PERFORMANCE OF THE BUILDING OF FACULTY OF ENGINEERING, TOHOKU UNIVERSITY, DURING THE 1978 MIYAGI-KEN-OKI EARTHQUAKE

by

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SYNOPSIS

A 9-story steel framed reinforced concrete building which houses the architectural and civil engineering departments of Tohoku University, was subjected to strong ground motion during the Miyagi-Ken-oki earthquake of June 12, 1978. The maximum acceleration of about 1G was recorded at the 9th floor of the building.

In this paper, the inelastic dynamic response analyses of this building are discussed considering the inelastic action of constituent members. The analysed results are compared with the damage caused by three strong earthquakes experienced by the building since its completion of 1969.

DAMAGE OF THE BUILDING CAUSED BY EARTHQUAKES

Description of the Building The plan and sections are shown in Fig.1. It is of steel framed reinforced concrete construction. The main part is 9 storied and the 2 storied wing parts are attached to the north and south sides. The seismic resistance is provided mainly by shear walls. The shear walls are located around the two cores and in the east and west sides. Foundations are of isolated footing type. Subsoil under the building is gravelly loam and concrete piles of 12m length are used.

Experienced Earthquakes Observation of earthquake response by SMAC accelerographs, shown in Fig.1., has been continued since its completion. The accelerograms for the three earthquakes having different intensity have been recorded to date. The maximum accelerations of each accelerograms are tabulated in Table 1.

No structural damage was seen in the earthquakes of September, 1970 and February, 1978. However, about 5.0% of window glasses were damaged in the February, 1978 earthquake.

In the earthquake of June 1978, small shear cracks and flexural cracks were observed in main shear walls, adjacent beams and a few columns of the third and fourth floor. Typical crack patterns of shear walls and adjacent beams are shown in Fig.2. The maximum width of shear cracks in shear walls and that of flexural cracks in columns and beams are about 1.0mm. The width of shear cracks in the adjacent beams with openings is about 1.5mm. Therefore, the structural damage of the building by the earthquake of June, 1978, is considered to be fairly minor and repairable. About 2.5% of window glasses were damaged and many furniture were overturned during this earthquake.

ASSUMPTIONS OF ANALYSES

Modelling of Members An analytical model of individual member is shown in Fig.3. In this analyses, inelastic flexural behaviour of beams, columns

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and shear walls, is modelled by the linear element(EI) and two inelastic rotational springs(k_{pA}, k_{pB}) placed at the ends of linear element. And inelastic shear behaviour of the bember is modelled by the nonlinear shear spring(k_s) placed at the center of linear element. Therefore, flexural and shear deformation of members are taken into account, while axial deformation is neglected.

The relationship between forces acting at the member ends (bending moments and vertical forces) and displacements can be stated as follows ;

$$\{ M_A \; M_B \; Q_A \; Q_B \}^T = [K] \{ \theta_A \; \theta_B \; u_A \; u_B \}^T \qquad \text{Eq. 1}$$

The matrix [K] is the stiffness matrix of the individual member. The deformations in flexural and shear springs are related to the displacements at the member ends in the following form ;

$$\{ \phi_{PA} \; \phi_{PB} \; \gamma \}^T = [X] \{ \theta_A \; \theta_B \; u_A \; u_B \}^T \qquad \text{Eq. 2}$$

