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Strength Design of Structural Walls with Particular Reference to Shear and Cyclic Loading*

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INTRODUCTION

The height-to-depth ratio is often regarded as the parameter governing the mode of response of a structural wall (shear wall). Short, stocky structural walls are supposed to be mostly governed by their shear strength, whereas slender walls are supposed to act very much like beam-columns controlled by flexure and axial load.

The Canadian code for the design of concrete structures for buildings (CSA Standard A23.3-1973)(1) was one of the first codes to introduce specific seismic provisions for structural walls to ensure that the structural properties required during extreme seismic conditions would be available. It went a step further in distinguishing between walls which respond primarily in a flexural mode and those which behave primarily in a shear manner. To quantify the difference in behavior, the deflections at the top of a linearly elastic homogeneous cantilever structure due to bending and due to shear were compared. Accordingly, a cantilever structure in which the deflection due to bending is at least 10 times the deflection caused simultaneously by shear was termed a "flexural wall". The dimensions of a flexural wall were thus required to be such that:(2)

$$\left(\frac{h_w}{\ell_w}\right)^2 \frac{1}{1 + 4\gamma} \geq 10 \tag{1}$$

where h_w is total height of wall from its base to its top, ℓ_w is horizontal length of wall, and γ is the ratio of flange width to total horizontal length. The factor γ recognizes the relative importance of shear distortions in flanged sections.

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It is evident that for sections with a particular shape (i.e., with a fixed value of γ), relation (1) reduces to a limitation on the h_w/ℓ_w ratio. For instance, flexural walls with a rectangular section ($\gamma = 0$) must have $h_w/\ell_w \geq \sqrt{10}$. Flexural walls having a flanged section with $\gamma = 1$ must have $h_w/\ell_w \geq \sqrt{50}$.

As has been pointed out by Paulay and Uzumeri,[2] requirements such as the one represented by Eq. (1) are not likely to serve the intended purpose of separating flexural behavior from shear type response. The reason is that the predominance of a particular mode of behavior of an idealized elastic structure has little, if any, relevance to the mode of energy dissipation in the actual structure. In general, the relation of the simultaneous flexural and shear distortion to each other in an elastic structure is significantly different from that during post-elastic response of the same structure. The reduction in shear stiffness of uncracked members due to diagonal cracking is much more than the corresponding reduction in flexural stiffness caused by flexural cracking, as shown by Park and Paulay.[3]

This paper intends to show that the parameters most directly governing the mode of response (flexural or shear) of a structural wall are the applied moment and shear at the critical section, the moment capacity of this section, and the shear capacity of a segment containing the section. A design approach based on this observation is suggested. This approach should make it possible to eliminate the probability of occurrence of brittle shear failure, whenever this is judged to be a necessary design criterion.

The possible failure types of reinforced concrete members are first broadly defined in this paper. The prediction of failure type under proportional loading is next discussed. A design approach based on such prediction and applicable to situations of non-proportional earthquake type loading is suggested. The approach requires reasonable estimates of the shear capacity of a member segment under variable reversing loads. A discussion of such capacity and possible estimates of it are included.

TYPES OF FAILURE OF REINFORCED CONCRETE MEMBERS

The following broad definitions of failure types are based on considerations of strength. Failure modes, conventionally defined in terms of the type of cracking and the resulting crushing and splitting of the concrete, constitute subclasses of the following failure types:

1. Flexural--The failure of a member segment by the exhaustion of the moment carrying capacity of its critical section(s), while there is still some reserve shear capacity, may be termed as flexural failure. This is a familiar failure type, and is usually subdivided into tension, balanced and compression failures.

2. Flexure-Shear--This type of failure implies the more or less simultaneous attainment of the flexural and shear capacities of the member segment and exhibits the characteristics of both a flexure and a shear failure.

3. Shear--The failure of a member segment by the exhaustion of its shear capacity before the attainment of the full flexural strength may be defined as shear failure. This may be in various modes, depending upon a large number of factors, and has been discussed in more detail elsewhere.[4]

The above definitions of failure types can be complete only if flexural strength (capacity) and shear strength (capacity) are properly defined. This is done here with reference to Fig. 1. Figure 1a is a schematic representation of the failure surface of a reinforced concrete member segment in axial load-bending moment-shear force space, failure being defined by the attainment of maximum strength (load carrying capacity). The axial load, bending moment and shear associated with any point on this surface may be termed the axial load at failure (P_f), the failure moment (M_f), and failure shear (V_f), respectively. For the purpose of this study, the axial load capacity is defined as the axial load at failure in the absence of any moment or shear:

$$P_{cap} = P_{f \mid M = 0,\, V = 0} \tag{2}$$

Moment capacity is defined as the failure moment in the absence of any shear and is thus a function of axial load:

$$M_{cap} = M_{f \mid V = 0} = M_{cap}(P) \tag{3}$$

Shear capacity is defined as the failure shear, with one important qualification. It should be noted that the failure surface is normal to the P-M plane, as long as shear does not exceed a certain limiting value. In this region the axial load-moment interaction diagrams are essentially the same as those under no shear, and failure is by the attainment of M_{cap}. To define shear capacity as the failure shear in this region of flexural failure is largely meaningless. Thus, shear capacity is defined as:

$$V_{cap} = V_{f \mid M < M_{cap}} = V_{cap}(P, M) \tag{4}$$

It is understood that $M < M_{cap}$ in Eq. (4) implies moment along the ascending branches of sectional moment-curvature diagrams as shown in Fig. 1b.

PREDICTION OF FAILURE TYPE, ASSOCIATED MOMENT AND SHEAR

A reinforced concrete member segment will fail in flexure, flexure-shear, or shear, depending on whether it satisfies the greater than, equal to, or less than sign in the following relationship:

$$M/V >, =, \text{ or } < M_{cap}/V_{cap} \tag{5}$$

where M and V are the moment and shear at the most critical section within the segment, M_{cap} is the moment capacity of the section and V_{cap} is the shear capacity of the segment under the M/V ratio on the left hand side. Relation (5) implies a given amount of axial load on the section, which is reasonably invariant.

Relation (5) also implies loading under which moment at a section is proportional to shear at the section, so that the M/V ratio is fixed, i.e., proportional loading--be it monotonic or cyclic. Under variable loads which cause nonproportional moments and shears, the satisfaction of relation (5) is to be checked only when either M or V is at a maximum and is equal to M_{cap} or V_{cap}, respectively.

The current state of the knowledge of material properties, and the available analytical tools,[5] permit a fairly accurate estimate of M_{cap}. Unfortunately, the same is not true of V_{cap}. It is, therefore, advisable in the prediction of failure types to use a reasonable lower-bound value of V_{cap} in relation (5). The right hand side, then, will be smaller in reality than estimated. This would ensure that a predicted flexural failure will always be flexural, although a predicted flexure-shear failure may also turn out to be flexural, and a predicted shear failure may actually be in flexure-shear, or even in flexure. Since a flexural failure is more desirable than a shear failure, any under-estimate of V_{cap} may be considered to be "on the safe side".

