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DAMAGE OF REINFORCED CONCRETE BUILDINGS FROM THE 1985 MEXICO EARTHQUAKE

HAJIME UMEMURA[1]
SHUNSUKE OTANI[2]

ABSTRACT

The 1985 Mexico Earthquake caused a significant damage to buildings in Mexico City approximately 400 km away from the epicenter. Reliable statistics on damaged and undamaged buildings over the selected metropolitan areas were established by Architectural Institute of Japan Investigation Team with an objective to identify the caracteristics and causes of damage. The damage rank of each building was determined by external appearance in accordance with uniform criteria.

A series of single-degree-of-freedom nonlinear earthquake response analyses were carried out to examine the ductility demand of the structure, designed under the Construction Regulations for the Federal District of Mexico (1977) and the 1985 Emergency Regulations, using the earthquake motions observed in Mexico City. Eight strong ground motion records measured in the firm ground, transition, and lake bed zones were analyzed to correlate the observed damage and the calculated response.

[1]Professor Emeritus, Faculty of Engineering, University of Tokyo.
[2]Associate Professor, Department of Architecture, Faculty of Engineering, University of Tokyo.

Introducción

An earthquake of magnitude 8.1 occurred on the Mexican west coast on September 19, 1985, followed by a large after shock of magnitude 7.5 on September 21. The two successive events caused a significant damage to mid to high-rise buildings in Mexico City approximately 400 km away from the epicenter; the severe damage in such a distant area was attributed to the magnification of ground motion by soft and deep soil deposit underlain in the Mexico Valley. Eleven strong motion stations recorded the acceleration waveforms in the two horizontal and one vertical directions in Mexico City and its outskirts by Instituto de Ingeniería, Universidad Nacional Autónoma de México (1, 2, 3, 4, 5).

Many Japanese researchers and engineers investigated the damage, and the Architectural Institute of Japan (AIJ) published a comprehensive report (6) in 1987. This paper introduces the statistical data on damaged buildings in Mexico City from the AIJ report. The data were gathered by a two-member Ohbayashi-gumi Research Institute team in the middle of October, 1985 (7), and by a 42-member AIJ team (Leader: Professor Yoshikazu Kanoh of Meiji University) in early November, 1985.

Nonlinear earthquake response analysis was carried out for a series of single-degree-of-freedom (SDF) systems to describe the observed damage statistics. The strength characteristics of the systems were determined by the existing building code at the time of the earthquake. The effect of the revised building code is also discussed by the analysis.

Damage investigation

After the disaster of the 1985 Mexico Earthquake, it was felt important to establish reliable statistics on damaged and undamaged buildings over the entire metropolitan area of Mexico City in order to understand the performance of the buildings of different types and constructions. Such data are useful to establish the earthquake resistant measures against future earthquakes. The investigation was conducted one to two months after the earthquake when some buildings had already been removed; the damage could not be identified in those buildings.

Area for Study

An investigation (8) inmediately after the earthquake reported severe damage in the lake bed zone (Fig. 1). Therefore, the AIJ investigation team decided to concentrate the efforts in selected areas in the lake bed zone within the limit of time and man power. A group of two to three experien-

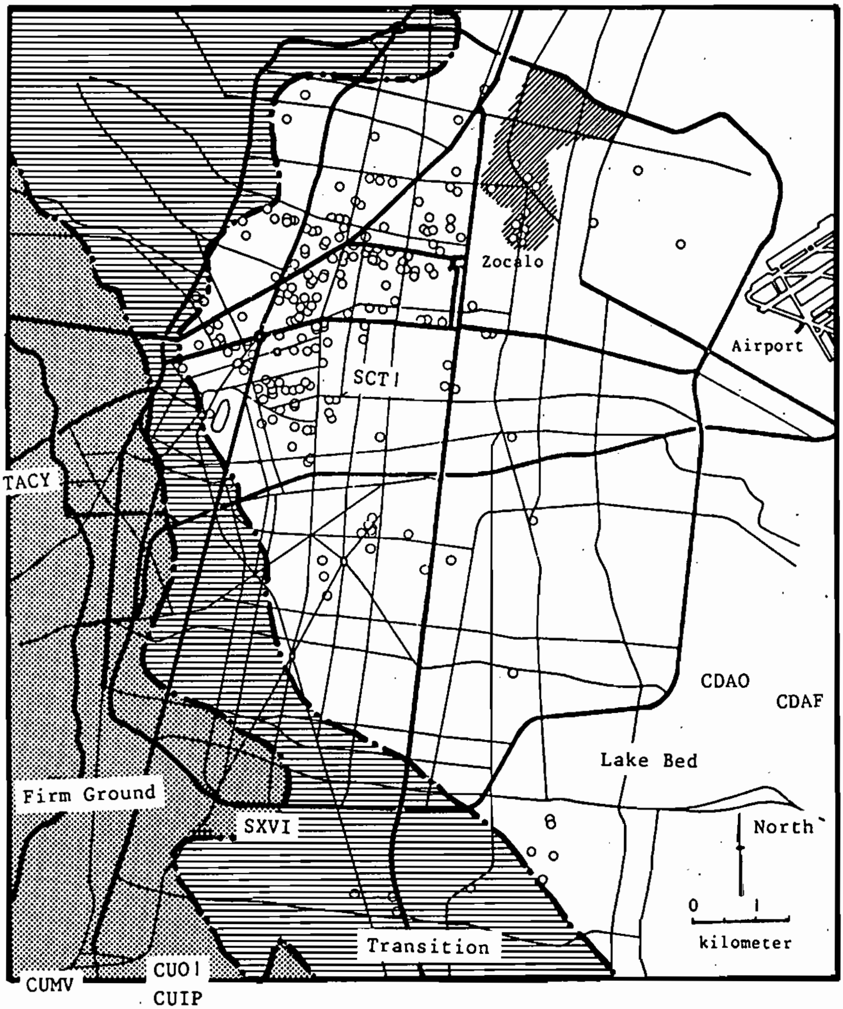


Fig. 1. Seismic Zoning, Severely Damaged Buildings, and Earthquake Recording Stations.

ced structural engineers and researchers covered each area, where the team went through every alley and surveyed the damage of each building from the external appearance. This investigation may be called a damage inventory survey.

Construction Types

The construction was classified into three types; masonry (M), reinforced concrete (RC), and steel construction (S). It was often difficult to distinguish the reinforced concrete and masonry constructions, especially of low-rise buildings (approximately 5 stories or lower); sometimes the two construction methods were mixed in a single building. The number of steel buildings was very small compared with reinforced concrete and masonry buildings. The numbers of masonry and reinforced concrete buildings were comparable, but the reinforced concrete construction was used in taller buildings.

The number of stories was identified easily from the external observation. However, in case of collapsed buildings in a stacked pancake manner, the number of slabs was counted to identify the number of stories. The largest number of stories was used for a set-backed building.

Damage Ranks

The AJI team classified the damage in accordance with guiding criteria given in Table 1, in which the damage was classified into six ranks; i.e., 1) light and no damage, 2) minor damage, 3) medium damage, 4) major damage, 5) partial collapse, and 6) total collapse. The criteria were originally developed to classify the damage after the 1978 Miyagi-kenoki Earthquake (9). After discussion on the applicability of the criteria in Mexico City, rank 5 (Partial collapse) was introduced. When the building showed inclination, the criteria in Table 2 were used on the basis of the experience from the 1964 Niigata Earthquake.

The Ohbayashi-gumi team (7) classified the damage into five levels; i.e., A) total collapse, B) partial collapse, C) large deformation or large cracking, D) small cracking or damage on window glasses, E) no external damage or minor damage on non-structural elements. For the damage statistics, the following equivalence was assumed;

AIJ Level ½: Ohbayashi-gumi Level D/E

AIJ Level ¼: Ohbayashi-gumi Level C

AIJ Level 5: Ohbayashi-gumi Level B

AIJ Level 6: Ohbayashi-gumi Level A

Note that the external observation tends to underestime the damage; e.g., severely damaged stiff architectural elements were often hidden inside.

TABLE 1

CLASSIFICATION OF DAMAGE LEVEL

(9)

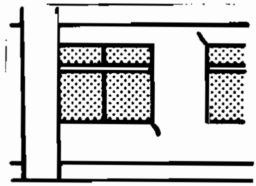
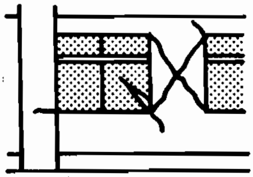
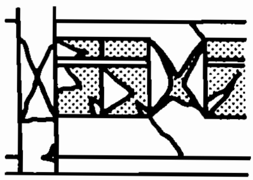
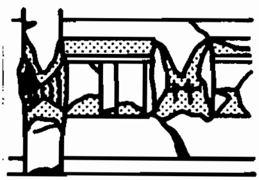
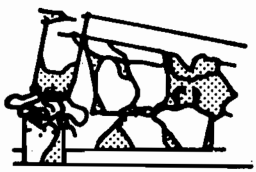
Rank	Description	Sketch
1 (Light)	Very light or no damage to columns and shear walls.	
2 (Minor)	Light damage on columns and walls, shear cracks on RC non-structural walls.	
3 (Medium)	Shear or flexural cracks on columns, appreciable damage on non-structural walls.	
4 (Major)	Reinforcement exposed and buckled in columns, large shear cracks in shear walls.	
5 (Partial)	Significant damage on columns and shear walls Collapse, and a part of the building collapsed.	
6 (Total)	Significant damage on columns and shear walls, Collapse, and the entire building collapsed.	

TABLE 2
DAMAGE RANK FOR TILTED BUILDINGS

Damage Level	Inclination Degrees	Comment
2	< 1.0	Minor damage
3	< 2.5	Medium damage
4	> 2.5	Major damage
6	Overturned	Collapse

RESULTS OF DAMAGE INVESTIGATION

The results of the inventory survey are presented here for all three types of buildings.

