INTERPRETATION OF STRONG-MOTION EARTHQUAKE RECORDS OBTAINED IN AND/OR NEAR BUILDINGS

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PREFACE

The material contained in this report constitutes the Proceedings of a Workshop on Interpretation of Strong-Motion Earthquake Records Obtained In and/or Near Buildings. The Workshop was sponsored by the National Science Foundation and was held on April 1 and 2, 1980 at the San Francisco Airport Hilton. The main purpose of the Workshop was to review existing building strong-motion instrumentation programs, to document existing procedures for processing and interpreting data from those programs, and to identify ways to improve data acquisition, analysis, and interpretation techniques.

The overall Workshop program and general background information are summarized in the Introduction (Chapter 1); Chapters II through V discuss the main subtopics of the Workshop; and Chapter VI provides a summary of comments plus specific recommendations.

Numerous individuals contributed to the success of the Workshop. We are particularly grateful to the members of the Steering Committee--Eric Elsesser, Robert E. Englekirk, Michael P. Gaus, and Anestis S. Veletsos--who provided valuable technical guidance and served as a sounding board for the organizers.

Gary C. Hart
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SUMMARY

This report presents a state-of-the-art summary of the various components of existing building strong-motion earthquake instrumentation programs. Instrumentation location, data analysis, and design applications are discussed. Recommendations are proposed.

Key words: earthquake engineering, buildings, structural dynamics, strong-motion, instrumentation.

CHAPTER I

INTRODUCTION

by

G. C. Hart*, C. Rojahn**, and J. T. P. Yao***

Prior to the 1964 Alaska earthquake, relatively few strong-motion earthquake records were available for use by structural engineers. Since then numerous strong-motion earthquake records obtained in and/or near buildings have been collected from various programs, including those of the U.S. Geological Survey, the California Division of Mines and Geology, the City of Los Angeles, and various other municipalities that adopted the strong-motion instrumentation provision of the Uniform Building Code. In the future, the need for insight into the interpretation of this data will become extremely important to the structural engineering profession. To gain a better understanding of existing procedures for collecting, processing, and interpreting strong-motion data from buildings and to identify ways to improve these procedures, the first author, with the assistance of the second and third authors, prepared a proposal and subsequently received a grant from the Problem-Focused Research Division of the National Science Foundation to support a Workshop on Interpretation of Strong-Motion Earthquake Records Obtained In and/or Near Buildings. The Workshop was organized with the guidance of a Steering Committee consisting of Eric Elsesser (Forell/Elsesser Engineers, San Francisco, California), Robert E. Englekirk (Ruthroff and Englekirk Consulting Structural Engineers, Los Angeles, California), Michael P. Gaus (National Science Foundation, Washington, D.C.), and Anestis S. Veletsos (Rice University, Houston, Texas). The organizers (Hart, Rojahn, and Yao) and the Steering Committee were responsible for the selection of participants and the establishment of the Workshop program.

While scientists are trying to improve their methods for the prediction of strong-motion earthquakes, engineers continue to learn more about seismic behavior or structures from available earthquake records and thereby improve their capability to design future structures. At present, there are three basic categories of building strong-motion instrumentation:

Category A: Ground Level Instrument In or Near Building
In this category on a single triaxial strong-motion instrument records the earthquake motion; it is located at ground level in or near the building under study.

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Category B: <u>Code-type Instrumentation</u>

In this category there are three triaxial instruments in the building located in the basement, near mid-height, and near the top. Such instrumentation is employed by the City of Los Angeles and other municipalities that adopted the strong-motion instrumentation provision of the Uniform Building Code (ICBO, 1979). Records from the 1971 San Fernando earthquake were obtained from buildings instrumented in this manner.

Category C: <u>Remote Accelerometer/Central Recording Instrumentation</u>

In this category the building is instrumented with a multi-channel accelerograph system consisting of remotely placed accelerometers connected by cable to one or two 13-channel recorders. Normally, the system contains 13 accelerometers but in rare instances may contain as few as 6 or as many as 26. This type of instrumentation is currently being employed in the structural instrumentation programs of the U.S. Geological Survey, the University of California at Los Angeles, and the California Division of Mines and Geology (Rojahn and Ragsdale, 1978). Most of these systems have been installed in accordance with the placement guidelines of Rojahn and Matthiesen (1977).

In the immediate future, most earthquake response records will be obtained from the above mentioned three categories of instrumentation, and it is therefore desirable to have a critique of each instrumentation type. More specifically, it is important to ask questions such as the following: Should we consider other types of instrumentation to learn more about structural behavior during strong-motion earthquakes? Is the present plan for the distribution and implementation of strong-motion earthquake instrumentation programs an optimum one? What possible improvements can we make in the future? Are the present analysis procedures and computer programs sufficient to give us all the desirable information from these records? Is it possible and feasible to make further improvements in this regard? How effective are research results involving available earthquake records used for structural design? How can practicing engineers communicate with researchers on research needs and practical application on a continuous basis? How should building damage be correlated with earthquake records? What conditions constitute various levels of damage, and how do these different levels of detected damage effect the structural safety and reliability in the future? Should there be standard testing and inspection procedures for new or repaired structures?

To help answer these and other questions, the following four state-of-the-art papers were prepared and presented during the Workshop:

"Building Instrumentation Programs" by R. B. Matthiesen

"Current Strong-Motion Ground Record Processing" by A. G. Brady

"Utilization of Strong-Motion Records in Building Design" by C. D. Poland

"Information Obtained From Strong-Motion Records" by J. D. Raggett

These papers were distributed to participants as they became available prior to the Workshop and were revised as necessary (based on the participants' review comments) during and after the Workshop; these proceedings contain the revised papers (Chapters II through V). Short papers (2 to 3 pages, approximately) on related topics submitted by Workshop participants, all of whom were invited to contribute, are in Appendix B. Chapter VI contains recommendations developed in the Workshop.

CHAPTER II

BUILDING INSTRUMENTION PROGRAMS

by

R. B. Matthiesen*

BACKGROUND

In 1932 the United States Coast and Geodetic Survey (C&GS) was authorized to initiate a program in engineering seismology intended to obtain the strong-motion data considered essential to the design of earthquake-resistive structures. In the first year of the program, several types of accelerographs had been designed and nine were installed at ground stations and in buildings in California. Less than 8 months later, the instruments installed at Long Beach, Vernon, and Los Angeles recorded the ground motions from the 1933 Long Beach earthquake. These first useful records of damaging earthquake ground motions indicated amplitudes as large as 0.25 g and provided an impetus for rapid development of the program. In the next three years, the network was expanded to 50 installations primarily in the San Francisco Bay area and in southern California but with a few in other seismically active regions of the western United States. The first building to be instrumented was the Bank of America Building in San Jose in September of 1932. The instrumentation consisted of a C&GS standard instrument located in the basement and another on the 13th floor. By 1935 instruments to record the response of buildings had been installed in 4 buildings in San Francisco, one in Oakland, one in San Jose, and 3 in Los Angeles. Between 1936 and 1963, the number of installations increased gradually while improvements were being made in the existing instrumentation.

In 1963 the first commercial accelerograph became available. The design of this instrument overcame many of the deficiencies in the design of the original C&GS instrument and ushered in an era of rapid development as several other agencies and organizations began to establish programs to serve their specific needs. The Bureau of Reclamation (USBR) initiated an expansion of the cooperative program it started in the 40's; the California Department of Water Resources (CDWR) began a cooperative program with the C&GS to obtain records of strong ground motion and the response of structures in the State Water Project; the Army Corps of Engineers districts in California started a cooperative program with the C&GS to instrument dams. In 1965 the City of Los Angeles began to require that strong-motion accelerographs be installed in all high-rise buildings. Subsequently a similar provision was added as an appendix to the Uniform Building Code (UBC) and adopted

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by many cities in California and other western states. Initially these instruments were maintained by the C&GS, but the number of instruments grew more rapidly than the number of personnel or budget to maintain them. With the transfer of the C&GS program to the Geological Survey (USGS) with funding from the National Science Foundation (NSF), the maintenance of instruments required by code has been gradually phased out of the federally supported program. In the late 60's and early 70's the Earthquake Engineering Research Laboratory (EERL) at Caltech acquired a number of accelerographs, many of which were incorporated directly into the C&GS program. Others were deployed in special arrays in the Imperial Valley, along the San Jacinto fault, throughout the transverse ranges, and in Bear Valley. Following the San Fernando earthquake, the Veterans Administration established a nationwide program to install instruments at each of its facilities in potentially active areas; the Corps of Engineers expanded its program to include active areas outside of California; UCLA initiated a program of building instrumentation at UCLA and Century City; the University of Nevada, University of Alaska, and Lamont-Doherty Geological Observatory acquired accelerographs for use in ground motion studies in their spheres of influence; and the Federal Highways Administration began to support the instrumentation of bridges. In 1972 the State of California established the California Strong-Motion Instrumentation Program (CSMIP) operated by the Office of Strong-Motion Studies (OSMS) of the California Division of Mines and Geology (CDMG). This is a general purpose program designed to obtain records of strong ground motions and the response of representative structures to such motions. Most recently, the University of Southern California (USC) began to develop an extensive network of instruments in the greater Los Angeles area to provide data for the study of the influence of subsurface geology and local site conditions. As a result of this general expansion of strong-motion programs there are over 2,000 accelerograph installations located in 38 states as indicated in figure 1. Most of these accelerographs are located at, in, or near buildings, although many are located at, in, or near other structures.

The various agencies supporting strong-motion instrument programs have had different objectives and are subjected to different constraints. Some desire research programs directed toward the understanding of basic problems in engineering seismology, whereas others are concerned with a regulatory function directed toward the monitoring of the response of a facility so as to provide a basis for a decision regarding the continued operation of the facility. An example of the former is the original C&GS program, whereas an example of the latter is the City of Los Angeles code program. The C&GS/USGS assisted in the planning and operation of these programs until they grew to such a size that the C&GS personnel were excessively overburdened; then the funding agencies provided or were requested to provide maintenance of their own instruments; but in many cases there is a sharing of instrument maintenance for the mutual benefit of both agencies. The USGS attempts to maintain files of information about all of the stations installed, events recorded, and records recovered by each of the

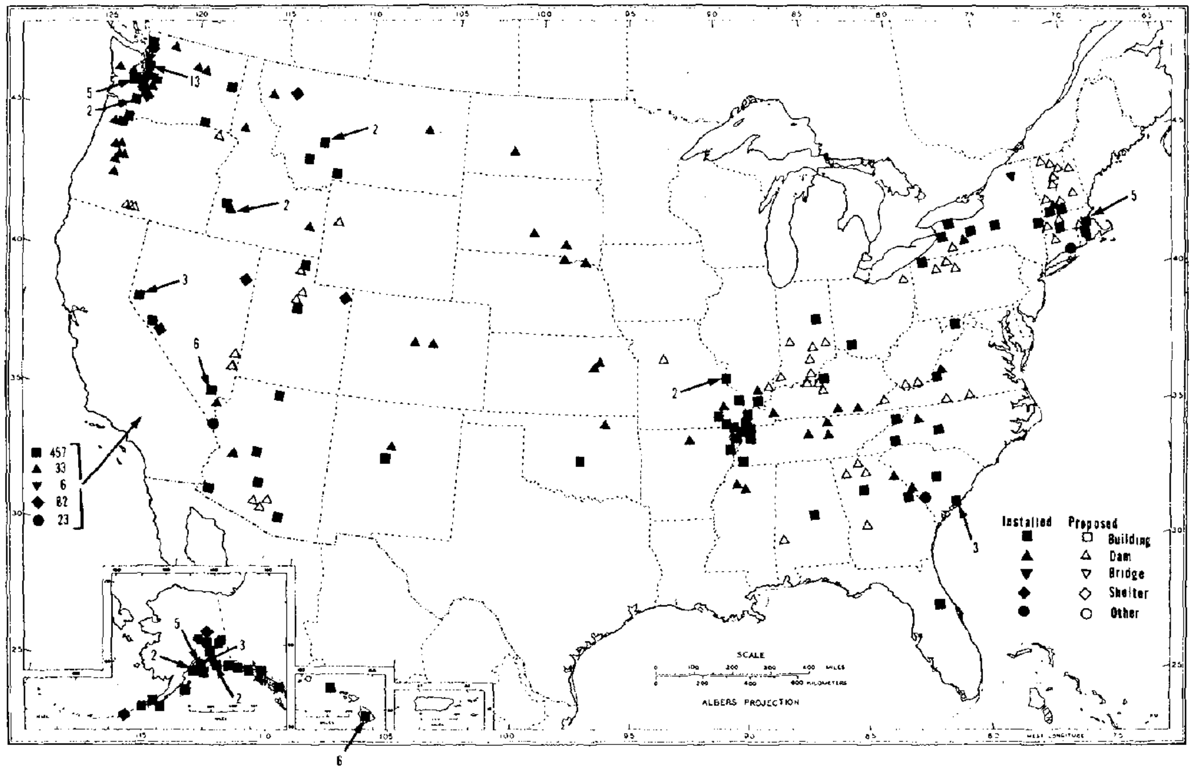

Figure 1.- Locations of accelerographs installed as of January 1, 1980.

programs and to provide this information to anyone interested through a computerized Strong-Motion Information Retrieval System (Converse, 1978).

As the nature of the strong ground motion and the response of structures to such motion has become better understood, the need for multiple ground motion records from each event and of carefully planned instrumentation of structures has been recognized. Concentrated arrays of instruments to measure ground motion have been planned in several of the more active areas (Matthiesen, 1978), and the instruments installed on structures are being tailored to the specific structure and to a particular aspect of structural response that is to be studied (Raggett and Rojahn, 1978; Rojahn and Matthiesen, 1977). The concentration of instruments in active areas has brought about a dramatic increase in the number of significant records that can be used for studies of ground motion and a less dramatic but significant increase in the number of records of building response.

CURRENT BUILDING STRONG-MOTION INSTRUMENTATION PROGRAMS

The following is a brief summary of those instrumentation programs that include the instrumentation of buildings as part of the program. The programs are described in order of their creation.

Seismic Engineering Program, NSF/USGS

Although the name and some of the objectives have been changed, this program is a continuation of the original program that began in 1932 under the Seismological Field Survey of the C&GS. As indicated above, building instrumentation was included in the program from the start, and the C&GS initially provided for maintenance of the instruments required by code. By 1972 the number of instruments required by code was approximately half of the total number of instruments then maintained by C&GS. During the budget review when the program was transferred to the USGS with program management and funds provided by NSF, a decision was made to phase the maintenance of code-required instruments out of the program. At the same time, a decision was made to include about 20 instrumented buildings in the program, but to improve the instrumentation over that which existed or was required by code. As a result of these decisions, general concepts for improved instrumentation of buildings evolved and instrument manufacturers were encouraged to develop multi-channel recorders with remote transducers so as to provide the measurements thought to be most significant for the analysis of building response. At the present time the program operated by the USGS for NSF has been reduced to only the 5 buildings indicated in table 1, while the personnel await additional funding to expand the building instrumentation part of the program. In addition, the program personnel advise and assist other agencies and organizations, process and archive any significant strong-motion records, and attempt to keep track of and disseminate information about all strong-motion programs in the western hemisphere.

Table 1.- Buildings maintained under NSF/USGS program

Station Identification				
USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
2750	Alaska Hospital Anchorage, AK		7-story bldg	Grnd level, 4th, 7th
2716	Anchorage Westward Hotel Anchorage, AK	Glcl out-wash	22-story bldg	Basement, roof
1162	Pleasant Vly Pumping Plant Fresno, CA	Alluvium	Pumping plant	Bsmt, grnd flr, roof, swtchyd
634	Bechtel Bldg Norwalk, CA	Alluvium	7-story bldg	Grnd level, roof, grnd (ff)
1446	Standard Oil Bldg San Francisco, CA		41-story bldg steel frame	2nd bsmt, grnd flr, 25th, 34th, 42nd

California Department of Water Resources (CDWR)

Since 1964 the CDWR has supported a cooperative program with the C&GS and subsequently with the USGS to obtain strong-motion records at facilities in the State Water Project. Ten pumping plants are included in this program as indicated in table 2. These buildings typically have a massive base below grade which includes an intake level and pump-room level. The motor room is generally above grade where the typical building is a single-bay moment-resistant steel frame in the transverse direction and a multi-bay braced frame in the longitudinal direction. The maintenance of the instruments in these buildings is shared by both CDWR and USGS personnel, but the significant records are processed and archived by the USGS.

City Building Code Programs

As indicated above, the City of Los Angeles began requiring that high-rise buildings be instrumented in 1965. Sixteen other cities in California and several in other states have a similar requirement based on an appendix to the UBC. Initially, the C&GS maintained all of these instruments and archived the records. Since 1973 the maintenance of the instruments in buildings in cities other than the City of Los Angeles has been the reponsibility of the building owner under the provisions of the UBC. At present, some building officials require that the instruments be maintained and others do not; some building owners want the instruments maintained and others don't care. There are at least 300 buildings that have been instrumented as a result of building code requirements (table 3). The USGS archives those records that are made available by the building owners.

California Institute of Technology (Caltech)

Faculty members at Caltech have been closely associated with the strong-motion instrumentation programs since the start of the C&GS program in 1932. As indicated above, the Caltech Earthquake Engineering Research Laboratory acquired numerous strong-motion accelerographs in the late 60's and early 70's and installed these in arrays to study ground motion or structural response. Most of the installations for obtaining ground motion data have become part of the NSF/USGS program. Caltech continues to operate about 20 instruments in the Pasadena area including those in Millikan Library and the JPL building (table 4). After processing, the records are turned over to the USGS for permanent archival.

Veterans Administration (VA)

This program began in 1972 as a part of the program of the Veterans Administration to review the seismic safety of VA buildings after the San Fernando earthquake. The basic program is nationwide and involves a single accelerograph located in the basement of each designated building. At five sites in California and at the VA Hospital in Salt

Table 2.- Buildings instrumented under the California Department of Water Resources program

Station Identification				
USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
1030	Delta Pumping Plant Alameda Co, CA		3-level plant	Basement, ground level
620	Devil Canyon Pwr Plnt San Bernardino Co, CA		Power plant	Basement, ground level
1142	Dos Amigos Pmpng Plnt Merced, CA	Sandstone	3-level plant	Levels 1 & 4
992	Edmonson Pumping Plant Kern Co, CA		3-level plant	Bsmt, grnd lvl, grnd station
994	Oso Pumping Plant Los Angeles Co, CA		3-level plant	Basement, ground level
585	Pearblossom Pump Plant Los Angeles, CA		3-level plant	Basement, main level
1144	San Luis Pump Plant Merced Co, CA	Sandstone & cnglmrt	5-level plant	2nd level, 5th level
1125	Wheeler Ridge Pumping Plant Kern Co, CA		4-level plant	Levels 1 & 4
1126	Wind Gap Pump Plant Kern Co, CA		4-level plant	Levels 1 & 4

Table 3.- City building code programs

Since the instruments that have been installed under the requirements of most of the city building code programs are not being maintained, only a brief summary is given of the total number of buildings in which instruments have been installed.

City	No. Bldgs	City	No. Bldgs
Alhambra, CA	1	Marina del Rey, CA	1
Anchorage, AK	1	Menlo Park, CA	1
		Montebello, CA	1
Bakersfield, CA[a]	3		
Berkeley, CA	2	Newport Beach, CA	7
Beverly Hills, CA	16	Norwalk, CA	1
Burbank, CA	1		
Burlingame, CA	2	Oceanside, CA	1
		Orange, CA	3
Coronado, CA[b]	7	Oxnard, CA	1
Culver City, CA[b]	3		
		Palo Alto, CA	3
El Segundo, CA	5	Pomona, CA	3
Emeryville, CA	2		
		Reno, NV	1
Fremont, CA	1	Riverside, CA	1
Fullerton, CA	2		
		San Bernardino, CA	1
Garden Grove, CA	1	San Dimas, CA	1
Glendale, CA	2	San Mateo, CA	2
		San Rafael, CA	1
Hayward, CA	1	Santa Ana, CA	5
		Santa Clara, CA	2
Inglewood, CA	3	Santa Maria, CA	1
Irvine, CA	1	Santa Monica, CA	3
		Santa Rosa, CA	3
Laguna Hills, CA	2		
Long Beach, CA	1	Torrance, CA	2
Los Angeles, CA[c]	200		
		West Covina, CA	1
		Whittier, CA	1

[a] Maintained by Bakersfield City College.
[b] Maintained by owners in three cases.
[c] Maintained by the City of Los Angeles.

Note: Many buildings instrumented under city building codes recently are being maintained by Kinemetrics, but no summary is available at this time.

Table 4.- Buildings instrumented under the California Institute of Technology (CIT) program

Station Identification USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
267	Jet Propulsion Lab Pasadena, CA		9-story bldg steel frame	Basement, roof
264	Millikan Library, CIT Pasadena, CA		9-story bldg rc frame	Basement, roof

Table 5. - Buildings Instrumented under the Veterans Administration program

Station Identification USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
1226	V A Hospital, Bldg. 62 Livermore, CA	Alluvium	6-story bldg	Basement, roof
1448	V A Hospital Martinez, CA		4-story bldg reinf conc	Basement, roof
1227	V A Hospital, Bldg 1 Palo Alto, CA	Alluvium	6-story bldg	Basement, roof
1447	V A Hospital, Bldg 5 Palo Alto, CA	Alluvium	4-story bldg reinf conc	Basement, roof
1225	V A Hospital San Francisco, CA	Franciscan rock	6-story bldg	Basement, roof
2210	V A Hospital Salt Lake City, UT	Alluvium	7-story bldg	Basement, 7th floor

Lake City, additional instruments have been installed to measure the response of one or more buildings (table 5). The program is operated by the USGS, which also processes and archives the records.

University of California at Los Angeles (UCLA)

The Full Scale Earthquake and Wind Laboratory created in 1974 at UCLA operates the UCLA and Century City Project to obtain records of both wind and seismic excitation in three buildings in Century City and one on campus all centrally recorded on magnetic tape on campus (table 6). In addition, UCLA took over the responsibility for maintenance of several buildings in Century City that had been instrumented under the City of Los Angeles code requirement. Harmonic, ambient, and seismic data from this project are stored on magnetic tape in the laboratory and are available to other researchers at reproduction cost.

California Strong-Motion Instrumentation Program (CSMIP), CDMG

The CSMIP program operated by the OSMS of CDMG includes instrumentation for studies of both strong ground motion and the response of representative structures to such motion. This program was an outgrowth of a recognition that the code-required building instrumentation programs do not provide for the instrumentation of low-rise buildings (which comprise the greatest number of buildings and house the greatest number of persons) nor did the federal program provide adequte arrays for the measurement of ground motion throughout California. If it fulfills its stated objectives, this program will be the largest operated by a single agency anywhere in the world. At the present time, over 300 instruments have been installed to measure strong ground motion and instrumentation tailored to study the specific response of each building has been installed in numerous buildings (table 7). Under the long range plan for this program, about 400 buildings will be instrumented. The planning of the program has included the development of general plans as to the types of buildings that should be instrumented, the relative numbers of buildings of each type, and the regions of the State in which an appropriate rate of return of data is to be expected (Rojahn and Ragsdale, 1978; Hart and Rojahn, 1979). The general plan as to the type and location of the instruments to be used follow the concepts developed under the NSF/USGS program. In each case, specific instrumentation plans are prepared for the particular building to be instrumented. A task group comprised of the design engineer, the building owner or his representative, a member of the Structural Engineers Association of California, a member of the advisory committee to the program, and a representative of the CSMIP meet at the building site to make detailed recommendations for the locations of the transducers to be installed in the building. In 1977, the CSMIP was assigned the responsibility to process and archive the data that it obtains.

City of Tacoma, Washington

In 1974, the City of Tacoma adopted an instrumentation program with

Table 6.- Buildings maintained under the UCLA program

Station Identification				
USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
184	Los Angeles 1900 Avenue of Stars	Alluvium	27-story bldg steel frame	Basement, 16th, roof
187	Los Angeles 1901 Avenue of Stars	Alluvium	20-story bldg steel frame	Basement, 9th, roof
985	Los Angeles 2020 Avenue of Stars	Alluvium	6-story bldg steel frame	Ground, 6th
984	Los Angeles 2040 Avenue of Stars	Alluvium	6-story bldg steel frame	Ground, 6th
983	Los Angeles Century City Plaza	Alluvium	7-level grge reinf conc	Basement
982	Los Angeles 2029 Century Park E	Alluvium	44-story bldg steel frame	30th, 43rd
981	Los Angeles 2049 Century Park E	Alluvium	44-story bldg steel frame	30th, 43rd
425	Los Angeles 1800 Century Park E	Alluvium	15-story bldg reinf conc	Basement, 5th, roof
685	Los Angeles 1801 Century Park E	Alluvium	26-story bldg steel frame	Basement, 14th, 26th
440	Los Angeles 1880 Century Park E	Alluvium	16-story bldg steel frame	Basement, 7th, roof
419	Los Angeles 1888 Century Park E	Alluvium	21-story bldg steel frame	Basement, 14th, 21st
193	Los Angeles 2080 Century Park E	Alluvium	17-story bldg rc mom frame	Basement, 10th, roof
542	Los Angeles 1801 Century Park W	Alluvium	17-story bldg steel frame	Basement, 6th, roof
5191	Los Angeles Life Sci Bldg, UCLA	Alluvium	7-story bldg steel frame, steel wall	Bsmt, 3rd, 4th, 6th, 7th
613	Los Angeles 10100 Santa Monica	Alluvium	26-story bldg steel frame	Basement, 14th

Table 7.- Buildings instrumented under the California Division of Mines & Geology (CDMG) program

Station Identification				
USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
1467	Envirotech Systems Belmont, CA	Alluvium	2-story bldg steel frame	Basement, roof
5090	Imperial County Services Building El Centro, CA	Alluvium	6-story bldg rc frm & shrwl	Grnd flr, 2nd, 4th roof, grnd (ff)
1419	Capwell's Dept Store El Cerrito, CA	Alluvium	3-story bldg	Grnd floor, roof
1524	Admin Bldg, HSU Hayward, CA	Alluvium	13-story bldg steel frame	Bsmt, 1st, 2nd, 5th, roof
1488	Cambiaso Winery Healdsburg, CA		1-story bldg cnc blk wls	Grnd floor, roof
5091	Hemet City Library Hemet, CA		1-story bldg msnry walls	Grnd floor, roof
5092	Hemet Valley Hospital Hemet, CA		4-story bldg rc shrwls	Bsmt, 2nd, roof
1489	LA DWP Bldg (Garage) Independence, CA		1-story bldg tilt-up walls	Grnd floor, roof
5150	Engineering Bldg, UCI Irvine, CA		7-story bldg shrwls	Bsmt, plz, mz, 3rd, 6th, roof
5149	Harbor Admin Bldg Long Beach, CA		7-story bldg steel frame	Grnd floor, 2nd, roof,
5144	Engineering Bldg, LBSU Long Beach, CA		5-story bldg rc shrwls	Bsmt, 2nd, roof
5152	Bullocks, Century City Los Angeles, CA		3-story bldg on 2-story garage	Bsmt, mall, 2nd, roof

Table 7.- Buildings instrumented under the CDMG program (continued)

Station Identification		Site Geology	Structure Type/size	Instrument location(s)
USGS Sta. No.	Name			
133	Hollywood Storage Bldg Los Angeles, CA	Alluvium	14-story bldg rc shrwl	Bsmt, 8th, 12th, roof, grnd (ff)
140	Math-Science Bldg, UCLA Los Angeles, CA		5-story bldg	Bsmt, 1st, 3rd, 5th, roof
1490	Mammoth Lakes High Schl Mammoth Lakes, CA		1-story bldg wood frame	Grnd floor, roof
1457	Calrus Bldg Oakland, CA	Alluvium	3-story bldg wood frame	Grnd floor, roof
1500	Oak Center Towers Oakland, CA		11-story bldg cnc blk wls	Grnd floor, 2nd, 6th, roof, grnd (ff)
1456	Title Insur & Trust Oakland, CA	Alluvium	2-story bldg st frm, shwl	Grnd floor, roof
5089	Holiday Inn Palmdale, CA		4-story bldg cnc blk shrwl	Grnd floor, 3rd, roof
5132	Kiewit Bldg Palm Desert, CA		4-story bldg pc fm & shrwl	Grnd floor, 2nd, roof
5135	Desert Hospital Palm Springs, CA		4-story bldg steel frame	Bsmt, 2nd, 3rd, roof
1469	1900 Embarcadero Bldg Palo Alto, CA	Bay mud	2-story bldg wood frame	Grnd floor, roof
1499	Piedmont Jr High Schl Piedmont, CA		3-story bldg rc shwls	Grnd floor, 2nd, roof, grnd (ff)
1510	Savings & Loan Pleasant Hill, CA		3-story bldg rc	Grnd floor, 3rd, roof

Table 7. - Buildings instrumented under the CDMG program (continued)

Station Identification USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
1468	Canada College Bldg Redwood City, CA		3-story bldg reinf conc	Grnd floor, roof
5145	Riverside Co Admn Cntr Riverside, CA		13-story bldg rc sw & st fm	Bsmt, 3rd, 7th, roof
5134	Hilton Hotel San Bernardino, CA		6-story bldg rc shrwls	Grnd floor, 3rd, roof
5133	Library, SBSC San Bernardino, Ca		5-story bldg rc flat slab	Bsmt, 3rd, roof
275	SDGE Office Bldg San Diego, CA		22-story bldg steel frame	Bsmt, 3rd, 12th, 20th, 21st, 22nd
1506	Great Western S & L San Jose, CA	Alluvium	10-story bldg rc & frm shwl	Bsmt, 2nd, 5th, roof
1508	Santa Clara Co Adm Bldg San Jose, CA		12-story bldg steel frame	Bsmt, 2nd, 7th, 12th, roof
1507	Town Park Towers San Jose, CA	Alluvium	10-story bldg rc shrwl	Grnd floor, 6th, roof
1509	Firemans Fund Ins Bldg San Rafael, CA		3-story bldg rc sw & st fm	Grnd floor, 2nd, 3rd, roof
1418	Eastman Kodak Bld San Ramon, CA	Alluvium	1-story bldg	Grnd floor, roof
5137	Freitas Bldg Santa Barbara, CA		4-story bldg steel frame	Bsmt, 2nd, roof
5093	North Hall, UCSB Santa Barbara, CA		3-story bldg rc shrwl	Grnd floor, 3rd, roof, grnd (ff)
1460	West Valley College Gym Saratoga, CA	Alluvium	1-story bldg	Grnd floor, roof

Table 7. - Buildings instrumented under the CDMG program (continued)

Station Identification USGS Sta. No.	Name	Site Geology	Structure Type/size	Instrument location(s)
5148	Union Bank Bldg Sherman Oaks, CA	Alluvium	12-story bldg rc frame	Bsmt, 7th, roof
1466	Kaiser Medical Bldg So San Francisco, CA	Fill	4-story bldg steel frame	Bsmt, grnd fl, 2nd, roof
1487	Truckee Elem School Truckee, CA		1-story bldg cnc blk walls	Grnd flr, roof
5200	Holiday Inn Ventura, CA		12-story bldg rc shrwl	Grnd floor, 4th, 8th, roof, grnd (ff)
5168	Hall of Justice Ventura, CA		4-story bldg st K-brc frame	Bsmt, 2nd, roof
1530	Fidelity Sav & Loan Walnut Creek, CA		10-story bldg rc fm & shwl	Grnd fl, 3rd, 8th, roof

funding patterned after that of the CSMIP in which a small fee is added to the building permit. This fund is to be used for the instrumentation of representative types of structures. Although a general plan for this program has been prepared, the funds that are collected accrue so slowly that no buildings have been instrumented as of this date. Discussion between the City of Tacoma and the USGS regarding the type of instruments and their maintenance continue to occur occasionally.

