

PERFORMANCE OF 20 EXTENSIVELY-INSTRUMENTED BUILDINGS
DURING THE 1994 NORTHRIDGE EARTHQUAKE

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ABSTRACT

In the aftermath of the January 17, 1994 Northridge earthquake hundreds of strong ground motion and building response accelerograms were retrieved from stations throughout the greater Los Angeles basin. Particularly important among the building response records were the data obtained from instrumented buildings which experienced relatively large ground motions. This paper provides a summary of the results obtained from an elaborate two-year project which included inspection of the buildings, damage assessment, performance evaluations. The forces, displacements, and dynamic characteristics interpreted from recorded data are contrasted with those suggested by building codes. Key response parameters and characteristics of each building are studied and where necessary observations are provided which may be used to improve future editions of the building codes.

INTRODUCTION

According to Nehru every theory must be tempered with reality. The January 17, 1994 Northridge earthquake ($M_L = 6.4$, $M_W = 6.7$, $M_S = 6.8$) provided ample opportunity for earthquake engineers to test their theories and practices of structural design and seismic performance against the realities of strong ground shakings. Hundreds of strong ground motion and building response accelerograms were recorded by and retrieved from instruments installed by California Division of Mines and Geology, Strong Motion Instrumentation Program (Shakal and others, 1994; CSMIP 1994-95), United States Geological Survey (USGS, 1994) and other agencies throughout the greater Los Angeles basin.

Particularly important among the building response records were the data obtained from 17 instrumented buildings distributed throughout the Los Angeles area which experienced peak base accelerations greater than 0.25 g, two instrumented downtown skyscrapers which experienced ground level accelerations of about 0.18g, and a two-story base isolated Fire Command Control building which experienced a peak base acceleration of about 0.22g.

As a part of this investigation, the above buildings were inspected to the extent possible and their performance were evaluated relative to various aspects of recorded ground motion and building configuration. Building superintends and structural engineers who had examined the buildings

were consulted and their observations were summarized. Detailed information on building structural systems, nonstructural systems, contents, construction history, extent and location of damage, and loss estimates were gathered.

For each building the code specified values for natural periods design base shears and drift indices were calculated. Two sets of code values were developed: one corresponding to the edition of the building code used in the actual design of each building, and the other based on the 1994 edition of the Uniform Building Code (ICBO, 1994). These values were compared with natural periods and maximum base shears interpreted from the earthquake records.

A unique feature of this project is development of a CD-ROM based interactive information system which contains all text, photos, sketches, earthquake records and most importantly all of the analytical tools which were developed and utilized for this study (Naeim, 1996). The companion SMIP Information System is a Microsoft Windows based system and is built around an open-architecture relational database system which can be modified and expanded by the users.

DESCRIPTION OF THE BUILDINGS

The following buildings were studied as a part of this investigation. The acronyms used for identification of these buildings in the rest of the paper are given in parenthesis:

Burbank 10-story residential building with 16 sensors (BURBANK 10) This building was designed and constructed in 1974. Its vertical load carrying system consists of precast and poured-in-place concrete floor slabs supported by precast concrete bearing walls. The lateral load resisting system consists of precast concrete shear walls in both direction. The foundation system includes concrete caissons which are 25 to 35 feet deep. The largest peak horizontal accelerations recorded at the base and at the roof are 0.34g and 0.77g, respectively. The peak velocity at the roof is about 63 cm/sec.

Burbank, 6-story commercial building with 13 sensors (BURBANK 6) This steel moment frame building was designed in 1976 and constructed in 1977. The vertical load carrying system consists of 3" concrete slab over metal deck supported by steel frames. The lateral load resisting moment frames are located at the perimeter of the building. The foundation system includes concrete caissons approximately 32 feet deep. The largest peak horizontal acceleration recorded at the base and at the roof are 0.36g and 0.47g, respectively. The peak velocity at the roof is about 48 cm/sec.

Los Angeles, 17-story residential building with 14 sensors (LARES 17) This building was designed in 1980 and constructed in 1982. Its vertical load carrying system consists of 4" or 8" precast, pretensioned concrete slabs supported by precast concrete walls. The lateral load resisting system consists of distributed precast concrete shear walls in both direction. The foundation system includes 44" diameter and 54 feet long drilled piles. The largest peak horizontal acceleration recorded at the base and at the roof are 0.26g and 0.58g, respectively. The peak velocity at the roof is about 48 cm/sec.

Los Angeles, 19-story office building with 15 sensors (LAOFFI 19) This office building has 19 stories above the ground and 4 stories of parking structure below the ground. It was designed in 1966-67 and constructed in 1967. The vertical load carrying system consists of 4.5" reinforced concrete slabs supported on steel frames. The lateral load resisting system consists of moment resisting steel frames in the longitudinal and X-braced steel frames in the transverse direction. The foundation system consists of 72 feet long, driven, steel I-beam piles. The largest peak horizontal acceleration recorded at the base, ground floor and roof are 0.32g, 0.53g and 0.65g, respectively. The peak velocity at the roof is about 65 cm/sec.

Los Angeles, 2-story Fire Command Control building with 16 sensors (LACC 2) This is a 2 story seismic isolated building. The isolation system is composed of elastomeric bearings. The vertical load carrying system is steel vented roof decking and steel decking with 3 to 4 inches of concrete fill at the first and second floors. The floor system is supported by steel frames and rubber bearings. The lateral load resisting system is perimeter chevron braced frames above the isolation interface. The foundation system is composed of spread footings. The building was designed in 1988 and constructed in 1989-90. In the E-W direction, the largest peak horizontal accelerations recorded below the isolation plane, at the floor directly above the isolation plane, and the roof are 0.22g, 0.35g and 0.77g, respectively. In the N-S direction, the largest peak horizontal accelerations vary from 0.18g at the base to 0.07g directly above the isolation system and 0.09g at the roof.