The matrices in the above equations are stated follows ;

$$[K] = [C]^T [B]^T [F]^{-1} [B] [C] \qquad \text{Eq. 3}$$

$$[X] = [f] [F]^{-1} [B] [C] \qquad \text{Eq. 4}$$

where

$$\left.\begin{aligned}
&[F] = \frac{l'}{6EI} \begin{bmatrix} 2+f_A+s & -1+s \\ -1+s & 2+f_B+s \end{bmatrix} \qquad [B] = \begin{bmatrix} 1+\lambda_A & \lambda_B \\ \lambda_A & 1+\lambda_B \end{bmatrix} \\
&[C] = \begin{bmatrix} 1 & 0 & 1/l & -1/l \\ 0 & 1 & 1/l & -1/l \end{bmatrix} \qquad [f] = \frac{l'}{6EI} \begin{bmatrix} f_A & 0 \\ 0 & f_B \\ s & s \end{bmatrix} \\
&f_A = \frac{6EI}{l'} \frac{1}{k_{PA}} \qquad f_B = \frac{6EI}{l'} \frac{1}{k_{PB}} \qquad s = \frac{6EI}{l'} \frac{1}{k_S \cdot l'}
\end{aligned}\right\} \qquad \text{Eq. 5}$$

Hysteresis Rules for Member Hysteresis rules of members for flexural and shear deformations are idealized by the piecewise linear hysteretic relationship with break-points at cracking and yielding points, as shown in Fig.4. Flexural cracking and flexural yielding moments are calculated based on reference (2). Yielding rotations at member ends are determined assuming that columns and beams are subjected to point symmetric bending moments and shear walls are subjected to mirror symmetric bending moments, as shown in Fig.5. A stiffness degradation ratio α_y of 0.25 at flexural yielding is chosen for all members.

Shear forces for shear cracking and shear yielding of columns and beams are calculated based on the Arakawa's equation (2). A half values of these forces are used for shear forces for adjacent beams with openings. Shear strengths for shear walls are calculated based on cracking shear stress τ_c=0.1Fc and ultimate shearing stress τ_u=0.25Fc, (Fc ; compressive strength of concrete), (3). The translational angles for shear cracking and yielding are assumed to be $\gamma_c=0.4\times10^{-3}$rad. and $\gamma_u=4.0\times10^{-3}$rad., respectively, for all members.

Idealized Structure The building is idealized into rectangular frames by centroidal axis for response analyses as shown in Fig.6. Only one half of the building frame is considered because of the symmetry of the building.

The stiffness and the strength of each shear walls are evaluated taking account of the flange effect of perpendicular walls. Rocking springs with reasonable values are attached to foundations of shear walls, which are determined from the observed rocking ratio in the forced vibration test (1). The calculated first natural periods for the transverse and longitudinal directions of the building in the elastic range are 0.59 and 0.55 second.

ANALYSES OF THE PERFORMANCE OF THE BUILDING

Analytical Method The inelastic response analyses of the idealized plane structure were done using the recorded accelerations at the first floor during the earthquakes of February, and June, 1978.

A step-by-step numerical integration procedure is used. The time interval for integration adopted in the analyses is 0.002 second.

Damping of 0.04 for the first elastic mode is considered in the level of overall structure.

Results Figs. 7 and 8 show the comparison of the recorded and calculated response waveforms at the ninth floor of the building subjected to the two earthquake motions. The calculated response waveforms are generally similar to the recorded response waveforms in terms of periods and amplitude of the responses. But considerable difference in response amplitude between calculated and recorded response is observed in several parts in the transverse direction. It is inferred that this difference is due to the effect of torsional vibration.

Distribution of member response for the earthquake of June, 1978 is shown in Fig. 9. The numerals on the figure indicate the flexural ductility factor μ at the maximum response.

For the frames in the transverse direction, ductility factors are 2∼3 at bottoms of walls in ② and ⑦ frames and at ends of beams in the fourth to the roof floor in ③∼⑥ frames. For the frames in the longitudinal direction, ductility factors are 1.5∼2.0 at beams in the third to the roof floor in Ⓓ frame.

Calculated distributions of shear cracks in shear walls in ② and ⑦ frames and in adjacent beams of ③∼⑥ frames agree well with the actual damage distribution caused by the earthquake of June, 1978.

CONCLUSIONS

The response analyses of the building of Faculty of Engineering, Tohoku University subjected to the earthquakes of February, and June, 1978, were done, considering the inelastic action of each constituent members.

