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Observation and Analysis on Damage of Reinforced Concrete Buildings from the 1985 Mexico Earthquake

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The 1985 Mexico Earthquake caused a serious damage to buildings in Mexico City, especially in the old lake-bed zone. Reliable statistics on damaged and undamaged buildings over the heavily damaged metropolitan areas were established by Architectural Institute of Japan and Ohbayashi-gumi Investigation Teams.

A series of single-degree-of-freedom (SDF) nonlinear earthquake response analyses were carried out to correlate the observed damage and the calculated response, using the earthquake motions observed in Mexico City; i.e., strong ground motion records measured in the firm ground, transition, and old lake-bed zones. The strength of SDF systems was assigned in accordance with the Construction Regulations for the Federal District of Mexico (1977) and the 1985 Emergency Regulations.

1. Introduction

An earthquake of magnitude 8.1 occurred on the Mexican west coast on September 19, 1985, followed by a series of aftershocks, notably one of magnitude 7.5 on September 21. The two successive events caused serious damages to mid- to high-rise buildings in Mexico City located approximately 400 km away from the epicenter; the severe demage in such a distant area was attributed to the ground motion amplified by soft and deep soil deposit underlain in the old lake-bed zone of 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 Ingenieria, Universidad Nacional Autonoma de Mexico.[1-5]

Many Japanese researchers and engineers investigated the damage, and the Architectural Institute of Japan (AIJ) published a comprehensive report.[6] The statistical data on damaged buildings in Mexico City 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 Y. Kanoh of Meiji University) in early November, 1985. The author served as secretary to the latter team.

Nonlinear earthquake response analysis was carried out for a series of single-degree-of-freedom systems to correlate the observed damage statistics and calculated response. The strength characteristics were determined by the existing building code at the time of the

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earthquake. The effectiveness of the revised building code is also discussed by the analysis.

2. 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. Such data are useful to prepare the earthquake resistant measures against future earthquakes.

2.1 Areas for Study

The metropolitan area is divided into three zones for an earthquake resistant design purpose; i.e., firm ground (Zone 1), old lake-bed (Zone 3) and transition (Zone 2) zones. In Zone 3, the old lake was gradually filled with a development of the city, and soil conditions are extremely poor.

An investigation[8] immediately after the earthquake reported severe damage in the lake-bed zone as shown in Fig. 1, in which each circle indicates the location of a severely damaged building. The AIJ investigation team decided to concentrate the investigation in selected areas (shaded areas in Fig. 1) in the lake-bed zone due to the limit of working time and man power. A group of two to three experienced structural engineers and researchers went through every alley and surveyed the damage of each building in the area by the external appearance. This investigation may be called a damage inventory survey.

2.2 Construction Types

The number of steel buildings was extremely small. 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.

2.3 Damage Ranks

The AIJ 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. When the building showed inclination, the criteria in Table 2 were used.

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 Rank 1/2: Ohbayashi-gumi Level D/E

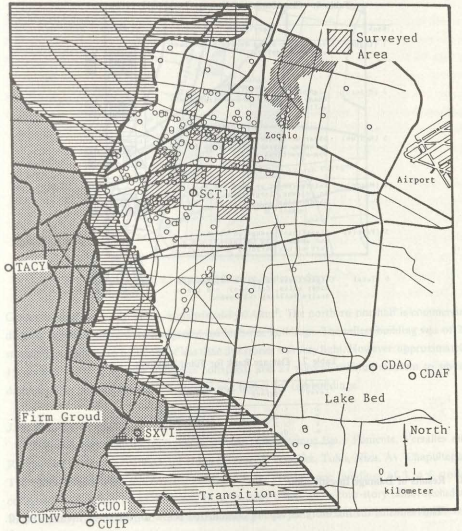


Fig. 1. Seismic Zoning, Severely Damaged Buildings, and Earthquake Recording Stations.

AIJ Rank 3/4: Ohbayashi-gumi Level C

AIJ Rank 5: Ohbayashi-gumi Level B

AIJ Rank 6: Ohbayashi-gumi Level A

Note that the external observation tends to underestimate the damage; e.g., severely damaged architectural and structural elements might have been hidden inside.

Table 1. Classification of Damage Level (AIJ, 1980).

