# EXPERIMENTAL STUDY OF SPANDREL WALL ASSEMBLIES

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## SYNOPSIS

The results of an experimental program to study the collapse of spandrel beam-column frames and to evaluate present U.S. code provisions, both with respect to design loads and design details is presented. The behavior and failure modes of half-scale, two-story, beam-column assemblages under cyclic induced lateral displacements and constant axial column loads is investigated. The brittle failure due to an inadequate shear capacity of the column sections is noted. Preliminary design recommendations to increase the lateral load resistance are presented.

## INTRODUCTION

Reinforced concrete spandrel wall frames with typical deep girders and short columns are commonly used to resist seismic loads. This system is mostly employed for buildings less than ten stories high. Hence, educational facilities, hospitals, and medium-rise office buildings are often designed with exterior spandrel walls. As substantiated by observations following a number of recent earthquakes, these systems exhibit little ductility in principle and mostly seem to fail predominantly in a brittle fashion through a shear failure in the short columns (1). The fact that the structures which suffered severe earthquake damage were mostly designed in accordance with existing local codes indicates the need for extensive research, particularly in order to establish means to develop the necessary ductility and strength for this specific type of structural frame. Hence, to study the behavior and collapse mechanism of spandrel beam-column systems and to evaluate the appropriateness of the present U.S. code provisions, both with respect to design loads and design details, an experimental program has been initiated at the University of California, Berkeley.

## PROTOTYPE STRUCTURE

In order to develop a realistic test element, a typical building frame, designed according to present U.S. code provisions, was considered. The prototype building was assumed to have a symmetric floor plan of 120 ft. by 120 ft. Since the majority of spandrel-wall buildings are of low and medium rise, the prototype structure was chosen to be ten stories high with a story height of 12 ft. The structural system in each principle direction was composed of two exterior spandrel wall frames and four interior typical beam-column frames. The lateral load resistance was provided by the exterior frames while the interior frames were designed to carry vertical loads only. The depth and width of the girders were determined by architectural considerations. The column cross-sectional dimensions were assumed identical throughout the building. The column reinforcement was designed to accommodate the varying axial loads.

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The lateral force analysis was based on the U.S. Uniform Building Code, assuming a horizontal force factor for the building of K = 1.00, and a zoning factor of Z = 1. Columns and beam-wall sections are designed according to the 1971 ACI Code using the ultimate-strength design method. Design details to meet confined-concrete requirements for the columns as would apply for a ductile moment resistant frame were disregarded.

## MODEL SELECTION AND DETAILS

In order to study the behavior of the spandrel wall frame under different lateral and vertical load combinations four test assemblages representing conditions existing at the first, fifth and tenth floor levels were selected.

In designing the actual test model the following factors were considered:

1) The end conditions of the model should represent load induced conditions of the prototype which are easily simulated in the test set-up, e.g. the inflection points of the structural members can be simulated by hinged supports in the model assembly.
2) The model should include one entire column to permit the study of this member under combined shear, moment, and axial load.
3) The model should be statically determinate in order that all forces and moments in any section can be evaluated.

Considering these factors and the basic lateral-load deformation pattern as shown in Fig. 1, a typical model configuration as presented in Fig. 2 was selected. The reduction in the depth at the ends of the beams was necessitated by the length of the hydraulic cylinders used in the testing system. This reduction seems to be totally acceptable since it will not affect the general behavior of the model. The selected test arrangement precludes a study of the static load effects in the beams. This decision seems to be justified since particularly at the lower stories, the earthquake forces are much higher than the static forces. Consequently, the beams were designed symmetrically (top and bottom) for earthquake forces only. In order to limit the load magnitude in the model and to permit the use of the actual structural materials a half-scale model size, allowing a simple scale reduction of the standard size reinforcement and aggregates, was selected. The reinforced concrete column and beam cross sections for the four tested models are shown in Fig. 3. Models 1, 2 and 3 represent the first, fifth and tenth floor conditions respectively. Model 4 also reflects the design of the 10th floor column, but with a column stirrup spacing of 8 in. rather than the 5 1/2 in. spacing required according to the ACI Code and introduced in Model 3. The column reinforcement models for 2, 3 and 4 are identical because of minimum percentage requirements for the longitudinal reinforcement in the upper story column. The main column and beam reinforcing steel had minimum specified yield stress values of 60 and 40 ksi respectively. The No. 2 deformed tie bars had a minimum yield stress of 40 ksi. The 28-day concrete strength of the four models was established as 5000, 4800, 5700 and 6400 psi respectively.

