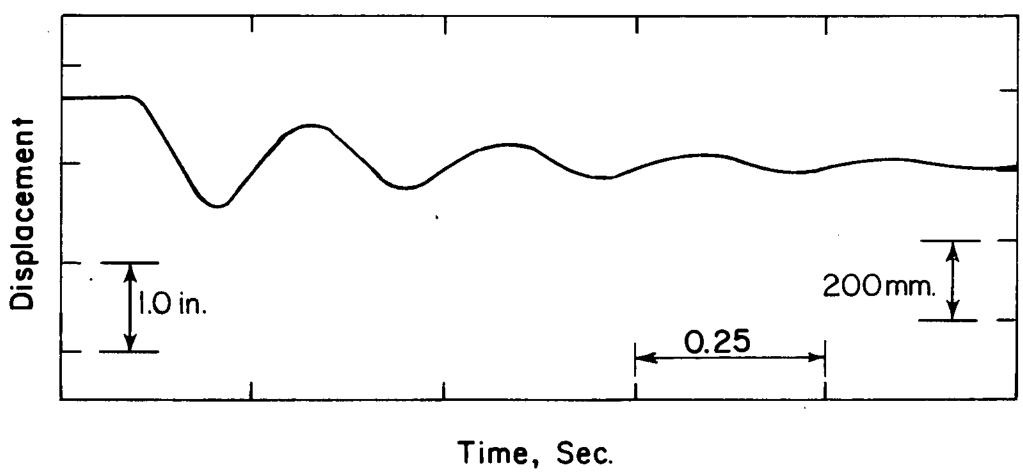
Fig. 5 Free Vibration Test Setup



a) Prior to Lateral Load Test



b) Prior $\Delta max / \Delta y = 1.0$



c) Prior $\Delta max / \Delta y = 5.6$

Fig. 6 Displacement Versus Time Relationships for Free Vibration Tests of Specimen B1

TABLE 2 - SUMMARY OF FREE VIBRATION TEST RESULTS

Specimen	Loading History		Excitation		Measured Fundamental Frequency (Hertz)	Measured Damping % of Critical[a]
	Prior No. of Load Cycles	Prior $\frac{\Delta_{max}}{\Delta_y}$	$\frac{P_i}{P_y}$	Initial Amplitude (in.)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
F1	0	--	0.05	0.017	33.8	2.0
	12	0.7	0.05	0.054	13.0	9.8
B1	0	--	0.17	0.016	30.0	2.2
	12	1.0	0.17	0.098	11.1	8.5
	24	5.6	0.17	0.681	3.9	9.1
B2	0	--	0.07	0.014	29.4	3.6
	15	1.0	0.07	0.065	13.0	10.0
	24	3.7	0.07	0.350	3.9	14.5
B3	0	--	0.17	0.015	29.7	2.7
	12	1.0	0.17	0.106	10.9	9.6
	24	5.8	0.17	0.598	4.3	8.1
	35	11.2	H.I.T.	(b)	5.2	9.0
B4[c]	0	--	H.I.T.	(b)	29.4	2.8
	0	--	0.13	0.018	29.4	2.4
	0	--	0.20	0.023	28.8	2.7
B5	0	--	H.I.T.	0.004	30.6	2.9
	0	--	0.07	0.026	29.5	4.0
	6	0.4	H.I.T.	0.004	20.4	9.2
	6	0.4	0.07	0.062	15.2	9.6
	15	1.1	H.I.T.	0.004	18.2	11.2
	15	1.1	0.07	0.090	12.0	12.0
	24	3.6	H.I.T.	0.004	11.8	3.2
	24	3.6	0.05	0.290	6.4	14.5
B5R[d]	0	--	H.I.T.	0.005	16.0	3.1
	0	--	0.07	0.075	13.3	4.0
	6	0.4	H.I.T.	0.005	13.2	4.0
	6	0.4	0.07	0.110	10.8	5.7
	15	1.1	H.I.T.	0.007	11.9	3.6
	15	1.1	0.07	0.151	8.3	11.0
R1	0	--	0.36	0.026	21.8	3.4
	6	0.5	0.36	0.095	10.5	6.7
R2	0	--	0.17	0.045	17.8	5.5
	15	1.0	0.17	0.121	8.8	6.8

NOTES:

(a) Based on Logarithmic decrement using five or more cycles.

(b) Initial amplitude not measured.

(c) Specimen B4 tested with monotonic lateral load.

(d) Specimen B5R was a repair of Specimen B5. Yielding in B5R taken at load P_y, for B5.

1 in = 25.4 mm

1. Excluding Specimen B5R, the measured frequency decreased by an average of 50% from the initial tests to the tests carried out after significant cracking, but prior to yielding. For the same conditions, the average damping coefficient increased from 3% to 9%.
2. Specimen B5 results indicate that relatively small decreases in frequency and increases in damping occurred for tests made after cracking compared to results for tests made close to yield.
3. Lateral load cycling through large inelastic deformations significantly reduced the frequency. However, the corresponding change in damping was generally small.
4. The initial tests on the repaired wall, B5R, indicate that frequency was approximately 50% of that of the original wall. Damping was the same order of magnitude in the original and the repaired wall.
5. In general, the smaller amplitude "Hammer Impact" tests gave higher frequencies and lower damping coefficients than "Initial Displacement - Sudden Release" tests. The results were particularly sensitive to the magnitude of the initial displacement after large inelastic lateral load cycles. This would be expected because of differences in crack closure that resulted from the magnitude of the initial displacement.

