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Shaking table tests of a six-story R/C frame with setback

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ABSTRACT: Measured responses to simulated earthquakes of a ductile moment-resisting reinforced concrete frame test structure are examined. The test structure is a 1/4 scale model of six-story, two-bay by two-bay frame with 50 percent setback at midheight, which was designed and detailed according to current seismic provisions. Resistance to unidirectional and bidirectional base motions is examined. Measured data did not reveal significant trends related to presence of the setback. The design resulted in a structural system with satisfactory behavior, but with significant overstrength. Sources of the overstrength are examined. Relatively simple procedures to compute stiffness and strength are evaluated and found to correlate well with the measured data.

1 INTRODUCTION

Behavior of a multistory building during strong earthquake motion depends on distribution of mass, stiffness, and strength in both horizontal and vertical planes of the building. Structural engineers have developed confidence in the design of buildings with regular configuration in which those distributions are more or less uniform. There appears to be less confidence about the design of irregular structures. A type of nonuniform structure that has repeatedly shown poor performance during past earthquakes (Berdis 1982, Suzuki 1977) is the building having setbacks at one or more levels. In most cases, the poor performance has been attributed to torsional effects and to concentration of inelastic action at the setback level.

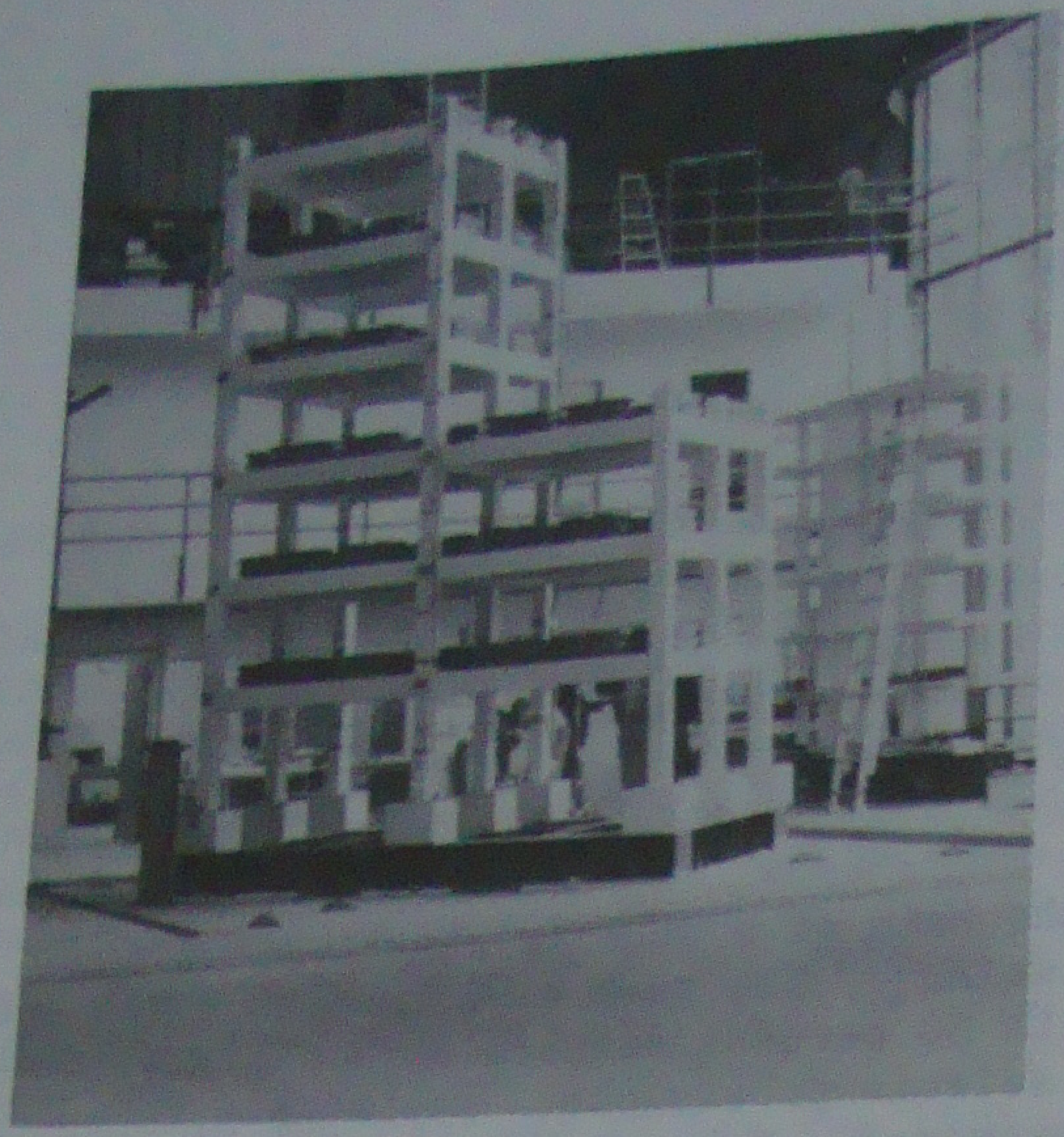
The unsatisfactory performance of several setback buildings warrants focused research to ascertain methods of improving performance of this class of structures. Among the issues to be addressed are (1) the influence of setbacks on dynamic response and (2) the adequacy of current design requirements for setback frames. In an effort to investigate these effects, an experimental and analytical study has been undertaken in which a complete moment-resisting reinforced concrete frame with floor slabs was constructed and subjected to simulated earthquake motions on a shaking table.

This paper documents the design, construction, testing, and analysis of the test structure. Experimental and analytical findings are presented to study the effects of setbacks, find the relation between design and actual strength, and check the adequacy of the current design and detailing provisions.

2 TEST STRUCTURE

The test structure modeled an imaginary prototype structure. The prototype is a six-story, two-bay by two-bay reinforced concrete ductile moment resisting frame having a fifty percent setback at the mid-height. The prototype is represented by the test structure depicted in Figs. 1 and 2.