It is apparent from the above discussion that at failure the moment and shear at a section must be as follows:

In the event of flexural failure,

$$M_f = M_{cap}, \qquad V_f = M_{cap}/(M/V) \tag{6}$$

In the event of flexure-shear failure,

$$M_f = M_{cap}, \qquad V_f = V_{cap} \tag{7}$$

In the event of shear failure,

$$M_f = (M/V)\, V_{cap}, \qquad V_f = V_{cap} \tag{8}$$

Relations (6), (7), and (8) imply that the moment and shear at a section at failure can be determined with the same accuracy with which V_{cap} can be estimated, if the M/V ratio is known or can be calculated, and if an accurate estimate of M_{cap} is available.

The above approach to the prediction of failure types, and the associated moment and shear, was tried on three series of structural wall specimens tested over the years at the Portland Cement Association laboratories[6-8] (Fig. 2). The geometric and material properties of these specimens are given in Refs. 6-8, and are not reproduced here. Six out of eight specimens in Barda's test series,[8] one specimen in Cardenas' test series,[7] and all the specimens in a current PCA investigation on structural walls[6] were tested under (essentially static, proportional) cyclic loading. The rest of the specimens were tested under proportional, monotonic loading. The flexural capacities listed in Table 1 were calculated by using the computer program described in Ref. 5. The shear capacities listed in Tables 1a, c, and d were calculated according to the provisions of Section 11.16 of the current ACI Code.[9] The ratios of these calculated moment and shear capacities are compared in Column 4 of each table with the actual moment-to-shear ratios to which the base sections of the test specimens are subjected. The failure types predicted on the basis of such comparison are listed in Column 5. There were two cases of anchorage failure in Cardenas' test series (Table 1c), and one specimen in the same series actually failed in flexure, rather than in flexure-shear as predicted (which is in accordance with prior discussion in this section). Apart from this, the actual failure types exactly matched the predicted ones. In cases of flexural failure, the failure shears predicted on the basis of Eq. (6) are compared with the actual shears at failure in Column 6. In the case of Cardenas' monotonically loaded specimens (Table 1c), the agreement is excellent, and reflects the accuracy of the flexural capacity estimates. The cyclically loaded specimens in the current PCA test series failed to attain the predicted flexural capacities due to premature buckling of the flexural reinforcement. This is reflected in the actual shears at failure being less than the failure shears predicted on the basis of Eq. (6). The actual moments at shear or flexure-shear failure are compared with moments predicted on the basis of Eq. (8) in Column 7.

The fairly large discrepancies reflect inaccuracies in shear capacity prediction. It should be noted that Table 1b is the same as Table 1a, except that the shear capacities are calculated using the provisions of Section 11.9 (special provisions for deep beams), rather than those of Section 11.16 (special provisions for walls) of ACI 318-71.[9] It can be seen that shear capacity estimates have improved somewhat in Table 1b, which implies that the proportions of Barda's specimens[8] make Section 11.9 more applicable perhaps than Section 11.16.

Table 1 demonstrates the validity of the present approach to prediction of failure type as well as the moment and shear associated with failure.

DESIGN UNDER VARIABLE REVERSING LOADS

In multi-degree of freedom systems subject to variable reversing loads (e.g., seismic excitations), the relationship between moment and shear at a given section is variable, due to the effect of the various modes of vibration. Figure 3a, which is a composite representation of the variation with time of the moment and shear at the base of an isolated structural wall subject to earthquake input motion, demonstrates clearly that the moment and shear are not in phase with each other. Thus, the M/V ratio varies continuously during the motion, as indicated in Fig. 3b. The various moment-shear combinations to which the critical section at the base is subjected during the motion may be plotted as shown in Fig. 3c. The envelope of these plotted points contains the critical moment-shear combinations that must be considered in design. On the basis of a preliminary analytical study conducted at Portland Cement Association, it has been determined[10] that for design purposes the critical portion of the envelope is covered if the following moment-shear combinations are considered:

1. The maximum moment, M_{max}, and the corresponding shear, V_M (point A, Fig. 3c).

2. The maximum shear, V_{max}, and the corresponding moment, M_V (point B, Fig. 3c).

3. The combination of M_{max} and V_{max} assumed to occur simultaneously (this combination approximates points intermediate between A and B).

The problem considered here is that of designing a structural wall, given M_{max}, V_M and M_V, V_{max}. Two procedures are suggested--one for intended failure in flexure and another for cases where the wall proportions and other factors make it impractical and unnecessary to aim for a flexural failure.

If M_{cap}, V_{cap}, are the design moment capacity of the critical section and the design shear capacity of the critical segment, respectively, then it is suggested that for intended flexural failure one ought to have:

$$M_{cap} = (\nless)\, M_{max} \quad \text{and} \quad V_{cap} > V_{max} \tag{9}$$

This would ensure, as required by relation (5), that

$$M_{max}/V_{max} > M_{cap}/V_{cap} \tag{10}$$

where V_{cap} must be the shear capacity available under the moment-to-shear ratio, M_{max}/V_{max} (V_{cap} is known to be a function of the moment-to-shear ratio[4]). Relations (9) should also ensure that

$$M_{max}/V_M > M_{cap}/V_{cap} \tag{11}$$

Any possible decrease in V_{cap} due to an increase in the moment-to-shear ratio from M_{max}/V_{max} to M_{max}/V_M must be considered in checking the satisfaction of relation (11). It should be noted that to ensure flexural failure it is not necessary to satisfy the relation

$$M_V/V_{max} > M_{cap}/V_{cap} \tag{12}$$

except when $M_V = M_{max}$, in which case (12) reduces to (10) which is satisfied.

When a design failing in shear appears to be the only practicable solution, one should make

$$V_{cap} = (\not<)\ V_{max} \quad \text{and} \quad M_{cap} > M_{max} \tag{13}$$

It can be easily checked that (13) ensures the satisfaction of the conditions for shear failure, as expressed by relation (5), with respect to load combinations 2 and 3 above. The condition need not be satisfied with respect to combination 1, except when $V_M = V_{max}$.

It should be noted that as V_{cap} approaches V_{max} in relation (9), or M_{cap} approaches M_{max} in relation (13), the mode of failure is going to approach flexure-shear. It is apparent that as far as strength is concerned, a design which fails in the flexure-shear mode is the most efficient and economical. However, if one considers deformability beyond failure (without a substantial loss of strength), a flexural failure may be more desirable than a flexure-shear failure. In such a case, it may be necessary to keep V_{max} below the shear which causes yielding of the shear reinforcement. This aspect of design for ductility is left to be discussed in a subsequent paper.

It is expected that in most design situations it will be fairly obvious whether it is necessary to aim for a flexural failure, or whether conditions are such (e.g., the case of a low, stocky wall) that a shear failure is not only permissible, but is about the only choice.

As to the determination of M_{max}, V_M and M_V, V_{max}, the most direct way of estimating them is of course through realistic analyses (including dynamic inelastic analysis if and when necessary) of the structure to be designed under the most probable input motion(s). Apart from this, an extensive investigation of earthquake-resistant structural walls currently under way at Portland Cement Association will hopefully result in design aids which would guide a designer to reasonable estimates of M_{max}, V_M, M_V, V_{max}, in the absence of refined analyses. It should be apparent, though, that unless one is in the range of M/V ratios (very low values) where V_{cap} is very much dependent on it, all that is necessary in design is a reasonable estimate of M_{max} and V_{max}.