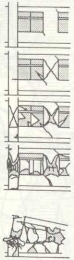
Rank	Description	Sketch
1 (Light)	Very light or no damage to columns and shear walls.	
2 (Minor)	Light damage on columns and walls, shear cracks on RC non-structural walls.	
3 (Medium)	Shear or flexural cracks on columns, appreciable damage on non-structural walls.	
4 (Major)	Reinforcement exposed and 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

3. Results of Damage Investigation

The results of the inventory survey are summarized below for the seven areas in Fig. 2.

3.1 Area 1

The area between Av. Insurgentes Nortes and Guerrero, south of Calz. Nonoalco and north of Puente de Alvarado. There were many low-rise masonry residential buildings, mid-rise masonry apartment buildings, and some factory buildings. The tallest reinforced concrete building was of 8 stories high. Less than 1.6 percent of the buildings surveyed suffered medium or severer damage; 7.4 percent suffered minor or greater damage. No or light damage was observed in 93 percent of the buildings. The damage was very light in this area.

3.2 Area 2

The area south of Arameda Park, bounded by Bucareli, Av. Juarez, Central Lazaro

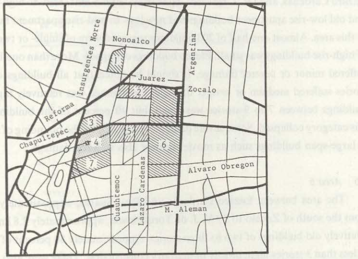


Fig. 2. Areas of Damage Inventory Survey.

Cardenas, and Arcos de Belen, approximately 0.4 km^2. The northern one-half is commercial district, in which there were many mid- to high-rise buildings. The tallest building was of 20 stories. The damage to buildings of less than five stories was very light. However, approximately 15 percent of 7- to 9-story buildings collapsed, and 27 percent suffered medium to severe damage. The ratio of damaged buildings was higher for tall buildings.

3.3 Area 3

The area bounded by Insurgentes Sur, Av. Cuauhtemoc Eje 1 Poniente, Versalles and Roma. The survey was carried out along Liverpool, Versalles, Tulin, Niza, Av. Chapultepec. The other streets were not covered in this survey. Ten low-rise buildings of 3 to 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.

3.4 Area 4

The area bounded by Av. Cuauhtemoc Eje 1 Poniente, Av. Chapultepec, Av. Insurgentes Sur, and Av. Alvaro Obregon. 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 were scattered in the area. Although the area is immediately adjacent to Area 7, severe damage was observed in 7- to 13-story buildings.

3.5 Area 5

The area bounded by Av. Cuauntemoc Eje 1 Poniente, Dr. Rio de la Loza, Eje Central

Lazaro Cardenas, and Dr. J. Navarro, approximately 0.6 km^2. 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. Eleven buildings collapsed. 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 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.

3.6 Area 6

The area between Lazaro Cardenas and Pino Suarez, approximately 1.0 km wide, and from the south of Zocalo to Fr Y. J. de Torquemade, 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. No to light damage was observed in 86 percent of the buildings surveyed, and only 1.1 percent of the buildings collapsed. The damage rate was light. Four percent of buildings of less than 4 stories suffered medium and heavier damage, while the ratio increased to 64.3 percent for buildings of more than 9 stories. The ratio of damaged buildings clearly increased with the number of stories, and the buildings taller than 10 stories suffered severe demage.

3.7 Area 7

The area bounded by Av. Cuauhtemoc Eje 1 Poniente, Av. Alvaro Obregon, Av. Insurgentes Sur, and Chiapas, approximately 0.85 km^2. 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. 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 rate 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.

3.8 Discussion

The area surveyed covered slightly more than 20 percent of the central area of the city, the results are summarized in Table 3 and Fig. 3 for all data obtained. The number of buildings less than 4 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 only in 1.8 percent of the buildings surveyed, however, in 9.0 percent of the buildings of more than 5 stories. Note the increase

Table 3. Summary of Damaged Buildings.

No. of Stories	Damage Rank 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

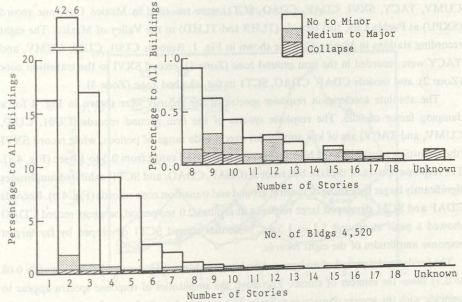


Fig. 3. Summary of Damage Inventory Survey.

in the percentage of damaged buildings with the number of stories; the damage was relatively light in low-rise buildings.