Design concrete strength was 4000 psi and all reinforcement was Grade 60 (minimum yield stress of 60 ksi). Design gravity loads comprised self weight and 40 psf service live load. Because current codes do not allow use of the equivalent lateral force static analysis for design of a structure having this degree of setback (ATC3-06 1978, UBC 1982), a modal analysis was performed using an acceleration spectrum. The spectrum ordinates were set so that first mode base shear was equal to the



Overall view

Plan view

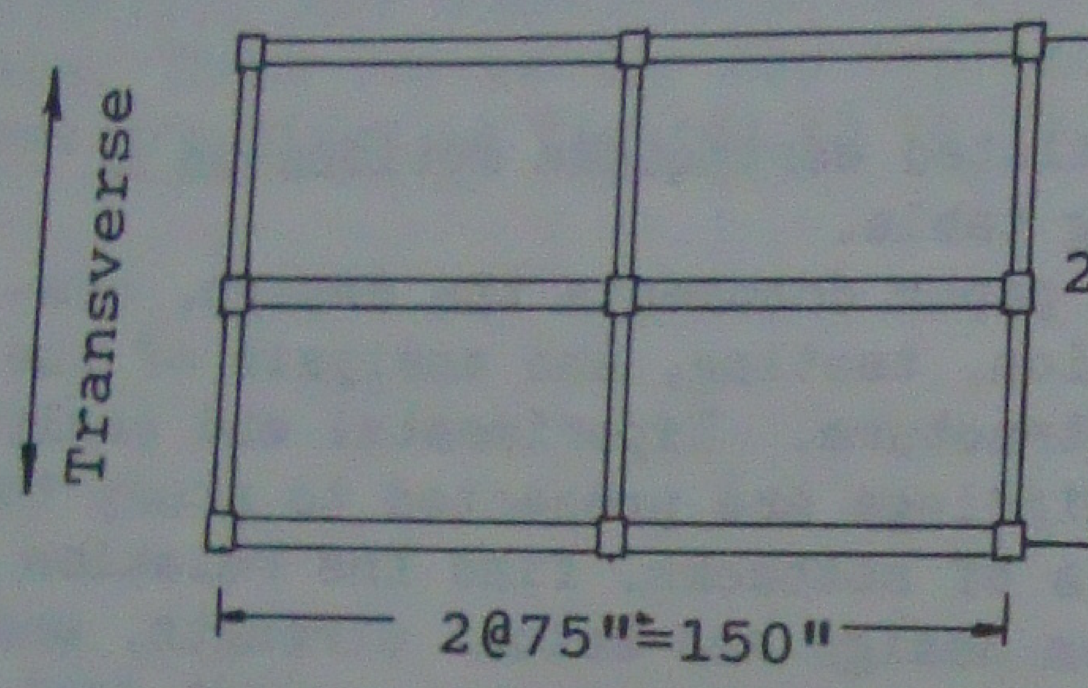


Fig. 1. Test structure.

Uniform Building Code (UBC 1982) design base shear for a building in zone 4.

Member proportions and details satisfy the seismic provisions of Appendix A of the ACI Building Code (ACI 318-83 1983). Connection design follows the ACI-ASCE Committee 352 recommendations (Comm. 352) (ACI 352R-85 1985). Column flexural overstrength ratios generally were in excess of the value of 1.4 recommended by Comm. 352, to account for anticipated effects of the slab which could enhance beam flexural strength, and higher modes which could affect plastic hinge patterns (Paulay 1986). Additional details of the design are given elsewhere(Shahrooz 1987).

All dimensions and details required for the prototype structure were scaled using the length scaling factor of 0.25 to obtain corresponding dimensions and details for the test structure. Typical column and beam details are shown in Fig. 2. The beam longitudinal reinforcement ratio ranges between 0.41% and 0.66% for beams spanning in the long direction and between 0.36% and 0.72% for those in the short span. This ratio is 1.5% and 2.3% for the exterior and interior columns, respectively. The column longitudinal reinforcing bars were continuous throughout the height except for the central column (Fig. 1). For this column, the longitudinal bars were spliced between the first and second floors and between the third and fourth levels.

Column longitudinal reinforcement was deformed #3 and #2 (0.25 in. diameter) bars. Beam longitudinal reinforcement was deformed nominal #2 and #1 bars, the latter fabricated in the laboratory to have a diameter of 0.178 in. Slab reinforcement was gauge #9 galvanized plain wire (0.148 in. diameter) which was lightly deformed to improve its bond strength. Transverse reinforcement for beams and columns was gauge #11 (0.120 in. diameter) and gauge #9 galvanized plain wire, respectively. Yield and ultimate stress capacities are summarized in Table 1. Concrete had maximum aggregate size of 3/8 in., and attained a mean compressive strength of 4200 psi at the time of the tests.

To avoid difficulty in casting the quarter scale columns, the test structure was cast in an unconventional manner. First, the three transverse frames (Fig. 1) were cast separately in a horizontal position atop a pivoting platform. Reinforcement for the longitudinal beams and slabs at the joints was tied in position before the transverse frames were cast. The transverse frames were subsequently pivoted to an upright position, and after forms and reinforcing cages for the longitudinal beams and slabs were completed, the remainder of the structure was cast. No unusual behavior was apparently attributable to this construction procedure.

Following construction, the model was prestressed to the shaking table at the Earthquake Engineering Research Center of the University of California at Berkeley. Lead pigs were fastened to the top surface of slabs to simulate effects of the service dead load expected for the prototype. Live load was not simulated. Total weight of the test structure, including lead pigs, was 73.3 kips.

3 TEST AND INSTRUMENTATION DESCRIPTION

Tests included static pull-back tests, low-amplitude free vibration tests, and earthquake simulations of varying intensity.

Table 1. Reinforcing bar properties.

Bar type	Yield stress (Ksi)	Ultimate stress (Ksi)
#2(type1)	64.4	86.0
#2(type2)	66.2	100.8
#2(type3)	73.1	96.5
#1	63.5	80.0
#3	65.0	95.7
Gauge#11	56.0	95.0
Gauge#9	53.7	88.5

Earthquake simulations were conducted in two stages. In the first, a series of horizontal base motions, of successively increasing intensity, was applied parallel to the longitudinal frames to simulate uniaxial response. In the second phase, unidirectional horizontal motions were input at an angle of 45 degrees relative to the principal axes of the frames to impart biaxial lateral-torsional response.

The input signals to the shaking table modeled acceleration and displacement histories of the 1940 El Centro NS record, the 1978 Miyagi-Ken-Oki S00E record, and the 1985 SCT Mexico City S60E record. Durations of the prototype El Centro and Miyagi-Ken-Oki records were compressed by a factor of 2 so that frequency content of the base motion and of the scaled model would be approximately in accord. The Mexico City record was compressed by a factor of 3 in order to resonate the test structure and achieve a desirable damage state. Peak base accelerations (Table 2) were varied to obtain base motions ranging from low to high intensity. Housner spectrum intensities (Housner 1959) (calculated using shifted frequency limits to account for time compression of the base motions) were calculated. Ratios between computed spectrum intensities and the scaled intensity of the prototype El Centro motion are presented in Table 2 for comparison. Test MO16.02 may be an unrealistically intense motion.

Instrumentation was used to determine motion of the shaking table, horizontal accelerations and relative displacements of the floors, and strains on selected beam and column bars. Data were recorded digitally at an interval of 0.005 sec. for each data channel.

4 RESPONSE OF THE TEST STRUCTURE

4.1 Responses to unidirectional tests

The test structure was initially subjected to a series of low-intensity tests to induce "linear" response. In the first test causing inelastic response, modeling the El Centro motion with a peak base acceleration of 0.176g, the peak base shear (computed as the sum of products between floor acceleration and tributary mass) reached 0.34W, and peak interstory drift reached 0.5% of story height. Cracking was more extensive, but no spalling was observed. Reinforcement in the central column at the footing level reached a peak strain corresponding to 67% of yield, and first-floor beam reinforcement at the interior joint was strained to 83% of yield.

