REPEATED AND REVERSED LOAD TESTS ON FULL-SCALE STEEL FRAMES

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ABSTRACT

The experimental hysteresis loops from testing two full size single-bay steel frames (one single-story and one three-story) under constant (working) gravity loads and a program of gradually increasing amplitudes of cyclic lateral displacements of the top of the frames are described. The results show a 40 percent increase in maximum load capacity over the maximum monotonically applied lateral load predicted by second-order elastic-plastic analysis. The significant effects on the maximum load capacity of strain hardening, the actual locations of the plastic hinges and the residual P-Δ moments existing in the frame when reversed loading begins, are also discussed. It is shown that the hysteresis loops are very stable for displacements greater than those even corresponding to maximum lateral load. Also included is a description of the problems that are being further investigated in a current investigation.

INTRODUCTION

In recent years, a considerable amount of tests have been conducted on structures and structural components under repeated and reversed loads. These studies have resulted in a better understanding of the response of structures during an earthquake. In a series of tests at the University of California (Berkeley) cantilever beams were tested to study the behavior of these beams near the connecting zone (Refs. 1 and 2). Additional reversed bending tests on different types of beams have been performed by others (Refs. 3, 4 and 5). Beam-columns have been tested under constant axial loads and alternating bending moments (Refs. 6 and 7). Reversed loading tests on beam-and-column subassemblages have also been conducted at Kyoto University (Ref. 8). Several reversed load tests of model frames and small frames of W cross section have been reported by Japanese researchers (Refs. 9, 10, 11 and 12). In addition, as an adjunct to recent tests of multi-story frames at Lehigh University to study the static behavior under a monotonic load application, four frames were tested under a reversed loading after large inelastic deformations due to the initial loading (Refs. 13, 14 and 15).

These latter tests showed that currently available methods of analysis are adequate to predict the static behavior of multi-story frames under the combined effect of gravity and monotonically in-

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creasing lateral loads. However, these methods were also shown to be inadequate to describe the static behavior of the frames under reversed loading even for relatively simple structures.

Therefore a research program has been initiated at Lehigh University as a continuation in plastic design research from static to dynamic problems and dealing primarily with the dynamic effects of earthquakes on multi-story frames. In the experimental portion of this program, two series of tests on full-scale single and multi-story frames are included. The experimental behavior of the frames tested in the first series and some theoretical interpretation of the results are discussed in this paper. Also included is a description of the essential features of the second series of tests.

DESIGN OF TEST FRAMES

The design of the two frames for the first test series was based on current aseismic design practice as applied to an eight-story, single-bay structure (Fig. 1). The columns of the prototype frame are likely to be bent in double curvature and have points of inflection at their midheights when lateral loads are applied. It is therefore possible to form assemblages by subdividing the frame at the inflection points of the columns. A three-story assemblage that would represent levels 5, 6 and 7 of the frame is shown in Fig. 1. A single-story frame (Frame A) was then selected as the lowest story of the three-story frame (Frame B) and loaded accordingly.

Eighty pounds per square foot were used for dead load and full live load with an average live load reduction factor of 40 percent applied to both beams and columns. Since the portion of the building selected is in a region of small variation in the total aseismic design shear, the working design shear was taken as the summation of the design shears through level 5 (this working shear is equal to 3 1/2 percent of the sum of the dead loads through level 7). The geometry of the frame and member sizes were selected to have representative ratio of column-to-beam stiffness.

The frame members were designed plastically (Ref. 15) by initially assuming no frame instability or P-Δ effect and a likely-to-occur mechanism. A plastic moment balancing analysis was then performed to check that all moments were less than or equal to their full plastic values (or reduced values in case of columns). Using the moment diagram which resulted from the analysis and the member sizes, the deflections were calculated. The frames were redesigned including the P-Δ moments and the preliminary member sizes were altered if necessary.

Both frames were then analyzed for their second-order elastic behavior. The analyses were carried out by a digital computer at the working values of the monotonically increasing horizontal load and gravity load. The results obtained permitted checking the adequacy of the beams and columns with the allowable stresses specified in the AISC (American Institute of Steel Construction) Specification. In addition, the frames were evaluated under the working values of gravity load acting alone. In both loading cases satisfactory results were obtained for the member sizes and frame geometry indicated in Fig. 2.

