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CORRELATIONS BETWEEN EARTHQUAKE DAMAGE AND STRONG GROUND MOTION

by

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ABSTRACT

The primary data for this study come from 40-odd buildings in Los Angeles from which both information as to damage ratio and strong motion records were obtained during the San Fernando earthquake in 1971. A few additional buildings, in Managua and near Los Angeles, are added: for these additional buildings, all of which were heavily damaged during earthquakes, the general level of earthquake shaking can be inferred with reasonable confidence. Damage ratio is correlated with spectral displacement, spectral velocity, spectral acceleration (these spectral quantities were averaged over periods from 10% less than the pre-earthquake fundamental period to 10% greater than the during-earthquake period) and to calculated interstory displacement. The most useful correlations related damage ratio to spectral velocity and spectral accelerations.

PREFACE

This is the 19th in a series of reports under the general title of Seismic Design Decision Analysis. The overall aim of the research is to develop data and procedures for balancing the increased cost of more resistant construction against the risk of losses during possible future earthquakes. The research has been sponsored by the Earthquake Engineering Program of NSF-RANN under Grant GI-27955X3. A list of previous reports follows this preface.

This report is identical with the thesis submitted by Mr. Wong in partial fulfillment of requirements for the degree of Master of Science. He served as research assistant during the work on this report. Dr. Whitman is Professor of Civil Engineering and is principal investigator for the overall study. Ms. Betsy Schumacker, Lecturer in Civil Engineering, assisted with the computer system for processing the data concerning ground motion and damage. Some clarifying comments by the principal investigator are appended at the very end of the report.

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16. Whitman,R.V., M.Yegian, J.Christian, and Tezian, "Ground Motion Amplification Studies - Bursa, Turkey,"

17. de Neufville, Richard, " Sensitivity Study using Utility Functions." Being written.

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Chapter 1. INTRODUCTION

1.1 BACKGROUND

Two principles are widely accepted by engineers as a basis for the seismic design of buildings. A building should be designed so as (A) not to collapse during the largest earthquake that is realistically imaginable, and (B) not to cause significant damage, that is economically unacceptable to an owner or to a community, from earthquakes which can be expected to take place during the lifetime of the building. These principles clearly imply a balancing of the risk of future damage against the added initial construction cost of providing a stronger building[36]. A precise implementation of these principles to the seismic design of buildings is an extremely difficult task with the present state of the art in Earthquake Engineering.

The Massachusetts Institute of Technology initiated a major research effort ("Seismic Design Decision Analysis") in the field of Earthquake Engineering in 1971. One of the major aims of this project is to develop a more explicit procedure to determine the optimum trade-off between higher initial construction cost and the long-term savings for designing for higher seismic loads against future earthquakes. The success of this research effort will, potentially, have a broad spectrum of applications. The development of this procedure should facilitate greatly in the implementation of the two principles mentioned above, in a more explicit manner.

This report is intended to aid the development of the procedure of balancing the risk of future damage against the added initial construction cost of providing a stronger building to resist future earthquakes.

1.2 SCOPE

The overall objective of this study is to compile statistics concerning structural and non-structural damage experienced by certain high-rise buildings in earthquakes, and to correlate these damage statistics to parameters derived from strong motion earthquake time histories. This report attempts to utilize these quantitative parameters to characterize the effects of earthquake ground motions on high-rise buildings rather than to use the conventional Modified Mercalli Intensity. The quantitative measures of intensity are useful for predicting damage and to distinguish different levels of ground shaking within the same level of Modified Mercalli Intensity.

For the purpose of this study, a high-rise building is defined in the following manner. Its height is five stories or greater, and it has been designed according to the seismic provisions of the Uniform Building Code. The structural system may be a steel moment-resisting frame, a steel braced frame, a concrete moment-resisting frame, a concrete shear wall, or a combination of the above systems. These requirements are necessary to insure all the buildings studied in this report have, in general, the same range of resistance to earthquake damage.

In the development of a method to correlate earthquake damage and strong ground motions, it was desired that the method could be applied to a building without a detailed knowledge of exactly how the building is constructed. Due to this limitation, it is necessary to limit the parameters which might be used as input to a method for the determination of earthquake damage.

Chapter 2. NATURE OF THE PROBLEM[11]

2.1 GENERAL COMMENTS REGARDING EARTHQUAKE MOTIONS AND THEIR EFFECT ON BUILDINGS

The detail and amplitude of any earthquake ground motions at a given site are related in a very complex manner to the nature of the faulting of the earthquake and that site. The complex interaction of the stress waves generated by the faulting and the geology of the region will undoubtedly result in a ground motion with varying amplitude, frequency and direction during the length of the shaking. The same reason also makes it impossible to predict with any great certainty the details of the ground motions, which might be observed at a given site during any given earthquake.

The occurrence of an earthquake will cause the structures in the vicinity of the causative fault to experience random dynamic forces felt, to some extent, throughout the whole structural system. These dynamic forces are directly related to the random cyclical distortions from earthquake motions at the base of the structure. It is these distortions which cause additional axial loads, bending moments, and shear forces other than those present under normal service loading conditions. The successful design of a building for earthquake effects depends on the engineer's understanding of the way the structure responds to dynamic loadings. The wrong approximations made in determining the dynamic characteristics of the building may cause individual members to behave very differently than the design engineer originally envisioned. Also, the available resistance of individual members at the time of the earthquake is subject to some uncertainties, i.e. the uncertain quantities of dead and live loads and possible local weakness due to the understrength of materials.

These uncertainties inherent in the structural system and the uncertainties in the dynamic forcing functions are just a few reasons why many people choose to treat earthquake problems in the context of probability. Although no probabilistic methods are used in the derivations of the materials presented in subsequent chapters, the methods developed are intended to be an approximate technique for the determination of damage values. Therefore, any results obtained through the utilization of these methods can better be described as mean values rather than deterministic quantities.

2.2 EARTHQUAKE DAMAGE CONSIDERATIONS IN THE DESIGN OF BUILDINGS

The general philosophy adopted by the Uniform Building Code, which in general governs the seismic design of buildings in the United States, does have some implications on the amount of damage a building might suffer during an earthquake.

It contains no explicit regulation concerning permissible values of damage to buildings during an earthquake, or how these values of damage can be computed. It is obviously not surprising to find no computation of damage to buildings from earthquakes in the seismic design of buildings. Certainly, steps are taken by many engineers and architects to try to limit the expected damage to the building system. Careful attention to the overall shape and proportions of buildings, their interior layouts, their orientation, and their architectural, structural, and construction details during design will certainly minimize the overall damage during an earthquake.

However, even when these techniques are utilized and carefully implemented into the building system, the design engineer will have only a very vague idea of the amount of damage the building might suffer during an earthquake. If one wishes to find the optimum trade-off between increased initial construction cost and the decreased earthquake damage, one would need a more accurate estimate of earthquake damage than what is obtained from the use of current building codes and present design procedures.

2.3 EXISTING SUBJECTIVE EARTHQUAKE INTENSITY SCALES

Since 1883, there have been many efforts to try to establish and define an earthquake intensity scale for the estimation of damage to buildings. Most of the scales proposed are descriptive and qualitative.

The first earthquake intensity scale was the Rossi-Forel Scale (1883). The Modified Mercalli Intensity scale shown in Figure 1.1 was initially propsed by Mercalli (1902) and was later modified by Wood and Neumann (1931). There are several Japanese scales and the recent MSK Scale (1964), developed by Medvedev, Sponheuer, and Karnik. All these intensity scales are subjective; they are based on the observed effects of earthquakes on human beings, on animals, on certain objects, on the ground and the landscape, and especially on buildings rather than on instrumental records. However, in many cases the assignment of an intensity rating to an earthquake took advantage of the information contained in instrumental records when these happened to exist.

A point worth mentioning is that subjective earthquake intensity scales tend to give too much emphasis to damage observed on buildings that have little or no seismic resistant properties. Some scales, such as the MSK scale, even exclude buildings designed to be earthquake resistant in the rating of damage for an earthquake.

Unfortunately, the damage description given in the Modified Mercalli Scale or any similar scales are far too general for use in any quantitative study. Therefore, several attempts have been made by many researchers to correlate

I Not felt except by a very few under especially favorable circumstances.

II Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.

III Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.

IV During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.

V Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles and other tall objects sometimes noticed. Pendulum clocks may stop.

VI Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.

VII Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.

VIII Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.

IX Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

X Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations, ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.

FIGURE 1.1 Abridged Modified Mercalli Intensity Scale these empirical intensity measures to the observed ground motions. One correlation suggested by Neumann is,

$$\text{Modified Mercalli Intensity, } I = \frac{\text{Log } 14V}{\text{Log } 2}$$

where v is the ground velocity in cm/sec. Any relationships, such as this one, do have some interesting implications. The use of the above relationship along with the Modified Mercalli Intensity scale can give a gross estimate of the amount of damage to buildings associated with a particular level of ground motion.

While there are needs for a more scientific approach to the definition of an earthquake intensity scale, subjective earthquake intensity scales will continue to be used for a long time. The reason is that in many high seismicity areas, there is a shortage or a complete lack of instrumentation for the recording of strong ground motions. Earthquake intensity scales are useful for the comparison of future earthquake damage to buildings to damage for past earthquakes.

Chapter 3. DATA COLLECTION

3.1 INTRODUCTION

Predicting the extent of building damage caused by strong earthquake ground motion is important in designing seismic resistant structures. In general, very little investigation has been done concerning the damage susceptibility of high-rise buildings subject to earthquake ground motions in a quantitative manner. The recognition that it is very difficult, if not impossible, to develop purely theoretical procedures to predict damage for high-rise buildings subject to strong ground motions make it worthwhile to conduct experimental investigations to determine the motion-damage relationship empirically. To correlate strong earthquake ground motion with building damage on a statistical basis, three types of information are necessary: dynamic structural response, building damage, and earthquake ground motion data.

3.2 THE SAN FERNANDO EARTHQUAKE OF FEBRUARY 9, 1971

3.2.1 GENERAL

The San Fernando, California, earthquake struck the northern part of the Los Angeles Basin at 6:01 a.m. on February 9, 1971. A 400 square mile area, which contained a population of over 1,200,000 was subjected to the strong ground shaking. The earthquake's epicenter was located in the San Gabriel Mountains, its strong motion lasted for about 12 seconds, and reached a magnitude of 6.4 on the Richter Scale. The earthquake casued 58 deaths primarily as a consequence of the collapse of the nonearthquake resistive Veterans Hospital and property damage estimated at 500 million dollars(5).

Due to the large number of structures and facilities which were subjected to strong ground tremors during the earthquake and the excellent seismic instrumental coverage of the area, the San Fernando earthquake provided a unique opportunity to conduct experimental evaluations in many aspects of Earthquake Engineering.

3.2.2 DYNAMIC STRUCTURAL RESPONSE DATA

In 1966 the Seismological Field Survey conducted a study to measure the fundamental periods of 43 newer high-rise buildings in the Los Angeles and the San Fernando Valley area. Immediately following the San Fernando earthquake of February 9, 1971, the same effort was repeated on these same buildings and also on 27 additional structures[22]. The fundamental periods presented in Table 3.1 were selected from this group of buildings that underwent low-amplitude wind or man-induced motion before and after the earthquake. The fundamental periods recorded during the relatively large-amplitude motions of the San Fernando earthquake are also presented in the same table. With few exceptions, all the buildings included in Table 3.1 are of modern design, constructed since 1960, are taller than six stories, and have strong-motion accelerographs installed in the basement and various other levels. This selected group of high-rise buildings can be considered to be well-designed against earthquake ground motions.

In view of the fact that the majority of the high-rise buildings listed in Table 3.1 were subjected to only moderate earthquake ground shaking during the San Fernando earthquake, a larger sample size should be obtained to include more

TABLE 3.1 - LOS ANGELES AREA BUILDING PERIODS--BEFORE, DURING, AND AFTER THE SAN FERNANDO EARTHQUAKE OF FEBRUARY 9, 1971.

No.	Address & Building Name	Year Built	Stories up/dwn.	Struct. Type	Miles to Epic.	Instr. Direct.	Bldg. periods, sec.		
							pre	during	post
1	9841 Airport Blvd. Tishman Airport Center	1967	14/1	RC(MR)	30	N--S E--W	1.13 1.32	1.6 1.8	1.21 1.43
2	1900 Ave. of the Stars	1968	27/4	ST(MR)	24	N44E S46E	3.30 3.30	-- --	3.60 3.60
3	1901 Ave. of the Stars	1966	20/4	ST(Br)	24	N46W S44W	2.63 2.45	3.5 3.6	2.80 2.72
4	633 E. Broadway, Glend. Glendale Muni. Service	--	4	ST	20	S70E S20W	0.60 0.64	-- --	0.79 0.80
5	Calif. Inst. of Tech. Millikan Library	1965	9	RC	23	E--W N--S	0.68 0.54	1.0 0.6	0.79 0.54
6	1800 Century Park E. Northrop Building	1970	16/3	RC(MR)	24	S36E N54E	-- --	1.1 1.6	0.95 1.30
7	1888 Century Park E. Shareholders Bldg.	1970	20/1	ST(MR)	24	S36E N54E	-- --	4.1 4.2	3.00 3.30
8	2080 Century Park E. Century City Med. Plaza	1969	17	RC	24	N50E N40W	-- --	2.1 3.1	1.58 2.38
9	4000 Chapman, Orange The City Center	--	19	RC	52	E--W N--S	-- --	1.14 2.04	1.05 1.94
10	222 Figueroa, L. A. Bunker Hill W. Tower	1967	17	RC	26	N53W S37W	-- --	1.1 0.6	0.91 0.54

TABLE 3.1 - continue

11	234 Figueroa, L. A. Bunker Hill S. Tower	1967	17	RC	26	S53E N37E	-- --	0.7 1.2	0.57 0.89
12	445 Figueroa, L. A. Union Bank Square	1965	39/4	ST(MR)	26	N52W S38W	2.84 3.29	-- --	4.10 3.70
13	250 E. First St., L. A. Kajima Building	1966	15/1	ST(MR)	26	N36E N54W	1.32 1.88	2.9 2.8	2.10 2.15
14	800 W. First St., L. A. Bunker Hill Cent. Tower	1967	32/0	ST(MR)	26	N53W N37E	-- --	3.3 3.4	2.58 2.62
15	533 S. Fremont, L. A. Coldwell Banker	1966	9/1	RC(MR)	26	N30W S60W	0.63 0.36	1.1 0.7	0.88 0.49
16	1025 N. Highland, L. A. Hollywood Storage	1926	14	RC	22	N--S E--W	1.20 0.49	-- --	-- --
17	7080 Hollywood Blvd. Muir Medical	1966	11/1	RC	21	N--S E--W	0.90 1.03	1.4 1.6	1.02 1.14
18	111 N. Hope St., L. A. L. A. Water & Power	1964	15	ST(MR)	26	N50W S40W	2.20 --	3.2 3.2	2.51 2.25
19	3838 Lankershim, L. A. Sheraton Universal	1967	19/1	RC(MR)	19	N--S E--W	1.22 1.26	2.3 2.2	1.40 1.50
20	1640 Marengo, L. A. Holliday Inn	1965	7/0	RC(FP)	26	N38W S52W	0.53 0.49	1.0 1.2	0.64 0.63
21	616 S. Normandie, L. A. Wilshire Christ. Manor	1969	16/1	RC(MR)	24	N--S E--W	-- --	1.1 1.8	0.85 1.17

TABLE 3.1 - continue

22	4800 Oak Grove Dr. Central Engineer., JPL	1962	9/0	ST	18	S08W S82E	0.99 0.93	1.4 1.3	1.16 1.01
23	1760 N. Orchid, L. A. Holliday Inn	1968	22	RC	21	N--S E--W	1.41 1.31	2.0 2.0	1.58 1.50
24	8244 Orion, Van Nuys Holliday Inn	1966	7/0	RC(FP)	13	N--S E--W	0.48 0.53	1.60 1.24	0.68 0.72
25	120 N. Robertson, L. A. Robertson Plaza	1965	9/2	RC(SW)	22	S02W S88E	0.63 0.46	0.8 0.6	0.70 0.49
26	611 W. Sixth St., L. A. Crockers Bank Plaza	1966	42/5	ST(MR)	26	N52W N38E	4.50 4.70	5.6 6.0	5.30 5.30
27	3407 W. Sixth St., L. A. Mutual Building	1965	9/0	ST(MR)	24	N--S E--W	1.03 1.05	1.6 1.4	1.20 1.24
28	6464 Sunset Blvd., L. A.	1968	11	ST	21	N--S E--W	-- --	2.3 2.7	1.50 1.75
29	3470 University Ave. Phillips Hall, USC	1966	11/1	RC(SW)	26	S71E N19E	0.66 0.71	0.9 1.0	0.86 0.86
30	15107 Vanowen St., L. A. Valley Presb. Hospital	1970	7	RC	15	N--S E--W	-- --	1.0 0.9	0.72 0.60
31	14724 Ventura, L. A. Certified Life Tower	1966	14/0	RC	17	N78W S12W	0.88 0.81	1.2 1.1	0.96 0.90
32	15250 Ventura, L. A. Bank of California	1970	12/0	RC(MR)	17	N11W N79W	-- --	2.2 3.0	1.70 1.60

TABLE 3.1 - continue

33	15433 Ventura, L. A. Howard Johnson Lodge	1966	13	RC	17	N12E N78W	0.62 0.71	1.4 1.3	0.77 0.91
34	15910 Ventura, L. A. Ventura Gloria Bldg.	1971	15/1	ST(MR)	17	S09W S81E	-- --	3.2 2.8	2.37 2.27
35	2500 Wilshire Blvd. Wilshire Coronado	1970	13/1	RC	25	N29E N61W	-- --	2.3 1.9	1.64 1.48
36	3345 Wilshire Blvd. Wilshire Square	1968	12/3	RC(T)	24	N--S E--W	0.84 0.68	1.2 1.0	0.99 0.80
37	3550 Wilshire Blvd. Tishman Plaza	1969	21	ST	24	N--S E--W	2.30 2.56	-- --	2.70 3.00
38	3710 Wilshire Blvd. Beneficial Plaza	1965	11/3	RC(SW)	24	E--W N--S	0.76 0.98	1.1 1.2	0.85 1.06
39	5900 Wilshire Blvd. Mutual Benefit Life Plaza	1970	30/3	ST	24	N83W S07W	-- --	5.0 5.0	4.55 4.60
40	9100 Wilshire Blvd. Wilshire Doheny Plaza	1969	10	RC	23	E--W N--S	1.09 1.09	1.8 1.7	1.23 1.26
41	2011 Zonal Ave. Hoffman Medical, USC	1966	9/1	RC	26	S28W S62E	0.48 0.48	0.6 0.6	0.53 0.53

Note: RC--Reinforced concrete, ST--Steel, MR--Moment-resisting frame, Br--Braced frame,SW--Shear wall, FP--Flat plate, T--Tubular structure.

