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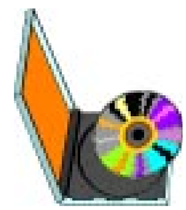
Performance of Extensively Instrumented Buildings During the January 17, 1994 Northridge Earthquake

An Interactive Information System



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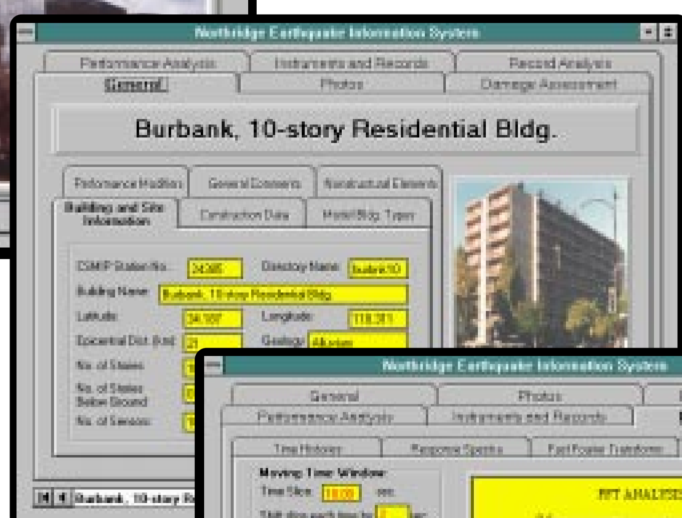

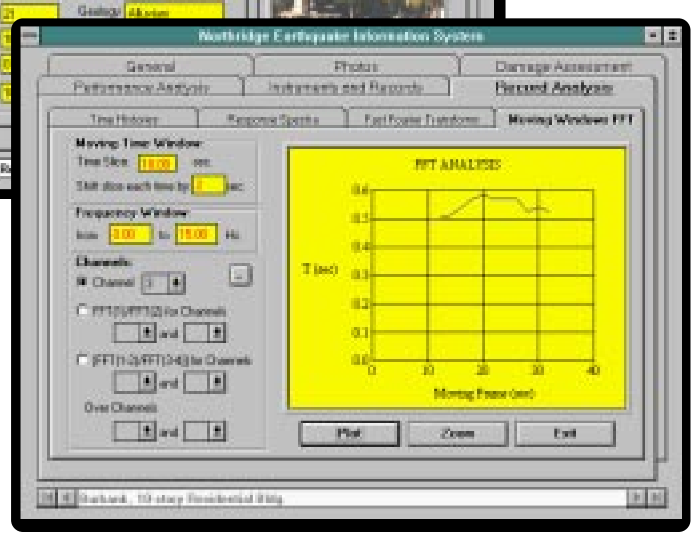

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ABSTRACT

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 17 instrumented buildings which experienced peak base acceleration greater than 0.25g, two downtown skyscrapers and a base isolated Fire Command Control building.

This report and its companion CD-ROM disc document the results of an elaborate two-year research and development project which included inspection of the buildings, damage assessment, performance evaluations, and contrasting forces, displacements, and dynamic characteristics interpreted from recorded data with those suggested by building codes.

A very unique feature of this project which is embodied in the companion interactive information system is publication of not only the results of the investigations – as is customary – but also the tools of the research. The relational database engine of the information system may be modified and expanded to accommodate new buildings and/or other seismic events with relative ease.

The primary objective of this report is to familiarize the reader with how the information and tools contained in the developed information system may be utilized and to provide an overall view of what author believes are the most significant aspects of the response of buildings covered by this investigation. 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.

ACKNOWLEDGMENTS

Funding for this project was provided in part by State of California, Department of Conservation, Division of Mines and Geology, Strong Motion Instrumentation Program (SMIP). Significant supplemental funding was provided by John A. Martin and Associates, Inc.

The author wishes to extend his gratitude and appreciation to DMG/SMIP technical staff, Dr. Moh Hung, and the project's advisory committee for their support and constructive suggestions. The dedication and total commitment of Mr. Hung Lee, Mr. Marvin Mittlestaedt and the rest of the staff at John A. Martin and Associates is gratefully appreciated.

We appreciate the efforts of Dr. Andrei Reinhorn, Dr. James C. Anderson, Mr. Frank McClure, Mr. William Staehlin, Dr. Marshall Lew, Dr. Gary Hart, Dr. Charles Kircher, Dr. Kenneth Honda, and Mr. John Robb who reviewed and commented on various pre-release alpha and beta versions of this information system.

Many of the pieces of the software incorporated in this information system have been developed by the author over the time span of more than a decade work on other projects. This project would not have been completed without the unconditional support given to research and development efforts by Jack and Trailer Martin and the rest of the management at John A. Martin and Associates, Inc.

The opinions expressed in this report are those of the author and do not necessarily reflect the views of the California Strong Motion Instrumentation Program.

TABLE OF CONTENTS

1. INTRODUCTION

On the morning of January 17, 1994 southern California was shaken by an earthquake (M_L = 6.4, M_W = 6.7, M_S = 6.8) which may prove to be one of the most significant earthquakes in the United States history (Naeim, 1995). 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) 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 a peak base acceleration 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. These buildings are:

- a 6-story hospital building located in Sylmar with 13 sensors
- a 7-story hotel in Van Nuys with 16 sensors
- a 13-story commercial building in Sherman Oaks with 15 sensors
- a North Hollywood 20-story hotel with 16 sensors
- a Los Angeles 19-story office building with 15 sensors
- a Burbank 10-story residential building with 16 sensors
- a Burbank, 6-story commercial building with 13 sensors
- a Los Angeles 3-story commercial building with 15 sensors
- a 6-story parking structure in Los Angeles with 14 sensors
- the 7-story UCLA Math-Science building with 18 sensors
- the Hollywood Storage Building in Los Angeles with 12 sensors

- a 6-story office building in Los Angeles with 14 sensors
- a Los Angeles 9-story office building with 18 sensors
- a Los Angeles 17-story residential building with 14 sensors
- a Los Angeles 5-story Warehouse with13 sensors
- a base isolated 7-story hospital building in Los Angeles with 24 sensors
- a 52-story office building in Los Angeles with 20 sensors
- a 54-story office building in Los Angeles with 20 sensors
- a base-isolated 2-story Fire Command Control building in Los Angeles with 16 sensors

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. Examples of application of this information system (hereafter referred to as the

SMIP Information System) may be seen throughout this report.

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. New buildings and data from other seismic events may be added to the information system with relative ease.

It is hoped that by publishing not only the results of the investigations – as is customary – but the tools of research at the same time, this report and the companion Information System will become widely used in teaching/learning earthquake engineering and seismic response principles and in further learning from response of instrumented buildings to the Northridge and other earthquake ground motions.

Detailed response analysis and system identification studies for each of the 20 buildings covered in this report will require one or more individual reports and is beyond the scope of this investigation. In contrast, the purpose of this report is to familiarize the reader with how the information and tools contained in the companion CD-ROM may be utilized and to provide an overall view of what the author believes are the most significant aspects of the response of buildings contained in this information system.

General information regarding the information system requirements and contents are presented in Chapter 2, followed by a more elaborate tutorial on the use of the information system in Chapter 3. Key response parameters and characteristics of each building is discussed in Chapter 4 which is followed by closing remarks in Chapter 5. Sketches of the buildings showing the spatial distribution sensors is presented in Appendix A. Simplified backup calculations may be found in Appendix B. An ATC-38 form which was utilized to complete damage assessment folders of the information system is included as Appendix C to this report.

2. WELCOME TO THE SMIP INSTRUMENTED BUILDINGS INFORMATION SYSTEM

2.1. HARDWARE AND SOFTWARE REQUIREMENTS

To successfully install and execute the SMIP Information System, you need:

- A personal computer using a 386 or higher processor (486-DX4 or Pentium recommended) with 8 MB RAM, 24 MB of available hard disk (for full installation,), CD-ROM drive, mouse, and a VGA or VGA+ color display capable of showing 256 or more simultaneous colors.
- Either
  - ⇒ MS-DOS® operating system version 3.1 or later running Microsoft Windows™ graphical environment version 3.1 or later.
  - ⇒ Microsoft® CD-ROM Extensions (MSCDEX) version 2.2 or later (provided with your CD-ROM drive).
- Or
  - ⇒ Windows 95® operating system with a CD-ROM drive installed and working.

2.2. INSTALLATION PROCEDURES

For Windows 3.1 or 3.11 or NT Environments:

- Turn on your computer and CD-ROM drive.
- Start Windows and, if necessary, display the Program Manager window.
- Insert the Information System disc in the CD-ROM drive.
- In Program Manager, choose Run from the File menu.
- Type the drive letter for the CD-ROM drive, a colon (:), and ***install*** (for

example, ***d:\install***).

- Press Enter.
- Follow the instructions that appear on your screen.

For Microsoft Windows 95 Environment:

- Turn on your computer and CD-ROM drive.
- Insert the Information System disc in the CD-ROM drive.
- Click the Start button, click Settings, and then click the Control Panel.
- Double-click the Add/Remove Programs icon.
- On the Install/Uninstall tab, click the Install button.
- Follow the instructions that appear on your screen.

The installation procedure creates a program group if needed, and places an icon for the SMIP Information System in the group. You can start the Information System at any time by double-clicking the icon.

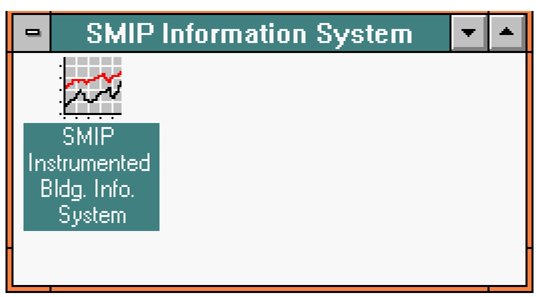


Figure 2-1. SMIP Information System icon after successful installation.

2.3. INSTALLATION OPTIONS

The installation program copies the essential programs and files to your hard disk. The bitmap images (photographs and sketches) and uncompressed versions of the earthquake records remain on the CD-ROM disc and are not copied to the hard disk. The photos and uncompressed record files may be found under the ***\photos and \Records*** subdirectories of the CD-ROM disc, respectively. The uncompressed version of the Information System exceeds 144 MB in size.

In order to minimize the required hard disk space while providing sufficient speed needed for tasks such as online response analysis and moving windows FFT, an innovative on-the-fly data compression/decompression scheme has been implemented. Using this technology, a compressed PKZIP-compatible file containing all Volume 2 and 3 data files is installed for each building. The particular uncompressed files needed for any operation are extracted from the compressed file, utilized, and then purged by the Information System. In this manner, the entire Information System when installed takes slightly more than 22 MB of hard disk space (Figure 2-2).

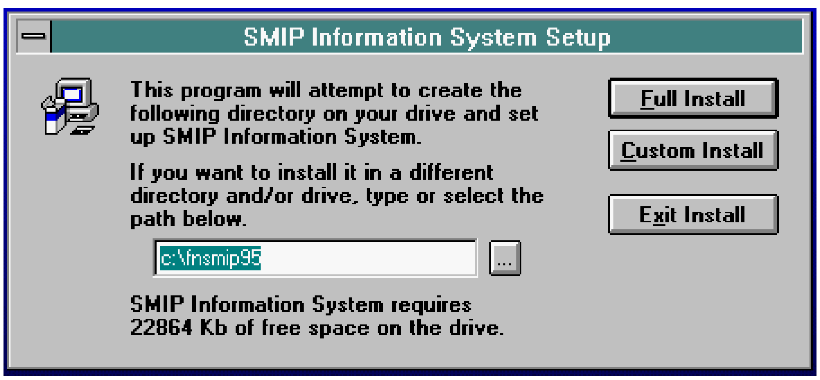


Figure 2-2. SMIP Information System installation options.

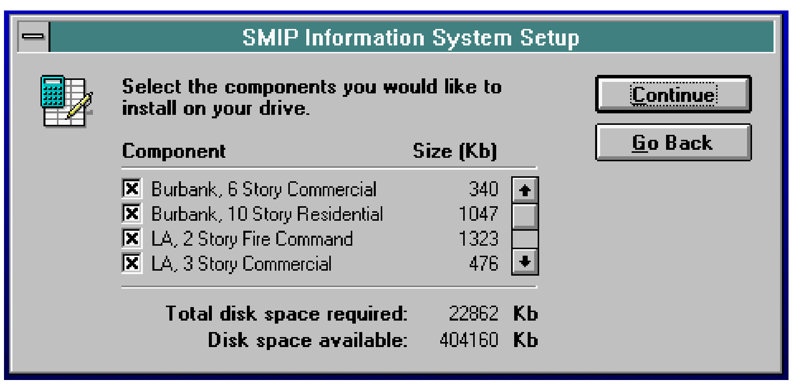


Figure 2-3. The Custom Installation Option

If your hard disk space is very limited, or if you are interested only in a few of the buildings contained in the information system, you may select the Custom Install Option which permits you to install only the data sets that you need on the hard disk (Figure 2-3). You can use the installation procedure to install more building data set at a later time. However, remember that the installation program does not remove the building data sets that you are not interested in. You can do this manually by deleting the corresponding compressed file (named SMIPDATA.ZIP) from you hard disk.

You can speed up the operation of the Information System at the expense of more hard disk usage. To achieve this all you need to do is to decompress (i.e., unzip) the compressed files of your interest or copy uncompressed files from the \records subdirectory of the CD-ROM disc and delete the corresponding zip file from your hard disk. Once this is done, the Information System recognizes that on-the-fly compression/decompression is not needed.

We strongly recommend that you perform a full installation and refrain from manipulating the files installed by the Information System. Always make backup copies of the information system directories before you proceed with any changes.

2.4. INFORMATION SYSTEM CONTENTS AND UTILITIES

The SMIP Information System is an interactive dynamic tool which not only contains a comprehensive amount of information regarding building response, instrumentation, damage photos and sketches on 19 instrumented buildings, it also embodies state-of-the-art analytical tools for on-line evaluation, manipulation, and interpretation of the data by the user.

The SMIP Information System is designed around a file cabinet/file-folders metaphor (See Figure 2-4). All information regarding each building is presented in a set of six main folders (or main tabs). Main folders have child folders of their own. The folder structure of the SMIP Information system is as follows:

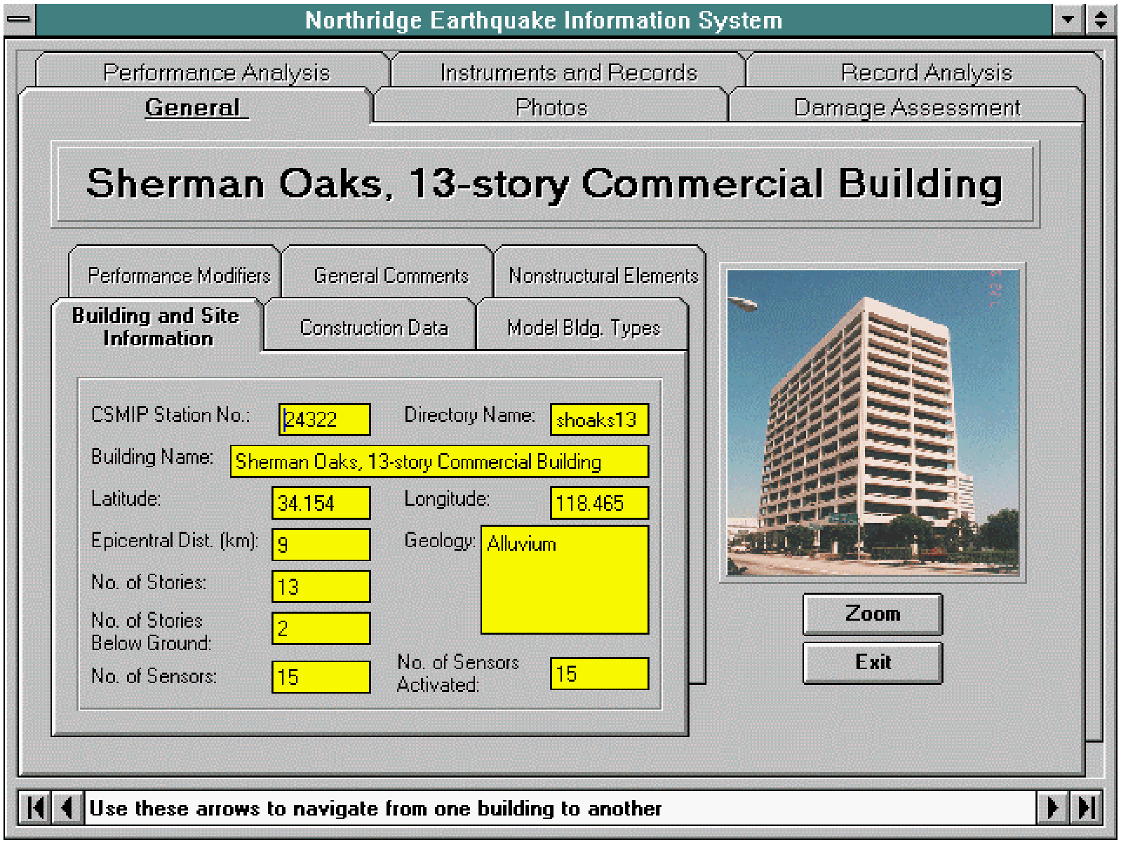


Figure 2-4. SMIP Information System uses a folder metaphor. The navigation controls at the bottom of screen are used to move from one building to another.

1. <u>General</u>
   a) Building and Site Information
   b) Construction Data
   c) Model Building Types
   d) Performance Modifiers
   e) General Comments
   f) Nonstructural Elements
2. <u>Photos</u>
3. <u>Damage Assessment</u>
   a) General Damage
   b) Nonstructural Damage
   c) Injuries/Fatalities/Functionality
   d) Observed Geotechnical Failures
   e) Detailed Damage Description

i) Vertical Elements

ii) Horizontal Elements

iii) Connections

iv) Foundations

v) Equipment/Systems

4. Performance Analysis

5. Instruments and Records

6. Record Analysis

a) Time Histories

b) Fourier and Response Spectra

c) Fast Fourier Transform (FFT)

d) Moving Windows FFT Analysis

Pointers to all information are contained in an open-architecture relational database. Detailed information on the structure of this database and database manipulation utilities developed as a part of this project are available from the author.

3. USING THE SMIP INSTRUMENTED BUILDINGS INFORMATION SYSTEM

3.1. STARTING THE INFORMATION SYSTEM

To start the SMIP information system in Windows 3.1 or 3.11 or NT environments, simply double-click on its icon. In Windows 95, click the ***Start*** button, then select SMIP Information system from the ***Programs*** task bar. The Information System's entry screen will appear on your display (Figure 3-1).

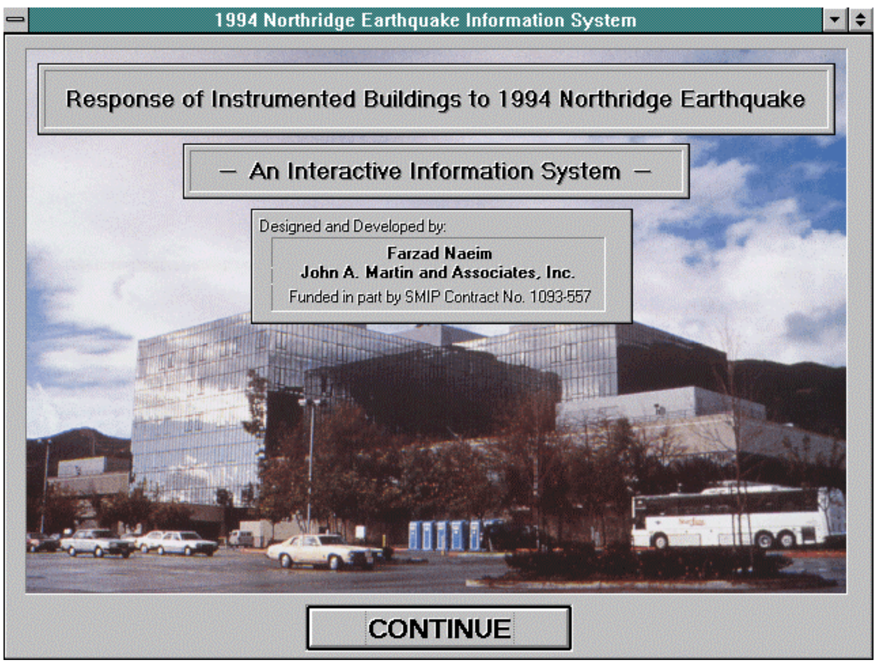


Figure 3-1. SMIP Information System's entry screen

Click on the Continue button to enter the information system. The folders for the first building contained in the information system appear on your screen (Figure 3-2).

The buildings are organized in an alphabetical order in the version of the database shipped by us. You can change the order of appearance, change the data, and add buildings to the information system (see Chapter 5 for details).

3.2. MOVING FROM ONE BUILDING TO ANOTHER

To move from one building to another click on the ***Next, Previous***, ***First***, or ***Last*** buttons displayed at the bottom of your screen (Figure 3-2).

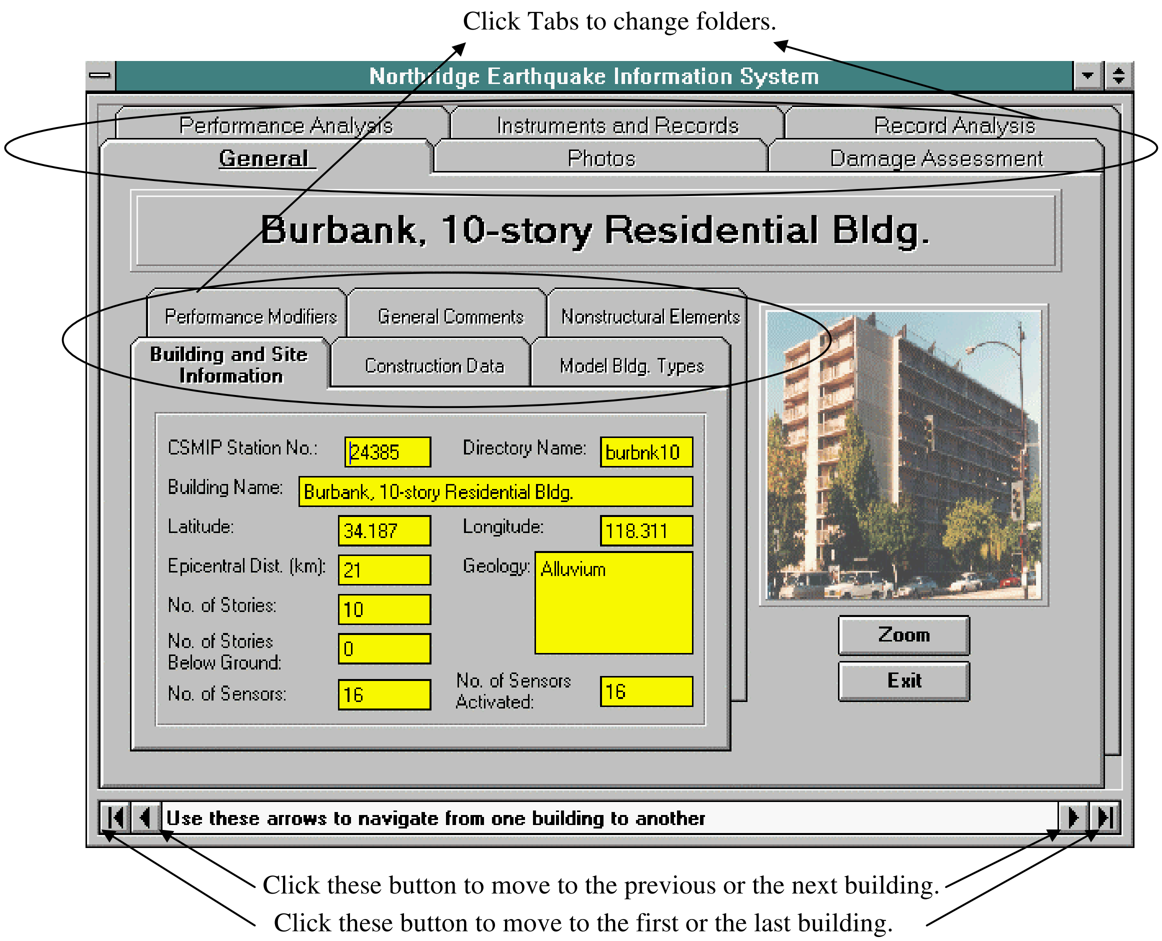


Figure 3-2. A typical building folder set.

3.3. BROWSING THE GENERAL FOLDER

Each building's general folder shows a photo of the building and a set of smaller folders containing general information regarding the building site and characteristics, design and construction dates, building type, pre-earthquake condition of the building, and the type of nonstructural elements of the building. The information contained in these smaller forms is consistent with what normally appears on the first page of an ATC-38 Post-

Earthquake Building Performance Assessment Form (see Appendix C).

You may click on the ***Zoom*** button to see an enlarged photo of the building (Figure3-3). Once the zoomed photo is displayed you may use the ***Print*** menu to obtain a hard copy of the screen or the ***Exit*** menu return back to the general folder.

Figure 3-3. Zoomed view of a General Folder photo.

3.4. BROWSING THE PHOTOS FOLDER

Click on the ***Photos*** tab to look at the photos of the building contained in the information. The number of photos included varies from one to more than a hundred per building. A typical view of a building's photo folder is shown in Figure 3-4.

You may use the ***Next, Previous***, ***First***, and ***Last*** buttons displayed directly above the ***Zoom*** button to move from on photo to another. You may also click the directional bars provided with the ***Available Photos*** list box for the same purpose. Notice that when you move from one photo to another the ***Caption pointer*** also moves to indicate the caption which corresponds to the photo you are looking at (see Figure 3-4). Also shown are the photos bitmap file name and the building level it relates to. As with the General Folder, you may use the Zoom button at any time to obtain an enlarged view of a photo and its caption (Figure 3-5).

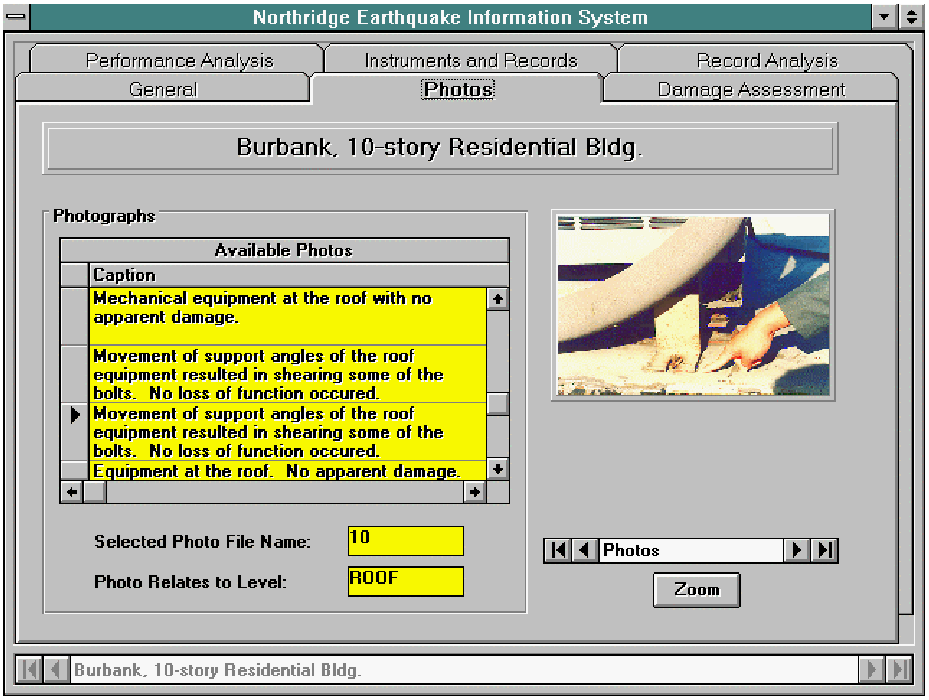


Figure 3-4. A typical view of a building's Photo Folder.

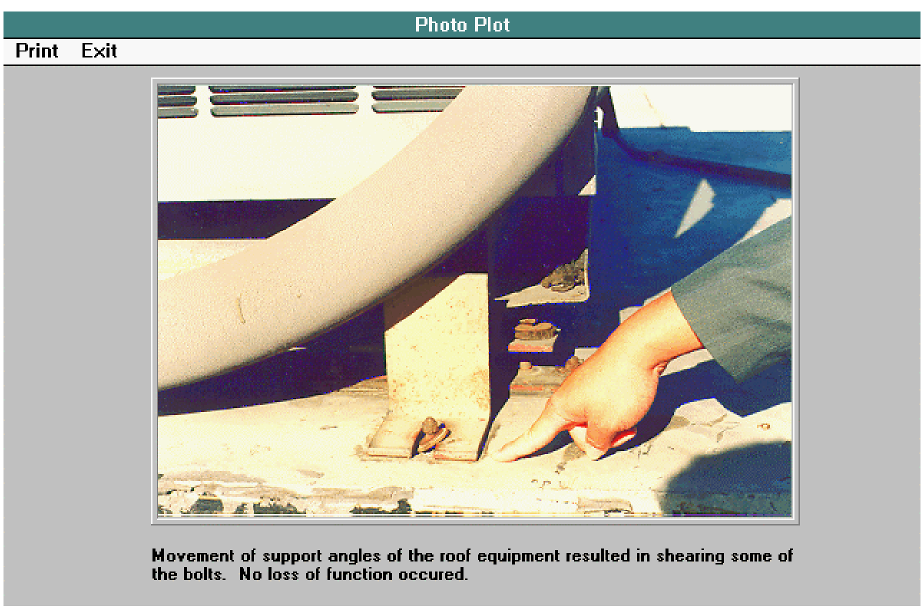


Figure 3-5. Zoomed view of a Photo Folder photo and its caption.

3.5. EVALUATING THE DAMAGE ASSESSMENT FOLDERS

Click the ***Damage Assessment*** tab and a set of five sub-folders will appear on the screen. You may click any of these tabs to see the associated information. The ***General Damage*** (Figure 3-6), ***Nonstructural Damage*** (Figure 3-7), ***Injuries/Fatalities/Functionality***, and ***Observed Geotechnical Failures*** folders together contain information which is consistent with the second page of an ATC-38 Post-Earthquake Building Performance Assessment Form (see Appendix A). The forms contained in the ***Detailed Damage Description*** folder (see Figures 3-8 and 3-9) contain information which is consistent with the items under similar heading in the ATC-38 forms. You may use the ***Next, Previous***, ***First***, and ***Last*** buttons displayed directly below the ***Notes*** button to examine other pages of information which may relate to various levels or directions of the building.

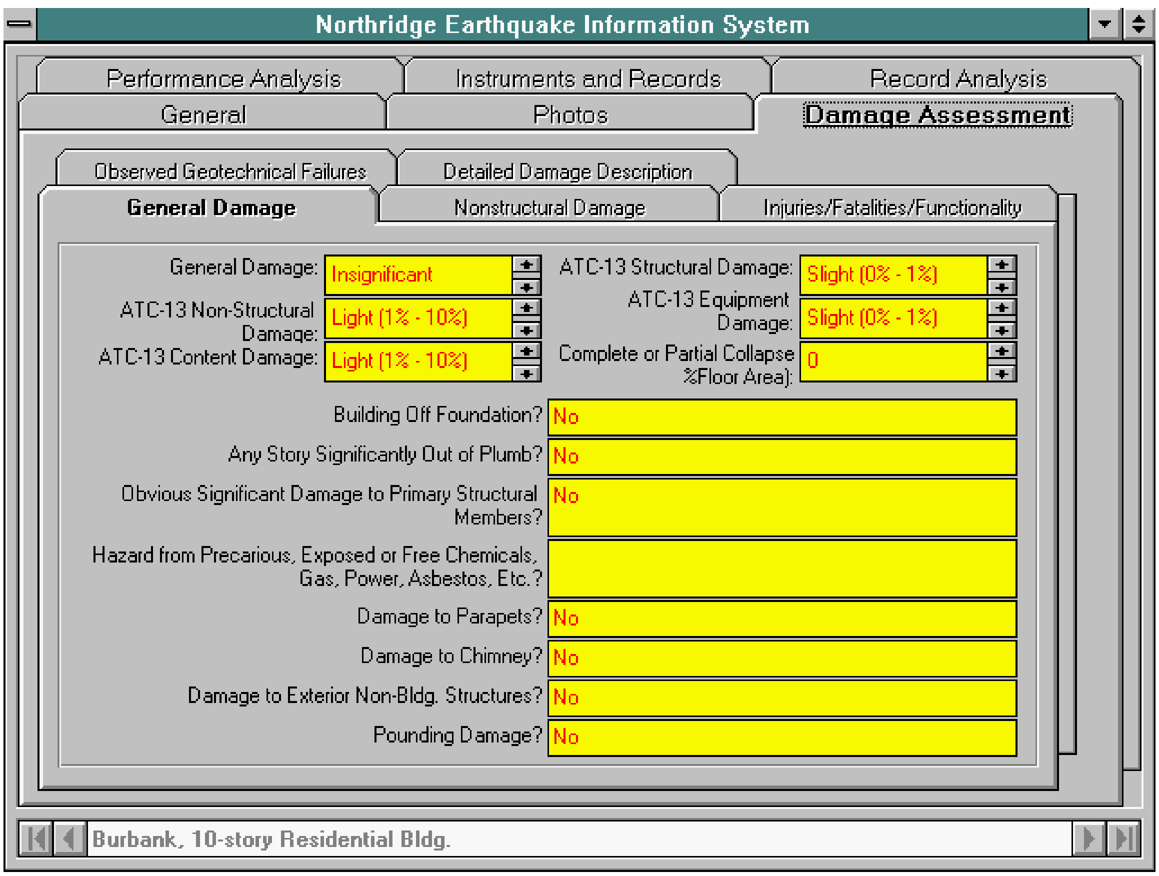


Figure 3-6. A typical view of a General Damage sub-folder.

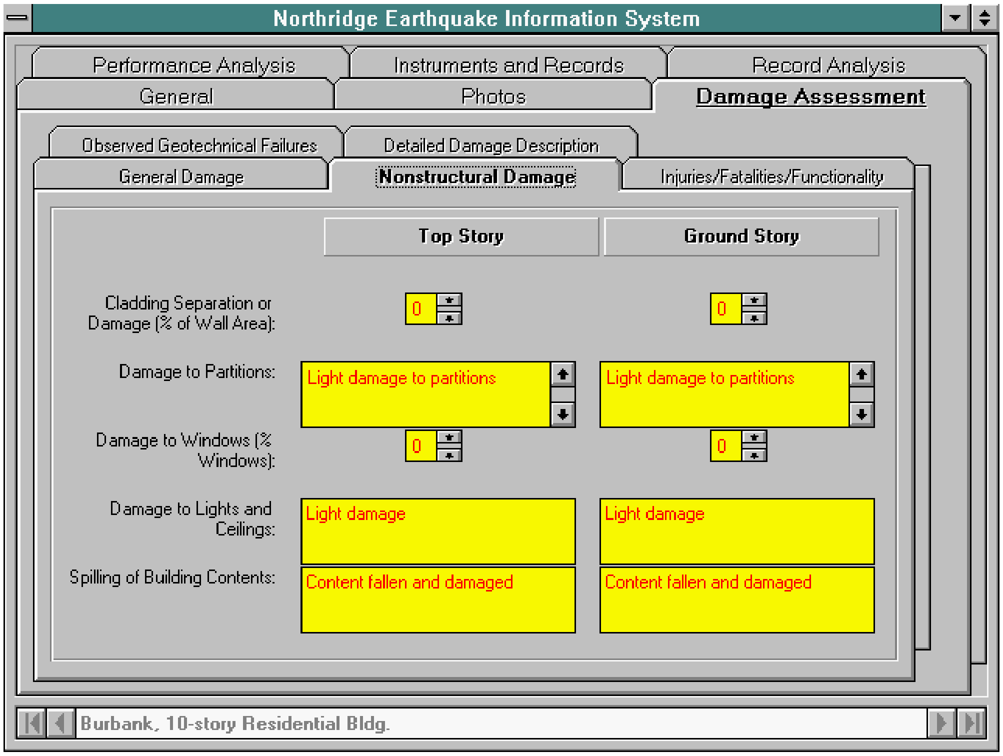


Figure 3-7. A typical view of a Nonstructural Damage sub-folder.