The analysed responses of the building to the two earthquakes agreed well with the recorded response accelerations and the observed distributions of cracked members.

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Table 1 Recorded and Calculated Maximum Accelerations

Earthquakes	*1 X (km)	*2 M	*3 I	*4 Direc.	Max. Accelerations (gal) Recorded 1FL	Recorded 9FL	Calculated 9FL
Sept.14, 1970	134	6.2	4	NS	46	79	-
				EW	33	103	-
Feb. 20, 1978	126	6.7	4	NS	170	421	463
				EW	113	298	318
Jun. 12, 1978	112	7.4	5	NS	258	1040	712
				EW	202	523	544

*1 : Distance from Epicenter
*2 : Magunitude
*3 : Intensity Scale at Sendai
(Japan Meterological Agency)

*4 NS : Transverse Direction
EW : Longitudinal Direction

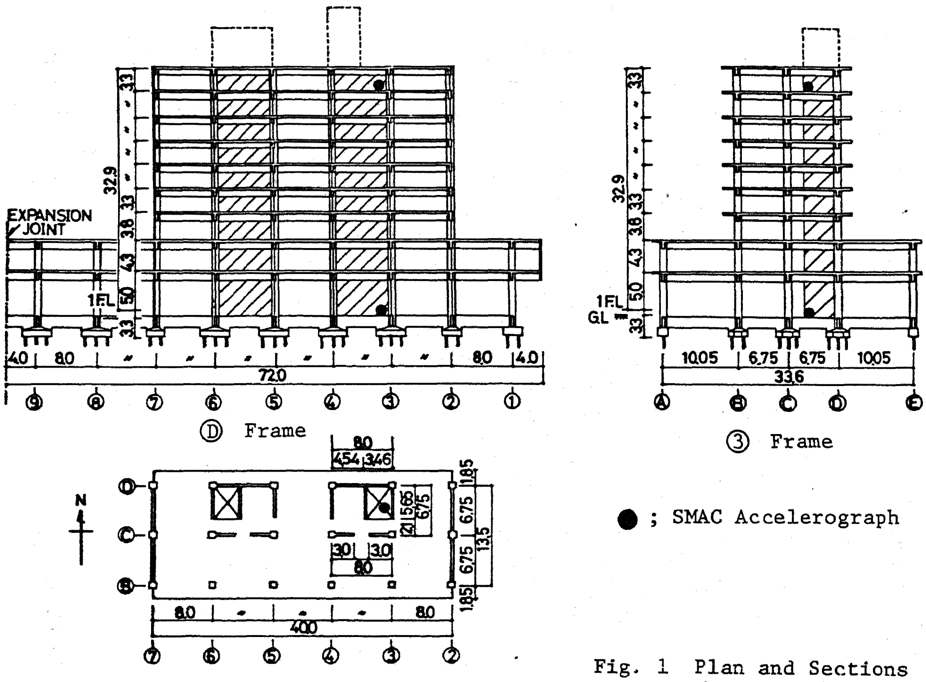
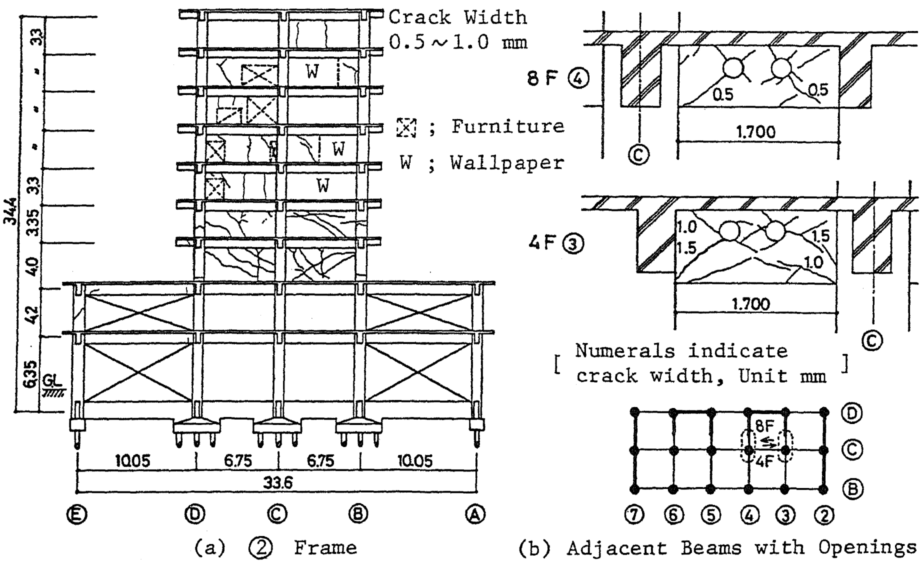