USEFUL INFORMATION OBTAINED TO DATE

In conjunction with building analysis and design, the studies which utilize strong-motion records include:

- studies of the spectral characteristics of strong ground motion at or above the levels associated with the initiation of damage,
- studies of the variations of strong ground motions over distances of the order of a characteristic plan dimension of buildings,
- studies of the influence of the supporting soil and foundation on building response at or above the initiation of damage, and
- studies of building response in the range at and above the initiation of damage.

This list deliberately emphasizes that records of ground motion and building response are desired at or above the levels associated with the initiation of structural damage. Such records are thought to be of highest priority and their acquisition must be emphasized at the planning stage. If plans are made for the acquisition of such data, records of lower level motion will be obtained in the normal course of data acquisition, but if such plans are not made the highest priority, records of such response may not be obtained in a reasonably short time.

The evolution of the concept of the response spectrum and the processing of the strong-motion data that began at Caltech in the late 40's with an analog computer and was extended in the late 60's utilizing digital computers provides the basis for much of the current state of knowledge about the spectral characteristics of strong ground motion. The variation of the spectral characteristics of ground motion with the type of source, the travel path, and the local site conditions is a topic of considerable current research in engineering seismology and earthquake engineering. The records recently obtained from the Imperial Valley earthquake provide a valuable set of near-field strong ground motion data that fills a gap in the range of data available for studies of the spectral characteristics of ground motion. Even though this research is far from complete, the concepts and preliminary conclusions from such research have been utilized in the development of building design regulations.

Only two sets of strong-motion records that may be used to study the variation of input motion across a building are known to exist. One set was obtained in Japan in 1975 and a second set was obtained from the Imperial Valley earthquake of October 1979. These two data sets are far from being sufficient, and additional arrays are required. In addition, the existing data sets do not allow for an accurate determination of whether or not significant ground rotation occurs over short distances as a result of earthquakes. The latter question has been raised by both engineers and seismologists.

Although the buildings that were instrumented in the 30's generally have not provided significant records of building response, those from the Southern Pacific Building in San Francisco, which is supported on piles on soft Bay mud, and those from the Hollywood Storage Building have provided records that are relevant to the study of the influence of the supporting soils on the structural response. In addition, an intepretation of the records of building response obtained during the San Fernando earthquake indicates that soil-structure interaction had a significant influence on the response of structures. A set of records obtained at the Pleasant Valley Pumping Plant in 1976 indicates a difference in the motion at the base of the building and the "freefield". During the October 1979 Imperial Valley earthquake, records were obtained from the Imperial County Services Building, which is pile supported on relatively soft soil. The dramatic failure of four columns of this building tends to cause the soil-foundation-structure interaction that is evident in the early portion of the record to be overlooked, but the record obtained at the base of the building is significantly different than that obtained from a nearby "freefield" instrument. Most recently, records obtained from the Livermore VA Hospital during the earthquake of January 1980 indicate that the motion at the base of the structure contains a significant component at the fundamental mode of the structure. This implies that significant soil-structure interaction occurred during the earthquake.

The buildings that were instrumented in the 30's provided some interesting records of low-level response, but no records of building response at levels at which damage initiates were obtained until 1971. The records obtained during the San Fernando earthquake from the Holiday Inn on Orion Blvd., the Bank of Califonia on Ventura Blvd., and the Holiday Inn on Morengo Street provided the first such data. The set of building response data obtained during the event is also important in that it provides an indication of the damping inherent in buildings at levels of response up to the level at which damage initiates. Subsequently, the records obtained by the CSMIP from the Imperial County Services Building during the October 1979 Imperial Valley earthquake have provided even more graphic data on building response that includes the failure of the first-story columns. The records obtained during the February 1980 earthquake from the Livermore VA hospital, in which the reinforced concrete shearwalls were cracked, and the one-story Eastman Kodak warehouse in San Ramon, which is instrumented to study the diaphragm action of the roof, provide additional evidence at high levels of structural response. This latter set of records is one of a series that are available from the CSMIP to study the response of low-rise buildings.

CHAPTER III

CURRENT STRONG-MOTION GROUND RECORD PROCESSING

by

A. G. Brady*

For the purposes of this workshop the precedures have been illustrated by a descriptive guide through the processing decisions for the set of records recovered from the Imperial Valley earthquake of October 15, 1979.

INTRODUCTION

Among the first strong-motion accelerographs installed in the early 1930's by the Seismological Field Survey (the predecessor of the Seismic Engineering Branch, U.S. Geological Survey) was the instrument in El Centro at the Southern Sierra Power Company Terminal Station, 302 Commercial Avenue, El Centro. This strong-motion station, subsequently known as the Imperial Valley Irrigation District Substation, has been occupied by a standard accelerograph ever since, as the USGS network of more recent 70-mm film recorders expanded throughout the Imperial Valley. The seismicity of this part of California, and the accompanying records recovered from the El Centro station since the thirties, have resulted in the placement of recorders throughout the Imperial Valley, including the El Centro array transverse to the Imperial Fault, a differential ground motion array of digital recorders (Bycroft, 1980) in El Centro, and instruments in several of the towns from the Mexico border north to the Salton Sea (Matthiesen and Porcella, 1980). The main shock records of the Imperial Valley, 1979, earthquake, provided by the 70-mm film recorders, are the subjects of this report. The recorders contain three accelerometers in two horizontal and the vertical directions. The three traces are written on photographic film, together with at least one reference trace and a time mark trace giving half-second time intervals after triggering. All have WWVB radio receivers for the recovery of absolute time, and are designed to run for at least 60 sec after triggering to allow the complete time code to be recorded.

The selection of twenty-two October 15 main-shock records to digitize from amongst the 30 triggered records in the USGS network (Matthiesen and Porcella, 1980), was made on the basis of the distance of the station from the nearest point on the 1940 Imperial Fault trace being less than approximately 30 km. Peak accelerations beyond this distance reached no more than 5 percent g except for Coachella Canal Station 4, which showed the amplification typical of soil-structure

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interaction phenomena (Porcella and Matthiesen, 1979), but is nonetheless included in this report. Two records within the 30 km distance are not included due to expected difficulties in digitization and processing (namely El Centro Station 9) or to less than 5 percent g peak acceleration (Salton Sea Wildlife Refuge). These two, and possibly the remaining 6 of the 30 main-shock records, will be processed at a later date, particularly if a specific research interest is discovered. The processed 22 records are the following (see figure 1, based on Matthiesen and Porcella, 1980): El Centro stations 1 through 8; El Centro stations 10 through 13; Bonds Corner; El Centro Differential Array; Brawley Airport; Holtville; Calexico; Parachute Test Site; Calipatria; Superstition Mountain; Plaster City; and Coachella Canal Station 4.

In the following sections we describe in some detail the special circumstances that contributed to the digitizing and processing decisions with regard to this set of 22 70-mm film recordings. Digitizing was performed on a laser-operated trace-following automatic scanner. The maximum duration processed for this report was 40 sec, initially digitized in approximately 10 sec frames. Difficulties with faint traces in the first 10 sec of several records close to the fault and confirmation of the quality of the reassembly of adjacent frames of digitized data are reported.

Prior to the processing of the digitized data, tests were performed to assist in choosing the parameters for the data correction procedures, namely, the long period limit (finally chosen as 6 sec), the high frequency limit (the reasonably standard 23 Hz), and the sampling frequency (finally chosen at 100 samples per second). The digital data are available on magnetic tape, originally provided as two seven-track tapes containing 11 records each, from the Environmental Data and Information Service, NOAA, Boulder, Colorado 80302, while the computer plots are available as a U.S. Geological Survey Open-File Report No. 80-703. In addition to referring to this Open-File Report, the reader is directed to the U.S. Geological Survey Professional Paper on the Imperial Valley Earthquake and to the preliminary report on the records (Porcella and Matthiesen, 1979).

DIGITIZATION PROCEDURES

Digitizing was carried out on contact prints prepared from the original records by a trace-following automatic digitizer capable of averaging 600 points per second at unequal time intervals. Operator intervention is required for decisions at trace intercepts, and for selecting visually located points when the trace is too pale for automatic detection. The frame size of the equipment limits the record duration scanned at one set-up to approximately 11 1/2 cm, corresponding to 11 1/2 sec of elapsed time. Vertical butting lines are scribed on the prints at 9 1/2 cm spacing, allowing a 1 cm overlay at each edge of the frame, and these are digitized with each frame's data to facilitate the reassembly of records longer than one frame.

On many of the mainshock accelerograms an aftershock arrival occurs between 35 and 40 sec after triggering. To avoid confusion between the displacements from this aftershock and noise from processing the extremities of the main shock, the processing described in the following sections have been carried out in nearly all cases for 36 seconds contained in four frames of digitizing. Three of the records most distant from the fault trace (greater than 25 km), namely Superstition Mountain, Plaster City, and Coachella Canal No. 4 have been digitized for less than 30 sec. At these distances, the acceleration amplitudes are down at the level of 2 percent g at the end of the digitized duration.

The quality of the reassembly procedures, outlined in Porter and others (1978), was verified by the checks carried out on the consistency of the locations of the intersections of butting lines with reference traces when these intersections were repetitively digitized in successive frames. In addition, subsequent Fourier analysis indicates that there is no abnormal content at 9 1/2 sec period, whose existence would have been the first indication, not only of faulty reassembly, but also of optical distortions within the digitizing system.

The preliminary computer run that reassembles the traces from the individual digitizing of each frame also carries out some elementary Hanning smoothing (Blackman and Tukey, 1958) and removal of points to reduce the average density to approximately 125 points per second. All local peaks were retained during this step, including the maximum digitized peak. An exception to this occurs when manual intervention is required during digitizing and individual points are chosen by the operator that are so widely separated in the y-direction that the elementary Hanning smoothing clips them. Points at the reduced density are transferred to the stage 1 processing for uncorrected data (Hudson, 1976) and for subsequent processing according to current U.S. Geological Survey practice (Basili and Brady, 1978).

CONVERSION TO CORRECTED ACCELERATIONS

The correction of the basic acceleration ordinates derived, as described in the previous section, from the film record, involves the removal by filtering of high frequency noise, the removal by filtering of long period noise, the selection of a sampling frequency compatible with the high frequency filter, and the correction to take account of the dynamic characteristics of the transducer. The selection of the filter corners and the interpolation rate are discussed in this section; the instrument correction is a standard mathematical procedure for a viscously-damped one-degree-of-freedom oscillator (see, for example, Trifunac, 1962).

Major efforts have been made in data processing since the Caltech project was completed with the San Fernando, 1971, earthquake accelerograms (Hudson, 1976; Trifunac and Lee, 1973). Particular emphasis has been placed on the selection of the long period limit, beyond which all Fourier content is removed (Basili and Brady, 1978; Fletcher and others, 1980; and Trifunac and Lee, 1978), and on the the detail of the filtering procedures.

Several factors influenced the selection of the long period limit for the Ormsby filter used in the processing of the Imperial Valley digitized data. We will restrict the discussion to the close-in records, namely those within approximately 10 km of the nearest point on the 1979 fault trace (see Matthiesen and Porcella, 1980, and figure 1).

The stations at Calexico, Holtville, and El Centro Array Stations 3 and 11 (figure 1) are approximately equidistant from each end of the ruptured fault, and their strong-motion durations (the time span between the first and last peak greater than 0.10 g) should consequently be a good estimate of the rupture duration. The strong-motion duration of the records at these stations, of 5.7 to 10.8 sec (Porcella and Matthiesen, 1979), indicates that we should attempt to force the long period limit out to 6 sec. The record length of 36 sec provides an upper bound to the termination of the cut-off ramp in the period domain. The corresponding ramp in the frequency domain is located between 0.03 and 0.17 Hz. An Ormsby filter with this ramp (Df = 0.14 Hz) requires a filter weighting function length, L, of 14 sec, (that is, 2/Df) which is a high fraction of the record length of 36 sec. As a result, and in particular for those records with maximum amplitudes occurring, in some cases, as early as 3 sec after triggering, we might expect that processing noise becomes predominant as the 6 sec long period limit is approached.

A series of test runs were performed on the El Centro Array Station 7 record, recovered at ground level from a 1-story building at the Imperial Valley College in El Centro, to investigate the effect of varying the long-period filter parameters. The long period limit (cut-off period, T_c) was varied from 3.7 sec to 6 sec as indicated in table 1, which also lists the additional details of the filter parameters applied to the three components of this record. For this preliminary test the data are interpolated at 50 pts per second, and have been filtered to remove high frequencies above 23 Hz. The ramp in the frequency domain lies between f_t and f_c, the termination and cut-off frequencies; or in the time domain, where the ramp falls off hyperbolically, between T_c and T_t, the cut-off and termination periods. As we attempt to lengthen the cut-off period T_c, so that more long period content can be examined, we are forced into longer filter weighting function lengths, L. The plots of figure 2 show the effects of these longer filters with nine diagrams containing the plots of acceleration, velocity and displacement. The top row represents the 230° component (transverse to the fault), the middle row vertically up, and the lower row represents the 140° component (along the fault). The three columns represent cut-off periods of 3.7, 5, and 6

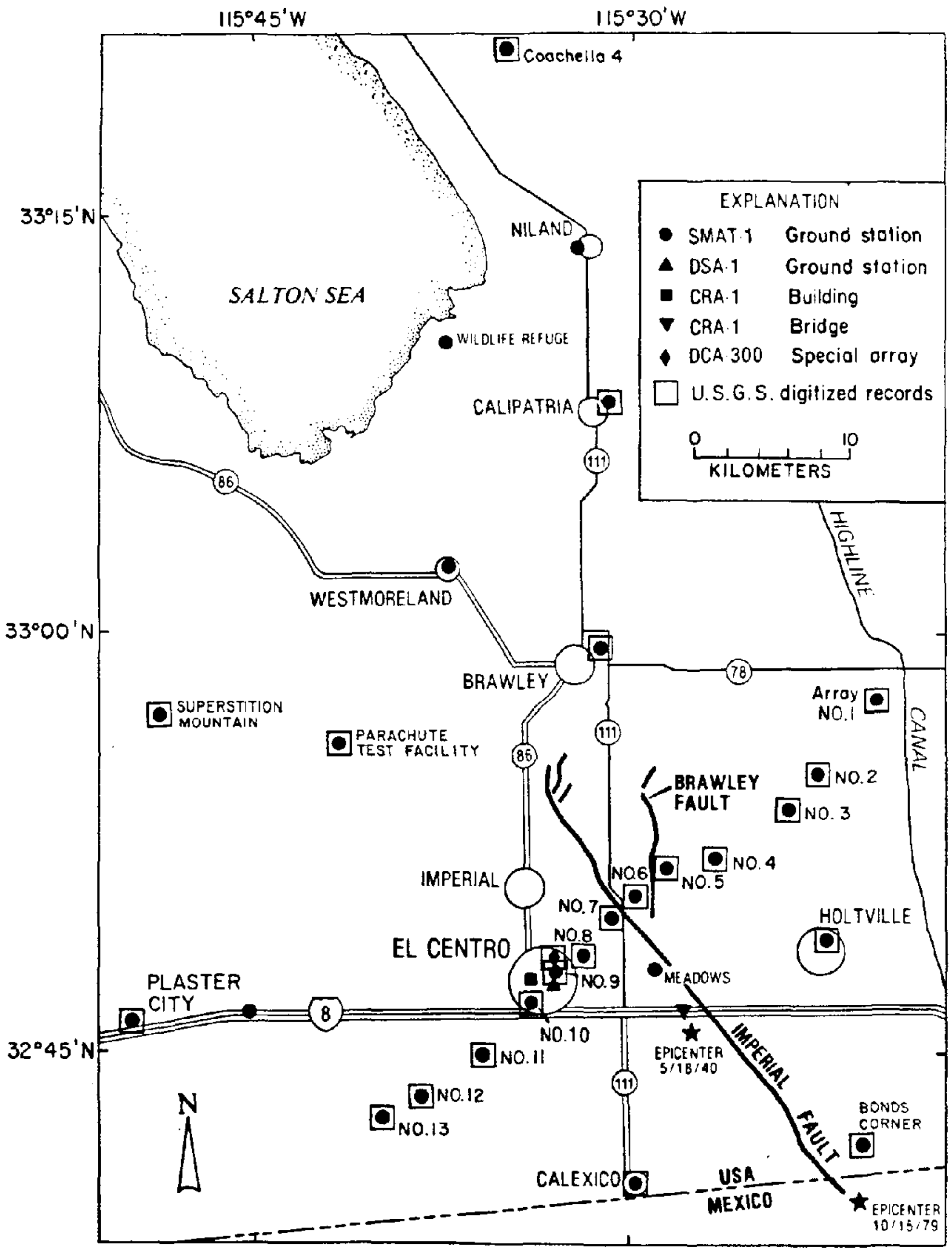


Figure 1.- Strong-motion stations in the Imperial Valley, California (from Porcella and Matthiesen, 1979).

sec. Taking into account the scale changes on the individual plots and our past experience with the detailed characteristics of this type of filter adjustment, we can assume that, so far as the first 5 sec are concerned, the first two components, 230° and Up, have not suffered unduly from lengthening the cut-off period, whereas the 140° component shows some additional motion arising either directly from filter processing, or from the additional entry of further long period signal contaminated to some extent by noise. Before we can conclude that the 6 sec filter is satisfactory, the following additional comments on the appearance of the time histories in figure 2 are necessary.

Firstly, the original recorded accelerogram 230° component does exhibit a pronounced trend of increasing acceleration amplitudes, in the negative direction, in the first 4 1/2 sec. This is transmitted through integration to the velocity and displacement which also exhibit the rather pronounced motion in this direction. Our previous experience with this behavior in corrected displacement time series has led us to conclude that the signal to noise ratio in the processed data has not been sufficiently high, and that these apparently excessive displacement amplitudes prior to the arrival of the shear wave were spurious. The appearance here of the motion in the original accelerogram, before any processing, confirms, however, that the displacements in figure 2 during the first 5 sec are based on a visible signal in the accelerogram, even though they change with the effects of the different filters.

Secondly, the 4.75-sec oscillations that are evident in the vertical component of displacement, from 5 sec into the record until the end, are visible also in the recorded acceleration. The amplitudes of the acceleration oscillations correspond to those evident here in displacement for the figures 2e and 2f, with 5- and 6-sec long-period filters. The 3.7-sec filter, on the other hand, reduces the content of a 4.75-sec Fourier component by 30 percent, as is confirmed in figure 2d. Thus these oscillations are true ground motion, and not processing noise.

Thirdly, the 140° component, in figures 2g, h, and i, exhibits the characteristic worsening behavior during the first 5 sec as the filter window length is increased. For the 6 sec filter, figure 2i, the filter is extending 7 sec on each side of its midpoint (table 1, L = 14 seconds), well into the high amplitude accelerations, and the resulting displacements caused some concern about forcing the long-period limit to 6 sec.

As a result of these preliminary tests, a selection of 11 close-in records was filtered according to the third scheme of table 1, with a view to ascertaining whether a search for 6-sec content was justified in light of the displacement distortions that could be expected, on some of the components (e.g., the 140° component above), due to mismatching of filter characteristics with record characteristics,

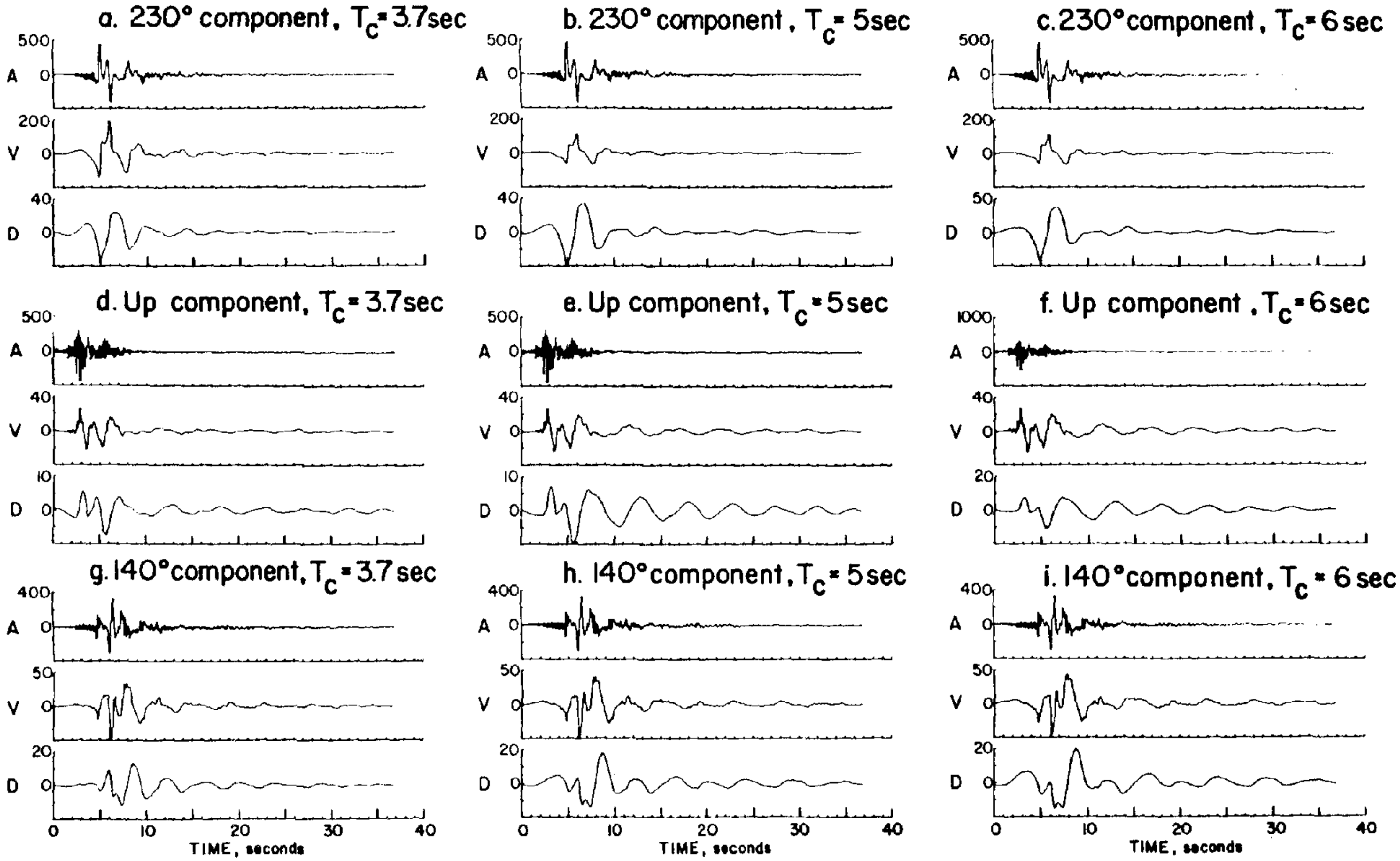


Figure 2.- Test-run plots of acceleration, velocity, and displacement time histories for 3 components recorded at El Centro Array Station 7. One set of plots is shown for each of 3 cut-off periods, T_c, studied.

Table 1.- Low frequency filter parameters for three filter schemes applied to the El Centro Array Station 7 October 15 main-shock record during test runs

Test Run	Frequency (Hz) Termination, f_t	Frequency (Hz) Cut-off, f_c	Period (sec) Cut-off, T_c	Period (sec) Termination, T_t	Df* (Hz)	L** (sec)
1.	0.07	0.27	3.7	14	0.20	10
2.	0.03	0.20	5	33	0.17	12
3.	0.03	0.17	6	33	0.14	14

*Df = filter ramp (f_c - f_t)
**L = weighting function length (2/Df)

Table 2.- Peak accelerations for several October 15 main-shock records at consecutive stages of processing

Station	Component	Scaled from original (g)	Uncorrected acceleration (g)	Corrected acceleration (cm/sec^2 (g)) 50 pts/sec	Corrected acceleration (cm/sec^2 (g)) 100 pts/sec
El Centro Array Station 7	230°	0.52	.47	450.7 (0.46)	453.6 (0.46)
	Up	0.65	.63	431.2 (0.44)	503.6 (0.51)
El Centro Array Station 6	230°	0.45	0.45	422.7 (0.43)	428.1 (0.44)
	Up	1.74	1.70	1566.5 (1.60)	1662.7 (1.69)
Bonds Corner	230°	0.81	0.78	766.0 (0.78)	770.4 (0.79)
	Up	0.47	0.44	318.6 (0.32)	347.7 (0.35)
El Centro Array Station 8	230°	0.50	0.48	436.0 (0.44)	457.4 (0.47)
	Up	0.55	0.48	339.3 (0.35)	405.9 (0.41)

and due to processing noise. Since the results were encouraging, we elected to utilize the filter parameters of scheme 3 (table 1) and proceeded with some further elementary testing at the high end of the frequency range of interest, which we shall now discuss, to determine if frequencies greater than 25 Hz ought to be included in the corrected data.

Until such time as the peak ground acceleration loses its currently held importance in the structural design and earthquake research professions, an explanatory note is required to discuss the discrepancies which exist between the peak acceleration values as they appear at different stages of processing. Table 2 lists these peaks for the transverse to the fault and vertically up components of four close-in records, namely El Centro Array Stations 6, 7, and 8, and Bonds Corner. The first column contains the scaled values (g units) from the original recordings, routinely listed in the Seismic Engineering Branch program report, and published earlier in Porcella and Matthiesen (1979). The second column contains the values (g units) from uncorrected automatic digitization, although some of the peaks were manually inserted by the operator at the correct locations because very faint traces caused automatic trace following to fail. The difference between these two sets of peaks can be attributed to the different way in which people and machines treat the detailed appearance of a peak on the record, and to the occasional partial loss of a lone manually digitized peak. In general, the automatic digitizer produces peak values slightly lower than the original scaled peaks by a few percent.

The third and fourth column-pairs list the peak values after the corrections for instrument characteristics and the low- and high-frequency filtering have been applied. In this case the low-frequency filter has a ramp from 0.03 to 0.17 Hz, and the high-frequency filter has a ramp from 23 to 25 Hz. The third column results from corrected data at 50 points per second, although internally the high-frequency filter and the calculations for the instrument corrections are performed at twice this density. The peak values are lost without exception, by as much as 30 percent, as reported by Hudson (1976) for the Caltech processing program. Possible reasons for this are interpolation at too wide a time interval, high-frequency filtering out of the high frequencies containing the peak accelerations, and the application of the instrument correction. The second and third can be mostly eliminated by noting from the original records that the approximate frequencies present during the recording of the peaks are within the range of 15 to 20 Hz, maximum, and that the instrumental natural frequency lies between 25 and 27 Hz. That interpolation density is probably the more important key in improving the relationship between corrected and uncorrected peaks, whether or not this improvement is warranted, is shown by interpolation at 100 points per second, which improves the recovery of the scaled peaks, as indicated in column 4 of table 2.

The detail of the highest peak value, El Centro Array Station 6 up component, scaled at 1.74 g, is indicated in figure 3 where the uncorrected and corrected points are plotted for a few hundredths of a second on either side of the critical portion. The 23-Hz filter has smoothed the uncorrected points as shown, eliminating the 1.74 g peak, and allowing a different peak to assume the role of the "corrected peak." The effect of interpolation at 100 points per second is evident, and the phase lag of the instrumental recording is clearly shown, being nominally one-quarter of the instrumental free period (here, 0.039/4 = 0.00975 sec). As a result of these interpolation tests the output for corrected acceleration, velocity, and displacement is given at equal time intervals of 0.01 sec.