In the last unidirectional test, which was intended to represent a design-level test, the peak acceleration of the El Centro motion was increased to 0.49g, resulting in a spectrum intensity ratio of 1.68 (Table 2). The peak base shear corresponded to 67% of structure weight, and the peak interstory drift reached 1.53% of the story height. Variation of roof displacement, base shear, and input acceleration with time is depicted in Fig. 3.

Table 2. Test and response summary.

Test	Spectrum intensity ratio	Base Acceleration(g)	Displacement (In.) X	Displacement (In.) Y	Interstory (In.) X	Interstory (In.) Y	Base shear (Kips.) X	Base shear (Kips.) Y	Twist (Rad.)	Period (Sec.)	Damping (%)
EC17.05	0.59	0.166	0.62	-	0.16	-	25.0	-	-	0.32	3.6
EC22.02	1.68	0.493	2.46	-	0.55	-	49.0	-	-	0.48	5.0
MO16.02	2.16	0.634	3.20	4.59	0.72	1.07	43.0	36.0	0.045	0.67	10.0
MX23.06	1.12	0.346	4.07	4.35	1.08	1.00	43.5	28.4	0.052	0.77	11.0

Notations:
X : Long direction
Y : Short direction
EC : El Centro NS 1940
MO : Miyagi-Ken-Oki S00E 1978
MX : SCT Mexico City S60E 1985

Notes:
Vibration period value is from the free vibration tests following the earthquake simulation.
Spectrum intensities are normalized with respect to 20% damped intensity of El Centro, NS 1940.

Table 3. Development of overstrength.

Analysis	Base shear (Kips)
A1	6.67 (0.14)
A2	7.77 (0.16)
B	10.0 (0.20)
C	14.9 (0.30)
D	23.8 (0.49)
E	26.9 (0.55)
F	30.0 (0.61)
G	42.0 (0.86)

Values given in ( ) are ratios between computed base shear strength and measured base shear strength.

Reinforcement in the central column at the footing was strained 3.5 times yield strain, and a first-floor longitudinal beam strain reached a peak equal to 4.3 times yield strain. Concrete cover spalled off a beam near a first-floor corner joint, and cracks as wide as 0.016 in. opened in a first-floor beam near the beam-column joint. Extensive cracking was observed in slabs, the most extensive apparent at the top surface of the first and fourth floors. The free-vibration period reached 0.48 sec., which is 1.8 times the value measured in the "uncracked" stage, suggesting that stiffness had been reduced to approximately one-third that of the uncracked structure.

Joint shears were approximated based on measured reinforcement strains and known cross-sectional and material properties, as follows. Knowing the tensile strains in beam and column reinforcement at a joint, flexural moments in columns above and below the joint and in beams having bottom reinforcement stressed in tension were estimated by comparison with calculated monotonic behaviors of these elements. Moments in beams with top reinforcement in tension were then deduced using equations of statics applied to the joint. Knowing the moments in beams and columns, joint shears could be estimated. During the third earthquake simulation, the maximum shear in the first-floor interior joint reached approximately $21\sqrt{f'_c}$, which is nearly equal to the design value of $20\sqrt{f'_c}$ recommended by Comm. 352. The maximum recorded shear at the first-floor corner connection reached $7\sqrt{f'_c}$, compared with corresponding recommended value of $15\sqrt{f'_c}$. Despite the large shears, there were no visible shear cracks or other signs of joint shear deterioration.

Peak base shears and top floor displacements, measured at successively increasing peak displacement amplitudes during the uniaxial tests, are depicted by asterisks

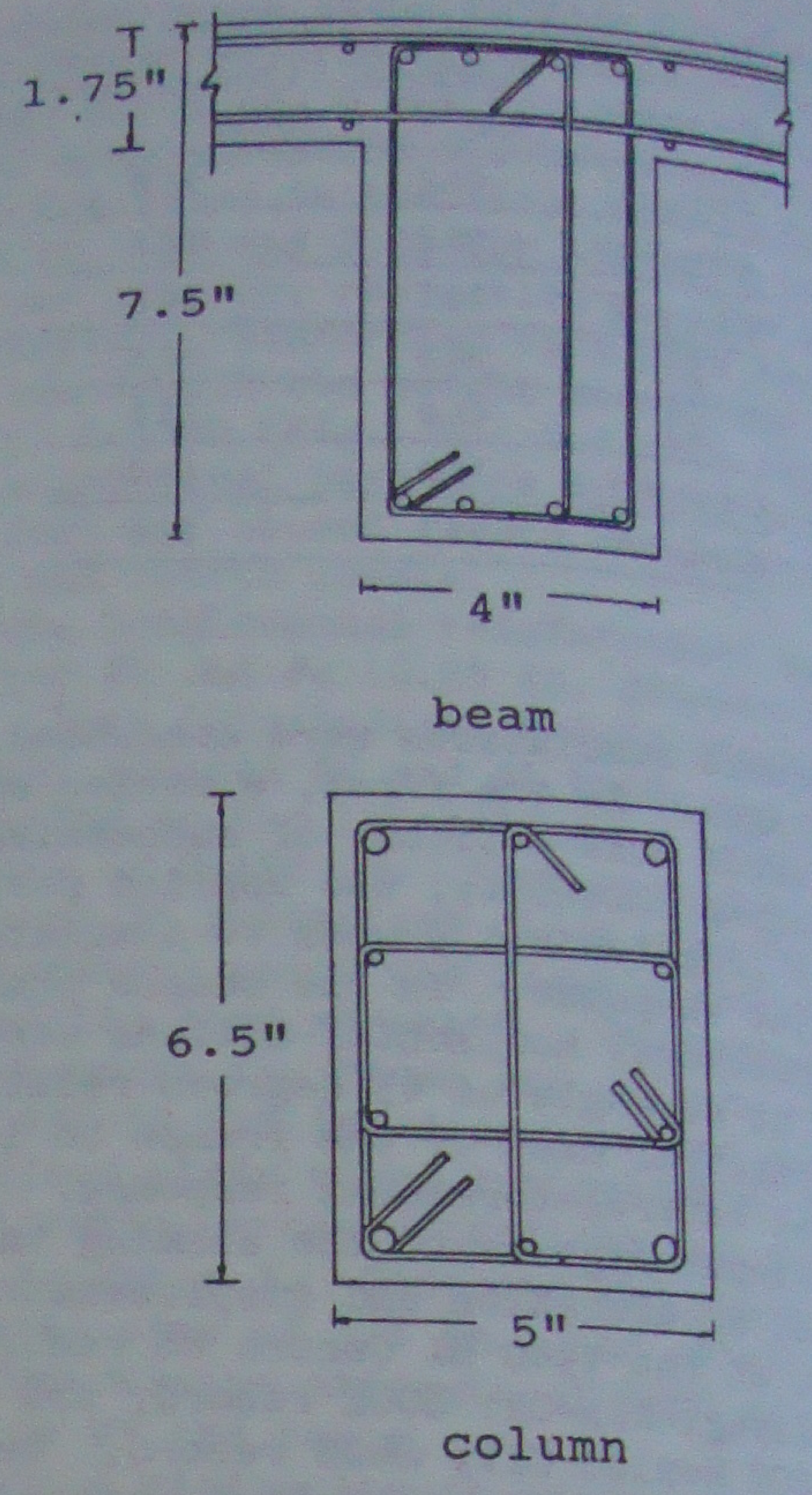


Fig. 2. Typical beam and column cross sections.

