STRONG-MOTION RECORDS FROM BUILDINGS DAMAGED IN EARTHQUAKES

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ABSTRACT

Strong-motion recordings have been obtained in several buildings damaged by earthquakes. For post-earthquake inspection of buildings, strong-motion records can provide important information on the integrity of the building structure. This paper examines records from two concrete and four steel buildings damaged in earthquakes. The characteristics of the building response that may be indicative of the structural damage are identified from the records. For these buildings, the fundamental period, the maximum drift between the roof and the base, and the maximum base shear are all higher than the corresponding design values. In addition, high-frequency spikes and highly nonlinear response can be seen in some of the records.

INTRODUCTION

Strong-motion records have been obtained from many buildings in California. Prior to the 1994 Northridge earthquake there were only a few records obtained in damaged buildings. The Northridge earthquake has increased the number substantially. In over 150 instrumented buildings throughout the Los Angeles area the recorded roof level accelerations exceeded 0.25 g during the Northridge earthquake. Some of these buildings were extensively instrumented by the California Strong Motion Instrumentation Program (CSMIP) and others have minimal instruments as required by the building code (Shakal et al., 1994 and 1996). Many of the buildings experienced high levels of structural response and some suffered structural damage. For most concrete buildings, the structural damage is visible and strong-motion records are not needed to determine that the building suffered damage. On the other hand, damage to steel buildings is mostly hidden unless the structure is out of plumb or has collapsed. Inspection of the steel members and connections requires removal of fireproofing material and non-structural elements. The cost can therefore be very high if many of the connections need to be inspected. In such cases, strong-motion records may be useful in providing information on whether the structure was damaged and where to start the inspection.

Table 1 lists 15 instrumented buildings that sustained structural damage in earthquakes. These buildings include 3 concrete and 12 steel buildings. All of the buildings were damaged in the Northridge earthquake except the Imperial County Services Building which was damaged in the 1979 Imperial Valley earthquake. The first

two concrete buildings were extensively instrumented by CSMIP and the other 13 were lightly instrumented by the owners as required by the building code. Only three of the 13 code buildings have both the base and the roof records from the Northridge earthquake. The list of the buildings in Table 1 is based on what the authors know at this time from either available publications or personal communication, and it is by no means complete. The number of buildings will certainly increase as more results of post-Northridge inspections of steel buildings become available.

Table 1. Summary of Recorded Data from Damaged Buildings

Building	No. of Stories	Building Direction 1				Building Direction 2			
		Base Accel (g)	Roof Accel (g)	Roof Drift (cm)*	T_1 (sec)**	Base Accel (g)	Roof Accel (g)	Roof Drift (cm)*	T_1 (sec)**
Concrete Buildings									
Imperial County Services(1979)	6	0.33	0.46	18	2.0	0.34	0.58	8	1.2
Van Nuys - Hotel	7	0.47	0.59	23	2.0	0.39	0.56	23	2.0
Northridge - Roscoe #1	7	0.42	0.58	24	1.4	0.39	0.39	30	1.3
Steel Buildings									
Woodland Hills - Oxnard #4	13	0.43	0.33	33	3	0.32	0.24	27	2.5
Woodland Hills - Oxnard #5	11	-	0.33	nd+		-	nr++		
Woodland Hills - Canoga #1	17	-	0.39	43	4.5	-	0.23	17	4
Woodland Hills - Canoga #2	17	-	0.49	45	4.6	-	0.26	16	4
Los Angeles - Olympic #1	9	-	0.69	15	~2	-	0.51	32	2
Los Angeles - Olympic #2	11	-	1.07	27	1.6	-	0.67	32	1.6
Los Angeles - Olympic #4	12	-	0.55	39	1.8	-	0.38	16	2.2
Los Angeles - Wilshire #7	14	-	0.29	28	2.6	-	0.34	17	2.4
N. Hollywood- Lankershim #1	7	-	0.33	20	2.2	-	0.30	29	2.2
Sherman Oaks - Ventura #7	22	-	0.46	34	4.5	-	0.34	29	4.5
Encino - Ventura #1	17	0.54	nr			0.54	nr		
Encino - Ventura #12	20	0.41	>0.6	nd	2.5	0.46	>0.5	nd	2.2

* - maximum roof drift is estimated from the roof record for the building without the base record.
** - T_1, largest value of the fundamental period obtained from the record
+ - nd, record has not been digitized
++ - nr, no roof record

To study the feasibility of identifying structural damage from strong-motion records, two concrete and four steel buildings from Table 1 are discussed herein. Characteristics of the building response, such as building period, base shear, drift, damping, high-frequency spikes or abnormal response that may be indicative of structural damage, are extracted from the records for these six buildings. The Imperial County Services Building has been studied before and the other five buildings were studied in

the SAC project (SAC, 1995) and the Seismic Safety Commission (SSC) project (SSC, 1996). The processed data for these buildings are available from CSMIP (Darragh et al., 1994 and 1995).

CONCRETE BUILDINGS

El Centro - Imperial County Services Building

The Imperial County Services Building, a 6-story concrete frame and shear wall building, suffered significant structural damage during the 1979 Imperial Valley earthquake (Figure 1). The building was demolished after the earthquake and replaced with a 2-story steel frame building. At the time of the earthquake, the building was instrumented by CSMIP with 13 sensors in the building and three sensors at a reference free-field site. This is the first case of an instrumented building sustaining significant structural damage.

Figure 2 shows the locations of the sensors in the building (Rojahn and Ragsdale, 1980). The earthquake resistance of the building was provided by shear walls in the transverse (N-S) direction and by moment-resistant frames in the longitudinal (E-W) direction. There were four interior shear walls below the second floor while the only shear walls above the second floor were the exterior walls. During the 1979 earthquake, all the beams and columns on the first story were damaged with the most severe failure occurring in the four columns at the east end of the building, as shown in Figure 1. More detailed description of the damage is given in the ATC-9 report (ATC, 1984)

The strong-motion records from this building were studied by numerous investigators (e.g., Housner and Jennings, 1982; Rojahn and Mork, 1982; Kreger and Sozen, 1989; Mau and Revadigar, 1994). The acceleration records in the longitudinal direction are shown in Figure 3. The records show that the response between 5 and 7 seconds is dominated by the second mode with a period of about 0.3 seconds. Between 7 and 11 seconds, the building responded with a period of about 1.6 seconds. In addition, high-frequency spikes appear at near 11 seconds and can be seen in all the upper floor records, in both directions. The time near 11 seconds has been interpreted as the time when the columns failed and the building dropped (Housner and Jennings, 1982; Rojahn and Mork, 1982). After the column failure, the building period was lengthened further to nearly 2 seconds. The building period from pre-1979 ambient measurements was 0.65 seconds (Rojahn and Mork, 1982).