Area 1

An AIJ team (S. Otani, T. Takahashi, M. Sakamoto and N. Izumi) surveyed the area (Fig. 2) between Av. Insurgentes Norte and Guerrero, south of Calz. Nonoalco and north of Puente de Alvarado, on November 5, 1985. The results are listed in Table 3. The depth to second hard soil layer was approximately 38 to 42 m (8).

There were many low-rise masonry residential buildings, mid-rise masonry apartment buildings, and some factory buildings. No steel construction was found, 67 reinforced concrete buildings were surveyed mostly of 3 to 5 stories; and the rest are masonry buildings. The tallest reinforced concrete building was of 8 stories high. Note that the number of single-story buildings was only one quarter of the number of two-story buildings.

Less than 1.6 percent of the buildings surveyed suffered medium or severer damage; 7.4 percent minor or greater damage. No or light damage was observed in 93 percent of the buildings. The damage rate was very light.

The damage to reinforced concrete buildings was very light; i.e., one eight-story and one four-story buildings out of 67 reinforced concrete buildings suffered medium damage. The three- to five-story buildings suffered minor damage.

Two hundreds and forty two masonry buildings were surveyed. Most masonry buildings were less than five stories high. Three three-story masonry buildings suffered major damage, but the other suffered minor or light damage. The percentage of buildings suffered minor or greater

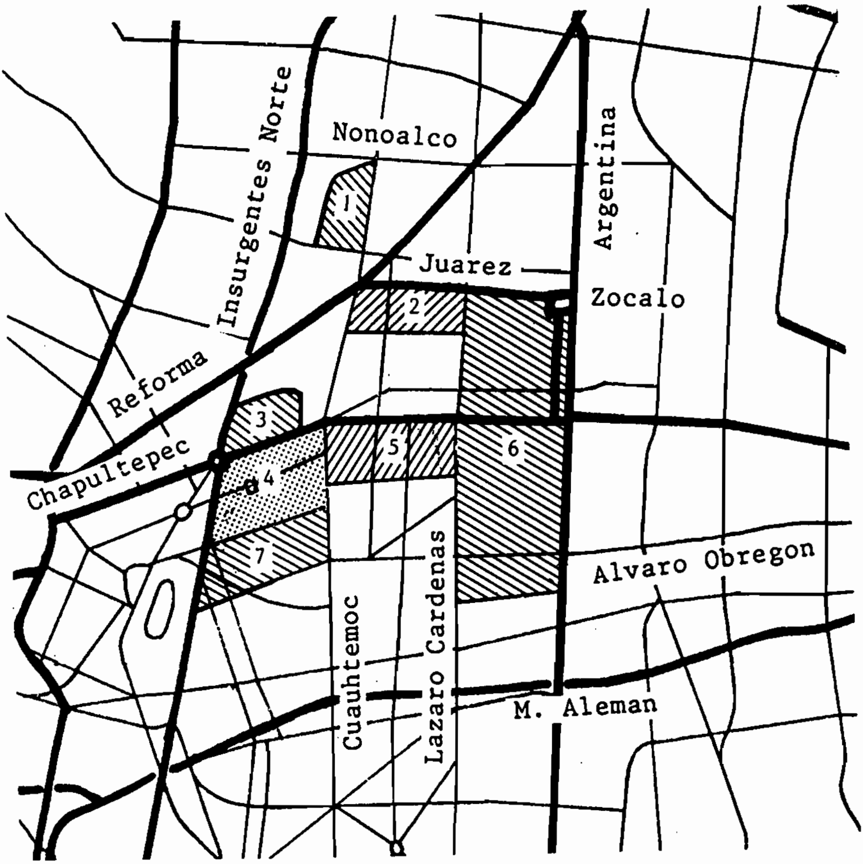


Fig. 2. Areas of Damage Inventory Survey.

damage was only 5 percent, and that suffered medium or more damage was 1.2 percent. The damage on masonry buildings was also small.

Area 2

An Ohbayashi-gumi Research Institute team (Y. Omote and H.Katsumata) surveyed the area (Fig. 2) south of Arameda Park, bounded by Bucareli, Av. Juarez, Central Lazaro Cardenas, and Arcos de Belen, approximately 0.4 km^2, from October 16-30, 1985. The northern one-half of the area is commercial district, in which there were many mid— to high-rise buildings. The percentage of buildings less than 3 stories high was only 39

TABLE 3

DAMAGED BUILDINGS IN AREA 1 (AIJ TEAM)

Nº of Stories	Damage Rank						
	1	2	3	4	5	6	Total
1	44						44
2	128	6		3			137
3	67	7					74
4	24	2	1				27
5	20	3					23
6	2						2
7	1						1
8			1				1
Total	286	18	2	3	0	0	309

percent, the ratio which was smaller than in the other areas. The tallest buildings was of 20 stories. The depth to second hard soil layer was approximately 42 to 46 m (8).

A total of 303 buildings were investigated (Table 4). The survey was more comprehensive between Av. Balderas and Central Lazaro Cardenas. Some heavily damaged buildings were observed along Independencia. The damage to buildings of less than five stories was very light. However, approximately 15 percent of 7-top 9-story buildings collapsed, and 27 percent suffered medium to severe damage. The ratio of damaged buildings was higher for tall buildings.

Four old masonry buildings, located closely together collapsed partially. Damage ratio (the ratio of buildings suffered medium and heavier damage to the existing buildings) of masonry construction was 10.3 percent.

Area 3

An AIJ team (Y. Kanoh, S. Otani, T. Takahashi, M. Sakamoto, and N. Izumi) surveyed the area (Fig. 2), bounded by Insurgentes Sur, Av. Cuauhtemoc Eje 1 Poniente, Versalles and Roma, on November 6 and 9, 1985. The survey was carried out along Liverpool, Versalles, Tulin, Niza, Av. Chapultepec. The other streets were not covered in this survey. Sixty four reinforced concrete buildings, fifty five masonry buildings, and

TABLE 4
DAMAGED BUILDINGS IN AREA 2
(OHBAYASHI-GUMI TEAM)

N° of Stories	Damage Rank				
	1/2	3/4	5	6	Total
1	9	1			10
2	60				60
3	48				48
4	46				46
5	55			2	57
6	20		2		22
7	12		1	1	14
8	3	3			6
9	9	1	2	1	13
10	6			1	7
11	5	3			8
12	2	1	1		4
13	1	1			2
14	1	1			2
15	1				1
16					0
17		1	1		2
20	1				1
Total	279	12	7	5	303

three steel constructions were investigated (Table 5). Five buildings were already demolished and removed from the sites. The depth to second hard soil layer was approximately 38 to 42 m (8).

Ten low-rise buildings of 3-6 stories collapsed, including a seven-story school, a six-story office, and a four-story office. Including those already demolished, 15 out of 121 buildings collapsed, the ratio which was significant.

Area 4

The Ohbayashi-gumi Research Institute team (Y. Omote and H. Katsumata) surveyed the area (Fig. 2) bounded by Av. Cuauhtemoc Eje 1

TABLE 5
DAMAGED BUILDINGS IN AREA 3 (AIJ TEAM)

Nº of Stories	Damage Rank						
	1	2	3	4	5	6	Total
1	7						7
2	22	12					34
3	16	2		1	2	1	22
4	8	7			5		20
5	11	1					12
6	3	1				1	5
7	1		1				2
8	1	1					2
9		1		2			3
10		1					1
11		1	1				2
12				1			1
13	2						2
14							0
15							0
16		1					1
17							0
18		1					1
unknown						5	5
Total	71	29	2	4	7	8	121

Poniente, Av. Chapultepec, Av. Insurgentes Sur, and Av. Alvaro Obregon, from October 16-30, 1985. An AIJ team (S. Otani, T. Takahashi, M. Sakamoto, and N. Izumi) also surveyed the area on November 6 and 7, 1985. The investigation by the AJI team was less extensive because the team placed more emphasis on detailed investigation of severely damaged buildings. However, it may be of interest to compare the results of the two surveys. The damage is summarized in Table 6 for the Ohbayashi-gumi team and in Table 7 for the AIJ team. The depth to second hard soil layer was approximately 38 to 46 m (8).

More than one half of the buildings surveyed were from 1 to 3 stories. The area is dominantly residential. High-rise buildings were concentrated in limited blocks. The damage was severe, and the damaged buildings

TABLE 6
DAMAGED BUILDINGD IN AREA 4
(OHBAYASHI-GUMI TEAM)

Nº of Stories	Damage Rank				
	1/2	3/4	5	6	Total
1	19				19
2	141			1	142
3	119	1	2		122
4	73		1		74
5	38	2	2	1	43
6	21		1		22
7	8	3	2	1	14
8	1	3			4
9	3	2		3	8
10		1	1		2
11	2	3			5
12	1	2		1	4
13					0
14	1				1
15	1				1
16					0
17	1				1
unknown				1	1
Total	429	17	9	8	463

were scattered in the area. A total of 463 buildings were surveyed by the Ohbayashi-gumi team. Although the area is immediately adjacent to Area 7, a severe damage was observed in 7 to 13 story buildings.

Six-to eight-story reinforced concrete buildings settled more than 1.0 m near the corner of Durango and Merida. More than one half of masonry construction were single-storied. Damage ratio of masonry buildings was 4.5. percent.

Area 5

An AIJ team (T. Endo, F. Watanabe, S. Hayama and J. Fukushima) surveyed the are (Fig. 2) bounded by Av. Cuauhtemoc Eje 1 Poniente, Dr.