A verification of the minor role that the inclusion of high frequencies (frequencies greater than 25 Hz) play in the recovery of scaled maximum peak values during correction procedures applied to this data set can be seen in table 3. Two components with particularly severe peak value loss during correction, namely the Bonds Corner up component and El Centro Array Station 8 (Cruickshank Road) 230° component, were passed through high-frequency filters of 35 Hz and 50 Hz with resulting peak acceleration, velocity and displacement as shown in table 3. The time spacing is at 0.01 sec for this test so that the 50 Hz results are meaningful, although as before the actual filtering step is performed at twice this density. Table 3 shows that for these cases there is no merit in raising the high frequency limit beyond 25 Hz solely to recover the scaled maximum peak accelerations. This is to be expected if the peak occurs during oscillations at a frequency considerably lower than 25 Hz which is the case for many records of this event. This result is not to say that there is no significant content in these records at frequencies higher than 25 Hz. Indeed, it is often possible, on the original records and on the uncorrected digitized versions, particularly the vertical components, to confirm that the highest frequencies are considerably in excess of 25 Hz, although their amplitudes are small.

As a result of these high-frequency tests we will continue to use a high frequency filter with a ramp falling from 23 to 25 Hz. The investigation of frequencies higher than 23 Hz can be carried out, if required, at a subsequent time, from the uncorrected data. The instrument correction plays a much more significant role at these frequencies, however, and a thorough testing of instrumental behavior and characterisics in this high-frequency range would be required if significant conclusions were to be reached.

REPRESENTATIVE SAMPLES OF ROUTINE PROCESSING

The previous sections have described the special testing carried out on representative samples of data from this earthquake, resulting in the selection of high frequency filtering at 23 Hz, terminating at 25 Hz, a corresponding density of 100 points per second, and long period filtering at 6 sec, terminating at 33 sec. This processing has

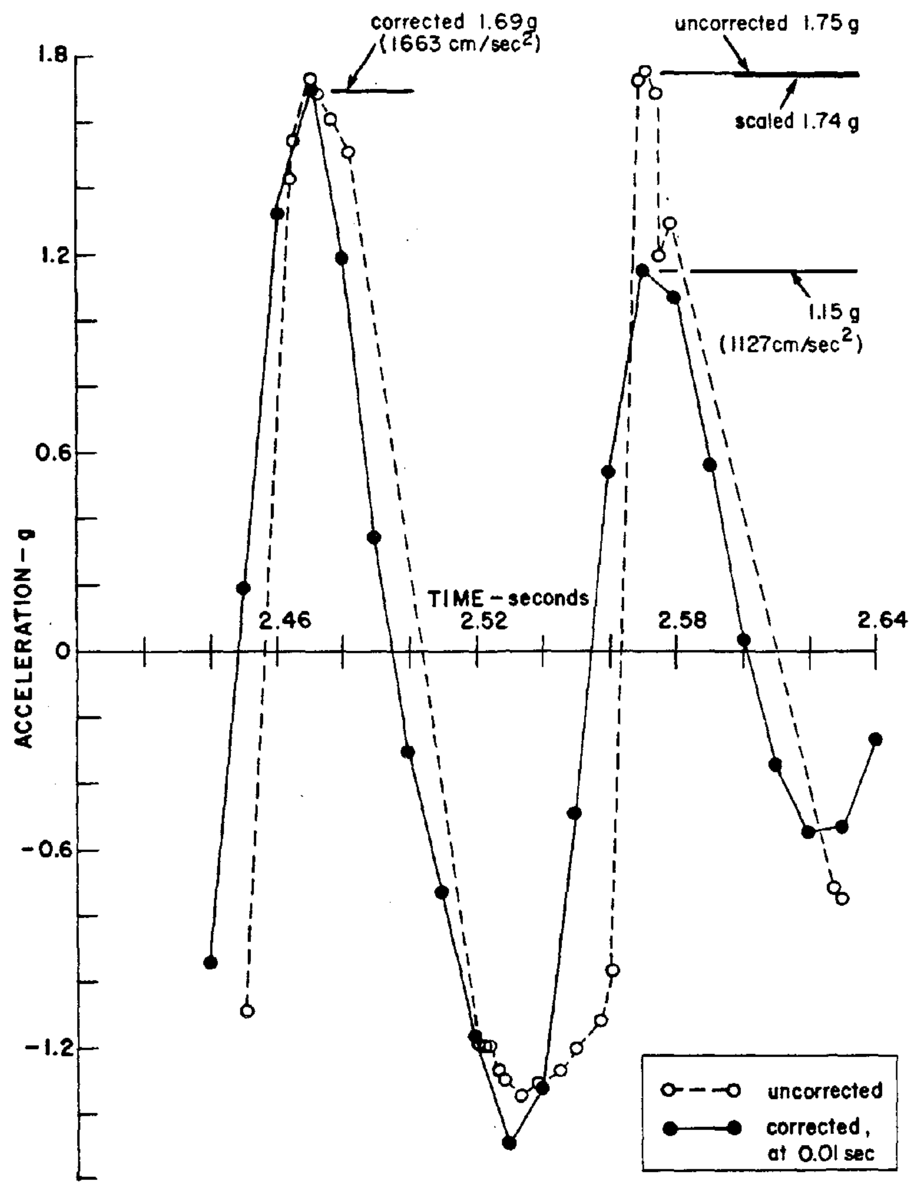


Figure 3.- Detail of October 15 main-shock El Centro Array Station 6 accelerogram, up component, peak values.

Table 3.- Comparison of peak acceleration, velocity and displacement for different filters applied to October 15 main-shock components with severe peak value loss during correction

Station and component	Peak Value	High-frequency filter (ramp) 25 Hz (23-25 Hz)	35 Hz (33-35 Hz)	50Hz (48-50 Hz)
Bonds Corner, Up	Accel. (cm/sec^2)	347.7	340.6	337.4
	Vel. (cm/sec)	12.17	12.15	12.15
	Disp. (cm)	2.46	2.46	2.46
El Centro Station 8, 230°	Accln. (cm/sec^2)	457.4	462.1	459.1
	Vel. (cm/sec)	47.71	47.72	47.72
	Disp.(cm)	29.34	29.34	29.34

Table 4. October 15 main-shock WWVB trigger times for nine stations of the El Centro Array

El Centro Array Station	Minutes:Seconds*
1	17:02.24
2	17:01.18
4	17:01.78
5	17:01.39
6	17:01.40
8	17:00.62
11	17:00.48
12	17:01.48
13	17:02.45

*After 288 days, 23 hours (UTC).

been performed on 22 strong-motion records of the main shock and provides, for each of the 66 components, a plot of corrected acceleration, velocity, and displacement time histories; response spectra, with both linear and tripartite axes; Fourier spectra with both linear and log-log axes; and three different interpretations of duration spectra. All but the linear Fourier spectra and duration spectra are contained in a U.S. Geological Survey Open-File Report (Brady and others, 1980). The digital data for uncorrected accelerations, corrected acceleration, velocity, and displacement time histories, and response spectra are available on two magnetic tapes from the Environmental Data and Information Service, NOAA, in Boulder, Colorado.

We present here a small representative sample of the corrected acceleration, velocity, and displacement time histories, the response spectrum on tripartite axes, and the Fourier spectrum on log-log axes calculated by the fast Fourier transform (FFT) algorithm. Three groups of specific components from records either close to the fault or within El Centro are considered, namely: (1) the vertical components at Bonds Corner and El Centro Array Stations 6 and 7; (2) the component transverse to the fault at El Centro Array Stations 6 and 7; and (3) the component transverse to the fault at El Centro Array Stations 8 and 10 and the El Centro Differential Array.

Plots of the processing of the vertical components of ground acceleration at the three stations closest to the fault, namely Bonds Corner (6 km epicentral distance, and within 3 km of the fault) and El Centro Array Stations 6, Huston Road, and 7, Imperial Valley College (within 1 km on either side of the fault) are included in figures 4 (corrected time-history data), 5 (response spectra) and 6 (Fourier spectra). This group includes the Station 6 component with the scaled peak of 1.74 g. These plots point to the presence of high-frequency energy, particularly between 10 and 20 Hz. The acceleration plot is not designed to show this particularly clearly since the entire record, almost 40 sec, is plotted on the one section of the time axis. None-the-less the arrival of the high amplitude packet at Stations 6 and 7 approximately 2.5 sec after triggering is particularly apparent. The long-period content of vertical motion discussed in previous sections is evident in the the displacement at all three sites, with a period at Stations 6 and 7 of approximately 4.75 sec (figs. 5b and 5c).

The second group of representative records consists of high amplitude horizontal accelerations close to the fault as portrayed by El Centro Array Stations 6 and 7. Figure 7 shows the corrected acceleration, velocity, and displacement time histories for the two components transverse to the fault, figure 8 shows their response spectra, and figure 9 shows their Fourier spectra. The lack of high-frequency ripple on the El Centro Array Station 7 record between 4.5 and 7.5 sec after triggering is reflected in the spectral plots. None of the other records, in the direction transverse to the fault, were as free of high-frequency content as the El Centro Array Station 7

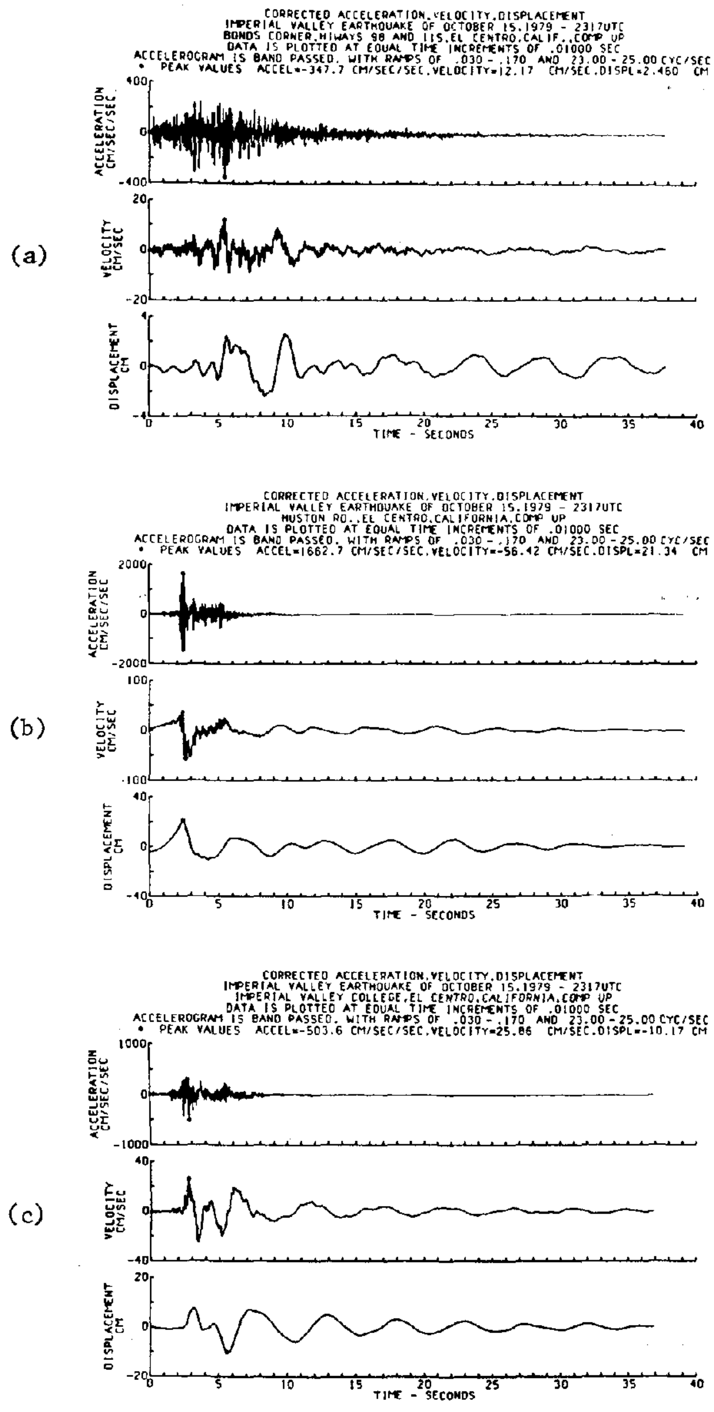
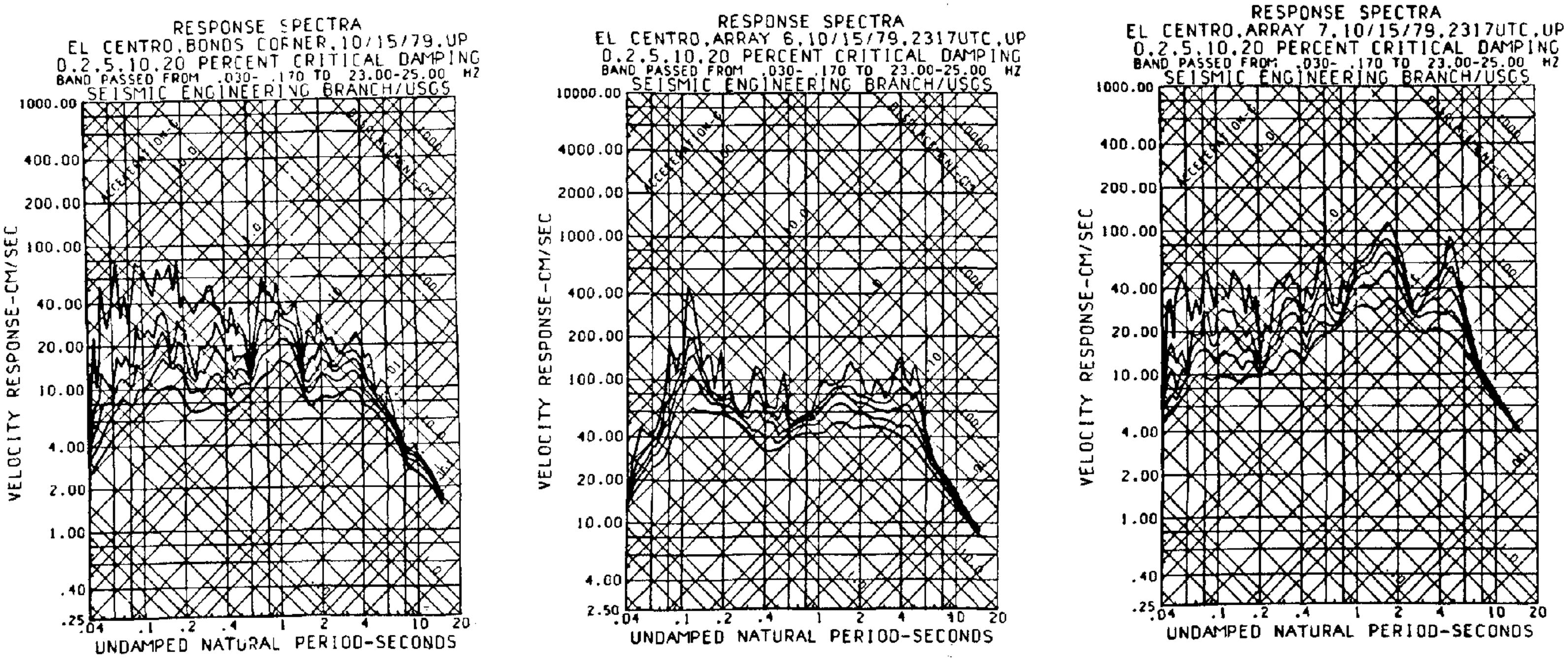


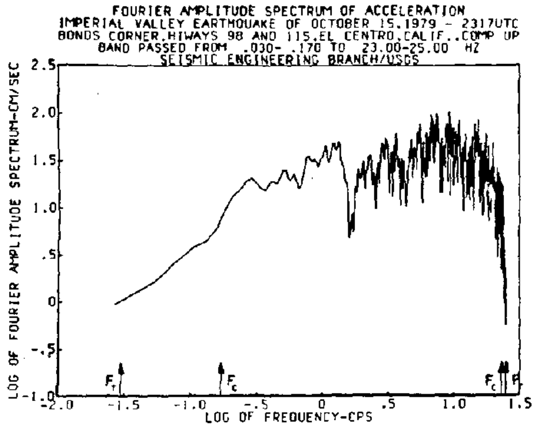
Figure 4.- Corrected October 15 main-shock acceleration, velocity, and displacement time histories for the vertical components at Bonds Corner (a), and El Centro Array Stations 6 (b) and 7 (c).



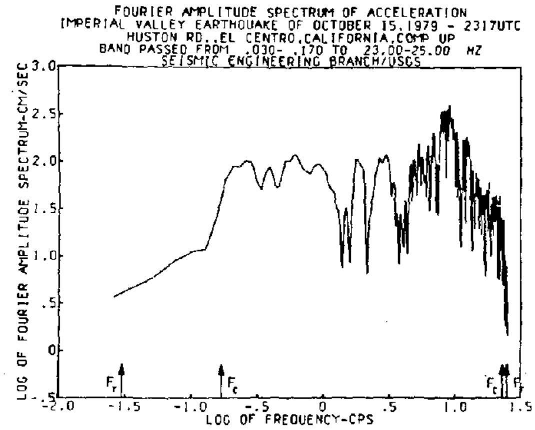
(a) (b) (c)

Figure 5.- Response spectra for the October 15 main-shock vertical components at Bonds Corner (a), and El Centro Array Stations 6 (b) and 7 (c).

(a)



(b)



(c)

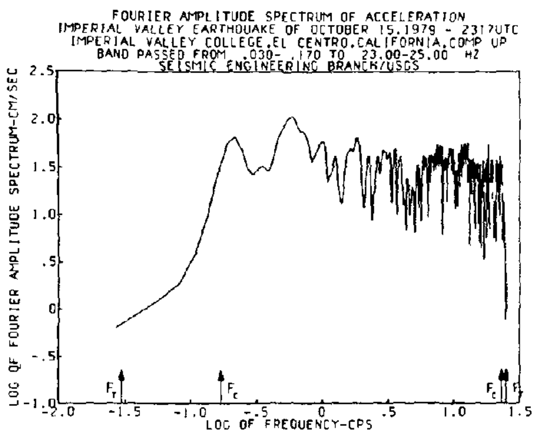
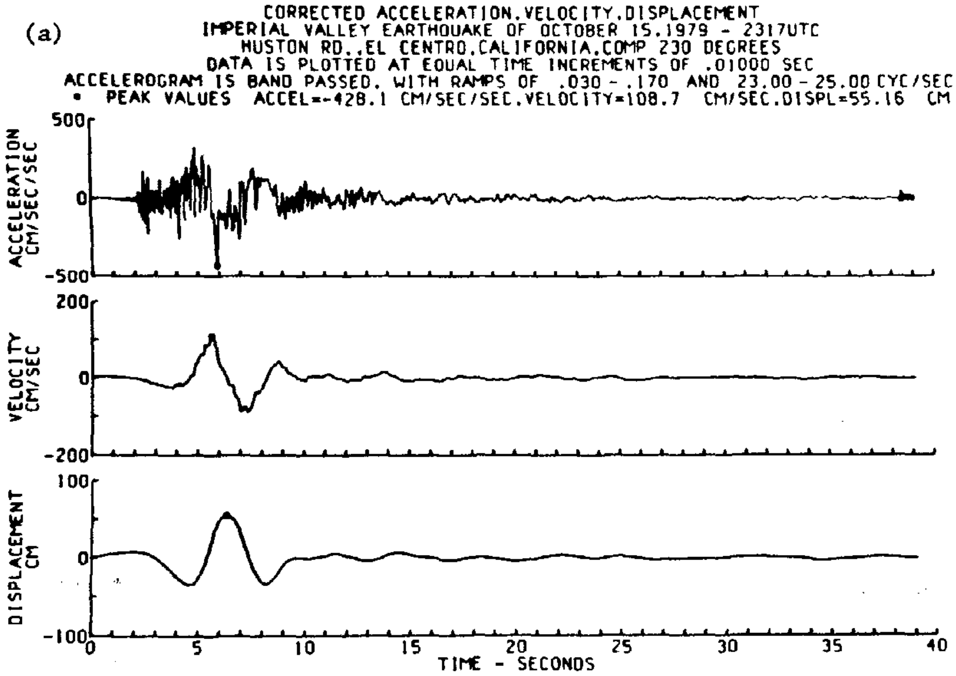


Figure 6.- Fourier amplitude spectra for the October 15 main-shock vertical components at Bonds Corner (a), and El Centro Array Stations 6 (b) and 7 (c).



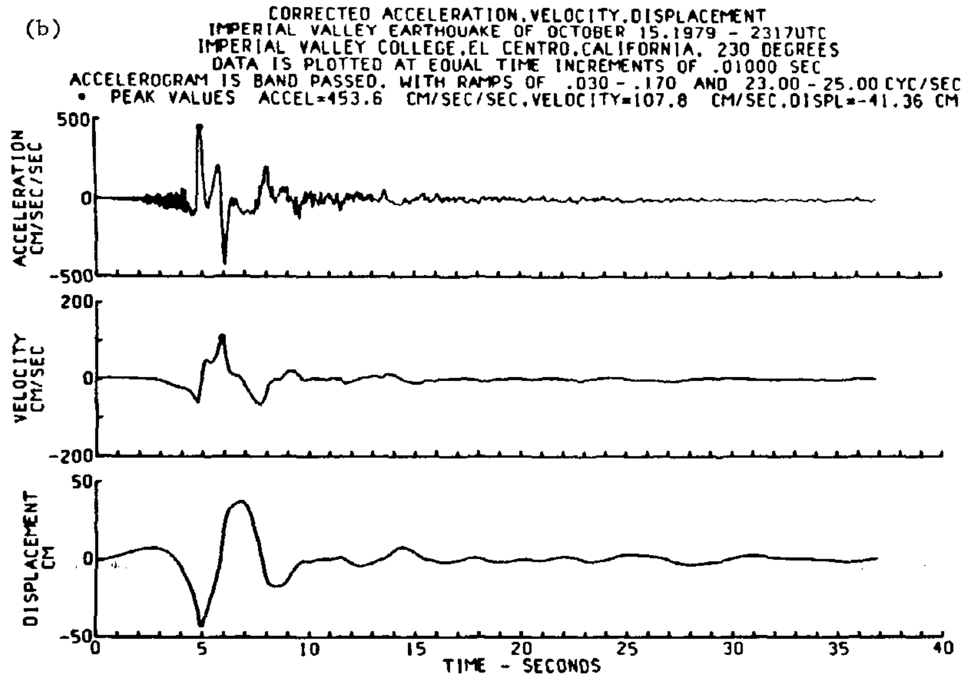
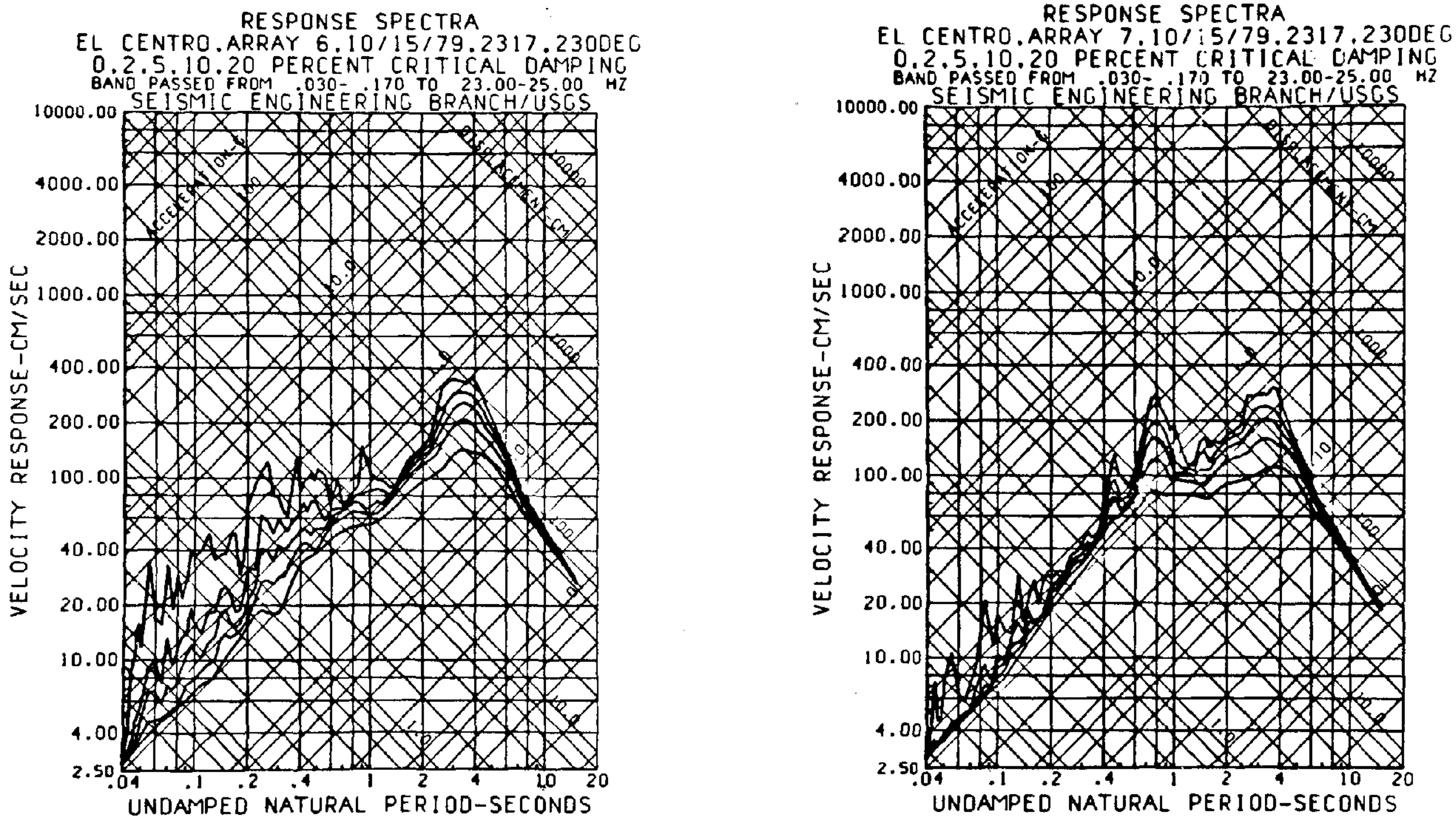


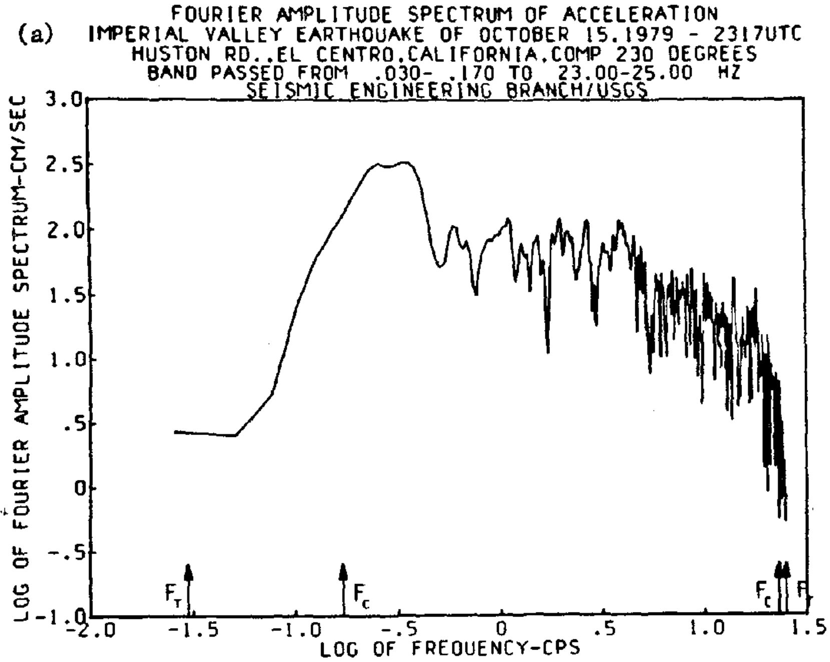
Figure 7.- Corrected October 15 main-shock acceleration, velocity and displacement time histories for the components transverse to the fault at El Centro Array Stations 6 (a) and 7 (b).



(a) (b)

Figure 8.- Response spectra for the October 15 main-shock components transverse to the fault at El Centro Array Stations 6 (a) and 7 (b).

(a)



(b)

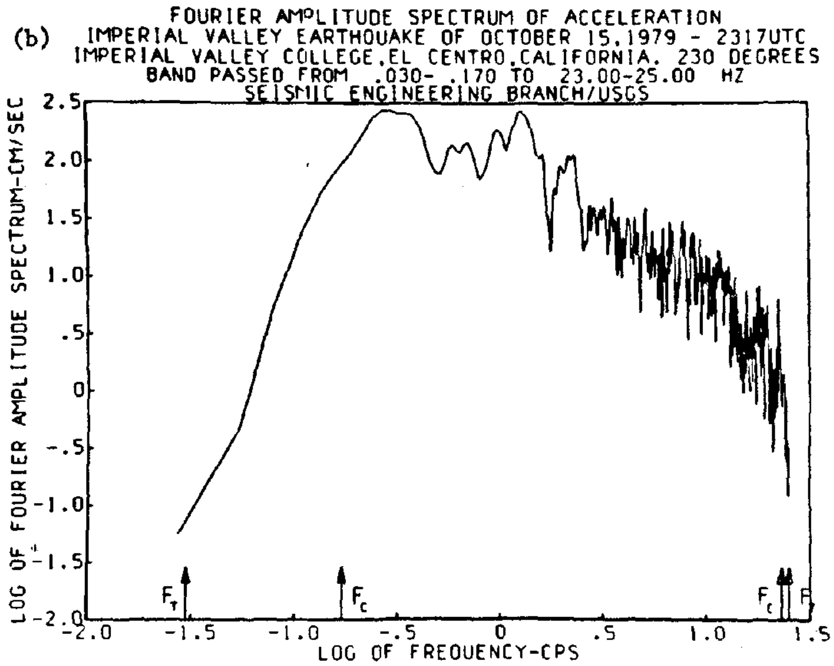


Figure 9.- Fourier amplitude spectra for the October 15 main-shock components transverse to the fault at El Centro Array Stations 6 (a) and 7 (b).

record during the large amplitude accelerations.