4. Earthquake Motions in Mexico City

Eleven earthquake records (Table 4), each containing two horizontal and one vertical components, were recorded in Mexico City and its outskirt.[1-5] Eight records (CU01, CUIP,

Table 4. Ground Motions Recorded in Mexico City[1-5].

No.	ID	Zone	Dir.	Max.Acc. (cm/s^2)	Max.Vel. (cm/s)	Max.Dspl. (cm)	Note
1	CU01	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.

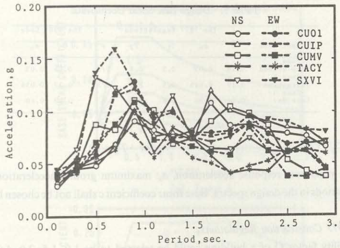
CUMV, TACY, SXVI, CDAF, CDAO, SCT1) were recorded in Mexico City, one record (SXPU) at Puebla City, two records (TLHB and TLHD) in the Valley of Mexico. The eight recording stations in Mexico City are shown in Fig. 1. Records CU01, CUIP, CUMV, and TACY were recorded in the firm ground zone (Zone 1), record SXVI in the transition zone (Zone 2), and records CDAF, CDAO, SCT1 in the lake bed zone (Zone 3).

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 (CU01, CUIP, CUMV, and TACY) are of low amplitudes over a wide range of periods, while record SXVI (the transition zone) exhibited high amplitudes at a period range from 0.5 to 1.0 sec (Fig. 4.a). The response spectra of the lake bed records (CDAF, CDAO, and SCT1) exhibited amplitudes significantly larger than those of the firm ground and transition zone records (Fig. 4.b). Records CDAF and SCT1 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 SCT1 developed by far largest response amplitudes of the eight records.

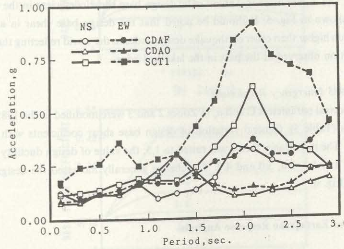
The fundamental period of undamaged buildings in Mexico City ranged approximately 0.08 to 0.11 times the number of stories. Therefore, the amplitudes of response spectra appear to coincide with the severer damage in taller buildings in the lake-bed zone demonstrated by the damage inventory survey. However, the response spectra simply represent the earthquake input force level, but do not reflect the resistance of buildings; i.e., further inelastic response analysis is needed to explain the damage.

5. 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.[9] The Emergency Regulations (Depertment del Distrito Federal, 1985)[10] were issued on October 18, 1985,



(a) Records obtained in Firm Ground Zone



(b) Records obtained in Lake Bed Zone

Fig. 4. Absolute Acceleration Response Spectra at 0.05 Damping.

within one month after the earthquake. The design base shear coefficient c in the two regulations was defined as a function of fundamental period T, selected ductility factor Q and spectral parameters C, a_o, r, T_1 and T_2 for the three seismic zones; i.e.,

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

$$c=C/Q \quad \text{for} \quad T_1<T<T_2$$

Table 5. Design Base Shear Coefficients.

Seismic zone	The 1977 Regulations					The 1985 Code	
	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

$$c = C(T_2/T)^r/Q \qquad \text{for} \quad T_2 < T$$

in which C: maximum response acceleration, a_o: maximum ground acceleration, T_1 and T_2 are corner periods in the design spectra. Base shear coefficient c shall not be chosen less than a_o.

5.1 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 by a structural engineer. The importance factor for buildings, which must maintain their 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 5. The design base shear coefficient in the three seismic zones are shown in Fig. 5. It should be noted that the design base shear in a long period range is much higher than other earthquake design codes in the world reflecting the long-period ground motion observed in the past in the lake-bed zone.

5.2 The 1985 Emergency Regulations

The spectral parameters C and a_o in Zones 2 and 3 were modified in the 1985 Emergency Regulations (Table 5). General variation of design base shear coefficients with periods was not altered. The importance factor was raised to 1.5; the value of design ductility factors were changed to 1.0, 1.5, 2.0, 3.0 and 4.0. The change generally increased the design base shear coefficient (Fig. 6).

