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論 文

[2133] メキシコ地震（1985）におけるコンクリート建物の被害

DAMAGE ON CONCRETE STRUCTURES FROM THE 1985 MEXICO EARTHQUAKE

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1. INTRODUCTION

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. More than 10,000 people were killed or injured from the events. This severe damage was attributed to the magnification of ground motion by deep and soft soil deposit underlain in the Mexico Valley. Many Japanese researchers investigated the damage to establish reliable statistics (Ref.1). Eleven stations recorded the ground motion in Mexico City and its outskirts by Instituto de Ingenieria, Universidad Nacional Autonoma de Mexico (UNAM; Ref.2). The records reflected the characteristics of soil conditions at the stations (Ref.3). A series of single-degree-of-freedom (SDF) nonlinear earthquake response analyses were carried out to correlate the observed damage and response of structures, designed in accordance with the 1977 Construction Regulations (1977 Code, Ref.4).

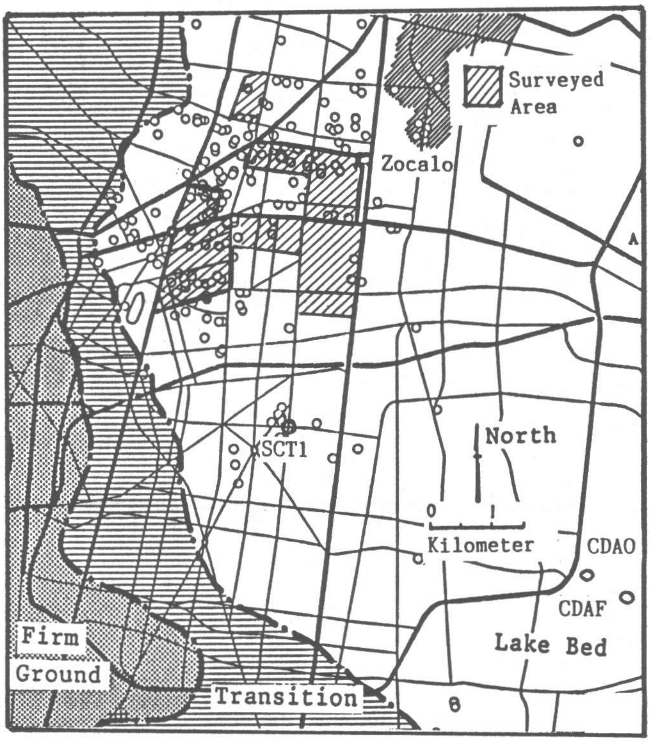

Fig. 1: Damage in Mexico City

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2. DAMAGE STATISTICS IN MEXICO CITY

An investigation (Ref.3) immediately after the earthquake reported that severe damage was observed to concentrate in the lake bed zone with deep and soft soil deposit, and minor damage was reported in the firm ground zone and the transition zone from the firm ground to the lake bed (Fig.1). One to two months after the earthquake, an extensive inventory damage survey was carried out by Japanese teams (Ref.1), in which a group of two to three experienced structural researchers went through all alleys of arbitrarily selected areas (Fig.1) in the lake bed zone and examined the damage of every building from external appearance. 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 results are summarized in Fig.2. The area surveyed covered slightly more than 20 percent of the Metropolitan area. The number of buildings decreased with the number of stories; the number of buildings less than 4 stories high was approximately 73 percent of the 4,520 buildings surveyed. No to light damage was observed in 82 percent of the buildings surveyed, or in 85.2 percent of the buildings of less than 6 stories high; the collapse occurred in 1.8 percent of the buildings surveyed, and in 9.0 percent of the buildings of more than five stories high. Note that the percentage of damaged buildings increased with the number of stories. The fundamental period of undamaged buildings in Mexico City ranged from approximately 0.08 to 0.11 times the number of stories (Ref.1).