SHEAR CAPACITY UNDER VARIABLE, REVERSING LOADS

The design approach outlined in the preceding section requires reasonable estimates of the shear capacity of a structural wall segment under variable reversing loads. Such estimates in turn require a clear understanding of the mechanism of shear resistance in structural walls subject to variable reversing loads, as well as of the various modes in which the resistance mechanism might fail.

Mechanism of Shear Resistance

The crack patterns in high rise, as well as (indirectly loaded) low rise structural walls subject to lateral loads reveals the formation of diagonal struts bound by the diagonal tension cracks (Figs. 4a and b). In the case of reversing loads, these struts are intersected by diagonal cracks formed by loading in the other direction, and are thus essentially discontinuous.

In high rise walls (directly as well as indirectly loaded), following the formation of diagonal cracks, much of the shear is resisted by truss action, with the diagonal struts acting in compression and the horizontal shear reinforcement actually carrying most of the horizontal shear. The horizontal shear reinforcement is thus extremely important in walls of this type. Vertical web reinforcement, if present, does not appear to play much of a role except in physically restraining the concrete in the compression struts from disintegrating at advanced stages of deformation.

In directly loaded low rise walls, the shear is disposed of along the shortest possible route (from load point to support) by arch action. Stirrups crossing the main diagonal crack, forming between load point and support, are not engaged in efficient shear resistance because no compression struts can form between stirrup anchorages.[3] This type of loading and the consequent shear carrying mechanism are uncommon in practice, and are therefore excluded from consideration in the remainder of this paper.

For the common structural wall in a building, the load is introduced along the joint between floor slabs and walls as a line load. No effective arch action can develop with this type of loading, even when the height is small. The crack pattern reveals the formation of diagonal struts (hence the engagement of stirrups), as sketched in Fig. 4c.[3] From consideration of equilibrium of the triangular free body, marked 1, it is evident that horizontal stirrups are required to resist the shearing stress applied along the top edge. The diagonal compression forces set up in the free body also require vertical reinforcement. In the free body bound by two diagonal cracks and marked 2, on the other hand, only vertical forces, equal to the shear intensity, need be generated to develop the necessary diagonal compression.[3] Figure 4c thus illustrates the role of vertical and horizontal bars in resisting shear forces in indirectly loaded low rise walls.

Modes of Shear Failure

Walls with no boundary element--In walls without boundary elements to offer restraining action, shear failure appears to be precipitated most often by sliding along a major diagonal crack after the shear reinforcement has yielded, (Fig. 5a). This type of failure, also encountered under monotonic loading, may be termed diagonal tension failure.

If and when the shear reinforcement is able to prevent sliding of the above kind, walls are observed to fail in direct sliding shear along their critical support sections. This is because, in the final stages of loading, irrespective of the amount of web reinforcement, the bulk of the shearing force has to be carried into the foundations across the concrete compression zone. However, by then the concrete in the compression zone areas is cracked as a result of the preceding load cycles, the cracks having opened and closed several times; therefore, the capacity to transfer shear is drastically reduced. A sliding shear failure of this kind, which is possible only under reversing loads, is illustrated in Fig. 5b.

Walls with boundary elements--When there are boundary elements of adequate stiffness to restrain sliding along diagonal cracks, the most commonplace shear failure appears to be by crushing of the compression struts. This type of failure must be distinguished from the usual web crushing failure under monotonic loading which is caused by inadequate thickness of the web. The failure being considered is the result of progressive 'softening' under repeated cycles of loading. When a diagonal strut formed by loading in one direction is intersected by diagonal cracks caused by loading in the other direction, the (axial) stiffness of the strut under loading in the first direction decreases. The amount of this decrease is a function of sliding movements along the intersecting cracks, which, in turn, are dependent on the shear reinforcement restraining this motion as well as the rigidities of the restraining boundary elements. One result of this loss of rigidity is that the force previously carried by the critically stressed compression strut gradually spreads to other adjacent struts, until an entire area of the web within intersecting diagonal struts is 'softened up' to the extent that it starts disintegrating. Figures 6a, b and d are illustrations of this type of failure.

The above process of disintegration usually causes a drastic reduction in the shear carrying capacity. However, depending upon the shear reinforcement and other factors, some or most of the shear may be sustained until the boundary elements fail, either in flexure (Fig. 6c) or in direct shear (Fig. 6e). The latter happens when the compression struts bearing against the compression flange cause a lateral bulging of the flange at a section above the base, where there is less restraint to such movement than at the base. This type of failure (also observed by Bertero[11] in a recent test conducted at the University of California, Berkeley) can be delayed with adequate stiffness and proper confinement of the boundary elements, so that they retain their strength and rigidity into advanced stages of deformation.

Estimates of Shear Capacity

The preceding section underscores the difficulties involved in arriving at reasonable estimates of the shear capacity of structural wall segments subject to repeated reversing loads of large amplitudes. Indications are that at least the following variables must be considered in a realistic prediction equation: geometric proportions of the cross-sections; concrete strength; axial load level; the amount, distribution and strength of the horizontal shear reinforcement; the amount, distribution and strength of the vertical web reinforcement; the moment-to-shear ratio, including the manner of loading (direct/indirect); the relative rigidities of the boundary elements and the web, as well as the strength and confinement of the boundary elements. An additional difficulty stems from the fact that some of the above variables assume more importance than others, depending upon the mode of failure. In view of the scarcity of available test results, one cannot even begin to develop a realistic prediction equation. For design purposes, the ACI Code(9) does contain prediction equations (Section 11.16) which are based on the monotonic loading tests carried out by Cardenas.(7) The specimens in this test series ranged in nondimensional moment-to-shear ratios ($M/V\ell_w$) from 1 to 2.4, did not contain any boundary elements; and the vertical web reinforcement was not considered as a variable. It is apparent that careful consideration must be given before applying the above provisions to situations not covered by Cardenas' tests.

It may be of interest to note here that the Japanese code for reinforced concrete recognizes the importance of the boundary elements in the shear transfer mechanism of structural walls. In fact, the commentaries to this code recommend certain minimum dimensions of the boundary elements relative to those of the web,(12) which are designed to ensure a minimum rigidity of the boundary elements relative to that of the web.

For the purposes of this paper it was thought worthwhile, instead of just pointing out the difficulties of arriving at shear capacity estimates, to check how the ACI code provisions, imperfect as they are, compare with the few test results that are available for structural walls subject to cyclic loading. The ratios of the shear capacities, as observed in tests and as computed according to Section 11.16 of ACI 318-71,[9] of eleven test specimens subjected to cyclic loading are plotted against their $M/V\ell_w$ ratios in Fig. 7. The only specimen with a test capacity lower than the calculated strength was one of Barda's walls containing no vertical web reinforcement. Such reinforcement has been shown to be extremely important in squat walls. For another of Barda's specimens, an unrealistically high ratio of test capacity to calculated capacity was obtained. This wall contained no horizontal reinforcement, so that the calculated capacity was very low. The vertical reinforcement in the specimen, not considered in the ACI equation, but in reality the principal shear carrying element in squat walls, produced a much higher test capacity.

