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AN ANALYSIS OF STRONG-MOTION DATA FROM A SEVERELY DAMAGED STRUCTURE, THE IMPERIAL COUNTY SERVICES BUILDING, EL CENTRO, CALIFORNIA

by

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Abstract

The Imperial County Services Building, a six-story reinforced-concrete frame and shear-wall office building 7.6 km southwest of the Imperial fault trace in El Centro, Calif., sustained significant structural damage during the magnitude 6.7, October 15, 1979, Imperial Valley, Calif., earthquake. Strong-motion instrumentation at the site, installed and maintained by the California Division of Mines and Geology, consists of a 13-channel remote-accelerometer central-recording accelerograph system in the building and a triaxial accelerograph located at ground level approximately 100 m to the east. Several features of the main-shock accelerogram recovered from the building, including abrupt changes in frequency content and bursts of high-frequency motion, provide important information on the mechanism of structural failure. A comparison of the main-shock motions recorded at the base of the building with those recorded at the adjacent ground site (intended to be free field) indicates that the motion recorded at the ground floor of the building incorporates to a significant extent the response of the building-soil-foundation system. The acceleration data also show that the building's fundamental periods changed significantly during the earthquake. A preliminary relative-displacement analysis indicates that the east-west interstory displacement between the second and ground floors was approximately 6.2 cm at the beginning of column collapse.

Introduction

The Imperial County Services Building, a six-story reinforced-concrete frame and shear-wall building in downtown El Centro, Calif., sustained significant structural damage during the shallow-focus, magnitude 6.7 (CIT) October 15, 1979 Imperial Valley earthquake. Strong-motion instrumentation at the site, which is located 7.6 km southwest of the nearest point on the Imperial fault and 27 km northwest of the October 15 main-shock epicenter (fig. 1) , consists of a 13-channel accelerograph system installed in the building and a triaxial accelerograph at ground level approximately 100 m east of the building. The instruments, installed and maintained by the California Division of Mines and Geology (CDMG), triggered and functioned properly during the earthquake.

The accelerograms recovered from the building are of great interest, not only because this is the first time an extensively instrumented building has sustained significant earthquake-induced structural damage, but also because the time and mechanism of damage can be inferred from the recorded data. The records therefore provide important information on forces, dynamic properties, and relative motions before, during, and after the time when damage was occurring. In conjunction with the records from the adjacent ground site, the building records also provide important insight into the extent to which the building and its foundation system influenced the motion recorded at the ground floor of the building.

Building description

The Imperial County Services Building (fig. 2), which serves as an officebuilding for Imperial County, is at 940 Main Street, El Centro, at lat 32.792° N., long 115.564° W. It was designed in 1968 (using the 1967 edition of the Uniform Building Code) and was completed in 1971 at a construction cost of $1.87 million (Randy Rister, oral commun., 1979). The building is 136 feet

10 inches by 85 feet 4 inches (41.71 by 25.92 m) in plan and is founded on a Raymond step-taper concrete pile foundation. The piles, which are interconnected with reinforced-concrete link beams, extend 14 to 18 m into the alluvial foundation material composed primarily of sand with interbeds of clay (based on logs from four 12- to 18-m-deep soil borings at the site).

Vertical loads are carried by reinforced-concrete floor slabs supported by reinforced-concrete pan joists spanning in the north-south direction; the joists are supported by four longitudinal five-bay reinforced-concrete frames at 7.6 m (25 ft) oncenter. Lateral loads are resisted by the four reinforced-concrete frames in the east-west direction and by reinforced-concrete shear walls in the north-south direction. The shear walls are discontinuous at the second floor. Below the second floor the shear walls are along three interior lines and at the west end; above the second floor they are at the east and west ends of the building only. The design requires that lateral loads at the east end of the building be transferred from the upper story east-end shear wall to the closest interior first-story shear wall (approximately 9.4 m to the west) through the second-floor diaphragm. It also requires that overturning at the east end of the building be resisted in the first story by a row of four reinforced-concrete columns approximately 2 m west of the upper story east-end shear wall.

Earthquake damage

The most significant damage to the Imperial County Services Building was the partial collapse just above ground level of the four reinforced-concrete columns along the building's east end (figs. 3, 4). Concrete at the base of each column was badly shattered, vertical reinforcing bars were severely buckled, and horizontal tie bars were widely splayed. On the basis of measurements by the Imperial County Department of Buildings and Grounds, the columns were shortened by approximately 23 cm (9 in.) during the main shock and by approximately 7 1/2 cm (3 in.) more during the strongest aftershock (Randy Rister, oral commun., 1979). Less significant damage elsewhere in the building included minor cracking in all columns beneath the second floor (just beneath the beams), minor cracking or spalling at the base of most columns (just above ground level), and a north-south line of severe cracking (fig. 5) in all upper story floor slabs just to the east of the first interior row of columns (from the east end). The pattern of column damage suggests frame yielding in the longitudinal (east-west) direction and axial-force failure due to north-south overturning of the east-end shear wall; the floor-slab cracks were due to settlement of the structure at the east end.

Strong-motion instrumentation

The building was originally selected for instrumentation under the California Strong-Motion Instrumentation Program because of its structural characteristics, size, and location in a known highly active seismic area (Rojahn and Ragsdale, 1980a). It was initially instrumented in May 1976 with a 9-channel Kinemetrics CRA-1 accelerograph system that was placed in accordance with recommendations by the following three groups: the Instrumentation Subcommittee of the Structural Engineers Association of Southern California (SEAOSC); the California Seismic Safety Commission's Subcommittee on Instrumentation for Buildings; and a site visitation committee composed of personnel representing the various organizations interested in the project (first author, CDMG and SEAOSC representatives, building owner, and design engineer).

The original 9-channel system was triggered by the November 4, 1976 Imperial Valley $M_L = 4.9$ earthquake, the epicenter of which was located approximately 32 km north of the Imperial County Services Building (Porcella and Nielson, 1977). After a review of the November 4 strong-motion record and on the basis of the first author's recommendations, the system was upgraded to its present 13-channel configuration, and a triaxial Kinemetrics SMA-1 accelerograph was added at ground level (intended to be a free-field site) approximately 100 m east of the building. The revised system was installed in May 1978 under the supervision of J. T. Ragsdale and on the basis of building strong-motion instrumentation guidelines developed by the U.S. Geological Survey (Rojahn and Matthiesen, 1977). The instrumentation is maintained by the CDMG office of Strong-Motion Studies.

The SMA-1 accelerograph at the ground-level site east of the building (hereafter referred to as the free-field site) is in a standard fiberglass instrument shelter (fig. 6) founded on a small reinforced-concrete pad. The three-component accelerograph is battery powered, is triggered by vertical motion that equals or exceeds 0.01 *g*, and records analog signals on 70-mm light-sensitive film. The 1-*g* accelerometers have a natural frequency of approximately 25 Hz and thus respond to frequency components nominally within the range 0-25 Hz. Real time is provided on each record by a radio WWVB time-code receiver and time-tick generator system.

The site for the SMA-1 accelerograph was selected on the basis of a distance (from the instrumented structure) criterion suggested by R. B. Matthiesen (oral commun., 1976), as well as its proximity to other buildings. This criterion specifies that sites intended to be free field should be at a distance from the instrumented structure equal to 1 to 1-1/2 times the estimated wavelength of a shear wave (at the surface) having a period equal to the fundamental period of the instrumented structure.

The 13-channel CRA-1 system in the building consists of nine FBA-1 single-axis force-balance accelerometers in various places throughout the upper stories; an east-west HS-0 horizontal starter at roof level; and one FBA-3 triaxial force-balance accelerometer package, one FBA-1 accelerometer, one 13-channel central-recording unit, and a VS-1 vertical starter at ground level. The FBA accelerometers, which have a natural frequency of approximately 50 Hz, are connected by low-voltage data cable to the central-recording unit. This unit is battery powered, is triggered by horizontal or vertical motion that equals or exceeds 0.01 *g*, and records on 178-mm (7 in.) light-sensitive film. The system is designed to record acceleration with frequency components nominally within the range 0-50 Hz and with maximum amplitudes of 1 *g*. Real time is provided by a radio WWVB receiver and a time-tick generator system; the recorder is not connected to the SMA-1 accelerograph east of the building.

The FBA accelerometers (fig. 7) were placed in order to provide information on overall building response as well as input ground motion. The primary purpose of the three north-south-oriented accelerometers at the roof and second floor (accelerometers 1-3, 7-9, fig. 7) is to obtain and isolate north-south translational, torsional, and in-plane floor-bending response. In conjunction with the north-south-oriented accelerometers at ground level (accelerometers 10, 11), these accelerometers provide translational- and torsional-response, mode-shape, and ground-to-second-floor interstory-motion information. Similarly, the accelerometers at the ground floor, second floor, fourth floor, and roof in the more flexible east-west (frame) direction (accelerometers 4-6, 13) provide east-west translational-response, mode-shape, and interstory-motion information. The two north-south-oriented accelerometers at ground level (accelerometers 10, 11) are intended to identify collectively the extent to which differential horizontal ground motion has occurred, and the vertical accelerometer at ground level (accelerometer 12) provides information on vertical motion. There are no vertically oriented accelerometers above ground level.

October 15 accelerograms

Both the 13-channel accelerograph system in the building and the triaxial SMA-1 accelerograph at the free-field site to the east provided complete high-quality strong-motion accelerograms (figs. 8, 9) of the October 15 earthquake. Peak accelerations (as read from the original accelerogram) at the ground floor near the center of the building were 0.29, 0.19, and 0.32 *g*, respectively, for the north (002°), up, and east (092°) components (traces 11, 12, 13, fig. 8). Peak north-south acceleration at the west end at the ground floor (trace 10, fig. 8) was 0.35 *g*, slightly higher than near the center of the building (trace 11), although both components were nearly identical in signature. At the free-field site to the east, peak accelerations were 0.24, 0.27, and 0.24 *g*, respectively, for the 002°, up, and 092° components (fig. 9). The maximum durations of motion between the first and last peak equal to or greater than 0.1 *g* were approximately 6 and 8 s for the vertical and horizontal components, respectively. In terms of frequency content, perhaps the most notable feature of these records is the long-period acceleration pulse in the east-west component at the building's ground floor (trace 13, fig. 8) between seconds 6 and 8. At that point, the acceleration remained positive and relatively high in amplitude (max acceleration, 0.26 *g*) for approximately 1 s and apparently generated large velocity and displacement pulses.