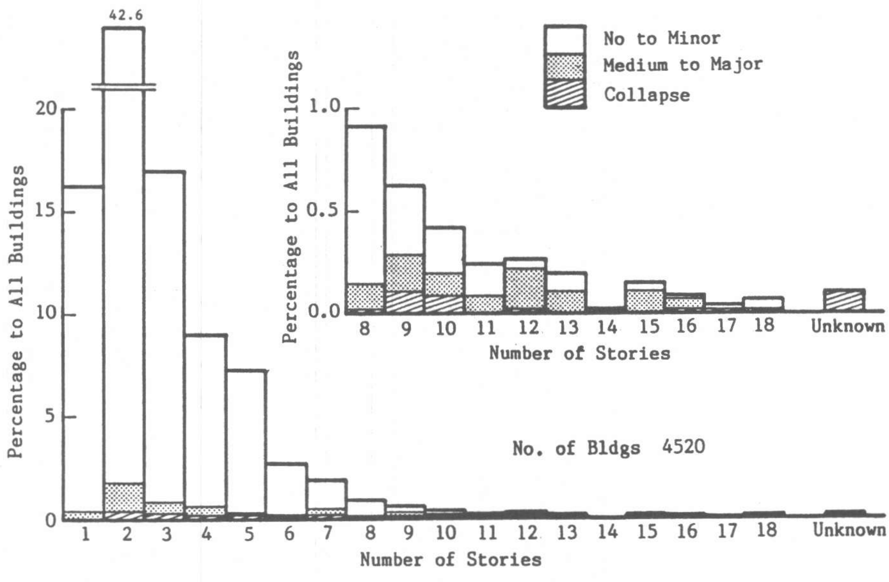


Fig.2: Summary of Damage Statistics

3. RESPONSE ANALYSIS

In order to correlate the observed damage with the characteristics of the ground motion and structures, a series of SDF nonlinear systems were analyzed under each horizontal component of the earthquake motion (Ref.2) recorded at the three stations (Fig.1) in the lake bed zone (Table 1; Ref.2). The absolute acceleration response spectra of the records are shown in Fig.3 for a damping factor of 0.05. Unlike the response spectra of normal earthquake motions, the response spectra have a peak in a period range longer than 1.0 sec, exhibiting the influence of soft ground. Record SCT1 developed by far the largest response amplitudes.

Table 1: Ground Motions (Ref.2)

No.	ID	Dir.	Max.Acc. (G)	Recording Station
1	CDAF	NS	0.081	Free
		EW	0.095	field.
2	CDAO	NS	0.069	One-story
		EW	0.080	building.
3	SCT1	NS	0.098	Free
		EW	0.168	field.

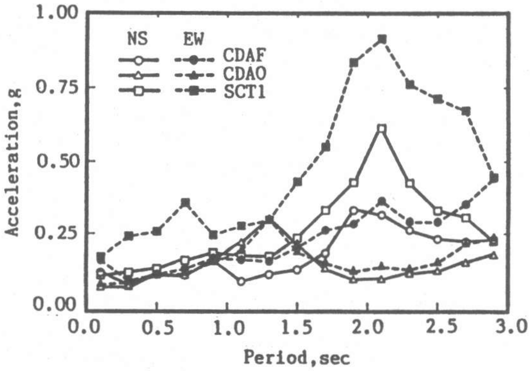


Fig. 3: Response Spectra at 0.05 Damping

Earthquake Design Loads: Earthquake resistant design code of Mexico City was first published in 1942, and revised in 1957 and 1977 after the disastrous earthquakes of 1957 and 1976, respectively. The 1957 Code introduced the effect of different soil conditions on the earthquake response; the dynamic analysis method in the 1966 Code; and the ultimate strength design in the 1972 Code. The Emergency Code (Ref.5) was issued approximately one month after the 1985 earthquake.

The 1977 Code (Ref.4), which prevailed at the time of the earthquake, determined the base shear coefficient c as function of fundamental period T and ductility factor Q (=1.0, 1.5, 2.0, 4.0, and 6.0); i.e.,

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

in which C: maximum response acceleration; a_o: maximum ground acceleration, selected on the basis of the maximum acceleration observed in the 1957 earthquake; and T_1 and T_2: corner periods (Table 2). Base shear coefficient c should not be less than a_o. The base shear coefficient for the lake bed zone is shown in Fig.4 for different values of Q. Note that (a) the elastic response acceleration (Fig.3) exceeds the elastic design base shear coefficient (Q=1.0; Fig.4) in a wide range of periods, and (b) the value a_o in the lake bed zone was significantly exceeded by the observed maximum ground acceleration (Table 1).