Overall, Fig. 7 appears to indicate that, pending the development of more refined prediction equations, the ACI provisions may be used to arrive at conservative lower bound estimates of the shear capacity of critical structural wall segments subject to repeated, reversing lateral loads.

CONCLUDING REMARKS

The strength design of structural walls with reference to high shear under variable, earthquake-type loading has been discussed in this paper. It has been pointed out that the parameters governing the mode of response of such members are the moment and shear at, and the moment capacity and shear capacity of, the critical segments. While the moments and shears can be determined from analysis, given the input motion(s) to be expected, and while realistic estimates of moment capacity are not difficult to arrive at,[5] difficulties in estimating the shear capacity leads to uncertainties and inaccuracies in the prediction of response. Uncertainties in the design of such walls to prevent shear failure under earthquake-type loading also stem from the same difficulties. A strong case can and should thus be made for further research directed towards the accurate determination of shear capacities of structural wall segments under repeated, reversed loading, particularly as such capacities are affected by the rigidities of boundary elements restraining deformations caused by shear.

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CAPTIONS

Table 1: Prediction of type of failure, associated moment and shear-comparison with list results: (a,b) Barda's tests,[8] (c) Cardena's tests,[7] (d) current PCA tests.[6]

Fig. 1: Schematic representation of: (a) an axial load-bending moment-shear force interaction diagram, (b) moment-curvature diagrams under different shear force levels.

Fig. 2: Specimens and loading in various PCA test series: (a) Barda's tests,[8] (b) Cardena's tests,[7] (c) current PCA investigation.[6]

Fig. 3: Results of inelastic dynamic anlaysis of an isolated structural wall: (a) variations with time of bending moment and shear force, (b) variations with time of moment-to-shear ratio, (c) bending moment vs. shear force plot.

Fig. 4: The formation of diagonal struts: (a) Barda's specimen B3-2,[8] (b) current PCA specimen F1,[6] (c) shear resistance of low-rise walls.[3]

Fig. 5: Shear failure of walls without boundary elements: (a) diagonal tension failure,[3] (b) sliding shear failure.[3]

Fig. 6: Shear filaure of walls with boundary elements: (a) Barda's specimen B3-2,[8] (b,c) current PCA specimen F1,[6] (d,e) current PCA specimen B2.[6]

Fig. 7: Comparison of shear capacities as measured in tests and as given by ACI Code[9] provisions.

Spec.	Calc. Flex. Cap. (Ghosh[7]) k-ft	Calc. Shear Cap. (ACI[9])* k	M/V ft vs. $\frac{M_{cap}}{V_{cap}}$ ft	Failure Mode	Shear at Failure Actual k	Moment at Failure Expected k-ft	Moment at Failure Actual k-ft	Remarks
B1-1	1575	155.5	3.125 << 10.13	Shear	273	185.9	856	Monotonic Loading
B2-1	3693	149.9	3.125 << 14.94	Shear	220	465.4	687	Monotonic Loading
B3-2	2620	163.6	3.125 << 15.01	Shear	249	511.3	779	
B4-3	2007	49.0	3.125 << 53.33	Shear	229	153.1	715	No hor. reinf.
B5-4	2623	167.2	3.125 << 15.69	Shear	157	522.5	491	No vert. web reinf.
B6-4	2911	156.0	3.125 << 18.65	Shear	197	487.5	616	
B7-5	3031	159.9	1.5625 << 18.97	Shear	256	249.7	420	
B8-5	2874	156.1	6.25 << 18.41	Shear	199	975.6	1244	

* Section 11.16, ACI 318-71

(a)

Spec.	Calc. Flex. Cap. (Ghosh[7]) k-ft	Calc. Shear Cap. (ACI-Deep Beam[9])* k	M/V ft vs. $\frac{M_{cap}}{V_{cap}}$ ft	Failure Mode	Shear at Failure Actual k	Moment at Failure Expected k-ft	Moment at Failure Actual k-ft	Remarks
B1-1	1575	219.7	3.125 << 7.48	Shear	273	658.4	856	Monotonic Loading
B2-1	3693	196.7	3.125 << 18.77	Shear	220	614.7	687	Monotonic Loading
B3-2	2620	216.9	3.125 << 12.08	Shear	249	677.8	779	
B4-3	2007	179.9	3.125 << 16.16	Shear	229	562.2	715	
B5-4	2623	131.1	3.125 << 20.01	Shear	157	409.7	491	
B6-4	2911	156.2	3.125 << 18.64	Shear	197	488.1	616	
B7-5	3031	211.8	1.5625 << 14.31	Shear	256	330.9	400	
B8-5	2874	205.3	6.25 << 14.00	Shear	199	1283.1	1244	

* Section 11.9, ACI 318-71

(b)

Spec.	Calc. Flex. Cap. (Ghosh[7]) k-ft	Calc. Shear Cap. (ACI[9])* k	M/V ft vs. $\frac{M_{cap}}{V_{cap}}$ ft	Failure Mode	Shear at Failure Expected k	Shear at Failure Actual k	Moment at Failure Expected k-ft	Moment at Failure Actual k-ft	Remarks
SW-1	394	69.4	15 > 6.46	Flex	20.29	28.71		406	
SW-2	634.5	59.1	15 > 19.71	Flex	42.30	44.45		625	
SW-3	1169	54.2	15 < 19.16	Shear-Flex		71.50	880.3	1073	
SW-4	1212	55.3	8.25 < 11.46	Shear-Flex		126.70	799.4	1077	
SW-5	1102	94.7	8.25 < 11.64	Shear-Flex		126.70	781.3	1078	
SW-6	1142	60.2	15 < 18.97	Shear-Flex	76.15	78.34		1179	Actual failure in flexure
SW-7	1156	75.4	6.25 < 15.33	Shear		116.70	471.3	729	
SW-8	1004	78.8	6.25 < 12.74	Shear		128.10	492.5	801	
SW-9	1094	142.7	6.25 < 7.34	Shear-Flex		152.70	891.9	954	
SW-10	842	32.7	6.25 < 25.98	Shear		68.70	329.4	429	
SW-11	1192	151.5	6.25 < 7.73	Shear-Flex		137.50		856	Anchorage failure
SW-12	1192	157.4	6.25 < 7.37	Shear-Flex		148.00		925	Anchorage failure
SW-13	1004	142.1	6.25 < 7.07	Shear-Flex		142.10	888.1	889	Cyclic Loading

* Section 11.16, ACI 318-71

(c)

Spec.	Calc. Flex. Cap. (Ghosh[7]) k-ft	Calc. Shear Cap. (ACI[9])* k	M/V ft vs. $\frac{M_{cap}}{V_{cap}}$ ft	Failure Mode	Shear at Failure Expected k	Shear at Failure Actual k	Moment at Failure Expected k-ft	Moment at Failure Actual k-ft
F1	252.0x15	140	15 << 27.06	Shear		187.9	140x15 [illegible]	187.9x15
S1	67.1x15	82	15 > 12.27	Flex	67.1 [illegible]	61		61x15
S2	159.1x15	127	15 < 18.79	Shear Flex		152.8	127x15 [illegible]	152.8x15
R1	29.1x15	82	15 >> 5.32	Flex	29.1 [illegible]	26.6		26.6x15
S3	67.1x15	82	15 > 12.27	Flex	67.1 [illegible]	62		62x15