TABLE 7
DAMAGED BUILDINGS IN AREA 4 (AIJ TEAM)

N° of Stories	Damage Rank						
	1	2	3	4	5	6	Total
1	7			1			8
2	103	19	1		2	1	126
3	44	10	1				55
4	37	8	1				46
5	28	9	1			1	39
6	10	5	1	1			17
7	6	2	2	2	1		13
8	1	1	1	1			4
9	1	2		1			4
10							0
11	1	1		1			3
12		2	1				3
13		2					2
14							0
15							0
16			1				1
17							0
18		1	1				2
Total	238	62	11	7	3	2	323

Rio de la Loza, Eje Central Lazaro Cardenas, and Dr. J. Navarro, approximately 0.6 km^2, on November 5, 1985. Mid- to high-rise office buildings and old low-rise apartment buildings and new low to mid-rise apartment buildings were mixed in this area. Almost one-half of 203 buildings surveyed were of single- or two story; the number of high-rise buildings was small. The depth to second hard soil layer was approximately 42 to 46 m (8).

The survey results are summarized in Table 8. Eleven buildings collapsed out of 203 buildings. More than one-half of the buildings suffered minor or severer damage. It should be noted that all buildings of more than eight stories suffered medium or severer damage; the damage was relatively high. The number of buildings between 7 to 9 stories was

TABLE 8
DAMAGED BUILDINGS IN AREA 5 (AIJ TEAM)

N° of Stories	Damage Rank 1	2	3	4	5	6	Total
1	28	10	1			1	40
2	33	14	4			1	52
3	18	10	2	3			33
4	7	7	2	2		1	19
5	10	12	2				24
6	1	5			1		7
7	3		2	2	2	2	11
8							0
9				2	1	1	4
10			3	1	1		5
11							0
12			3	1			4
13							0
14							0
15			4				4
Total	100	58	23	11	5	6	203

small, but 40 percent of the buildings (6 buildings) in this category collapsed. Some severe damage was caused by the pounding of adjacent buildings. A large-span buildings such as movie theaters also suffered damage. The soil conditions were poor and the settlement of the foundation was observed.

Area 6

An AIJ team (R. Tanaka, I. Shiraishi, M. Yanagisawa, K. Taga, and M. Fujimura) surveyed the area (Fig. 2) between Lazaro Cardenas and Pino Suarez, approximately 1.0 km wide, and from the south of Zocalo to Fr. Y.J. de Torquemada, approximately 2.8 km long. There were relatively old buildings of two to three stories. Approximately 80 percent of the buildings were of less than 3 stories high, mostly of masonry construction. There were also high-rise buildings taller than of 10 stories. The total number of buildings surveyed was 2,536. Table 9 summarizes the damage

TABLE 9
DAMAGED BUILDINGS IN AREA 6 (AIJ TEAM)

N° of Stories	Damage Rank 1	2	3	4	5	6	Total
1	523	38	9	2			572
2	903	90	34	6	5		1.038
3	353	43	13	6	3	5	423
4	191	12	5	3	2		213
5	119	8	4		2	1	134
6	51	7	1	1			60
7	23	5	3			4	35
8	17	2	1		1		21
9	6	1	1	2		2	12
10	3	2	1		3		9
11	2	1		1			4
12			3			1	4
13			4	1			5
14			1				1
15		2	1				3
16				1	1		2
Total	2,191	211	81	23	17	13	2,536

rank and the number of stories. The depth to second hard soil layer was approximately 46 to 50 m (8).

No to light damage was observed in 86 percent of the buildings surveyed, and only 1.1. percent of the buildings collapsed (damage ranks 5 and 6). The damage rate was light. Four percent of buildings of less than four stories suffered medium and heavier damage, while the ratio increased to 64.3 percent for buildings of more than nine-stories. The ratio of damaged buildings clearly increased with the number of stories, and the buildings taller than 10 stories suffered severe damage.

Area 7

An AIJ team (T. Takahashi and M. Sakamoto) surveyed the area (Fig. 2) bounded by Av. Cuauhtemoc Eje 1 Poniente, A. Alvaro Obregon, Av. Insurgentes Sur, and Chiapas, approximately 0.85 km^2, on November 8,

TABLE 10

DAMAGED BUILDINGS IN AREA 7 (AIJ TEAM)

N° of Stories	Damage Rank						
	1	2	3	4	5	6	Total
1	55	5	1	3			64
2	475	40	8	5	8	2	538
3	148	13		1	1		163
4	56	20	3	1	1		81
5	60	30	1		2	2	95
6	23	12			2	1	38
7	15	8		1			24
8	6	5	2				13
9	1	3			1		5
10	1	3					4
11		1	1				2
17		1				1	2
Total	840	141	16	11	15	6	1,029

1985. There were mostly low-rise masonry buildings, and few high-rise buildings. More than one half of the buildings were single- or two-story residential buildings. A total of 1,029 buildings were surveyed (Table 10). The depth to second hard soil layer was approximately 38 to 42 m (8).

The trend in damage is similar to Area 6. The buildings with major (rank 4) and severer damage was less than 10 percent except for the buildings of 9 and 14 stories. The damage ratio was relatively light. No to light damage was observed in 85 percent of the buildings of less than four stories. However, the damage increased with the number of stories. Out of 1,029 buildings, 702 buildings were masonry; damage ratio to masonry buildings was 4.3 percent.

DISCUSSION OF OBSERVED DAMAGE

The area surveyed covered slightly more than 20 percent of the central area of the city, the results are summarized in Table 11 and Fig. 3 for all data obtained by the AIJ teams. The number of buildings less than 4

TABLE 11

SUMMARY OF DAMAGED BUILDINGS

N° of	Damage Rank						
Stories	1	2	3	4	5	6	Total
1	664	53	11	6		1	735
2	1,664	181	47	14	15	4	1,925
3	646	85	16	11	6	6	770
4	323	56	12	6	8	1	406
5	248	63	8		4	4	327
6	90	30	2	2	3	2	129
7	49	15	8	5	3	6	86
8	25	9	5	1	1		41
9	8	7	1	7	2	3	28
10	4	6	4	1	4		19
11	3	4	2	2			11
12		2	7	2		1	12
13	2	2	4	1			9
14			1				1
15		2	5				7
16		1	1	1	1		4
17		1				1	2
18		2	1				3
unknown						5	5
Total	3,726	519	135	59	47	34	4,520

stories high was approximately 73 percent of the 4.520 buildings surveyed; the number of buildings decreased with the number of stories.

No to light damage was observed in 82 percent of the all buildings, and in 85.2 percent of the buildings of less than 6 stories. The collapse occurred in 1.8 percent of the buildings surveyed, and in 9.0 percent of the buildings of more than five stories. Note the increase in the percentage of damaged buildings with the number of stories.

The damage was relatively light in low-rise buildings. For a total of 1,245 masonry buildings surveyed, the average damage ratio was 4.2 percent.

The fundamental period of undamaged buildings in Mexico City ranged approximately 0.08 to 0.11 times the number of stories (6). It was

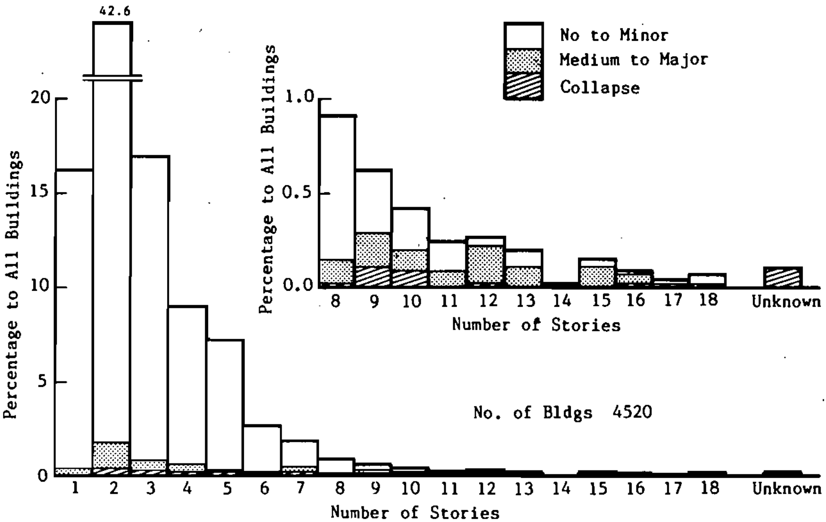


Fig. 3. Summary of Damage Inventory Survey.

pointed out by many researchers that the dominant long period of ground motion in the lake bed zone should be responsible for the higher damage rate in taller buildings.

Nonlinear earthquake response analysis

Objectives

The objectives of the nonlinear earthquake response analysis study were to correlate the observed damage with the characteristics of the ground motions and the structures designed in accordance with the existing building codes in Mexico City.

Analytical Models

A series of single-degree-of-freedom (SDF) systems with nonlinear hysteretic characteristics were designed for yielding periods ranging from 0.1 to 3.0 sec. The resitance of the systems were determined for three soil conditions in Mexico City and three ductility factors in accordance with the 1977 Construction Regulations for the Federal District of Mexico (10) and the 1985 Emergency Regulations (11). The systems were subjected to each horizontal component of the observed earthquake motions recorded in the corresponding earthquake zone.

Earthquake Motions

Eleven earthquake records (Table 12) (1, 2, 3, 4 and 5), each containing two horizontal and one vertical components, were recorded in Mexico City and its outskirts. Eight records (CUOI, CUIP, CUMV, TACY, SXVI, CDAF, CDAO, SCTI) were recorded in Mexico City, one record (SXPU) at Puebla City, two records (TLHB and TLHD) in the Valley of Mexico. The eight recording stations in Mexico City are shown in Fig. 1. Records CUOI, CUIP, CUMV, and TACY were recorded in the firm ground zone, Record SXVI in the transition zone, and Records CDAF, CDAO, SCTI in the lake bed zone.