The base shear force, estimated from the acceleration records, has a maximum value of about 0.24W for the east-west direction, which is about 4 times the design base shear (0.06W). The absolute displacements, obtained from the accelerations in Figure 3, are shown in Figure 4. These displacements are dominated by the ground motion with periods longer than 3 seconds. The building response is indicated by the relative displacements between the upper floors and the ground floor, which have a period of 1.6 to 2 seconds. The drift can be computed by differencing the displacements at various floors. The maximum drift is about 18 cm, 0.7% of the building height, between the roof

Fig. 1. Line of columns that failed at east end of the Imperial County Services Building during the 1979 Imperial Valley earthquake.
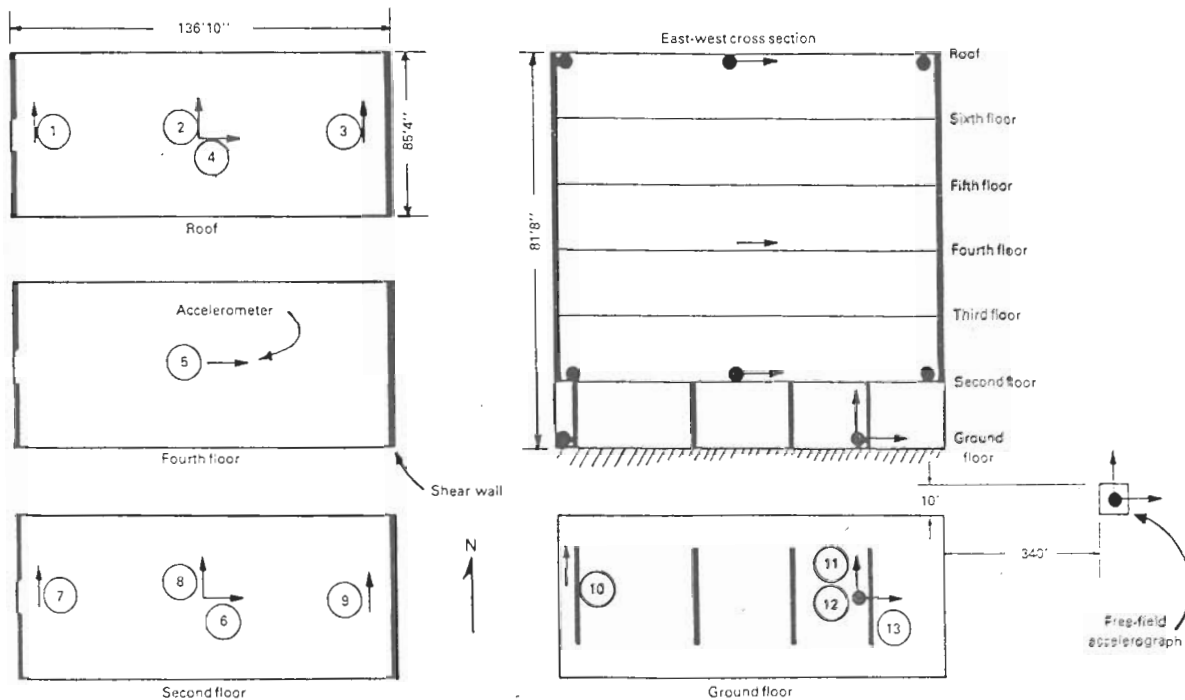

Fig. 2. Locations of accelerometers in Imperial County Services Building and reference free-field site during the 1979 Imperial Valley earthquake (Rojahn and Ragsdale, 1980).
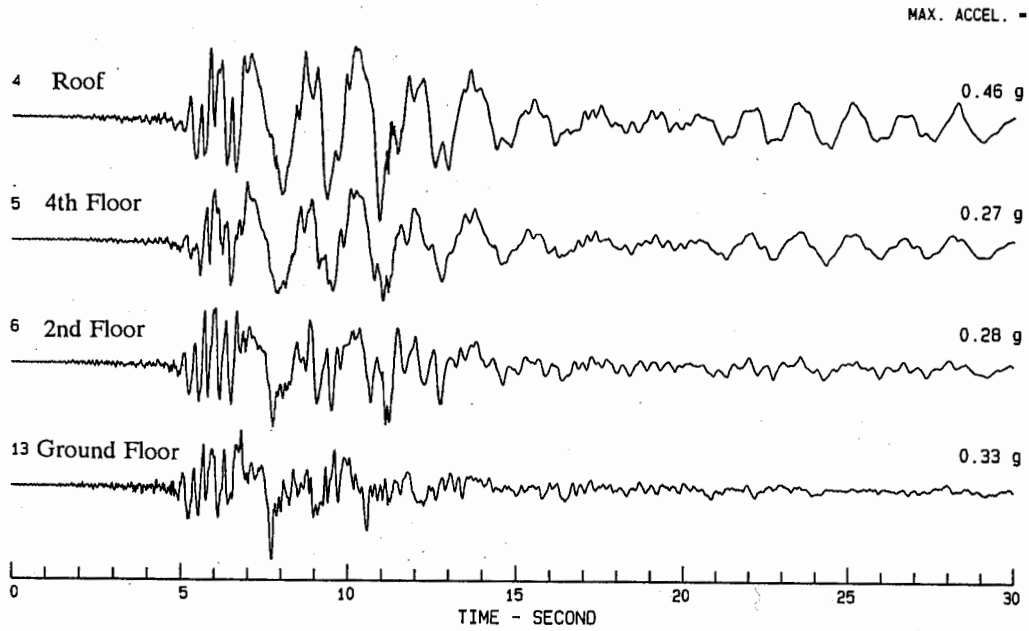
MAX. ACCEL. =

4 Roof   0.46 g

5 4th Floor   0.27 g

6 2nd Floor   0.28 g

13 Ground Floor   0.33 g

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0        5        10       15       20       25       30
              TIME - SECOND
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Fig. 3. Accelerations in the longitudinal (E-W) direction recorded at the Imperial County
         Services Building during the 1979 Imperial Valley earthquake. Failure of columns is
         inferred to have occurred at about 11 seconds.