## TEST ASSEMBLY

A general plan of the test assembly is shown in Fig. 2. The specimens were placed in a horizontal position above the test floor and supported at five points on low-friction Teflon bearing pads. The reactions to the applied cylinder loads were provided by five reinforced concrete "reaction blocks", each bolted to the test floor by nine prestressing rods. Fig. 4 gives a general view of the test set-up. The cylinder load capacities for the axial and lateral loads on the column were 300 and 200 kips respectively.

Two hydraulic cylinders, with a capacity of 120 kips each were used at the ends of the beams and were operated electronically according to the signal taken from the potentiometer monitoring changes in the total column length of 6 feet. This closed-loop servo-controlled arrangement served two purposes, namely:

1) to keep the two beams parallel to each other during the horizontal cyclic displacements; and
2) to reduce the distance between the beams when any axial shortening of the column would occur and additional moments in the beams would be introduced.

As a result of these displacement-controlled conditions appropriate shear forces in the beams were introduced.

## INSTRUMENTATION

The test assembly was designed with a sufficient number of load-cells and linear potentiometers to define the forces on any part of the model and to determine in detail the deformed shape of the system. Hence, the axial and lateral loads acting on the column, the forces in both the beam-end cylinders and in the reaction links were registered throughout the test. The total deflections at various locations of the model, were recorded simultaneously. The model was instrumented externally to measure the average curvature distribution along the column and beams. For this purpose, "clip gages" were installed between the aluminum frames which were mounted onto the test model. This set-up, shown in Fig. 4, also permitted evaluating the moment-curvature relations at various sections of the beams and the column.

To measure internal strains some sections were instrumented internally by means of waterproof electrical strain gages welded to the steel bars. This instrumentation allowed a comparison of the local curvature with the average values obtained from the clip gage data.

## TEST SCHEDULE

Since the loading sequence affects the ultimate behavior of structural elements, different load histories for a single assembly could have been studied. However, since the primary objective of these initial studies was to evaluate the response of the beam-column assemblies at different levels in the building, each specimen was subjected to the following load and displacement schedule:

1) Application of an axial column load;
2) Application of a symmetric lateral displacement cycle ($\Delta$, $-\Delta$), by means of the laterally acting 200 kips cylinder and repetition of the same for total of five cycles;
3) Introduction of the same sequence as under 2) for stepwise increased

displacements until failure occurred.

Complete data recording in each sequence was limited to the first and fifth cycle and involved the recording of all data for lateral column displacements in the model of zero, $\Delta$, zero, $-\Delta$, and zero inches. The horizontal displacements $\Delta$ at the top of the column and the associated horizontal load were monitored continuously throughout the test on the X-Y recorder. This data was a direct indication of the stiffness degradation of the spandrel wall assembly. Similarly, a limited number of clip-gage data versus the horizontal load were recorded continuously on X-Y-Y recorders.