Fig. 1 Plan and Sections of the building



(a) ② Frame (b) Adjacent Beams with Openings

Fig. 2 Crack Patterns Caused by The Earthquake of June 12, 1978

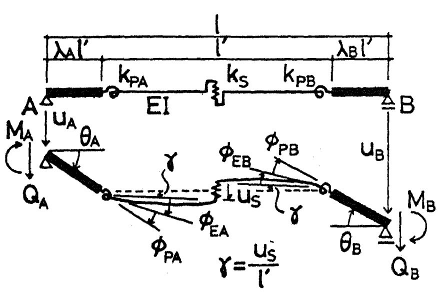
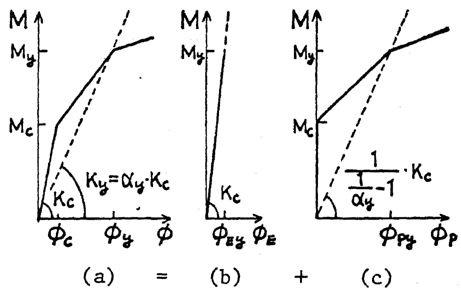
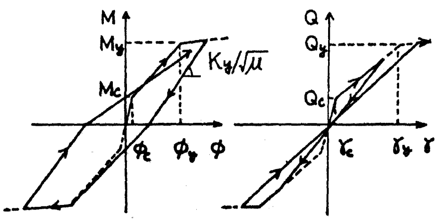