MAXIMUM LOAD ANALYSES OF TEST FRAMES

To find the complete load-deflection curve for each frame under monotonically applied horizontal force and constant (working) gravity loads (Fig. 3), the previous combined load analyses were continued into the inelastic range past the point of frame instability. An automated computer program developed in Ref. 17 was used. The complete results are given in Fig. 4. The load-deflection curve for Frame A indicates that the frame instability load and the plastic mechanism load coincide at a lateral load of 14.8 kips. However the behavior of the three-story frame shows the frame to be unstable at a load of 15.3 kips before a mechanism is formed. All of the analyses were based on handbook values for cross sectional properties and on an assumed static yield stress level for ASTM-A36 steel of 36 ksi.

Comparing the working value of lateral load of 5.2 kips with the maximum capacity of either frame shows a considerable reserve of strength and source of possible material savings.

TEST SETUP AND LOADING PROGRAM

Both frames were tested under constant (working) gravity loads and a program of statically applied cyclic lateral displacements of the top of the frames similar to those used by E. P. Popov on cantilever beams (Refs. 1 and 2).

Two unique devices ere used to load and to brace the frames without offering any restraint to in-plane movements. Gravity load simulators were used to apply the constant gravity loads to the quarter-points of the beams through a spreader beam and to the column tops, and bracing linkages were used to prevent out-of-plane movements of the members of the frames (Ref. 18). The lateral displacement was produced by mechanically displacing the top of the frame. The testing arrangement for Frame A is shown in Fig. 5. The test setup for Frame B is shown in Fig. 6 in which the gravity load-simulators are shown positioned under the load spreader beams.

The boundary conditions imposed on the frames required no moment at the assumed points of inflection above and below the main portions of each frame. Pinned-bases utilizing roller bearings were used at the lower end of each of the lower half-story columns. A pinned-end tie beam between the two ends of the top half story columns was used to distribute the lateral force.

Displacements and rotations of various points throughout the frames were measured mechanically and electrically. Strain gages were used extensively throughout the structure. Computations from the strain gage readings and the measured deflections of the gaged points reduce the frames to determinate components. During the tests, complete sets of static readings were taken at suitable intervals to permit construction of the hysteresis loops.

Initially the gravity loads were applied to the frames and then sets of lateral displacements of increasing amplitudes were applied to the frames in a cyclic manner. In each case, the amplitudes to be cycled were selected to bracket the plastic hinge occurrences and other intermediate points on the respective load-deflection curves. For displacements in the elastic range three cycles were used at each amplitude and for inelastic range displacements five cycles were used. The number of replications at each amplitude was set to observe the stability of the hysteresis loops at the various amplitudes of deflection and inelastic conditions of the frames. The amplitudes selected for Frame A are superimposed on the load-deflection curve and the resulting displacement program is given in Fig. 7.

TEST RESULTS

Sixty cycles of increasing amplitudes of lateral displacements were applied to Frame A with a maximum displacement of 5.2 inches which is 14 times the deflection at the maximum lateral load. The three-story frame had 54 cycles at various amplitudes of displacement applied to it with a maximum cycled displacement of 10 inches. (At the conclusion of the test of Frame B, the top of the frame was displaced 13.5 inches east.) The largest cycled displacement is 9 times the displacement at working load and 1.5 times the deflection at the maximum predicted load. The approximate ratios above give an indication of the toughness and ductility of these steel frames.

The shape of the characteristic hysteresis loops are shown at selected displacement amplitudes for the three-story test in Fig. 8 and for the test of Frame A in Fig. 9a. In addition, the stability of the hysteresis loops are typified by the five cycles at the largest amplitude in the test of Frame A as shown in Fig. 9b. The replications of cycles at all amplitudes indicated stable hysteresis loops and, significantly, even when the amplitudes were larger than those corresponding to the maximum load.

Both tests showed a considerable increase in load carrying capacity for steel frames when subjected to cyclic lateral displacements. In each case, the maximum load the frames could withstand was about 40 percent greater than that predicted by the second-order elastic-plastic analysis of the frames under monotonically increasing lateral loads. (This percentage was computed after the analysis was redone with the actual experimental plastic moment values.) Also, for the single-story frame the deflection when the maximum load was reached was predicted closely by the monotonic analysis. But, for the three-story frame the maximum load occurred at a somewhat higher deflection (about 8 inches compared to the 6.8 inches predicted.)