CONCLUSIONS

Small amplitude free vibration tests of isolated structural walls indicate that frequency and damping characteristics are sensitive to the development of structural cracks in the walls. This corresponds to the large change in stiffness that occurs at this stage. With increasing damage levels, frequency decreases and damping increases. These observations should be considered in analyzing the dynamic response of reinforced concrete wall systems. A detailed analysis of the free vibration test results is presented in a companion paper.[1]

ACKNOWLEDGMENTS

This work was part of a combined experimental and analytical investigation on the earthquake resistance of structural walls under the direction of Mr. M. Fintel and Dr. W. G. Corley. The project was supported by the National Science Foundation under Grant Nos. ENV 74-14766 and PFR-7715333 and by the Portland Cement Association. Any opinions, findings and conclusions expressed in this paper are those of the authors and do not necessarily reflect the views of the National Science Foundation. The tests were carried out in the Structural Development Department of the Portland Cement Association, Dr. H. G. Russell, Director.

APPENDIX I - REFERENCES

1. Aristizabal-Ochoa, J.D., "Analysis of Free Vibration Tests of Structural Walls," Report to National Science Foundation, Construction Technology Laboratories, A Division of the Portland Cement Association, Skokie, Illinois, February 1980.

2. Oesterle, R.G., et al, "Earthquake Resistant Structural Walls - Tests of Isolated Walls," Report to National Science Foundation, Portland Cement Association, Skokie, Nov. 1976, 44 pp.; Appendix A, 38 pp.; Appendix B, 233 pp. (Available through National Technical Information Service, U.S. Department of Commerce, 5285 Port Royal Rd., Springfield, Va., 22161, NTIS Accession No. PB271467.)

3. Oesterle, R.G., et al, "Earthquake Resistant Structural Walls - Tests of Isolated Walls - Phase II," Report to National Science Foundation, Construction Technology Laboratories, A Division of the Portland Cement Association, Skokie, Illinois, October 1979, (Available through National Technical Information Service, U.S. Department of Commerce, 5285 Port Royal Rd., Springfield, Va., 22161).

4. American Concrete Institute, "Building Code Requirements for Reinforced Concrete (ACI 318-71)," Detroit, 1971, 78 pp.

APPENDIX II - NOTATION

The following symbols are used in this report:

E_c	=	modulus of elasticity of concrete
H.I.T	=	hammer impact test
P_1	=	load applied to top of wall to initiate vibrations
P_y	=	load applied at top of wall corresponding to Δ_y
f'_c	=	concrete compressive strength
Δ_{max}	=	maximum deflection at top wall during prior lateral load cycles
Δ_y	=	deflection at top of wall at which first yielding of main flexural steel was observed during lateral load tests
ρ_f	=	ratio of main flexural reinforcement area to gross concrete area of boundary element
ρ_h	=	ratio of horizontal shear reinforcement area to gross concrete area of a vertical section of wall web
ρ_n	=	ratio of vertical web reinforcement area to gross concrete area of a horizontal section of wall web
ρ_s	=	ratio of effective volume of confinement reinforcement to the volume of core in accordance with Eq. A.4 of ACI 318-71.

ANALYSIS OF FREE VIBRATION TESTS OF STRUCTURAL WALLS

by

J. D. Aristizabal-Ochoa*

INTRODUCTION

Determination of natural frequencies of a reinforced concrete structural system and the implications of cracking and yielding on dynamic characteristics are important in earthquake resistant design. This paper evaluates free vibration tests of nine reinforced concrete structural walls constructed and tested at the Portland Cement Association. The free vibration tests were carried out to determine the fundamental frequency and critical damping ratio. Test specimens and free vibration tests are described and reported elsewhere.(1)

This paper compares the measured frequencies of the undamaged walls with values calculated by two methods. The first method considers flexural deformations only. The second method considers shear deformations, rotary inertia, and axial load in addition to flexural deformations. Comparisons of measured and calculated data show the effects of shear deformation on the natural frequencies of the reinforced concrete walls. Based on test results described in this paper, the first method modified to include shear deformations is recommended to calculate the fundamental frequency of thin-webbed structural walls.

EXPERIMENTAL WORK

Figure 1 shows schematically the free vibration setup. Flanged, barbell and rectangular walls were tested. Figure 2 shows the nominal dimensions of the test specimens. The free vibration tests were conducted at selected stages after apply-

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* Former Structural Engineer, Structural Development Department; Portland Cement Association, Skokie, Illinois.