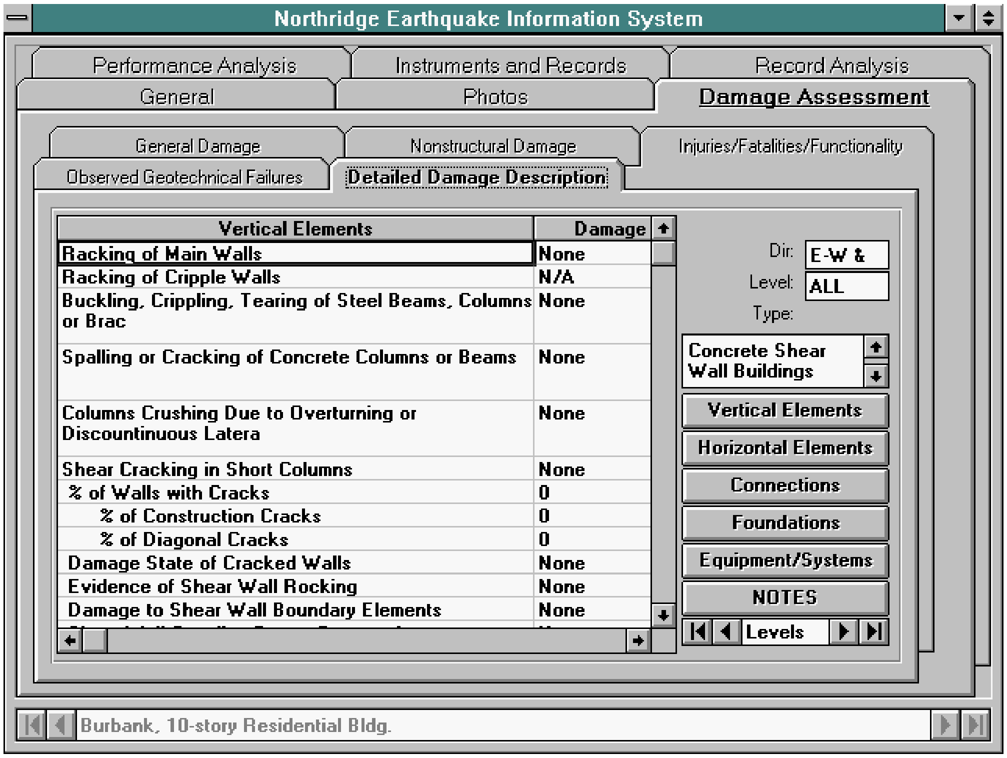


Figure 3-8. A typical view of a Detailed Damage folder (Vertical Elements selected).

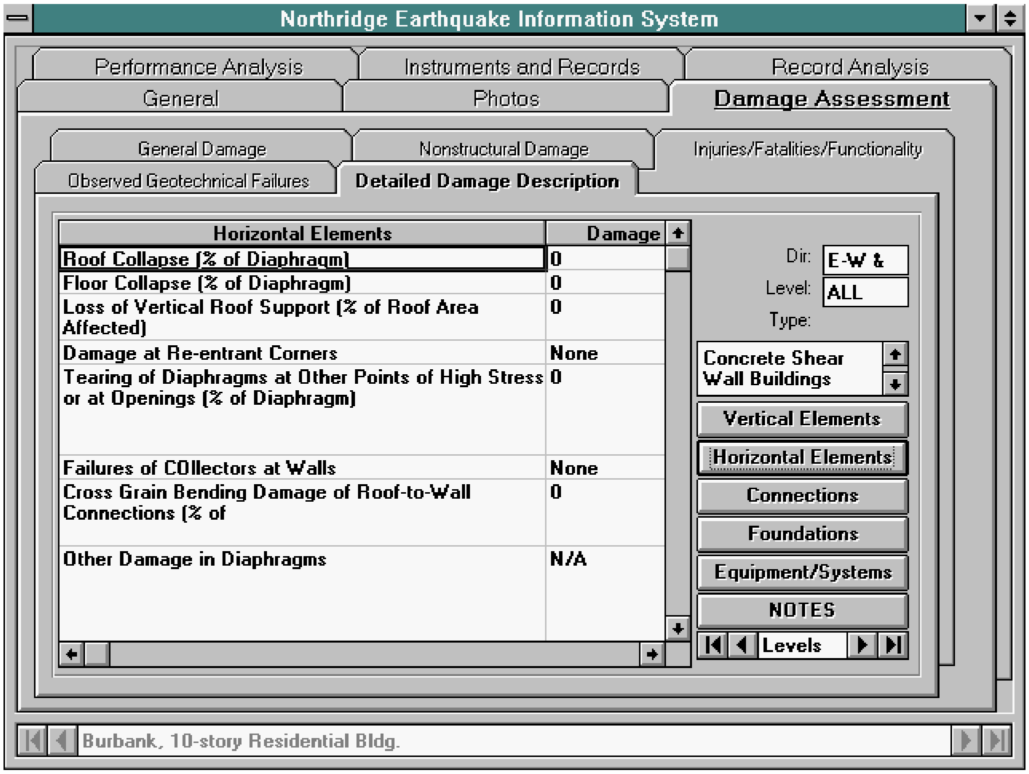


Figure 3-9. A typical view of a Detailed Damage folder (Horizontal Elements selected).

3.6. USING THE PERFORMANCE ANALYSIS TOOLS

The ***Performance Analysis*** folder provides innovative ways of evaluating approximate values of key response and performance parameters of a building at any arbitrary time into the ground motion excitation, or at the time of a given maximum response parameter of interest. The North-South, East-West, and torsional components of the following key parameters may be evaluated:

- Inertial Story Forces.
- Story Shears
- Overturning Moments
- Lateral Displacements
- Story Drifts
- Story Drift Indices

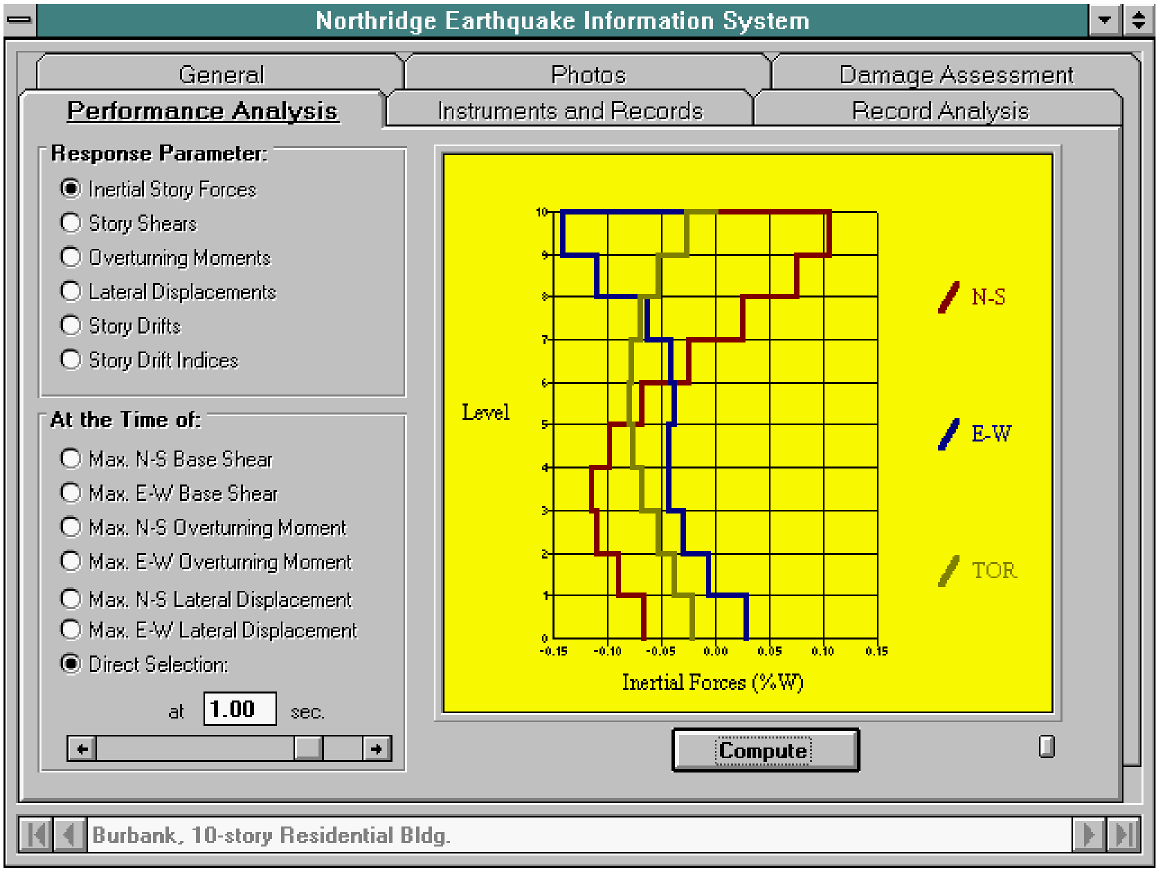


Figure 3-10. Approximate distribution of inertial story forces for a building at 1.00 seconds into the ground motion.

You may evaluate any of the above parameters at any desired time by selecting the Direct Selection option and typing in the time instant or using the scroll bar at the bottom of the time selection box (see Figure 3-10). In addition, you may evaluate any of the above parameters at any of the following times of maximum response:

- At the time maximum N-S base shear
- At the time of maximum E-W base shear
- At the time maximum N-S overturning moment
- At the time maximum E-W overturning moment
- At the time maximum N-S lateral displacements
- At the time maximum E-W lateral displacements

For example, Figure 3-11 shows distribution of inertial story forces at the time of maximum N-S base shear for the same building of Figure 3-10. Notice that as soon as you select one of the instances of maximum response, the corresponding time appears on the selection time box (i.e., 8.88 seconds in Figure 3-11).

Representative screens showing results of analysis for distribution story shears, lateral displacements and story drift ratios at various times of response maxima are shown in Figures 3-12, 3-13, and 3-14, respectively.

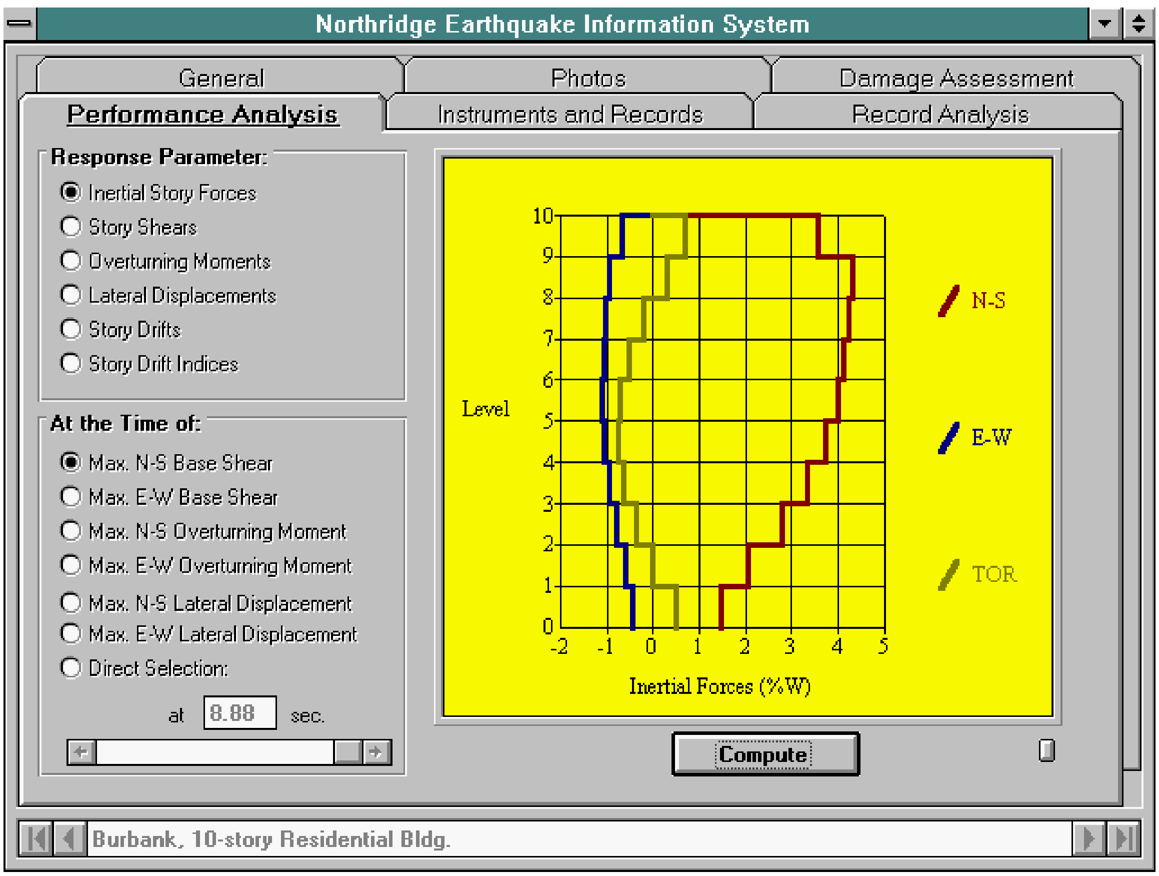


Figure 3-11. Approximate distribution of inertial story forces for a building at the time of maximum N-S base shear.

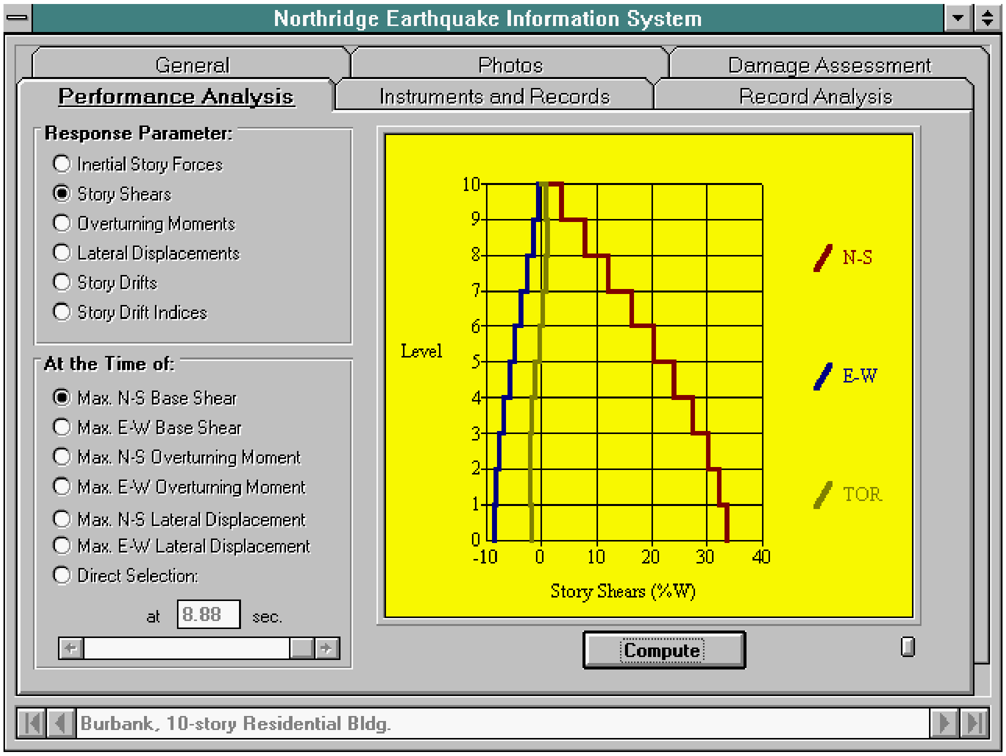


Figure 3-12. Approximate distribution of story shears for a building at the time of maximum N-S base shear.

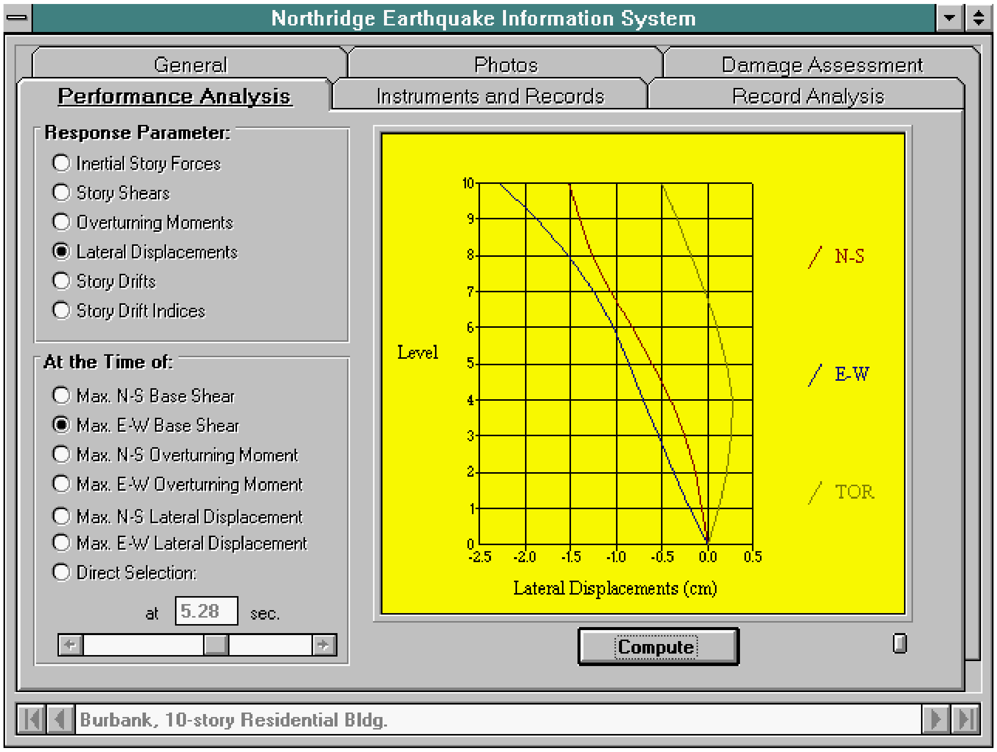


Figure 3-13. Approximate distribution of lateral displacements for a building at the time of maximum E-W base shear.

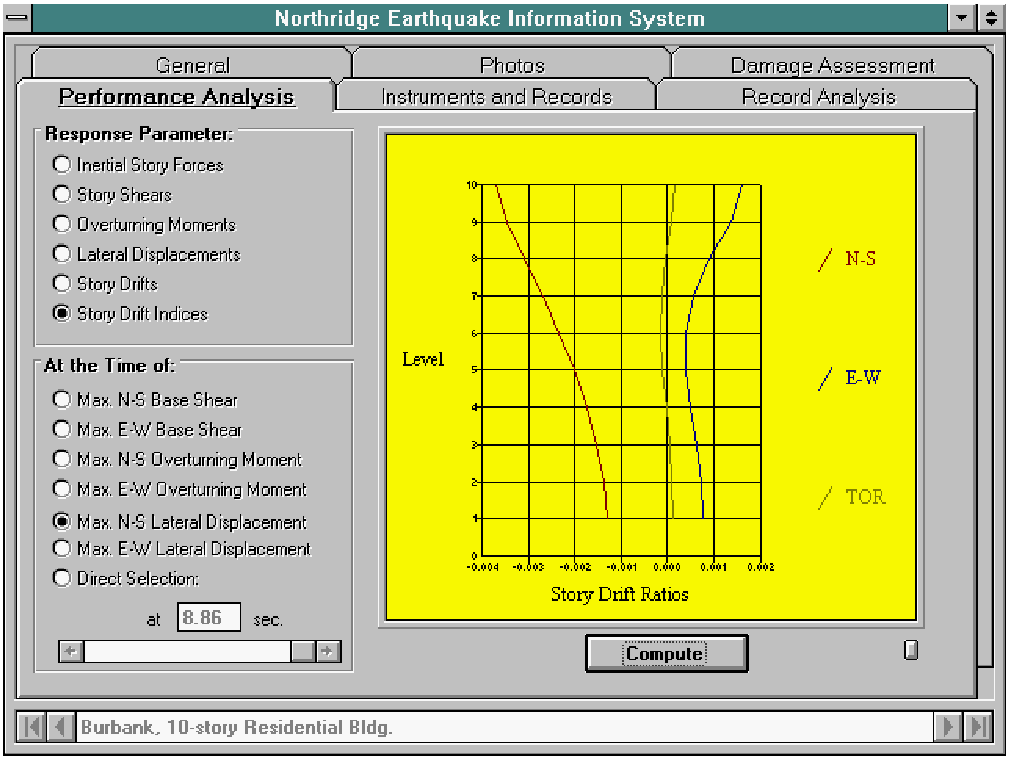
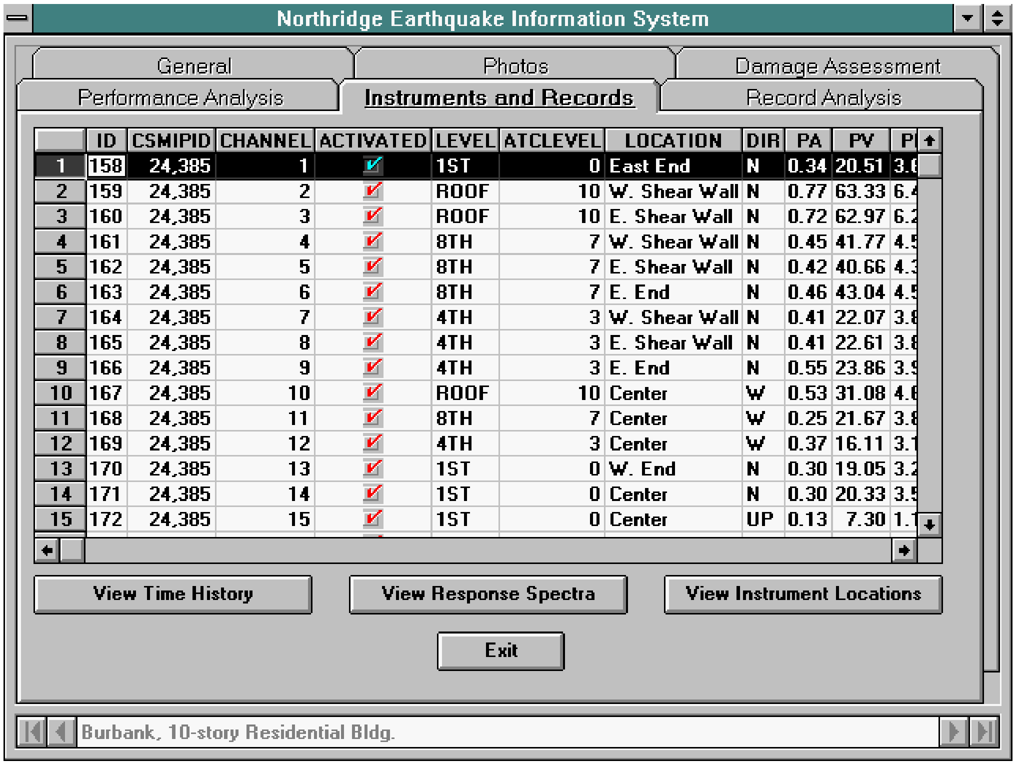


Figure 3-14. Approximate distribution of drift ratios for a building at the time of maximum N-S roof lateral displacement.

For an overview about the nature of analytical assumptions and approximations embodied in the Performance Analysis and Record Analysis folders, please see Section 4.1 of this report.

3.7. UTILIZING THE INSTRUMENTS AND RECORDS FOLDER

Click the ***Instruments and Records*** folder to evaluate the earthquake records that SMIP instrumentation had made available for a building (Figure 3-15). The channel number, activation status, floor level, location in plan, direction, and peak values of acceleration (PA), velocity (PV) and displacement (PD) of each channel is shown. You may view the time history, or response spectra of a record by clicking the ***View Time History*** and ***View Response Spectra*** buttons as shown in Figures 3-16 and 3-17, respectively. In addition, you may view the location of each instrument in the building by clicking the ***View Instrument Location*** button.



	ID	CSMIPID	CHANNEL	ACTIVATED	LEVEL	ATCLEVEL	LOCATION	DIR	PA	PV	P
1	158	24,385	1	✓	1ST	0	East End	N	0.34	20.51	3.6
2	159	24,385	2	✓	ROOF	10	W. Shear Wall	N	0.77	63.33	6.4
3	160	24,385	3	✓	ROOF	10	E. Shear Wall	N	0.72	62.97	6.2
4	161	24,385	4	✓	8TH	7	W. Shear Wall	N	0.45	41.77	4.5
5	162	24,385	5	✓	8TH	7	E. Shear Wall	N	0.42	40.66	4.3
6	163	24,385	6	✓	8TH	7	E. End	N	0.46	43.04	4.5
7	164	24,385	7	✓	4TH	3	W. Shear Wall	N	0.41	22.07	3.8
8	165	24,385	8	✓	4TH	3	E. Shear Wall	N	0.41	22.61	3.8
9	166	24,385	9	✓	4TH	3	E. End	N	0.55	23.86	3.9
10	167	24,385	10	✓	ROOF	10	Center	W	0.53	31.08	4.6
11	168	24,385	11	✓	8TH	7	Center	W	0.25	21.67	3.8
12	169	24,385	12	✓	4TH	3	Center	W	0.37	16.11	3.1
13	170	24,385	13	✓	1ST	0	W. End	N	0.30	19.05	3.2
14	171	24,385	14	✓	1ST	0	Center	N	0.30	20.33	3.5
15	172	24,385	15	✓	1ST	0	Center	UP	0.13	7.30	1.1

Figure 3-15. A typical Instruments and Records folder where Channel 1 is selected.

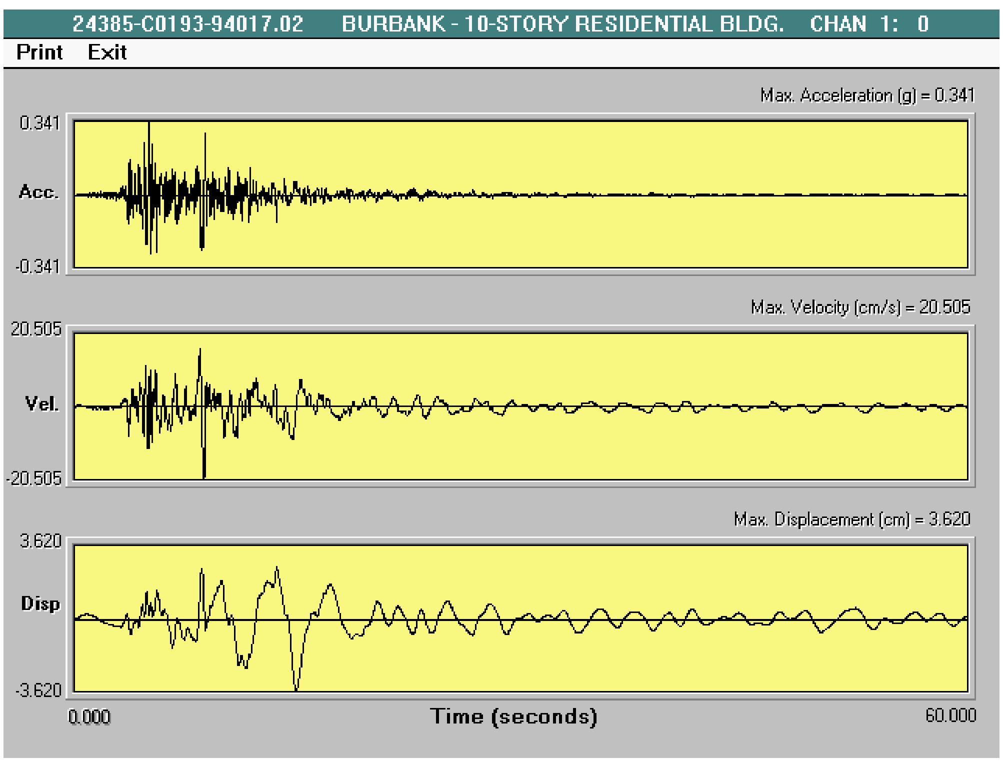


Figure 3-16. Clicking the ***View Time-History*** button has resulted in displaying a time series.

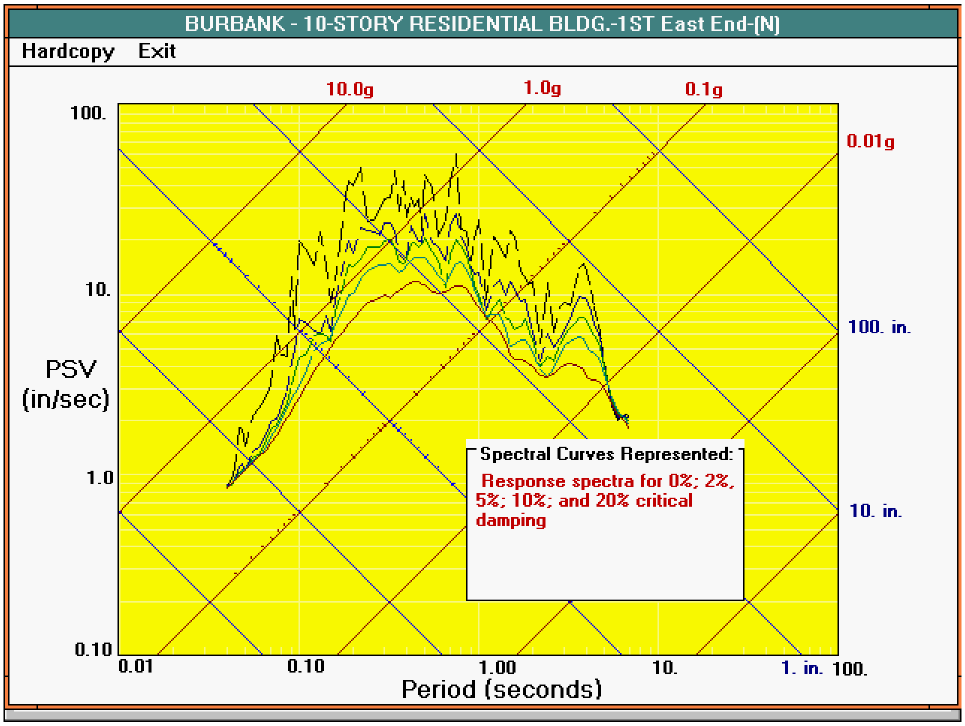


Figure 3-17. Clicking the ***View Response Spectra*** button has resulted in displaying a response spectra plot.

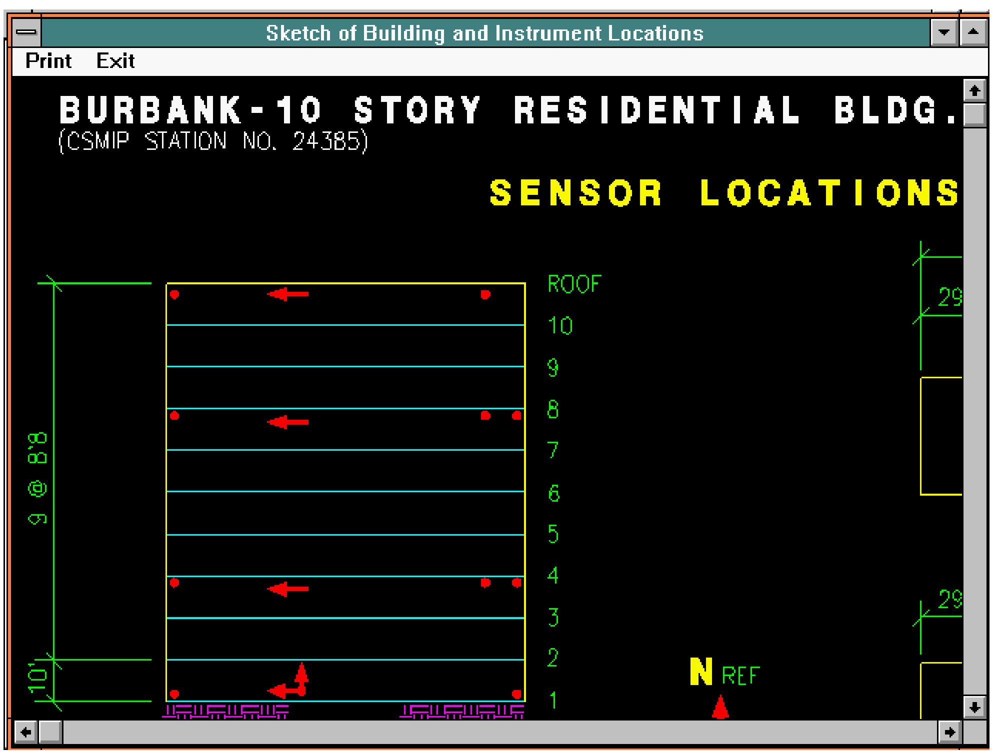


Figure 3-18. Instrument location may be viewed on plan and elevation sketches (horizontal and vertical scroll bars are used to pan and view larger than screen images).

3.8. USING THE RECORD ANALYSIS TOOLS

Click on the Records Analysis tab and four sub-folders appear on screen each providing state-of-the-art tools for viewing and analysis of a single channel or a combination of channels of instrument records. The ***Time Histories*** sub-folder lets you view any portion of an acceleration, velocity, or displacement record (Figure 3-19). You may also look at average of two channels, difference of two channels, average of two channels minus average of another two channels, or difference of two channels minus difference of two other channels (see Figure 3-20 for example). All of this is done by simply selecting the desired channels from the drop-down lists. There is a +/- button at the top right of the Channels frame which you can click to toggle between various average and difference options. This is true for all sub-folders of the Records Analysis folder.