Average expolation ratio for pre to during earthquake period: RC buildings(0.616) and ST buildings(0.690).

severely shaken buildings. Only then can one better assess the motion-damage relationship. There are three other newer high-rise buildings in the northern part of the San Fernando Valley located at distances much closer to the epicenter of the earthquake than those buildings cited in Table 3.1. The relative locations of all the buildings cited above are shown in Figure 3.1 and 3.2. These buildings are: Holy Cross Hospital, Indian Hill Medical Center, and Olive View Hospital.

The main building of the Holy Cross Hospital is a seven story tower with basement and was constructed between 1959-1963. This building was designed as a reinforced concrete structure with concrete shear walls in both directions to resist lateral seismic forces. No fundamental period measurement was conducted for the original undamaged structure due to the absence of accelerograph in the building. Dynamic analysis, using a simplified model of the complex lateral force-resisting system of the main tower, indicates that the fundamental period would be 0.65 seconds in the north-south direction and 0.80 seconds in the east-west direction. Small amplitude transient vibration measurements were made immediately following the San Fernando earthquake and showed fundamental periods of approximately 0.66 and 0.80 seconds for this tower, respectively. Due to the heavily damaged

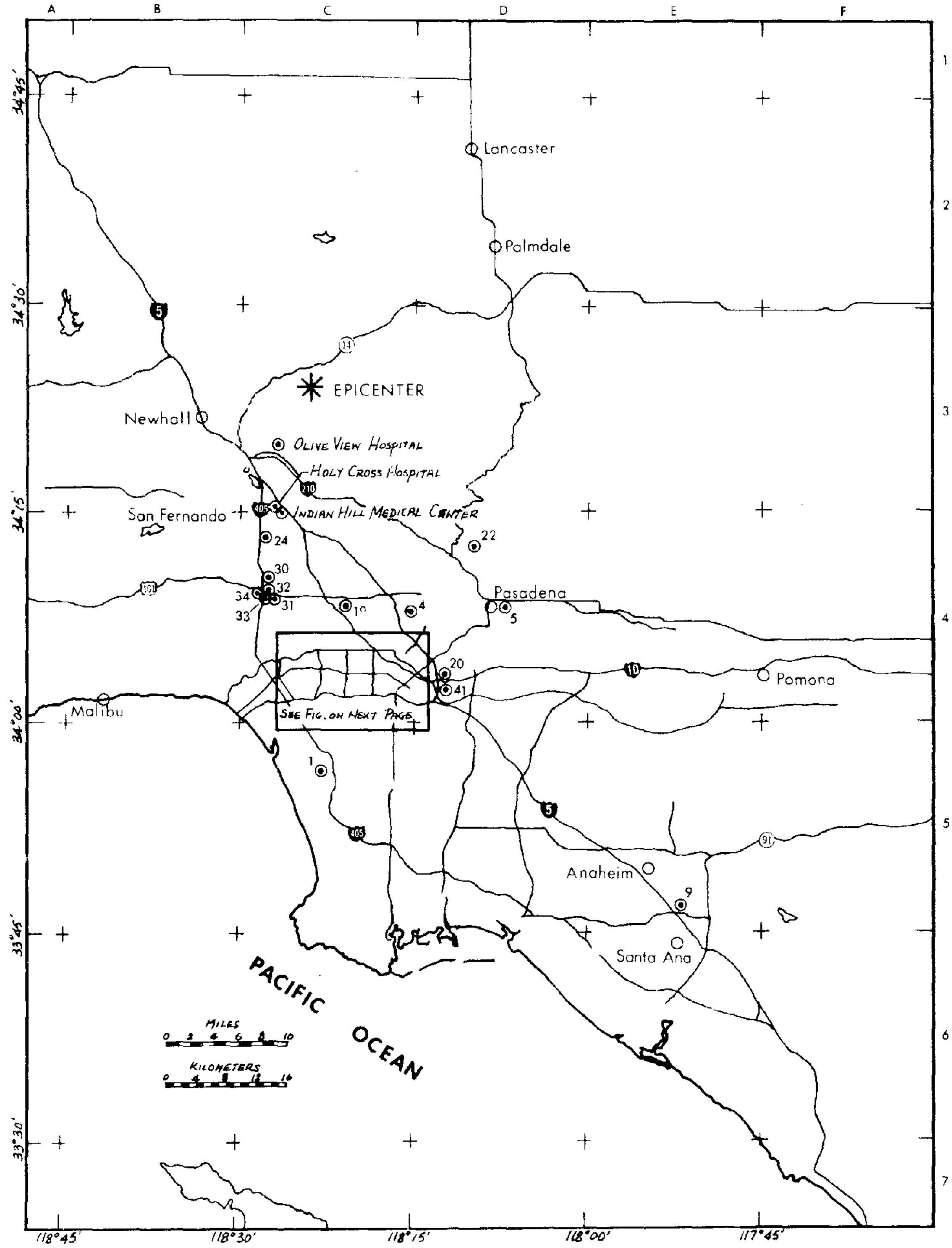


FIGURE 3.1 Accelerograph stations in extended Los Angeles area during the San Fernando earthquake.[16]

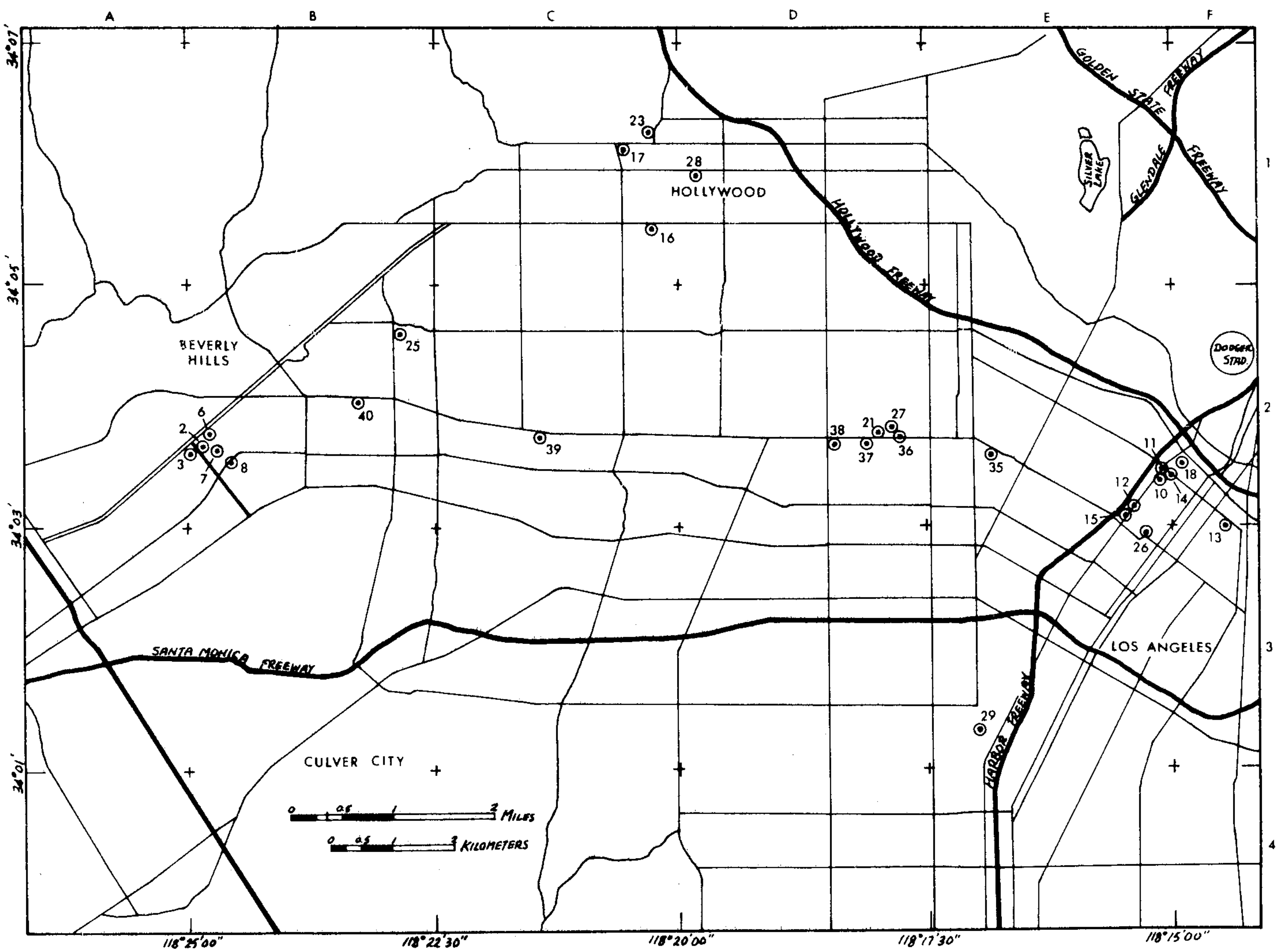


FIGURE 3.2 Accelerograph stations in central Los Angeles area during the San Fernando earthquake.[16]

condition of the structure at the time of the measurement, the validity of using these measured periods might be questionable. However, the original small amplitude period could have been only somewhat shorter than these measured values. As the calculated periods of the original structure have nearly the same magnitude as the measured ones, it seems reasonable to use them as the approximate fundamental period of vibration for strong-motion elastic respone[(3)].

The Indian Hills Medical Center is located about 300 feet from the Holy Cross Hospital. It is a seven story reinforced concrete shear wall structure with a complete vertical load carrying frame. It was designed in accordance with the 1966 edition of the Los Angeles City Building Code. In general, the structural system is regular and symmetrical. It is a structure that can be easily modeled to predict elastic structural response to earthquake ground motion. There were no accelerographs at or near the building, which is located between the instrumented locations at Pacoima Dam, the Holiday Inn on Orion Boulevard, and Castaic Dam. Dynamic analysis, using some simplifications in modeling the overall structural system, indicate that this structure would have fundamental periods of 1.25 seconds in both the north-south and east-west directions[(4)].

The Olive View Hospital lies at the base of the San Gabriel Mountains within about 6 miles southwest of the epicenter of the February 9, 1971, earthquake. Although there was no apparent tectonic surface rupture of the soil within the boundaries of the property, there was considerable permanent ground movement, both vertically and horizontally[18].

The main building of the Olive View Hospital is a single unit structure consisting of four symmetrical five-story wings supported on a single, large one-story base and was constructed during 1964-1970. The basic framing scheme is a two-way flat slab reinforced concrete system supported either on tied or spirally-reinforced columns. The lateral forces were resisted by a system of shear walls above the second floor and moment-resisting concrete frames in the lower two stories. In effect, the scheme may be described as a four story box structure supported on two levels of beam and column rigid frames[18].

There were four stair towers at the ends of the four wings, designed as free-standing cantilever towers, which were structurally separated from the main building by 4 inches. The walls on these box type concrete structures on three of the four towers terminated at the first floor level and were supported on a beam and column framing system. In the fourth tower the walls extended to the foundation pedestals. In general, the stair towers were designed to

TABLE 3.2 - COMPUTED BUILDING PERIODS FOR OTHER BUILDINGS IN THE LOS ANGELES AREA

Address & Bldg. Name	Year Built	Stories Up/Down	Struct. Type	Dist.to Epicenter	Directions	Periods
15031 Rinaldi St., Los Angeles Holy Cross Hospital	1959	7/1	RC(SW)	9 miles	N-S E-W	0.65 sec 0.80 sec
14935 Rinaldi St., Los Angeles Indian Hills Medical Center	1966	7	RC(SW)	9 miles	N-S E-W	1.25 sec 1.25 sec
14445 Olive View Dr. Sylmar Olive View Hospital	1964*	6	RC(MR,SW)	6 miles	N-S	0.60 sec

*The Olive View Hospital complex of modern reinforced concrete structures was designed in 1964 and completed in 1970.

resist lateral forces with reinforced concrete shear walls[8].

A three dimensional elastic analysis of a model of the main building of the Olive View Hospital indicates that the fundamental mode of vibration was primarily translational in the north-south direction (direction of major shaking), and the fundamental period was 0.59 seconds in this direction. Dynamic analysis on two of the three collapsed stair towers indicates that these two stair towers had approximately the same periods as the main building. Therefore, for all practical purposes, a fundamental period of 0.60 seconds was assumed for these three structures[8].

3.2.3 BUILDING DAMAGE DATA

For any motion-damage correlation studies of high-rise buildings, a general damage parameter must be used to describe the amount of damage incurred on a building during an earthquake. This requirement can be adequately satisfied by the parameter, damage ratio, DR, defined as

$$DR = \frac{\text{Damage Repair Cost}}{\text{Replacement Cost of Building}}$$

This nondimensional parameter is quite general and may be used for a geographical area as well as for various building classifications or types of constructions.

While a considerable number of tall buildings had been inspected for damage by many researchers immediately following the San Fernando earthquake (i.e.,Steinbrugge, 1972), a parallel effort was undertaken by M.I.T. to gather damage information. This was done through questionnaires and field trips to the building sites. The objective of the effort was to determine repair costs for the buildings under study and the amount of damage experienced by them. Many of the repair costs were revised several times by subsequent interviews in an attempt to single out the repair cost due to earthquake damage.

There are at least two methods of assessing the present building value: (1) the current market value, and (2) the replacement cost. The difference between these two values, which arise from depreciation, market conditions, and inflation, may not be significant for newer buildings constructed in recent years, while it will be great for older buildings. From the standpoint of this study, the replacement cost is considered more appropriate since this value is more definite than the current market value. The replacement cost is unaffected by such changeable factors as market conditions, assessing practice, and inflation.

At the outset of this M.I.T. damage data gathering effort, it was recognized that this form of experimental research would be every bit as difficult as any good research in the laboratory. Indeed, it proved to be difficult and expensive to compile accurate and complete statistics of this form. In any case, it is believed that the damage figures available at the time of writing this report represent the best estimate one can derive for such information. All the damage values for the buildings listed in Table 3.1 are presented in Table 3.3. These damage values are extracted from the M.I.T. data base (Whitman, 1973). The damage values for other buildings in the San Fernando Valley area for the San Fernando earthquake are obtained from other sources and are listed in Table 3.4.