Among the most notable features of the acceleration traces in the upper stories of the building (traces 1-9, fig. 8) are the following: (1) peak roof-level accelerations of approximately 0.59 *g* at 8.9 s and 0.48 *g* at 10.9 s in the north-south and east-west directions (traces 3 and 4, fig. 8), respectively; (2) the abrupt occurrence of long-period motion in the east-west components at the roof, fourth floor, and second floor at 6.8 s (traces 4-6, fig. 8); (3) bursts of low-amplitude high-frequency (approximately 50 Hz) motion at various times in all upper story records at and after 6.8 s; (4) a 0.5-s-long burst of high-amplitude high-frequency (approximately 50 Hz) motion near 11 s in the north-south direction on the second floor (trace 9, fig. 8) directly above the columns that failed; and (5) the continuation of high-amplitude motion in the upper story components after the high-amplitude ground motions had subsided (after 11 s). Most of these features denote critical times in the performance of the building and are discussed later.

Comparison of free-field and building ground-floor motions

The signatures of corresponding components of motion recorded at the free-field site and at the ground floor of the building differ significantly (fig. 10). Vertical motion at the free-field site is generally higher in amplitude than at the ground floor, whereas for the horizontal motion the opposite is true. A comparison of corrected acceleration, velocity, and displacement peak amplitudes on the two records (Porter, in press) indicates that the differences are most noticeable in acceleration (table 1). The maximum vertical acceleration at the free-field site (0.24 _g_), for example, is approximately 30 percent higher than at the ground floor (0.18 _g_); corresponding differences in velocity and displacement for the same component are approximately 5 and 15 percent, respectively. The trend is even more striking for the horizontal components. In the case of the 002° component, the maximum acceleration at the west end of the ground floor (0.34 _g_) is approximately 60 percent higher than at the free-field site (0.21 _g_); corresponding differences in velocity and displacement, however, are substantially smaller (approximately 20 and 5 percent, respectively). Corresponding differences for the 092° component are similar.

There are also significant differences in the frequency content of the two records. The horizontal motions recorded at the ground floor contain relatively high amplitude low-frequency (about 3-4 Hz) components that do not exist in the free-field motion. These frequencies correspond to those of the second mode of building response in both directions (note that the motions at the ground and second floor in fig. 8 are 180° out of phase with that at the roof). This difference in horizontal motion is most noticeable in the 002° (north) component, the direction in which lateral forces are resisted by the shear walls. The difference in motion is evidently related to the fact that the building is founded on piles and suggests that the effective base of the building is below ground level (in other words, that ground level is effectively above the base of the structural system).

Evidence of damage initiation and subsequent column collapse in the October 15 building accelerogram

Various features of the main-shock accelerogram recovered from the building (fig. 8) provide important information on the mechanism of structural failure. These features are clearly evident in a twofold enlargement of the original record (fig. 11) but are not apparent in the plots of corrected digitized data (Porter, in press). The first major feature, an abrupt change in frequency content in the upper story east-west components, occurs at 6.8 s (traces 4 (shaded area A) -6, fig. 11). At that time the predominant period of vibration in all three components abruptly lengthens to approximately 1.6 s (average value for the first 2-1/2 cycles after 6.8 s). This period elongation, which occurs during the long-period acceleration pulse in the east-west component at ground level (max pulse acceleration, 0.24 _g_ (uncorrected)), denotes a sudden decrease in the stiffness of the east-west reinforced-concrete frames. We therefore interpret this feature as the time at which damage begins. Immediately after 6.8 s, bursts of low-amplitude high-frequency motion begin to appear in all upper story components (especially traces 4, 6, 9, fig. 11). These bursts probably reflect the continuation of damage.

The second major feature, a 0.5-s-long burst of high-frequency (approximatey 50 Hz) high-amplitude motion, occurs at 11.0 s in trace 9 (shaded circle B, fig. 11), which is the acceleration time history of the north-south component at the second floor directly above the columns that failed. We interpret this feature to denote the time at which the columns along the building's east face collapsed. At the same time, the frequency content of the north-south-component time histories recorded at both ends of the roof changed (traces 1, 3, fig. 11). Before 11.0 s the frequency of these two traces is roughly equivalent, that is, both ends of the roof are vibrating in phase; but after 11.5 s their frequencies are no longer the same. The predominant frequency at the west end of the roof after 11.5 s (trace 1, fig. 11) is approximately 1.7 Hz, whereas that at the east end of the roof (trace 3) is approximately 0.8 Hz. Corresponding predominant periods for the two components are 0.6 s (west end) and 1.2 s (east end). These data imply that the stiffness characteristics of the building in the north-south direction at the east and west ends were altered substantially between 11.0 and 11.5 s, and because the columns collapsed at the east end, this sudden difference in stiffness must denote that collapse.

Ground shaking amplitudes before damage initiation and column collapse

Because the intensity of ground shaking and the building response before and during the times of damage initiation and column collapse are of great importance and because there is little documentation on the subject, it is of interest to examine selected time histories of ground motion and building response for these critical moments. Our discussion here is limited to the recorded acceleration and calculated velocity time histories (Porter, in press) and excludes calculated displacement time histories because of the uncertain effect of record processing on displacements obtained from doubly integrated accelerograms. For brevity, we also limit our discussion to ground-floor and roof-level records (components relevant to response-spectra studies).

The acceleration and velocity time histories for the three ground-level components recorded near the east end of the building (traces 11-13, fig. 11; fig. 12) indicate that the time of damage initiation (6.8 s) was approximately 2 s after the onset of high-amplitude horizontal motion and that the beginning of column collapse at 11.0 s occurred 4 s later, near the end of these high-amplitude motions, that is, just after the strongest motions had subsided. At the time of damage initiation, for example, ground accelerations in the north-south, vertical, and east-west directions were approximately 0.26, 0.12, and 0.24 g, respectively, whereas at the beginning of column collapse they were approximately 0.07, 0.02, and 0.03 g, respectively. More significant, perhaps, is the evidence that damage initiation occurred during the long-period (greater than 1 s) acceleration pulse in the east-west direction (trace 13, fig. 11) and that it immediately followed the maximum ground velocity of 42.4 cm/s in the north-south direction (fig. 12). It is also noteworthy that (1) the maximum east-west velocity of 64.6 cm/s occurred approximately 1 s after damage initiation and 3 s before column collapse, and (2) peak velocities exceeded 40 cm/s on two occasions (once in the east-west and once in the north-south direction) before damage initiation and on four occasions before column collapse (table 2). These observations, particularly those regarding the number of velocity peaks greater than 40 cm/s and the times of occurrence of the long-period acceleration-pulse and maximum-velocity amplitudes, should assist in specifying the ground motions that may be damaging to structures having dynamic properties similar to those of the Imperial County Services Building. The data also suggest that, had the high-amplitude ground shaking lasted longer, as it did during the 1940 Imperial Valley earthquake (Matthiesen and Porcella, in press), column collapse would have occurred during (not after) these strong motions, and the resulting damage might have been far more severe.

The acceleration and velocity time-histories for the roof-level components (traces 1-4, fig. 11; fig. 13) indicate that the time of damage initiation (6.8 s) was approximately 2 s after the onset of high-amplitude horizontal motions, as was the case at ground level. The plots also indicate that the beginning of column collapse at 11.0 s occurred near the end of high-amplitude response in the north-south direction, whereas in the east-west direction the response level remained high for several seconds after column collapse. In addition, the data indicate that: (1) at damage initiation (6.8 s), peak east-west and north-south accelerations were approximately 0.28 and 0.44 g (avg for traces 1-3), respectively; (2) the maximum east-west acceleration (0.48 g) occurred just before column collapse (11.0 s); (3) the maximum north-south acceleration (0.59 g) occurred midway between damge initation and column collapse; (4) the building underwent roof-level torsional response approximately 1.3 s before column collapse (at 9.7 s the north-south acceleration at the east end of the building was approximately 0.53 g, whereas at the west end it was 0.26 g); (5) both the east-west and north-south (trace 3, east end) peak velocity responses exceeded 40 cm/s on one occasion before damage initiation and on six occasions before column collapse (table 3); (6) peak north-south (trace 3, east end) velocity response exceeded 60 cm/s on one occasion before column collapse (table 3); and (7) peak east-west velocity response exceeded 80 cm/s on three occasions before column collapse (table 3). These latter observations provide information on the number of cycles of motion (each peak corresponds to half a cycle) above various response thresholds before damage initiation and column collapse; they should therefore aid interpretation of the velocity response envelope spectra (Perez, 1973) obtained from other sites during other earthquakes. In general, the above observations should also help in determining the response levels at which damage can be expected to occur in structures with dynamic properties similar to those of the Imperial County Services Building.

Building fundamental-period changes

Preearthquake ambient-vibration data and the October 15 accelerogram recovered from the building indicate that the building's fundamental period in both principal directions changed over time and with the strength of ground shaking. Results from an ambient vibration study conducted in spring 1979 (Pardoen, 1979) indicated that the fundamental periods in the north-south and east-west directions under ambient conditions were 0.45 and 0.65 s, respectively. The October 15 accelerogram (fig. 11), on the other hand, indicates that these periods were substantially longer during earthquake excitation and that they fluctuated during the earthquake. During the initial strong ground shaking but before the initiation of damage (between 5.0 and 6.8 s), the east-west fundamental period is estimated to have been approximately 1.0 s (we were unable to infer from the original accelerogram the north-south fundamental period during this time). After damage initiation but before column collapse (between 6.8 and 11.0 s) the north-south fundamental period steadily lengthens from about 0.6 to 0.8 s (traces 1-3, fig. 11), whereas the east-west fundamental period is about 1.6 s (trace 4, fig. 11). After column collapse and until the amplitudes of building response subside (after 11.5 s), the fundamental period in the east-west direction is estimated to be 1.7 s, and that in the north-south direction is no longer consistent throughout the length of the building. During this time, as indicated earlier in this report, the north-south fundamental period at the west end of the building is estimated to be about 0.6 s, and that at the east end 1.2 s.

A comparison of the preearthquake and earthquake periods (table 4) indicates that: (1) during the initial strong ground shaking but before the initiation of damage, the building's east-west fundamental period increased by approximately 50 percent relative to that under ambient conditions; (2) after damage initiation but before the time of column collapse, the east-west fundamental period increased from that under ambient conditions by approximately 150 percent, and the north-south fundamental period increased with time by approximately 30 to 80 percent; and (3) during the final excursions of high-amplitude building response after column collapse, the east-west fundamental period increased by about 160 percent, and the north-south fundamental periods about 30 and 170 percent at the west and east ends of the building, respectively.

Preliminary relative displacement analysis

Our analysis of the building accelerogram also includes computing and interpreting the relative-displacement time histories that are obtained by differencing the displacement time histories of various acceleration traces (fig. 8). We refer to this relative-displacement analysis as a preliminary effort to stress that uncertainties are introduced when displacements are obtained by doubly integrating accelerograms. We believe we have minimized the uncertainties by differencing and careful filtering.