The third group of records are those from three USGS-operated stations that are closest to the Imperial County Services Building in El Centro, and which will be used in future comparative investigations of the detailed ground motion at the County Building as it is deduced from recordings made at the ground floor level and at a ground site 103 meters from the building (Rojahn and Mork, 1980). The three stations in this group are El Centro Array Station 8 (95 E. Cruickshank, approximately 4 km northeast of the County Building), the El Centro Differential Array (Dogwood Road; a film recorder placed near the recorders of the digital array, approximately 3 km east of the building), and El Centro Array Station 10 (the Community Hospital at Imperial and Ross, approximately 1.5 km south of the building). Figure 10 shows the corrected acceleration, velocity and displacement time histories for the component transverse to the fault for stations 8 and 10, and the west component for the El Centro Differential Array. Figure 11 shows their response spectra and figure 12 shows their Fourier spectra. Taking into account the 180° difference in direction at Stations 8 and 10, it is clear that the displacement pulse associated with the transverse shear wave arrival is remarkably coherent across this section of the city, with a peak displacement within a few centimeters of the average of 30 cm. The high-frequency content, on the other hand, although obviously present to the extent expected for stations this close to the causative fault, does attenuate from Station 8 to 10, as indicated by the peak acceleration falling from 457.4 to 168.2 cm/sec/sec. The spectral plots of both kinds for stations 8 and 10 show clearly the consistent level of long period content (longer than 1 sec, for example) and the gradual decay of high-frequency content (between 3 and 10 Hz).

EL CENTRO ARRAY CALCULATED GROUND DISPLACEMENTS

The 13 stations comprising the El Centro Array provide an opportunity to check the long-period performance of the processing described in this paper. Of those stations numbered 1 to 13, we at present exclude Station 9, not yet digitized, and add the film recorder stationed next to the digital recorders for the differential array (see Matthiesen, and Porcella, 1980, and Bycroft, 1980). All except the differential array instrument are aligned approximately along and transverse to the Imperial fault. All the recordings show an exceptionally clear transverse displacement pulse, identified as the direct S-wave in the southwest direction, with a maximum peak-to-peak amplitude, occurring at Station 6 within a kilometer of the fault, of 90 cm, and period of 3.6 sec. To portray this pulse in a precise way, figure 13 has been prepared using only those records with a legible WWVB time code. Nine such records, with trigger times read to 0.01 sec, are listed in table 4. Figure 13 shows the transverse displacement time-history plots for the nine records, adjusted in the time direction such that all records are synchronized to WWVB time. The displacement scales remain unchanged from those used in the regular

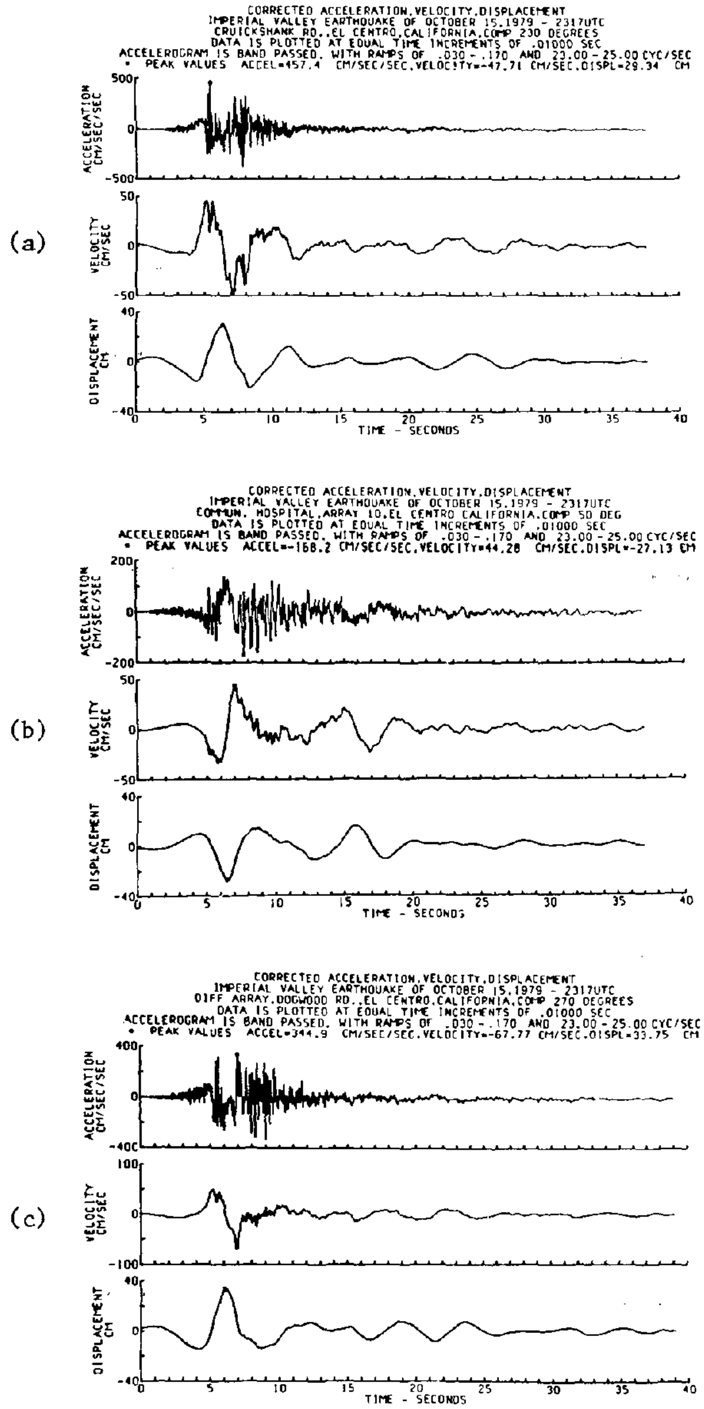
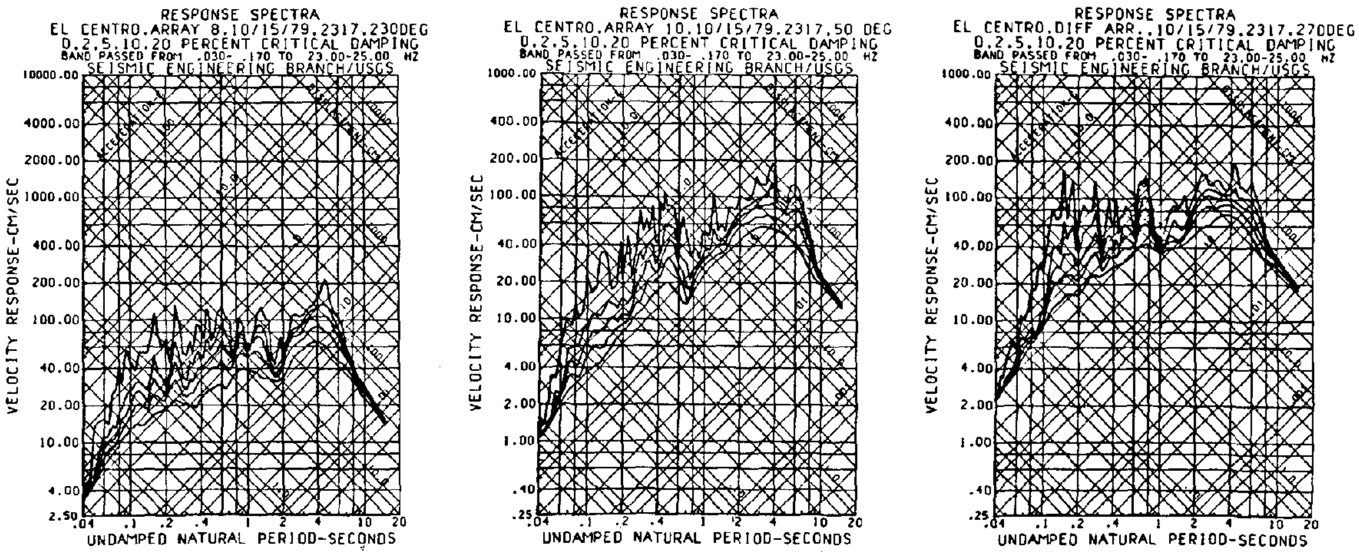


Figure 10.- Corrected October 15 main-shock acceleration, velocity and displacement time histories for the components transverse to the fault at El Centro Array Stations 8 (a) and 10 (b), and for the west component at the El Centro Differential Array (c).



(a) (b) (c)

Figure 11.- Response spectra for the October 15 main-shock components transverse to the fault at El Centro Array Stations 8 (a) and 10 (b), and for the west component at the El Centro Differential Array (c).

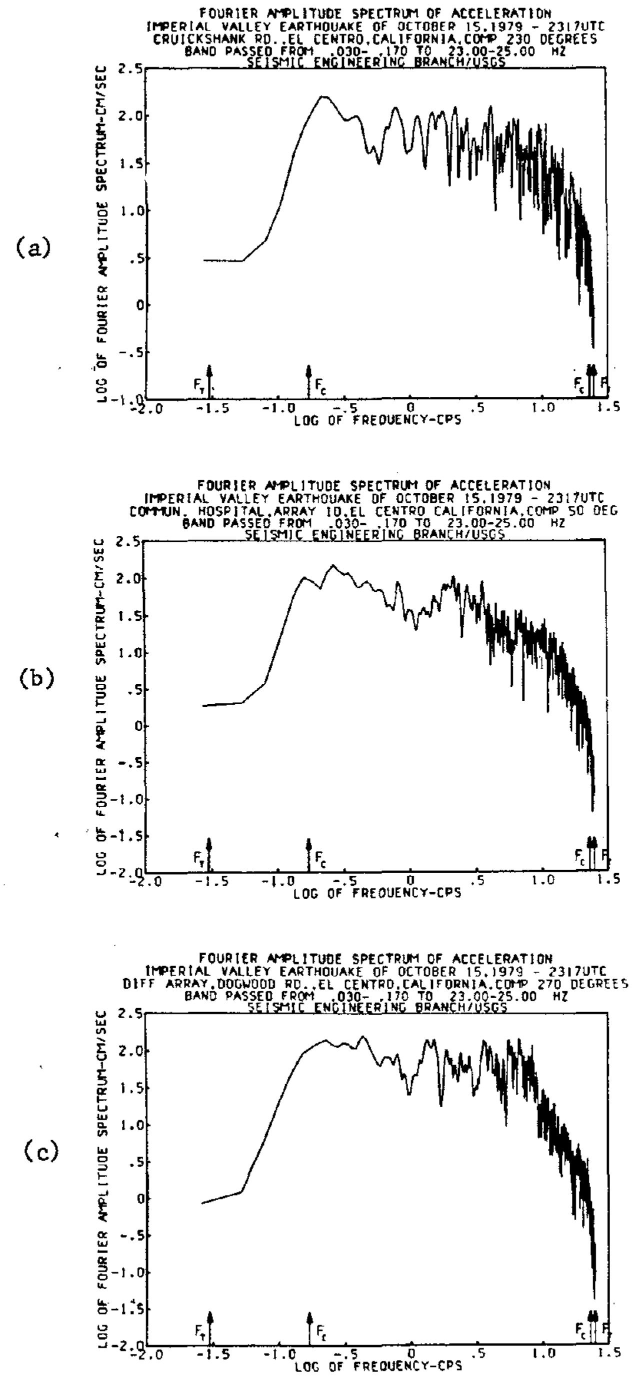


Figure 12.- Fourier amplitude spectra for the October 15 main-shock components transverse to the fault at El Centro Array Stations 8 (a) and 10 (b), and for the west component at the El Centro Differential Array (c).

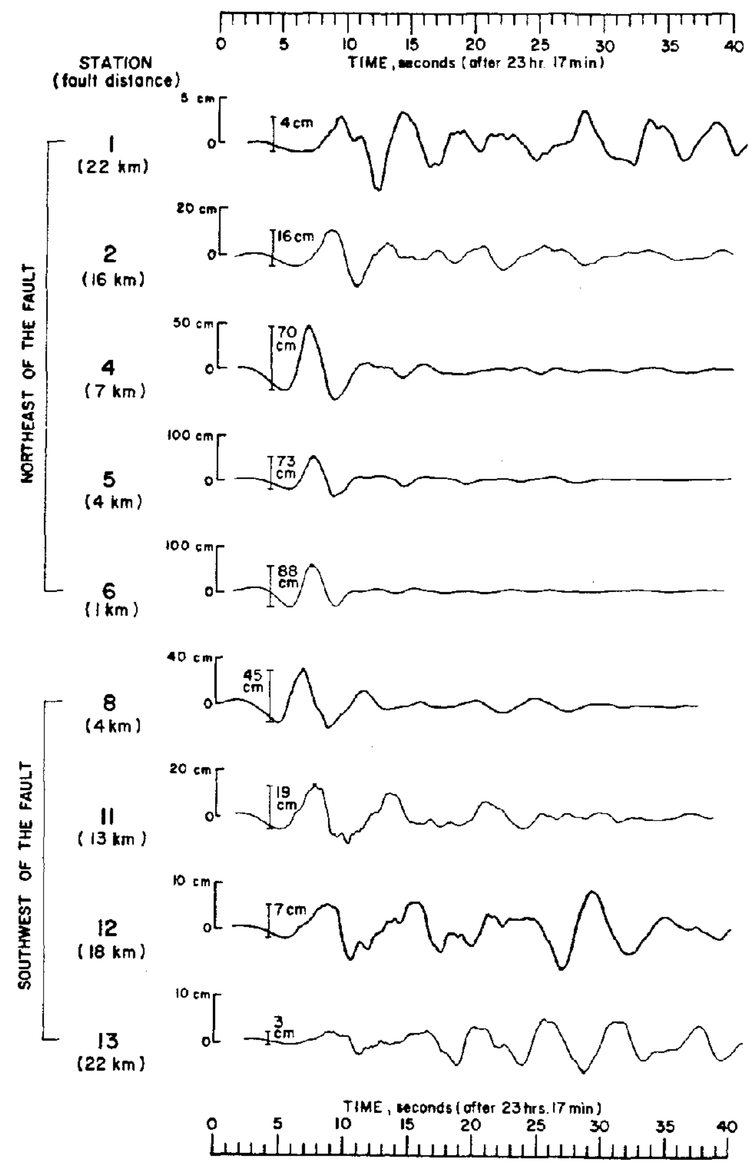


Figure 13.- October 15 main-shock ground displacements transverse to the Imperial fault trace, El Centro Array, 230° direction.

corrected acceleration, velocity, and displacement time-history plots. From the value of 88 cm at Station 6, the peak-to-peak amplitude falls off until at Stations 1 and 13, 22 km on either side of the fault, the pulse measures 3 cm.

Identifying the S-wave pulse on these 9 records and its arrival times at the different stations can be performed with confidence. This pulse's ground motion in the 230° direction (approximately southwest) begins at 17m 05.0s at Station 8, 17m 06.1s at Station 6 on the other side of the fault, and as late as 17m 07.4s at Station 1, 22 km to the northeast of the fault. The displacement character both before and after the S-wave passage is surprisingly coherent along the entire array. All show the preliminary motion to the northeast (downward on the fig. 13 plots), in various amounts, as was expected from the original accelerograms where the motion was first noted. Stations distant from the fault indicate some signal to noise problems although the oscillatory motion when it occurs at 4.75 sec period is undoubtably associated with quite visible vertical motions (figure 4) at the same period. Subsequent investigations will allow comments to be made on the seismological significance of this data when considered together with outlying strong-motion recordings.

CONCLUSIONS

Tests on the high-frequencies in the October 15 main-shock Imperial Valley earthquake accelerograms have resulted in a high-frequency limit of 23 Hz and a final sampling rate of 100 per second. The long period limit was chosen at 6 sec, with roll-off termination at 33 sec. The processed length of the records for the most part is approximately 36 sec.

The entire set of digital data is available on tape, and the graphed results of the processing are available in a U.S. Geological Survey Open-File Report. Included in the digital data is the uncorrected phase 1 version, for use by those researchers who would prefer to carry out their own analyses if different from those reported here.

The great interest shown by the engineering and seismological communities in the strong-motion records obtained from the Imperial Valley earthquake, and more particularly, in their digitization and processing has caused us to exert an unusual amount of time and effort in preparing the data for their use. We have described here most of the decisions that were made concerning digitizing and processing, and have shown the effects on the processed data that have resulted from these decisions. There is no doubt that subsequent analysis of the data might lead to discoveries which in turn could lead to minor reassessments of the processing philosophy. We expect that the work and data described in this paper will lead to a great deal of investigative research by both the engineering and seismological communities. The additional processing of longer durations of the mainshock recordings, and of the more significant aftershocks, will be carried out by the U.S. Geological Survey as time and funding permit.

ACKNOWLEDGMENTS

The research performed by the data management project within the Seismic Engineering Branch is funded in part by the National Science Foundation, Grant CA-114, under the terms of an interagency agreement with the U.S. Geological Survey. The assistance of the personnel of IOM-TOWILL in the digitization, particularly of the faint traces on some of the close-in recordings, is also acknowledged.

CHAPTER IV

UTILIZATION OF STRONG-MOTION RECORDS IN BUILDING DESIGN

by

Chris D. Poland*

INTRODUCTION

A discussion of the utilization of strong-motion records in building design could obviously pursue a variety of courses and extend for a lengthy period of time. In the spirit of the theme of this workshop though, and in an attempt to be responsive to the questions raised by the organizers in their introduction, this chapter will be brief and avoid an analytical presentation of the available analysis and design techniques in use today. Rather, it will concentrate on the actual application of strong-motion records to building design and the assumptions inherent in such an application. With this task, it should be observational where we are today, where seismic design fits into the overall process of building design, where the areas requiring major assumption are, and possibly where the future interpretation of strong-motion records might provide additional insight into building design.

Following the lead of the Uniform Building Code (ICBO, 1979), it will be useful to define a building as a "...structure used or intended for supporting or sheltering any use of occupancy". This definition excludes a large number of other "structures" which are also appropriately defined by the UBC as "...any piece of work artifically built up...". While the use of strong-motion records and the techniques described herein certainly offer a basis for the design of such "other structures", their uniqueness and high degree of specialization make sweeping statements, such as those that follow, meaningless and without application.

As further clarification of direction, this discussion is specifically oriented toward the analysis and design of new and existing buildings for future load conditions. We are therefore looking from an engineer's point of view at describing and designing for the various possible structure oriented, occupancy, and environmental loading conditions and their possible combinations during the life of the structure. As will become obvious, not all of these comments are appropriate when looking at the inverse condition, i.e., the "after-the-fact look" at why a building performed as it did given a specific event and physical status of time.

To better understand why engineers choose and carry out the

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techniques they do for seismic design of buildings, it is useful to set the seismic design process in perspective with the overall process of building design. The engineer's involvement in a building generally begins with a given site, building function, functional layout and planned exterior and interior appearance. As is indicated in figure 1, his effort can be thought of in four phases, all of which must be carried out such that the intended function and use of the building are supported. As a general indication of effort, we know that roughly five to 10 percent of the engineer's effort is in system development, 10 to 15 percent in structural analysis, 60 to 70 percent in design, detailing and checking, and 10 to 15 percent in construction services. With the structural analysis effort, balanced consideration must be given to the effects of dead and live loads, wind loads, lateral loads due to soil pressure, loads related to the volume change effects of thermal expansion, thermal contraction, shrinkage and creep, and the force and deflection demands of seismic motions. Each loading condition must be considered with roughly equal accuracy and in such a way that their effects can be rationally combined and designed for.

As is true for all engineering related problems, the design solution for the earthquake problem must be based on past observations applied to the available analysis techniques and proportioned by judgement. In general, the relationship between the reliance on an analysis technique and the amount of judgement applied can be thought of as being inversely proportional to the number of unknowns and resulting assumptions required to validate the analysis, or at least the number of such variables recognized by the user. In the specific case of seismic design, observations of past performance coupled with the available analysis techniques have led to design criteria that generally set standards for structural systems, their composition, completeness and details as judgement calls, and set strength criteria that use the available analytical techniques to establish and apply a consistent set of equivalent design forces and displacements throughout the structural system. These values are equivalent in the sense that they recognize and attempt to account for the difference in the ultimate strength and deformability of a particular structural system as observed and the strength and deformability of the system as analyzed using normally available elastic techniques.

With the acquisition of strong-motion records beginning in the 30's, and the development of dynamic analysis techniques in the early 40's, two basic seismic analysis techniques have developed and are in use. One technique, considering earthquake motion in terms of response spectrum, has led to the development of the most generally applied design standards. These include the Recommended Lateral Force Requirements of the Structural Engineers Association of California (SEAOC, 1975), the Tentative Provisions of the Development of Seismic Regulations for Buildings by the Applied Technology Council (ATC-3, 1978), and the general application of Response Spectrum Analysis (Blume, Newmark, & Corning, 1961; Newmark & Hall, 1973). The other technique, using earthquake motion in terms of time history records

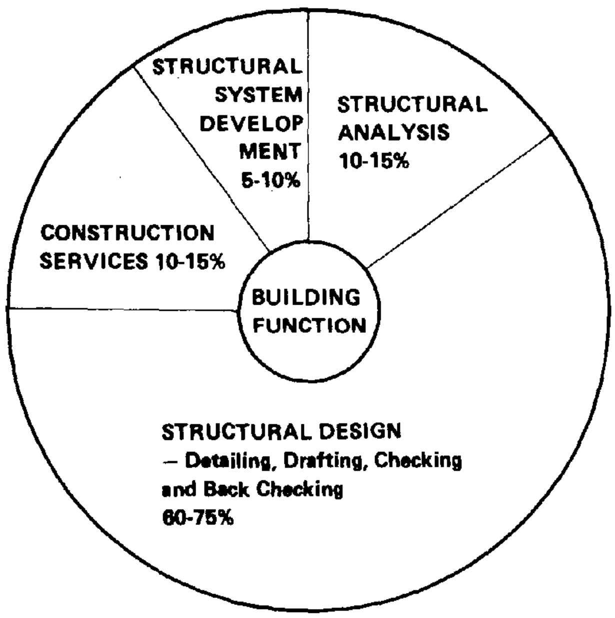

Figure 1.- Building design effort.

of expected accelerations, have led to the linear and non-linear solutions of building response in time.

Given a particular building design, the analysis technique chosen must meet two basic criteria. First, the reliance upon accuracy of the solution must not exceed the accuracy of the input information for the given problem. Secondly, the solution must be compatible and combinable with solutions for the other structure oriented, occupancy, and environmental loading conditions under consideration in design. It is not surprising, then, that engineers consistently tend to choose the enveloping techniques of response spectrum approach over the time history approach.

Major emphasis and consideration will be given in the following text therefore, to the actual uses of response spectrum analysis and the companion SEAOC and ATC-3 recommendations. This will be followed by some specific comments on time history related analysis.

DESIGN PROCEDURES USING RESPONSE SPECTRA

The details of the Response Spectrum technique of analysis are well documented, taught and generally known (Biggs, 1964; Clough & Penzien, 1975). Simply put, a mathematical model of the structure of some level of complexity is conceived that includes a representation of the stiffnesses and masses of the structural system and their respective distributions. Given such a model, it is assumed that its vibrational characteristics and deflected shape in time for a given forcing function can be described in terms of a finite number of deflected shapes, each of which represents a unique mode of harmonic motion characteristic of the building. Then, with the additional assumption that the system is linearly elastic, a set of simultaneous differential equations can be derived and rewritten into an uncoupled set of differential equations, one for each harmonic mode. Since the resulting uncoupled equations are the same as the equation of a single-degree-of-freedom system, we can use a known maximum response for each mode as is available from a design response spectrum, assign it to the appropriate mode and back figure the maximum displaced shape for each mode for that particular spectrum. Since the response spectrum is independent of time, and since the maximum response of each mode occurs at different times, direct addition of the modal responses, while judged conservative, is considered excessive. Thus, the results of these modal responses are usually combined by square root sum of the squares of the individual modal contributions. This technique provides force and displacement information, consistent with the design response spectrum used, which can be stated in terms of base shear, horizontal and vertical force distribution in the structural elements modeled, story drifts and overall building displacements. These derived loads are available for combination with other load conditions and use in normal building design.

The design response spectrum used in the analysis is generally the result of taking a predicted elastic response spectra and reducing it to account for the previously mentioned difference in a building's ultimate strength and its strength as predicted by a normal elastic analysis and design at working stress levels. The elastic spectra is often developed by a special consultant to the engineer. It is generally based on a geological and seismological study of the site, an establishment of the average recurrence rate and probabilistic description of the expected earthquake events, and a determination of expected ground motion characteristics. The procedures used vary widely, though they are generally based on statistical studies of available strong-motion ground records. By varying the records considered and the amount of probabilistic uncertainty, the resulting elastic response spectra can be made representative of a specific site and performance criteria. The Structural Engineers Association of California has recently published their second edition of "Suggested Procedures for Developing Seismic Ground Motion" which not only strives for uniformity in the evaluation of seismic exposure, but provides an extensive bibliography on this subject.

Given a specific elastic response spectra at an appropriate damping level, the task at hand involves properly reducing it to a level suitable for use in a linear elastic analysis. While the basic methods and assumptions for carrying out such a reduction have been available since the early 60's, the latest refinement into an actual procedure, that is still commonly used, was proposed by Newmark and Hall (1973). Defining the amount of additional strength and deformability a building system has over its amount of calculated yield strength as the building ductility factor, μ, they reduce the elastic response spectra to an inelastic acceleration response spectra or design response spectra by reducing the spectral accelerations in their displacement and velocity regions by μ, and in the acceleration region by $\sqrt{2\mu-1}$. They further define the inelastic displacement response spectra as a constant multiple of the inelastic acceleration spectra. Figure 2 shows an example of this process with a ductility of 5 applied to their normalized 1.0 g, 5 percent damped elastic spectra. Also shown in the same figure is a plot of the variation of the actual reduction due to ductility with the building period.

Within the text of presentation, Newmark and Hall are careful to point out that their technique in the strictest sense, only applies to single-degree-of-freedom systems and further caution that "...the elasto-plastic or other inelastic response spectra can be used only as an approximation for multi-degree-of-freedom systems". This caution demands that the application of this technique be limited to those buildings that behave similarly to single-degree-of-freedom systems with a constant ductility, i.e., to buildings with a uniformly distributed reserve capacity, uniformly distributed stiffness, and uniformly distributed mass.

Given a reduced design (or inelastic) acceleration spectra that is representative of a rational force level for building design, two other

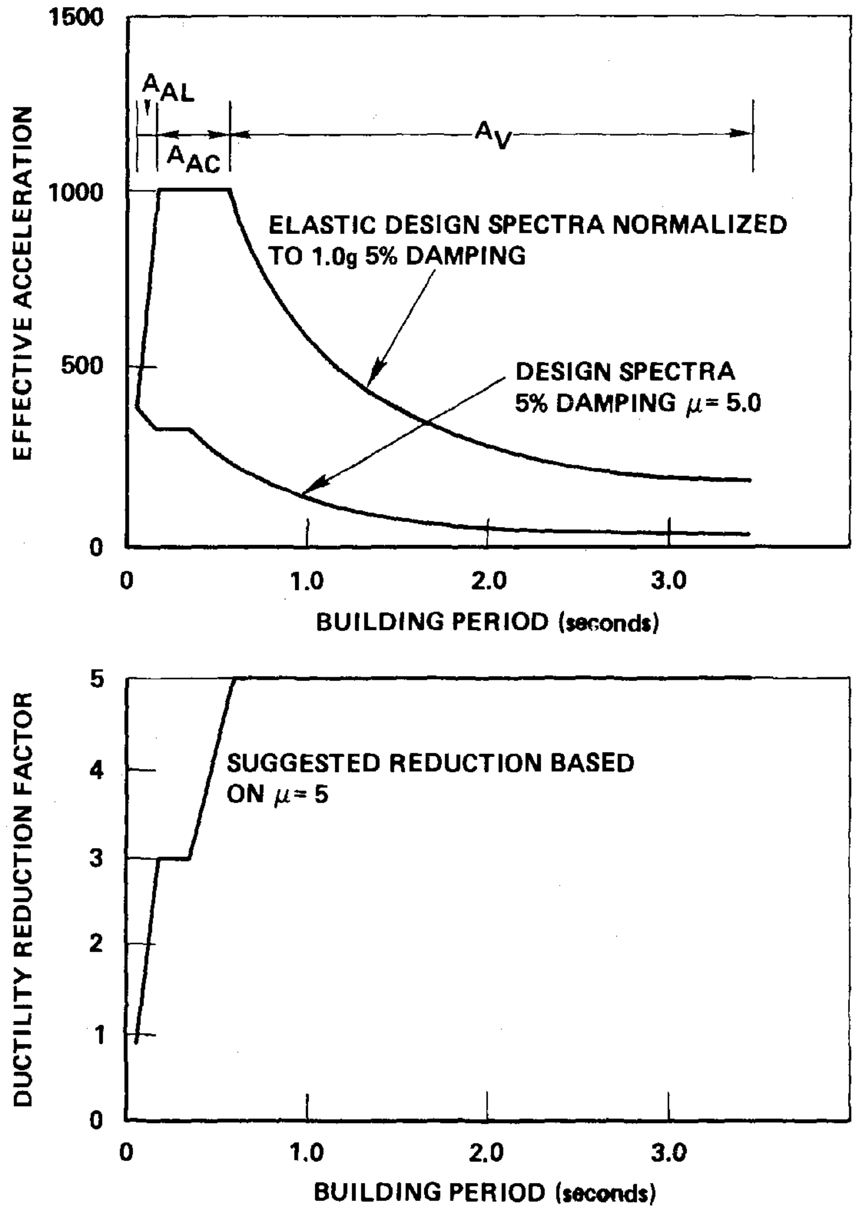

Figure 2.- Suggested reduction of elastic design spectrum.

reductions are inherent in the resulting design process. They are reductions based on material strength characteristics, and reductions due to the actual distribution of the mass throughout the structure. Recalling that the reduction due to ductility accounted for the difference in ultimate strength and yield strength, and recognizing that traditional design is carried out at working stress levels, an additional reduction in forces that accounts for the difference in yield strength and allowable design strength is appropriate. Such a reduction must be tailored to the material under consideration and the type of loads involved.