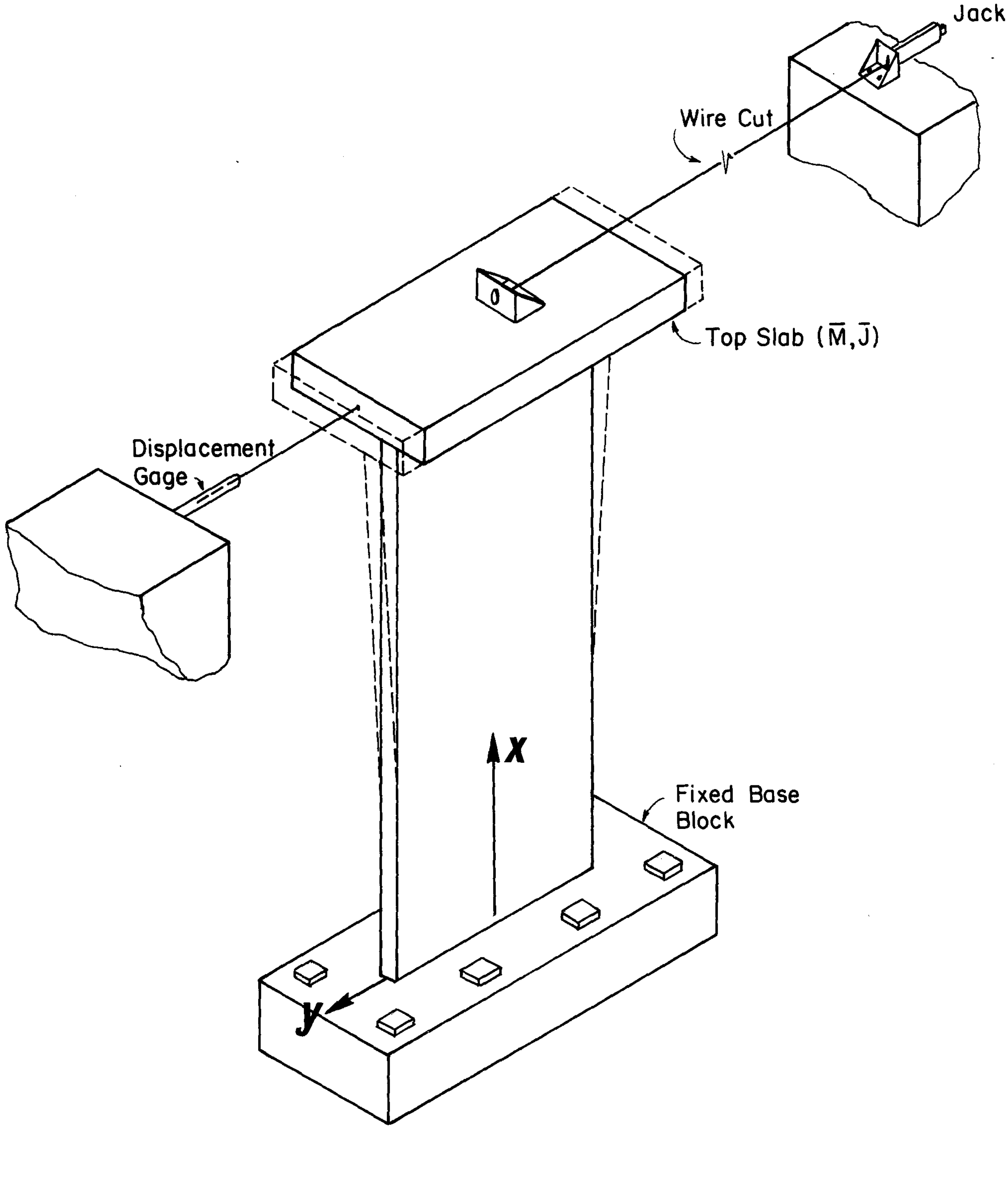
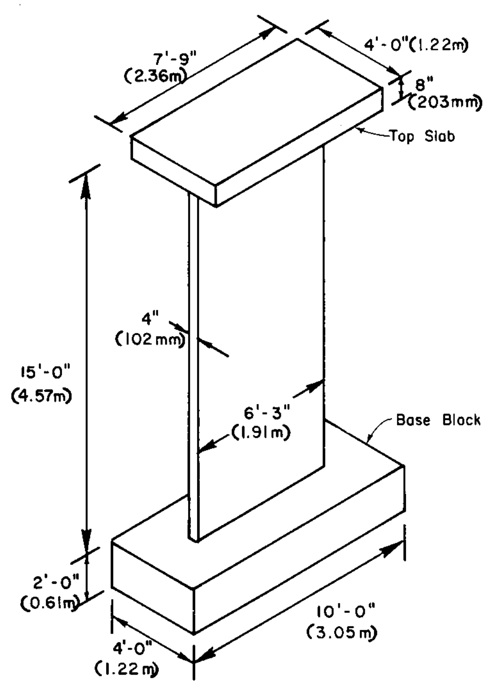
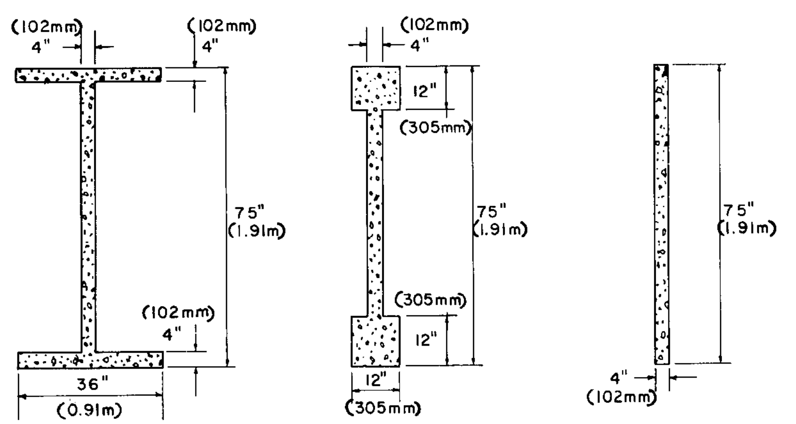


Fig. 1 Free Vibration Test Setup



(a) Nominal Dimensions of Test Specimen with Rectangular Cross Section



Flanged
k = 0.52

Barbell
k = 0.58

Rectangular
k = 0.83

(b) Nominal Cross-Sectional Dimensions and Shear Factors of Test Specimens

Fig. 2 Nominal Dimensions of Test Specimens ing reversing in-plane lateral loads to the specimens. Small amplitude vibrations were initiated by pulling the specimens with a 1/4-in. (6.4 mm) diameter wire and then cutting the wire as shown in Fig. 1. These are termed "Initial Displacement-Sudden Release Tests." Another method to excite the walls was to hit the specimens with an 8-lb (3.63 kg) hammer. These are termed "Hammer Impact Tests." In both cases, the top lateral deflection of the test specimens was plotted against time using an oscillographic recorder. Fundamental frequencies and damping ratios were calculated from the recorded signals using five or more cycles. Damping ratios were based on logarithmic decrement.