Rather than using the corrected displacement time histories computed by the CDMG (Porter, in press), we elected to process the uncorrected acceleration data using different Ormsby-filter limits (Ormsby, 1961). The filter limits were selected on the basis of recent studies by Hanks (1975) and Basili and Brady (1978) which suggest that the uncertainties in displacements calculated using the Ormsby filter (Trifunac and Lee, 1973; U.S. Geological Survey, 1976) increase with the period. Therefore, we chose limits to exclude most components of motion with periods longer than the longest fundamental period of the building observed in the original accelerogram (1.7 s). The resulting filter had a gain of unity between the frequencies 0.5 to 23 Hz, and fell linearly to zero from 0.50 to 0.10 Hz and from 23 to 25 Hz. In terms of period, the removal of the long-period content commenced at 2 s and was complete at 10 s.

The time history of north-south differential motion at the ground floor (fig. 14), computed by differencing displacement time histories for traces 10 and 11 (fig. 8), indicates that the maximum differential motion between the west and east ends of the building (accelerometers 10, 11, fig. 7) was approximately 0.6 cm. Since accelerometers 10 and 11 are 30 m apart, these data suggest that the maximum horizontal shear strain at the ground floor was 0.6 cm/30 m, or about 2×10^{-4}. Because the cyclic relative motion is similar in frequency content to that of the upper story north-south components, the extent of relative motion is probably influenced, to a large degree, by the structure itself.

East-west relative-displacement time histories, computed by differencing east-west displacement time histories for the various floor levels, provide insight into the overall extent of building response. Roof minus ground-floor, fourth-floor minus ground-floor, and second-floor minus ground-floor east-west relative-displacement data (fig. 15), computed by differencing displacement time histories for traces 4 and 13, 5 and 13, and 6 and 13 (figs. 7, 8), respectively, suggest that the east-west relative displacements between the second and ground floors (hereafter referred to as the first story) were substantially larger than those between any other adjacent floors (table 5). More specifically, the data suggest that the maximum interstory displacement was approximately 2 cm (average value) in the fourth, fifth, and sixth stories, approximately 3 cm in the second and third stories, and approximately 8 cm in the first story.

North-south relative-displacement time histories, computed by differencing north-south displacement time histories, for various places (west end, center, and east end) at the roof, second floor, and ground floor (fig. 16, table 6), suggest that interstory motion in the north-south direction was substantially less than in the east-west direction (fig. 15, table 5). Between the roof and ground floor at the east end of the building, for example, the maximum north-south relative displacement was about 9 cm, whereas in the east-west direction it was approximatey 20 cm. These data also suggest that the north-south relative displacements between the second and ground floors at the east end of the building (max amplitude, 2.7 cm; fig. 16), where the columns collapsed, were substantially larger than at the center and west end (max amplitudes, 0.7 and 1.1 cm, respectively; fig. 16). These data imply, then, that the row of columns at the east end of the building underwent larger north-south relative displacements between the ground and second floors than did any other north-south row of columns in the first story. These data also imply that that part of the second-floor diaphragm between the east end of the building and the easternmost first-story shear wall (9.4 m to the west) did not remain rigid in its own plane but rather sustained relatively large north-south bending or shear distortions. These diaphragm distortions are clearly evident in figure 17, which shows the time histories of north-south relative displacements between the center and east end (max amplitude, 2 cm) and center and west end (max amplitude, 1 cm) of the second-floor diaphragm.

The superposition of north-south and east-west time histories of relative displacement at the east end between the second and ground floors (fig. 18) provides insight into the two-dimensional horizontal motion there before, during, and after the beginning of damage and the collapse of the columns. This plot indicates that: (1) at the time of damage initiation, the amplitudes of north-south and east-west relative displacement between the second and ground floors were approximately 0.9 and 1.1 cm, respectively; (2) the maximum north-south first-story relative displacement of approximately 2.7 cm occurred approximately 1.3 s before column collapse began; (3) column collapse apparently began when the east-west relative displacement was approximately 6.2 cm, or 81% percent of the maximum (7.7 cm) that occurred approximately 0.1 s later; and (4) the north-south first-story relative displacement was relatively small (0.4 cm) when column collapse began. Also, the amplitude of north-south relative displacement between the roof and ground floor at the east end (fig. 16) when column collapse began was also small (0.8 cm); in other words, at the time of column collapse, the building was alined nearly vertical in the north-south direction.

Summary and additional conclusions

The October 15 accelerograms recovered from the structurally damaged Imperial County Services Building and adjacent free-field site constitute a valuable data set for earthquake engineering studies. The 13-channel remote-accelerometer central-recording CRA-1 accelerograph system in the building performed excellently and provided a 90-s-long accelerogram in analog form that is of great interest, primarily because the time and mechanism of damage, as well as the associated ground shaking and structural response, can be inferred from the recorded data.

The accelerograms from the building and adjacent free-field site indicate that the motion recorded at the ground floor of the building incorporates to some significant extent building-soil-foundation system response. More specifically, the data suggest that the effective base of the building, founded on piles, is below the ground floor. The records also show that the maximum horizontal acceleration recorded at the ground floor was approximately 60 percent higher than at the free-field site. Corresponding increases in velocity and displacement, however, were substantially less.

Features of the building accelerogram that provide insight into the mechanism of structural damage include abrupt changes in frequency content in the upper story north-south and east-west components, as well as bursts of high-frequency motion. These features suggest that damage began 6.8 s after the instrument was triggered and that the collapse of the four columns along the building's east face began 4.2 s later, or 11.0 s after triggering.

The recorded acceleration data and calculated velocity time histories (Porter, in press) indicate that the time of damage initiation was approximately 2 s after the onset of strong ground motion during a long-period acceleration pulse in the east-west direction, and that it immediately followed the maximum ground velocity of 42.4 cm/s in the north-south direction. The time of column collapse was immediately after the strong ground motions. Before damage began, peak horizontal ground velocity exceeded 40 cm/s on two occasions (once in each direction), and before column collapse, on four occasions (twice in each direction). Also before column collapse, roof-level peak velocity exceeded 40 cm/s on 12 occasions (six in each direction), 60 cm/s on 4 occasions (once in the north-south direction and three times in the east-west direction), and 80 cm/s on 3 occasions (all in the east-west direction). The roof-level data also indicate that the peak east-west and north-south accelerations at the time of damage initiation were approximately 0.28 and 0.44 *g*, respectively, that the maximum east-west acceleration (0.48 *g*) occurred just before the time of column collapse, that the maximum north-south acceleration (0.59 *g*) occurred approximately 2 s before column collapse, and that the roof underwent torsional response approximately 1.3 s before column collapse.

The original accelerogram also suggests that the building's fundamental period in both principal directions changed significantly during the earthquake. During strong ground shaking but before damage initiation, for example, the fundamental period in the east-west direction (the direction in which lateral forces are resisted by frame action) was approximately 1.0 s, or 50 percent longer than that measured under ambient conditions (0.65 s) in spring 1979. After damage initiation and before column collapse, the period was approximately 1.6 s, or approximately 150 percent longer than the ambient period; and during the final excursions of high-amplitude building response after column collapse approximately 1.7 s, or 160 percent longer. Corresponding increases in the north-south fundamental period (lateral forces are resisted by shear walls in this direction) were generally smaller with one exception: after column collapse, the fundamental period in the north-south direction at the east end of the building was approximately 1.2 s, or 170 percent longer than that measured under ambient conditions (0.45 s). At the west end during the same interval it was approximately 0.6 s, or 30 percent longer.

Results of a preliminary relative-displacement analysis, based on data corrected using specially selected Ormsby-filter limits, suggest that north-south differential motion between two points 30 m apart at ground level was small (max 0.6 cm, which corresponds to a shear strain of 2×10^{-4}), that the four columns at the east end of the building sustained larger relative displacements between the second and ground floors in the north-south direction (2.7 cm) than did any other north-south row of first-story columns, and that the large interstory displacements between the second and ground floors in the east-west direction (7.7 cm maximum) played a key role in the column collapse mechanism. At the time of damage initiation (6.8 s), the maximum north-south and east-west first-story relative displacements at the east end of the building were approximately 0.9 and 1.2 cm, respectively. At the beginning of column collapse (11.0 s), the east-west first-story relative displacement was approximately 6.2 cm, or 15 times that in the north-south direction at the east end (0.4 cm). The relative-displacement data also indicate that the building was alined nearly vertical in the north-south direction at the beginning of column collapse. This orientation implies that axial loads due to north-south overturning moments played only a minor role in the actual collapse beginning at 11.0 s, whereas stresses due to large east-west interstory displacements played a major role. Before column collapse, however, the large north-south relative displacements between the roof and ground floor (8.5 cm maximum) suggest that north-south overturning moments played a more significant role. Collectively, the relative displacement data suggest that, from 6.8 to 11.0 s, stresses in the east-end first-story columns were higher than in any other row, presumably because these columns underwent the largest axial strains (from north-south and east-west overturning moments) as well as the largest flexural stresses (from north-south interstory displacements), in addition to the flexural stresses (due to east-west frame action) that affected all the first-story columns about equally. In short, the relative-displacement data suggest that north-south overturning moments and north-south relative displacements between the second and ground floors played a significant role during damage initiation and the continuation of yielding before column collapse (in weakening the east row of first-story columns), but that collapse was primarily caused by stresses due to large east-west interstory displacements--that is, by a combination of east-west flexural stresses and east-west overturning-moment axial stresses.

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References cited

Basili, M., and Brady, A. G., 1978, Low frequency filtering and the selection of limits for accelerogram corrections: European Conference on Earthquake Engineering, 6th, Dubrovnik, Yugoslavia, 1978, Proceedings, p. 251-258.

Hanks, T. C., 1975, Strong ground motion of the San Fernando, California, earthquake--ground displacements: Seismological Society of America Bulletin, v. 65, no. 1, p. 193-225.

Matthiesen, R. B., and Porcella, R. L., (in press), Strong-motion data recorded in the United States, in Johnson, C. E., Rojahn, C., and Sharp, R. V., eds., The Imperial Valley, California, earthquake of October 15, 1979: U.S. Geological Survey Professional Paper.

Ormsby, J. F. A., 1961, Design of numerical filters with applications to missile data processing: Association for Computer Machinery Journal, v. 8, p. 440-466.

Pardoen, G. C., 1979, Imperial County Services Building ambient vibration test results: Christchurch, University of Canterbury Report 79-14, 21 p.

Perez, Virgilio, 1973, Velocity response envelope spectrum as a function of time, in San Fernando, California, earthquake of February 9, 1971: National Oceanic and Atmospheric Administration, Environmental Research Laboratories, v. 3, p. 393-401.

Porcella, R. L., and Nielsen, J. D., 1977, Preliminary report on the Calipatria, California earthquake swarm: November, 1976, in Porcella, R. L., ed., Seismic engineering program report, October-December 1976: U.S. Geological Survey Circular 736-D, p. 1-3.

Porter, L. D., (in press), Data processing procedures for the main-shock motions recorded by the California Division of Mines and Geology strong-motion network, in, Johnson, C. E., Rojahn, C., and Sharp, R. V., eds., The Imperial Valley, California, earthquake of October 15, 1979: U.S. Geological Survey Professional Paper.