The absolute acceleration response spectra of the records were shown in Fig. 4 for a damping factor of 0.05. The response spectra of the firm ground records (CUOI, CUIP, CUMV, and TACY) are of low amplitudes over a wide range of periods, while Record SXVI (the tansition zone) exhibited high amplitudes at a period range from 0.5 to 1.0 sec. The response spectra of the lake bed records (CDAF, CDAO, and SCTI) exhibited amplitudes significantly larger than those of the firm ground and transition zone records. Records CDAF and SCTI developed large response at around 2.0 sec period, whereas Record CDAO showed a peak at around 1.3 to 1.5 sec. Note that Record SCTI developed by far largest response amplitudes of the eight records.

The amplitudes or response spectra appear to coincide with the severer damage in Taller (longer-period) buildings demonstrated by the damage inventory survey. However, the response spectra do not consider the strength of buildings; i.e., the buildings did not posesse a uniform resistance.

DESIGN EARTHQUAKE LOADS IN MEXICO CITY

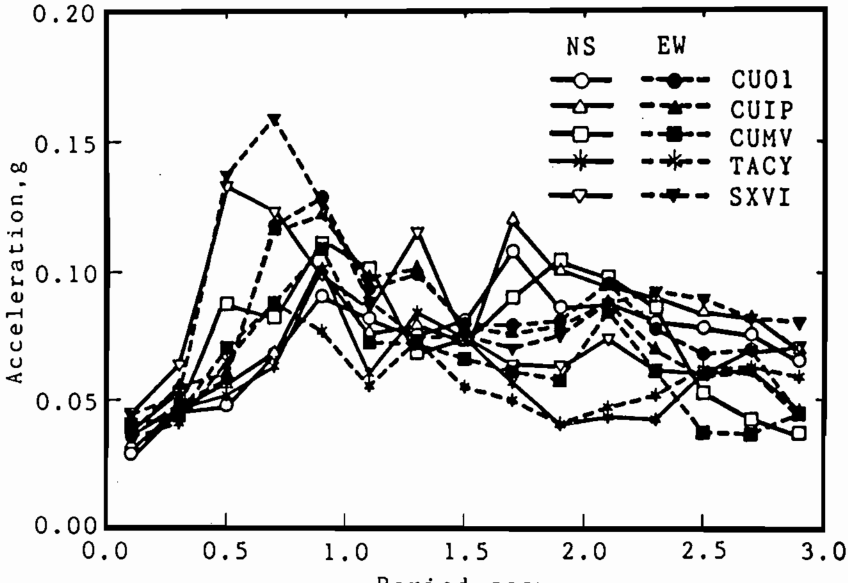
The earthquake resistant design before the 1985 Mexico Earthquake was governed by the Construction Regulations for the Federal District of Mexico 1977 (10). The Emergency Regulations (11) were issued on October 18, 1985, within one month after the earthquake. The design base shear coefficient in the two codes was determined as function of fundamental period T, selected ductility factor Q and spectral parameters C. a_0, r, T_1 and T_2 for the three seismic zones; i.e.,

TABLE 12

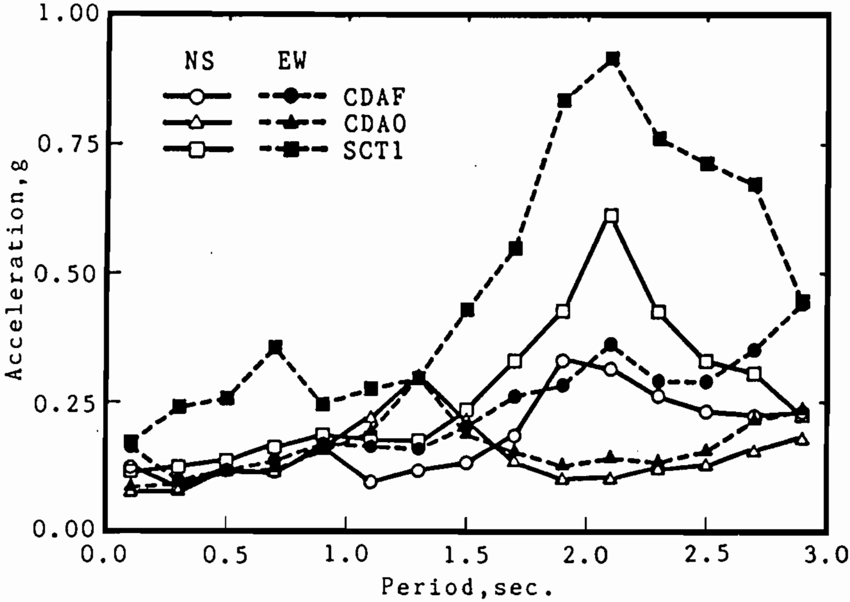
GROUND MOTIONS RECORDED IN MEXICO CITY

(2, 3)

N°	ID	Zone	Dir.	Max. Acc. (cm/s^2)	Max. Vel. (cm/s)	Max. Dspl. (cm)	Note
1	CUO1	1	NS	28.1	10.2	5.5	Rock basalt.
			EW	33.5	9.4	7.2	Building (1st fl.)
2	CUIP	1	NS	31.7	10.3	6.2	Rock basalt.
			EW	34.6	9.4	7.7	Free field.
3	CUMV	1	NS	37.4	9.2	5.7	Rock basalt.
			EW	38.8	11.0	4.5	Shaking table.
4	SXPU		NS	29.6	7.2	3.1	Hard soil.
			EW	32.6	6.6	2.7	Free field.
5	TACY	1	NS	34.4	14.3	12.0	Hard soil.
			EW	33.2	9.8	8.6	Free field.
6	SXVI	2	NS	44.1	11.5	9.1	Soft soil.
			EW	42.4	12.2	7.5	Free field.
7	CDAF	3	NS	80.5	24.8	15.0	Very soft soil.
			EW	94.6	37.6	18.9	Free field.
8	CDAO	3	NS	69.2	35.0	25.0	Very soft soil.
			EW	80.4	41.9	24.7	One-story building.
9	SCT1	3	NS	98.0	38.8	19.1	Very soft soil.
			EW	167.9	60.5	21.9	Free field.
10	TLHB		NS	135.9	64.1	36.6	Soft soil.
			EW	106.7	44.6	39.3	Free field.
11	TLHD		NS	117.7	34.9	20.8	Soft soil.
			EW	111.6	36.1	22.1	Free field.



(a) Records obtained in Firm Ground Zone



(b) Records obtained in Lake Bed Zone

Fig. 4. Absolute Acceleration Response Spectra at 0.05 Damping
a) Records obtained in Firm Ground Zone
b) Records obtained in Lake Bed Zone

$$c = [a_o + (C - a_o)\, T/T_1] \,/\, [1 + (Q - 1) T/T_1] \qquad \text{for} \quad T < T_1$$
$$c = C/Q \qquad \text{for} \quad T_1 < T < T_2$$
$$c = C(T_2/T)^r/Q \qquad \text{for} \quad T_2 < T$$

in which C: maximum response accleration, a_o: maximum ground acceleration, T_1 and T_2 are corner periods. Base shear coefficient c cannot be chosen less than a_o.

The 1977 Construction Regulations

A ductility factor Q of a building could be selected to be 1.0, 1.5, 2.0, 4.0 or 6.0. The importance factor for buildings, which must maintain its function even after a strong earthquake, was introduced to be 1.3. The values of the spectral parameters C, a_o, r, T_1 and T_2 are listed in Table 13. The design base shear coefficient in the three seismic zones are shown in Fig. 5.

The 1985 Emergency Regulations

The spectral parameters C and a_o in Zones 2 and 3 were modified in the 1985 Emergency Regulations (Table 13). General variation of design base coefficient with periods was not altered. The importance factor was raised to 1.5; ductility factor was changed to 1.0, 1.5, 2.0, 3.0 and 4.0. The change increased the design base shear coefficient (Fig. 6).

TABLE 13

DESIGN BASE SHEAR COEFFICIENT

	The 1977 Regulations					The 1985 Code	
Seismic zone	C	a_o	T_1	T_2	r	C	a_o
Zone 1 Firm Ground	0.16	0.03	0.3	0.8	1/2	0.16	0.03
Zone 2 Transition	0.20	0.045	0.5	2.0	2/3	0.27	0.054
Zone 3 Lake Bed	0.24	0.06	0.8	3.3	1	0,40	0.10

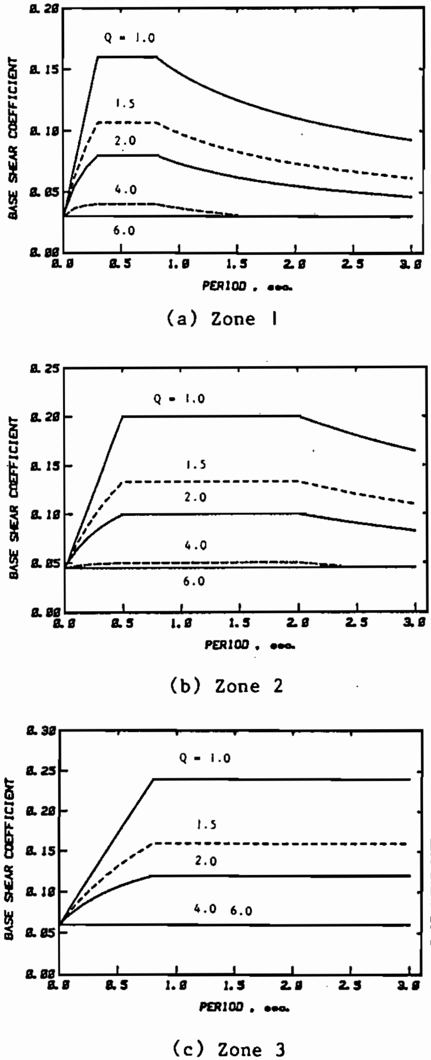


Fig. 5. Design Seismic Load for the 1977 Code.