Further details of the specimens and test setup are given elsewhere.[1] Table 1 summarizes concrete properties and vertical reinforcement percentages used in the analysis reported in this paper.

FREQUENCY CALCULATIONS

In the analysis of response to dynamic loadings, determination of undamped natural frequencies of a structure is of a vital importance. For this reason it is useful to develop methods of evaluating natural frequencies. The "correctness" of the assumptions may be determined by comparing the calculated results with experimental values.

Two methods of analysis were used to estimate the fundamental frequency of undamaged walls in this program. Method 1 is an approximate analysis. Method 2 is a more elaborate analysis developed primarily to investigate the effect of shear deformations.

In Method 1, fundamental frequency was calculated using the Rayleigh method. The derivation is available in standard textbooks.[2,3] Only flexural deformations and gross section properties were considered. The method was relatively simple to apply.

TABLE 1 - CONCRETE PROPERTIES AND REINFORCEMENT PERCENTAGES

Specimen (1)	Compressive Strength, f'_c[a] (psi) (2)	Modulus of Elasticity,[a] E_c (ksi) (3)	Reinforcement Percentage[b] (%) (4)
F1	5580	3690	2.16
B1	7690	4080	0.77
B2	7780	4200	2.27
B3	6860	3960	0.77
B4	6530	4100	0.77
B5	6570	3970	2.27
B5R	6200	4010	2.27
R1	6490	4030	0.49
R2	6740	3890	1.00

(a) Average properties are for lower 6 ft (1.83 m) of wall. For Specimen B5R average properties are for replaced web concrete

(b) Gross reinforcement ratio based on the total vertical reinforcement and the gross sectional area.

1000 psi = 1 ksi = 6.895 MPa

In Method 2, the natural frequency was calculated using Timoshenko's theory[4] of beam vibrations. Effects of flexural and shearing deformations, rotary moment of inertia and axial loading were taken into account in these calculations. The derivation is given in Appendix III.

COMPARISON WITH TEST RESULTS

Tables 2 and 3 list the initial "uncracked" fundamental frequencies and calculated values. The calculated values are based on the assumptions described above using the concrete properties shown in Table 1 and the gross section dimensions shown in Fig. 2. Concrete was assumed to weigh 150 pcf (2400 kg/m^3). The compressive force, N, was assumed to be equal to that of the top slab plus one third of the weight of the cantilever. The influence of axial force on the calculated fundamental frequency was negligible (less than 0.03). Shear distortion factors, k, shown in Fig. 2b were based on strain energy considerations as described by Langhaar.[5]

Fundamental Frequency

Calculated frequencies based on Timoshenko's theory (Method 2) gave better agreement with measured frequencies than those based on the Rayleigh method (Method 1). Except for Specimen R2, the measured initial fundamental frequency varied from 90% to 100% of the calculated values based on Timoshenko's theory.

The difference between calculated values based on Method 1 and Method 2 is due mainly to shear deformations. This difference is more significant in specimens with boundary elements particularly the flanged specimens.

Calculated values based on the Rayleigh method can be improved by modifying the fundamental frequency calculated in Method 1 by a factor of $\sqrt{1 + 4EI/kAGL^2}$ as shown in Columns 5 and 8 of Table 3. The term, $(4EI/kAGL^2)$, represents the ratio of shear deformations to bending deformations at the free end of a cantilever beam with uniform load along its span. The advantage of Method 1 and Modified Method 1 is the simplicity of the calculation.

TABLE 2 - SUMMARY OF FREE VIBRATION TEST RESULTS

Specimen	Loading History		Excitation		Measured Fundamental Frequency (Hertz)	Measured Damping % of Critical[a]
	Prior No. of Load Cycles	Prior Δ_{max}/Δ_y	P_i/P_y	Initial Amplitude (in.)		
(1)	(2)	(3)	(4)	(5)	(6)	(7)
F1	0 12	-- 0.7	0.05 0.05	0.017 0.054	33.8 13.0	2.0 9.8
B1	0 12 24	-- 1.0 5.6	0.17 0.17 0.17	0.016 0.098 0.681	30.0 11.1 3.9	2.2 8.5 9.1
B2	0 15 24	-- 1.0 3.7	0.07 0.07 0.07	0.014 0.065 0.350	29.4 13.0 3.9	3.6 10.0 14.5
B3	0 12 24 35	-- 1.0 5.8 11.2	0.17 0.17 0.17 H.I.T.	0.015 0.106 0.598 (b)	29.7 10.9 4.3 5.2	2.7 9.6 8.1 9.0
B4[c]	0 0 0	-- -- --	H.I.T. 0.13 0.20	(b) 0.018 0.023	29.4 29.4 28.8	2.8 2.4 2.7
B5	0 0 6 6 15 15 24 24	-- -- 0.4 0.4 1.1 1.1 3.6 3.6	H.I.T. 0.07 H.I.T. 0.07 H.I.T. 0.07 H.I.T. 0.05	0.004 0.026 0.004 0.062 0.004 0.090 0.004 0.290	30.6 29.5 20.4 15.2 18.2 12.0 11.8 6.4	2.9 4.0 9.2 9.6 11.2 12.0 3.2 14.5
B5R[d]	0 0 6 6 15 15	-- -- 0.4 0.4 1.1 1.1	H.I.T. 0.07 H.I.T. 0.07 H.I.T. 0.07	0.005 0.075 0.005 0.110 0.007 0.151	16.0 13.3 13.2 10.8 11.9 8.3	3.1 4.0 4.0 5.7 3.6 11.0
R1	0 6	-- 0.5	0.36 0.36	0.026 0.095	21.8 10.5	3.4 6.7
R2	0 15	-- 1.0	0.17 0.17	0.045 0.121	17.8 8.8	5.5 6.8

NOTES:

(a) Based on Logarithmic decrement using five or more cycles.