## TEST RESULTS

The test results for the four models indicated clearly that prior to failure the load-displacement hysteresis loops remained virtually constant for each series of five induced displacement cycles. Furthermore, it was observed that the first and fifth floor assemblages (Models 1 and 2) failed at almost twice the displacement at which the maximum load (Vm) was reached. For Model 1 the maximum registered load of about 60 kips occurred under a displacement of $\Delta$ = 1.25 in. Failure occurred after a displacement of 2.5 in. had been reached. For Model 2 the corresponding values were: 56 kips, 1.25 in. and 2.5 in. respectively. The load at failure in both instances was approximately 20% less than the maximum value observed. Both 10th-floor assemblies (Models 3 and 4) failed rather abruptly. After attaining a displacement of 1.25 in. and a maximum load of 40 kips, Model 3 failed after having reached a displacement of 1.5 in. The load at failure was virtually identical to the maximum load value. Model 4 failed abruptly after having reached a maximum load of 38 kips and a corresponding displacement of 1.25 inches. A typical partial composite of the first cycle load-displacement loops for Model 1 is presented in Fig. 5. The curves for $\Delta = \pm 0.20$ and $\pm 0.40$ in. have been omitted for sake of clarity. The consistent drop in the load ($\Delta V$) can be attributed to the friction of the assembly over the Teflon bearing pads at locations A, B, C, D and E as identified in Fig. 2. The observed failure modes as shown typically in Fig. 6 clearly exhibited the diagonal-tension shear cracking. With this shear crack development, the lateral load is virtually entirely carried by the column-tie stirrups and the dowel action of the longitudinal column reinforcing steel. It should be noted that except for Model 4 the column-tie design was based on standard ACI Code provision, using the ultimate strength design method.

The basic results of the four model tests are presented graphically in Fig. 7. This figure presents the typical column interaction curves - ultimate axial load $P_u$ versus ultimate Moment $M_u$ - for each of the four models. Also shown are the maximum shear loads $V_m$ for each of the models. These values are shown to scale in terms of the column moment $M = (h/2)(V)$, where $h$ is the clear length of the column, or 36 in. In addition, Fig. 7 presents graphs of the nominal permissible shear loads $V_c$ as based on the shear stress values $v_c$ specified by the ACI code. The straight lines show the increasing allowable shear load for an increasing axial load while the curved lines present the allowable shear load as influenced by the axial load and the moment.

The results for the four models indicate that the observed maximum loads $V_m$ are less than the ultimate shear loads $V_u$, namely: 86, 99, 93 and

87%. Hence, without reaching the ultimate moment capacity $M_u$ or $(V_u)(h/2)$, failure is invariably due to a premature shear failure of the column, as substantiated by the observed failure modes. A conclusion that, based on these rather high percentages, the different designs were virtually balanced (100%) is wrong and highly dangerous. The same applies to any observation as that the maximum loads $V_m$ safely surpass the code predicted shear capacities of the concrete $V_c$. In reality, the column before reaching $V_m$ has been cracked severely. The shear load at that stage is virtually entirely resisted by the strength of the stirrups and the dowel action of the longitudinal reinforcement. Actually, failure is invariably due to a tensile failure of the stirrup bars. Therefore, underdesigned stirrups will allow a brittle failure despite the high maximum shear load capacity ($V_m$ versus $V_u$). This is supported by the correlation between the yield load capacity $V_{ys}$ of the stirrups in terms of the maximum observed load $V_m$ and the "ductility ratio" at failure as expressed in terms of the ratio of the lateral displacements at failure and at the maximum observed load $V_m$. The definition of the ductility ratio implies a ratio of 1.00 in case of a totally brittle failure. Consequently increasing ratios reflect an increasingly ductile behavior. In that respect it can be noted that Model 4 which failed abruptly without exhibiting an increase of the maximum displacement - ductility ratio 1.00 - had a shear yield load $V_{ys}$ of only 0.32 $V_m$. Model 3 with displacements of 1.50 in. at failure and 1.25 in. at the maximum load, or a slightly larger ductility ratio of 1.20 also exhibited a slightly larger shear load capacity of $V_{ys} = 0.42\ V_m$. Remarkably enough the identical ductility ratios of 2.00 - or the ratio 2.50 in/1.25 in. of the displacements at failure and $V_m$ - observed for Models 1 and 2, corresponded with identical shear yield load capacities of the stirrups of $V_{ys} = 0.55\ V_m$. The shear yield load above is defined as: $V_{ys} = (d/s)(A_s)(F_{ys})$, where d = width of column, s = vertical stirrup spacing, $A_s$ = total stirrup cross sectional area and $F_{ys}$ = yield stress steel.