Los Angeles, 3-story commercial building with 15 sensors (LACOMM 3) This department store building has three stories above and two parking levels below the ground. The building was designed in 1974 and constructed in 1975-76. The vertical load carrying system consist of 3.25 inches of light-weight concrete slab over metal deck in upper three floors and 18 inches thick waffle slabs in the basement floors. The lateral load resisting system is steel braced frames in the upper three stories and concrete shear walls in parking floors. The foundation system consists of spread footings and drilled bell caissons. The largest peak horizontal accelerations recorded at the base is 0.33g. At the roof peak horizontal acceleration of 0.97g and peak velocity of 57 cm/sec were recorded.

Los Angeles, 5-story Warehouse with 13 sensors (LAWH 5) This is a 5-story reinforced concrete building was constructed in 1970 with perimeter ductile concrete frames acting as its lateral system. The largest peak horizontal accelerations recorded at the base and at the roof are 0.25g and 0.28g, respectively. The peak velocity at the roof is about 34 cm/sec.

Los Angeles, 52-story office building with 20 sensors (LAOFFI 52) This office building has 52 stories above and 5 levels below the ground. It was designed in 1988 and constructed in 1988-90. The vertical load carrying system consists of 3 to 7 inches of concrete slabs on steel deck supported by steel frames. The lateral force resisting system consists of concentrically braced steel frames at the core with moment resisting connections and outrigger moment frames in both directions. The foundation is composed of spread footings of 9 to 11 feet thickness. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.15g and 0.41g, respectively. The peak velocity at the roof is about 40 cm/sec.

Los Angeles, 54-story office building with 20 sensors (LAOFFI 54) This office building has 54 stories above and 4 levels below the ground. It was designed in 1988 and constructed in 1988-90. The vertical load carrying system consists of 2.5 inches of concrete slabs on a 2inche metal deck supported by steel frames. The lateral force resisting system consists of perimeter tubular moment resisting frames which step in twice in elevation.. The foundation system consists of a 9 feet deep mat foundation. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.14g and 0.19g, respectively. The peak velocity at the roof is about 34 cm/sec.

Los Angeles, 6-story office building with 14 sensors (LAOFFI 6) This office building has five stories above and one level below the ground. It was designed in 1988 and constructed in 1989. The vertical load carrying system consists of composite construction of concrete slabs over metal decks supported by steel frames. The lateral load resisting system is a combination of Chevron braced and moment resisting steel frames. Mat foundations are utilized beneath the four towers and spread footings are used elsewhere. The largest peak horizontal accelerations recorded at the base and at the roof are 0.24g and 0.48g, respectively. The peak velocity at the roof is about 70 cm/sec.

Los Angeles, 6-story parking structure with 14 sensors (LAPARK 6) The first three stories of this concrete parking structure were constructed in 1977. The upper three floors were added in 1979 based on designs dated 1975 and 1978. The vertical load carrying system consists of 5.75 in. concrete slabs and 5 in. post-tensioned concrete slabs supported by precast concrete beams and columns (see the Information System photos). The lateral force resisting system consists of six cast-in-place reinforced concrete shear walls in the North-South direction and two in the E-W direction. The foundation system is made of drilled concrete caissons. The largest peak horizontal acceleration recorded at the base, near the north-east shear is 0.29g. Channel 1 at the base of the North shear wall recorded a peak vertical acceleration of 0.22g. At the roof, the sensor placed on the mid-span of a girder recorded a peak vertical acceleration of 0.52g. Elsewhere on the roof, a sensor attached to a parapet on the North side recorded peak horizontal and vertical accelerations of 1.21g and 0.52g, respectively.

Los Angeles, UCLA Math-Science building with 18 sensors (UCLA7) This Math-Science addition to the engineering school building at UCLA is a 7 story building with no basement. It is separated by seismic joints from adjacent wings of the building. The vertical load carrying system for the upper floors (third and higher) consists of 2.5 inches of concrete slab over metal deck supported by steel frames. At the third floor a thick concrete slab supported by concrete walls make up the gravity system. The lateral load resisting system consists of concrete shear walls between the base and the third floor and moment resisting steel frames from the third floor to the roof. The largest peak horizontal accelerations recorded at the base and roof are 0.29g and 0.76g, respectively. The maximum velocity recorded at the roof is about 73 cm/sec.

Los Angeles, 7-story hospital building in with 24 sensors (LAHOSP 7) This structure is the first base isolated hospital building in the United States. It was designed in 1988 and constructed between 1989 to 1991. The vertical load carrying system consists of concrete slabs on metal decks supported by steel frames and rubber isolators. The lateral force resisting system consists of diagonally braced perimeter steel frames isolated by lead-rubber and elastomeric isolator units.

Foundation system consists of continuous and isolated spread footings.. The largest free-field peak horizontal acceleration recorded adjacent to the building is 0.49g in the N-S direction. The largest horizontal peak acceleration recorded at the foundation, immediately above the isolation plane, and at the roof of the building are 0.37g (N-S), 0.14g (E-W), and 0.21g (N-S).

Los Angeles, 9-story office building with 18 sensors (LAOFFI 9) This office building was designed and constructed in 1923. It consists of concrete frames with unreinforced masonry infill walls. It consists of one level of basement and 9 floors above the ground. The largest peak horizontal accelerations recorded at the basement and roof are 0.18g and 0.34g , respectively. The maximum velocity recorded at the roof is about 45 cm/sec.

Los Angeles, Hollywood Storage Building with 12 sensors (HWSTOR) This building has 14 stories above and one level below the ground. It was designed in 1925. The vertical load carrying system consists of 8 in. thick concrete slabs supported by concrete frames. The lateral load resisting system, consists of reinforced concrete frames in both directions. The deep foundation system consists of concrete piles. The “free-field” station adjacent to the building recorded peak accelerations of 0.41g in the N-S direction, 0.19g in the E-W direction, and 0.19g in the vertical direction. The maximum peak horizontal accelerations recorded at the base and at the roof are 0.28g and 0.49g, respectively. The uncorrected trace of Channel 11 at the roof shows a peak acceleration of 1.61g. However, at the time of publishing this paper the corrected version of this record was not available. It is possible that this sensor was not properly calibrated at the time of the earthquake since it has high frequency content which is not corroborated by other instruments. The peak velocity at the roof is about 38 cm/sec.