6. Nonlinear Earthquake Response Analysis

The nonlinear earthquake response analysis was carried out 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. A structure was represented by a single-degree-of-freedom (SDF) system having nonlinear hystertic characteristics to study a general trend in the response.

6.1 Response Calculation

A series of SDF systems with nonlinear hysteretic characteristics were designed for periods ranging from 0.1 to 3.0 sec, in which the fundamental periods of most buildings in Mexico City fall.

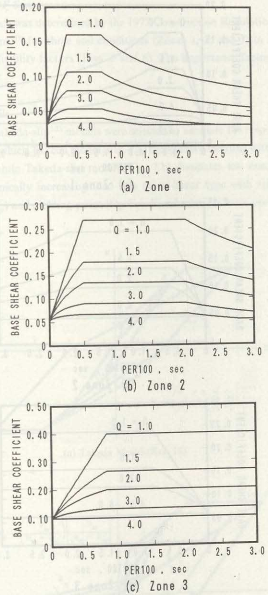


Fig. 5. Design Seismic Load for the 1977 Code.

Mass of the systems was assumed to be unity; i.e., the resistance was normalized yield base shear coefficients. The damping coefficient was assumed to vary proportional to instantaneous stiffness, and the value was selected to yield a dampling 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.

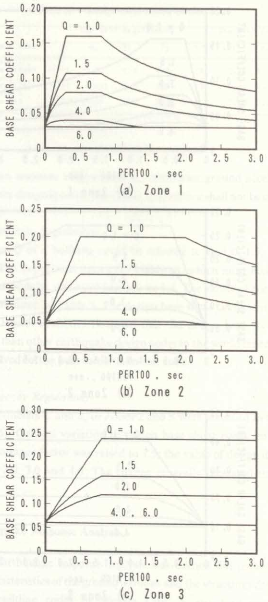


Fig. 6. Design Seismic Load for the 1985 Code.

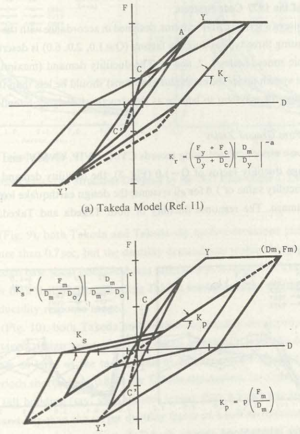
The systems were subjected to each horizontal component of the observed earthquake motions recorded in the corresponding earthquake zone. Response computation was carried out by the Newmark-beta method with iterations to satisfy both the balance of forces and the hysteretic relation at each time step.

6.2 Resistance of SDF Systems

The yield resistance was determined by the 1977 Construction Regulations[9] and the 1985 Emergency Regulations[10] for three soil conditions (Zones 1, 2, and 3) in Mexico City and three representative ductility factors (Figs. 5 and 6). The importance factor was taken to be unity for all systems.

6.3 Hysteresis Models

Takeda[11] and Takeda-slip[12] models were selected to simulate the response of reinforced concrete buildings, in which Takeda model (Fig. 7.a) represents a building with a large hysteretic energy dissipation, while Takeda-slip model (Fig. 7.b) 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;



(a) Takeda Model (Ref. 11)

(b) Takeda-slip Model (Ref. 12)

Fig. 7. Hysteretic Models.

$$F_e = F_y/3$$

$$D_e = D_y/12$$

The period corresponding to the secant stiffness at 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[11]. The slip stiffness degradation parameter was 1.0, reloading stiffness parameter of Takeda-slip model[12] were selected to be 1.5 and 1.0, respectively.

The previous study[13] indicated that the inelastic response is not sensitive to the properties before yielding; consequently, the period of the system was selected on the basis of secant stiffness at the yielding in this study.

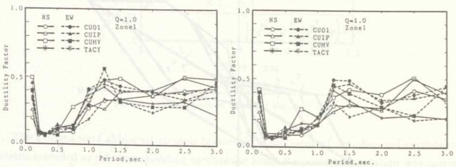
7. Response of the 1977 Code Systems

The response of a series of SDF systems, designed in accordance with the 1977 Construction Regulations[9] using three typical ductility factors ($Q=1.0$, 2.0, 6.0) is described separately for the three seismic zones (Zones 1, 2, and 3). The ductility demand (maximum ductility factor calculated for a system under the earthquake motions) should be less than the ductility factors used in the design for a system to behave as the structural designer intended.