Rojahn, Christopher, and Matthiesen, R. B., 1977, Earthquake response and instrumentation of buildings: American Society of Civil Engineers Proceedings, Technical Councils of ASCE Journal, v. 103, no. TC1, p. 1-12.

Rojahn, Christopher, and Ragsdale, J. T., 1980a, Building instrumentation phase of the California Strong-Motion Instrumentation Program: Annual Convention of the Structural Engineers Association of California, Lake Tahoe, Calif., 1978, Proceedings, p. 21-39.

__________, 1980b, Strong-motion records from the Imperial County Services Building, El Centro, in Leeds, D. J., ed., Imperial County, California, earthquake, October 15, 1979: Berkeley, Calif., Earthquake Engineering Research Institute Reconnaissance Report, p. 173-184.

Trifunac, M. D., and Lee, V. W., 1973, Routine computer processing of strong-motion accelerograms: Pasadena, California Institute of Technology, Earthquake Engineering Research Laboratory Report EERL 73-03, 360 p.

U.S. Geological Survey, 1976, Strong-motion earthquake accelerograms, digitization and analysis, 1971 records: Open-File Report 76-609, 135 p.

Table 1.--Free-field and ground-floor motion comparisons, Imperial County Services Building, October 15, 1979

Component	Maximum acceleration (g)		Maximum velocity (cm/s)		Maximum displacement (cm)	
	Free field	Ground floor	Free field	Ground floor	Free field	Ground floor
002°*	0.21	0.34	36.2	43.3	16.4	16.0
Up	.24	.18	17.4	16.2	8.0	7.0
092°	.24	.33	64.4	64.6	28.2	27.4

*Trace 10, figure 8.

Table 2.--Peak ground-floor velocities before beginning of column collapse, Imperial County Services Building, October 15, 1979

Peak Number	Time[1] (s)	Amplitude (cm/s)
East component (accelerometer 13)[2]		
1	5.60	-40.3
(damage initiation)	6.8	----
2	7.54	64.6*
3	9.30	-31.4
4	10.32	26.6
(column collapse)	11.0	----
North component (accelerometer 11)[2]		
1	5.94	-29.4
2	6.62	42.4*
(damage initiation)	6.8	----
3	7.20	-9.7
4	8.01	28.6
5	8.80	-18.5
6	9.38	22.2
7	9.91	-42.0
8	11.00	12.9
(column collapse)	11.0+	----

[1]After triggering.
[2]See figure 7.
*Maximum component amplitude.

Table 3.--Peak roof-level velocities before beginning of column collapse, Imperial County Services Building, October 15, 1979

Peak Number	Time[1] (s)	Amplitude (cm/s)
East component (accelerometer 4)[2]		
1	5.70	59.5
2	6.28	18.0
3	6.76	-38.0
(damage initiation)	6.8	----
4	7.49	95.9
5	8.40	-58.9
6	9.18	49.8
7	9.84	-81.0
8	10.72	98.1*
(column collapse)	11.0	----
North component (accelerometer 3)[2]		
1	5.96	-37.8
2	6.70	58.5
(damage initiation)	6.8	----
3	7.06	-23.9
4	8.13	46.4
5	8.43	-45.9
6	8.74	35.1
7	8.99	-43.8
8	9.42	52.5
9	9.84	-72.5*
10	10.31	15.9
11	10.68	-4.3
(column collapse)	11.0	----

[1]After triggering.
[2]See figure 7.
*Maximum component amplitude.

Table 4.--Fundamental periods, Imperial County Services Building

Measurement date and type	North-south direction (s)	East-west direction (s)
Spring 1979--ambient (Pardoen, 1979) - - - - - - - - - - - -	0.45	0.65
October 15, 1979 Earthquake--		
Before initiation of damage (during high-amplitude ground shaking)- - -	*	1.0
After initiation of damage but before column collapse- - - - -	0.6 to 0.8	1.6
After column collapse (during high-amplitude building response) -	0.6 (west end) 1.2 (east end)	1.7

*Not able to infer from original accelerogram.

Table 5.--Selected east-west relative displacements, Imperial County Services Building, October 15, 1979

Time* (s)	Relative horizontal displacement (cm)		
	2nd floor minus ground floor	4th floor minus 2nd floor	Roof minus 4th floor
6.8 (damage initiation)	1.1	0.6	0.4
9.4	5.1	5.9	5.8**
10.3	-7.4	-6.3**	-5.8
11.0 (beginning of column collapse)	6.2	5.8	5.2
11.1	7.7**	4.4	3.9

*After triggering.
**Maximum.

Table 6.--Selected north-south relative displacements, Imperial County Services Building, October 15, 1979

Time* (s)	Relative horizontal displacement (cm) West end	Center	East end
	2nd floor minus ground floor		
6.8 (damage initiation)	-0.1	0.1	0.9
8.8	.7	.4	1.6
9.1	-1.1**	-.4	-1.0
9.2	-.7	-.3	-1.9
9.7	.2	.5	2.7**
11.0 (beginning of column collapse)	.6	.3	-.4
11.2	.5	.7**	2.1
	Roof minus second floor		
6.8 (damage initiation)	1.6	1.6	0.4
8.8	4.3**	4.7	1.7
9.1	-3.2	-3.5	2.4
9.2	-3.4	-4.9**	-3.8
9.7	2.3	4.8	5.9**
11.0 (beginning of column collapse)	.0	-1.2	-.4
11.2	1.4	-.2	-1.4

*After triggering.
**Maximum.

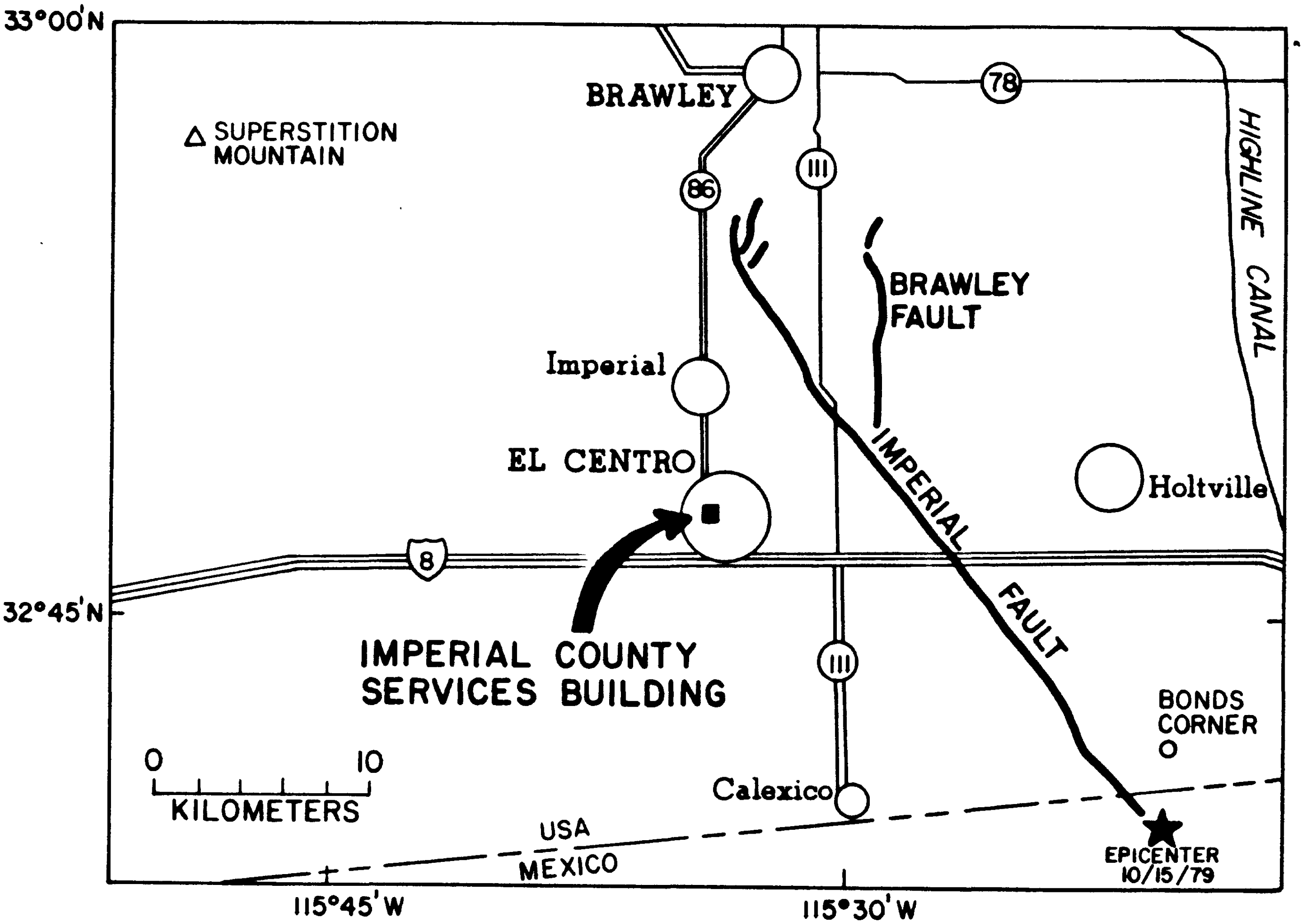

Figure 1.--Location of Imperial County Services Building relative to Imperial fault trace and October 15 main-shock epicenter.

Figure 2.--View northwestward of Imperial County Services Building, El Centro, Calif.

Figure 3.--View northward of east end of Imperial County Services Building, showing row of columns (far right) that failed during October 15 main shock.

Figure 4.--One of four damaged reinforced-concrete columns along east end of Imperial County Services Building between ground and second floors. Damage to other three columns was similar.

Figure 5.--North-south-trending crack in sixth floor just east of first interior row of columns (from east face).

Figure 6.--Fiberglass instrument shelter (foreground) housing SMA-1 accelerograph located east of Imperial County Services Building (background). Note solar cells mounted on pole adjacent to shelter; these cells provide current for accelerograph's battery charger.