North Hollywood, 20-story hotel with 16 sensors (NHHOTEL 20) This hotel has 20 stories above and one level below the ground. It was designed in 1967 and constructed in 1968. The vertical load carrying system consists of 4.5 to 6 inches thick concrete slabs supported by concrete beams and columns. The lateral load resisting system consists of ductile moment resisting concrete frames in the upper stories and concrete shear walls in the basement. The exterior frames in the transverse direction are infilled between the second and the 19th floors. The building rests on spread footings. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.33g and 0.66g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

Sherman Oaks, 13-story commercial building with 15 sensors (SHERMAN 13) This office building has 13 stories above and two floors below the ground. It was designed in 1964. The vertical load carrying system consists of 4.5 inches thick one-way concrete slabs supported by concrete beams, girders and columns. The lateral load resisting system consists of moment resisting concrete frames in the upper stories and concrete shear walls in the basements. The foundation system consists of concrete piles. The first floor spandrel girders were modified by post-tensioning after the 1971 San Fernando earthquake. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.46g and 0.65g, respectively. The middle floors experienced large acceleration in the neighborhood of 0.6g. The largest velocity recorded at the roof is about 68 cm/sec.

Sylmar, 6-story hospital building with 13 sensors (SYLMAR) The Sylmar County Hospital Building is a unique building built on the site of the old Olive View hospital building which suffered major and irreparable damage during the 1979 San Fernando earthquake. This six story cruciform shaped building has no basements. It was designed in 1976 and was constructed during the period of 1977 to 1986. Its vertical load carrying system consists of concrete slabs over metal deck supported by steel frames. The lateral load resisting system consists of concrete shear walls in lower two floors and steel shear walls encased in concrete at the perimeter of the upper four floors. The building rests on spread footings. The "free-field" station located at the parking lot adjacent to the building recorded 0.91g, 0.61g, and 0.60g in the N-S, E-W, and vertical directions, respectively. The largest peak horizontal accelerations recorded at the ground floor and at the roof of the building are unprecedented at 0.80g and 1.71g, respectively. The largest velocity recorded at the roof was as large as 140 cm/sec.

Van Nuys, 7-story hotel with 16 sensors (VAN NUYS 7) This reinforced concrete structure with no basements was designed in 1965 and constructed in 1966. Its vertical load carrying system consists of 8 in. and 10 in. concrete slabs supported by concrete columns, and spandrel beams at the perimeter. The lateral load resisting system consists of interior column-slab frames and exterior column-spandrel beam frames. The foundations consist of 38 inch deep pile caps, supported by groups of two to four poured-in-place 24 inch diameter reinforced concrete friction piles. The largest peak horizontal accelerations recorded at the basement and at the roof are 0.45g and 0.58g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

RESULTS OBTAINED

The 20 instrumented buildings exhibited structural and nonstructural damages ranging from *None* to *High* based on the ATC-38 post-earthquake evaluation procedure. Hundreds of photos exhibiting various types of damage to these buildings and a wide variety of analytical tools developed as a part of this project are available on the CD-ROM based information system which was developed as a part of this investigation (Naeim, 1996). A few sample representatives are reproduced in Figures 1 to 9.

Interpreted maximum direct (N-S or E-W) and differential (torsional) base shears and drift indices are presented in Table 1 where interpreted base shears are compared with recommended code strength design values. Overall levels of structural and nonstructural damage are also indicated on this table.

Interpreted vibration periods of these buildings are compared to code recommended period estimates in Table 2 where predominant periods --if significantly different from fundamental periods-- are identified. This table also shows the shifts in building periods during and after the earthquake.

In light of the results of this investigation the following observations are made:

1. Building code estimates of building periods are consistently less than both the initial and final fundamental periods obtained from interpretation of recorded data. UBC-94 estimates,

however, are much better than the estimates provided by the older editions of the code. It may be necessary to further calibrate code period estimation formulas to reduce this inconsistency.

2. Except for the two base isolated buildings and the two downtown skyscrapers, the building base shears obtained from interpretation of recorded data are larger, sometimes substantially, than the base shears they have been apparently designed for. With the exception of the Van Nuys 7-story hotel, however, these buildings behaved remarkably well given the magnitude of forces they were subjected to. One could suggest that all these buildings performed much better than what would have been expected by routine design analysis techniques. Design procedures need to be modified to take advantage of this excess capacity which is not ordinarily addressed in design analysis schemes.
3. The ratio of the base shears experienced to design code base shears does not correlate very well with the extent of damage observed. The overall drift ratio, however, does correlate rather well. This statement, however, needs further clarification through system identification studies since it is not clear at this time whether the large drifts are contributing to damage or are caused by it.
4. Given the level of forces the building experienced, the overall drift ratios experienced are significantly less than what would have been expected from ordinary design analysis techniques.
5. While structural damage was generally less than what would have been expected, the content damage was generally extensive and usually the dollar value of the content damage and loss of occupancy far out-weighed the cost of structural repair.
6. In seismic response of majority of the buildings, different modes became predominant at different times during the response. In many cases, particularly for taller buildings such as the downtown skyscrapers, the Sherman Oaks 13-story office building, and the North Hollywood 20-story hotel, 2nd and or 3rd modes had more contribution to the overall response than the fundamental mode. In such cases application of the lateral story force profiles as suggested by static lateral force procedures may grossly underestimate the demand on the middle floors of the building. This can be further illustrated by examining the story force diagrams at the time of maximum base shear which indicate that except for the shorter buildings, the story force profile at the instant of maximum base shear is radically different from that recommended by static lateral force procedures. Lateral force distribution over the height of the building as suggested by static lateral force procedures is generally based on the static deflected shape of the building. Evaluation of the deformed shape at the time of maximum lateral displacement shows that the lateral deformation at this instant almost always follows a shape similar to the first mode of vibration. Our studies indicate, however, that in most cases maximum forces and maximum displacements are not concurrent. In most cases the maximum force response occurs first and the maximum displacement response occurs many seconds later. Current edition of the UBC code requires dynamic (i.e., response spectrum) distribution of forces for irregular structures. In light of observations presented

here it might be prudent to require dynamic distribution of forces for buildings exceeding a certain height (65 feet for example) and limiting the application of static lateral force distribution to the regular buildings of less height.