TABLE 3.3 - DAMAGE VALUES FOR BUILDINGS IN THE LOS ANGELES AREA (35)

M.I.T. Bldg. No.	Damage Ratio %	M.I.T. Bldg. No.	Damage Ratio %
2	3.490	755	0.142
11	0.007	876	0.057
77	0.062	904	0.201
97	0.010	920	0.601
204	0.436	923	1.293
221	0.118	925	0.205
222	0.071	928	1.754
347	0.033	1008	0.004
350	0.086	1026	0.002
353	0.021	1041	0.232
387	0.759	1046	0.467
388	0.0	1073	0.277
430	1.463	1118	0.0
472	0.0	1184	0.0
544	0.239	1407	3.200
550	0.205	1408	1.782
592	0.021	1438	1.935
661	3.772	1643	0.0
687	0.242	1682	0.0
732	0.125	1841	0.851
733	5.103		

TABLE 3.4 - DAMAGE VALUES FOR OTHER BUILDINGS IN THE LOS ANGELES AREA

Address & Bldg. Name	Damage Ratio, %
15031 Rinaldi Street, L.A. Holy Cross Hospital	26 [34]
14935 Rinaldi Street, L.A. Indian Hill Medical Center	48 [4]*
14445 Olive Drive, Sylmar Olive View Hospital	100

*The reference did not explicitly state whether damage ratio for Indian Hill Medical Center was for structural and/or nonstructural damage.

3.2.4 STRONG EARTHQUAKE GROUND MOTION DATA

The strong earthquake ground motion that a building was subjected to is conventionally identified by three independent orthogonal time-histories of motion. Although these earthquake time histories are usually represented in terms of acceleration, they can also be represented in units of displacement, or velocity.

In consideration of the classical damage criteria (Crandall and Mark, 1963) and the nature of dynamic response of buildings, the most important ground motion characteristics that influence building damage are: frequency content, motion amplitude, periodicity, and duration. Insofar as these influence building damage, all these factors are included in the response spectrum characterization of ground motion, except duration. Therefore, it is obvious that the use of response spectrum to characterize ground-motion in the motion-damage correlation is an improvement over the use of peak ground-motion value. The only limitation on the use of spectral values is that they do not provide a description of building response duration[27].

The strong ground motion of the San Fernando, California, earthquake of February 9, 1971 was recorded on over 200 accelerographs of the Southern California strong-motion accelerograph network. These accelerographs were located at various ground sites, dams, and buildings. The majority of the instrumented buildings were located in the greater Los Angeles Metropolitan area. By number and quality of these strong motion records, the instrumental coverage of the San Fernando earthquake is by far the most complete and extensive in the history of strong motion seismology (Hudson, 1971). This earthquake provides a very unique opportunity to conduct a quantitative motion-damage study of high-rise buildings.

Immediately following the San Fernando earthquake, the California Institute of Technology undertook the effort to digitize the accelerograms recorded during the San Fernando earthquake and to generate response spectra for these accelerographs. This effort made available response spectrum curves for all buildings cited in Table 3.1, except three buildings: the Howard Johnson Lodge at 15433 Ventura Boulevard, L.A.; Shareholders Building at 1888 Century Park East, L.A.; and Century City Medical Plaza at 2080 Century Park East, L.A. The basement accelerograms of these three buildings were not recorded properly due to some instrument malfunction. Therefore, response values for these three buildings were obtained by using the average values of two response spectrum curves at sites nearest the building.

This method used in the evaluation of spectral values for these three buildings assumes soil condition does not vary drastically as to affect the recorded time histories at these particular sites. The results presented in the next chapter shows insignificant differences between the two spectral values, indicating the assumption is more-or-less correct.

The Holy Cross Hospital and the Indian Hill Medical Center are situated approximately one mile southeast of the lower Van Norman Dam. There are no accelerographs in these two buildings. The location of these buildings indicates that the ground motion would be intermediate to the instrumented motion at the Pacoima Dam, approximately 5 miles from the epicenter, and at the Holiday Inn on Orion Avenue, 9 miles farther away from the epicenter. Due to the lack of strong-motion accelerometers at this location, a satisfactorily accurate estimate of response for these buildings during the earthquake is difficult without information on the ground motion at this site. However, at the east abutment of the lower Van Norman Dam, a seismoscope* (S-213) recorded the shaking during the San Fernando earthquake. By using mathematical simulation techniques and the

*The seismoscope is a simple instrument designed originally to give a largely qualitative indication of the strong ground motions produced by an earthquake.

seismoscope records (S-213), Scott derived time histories and corresponding response spectra for the lower Van Norman Dam site (Scott, 1973). The result from this technique is demonstrated to be valid with some uncertainty, especially for high frequency ground shaking.[28] It is recognized that the spectral values obtained for the two medical buildings cited above from the derived spectra will have a higher degree of inaccuracy than for those buildings with actual on site measured accelerograms. In any case, the intent of this study is to develop an approximate technique for the determiniation of mean damage values rather than deterministic quantities. The derived spectra are shown in Figure 3.3 for the two components of earthquake motion. It is seen that the north component gives the larger spectral values than the east component.

The Olive View Hospital is located three miles west of Pacoima Dam at the north edge of an alluvial fan at the base of the San Gabriel mountains.

Horizontal accelerations over 0.7 g were recorded for the Pacoima Dam instrument, which was founded on a steep rock ridge directly over the subsurface zone of extended faulting of the San Fernando Earthquake. A few isolated peaks of horizontal motion exceeding 1.0 g were also recorded.[17] Another instrument located seven miles south of the Olive View Hospital complex at the Holiday Inn on

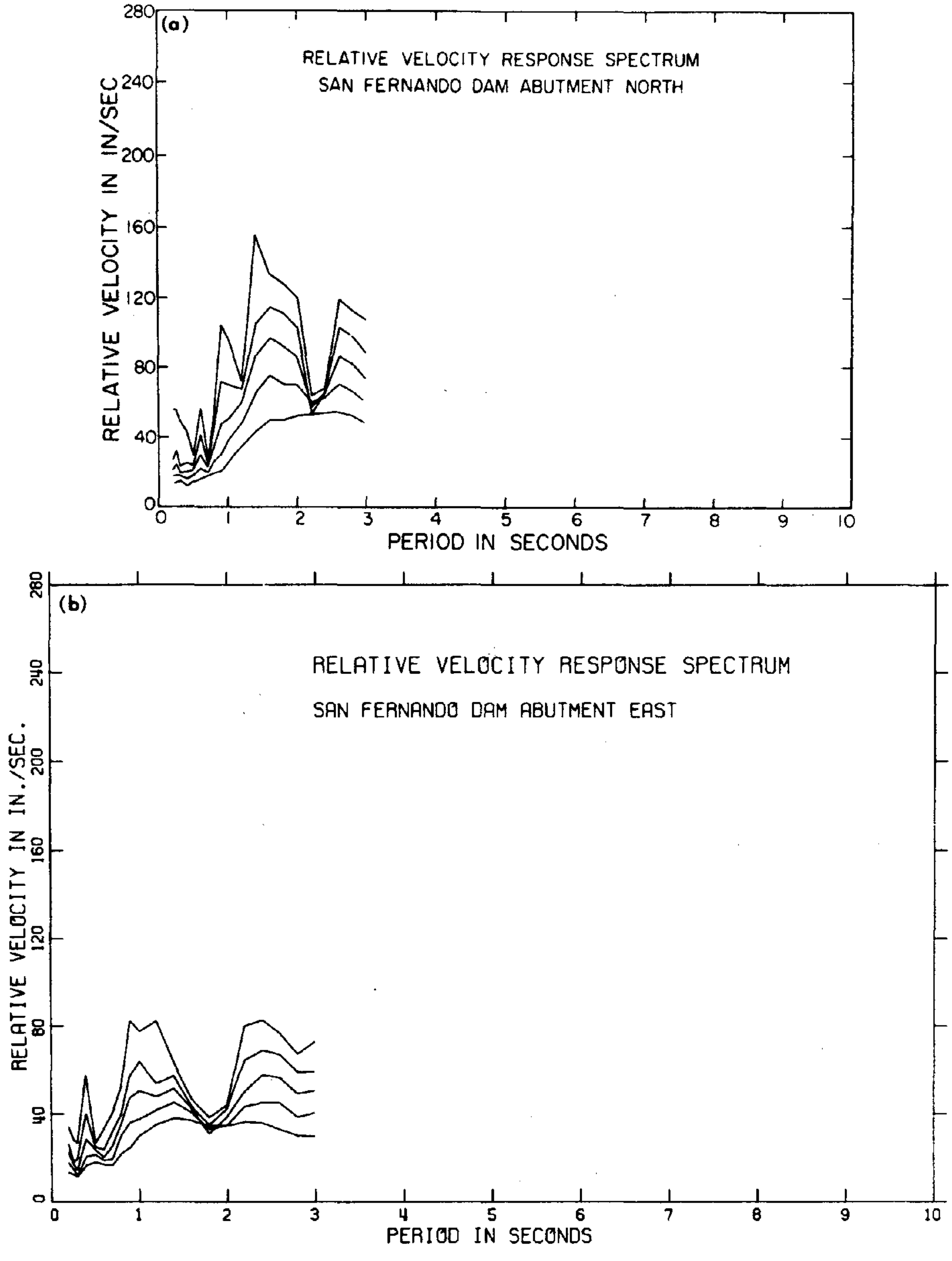


FIGURE 3.3 Relative velocity response spectra for computed acceleration components, seismoscope S-213, the curves are for 0, 2, 5, 10, and 20 percent damping.

8244 Orion Boulevard recorded a peak ground acceleration of 0.28 g for the earthquake.

It is apparent from the level of damage at the Olive View site that peak accelerations exceeded the measured value of 0.28 g recorded at the Holiday Inn site; it appears very likely that peak accelerations on the order of twice this recorded value may have occurred at the Olive View Hospital.[14] Unfortunately, just how intense the ground shaking was at this site cannot be accurately assessed. A reasonable estimate of the spectral value might be obtained for the Olive View hospital by taking some average value of the Pacoima Dam record, the Holiday Inn record, and the derived seismoscope (S-213) record. The horizontal components of the response spectra for the Pacoima Dam and the Holiday Inn accelerograms are shown in Figure 3.4 and 3.5 respectively.

3.3 DATA FROM MANAGUA, NICARAGUA EARTHQUAKE OF DECEMBER 23, 1972[20,21]

The city of Managua, Nicaragua was severely damaged by the seismic activity on December 23, 1972; two heavy shocks occurred in a span of fourteen seconds. The magnitude was estimated at 6.25 on the Richter scale. The epicenter of

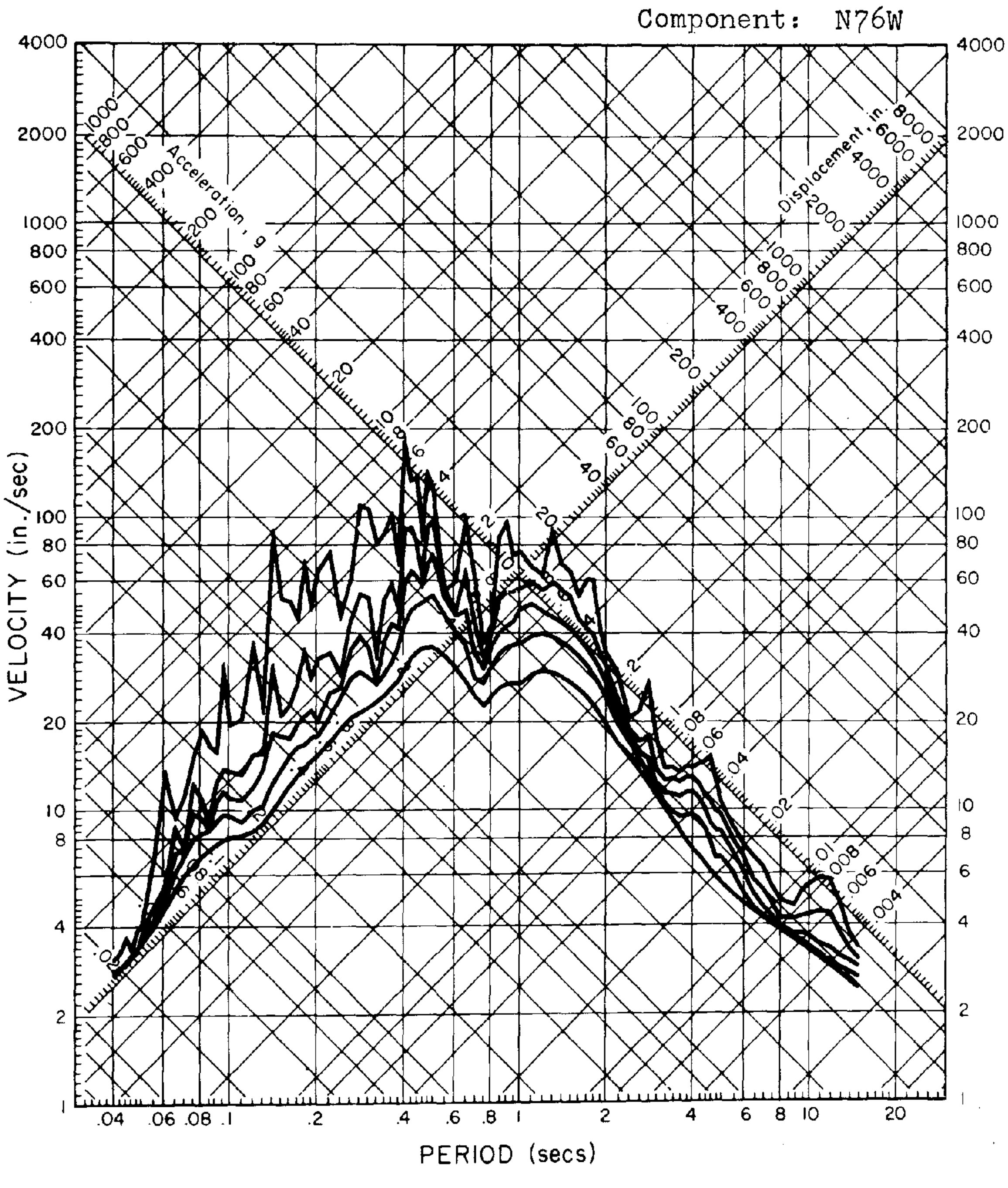


FIGURE 3.4a San Fernando Earthquake response spectrum, Pacoima Dam, the curves are for 0, 2, 5, 10, and 20 percent damping.

Component: S14W

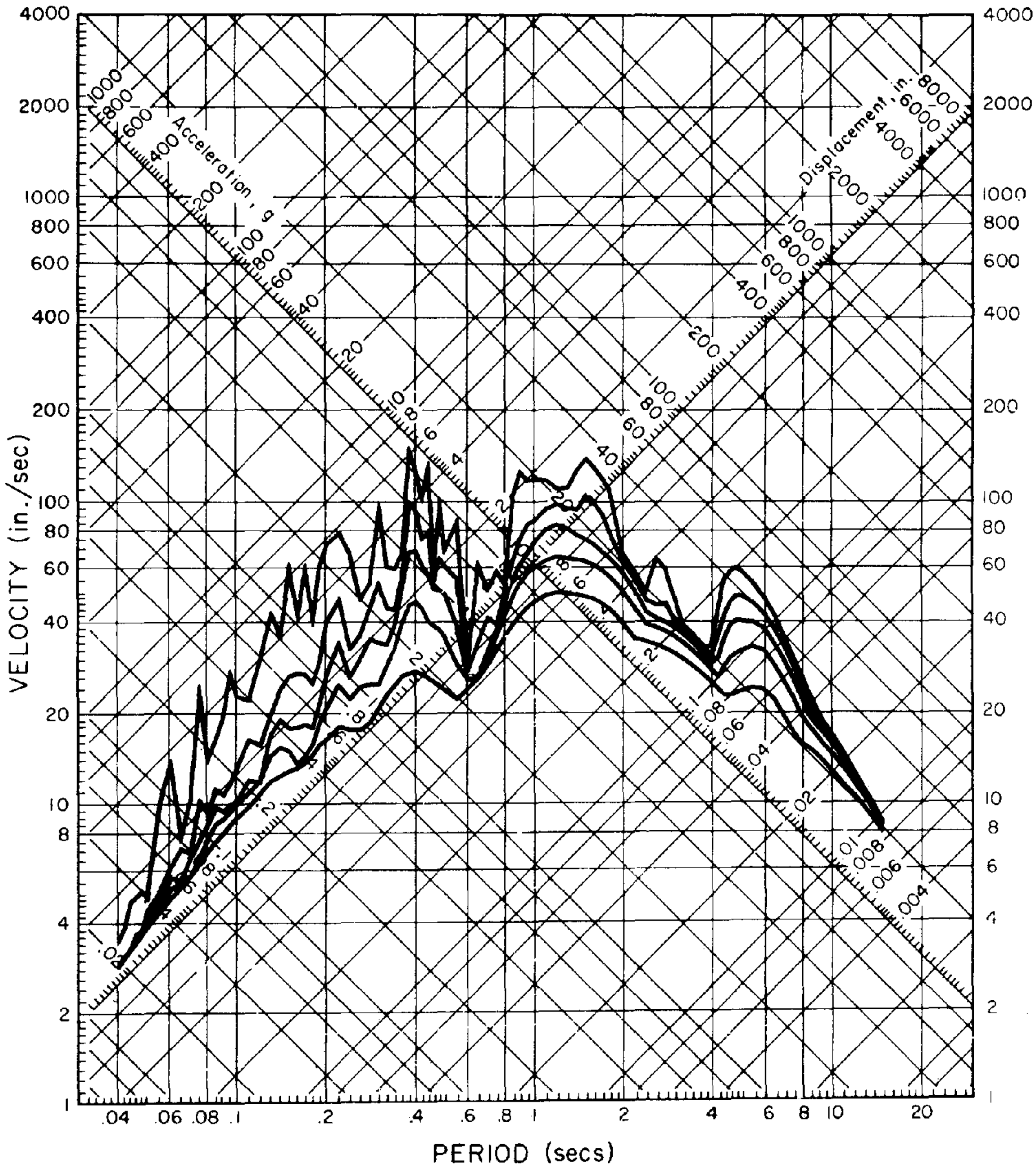


FIGURE 3.4b San Fernando Earthquake response spectrum, Pacoima Dam, the curves are for 0, 2, 5, 10, and 20 percent damping.