* Section 11.16, ACI 318-71

(d)

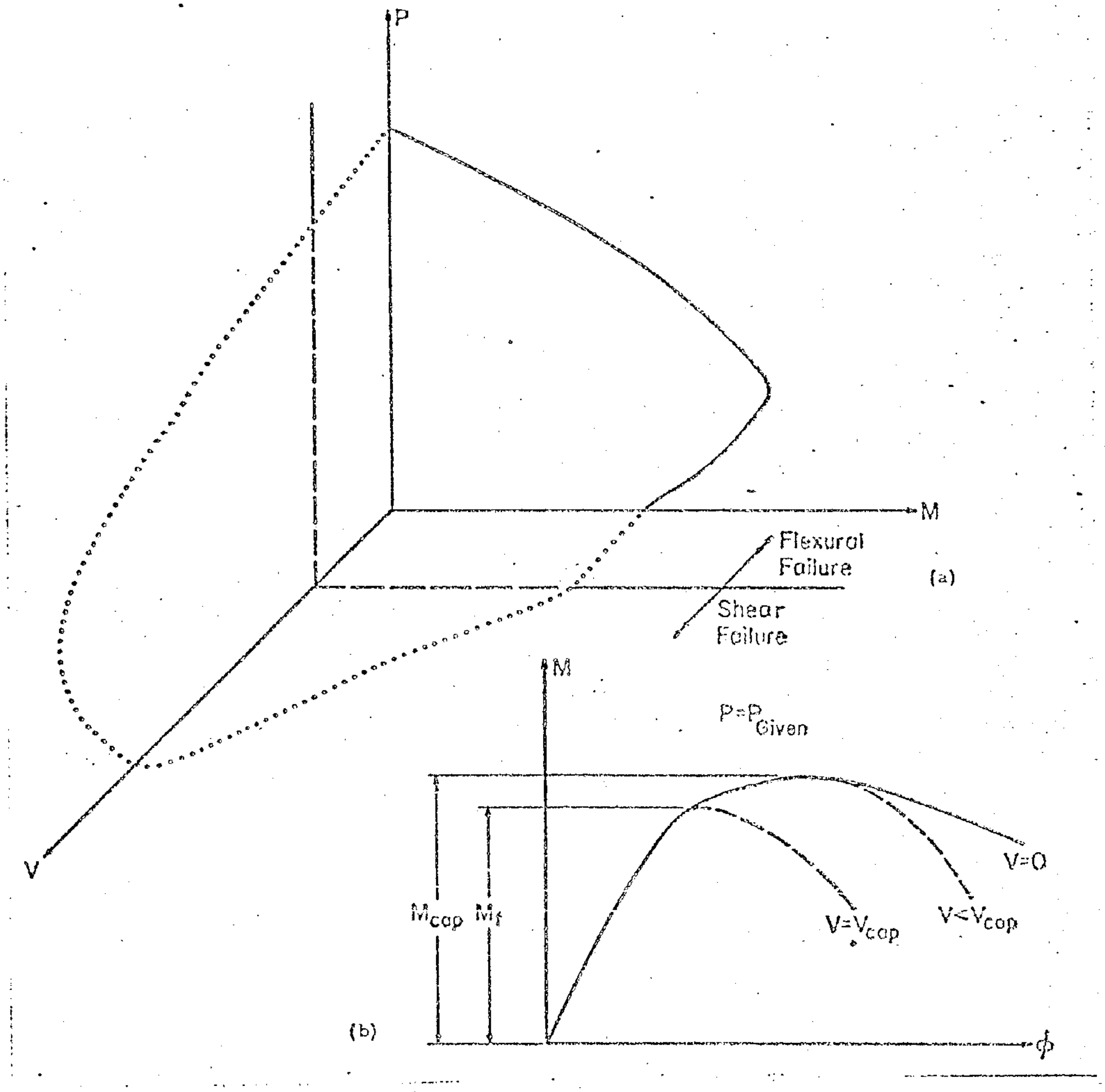

P
M
V
Flexural Failure
Shear Failure
(a)
M
$P = P_{Given}$
V=0
V=V_cap
V<V_cap
M_cap
M_f
(b)
φ