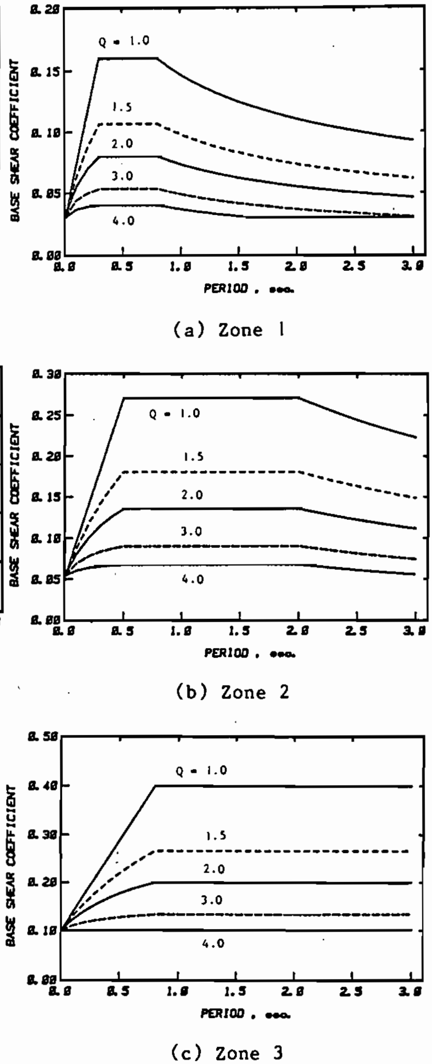


Fig. 6. Design Seismic Load for the 1985 Code.

Properties of SDF systems

Takeda and Takeda-slip models (12, 13) were selected to simulate the response of reinforced concrete buildings, in which Takeda model (Fig.

7) represents a building with a large hysteretic energy dissipation, while Takeda-slip model (Fig. 8) dissipates less energy. The skeleton curve under monotonically increasing load was of tri-linear type with stiffness changes at cracking point (D_c, F_c) and yielding point (D_y, F_y). Fixed relations were used for the cracking and yielding points;

$$F_c = F_y/3$$
$$D_c = D_y/12$$

The period corresponding the secant stiffness at the yielding is two times longer than the initial elastic period. The post-yield stiffness was assumed to be zero. The unloading stiffness degradation parameter, which controls the fatness of a hysteresis loop after yielding, was chosen to be 0.5 for Takeda model (12). The slip stiffness degradation parameter was 1.0, reloading stiffness parameter of Takeda-slip model were selected to be 1.5 and 1.0, respectively (13).

Mass of the systems was assumed to be unity. The yield resistance F_y was determined by the 1977 Construction Regulations or the 1985 Emergency Regulations. The previous study (14) indicated that the inelastic response is not sensitive to the elastic stiffness, the period of the system was selected on the basis of secant stiffness at the yielding. The damping coefficient was assumed to vary proportional to instantaneous stiffness and the value was selected to yield a damping factor of 0.05 at an initial elastic stage.

The base of SDF systems was assumed to be fixed on the infinitely rigid foundation; i.e., the structure-foundation interaction was not included in the analysis. Response computation (14) was carried out by the Newmark-beta method with interations to satisfy both the balance of forces and the hysteretic relation at each time step.

Response of the 1977 Code Systems

A series of SDF systems were designed in accordance with the 1977 Construction Regulations for three typical ductility factors (Q = 1.0, 2.0, 6.0) and for the three seismic zones. The period was varied from 0.1 to 3.0 sec.

Zone I (Firm Ground Zone, Fig. 9)

Nonlinear SDF earthquake response was calculated using Records CV01, CVIP, CVMV, and TACY.

For a design ductility factor of Q = 1.0, the ductility demand by the

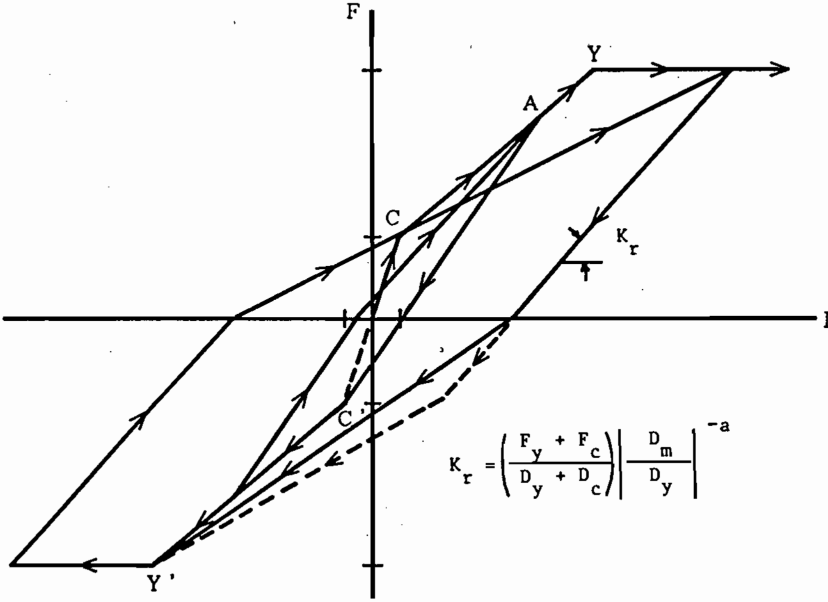


Fig. 7. Takeda Model (12).

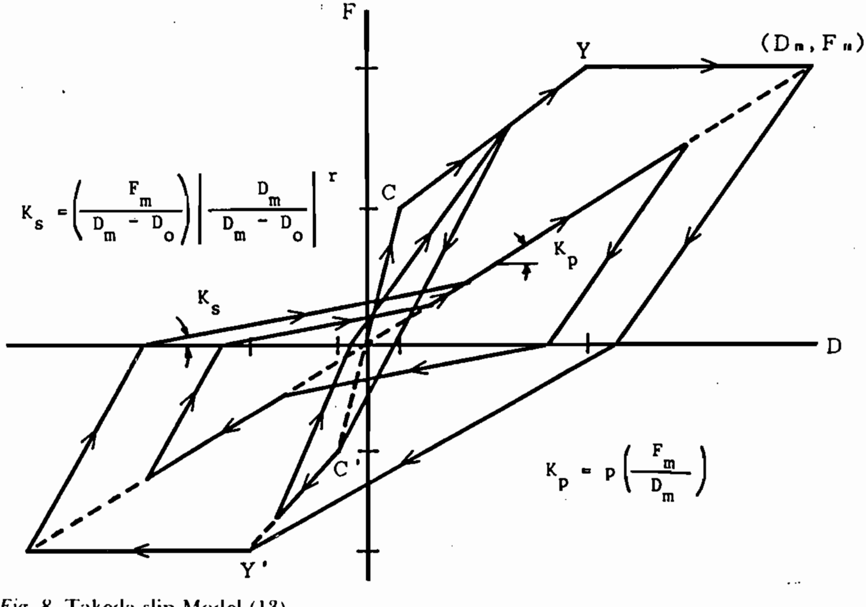


Fig. 8. Takeda-slip Model (13).

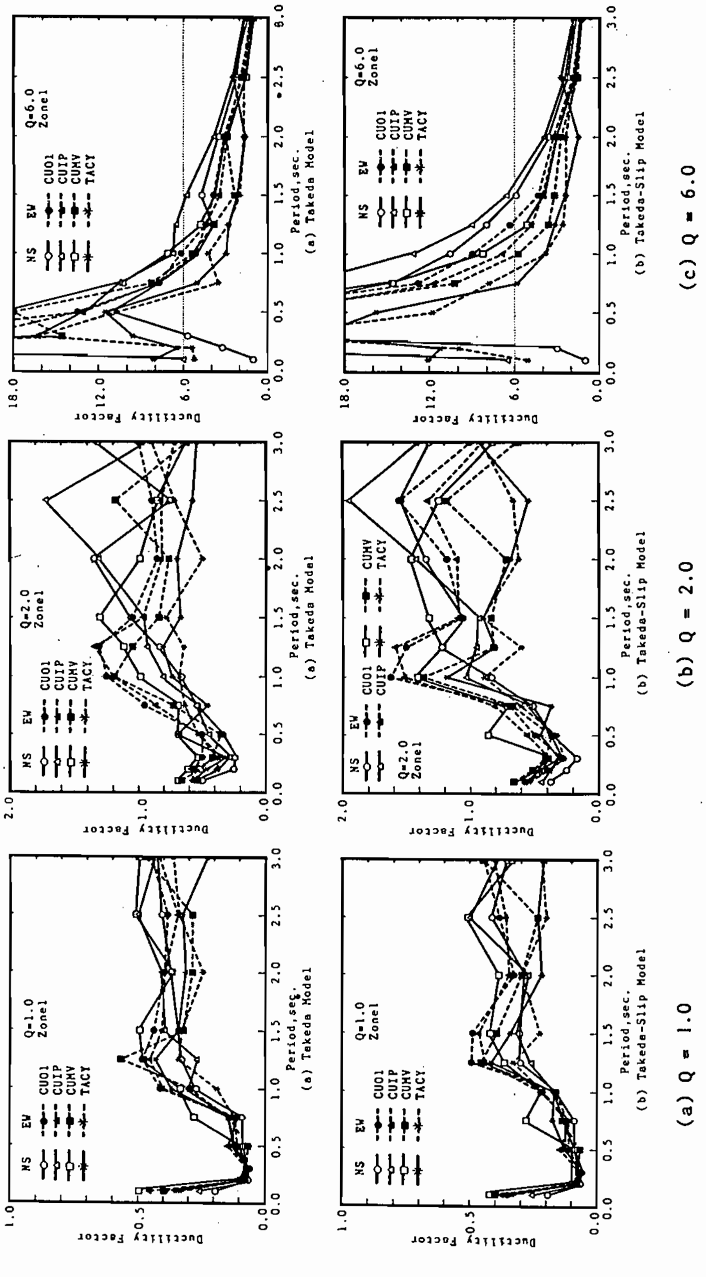


Fig. 9. Ductility Demand of Structures Designed by the 1977 Code (Zone 1).

earthquake motions was less than 2/3 for all SDF systems; the design earthquake load was satisfactory. The response ductility of both Takeda and Takeda-slip models was comparable.