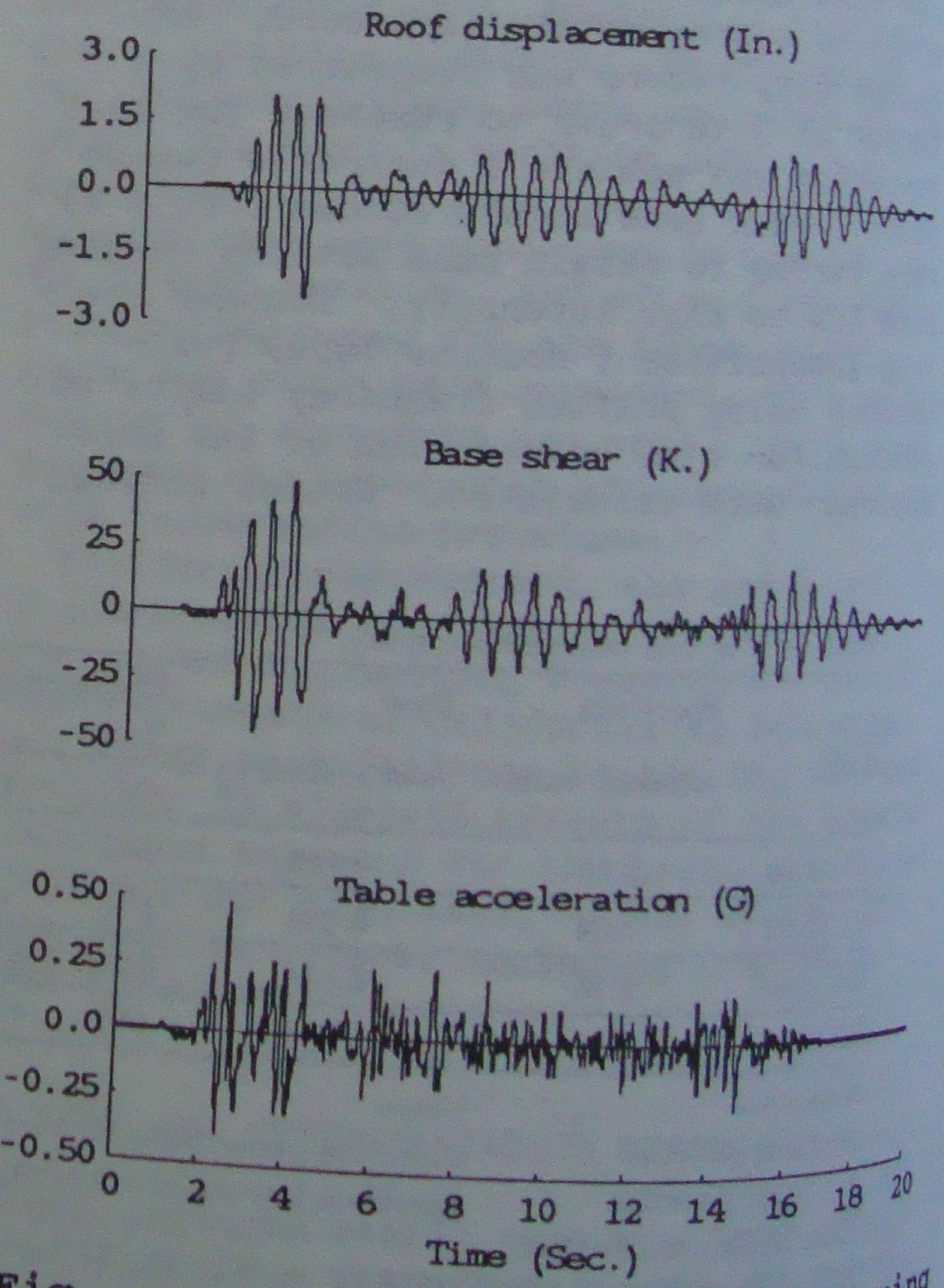


Fig. 3. Response history during test EC22.02.