Component: S90W

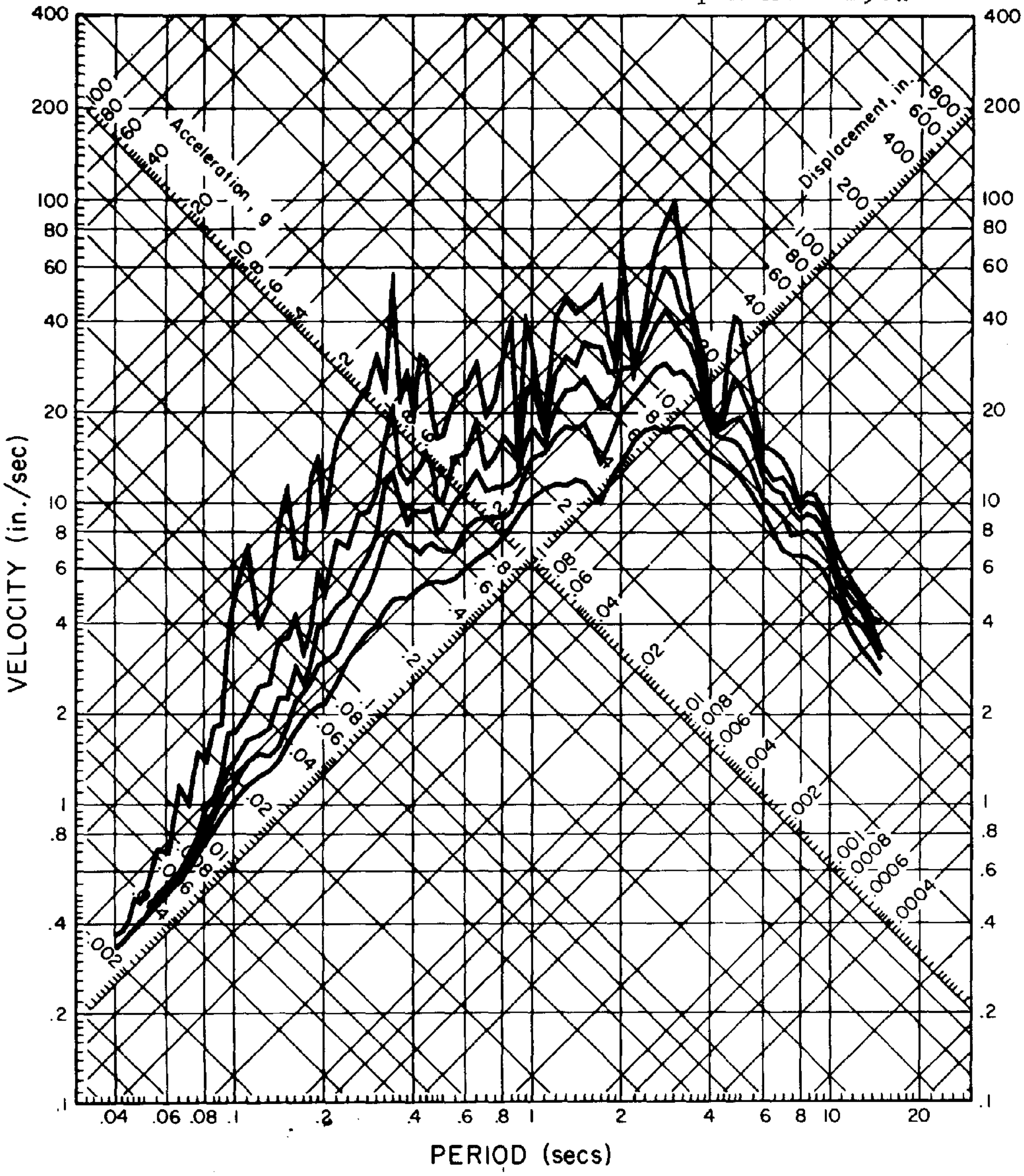


FIGURE 3.5a San Fernando Earthquake response spectrum, Holiday Inn at Orion Blvd., the curves are for 0, 2, 5, 10, and 20 percent damping.

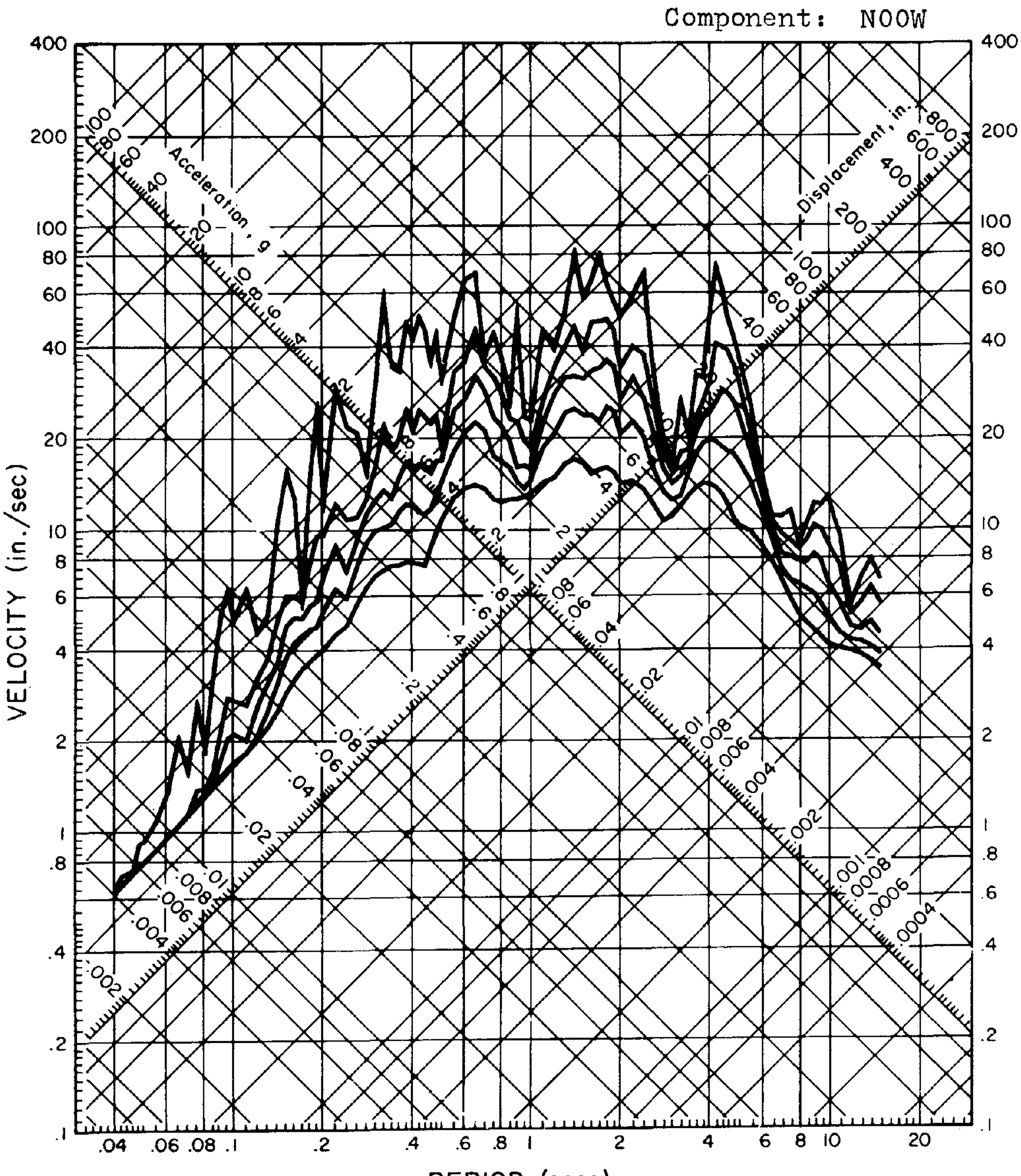


FIGURE 3.5b San Fernando Earthquake response spectrum, Holiday Inn at Orion Blvd., the curves are for 0, 2, 5, 10, and 20 percent damping.

this earthquake is estimated to be in the vicinity of the downtown area and the focus is estimated to be at two miles in depth. This earthquake devastated 600 to 700 square blocks of the city of Managua and caused approximately 7,000 deaths, 20,000 injuries, and property damage exceeding $700 million (U.S.).

Prior to the earthquake, the city of Managua had no established building code or any building departments that governed the design and/or construction of buildings. In spite of this many of the major buildings constructed in Managua used the Uniform Building Code, the SEAOC Lateral Force Recommendations, and the ACI Code in the design.

The Banco Central de Nicaragua and Banco de America were the two tallest buildings in Managua at the time of the earthquake. These two buildings are of special interest in this study due to the significant level of structural damage as well as nonstructural damage suffered during the Managua earthquake. The Banco Central Building is a 16-story reinforced concrete moment-resisting frame structure. It consists of a three-story base with a twelve story tower on top and was built during 1962-1964. The lateral design loading criteria apparently used in the design of this building was considerably in excess of any building code requirement in the U.S. at that time, but the structural damage of this building was considerable from the Managua earthquake. The Banco de America Building is an eighteen story reinforced concrete coupled shear wall structure designed in accordance with standards of the Uniform Building Code and built in 1968. This building is approximately a 22.7 meter square building. It has four shear wall cores and each core is connected to the other by two reinforced concrete beams. The Banco de America building sustained less structural damage than the Banco Central building during the Managua earthquake.

Ambient dynamic measurements were conducted on these two tall buildings after the Managua, Nicaragua earthquake by the "Fourier Analyzer System" (Shah, 1973) to determine the natural periods of these buildings.[30] A summary of the measured fundamental periods and the damage ratios for these two buildings are presented in Table 3.5.

Only one accelerograph was functioning at the time of the Managua earthquake of December 23, 1972. This was located at the Esso Refinery, 3 1/2 miles west of downtown Managua, where ground accelerations of 0.39 g E-W, 0.34 g N-S, and 0.33 g vertical were recorded. The lack of a strong-motion record for the downtown area is unfortunate, since the record at the Esso Refinery does not represent the precise ground motion in downtown Managua. Available

TABLE 3.5 - MEASURED FUNDAMENTAL PERIOD
AND DAMAGE RATIO FOR BUILDINGS IN MANAGUA, NICARAGUA

Building Name	Year Built	Direction	Measured[30] period, sec.	Damage ratio, %[33]
Banco Central de Nicaragua	1962-64	E-W	2.0	38
		N-S	1.92-2.0	
Banco de America	1968	E-W	2.10	33*
		N-S	1.82	

*The Damage Ratio for the Banco de America Building includes the effect of fire which may not be directly attributable to the earthquake.

seismoscope traces indicate an increase in the ground motion in downtown Managua with respect to that of the Esso Refinery. Despite the record's limitations and exclusions, it does represent an invaluable reference point for the understanding of the ground motion in Managua.

Aftershocks ranging in magnitude from approximately 0.5 to about four were recorded on an extensive seismic array deployed following the 1972 Managua earthquake. The locations for aftershock recording were selected so as to obtain data which could be incorporated in soil amplification and structural response research studies. From these aftershock data and the use of a recently developed technique (Hays, 1973), estimates of the main shock response spectra were conducted for a number of sites where no main shock motion was recorded. This technique uses the main shock record at the Esso Refinery as a reference point. The technique can be applied independently of any knowledge of the site geologic configuration. The application of this technique provided reasonable estimates of the main shock ground motion at various locations; each estimated main shock response spectrum has an uncertainty of about 50% independent of period.[15]

The estimated main shock norizontal response spectrum for the station at Banco Central Building differs significantly from that of the station at Esso Refinery even if the 50% level of uncertainty is taken into account. The difference between the response spectrum of these two locations is the greatest between periods of 0.3-2.0 seconds with spectral values for the Banco Central site ranging from about 1.5 to almost 4 times greater than that for the Esso Refinery site. This large difference between the two locations is reasonably consistant with the seismoscope traces measured for the Managua earthquake. The estimated response spectrum for the Banco Central site is shown in Figure 3.6. This estimated response spectrum is also used for the Banco de America building, since the Banco de America and the Banco Central buildings are only separated by a city street.

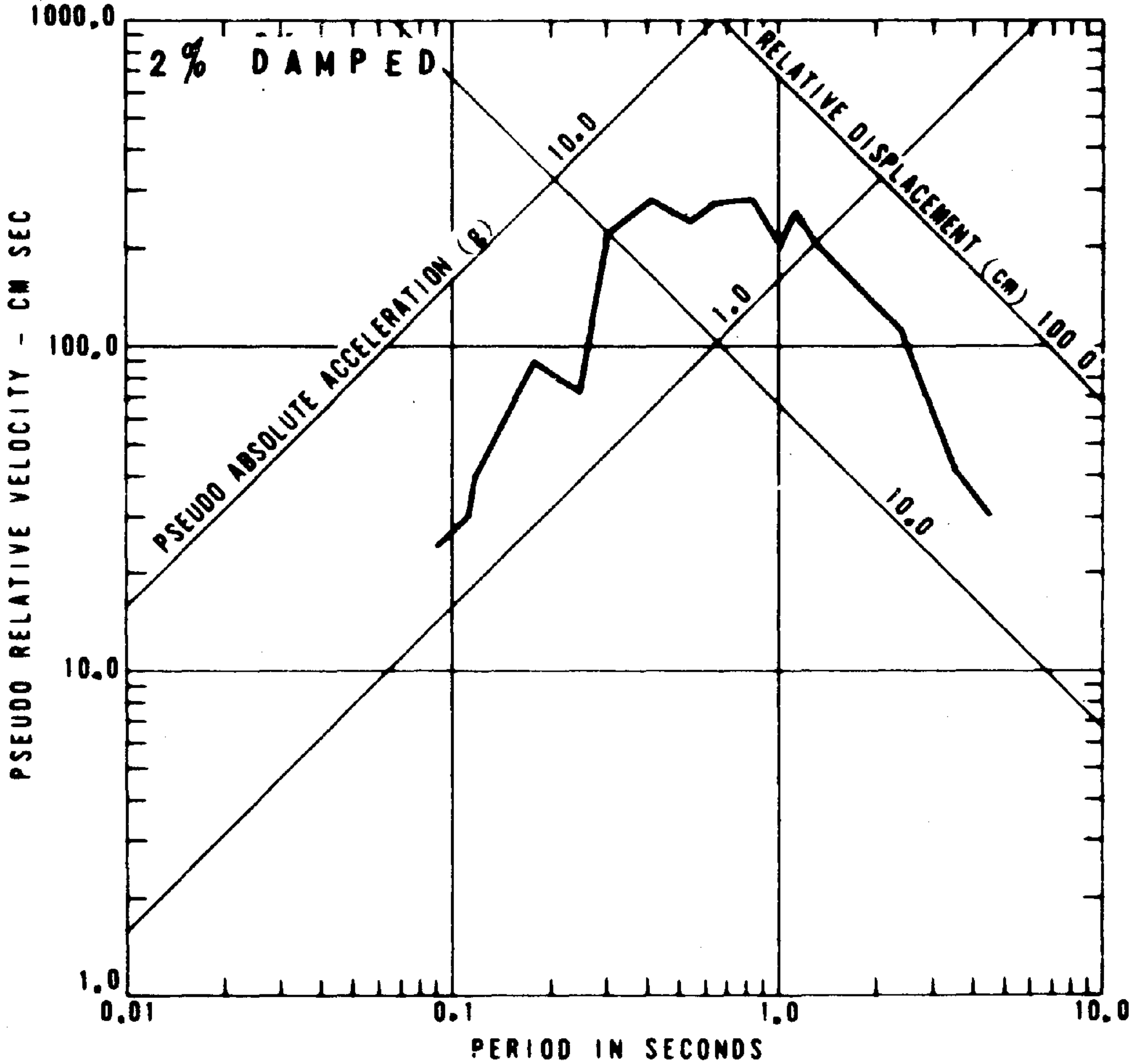


FIGURE 3.6 Estimated Managua earthquake horizontal response spectrum, for station at the Banco Central de Nicaragua building. (after Hays, 1973)

Chapter 4. CORRELATIONS OF BUILDING RESPONSE AND DAMAGE

4.1 INTRODUCTION

This chapter presents the development of motion-damage correlations for estimating earthquake damage to high-rise buildings based on the building's dynamic response. Many parameters are considered in the process of investigating the earthquake ground motion-damage problem. For the sake of brevity only those parameters that give the best indication of motion-damage relations will be presented in subsequent sections of this chapter.