MAX. DISPL. =

4 Roof   33.4 cm

5 4th Floor   29.5 cm

6 2nd Floor   28.4 cm

13 Ground Floor   27.4 cm

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0        5        10       15       20       25       30
              TIME - SECOND
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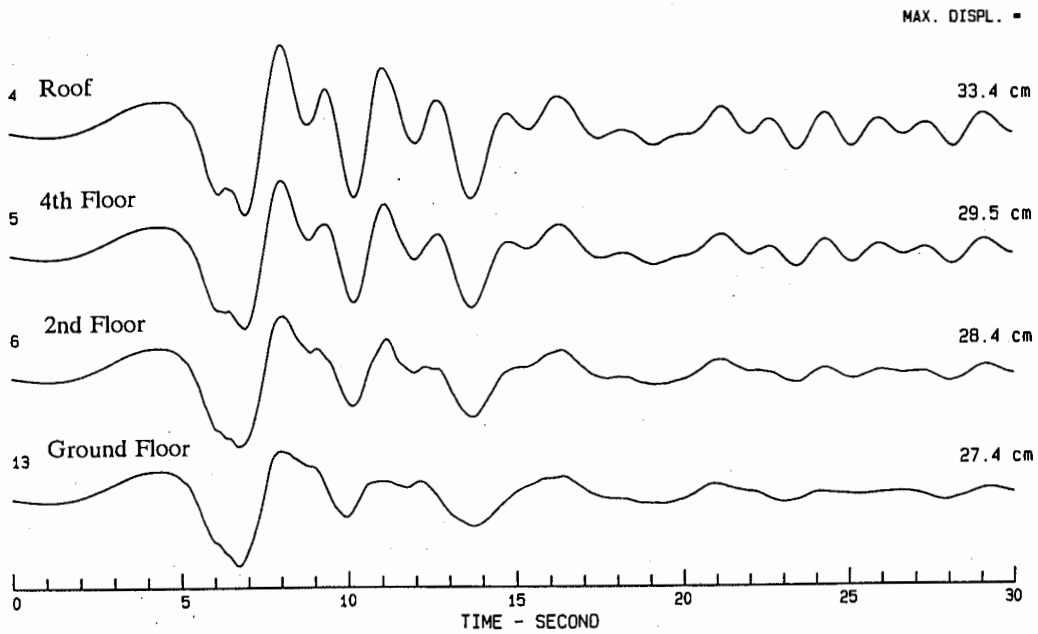
Fig. 4. Displacements corresponding to the accelerations in Fig. 3.

and the ground floor, and is about 8 cm, 1.8% of the story height, between the second and the ground floors. These drifts are larger than the 0.5% code limit

In summary, the records in the longitudinal direction indicate that: (a) the maximum base shear was 4 times the design base shear; (b) the building period was lengthened during the earthquake shaking, up to 3 times the ambient period; (c) large drift occurred in the first story; and (d) high-frequency spikes can be seen in the records. These features imply that the building response was highly nonlinear, and in the absence of inspection or visible damage they may suggest that structural damage might have occurred.

Van Nuys - 7-story Hotel

A hotel in Van Nuys, a 7-story concrete frame structure, suffered structural damage during the 1994 Northridge earthquake (Figure 5). The building also suffered minor structural damage in the 1971 San Fernando earthquake. It is the closest instrumented building to the epicenter of the 1971 earthquake and was later instrumented more extensively with 16 sensors by CSMIP. Several distant earthquakes including the 1987 Whittier, the 1992 Landers and the 1992 Big Bear earthquakes were recorded at this building. The recorded peak accelerations in the longitudinal direction from these earthquakes are listed in Table 2. The Northridge shaking is the strongest and had the largest drift, as shown in the table. After the Northridge earthquake the building was repaired and strengthened with new concrete shear walls.

Table 2. Summary of Recorded Accelerations, Drifts and Fundamental Periods from Several Earthquakes for the Van Nuys 7-story Hotel (Longitudinal Direction)

	Max. Base Acceleration (g)	Max. Roof Acceleration (g)	Max. Drift Roof-Base (cm)	Fundamental Period (second)
Pre-1971 Ambient Measurement[+]	-	-	-	0.52
1971 San Fernando (M6.5, d[*]=20 km)	0.14	0.32	7.8	1.3
Post-1971 Ambient Measurement[+]	-	-	-	0.70
1987 Whittier (M6.1, d=41 km)	0.14	0.17	2.8	1.1
1992 Landers (M7.5, d=187 km)	0.04	0.13	3.2	1.2
1992 Big Bear (M6.6, d=152 km)	0.02	0.06	1.6	1.2
1994 Northridge (M6.7, d=7 km)	0.45	0.58	23.0	1.5 - 2.0

+   from Cloud and Hudson (1975)
*   d - distance from the epicenter

The sensor locations and the structural system for the building are shown in Figure 6. The earthquake resistance of the building in each direction was provided by perimeter column-spandrel beam frames and interior column-slab frames. The structural damage during the Northridge earthquake was mainly in the longitudinal perimeter frames. As shown in Figure 5, the most severe damage occurred in the columns just below the fifth floor spandrel beam on the south side of the building. More detailed description of the damage is given in the Earthquake Engineering Research Institute (EERI) Reconnaissance Report (EERI, 1995) and in the Seismic Safety Commission Case Studies Report (SSC, 1996).

The Northridge records from this building have been used to study analytical techniques by Islam (1996) in the SSC Case Studies project, and by De la Llera and Chopra (1995) in a CSMIP-funded study. The records indicate that the building experienced significant torsional response. Figure 7 shows the accelerations recorded at the roof, the 6th, the 3rd, and the ground floor levels in the longitudinal (E-W) direction. It can be seen from these records that the second mode response with a period of about 0.4 seconds is significant and determined the peak accelerations for all floors. The high-frequency spikes seen in the Imperial County Services building records (Fig. 3) do not appear in these records. This may be due to the fact that the building did not drop vertically and the sensors were not close to the locations of concrete spalling below the fifth floor.