7. Except for the case of the 6-story Sylmar County hospital, behavior of mounted mechanical equipment was not a strong function of the severity of the ground motions but rather the quality of design and construction. (see for example photos of equipment mounted at the roof of the 3-story commercial building or the Van Nuys 7-story hotel in the Information System developed as a part of this project).
8. Except for buildings with observed structural damage, the period of the building as interpreted from the recorded data did not elongate significantly and when elongation occurred the period came back to the vicinity of the initial value towards the end of the ground motion. The period of damaged buildings however did decidedly elongate.
9. For several buildings, torsion contributed significantly to the seismic response. In one of these cases (Van Nuys 7-story hotel) the building experienced major damage.

CONCLUSIONS

An interactive information system was developed which contains all of the gathered information, inspection results, recorded data, performance analysis results, and analytical tools utilized for evaluation of twenty extensively-instrumented buildings which were subjected to significant ground shaking during the January 17, 1994 Northridge earthquake (see Naeim, 1996). This CD-ROM based interactive information system can be a very valuable tool in teaching and learning earthquake engineering and seismic response principles as well as a tool for further research on response of buildings to strong earthquake ground motions. A brief overview of the information generated regarding the seismic performance of these buildings were presented. The interested reader is referred to Naeim (1996) report to California Strong Motion Instrumentation Program and its companion CD-ROM based information system for more details.

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TABLE 1. Base Shear, Drift and Overall Damage Summary

Building Acronym	Maximum Direct Base Shear Interpreted (% Total Weight) ^a	Maximum Differential Base Shear Interpreted (% Total Weight) ^a	Design Code Strength Level Base Shear (% Total Weight) ^b	UBC-94 Strength Level Base Shear (% Total Weight) ^c	Maximum Overall Drift Index Interpreted (in./in.) ^a	Overall Structural Damage ^d	Overall Non-structural and/or Equipment Damage ^d
BURBANK 10	34	14	14	20	0.0023	Insignificant	Insignificant
BURBANK 6	22	7	10	7	0.0039	Insignificant	Moderate
LARES 17	17	16	18	18	0.0022	Insignificant	Insignificant
LAOFFI 19	34	22	8	12	0.0039	Moderate	Insignificant
LACC 2	7	11	-- ^e	-- ^e	--	None	Moderate
LACOMM 3	49	27	18	27	0.0111	None	Insignificant
LAWH 5	17	10	8	17	0.0020	Insignificant	Insignificant
LAOFFI 52	9	4	-- ^e	-- ^e	0.0011	None	None
LAOFFI 54	4	4	-- ^e	-- ^e	0.0008	None	None
LAOFFI 6	-- ^f	-- ^f	-- ^f	-- ^f	--	None	None
LAPARK 6	27	13	12.5	18	0.0016	Insignificant	Insignificant
UCLA 7	27	21	-- ^f	-- ^f	0.0047	Moderate	Insignificant
LAHOSP 7	-- ^g	-- ^g	-- ^e	-- ^e	--	None	None

TABLE 1. (Continued)

Building Acronym	Maximum Direct Base Shear Interpreted (% Total Weight) ¹	Maximum Differential Base Shear Interpreted (% Total Weight) ¹	Design Code Strength Level Base Shear (% Total Weight) ²	UBC-94 Strength Level Base Shear (% Total Weight) ³	Maximum Overall Drift Index Interpreted (in./in.) ⁴	Overall Structural Damage	Overall Non-structural and/or Equipment Damage
LAOFFI 9	20	18	-- ^f	-- ^f	0.0018	Insignificant	Insignificant
HWSTOR	10	10	-- ^f	-- ^f	0.0013	Insignificant	Insignificant
NHHOTEL 20	12	9	6	6	0.0036	Insignificant	Moderate
SHERMAN 13	13	3	6	6	0.0067	Moderate	Insignificant
SYLMAR	97	63	-- ^e	-- ^e	0.0022	Insignificant	High
VAN NUYS 7	33	41	7	21	0.0117	High	Moderate

NOTES:

- For analytical assumptions see (Naeim, 1996).
- Estimate of the code WSD value times 1.4 at the time of building design.
- Estimate of the UBC-94 code WSD value times 1.4.
- ATC-38 Definitions of overall damage states are used.
- Design was not based on code static lateral force procedures.
- Sufficient information not available for compiling this value.
- Approximate methods used are not applicable to this case.