## CONCLUSIONS AND RECOMMENDATIONS

Laboratory studies of spandrel wall frame assemblages invariably showed a shear failure of the column section, despite the fact that the maximum shear loads virtually permitted the development of the ultimate moment capacity of the column section. Particularly for columns with low axial loads the failure is very brittle. The main cause of the sudden brittle column failure is an inadequate design of the column-tie stirrups. Ideally, to develop a ductile design it is necessary to develop a stirrup shear yield load resistance $V_{ys}$ equal to the ultimate shear load $V_u = (M_u)(h/2)$, where $M_u$ = ultimate column moment capacity and h = clear length of column. Hence, the shear yield load should be $V_{ys} = V_u = (\ell_e/s)(A_s)(F_{ys})$, where $\ell_e$ = vertical projection of the inclined shear crack. Depending on the relative axial load and moment values the shear crack pattern and the effective length $\ell_e$ will vary. However, at present it is recommended to determine the stirrup cross section $A_s$ by setting $\ell_e = d$ in the above formula. In order to engage a sufficient number of stirrups in resisting the shear load transfer across the shear-cracked section, the stirrup spacing should not be larger than 1/3d. Based on the present results one can expect that a spandrel wall system of the type studied and designed according to the above recommendations will perform significantly more ductile than observed in the reported tests.

In order to develop more specific design recommendations, further experimental and analytical studies are presently under way at the University of California, Berkeley. The support of the National Science Foundation Grant GI-31883 to carry out these studies is gratefully acknowledged.

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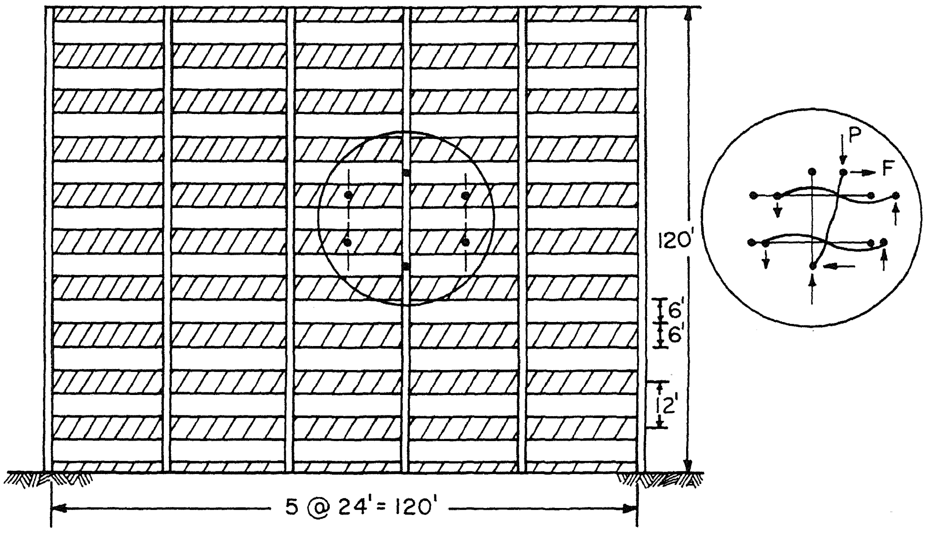


FIG. 1 SCHEMATIC FRAME AND LATERAL DISPLACEMENT PATTERN.