The fundamental mode response is more apparent in the displacement records, as shown in Figure 8. The fundamental period of the building was lengthened from about 1.5 seconds in the first 10 seconds of the record to about 2 seconds later. The inter-story drift ratio is 1% for the first story and 1.9% for the second story. The maximum drift between the roof and the ground floor is about 23 cm, 1.2% of building height, which occurred at 9.34 seconds. It has been interpreted by Islam (1996) that many structural elements may have yielded at approximately 4 to 5 seconds and the most severe damage to the columns may have occurred at about 9 seconds, which corresponds to the time of maximum acceleration. After the columns were damaged, the building period was lengthened to nearly 2 seconds. The maximum base shear from the first mode is about 0.20W which is about 5 times the design base shear (0.04W, according to 1967 UBC).

The building is quite unique in that various levels of structural response, ranging from ambient vibration to strong earthquake response, have been recorded, as summarized in Table 2. One of the important results from studies of these data is the lengthening of the building period during earthquakes relative to the ambient values. For the longitudinal direction, the period was lengthened to three times the ambient period in the San Fernando earthquake, and to four times in the Northridge earthquake.

In summary, the Northridge records in the longitudinal direction indicate that: (a) the maximum base shear was 5 times the design base shear; (b) the building period was lengthened during the earthquake shaking, from 3 times to 4 times the ambient period; and (c) large story drifts occurred.

Fig. 5. Partial south elevation of Van Nuys 7-story Hotel showing damage to columns (indicated by an arrow) below fifth floor spandrel beams during the 1994 Northridge earthquake.
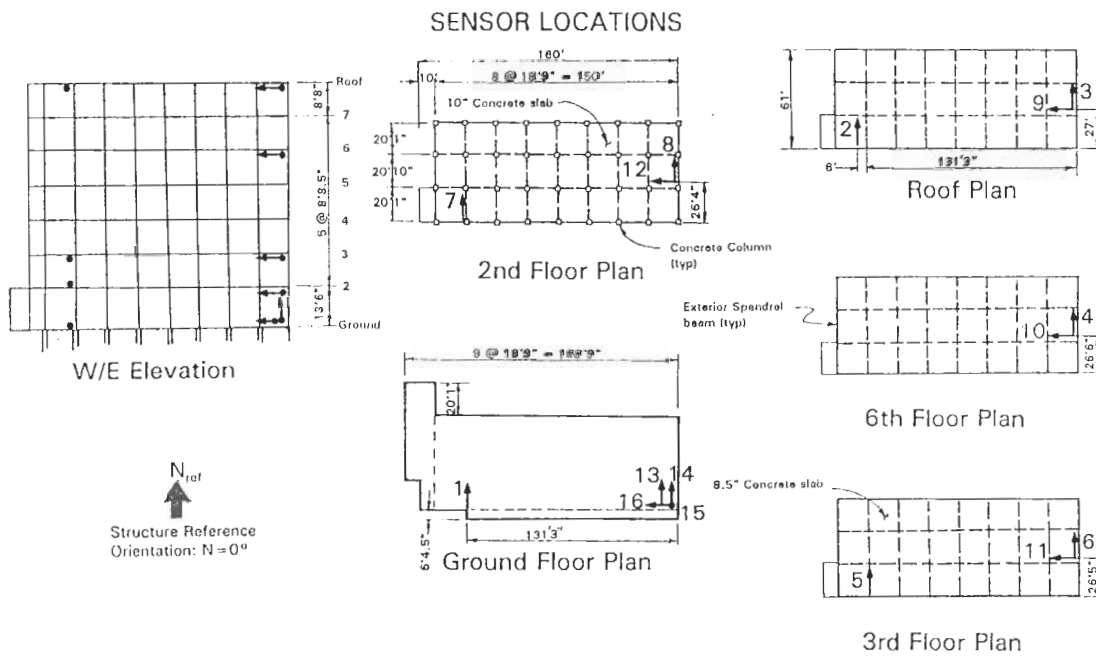

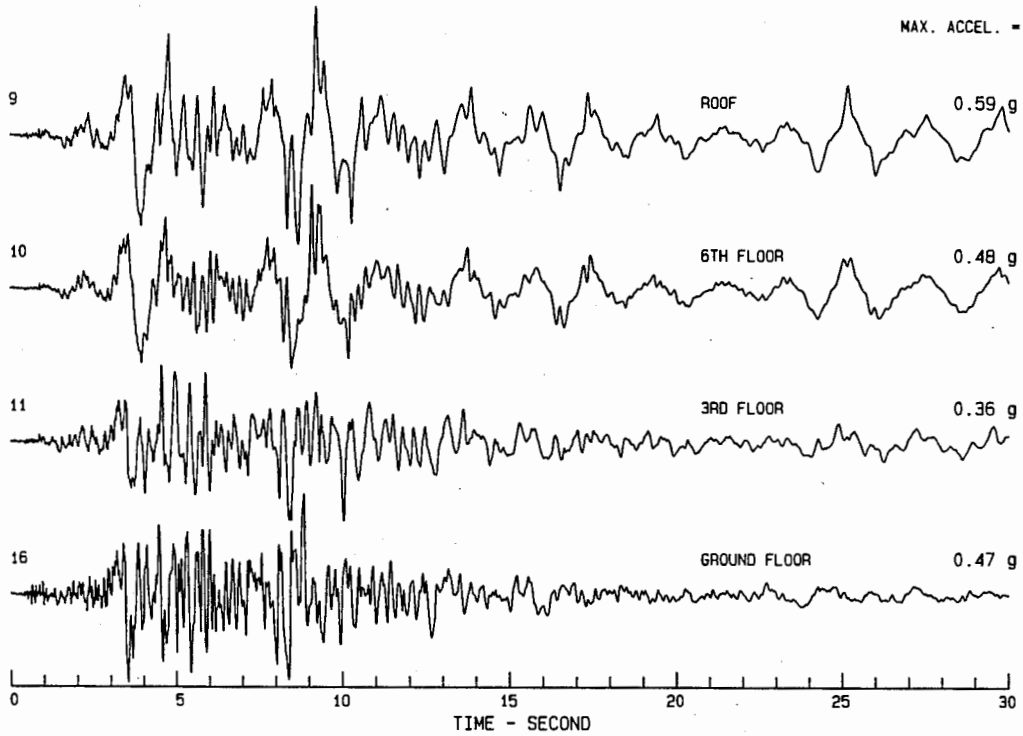Fig. 6. Locations of accelerometers in Van Nuys 7-story Hotel.