TABLE 2. Vibration Periods (seconds)

Building Acronym	Interpreted N-S & E-W Fundamental Periods ^a	Interpreted Predominant Response Periods ^a	Significant Period Elongation? ^a	Design Code Period Estimates ^a	UBC-94 Period Estimates ^a
BURBANK 10	0.57 - 0.62	Same	No	0.30	0.58
BURBANK 6	1.28 - 1.28	Same	No	0.60	0.95
LARES 17	0.80 - 1.20	Same	Moderate	-- ^b	-- ^b
LAOFFI 19	2.60 - 3.41	Same	No	0.76 - 1.90	1.24 - 2.33
LACC 2	1.28 - 0.2 to 1.14	Same	Yes in E-W Dir.	-- ^b	-- ^b
LACOMM 3	0.55 - 0.51	Same	No	0.16	0.40
LAWH 5	1.46 - 1.37	Same	No	0.60	0.73
LAOFFI 52	6.0 - 6.0	1.6 to 2.0	No	-- ^b	-- ^b
LAOFFI 54	6.0 - 6.0	1.0 to 2.0	No	-- ^b	-- ^b
LAOFFI 6	0.85 - 0.85	Same	No	0.56	0.56
LAPARK 6	0.35 - 0.40	Same	No	0.18	0.44
UCLA 7	0.66 - 1.02	Same	No	-- ^b	-- ^b
LAHOSP 7	0.64 to 1.5 - 0.79 to 1.5	Same	Yes	-- ^b	-- ^b
LAOFFI 9	1.21 - 1.71	Same	No	-- ^f	-- ^f
HWSTOR	-- ^b	-- ^b	-- ^b	-- ^b	-- ^b
NHHOTEL 20	2.20 - 2.50	Same & 0.70	Moderate	1.20	1.60
SHERMAN 13	2.6 - 2.9	Same & 1.0	Yes	1.27	1.60
SYLMAR	0.46 - 0.46	Same	No	-- ^b	-- ^b
VAN NUYS 7	1.1 to 1.8 - to 2.2	Same	Yes	0.70	0.70

a) for analytical assumptions, methods and procedures see (Naeim, 1996)

b) Either sufficient information not available for compiling this value or value not applicable.

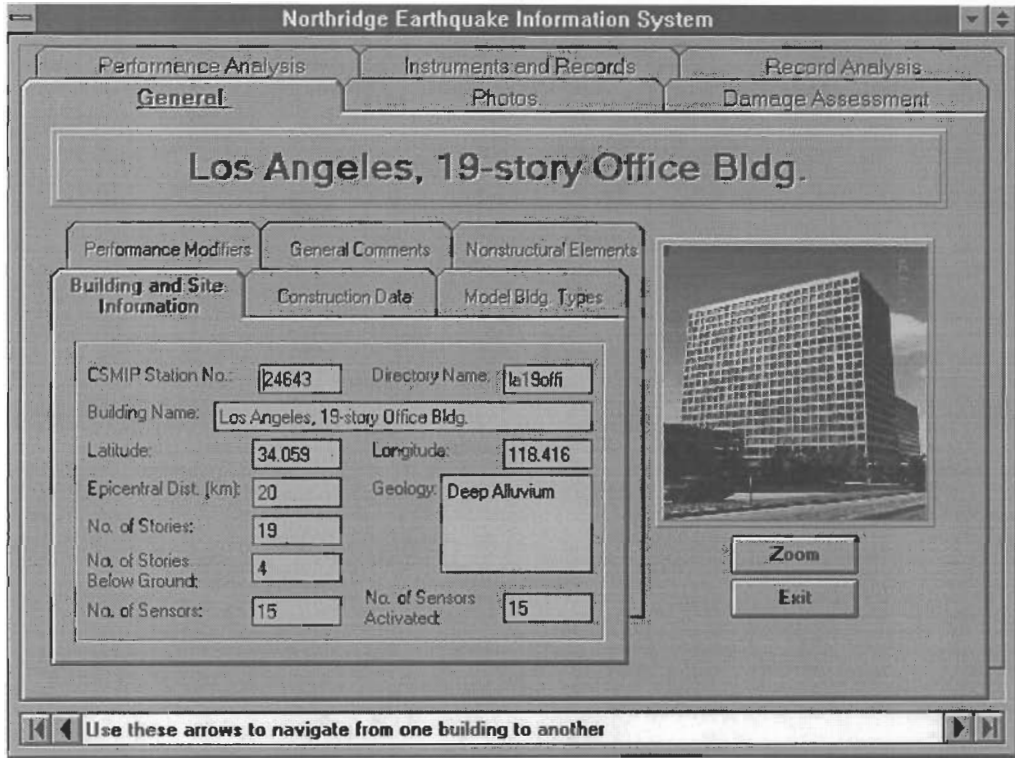


Figure 1. The Information System main folder LAOFFI 19 building.

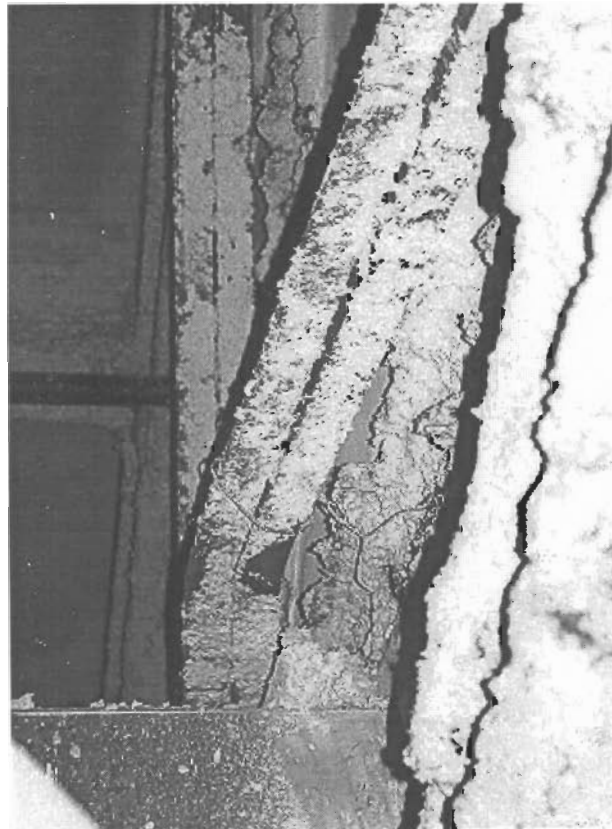


Figure 2. Buckled brace at the penthouse of the LAOFFI 19 building.