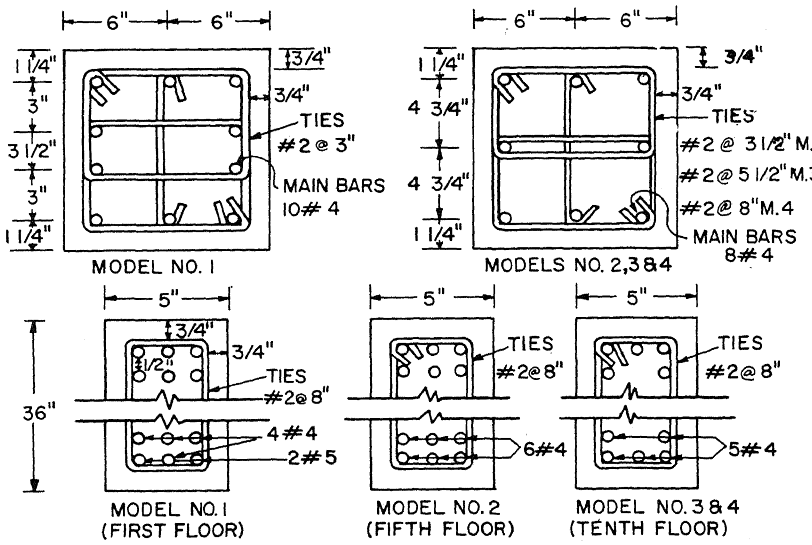


FIG. 3 COLUMN AND BEAM SECTIONS FOR THE TEST MODEL

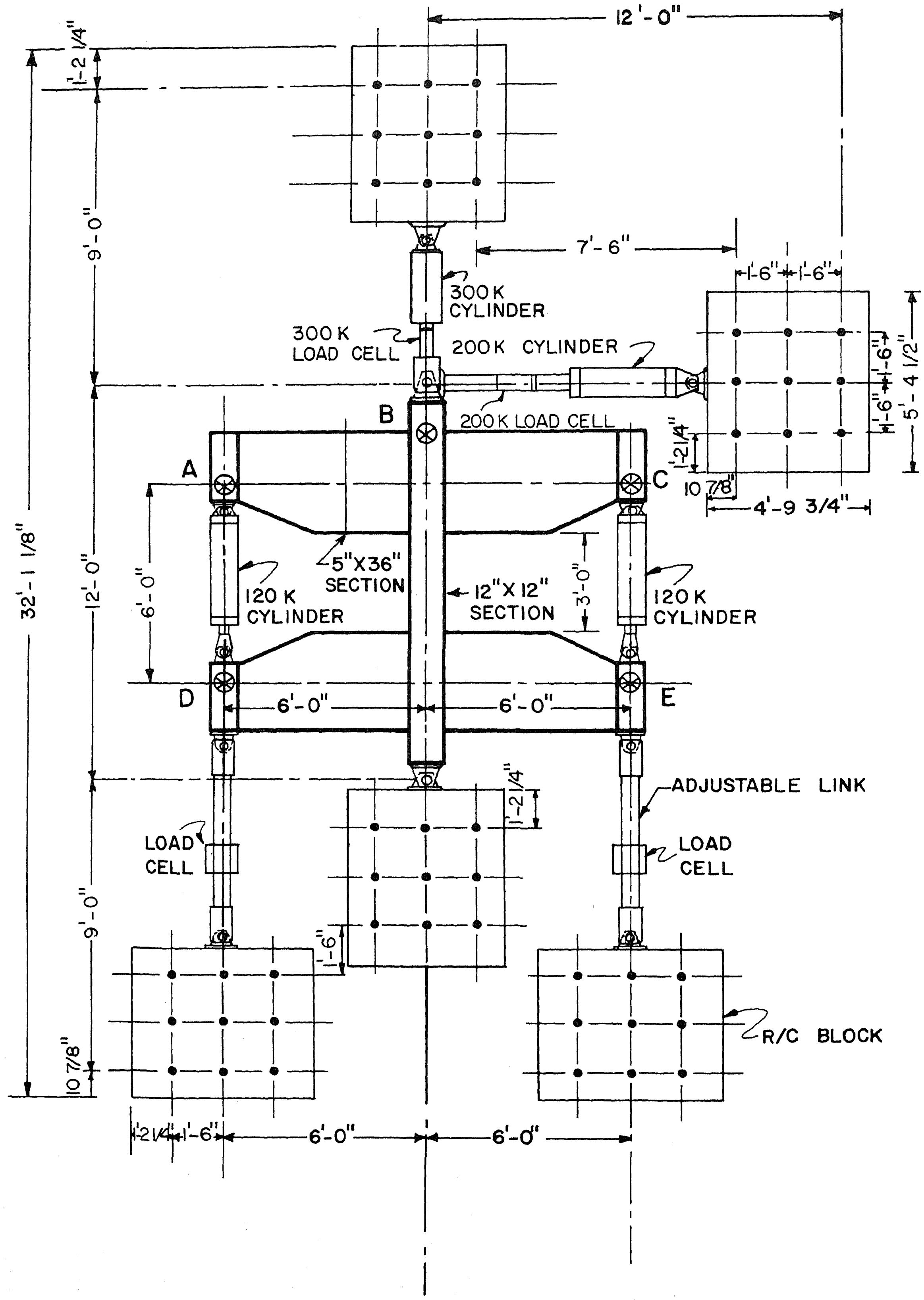


FIGURE 2
GENERAL PLAN OF THE TEST ASSEMBLY