Fig. 7. Accelerations in the longitudinal (E-W) direction recorded at the Van Nuys 7-story Hotel during the 1994 Northridge earthquake. Failure of columns occurred between the 4th and 5th floors.
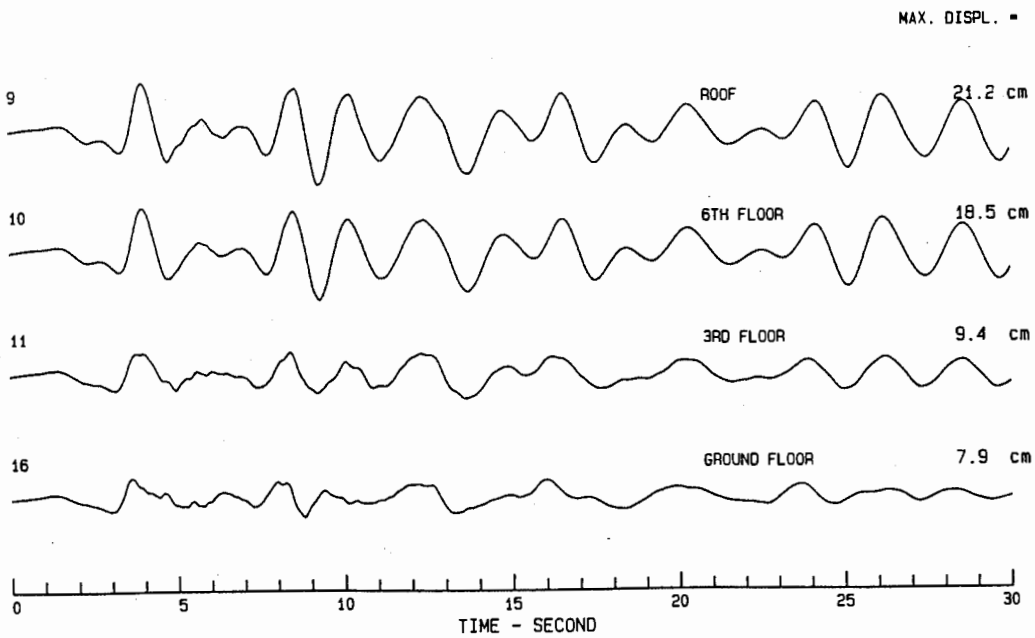

Fig. 8. Displacements corresponding to the accelerations in Fig. 7.

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STEEL BUILDINGS

Woodland Hills - Oxnard #4 Building

The Oxnard #4 Building in Woodland Hills, built in 1976, is a 13-story office building. The earthquake resistance of the building is provided by steel moment frames on the perimeter. The building is 160' square in plan. After the Northridge earthquake, 54 of the 551 inspected joints were found to be damaged (Uang et al., 1995a). The west side frame that provided the resistance in the reference north-south direction has more fractured joints than other three frames. Detailed locations of the fractured joints are given in the report by Uang et al. (1995b).

The building was instrumented with three accelerographs by the owner as required by the Los Angeles Building Code. They were located at the basement, the 6th floor and the roof level. Figure 9 shows the acceleration records in the reference N-S direction. The building fundamental period can not be easily determined by a visual inspection of the acceleration records due to prominent higher mode response in the roof record. Only the first 22 seconds of the roof record could be digitized in the initial digitization. The corresponding displacement records are shown in Figure 10. The building's largest response, with a period of about 3 seconds, occurred between 4 and 8 seconds. The ground shaking was so energetic in the beginning that the structural members apparently yielded right at the beginning. From the computer modelling of the structure, Uang et al. (1995) concluded that a significant number of panel zones at the joints yielded and that the panel zones were a major source of energy dissipation during the Northridge earthquake. The total drift between the roof and the basement, computed from the displacement records, has a maximum value of 33 cm in the reference N-S direction, which is about 0.6% of building height. Design of this steel building was probably controlled by the drift limit, 0.5%, rather than by the force. The maximum base shear is about 0.12W which is about 4 times the code-specified force (0.03W).