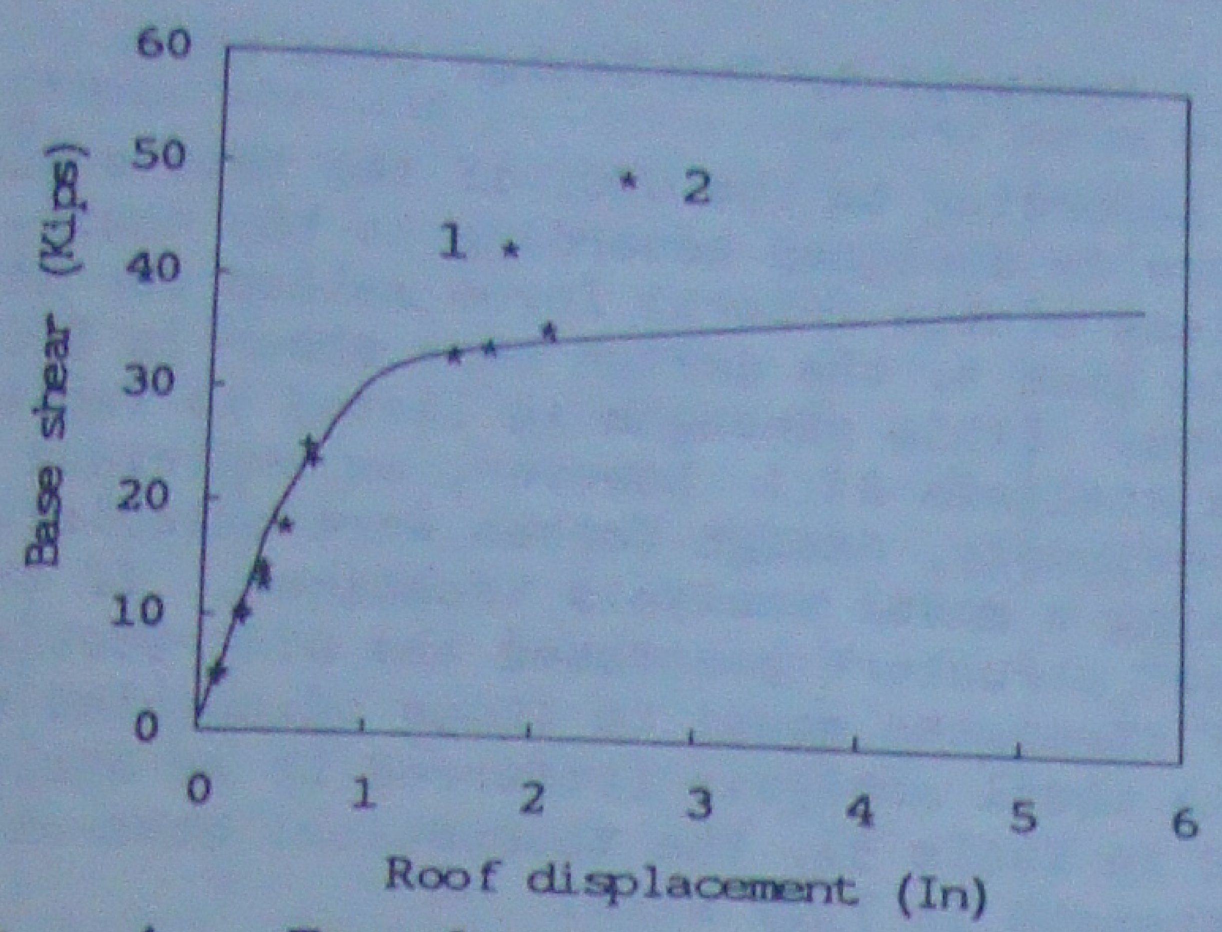


Fig. 4. Envelope relation between base shear and roof displacement.

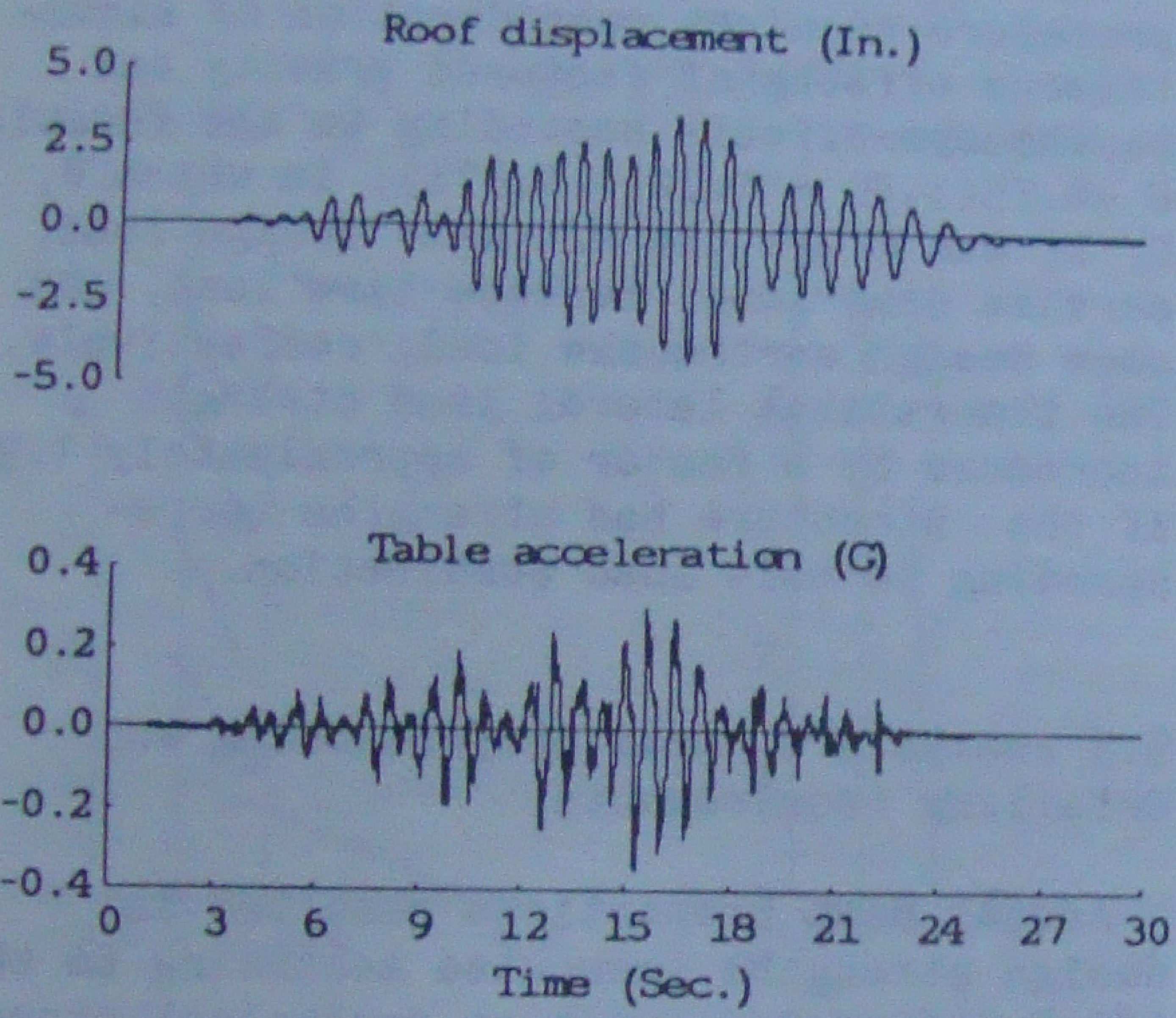


Fig. 5. Response history during test MX23.06.