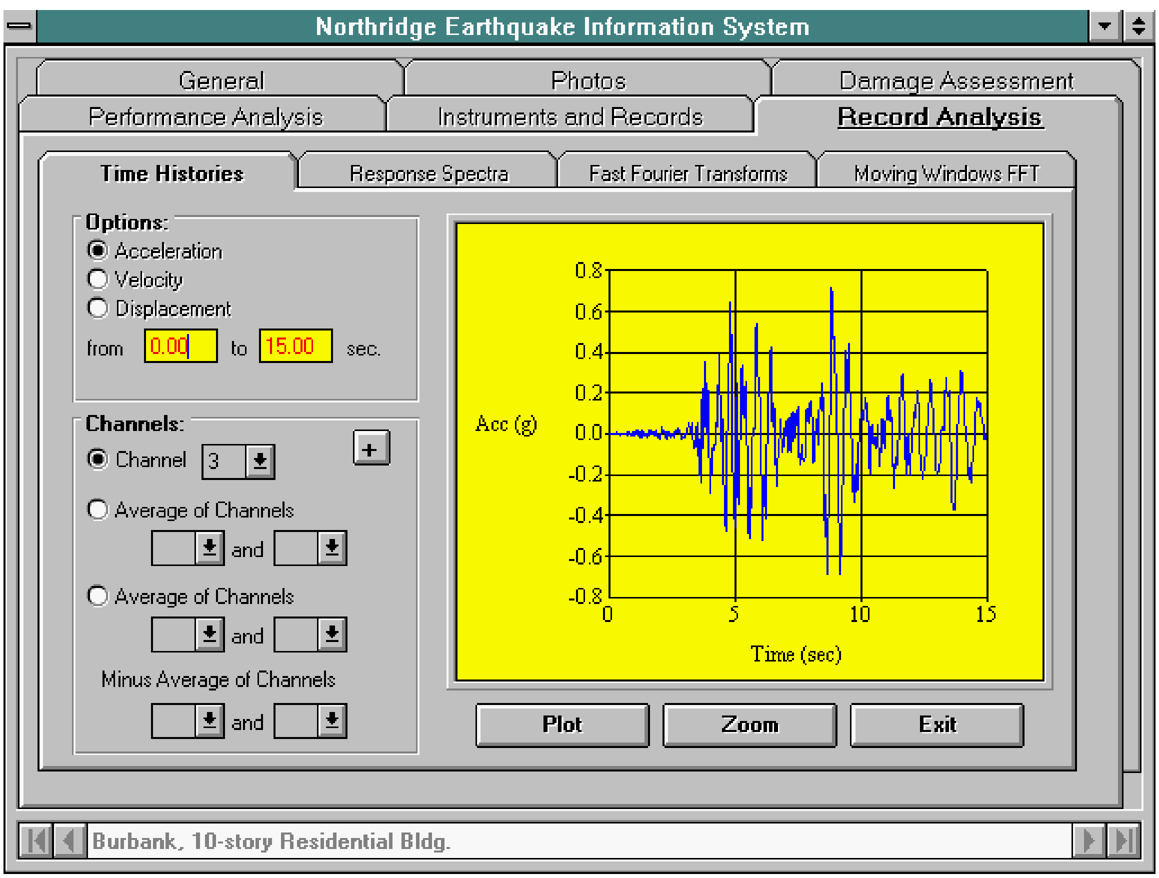


Figure 3-19. A typical view of the Time Histories sub-folder of Record Analysis tool

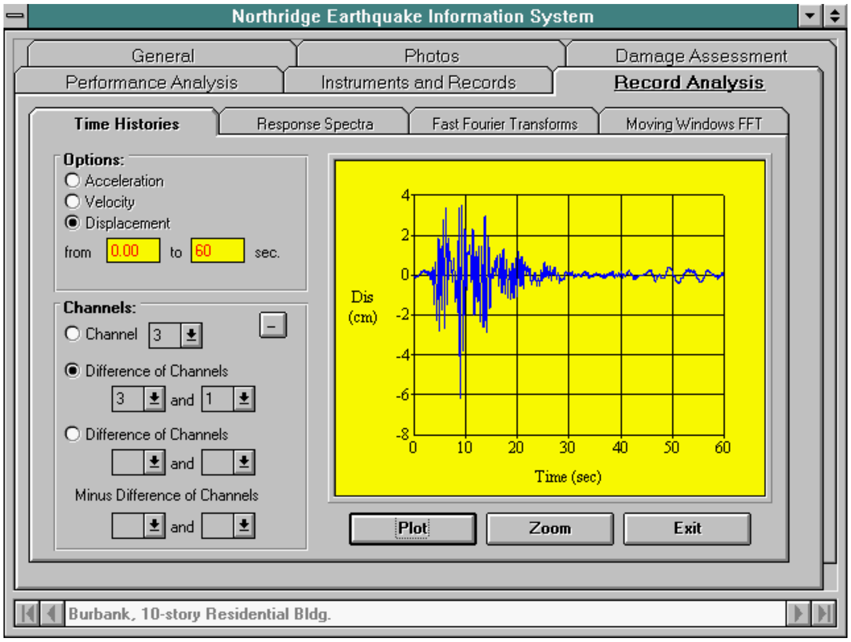


Figure 3-20. Various manipulations may be made online to a channel or a combination of channels. Here the relative displacement of two channels(i.e., roof and base) are viewed.

Click the ***Response Spectra*** tab and you may examine plots of spectral displacements, spectral velocity, spectral acceleration, pseudo-velocity, corresponding to the desired level of damping, or the Fourier amplitude spectrum of any data channel (Figure 3-21).

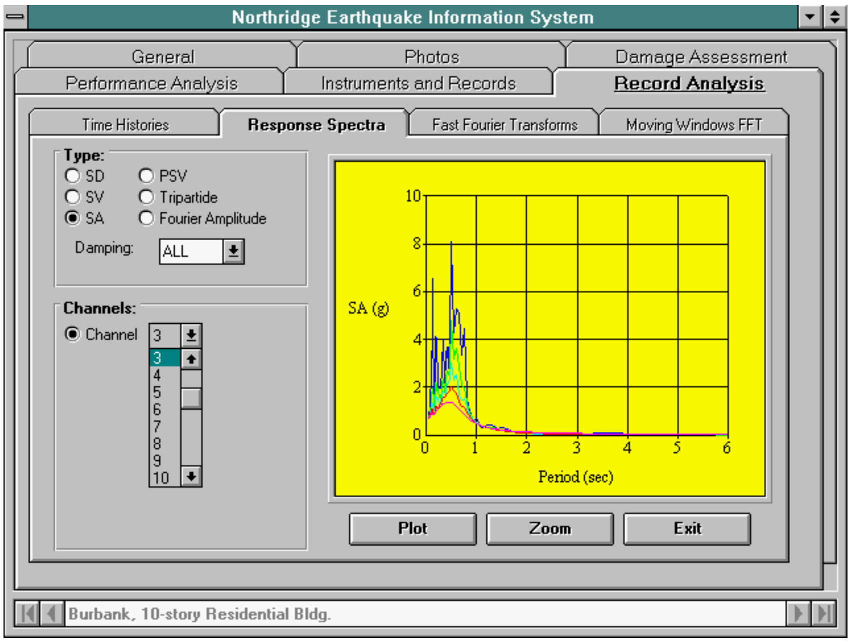


Figure 3-21. The Response Spectra folder provides means of examining various

frequency domain characteristics of a record.

All Record Analysis folders contain a ***Zoom*** button which may be used to enlarge the plots for better viewing (Figure 3-22). Once enlarged, a hard copy of the plot may be obtained via the ***Print*** menu. The ***Fast Fourier Transform*** (FFT) sub-folder may be used to obtain online a variety of FFT amplitude plots for individual records and/or various transfer functions (Figure 3-23).

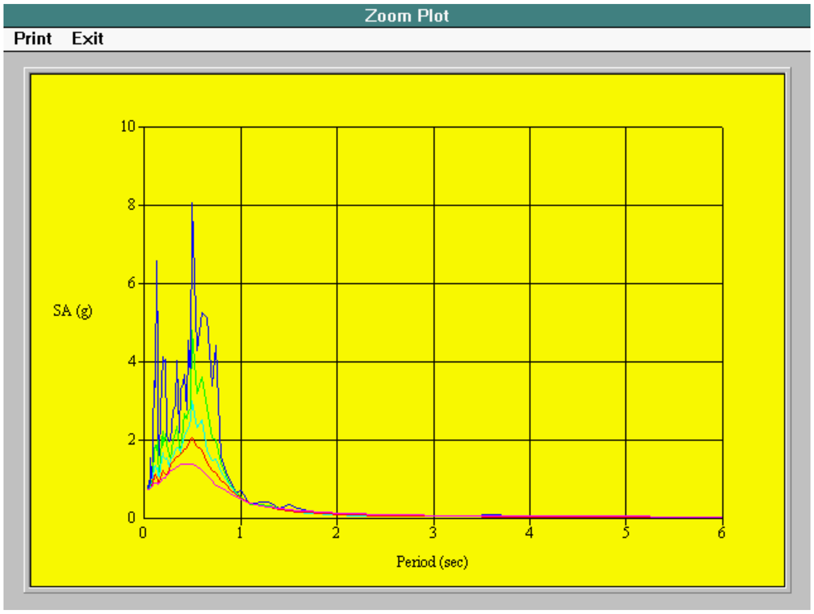


Figure 3-22. The Zoom button may be used to enlarge the plot and obtain hard copies.

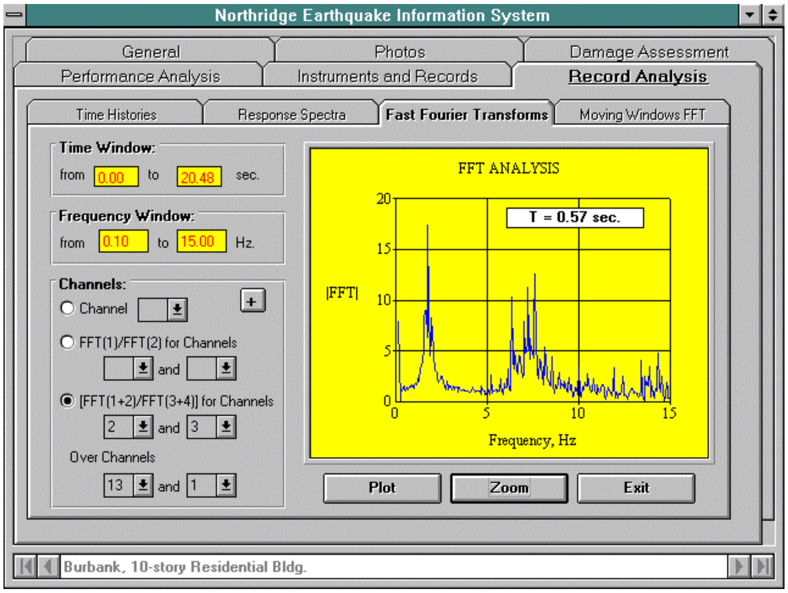


Figure 3-23. A typical online Fast Fourier Transform analysis.

Once the Zoom button is clicked, the FFT phase values are shown side-by-side the amplitude plot (Figure 3-24).

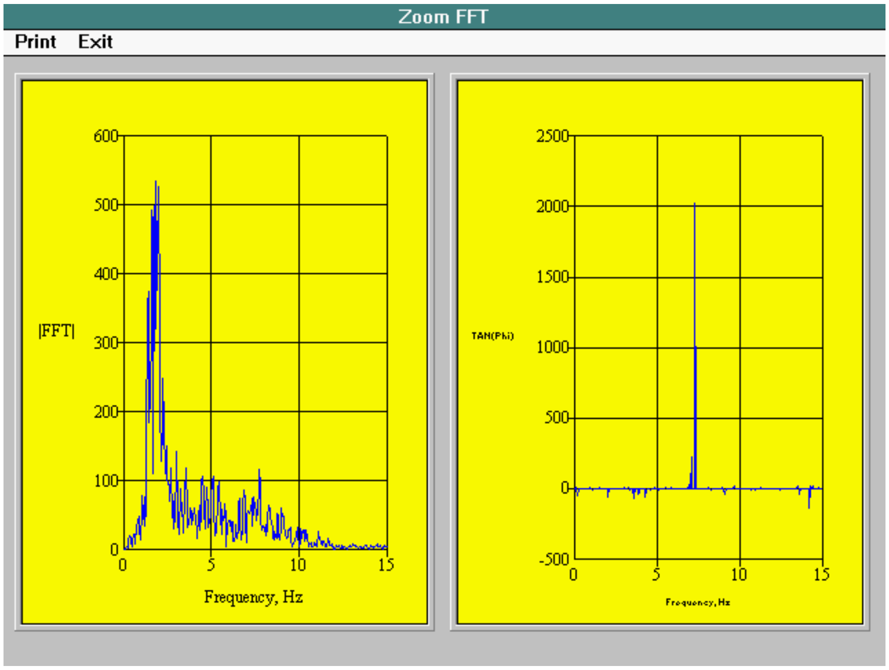


Figure 3-24. Zoomed FFT view shows phase values as well as a print option.

Finally, the ***Moving Windows FFT*** sub-folder permits online dynamic evaluation of frequency characteristics of records and transfer functions as a function of time. You select a time window or a slice of time, and a time increment by which this time window or slice is moved each time from the beginning to the end of earthquake excitations. The information system calculates the most dominant frequency (and hence vibration period) within each selected time window. The end result is a graph showing the changes in dominant period/frequency during the course of an earthquake (Figure 3-25). As we will see later in Chapter 4, such analysis can be very useful in evaluating the performance of buildings and shifts in the natural periods of the structure as a result of earthquake ground motions.

You may also look at average of two channels, transfer function of two channels, transfer function of average of two channels and average of another two channels, or difference of two channels and difference of two other channels. There is a +/- button at the top right of the Channels frame which you can click to toggle between various average and difference options.

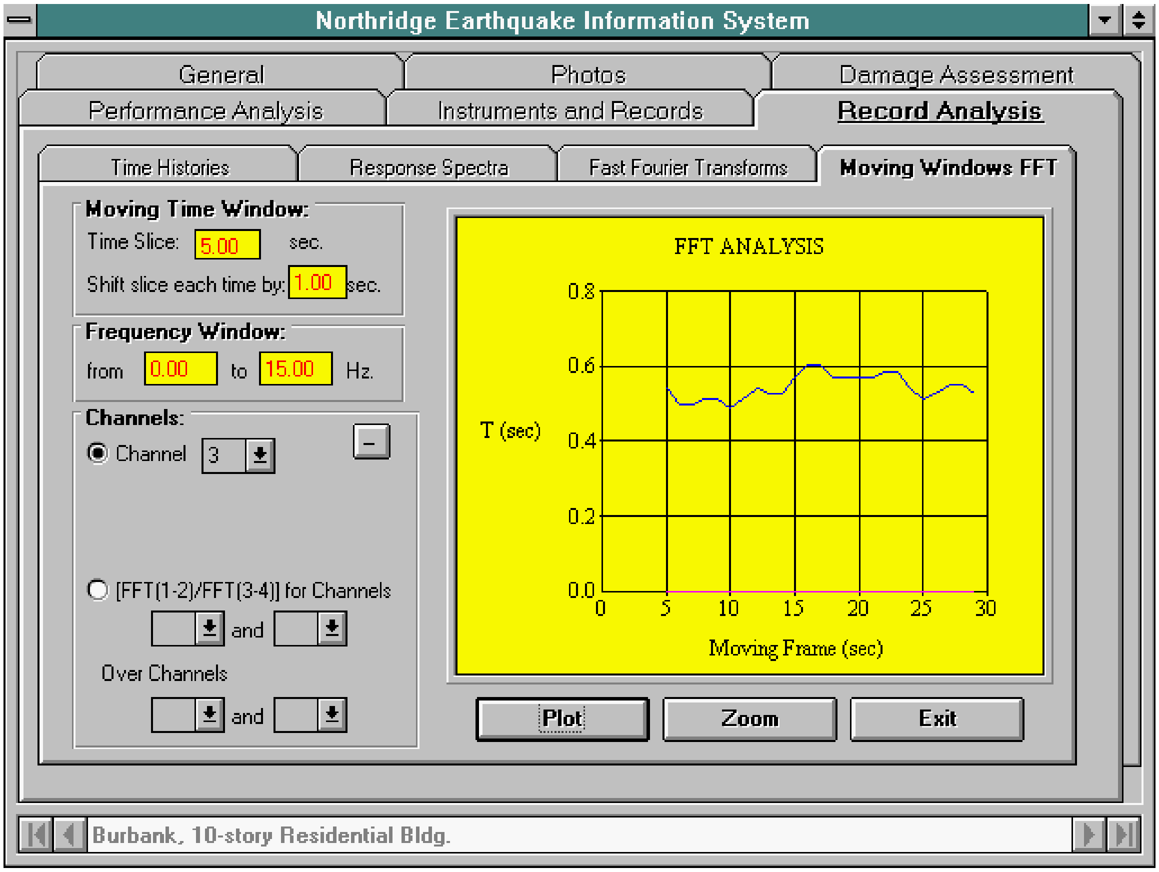


Figure 3-25. Results of a typical moving windows FFT analysis.

4. KEY FEATURES AND RESPONSE CHARACTERISTICS OF BUILDINGS INCLUDED IN THE SMIP INFORMATION SYSTEM

This chapter provides a basic understanding of the key performance and response characteristics for each of the buildings contained in the SMIP information system. Analytical assumptions and approximations used in these evaluations are also presented.

There is much more to subject to evaluation and interpretation on the CD-ROM disc accompanying this report. By highlighting what the author believes are the most important response features of these buildings, it is hoped that the readers will be encouraged to perform their own evaluations and interpretations and hence maximize the potential benefits of the SMIP Information System.

For each building the following information is summarized in this chapter:

1. Number of sensors activated and number of photos contained in the information system

2. A general description of the building and its structural system

3. A comparison of maximum force and displacement response parameters with those recommended by the pertinent edition of the UBC code at the time of design and the 1994 edition of the UBC code.

4. An evaluation of fundamental and predominant periods and comparison with code formulas for estimating building periods.

5. Status of damage or changes in dynamic characteristics of the building, if any.

6. A table summarizing maximum displacements, drift indices, base shears, overturning moments, and the corresponding times of maximum response.

4.1. ANALYTICAL ASSUMPTIONS AND APPROXIMATIONS

The SMIP information system extracts the time histories and response spectra directly from appropriate SMIP Volume 2 and 3 data files. All instrument data files for each building are stored in a compressed file named "SMIPDATA.ZIP". The program extracts, decompresses, utilizes, and discards on-the-fly the decompressed files it needs for any given operation.

The transformation from the time domain to the frequency domain is based on the *Fourier Transform* defined as

$$S_x(f) = \int_{-\infty}^{\infty} x(t)\, e^{-j2\pi ft}\, dt \tag{4-1}$$

where $x(t)$ is the time domain representation of the signal x (i.e., the sensor time history); $S_x(f)$ is the frequency domain representation of the signal x *and* $j = \sqrt{-1}$.

Since the sensor time histories are given at distinct intervals (i.e., 50 or 100 data points per second), numerical integration techniques need to be used

$$S_x(m\,\Delta f) = \int_{-\infty}^{\infty} x(t)\, e^{-j2\pi ft}\, dt \tag{4-2}$$

where $m = 0,\ \pm 1,\ \pm 2$ etc., Δf is the frequency spacing of the lines and Δt is the time interval between samples

Furthermore, since we can not evaluate integrals from minus to plus infinity, we must limit the transform to a finite time interval and hence we can rewrite Eq. (4-2) as

$$S_x(m\,\Delta f) = \Delta t \sum_{n=0}^{N-1} x(n\,\Delta t)\, e^{-j2\pi m\,\Delta f n\,\Delta t} \tag{4-3}$$

which can be re-written as the *Discrete Fourier Transform (DFT)*, *S'x*

$$S'_x(m\,\Delta f) = \frac{T}{N} \sum_{n=0}^{N-1} x(n\,\Delta t)\, e^{-j 2\pi m n / N} \tag{4-4}$$

Fast Fourier Transforms (FFT) are algorithms for computing DFT with reduced amount of computational effort. Our program uses the *Danielson-Lanczos* or *bit reversal* technique for computing the FFT of time series. FFT algorithms, however, require that the number of data points (*N*) be a multiple of 2. Our information system automatically enforces this rule. The maximum number of data points which is accepted by the program for each FFT analysis is internally set to 2^{10} or 1024 points. The program offers a simple weighted-average smoothing filter in the form of

$$S'_x(i) = \left[S'_x(i-1) + 2\,S'_x(i) + S'_x(i+1) \right] \Big/ 4 \tag{4-5}$$

which may be repeated as many time as necessary. Care must be taken in applying this smoothing filter since over-smoothing may result in elimination of some important frequency information.

To distinguish the frequency response characteristics of the structure from those represented in the input motions, the program has facilities for calculating *Transfer Function H(f)* as

$$H(f) = \frac{S_y(f)\, S_x^*(f)}{S_x(f)\, S_x^*(f)} \tag{4-6}$$

where * indicates the complex conjugate of the function, and $S_x(f)$ *and* $S_y(f)$ correspond to the input and response channels, respectively. The amplitude of *H(f)* is what is shown in the FFT plot window. Both amplitude and phase values are displayed when the *zoom* button is pressed.

When multiple channels are selected for calculation of FFT of average or difference functions, the averaging or deducting is performed in the time-domain prior to application of FFT calculations.

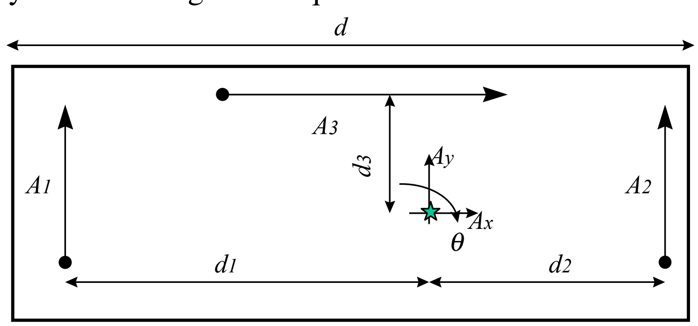
When *Moving Windows FFT* calculations are requested, the program calculates FFT once for the selected time-slice and successively shifts the time slice by the specified time-shift reporting the frequency which corresponds to maximum FFT amplitude for the time-slice considered. Notice that various building modes might be predominant at different times during the response of the building to strong ground motion. When that is the case, the moving windows FFT plot will show jumps between the periods that correspond to these modes. For example, see the difference in the relative amplitude of the two modes depicted in Figure 4-1a and 4-1b where the time slices are moved by about one second. If this is repeated for the entire duration of strong ground motion, the moving windows plot of Figure 4-1c results which simply indicates that there are two predominant modes in the response of this building in the direction considered. If one needs to track a period which corresponds to a particular mode, then the frequency window must be narrowed to eliminate the jump between different modes in the moving windows FFT plots.

The maximum response tables presented in this chapter and in the *Performance Analysis* folders of the information system utilize as many sensors throughout a building as possible and are created using certain simplifying assumptions. Locations of selected sensors for each building, normalized weight of the floors and rules for calculating direct response parameters as well as differential (torsional) parameters for each building are contained in a *key* file named SENSORS.KEY. Contents of a representative key file is shown on Plate 4-1.

Direct response values are calculated using average instantaneous sensor displacements or accelerations of unidirectional sensors located on each elevation. Differential values produced by the information system are based on the difference of the same values for the unidirectional sensors located furthest from each other for each elevation.

While this simple approach is accurate enough for capturing direct response parameters, it may underestimate torsional response substantially for cases where the distances of the unidirectional sensors considered from the floor center of mass is significantly different. You may change this by modifying the building's SENSORS.KEY file and deleting the MAXRESP.KEY file from the building's directory. Once this is done, pressing the small button on the upper right hand side of the *Performance Analysis* folder of the building will regenerate performance response parameters according to your specifications contained in the modified SENSORS.KEY file. Assuming that floor diaphragms are perfectly rigid in their own plane, an accurate estimate of torsional response may be obtained using the following formulation:

If A_1, A_2 and A_3 are the values of response measured by three sensors on the same floor, and d_1, d_2 and d_3 are the respective distance of these sensors from the center of mass, then A_x, A_y and A_θ (the response of the center of mass in the x , y, and t directions) are related to A_1, A_2, and A_3 by the following set of equations:



$$\begin{aligned} A_1 &= A_y + d_1\theta \\ A_2 &= A_y + d_2\theta \\ A_3 &= A_x + d_3\theta \end{aligned} \qquad (4\text{-}7)$$

In matrix form,

$$\begin{Bmatrix} A_1 \\ A_2 \\ A_3 \end{Bmatrix} = \begin{bmatrix} 0 & 1 & d_1/d \\ 0 & 1 & -d_1/d \\ 1 & 0 & d_3/d \end{bmatrix} \begin{Bmatrix} A_x \\ A_y \\ \theta\, d \end{Bmatrix} \text{ or } \begin{Bmatrix} A_x \\ A_y \\ \theta\, d \end{Bmatrix} = \begin{bmatrix} 0 & 1 & d_1/d \\ 0 & 1 & -d_1/d \\ 1 & 0 & d_3/d \end{bmatrix}^{-1} \begin{Bmatrix} A_1 \\ A_2 \\ A_3 \end{Bmatrix} \qquad (4\text{-}8)$$

To calculate story forces at each time step, instantaneous accelerations in the desired directions are multiplied by the normalized masses to obtain story forces as a fraction of total building weight.

In order to estimate response parameters for floors in between the floors housing sensors (nodes) a cubical spline interpolation technique is used. A cubic spline is a third-order curve applied to subsets of pre-defined h and $f(h)$ values(i.e., sensor elevations and response parameters, respectively). A cubic spline results in smooth transition between data points. This property is particularly desirable for conventional buildings but it is not suited for base isolated buildings where enough number of floors above the isolation base are not instrumented. Given a complete third order polynomial in the form

$$f(h) = ah^3 + bh^2 + ch + d \tag{4-9}$$

the coefficients a, b, c and d are determined by forcing the $f(h)$ values and their derivatives be equal at each node when calculated from adjacent sub-interval polynomials. The computation of spline coefficients for each-sub interval (the distance between two adjacent nodes) involves the solution of a tri-diagonal system of linear equations. Once the interval i containing the h value is determined, the value of the interpolated function is determined from

$$f_i(h) = a_i(h - h_i)^3 + b_i(h - h_i)^2 + c_i(h - h_i) + d_i \tag{4-10}$$

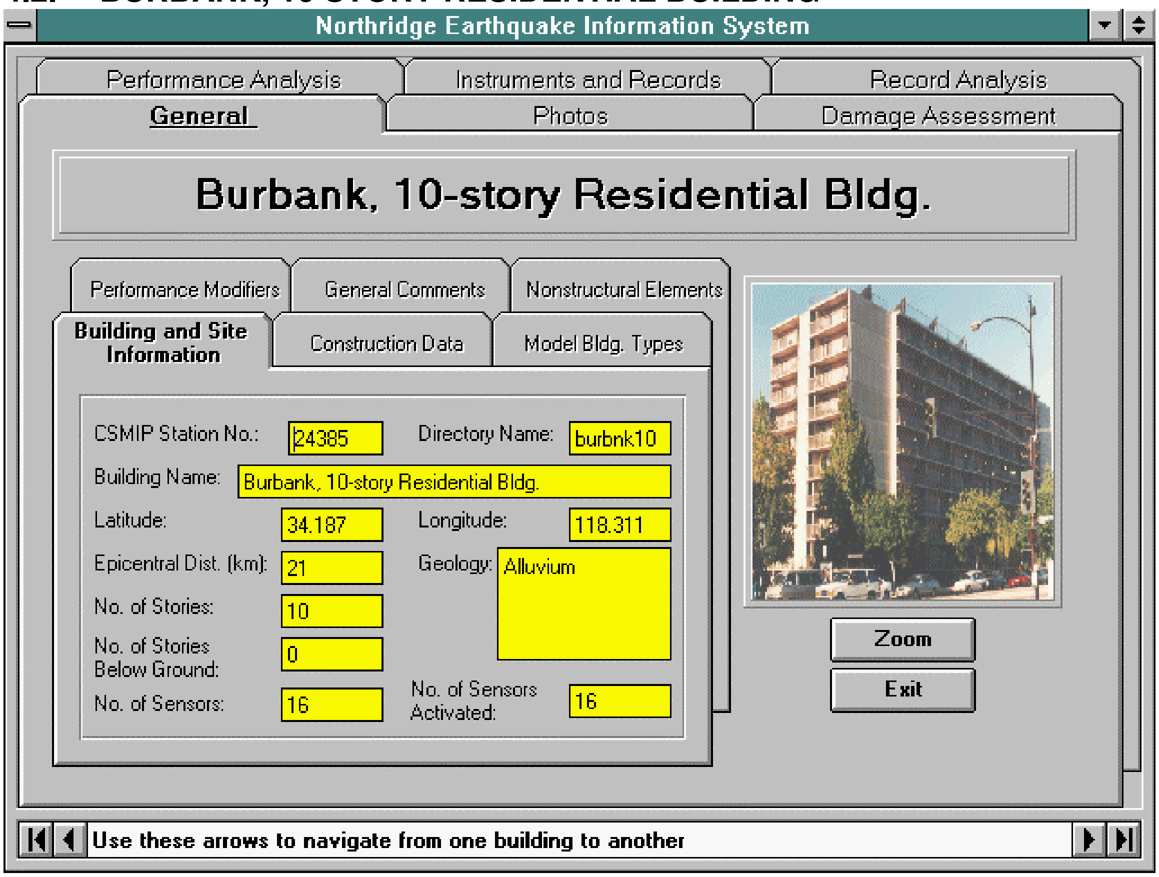
This operation is performed on-the fly by the information system for any requested time instant during the strong ground motion. Response values are also calculated for each time step for each building and the maximum response values and their time of occurrence for each building are saved in a file named MAXRESP.KEY. These maximum response values are also reported for each building in this chapter.

```
1234567890123456789012345678901234567890123456789012345678901234567890123456789012
0         1         2         3         4         5         6         7
No. of Floors in the Bldg.= 10
Floor         H (ft)     %mass      %MMI
    0           0.0     0.0       0.0
    1          10.0     0.118     0.118
    2          18.7     0.100     0.100
    3          27.3     0.100     0.100
    4          36.0     0.100     0.100
    5          44.7     0.100     0.100
    6          53.3     0.100     0.100
    7          62.0     0.100     0.100
    8          70.7     0.100     0.100
    9          79.3     0.100     0.100
   10          88.0     0.082     0.082

No. of Elevations with Sensors=        4
   Height #1 =              0.0
   # of Sensors =             3
   Torsional Arm =        210.0
   Sensor:               Chan01
   Sensor:               Chan13
   Sensor:               Chan16
   N-S Direct Rule:                0.50      0.50      0.00
   E-W Direct Rule:                0.00      0.00      1.00
   Torsional Disp. Rule:           1.00     -1.00      0.00
--
   Height #2 =             36.0
   # of Sensors =             3
   Torsional Arm =        181.0
   Sensor:               Chan09
   Sensor:               Chan07
   Sensor:               Chan12
   N-S Direct Rule:                0.50      0.50      0.00
   E-W Direct Rule:                0.00      0.00      1.00
   Torsional Disp. Rule:           1.00     -1.00      0.00
--
   Height #3 =             70.7
   # of Sensors =             3
   Torsional Arm =        181.0
   Sensor:               Chan06
   Sensor:               Chan04
   Sensor:               Chan11
   N-S Direct Rule:                0.50      0.50      0.00
   E-W Direct Rule:                0.00      0.00      1.00
   Torsional Disp. Rule:           1.00     -1.00      0.00
--
   Height #4 =             88.0
   # of Sensors =             3
   Torsional Arm =        152.0
   Sensor:               Chan03
   Sensor:               Chan02
   Sensor:               Chan10
   N-S Direct Rule:                0.50      0.50      0.00
   E-W Direct Rule:                0.00      0.00      1.00
   Torsional Disp. Rule:           1.00     -1.00      0.00
[END]
```

Plate 4-1. A sample SENSOR.KEY file contents.

4.2. BURBANK, 10 STORY RESIDENTIAL BUILDING



No of Sensors Activated: 16 No. of Photos in the Database: 12

This 10 story 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the base (Channel 1, N-S) and at the roof (Channel 2, N-S) are 0.34g and 0.77g, respectively. The peak velocity at the roof is about 63 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-1. The

0.34W maximum base shear apparently experienced by the building in the N-S direction significantly exceeds both the 1973 and 1994 UBC strength design base shears of 1.4x0.10W= 0.14W for UBC-73, and 1.4x0.145W =0.20W for UBC-94 (see Appendix B for backup calculations). As documented in the information system, however, no sign of structural damage were observed during our inspections. See information system photos which document minor damage to the nonstructural equipment on the roof (one such photo is also shown in Figures 3-8 and 3-9 of this report).

TABLE 4-1. Response Summary for Burbank 10-story Residential Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear	N-S	8.88	33.63
(% Total Weight)	E-W	5.28	20.39
	DIFF	9.40	14.32
Overturning Moment	N-S	8.90	1518
(% Total Weight x feet)	E-W	5.90	804
	DIFF	4.70	522
Roof Lateral Displacement	N-S	8.86	6.19 (0.0023)*
Relative to the Base (cm)	E-W	5.94	2.76 (0.0010)*
	DIFF	4.54	1.43 (0.0053)*

* Overall drift index values are shown in brackets

Our FFT analysis of the recorded data indicates a N-S fundamental period of about 0.57 seconds (see Figure 3-23), an E-W fundamental period of about 0.62 seconds (Figure 4-1).. The fundamental periods implied by the sensor data compare well with the UBC-94

Method A estimate of 0.58 seconds. The UBC-73 estimate of the periods at 0.30 sec. and 0.51 sec. while not poor are not as good (see Appendix B for backup calculations).

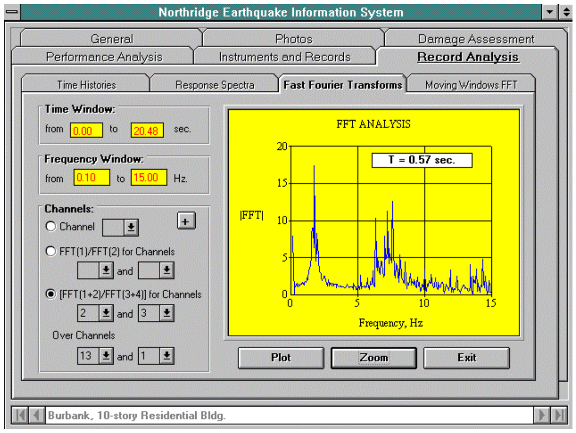


Figure 4-1(a). An FFT analysis for the E-W direction response (from 0 to 20.48 sec.)

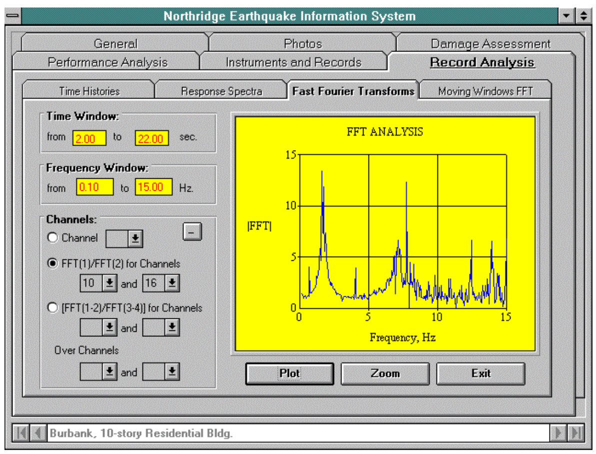


Figure 4-1(b). An FFT analysis for the E-W direction response (from 2 to 22 seconds).

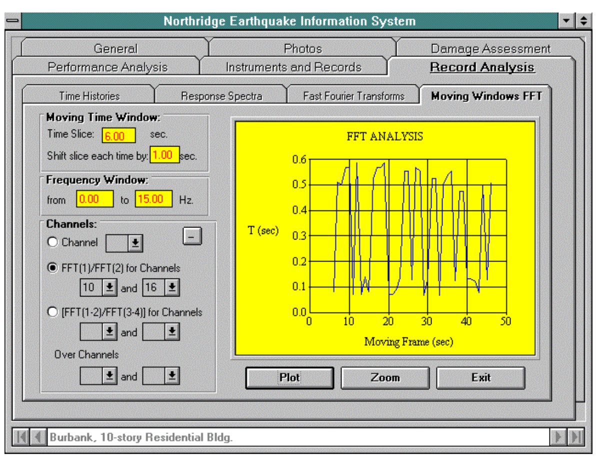
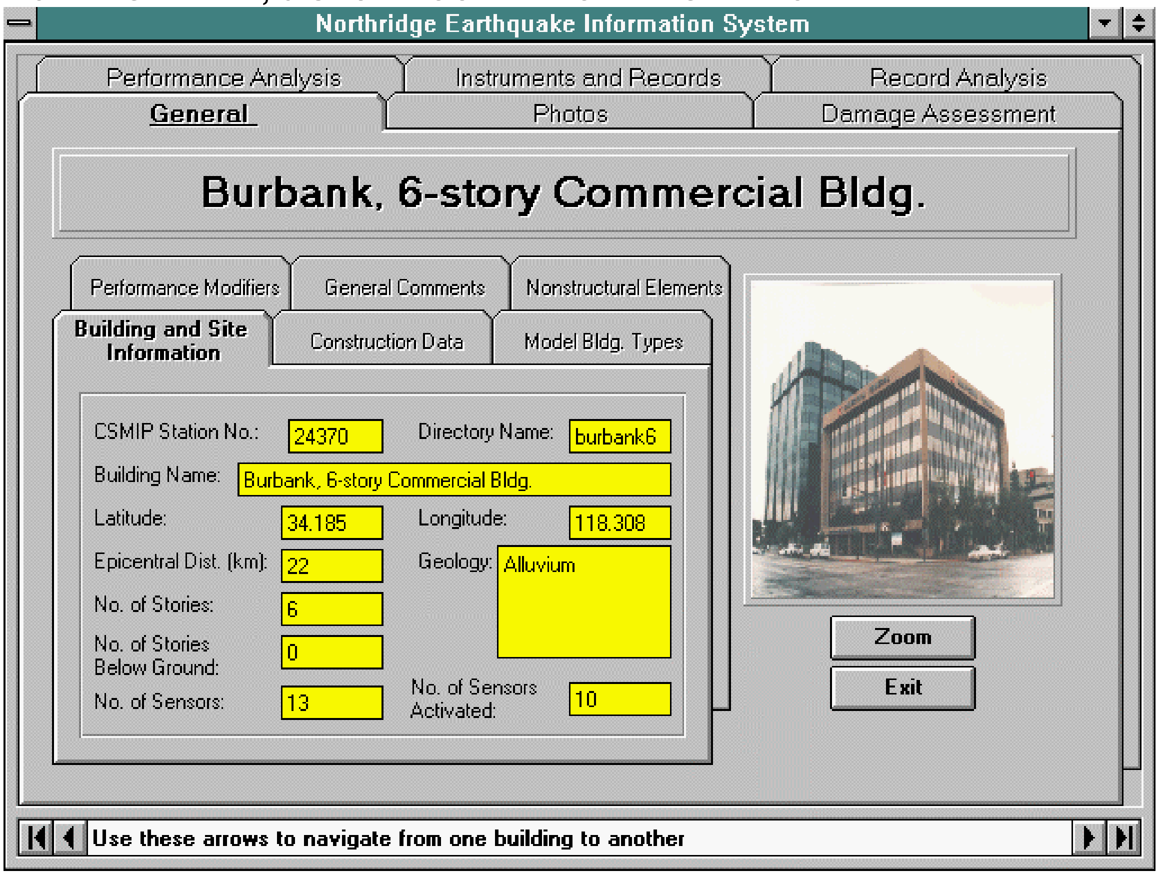


Figure 4-1(c). Predominant periods as a function of time (moving windows FFT).