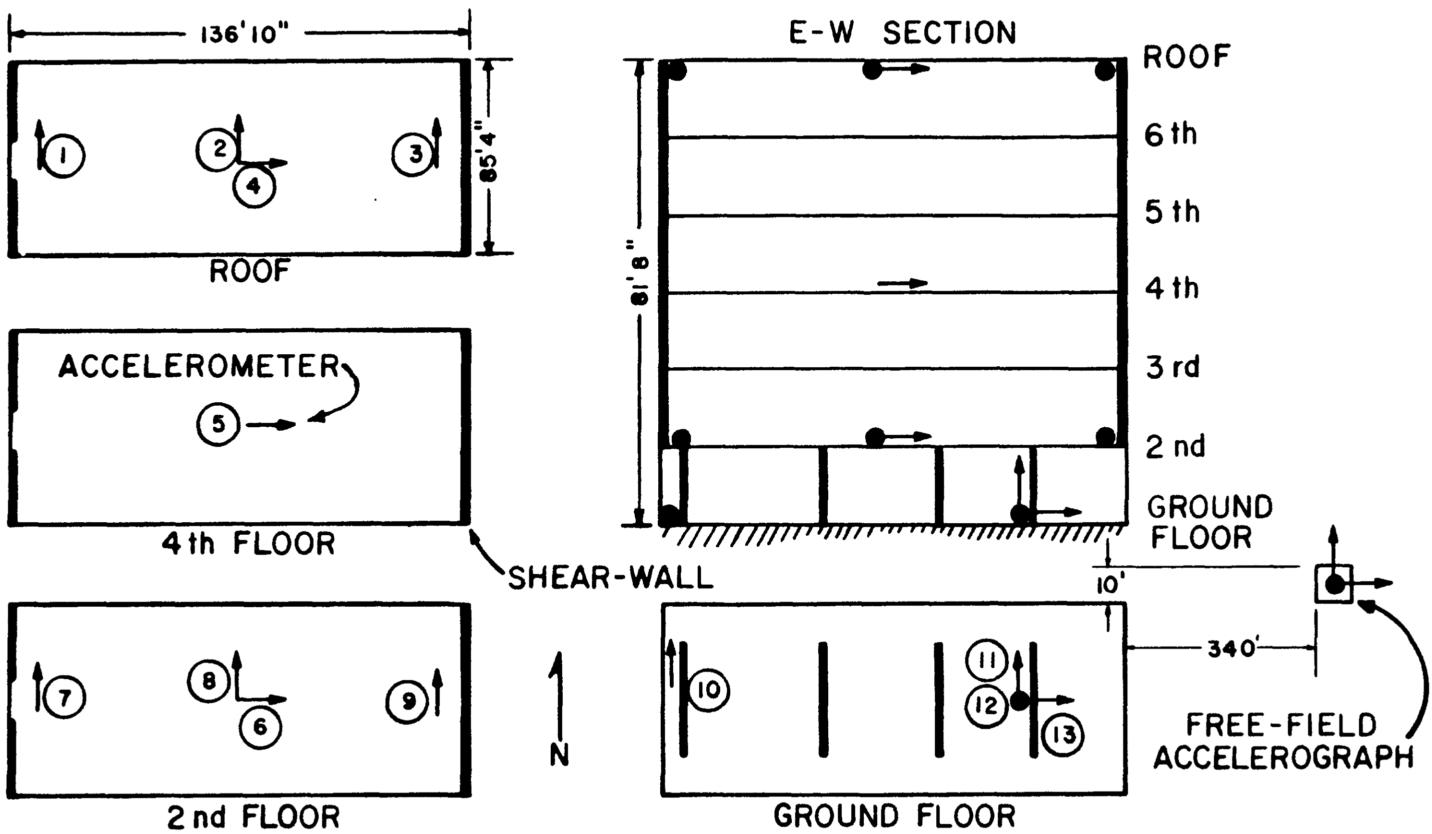

Figure 7.--Locations of FBA accelerometers (arrows with numbers) and SMA-1 accelerograph at Imperial County Services Building and adjacent free-field site (after Rojahn and Ragsdale, 1980b). Arrows denote direction of positive acceleration (on trace).

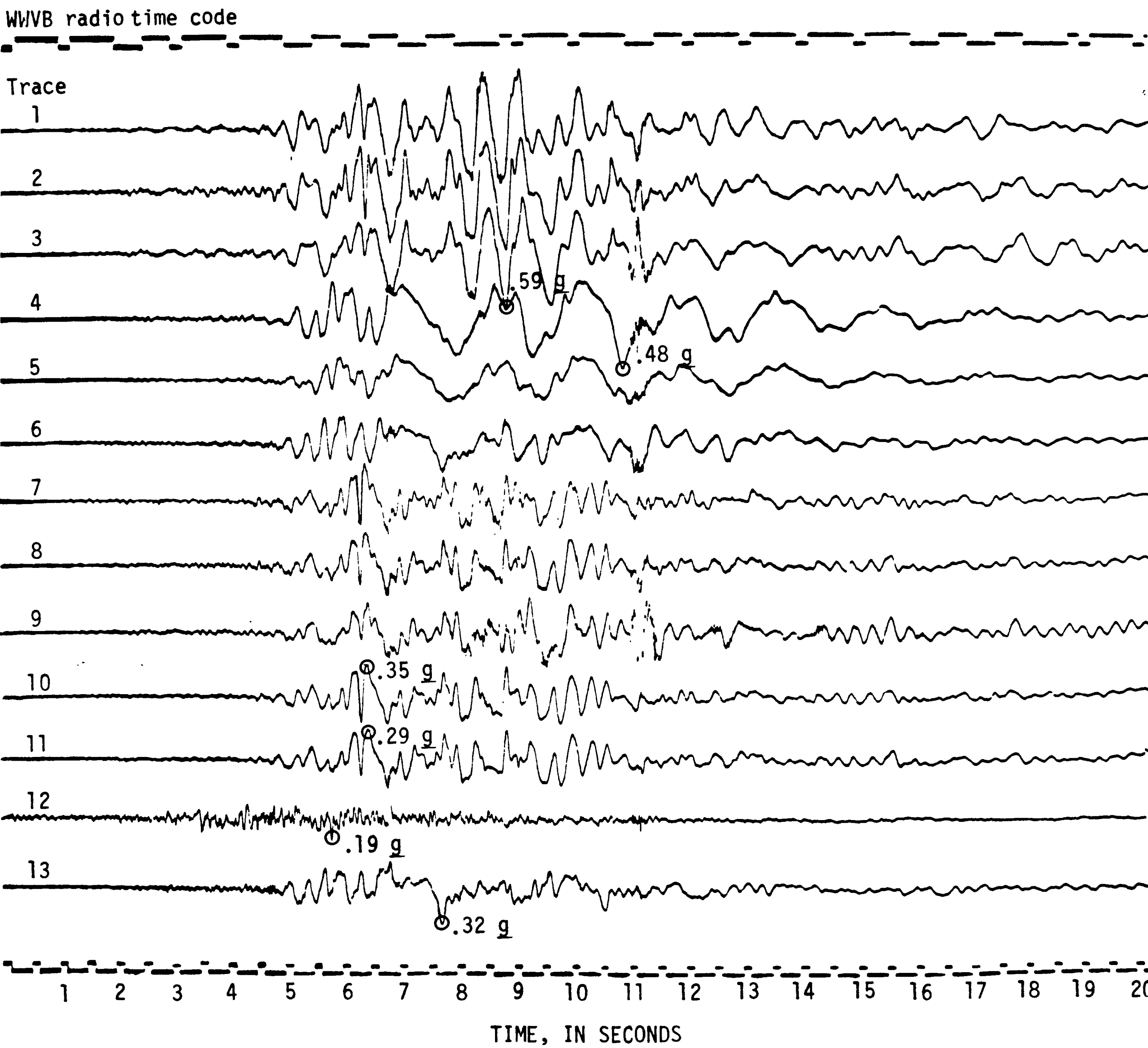

Figure 8.--Part of CRA-1 strong-motion accelerogram recorded on October 15 in Imperial County Services Building. Total record length was 90 s (Porter, in press). Trace number at start of record (left of figure) corresponds to accelerometer numbers in figure 7.

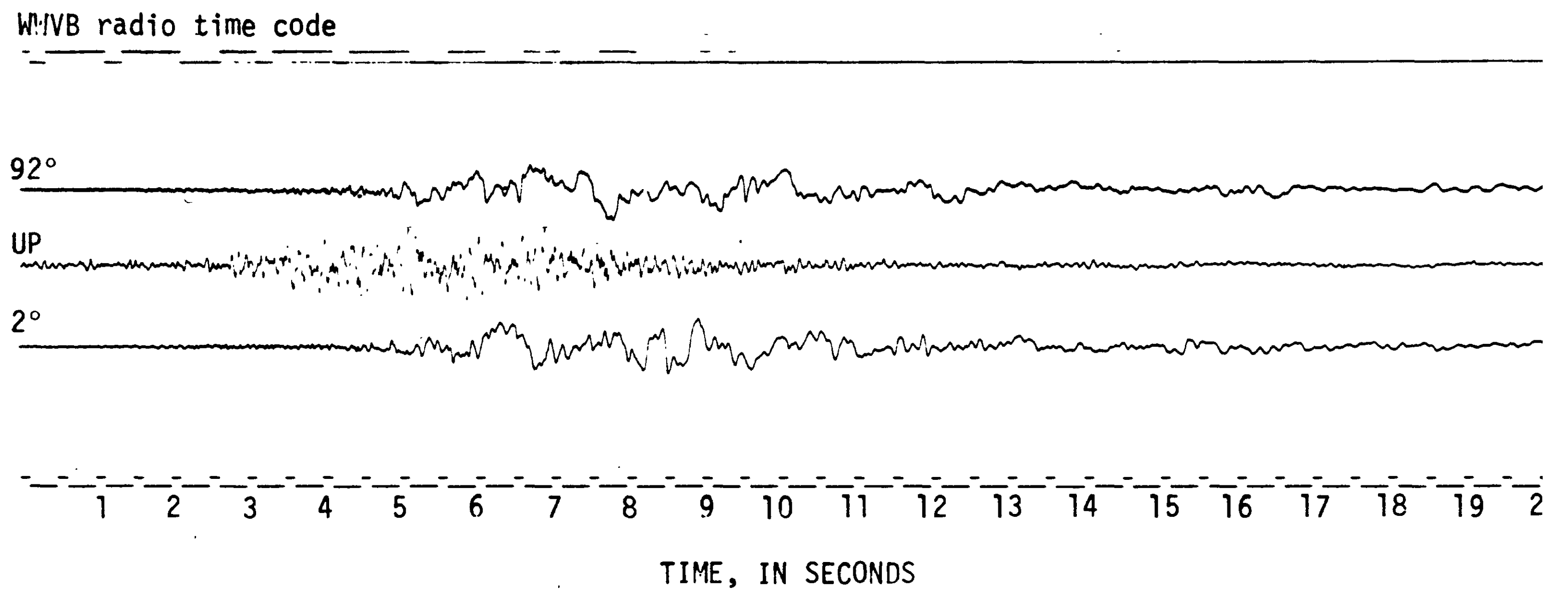

Figure 9.--Part of SMA-1 strong-motion accelerogram recorded on October 15 at free-field site 100 m east of Imperial County Services Building. Direction of positive acceleration given at beginning of trace.