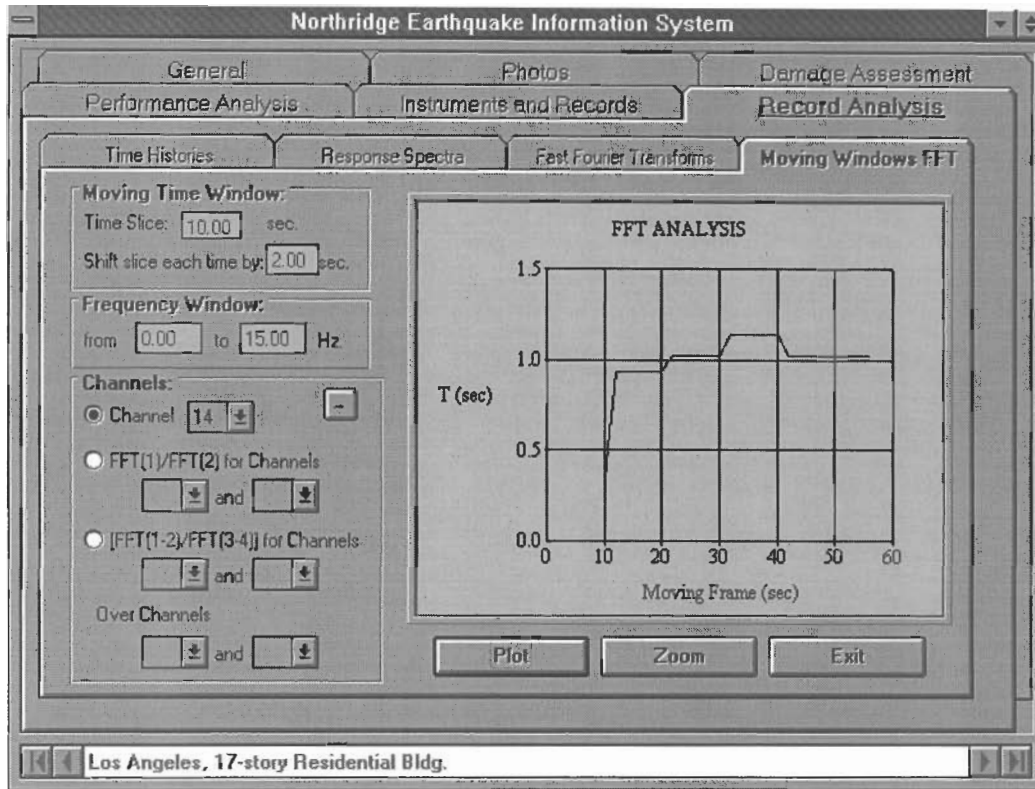
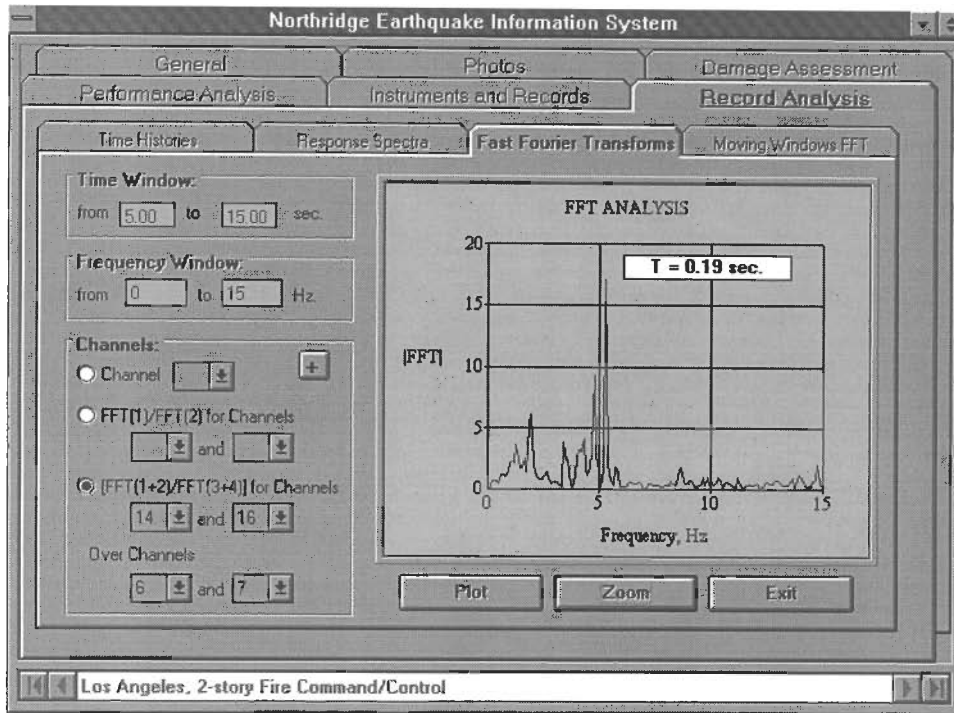