4.3. BURBANK, 6-STORY COMMERCIAL BUILDING



No of Sensors Activated: 13 No. of Photos in the Database: 13

This 6 story 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal acceleration recorded at the base (Channel 9, E-W) and at the roof (Channel 3, E-W) are 0.36g and 0.47g, respectively. The peak velocity at the roof is about 48 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-2. The

0.22W maximum base shear apparently experienced by the building in the E-W direction significantly exceeds both the 1976 and 1994 UBC strength design base shears of 1.4x0.07W= 0.10W for UBC-76, and 1.4x0.052W =0.07W for UBC-94 (see Appendix B for backup calculations). At the time of writing this report, we are not aware of any inspections performed on the beam-column joints of this building. Our visit to the building and interviews conducted revealed no sign of structural damage. Most of the content damage was caused by tearing of a small water pipe at the penthouse which resulted in flooding of the building. The anchorage of a roof mechanical equipment was also damaged (Figure 4-2). Consult the information system for more photos and observations.

TABLE 4-2. Response Summary for Burbank 6-Story Commercial Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear	N-S	14.98	12.37
(% Total Weight)	E-W	5.10	22.07
	DIFF	5.96	7.11
Overturning Moment	N-S	14.96	546
(% Total Weight x feet)	E-W	8.84	807
	DIFF	5.78	231
Roof Lateral Displacement	N-S	16.30	9.63 (0.0038)*
Relative to the Base (cm)	E-W	15.78	9.68 (0.0039)*
	DIFF	13.02	1.54 (0.0006)*

* Overall drift index values are shown in brackets

Our FFT analysis of the recorded data indicate fundamental periods of about 1.28 seconds in both N-S and E-W directions (see Figure 4-3 for example). These periods are twice the 0.6 second value suggested by UBC-76. The UBC-94 Method A provides a much better period estimation at 0.95 (see Appendix B for backup calculations).

Figure 4-2. Mechanical equipment damage at the roof.

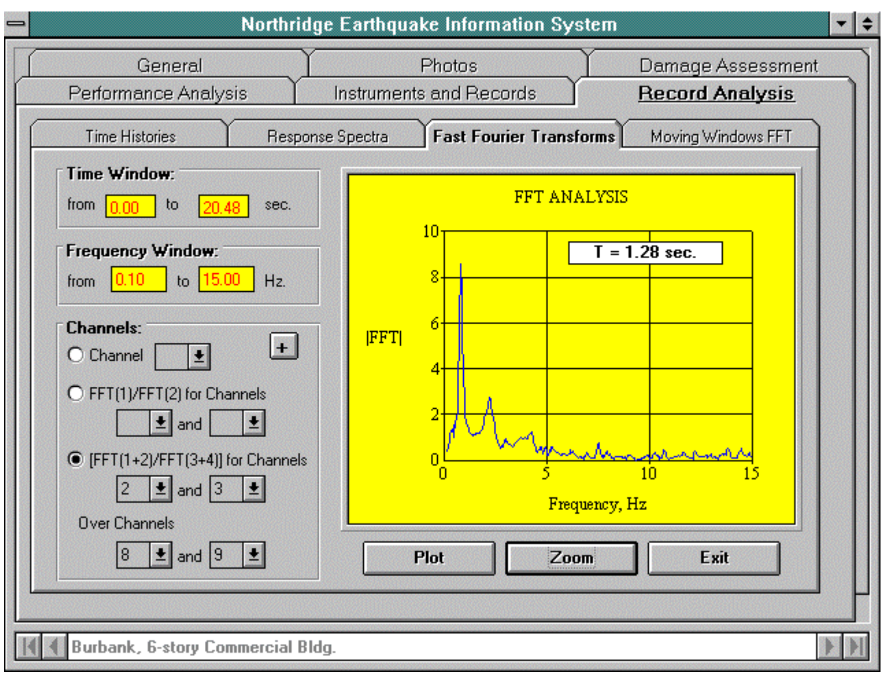
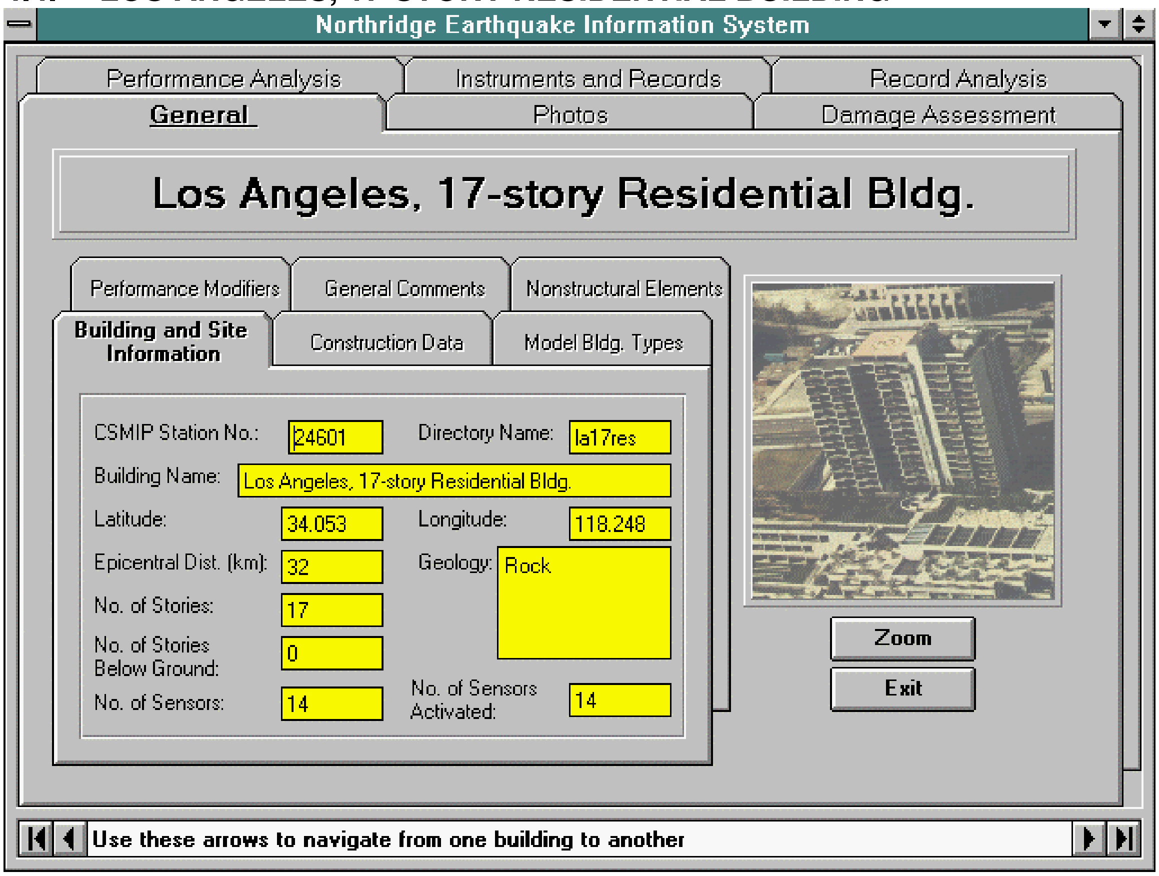


Figure 4-3. An FFT analysis for the N-S direction response.

4.4. LOS ANGELES, 17-STORY RESIDENTIAL BUILDING



No of Sensors Activated: 14 No. of Photos in the Database: 1

This 17 story 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 (Channel 5, N-S) and at the roof (Channel 13, E-W) are 0.26g and 0.58g, respectively. The peak velocity at the roof is about 48 cm/sec. . Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

Performance analysis calculations for this building are summarized in Table 4-3. The

0.18W maximum base shear apparently experienced by the building is equal to the working stress base shear used for design of shear wall buildings in the recent editions of the UBC code. The overall drift experienced by the building is also very modest.

TABLE 4-3. Response Summary for Los Angeles 17-Story Residential Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	16.12	15.89
	E-W	15.38	16.56
	DIFF	15.01	16.15
Overturning Moment (% Total Weight x feet)	N-S	14.88	1301
	E-W	14.90	1582
	DIFF	14.91	1539
Roof Lateral Displacement Relative to the Base (cm)	N-S	14.90	8.55 (0.0020)*
	E-W	14.91	9.61 (0.0021)*
	DIFF	14.92	9.75 (0.0022)*

* Overall drift index values are shown in brackets

Moving windows FFT analysis for the N-S direction implies initial and final fundamental periods of about 0.80 seconds which rise to about 1.20 seconds in the midst of strong ground motion (Figure 4-4). Similar analysis indicates a fundamental period of about 1.0 seconds in the E-W direction (Figure 4-5). Higher modes have a significant contribution to seismic response of this building as may be seen in Figure 4-5.

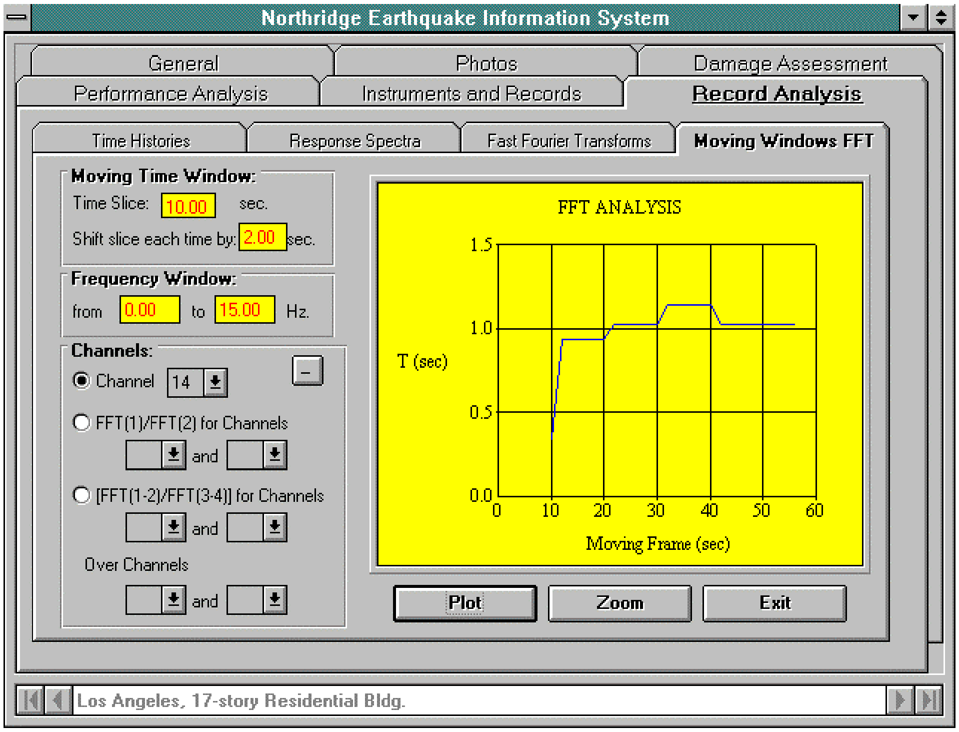


Figure 4-4. A moving windows FFT analysis for the N-S direction response.

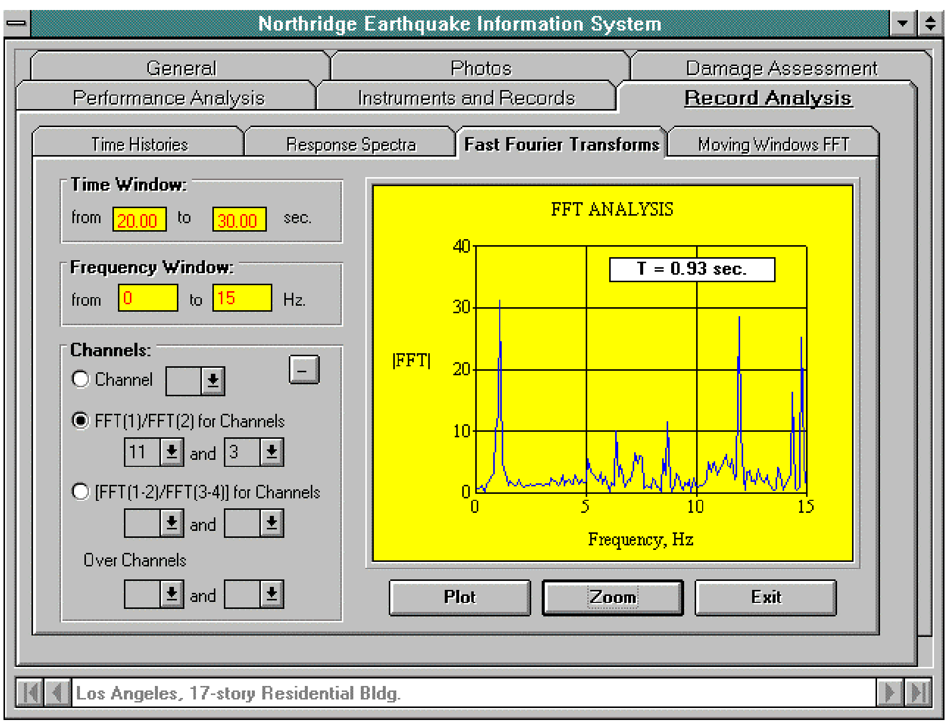
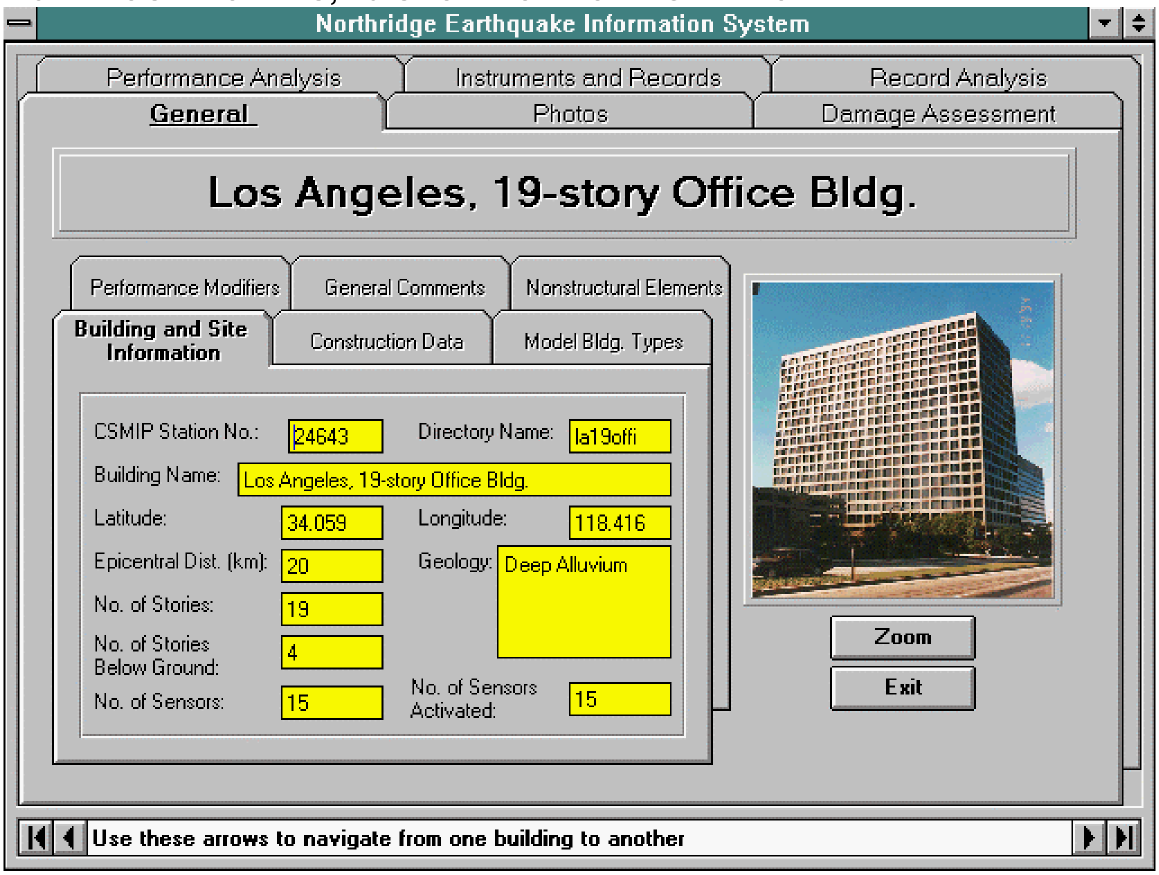


Figure 4-5. An FFT analysis for the N-S direction response.

4.5. LOS ANGELES, 19-STORY OFFICE BUILDING



No of Sensors Activated: 15 No. of Photos in the Database: 8

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 (Channel 18, E-W), ground floor (Channel 7, E-W) and roof (Channel 14, N-S) are 0.32g, 0.53g and 0.65g, respectively. The peak velocity at the roof is about 65 cm/sec. . Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

Performance analysis calculations for this building are summarized in Table 4-1. The 0.34W and 0.22W maximum base shears apparently experienced by the building are two to four times larger than the strength design values suggested by the 1967 and 1994 UBC strength design base shears of 1.4x.055W= 0.08W for UBC-67, and 1.4x0.083W =0.12W for UBC-94 (see Appendix B for backup calculations). As documented in the information system, the building suffered from buckling of some braces at the penthouse level (Figure 4-6). No other significant structural damage were observed. The mechanical equipment at the roof did not suffer any significant damage.

TABLE 4-4. Response Summary for Los Angeles 19-Story Office Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S E-W DIFF	15.27 14.50 16.81	33.79 22.70 21.58
Overturning Moment (% Total Weight x feet)	N-S E-W DIFF	15.27 14.49 16.81	3781 2257 2867
Roof Lateral Displacement Relative to the Base (cm)	N-S E-W DIFF	21.31 29.37 18.15	27.82 (0.0034)* 32.52 (0.0039)* 5.01 (0.0006)*

* Overall drift index values are shown in brackets

FFT analyses of the recorded data indicate very significant participation of higher modes in seismic response of this building. These analyses (Figures 4-7 and 4-8) indicate a N-S fundamental period of about 2.6 seconds and an E-W fundamental period of about 3.41 seconds. Both these periods are significantly larger than the UBC-67 estimates of 0.76 and 1.9 seconds and UBC-94 estimates of 1.24 and 2.33 seconds, respectively. Here again the UBC-94 estimates are closer to the observed ones than those predicted by the earlier editions of the code. As may be seen by examining the Record Analysis options of the information system, one would make grave mistakes if he/she evaluates the seismic response of this building based solely on the fundamental periods.

Figure 4-6. Buckled brace at the penthouse.

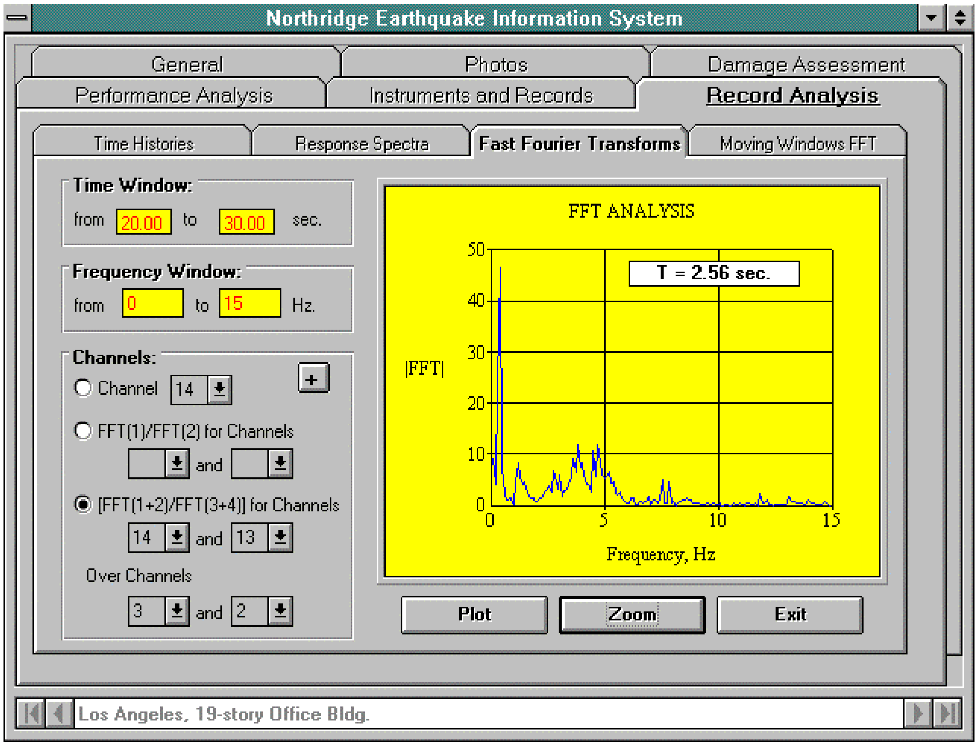


Figure 4-7. An FFT analysis for the N-S direction response.

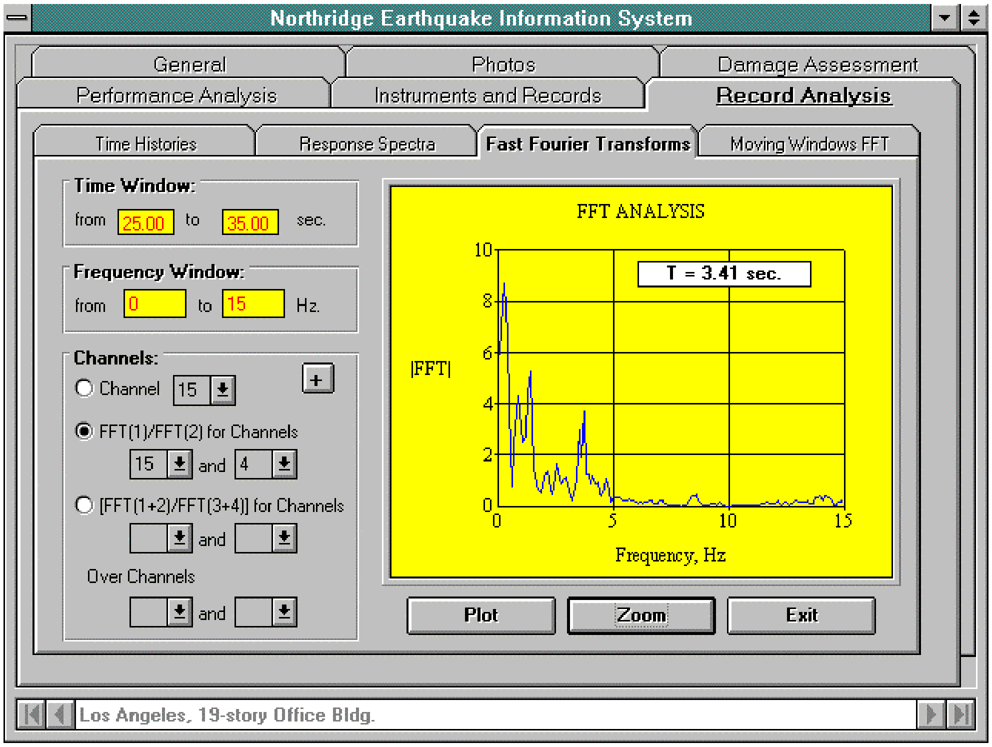
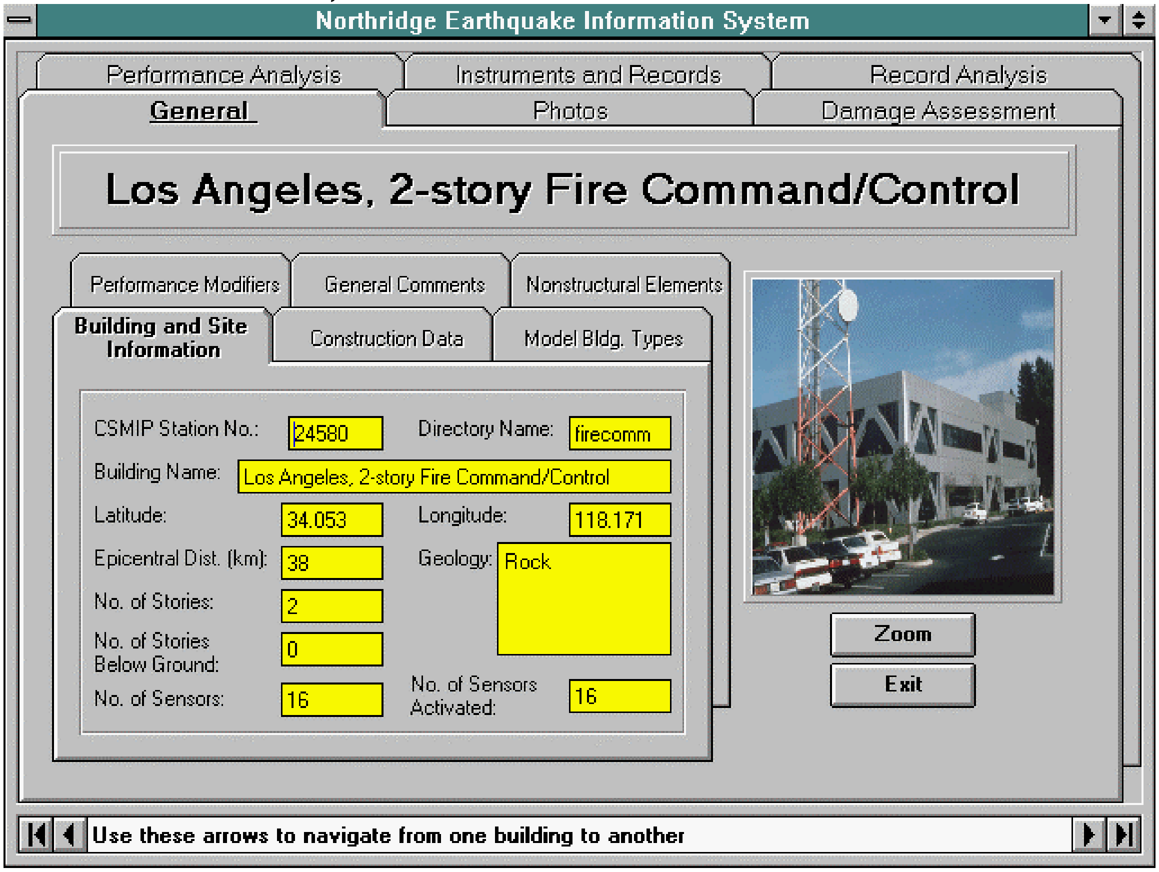


Figure 4-8. An FFT analysis for the E-W direction response.

4.6. LOS ANGELES, 2-STORY FIRE COMMAND/CONTROL BUILDING



No of Sensors Activated: 16 No. of Photos in the Database: 4

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

In the E-W direction, the largest peak horizontal accelerations recorded below the isolation plane (Channel 6), at the floor directly above the isolation plane (Channel 11) and the roof (Channel 15) are 0.22g, 0.35g and 0.77g, respectively. At first glance it seems that this building did not behave as an isolated system in this direction. The reasons, including some photos, are presented in the information system. In summary, a construction mistake of pouring concrete with mesh reinforcement over a segment of the isolation pit prevented the building from behaving as an isolated system in this direction up to about 16 seconds into the ground motion (see Figure 4-9). At this time, the building is separated from the covered pit and behaves very much as an isolated system. In the N-S direction, the behavior is much closer to what is expected from an isolated system and the largest peak horizontal accelerations vary from 0.18g at the base (Channel 4) to 0.07g directly above the isolation system (Channel 8) and 0.09g at the roof (Channel 13).

TABLE 4-5. Response Summary for 2-Story Fire Command Control Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	16.26	3.99
	E-W	15.85	7.26
	DIFF	13.75	11.18
Overturning Moment (% Total Weight x feet)	N-S	16.25	66
	E-W	12.35	107
	DIFF	13.79	223
Roof Lateral Displacement Relative to the Ground (cm)	N-S	16.87	2.48 ( )*
	E-W	18.78	3.53 ( )*
	DIFF	16.11	1.62 ( )*

* Displacements relative to the top of isolators are shown in brackets

Performance analysis calculations for this building are summarized in Table 4-5. The 0.04W and 0.07W maximum base shears apparently experienced by the building are small compared to 0.40 to 0.5g design base shear values (see Bachman, Gomes and Chang, 1990).

Our FFT analyses of the response in the E-W direction indicates an essentially fixed-base response with a fundamental period of about 0.20 seconds during the first 15 seconds of ground motion (Figure 4-10). After about 15 seconds into the ground motion the building exhibits a fundamental period of about 1.14 seconds in the E-W direction (Figure 4-11). A moving windows FFT analysis presents a more clear picture of the response in the E-W direction where higher mode participation in the response can be clearly seen (Figure 4-12.) The N-S response although showing some level of higher mode participation due to coupling, is much more simple and exhibits a fundamental period is about 1.28 seconds (Figure 4-13).

Figure 4-9. Tiles over isolation pit after the earthquake.

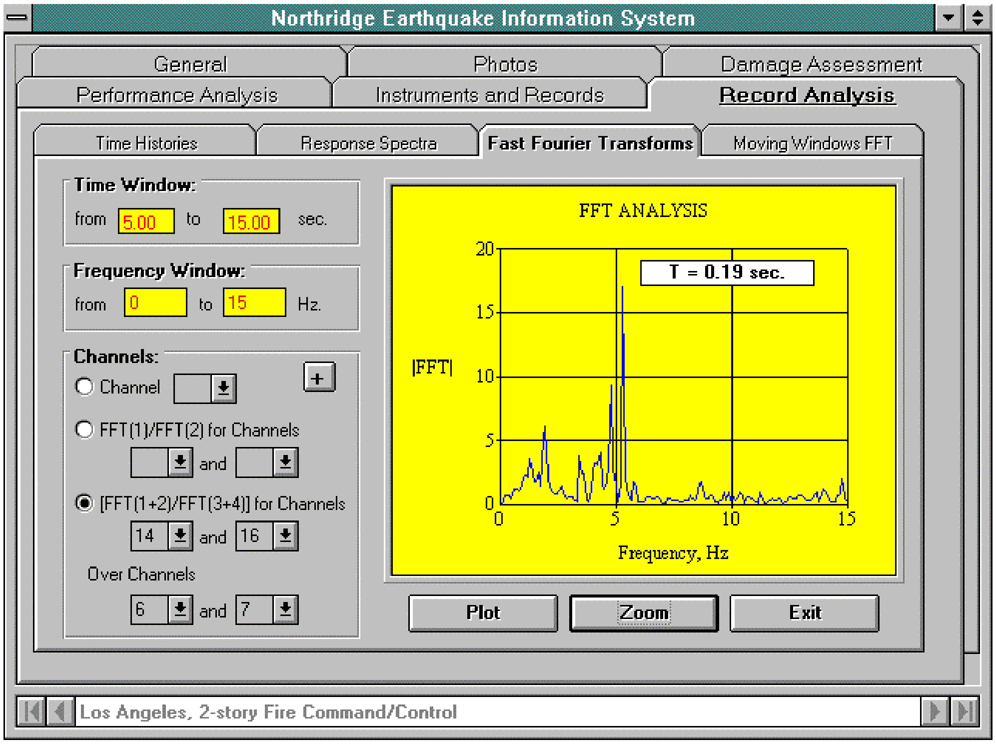


Figure 4-10. FFT of E-W response up to 15 seconds into the strong ground motion.

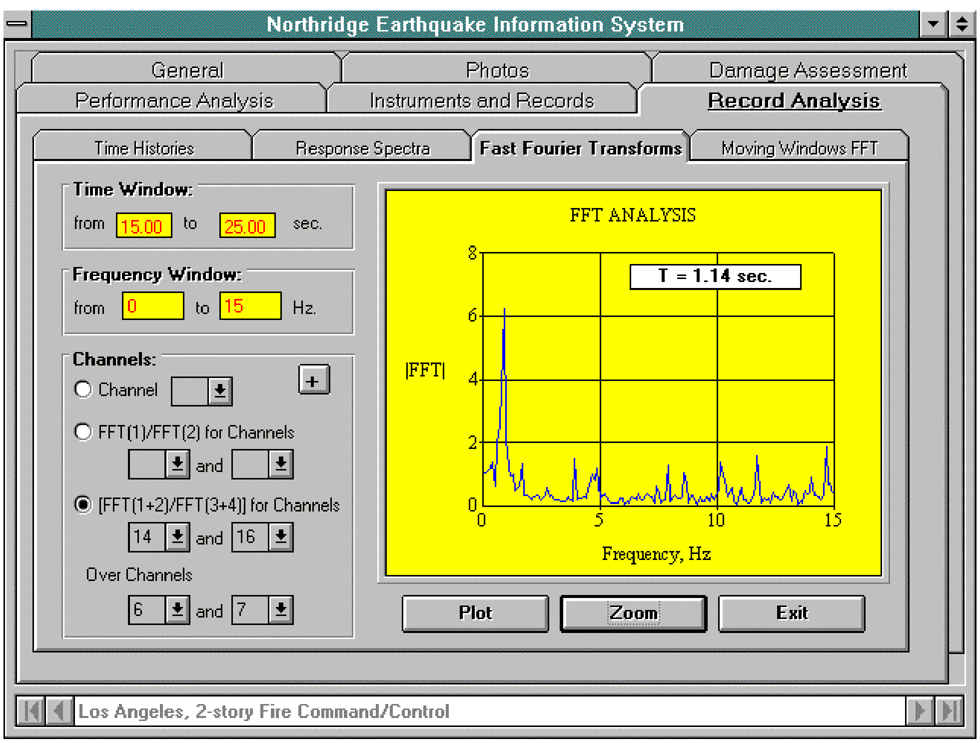


Figure 4-11. FFT for the E-W direction response (15-25 sec. into strong ground motion).