Woodland Hills - Canoga #1 and #2 Buildings

The Canoga #1 (West Tower) and Canoga #2 (East Tower) buildings in Woodland Hills are two identical 17-story office buildings. They were designed in 1984. The lateral force resistance is provided by four two-bay steel moment frames. Three moment frames are located on the building perimeter and the fourth moment frame is located one bay from the north face of the building. Although the two towers have identical structural systems, the damage to the East Tower was more severe than the West Tower (Anderson, personal communication). For the East Tower, all fractures occurred at the connections in the N-S frames, with 23 occurring in the frame on the west face and 6 in the frame on the east face (Anderson and Filippou, 1995). The majority of the cracked connections were located between the 12th and the 15th floor levels. Yielding of the beam flanges was also found near the connections at the 13th floor level. A check for vertical plumb of the elevator shaft revealed that the building had a permanent deformation of approximately six inches to the North at the roof level.
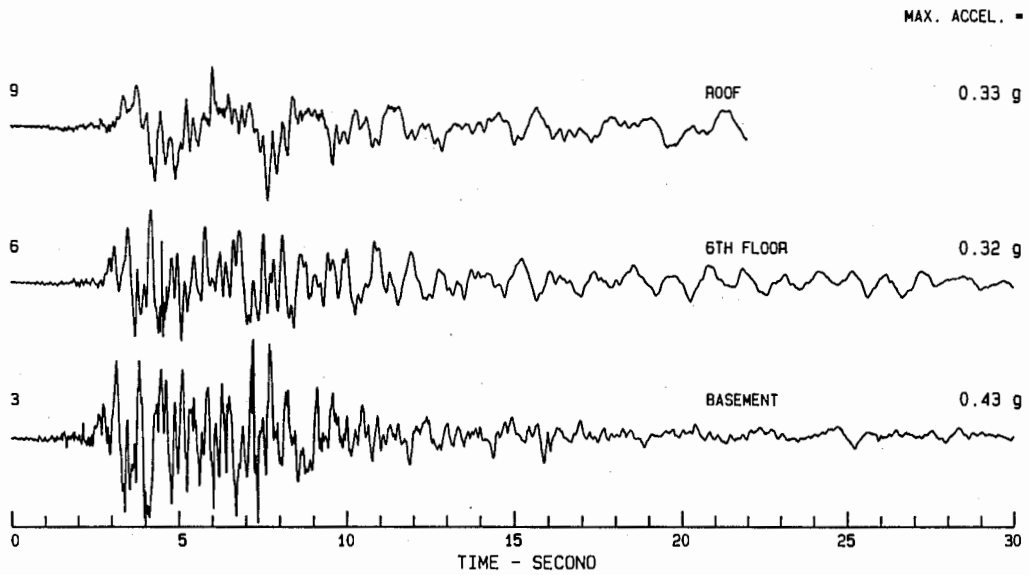
Fig. 9. Accelerations in the reference north-south direction recorded at the Woodland Hills - Oxnard #4 Building during the 1994 Northridge earthquake. After the earthquake, about 10% of the connections in this 13-story steel frame building were found damaged.
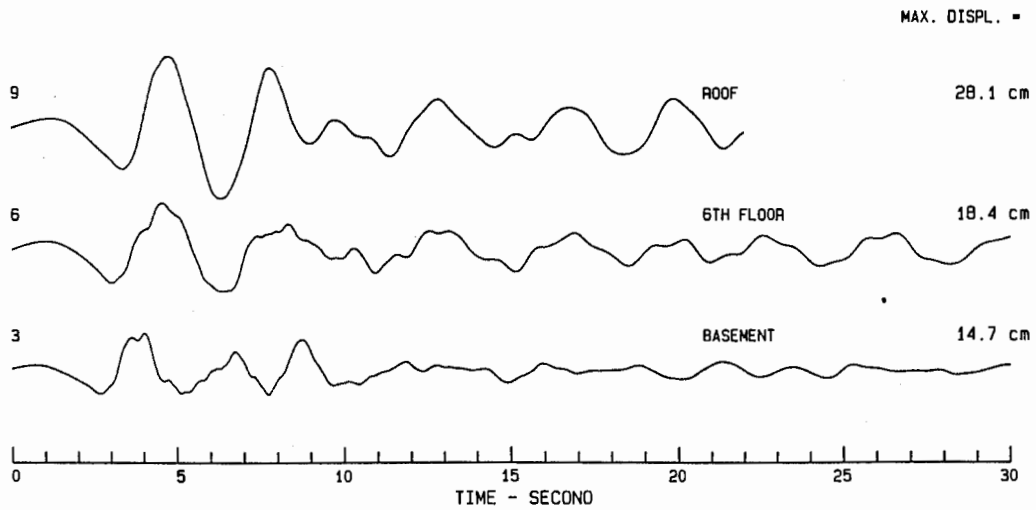

Fig. 10. Displacements corresponding to the accelerations in Fig. 9.

Detailed description of the damage is given in the EERI Reconnaissance Report (EERI, 1995) and the SAC report (Anderson and Filippou, 1995).

Both buildings were instrumented by the owner with one accelerograph at the roof level by the owner as required by the Los Angeles Building Code. The roof acceleration records for the two buildings are shown in Figure 11. In general, the response of the buildings are quite similar with much stronger response in the N-S direction. The peak acceleration near 9.5 seconds is slightly larger for the East Tower than the West Tower. The building is a long-period structure, so the acceleration records were dominated by the higher mode response. In addition, many high-frequency spikes can be seen in the records. The roof displacements for the first 30 seconds are shown in Figure 12, which indicate that the building period was about 4.6 seconds in the N-S direction and about 4 seconds in the E-W direction. By using the basement record of the Oxnard #4 building, which is across the street from this building, one can estimate the maximum drift between the roof and the base. The maximum drift for the East Tower in the N-S direction is about 47 cm, 0.7% of building height, and about 2.5 times that in the E-W direction. The base shear from the first mode, estimated from the roof record, is about 0.05W. The contribution of higher modes to the base shear is significant as compared to the first mode. The design base shear is about 0.05W. The damping ratio for the N-S direction, estimated from the 60-second long displacement records, is about 4%.

Los Angeles - Olympic #2 Building

The Olympic #2 Building in west Los Angeles, built in 1982, is an 11-story building. The building consists of six levels of office space over five levels of parking space. The building has vertical setbacks and re-entrant corners. The earthquake resistance of the building is provided by four steel moment-resisting frames in each direction. Post-earthquake inspection revealed that 258 of the 913 inspected connections suffered varying degrees of damage and the tenants were evacuated (Naeim et al., 1995a). Detailed description of the damage are presented in the SAC report (Naeim et al., 1995b).