4.2 METHOD OF ANALYSIS

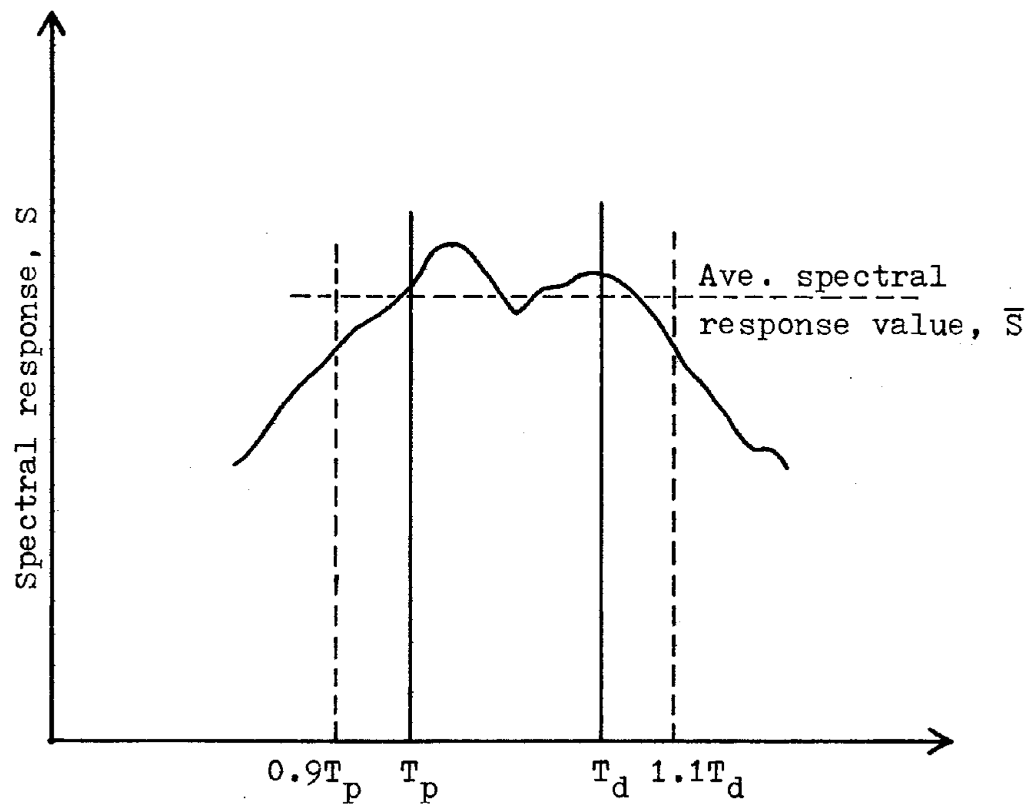
To effectively correlate ground motion with building damage requires consideration of the nature of dynamic structural response to ground motion, i.e., dynamic response amplification. For earthquake ground motion correlation purposes, it would be too involved and expensive to compute total dynamic response behavior of tall buildings to ground motion as indicators of damage. Therefore, it is reasonable to idealize a tall building as a single-degree-of-freedom system and to use its vibration properties (i.e., spectral values, damping, and period) as the fundamental mode vibration of the tall building. This idealization for tall buildings is appropriate because it includes consideration of dynamic response behavior of buildings and also provides the flexibility of allowing for different motion-damage relationships for various building construction types. Although the fundamental mode response of a tall building generally does not represent the total dynamic response; theoretical considerations of structural dynamics indicate that a large portion of the total response is usually included in the fundamental response. Using the fundamental mode model and a given set of response spectral curves, there are only two parameters that need to be considered explicitly to determine the spectral response of the one-degree-of-freedom system: fundamental period, and damping.

There are several schemes that might be used to establish viscous modal damping values for different classes of buildings. For example, one might select the steel buildings in one class and the reinforced concrete buildings in another class. However, considering the high level of uncertainties in the determination of the damping values for buildings and the immediate goal of this study, it was decided to simply group all of the buildings in this study into a single class having a viscous modal damping value of 5 percent of critical. Five percent damping was used because it is reasonably applicable and because of its standardized use in many aspects of earthquake engineering problems.

The techniques used to measure fundamental building periods of the majority of the high-rise structures in the Los Angeles and San Fernando Valley area involve a great deal of judgement in selecting the appropriate record segments for analysis. The selection of these record segments is based on one's judgement of the regularity and duration of the nearly sinusoidal data traces recorded from ambient or earthquake vibrations on the roof or top floor of each building. In structures with a fundamental period of approximately 1 second or less, it is usually possible for independent investigators to obtain results within and uncertianty of 0.01 to 0.02 second. In the longer period buildings the precision is not as great, and repeatability of results may vary by as much as 0.1 second or more for the investigation. Therefore, inherent in the measured fundamental building periods of all the buildings cited in Table 3.1, there is an average uncertainty level of about 10 percent.[22]

To characterize the ground motion at a particular building site, one must arrive at a single-valued number from the response spectrum curve. However, this is complicated by the fact that the fundamental periods of buildings change significantly during and after an earthquake. That is, building period changes from a shorter, pre-earthquake period to a longer, during-earthquake period as stiffness of the structure degrades. This is accomplished by averaging the response spectrum for each particular site over the period band from pre-earthquake period less 10 percent to during-earthquake period plus 10 percent. The 10 percent corrections allow for uncertainties in the two periods due to the inaccuracy in period measurements. This technique, undoubtedly, covers the fundamental periods the building underwent for the total duration of the earthquake ground motion. The technique can be better understood by the illustration in Figure 4.1.

The ground motion to which a building is subjected can be completely identified by three independent orthogonal time-histories of motions. From these time-histories one is able to obtain three independent response spectra for a particular building site. The task of selecting an appropriate response spectrum for a motion-damage correlation



T_p= Pre-earthquake building period.

T_d= During-earthquake building period.

FIGURE 4.1 Definition for period banding of response spectrum.

analysis is complex. Considering the nature of dynamic structural response to ground motion and the method of construction for high-rise buildings, Czarnecki (1973) was able to deduce that a large proportion of earthquake damage is contributed by horizontal rather than vertical motion. Indeed, since there is no present evidence to substantiate the importance of vertical motion to building damage, this component of response was not considered in the motion-damage correlation study. This, in essence, reduced the number of response spectra curves used for each building to only two horizontal components. Since our analysis requires a single response value, we needed to reduce these two components into a single representative one. This was achieved by conducting the correlation studies using various approaches to combine the two motions and determine which scheme gave better motion-damage correlation. Specifically, two schemes were used to simplify the two components of response into a single value so that it can be used in the motion-damage correlations. First, use the larger of the two horizontal components of response. This can be justified if we recognize that while the larger component may not accurately represent the total damage, it does contribute to a major portion of the overall damage. Second, use the vector sum of the two horizontal components of response. In other words, compute the vector response by taking the square root of the sum of the squares of the two independent horizontal components of response. Although the probability for the simultaneous occurrence of the two spectral components is very low, even if the two components were to occur independently of each other, the resulting total building damage is equivalent to applying the two spectral components simultaneously in their respective directions. This conclusion for the vector component approach is true only if the assumptions that the vertical ground motion has no contribution to building damage and the particular response parameter under study is an indicator of damage are also true. In subsequent sections of this chapter, the two schemes depicted above will be referred to as the critical component of response and the vector component of response, respectively.

4.3 MOTION-DAMAGE CORRELATION RESULTS

Recognizing the fact that there are many possible expressions for the ground motion that the buildings at the various sites were subjected to during an earthquake, four characterizations of ground motion were selected for this study. These parameters are:

1) Relative Displacement, S_d

2) Spectral Velocity, S_v

3) Spectral Acceleration, S_a

4) Inter-story Displacement.

Although there are other ground motion parameters that might be considered, the ones selected constitute a substantial share of those that are readily available and applicable to many aspects of earthquake engineering today.

This motion-damage correlation study was conducted in two parts due to the superior quality of data for those buildings presented in Table 3.1 than all other buildings cited in Chapter 3. The first part of this study will utilize the data for the buildings listed in Table 3.1 to conduct a parameter study to determine the parameters that are the better indicators of damage. Once these better parameters are determined, other data in Chapter 3 will be correlated with these data and an overall motion-damage relationship for the estimation of damage will be derived. This two-part procedure of analysis will, hopefully, eliminate any erroneous results that might be caused by the relatively higher level of uncertainties inherent in the data for those severly damaged buildings compared to those buildings in Table 3.1.

With the response spectra derived from time-histories recorded at each building sites for all the buildings in Table 3.1(2) and the method of analysis described in Section 4.2, it is a simple matter to compute response values and correlate them with the corresponding damage ratio, DR, in Table 3.3. These correlation plots are shown in Figure 4.2 (Relative Displacement), Figure 4.3 (Spectra Velocity), Figure 4.4 (Spectral Acceleration), and Figure 4.5 (Inter-story Displacement). Inter-story displacements are computed, considering only the fundamental modal component of relative displacement, with the assumptions that the tall buildings have uniform mass distribution along the height, the story height is constant for the whole building, and the mode shape is linear. With these assumptions, computations of inter-story displacements can be easily performed without any knowledge of the structural system, except the total number of stories. Although in general this idealization is quite crude, considering the immediate goal of this study and the level of uncertainties in other data, these approximations are reasonable for most tall buildings.

After considering various ways to perform regression analysis on the data (linear, nonlinear, etc.) it was concluded that the most readily interpreted and perhaps the