In addition, on each of the large cycles once the deflection at the maximum load had been exceeded the load carrying capacity dropped and repeated loads will investigate the following three basic problems:

1. The effect of cross sectional instability in a beam member (local buckling).
2. The behavior of weak-axis connections and columns oriented for weak-axis bending with significant inelastic strains.
3. The behavior of plastic hinges in columns oriented for strong axis bending.

The first problem will be studied in a single-story frame which will be a duplicate of Frame A except that a non-compact section (in the context of plastic design) will be used for the beam. The section will have the same moment of inertia and fully plastic moment as the 10W29 section used in Frame A, but its flange width-to-thickness ratio will be about 21. The effects of flange buckling on the frame behavior will be found by comparison with the previous test.

The second frame will also be a single-story frame with the columns oriented for weak-axis bending. This frame will have beam-to-columns connections similar to the fully welded connections used by E. P. Popov in his cantilever beam tests. In addition, part of the considerable inelastic strength of the columns will be utilized. The frame drift and strength have been maintained at the value used previously by utilizing a deeper beam.

A two-story frame with columns oriented for strong-axis bending will be tested to study the third problem. The failure mechanism of the frame is similar to that of the previous tests. The difference will be that instead of a hinge forming at the end of the beam initially, hinges will form above and below the joint in the column at about the same load.

CONCLUSIONS

The following tentative conclusions may be drawn from the preliminary results presented in the paper:

1. The hysteresis loops are very stable even for deformations greater than those corresponding to the maximum lateral load.
2. A considerable increase in lateral load carrying capacity over that expected from a monotonic analysis is possible.
3. Strain-hardening plays an important role in the behavior of the frames for displacements greater than those at the maximum loads.
4. The presence of the residual P-Δ moments has significant effects on frame behavior and must be included in developing a rational method of analysis for repeatedly loaded frames.
5. The shape of the hysteresis loops is affected by the reduction of frame stiffness under reversed loading due to spread of yielding in the plastic hinge locations and the Bauschinger effect in the material.

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REFERENCES

1. Popov, E. P. and Franklin, H. A.
STEEL BEAM-TO-COLUMN CONNECTIONS SUBJECTED TO CYCLICALLY REVERSED LOADING, Steel Research for Construction, AISI, February, 1966.

2. Popov, E. P. and Pinkey, R. B.
BEHAVIOR OF STEEL BUILDING CONNECTIONS SUBJECTED TO REPEATED INELASTIC STRAIN REVERSAL - Experimental Data - Report No. 67 - 31, University of California at Berkeley, December 1967.

3. Sherbourne, A. N.
SOME PRELIMINARY EXPERIMENTS ON THE BEHAVIOR OF DUCTILE STRUCTURES UNDER REPEATED LOADS, Experimental Mechanics, Vol. 3, No. 5 p. 119 (May 1963).

4. Krishnasamy, S. and Snerbourne, A. N.
RESPONSE OF A "PLASTIC HINGE" TO LOW CYCLE ALTERNATING DEFLECTIONS, Experimental Mechanics, Vol. 8, No. 6, p. 241 (June 1968).

5. Royles, R.
INCREMENTAL EXTENSION OF MILD STEEL BEAMS IN REVERSED BENDING J. Strain Analysis, Vol. 1, No. 2, p. 133 (1966).

6. Hagura, H.
RESEARCH ON THE ELASTO-PLASTIC ANALYSIS OF STEEL SECTIONS SUBJECTED TO ALTERNATIVE LOADS, Trans. Arch. Inst. of Japan, No. 125, p. 8 (July 1966).

7. Naka, T., Kato, B., Akiyama, H. and Uchida, N.
ULTIMATE STRENGTH OF COLUMNS IN MULTI-STORY RIGID FRAMES, Yawata Technical Report, September 1966, No. 256, p. 82-90.

8. Wakabayashi, M., Nonaka, T., Matsui, C., and Mitani, I.
AN EXPERIMENTAL STUDY OF THE ELASTIC-PLASTIC STABILITY OF CROSS SHAPED STRUCTURAL SYSTEMS, Kinki Branch of the Arch. Institute of Japan, 1967.

9. Wakabayashi, M.
THE RESTORING FORCE CHARACTERISTIC OF MULTI-STORY FRAMES, Bulletin of the Disaster Prevention Research Institute (Japan), Vol. 14, Part 2, February 1965.