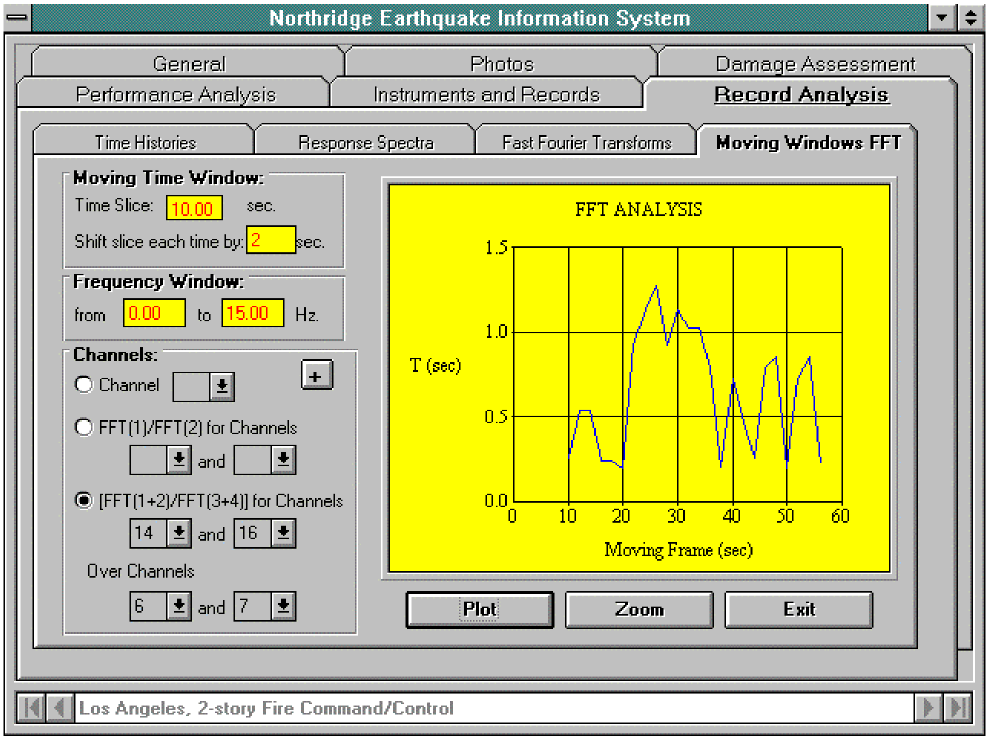


Figure 4-12. A moving window FFT analysis for the E-W direction response.

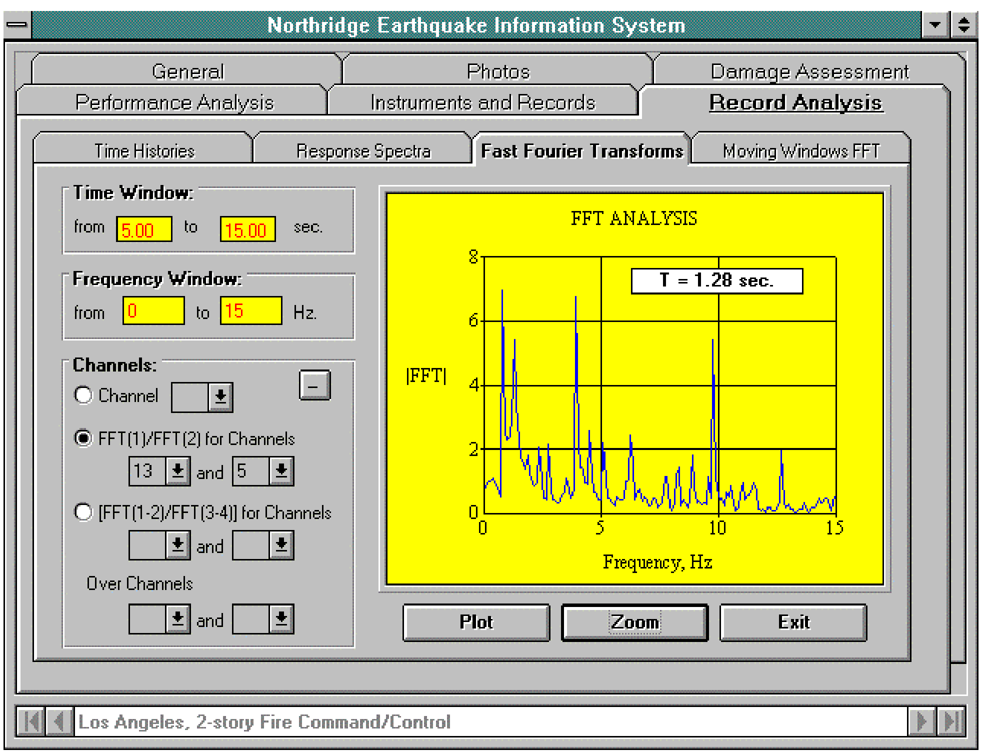
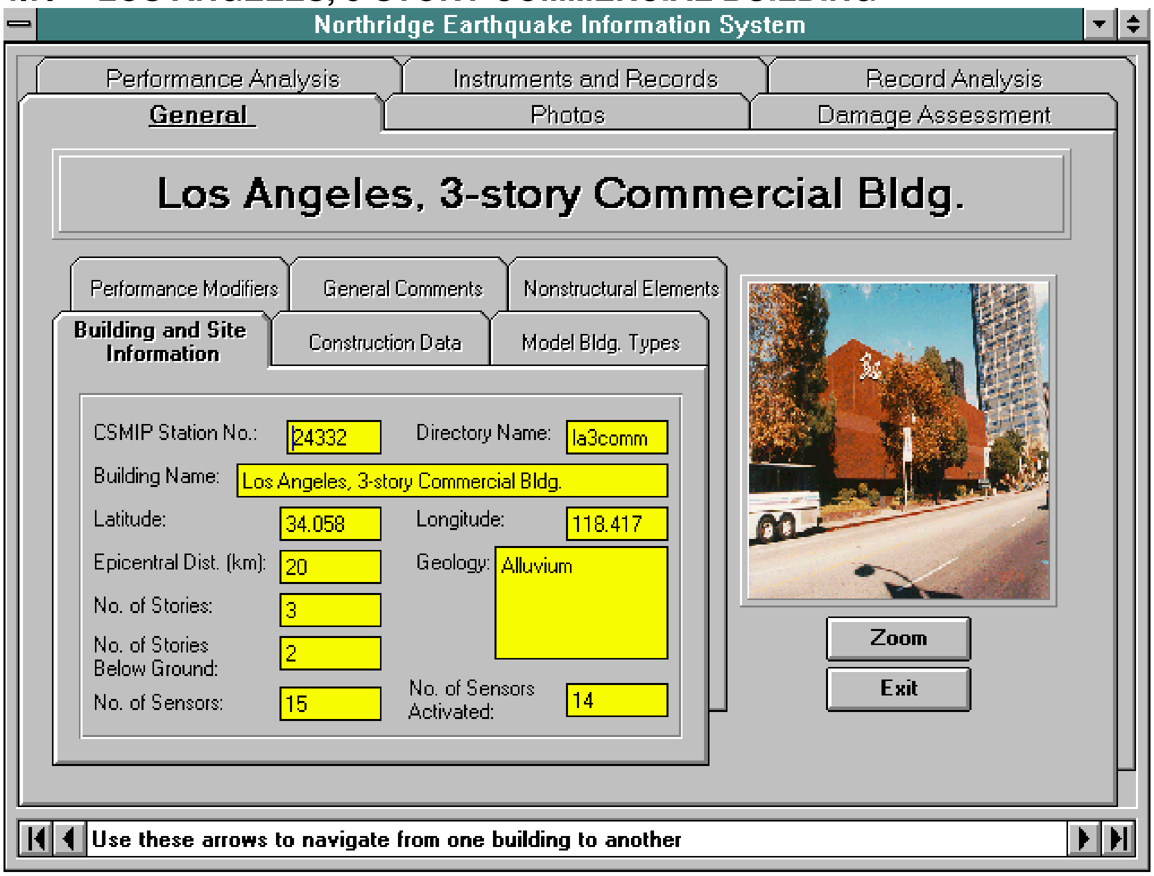


Figure 4-13. An FFT analysis for the N-S direction response.

4.7. LOS ANGELES, 3-STORY COMMERCIAL BUILDING



No of Sensors Activated: 15 **No. of Photos in the Database: 23**

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 consists 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 the parking floors. The foundation system consists of spread footings and drilled bell caissons. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the base (Channel 13, E-W, and Channel 15, N-S) is 0.33g. At the roof, Channel 2 (E-W) recorded a peak horizontal acceleration of 0.97g and a peak velocity of 57 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-6. The 0.49W maximum base shear apparently experienced by this building is very high compared to any code of practice. The 1973 and 1994 UBC strength design base shears of 1.4x0.125W= 0.175W for UBC-73, and 1.4x0.192W =0.27W for UBC-94 are significantly less. Notice that the UBC-94 base shear is 53% larger than the UBC-73 value which was in effect when the building was designed. The overall drift index of more than 1% is rather large for a braced frame system.

TABLE 4-6. Response Summary for Los Angles 3-Story Commercial Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S E-W DIFF	9.24 10.34 9.88	48.73 42.62 27.35
Overturning Moment (% Total Weight x feet)	N-S E-W DIFF	9.84 10.08 9.22	890 953 441
Roof Lateral Displacement Relative to the Base (cm)	N-S E-W DIFF	9.86 10.36 9.90	5.65 (0.0111)* 4.12 (0.0081)* 0.96 (0.0019)*

* Overall drift index values are shown in brackets

In contrast with such large base shears and drifts, and a recorded roof acceleration of almost 100%g, as documented in the information system no sign of structural damage were observed during our inspections. More interesting is the fact that the roof mounted equipment experienced very little damage, if any. The building experienced heavy content damage and some nonstructural damage to the hung ceilings, lights, and flooring. Consult the Information System for more facts.

Our FFT analysis of the recorded data indicates a N-S fundamental period of 0.55 seconds and an E-W fundamental period of 0.51 seconds (Figures 4-14 and 4-15). These periods as implied by the sensor data compare well with the UBC-94 Method A estimate of 0.4 seconds. However, they are very far from the UBC-73 estimates of 0.15 and 0.17 seconds. (see Appendix B for backup calculations).

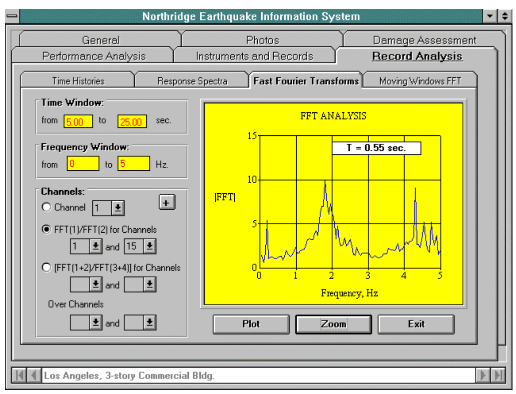


Figure 4-14. An FFT analysis for the N-S direction response.

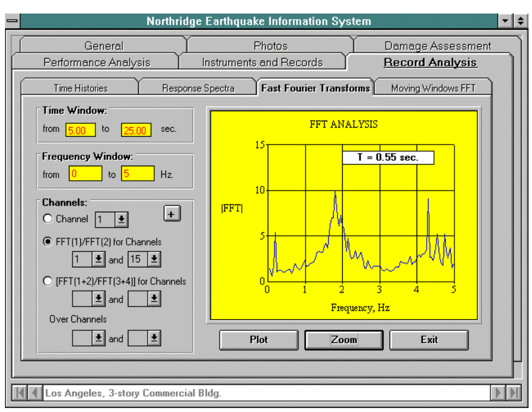
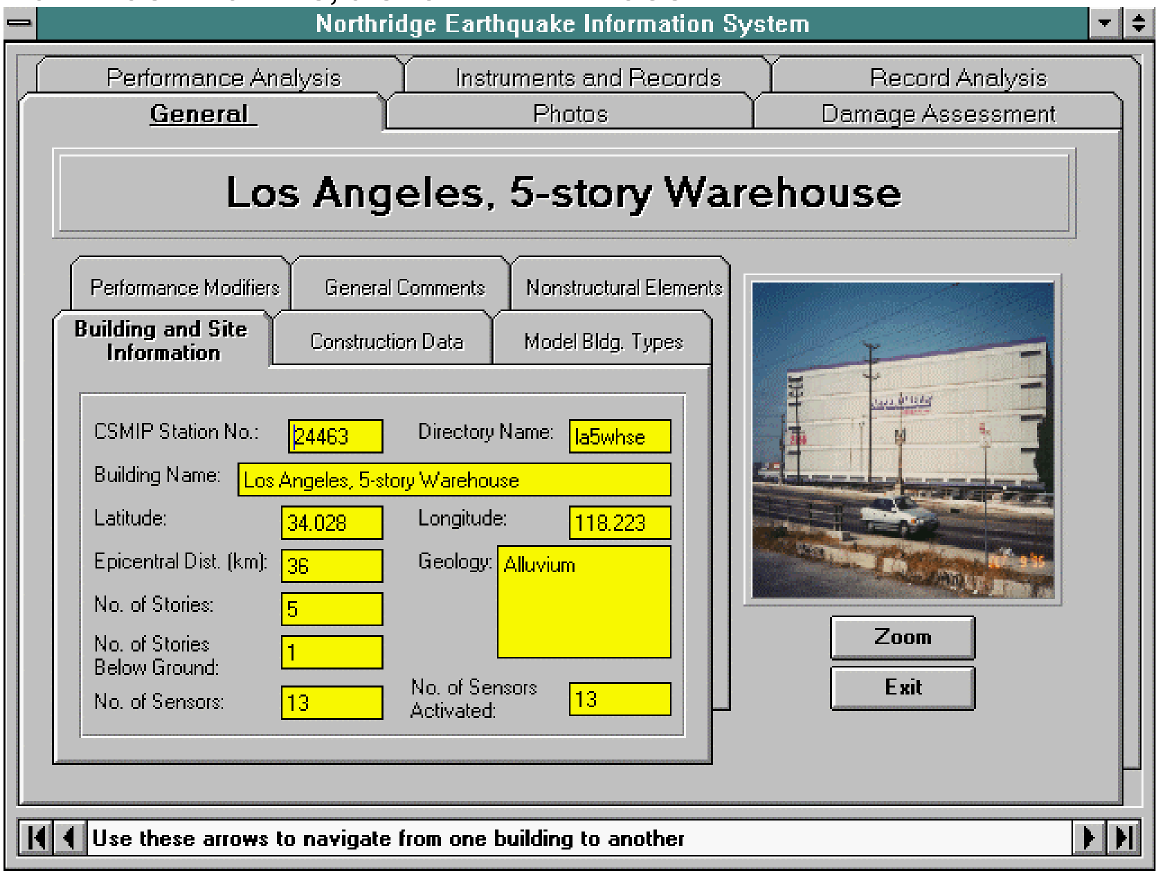


Figure 4-15. An FFT analysis for the E-W direction response.

4.8. LOS ANGELES, 5-STORY WAREHOUSE



No of Sensors Activated: 13 No. of Photos in the Database: 1

This 5-story reinforced concrete building was constructed in 1970 with perimeter ductile concrete frames acting as its lateral system. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the base (Channel 9, N-S) and at the roof (Channel 3, N-S) are 0.25g and 0.28g, respectively. The peak velocity at the roof is about 34 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-1. The 0.17W maximum base shear apparently experienced by the building in the N-S direction is very close to the UBC-94 strength design base shear of 1.4x0.124W= 0.17W (R_w =6). The UBC-67 strength design base shear of 1.4x0.059W =0.08W, however, is significantly less (see Appendix B for backup calculations).

TABLE 4-7. Response Summary for Los Angeles 5-Story Warehouse.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear	N-S	11.70	10.65
(% Total Weight)	E-W	11.50	17.36
	DIFF	12.18	9.64
Overturning Moment	N-S	11.06	674
(% Total Weight x feet)	E-W	11.52	1028
	DIFF	12.18	615
Roof Lateral Displacement	N-S	22.58	7.11 (0.0020)*
Relative to the Base (cm)	E-W	16.38	5.94 (0.0016)*
	DIFF	12.18	2.19 (0.0006)*

* Overall drift index values are shown in brackets

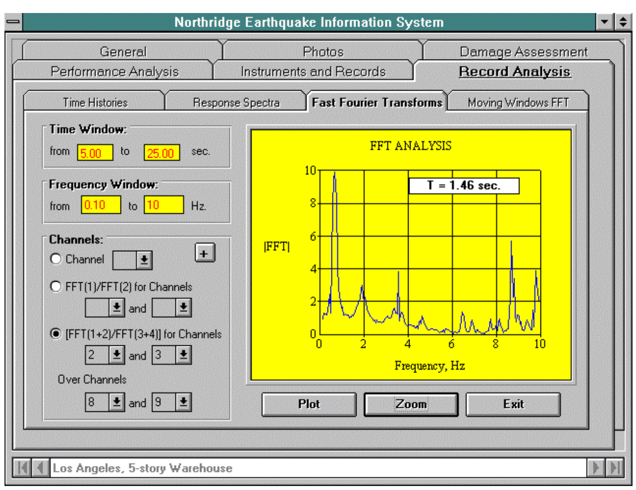


Figure 4-16. An FFT analysis for the N-S direction response.

Our FFT analysis of the recorded data indicates N-S and E-W fundamental periods of about 1.46 and 1.37 seconds, respectively (Figures 4-16 and 4-17). The first torsional period is about 1.0 seconds (Figure 4-18). The UBC-94 estimated period is 0.73 seconds while the UBC-67 estimated period is 0.60 seconds. (see Appendix B for backup calculations).

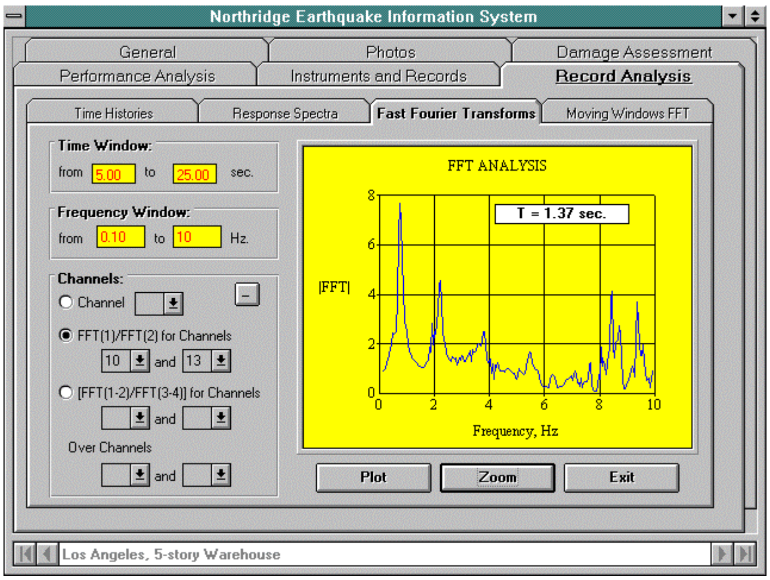


Figure 4-17. An FFT analysis for the E-W direction response.

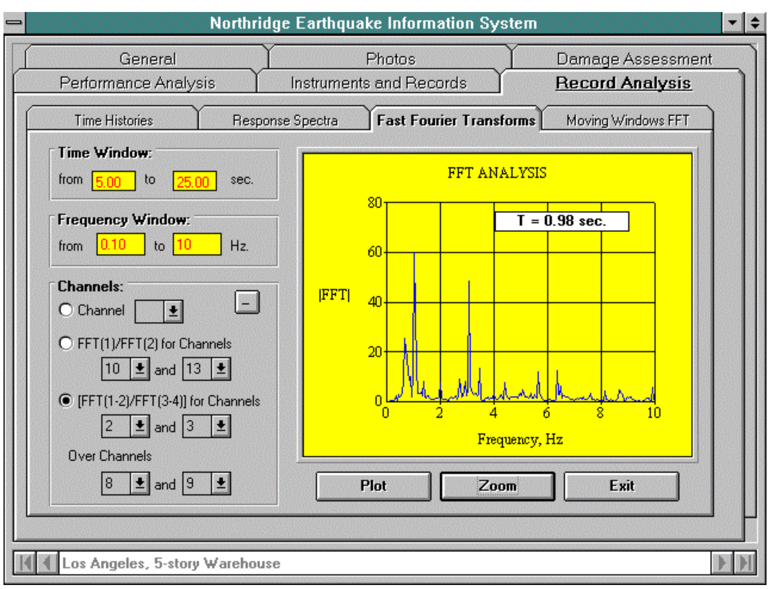
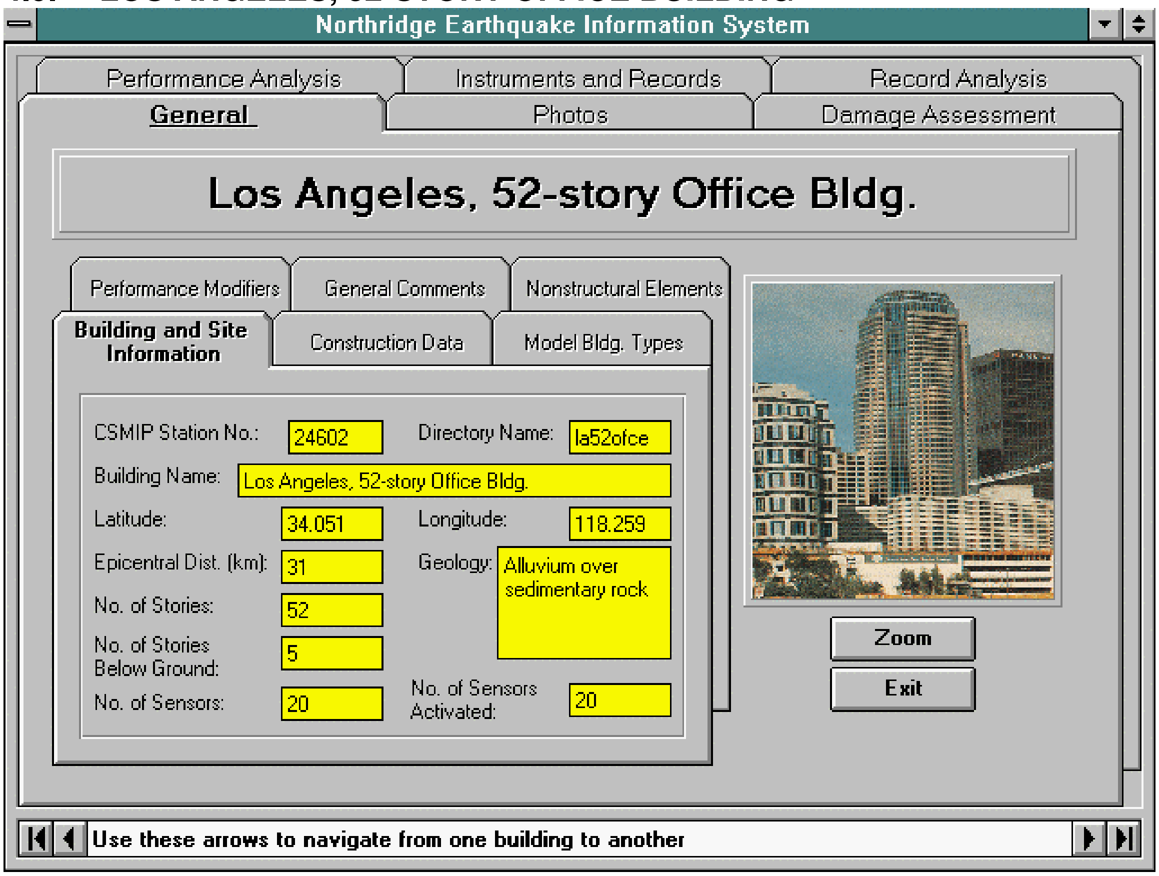


Figure 4-18. An FFT analysis for the torsional response.

4.9. LOS ANGELES, 52-STORY OFFICE BUILDING



No of Sensors Activated: 20 No. of Photos in the Database: 1

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the basement (Channel 4, N-S) and at the roof (Channel 20, N-S) are 0.15g and 0.41g, respectively. The peak velocity at the roof is about 40 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-8. Torsional components are not calculated because of the existence of only one sensor in each direction at the roof. The maximum base shear experienced during the Northridge earthquake is estimated at about 0.09W. The overall drift experienced is very low. No damage was reported for this building.

TABLE 4-8. Response Summary for Los Angeles 52-story Office Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S E-W	14.95 14.94	7.30 8.75
Overturning Moment (% Total Weight x feet)	N-S E-W	16.45 16.64	1320 1907
Roof Lateral Displacement Relative to the Base (cm)	N-S E-W	40.12 29.62	14.08 (0.0006)* 23.72 (0.0011)*

* Overall drift index values are shown in brackets.

Fourier amplitude spectra analyses of the roof instruments (see Figure 4-19) indicate fundamental translational periods of about 6.0 seconds in both direction. These long fundamental periods get barely excited however. The predominant modes of response correspond to periods between 1.6 and 2.0 seconds (Figure 4-20).

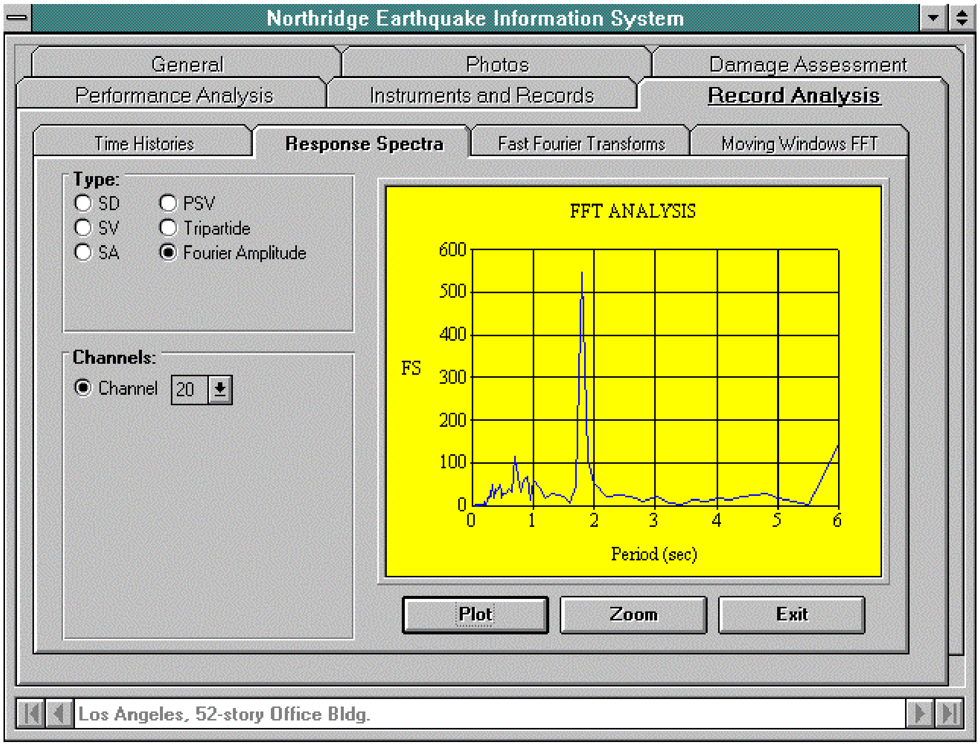


Figure 4-19. Fourier spectrum indicating a fundamental building period at 6.0 sec..

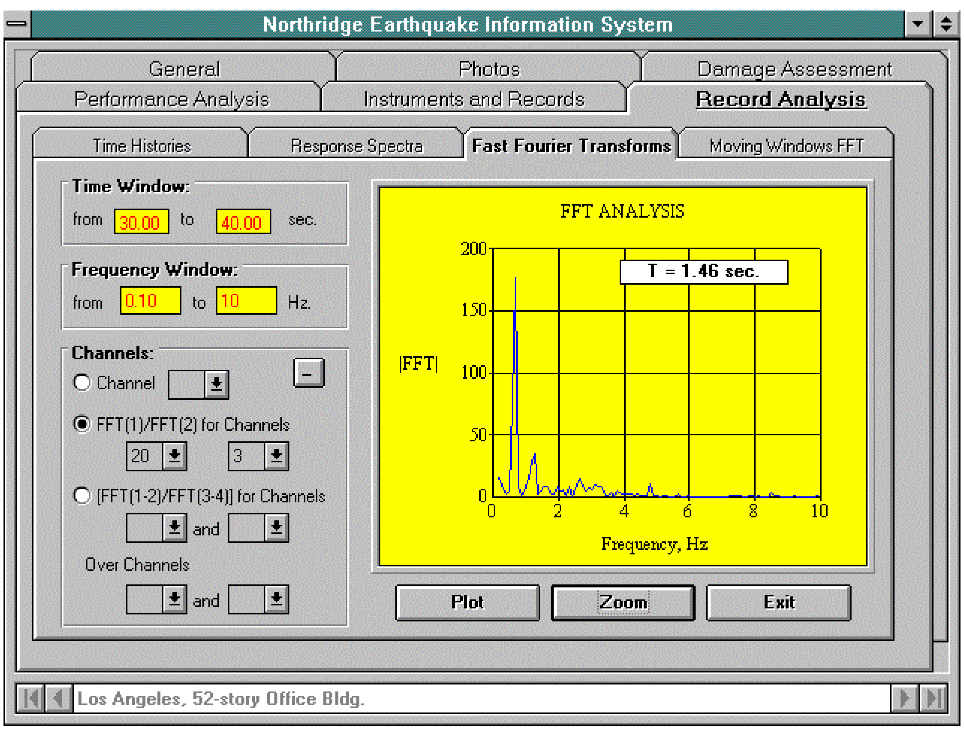
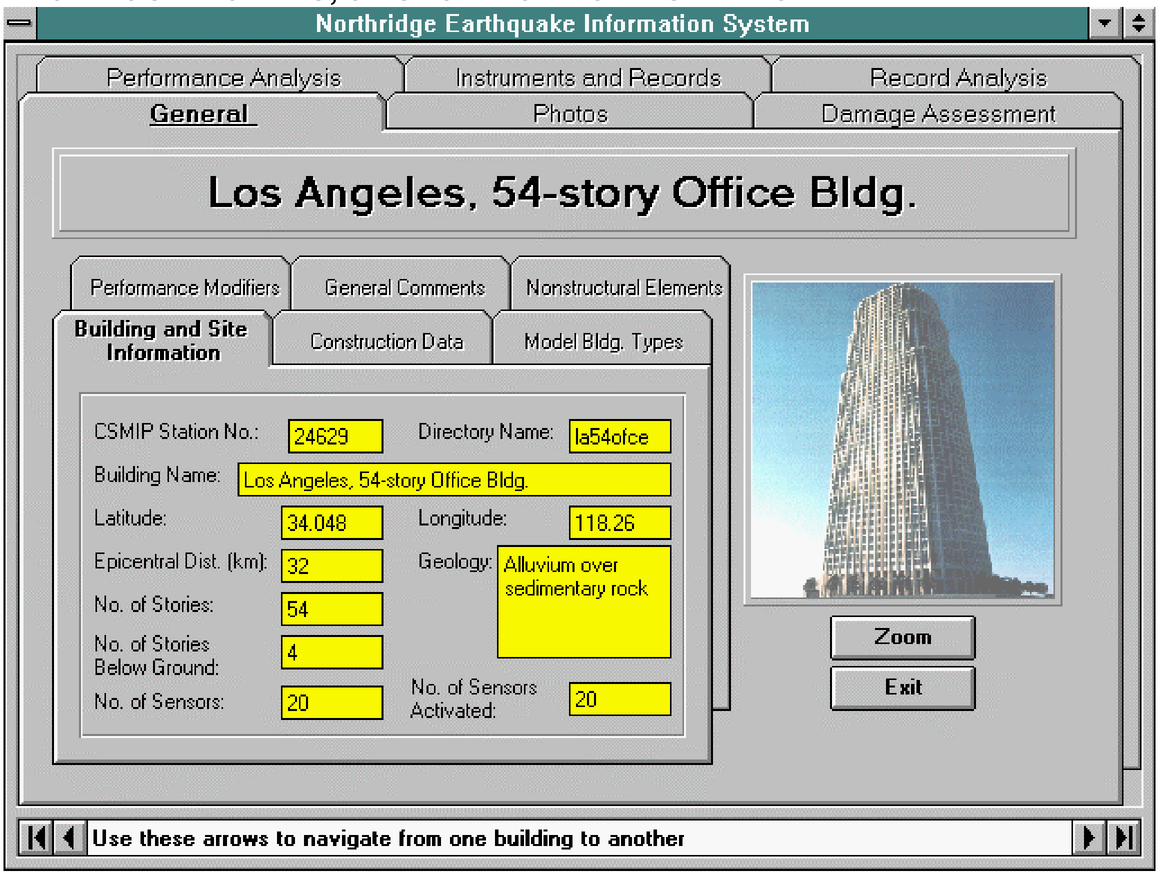


Figure 4-20. An FFT analysis for the N-S response.

4.10. LOS ANGELES, 54-STORY OFFICE BUILDING



No of Sensors Activated: 20 **No. of Photos in the Database: 1**

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the basement (Channel 4, N-S) and at the roof (Channel 19, N-S) are 0.14g and 0.19g, respectively. The peak velocity at the roof is about 34 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-9. The maximum base shear experienced during the Northridge earthquake is estimated at about 0.04W. The overall drift experienced is very low. No damage was reported for this building.

TABLE 4-9. Response Summary for Los Angeles 54-Story Office Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	15.43	3.79
	E-W	17.14	3.57
	DIFF	14.88	4.27
Overturning Moment (% Total Weight x feet)	N-S	16.30	878
	E-W	16.10	955
	DIFF	15.46	753
Roof Lateral Displacement Relative to the Base (cm)	N-S	29.99	13.28 (0.0006)*
	E-W	104.71	16.76 (0.0008)*
	DIFF	24.37	3.60 (0.0002)

* Overall drift index values are shown in brackets

Fourier amplitude spectra analyses of the roof instruments (see Figures 4-21 and 4-22) indicate fundamental translational periods of about 6.0 seconds in the N-S direction and 4.8 seconds in the E-W direction. However, the modes really amplified by the ground motion correspond to periods of about 1.0 and 2.0 seconds. Moving windows FFT analyses (Figures 4-23 and 4-24) exhibit the predominance of the higher modes in the seismic response of this structure.

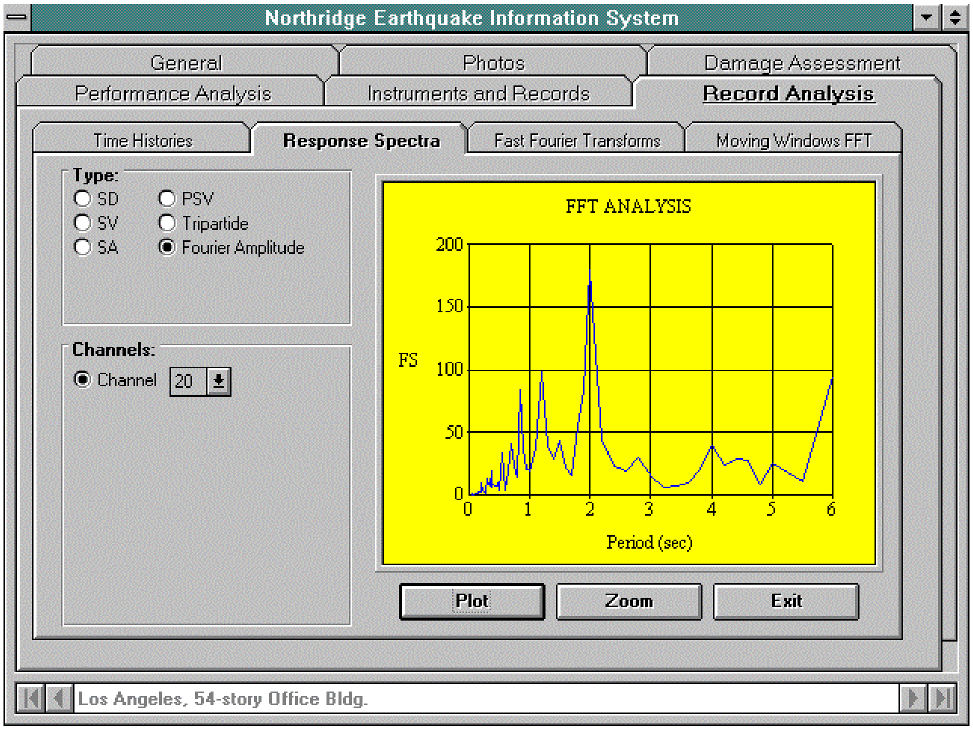


Figure 4-21. Fourier spectrum indicating a fundamental N-S building period of about 6.0 and a predominant mode at about 2.0 seconds.