7.1 *Zone 1 (Firm Ground Zone)*

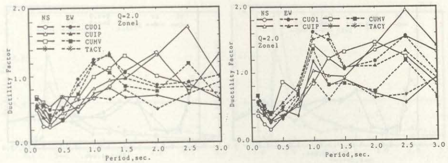
The response was calculated using records CV01, CVIP, CVMV, and TACY.

For a design ductility factor of $Q=1.0$ (Fig. 8), the ductility demand was less than 2/3 of the design ductility value of 1.0 for all systems; the design earthquake load was satisfactory to limit the damage. The response ductility of both Takeda and Takeda-slip models was comparable.



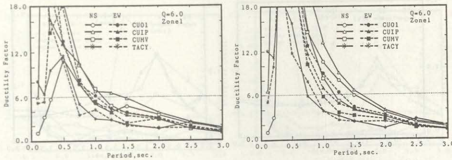
(a) Takeda Model (b) Takeda-slip Model

Fig. 8. Ductility Demand of Structures Designed by the 1977 Code (Zone 1, Design Ductility Factor = 1.0).



(a) Takeda Model (b) Takeda-slip Model

Fig. 9. Ductility Demand of Structures Designed by the 1977 Code (Zone 1, Design Ductility Factor = 2.0.)

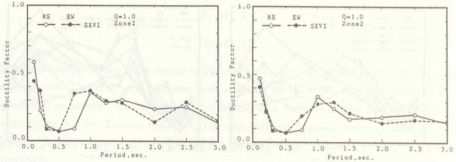


(a) Takeda Model (b) Takeda-slip Model

Fig. 10. Ductility Demand of Structures Designed by the 1977 Code (Zone 1, Design Ductility Factor = 6.0).

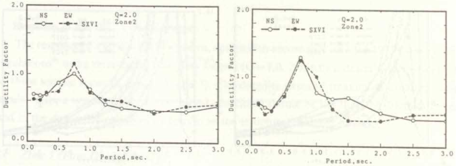
For $Q=2.0$ (Fig. 9), both Takeda and Takeda-slip models developed yielding in systems with periods greater than 0.7 sec, but the ductility demand was within the target design value of 2.0; i.e., the design base shear coefficient was satisfactory. Response of Takeda-slip models was slightly larger than that of corresponding Takeda models because slip-type behavior was small in a small ductility response range.

For $Q=6.0$ (Fig. 10), 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 models. In other words, the design base shear for a tall building (say, yield periods longer than 0.5 sec) was too small to limit the ductility demand less than the target ductility factor of 6.0. The response of Takeda-slip model was generally larger than that of Takeda models because the slip behavior was conspicuous in a diplacement range much after yielding.



(a) Takeda Model (b) Takeda-slip Model

Fig. 11. Ductility Demand of Structures Designed by the 1977 Code (Zone 2, Design Ductility Factor = 1.0).



(a) Takeda Model (b) Takeda-slip Model

Fig. 12. Ductility Demand of Structures Designed by the 1977 Code (Zone 2, Design Ductility Factor = 2.0).

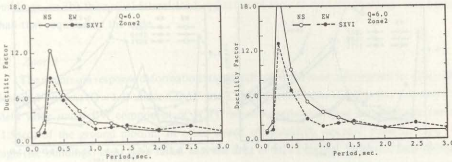
7.2 Zone 2 (Transition Zone)

Record SXVI was used as an input motion.

For a design ductility factor of $Q=1.0$ (Fig. 11), the response was less than 2/3 of the yield displacement; i.e., the amplitude of design base shear was satisfactory to control the damage.

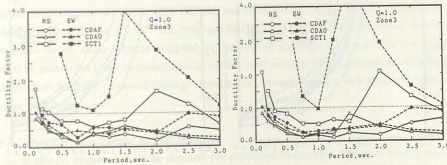
For Q = 2.0 (Fig. 12), the ductility demand was less than 2/3 of the target design value of 2.0. The response of both Takeda and Takeda-slip models peaked at around 0.7 sec, at which the linearly elastic acceleration response spectra also exhibited high amplitudes. The variation of response amplitudes with periods was similar for the two models. The amplitude of design base shear was also satisfactory to control the damage using $Q=2.0$.