The fact that each mode of vibration is considered, in essence, as a single-degree-of-freedom system with a concentrated mass and realizing that the actual building is a set of distributed masses, it is not surprising that built within the mathematics of the modal analysis is the variation of inputed spectral acceleration as it is applied to the distributed masses of the structure and defined by the proportion inherent in the individual mode shapes. As a result, some masses in the system, for a given mode, experience effective acceleration greater than the related spectral value and some receive less. Blume (1973) showed that the net effect of this characteristic could account for as much as a 58 percent difference (decrease) between spectral acceleration as obtained for a given mode, and the actual ration of subsequent base shear to total weight.

Any proper application of the response spectrum technique to building design will recognize the few areas not adequately considered within the analysis technique. These areas are either considered independently and included at the point of design, or they are used as a basis for modifying the basic process, or they include the effects of inputed torsional motion, out-of-phase input motion, secondary effects of large structural displacements (such as P-Δ effects), the expected ductility demand in a very long period tall building frame, and structural elements subjected to seismic forces due to motion in both principal directions which are normally considered non-concurrently.

The key to the success of the response spectrum technique lies in the validity of the assumptions, and the accuracy of the analytical model used in the design process. For reference, the basic assumptions required for a response spectrum analysis are listed below. Their validity is open for and continually subject to debate. The accuracy of the analytical models is beyond the scope of this discussion, but it is also debatable and worthy of continuing review. Analytical models have been shown to be very sensitive to non-structural elements (Commentary, Chapter 5, ATC-3, 1978), variations in material properties, variations in mass distributions in time (Gates, 1977), inconsistencies in analytical modeling techniques (Gates, 1977; Poland, 1980), etc.

RESPONSE SPECTRA ASSUMPTIONS

Assumptions inherent in the Response Spectrum Technique for determining structural response to earthquake motion are as follows:

1. Earthquake motion as recorded on an accelerometer, and the effect it has on buildings can be described in terms of the maximum elastic displacement of a set of single-degree-of-freedom oscillators with viscous damping subjected to the same motion, i.e, a response spectrum is representative of earthquake motion.

2. The effects of duration are adequately accounted for within the parameters of a response spectrum.

3. A design elastic response spectra can be developed for a given site using statistical studies of response spectra derived from actual strong-motion records that have been scaled to account for local geologic and seismologic conditions, regardless of source mechanism.

4. There are sufficient strong-motion records avaiable from reliable sources to carry out meaningful statistical studies.

5. The design elastic response spectra can be significantly adjusted by a single parameter to account for the post-elastic strength and post-yield deformation ability of the overall structural system. In this way, the building may be analyzed elastically at normal working stress levels and the resulting forces combined with other loading conditions for purposes of design without neglecting its post-elastic strength.

6. The magnitude of such a single parameter can be extrapolated from both laboratory tests of structural subassemblages and observation of structural damage in major earthquakes.

7. There is a direct relationship between earthquake damage and response spectrum.

8. This ability of the structural system to develop post-elastic strength and post-yield deformations is uniformly distributed throughout the structure.

9. A building's maximum deflected shape and required strength under the reduced design spectrum can be described as some combination of a finite set of harmonic uncoupled mode shapes that are characteristic of the building's geometry, mass, stiffness, and foundation conditions.

10. For purposes of design, the single reduction parameter of assumption 5 reflects the effect of a changing period due to inelastic deformation, lack of complete coupling with the ground, and the hysteric degradation of material properties under repeated stress reversals.

11. The strength and deformability demands on a building can be

adequately defined by considering earthquake motion along each of its principal axes non-concurrently. Adjustments can be made as required for the three-dimensional input effects of torsion and out-of-phase motion, and for structural elements carrying loads concurrently from excitation in more than one direction.

12. The effects of large displacement such as the P-Δ effect and basic geometry changes do not affect the overall building response, and therefore may be considered separately and added.

THE SEAOC RECOMMENDATIONS

Without getting involved in the history of the SEAOC Recommendations, suffice it here to say that they have develped over the years from the observations of the earthquake performance of buildings, the strong-motion records taken, and the analysis techniques available. Through the years, SEAOC has modified its design standards to reflect the recognition of the value of certain structural systems, detailing techniques, and the validity of certain principles of structural dynamics. This recognition, in its latest completed effort, was published as their fourth edition in 1974. That edition includes, in addition to the judgment calls on required structural systems and specific detailing requirements, a complete lateral force requirement that was distilled from the standard procedures of response spectrum analysis.

The heart of the SEAOC lateral force evaluation technique is the well-known formulas for base shear and lateral force distribution throughout the structure. These forces, when derived, distributed and analytically applied to a structure will produce a displaced shape similar to that derived through a response spectrum analysis which is suitable for design.

It is worth mentioning that these standards are accepted and used for the vast majority of buildings designed for seismic loads. This acceptance naturally includes acceptance of the assumptions behind the techniques, assumptions that need to be understood by their users and continually reviewed in light of new data as it is acquired. Needless to say, any improvement in the SEAOC Recommendations, that maintains their basic style (and insures their continued application) will have a profound effect on overall structural safety under earthquake activity.

Contained within the base shear formula is the base shear coefficient C and two modifying coefficients for soil conditions, S, and variations in ductility, K. When combined, they relate to a design response spectrum, in a general sense, that has been reduced and stated in terms of base shear coefficients. The method of computing the design base shear coefficients was the result of a consensus of the Seismology Committee and respresents compromises between various opinions (S. A. Freeman, oral commun., April 1980).

In arriving at the agreed-upon KCS design coefficient curve, there appears to have been at least two thought processes used. To some committee members, it was rationalized in terms of existing response spectra. To others, the new shape was based on needed modifications to the then existing design criteria. It was reported by Kariotis (1975) that the basic elastic design spectra was taken from some average of four maximum credible earthquake spectra developed for the Los Angeles basin and the 1940 El Centro N-S record. The upper bound of these elastic spectra at 5 percent damping is shown in figure 3. This elastic spectra was initially reduced for increasing damping to 10 percent and for multi-mode effects. This reduction considered only the building's fundamental mode for building periods longer than 0.6 seconds (SEAOC, 1975). Additional reductions for material strength, equal to 1.7/1.33 = 1.28, and ductility, 3.4 to .75 were taken and are also indicated on figure 3. Kariotis reported that the ductility factor "...is not a ductility factor as expressed in usual terms, but is a combined judgment factor based on ductility, reserve energy, energy absorption, redundancy of systems, and personal judgements" (Kariotis, 1975). Also indicated in figure 3 is a plot of the total reduction factor applied to the elastic response spectrum. As can be seen, it varies from a high of 10.9 to a low of 1.0 and is period dependent. This reduction factor, more than anything else, represents the results of committee action regardless of its derivation.

The previously mentioned areas of loading not properly covered by the response spectrum approach are dealt with specifically in the SEAOC Recommendations. Their provisions contain specific loading conditions related to torsion including a minimum value. The large displacement effects, such as P-Δ, appear to be accounted for in the selected reduction in ductility in the longer period ranges.

In addition to the basic assumptions previously listed for the response spectrum technique, the SEAOC Recommendations include the four additional assumptions:

1. Elastic Response Spectra developed for the Los Angeles Basin and El Centro 1940 are valid spectra and are representative of all zones of highest seismicity in California.

2. The effects of distributed masses and higher modes (multi-mode effects) can be enveloped and accounted for by reducing the design acceleration response spectrum.

3. All materials normally used in design can accommodate a 28% increase in load over the normal 1/3 increase before reaching yield.

4. The ductility factors, as assigned, are properly conceived and representative of all structural systems and material types.

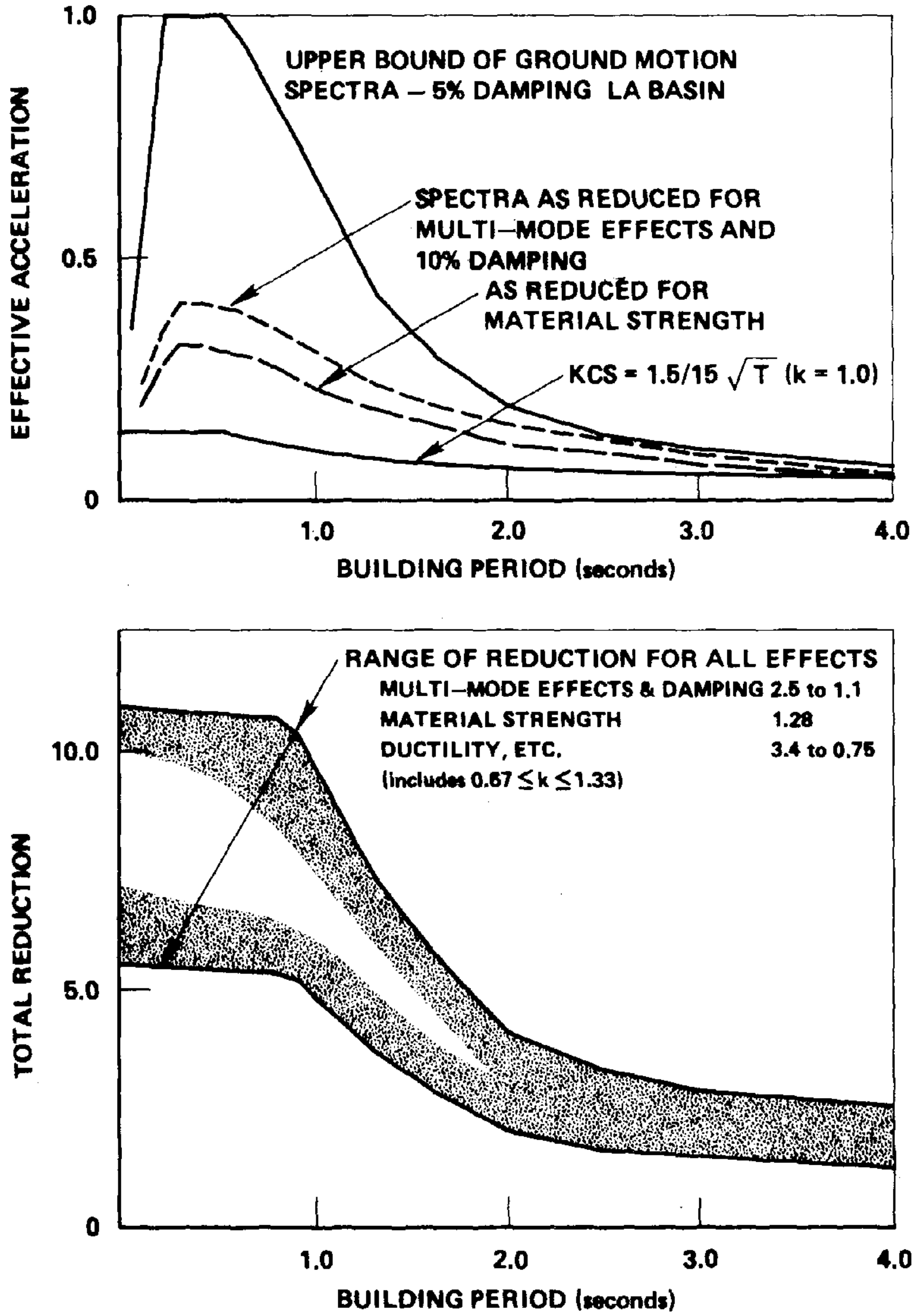

Figure 3.- Design spectra: SEAOC.

ATC-3 RECOMMENDATIONS

Without getting very deeply involved in the description of these tentative provisions, it would be instructive to view their approach to applying strong ground motion records to building design as a measure of some very current thinking. Its basic approach follows the reasoning previously described, i.e., setting standards for structural systems and detailing as judgment calls and defining a system of lateral forces and displacements as a strength requirement. ATC-3 develops and presents a lateral force coefficient type approach to building design, the details of which avoid some of the sweeping generalizations of the SEAOC Recommendations, though the intent remains the same (that is, to develop a set of design forces that when applied to a building produce a deflected shape representative of its response to seismic motion reduced by ductility and damping).

The development of their lateral force design coefficient begins with three response spectra. All three have the recognizably smooth shape of spectra as recommended by Newmark (1973) after having been adjusted to match three classifications of soil type as developed by Seed, et al (1974). For a given site, the spectra are scaled such that their acceleration dependent portions follow a set of acceleration attenuation relationships and their velocity dependent portion follows a set of derived velocity attenuation relationships. The absolute ground acceleration selected generally follows the effective peak acceleration prediction of Algermissen and Perkins (1976) except in the areas of highest seismicity where their 0.6 *g* peak acceleration contour and reported 0.8 *g* local maximum along major faults were reduced to 0.4 *g* effective peak acceleration. This maximum acceleration reduction represents a judgmental and design oriented reduction in the available elastic response spectra data. As reported in the Commentary, "...(it was) based partially on scientific knowledge and in part on judgement and compromise".

Given an elastic response spectra, suitable for building design, the process of reducing it to an inelastic design spectra was limited to the definition of a response modification coefficient R. These coefficients were derived from the available material tests, building observation, and committee action. They were directly based on the expected ductility of the structural system, material used and damping expected. The R factors were set in a range from 1-1/4 for unreinforced and partially reinforced masonry walls in bearing wall building systems to 8 for special steel moment frames in moment resisting frame systems. For comparison, figure 4 shows the design response spectrum and applicable zone for the related lateral design force coefficients. Also shown in Figure 4 is the effective reduction as a function of building period. The net decrease in allowable reduction with longer period comes from the use of $T^{2/3}$ in the denominator of the force coefficient formula. The decrease inherent in the ground spectra in the same region generally varies with T. The Commentary justifies this change in light of a number of reasons, all

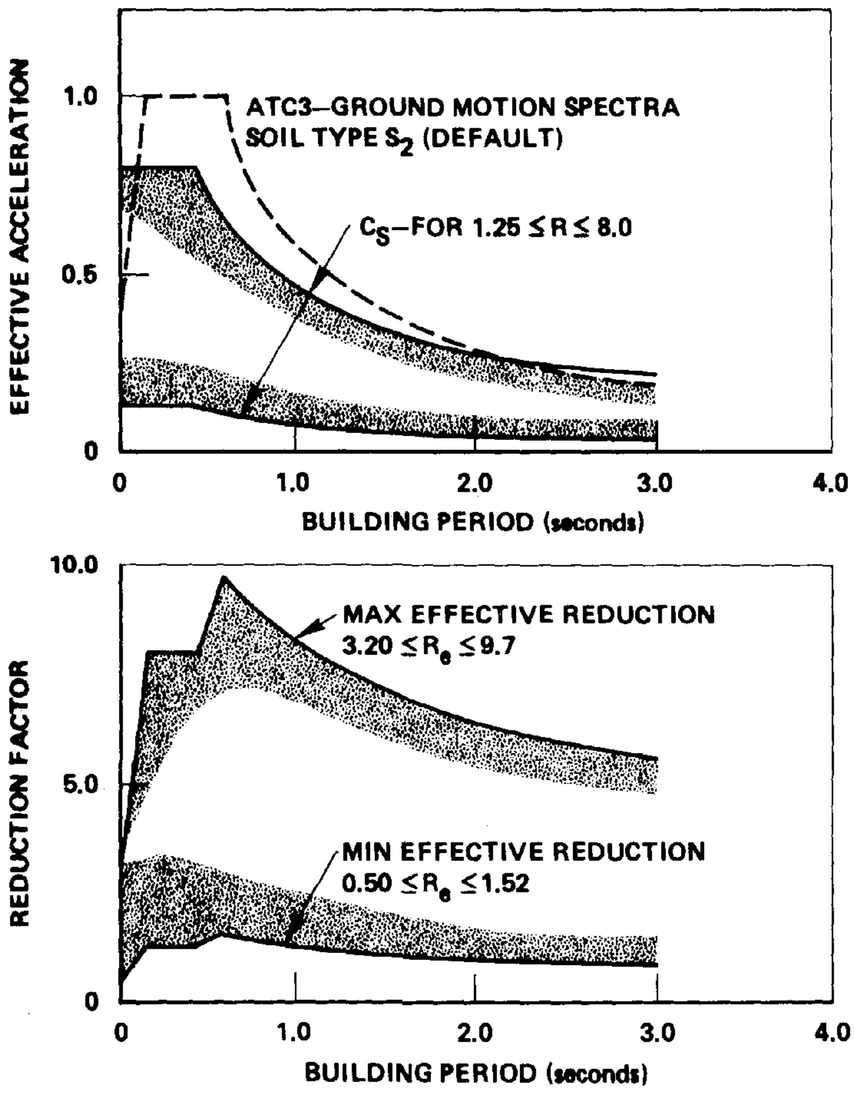

Figure 4.- Design spectra: ATC-3.

associated with the observed structural behavior of long period buildings (Commentary, ATC-3, 1978). Generally, the reasons are related to the possible concentrations of high ductility requirements, the increase in the number of potential modes of failure, the increase in the instability of the building, and the tendency to overestimate the period, all of which increase as the building period does.

Other reductions available to adjust the elastic response spectra to a design spectra such as variations in material strengths and multi-mode effects are handled in ATC-3 on a building by building basis. The material strength is accounted for at the point of design where the allowable design stresses are increased to yield levels. The reduction due to multi-mode effects are determinable by a basic modal analysis, specified in Chapter IV-5, that will properly account for the unique characteristics of the system.

The previously mentioned areas of loading definition not properly covered by a response spectrum approach are also dealt with in ATC-3 on a specific building by building basis. These include the effects of torsion, out-of-phase input motion, orthogonal loading, P-Δ effects, etc.

DESIGN USING TIME HISTORIES

In the most general sense, building design using a time history record as an indication of the expected earthquake motion and threat to the structure represents a substantial increase in sophistication in the analysis process. Since it analyzes structural response in time, it provides the option of varying key structural parameters in time. This allows, as far as is analytically possible, for avoiding excessive conservatism due to arbitrarily combining mode shapes to derive design deflected shapes and empirically assigned ductility reduction factors.

The basic procedure for carrying out a building analysis with a time history record, put as simply as possible, implies solving a structural system using available techniques for a set of constant building parameters over a short time duration for a pulse of input motion. Each solution then becomes the initial condition for the next time step and the process continues as long as required to determine the response of a building to a particular motion.

There are serious differences of opinion in the practising profession regarding the use of time history analysis in the design process. Many engineers believe that the accuracy of the selected time-history ground motion data and time-dependent material characteristic information is not in balance with the accuracy of other loading information under simultaneous consideration, and that the analytical techniques are not sufficiently developed to be useful in the building design process. Such engineers (this author is among them) therefore believe that the use of time-history analysis in the design phase may not be justifiable. Others believe that time-history analysis procedures are sufficiently developed, that they can be used effectively to identify structure components most likely to be damaged, and that time-history analysis complements and can provide a basis for confidence in the response spectrum design procedure. In either case, it is recognized that the accuracy of any time-history analysis is dependent, to a significant extent, upon the accuracy of the ground-motion time-history utilized. The acquisition of site-specific ground-motion time histories that reflect reality in the extreme load case are therefore of utmost importance.

CHAPTER V

INFORMATION OBTAINED FROM STRONG-MOTION RECORDS

by

Jon D. Raggett*

INTRODUCTION

The intent of this chapter is to outline and summarize the information that is likely to be obtained from strong-motion records, whether they be analog or digital. It is a far too common practice for some individual or agency to specify that strong-motion instrumentation is desired, wanted, or needed, while at the same time that individual or agency has no idea whatsoever what information can be obtained from that instrumentation so specified. There is, of course, some value to the collection of strong-motion records when the purpose of such a collection is unknown simply because sometime, down the line, someone will likely know what to do with them (assuming that the state-of-the-art advances) and will be able to extract some informaton from them. However, if there is no specific purpose for the instrumentation (other than to collect strong-motion records), then the use of the instrumentation typically is very inefficient, and invariably, that one bit of information needed to make the strong-motion records of great value is not obtained. Therefore, by summarizing and outlining what information is likely to be obtained from strong-motion instrumentation, the instrumentation can be designed to obtain that information efficiently and accurately.

Anyone desiring to install strong-motion instrumentation must first determine what information they want to obtain from such instrumentation, must review the analysis techniques for obtaining that information from the strong-motion records, and must then design an instrumentation plan compatible with the chosen analysis procedure or procedures to get that information. Only if these steps are taken can the desired information from strong-motion records be obtained with success.

PURPOSE FOR STRONG-MOTION INSTRUMENTATION

This section and the remainder of this chapter are divided into two categories: strong-motion instrumentation for ground motion studies and strong-motion instrumentation for structural response studies (which necessarily involve ground motion transducers as well as transducers in the structure).

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Ground Motion Studies

The most specific purpose for instrumenting a ground site is to evaluate the seismicity of the site in question. The use of any information obtained at such a site is solely to create a data bank of seismic activity for that site in question. A specific example might include the site for a future LNG terminal. If a site is chosen, and yet construction of such a facility is not likely to commence for several years, it would be desirable to instrument the site to obtain information about the seismic activity at the site in the intervening years, as well as the years after construction commences.

Another purpose for instrumenting ground motion sites is to collect ground motion data for use in a general data bank of ground motion information for the design of structures. There may never be a structure of consequence other than a head of lettuce at a particular ground site in the Imperial Valley, but the proximity of that site to the Imperial fault may make it a valuable site for collection of ground motion records. The probability of obtaining significant strong-motion records from such a site in a short time interval makes such a site valuable. In essence, the return for an instrumentation dollar is likely to be high. Any information obtained, however, from such a site would be used by the engineering profession for the design of structures not located there, but located on similar sites geologically.

Another purpose for instrumenting a ground motion site is to verify, to identify, or to generate mathematical models of soil dynamic behavior during seismic disturbance. Again, for such a purpose, there may never be a need to use the ground motion data for the design of a structure at the site in question. However, in this case the purpose of the instrumentation is to verify experimentally some mathematical description of the dynamic behavior of the site in question so that the mathematical description may be used to predict the ground motion with accuracy to be experienced at some other site. Of interest may be the mathematical description of a column of soil above bedrock, or the mathematical description of the spatial variation of the surface ground motion in the vicinity of a site in question.

For each purpose mentioned above, the information obtained will be different, the instrumentation needed to obtain that information will be different, and the required knowledge of the surrounding area will be different.

Structural Response Studies

A specific and very important purpose for instrumenting a building structure is to determine, in the event of an earthquake, whether or not damage has occurred.

Following this purpose is that to determine, again in the event of an earthquake, the reliability of the building structure to withstand future earthquakes. This reliability or safety analysis can, of course, be performed for structures that have not yet experienced damaging earthquake motions. Hopefully, the need for such analysis is slight.

The third and most common purpose for instrumenting structures is to understand the dynamic behavior of structures to earthquakes so that mathematical models used to describe such behavior can be improved. In may instances it is important to have an improved mathematical model for the dynamic behavior of a specific structure. For example, if the mathematical model describing the dynamic behavior of a nuclear power plant is improved from observations of its response to a small earthquake, then its predicted response to a major earthquake can be made with greater assurance. For such a critical structure, there is great value to improve continually the mathematical model of the dynamic behavior as the structure ages and is modified.

More than likely, the purpose for instrumenting structures is to improve the body of knowledge used to formulate mathematical models of dynamic behavior. By fully understanding the relationship between the design of a structure and its dynamic response to an earthquake, one can then predict with greater assurance the response of a new design to a suitable, hypothetical, design earthquake.

SPECIFIC INFORMATION OBTAINED

This section, the body of this paper, will be divided between ground motion studies and structural response studies, and will further be divided according to the purpose of the instrumentation, when possible. There obviously will be overlapping because data from a model improvement study may be used, some time in the future, for damage assessment and reliability analyses in some yet to be determined manner.

It should be noted now that there is value in the collection of data even though it is not known in the least bit what to do with that data in hopes that someone will know what to do with it later. Such a haphazard, unguided approach has more credibility in the field of earthquake engineering than in many other scientific fields, because significant earthquakes in populated areas occur rather infrequently. Therefore, the volume of data from strong earthquakes is still rather small (particularly for statistical analysis purposes) but is steadily growing due to the mushrooming network of strong-motion instruments (described by Matthiesen in Chapter II of these Proceedings).

Ground Motion Studies

Ground motion studies for the purpose of evaluating the seismicity of a site typically incorporate, as a minimum, one triaxial package of accelerometers at the site in question, whether that site be on the surface or below the surface.

The information to be obtained from the triaxial package of accelerometers is well-known, but will nonetheless be reviewed for completeness and for the purpose of emphasizing the amount of information that can be obtained from such time histories at a level of sophistication often overlooked.

Specifically, acceleration time histories (analog or digital) in three mutually perpendicular directions are recorded. As described in another chapter of these Proceedings these raw recordings need a certain amount of processing in order to eliminate recording and transducer errors. They may then be integrated to yield velocity and displacement time histories for each of the three mutually perpendicular directions. As easy as it is to say this, until relatively recently (the last ten years), displacement time histories so integrated were not obtained with reliability. Although some displacement time histories may have been integrated accurately before this time, in general it could not be said that displacement time histories could be included in the body of information obtained if accelerations were recorded. Due to the efforts of Brady and Perez (1976), Trifunac and Lee (1973), now it can be assumed that velocity and displacement time histories are also obtained if acceleration time histories are recorded.

Time histories by themselves are of great value as input motions for dynamic analyses of structures. Of great value too are simple characterizations of time histories often overlooked. Typical characterizations are peak values, duration of strong-motion, number of peaks greater than a specified value, dominant frequency, etc. If dynamic amplification of a structure is not expected to be significant, then all the information needed for a seismic analysis is contained in peak values. Even for structures for which dynamic amplification is significant, that dynamic amplification can often be estimated and the structure can be designed using the simple characterizations of the time history described above. Often this simplistic approach is sufficiently accurate for many structures, but often this approach is overlooked.

Of course, characterizations of earthquake time histories, other than the simplistic one I have made in the previous paragraph, are important (only after the need for them is warranted). The time history (of acceleration velocity, or displacement) can be converted into its Fourier Transform (Robson, 1964). A Fourier Transform is a description of a ground motion in the frequency domain every bit as complete as the more usual time history in the time domain. Transforming from one to the other is completely reversible without loss of information. The significant information to be gained by making such a transformation is to identify, more accurately than by observing time histories, the frequency content of the ground motion. Dominant ground motion frequencies become immediately apparent. Specifically, the energy in the ground motion between two frequencies is proportional to the area under the square of the Fourier Transform. Obviously, the relationship of dominant ground motion frequencies to natural frequencies of vibration of the structure are of utmost importance.

A third and most common characterization of a ground motion is by its response spectrum; a characterization of the ground motion by the peak response of a simple oscillator when subjected to that ground motion as a function of the natural frequency and damping ratio of that oscillator (Clough and Penzien, 1975). Less information is contained within a response spectrum than in a Fourier Transform (one cannot recreate a time history from a response spectrum), but it contains its information in a very usable form. Specifically, maximum modal responses of a structure whose dynamic motion is modeled as a linear multi-degree-of-freedom oscillator can be obtained directly from a response spectrum.

Ground motions recorded at a site and characterized in one of the manners described can then be used in any manner to evaluate the seismicity of the site. Specifically, if several time histories are obtained, an average normalized response spectrum can be obtained which yields the frequency content of the ground motion that can be expected at the site. Furthermore, peak values, their frequency of occurence, and extreme value distributions can be used to determine the occurrence interval of large, damaging earthquakes.

Ground motions recorded at many sites and characterized in one of the manners described can be normalized and averaged to be used by the design profession in general. It is in general, very unlikely that several time histories of ground motion will be available for one site in question. More than likely, there will be several time histories available for similar sites, whether that site is on bedrock, on alluvium, on sand, etc. Response spectra for many ground motions at similar sites can be normalized, averaged, and then rescaled to yield a suitable design spectrum.

If the seismic history is known at a site, as described in the first part of this section, it is not necessary to determine the transmission path characterstics of seismic waves from a source to the site, because the seismic characteristics of the site (the end product) are already known. However, this is a very rare situation to have seismic records in a sufficient quanitity to be available at a site in question. It is far more common to know something about the bedrock motion, to know something about the soil characteristics above the bedrock, and then create from fundamental principles the transmission path and the modification to the seismic waves between the bedrock and the site in question. In order to further the state-of-the-art (of soil dynamics) special soil dynamic studies can be made. A discussion of what information can be expected from such studies follows.