10. Igarashi, S., Toga, N., Takada, S. and Koyanagi, Y.
PLASTIC BEHAVIOR OF STEEL FRAMES UNDER CYCLIC LOADINGS, Trans. of the Architectural Institute of Japan, No. 130, Dec. 1966.

11. Igarashi, S. and Toga, N.
HYSTERESIS CHARACTERISTICS AND STRUCTURAL DAMPING OF STEEL STRUCTURES UNDER ALTERNATE LATERAL LOADING, Trans. of the Architectural Institute of Japan, No. 120, February 1960.

12. Yokoo, Y., Wakabayashi, M., Matsui, C. and Tsuji, B.
HORIZONTAL-FORCE-RESTRAINT PROPERTIES OF MULTI-STORY STEEL FRAMES, Yawata Technical Report, September 1966, No. 256, p. 43-58.

13. Yarimci, E.
INCREMENTAL INELASTIC ANALYSIS OF FRAMED STRUCTURES AND SOME EXPERIMENTAL VERIFICATIONS, Ph.D. Dissertation, Lehigh University, 1966.

14. Arnold, P., Adams, P. F. and Lu, L. W.
THE EFFECT OF INSTABILITY ON THE CYCLIC BEHAVIOR OF A FRAME, Fritz Laboratory Report No. 297.24, September, 1966.

15. Beedle, L. S.
REVERSED LOADING OF FRAMES - PRELIMINARY TESTS, Proceedings, Structural Engineers Association of California, 1965, p. 87.

16. Driscoll, G. C., Jr., et al
PLASTIC DESIGN OF MULTI-STORY FRAMES, Fritz Laboratory Report No. 273.20, Summer, 1965.

17. McNamee, B. M.
THE GENERAL BEHAVIOR AND STRENGTH OF UNBRACED MULTI-STORY FRAMES UNDER GRAVITY LOADING, Ph.D. Dissertation, Lehigh University, June 1967.

18. Yarimci, E., Yura, J. A. and Lu, L. W.
TECHNIQUES FOR TESTING STRUCTURES PERMITTED TO SWAY, Experimental Mechanics, Society for Experimental Stress Analysis, Vol. 7, No. 8, Aug., 1967, pp. 76-84.