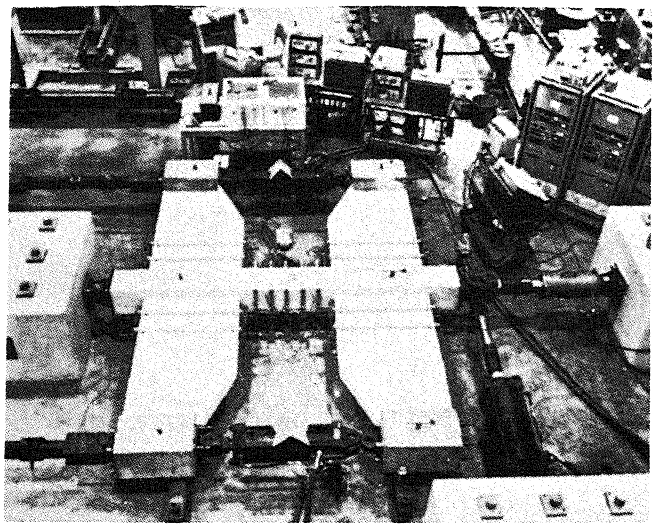

FIG. 4 GENERAL VIEW BEFORE TEST ASSEMBLY

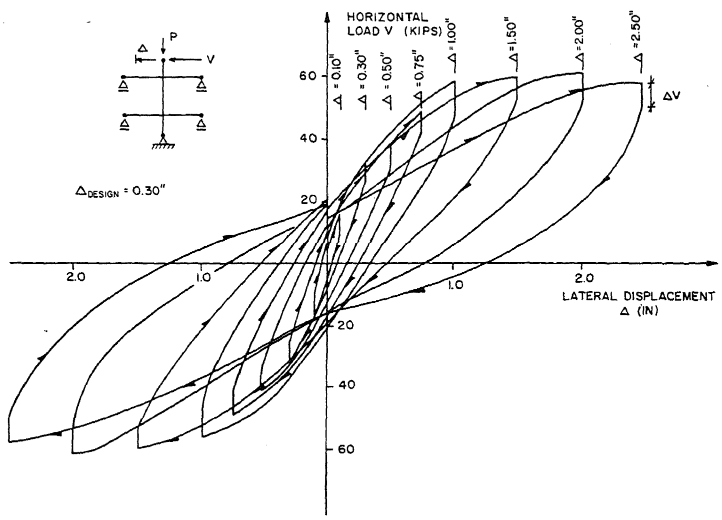


FIG. 5 COMPOSITE OF FIRST LOAD VS. DISPLACEMENT CYCLES FOR MODEL 1