If a pair of time correlated triaxial-packaged accelerometers exists, one at the surface and one at the nearest bedrock location, then the characteristics of the soil structure between the bedrock station and the soil surface can be determined. The usual information which can be obtained from such a pair of transducer stations is a transfer function. If it can be determined that one component of a surface ground motion is caused directly by one component of bedrock motion, and if the motions are sufficiently low in amplitude so they can be approximated with a linear model, then a complex, frequency dependent transfer function can be defined and found relating those two motions. Specifically, for:

$X(w)$ = Fourier Transform of the input time history, $x(t)$
$Y(w)$ = Fourier Transform of the output time history, $y(t)$
$H(w)$ = Transfer function

then

$$H(w) = Y(w)/X(w)$$

Under the assumptions outlined, the transfer function $H(w)$ is an earthquake independent, amplitude independent characteristic of the soil structure dynamics. Of course, its value lies in its use in determining site ground motions for a future, hypothetical design earthquake (likely to be much larger than the earthquake used to determine the soil structure transfer function - unless one is so lucky). Specifically, if $x^*(w)$ is the design earthquake time history for the bedrock motion and $X^*(w)$ is its Fourier Transform, then

$$Y^*(w) = H(w)X^*(w)$$

where $Y^*(w)$ is the Fourier Transform of $Y^*(t)$, the desired time history of the soil surface motion which is readily available through inverse transformation (Robson, 1964).

Although such a procedure sounds simple and very useful, its unguided use can be misleading and dangerous. The key to the validity of a transform is that there exists a one-to-one relationship between the input and the output. If the soil structure is a relatively thin layer of soil above bedrock, it is reasonable to assume that one component of the surface motion is related directly to the corresponding component of the bedrock beneath. However, if a deep alluvium deposit overlays an irregular bedrock base, the likely source of one component of the surface motion is the motion of the entire bedrock surface at the alluvium/bedrock interface. It is an impossible task, at least at the present time, to obtain the proper input/output relationship from such strong-motion records. It is particularly discouraging because mathematically, a transfer function can be obtained from any two time histories; it is simply a mathematical exercise. However, unless a one-to-one relationship exists between the two, the transfer function so obtained is not unique, and cannot be used as a characteristic of the soil structure dynamics.

One other assumption made regarding transfer functions is that the soil dynamics must be able to be modeled accurately with a linear model. Unfortunately, for the high level, damaging motions of real interest to earthquake engineers (and related professionals) the linear model breaks down. Granular displacement and shifting occurs invalidating the assumptions that must be made while formulating a linear mathematical model. It immediately would be asked, "Can one obtain meaningful information from strong-motion records to formulate and improve non-linear models of soil structure dynamics?". My response would be, at the present time, very little informaton really could be gained. The most that one could hope for is the verification of a preconceived and formulated non-linear model. For example, a finite element model including non-linear effects could be formulated for the soil structure dynamics (Idriss, 1968), and a predicted response could be compared to the measured response when the model is subjected to the measured input (if it can be defined and recorded). If the match is good, then the model is verified, at least for this case. If the match is poor, it is unlikely that the proper model parameters (and there will be many) can be identified from the strong-motion records. A Fritzonian Delta method (R. B. "Fritz" Matthiesen, oral commun.) can be employed to improve the model, i.e., tweek modeling parameters until a better match between predicted response and measured response exists. The more complex the model is, the less likely that such a procedure will succeed. Unfortunately, the simplest of realistic non-linear soil structure models is very complex (I am considering a single-degree-of-freedom linear oscillator to be simple).

Another purpose for collecting information regarding soil structure dynamics is to determine the spatial vibration in ground motion over a large site. The information is only now being collected and as of yet has not been incorporated into the design phase of structures.

Typically, the ground motion at a site is characterized by three mutually perpendicular time histories at a point. It is then assumed (rightfully or wrongfully, but really out of necessity) that the entire site undergoes this identical three dimensional motion. This may be bedrock motion or it may have been modified to take into consideration the soil structure dynamics above the bedrock (if such a soil structure exists). Such an assumption may be valid for tall, high-rise structures for which the plan dimensions are relatively small and the natural frequencies of the structure are low. However, this assumption may not be valid, and is not likely to be valid for long, low structures (buildings and bridges) whose plan dimensions are large and whose natural frequencies are relatively high. The reason why this assumption is not valid is, of course, because the seismic waves are traveling waves that take a finite time to get from point to point.

How can phase relationships be obtained and what information is to be gained from such a study? Phase relationships amongst nearby ground motion have just recently been recorded. One, two-dimensional horizontal array of time correlated accelerometers is located in Tokyo, Japan (Hayaski, Tsuchida and Kurata, 1977), with six transducer locations, each spaced 500 meters apart with two points also having downhole instruments. Another one-dimensional horizontal array of triaxial packages is located in the Imperial Valley (Bycroft, 1980). In this array there are six stations with spacings of 60, 120, 240, 280 and 300 feet between each.

The most valuable information to be gained from such an array of time histories, at the present time, is to simply have a set in one's possession. One could then use these ground motions to compute the response of, for example, a bridge (one would most likely have to write a program that would accept variable ground motion inputs) and compare the results obtained to the response of the same structure to a single, representative ground motion. Such information published would be exceedingly important just to see if there are similarities in the two responses, and to see if what has been done in the past (using a single ground motion) has value or not. It is premature to say what form the information should take from an array of time histories other than to have an array of time histories on hand. One could compute all sorts of correlations, cross-correlations, spectra, co-spectra, etc., but I am not sure I would know what to do with them if they were obtained. Unfortunately, there is not an algebra developed which can handle such an array, as there is for an array of stationary ergodic random processes (Robson, 1964) (except as noted in a very specific deterministic sense).

Another bit of information about soil structure dynamics that can be obtained, and rarely is, is the spatial variation in amplitude (not phase) over a particular building site. Typically, rotational components of ground motions in a horizontally homogeneous soil structure are small and of little significance structurally. However, on a site where the soil structure is not horizontally homogeneous, rotational input motions to the structure may be significant and are often overlooked. Two buildings on sites within a few blocks of my office come to mind immediately. One building (really four structures loosely connected through expansion joints) is spread horizontally over a site approximately 200 feet wide. The eastern-most section is founded directly on bedrock with spread footings. The west side of the building, on the other hand, is founded on 60-foot-deep drilled caissons to bedrock. The lengths of caissons gradually decrease as the caisson location approaches the eastern side. While it can be expected that the east and west bedrock motions at this site will be similar, it is certainly not expected that the east and west surface motions will also be similar. Certain frequencies of bedrock motion will be highly amplified by the 60 foot overlayment of loose material, and other frequency ground motions will be completely filtered by it. This spatial variation in ground motion amplitude (not phase) over a relatively small separation distance is equivalent to a translational ground motion and a significant rotational ground motion. At certain locations in the structure, responses may be greatly in error if this rotational motion is not modeled correctly. Where building sites (or instrumentation sites) are not horizontally homogeneous, it should become standard practice to obtain time correlated ground motions at nearby locations to determine quantitatively the importance of spatial variation in ground motion amplitudes. Except for sites in the middle of the Imperial Valley, I would expect that this spatial variation is far more prevalent than is considered at the present time.

Structural Response Studies

The information that can be obtained from strong-motion studies of structural response will be discussed in a reversed order from what was presented earlier when the purpose of the instrumentation program was discussed. Considered first will be the information to be obtained for model improvement (advancement of the state-of-the-art). Then discussed will be the information to be gained from strong-motion records for damage surveys and reliability analysis. Information for these two analyses can best be described in terms of information found for use in model improvement.

The information to be obtained for model improvement is divided into three groups of ascending sophistication: peak values, information for model verification, information for model indentification.

Peak values are by far the simplest quantities to obtain from strong-motion records once they have been corrected. In fact, they can be obtained directly from simple peak value recorders rather than complex strong-motion recorders. The value of recording peak values only is often overlooked in light of the glamour of fully instrumenting a high-rise structure with several accelerometers. Even with the glamour of such instrumentation, relatively few buildings are heavily instrumented and only now are a sufficient number of structures being instrumented so that statistics of response data can be generated (although they are not really being generated now, and should be). Hundreds of very simple peak value recorders could be used to literally blanket an entire area for one earthquake. Significant, useful data could be obtained from such a blanket.

Peak accelerations only are still used in almost all building codes, and structures have been built safely to such standards for years (some structures have not been, but some structures probably have not been designed well with sophisticated dynamic analyses either). If dynamic amplification of the response is not expected, then a peak ground acceleration, or peak structure acceleration (since in this case the two should be the same) tell the entire story, i.e., from peak accelerations are obtained peak inertial forces from which peak member actions can be found with accuracy throughout the entire structure. For such a structure, no matter how sophisticated the dynamic analysis is, the final results, i.e., member actions and displacements, will be only marginally different from those values obtained from a simple peak value analysis. It should also be kept in mind that for structures for which dynamic amplification is significant for earthquake motions the peak response is rarely more than two or three times the peak ground motion (Newmark and Rosenblueth, 1971). One rarely obtains the high dynamic amplification associated with resonant mechanical vibrations of a lightly damped system (Clough and Penzien, 1975). For this reason, peak value studies alone are of greater importance for earthquake vibrations than for mechanical vibration problems and should not always be overlooked in favor of strong-motion recorder installations, particularly when the cost and time required to make such an installation may prevent or delay the installation from occurring.

So far, peak accelerations alone have been discussed as the usual peak quantity obtained. Peak displacements and relative displacements can be obtained for structures and be of vital interest. Gaps between elements of critical structures are of great importance. Peak relative displacements across expansion joints are of great concern, particularly when unseating on one side may lead to the total collapse of the structure, as was the case for many bridge failures in the San Fernanado earthquake in 1971 (Elliott and Nagai, 1973).

I do not want to belabor the collection of peak response information (acceleration and displacement particularly) because it is relatively simple, particularly if a peak is simply read from a strong-motion history. However, I simply want to make the point that peak value data alone can be of vital interest for many structures, and should not be overlooked, particularly if the collection of peak value data is possible when the collection of strong-motion time histories may not be.

The next, more sophisticated level of information gathering from strong-motion records of structural response is model verification. Historically, this is the most common level of data accumulation from strong-motion records and for many structures it may be the best. What information can be expected to be obtained from a model verification?

Several examples of a model verification approach are exemplified by the structural response analyses included in the NOAA Reports of the San Fernando Earthquake (Murphy, 1973). Specifically, the process of model verification is defined as follows: a) complete input time histories are recorded, b) at least one time correlated response time history is recorded, c) mathematical model of the structure dynamics is formulated, d) the model is subjected to the recorded input motions, and e) the model response is compared to the recorded response. If the two responses do not match well, then the model is not verified and a new and improved model must be found. To find a new and improved model and the mismatch of response records can be very difficult and very frustrating. It may also be totally without success. If the structure is simple and if its motion can be described with relatively few degrees-of-freedom, then the likelihood is high of identifying the best-fit mathematical model of the structure dynamics using this trial and error procedure of model verification (alluded to earlier as the Fritzonian Delta Method). For example, if the structure dynamics can be described accurately with a mathematical model having three degrees-of-freedom, and the mass distribution is known, then it may be possible to obtain a reasonable fit between theoretical and measured response records by tweeking stiffnesses and damping coefficients, providing that a linear model of the structure can be used. If the response is definitely non-linear, the likelihood in finding a good model is not so great. If the structural response is non-linear and the structure dynamics can be described with a model having only one- or two-degrees-of-freedom, then it may very well be possible to identify a good model through these trial and error procedures.

Once a model has been verified, then what information really has been obtained from the strong-motion records? In answer, literally every parameter of the model has been "obtained" from the strong-motion records whether it is a mass, a stiffness, a damping coefficent, a frequency, a damping ratio, etc. Literally, every characteristic of that model has been "obtained" from the strong-motion records although it required a great amount of rational analysis, intuition, experience and general know-how to "obtain" those model characteristics from the time histories. Obviously, if the analyst cannot formulate a mathematical model from the fundamental principles first, he or she can never identify a mathematical model from strong-motion records using these trial and error techniques.

What are the chances for success if the model has 2,000 degrees-of-freedom? I would say the chances are not too great although some significant information still could be gained from the records. Even though the mathematical model can most easily be formulated using 2,000 degrees-of-freedom, the response may be dominated by the response of three or four modes of vibration. Our choice of using 2,000 degrees-of-freedom to describe the structure dynamics was then not very efficient if we could have described the structure dynamics with three of four modal degrees-of-freedom. The model parameters then to be tweeked and eventually identified are masses, stiffness, and damping coefficients in modal coordinates rather than in physical, measureable coordinates. Although this may sound complex, these model parameters in modal coordinates can, of course, be transformed into other model parameters in modal coordinates which do have physical significance, namely natural frequencies, damping ratios, and participation factors. Since the equations of motion, written in modal coordinates, are uncoupled, the total number of parameters to be identified are only three per mode per response time history (for other response time histories only the participation factors should vary).

Unfortunately, but as expected, one cannot retransform back to a model with 2,000 degrees-of-freedom from a model which has been identified with three degrees-of-freedom. Therefore, of what value is a three degree-of-freedom model, which has been identified, in furthering the state-of-the-art of modeling building structures? Well, so far, very little. Only when a relationship is found between the model of the structure in physical coordinates (2,000) and the model identified in modal coordinates (Clough and Penzein, 1975) has the state-of-the-art been advanced. Unfortunately, models of structural dynamics behavior cannot be formulated in modal coordinates from the dimensions and properties of the building materials. Although hundreds if not thousands more degrees-of-freedom are required, it is far easier to formulate a model from the dimensions and materials of the building using physical coordinates. It is at this time when the benefits of experimental data are needed. It is at this time that answers are needed to design formulation questions such as, 1) shall I use center-to-center or clear span length, 2) shall I used cracked section or uncracked section properties, 3) what portion of the slab shall I use in modeling a beam stiffness?. Therefore, there still exists a major model verification problem even if some mode shapes, frequencies, and damping ratios of a structure have been found from strong-motion time histories. Instead of verifying a model by fitting its earthquake response to a measured response, the problem exists of verifying a model of a structure by fitting its first three modal parameters to those modal parameters which have been identified from the strong-motion records. A step has at least been taken in the right direction as this latter fitting problem is somewhat easier than the former. Also, it is somewhat easier to see what parameters should be tweeked to obtain a better fit, i.e., 1) if the measured frequencies are higher than the model frequencies, increase the model stiffnesses; 2) if the measured mode shapes are more shear-beam-like than the model mode shapes, increase the floor structure stiffnesses relative to the column stiffnesses; and so on. After model mode shapes, frequencies, and damping ratios are fitted to the identified mode shapes, frequencies, and damping ratios, then some of the modeling guide questions mentioned before can be answered.

Absolutely, the most important requirement for any model verification scheme to succeed is that a description on the complete input time history must be recorded, even if there is only one response time history. Unless the complete input is recorded for a recorded output, there is no chance whatsoever of verifying a model relating an output to an input. At times, one becomes preoccupied with the collection of a complete response data while overlooking the completeness of the input. If the input is incomplete and not correctly identified, no matter how many response time histories are recorded, the most information that anyone can expect to obtain are peak values, some natural frequencies (from Fourier Transforms of response records alone) and approximate mode shapes (maybe one or two from relative response amplitude at several transducer locations). All, certainly is not lost, but the great value of the instrumentation (identification of the model relating output to input) has been lost.

The highest level of information collection from strong-motion time histories of structure motions (as I have grouped them) is what is called model identification, or system identification. What differentiates model identification from model verification is that for the former, a model is systematically identified relating the output to the input according to some goodness-of-fit criteria. Ideally, the information to be gained from identification procedures is the same as what is expected to be obtained from verification procedures, although the likelihood of success is far greater for identificaton procedures than for verification procedures. One would hope to obtain, from the output records and time correlated input, a complete description of the mathematical model which best relates the two. This can be a very difficult objective to achieve, even for a very simple structure. The information that can be expected to be obtained, in essence, is the same which was discussed in the previous section on model verification. The difference between the two is in the manner in which that information is obtained. The actual information to be obtained is dependent in some degree upon the procedure used to obtain that information. Some of the procedures, and the information which can be obtained from each will be discussed as follows.

Complete and descriptive papers describing several of the model identification procedures have been written by Hart and Yao (1977) and Ibanez (1977), in particular. Although both are somewhat dated, and obviously new procedures with new hopes have appeared on the scene, these references both include significant categories which exist and both include good examples of procedures for each category. There are several procedures and several ways to categorize them. Typical divisions in these two papers are as follows: 1) procedures in the time domain and procedures in the frequency domain; 2) direct methods and iterative methods; 3) methods which require a priori model which is to be improved and methods which do not; 4) methods which do or do not account for uncertainties in the model; and 5) methods which do or do not account for uncertainties in the data.

Frequency domain methods are the oldest methods having been developed years ago primarily for mechnical-electrical systems for which a linear model is most valid. These methods can be applied to structure vibrations with great success when the input is a mechanical vibrator force rather than a strong-motion earthquake. Discussions of results from mechanical vibrator-induced structural motions include Jennings, Matthiesen and Hoerner (1971), Ibanez (1977), Chen, Czarnecki and Scholl (1977), and Galambos and Mayes (1978). All of these discussions, however, are for cases with mechanical vibrator input forces. Of interest here, is what information can be expected to be obtained from earthquake induced strong-motion records using these frequency domain procedures?

For each response record, if it is the response to a singly defined input record, a transfer function can be obtained for the response of the structure at that point as the ratio of the response Fourier Transform to the input Fourier Transform. Typically, what can be expected to be obtained from such a transfer function are a few natural frequencies of the structure from the peak locations, and the damping ratios for each frequency from the peak widths. If more than one response record is obtained, mode shape data for each significant peak can be obtained from relative peak amplitudes. All results are average, best-fit modal parameters. It is possible for several natural frequencies of vibration to be identified. It is likely that more than a couple of damping ratios and mode shapes will be identified with acceptable accuracy. The probable source of error in these procedures are the non-linearities in the response of most structures (even at relatively low amplitudes of motion) and the omission of spurious input motions. Even for the most symmetric designs, asymmetries in the materials, detailing and base motions will often introduce rotation and coupling of response motions to orthogonal input motions. When such is the case, one is measuring an effect without the cause, which obviously will lead to errors. Because of these limitations, classical frequency domain procedures have not been used extensively in reduction of strong-motion response records of structures.

Perhaps the simplest of the time domain procedures is the direct procedure attributed to Kozin and Kozin (1968). Specifically, a best-fit linear model of the dynamics of a structure is obtained directly, given the mass matrix of the structure, given the input time histories (as many as are needed to describe the input accurately), and given a response time for each pertinent degree-of-freedom necessary to describe the response. At first, this appears to be a mind-boggling set of requirements, but often it is not. Typically, the motion of a structure in one vertical plane can be described adequately with three or four modes of vibration. Typically, too, for the structures of particular interest there will be as many accelerometers to record independent motions. One can then set about to identify a best-fit linear model of the structure dynamics using a set of three or four generalized coordinates. Knowing the mass distribution, one can then solve for the mass matrix associated with such a coordinate system as well as each vector of participation factors, one for each generalized degree-of-freedom for each input time history. What one solves for, and solves for directly, is a damping matrix and stiffness matrix, again in generalized coordinates. With such a complete model, one can then make a coordinate transformation to modal coordinates, just as one would make from the more usual physical coordinates to obtain the usual modal parameters of mode shapes, frequencies, and damping ratios.

Required for the analysis above are a set of response velocities and displacements as well as the recorded response accelerations. The velocities and displacements are readily obtained from the accelerations by integration as described by Brady (in these proceedings). The damping and stiffness matrices are, in fact, obtained (by this author) with greater accuracy if the response accelerations, first derivative of response accelerations, second derivative of response accelerations, and the second derivative of ground accelerations are used in lieu of the response displacements, response velocities, response accelerations, and the ground accelerations. The mathematical model relating the first set is the same model which relates the second set of time histories. The first set of time histories yields more accurate results than the latter set, most likely for the following reasons: small amplitude high frequency response information is not lost through differentiation, while it is lost through integration. Unfortunately, high frequency errors or noise are amplified through differentiation. However, least squares curve fitting procedures are not seriously affected adversely with the introduction of high frequency noise while the same procedures are greatly affected adversely by the introduction of low frequency noise, the product of two integrations.

I have discussed the above procedure at great length even though it appears to have the two principle failings attributed previously to the classical frequency approach. A linear model of the structure dynamics has been assumed, and average properties only are obtained over the entire length of the time histories. However, the length of the time history required for analysis can be very small, equal to or even less than a period of the fundamental mode. Therefore, the best-fit model parameters which are obtained can be average values for a very short length of time only. Some non-linearities in the response of the structure can then be obtained from this procedure as time variations in the best-fit linear model parameters. Raggett and Rojahn (1978) discuss further the use of this procedure with applications to the analysis of bridge motions. It is particularly well-suited for such motions because it can identify modal properties of structures having closely spaced natural frequencies (common to bridges) and it can handle three-dimensional responses to three-dimentional input motions (common to bridges) as easily as it can handle one-dimensional responses to one-dimensional inputs.

Although complete damping and stiffness matrices are the product of the analysis described above, except for simple structures, these matrices must be in terms of generalized coordinates and must then be transformed to modal coordinates to have the greatest significance. For structures with several degrees-of-freedom, one then still has the problem of identifying the model in physical coordinates from the model which has been identified in generalized and modal coordinates. It is, again, only after such identification has been made, has the state-of-the-art really been advanced from information obtained from strong-motion records.

Another time domain analysis procedure is an iterative procedure developed by Raggett (1974). For such an analysis, modal responses must first be filtered from the response. (Obviously, for structures having closely spaced natural frequencies this procedure cannot be used). Properties of a single-degree-of-freedom oscillator are then obtained iteratively from initial estimates in order that the single-degree-of-freedom oscillator response matches the modal response (which has been filtered from the recorded response time histories) in a least squares sense. One modal response from one response time history caused by one or more input time histories is analyzed at a time. Expected from such an analysis are a natural frequency, a damping ratio, and participation factors. Note that properties of more than one mode may be obtained from a single response time history. After the analysis of several time histories, mode shapes can be obtained from the relative amplitudes of the participation factors for each mode obtained. From the analyses of several structure motions using this procedure, modal parameters for two or three modes of vibration can be expected to be obtained.

Similarly, for this procedure, short time histories taken from anywhere in the total record may be obtained. Therefore, again, piece-wise linear modal properties may be obtained and some non-linearities may be identified as time variations in the linear modal parameters obtained.

This writer has had the experience of using both of these time domain procedures with time histories of real buildings and believes that the former of these two time history procedures is by far the most informative. Furthermore, the former method can be readily modified to identify modal properties of a single-degree-of-freedom oscillator and be used in the second procedure.

Torkamani and Hart (1978) have developed a procedure to identify an impulse response function from time histories. Again, the same restrictions apply to the identified impulse response function that applied to the transfer function so obtained in the frequency domain approach. Specifically, the response must be the response to a single input, the system must be modeled as being linear (otherwise the use of the impulse response function is severely restricted because super-position is no longer valid) and in order that the impulse response function be defined with acceptable accuracy, it must be the average of a long time history. Identification of an impulse response function, while informative in its own sense, is really only an intermediate step in the previous cases. Specifically, model parameters in physical coordinates must then be identified from the impulse response function before the state-of-the-art of earthquake resistant design of structures can be improved.

The identification of structural response parameters assuming a non-linear model is of particular importance because the behavior of most structures is in a non-linear manner at high amplitudes of motion, because the behavior at high amplitudes of motion is at the present time somewhat uncertain, and because the ultimate capacity of the structure to withstand earthquakes depends upon its behavior at high amplitudes of motion. Two basic approaches are taken in the identification of non-linear model parameters. The first approach is to assume the form of a non-linear model in terms of unknown coefficients. A first usual extension from a linear model is to assume, for example, that there exists a stiffness term proportional to the cube of the relative displacement. The other approach is to assume a simple model, most likely linear, and identify non-linearities as time dependent best-fit linear properties such as frequencies and damping ratios. This approach is similar to that discussed previously by Kozin and Kozin (1968), except for that case, best-fit linear model parameters were found for time segments, while the methods discussed here consider continually varying model parameters.

Distefano and Rath (1975) have presented a procedure for identifying unknown parameters of an assumed non-linear model for a single-degree-of-freedom oscillator. The procedure is identical to that suggested by Kozin and Kozin (1968) with the addition of cubic stiffness and damping terms. The time histories of the cube of the relative displacement and the cube of the relative velocity are known time histories, inputs, and only their constant coefficients are unknown. Therefore, one can solve directly for the unknown coefficients as was done by Kozin and Kozin because the equations are linear with respect to the unknown coefficients, even though the models are non-linear with respect to displacements and velocities. What can be obtained from such an analysis, for a single-degree-of-freedom oscillator are the unknown model stiffness and damping parameters. Since the procedure is the same as that discussed earlier (Kozin and Kozin, 1968), this least-squares-best-fitting can be done for short segments of time, so in addition variations in the best fit models may be obtained. The use of a non-linear model eliminates the real necessity to do this. McNiven and Matzen (1976) have used a similar procedure to identify non-linear model parameters which describe the dynamic behavior of a model structure on the shake table at the Richmond Field Station (University of California, Berkeley).

More recently, Distefano and Peno-Pardo (1976) have extended this procedure to identify the non-linear model of a three-degree-of-freedom shear building. Model parameters, including cubic displacement and velocity terms, were found using dynamic programming procedures for records obtained from shake table tests, again at the Richmond Field Station. While this is an improvement upon the procedure used to obtain model parameters for a single-degree-of-freedom model, it is still far from being particularly useful in determining model parameters for full-scale structures, unless the motions of the full-scale structure can be described accurately with three physical degrees-of-freedom. The motions of a 2,000 degree-of-freedom structure can be modeled accurately with three modal degrees-of-freedom if the structural response remains in the linear range. Since this transformation of coordinates is not possible for non-linear systems, the use of procedures limited to a few degrees-of-freedom cannot readily and meaningfully be used on structures with more than a few physical degrees-of-freedom.

An identification procedure which identifies continuous time variations in modal parameters has been recently used and developed by Udwadia and Jerath (1980). From such an analysis, a time dependent fundamental frequency and a time dependent fundamental damping ratio were obtained from strong-motion records of the Milliken Library response to the 1971 San Fernando earthquake. Such a procedure may be used to identify time variations in more than the fundamental mode, although that capacity was not demonstrated.

There are several other identification procedures (see Hart and Yao, 1977; and Ibanez, 1977) that were not mentioned but these do produce basically the same information as those procedures which were mentioned. Specifically, using one or more of the procedures one can expect to obtain the following: 1) damping matrix and stiffness matrix whether it be linear or non-linear for simple (3 degrees of freedom or less) fully instrumented structure; 2) time variation in the linear model matrices mentioned before (at least piecewise if not continuous variation); 3) for larger structures with several-degrees-of-freedom, characteristics of three or four natural modes of vibration; and 4) continuous or piecewise time variation in those model characteristics obtained.

The big problem remains to extract useful design information from the information obtained above to improve the design of new structures to withstand earthquake motions. Only when the information so obtained directly or indirectly influences how a designer models a new structural design will the information be of more than academic value. Damping ratios used in design were obtained in such a manner. A report by J. A. Blume & Associates (1970) is a good summary of damping ratios obtained experimentally, but that data can be updated considerably. It is time for someone to take two or three of the most promising of the model identification procedures and run through all of the structural response strong-motion records and start generating statistics of structural parameters so identified. This data bank can then be expanded as records of strong-motion structural response become available. Others can take this data and systematically fit theoretical models to the data for the purpose of improving modeling techniques. It is time that there is a real return on the time and money invested in the instrumentation of high-rise structures.

How can strong-motion records yield information that is informative in the evaluation of damage and reliability to withstand future motions? At the present time, relatively little can be gained. Reliability or safety analyses can be performed analytically after the damage to the structure is correctly assessed. Therefore, any input to reliability analyses from strong-motion records will come from damage assessment.

Damage to a structure is usually assessed after a visual inspection or some other more sophisticated inspection, such as ultra-sonic inspection of welds. After assessing the damage to the individual elements of the structure, for example see Yao (1979), the damage to the entire structure can be assessed.

Contributions to damage assessment from strong-motion records comes in the form of correlating variations in structural parameters with damage. Galambos and Mayes (1978) have correlated variations in natural frequencies with damage for a structure mechanically vibrated to failure. The same held true for the four-story test structure vibrated to failure by Chen, Czarnecki and Scholl (1977). However, some modal properties vary significantly even though actual failure is not present, as reported by Raggett (1971) for the four-story reinforced concrete test structure mentioned previously. The modal parameters can vary significantly when hairline cracks first form in the structure, or when non-structural elements become unbonded from structural elements (when they should not have been bonded in the first place). Such modal variations can be significant (damping ratios and natural periods can vary by a factor of two) even though no significant damage has occurred. The same variations were observed for several high-rise structures in Las Vegas and are reported by Blume (1969). Although, again, for those structures the modal parameter variations were significant, no real damage was observed.

For very simple structures, damage and the mechanism of damage can be identified more specifically. Rojahn and Negmatullaev (paper in preparation) have vibrated a three-story test structure using conventional explosives. From preliminary analyses of the structure responses, the stiffness and damping matrices have been identified for time segments taken through the entire strong-motion history using the direct identification method of Kozin and Kozin. It is clear, looking at the records, how pre-cast concrete panels become unbonded from the frame at a certain level of motion. Ideally, such data will ultimately be extracted from strong-motion records from multi-degrees-of-freedom, fully instrumented high-rise structures.