Fig. 1

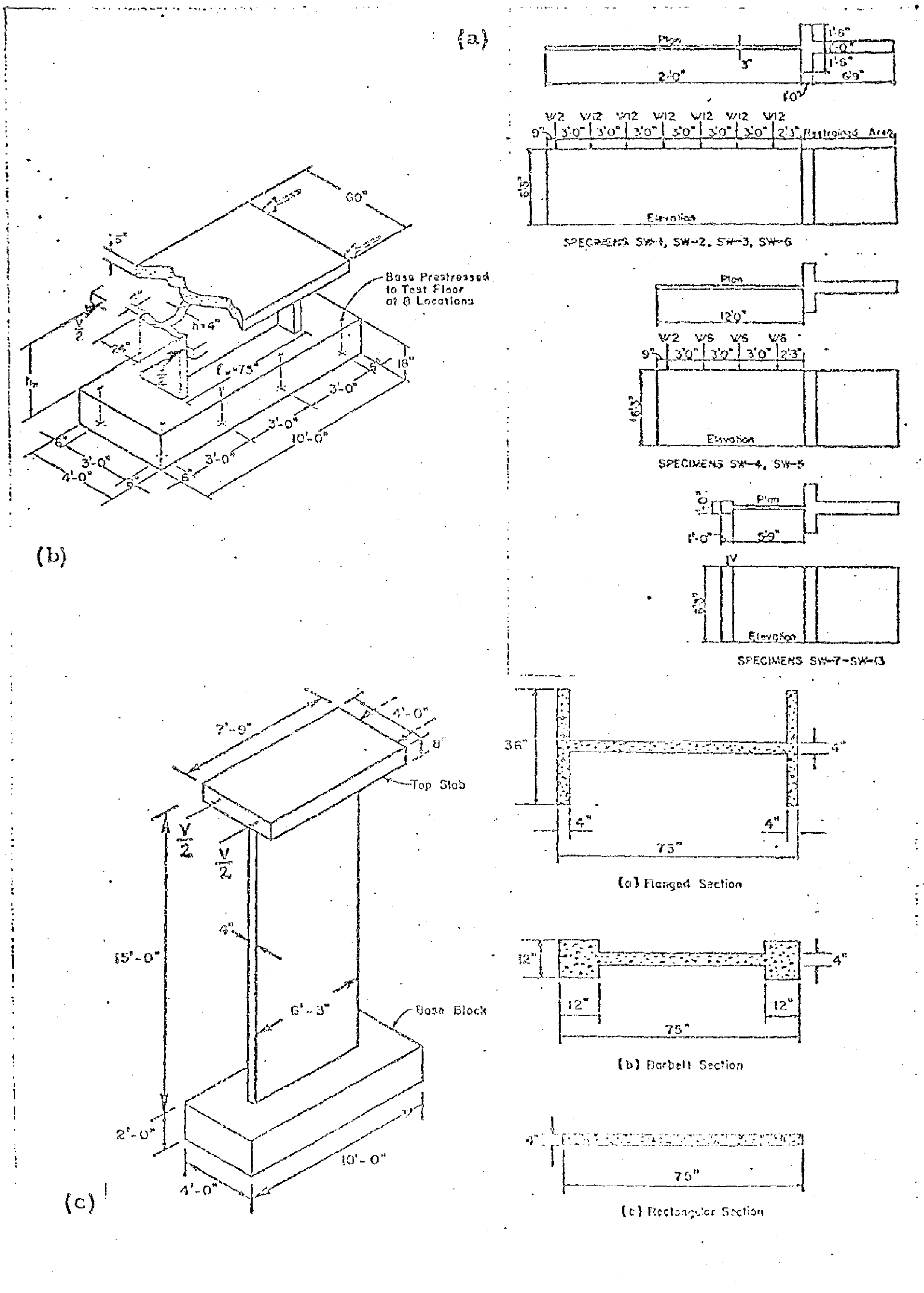

(a)
Plan
21'0"
1'6"
1'0"
1'6"
6'9"
1'0"
W2 W12 W12 W12 W12 W12 W12
9" 3'0" 3'0" 3'0" 3'0" 3'0" 3'0" 2'3" Restrained Area
Elevation
SPECIMENS SW-1, SW-2, SW-3, SW-6
Plan
12'0"
W2 W6 W6 W6
9" 3'0" 3'0" 3'0" 2'3"
Elevation
SPECIMENS SW-4, SW-5
Plan
1'-0"
5'9"
V
Elevation
SPECIMENS SW-7-SW-13
(b)
60"
Base Prestressed to Test Floor at 8 Locations
24"
3'-0"
3'-0"
10'-0"
3'-0"
4'-0"
18"
(c)
7'-9"
4'-0"
8"
Top Slab
V/2
V/2
15'-0"
4"
6'-3"
Base Block
2'-0"
10'-0"
4'-0"
36"
4"
4"
4"
75"
(a) Flanged Section
12"
4"
12"
12"
75"
(b) Barbell Section
4"
75"
(c) Rectangular Section

Fig. 2

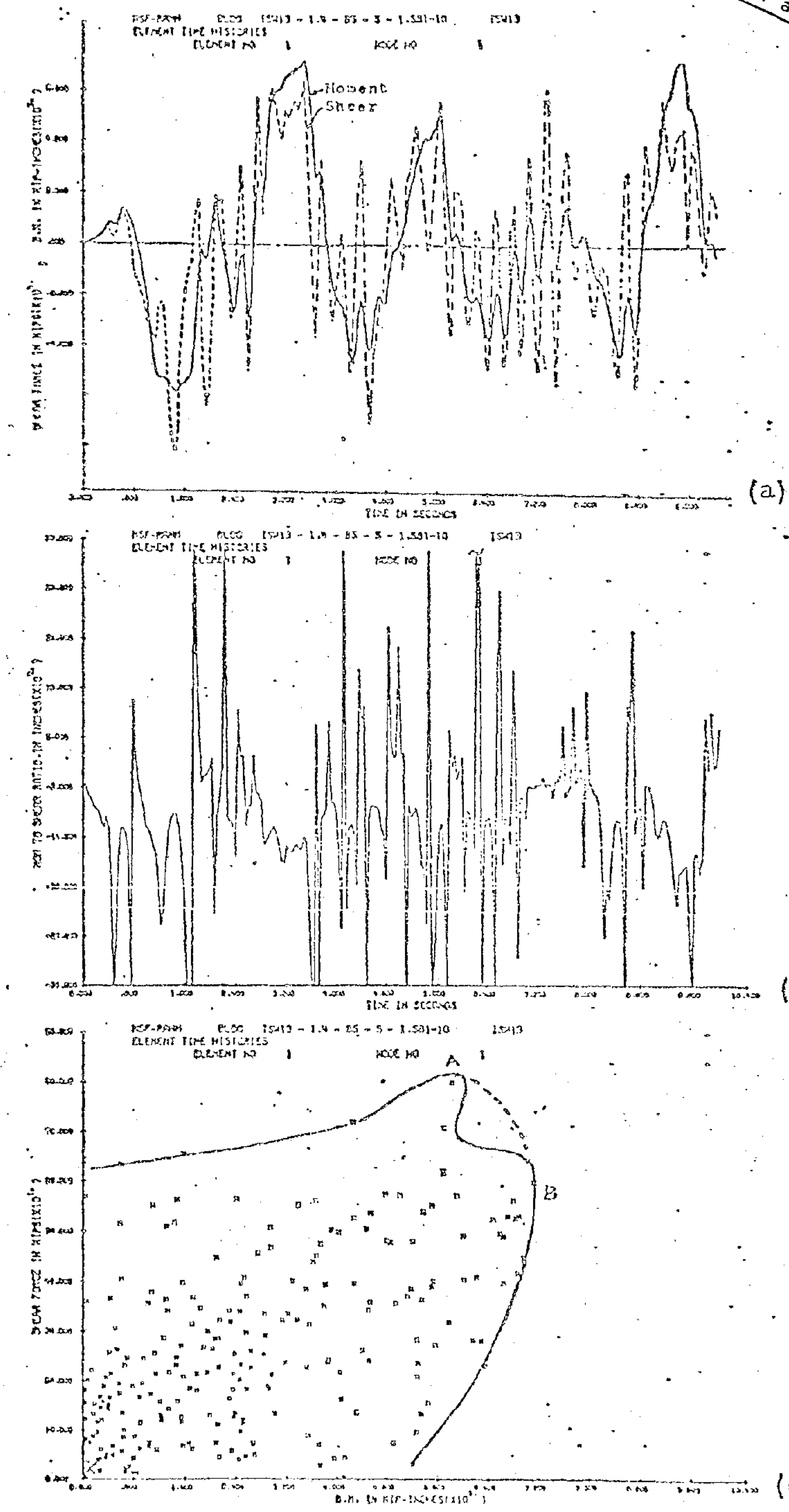
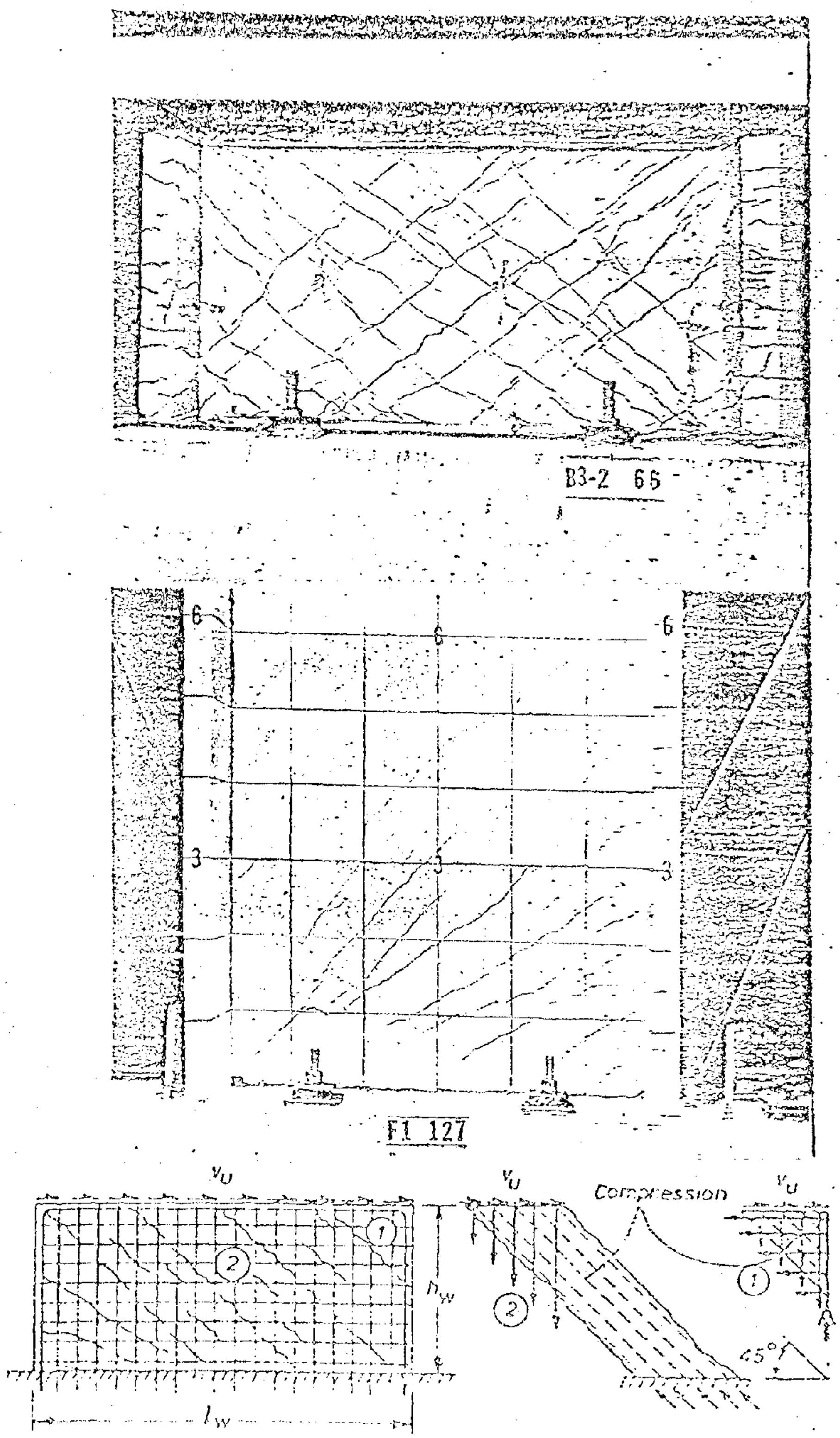