For Q = 2.0, both Takeda and Takeda-slip models developed yielding in systems with periods greater than 0.7 sec, but the ductility demand was within the target design value of 2.0; i.e., the design base shear coefficient was satisfactory. Response of Takeda-slip models was slightly larger than that of corresponding Takeda models.

For Q = 6.0, both Takeda and Takeda-slip models developed ductility factors greater than the target design value of 6.0 at periods less than 1.5 sec; the ductility demand reached three times as large as the target value at periods shorter than 0.5 sec for Takeda models and at periods shorter than 0.8 sec for Takeda-slip model. In other words, the design base shear of a tall building (yield period longer than 0.5 sec) was too small for a ductility factor of 6.0. The response of Takeda-slip model was generally larger than that of Takeda models.

Zone 2 (Transition Zone, Fig. 10)

Record SXVI was used as an input motion in the analysis.

For a design ductility factor of Q = 1.0, the response was less than 2/3 of the yield displacement; i.e., the amplitude of design base shear was satisfactory.

For Q = 2.0, the attained ductility demand was less than 2/3 of the target design value of 2.0. The response of both Takeda and Takeda-slip models peaked at around 0.7 sec, at which the linearly elastic acceleration response spectra also exhibited high amplitudes. The variation of response amplitude with periods wasimilar for the two models.

For Q = 6.0, the maximum ductility demand exceeded the target design value of 6.0 at periods between 0.2 and 0.7 sec for both Takeda and Takeda-slip models. The response of Takeda-slip models was generally larger than that of corresponding Takeda models, especially in a period range where the ductility demand exceeded the target.

Zone 3 (Lake Bed Zone, Fig. 11)

Significant damage was observed in this zone especially for mid-to high-rise buildings. Records CDAF, CDAO, and SCT1 were used in the analysis.

For a design ductility factor of Q = 1.0, both Takeda and Takeda-slip models did not yield under CDAF and CDAO motions except at very short period. However, Record SCT1, especially EW component, caused yielding for systems in most period range.

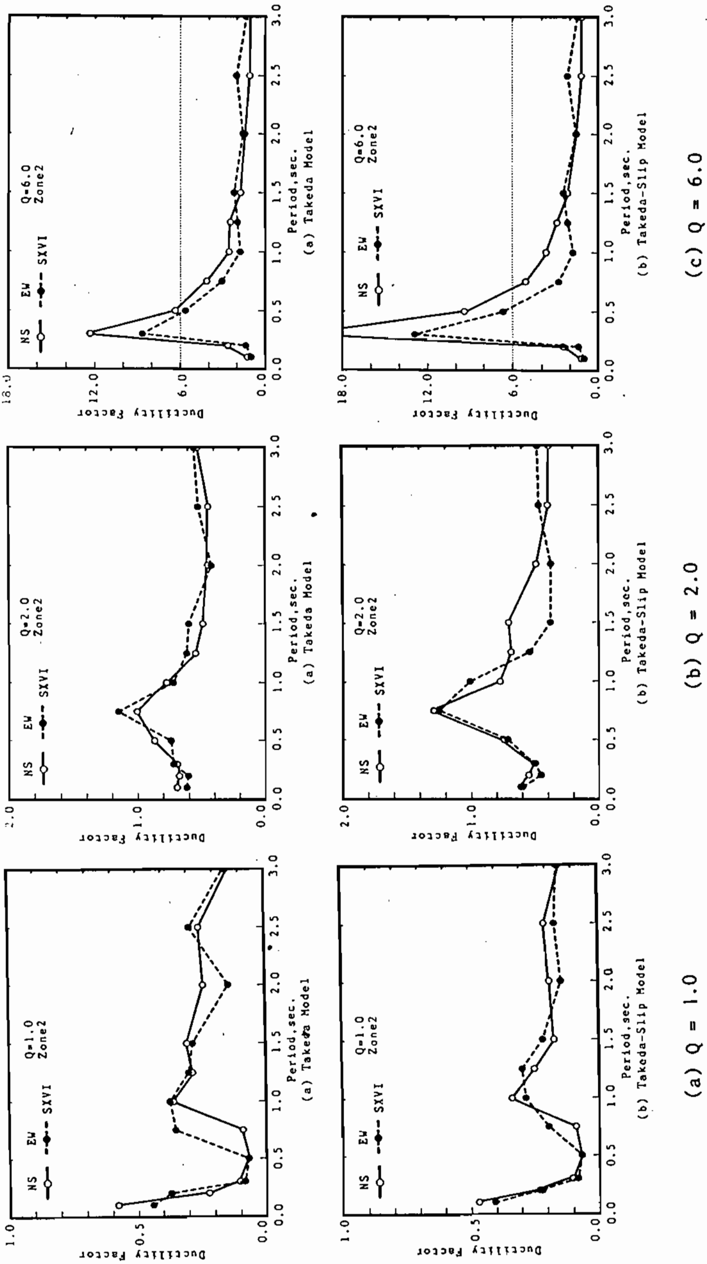


Fig. 10. Ductility Demand of Structures Designed by the 1977 Code (Zone 2).

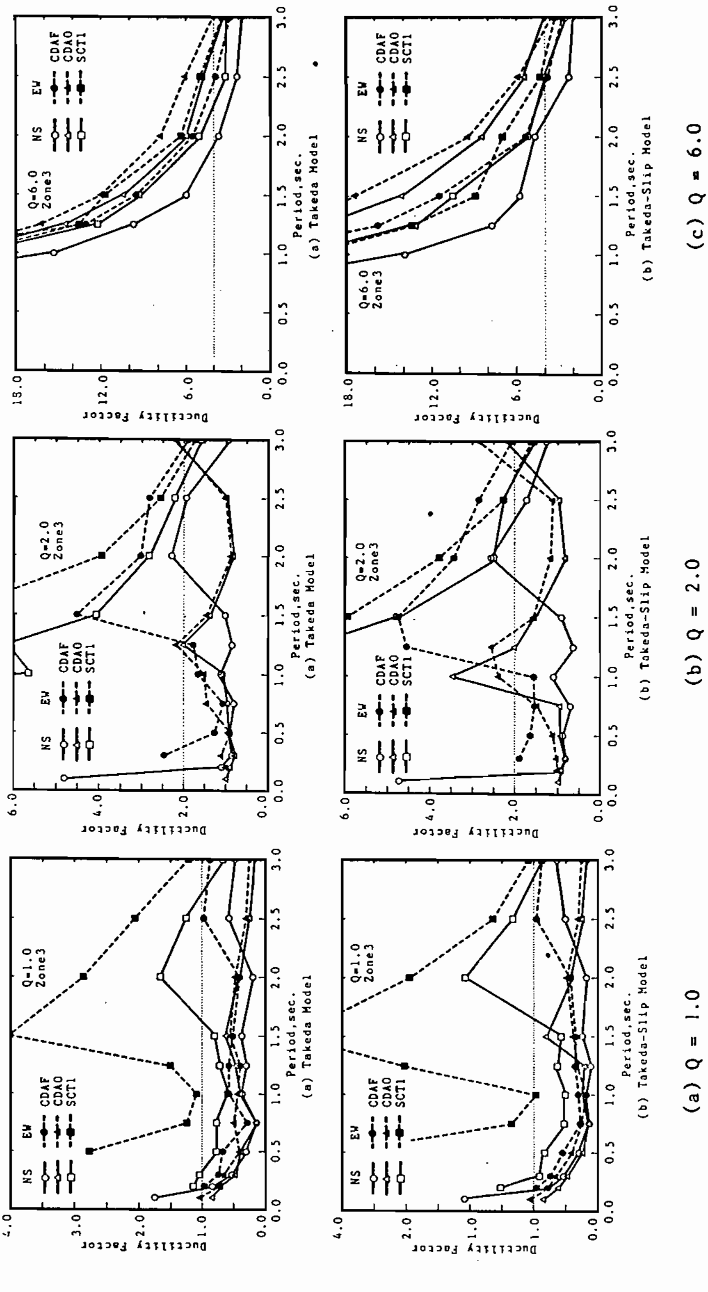


Fig. 11. Ductility Demand of Structures Designed by the 1977 Code (Zone 3).

For Q = 2.0, the ductility demand exceeded the target design values of 2.0 under CDAF and CDAO motions for a period range longer than 1.3 sec for Takeda models, and for a period range longer than 0.9 sec for Takeda-slip models. Record SCT1 required ductility demand grater than the design value in all range of periods.

For Q = 6.0, the maximum response was comparable for the three records. The ductility demand exceeded the design target for a period range less than 2.4 sex for Takeda models, and 2.5 sec for Takeda-slip models. The response increased as the system period decreased. The attained ductility exceeded three times the target design value for a period range shorter than 1.2 sec for Takeda models and 1.5 sec for Takeda-slip models.

The significant exceedance of ductility in short period systems is attributable to the fact that the ground acceleration oscillated in a period much longer than the elastic period of the short period systems and furthermore, at amplitudes much larger than the base shear coefficient (c = 0.06) of ductile systems (Q = 6.0). Even without dynamic response magnification, the inertia forces corresponding to this large-amplitude and long-period ground acceleration acted almost statically on the weak short-period systems causing a dramatic plastic deformation.

In order to illustrate this, the response of two systems having yield periods of 0.3 sec (stiff system) and 1.5 sec (flexible system) was calculated under the EW component motion of SCT1 record (Fig. 12a). The design base shear coefficient was selected to be the same (c = 0.10) in the two systems. Consequently, the yield displacement (= 0.22 cm) of the stiff system was one-twenty-fifth of the yield displacement (= 5.6 cm) of the flexible system. The ground motion oscillated at a dominant period (approximately 2.1 sec) of the site.