SUMMARY

The mushrooming network of strong-motion instruments, as described by Matthiesen elsewhere in these proceedings, insures a wealth of strong-motion records for use in the future. Even to date, there are hundreds if not thousands of strong-motion records available for analysis, and yet at the same time, the information which has been extracted from those records for use by designers is relatively meager.

By far and away, the most valuable information which has been obtained from strong-motion records is a clear definition of an earthquake motion. There are several strong-motion records available, having motions of significant amplitude, on base rock and alluvium that can be used to generate statistics of a suitable design earthquake. There is no dirth of design time histories or design response spectra from which to choose. With this information on hand, one then uses these data to design structures to withstand earthquake motions as discussed by Poland elsewhere in these proceedings. Experimentally obtained input motions (in one form or another) are used in conjunction with a mathematical model of the structural dynamics to generate a prototype response to the design earthquake motions. But how good is that model of the structural dynamics? Fortunately, that information too, is readily extracted from strong-motion records from building motions. For a measured input motion, a model response can also be generated and compared to the measured building motions. If the comparison is good, the mathematical model is verified, and the same modeling procedures used for that structure can be used with confidence to model the structural dynamics of a similar structure. But if the comparison between model response and measured response is poor, the model is not verified and must be improved. It is, for this function, the improvement of the model of structure dynamics, that the building strong-motion records are of greatest value, and yet so far have produced so few improvements. It is true that mode shapes, natural frequencies and damping ratios have been obtained from strong-motion records, and is shown by Udwadia and Jerath (1980) that variations in those properties (at least for the Millikan Library at the California Institute of Technology) have also been obtained. However, until this information is used to improve the procedures for modeling structural dynamics for use by the designers of future buildings, this information so obtained is of academic value alone. Great improvements have been made in modeling procedures, but these improvements have come from theoreticians, laboratory experimentalists, and computer scientists, and have come from the observance of over-all structural behavior during earthquakes. Relatively few improvements to modeling procedures have come from information obtained from strong-motion records. It is time for someone to take all the available building motion records and, systematically, using two or three analysis procedures, create a data bank of dynamic response characteristics so that improvements to modeling procedures can be made based upon genuine and systematic trends. It would be of great value to say "Based upon the results of 64 analyses, the dynamic response of a concrete structure is most accurately modeled if 97 percent of the center-to-center beam lengths are used". Until such statements can be made, the benefits of strong-motion instrumentation have not been fully realized by the design profession.

CHAPTER VI

SUMMARY AND RECOMMENDATIONS
(Compiled by the Workshop Organizers)

OVERVIEW

The second day of the workshop focused on a discussion of the actions which are required to optimize future benefits from strong-motion instrument records obtained in and/or near buildings. The discussion was based on the recognition that the ultimate goal of strong-motion instrumentation is to obtain data that can be used to improve the design of safe and economical structures. The former relates to life safety. The latter relates to cost effective design which is essential to maximize United States productivity.

The recommended actions were developed by practicing, licensed structural engineers, university structural engineering faculty, earthquake data analysis experts, and engineers experienced in the design, deployment and maintenance of strong-motion instrumentation systems. The recommendations are divided into two categories. The first category is that of fundamental policy recommendations and the second deals with important specific technical recommendations on instrumentation, data processing, and research needs.

FUNDAMENTAL POLICY RECOMMENDATIONS

(1) The emphasis of future strong-motion instrumentation programs should be on the instrumentation of buildings and other structures rather than the instrumentation of ground sites. This recommedation is made in light of existing ground motion instrument arrays and data, and the recognition that (1) there is precious little data on the response of real buildings during earthquakes, particularly damaged buildings; (2) there are a vast number of types of existing buildings (the Structural Engineers Association of California has identified, for California, 29 typical building types and heights for study purposes), and in each, structural behavior can be expected to be complex; and (3) without more data on structural response, definitive statements on life safety in and/or near buildings cannot be made.

(2) Buildings should be instrumented in regions of the United States that have significant seismic exposure. This includes states other than California. Consideration should also be given to the instrumentation of buildings that are located both in seismically active and severe wind speed environments.

(3) The instrumentation of typical buildings and non-typical buildings of special importance should be emphasized.

(4) The ATC-3 building design review studies should include several buildings with existing or possible future strong-motion instrumentation. A site specific response spectra analysis should be performed on these buildings if they do not already exist. This recommendation is made to attempt to strengthen the link between structural engineering practice and strong-motion programs.

(5) Detailed ambient vibration measurements should be performed on all instrumented buildings before and after exposure to severe earthquake induced strong ground shaking. This program should be supplemented by a limited forced-vibration program that focuses on unusual or critical buildings, or on classes of buildings that have not yet been force-vibrated. In either case, the acquisition of mode shapes and frequencies should be emphasized, particularly for the fundamental and other lower modes of vibration.

(6) Descriptions of instrumented buildings and installed strong-motion instrumentation systems should be compiled and archived by the organization that assumes overall responsibility for the instrumentation. Additional documentation should include building structural plans (showing precise locations of all strong-motion instruments), design calculations, ambient and forced-vibration studies, and a list of pertinent technical publications.

(7) The value of a computer-based strong-motion information retrieval system such as that developed and operated by the Seismic Engineering Branch of the U.S. Geological Survey is recognized. In regard to instrumented buildings, such a system should provide detailed information on building geometry, lateral force resisting elements, strong-motion instrumentation type and location, history of ambient-vibration, forced-vibration, and earthquake studies, and a citation of pertinent technical publications.

(8) The organization that has overall responsibility for the strong-motion instrumentation should prepare preliminary data description and interpretation reports for all significant records obtained from buildings instrumented under its program. Such reports should be written as soon as possible after the data is acquired and should be designed to point out to the research community and practicing profession the most significant aspects of that data.

TECHNICAL RECOMMENDATIONS

Instrumentation

(1) Strong-motion instrumentation in buildings should include

instrumentation for measuring inter-story displacements. The feasibility of using simple scratch gauges or other displacement devices should be carefully studied in order to provide information supplemental to strong-motion acceleration time history data.

(2) Strong-motion accelerograph systems in buildings should be located in order to obtain data on building translation and torsion (including mode shapes), rocking motion (overturning), floor load distribution (in-plane bending) and overall base input motion. It is recognized that the strong-motion programs of the California Division of Mines and Geology, the U.S. Geological Survey, and U.C.L.A. emphasize the acquisition of such data; they are encouraged to continue to do so.

(3) If site conditions warrant, accelerographs should be installed at "free-field" ground sites adjacent to each instrumented building.

(4) In selected instances, down-hole or additional ground level accelerographs should be installed at building sites where soil-structure interaction or rocking motion is expected to occur. The ground level accelerographs should be located within five to 10 feet of the shear-wall foundations of interest.

(5) In selected instances it may be desirable to install instruments to record strain, accoustical emissions, or other variables (such as temperature changes) during earthquake excitation.

Data Processing

(1) Existing procedures at the U.S. Geological Survey and the California Division of Mines and Geology for processing and analyzing strong ground motion and building base motion records should be continued. Those agencies should be encouraged to document (in descriptive terminology written for the practicing profession) those procedures and to provide estimates of associated errors. The documentation should include a description of the basic assumptions and a discussion of these assumptions upon calculated motion time histories and spectra.

(2) In addition to providing acceleration, velocity, and displacement time histories of strong-motion data recorded in buildings, those agencies that perform standard processing (see above) should also compute important relative displacement time histories (e.g., second floor motion less ground level motion).

(3) Baseline correction procedures used for ground strong-motion records have not been adequately verified for building application. The area requires immediate and thorough study if building displacements are to be qualified with confidence.

Research

(1) System identification research is strongly encouraged and should focus on the utilization of strong-motion data, ambient-vibration data, and forced-vibration data.

(2) Damage indicators, derivable from ambient and/or strong-motion records are needed. Possible indicators include earthquake/ambient period ratios, and earthquake/design base shear ratios.

(3) Technical reports associated with building strong-motion studies, including those that document procedures for systems identification should be available to researchers from a central source. NTIS is not an acceptable source.

(4) The operating characteristics of all strong-motion instruments utilized, such as the Kinemetrics CRA-1 and newly developed digital systems, should be thoroughly documented and evaluated in the laboratory. The laboratory effort should include shaking table tests as well as tests involving expected field environmental conditions.

CHAPTER VII

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APPENDIX A

WORKSHOP PROGRAM

INTERPRETATION OF-STRONG-MOTION EARTHQUAKE RECORDS OBTAINED IN AND/OR NEAR BUILDINGS

April 1-2, 1980

San Francisco Airport Hilton
San Francisco, CA.

Tuesday, April 1, 1980 (Vintage Room)

8:00 AM	Registration (Vintage Room)
9:00	Introduction (Gaus, Hart, Rojahn, Yao)
9:30	Building Instrumentation Programs (Matthiesen)
10:00	Strong-Motion Record Analysis Procedures (Brady)
10:30	Coffee Break
11:00	Utilization of Strong-Motion Records in Building Design (Poland)
11:30	Information obtained from Strong Motion Records (Raggett)
12:00 noon	Lunch
2:00 PM	Written Contributions - Session I (Veletsos)
3:30	Coffee Break
4:00	Written Contributions - Session II (Englekirk)

Wednesday, April 2, 1980 (Vintage Room)

8:30 AM	Subgroup I (Instrumentation; Discussion Leader Ragsdale) Subgroup II (Structural Design; Discussion Leader Hall)
10:00	Coffee Break
10:30	Subgroup III (Computer Programs and Data Processing; Discussion Leader Schiff) Subgroup IV (Damage and Structural Identification; Discussion Leader Sozen)

12 noon	Lunch
2:00 PM	Summary Reports by Subgroup Discussion Leaders
3:30	Coffee Break
4:00	Discussion and Conclusion (Gaus, Hart, Rojahn, Yao)

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PARAMETER IDENTIFICATION

by

Staff of ANCO Engineers*

Estimation of structural parameters of buildings from seismic response is a parameter identification task. ANCO Engineers has a parameter identification capability and is at the present time developing its skills in this area. This brief memo describes some of ANCO's parameter identification skills, software, application experience, and the directions ANCO is taking in this area.

The parameter identification group at ANCO is divided into essentially three groups: (1) eigenparameter identification; (2) model updating; and (3) data acquisition and manipulation.

Eigenparameter identification deals with determining eigenparameters (resonant frequencies, damping, mode shapes, etc.) from test data. The data are not related to a finite element or difference model of the system. All the "fitting" of model response to data is done at the modal level. The ANCO software available to do this is XSHAPE (mode shapes from frequency response data), XTSHP (mode shapes from transient response data--uses the FFT [Fast Fourier Transform]), ANSPI (eigenparameters [ω, β, ϕ] are identified from frequency response data), and ANTPI (eigenparameters are determined from transient response data).

The theory of ANSPI is described briefly. Let S be an ℓxm shape matrix from the *a priori* model (or other source). X is the response to harmonic forcing. Define the transformation $X = SZ$ with $Z = S^iX$ (S^i is the pseudo-inverse of S). Z is near pure modal response. If S were the correct matrix for the test, Z would be pure modal response. Assuming different confidence levels for the elements of X, the following is written: $Z = (\sigma^{-1}S)^i\sigma^{-1}X$. ANSPI breaks the parameter identification scheme into two parts. Assuming Z to be essentially modal response, it seeks to find the best eigenfrequency, damping, and effective mass associated with each component of Z. Once good estimates are obtained for these parameters, a second procedure seeks the matrix α such that the transformation $Z = \alpha Y$ yields true modal response in Y. The overall transformation from X to Y is given by $X = S\alpha Y$, so that the correct mode shapes for the tested structure are given by $S\alpha$. Once this has been done the procedure is used again. This is done until covergence is achieved. The theory for ANTPI is very similar to that for ANSPI; the only major difference is that X is the FFT of the transient response data.

*ANCO Engineers, Santa Monica, California

Another parameter identification effort at ANCO is model updating. This is broken into two sections: (1) Bayesian identification; and (2) sensitivities. Some of the areas of Bayesian identification that have been developed or are being developed are: investigation of covariance matricies (effect on solution), method applied to transient response data, linear constraint relations among parameters, and force identification. For transient response data the criterion function E is defined by:

$$E(P,R) = (P-\bar{P})^T\sigma_P^{-2}(P-\bar{P}) + \frac{1}{\tau}\int_0^{\tau} (R(t) - \hat{R}(t)^T\sigma_R^{-2}(R(t) - \hat{R}(t))dt$$

where

P = vector of model parameters;
$\bar{P}$ = *a priori* model parameter estimates;
$R(t)$ = calculated response vector (from model);
$\hat{R}(t)$ = measured response vector;
σ_P = uncertainties in model parameter estimates; and
σ_R = uncertainties in measured response.

The solution to this problem (values of model parameters, P_0, which correspond to the minimum value of the criterion function):

$$P_0 - \bar{P} = (\sigma_P^{-2} + \frac{1}{\tau}\int_0^{\tau} S^T\sigma_R^{-2}S dt)^{-1}\frac{1}{\tau}\int_0^{\tau} S^T\sigma_R^{-2}(\hat{R} - R)dt$$

This is a Bayesian scheme using a vector of time dependent data.

Another area of Bayesian identification that has been developed is for the case where there are linear constraint relations for some of the model parameters. The criterion function used is:

$$E(R,C) = (R - R^e)^T W_r^{-2}(R - R^e) + (C - C^{(0)})^T W_c^{-2}(C - C^{(0)})$$

where

R = model response vector;
R^e = experimental data response vector;
C = model parameter vector;
$C^{(0)}$ = nominal estimate of parameters;
W_r = uncertainties in response data; and
W_c = uncertainties in nominal model parameter values.

Define X and Y as the independent and dependent parameter vectors, respectively; so

$$C = \left\{ \frac{X}{Y} \right\}$$

The constraint relation is given by Y = AX + B. One of the solutions that was obtained is:

$$X^{(i+1)} = X^{(i)} + (S_z^{(i)T} W_r^{-2} S_z^{(i)} + W_{cxx}^{-2} + A^T W_{cyy}^{-2} A)^{-1}$$

$$\times \; (S_z^{(i)T} W_r^{-2} (R^e - R^{(i)}) + W_{cxx}^{-2} (X^{(0)} - X^{(i)})$$

$$+ A^T W_{cyy}^{-2} (Y^{(0)} - Y^{(i)}))$$

where

$X^{(i)}$, $Y^{(i)}$ = solution from the i^{th} iteration; and

$S_z^{(i)} = S_x^{(i)} + S_y^{(i)} A$ where the sensitivity matrix $S^{(i)}$ is partitioned as $[S_x^{(i)} \mid S_y^{(i)}]$

Work dealing with developing the sensitivity matrix involves: (1) developing an interim computer program where the model parameters are at the level of terms such as AE and IE (area or sectional modulus times elastic modulus); (2) developing a computer program where the model parameters are at the level of such terms as plate thickness t or outside pipe diameter d_o; and (3) numerous studies involving low sensitivities and limits of perturbation theory.

The first item (the interim program) involves defining the sensitivity matrix as $\left[\frac{\partial \lambda}{\partial r}\right]$ where λ are the "responses" and r are the parameters. This matrix is developed using:

$$\left[\frac{\partial \lambda}{\partial r}\right] = \left[\frac{\partial \lambda}{\partial k}\right] \left[\frac{\partial k}{\partial r}\right]$$

Both of these matricies will be developed in closed form and be evaluated by the computer program. The matrix k is a combination of the mass and stiffness matrix, i.e., k = [M ¦ K]. The second item is similar but involves determining $\left[\frac{\partial \lambda}{\partial t}\right]$ where t are the parameters such as thickness and diameter:

$$\left[\frac{\partial \lambda}{\partial t}\right] = \left[\frac{\partial \lambda}{\partial k}\right] \left[\frac{\partial k}{\partial r}\right] \left[\frac{\partial r}{\partial t}\right]$$

The last major area of parameter identification involves improving ANCO's data acquisition and manipulation capability. There are two areas: (1) hardware, and (2) software. The hardware involves procuring such items as: (1) array processor, (2) anti-aliasing filters, and (3) CRT plotter. The area of software involves improving programs used to obtain and analyze the data, i.e., XSINE, XFAST, XPOST, XFILT, and XPLOT. Some new software will be developed, i.e., chirpy transform, digital filters, and transfer functions.

ANCO Engineers is at present involved in several jobs which require the use of parameter identification.

- Soil Dynamic Studies: This involves applying a time dependent load to a body of soil. Various responses will be measured. A model of the soil will be generated using the Cap model. A parameter identification (Bayesian) will be done using the approximately 30 soil parameters in the Cap model.

- Nonlinear Pipe Dynamics: A pipe system (part of a nuclear system) is being tested. Various modern restraint configurations will be installed (one at a time). For the different configurations nonlinear models will be constructed. Parameter identification will be done to help study the effect the modern restraints have on the system.

- Offshore Platform: The parameter identification done here is playing two roles. First, it is being used to verify the finite element model of the platform. Second, it will be used to help determine which members or groups of members will have been damaged by the environment in the future.

- Water Dam Studies: A dam will be tested and the results compared with model results to determine how well the added mass in the model compares with reality.

At ANCO Engineers parameter identification (and system identification) is a matter of high priority. We are committed to continual development in this area.

ON THE TORSIONAL ACCELEROGRAPH

by

Martin E. Batts*

To the writer's knowledge no torsional accelerographs have been developed so far. (The Wood-Anderson type seismograph measures primarily translational motions).

Torsional building response can be caused (a) as a result of an eccentricity of the center of mass with respect to the center of stiffness (Hoerner, J.B, 1971; Kan, C L., and Chopra, A.K., 1976; Batts, M.E., 1978), and (b) as a result of torsional ground motions. As shown in References (Batts, M.E., 1978; Luco, J.E., 1966; DiJulio, R.M., and Hart, G.C., 1974), the response generated by torsional ground motions can be quite significant, indeed, according to Luco, of the same order as the translational response (1966).

Surface torsional ground motion originates from horizontally polarized shear waves in the surface layer (Love waves). The rotation of a point about a vertical axis can be computed from the theory of elasticity as (Newmark, N.M., Rosenblueth, E., 1971).

$$\phi = \frac{1}{2}\left(\frac{\partial u}{\partial y} - \frac{\partial v}{\partial x}\right) \qquad (1)$$

Since $\ddot{u}$ and $\ddot{v}$ can be obtained for any given earthquake from measurements of ground translation accelerations, use of Equation 1 in conjunction with the assumption

$$u = \dot{u}\left(t - \frac{y}{c_s}\right) \qquad (2)$$

where c_s = shear wave speed at the free surface provides a means to calculate the rotational acceleration $\ddot{\phi}$. It can be shown that, in general

$$\ddot{\phi} = \frac{1}{2c_s}\left(\dddot{u}\ \sin(\theta) - \dddot{v}\ \cos(\theta)\right) \qquad (3)$$

where θ = angle between the shear wave propagation path and the x-axis. One problem associated with the use of Equation 3 lies in the determination of c_s.

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Newmark (1971) argues that, since the refraction of waves at rock-soil interfaces tends to make shear waves travel upward as they approach the ground surface, the shear waves at the free surface travel practically with the same speed as the shear waves in the underlying rock. However, viewing the surface lateral motion as Love waves (horizontally polarized shear waves) a more appropriate choice for c_s might be the Love wave velocity, which is much lower than the shear wave velocity of the rock. This would significantly increase the predicted ground torsional motion.

One approach for obtaining the torsional accelerogram indirectly from a translation accelerogram is by appealing to Equation 1. By a finite difference approach

$$\frac{\partial u(x,y,t)}{\partial y} \cong \frac{u(x,y+dy,t) - u(x,y,t)}{dy} \tag{4}$$

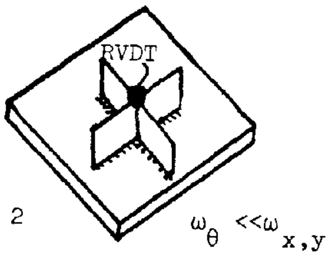
As shown in figure 1, if only two translational accelerographs were used, in the case in which the shear waves travel in the direction of the axis of the accelerometer response the ground motion would not be reflected in the measurements. Therefore, four accelerometers are required for this approach. Several factors must be taken into account. The first is accuracy. For a system with digital recording when the accelerometers were placed say one meter apart, the typical differences as expected from Equation 3, should be significantly larger that the resolution of the A/D converter. The second consideration is wavelength. The separation of the accelerometers should be significantly less that the smallest wavelength of interest.

Fig. 1

Another approach is to obtain the torsional accelerogram directly. This would require a torsional accelerograph that responds onlv to torsional motion, i.e., not an eccentric mass type. There are several manufacturers that produce angular accelerometers; but these are typically eccentric mass type. In addition, their range is 50 radians/sec^2 and their cross-axis sensitivity is 0.2 radians/sec^2 which are much too high for our purposes.

One novel approach (proposed by A. Schiff) is shown if Figure 2, a cruciform cantiflever. It is stiff in the two translation directions and flexible in rotation. An RVDT (a rotary variable differential transformer) could be placed at the intersection of the plates. Damping

Fig. 2



would probably have to be by fluid.

Another design is shown in Figure 3, a torus. A small arc of of the torus could form the core of an LVDT to measure the rotation. Damping could be provided electromagnetically. A watchspring, like in the common strong motion accelerometers, provides the stiffness. The use of an LVDT makes it easier to record the accelerogram digitally. It would be best to record this simultaneously with four translation accelerographs in order to compare the torsional accelerogram with that implied from Equation 1. The results could be recorded using a microcomputer with a 12 bit A/D converter and stored on magnetic bubble memory or cassette tape (possibly vibration sensitive).

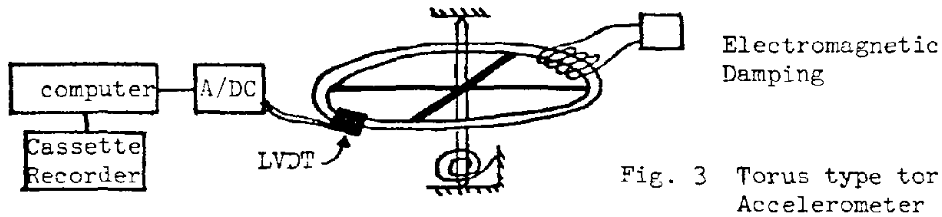


Fig. 3 Torus type torsional Accelerometer

Given the significance of torsional ground motions in earthquake engineering and the fact that no satisfactory device for measuring such motions currently exists, the development of a reliable torsional accelerograph is an important task.

IMPLICATIONS OF STRONG MOTION RECORDS ON THE ASSESSMENT OF SEISMIC HAZARDS IN EXISTING BUILDINGS

by

Craig D. Comartin*

INTRODUCTION

The overwhelming majority of existing buildings do not conform to the seismic design provisions of current building codes. A large number of existing buildings were built prior to the inclusion of seismic design provisions in building codes. The fact that a given building does not conform to current state-of-the-practice for seismic design does not, in itself, constitute a seismic hazard.

A rational assessment of seismic hazard in an existing building may be made by predicting the damage that would occur to the building when subjected to seismic ground shaking of varying intensity. For example, a procedure for assessing the damageability (potential for damage) in existing buildings has been developed by Blejwas and Bresler (1979). Local damageabilities for each element of a building are determined for a given intensity of seismic ground motion. These local damageabilities are then combined using weighting factors which reflect relative importance of building elements to form a global damageability index. This global damageability index may be plotted for a given building, or a category of buildings, versus a seismic demand parameter (see Figure 1). This parameter represents the intensity of ground shaking during earthquakes with different probabilities of occurrence.

Records of strong ground motions of building response, coupled with observations of damage which occurred in buildings subjected to real earthquakes, can be used to verify and refine the process of seismic hazard assessment in existing buildings. In addition, these records provide an opportunity to study, analytically and experimentally, the potential for damage during seismic events for various categories of existing buildings.

VERIFICATION AND UTILIZATION OF PROCEDURES FOR DAMAGE PREDICTION

The process in which strong motion records may be utilized to verify and refine procedures for predicting seismic damage in existing buildings is illustrated in Figure 2. During an earthquake, the existing building of interest is subjected to the actual ground motion. As a result of this motion, the structure responds and damage may or may not occur. The condition of the building after the earthquake is observed

* Wiss, Janney, Elstner & Associates, Emeryville, CA.

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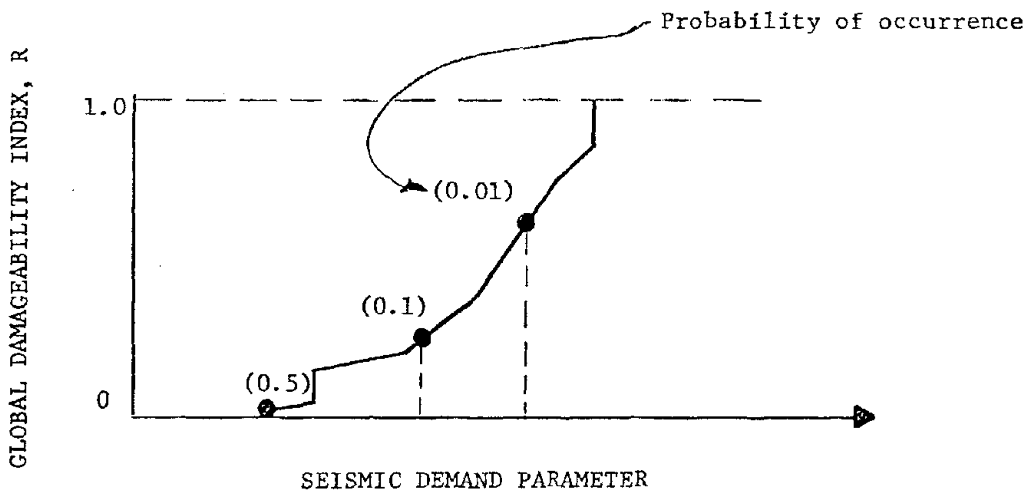

FIG. 1: Damageability for Earthquakes of Varying Probability of Occurrence

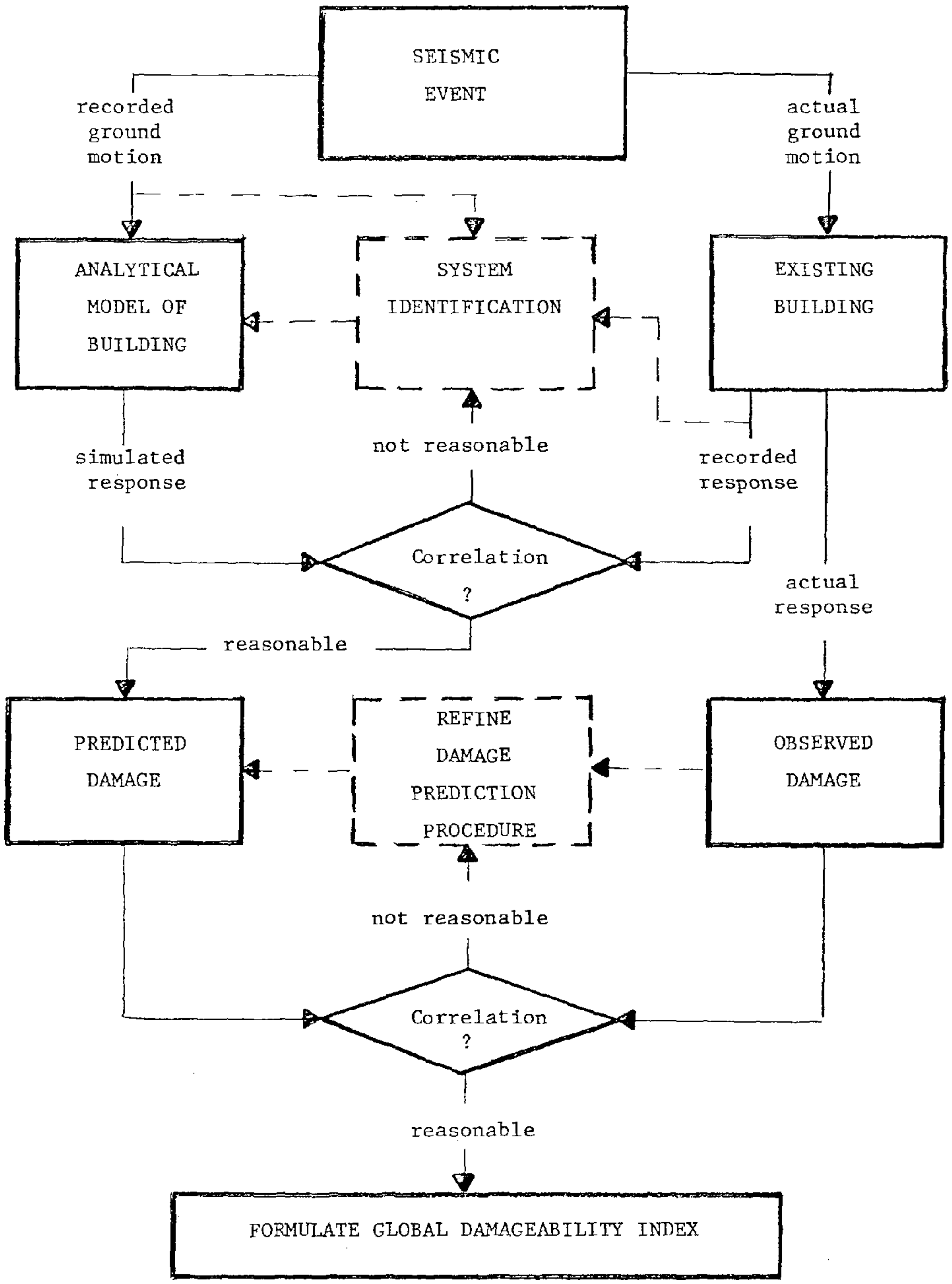

FIG. 2: Verification and Refinement of Procedures for Seismic Damage Prediction

and documented in the field. An analytical model of the existing building is developed and subjected to records of ground motion obtained near the site. If the response of the building was also recorded by instrument, a comparison between this record and the simulated response from analysis may be made. If the correlation between the two is not reasonable, system identification techniques might be used to revise the analytical model (Ting, Hong-Chen and Yao, 1978; Hart, Yao, 1977).