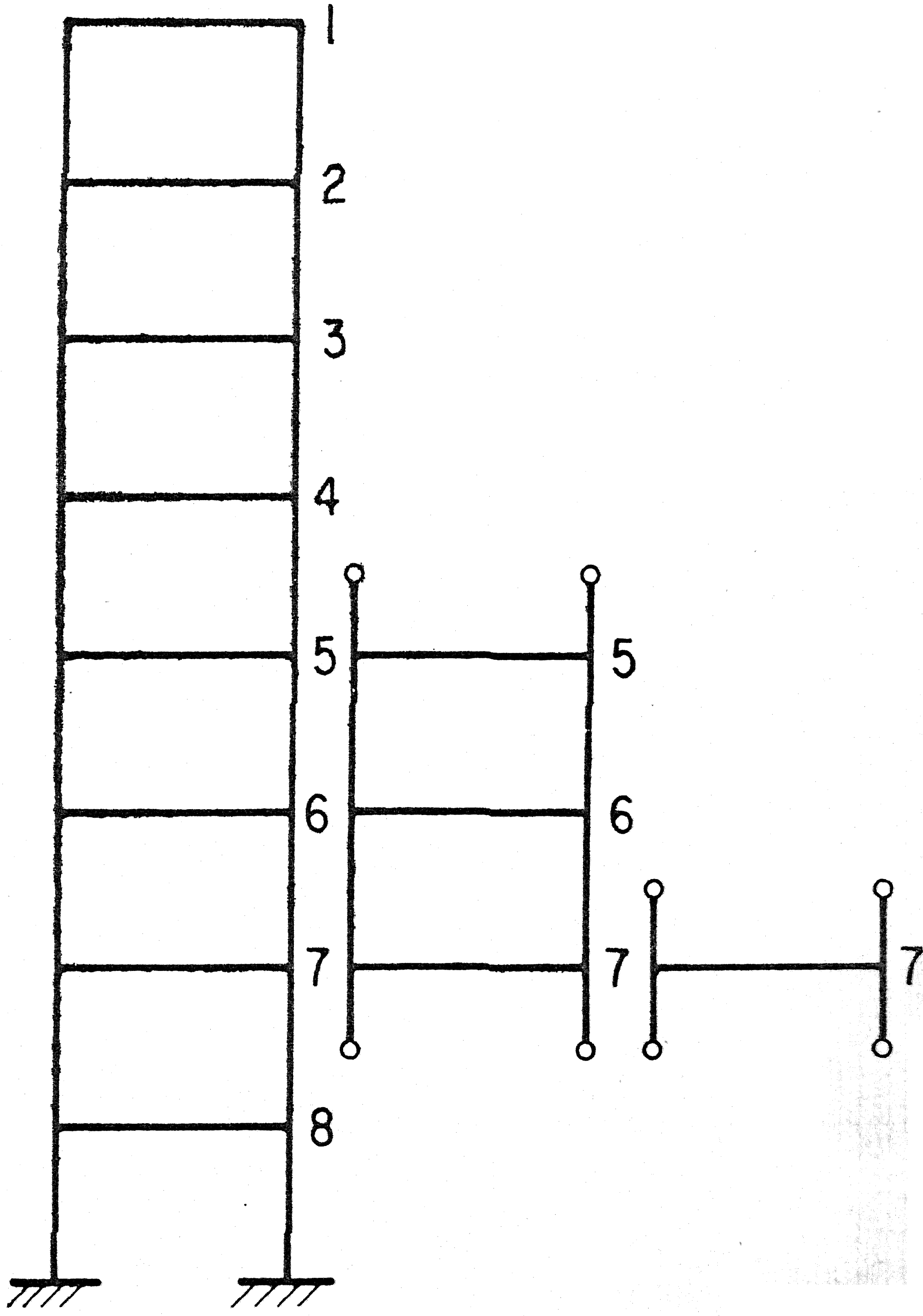

FIG. 1 PROTOTYPE FRAME AND TEST FRAMES

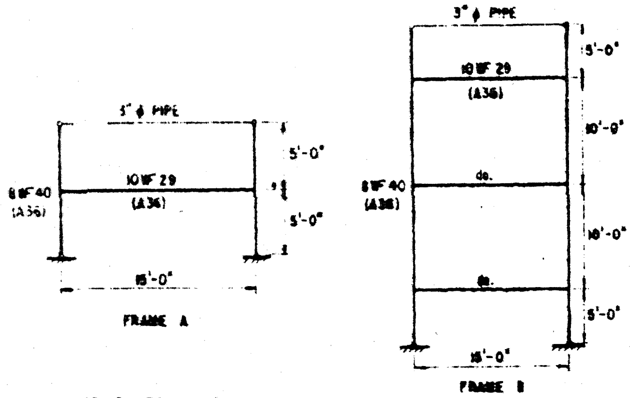

FIG. 2 GEOMETRY AND MEMBER SIZES FOR TEST FRAMES

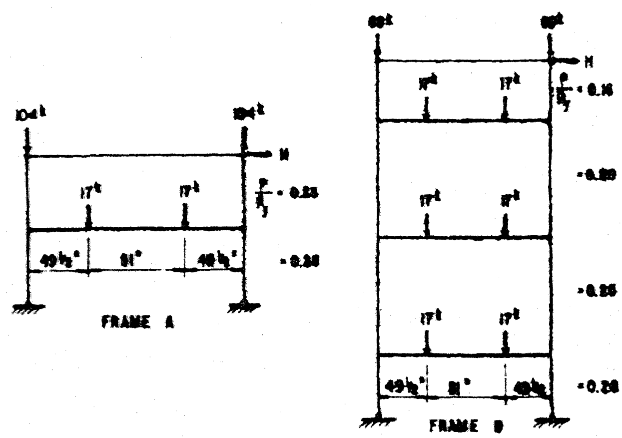