For Q = 6.0 (Fig. 13), 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.



(a) Takeda Model (b) Takeda-slip Model

Fig. 13. Ductility Demand of Structures Designed by the 1977 Code (Zone 2, Design Ductility Factor = 6.0).



(a) Takeda Model (b) Takeda-slip Model

Fig. 14. Ductility Demand of Structures Designed by the 1977 Code (Zone 3, Design Ductility Factor = 1.0).

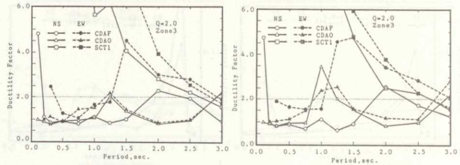
7.3 Zone 3 (Lake Bed Zone)

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 (Fig. 14), 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 an almost entire period range.

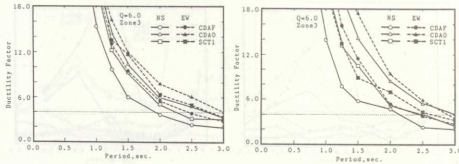
For Q = 2.0 (Fig. 15), the ductility demand exceeded the target design values of 2.0 under CDAF and CDAO motions for a period range longer than 1.3 sec for Takeda models, and for a period range longer than 0.9 sec for Takeda-slip models. Record SCT1 required ductility demand greater than the design value in all range of periods; i.e., all the buildings designed using a ductility factor of 2.0 must have suffered a damage greater than the structural engineer conceived in the design.

For Q = 6.0 (Fig. 16), the maximum response was comparable for the three records. The ductility demand exceeded the design target for a period range less than 2.4 sec for Takeda



(a) Takeda Model (b) Takeda-slip Model

Fig. 15. Ductility Demand of Structures Designed by the 1977 Code (Zone 3, Design Ductility Factor = 2.0).



(a) Takeda Model (b) Takeda-slip Model

Fig. 16. Ductility Demand of Structures Designed by the 1977 Code (Zone 3, Design Ductility Factor = 6.0).

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.

8. Correlation of Damage and Calculated Response

8.1 *Zones 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 resistance of low- to mid-rise buildings was higher than that required by the code.

8.2 *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,[8] 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 mainly in taller buildings.

8.3 *Ductility Requirement in Tall 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 needed 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. 17) 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 because the design base shear coefficient is not reduced significantly for a tall building in Mexico City. Then, the elastic design base shear of the tall building becomes 1,500 tonf, whereas that of the low building is only 200 tonf. 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 the tall building to reduce the design earthquake loads as much as possible relying on ductility even if complicated structural detailing requirements must be satisfied.

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 the design, causing a severer damage. However, a low building, easily designed for a high base shear coefficient, will develop a small ductility and survive a strong earthquake without much damage.

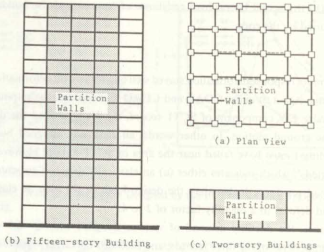


Fig. 17. Tall and Low Buildings.

8.4. Effect of Additional Resistance

Furthermore, an additional lateral load resistance, for example, provided by non-structual 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 (Fig. 17), 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 further reduces the plastic deformation during an earthquake.

8.5 Higher Damage Rate in Tall Buildings

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 structures from, for example, non-structural partitions. Therefore, a plastic deformation and associated damage could be significantly reduced in these structures.

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 will develop intended plastic deformation and must suffer associated damage from an earthquake.

9. Effect of Long-Period Ground Motion

The significant exceedance of ductility demand of a system having a short period of oscillation under the Zone 3 motion is attributable to the fact that the ground acceleration oscillated in a period (approximately 2.1 sec) much longer than the elastic period of the short period systems and at amplitudes (approximately 0.2 g) much larger than the design base shear coefficient ($c=0.06$) of ductile systems ($Q=6.0$). Therefore, 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. 18.a). 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. 18.b) corresponded to the initial elastic period of the system. At approximately 16 sec, when the ground acceleration reached the design base shear coeflcient ($=0.10$ 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 acceleration 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 to 10 (Fig. 18.c). The displacement waveform of the stiff system became similar to that of the flexible system in amplitudes and periods after 32 second (Fig. 18.c 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.