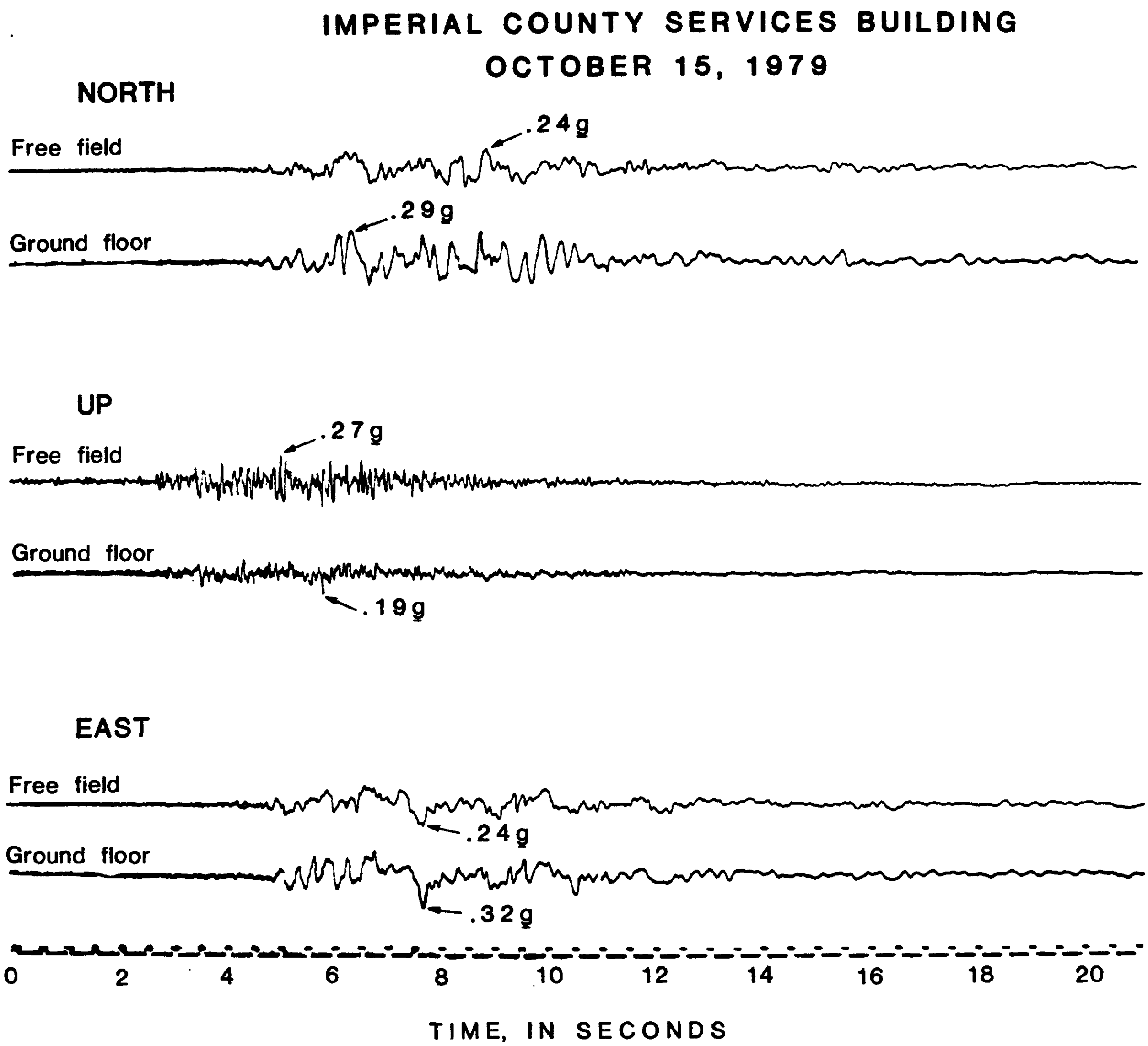

Figure 10.--October 15 acceleration-time histories recorded at Imperial County Services Building and adjacent free-field site.

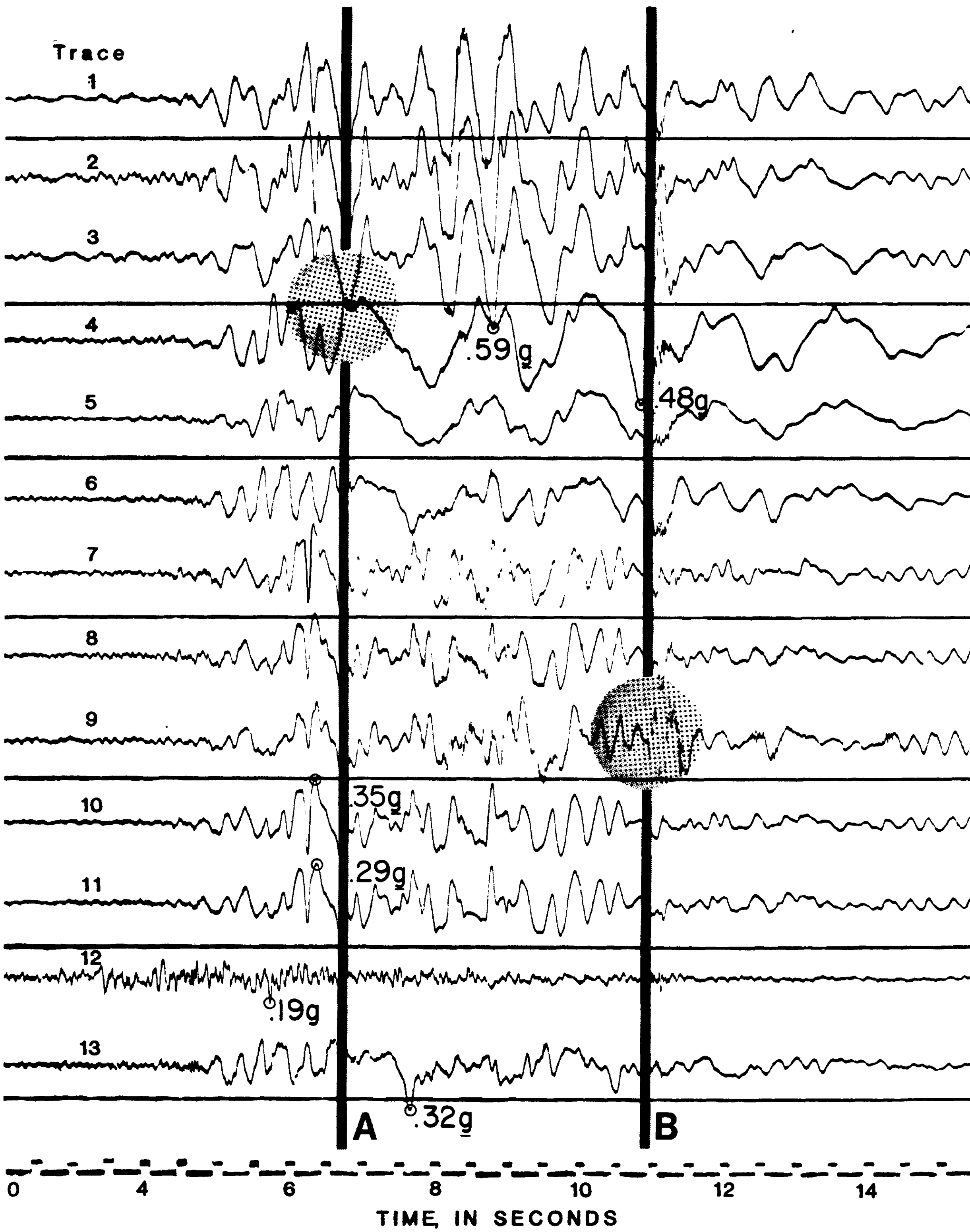

Figure 11.--Enlargement of critical portion of October 15 accelerogram recorded in Imperial County Services Building. Shaded circles denote features discussed in text. Times at which damage initiation occurred and column collapse began are labeled A and B, respectively.

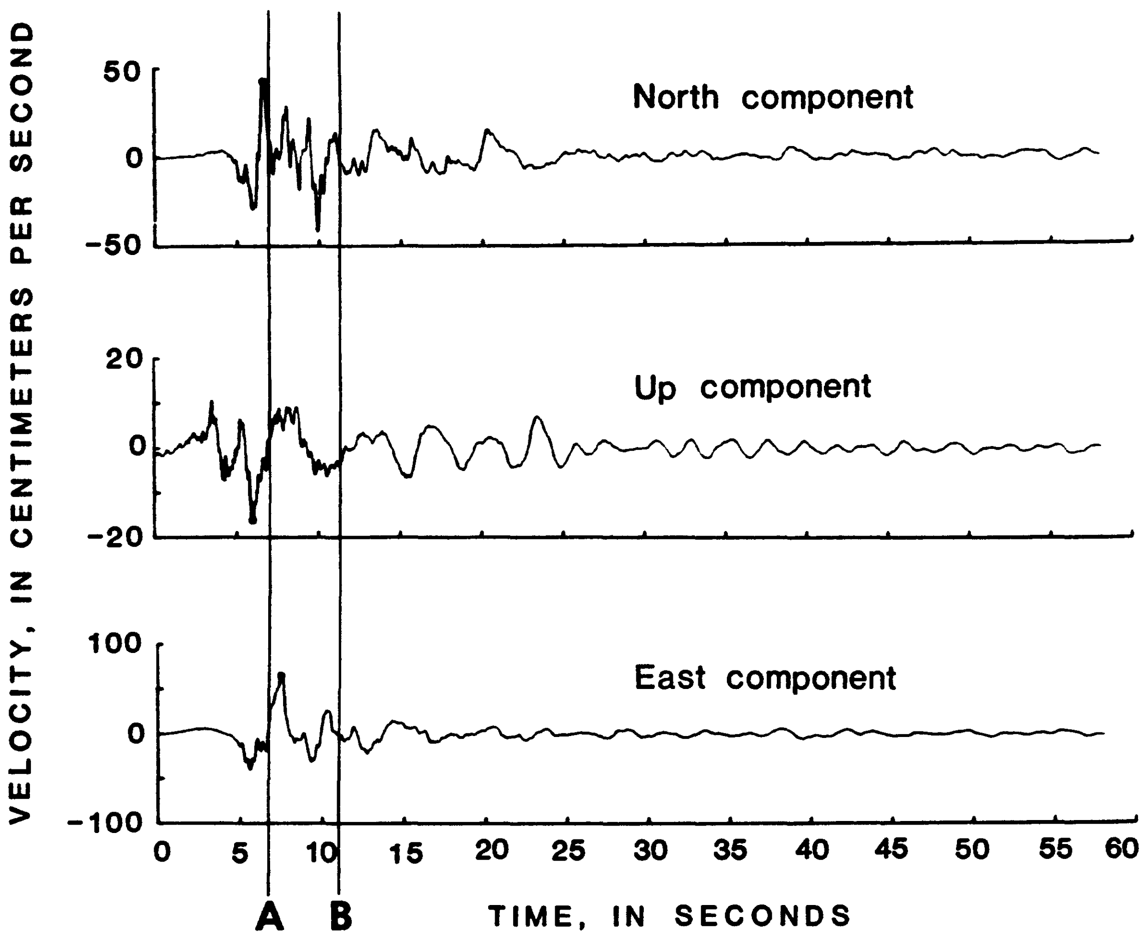

Figure 12.--Velocity-time histories for north, up, and east components (accelerometer 11) recorded at ground level (after Porter, in press). Times at which damage initiation occurred and column collapse began are labeled A and B, respectively.