(b) Initial amplitude not measured.

(c) Specimen B4 tested with monotonic lateral load.

(d) Specimen B5R was a repair of Specimen B5. Yielding in B5R taken at load P_y, for B5.

1 in = 25.4 mm

TABLE 3 - MEASURED AND CALCULATED FREQUENCIES

Specimen	Measured Fundamental Frequency Hertz	Calculated Fundamental Frequency, Hertz			(Measured/Calculated) Fundamental Frequency		
		Method 1[a]	Method 2[b]	Modified Method 1[c]	Method 1	Method 2	Modified Method 1
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
F1	33.8	41.1	33.9	34.1	0.82	1.00	0.99
B1	30.0	37.4	32.2	32.4	0.80	0.93	0.93
B2	29.4	37.9	32.7	32.8	0.78	0.90	0.90
B3	29.7	36.8	31.7	31.9	0.81	0.94	0.93
B4	29.2[d]	37.5	32.3	32.5	0.78	0.91	0.90
B5	30.1[d]	36.9	31.8	32.0	0.81	0.95	0.94
B5R	14.7[d]	--	--	--	--	--	--
R1	21.8	25.9	23.8	24.1	0.84	0.92	0.90
R2	17.8	25.4	23.4	23.6	0.70	0.76	0.75

NOTES:

(a) Fundamental Frequency = $F = \frac{1}{2\pi}\sqrt{\frac{3EI/L^3}{M + 33\ mL/140}}$

(b) Based on Timoshenko's theory.

(c) Fundamental Frequency = $F/\sqrt{1+4EI/kAGL^2}$

(d) Average initial measured fundamental frequency.

The measured fundamental frequency of Specimen R2 was low even compared to that of similar Specimen R1. Measured initial damping ratio of Specimen R2 indicates that it may have had more cracks than the rest of the specimens before the first free vibration test. These cracks may have been caused as the specimen was prepared for test.

Stiffness Changes

A key characteristic of the test specimens was the change in measured fundamental frequency with the reduction in stiffness caused by the reversing lateral loads. Figure 3 shows the change in the fundamental frequency with the damage ratio, defined as $\mu = \Delta_{max}/\Delta_y$. The figure shows that small amounts of damage significantly reduced the fundamental frequency of each specimen.

Most of the reductions in the measured frequencies were caused by cracking before first yielding of main flexural reinforcement ($\mu < 1$). This is expected since the test structures were lightly reinforced for flexure. Additional damage ($\mu > 1$) had relatively less influence on the fundamental frequency.

As seen in Table 2, the magnitude and changes in the measured frequency of repaired Specimen B5R are significantly lower than those of Specimen B5. This is because only the web of the Specimen B5R was repaired. The boundary elements were already cracked and the reinforcement had experienced yield excursions.

The lateral resisting stiffness of each specimen decreased as the load level increased.[1] This explains the decrease in the measured frequencies of Specimens B4, B5, and B5R with increasing initial amplitude of free vibration.

Damping

Because of its convenience in dynamic analysis, viscous damping is used traditionally to represent energy dissipation in the linear range of response of structures subjected to dynamic loading. Viscous damping is considered as a percentage