The building was instrumented by the owner with one accelerograph on the roof level as required by the Los Angles City Building Code. The roof acceleration records and the corresponding displacements are shown in Figure 13. As seen in these records, between 5 and 10 seconds the building responded predominantly in the second mode with a period of about 0.6 seconds. High-frequency spikes appear near 10.5 and 11 seconds and can be seen on the acceleration records in both directions. This corresponds to the time when the maximum displacement occurred. This time may be interpreted as the time when damage of the structural members occurred. After the damage, the building responded mainly in the fundamental mode with a period of about 1.6 seconds, although the second mode response can be seen in the acceleration records. The response in the reference north-south direction was larger than that in the reference east-west direction and does not have a distinct period of vibration. This may imply that the damage in the north-south direction is more severe than that in the east-west
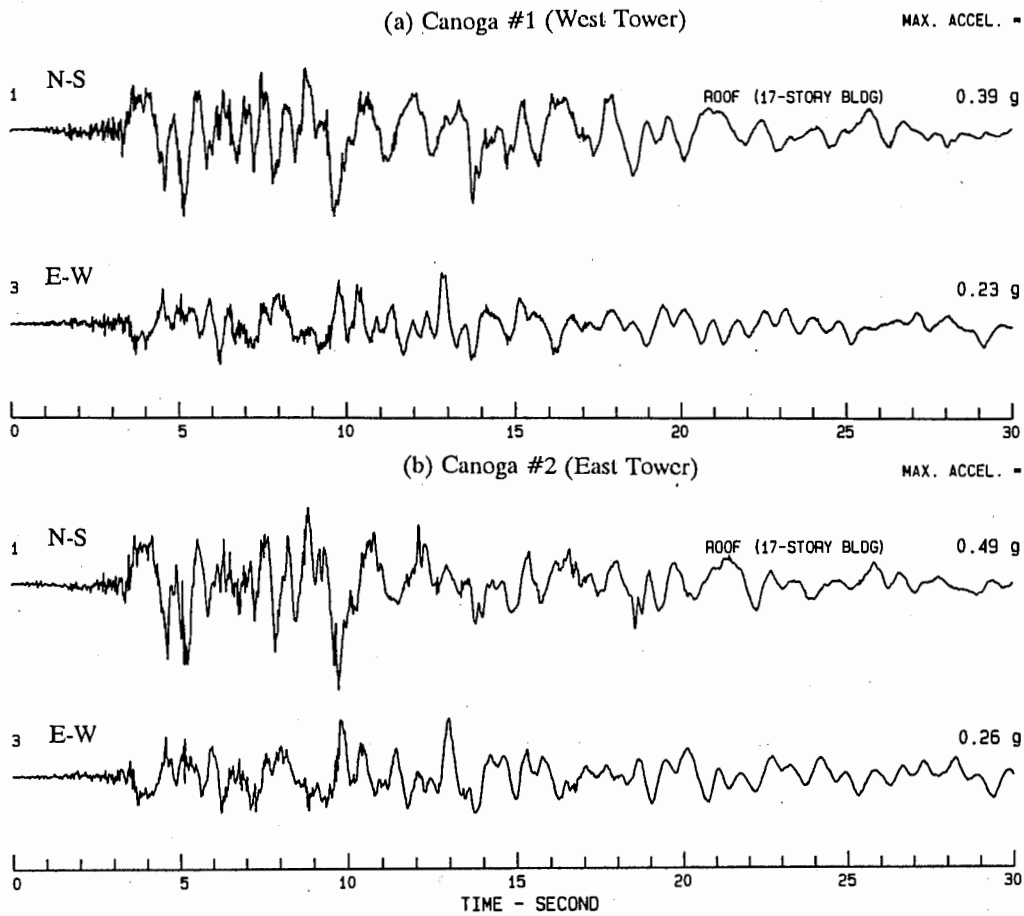
Fig. 11. Horizontal accelerations recorded at the roof level of the Woodland Hills - Canoga #1 (West Tower) and Canoga #2 (East Tower) during the 1994 Northridge earthquake. The East Tower was more severely damaged than the West Tower.
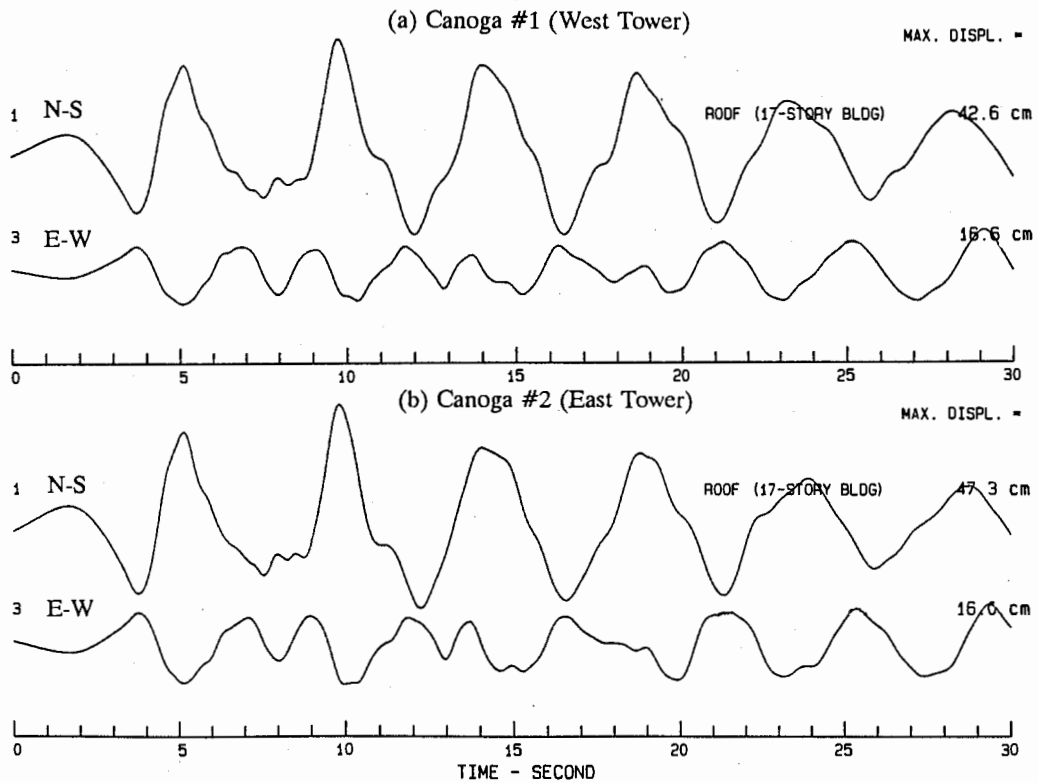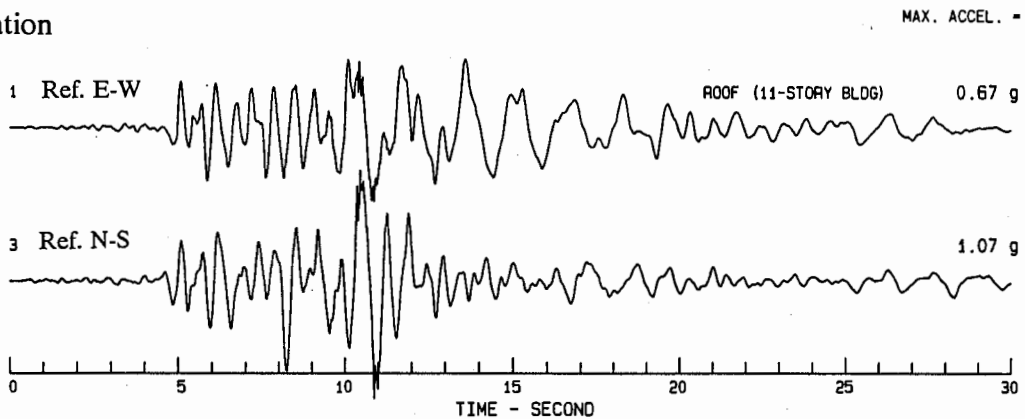