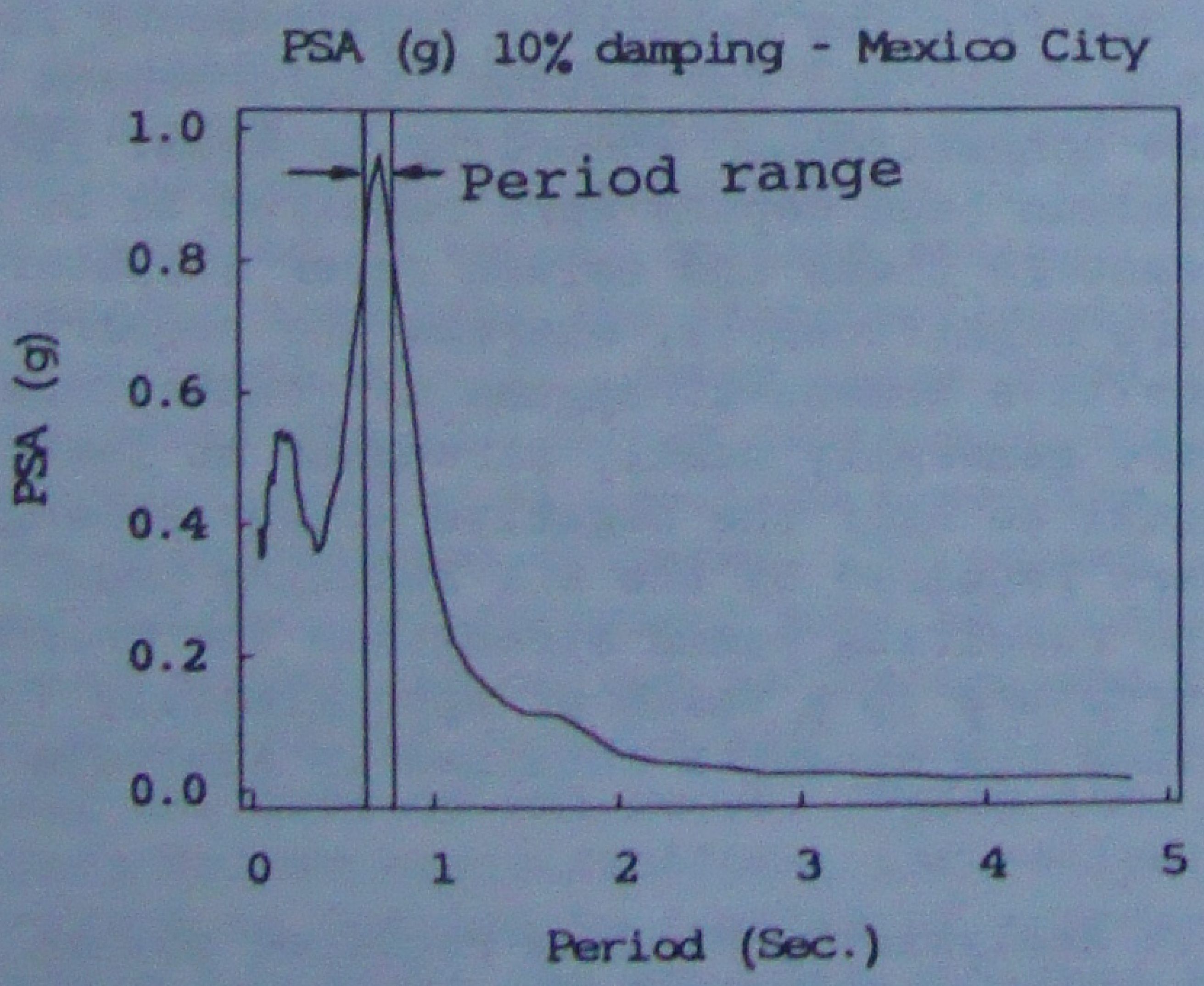


Fig. 6. Acceleration response spectrum.

in Fig. 4. The data points reveal an envelope relation having decreasing stiffness with increasing displacement. It is likely that the structure was at or near the base-shear strength during the last uniaxial test.

4.2 Biaxial tests

After subjecting the test structure to a base motion similar to that used in the last uniaxial test, the base motion was changed to Miyagi-Ken-Oki because its excitation potential would be higher in the period range of the structure. This test is denoted by MO16.02 in Table 2. The peak base acceleration for the test reached 0.643g. The peak base shear reached 0.59W and 0.49W along the longitudinal and transverse axes of the structure. The peak interstory drift was nearly 3.0% of interstory height. Torsional response was visually clear during the test, the roof twisting as much as 0.045 rad. Major cracking and spalling were observed at several locations in the first, second and third levels. Column reinforcing bars were strained to a maximum of 6 times of yield strain. Spalling of other column covers was also detected at the second and third floors. Minor torsional cracking was found in the short direction beams, suggesting that they had contributed in resistance to loads in the longitudinal direction. Diagonal shear cracking formed at some exterior joints in both principal directions. However, joint shears cannot be estimated reliably with the available data.

As a final test, the test structure was subjected to the Mexico City. As explained earlier, this record was intended to resonate the structure. The resonance is seen from response history of roof displacement (Fig. 5). At the beginning of the test, the fundamental period (left vertical line in Fig. 6), was less than that corresponding to the peak of the acceleration spectrum. During the test, the period elongated into the range of increased spectral acceleration (Fig. 6) with consequent amplification of response (Fig. 5) and damage. During this test, peak base shear reached 0.59W. The maximum interstory drift (in the longitudinal direction) reached of 3% of story height. The first mode vibration period elongated, confirming loss of stiffness due to inelastic response. Extensive spalling and cracking were apparent in beam ends and columns at footing level. At the fourth and second level, the longitudinal reinforcement of the longitudinal became visible where cover had spalled at the joints. Cover over an exterior joint spalled subsequent to formation of significant diagonal shear cracking. Reinforcement in the longitudinal beam at the fourth-level interior joint was strained as much as 11 times yield. Column reinforcing bars at the footing level reached strains corresponding to 7 times of yield.

5 LATERAL LOAD STRENGTH

During the uniaxial dynamic tests, the test structure sustained a maximum base shear equal to 0.67W. During the biaxial tests, the maximum shear along any of the principal axes of the structure was 0.59W. Compared with the design base shear of 0.091W (calculated using the UBC design formula without load factors), it is apparent that the structure possessed a significant overstrength. The overstrength is in some ways advantageous. For example, the increased strength is likely to result in reduced ductility demands during strong shaking. However, the overstrength also indicates that current design and analysis methods are capable of producing a structure that is significantly different from that which was intended. It is conceivable that in some cases, the structure will differ sufficiently that undesirable and unpredictable failure modes might result using the current methods.

Because the test structure was designed according to currently applied design algorithms, and because it comprises and reasonably replicates the essential primary structural elements of a real building, it is possible to trace through the structural design and analysis methods to ascertain the sources of overstrength that might influence real buildings. The findings will be of value in developing refined analysis and design techniques that more closely represent the real building.

To arrive at an understanding of the source of overstrength in the test structure, a series of static limit analyses was conducted. In each analysis, different design and analysis provisions and their effects on strength are considered separately. The lateral-forced distribution was the same as that specified for static design in the UBC. The selection was based on measured lateral force distribution. Analytical results for other lateral force distributions are discussed elsewhere (Shahrooz 1987). Results of the analyses are summarized in Table 3, and discussed in the following paragraphs.

5.1 Analysis A, The design base shear

According to the UBC, if the test structure is designed according to the equivalent static lateral force method for seismic zone 4, the design base shear is 6.67 kips. (This strength is listed in Table 3 as Analysis A1.) However, as explained previously, design forces were calculated using a modal analysis technique. If the test structure possessed the distribution of strengths equal to those calculated with the modal analysis (referred to as Analysis A2 in Table 3), the theoretical base-shear strength is 7.77 kips.