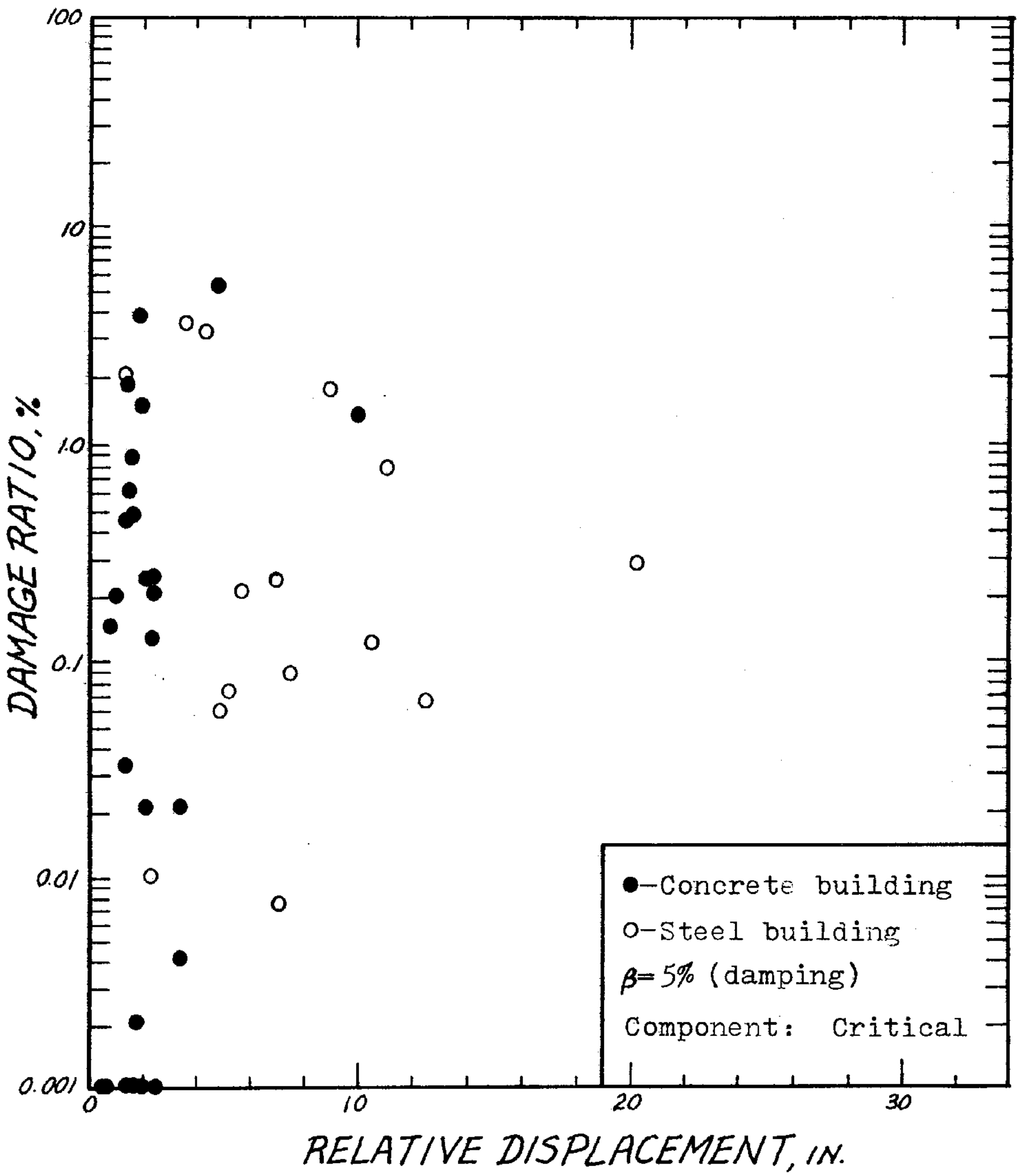


FIGURE 4.2a Correlation of Relative Displacement with Damage Ratio

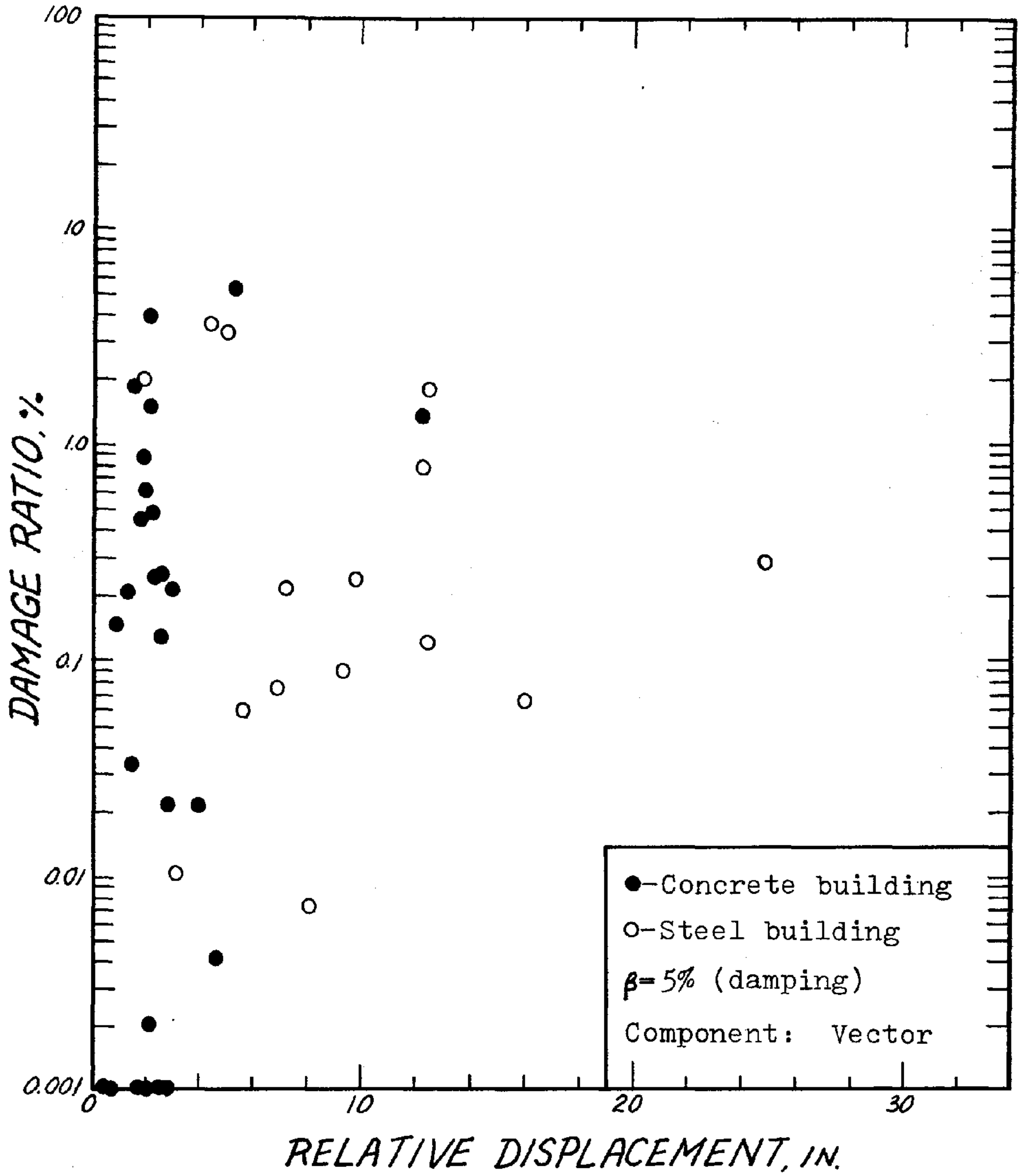


FIGURE 4.2b Correlation of Relative Displacement with Damage Ratio

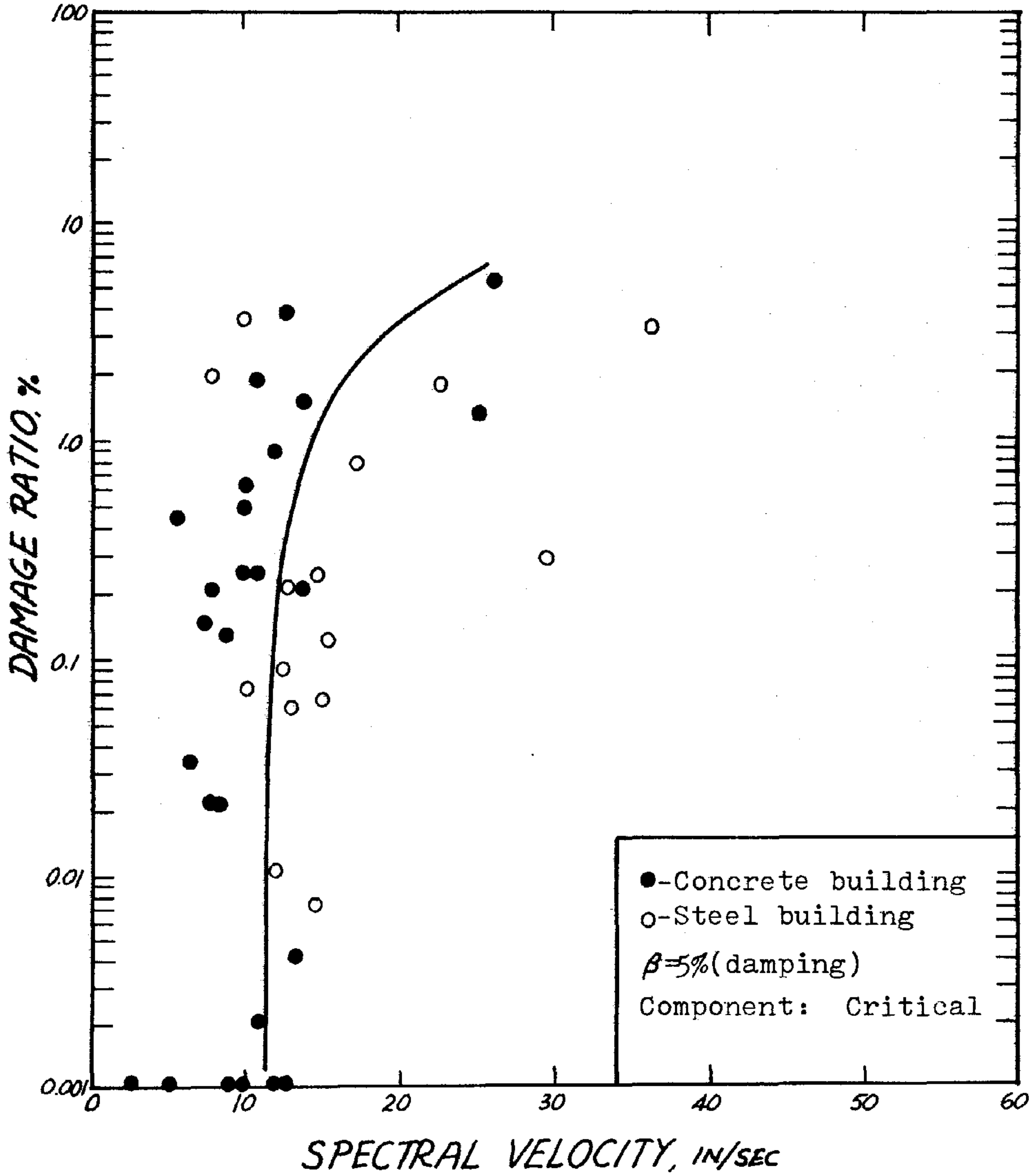


FIGURE 4.3a Correlation of Spectral Velocity with Damage Ratio

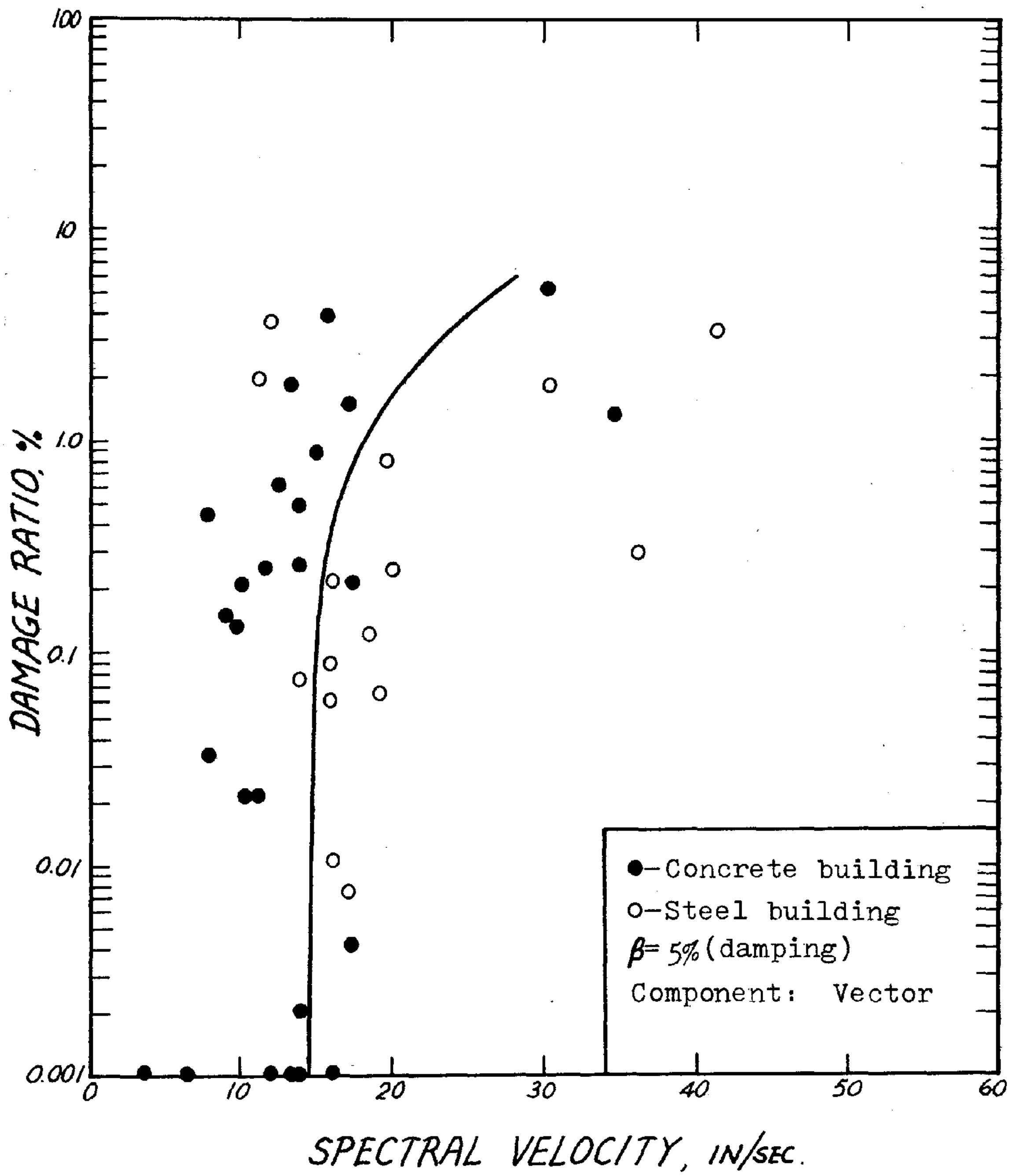


FIGURE 4.3b Correlation of Spectral Velocity with Damage Ratio

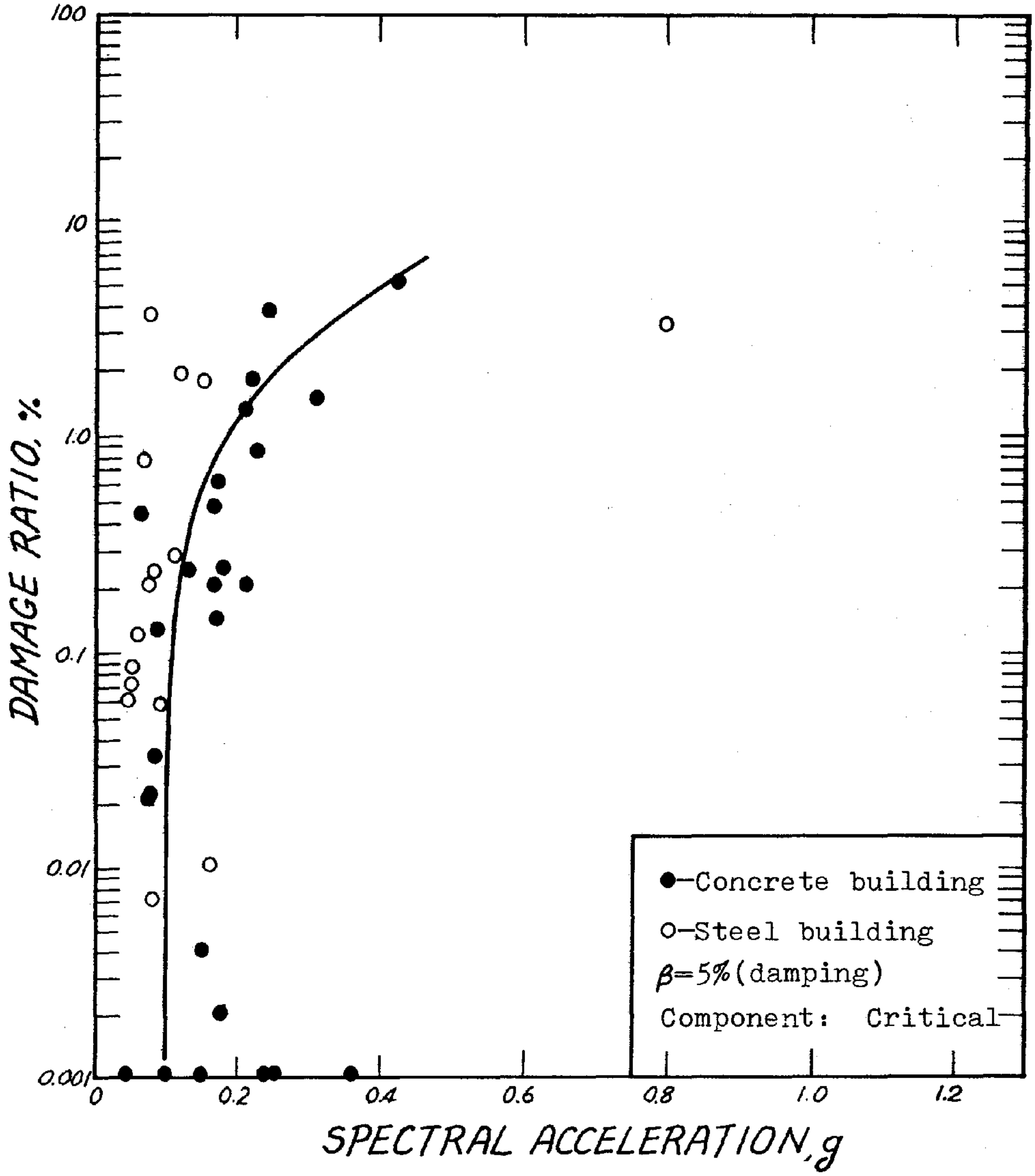


FIGURE 4.4a Correlation of Spectral Acceleration with Damage Ratio

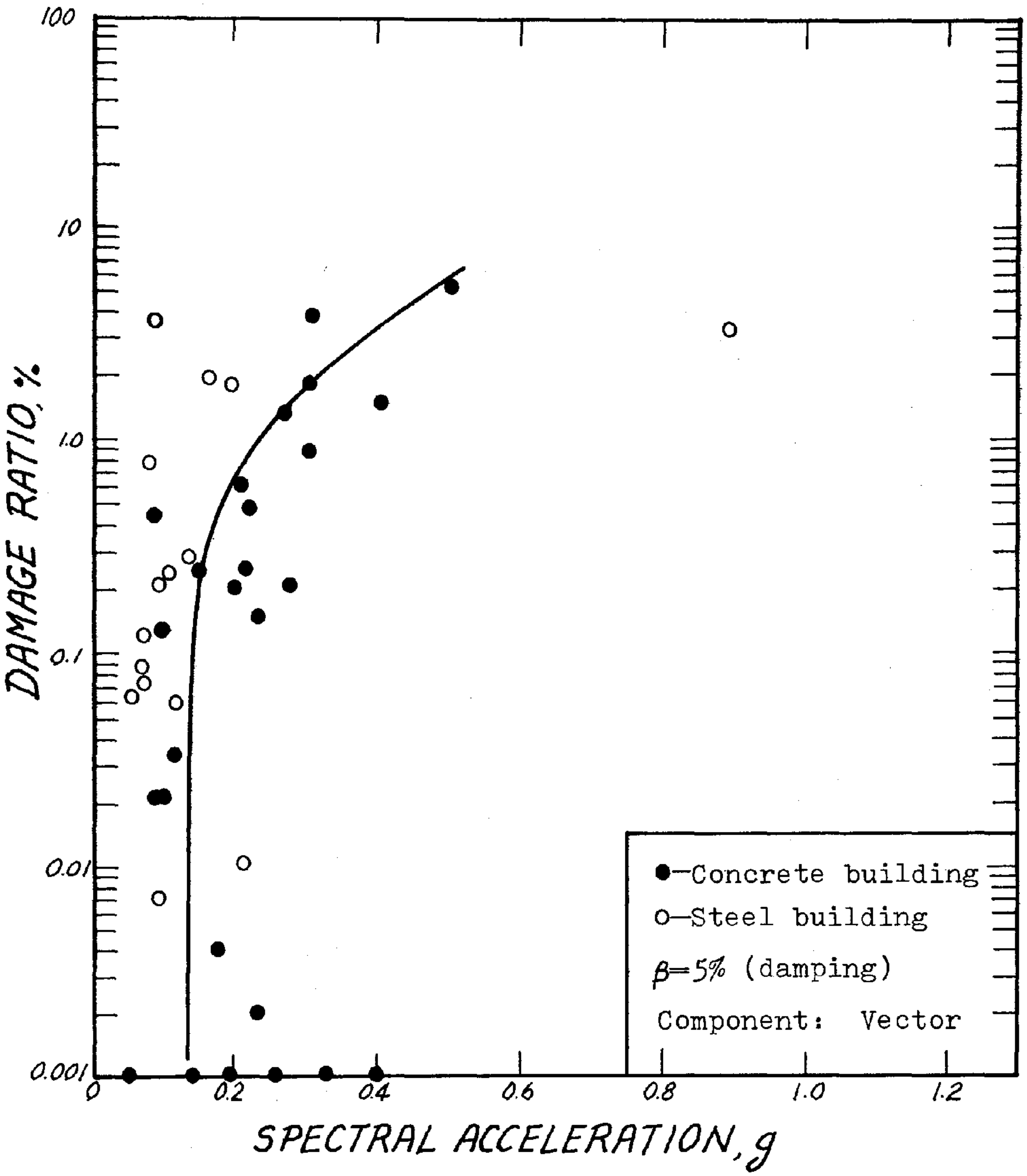


FIGURE 4.4b Correlation of Spectral Acceleration with Damage Ratio

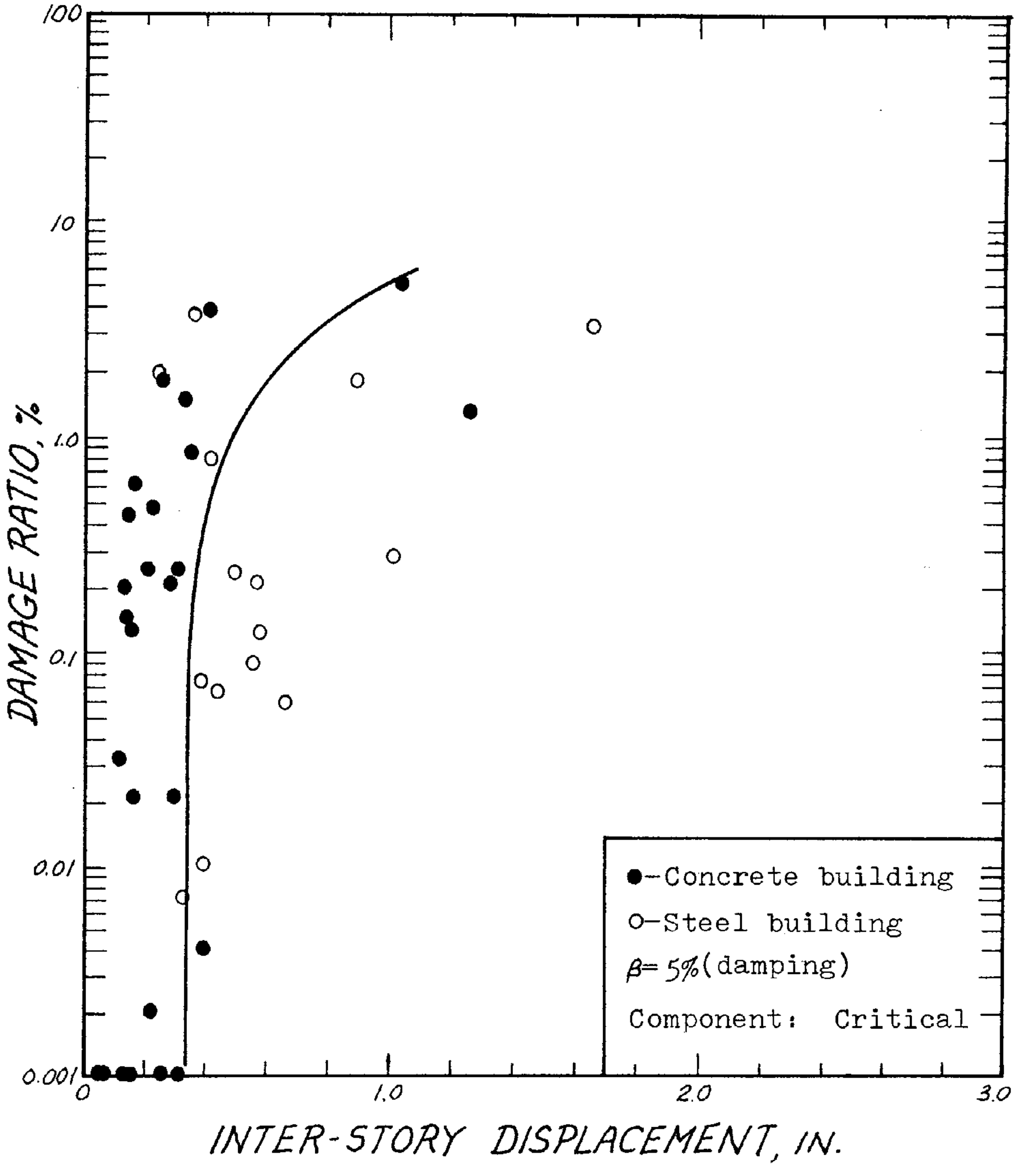


FIGURE 4.5a Correlation of Inter-story Displacement with Damage Ratio

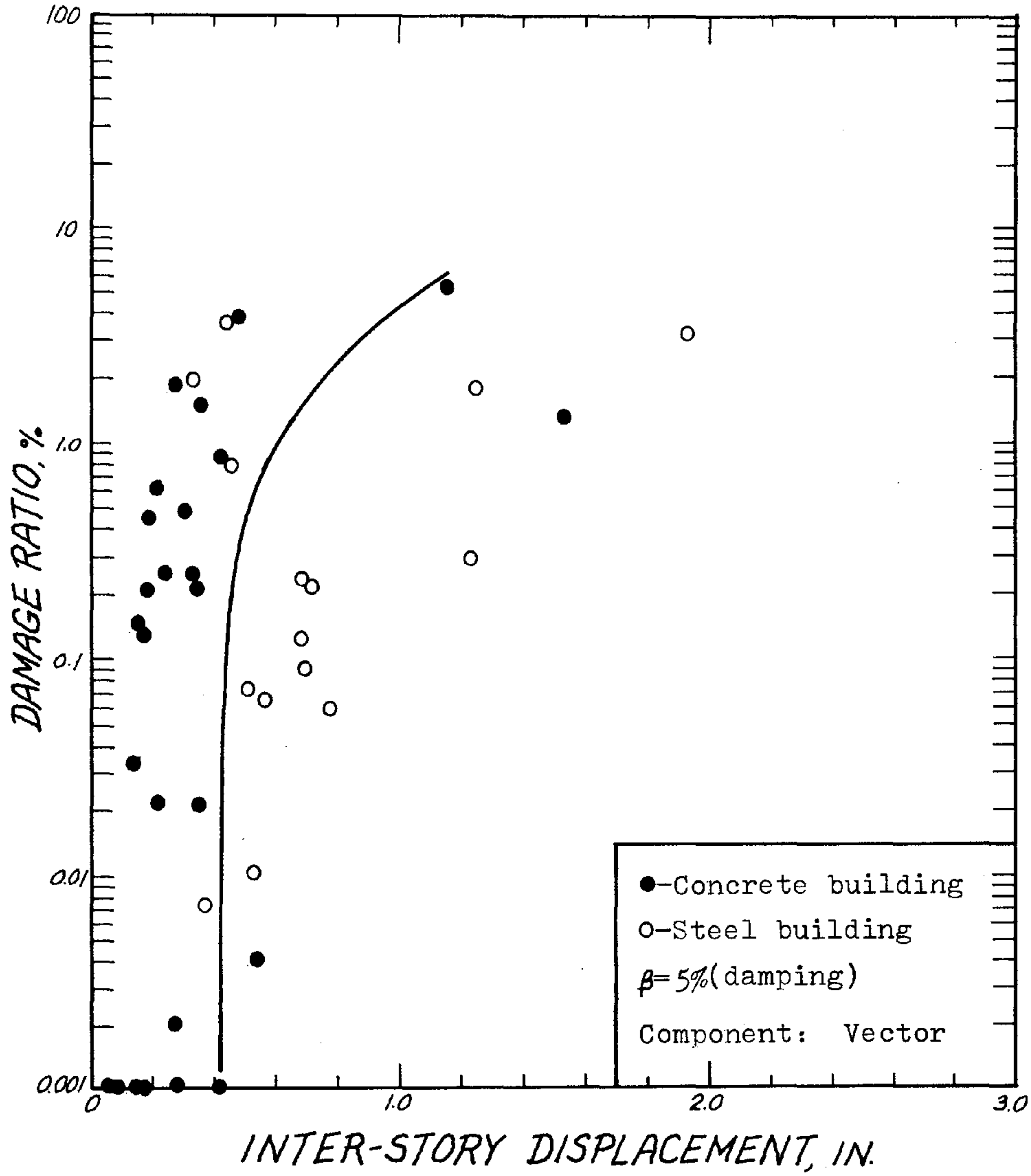


FIGURE 4.5b Correlation of Inter-story Displacement with Damage Ratio

best analytical treatment for the data was to perform regression analysis in the arithmatic domain for a second order polynomial. Accordingly, the general relationship between damage and building response can be expressed by

$$S = A + B(DR) + C(DR)^2 \qquad (1)$$

where S is a building response parameter, DR is a building damage parameter (damage ratio), and A,B, and C are constants computed from the observed data. The constants A,B, and C are evaluated by regression analysis using the least square method performed on the observed data in the arithmatic domain. The resulting constants are the coefficients for the best-fit second order polynomial curve for the observed data.

By the application of this regression method, there was no logical motion-damage relationship found for relative displacement and it was concluded that this parameter was not an indicator of damage on the basis of the observed data and the regression technique used. Although inter-story displacement was also found to be an indicator of building damage, it is not as readily available in earthquake engineering unless some very stringent assumptions are made or some very elaborate computations are performed. Therefore, it was decided to investigate only the two parameters, spectral velocity and spectral acceleration, further in the second part of this study, where more severely damaged buildings are included. The plots in Figures 4.3 to 4.5 revealed no significant difference between the critical component of response and the vector component of response, except in general the response values of the vector component is larger than the critical component. Hence, it was decided to investigate both the critical component and the vector component of response further in the next section.