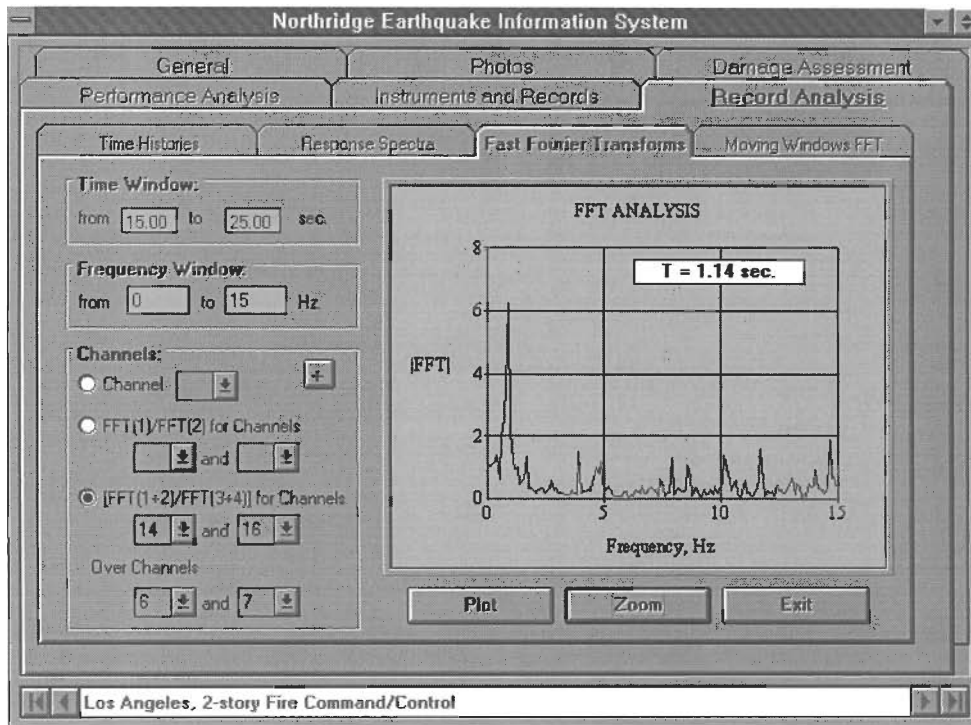
Figure 3. Moving windows FFT analysis for the N-S response of the LARES 17 bldg.



Figure 4. Tiles over isolation pit of the LACC 2 bldg. after the earthquake (photo courtesy of Robert Bachman).



(a) time = 0 to 15 seconds



(b) time = 15 to 25 seconds

Figure 5. Fast Fourier Transform of E-W response of LACC 2 building shows that the pit separation shown in Figure 4 has permitted the building to behave as an isolated system after 15 seconds into the ground motion.

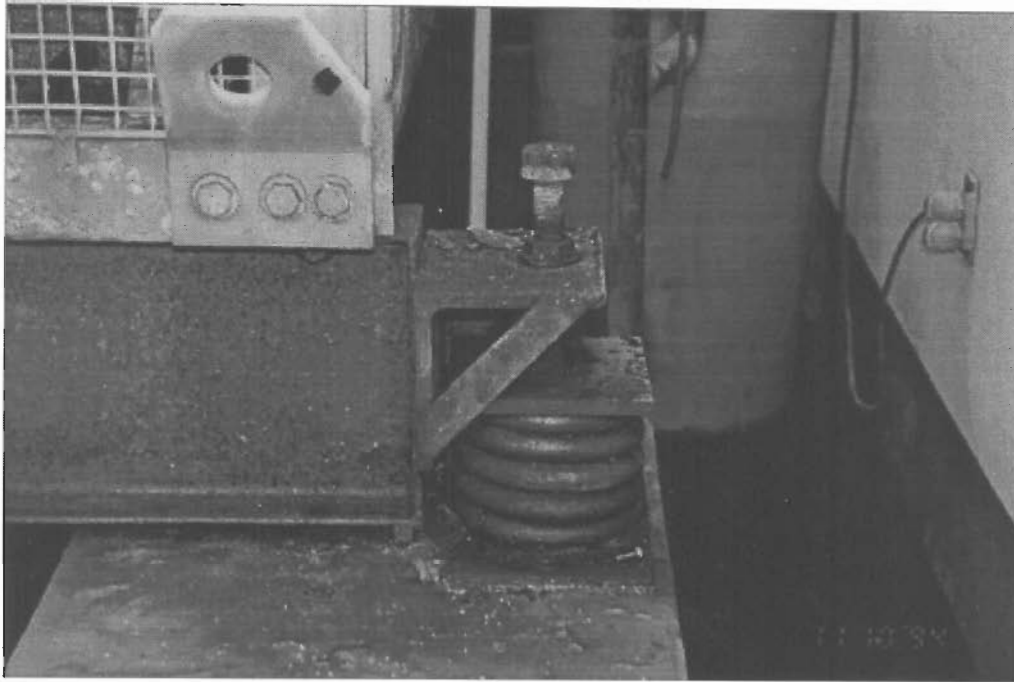


Figure 6. Mechanical equipment damage at the roof of BURBANK 6 building.