5.2 Analysis B, Load factors

The ACI Building Code strength design procedure requires consideration of simultaneous effects of factored gravity and earthquake effects according to the formula $U = 0.75(1.4D + 1.7L + 1.87E)$, in which U, D, L, and E are the ultimate design load, service dead load, service live load, and code design earthquake load, respectively. The theoretical lateral load strength increases by a factor of approximately 1.3 if the structure has strengths corresponding to this load combination.

5.3 Analysis C, Beam proportioning and detailing requirements

Actual beam proportions resulted in design strengths (computed according to the ACI Building Code with an equivalent stress block and nominal material properties) significantly exceeded required strengths. Although limitations in available model reinforcement resulted in a some overstrength, the majority of overstrength arose from the detailing requirements for beam depth and for bottom reinforcement at the connections. According to Comm. 352, minimum beam depths were required to be at least 20 times the column rebar diameter. More significantly, whereas the required positive moment strengths at connections were generally small, strengths at least equal to half the negative moment strengths were required by the ACI Building Code. The resulting beams strengthen the entire structure to a value of approximately 1.5 times the strength obtained in Analysis B.

5.4 Analysis D, Minimum required column overstrength and actual column strengths

The report of Comm. 352 recommends that

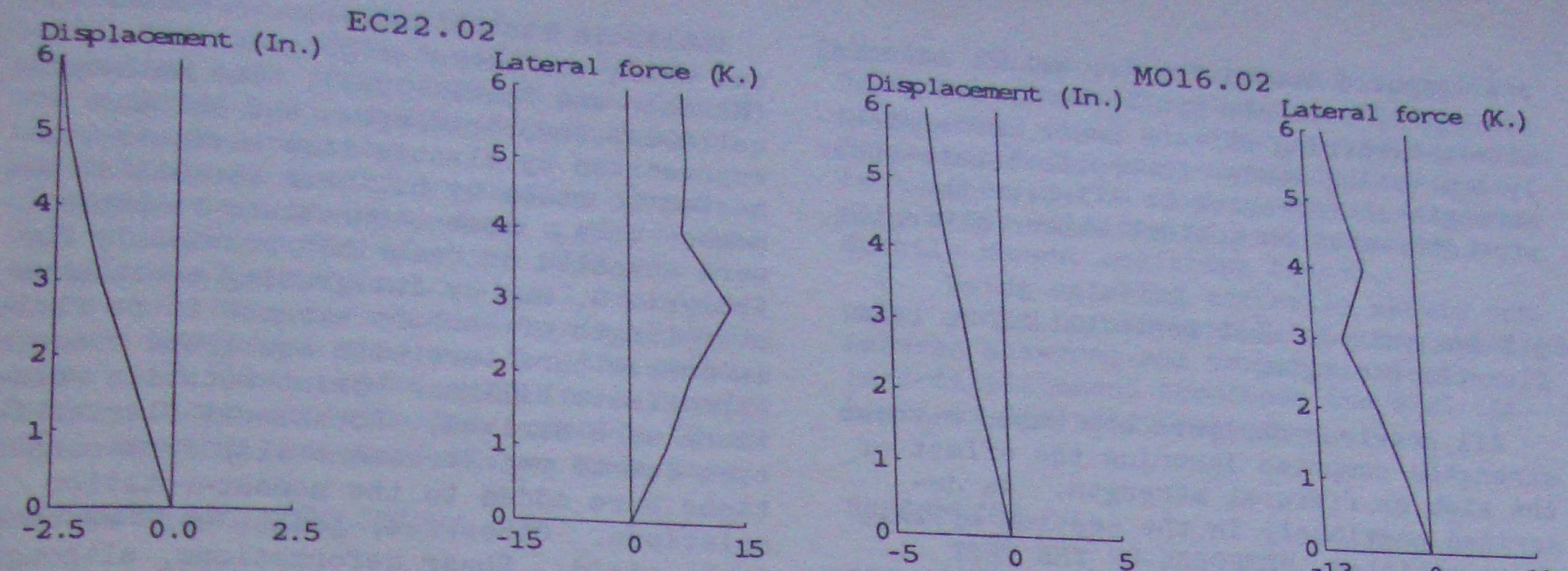


Fig. 7a. Displacement and lateral force distribution.

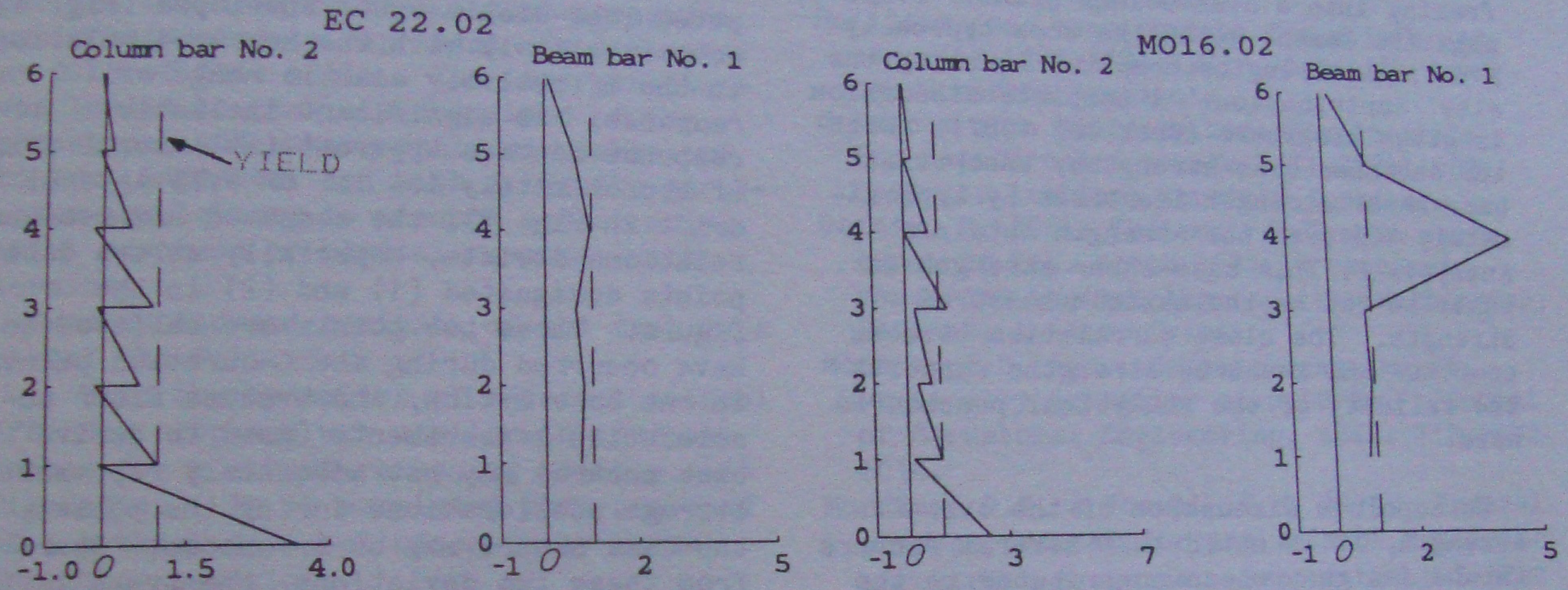


Fig. 7b. Strain gauge profile for central column.

the sum of nominal column strengths should be at least 1.4 times the sum of nominal beam strengths at a connection. Nevertheless, additional recommendations of Comm. 352 resulted in actual columns having strengths exceeding the minimum flexural overstrength of 1.4. Among these detailing provisions are (1) column cross-sectional dimension must be at least 20 times the beam reinforcement diameter, (2) column longitudinal reinforcement must be closely spaced around the column perimeter, and (3) joint dimensions must be such that joint shear failures do not occur. Using the actual column cross sections, theoretical strength is boosted to a value of approximately 1.6 times the strength obtained in Analysis C.