Table 2: Design Base Shear Coefficient

Seismic Zone	C	a_o	T_1	T_2	r
Firm Ground	0.16	0.03	0.3	0.8	1/2
Transition	0.20	0.045	0.5	2.0	2/3
Lake Bed	0.24	0.06	0.8	3.3	1

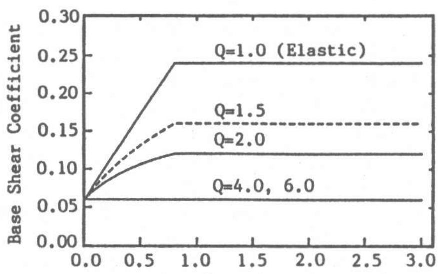


Fig. 4: Base Shear Coefficient (Lake Bed Zone)

SDF Systems: Takeda-slip Models (Ref.6) was selected to simulate the response of reinforced concrete buildings with small hysteretic energy dissipation. The skeleton curve under monotonically increasing lateral load was idealized by a tri-linear relationship with stiffness changes at "cracking" point (D_c, F_c) and "yielding" point (D_y, F_y), where $F_c=F_y/3$, and $D_c=D_y/12$. The yield resistance F_y was determined for the lake bed zone and for two extreme ductility factors (Q=1.0 and 4.0) using the 1977 Code (Ref.4). 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; the slip stiffness degradation parameter and reloading stiffness parameter were selected to be 1.5 and 1.0, respectively (Ref.7). The period was defined for the secant stiffness at yielding; mass was assumed to be unity, and the damping coefficient proportional to instantaneous stiffness with an initial elastic damping factor of 0.05. The structure-foundation interaction was not included in the analysis.

5. RESULTS OF RESPONSE ANALYSIS

For a design ductility factor of Q=1.0 (Fig.5), the ductility demand was well below the target for all range of periods under CDAF and CDAO motions. However, SCT1 record, especially EW component, caused yielding for systems at most period range. Note that ductility demand at periods less than 1.0 sec also exceeded the design target, which contradicts with the observed small damage in low-rise buildings. For Q=4.0 (Fig.6), the maximum response was comparable for the three records. The ductility demand significantly exceeded the design target (Q=4) for a period range shorter than 2.2 sec. The attained ductility exceeded three times the target design value for a period range shorter than 1.0 sec. The response increased as the system period decreased, the phenomenon which cannot be described by the elastic response spectra of the records.

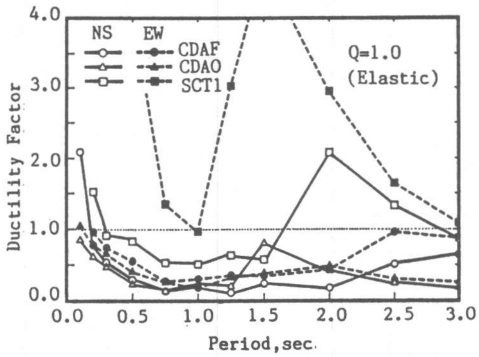


Fig.5: Response of Systems (Q=1.0)

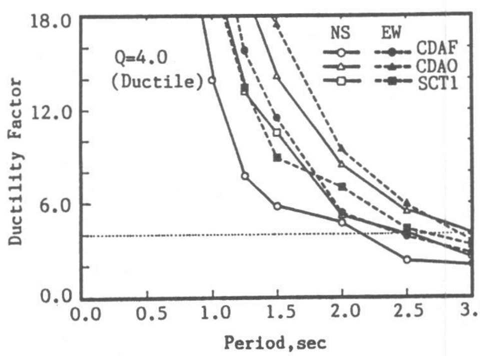
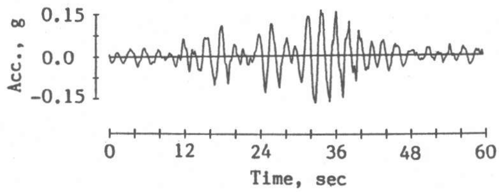