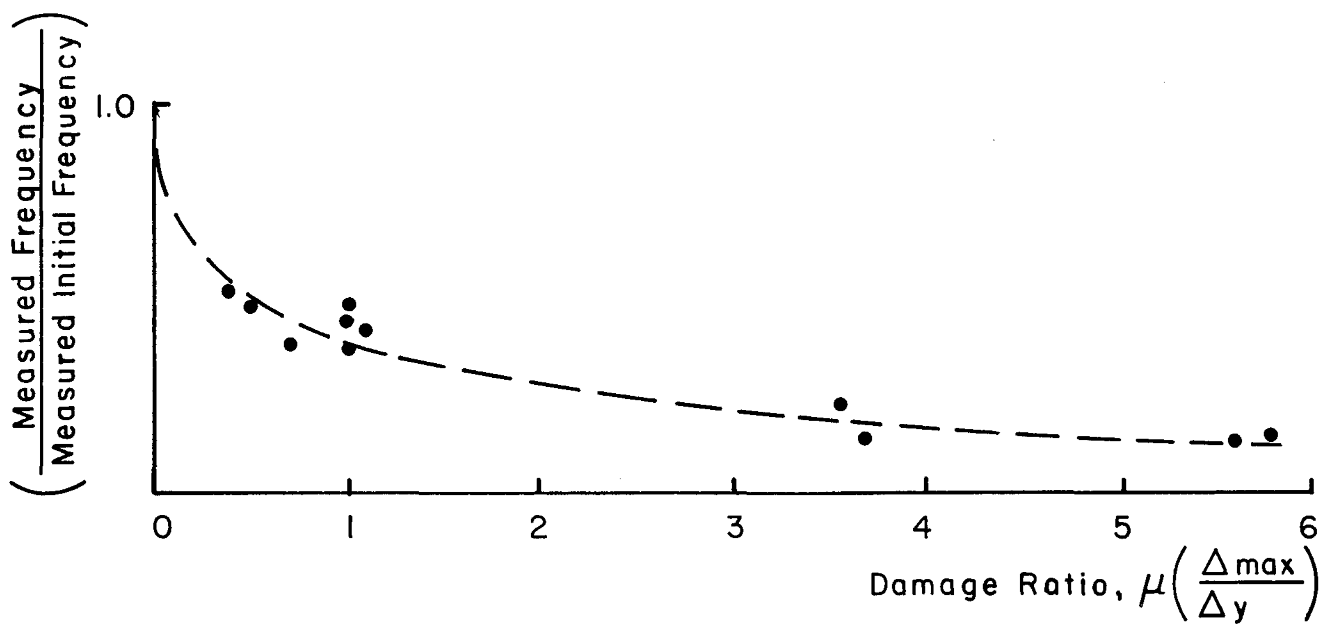


Fig. 3 Variation of Measured Fundamental Frequency with Damage Ratio

of the critical viscous damping. Critical damping is defined as the smallest amount for which no oscillation occurs in a system subjected to an initial disturbance.

Measured damping shown in Column (7) of Table 2 represents the percentage of critical viscous damping of the test specimens at very low amplitudes only. The magnitude of the initial amplitude is given in Columns (4) and (5) of the same table. It should be noted that the damping percentages are for the structural walls alone. Therefore, they do not represent overall damping of reinforced concrete buildings. Damping of a building would depend on the damping of the structural systems and nonstructural elements as well as on friction between different elements.

Measured damping after the specimens were cracked was primarily due to the energy dissipated by friction between crack surfaces. Figure 4 shows the variation of damping measured in the "Initial Displacement-Sudden Release Tests"[1] with the damage ratio. As shown in Fig. 4, damping increased significantly with initial cracking ($\mu < 1$). Further damage did not significantly influence measured damping.

Table 2 shows that the amount of damping increased with initial amplitude of the free vibration. For "undamaged" specimens or for very small amplitude vibrations, little friction developed along the cracked surfaces and the measured damping was small. This can be seen from the test results for Specimens B4, B5, and B5R.

CONCLUSIONS

Initial frequencies are a good indicator of the importance of the different structural actions on a specific structural system. For structural walls similar to those tested, the inclusion of shear deformations in the calculation of natural frequencies has a significant effect. This is particularly true for walls with large boundary elements.

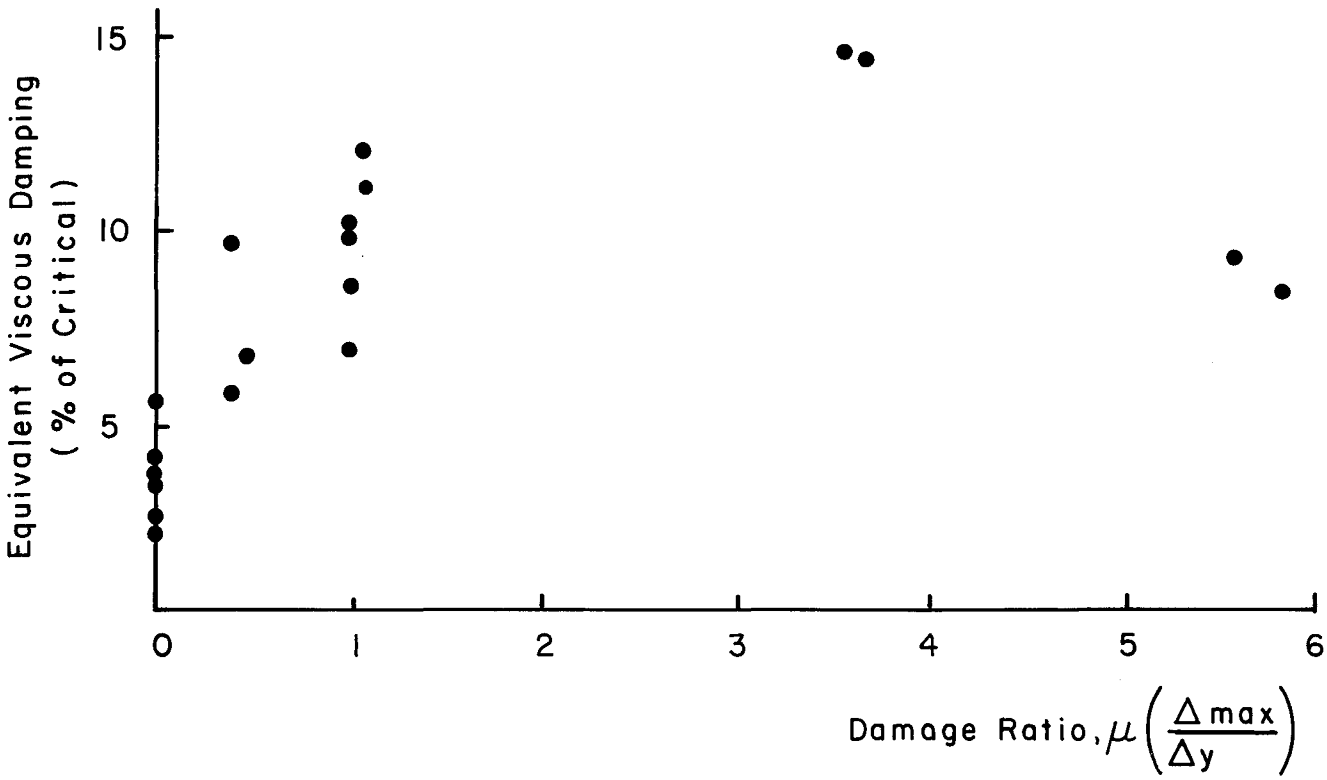


Fig. 4 Variation of Measured Damping with Damage Ratio

Two methods are presented for the calculation of fundamental frequency. The Rayleigh method when modified and the Timoshenko method gave good comparison between calculated and measured values. The modified Rayleigh method is preferred because the values can be calculated easily.