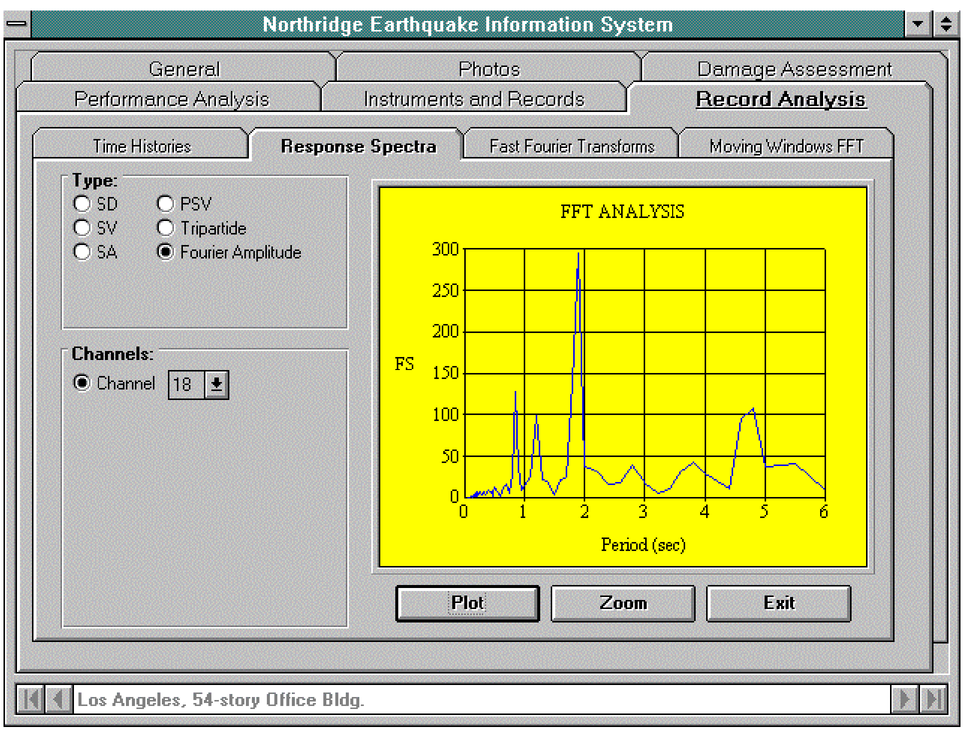


Figure 4-22. Fourier spectrum indicating a fundamental E-W building period of about 4.8 and a predominant mode at about 1.8 seconds.

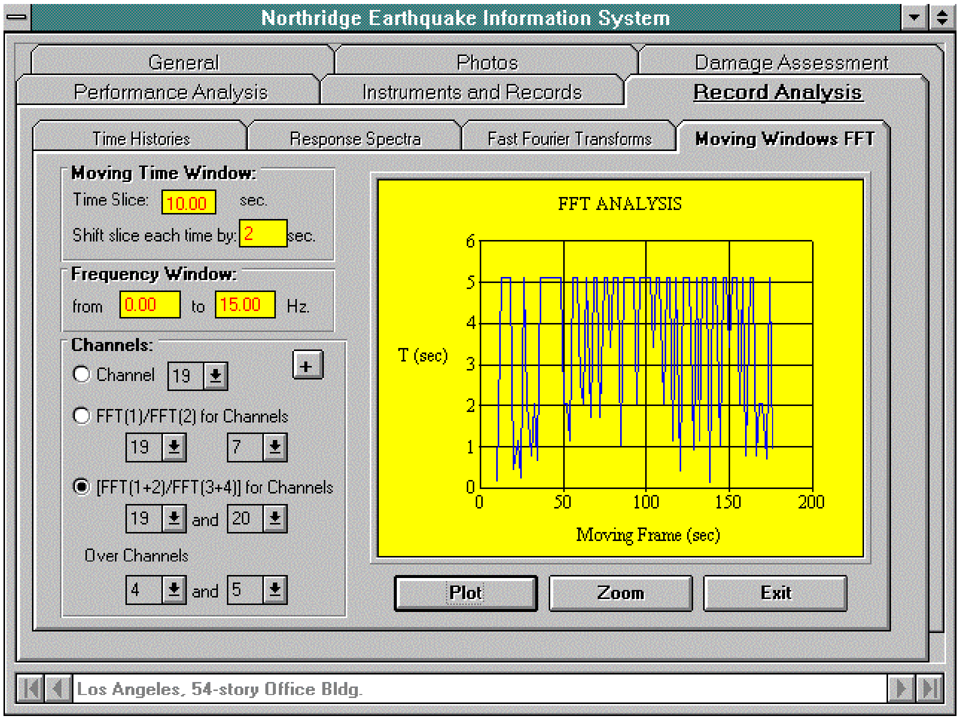


Figure 4-23. Moving windows FFT analysis in the N-S direction shows the predominance of higher modes and no softening of structure.

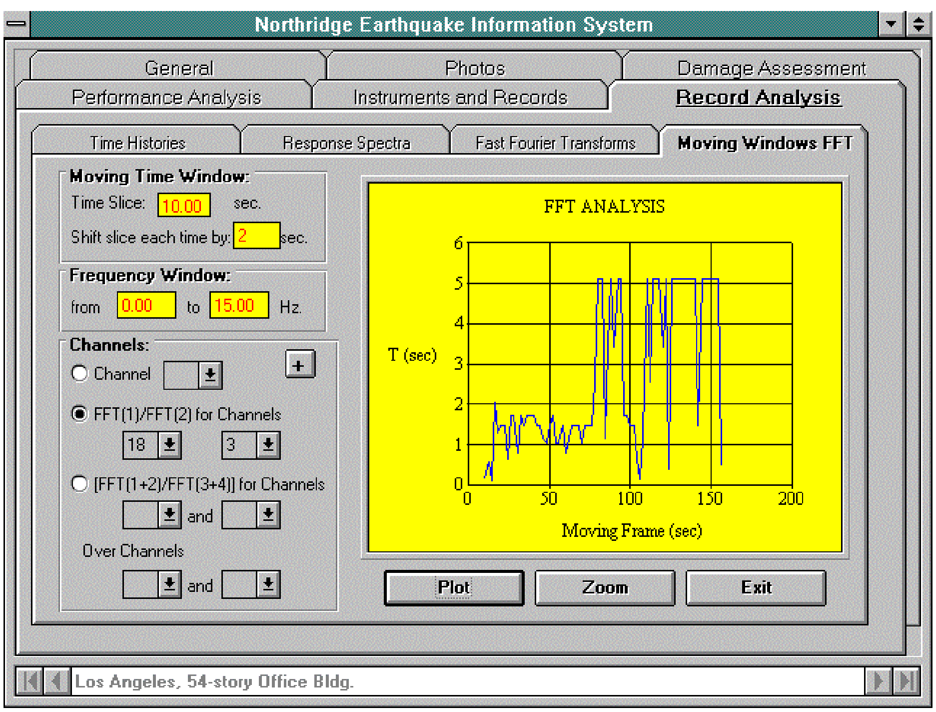


Figure 4-24. Moving windows FFT analysis in the E-W direction shows the predominance of higher modes and no softening of structure.

4.11. LOS ANGELES, 6-STORY OFFICE BUILDING

No of Sensors Activated: 15 **No. of Photos in the Database: 1**

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the base (Channel 3, E-W) and at the roof (Channel 12, E-W) are 0.24g and 0.48g, respectively. The peak velocity at the roof is about 70 cm/sec.

At the time of publication of this report the corrected versions of instrument recordings for Channels 6 to 10 were not available. SMIP report of 5/2/95 on this building (OSMS 95-01R) cited remaining instrument problems on these channels. For these reasons, performance analysis results for this building are not included in the Information System at this time. An upgrade patch will be issued in the future as this data becomes available.

Our FFT analysis of the recorded data indicates N-S and E-W fundamental periods of about 0.85 seconds (see Figure 4-25). The observed translational periods are not far from theUBC-88 and UBC-94 estimate of 0.56 seconds. However, the UBC-85 period estimate of about 0.35 seconds is significantly shorter than observed values (see Appendix - B for backup calculations). Moving Windows FFT analysis of the roof transfer functions shows no significant changes in the natural periods of the building during and after the earthquake.

The maximum drift experienced by the building in the E-W and N-S directions are about 12 cm and 5 cm, respectively. This translates into a modest maximum overall drift index of 0.0023.

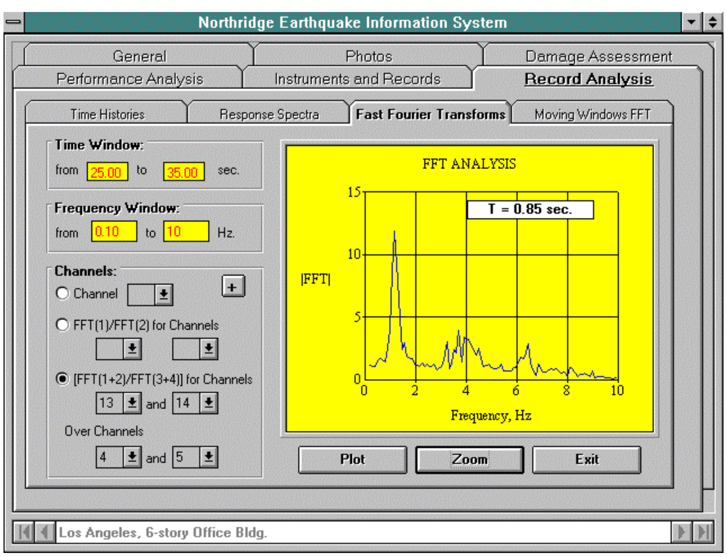
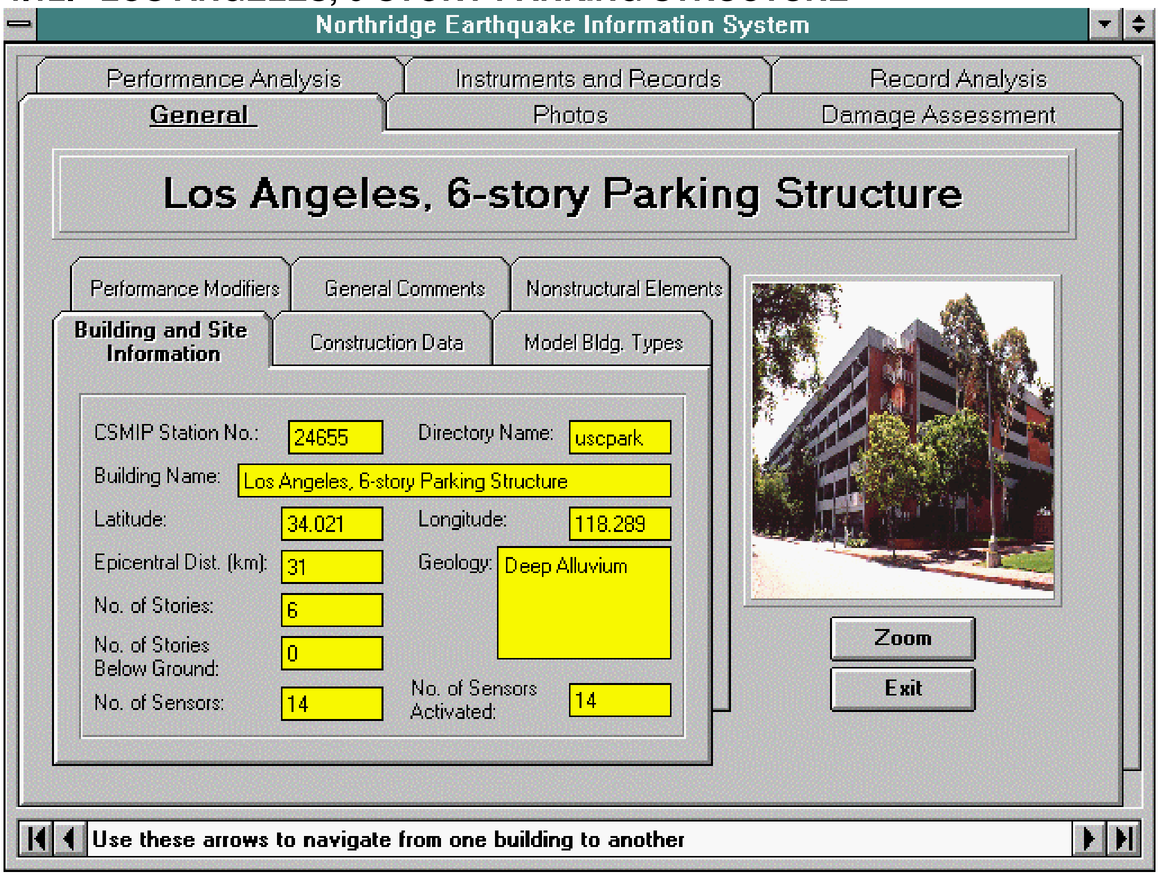


Figure 4-25. An FFT analysis of the N-S response (E-W picture is very similar and hence not reproduced here).

4.12. LOS ANGELES, 6-STORY PARKING STRUCTURE



No of Sensors Activated: 16 No. of Photos in the Database: 12

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal acceleration recorded at the base, near the north-east shear wall (Channel 4, N-S) 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 (Channel 13) recorded a peak vertical acceleration of 0.52g. Elsewhere on the roof, a sensor attached to a parapet on the North side (Channel 14, N-S) recorded a peak horizontal acceleration of 1.21g. and 0.52g, respectively.

Performance analysis calculations for this building are summarized in Table 4-10. The 0.27W maximum base shear apparently experienced by the building in the N-S direction significantly exceeds both the 1976 and 1994 UBC strength design base shears of 1.4x0.089W= 0.125W for UBC-76, and 1.4x0.13W =0.18W for UBC-94 (see Appendix B for backup calculations). The maximum overall drift ratios observed, however, are rather modest.

TABLE 4-10. Response Summary for Los Angeles 6-Story Parking Structure.

Response Parameter	**Direction**	**Time of Maxima (seconds)**	**Maximum Value**
Base Shear (% Total Weight)	N-S	15.41	27.01
	E-W	15.28	20.52
	DIFF	18.01	12.57
Overturning Moment (% Total Weight x feet)	N-S	15.74	780
	E-W	15.28	593
	DIFF	18.01	427
Roof Lateral Displacement Relative to the Base (cm)	N-S	15.49	3.05 (0.0016)*
	E-W	15.28	1.26 (0.0007)*
	DIFF	18.03	1.03 (0.0006)*

* Overall drift index values are shown in brackets

In-spite of large accelerations recorded at the roof and base shears exceeding design values, no apparent sign of structural damage were observed. See Information System photos and damage assessment folders.

Our FFT analyses of the recorded data indicates a N-S fundamental period of about 0.5 seconds and an E-W fundamental period of about 0.4 seconds (see Figures 4-26 and 4-27). Recorded data also indicates a fundamental period of 0.25 seconds for vertical vibration of a roof girder and 0.47 seconds for lateral vibration of a parapet (Figure 4-28). Consult the Information System for more facts. There is no sign of significant lengthening of the periods as a result of the Northridge earthquake. The observed translational periods implied by the sensor data compare very well with the UBC-94 Method A estimate of 0.44 seconds. The UBC-76 period estimates, however, are poor at 0.17 and 0.19 seconds (see Appendix B for backup calculations).

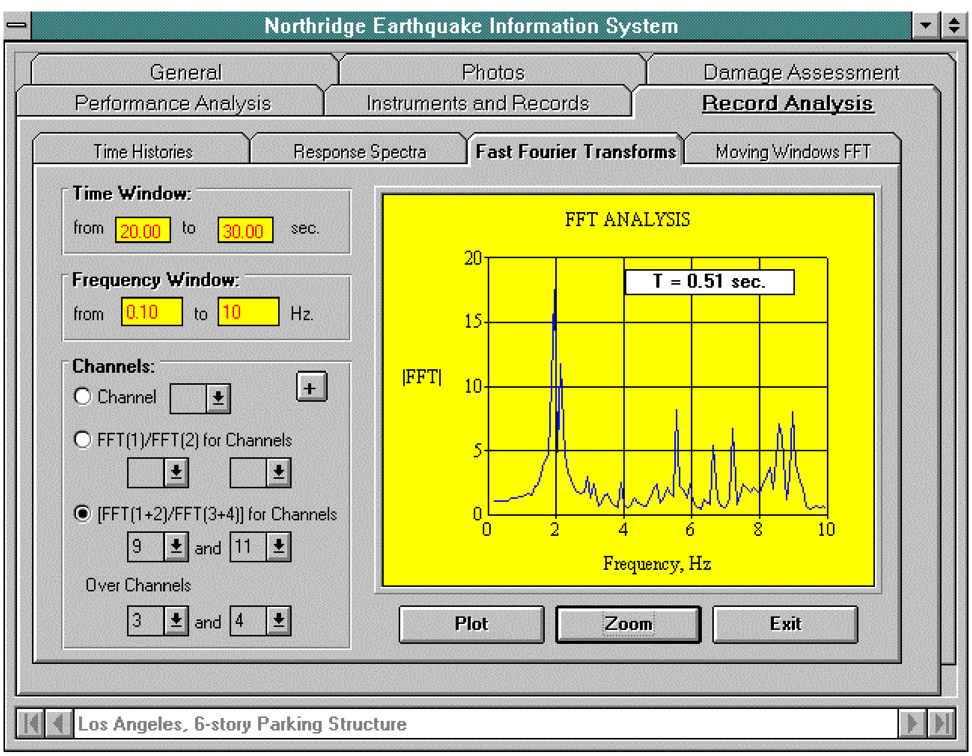


Figure 4-26. An FFT analysis of the N-S response

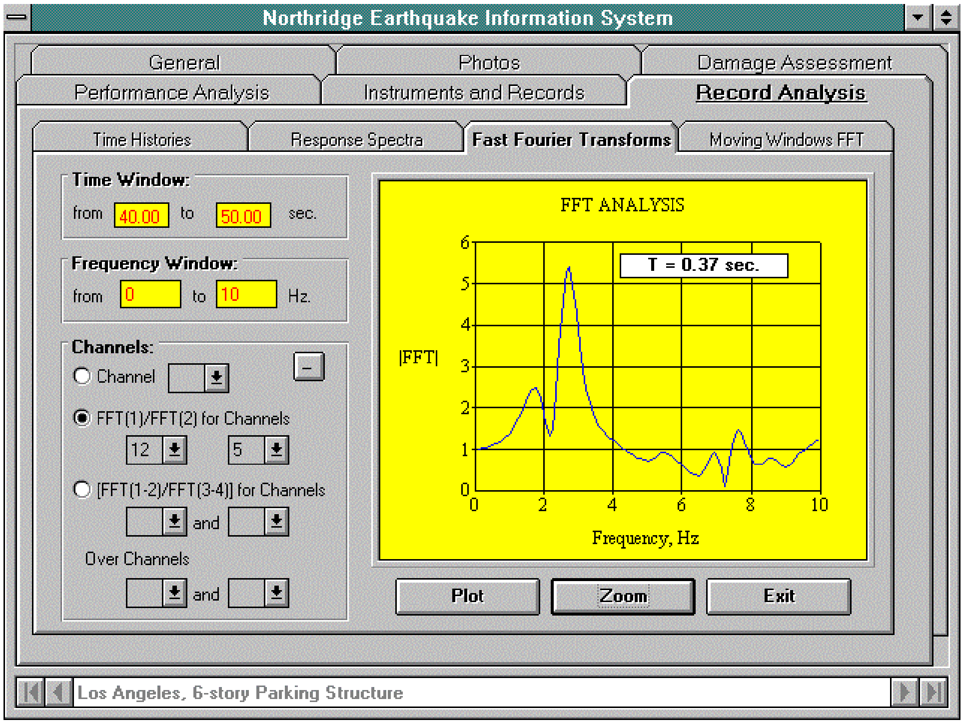
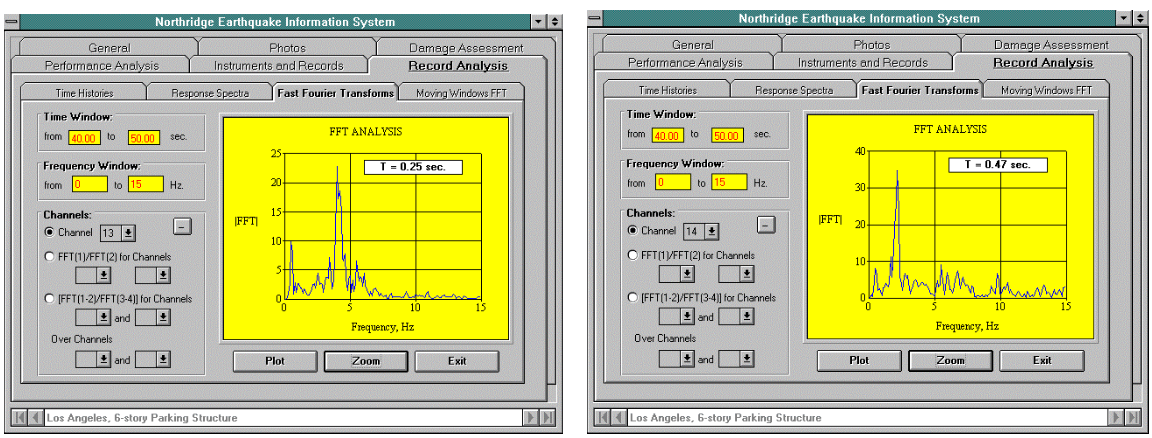


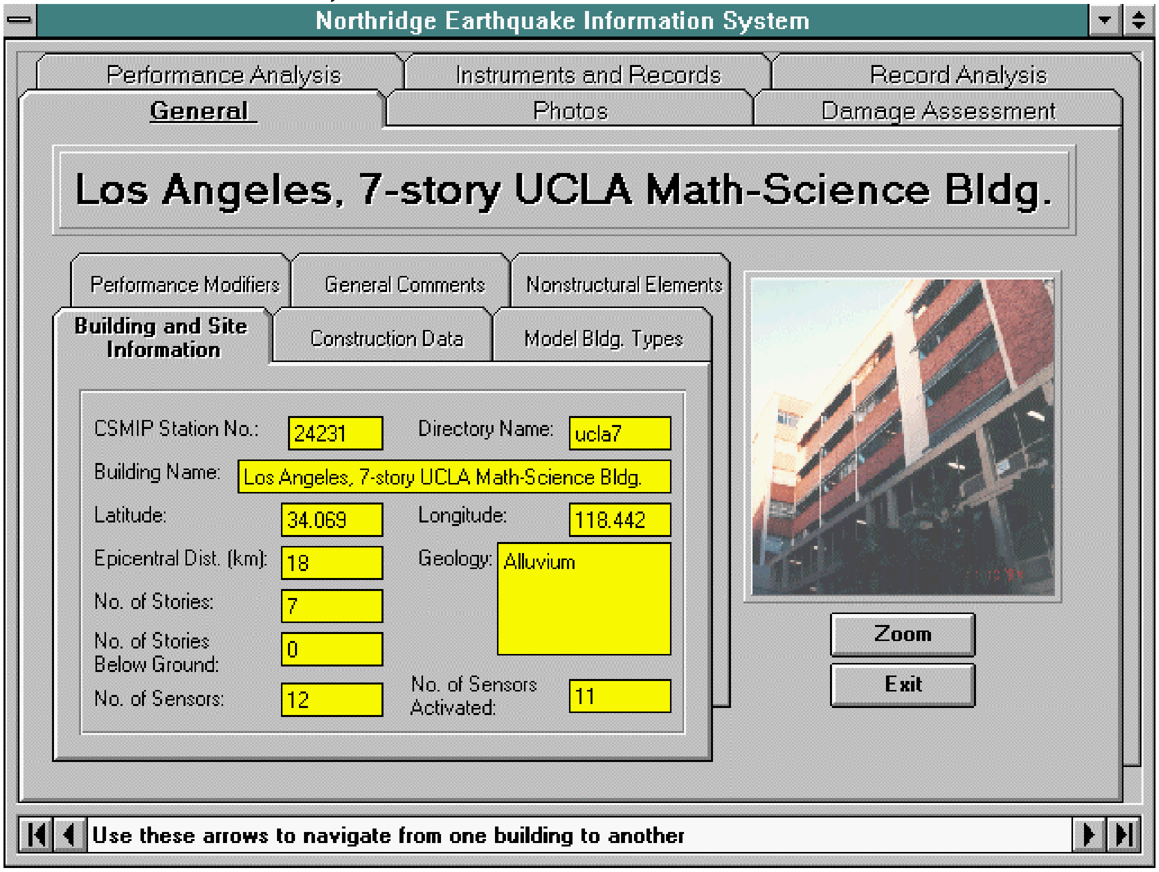
Figure 4-27. An FFT analysis for the E-W response (an smoothed FFT curve is shown).



(a) (b)

Figure 4-28. FFT analysis of sensor data: (a) vertical vibration of a roof girder; (b) lateral vibration of a parapet.

4.13. LOS ANGELES, 7-STORY UCLA MATH-SCIENCE BUILDING



No of Sensors Activated: 11 No. of Photos in the Database: 7

The 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the base and roof are 0.29g (Channel

1, N-S) and 0.76g (Channel 12, N-S), respectively. The maximum velocity recorded at the roof is about 73 cm/sec.

Since no construction plans were available for this building, our performance analysis calculations should be considered as very preliminary estimates (see Table 4-11). Based on these analyses, the building experienced a maximum base shear of about 0.27W. The overall drift ratio experienced by the building is about 0.0047. The maximum differential between the 3rd floor and roof is about 10 cm. Signs of some permanent tilting could be observed and are documented in the Information System.

TABLE 4-11. Response Summary for UCLA Math-Science Addition.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	9.70	22.19
	E-W	8.58	27.42
	DIFF	9.70	21.21
Overturning Moment (% Total Weight x feet)	N-S	9.70	1279
	E-W	9.54	1194
	DIFF	9.70	1213
Roof Lateral Displacement Relative to the Base (cm)	N-S	9.28	7.46 (0.0029)*
	E-W	9.06	12.19 (0.0047)*
	DIFF	6.54	6.76 (0.0026)*

* Overall drift index values are shown in brackets

FFT analysis indicates a fundamental period of about 0.66 seconds for this building in N-

S direction and about 1.02 seconds in the N-S direction (see Figures 4-29 and 4-30). The N-S response generally exhibits a more pronounced high frequency content which may be caused by impact on the seismic separation covers or pounding on the adjacent wings. See the Information System for more details.

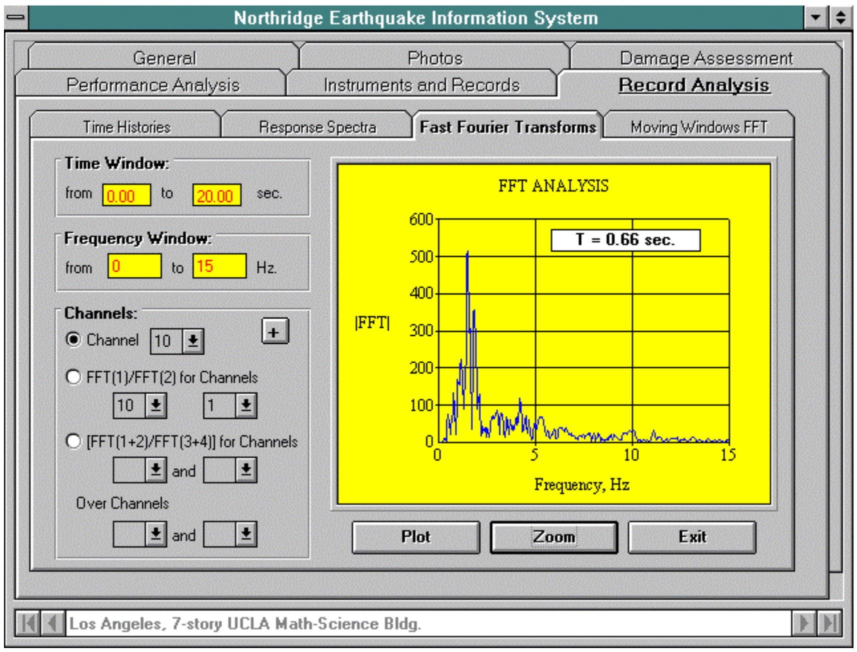


Figure 4-29. An FFT analysis for the N-S response.

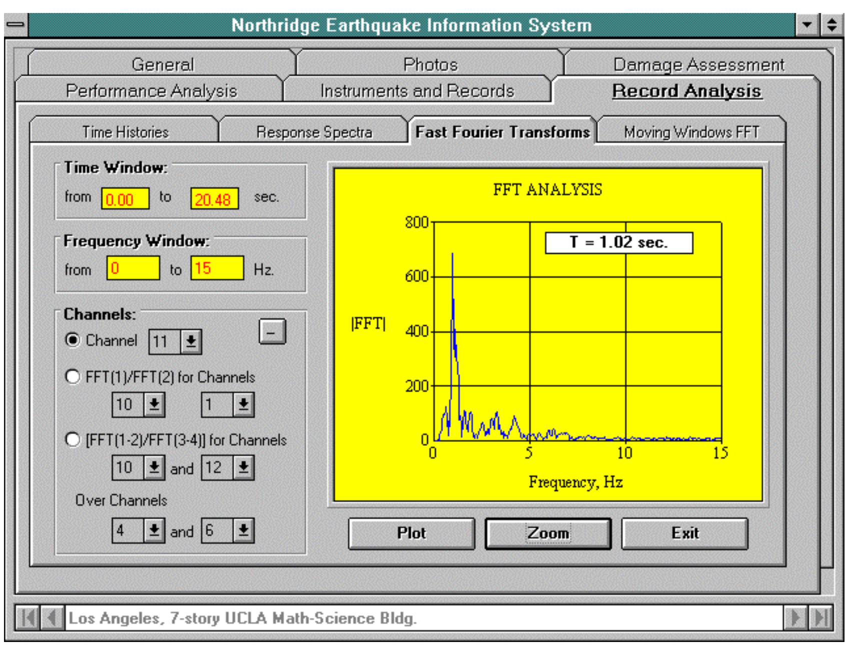
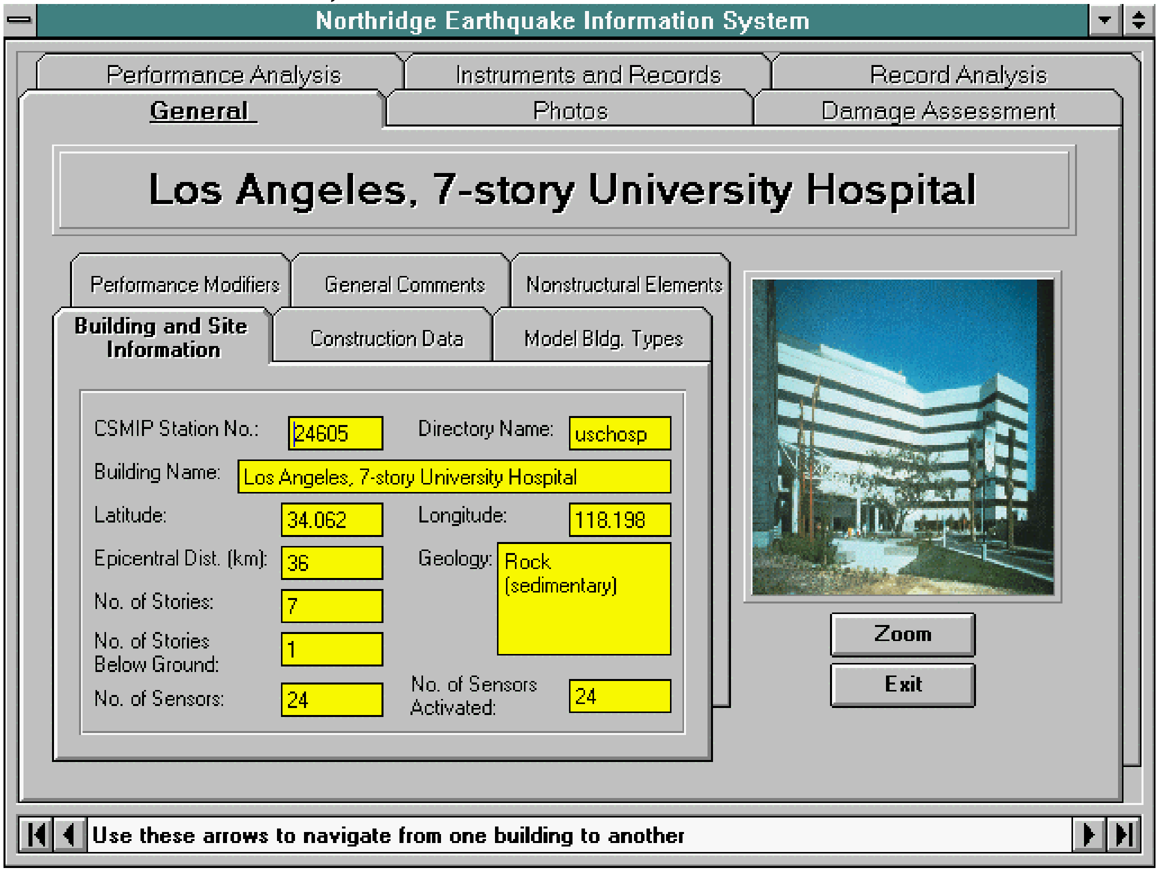


Figure 4-30. An FFT analysis for the E-W response.

4.14. LOS ANGELES, 7-STORY UNIVERSITY HOSPITAL



No of Sensors Activated: 24 No. of Photos in the Database: 1

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Figure 30.

An extensive study of the response of this building to Northridge earthquake ground motions has been sponsored by CSMIP (Nagarajaiah, in press). Once the results of that investigation are available they will be integrated in the Information System by issuing an upgrade patch. 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 (Channel 5, N-S), 0.14g (Channel 11, E-W), and 0.21g (Channel 21, N-S). Notice that the isolation system was effective and managed to reduce peak accelerations from the base to the super-structure.

The cubical-spline interpolation technique incorporated in the performance analysis folder of the information system at this time is not appropriate for approximating the response of isolated systems and hence is not activated for this building.

FFT analyses indicate predominant fixed-base periods of 0.64 and 0.79 seconds in the N-S and E-W directions, respectively (Figures 4-31 and 4-32). Moving windows FFT analysis indicates a building response which begins as a fixed-base system with period elongating to slightly less than 1.5 seconds during the response and decreasing back to a fixed-base response towards the end of the event (Figure 4-33).

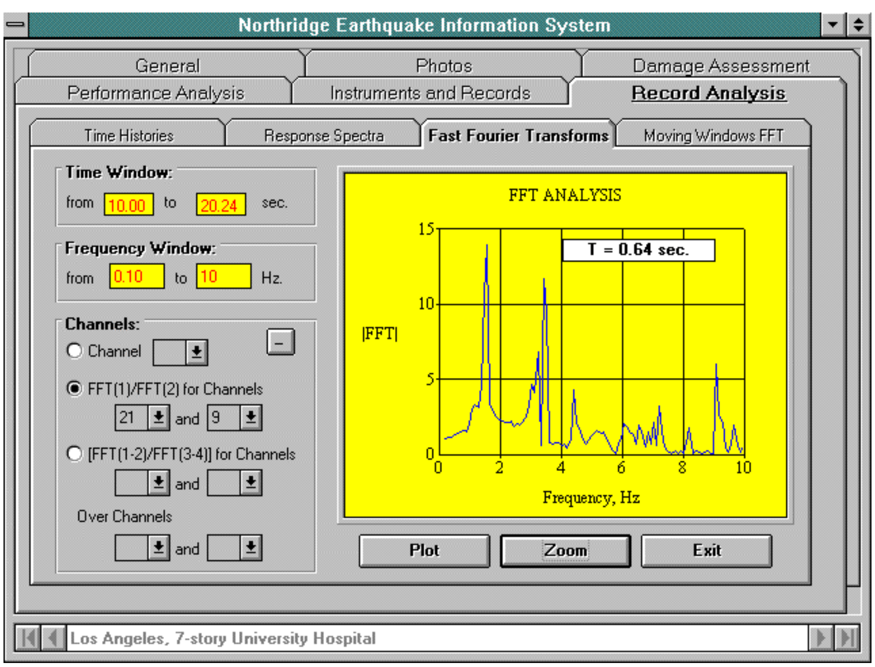


Figure 4-31. An FFT analysis for the fixed-base N-S response.