The simulated response obtained from the analysis is used to predict the damage which would occur in the model according to the procedure similar to that discussed previously. If the predicted damage does not correlate with the damage that was observed in the actual building in the field, these procedures may be revised.

A number of researchers have obtained remarkably good correlation between observed and predicted damage in buildings utilizing similar techniques (Mahin, Bertero, Chopra and Collins, 1976; Selna, Cho, 1973).

Once procedures for prediction of seismic damage have been verified for a particular category of buildings, analytical models may be subjected to earthquake records representative of various intensities. Curves similar to Fig. 1 may be thus generated. These curves may be utilized to assess seismic hazards in particular buildings.

FUTURE REQUIREMENTS

As more records of ground and building motions during actual earthquakes become available, procedures for seismic hazard assessment in existing buildings may be greatly improved. Efficient development of these methods requires the following:

(1) Programs for instrumentation of buildings which are typical of various types of construction must be continued and expanded. Categories of existing buildings which are suspected to be hazardous deserve particular scrutiny.

(2) Detailed investigations of damage to typical buildings following seismic events must be expanded and thoroughly documented. It is important to realize that the lack of damage to a particular type of building is as significant as major damage to others.

(3) The formulation of indices similar to global damageability (Blejwas, Bresler, 1979) must be developed and refined. These indices should be calibrated to identify various levels of life safety and expected economic loss.

SOME INSTRUMENTATION NEEDS

by

Douglas A. Foutch*

If a magnitude 7 earthquake occurred today in downtown Los Angeles, literally millions of data points would be generated. In spite of this wealth of information, several questions of interest to research and design engineers would go unanswered because of the type and placement of instruments currently in the field. Given below is a brief summary of some of these questions and a general description of a very expensive instrumentation system that might enable many of the questions to be answered. The unanswered questions and proposed solutions are not original nor exhaustive.

INFORMATION NEEDS FOR ASEISMIC DESIGN

The goal of the instrumentation system described in the next section of this paper is to provide answers to the following questions:

1. What characteristics define the gross nonlinear dynamic behavior of the building during structurally damaging earthquake motions?

2. Are the torsional motions in the building significant; and are they due to structural nonsymmetry or rotational components of ground motion?

3. Is there significant rocking of the building due to foundation compliance? Does uplift occur?

4. What effects do the local geology and site conditions have on the incoming seismic waves; and when are these effects significant for design?

5. What are the spatial and time correlations between motions at different points in the free-field?

6. How does the presence of the building modify the free-field motions?

7. How does the soil-structure interaction influence the building response?

8. How well do our analytical models predict the answers to all of the above?

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REQUIREMENTS FOR THE INVESTIGATION

The building chosen for study should be several stories tall, rectangular in plan and should have relatively simple structural systems. The foundation system should also be simple and, if possible, at grade level to reduce the analytical complexity of embedment. The site should have a relatively uniform geology with at least a few hundred feet of soil over bedrock. The site should obviously be located in a very seismically active area and, ideally, in or near an existing strong motion array.

Full-scale dynamic tests and a complete analytical study of the building and surrounding ground should be undertaken to determine the optimum location for the instrument placements in and around the building. These studies would also serve as a benchmark for observed earthquake motions and subsequent analytical and experimental studies.

At least three standard triaxial instruments should be placed on the ground floor at three of the four corners of the building. Some thought should also be given to placing instruments that measure rotations directly. Determining rotations from the difference in two translations may be a very ill-conditioned process if the rotations are small. The error in each displacement (or acceleration) measurement may be larger than the expected difference, rendering the results useless. The second floor of the building should be the next floor to be chosen for instrumentation. This data along with the base motion will provide the best information from a system identification point of view or from the point of view of inferring damage. The majority of the damage is likely to occur between the first two floors. By the time that information from this process reaches the top of the building, or even an intermediate floor, the effects are integrated over such an extent that useful inferences may not be possible. The third floor chosen for instrumentation should be the roof.

Free field motions would need to be recorded to answer many of the above questions. This would require that the building chosen be relatively isolated. Triaxial instruments at two to three depths and twelve to sixteen locations in plan would be ideal. For the correlation studies to be meaningful a common recording system for the entire array, or at least an extremely reliable time trace for each instrument, would be required.

CONCLUDING REMARKS

The challenge of finding a building and site that meets all of the above qualifications and, in addition, is highly accessible would be formidable. The most likely candidates would be government or university buildings.

The above instrumentation system might result in the simultaneous measurement of 90 - 125 components of motion. The total investigation described above would probably cost between one and two million dollars with an additional yearly maintenance fee for the instrumentation system. This may appear to be an excessive amount to spend on one project. However, if all of the above questions are answered, the return on the research investment in terms of savings in construction costs could be enormous.

COMMENTS

by

King-Sun Fu*

My main interest in earthquake engineering is from the viewpoint of data analysis. Pattern recognition deals with information extraction (description and interpretation) from available data for modeling and/or decision-making purposes (K.S. Fu, 1976; K.S. Fu, 1980). At present, I am interested in using limited data to determine the occurrence of a strong-motion earthquake. There are three major problems in pattern recognition:

(1) The pre-processing problem
(2) Feature extraction and pattern
(3) The classification and interpretation problem

All these three problems are inter-related. I believe that it is important to design the experiments so that we will know what to measure among other things. It can also be helpful to obtain time-sequencing information, spatial information, statistical information, and structural information. In the last item, linguistic or syntactic techniques (K.S. Fu, 1974) can be very useful. When the uncertainty involved in the data is primarily non-statistical, techniques based on the theory of fuzzy sets can be applied (L.A. Zadeh, K.S. Fu, 1975).

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SOME COMMENTS ON STRONG MOTION INSTRUMENTATION NEEDS

by

W. J. Hall* and T. F. Zahrah*

Our understanding of the nature of earthquake motions and their effect on structures has improved in recent years for many reasons, not the least of which is the fact that we now have an expanded data base arising from measurements during earthquakes, especially in the free-field. Even so, our data base arising from measurements in buildings is lacking and it is to this point that the following comments are directed.

In discussing deployment of instruments in buildings, it is not realistic to think of installing a large number of instruments. Obviously the cost in such cases would be great, and the cost and logistics associated with the maintenance, and perhaps even data reduction, would be prohibitive and not cost effective. Thus the goal should be to develop a carefully thought out plan and to work towards that goal.

Such a study was undertaken for arrays in the Proceedings edited by W.D. Iwan (1978) and this document should be reviewed when considering global instrumentation. One section, Chapter 4, deals with local arrays and in particular presents some ideas on laboratory arrays which could be employed to gain an understanding of spatial ground motions, with instruments on the surface and at depth. If such an array were located near instrumented structures the value of the data obtained would be greatly enhanced.

As noted in the work edited by W.D. Iwan (1978), a highly desirable (and almost necessary) feature of the array is to have time-synchronized motions especially if wave-passage and wave-form features are to be studied. A similar observation pertains to measurements in structures; the most valuable data will be obtained if the nearby free-field instruments, and the instruments in the building are synchronized with respect to time. Time-synchronized data would be even more valuable if we were able to develop relatively wide-band velocity instruments for installation in structures.

While isolated single instruments provide some information of value, a carefully structured array (even limited in number of instruments) is much more useful in terms of studying special topics such as soil-structure interaction and building response. Preliminary thinking suggests that a few selected buildings should be instrumented in areas where there is reasonable expectation of obtaining data. These buildings should be instrumented in some significant degree along the following lines:

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(a) a minimum 2 or 3 free-field instruments around the building, preferably at distances of 2D to 3D (where D is the greatest building plan dimension)

(b) a similar number in the basement on a common thick slab tied onto the walls with one at the center of the basement and 2 in diagonal corners (even attached to the walls if this appears more desirable)

(c) the same array repeated on the first, second and third floors as may appear desirable

(d) at elevations in the building,instruments should placed in a regular pattern (perhaps single instruments on selected floors)

(e) a three instrument array (center and diagonally at corners) near the top.

The goal is to help sort out translational and torsional components of motion and perhaps obtain information on tilting. The latter might be studied with some type of long-based special tilt meter and some type of special extensometers anchored at depth (preferably on all four sides of the building); these instruments would be used to measure peak values of displacement (perhaps on scratch surfaces) of the foundation of the structure at different locations in order to obtain an estimate of the relative motion of the structural foundation. If possible, such data should be supported by data from free-field instruments located at some reasonable depths near and below the foundation.

For damage (or gross strain) determination one suspects that passive instrumentation (for example scratch gages) may be highly useful; such instruments are cheap, easy to install, need little maintenance, and can be employed to provide a measure of the relative deformation or strain in structural elements. Also, in some cases, diagonal story deformation gages might be employed to advantage.

The selected buildings should be in areas where there is a high likelihood of excitation, be simple in structural form, and not be encumbered by problems associated with release of information after the earthquake. Obviously careful selection and study of the structures to be instrumented is required. Measurements of ambient vibration (wind induced) or shaker induced motions also would be helpful in arriving at interpretation of later data. If movable arrays were developed for rapid deployment, even aftershock data of the type noted could possibly be obtained and be of great value.

VIBRATION ANALYSIS STUDIES OF BUILDINGS INSTRUMENTED FOR STRONG-MOTION EARTHQUAKES

by

Charles A. Kircher*

INTRODUCTION

Strong-motion earthquake records obtained from instruments located in buildings have been collected for several events in the last two decades. While these records by themselves are important contributors to our knowledge of structural response behavior, consideration must be given to means of increasing the usefulness of these measurements particularly for applications which will lead to a discernible reduction in the risk to life and property due to earthquakes. One means for increasing the usefulness of records of strong-motion building response is the incorporation of these records with the results of separate vibration analysis studies (i.e., dynamic tests) of instrumented buildings. These vibration analysis studies would provide several benefits which would include the improved understanding of building dynamic properties (i.e., natural frequencies, mode shapes and damping). An immediate payoff from dynamic tests is the use of experimentally determined mode shapes to assist in the optimal placement of accelerometers for multi-channel accelerograph systems (e.g., as for California Strong-Motion Instrumentation Program-instrumented buildings). Further, long-term benefits from vibration analysis studies may provide the means by which post-earthquake damage can be assessed and structural reliability evaluated through pre-determined relationships of measured structure properties (i.e., from dynamic tests) and established building performance (i.e., from strong-motion records).

DAMAGE ASSESSMENT FROM VIBRATION ANALYSIS STUDIES

In essence, a procedure is described herein for assessing the degree of earthquake damage sustained in a building by comparing the results of dynamic tests made before and after an event. This procedure would utilize the results of the analysis of either forced or ambient vibrations, although the ease of performing ambient vibration analyses would make it more practical. The critical element in quantification of damage is the relationship between degree of damage (structural and non-structural) and measurable change observed in the dynamic properties. The records of buildings damaged during strong-motion events are of particular importance since the analysis of these records, in conjunction with other studies, would provide the necessary information to relate the change in building dynamic

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characteristics and degree of damage. Required groundwork for these studies would involve detailed pre-earthquake vibration analysis studies of those buildings which are currently instrumented with multi-channel strong-motion accelerograph systems. Following a strong-motion event, detailed post-earthquake vibration analysis studies of damaged buildings would be made and the results compared with the pre-earthquake measurements. In this manner, each recorded event would provide additional information on building response, damage and the relationship to measured changes in building dynamic characteristics. Hence, information obtained from studies of recorded strong-motion events in buildings and from pre-earthquake and post-earthquake vibration analysis studies would provide an accurate assessment of damage and a basis for quantifying structure reliability.

CONCLUSION

Strong-motion building instrumentation, particularly the more sophisticated multi-channel accelerograph systems, produce significant records of building response to earthquakes and contain information relating sustained damage to changes in the dynamic characteristics. Pre-earthquake and post-earthquake vibration analysis studies, in conjunction with the information contained in these strong-motion building records, show promise of providing a basis for accurately assessing damage and evaluating building reliability.

COMMENTS ON STRONG MOTION EARTHQUAKE DATA IN OR NEAR BUILDINGS

by

Frank Kozin*

Recent studies have led to the possibility of identifying the structural parameters of buildings, allowing for the formulation of relatively accurate dynamical models of the real structure. The structural parameter estimation techniques are based upon the relatively fundamental statistical procedures of maximum likelihood estimates as well as recursive estimation techniques.

The need for dynamical models is clearly motivated by the potential for controlling buildings in order to maintain their structural integrity, to enhance the comfort of the occupants especially in tall buildings and, finally, to allow for radically new building design philosophies and concepts as a result of the use of controllers.

A further need for accurate structural parameter estimates as well as dynamical models is the requirement to determine as accurately as possible changes in the dynamical characteristics of the building over its life span, and, more importantly, during or after a strong motion excitation. This is one important means by which information can be obtained to help assess the damage to the building as a result of strong motion excitations.

Finally, we may mention other needs for data in or near buildings in order to determine how the foundation of the building feeds back to, or changes the local geological properties. This may modify the strong motion excitations that are fed into the structure.

We now motivate to some extent the nature of the information that is required for the control problem.

If we view the structure as a linear system, then we can write

$$(1.1)\quad \dot{\underline{q}} = A\underline{q} + B\underline{u} + \underline{\omega},$$

where $\underline{q}$ denotes the "state vector" of displacements and velocities of the N-story structure, $\underline{\omega}$ represents the external excitations to the structure (earthquakes, wind loadings, etc.) and $\underline{u}$ represents the control forces that must be applied to the structure in order to achieve the desired dynamical behavior.

In particular

$$(1.2)\quad A_{2N\times 2N} = \begin{bmatrix} 0_N & I_N \\ -M^{-1}K & -M^{-1}C \end{bmatrix},$$

where M is the mass matrix, K is the stiffness matrix and C is the damping matrix; and B is $2N \times m$.

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It is known that under certain conditions one can control a linear system by pole assignment through state feedback.

The exact statement is as follows: If the system (1.1) is controllable, which is equivalent to requiring the 2Nx2Nm controllability matrix.

$$(1.3) \quad L = [B, AB, A^2B, \ldots, A^{2N-1}B]$$

be of rank 2N, then given any prescribed set of 2N eigenvalues (complex conjugates must always be included), a matrix F can be determined so that the state feedback control vector

$$u = F\underline{q},$$

gives us the system

CITY OF LOS ANGELES STRONG-MOTION

INSTRUMENTATION PROGRAM

by

John O. Robb*

INTRODUCTION

In 1965, the City of Los Angeles adopted an ordinance requiring building owners to install strong-motion accelerographs at three locations (i.e., basement, mid-height and roof) in buildings which exceed 6 stories. These instruments were installed to enable the building owner and the Department to ascertain the structural condition of the buildings after a large earthquake. At the present time (March, 1980) there are 170 code instrumented buildings in the City with approximately 25 new buildings under permit.

PRESENT STATUS

When the Los Angeles instrumentation system was first adopted the United States Geological Survey (USGS) installed the instruments and maintained them as part of the national network. Starting in the mid 70's the USGS started to drop the City instruments from their maintenance program. This was due to budget problems, the large number of instruments being installed and the limited national need for such a concentration of instrumented buildings.

The City of Los Angeles code program was not designed to have the building owner maintain their own instruments; therefore, when the USGS dropped their maintenance program the City Department of Building and Safety initiated an interim maintenance program to ensure a minimum level of operational instruments. It should be noted that in the event of a major earthquake, no agency is responsible for retrieving these records and unless the owner is interested in contracting the processing of the film, there will probably be considerable delay before the record would become available, if at all.

The State of California adopted a strong-motion instrumentation program in 1971. This program is providing a network of free field instruments which will yield records of ground motion unaffected by the mass of large buildings. The City's required instrumentation in basements, therefore, becomes redundant and serves minimal purpose. In addition, experience has shown that records obtained from instruments located at mid-height primarily yield information regarding the higher modes of vibration. For scientific studies of response of buildings the set of three triaxial accelerographs is not the ideal instrumentation.

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At the present time the City is exempt from participating in the State of California program because of the existence of a local program prior to January 20, 1972.

PROPOSED CHANGE

It is proposed to reduce the number of required accelerographs in a building from three to only one. The single triaxial instrument would be located near the top floor. The reduction would amount to a substantial savings to building owners, not only in the change from three to one instrument but also in the elimination of the presently required interconnecting electrical circuitry and the acquisition of additional floor space required by the basement and mid-height instruments. The proposal would permit the owner to either provide for an approved agency to maintain the instrument or to have the City provide the service for a fee. The proposal would also permit buildings with existing accelerographs to modify their system to the single instrument. Under this proposal the City would join the State of California Strong-Motion Instrumentation Program.

It would be expected that the State of California Strong-Motion Instrumentation Program would agree to collect and develop the film from the single instrument of the City program buildings in the event of a major earthquake and provide the owner with a copy, thus ensuring a timely evaluation of the structure.

CONCLUSION

Although the proposal lessens the existing requirements for strong-motion instrumentation, the proposal will still result in obtaining adequate records to ascertain the structural condition of high-rise buildings after a major earthquake. This would be in agreement with the intent of the original code requiring instrumentation of buildings in Los Angeles.

SOME IDEAS ON THE INSTRUMENTATION PROGRAM

by

Jose M. Roesset*

Earthquake instrumentation programs have two distinct objectives:

1. To obtain information on the characteristics of ground motions in the epicentral region and at various distances, which will allow us to understand and model better the seismic input to structures.

2. To measure the response of actual structures to earthquakes and to correlate these records with observed damage and results of analyses in order to gain a better understanding of structural behavior (particularly in the nonlinear range).

With respect to the first point the instrumentation programs already existing and described in this workshop have already furnished, during the San Fernando and the El Centro earthquakes, a considerable amount of valuable information. One of the main questions which still limits our ability to conduct soil structure interaction analyses is the lack of knowledge on the wave content of a potential earthquake. The display of instrumental arrays on a horizontal plane, which has been recently started, would allow us to estimate the apparent velocity of propagation of the waves or an effective angle of incidence, and will help to answer this question. It appears that some information of this nature has already been obtained in a study at Rensselaer Polytechnic Institute and additional data related to the recent El Centro earthquake are now being processed. Thus, while more data are necessary to improve the present state of knowledge, it would appear that the number of instruments available at this time and their placement are quite satisfactory and that there is no urgent need for expansion of this part of the instrumentation program.

The second point of concern is the measurement and the response of actual buildings to seismic excitation. During the San Fernando earthquake a large number of records were obtained. A dozen or so buildings were extensively studied making use of these records. There were maybe 40 additional buildings which were instrumented but which were not studied. This points out a major problem of having an excessive amount of information. It is quite likely, in addition, that only a few of the many buildings instrumented will yield interesting subjects of study. From the point of view of understanding the behavior of buildings under earthquakes and of improving present design methodologies it would appear that controlled experiments with large scale models either under cyclic loading or in shaking tables would produce faster and more reliable results. On the other hand the ultimate test of the applicability of analyses and design techniques must come from the observation

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of damage experienced by actual structures. It is important to maintain therefore a building instrumentation program and perhaps to look at it from a different perspective.

Attempting to instrument all important buildings, or buildings over a certain height, with a sufficient number of instruments to be able to explain their response in detail would lead to tremendous maintenance problems. A better solution might be to place only a minimum amount of instruments (preferably simple instruments) in all buildings which are now being instrumented, and to concentrate the program on a few selected structures which would serve as living laboratories. It would be particularly convenient if these buildings could be selected before construction starts to make sure that all needed information on the soil properties is available or to conduct additional tests. The analysis and design of the buildings should be documented and kept available for future reference. Preliminary dynamic analyses should be conducted to select the most appropriate placement of instruments so as to obtain not only horizontal accelerations at various floor levels, but also measures of the foundation motions (preferably six components of motion), torsional motions, interstory displacements or shears etc. . . Tests of the foundation and ambient vibration tests of the structure at various phases of the construction process could be carried out and the structures could be used by researchers for their studies when comparing methods of linear or nonlinear analyses system identification techniques etc. . . (a large number of studies of this kind are performed each year on fictitious and often unrealistic structures). For this type of program to be successful the number of buildings considered has to be very limited; in addition a university, hopefully located near the site of the building or buildings should take up the responsibility for coordinating the studies, supervising the placement and maintenance of the instruments and distributing all the resulting information.

A COMPARATIVE STUDY OF THE RESPONSE SPECTRA FOR

THE THREE COMPONENTS OF STRONG MOTION RECORDS

by

K. R. Sadigh*

The purpose of this study was to examine the relationship among the response spectra for the three components of motion for recordings obtained on soil sites. The relationships presented are for moderately strong earthquakes in the Western United States with particular emphasis on the 1971 San Fernando earthquake. The relationships are expressed in terms of the peak accelerations, a , and the response spectra, S , between the two horizontal component (V) and the average of the horizontal components (H).

The ratio of S(H1)/S(H2) was found to be roughly porportional to the ratio a(H1)/a(H2) up to periods less than about 1 sec. For longer periods, the ratio S(H1)/S(H2) was found to decrease to a value of about 1 or smaller. The ratios a(H1)/a(H2) and S(H1)/S(H2) were found to be weakly dependent on the distance.

The ratio S(V)/S(H) was found to be very strongly dependent on the period and the distance. At a period of about 0.1 sec., this ratio was found to vary from about seven-tenth at moderate distances (about 40 km) to about one at clost distances (about 10 km). At longer periods (between 0.3 and 10 sec.) this ratio was found to vary from about one-fourth to two-thirds.

* Woodward-Clyde Consultants, San Francisco, CA

COMMENTS ON STRONG-MOTION INSTRUMENTATION

by

Anshel J. Schiff*

The comments given below are a collection of items which relate to various aspects of the collection, analysis, and interpretation of strong-motion earthquake data.

PRE-EARTHQUAKE TESTING

To maximize the information gleaned from future strong-motion structural records, knowledge of the pre-earthquake response is necessary. Thus, there is a need to establish the practice of testing structures in which strong-motion instrumentation is to be installed. The results of such tests should be used to find preferred positions for strong-motion instrumentation within the structure. Because of the economics and the problems of securing permission to conduct forced testing, ambient data may be required. After strong-motion instrumentation has been installed, ambient data should be obtained to get a pre-earthquake signature for the instrument locations. With the collection of post-earthquake data, a total of three sets of data (pre-earthquake, earthquake, and post-earthquake) for the particular transducer locations would be available.

STRONG-MOTION INSTRUMENTATION DOCUMENTATION

Structures with strong-motion instrumentation should have the following information available to researchers:

(1) A structural plan should be available along with a written description of the structural system which provides lateral load resistance. The description of the structural system should be detailed enough to allow finite element modeling. Transducer locations should be indicated.

(2) The rationale used to locate the strong-motion instrumentation within the structure should be on file along with the data analysis plan. Owing to the long delays between installation and data recovery, the instrumentation and data analysis plan may not be recalled.

(3) Test data for the structure should be well-documented. This would include pre-earthquake data and results from independent testing and analysis. If possible, raw data from tests should be available so that it could be re-evaluated as improved methods of data analysis become available.

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AVAILABILITY OF SYSTEM IDENTIFICATION PROGRAMS

The number of methods of system identification using test or strong-motion data continues to increase. While the methodologies are published, implementation is a major effort often requiring special procedures and tricks to get the method to work. Also, as often as not, the method is evaluated using computer simulation data or laboratory models. As a result, various methods are not meaningfully compared or even evaluated and, from a practical point of view, the state-of-the-art is not advancing. It is unreasonable to expect graduate students to provide documentation for general use. Even if "standard" FORTRAN is used for coding, the transportability between computer systems, even with the same main-frames, is usually limited. This problem needs to be addressed.

FINDING DISSIPATION FROM RESPONSE DATA

In the analysis of data, more emphasis should be given to identifying where in a structure energy is being dissipated. Dissipation is one of the vital parameters in the serviceability of a structure. The identification of features in actual designs which account for the most dissipation would allow such features to be designed into the system.

OTHER TYPES OF DATA

It is suggested that torsional accelerometers be developed and used. The present method of deploying two accelerometers at opposite ends of a structure adds expense to the installation and, for many structures, the rigidity of the floor as a diaphragm can be brought into question. It is also suggested that strain be used as a response variable. In upper floors the response will primarily be in first or first and second mode. With appropriate instrumentation a lower sample rate could be used and thus more channels of data could be digitally recorded and stored.

SOME THOUGHTS ON FINDING OPTIMAL SENSOR LOCATIONS IN STRUCTURES

by

F.E. Udwadia

In this brief communication, I would like to address myself to just one aspect of the problem of instrumenting building structures, to obtain records of structural motion during strong ground shaking. The question addressed will be, "Given M instruments, where should they be distributed within a structure?"

The answer to this question has, in the past, been primarily motivated by two thoughts: (a) the scarcity of building records and (b) the reliability, or rather the lack thereof, of proper instrument operation. The location of sensors were, it seems, selected so that good quality large amplitude (high signal to noise ratio) records could be obtained.

Though the abovementioned criteria may well be still applicable today, with improvements in instrument design and the need for improved prediction of structural response during strong ground shaking, perhaps an additional criterion may be considered. Thus, one may want, within the signal to noise ratios prescribed by the instrument and data processing error limitations, to locate sensors so that the "best" structural models could be arrived at, from the measured time histories.

The optimal sensor location problem may be thought of in terms of: (1) the class of models, M, to which the physical system is assumed to belong; (2) the class of functionals (parameters), F, to be identified to improve the present knowledge of the structural model; and (3) the class of inputs to be used. Also, system identification schemes, using the time histories of response normally depend on an error criterion, E. This is often a norm of the difference between the model prediction and the response measurement. These aspects are often interrelated to each other [Udwadia and Shah; 1976].

The iterative nature of most identification schemes require us to differentiate between two distinct, and often times confusing, criteria for the sensor location problem.

(a) Uniqueness of Identification

The idea here is to locate sensors in such a manner that no matter what initial guess (of the parameters or functionals) one starts off with in the iterative scheme, the identification will converge to the unique "actual" system. Alternately put, one wants to locate sensors at locations which yield information that can unequivocally tell us of

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the actual system. Using records obtained from such locations, different initial guesses would not yield different estimates of the parameters (or functionals) and these estimates would not depend on the proximity (measured by a suitable norm) of the initially guessed parameters (or functionals) to those of the actual system.

Such nonuniqueness problems from a practical standpoint would indeed lead to not only incorrect identification, irrespective of the actual estimator used, but also to erroneous base shear values (Udwadia, Sharma and Shah; 1978).

(b) Local Optimization

If however, one has a fairly good idea of the parameter values, the initial guesses would be fairly close to the actual values. Thus, one would not be likely to converge to a solution other than that represented by the actual system. Having restricted the search space through the use of a good initial guess, one needs to locate sensors in such a manner that having started with approximate (and good) estimates, the records obtained have the greatest information to improve the estimates. As opposed to the "global" convergence (starting from any initial guess), this method settles for a "local" convergence (starting from a "close" initial guess) criterion (Udwadia and Shah; 1978 and Udwadia and Tabaie; 1980).

CONCLUSIONS

Two different criteria, both related to structural identification, have been proposed for finding optimal sensor locations in building structures. The use of either depends on the amount of apriori information available regarding the system. The optimal sensor locations may depend not only on which parameters (functionals) are to be identified from the records, but also on the actual parameter values involved, the nature of the input and the structure of the model. Finally, a combination of the two criteria is also possible.

COMMENTS ON STRONG-MOTION RECORDS AND STRUCTURAL IDENTIFICATION

by

James T. P. Yao*

It is encouraging to note that more strong-motion earthquake records have been collected during recent years. Valuable information has been obtained from the analysis and interpretation of these records. While many important activities concerning the current planning and immediate application of strong-motion instrumentation programs are discussed during this Workshop, the following comments are presented to explore the possible application of pattern recognition and fuzzy sets (see respective Comments as presented by K.S. Fu and L.A. Zadeh in these Proceedings). Although the discussion is mainly concerned with a specific application, it is hoped that these comments will serve the purpose of generating further interest among earthquake engineers to study and apply these relatively new approaches.

The application of system identification techniques in structural dynamics was reviewed by Hart and Yao (1977) among others. They also suggested that a broad sense of system identification in structural engineering should include the estimation of damage along with the equations of motion for the given structure. The concept of structural identification was further discussed by Liu and Yao (1978). Recently, several possible approaches making use of full-scale dynamic test data for the assessment of structural damage were reviewed and discussed by Yao (1979).

Because existing structures are usually complex systems, it is difficult to define the damage state of various structures in a manner which is both clear and meaningful. Consequently, a possible approach to solving this problem is the application of the theory of fuzzy sets (see Comments by L.A. Zadeh).

To explore such an approach and to introduce the elementary theory of fuzzy sets, a paper has been prepared and is now in the process of being published by Yao (1980). It is obvious that much work remains to be done before such an approach becomes practical in the civil engineering profession. Nevertheless, there exists a great potential for such applications.

In the theory of pattern recognition (see Comments by K.S. Fu), (a) data are collected, (b) a feature space is extracted from these data, and (c) a decision function is then applied to obtain the appropriate classification. It is believed that such a methodology will be very useful in the interpretation of strong-motion earthquake records in and/or near buildings.

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