FIG. 3 LOADS AND AXIAL LOAD RATIOS FOR TEST FRAMES

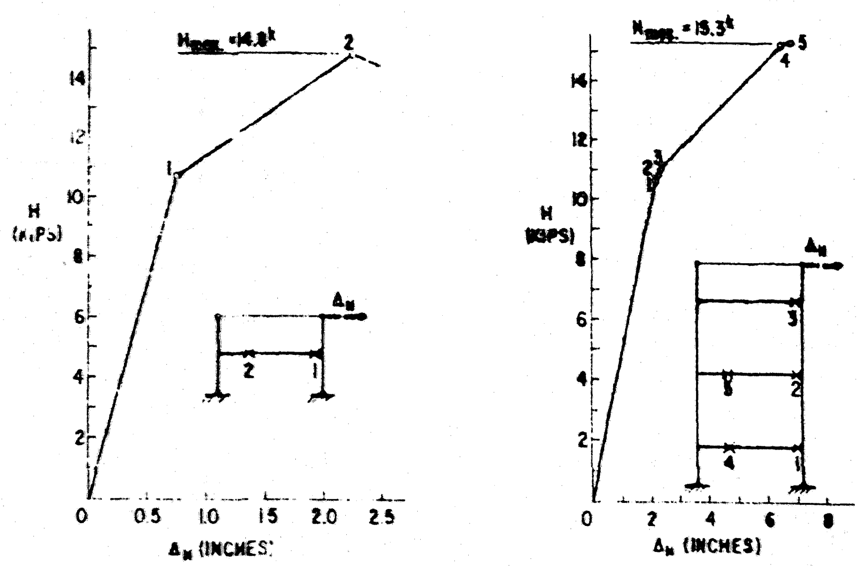

FIG. 4 LOAD-DEFLECTION CURVES FOR FRAME A AND FRAME B

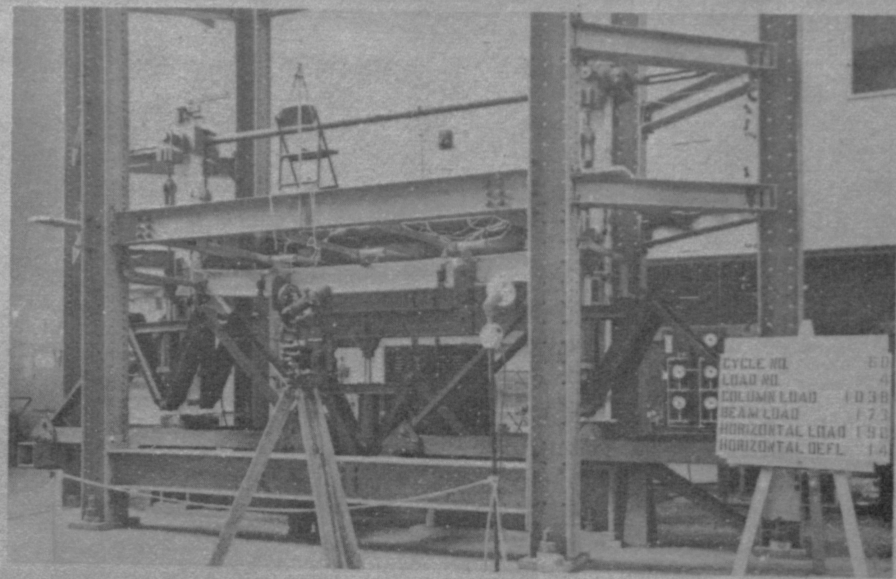

FIG. 5 TEST SETUP FOR FRAME A