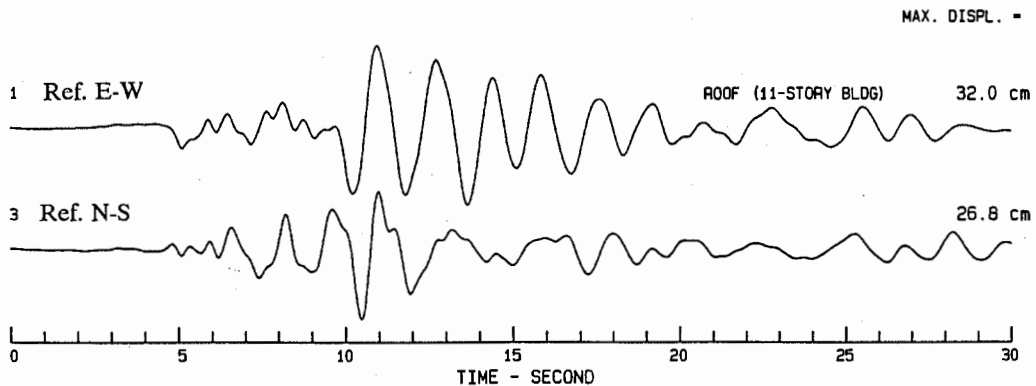Fig. 12. Displacements corresponding to the accelerations in Fig. 11.

direction, although this has not been confirmed from the inspection. The lack of base records prevents computation of the total roof drift, but the maximum roof drift is estimated to be about 32 cm, 0.8% of building height, in the east-west direction. The maximum base shear from the first mode is about 0.20-0.25W, as compared with the design base shear of about 0.06W.

In summary, the Northridge records from the above four damaged steel buildings indicate that: (a) the period for two perimeter moment frame buildings is longer than the period given by 0.2N, N being number of stories; (b) higher mode response is dominant in all acceleration records; (c) the maximum base shear from the first mode is about 1 to 4 times the design base shear; (d) the total roof drift was 20% to 60% larger than the design drift limit, but the story drift due to higher modes can not be determined directly from the records, and (e) high-frequency spikes are seen in some of the records.

Acceleration

MAX. ACCEL. =

1 Ref. E-W   ROOF (11-STORY BLDG)   0.67 g

3 Ref. N-S   1.07 g

0   5   10   15   20   25   30
TIME - SECOND

Displacement

MAX. DISPL. =

1 Ref. E-W   ROOF (11-STORY BLDG)   32.0 cm

3 Ref. N-S   26.8 cm

0   5   10   15   20   25   30
TIME - SECOND

Horizontal accelerations and displacements recorded at the roof level of the Olympic #2 Building in west Los Angeles during the 1994 Northridge earthquake. After the Northridge earthquake, over 25% of the connections in this steel-frame building were found damaged.

EVALUATION OF BUILDING RESPONSE

Following a destructive earthquake, strong-motion records from a building provide useful information for post-earthquake evaluation of the building. Analysis of the response of steel frame buildings using data recorded at the base and other floors have been shown to be very useful in defining the zone of the building where earthquake damage is most likely (e.g., Kariotis, 1996). In the SAC steel building project, the damaged steel buildings were analyzed using available data to test the ability of currently available computer modeling techniques in predicting the locations of fractured joints. Computer modeling of the building is usually time-consuming and requires years of experience to build a realistic computer model, and also requires a good understanding of the capability of the program used. In general, the results are highly dependent on the accuracy of modeling the existing structural conditions.

Examination of the building records can yield insight into the nature of the response as well as quantitative information on the parameters of the structural response such as building period, effective damping, drift, base shear and overturning moment. For most buildings, some of these parameters can be estimated by visual inspection of the records and a simple calculation. For some buildings, especially when the response is highly nonlinear, more rigorous analysis can be carried out by using system identification techniques to determine the parameters of a linear or nonlinear model of the structure that best fit the recorded response when the model is subjected to the recorded base motion (e.g., Beck and Jennings, 1980).

Most buildings are designed to remain elastic for code-specified forces and drifts, and some structural members are expected to yield to larger earthquake forces. Therefore one can expect no structural damage in a building if the records show that the building response is linear and the earthquake force and drift are smaller than the design values. The period lengthening during the shaking, larger base shear than the design base shear, or large drift is an indication of structural nonlinearity. Structural damage is very likely to occur when the response is highly nonlinear. A failure analysis of the building can be carried out to determine the maximum story drift that each story can withstand, the maximum base shear that causes the structural members to fail, and the period of vibration at which structural damage occurs. These values can then be compared with the corresponding values derived from the records. A building is expected to be severely damaged if the values derived from the record are larger than the estimated damage values. However, due to the uncertain nature of the actual strength in an existing building such a failure analysis will be approximate.

To determine how nonlinear a building response is, application of the system identification technique to the recorded data using a linear or nonlinear model is probably the most economic and systematic approach. Results can be obtained in a timely manner for post-earthquake evaluation. The identification techniques for linear models have been successfully applied to the San Fernando records (e.g., Beck and Jennings, 1980). Many researchers have applied system identification techniques to the detection of changes in structural parameters with a goal of damage detection (e.g.,

Agbabian, et al., 1991; Mau and Revadigar, 1994; Loh and Tou; 1994). As more building performance information is available, the system techniques can be applied to many Northridge records from damaged as well as undamaged buildings. The results can be correlated with the building performance and some damage indices or criteria may be derived at least statistically.

SUMMARY

The records from two concrete and four steel buildings are reviewed for possible indications of structural damage from these records. Although no single character in the record can be used to indicate the structural damage, each record has some but not all of the following characteristics: (a) period lengthening, (b) large base shear, (c) large roof drift, c) nonlinear response and (d) high-frequency spikes. More rigorous analyses of these records, such as application of system identification techniques, are needed to study damage indices for detecting structural damage from strong-motion records. As the results of post-Northridge evaluation of more damaged and undamaged buildings become available, these indices may be derived statistically.

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