For the first 16 sec, the stiff system developed very small deformation (less than the yield deformation of 0.22 cm) and the resistance completely out of phase with the ground motion, the characteristics which can be observed in the response of a rigid body. The short period component in the resistance waveform (Fig. 12b) corresponded to the initial elastic period of the system. At aproximately 16 sec, when the ground acceleration reached the design base shear coefficient (= 0.10 g) of the system, the response base shear coefficient reached the yielding capacity and a significant plastic deformation took place.

At approximately 25 sec, when the ground acceleratin exceeded 0.1 g for the second time but in a longer duration, a dramatic plastic deformation took place, exhibiting a deformation of 10 to 20 cm (ductility factor of 45 to 90), and elongating an effective period of oscillation by a factor of 7

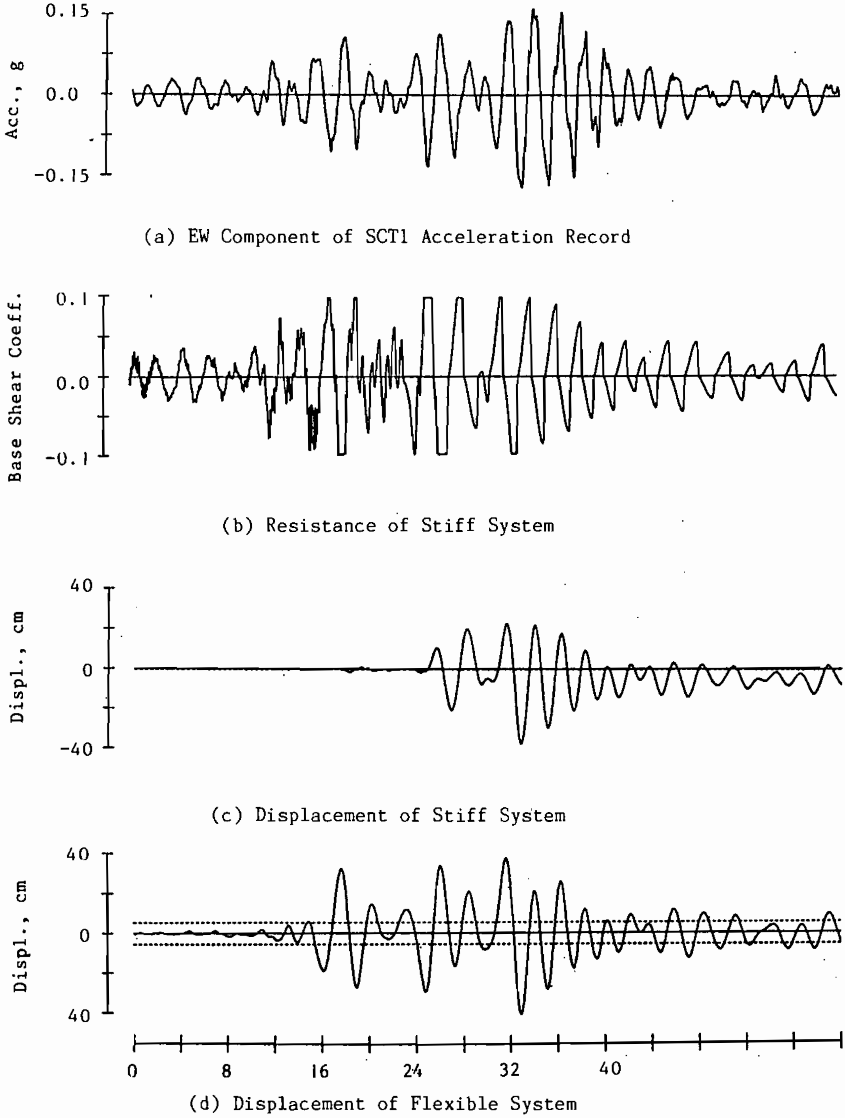


Fig. 12. Response Waveforms of Stiff and Flexible Systems.

a) EW componet of SCT1 Acceleration Record.
b) Resistance of Stiff System.
c) Displacement of Stiff System.
d) Displacement of Flexible System.

to 10 (Fig. 12c). The displacement waveform of the stiff system became similar to that to the flexible system in amplitudes and periods after 32 second (Fig. 12c and d).

Therefore, the maximum response displacements were comparable for the stiff and flexible systems, but the ductility factor of the stiff system became by far larger than the flexible system. For design, the systems with periods much shorter than the dominant ground period at the construction site, must be provided with the resistance at least equal to the maximum acceleration amplitude of the expected ground motion.

Correlation of damage and calculated response

In Zone 1 and 2, the response of systems for Q = 1.0 and 2.0 stayed within the intended deformation, and the systems could be judged to survive the observed strong motion. For short period systems designed with a large ductility (Q = 6.0), however, the ductility demand exceeded the target design ductility factor in low- to mid-rise buildings (yield periods less than 1.5 sec in Zone 1 and yield periods between 0.2 and 0.7 sec in Zone 2). It should also be noted, contrary to the analysis, that a serious damage was not reported in Zones 1 and 2 (8). Therefore, it appears that either a) a large design ductility factor was not used in the design of low-to mid-rise buildings, or b) the actual lateral load resitance of low- to mid-rise buildings was higher than that required by the code.

In Zone 3, the maximum response deformation stayed well below the yield deformation for elastically designed systems (Q = 1.0) under CDAF and CDAO motions, but the response exceeded the yield value under EW component of SCT1 record, especially around the dominant period (1.5 sec) of the ground motion. In other words, all elastically designed buildings (low- to high-rise buildings) must have failed near the area of SCT1 station. However, such damage was not reported (Instituto de Ingerniería, 1985), which indicates either a) an elastically designed structure was provided with lateral load resistance higher than the design load, or b) even an elastically designed structure could deform to a ductility factor of 2 to 4.

Furthermore, the ductility demand of systems designed with Q = 6.0 was comparable under both components of the three records, and exceeded the target value for systems having yield periods less than 1.5 sec. The response increased as the system yield period decreased. In other words, those systems designed on the base of large ductility must fail; the damage must be greater for lower buildings having shorter yield periods. The analytical results definitely contradicts with the observed damage statistics; i.e., severer damage was observed in taller buildings.

It should be noted that the demand for reduction in design earthquake load is not the same for low-rise and high-rise buildings in real life; i.e, larger reduction is normally requested in the design of a taller building.

Consider a 15-story building (tall building) and a 2-story building (low building), both having floor area of 1,000 m^2 (Fig. 13) for simplicity in comparison. Let us assume the same unit floor weight of 1.0 ton/m^2 and elastic design base shear coefficient of 0.10 for the two buildings for simplicity. Then, the elastic design base shear of the tall building becomes 1,500 ton, whereas that of the low buildings is only 200 ton. There should not be much problem to design the low building for the elastic earthquake load. However, the taller building requires much larger lateral resistance if it were to be designed elastically. Therefore, it becomes essential in the design of a tall building to reduce the design earthquake loads as much as possible relying on ductility even if a complicated structural detailing requirements must be satisfied.

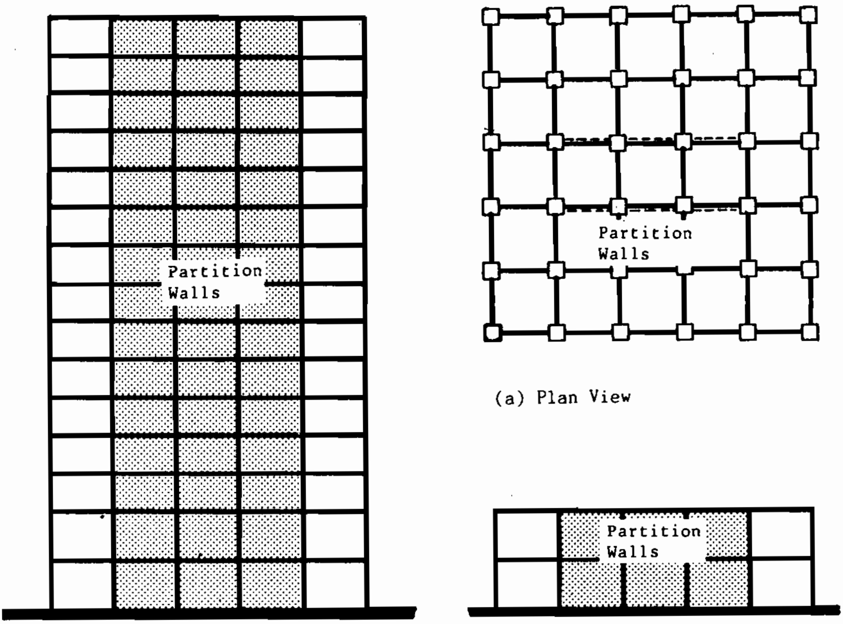


Fig. 13. Tall and Low Buildings.
a) Plan View
b) Fifteen-story Building.
c) Two-story Building.

In other words, the demand to use a larger design ductility factor Q is higher in a tall building than in a low building; i.e., there is a tendency to use smaller design base shear coefficient c in a taller building. Therefore, at a time of a strong earthquake, the tall building has to develop a large ductility intended in te design, causing a severer damage. However, a low building, which can be easily designed for a high base shear coefficient, will develop a small ductility and could survive a strong earthquake.

Furthermore, an additional lateral load resistance, for example provided by non-structural walls, also influence the earthquake response. The amount of the non-structural walls is normally associated with the floor area rather than the height of a building. Let us assume that there exist non-structural partition walls of 10-cm thick and total of 30-m long in each floor of the tall and low buildings, and that unit resistance of the non-structural walls is 2.0 kgf/cm^2. Then, the additional lateral load resistance by the non-structural wall is 60 ton at each floor, which amounts to only 4 percent of the elastic design base shear in the tall building and as much as 30 percent in the low building. The increase in lateral load resistance by non-structural as well as structural elements is more pronounced in the lower building, and the additional resistance in the low building could reduces the plastic deformation during an earthquake.