Fig. 3 Analytical Model of Members

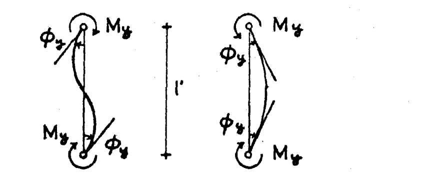


$\mu = \phi/\phi_y = (\phi_E+\phi_P)/(\phi_{Ey}+\phi_{Py})$



(a) Flexural (b) Shearing

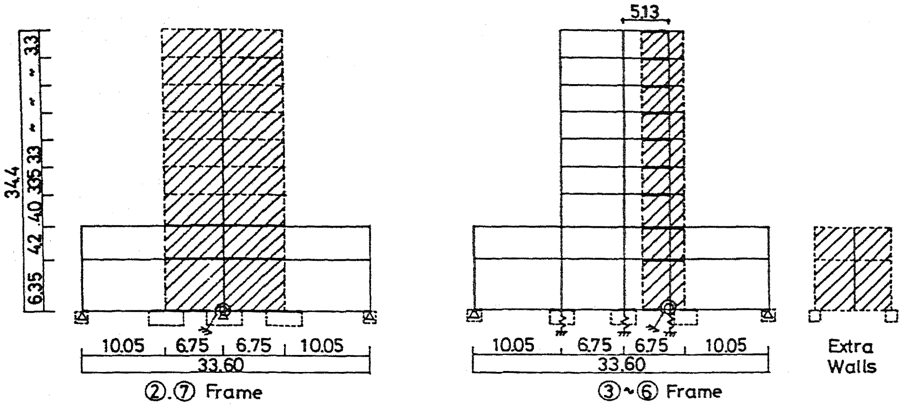
Fig. 4 Idealized Hysteresis Curves



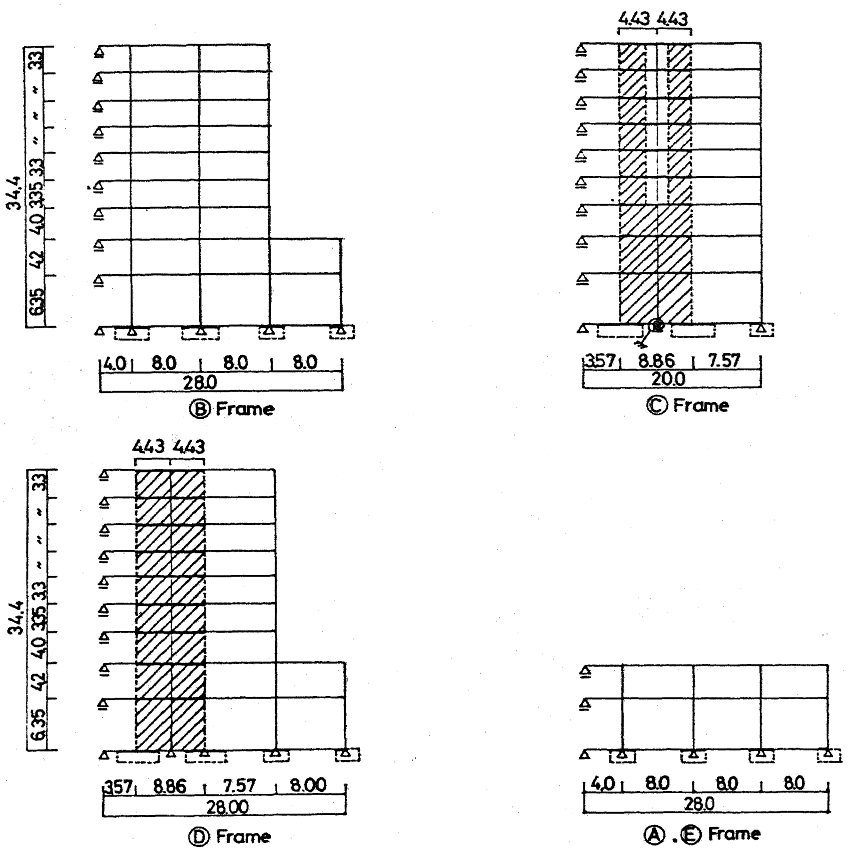
$\phi_y=(M_y \cdot l')/(\alpha_y \cdot 6EI)$ $\phi_y=(M_y \cdot l')/(\alpha_y \cdot 2EI)$