ELEMENT TIME HISTORIES
Moment
Shear
TIME IN SECONDS
(a)
ELEMENT TIME HISTORIES
TIME IN SECONDS
(b)
ELEMENT TIME HISTORIES
A
B
(c)

Fig. 3



(a)

(b)

(c)

Fig. 4

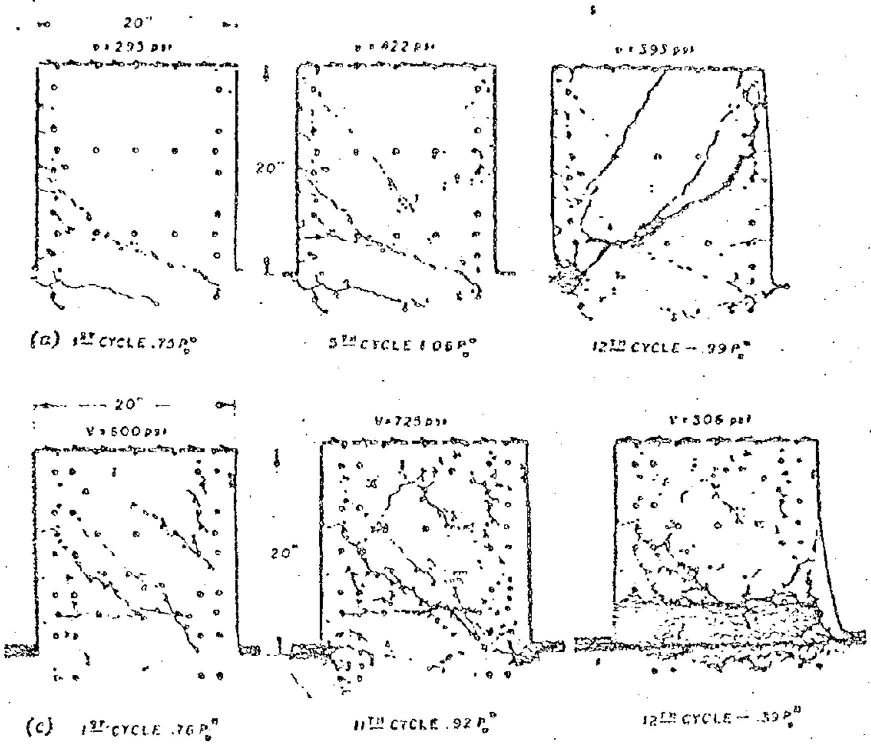

20"
v = 293 psi
v = 422 psi
v = 595 psi
20"
(a) 1ST CYCLE .75 P_o^o
5TH CYCLE 1.06 P_o^o
12TH CYCLE → .99 P_o^o
20"
V = 600 psi
V = 725 psi
V = 306 psi
20"
(c) 1ST CYCLE .76 P_o^o
11TH CYCLE .92 P_o^o
12TH CYCLE → .39 P_o^o