It should be noted that an exact calculation of the initial natural frequencies of reinforced concrete structures is of limited usefulness in predicting response to strong dynamic motions. This is because of the considerable reductions in frequency caused by cracking of the concrete and yielding of the reinforcement.

Implications of various levels of structural damage are particularly important in considering the response of reinforced concrete structures subjected to earthquake motions. In some cases reinforced concrete elements particularly structural walls must remain elastic or nearly elastic to perform their allocated safety function. Test results considered in this paper indicate that nonlinearity occurs at load levels lower than initial yield ($\mu < 1$). This is sufficient to reduce considerably the required design values. Therefore, linear elastic analysis based on "uncracked" properties may be unreasonably conservative particularly for lightly reinforced concrete members.

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APPENDIX I - REFERENCES

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APPENDIX II - NOTATION

The following symbols are used in this report:

A = cross-sectional area of wall

C_1, C_2, C_3, C_4 = unknown coefficients in Eq. (9)

E_c = modulus of elasticity of concrete

$G = \frac{E}{2(1 + \upsilon)}$ = shear modulus of concrete

I = moment of inertia of wall cross section

$\bar{J}$ = rotary moment of inertia of top slab

L = span or wall height

M = flexural moment

$\bar{M}$ = mass of top slab

N = axial force

V = shear force

f'_c = concrete compressive strength

k = shear distortion coefficient (= 5/6 for rectangular section)

m = mass of wall per unit length

r = radius of gyration of wall cross section

t = time

x = independent variable

y = total deflection of the center line (including bending and shear deformation)

$\frac{\partial^2 y}{\partial t^2}$ = acceleration of wall center line

$\frac{\partial y}{\partial x}$ = total slope of wall center line

Δ_{max} = maximum deflection at top of wall during prior lateral loads cycles.

Δ_y = deflection of top of wall at which first yielding of main flexural steel was observed during lateral load tests.

μ = damage ratio = $\frac{\Delta_{max}}{\Delta_y}$

υ = Poisson's ratio (taken as 0.15)

Ψ = slope of wall center line due to bending

Φ = eigenfunction for total deflection

θ = eigenfunction for bending slope

ω = angular frequency

APPENDIX III - FREQUENCY ANALYSIS

To develop the second method, the free-body diagram and geometry for a differential wall element shown in Fig. A were used. Assuming as a first approximation that shear force causes the element to deform into a diamond shape without rotation of the cross sections, the slope of the center line caused by flexure is diminished by the shear distortion ($\psi - \partial y/\partial x$). If the shear distortion is zero, the center line will coincide with a line perpendicular to the face of the cross section.

Using notation in Appendix II, the equations of motion for rotation and translation of the differential element are, respectively:

$$mr^2 \frac{\partial^2 \psi}{\partial t^2} = \frac{\partial M}{\partial x} - V + N \frac{\partial y}{\partial x} \tag{1}$$

$$m \frac{\partial^2 y}{\partial t^2} = - \frac{\partial V}{\partial x} \tag{2}$$

Axial force, N, is assumed to be constant with respect to both time and position.

The bending moment, M, and shear force, V, are related to deformations by two relationships from elastic theory:

$$\frac{\partial \psi}{\partial x} = \frac{M}{EI} \tag{3}$$

$$\psi - \frac{\partial y}{\partial x} = \frac{V}{kAG} \tag{4}$$

Eliminating M and V from the four relationships above gives the following pair of coupled equations:

$$\frac{\partial}{\partial x}\left(EI \frac{\partial \psi}{\partial x}\right) + kAG\left(\frac{\partial y}{\partial x} - \psi\right) + N \frac{\partial y}{\partial x} - mr^2 \frac{\partial^2 \psi}{\partial t^2} = 0 \tag{5}$$

$$\frac{\partial}{\partial x}\left[kAG\left(\frac{\partial y}{\partial x} - \psi\right)\right] - m \frac{\partial^2 y}{\partial t^2} = 0 \tag{6}$$