Fig.6: Response of Systems (Q=4.0)

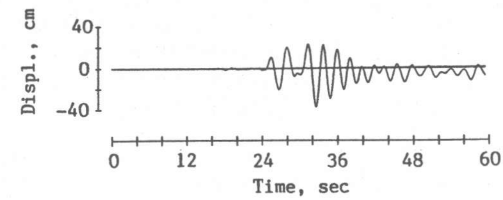
The significant exceedance of ductility in short period systems is attributable to the fact that the ground acceleration oscillated in a period much longer than the elastic period of systems and furthermore, at amplitudes (Table 1) larger than the lateral resistance of ductile systems (c=0.06 for Q=4.0). 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. 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.

In order to demonstrate 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.7). The design base shear coefficient was selected to be the same (=0.10) in the two systems. Consequently, the yield displacement (=0.22 cm) of the stiff system was twenty-fifth of that (=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 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.7.c) corresponded to the initial period of the system. At approximately 16 sec, when the ground acceleration reached the design base shear coefficient (=0.10), the response base shear coefficient reached the 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 by a factor of 7 to 10. The displacement response waveform of the stiff system became similar to that of the flexible system in amplitudes and periods after 32 sec.

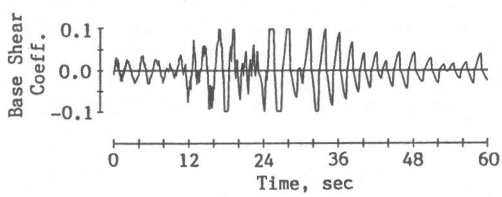
The above discussion appears to contradict the statistics of less damage in low-rise buildings in the lake bed zone, which indicates that the low-rise buildings generally were provided with lateral resistance much higher than what the code required for ductile buildings. The actual resistance of buildings is normally greater than the resistance required by the code attributable to inherent additional resistance of the structural as well as non-structural elements. In a low-rise buildings, the ratio of this increased resistance to the code required resistance is larger and the additional resistance can be readily provided because the total dead weight corresponding to the seismic inertia force is small. Furthermore, simple low-rise buildings are normally designed for a larger resistance, rather than for a larger ductility, to avoid the complex structural design and detailing requirements. Therefore, less damage must have occurred in low-rise buildings. On the contrary, heavy mid- to high-rise buildings are normally designed for a higher ductility to reduce the design earthquake load, and naturally the damage corresponding to the ductility developed in these buildings. The damage could have been reduced significantly by providing higher resistance.



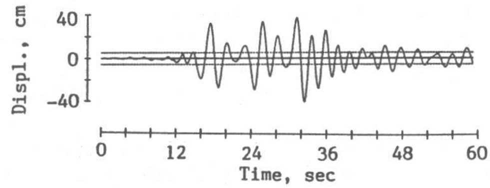
(a) EW Component of SCT1 Record



(b) Displacement of Stiff System



(c) Resistance of Stiff System



(d) Displacement of Flexible System

Fig.7: Response Waveform

6. CONCLUSIONS

The damage inventory survey was carried out in a limited number of areas in the severely damaged lake bed zone in Mexico City (Ref.1). 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 the planned earthquake resistant design for taller buildings.

Nonlinear earthquake response analysis was carried out for simple single-degree-of-freedom systems designed for the lake bed zone in accordance with the 1977 Construction Regulations for the Federal District of Mexico (Ref.4). The response of elastically designed systems was satisfactory, whereas the response of ductile systems, with yield periods shorter than 2.2 sec, exceeded the design target ductility significantly because the ground acceleration oscillated in a period much longer than the period of systems and furthermore, at amplitudes larger than the lateral resistance of ductile systems.

The response analysis results and the earthquake damage statistics indicates that the low-rise buildings generally were provided with lateral resistance much higher than what the code required for ductile buildings attributable to inherent additional resistance of the structural as well as non-structural elements and to design practice for a larger resistance to avoid the complex design requirements. On the contrary, mid- to high-rise buildings, normally designed for a higher ductility, had to develop the damage corresponding to the expected ductility. The damage could have been reduced significantly by providing higher resistance.

ACKNOWLEDGMENT

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