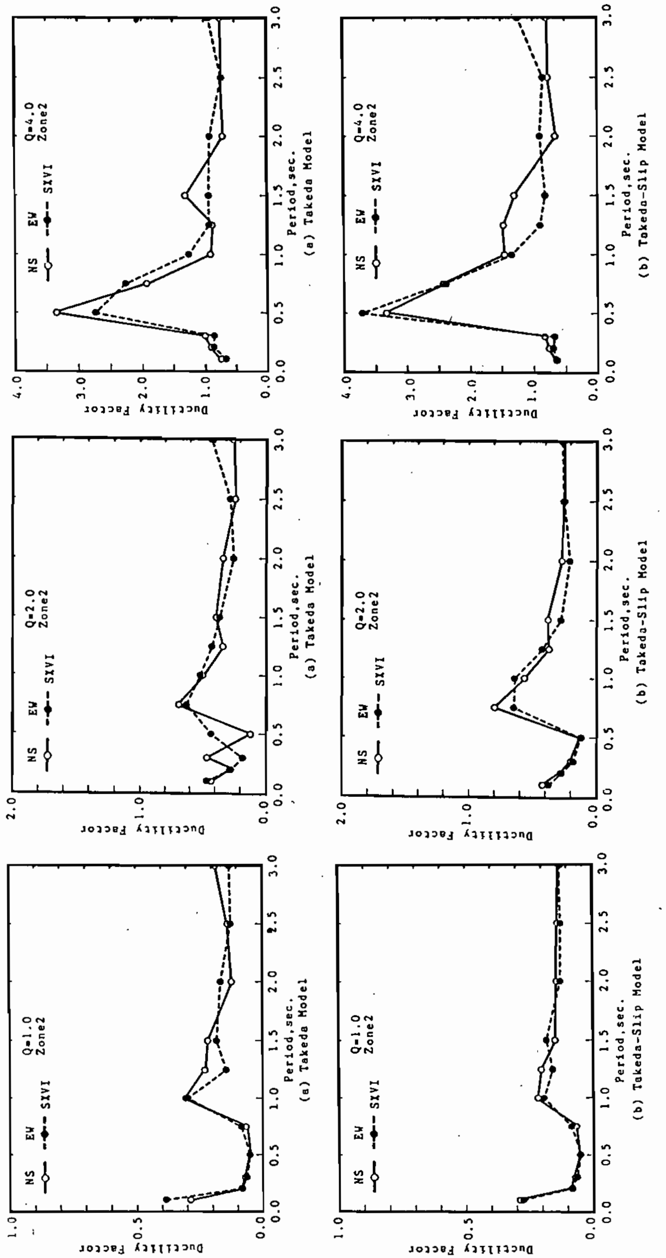
As discussed above, low- to mid-rise buildings could be designed for higher lateral load resistance using a smaller design ductility, and an appreciable lateral load resistance can be added to these structure from, for example, non-structural partitions. Therefore, a plastic deformation and associated damage could be significantly reduced in these structure. On the other hand, it is essential in the design of high-rise buildings to reduce the design earthquake loads as much as possible even counting on expected ductility, hence these tall buildings must develop intended plastic defomation (damage) in an earthquake.

RESPONSE OF 1985 CODE SYSTEMS

A series of SDF systems were designed in accordance with the 1985 Emergency Regulations for three ductility factors (Q = 1.0, 2.0, 4.0). The fundamental period was varied from 0.1 to 3.0 sec. The design requirements for zone 1 was not altered in the 1985 Emergency Regulations; hence, the analysis was carried out for Zones 2 and 3.

Zone 2 (Transition Zone, Fig. 14)

The design spectral parameters for this zone were raised in the 1985



(a) Q = 1.0

(b) Q = 2.0

(c) Q = 4.0

Fig. 14. Ductility Demand of Structures Designed by the 1985 Code (Zone 2).

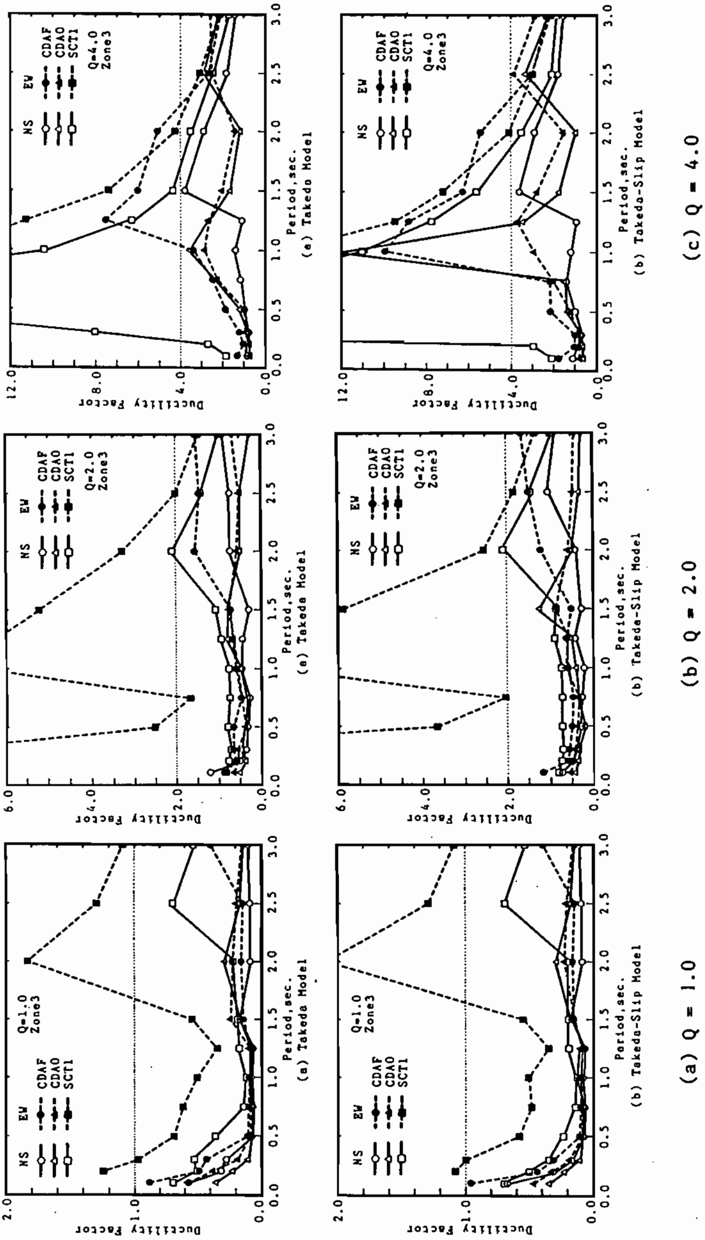


Fig. 15. Ductility Demand of Structures Designed by the 1985 Code (Zone 3).

Emergency Regulations. The 1977 Construction Regulations were judged satisfactory for design ductility factors of 1.0 and 2.0 in this study. Therefore, the ductility demand of systems for a design ductility factor of 4.0 was studied here. Record SXVI was used as an input motion.

For a design ductility factor of Q = 4.0, the system yielded in almost all range of periods, but the ductility demand remained within the design target (ductility factor of 4.0).

Zone 3 (Lake Bed Zone, Fig. 15)

The design earthquake load was significantly increased in this zone. Records CDAF, CDAO, and SCT1 were used in the analysis.

For a design ductility factor of Q = 1.0, the ductility demand was well below the target for all range of periods except for the response under EW component of Record SCT1. The response to ground motion SCT1 (EW) exceeded the target at a period above 1.6 sec for Takeda and Takeda-slip models.

For Q = 2.0, the ductility demand stayed around and below the design target for all range of periods except for the response under EW component of Record SCT1. The response to ground motion SCT1 (EW) exceeded the target in all range of periods, and exceeded three times the target at around 1.2 sec and below 0.4 sec.

For Q = 4.0, the ductility demand exceeded the design target for a period range less than 2.2 sec for Takeda models, and 2.5 sec for Takeda-slip models. The response amplitudes were significantly reduced for this category of systems by the introductions of the Emergency Regulations, but the response ductility exceeded three times the target for a period range less than 1.3 sec.

The reason for a larger ductility demand for short period systems in Zone 3 was already described with respect to the response of the 1977 code systems. For a design ductility factor of 4.0, the required base shear coefficient (= 0.10) is smaller than the peak acceleration amplitude of SCT1 (EW) record.

The introduction of the 1985 Emergency Code was effective reducing the plastic deformation by requiring higher earthquake loads. However, the use of large design ductility need be further studied.

Concluding remarks

The damage inventory survey was carried out in a limited number of areas in the severely damaged lake bed zone in Mexico City. The damage to low-rise buildings (less than 5 stories) was relatively light, whereas the damage was heavier in mid- to high-rise buildings. The survey indicated the importance of careful earthquake resistant design for taller buildings.

A system, with periods much shorter than the dominant ground period at the construction site, must be provided with the resistance at least equal to the maximum acceleration amplitude of the expected ground motion.

Low- to mid-rise buildings could be designed for higher lateral load resistance using a smaller design ductility, and a appreciable lateral load resistance can be added to these structure from, for example, non-structural partitions. Therefore, a plastic deformation and associated damage could be significantly reduced in these structure. On the other hand, it is essential, in the design of high-rise buildings, to reduce the design earthquake loads as much as possible counting on expected ductility; hence the tall buildings must suffer from intended plastic defomation, that is damage, in an earthquake.

A significant improvement of response designed after the 1985 Emergency Regulations was observed in nonlinear earthquake response analysis. However, the resistance of buildings for a ductility factor of 4.0 in the lake bed zone was observed insufficient to limit the response within the design target.

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