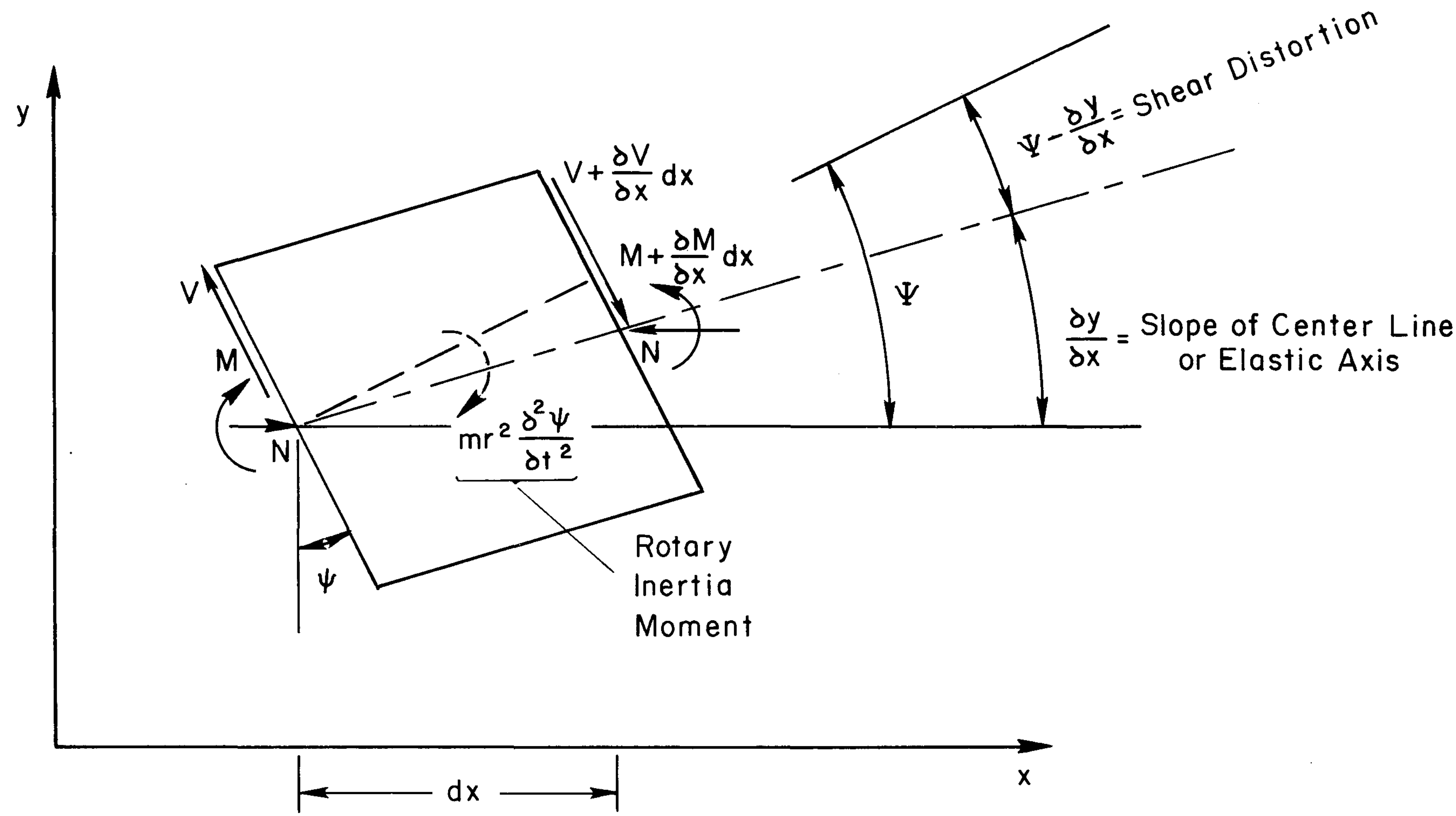

Fig. A Deformations and Forces Acting on Differential Element

Equations (5) and (6) can be solved by assuming a solution of the form:

$$y = \Phi\ (x)\sin \omega t \tag{7}$$

$$\psi = \theta\ (x)\sin \omega t \tag{8}$$

Substituting into (5) and (6) and solving the resulting pair of ordinary differential equations leads to the following general expressions for the eigenfunctions Φ and θ:

$$\Phi(x) = C_1 \sin \xi x + C_2 \cos \xi x + C_3 \sinh \delta x + C_4 \cosh \delta x \tag{9}$$

$$\theta(x) = \frac{a^4 r^2 (\frac{E}{kG}) - \xi^2}{\xi}\left[C_2 \sin \xi x - C_1 \cos \xi x\right] + \tag{10}$$

$$\frac{a^4 r^2 (\frac{E}{kG}) - \delta^2}{\delta}\left[C_4 \sinh \delta x + C_3 \cosh \delta x\right]$$

where

$$a^4 = m\omega^2/EI$$

$$\left.\begin{matrix}\xi^2 \\ \delta 2\end{matrix}\right\} = \pm \frac{a^4 r^2}{2}\left(1 + \frac{E}{kG} + \frac{N}{m\omega^2 r^2}\right) + \sqrt{\frac{a^8 r^4}{4}\left(1 + \frac{E}{kG} + \frac{N}{m\omega^2 r^2}\right)^2 + a^4\left(1 - a^4 r^4 \frac{E}{kG}\right)}$$

The three terms within the first parentheses of last expression represent the effect of rotary moment of inertia, shear distortion and axial force, respectively. The shear distortion is E/kG times as important as rotary inertia.

Substituting Eqs. (9) and (10) into the four boundary conditions gives a set of four homogeneous algebraic equations in the unknowns C_1, C_2, C_3, C_4. The frequencies are obtained by equating the determinant of the set to zero.

The boundary conditions for the structural wall as a vertical cantilever shown in Fig. 1 are as follows:

(a) at the fixed end ($x = 0$); $y(0) = 0$ and $\psi(0) = 0$

(b) at the free end ($x = L$);

$$\text{Shear Force} = kAG\left(\psi - \frac{\partial y}{\partial x}\right) = -\bar{M}\omega^2 y$$

$$\text{Bending Moment} = EI\,\frac{\partial \psi}{\partial x} = \bar{J}\omega^2\psi$$