10. Response of the 1985 Code Systems

A series of SDF systems were designed in accordance with the 1985 Emergency Regulations[10] for three ductility factors ($Q=1.0, 2.0, 4.0$). 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.

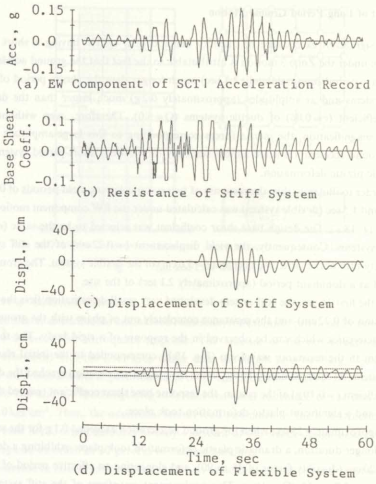


Fig. 18. Response Waveforms of Stiff and Flexible Systems.

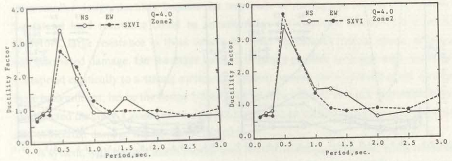
10.1 Zone 2 (Transition Zone)

The design spectral parameters for this zone were raised in the 1985 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 (Fig. 19), the system yielded in almost all range of periods, but the ductility demand remained within the design target (ductility factor of 4.0).

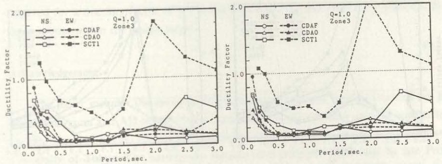
10.2 Zone 3 (Lake Bed Zone)

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



(a) Takeda Model (b) Takeda-slip Model

Fig. 19. Ductility Demand of Structures Designed by the 1985 Code (Zone 2, Design Ductility Factor = 4.0).



(a) Takeda Model (b) Takeda-slip Model

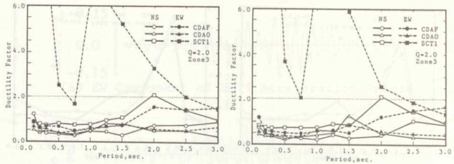
Fig. 20. Ductility Demand of Structures Designed by the 1985 Code (Zone 3, Design Ductility Factor = 1.0).

For a design ductility factor of Q = 1.0 (Fig. 20), 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 (Fig. 21), 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.

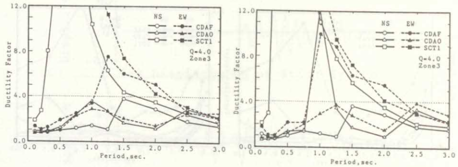
For Q = 4.0 (Fig. 22), the ductility demand exceeded the design target for a period range less than 2.2 sec for Takeda models, and 2.5 sec for Takeda-slip models. The response amplitudes were significantly reduced for this category of systems by the introductions of the Emergency Regulations, but the 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



(a) Takeda Model (b) Takeda-slip Model

Fig. 21. Ductility Demand of Structures Designed by the 1985 Code (Zone 3, Design Ductility Factor = 2.0).



(a) Takeda Model (b) Takeda-slip Model

Fig. 22. Ductility Demand of Structures Designed by the 1985 Code (Zone 3, Design Ductility Factor = 4.0).

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 cautioned.

11. Concluding Remarks

The damage inventory survey was carried out in a limited number of areas in the severly 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.

Low- to mid-rise buildings could be easily designed for higher base shear coefficient relying

on a smaller design ductility because the amplitude of earthquake load is controlable, and the lateral load resistance could be enhanced by, for example, non-structural partitions. Therefore, large resistance in these structures could significantly reduce plastic deformation and associated damage. On the other hand, it is hardly possible to design high-rise buildings to respond elastically to a strong earthquake motion because the amplitude of the earthquake force is significant, hence the design lateral load must be reduced as much as possible counting on expected ductility. Consequently, the tall building must suffer intended plastic deformation and associated damage during an earthquake.

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.

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

Acknowledgement

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