Fig. 5

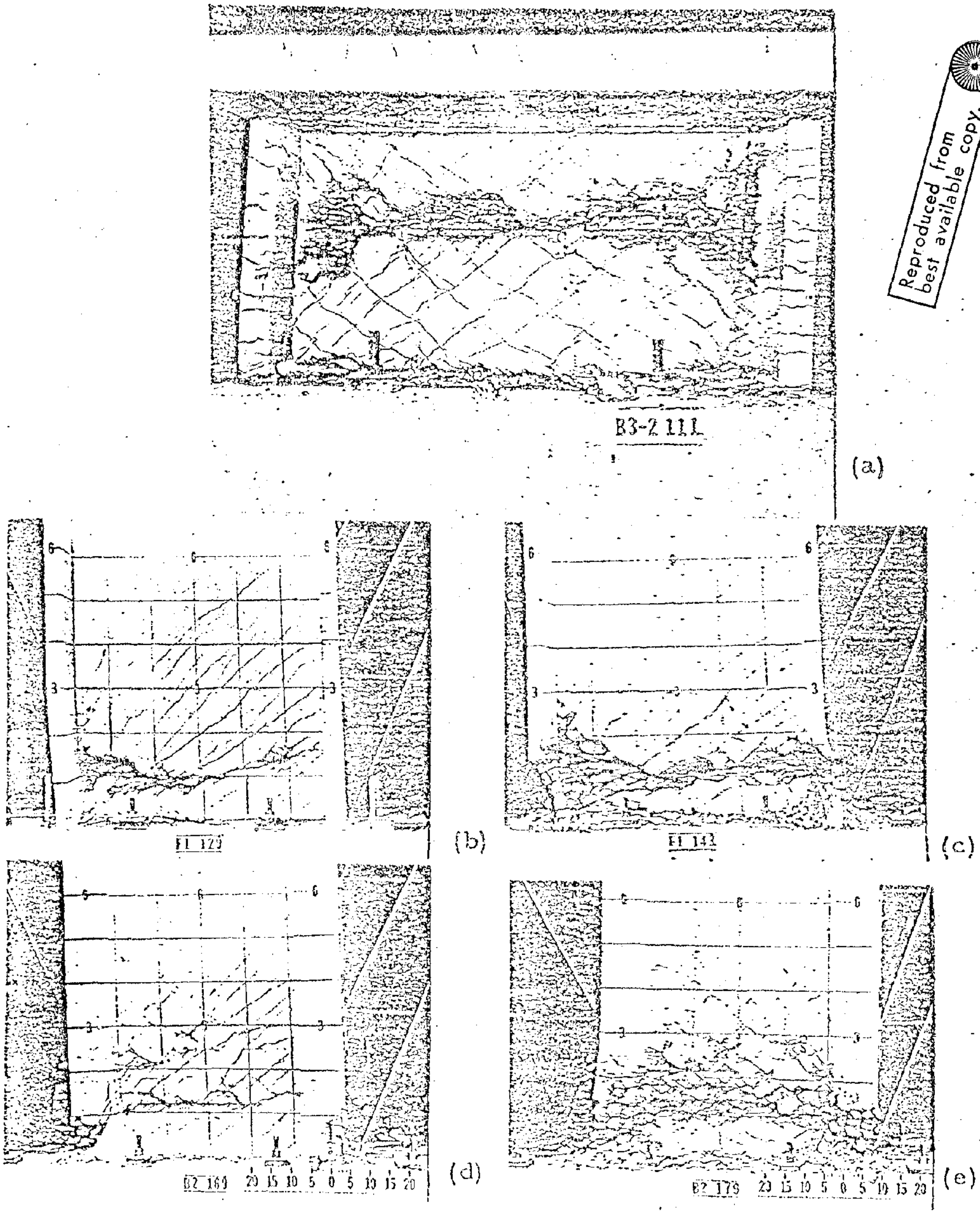
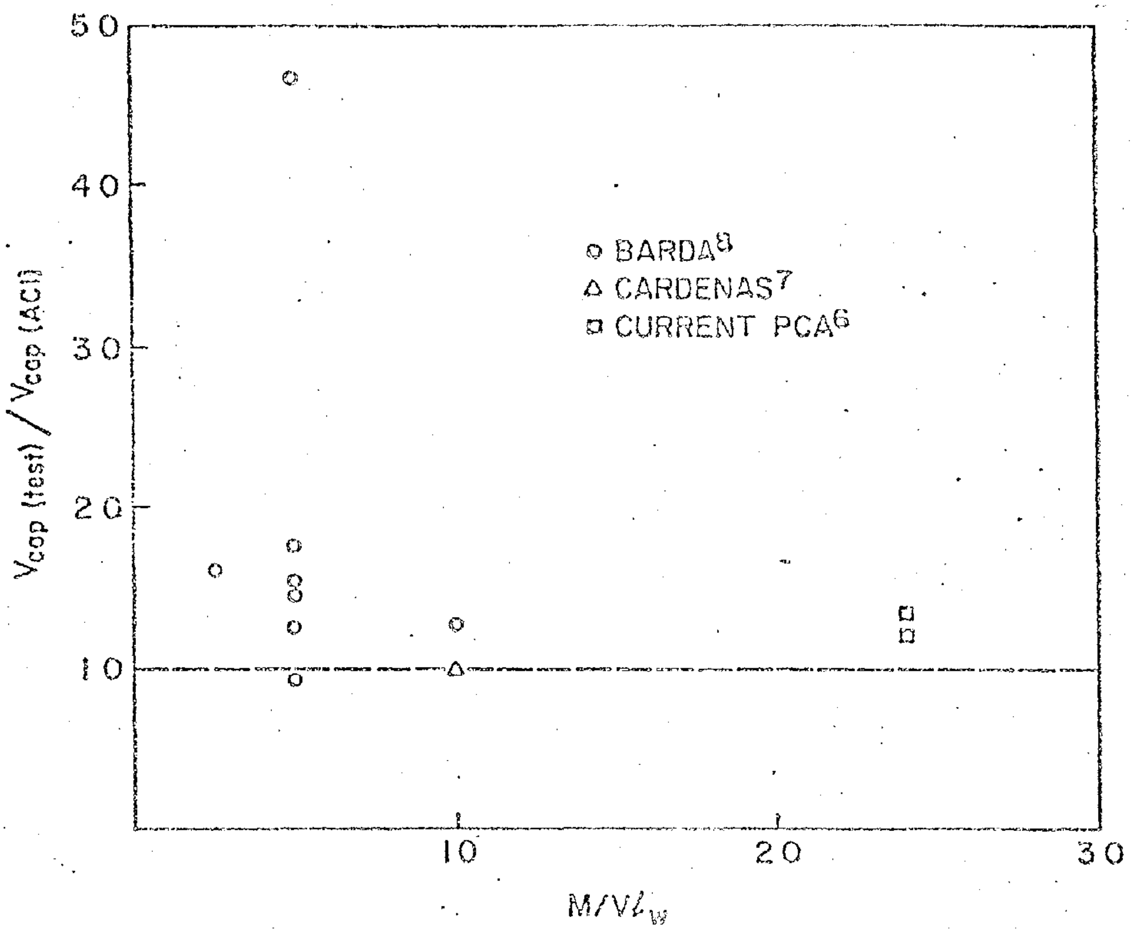


Fig. 6



50
40
30
20
10
10
20
30
○ BARDA8
△ CARDENAS7
□ CURRENT PCA6
Vcap (test) / Vcap (ACI)
M/Vlw

Fig. 7

272-101

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. Abstract (Limit: 200 words)

The strength design of structural walls with reference to high shear under variable, earthquake-type loading is discussed in this paper. The height-to-depth ratio has been considered as the parameter governing the mode of response of a structural (shear) wall. Short, stocky structural walls are supposed to be governed mainly by their shear strength, whereas slender walls are supposed to act like beam-columns controlled by flexure and axial load. It was shown that the parameters governing the mode of response of structural walls are the moment and shear at, and the moment capacity and shear capacity of, the critical segments. While the moments and shears can be determined from analysis, given the input motion to be expected, and while realistic estimates of moment capacity can be determined, difficulties in estimating the shear capacities leads to uncertainties and inaccuracies in prediction of response. The same difficulties may result in uncertainties in the design of such walls to prevent shear failure under earthquake-type loading. Further research should be directed towards the accurate determination of shear capacities of structural wall segments under repeated, reversed loading since such capacities are affected by the rigidity of boundary elements restraining deformation caused by shear.

. Document Analysis a. Descriptors

Walls
Shear stress
Earthquake resistant structures
Structural analysis
Illinois
Structural design
Loads (forces)

b. Identifiers/Open-Ended Terms

Structural walls
Shear walls
Skokie, Illinois

c. COSATI Field/Group

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