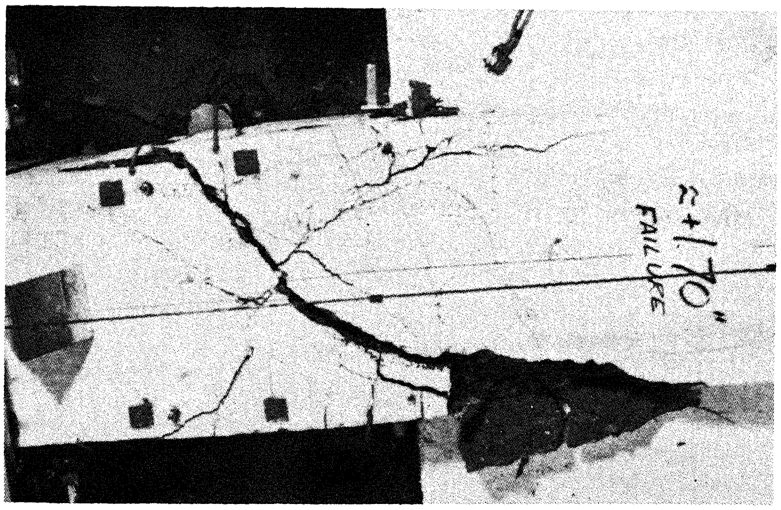


FIG. 6 TYPICAL FAILURE OF THE COLUMN

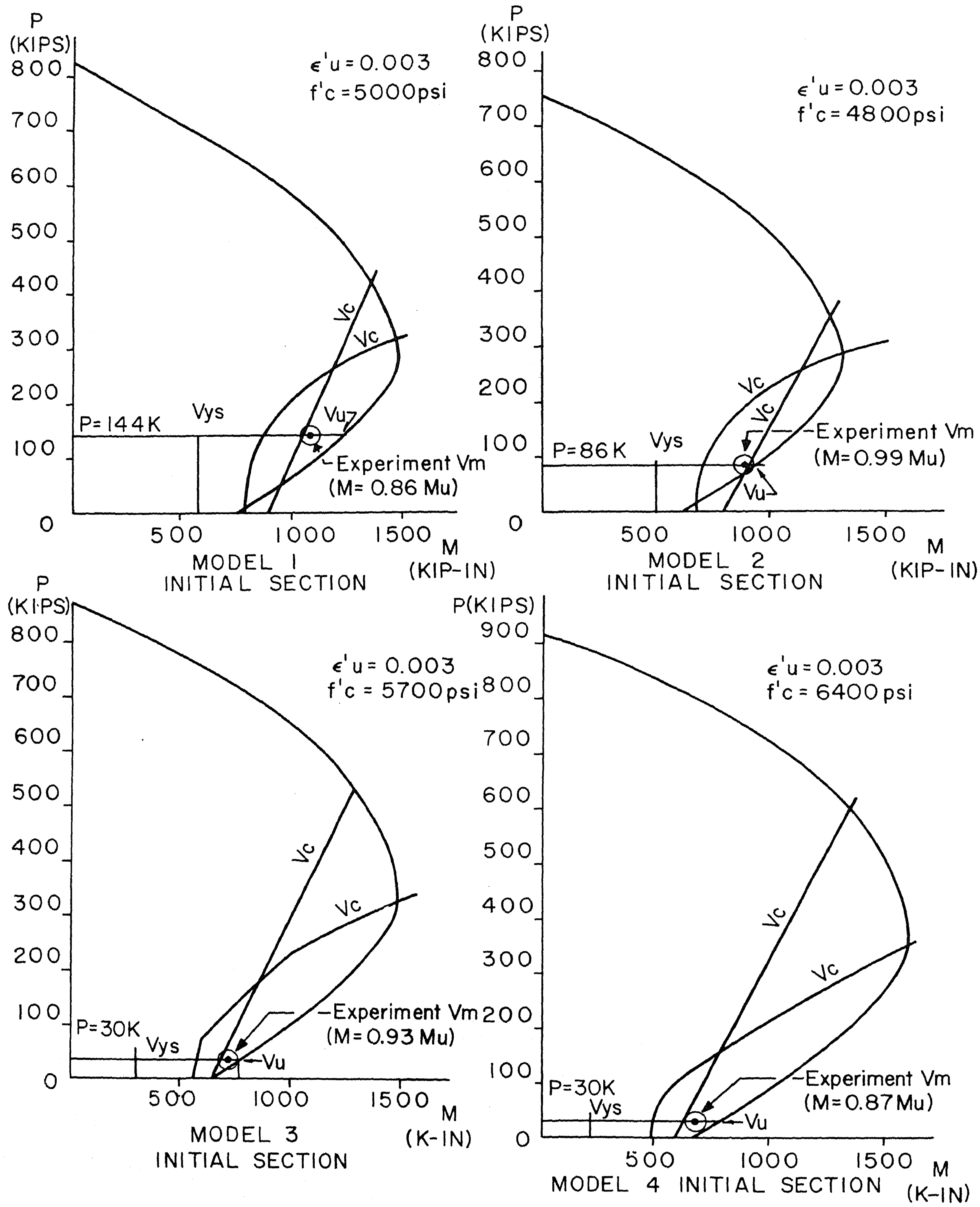


FIGURE 7

GRAPHICAL REPRESENTATION OF RESULTS