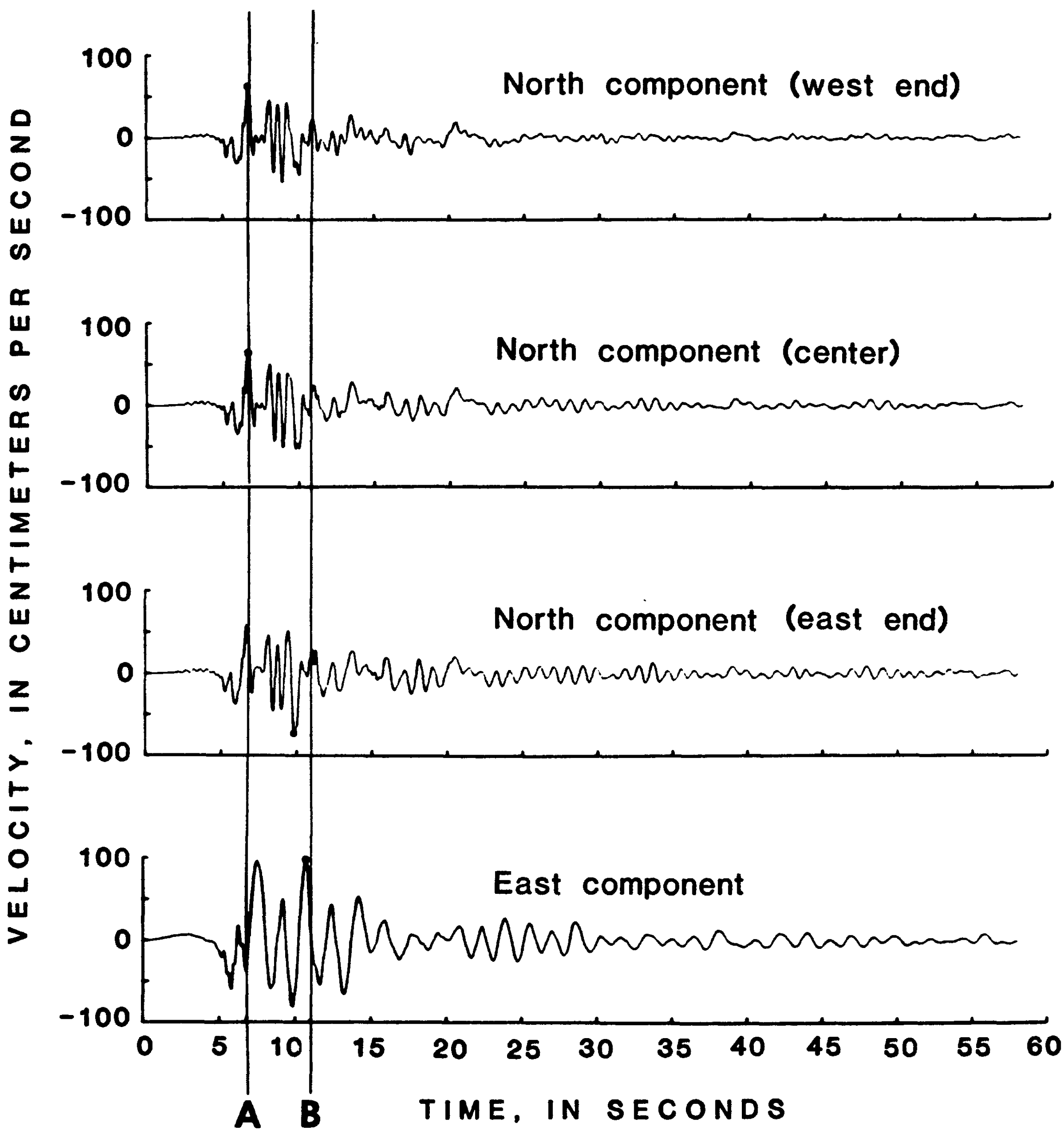

Figure 13.--Velocity-time histories for north and east components recorded at roof level (after Porter, in press). Times at which damage initiation occurred and column collapse began are labeled A and B, respectively.

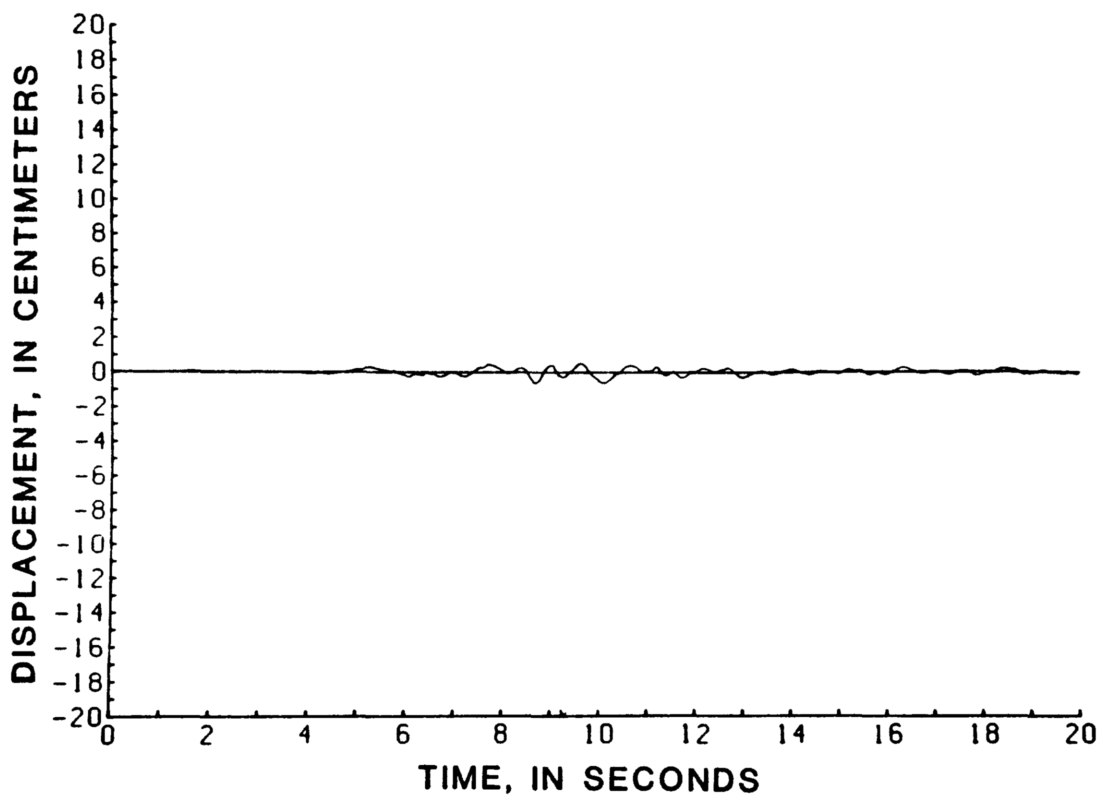

Figure 14.--Time history of north-south relative displacements at ground level, computed by differencing displacement-time histories for acceleration traces 10 and 11 (fig. 8).

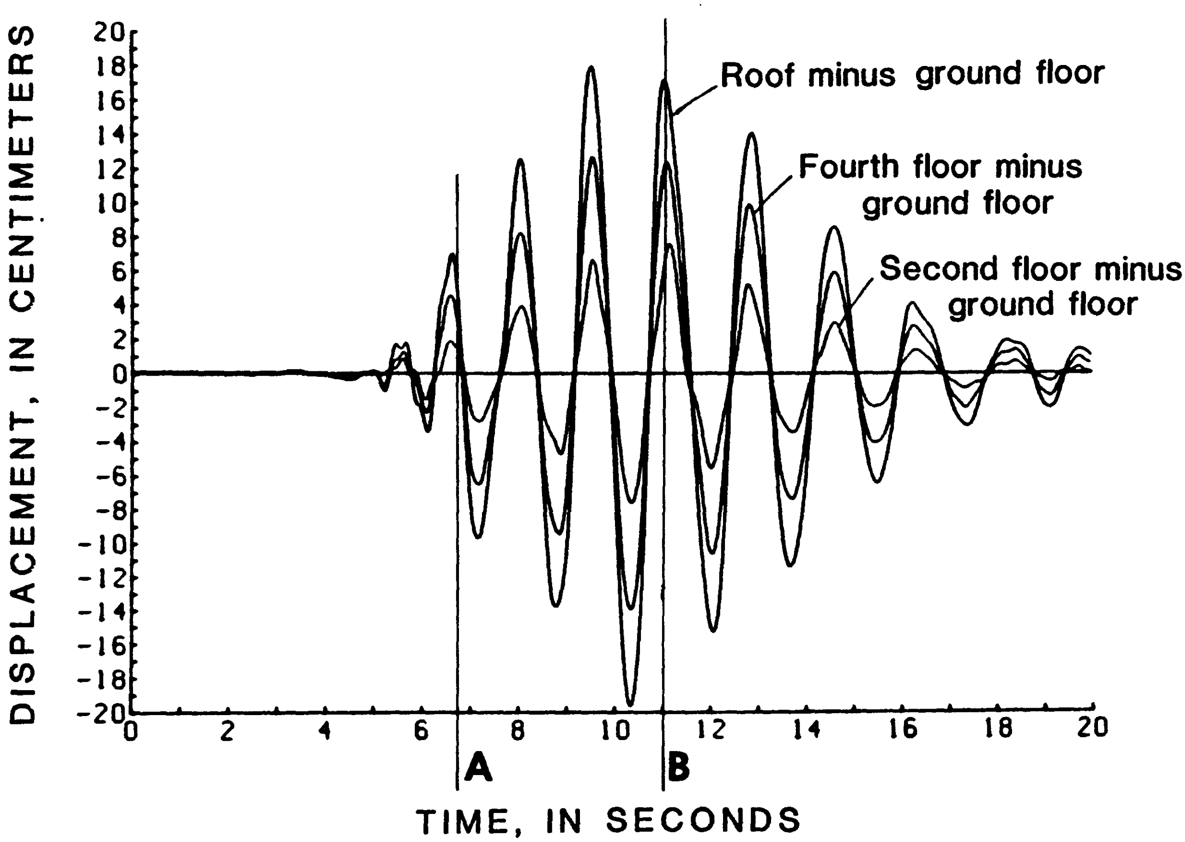

Figure 15.--Time history of east-west relative displacements between roof and ground floor (difference of data from accelerometers 4 and 13), fourth and ground floor (difference of data from accelerometers 5 and 13), and second and ground floor (difference of data from accelerometers 6 and 13). Times at which damage initiation occurred and column collapse began are labeled A and B, respectively.