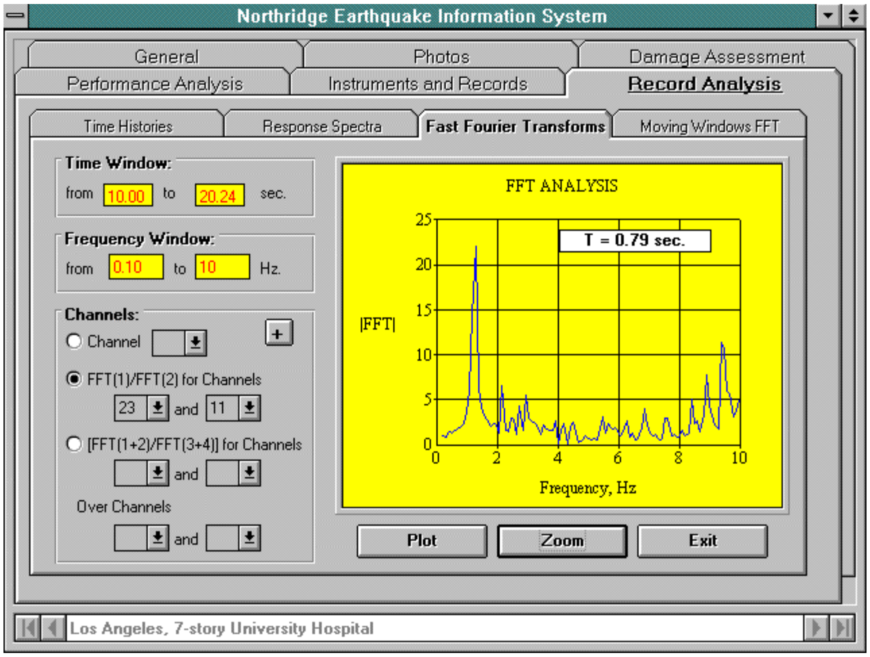
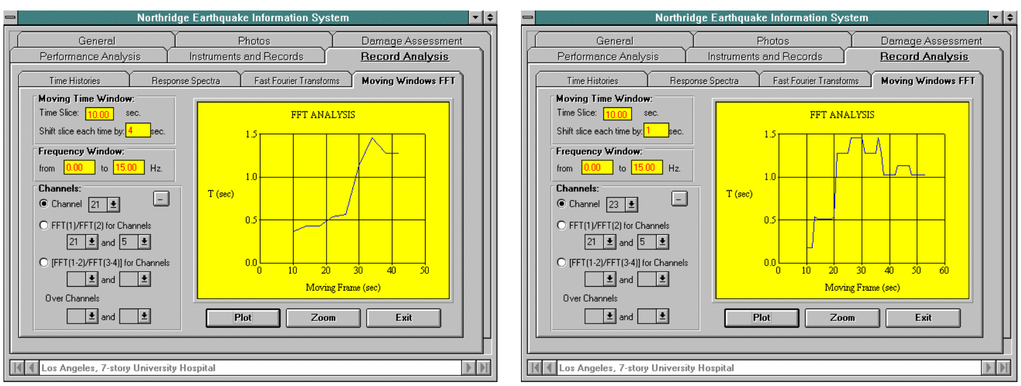


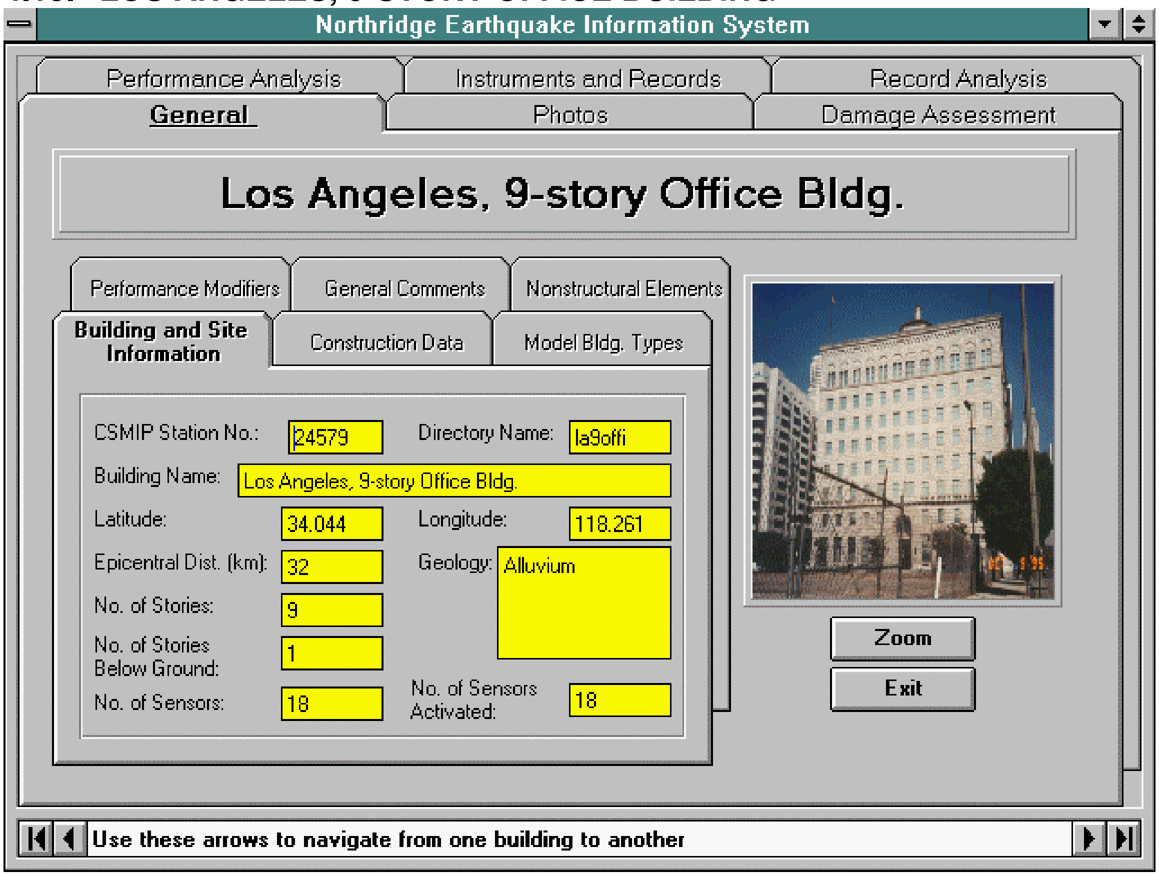
Figure 4-32. An FFT analysis for the fixed-base E-W response.



(a) (b)

Figure 4-33. Moving windows FFT analysis: (a) N-S direction; (b) E-W direction.

4.15. LOS ANGELES, 9-STORY OFFICE BUILDING



No of Sensors Activated: 18 No. of Photos in the Database: 1

This 9-story 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the basement and roof are 0.18g (Channel 7, N-S) and 0.34g (Channel 14, E-W), respectively. The maximum velocity recorded at the roof is about 45 cm/sec.

Since no construction plans were available for this building, our performance analysis calculations should be considered as preliminary estimates (see Table 4-12). Based on these analyses, the building experienced a maximum direct base shear of about 0.17W.

However, apparently torsion played a major role in the response of the building since the differential base shear between the east and west side of the building is significantly more at 0.27W. The overall drift ratio experienced by the building is modest at 0.0020.

TABLE 4-12. Response Summary for Los Angeles 9-Story Office Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear	N-S	20.00	17.19
(% Total Weight)	E-W	15.68	15.08
	DIFF	18.45	27.51
Overturning Moment	N-S	20.01	1414
(% Total Weight x feet)	E-W	18.07	1232
	DIFF	18.45	2079
Roof Lateral Displacement	N-S	20.04	7.60 (0.0018)*
Relative to the Base (cm)	E-W	20.19	5.28 (0.0012)*
	DIFF	16.18	8.78 (0.0020)*

* Overall drift index values are shown in brackets

FFT analyses indicate predominant N-S and E-W periods of about 1.28 and 1.71 seconds (Figures 4-34 and 4-35). The first torsional period appears to be at about 1.0 seconds. As indicated by Figures 4-34 and 4-35, higher modes played a significant role in seismic response of this building. Further confirmations of this fact are obtained by performing moving windows FFT analysis or evaluating frequency domain response at various time-spans during the strong ground motion.

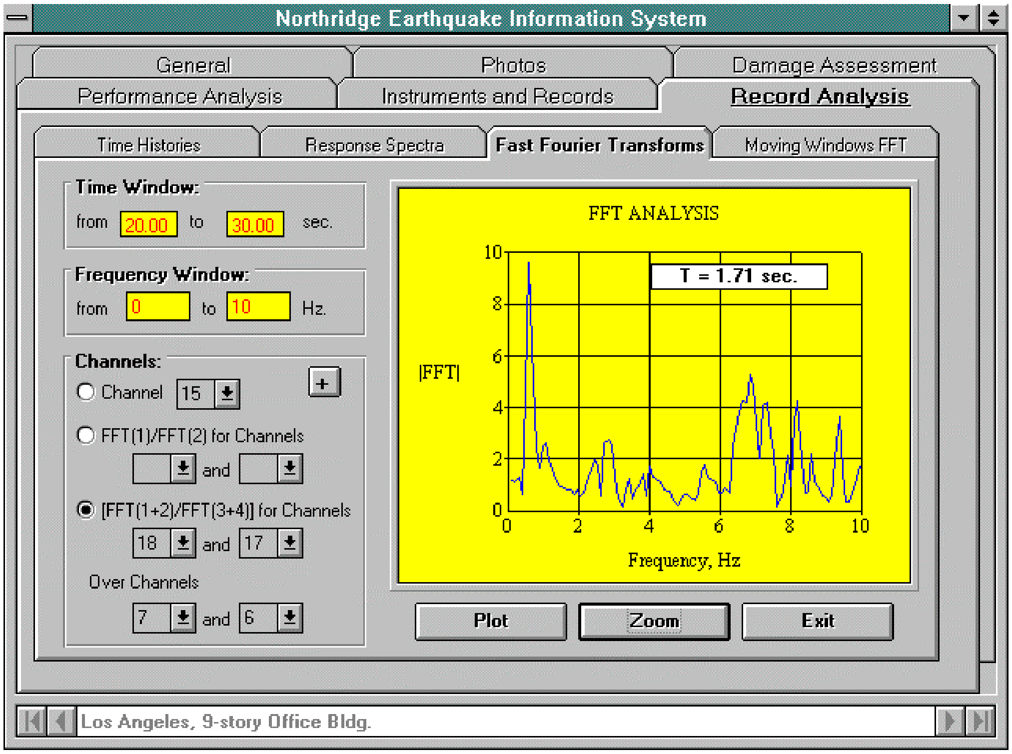


Figure 4-34. An FFT analysis for the N-S response.

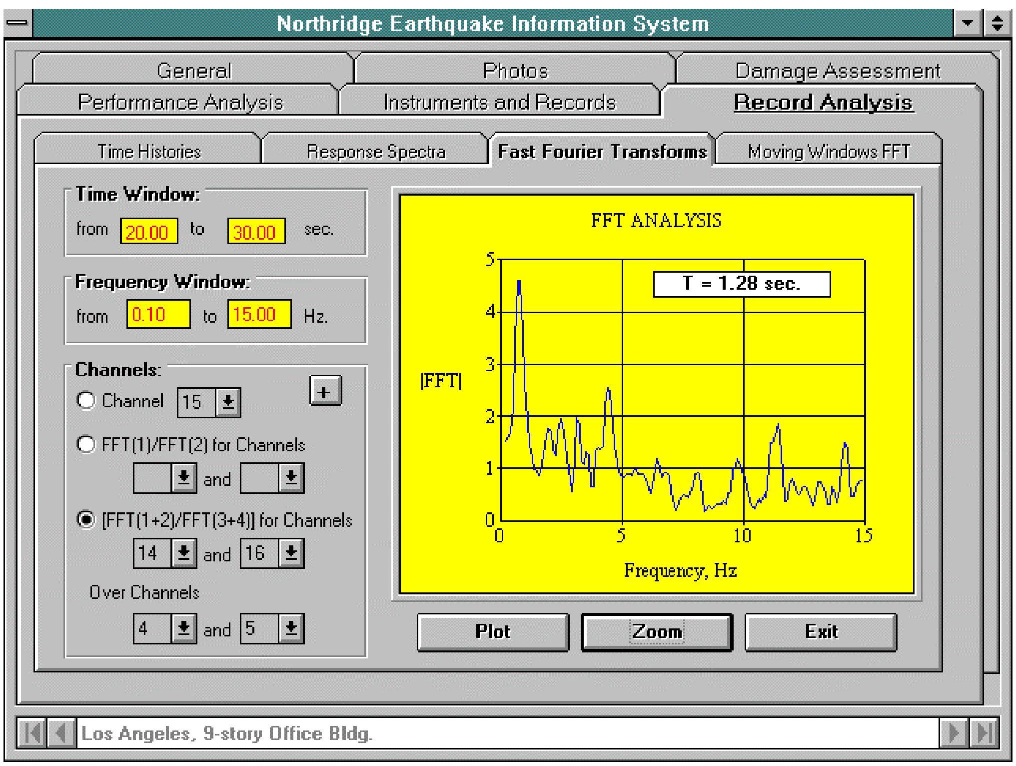
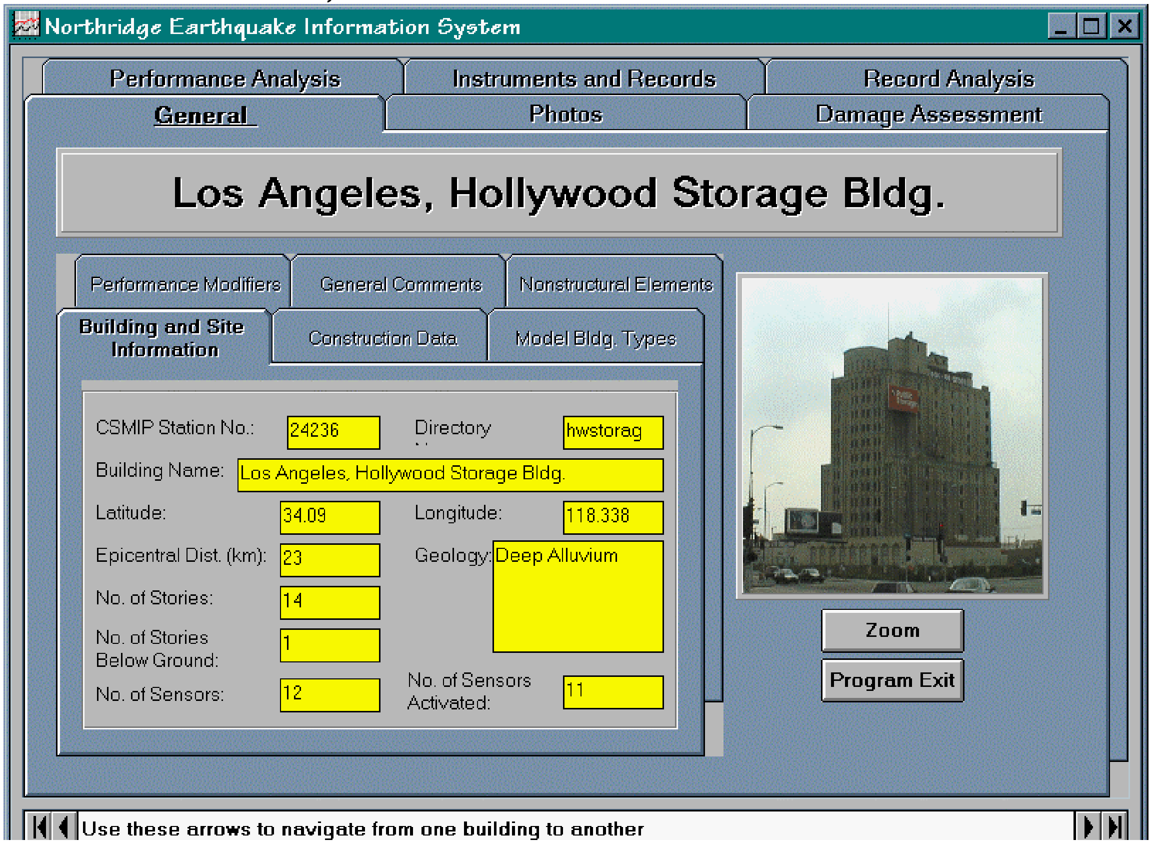


Figure 4-35. An FFT analysis for the E-W response.

4.16. LOS ANGELES, HOLLYWOOD STORAGE BUILDING



No of Sensors Activated: 12 + 3(FF) No. of Photos in the Database: 1

The Los Angeles Hollywood Storage 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

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 (Channel 1, N-S) and at the roof (Channel 12, N-S) 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 report 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.

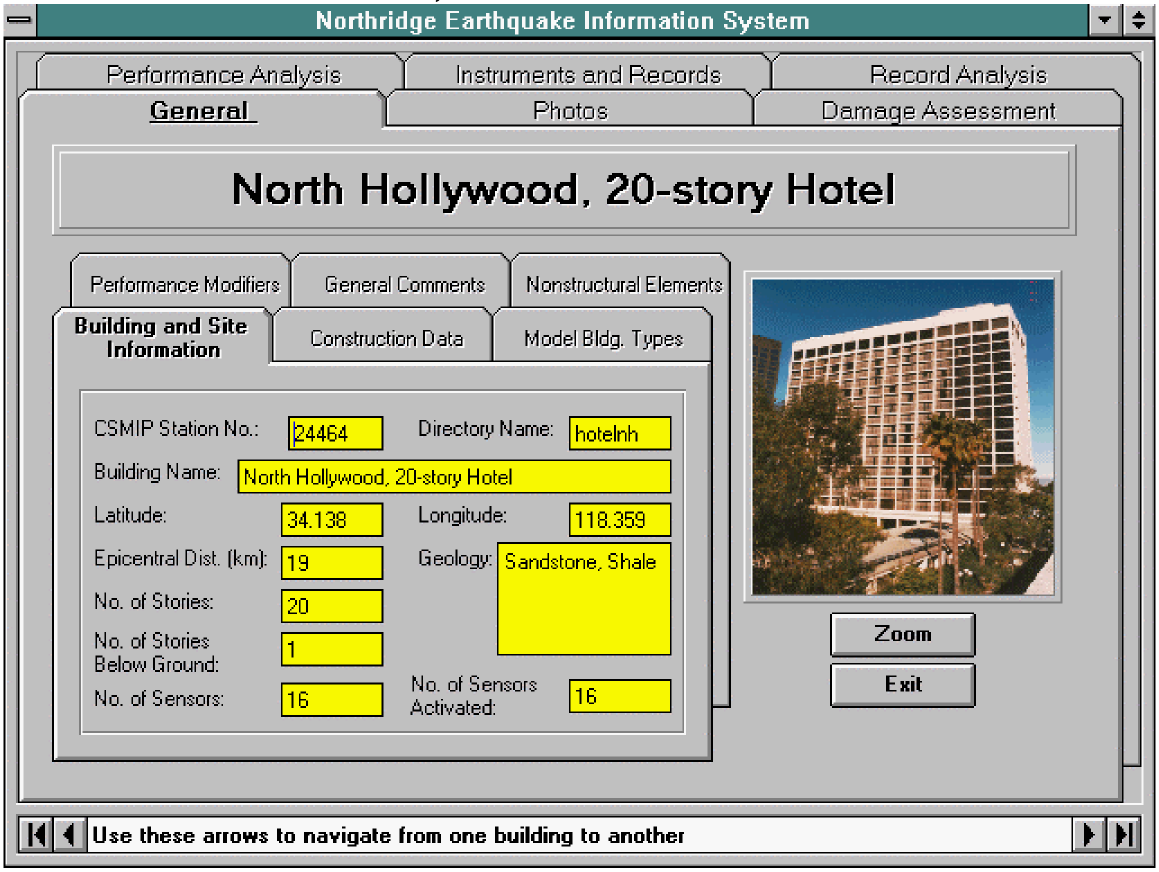
TABLE 4-13. Response Summary for Hollywood Storage Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear	N-S	9.68	13.08
(% Total Weight)	E-W	◆	◆
	DIFF	9.60	28.97
Overturning Moment	N-S	9.68	1065
(% Total Weight x feet)	E-W	◆	◆
	DIFF	9.08	1974
Roof Lateral Displacement	N-S	9.92	6.10 (0.0013)*
Relative to the Base (cm)	E-W	◆	◆
	DIFF	10.76	5.82 (0.0013)*

◆ Not computed due to the lack of Channel 11 data.
* Overall drift index values are shown in brackets.

Performance analysis calculations for this building are summarized in Table 4-13. Notice that due to the lack of data from Channel 11 at the roof, the E-W response is not represented. The maximum direct base shear apparently experienced by the building in the N-S direction is 0.13W. However, this table implies that torsion contributed significantly to the response of this building. The overall drift experienced by the building is small.

4.17. NORTH HOLLYWOOD, 20-STORY HOTEL



No of Sensors Activated: 16 No. of Photos in the Database: 25

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the basement (Channel 1, N-S) and at the roof (Channel 2, N-S) are 0.33g and 0.66g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-14. The 0.11W maximum base shear apparently experienced by the building in the N-S direction is more than twice the UBC strength design base shears of 1.4x0.04W= 0.056W (see Appendix B for backup calculations). The maximum overall drift index experienced by the building was moderate at 0.0036.

TABLE 4-14. Response Summary for North Hollywood 20-Story Hotel.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear	N-S	8.62	10.61
(% Total Weight)	E-W	11.76	5.77
	DIFF	8.58	11.07
Overturning Moment	N-S	8.76	1320
(% Total Weight x feet)	E-W	8.14	613
	DIFF	8.60	1202
Roof Lateral Displacement	N-S	9.02	21.12 (0.0036)*
Relative to the Base (cm)	E-W	18.98	10.63 (0.0018)*
	DIFF	9.08	6.76 (0.0011)*

* Overall drift index values are shown in brackets.

As documented by the information system photos building experienced heavy nonstructural and content damage. According to our interviews, however, no sign of significant structural damage were observed. The building was scheduled to undergo a major renovation in early February 1994. The advent of the Northridge earthquake accelerated the process. The building was unoccupied for a period of three months after the earthquake for a renovation which according to some sources cost about $24,000,000. Nonstructural damage varied from damage to partitions, doors, bathroom fixtures and tiles, and chandeliers. Six to eight glass panels were broken. Cracks were clearly visible on the sidewalk slabs on grade near the entrance of the building. Some oil spillage occurred at the basement equipment room. Aside from that, damage to mechanical equipment was minimal. Consult information system for a large number of photos and further damage information about this building.

Participation of higher modes in response of this building can be clearly seen in a zoomed FFT of the N-S response shown in Figure 4-36.

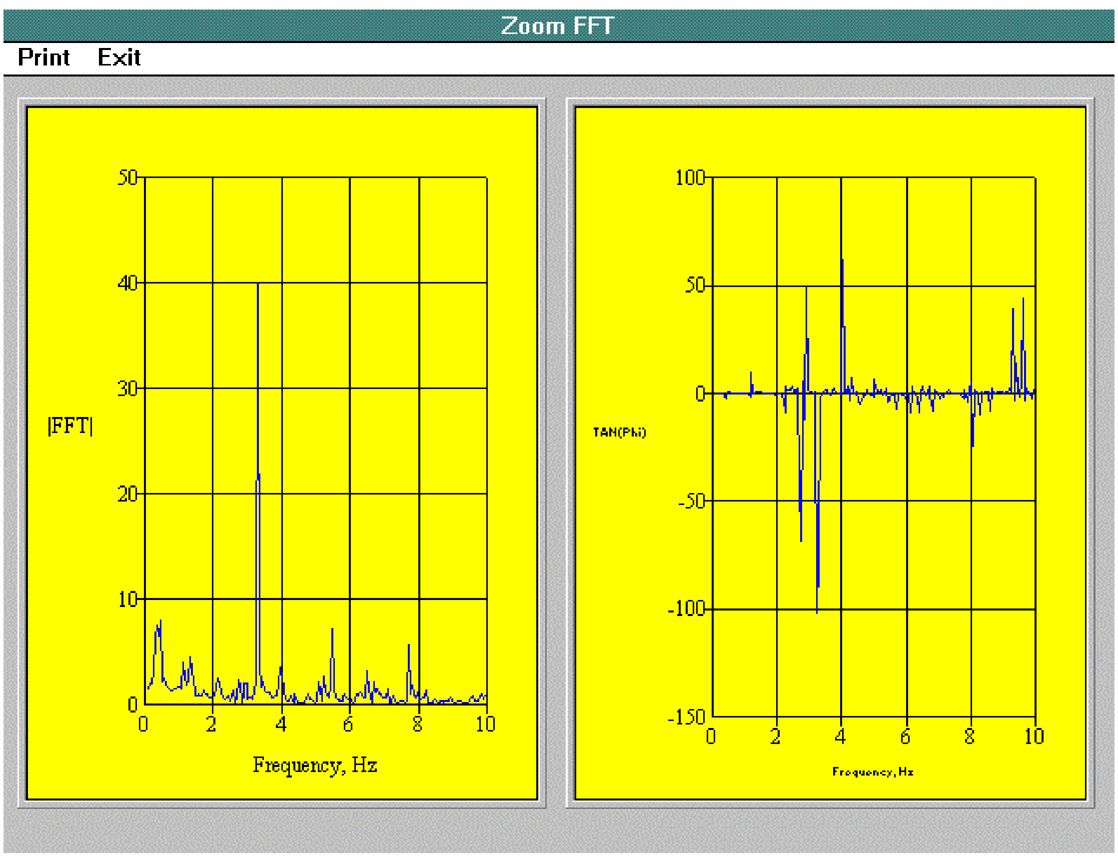


Figure 4-36. A zoomed view of an FFT analysis for the N-S response.

Our analysis indicates a fundamental period between 2.20 to 2.50 for the N-S and E-W direction (Figure 4-37). These periods are significantly larger than UBC-67 estimate of 1.2 and UBC-94 estimate of 1.6 seconds. Response of the building however is significantly influenced by higher mode participation. A torsional first period of about 0.71 sec. is depicted by FFT analysis (Figure 4-38).

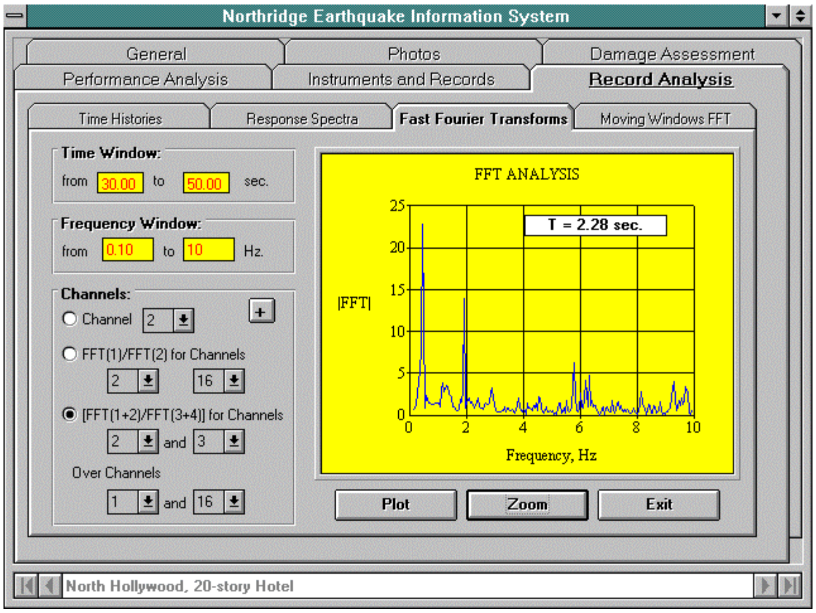


Figure 4-37. An FFT analysis for the N-S response.

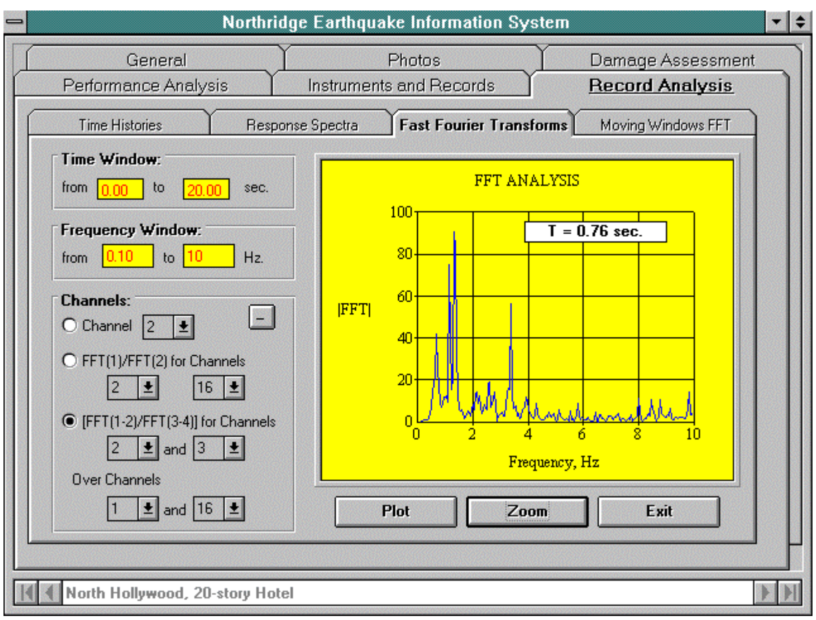
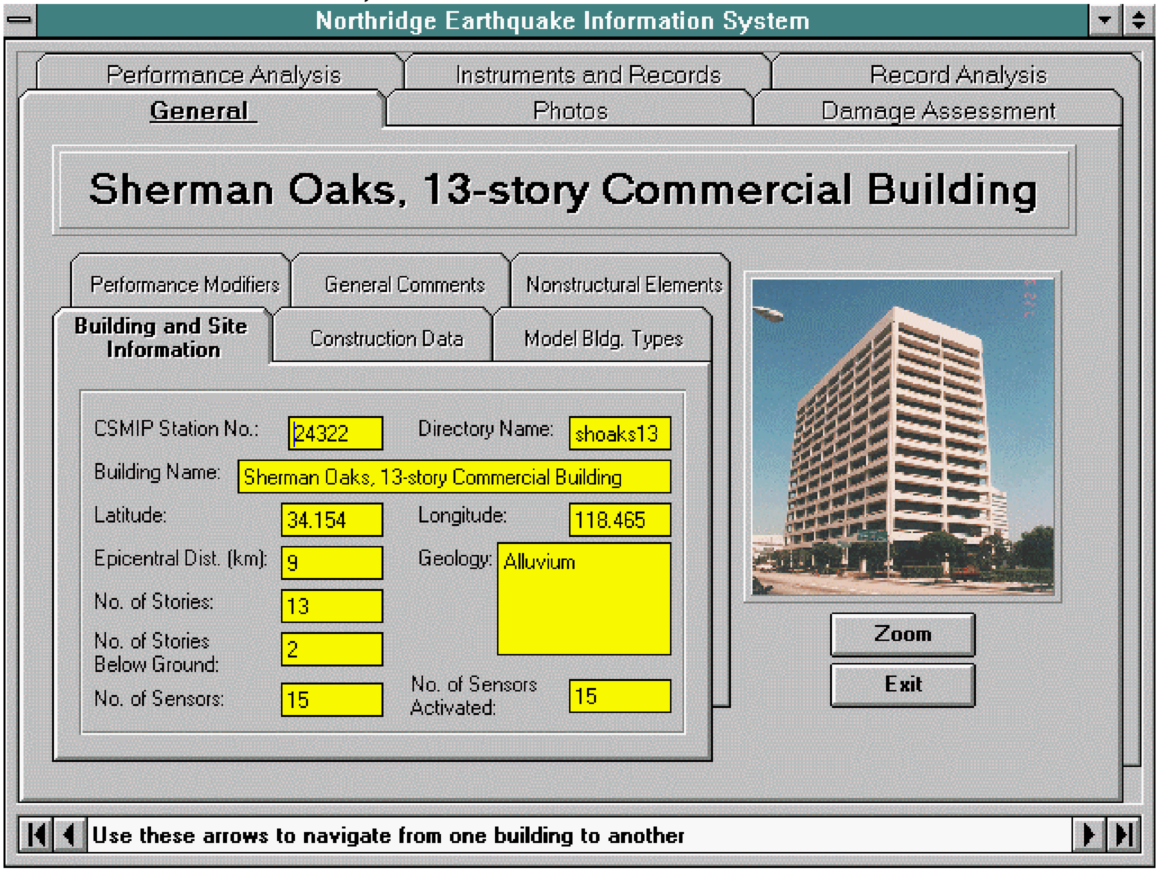


Figure 4-38. An FFT analysis for the torsional response.

4.18. SHERMAN OAKS, 13-STORY COMMERCIAL BUILDING



No of Sensors Activated: 15 No. of Photos in the Database: 29

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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the basement (Channel 15, N-S) and at the roof (Channel 3, N-S) are 0.46g and 0.65g, respectively. The middle floors (see sensor data on the 2nd and 8th floors) experienced large acceleration in the neighborhood of 0.6g. The largest velocity recorded at the roof is about 68 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-15. The 0.19W maximum base shear apparently experienced by the building in the N-S direction is significantly larger than UBC strength design base shear of about 1.4x0.04W= 0.06W for a ductile moment resisting frame with this configuration (see Appendix B for backup calculations). Notice that while the maximum base shear is experienced in the N-S direction, the maximum lateral displacement and an overall drift index of 0.0067 occurs in the E-W direction.

TABLE 4-15. Response Summary for Sherman Oaks 13-Story Office Building.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	5.14	18.70
	E-W	12.72	7.57
	DIFF	3.24	6.69
Overturning Moment (% Total Weight x feet)	N-S	3.22	1304
	E-W	11.52	771
	DIFF	3.22	615
Roof Lateral Displacement Relative to the Base (cm)	N-S	10.86	24.10 (0.0048)*
	E-W	37.98	33.42 (0.0067)*
	DIFF	11.00	4.30 (0.0009)*

* Overall drift index values are shown in brackets.

As documented by the information system photos the building experienced noticeable but repairable structural damage in the form of cracks in the beams, slabs, girders, and walls. According to one source the repair costs exceeded $7,000,000. In contrast, no mechanical equipment damage was observed either at the roof or the basement. Thanks to proper mounting and anchorage details.

As can be seen in Figure 4-39, participation of higher modes were particularly significant in the response of this building to Northridge earthquake. The N-S period of about 2.6 seconds is significantly larger than code estimated periods of 1.27 per UBC-67 and 1.60 seconds per UBC-94. In the E-W direction, a fundamental period of about 2.9 seconds is implied by FFT analysis (Figure 4-40). Our moving windows FFT analysis points to a softening of the structure which may be attributed to the concrete cracking (Figure 4-41).