5.5 Analysis E, Capacity reduction factors

The preceding analyses were based on design member strengths, which are equal to nominal strengths multiplied by capacity reduction factors. Using nominal strengths rather than design strengths, theoretical strength of the structure is boosted by another 13%. It is noted that, at this point, where strengths are computed using the ACI Building Code nominal member strengths, the computed structure strength 55% of maximum measured base shear.

5.6 Analysis F, Actual material properties

If the nominal strengths of the ACI Building Code are abandoned, and strengths are computed accounting for actual material strengths, concrete confinement, and strain-hardening effects under monotonically increasing loads, theoretical base-shear strength is increased to 11% over the strength based on nominal member strengths.

5.7 Analysis G, Slab contribution to beam flexural strengths

All previous analyses are based on beam strengths computed ignoring the effect of the slab on flexural strength. As described previously in the section of this paper entitled RESPONSE OF THE TEST STRUCTURE, beam negative (hugging) moment strengths, including slab contributions, were estimated based on measured strains and statical considerations for elements framing into a beam-column joint. Computed negative moment strengths were typically 2.5 times strengths computed ignoring the slab contribution. A complete discussion is given elsewhere (Shahrooz 1987). With the enhanced beam strengths, theoretical base-shear strength increases by approximately 40% over the strength obtained in Analysis F. This base-shear strength is equal to 86% of the maximum measured strength. The close correlation between computed and measured strengths supports the validity of the analytical procedures used.

To conclude discussion of the overstrength, it is noted that several factors in the design process contributed to the strength increase of the structure. Taken individually, no single design step can be identified as having caused the large overstrength observed for the test structure. Taken together, and recognizing that the individual factors are multiplicative, the overstrength can be plausibly explained. Further research on this subject is recommended so that design methods can be developed that explicitly account for these effects, and consequently, structures can be designed that will respond in a manner more similar to that which was intended.

6 LATERAL LOAD-DISPLACEMENT RELATION

To verify analytical procedures for computing lateral load response envelopes for reinforced concrete structures, results of inelastic static analysis were correlated with the measured relation represented by asterisks in Fig. 4. The analysis is documented in the following paragraphs.

Inelastic static analyses were carried out using the computer program ANSR (Mondkar and Powell 1975). The mathematical model comprised beams and columns represented by elastic line elements connected to nodes by bilinear springs at the member ends. Moment-curvature responses were computed as described previously for Analysis G, and by integrating curvatures over length of members assumed to be flexed in double curvature with equal end moments, approximate bilinear moment-rotation relations were derived. Rotational flexibilities due to reinforcement slip from connections were added to the moment-rotation relations. Otherwise, joints were assumed to be rigid. Shear deformations, although computed to be small, were included in computing member stiffnesses.

The mathematical model was loaded with lateral loads according to the UBC. Computed load-displacement envelopes (Fig. 4) compare closely with the measured relation in the effectively elastic range of response. As significant inelastic response becomes apparent (at lateral drift of approximately 1.5 in. or 0.7% lateral drift in Fig. 4), the computed and measured relations deviate, especially at two data points designated (1) and (2) in the figure. These two points are believed to have occurred during short-duration pulses in the base motion, and because floor acceleration measurements (used to derive base shears) may not adequately represent average accelerations during the pulses, the base shears may be erroneous. Apart from these two deviations, the overall correlation is reasonably good.

7 APPARENT EFFECTS OF THE SETBACK

Examination of response waveforms and distributions of response over height did not indicate that the tower responded differently from the base in either the uniaxial or biaxial tests. Typical response distributions (Fig. 7a) do not show discontinuity in the displacement profiles. Similarly, typical lateral force distributions are nearly the same as the static design distribution of the UBC. Beam and column strain profiles (Fig. 7b) indicate a relatively smooth distribution of internal actions, except for the fourth-level beam, which sustained larger strains than adjacent beams. This discontinuity is not unexpected, as there is only one beam to resist column moments, whereas below this level there are two beams framing into the column. Inelastic static analysis of the test structure, as reported in the preceding section, shows a similar trend, suggesting that this observed behavior is not a peculiarity of dynamic response of the setback structure.

More study is needed to confirm these observations. However, from a global as well as local point of view, with the exception of moderate torsional response, there were no apparent differences between dynamic responses observed for the setback frame and those anticipated for a regular frame. Other experimental results (Wood 1986) support the observations made here.

8 SUMMARY AND CONCLUSIONS

A six-story, two-bay by two-bay reinforced concrete frame with 50 percent setback at the midheight was designed and detailed to satisfy gravity and seismic requirements of current building codes. A test structure was constructed to represent the prototype at 0.25 of full scale. The test structure was loaded to simulate dead load of the prototype, and subsequently was subjected to low, moderate, and high intensity earthquake simulations to produce either uniaxial response or biaxial response with torsion.

Examination and analysis of the test data reveal the following:

1. With the exception of modest torsional effects, response was similar to that expected for a structure having regular configuration. It is concluded that presence of a setback does not necessarily cause irregular behavior.

2. Lateral drift exceeded 2% of building height, and interstory drifts reached a maximum of 3% of story height, without signs of imminent collapse. Code-specified procedures for proportioning and detailing of beams, columns, and connections were apparently adequate.

3. The experiments show clearly the tendency for current design techniques to result lateral-load strengths significantly exceeding the design strength. Several factors that contributed to the overstrength are identified, including contributions of the slab to beam flexural strength, and column overstrength resulting from detailing requirements. The overstrength reduced the demand for ductility. Undesirable modes of failure are possible because of changes in strength distribution.

4. Strengths of the components and structure were studied following existing analytical methods. Using component strengths of the ACI Building Code, computed base-shear strength using standard limit analysis was 55 percent of measured strength. Using more refined techniques, 86 percent of measured shear strength was accountable. It is concluded that currently available techniques can be used to obtain a reasonably close estimate of real strength of ductile moment resisting frames.

6. Using existing inelastic static computer codes, close correlation was obtained between measured and computed uniaxial load-displacement envelopes and distributions of inelastic response.

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