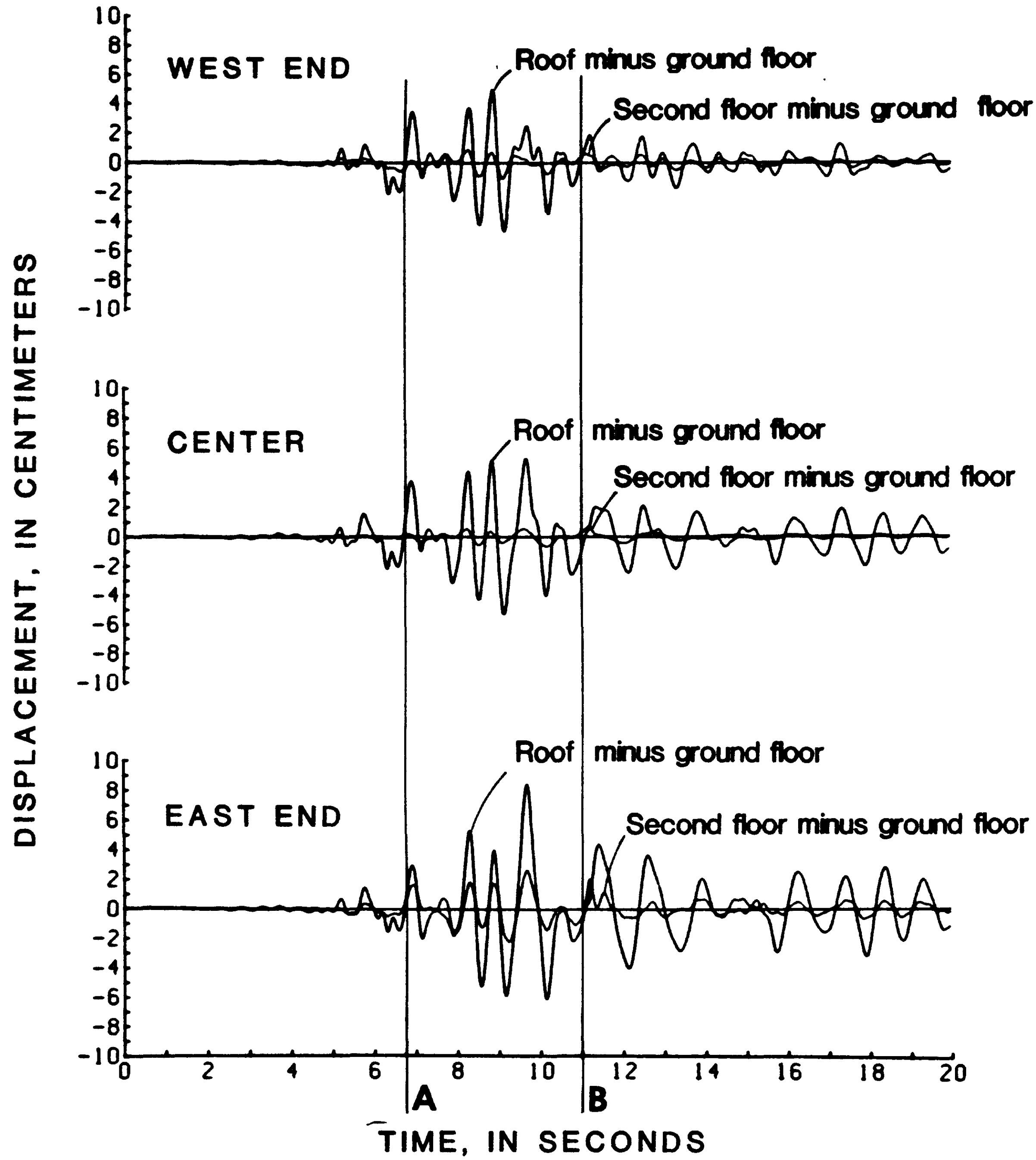

Figure 16.--Time histories of north-south relative displacements between roof and ground floor and second and ground floor at west end of building (difference of data from accelerometers 1 and 10, and 7 and 10), center (difference of data from accelerometers 2 and 11, and 8 and 11), and east end of building (difference of data from accelerometers 3 and 11, and 9 and 11). Times at which damage initiation occurred and column collapse began are labeled A and B, respectively.

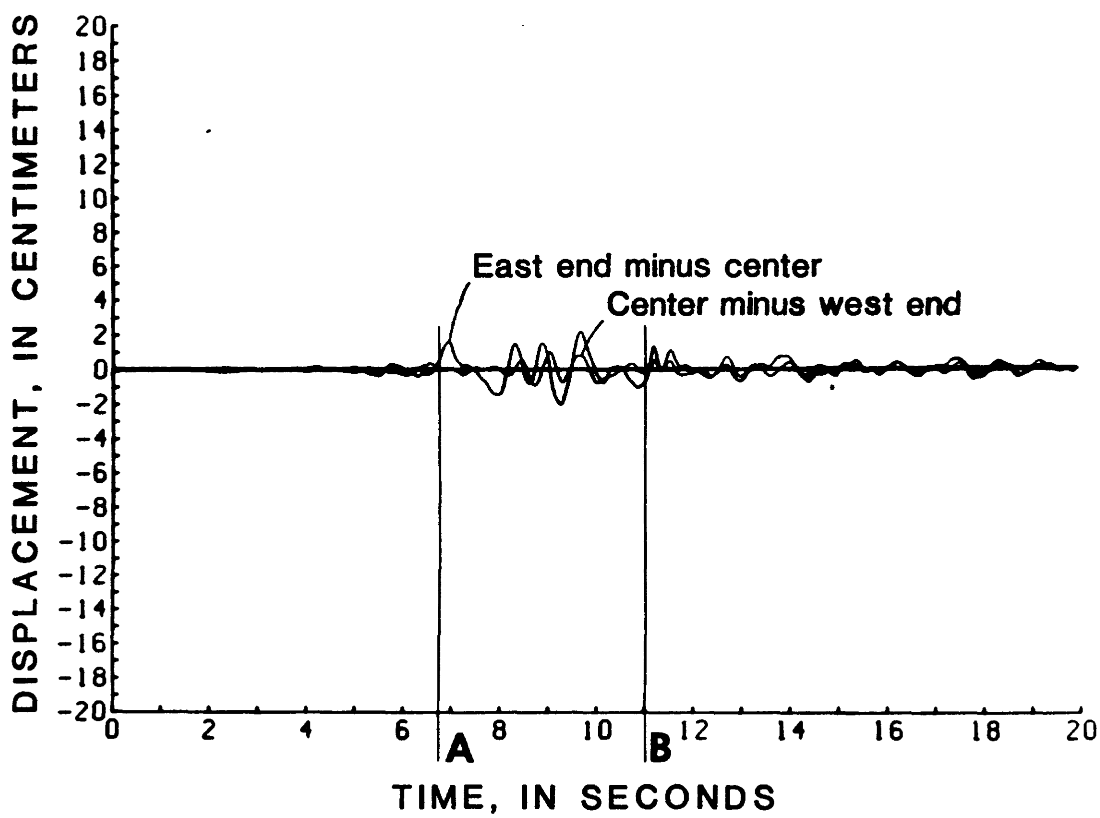

Figure 17.--Time histories of north-south relative displacements between center and east end of second-floor diaphragm (difference of data from accelerometers 8 and 9) and center and west end of second-floor diaphragm (difference of data from accelerometers 7 and 8). Times at which damage initiation occurred and column collapse began are labeled A and B, respectively.

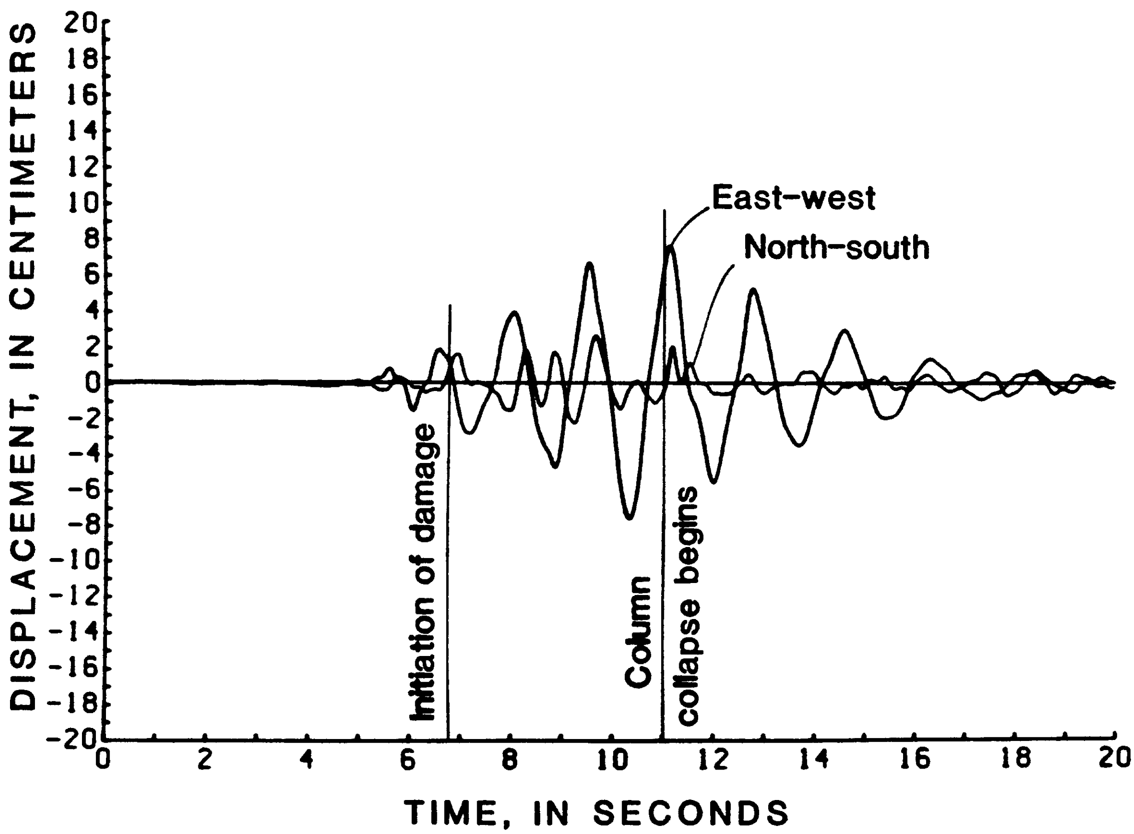

Figure 18.--Time history of east-west and north-south relative displacements between ground and second floor (difference of data from accelerometers 6 and 13, and 9 and 11), showing times of damage initiation and beginning of column collapse.