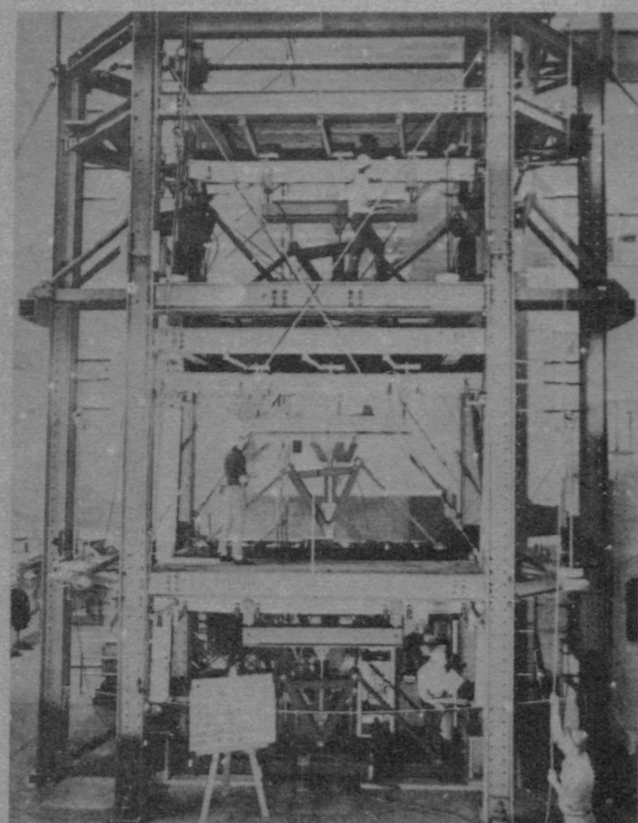

FIG. 6 TEST SETUP FOR FRAME B

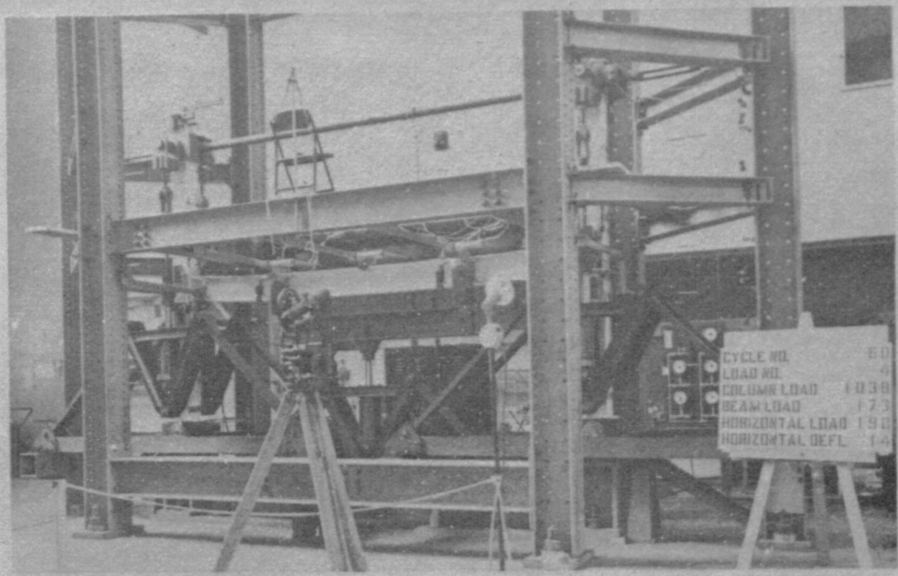

FIG. 5 TEST SETUP FOR FRAME A

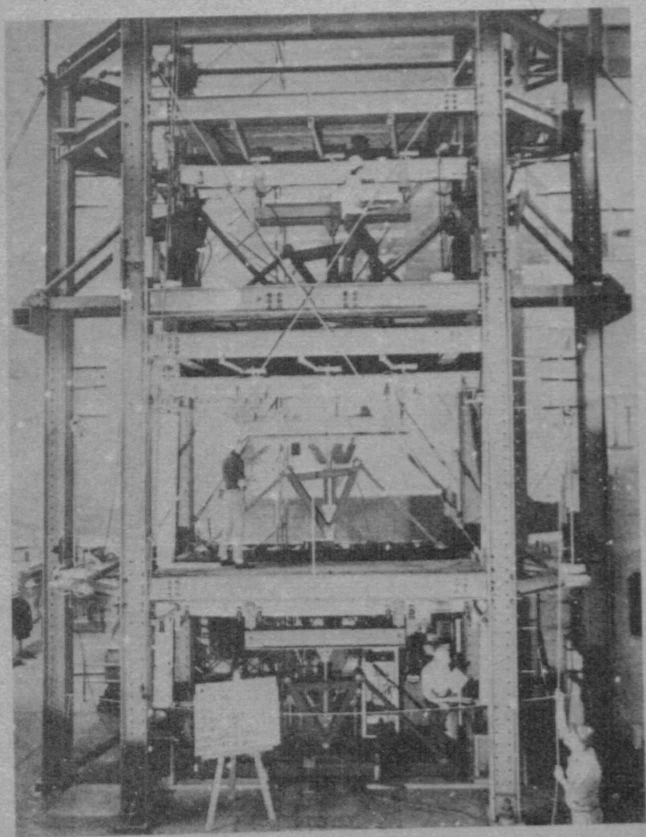

FIG. 6 TEST SETUP FOR FRAME B