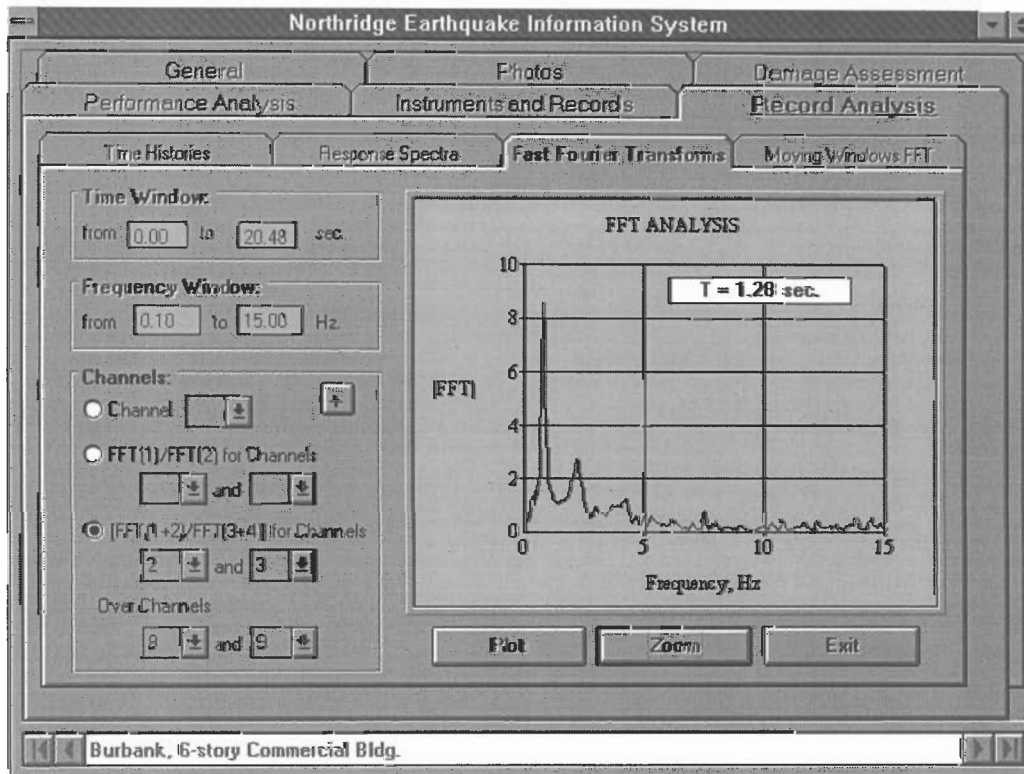


Figure 7. FFT analysis depicting fundamental N-S period of BURBANK 6 building.

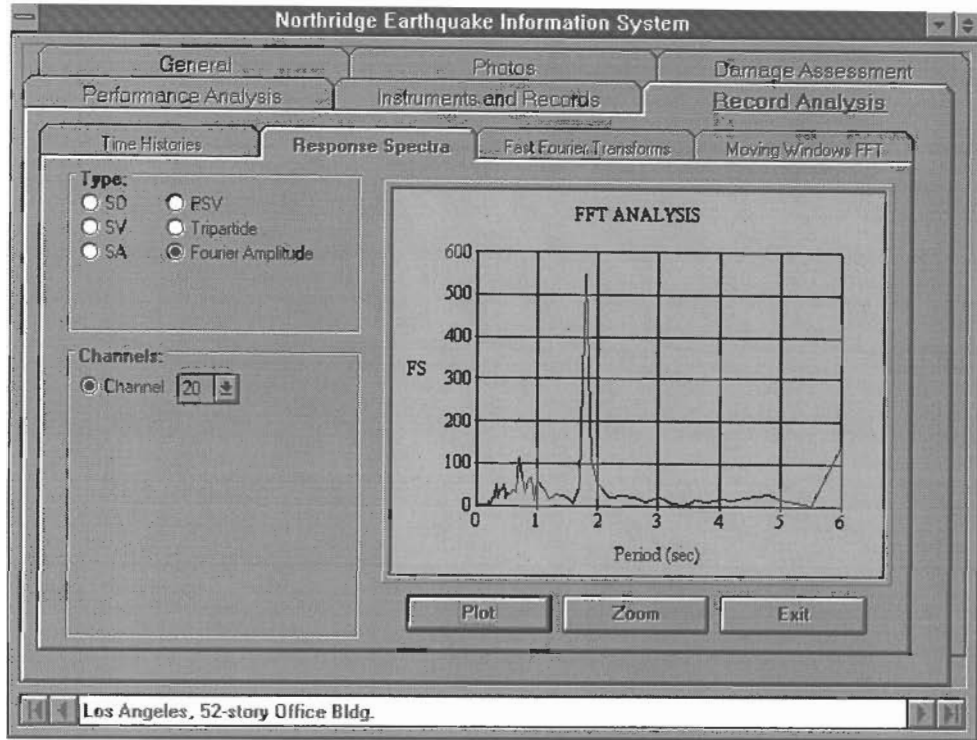


Figure 8. Fourier spectrum indicating a fundamental period of 6.0 sec. For LAOFFI 52 building. Notice that the predominant period however is slightly below 2 seconds.

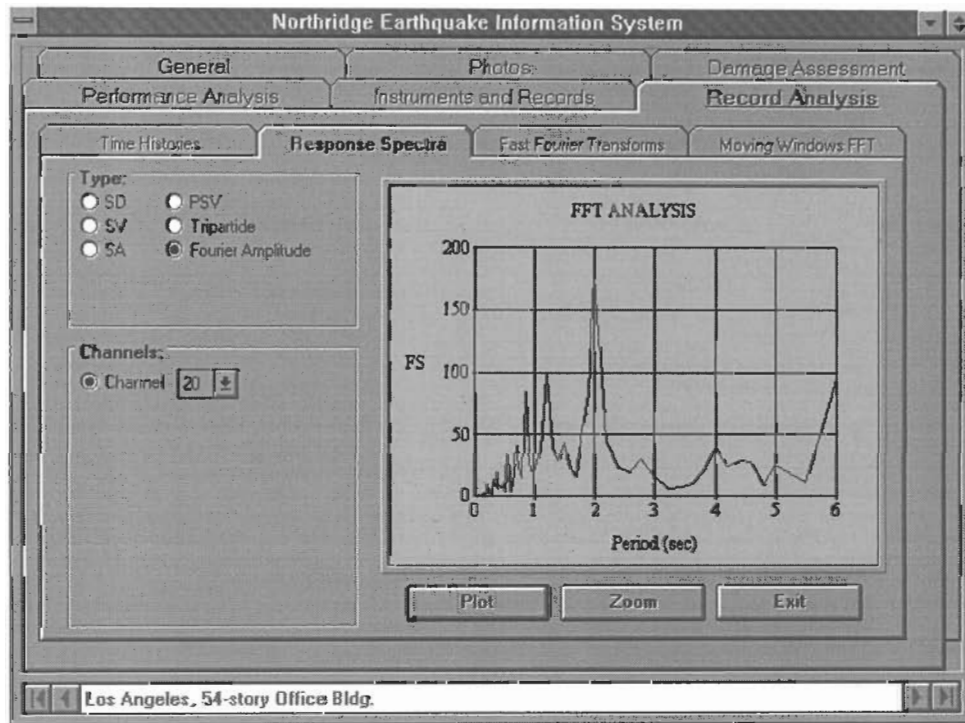


Figure 9. Fourier spectrum indicating a fundamental period of about 6.0 sec. For LAOFFI 54 building. Notice that the predominant period however is at about 2 seconds.