4.4 MOTION-DAMAGE CORRELATION, INCLUDING SEVERLY DAMAGED BUILDINGS

In general, the same basic analysis techniques as described in Section 4.2 was used for the severly damaged buildings in relation to the buildings studied in the last section, except there are some slight variations in obtaining the building response curves and the fundamental building periods. In all cases, building response values for these buildings estimated either from derived ground motion records or took the average of several records. For example, the response values for the Holy Cross Hospital and the

Indian Hill Medical Center are determined from the derived seismoscope record at the east abutment of the lower Van Norman Dam (Scott, 1973), that for the Olive View Hospital is obtained from the average of three records (Pacoima Dam record, Holiday Inn record, and derived seismoscope record), and that for the Banco Central and Banco America buildings is obtained from the estimated record from aftershock measurements (Hays, 1973). The fundamental building periods obtained from elastic dynamic analysis or results from after-earthquake period measurements (Table 3.2 and 3.5) were used as the pre-earthquake fundamental building period. This assumption is reasonable, since the calculated period or the after-earthquake period could have been only somewhat shorter than the actual measured pre-earthquake building period. The results obtained from the above analysis procedures and some other pertinent information are tabulated in Table 4.1. Correlations of these data along with data from the previous section are shown in Figure 4.6 and 4.7. In cases where more than one ground motion record is used (Olive View Hospital) or where the uncertainty for the estimated record can be evaluated (Banco Central and Banco America buildings) a range of spectral values are plotted along with their mean values. This is only an attempt to differentiate the higher level of uncertainties for these

TABLE 4.1 - SPECTRAL RESPONSE VALUES OF HEAVILY DAMAGED BUILDINGS

Building Name	Damage Ratio,%	Direct.	Bldg. Period, sec pre	Bldg. Period, sec during*	Response S_v, in/sec	Vector S_v, in/sec	Response S_a, g	Vector S_a, g
Holy Cross Hospital	26	N-S	0.65	1.06	38	59	0.70	0.98
		E-W	0.80	1.30	45		0.68	
Indian Hill Medical Center	48	N-S	1.25	2.03	80	92	0.77	0.89
		E-W	1.25	2.03	45		0.44	
Olive View Hospital	100	N-S	0.60**	0.97	44	55	0.88	1.12
		E-W	0.60	0.97	34		0.69	
Banco Central	38	N-S	1.92	3.12	35	48	0.22	0.30
		E-W	2.0	3.25	33	48	0.20	
Banco America	33	N-S	1.82	2.96	42	51	0.28	0.32
		E-W	2.10	3.41	28		0.16	

*Derived during-earthquake periods, using average period extrapolation ratio from Table 3.1 (RC--pre to during period ratio=0.616).