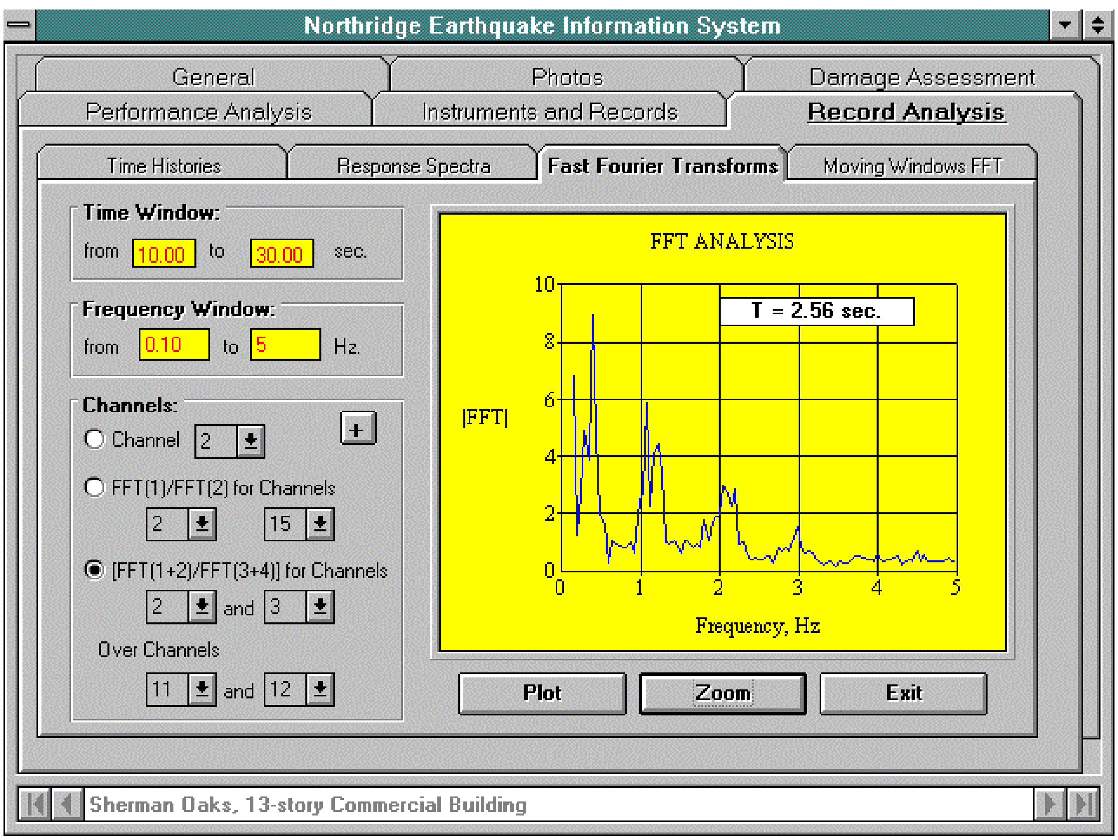


Figure 4-39. An FFT analysis for the N-S response.

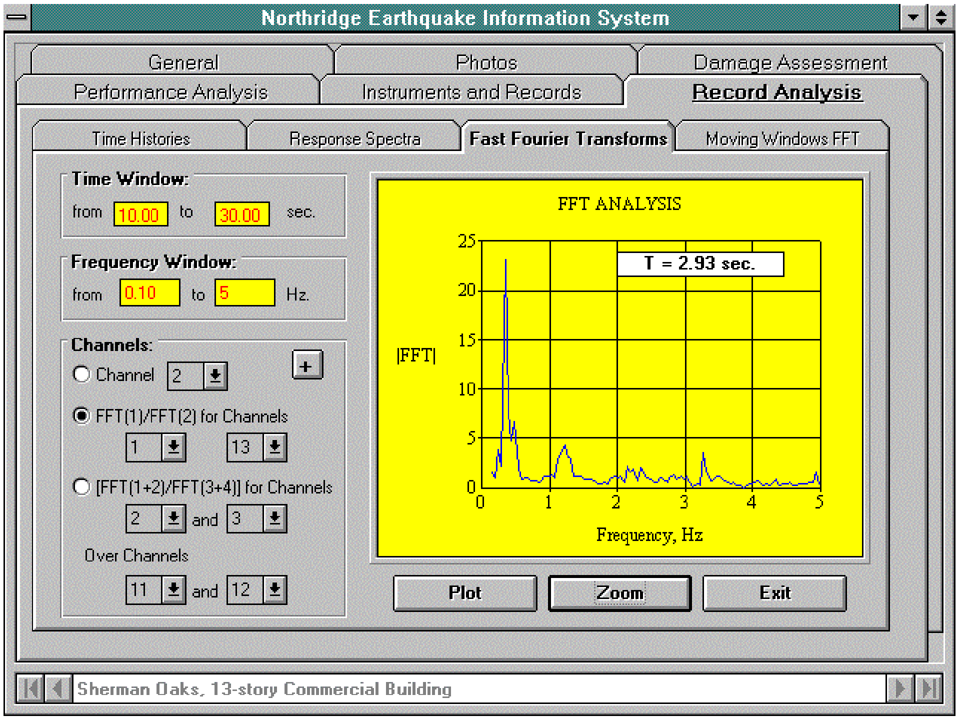


Figure 4-40. An FFT analysis for the E-W response.

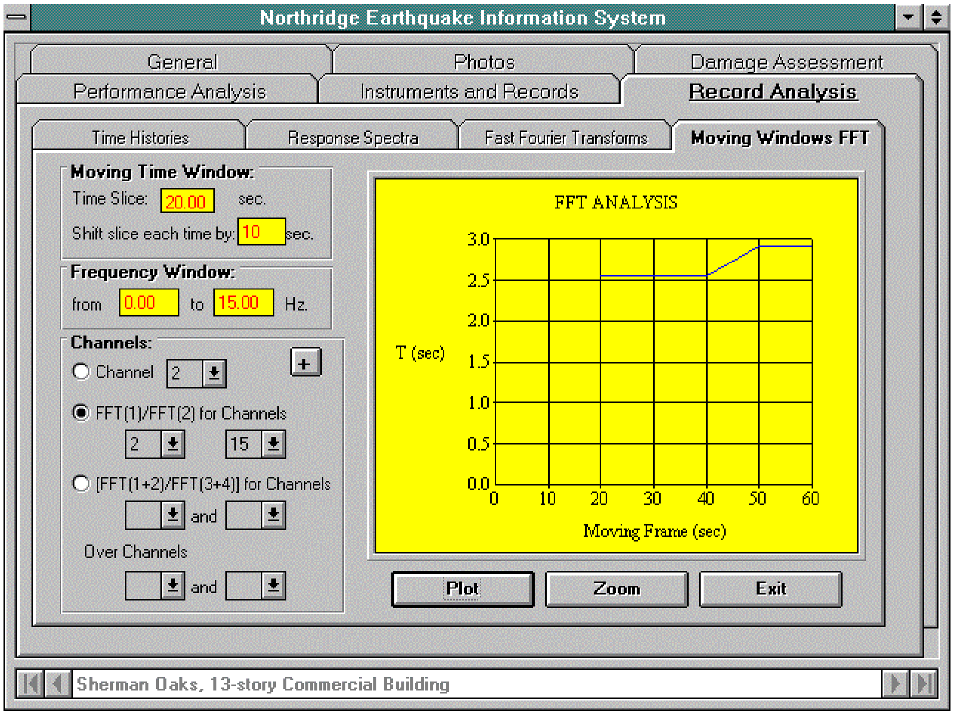
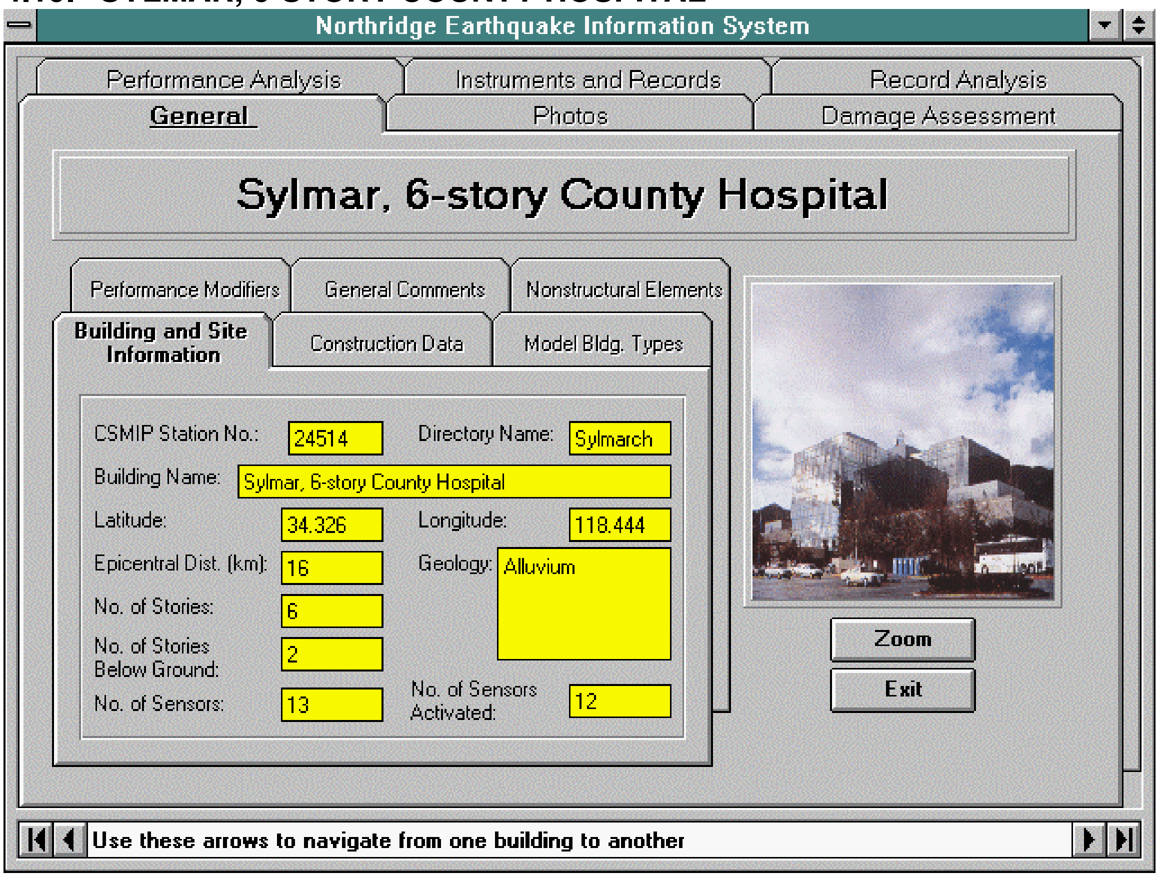


Figure 4-41. Moving windows FFT analysis indicates period elongation.

4.19. SYLMAR, 6-STORY COUNTY HOSPITAL



No of Sensors Activated: 12 No. of Photos in the Database: 62

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 1971 San Fernando earthquake. Designed with the explicit intention of resisting the most damaging earthquakes as perceived at the time, during the Northridge earthquake the structure passed the test of time with flying colors. What happened to the contents, however, as documented by dozens of photos contained in the information system is an entirely another story.

This six story cruciform shaped building has no basement. 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

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 (Channel 9, N-S) and at the roof of the building (Channel 2, N-S) are unprecedented at 0.80g and 1.71g, respectively. The largest velocity recorded at the roof was as large as 140 cm/sec.

TABLE 4-16. Response Summary for Sylmar County Hospital Building.

Response Parameter	**Direction**	**Time of Maxima (seconds)**	**Maximum Value**
Base Shear (% Total Weight)	N-S E-W DIFF	4.24 4.36 4.44	96.89 53.76 62.76
Overturning Moment (% Total Weight x feet)	N-S E-W DIFF	4.22 4.36 4.44	3646 1786 2197
Roof Lateral Displacement Relative to the Base (cm)	N-S E-W DIFF	4.22 6.62 4.48	6.31 (0.0022)* 2.12 (0.0007)* 1.48 (0.0005)*

* Overall drift index values are shown in brackets.

Performance analysis calculations for this building are summarized in Table 4-16. The

0.97W maximum base shear apparently experienced by the building in the N-S direction is several times larger than any value generally used in engineering practice. Although steel plate walls are not specifically covered by the building codes, assuming a shear wall configuration for code comparisons yields a UBC-94 strength design base shear of about 1.4x0.17W= 0.24W (see Appendix B for backup calculations). The building was designed using a site-specific design spectrum rather than equivalent static lateral forces. Considering the severity of the motion the building experienced the observed overall drift indices are surprisingly low. Contrasting the maximum response times in Table 4-16 to the similar values listed for other buildings in this report clearly distinguishes the near-field effect or the "fling" of the ground motion at the site. Here all force and displacement related maximum response values occur within a time window of about two seconds. While for virtually all other buildings studied in this report the maximum displacements occur several seconds later than maximum forces.

As documented by the information system, the structural system experienced negligible damage, if any. Post earthquake survey of some of the steel plate welds showed signs of minor cracking. It is not clear, however, if these cracks were caused by the Northridge earthquake. The content damage was wide-spread and very significant as represented by numerous photos contained in the information system (see Figure 4-42 as an example).

Our FFT analysis point to a fundamental translational period of about 0.46 seconds (Figure 4-43) and a torsional period of about 0.23 seconds (Figure 4-44). Higher mode participation were significant as may be seen in Figure 4-44 and moving windows FFT analysis shown in Figure 4-45).

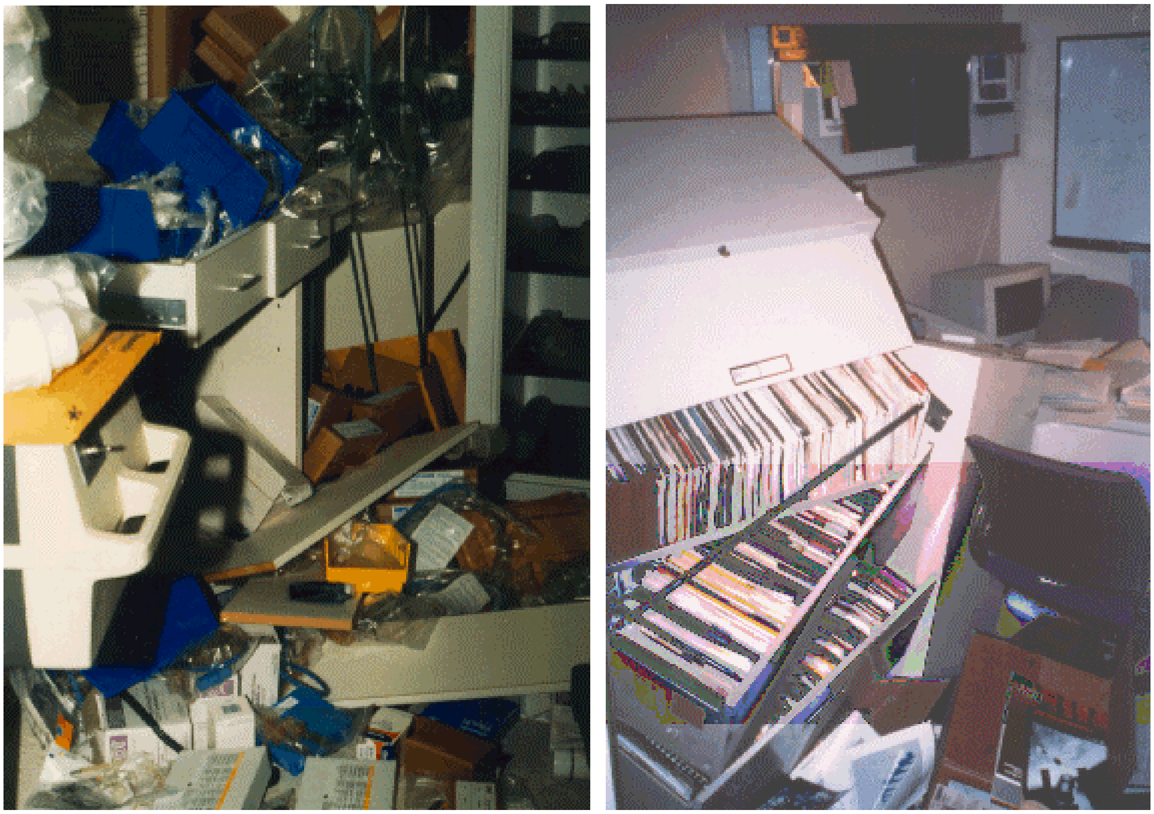


Figure 4-42. Examples of content damage

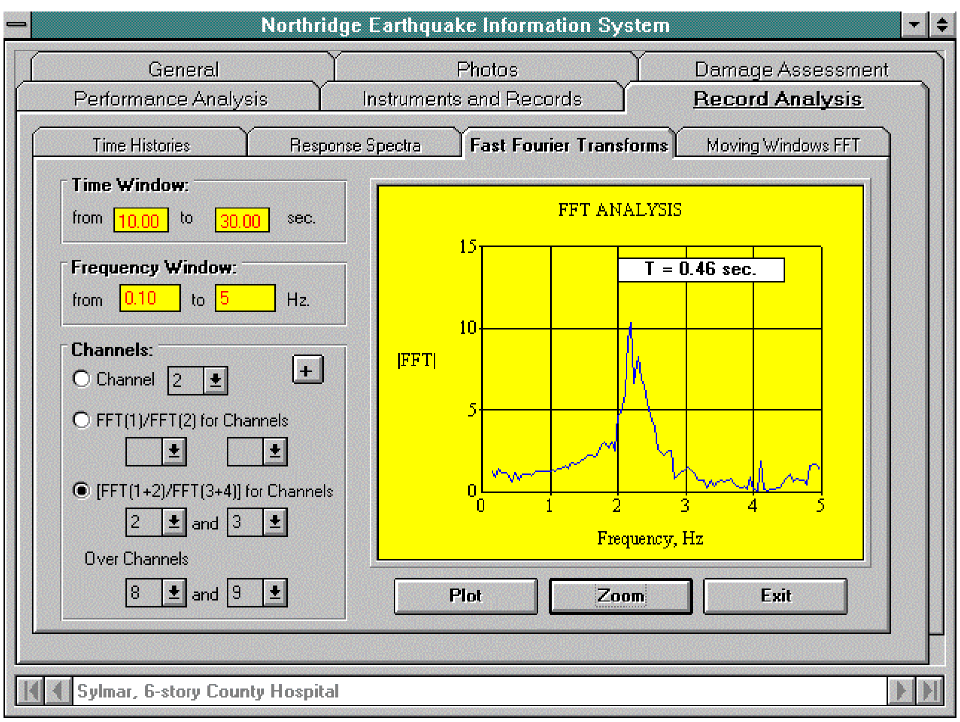


Figure 4-43. FFT analysis for response in the N-S direction.

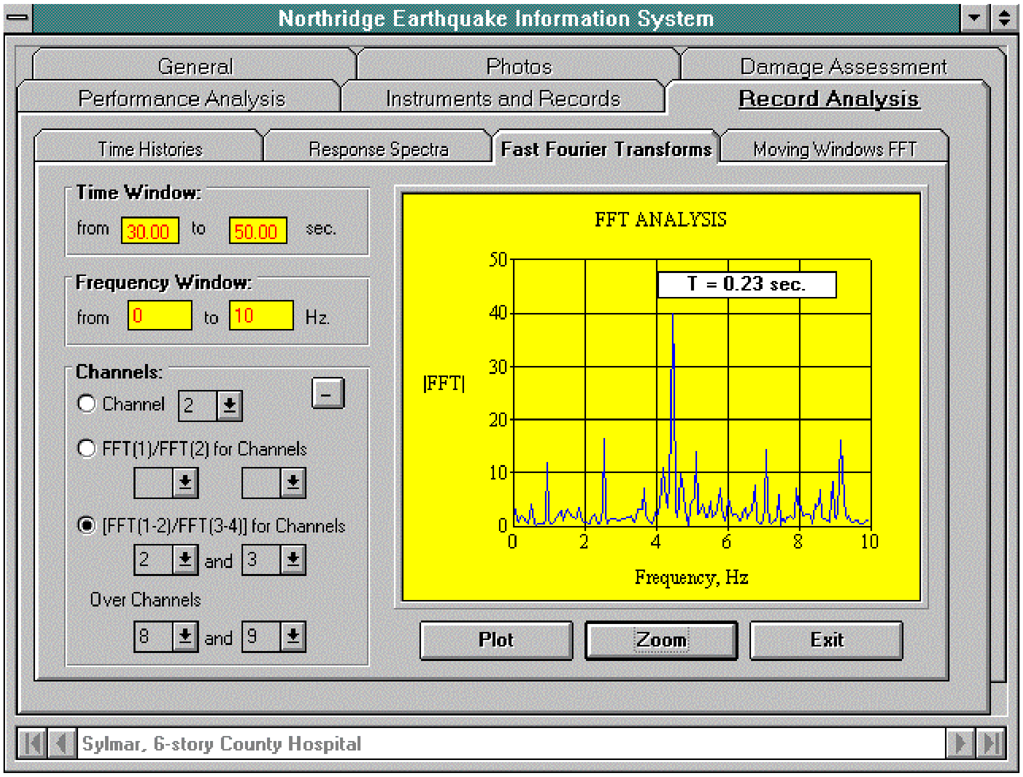


Figure 4-44. FFT analysis for torsional response.

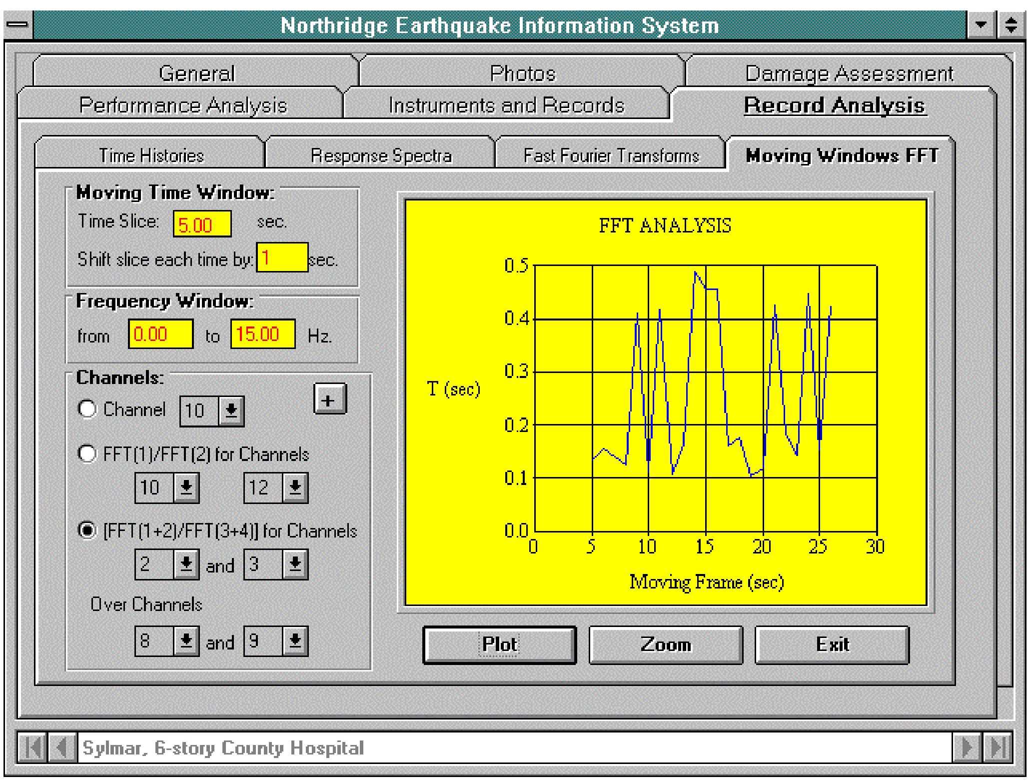
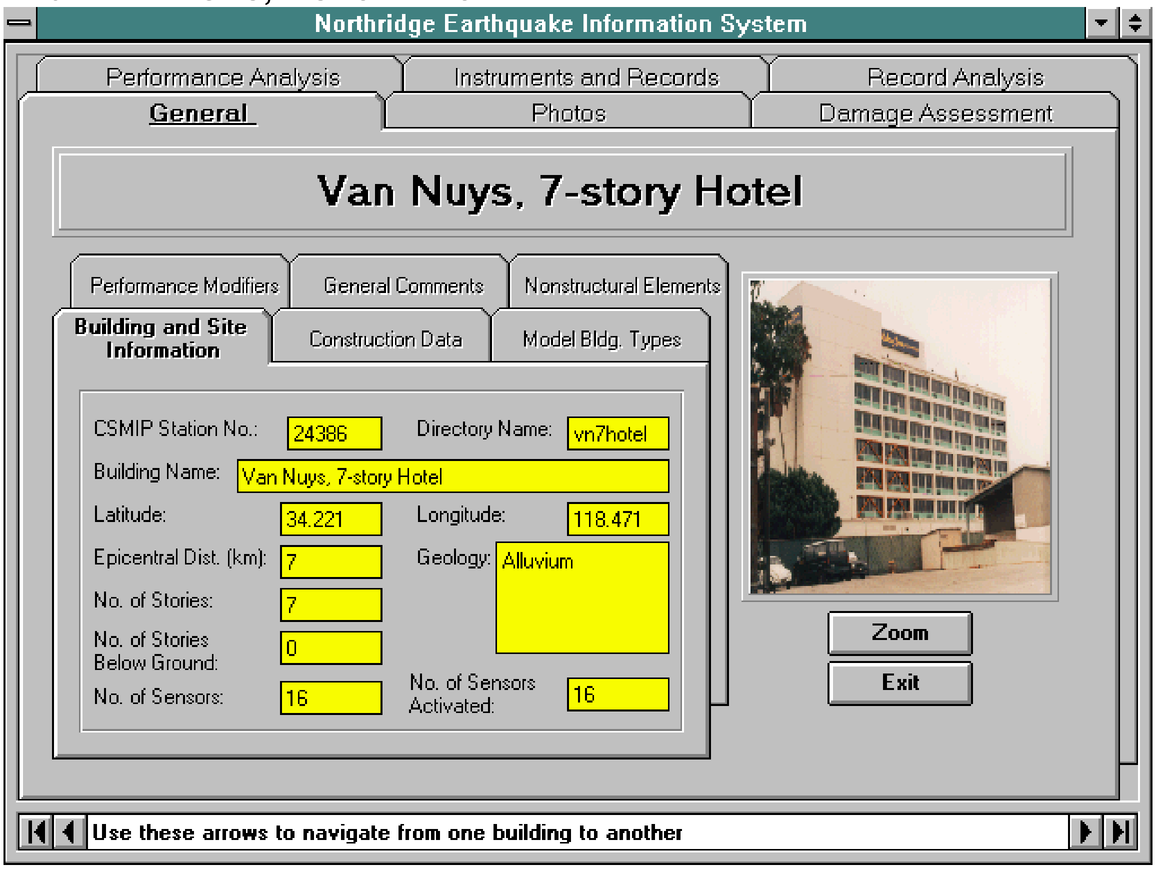


Figure 4-45. Moving windows FFT analysis showing the significance of higher modes.

4.20 VAN NUYS, 7-STORY HOTEL



No of Sensors Activated: 16 No. of Photos in the Database: 25

This 7 story 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. Sketches of plan and elevation of the building showing the location of sensors are presented in Appendix A.

The largest peak horizontal accelerations recorded at the basement (Channel 16, E-W) and at the roof (channels in both directions) are 0.45g and 0.58g, respectively. The largest velocity recorded at the roof is about 77 cm/sec.

Performance analysis calculations for this building are summarized in Table 4-17 where the significance of torsion (or differential response) to overall seismic behavior may be clearly seen . The 0.33W maximum base shear apparently experienced by the building in the E-W direction is significantly larger than the 1964 UBC strength design base shear of about 1.4x0.05W= 0.07W and somewhat larger than the UBC-94 value of about 1.4x0.15= 0.21W for a non-ductile moment resisting frame system (see Appendix B for backup calculations). Notice that the building experienced significant deformation particularly in the E-W direction with an overall drift index exceeding one percent of the height.

TABLE 4-17. Response Summary for Van Nuys 7-Story Hotel.

Response Parameter	Direction	Time of Maxima (seconds)	Maximum Value
Base Shear (% Total Weight)	N-S	8.38	27.68
	E-W	9.24	33.30
	DIFF	8.56	40.46
Overturning Moment (% Total Weight x feet)	N-S	8.38	830
	E-W	9.24	1058
	DIFF	4.56	1070
Roof Lateral Displacement Relative to the Base (cm)	N-S	10.68	19.82 (0.0099)*
	E-W	9.36	23.36 (0.0117)*
	DIFF	8.74	13.91 (0.0069)*

* Overall drift index values are shown in brackets.

The building had suffered minor structural damage and extensive nonstructural damage during the 1971 San Fernando earthquake which was subsequently repaired. As documented by the information system photos the building experienced heavy damage during the Northridge earthquake where the South side exterior columns at fourth floor failed in shear (Figures 4-46 and 4-47). The building is currently undergoing repairs which are changing the structural system in the E-W direction to a shear-wall frame interaction system. The content damage was also heavy as documented in the information system. Surprisingly however, the mechanical equipment installed at the roof did not suffer any noticeable damage. The racking of the fourth floor was so significant that some of the hotel room doors needed to be opened using sledge hammers to get the occupants out (Figure 4-48).

Our moving windows FFT analysis indicates an initial E-W fundamental period of about 1.4 seconds which elongates to about 2.2 seconds towards the end of the strong motion (Figure 4-49). Similar analysis shows a more moderate period elongation in the N-S direction from about 1.3 to 1.8 seconds, except when the kick from the E-W failure is captured (Figure 4-50). These periods are more than twice the code estimated periods of about 0.7 seconds (see Appendix B).

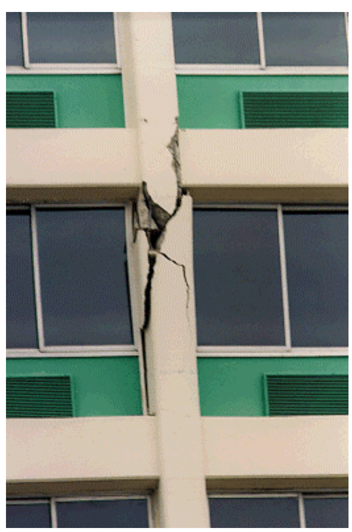


Figure 4-46. Shear failure of column at the fourth floor (view from outside).

Figure 4-47. Shear failure of column at the fourth floor (view from inside).

Figure 4-48. Doors had to be opened with sledge hammers.

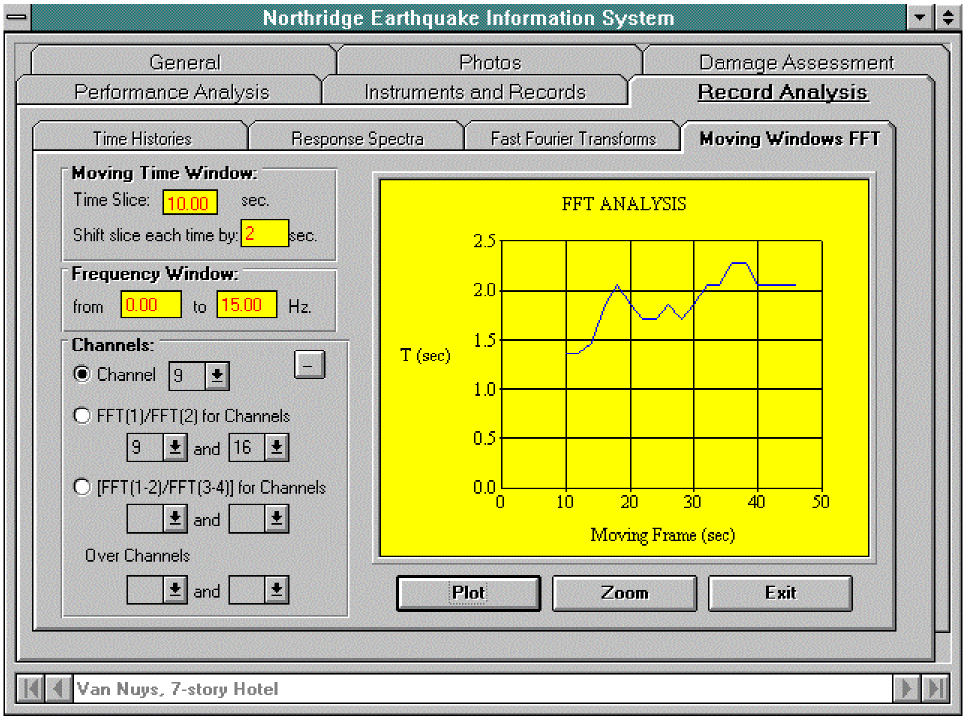


Figure 4-49. Moving windows FFT analysis for E-W response.

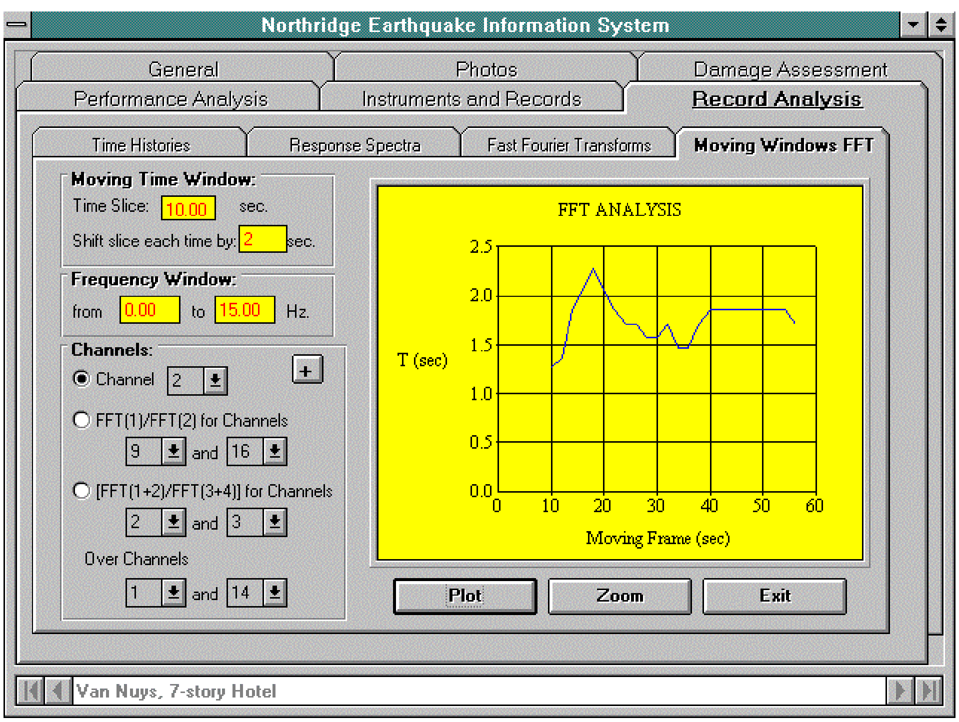


Figure 4-50. Moving windows FFT analysis for E-W response.

5. CLOSURE

An interactive information system was presented which contains all of the gathered information, inspection results, recorded data, performance analysis results, and analytical tools utilized for evaluation of twenty instrumented buildings which were subjected to significant ground shaking during the January 17, 1994 Northridge earthquake. 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 instrumented buildings to strong earthquake ground motions.

For each building the code recommended values for natural periods design base shears and drift indices were compared with those experienced by the buildings during the Northridge earthquake. Key response parameters and characteristics of each building was discussed. In light of the results presented in this report the following observations are offered:

1. Building code estimates of building periods were 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 were 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 forced 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 may 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 were contributing to damage or where caused by it.

4. Given the level of forces the building experienced, the overall drift ratios experienced were less than what would have been expected from ordinary design analysis techniques.

5. While the 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 the seismic response of a 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, 2^{nd} and/or 3^{rd} 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 (see the Performance Analysis folder of the information system) which indicates 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. As mentioned earlier, however, in most cases maximum forces and maximum displacements do not occur at the same time. The 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 limit the application of static lateral distribution to the regular buildings of less height.

7. Except for the case of the 6-story Sylmar 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 examples photos of equipment mounted at the roof of the 3-story commercial building or the Van Nuys 7-story hotel.

8. Except for buildings with observed structural damage, the period of the building as interpreted from the recorded data did not elongate significantly. In these cases, when period elongation did occur, the period came back to the vicinity of the initial value towards the end of the 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.

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APPENDIX A

-- SENSOR LOCATION MAPS --

APPENDIX B

-- BACKUP CALCULATIONS FOR SIMPLIFIED ANALYSES --

APPENDIX C

– ATC-38 DAMAGE ASSESSMENT FORM --