**Assume period to be equal to the N-S direction of the Olive View Hospital main building (building is symmetrical).

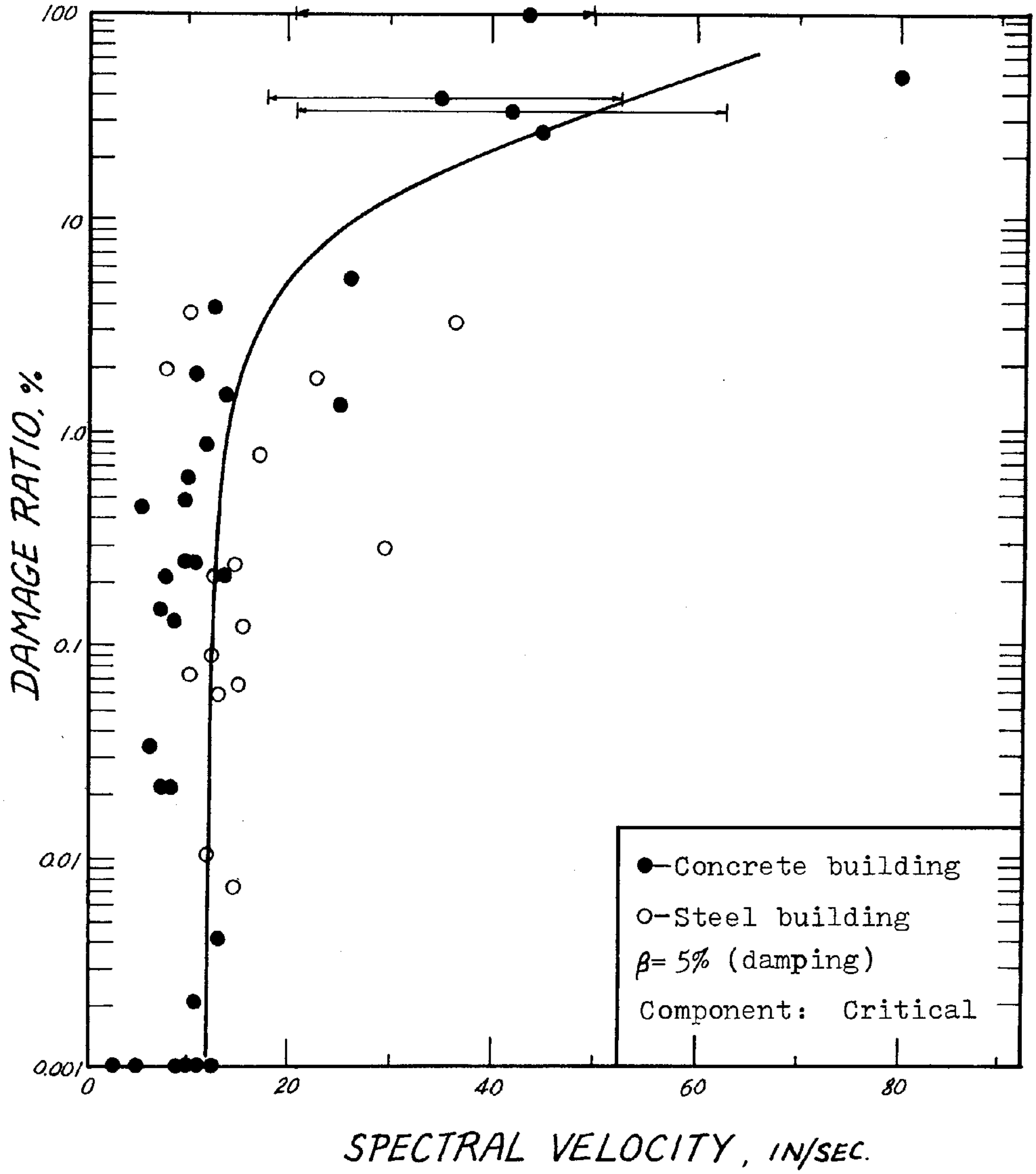


FIGURE 4.6a Correlation of Spectral Velocity with Damage Ratio

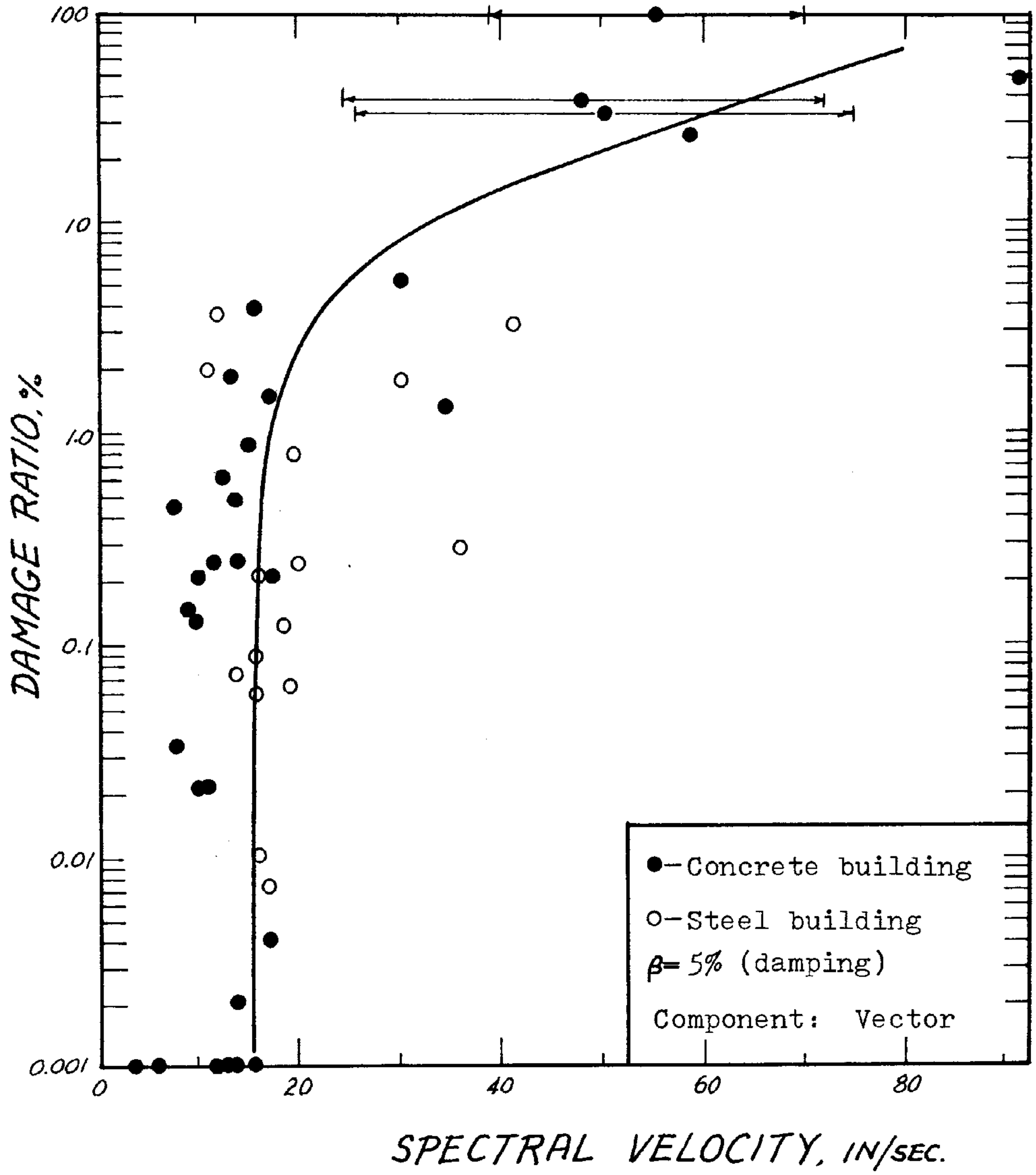


FIGURE 4.6b Correlation of Spectral Velocity with Damage Ratio

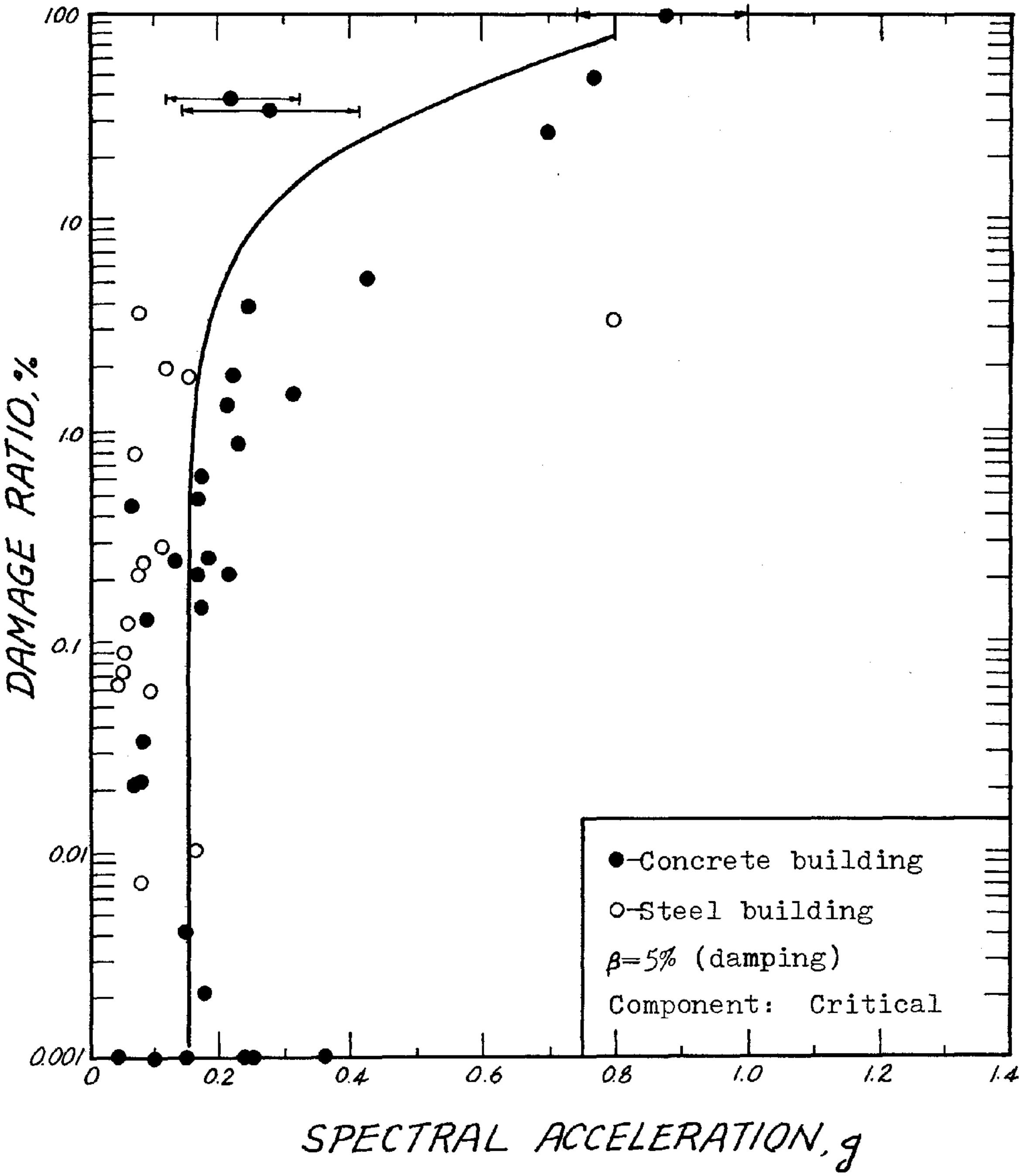


FIGURE 4.7a Correlation of Spectral Acceleration with Damage Ratio

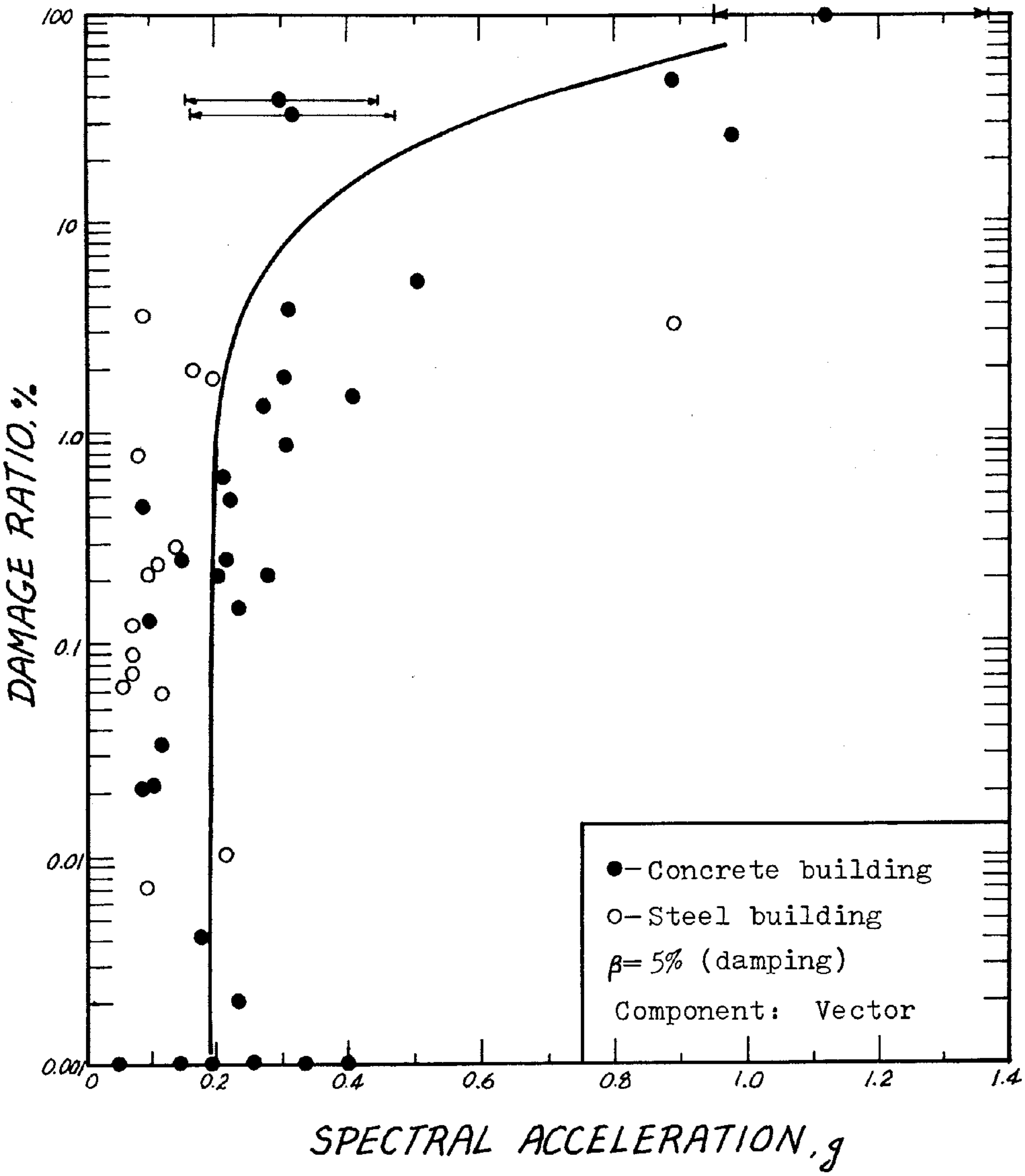


FIGURE 4.7b Correlation of Spectral Acceleration with Damage Ratio

observations relative to the other, not an implication of the overall uncertainty for these observed data.

It is interesting to note that except for the two data points from the Managua earthquake which have substantially large damage with relatively low spectral response values, the overall correlation for this study is consistant with the best-fit mean curve of the form shown in equation (1). The substantial difference between the Managua buildings relative to the mean damage curve as shown for the parameter, spectral acceleration, can be accounted on the basis that the actual pre-earthquake periods for these two buildings might be significantly shorter than the measured post-earthquake period, contrary to the assumption made in this study. Since there is a very steep slope in the response spectrum (Figure 3.6) at the particular range of periods considered, a slightly shorter period will increase the spectral acceleration response drastically in this period range. If this reasoning is correct, response values equal to or even higher than those for the other severly damaged buildings can be anticipated.

It is worthwhile to demonstrate the validity of the motion-damage relationships derived from this study effort. One early useful damage criteria, established by Crandell (1949) for low-rise buildings was that for well-built residential construction, damage would not occur for peak ground velocities less than 3.3 in/sec. Duvall and Fogelson (1962) reviewed damage data and criteria established by several researchers and developed a damage criterion of 2.0 in/sec (peak ground velocity) as the threshold damage for low-rise buildings.[27] In general, spectral velocity is significantly higher than peak ground velocity for the same ground motion (Dowding, 1971), the amplification factors can be from 2.5 to 5.5 times for periods of 0.1 to 0.05 (5 to 15 inches/sec) seconds, respectively. This value of spectral velocity for the damage threshold of low-rise buildings is in close agreement with the damage threshold for high-rise buildings as indicated in Figures 4.6a and 4.6b.

Chapter 5. CONCLUSIONS

In course of this study, a motion-damage relationship was developed by correlating various building response parameters to damage. Study of the observed data revealed that relative displacement was not an indicator of building damage for earthquake ground motion. On the other hand, spectral velocity and spectral acceleration appear to be good damage indicators. Correlations between the expected increase in damage and increasing building response as measured by these two parameters are very obvious. Although the parameter, inter-story displacement, demonstrates a similar trend between building response and damage as spectral velocity and spectral acceleration, it was not considered in the final developments of motion-damage relationships. The drawback of using this parameter comes from requiring stringent approximations or elaborate computation procedures in order to obtain such values for inter-story displacement.

From the mathematical standpoint, none of the correlations can be considered exact. However, considering the level of uncertainty involved and the limited number of data available for this study, any more-elaborate formulation was unjustified. Another reason for the use of a simple analysis technique was the consideration of the trade-offs between sophistication of analysis and the need to keep a reasonable sample size of data.

This study effort presents a first attempt to develop motion-damage relationships for tall buildings. Since little information is currently availab for the development of such relationships, it would be unrealistic to expect perfect agreement in the correlations. However, it is apparent that the data presented does demonstrate some general trends of building damage for the two parameters (spectral velocity and spectral acceleration) at a given seismic design level. The erratic scatter which was observed in the motion-damage correlations may be partly explained on the basis that damage to tall buildings is more often influenced by the details of the design than is by the strength of the structural frame. Hence, designing various buildings for the same seismic design level using the same gross structural elements without control over the details does not necessarily provide the same level of building damage.

The motion-damage relationships developed by statistical correlations from a small data sample may not have a high confidence level. However, the validity of the relationships derived on this basis was demonstrated to be in close agreement with motion-damage criteria established for low-rise buildings in the low-damage region. Moreover, the intent of this study was to develop relationships for the estimation of mean damage values rather than deterministic quantities.

It is significant to point out a serious limitation for the motion-damage relationships developed in this report. Recognizing the fact that a thorough repair of a severely damaged building may result in a repair cost which can be several times more than the initial value of the building, the damage relationships presented are only valid up to the point where the repair cost of the building equals a fraction of the replacement cost of the building. At large damage, there is the tendency for the building owners to replace the whole building rather than to initiate an elaborate repair program. The cut-off level for this decision is very dependent upon the benefit and cost of the project and on how much money the building owner is willing to spend.

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APPENDED COMMENTS BY PRINCIPAL INVESTIGATOR

page 55: The average response was computed as a simple arithmetic average over the range of periods (see figure on p. 56), using spectral ordinates computed for equal intervals between periods.

page 60: The participation factor for a structure did not enter into the evaluation of the relative displacement (spectral displacement), spectral velocity or spectral acceleration; these values come directly from the response spectrum. The participation factor did enter into the evaluation of the interstory displacement; hence the assumptions concerning mode slope and mass distribution.

page 71: The spectral ordinates were average from 0.9 times the pre-earthquake period to 1.1/0.616(RC) or 1.1/0.69(steel) times the pre-earthquake period. The numbers in the denominator come from the last line on p.24.

page 73: The average curves on this and subsequent figures are not the same as those on the corresponding earlier figures. That is, a new regression analysis has been made including the effect of the additional buildings.
The coefficients from the regression analyses (see Eq. 1 on p. 69) are as given in the following tables.

Figure No.	A	B	C
4.3a	11.70	2.726	-0.0824
4.3b	14.69	3.868	-0.3090
4.4a	0.1019	0.0880	-0.00542
4.4b	0.1290	0.1116	-0.00867
4.5a	0.3426	0.1500	-0.00603
4.5b	0.4233	0.1971	-0.01436
4.6a	12.38	1.538	-0.0121
4.6b	15.58	1.861	-0.0145
4.7a	0.1510	0.0119	-0.00047
4.7b	0.1890	0.0147	-0.00056

In computing these coefficients, the buildings having zero damage were excluded. Coefficients were also computed considering the buildings with zero damage, but these results are not reported here. In all cases, the regression minimized the quantity

$$o = \left\{ \frac{1}{n-2} \sum_{i=1}^{n} \left\{ S_i - [A + B(DR)_i + C(DR_i)^2] \right\}^2 \right\}^{1/2}$$