(a) Columns and Beams (b) Shear Walls

Fig. 5 Assumed Flexural Deformed Shapes for ϕy

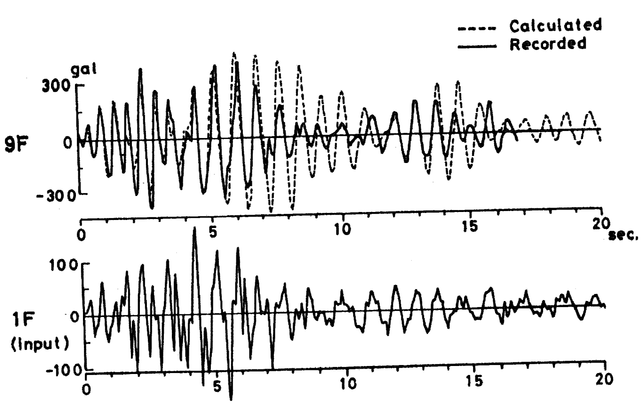


(a) Transverse Frames

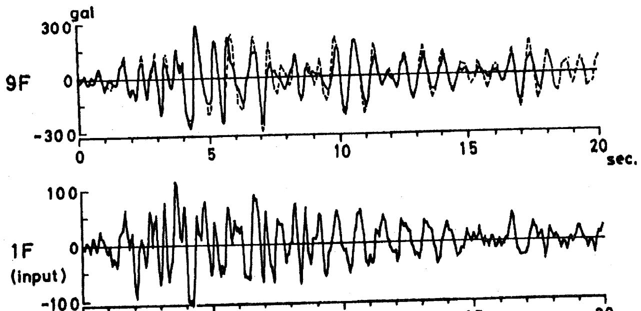


(b) Longitudinal Frames

Fig. 6 Idealized Model of the Building

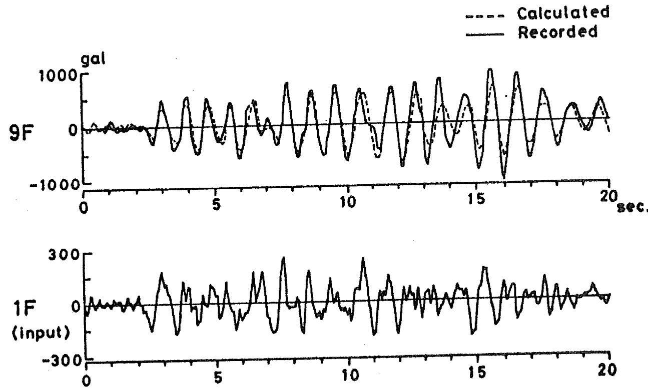


(a) Transverse Direction

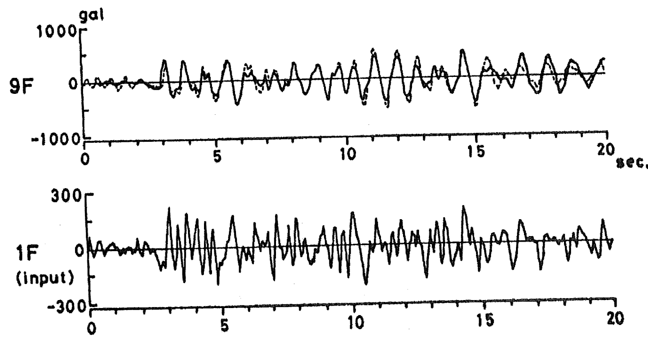


(b) Longitudinal Direction

Fig. 7 Response Accelerations to
The Earthquake of February, 1978

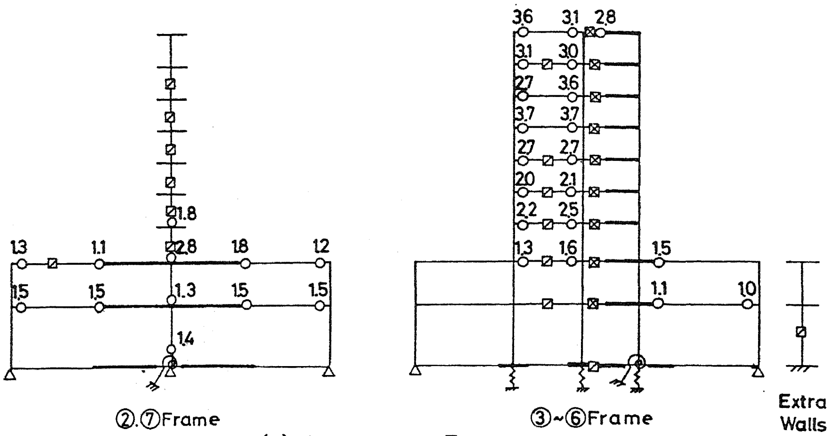


(a) Transverse Direction

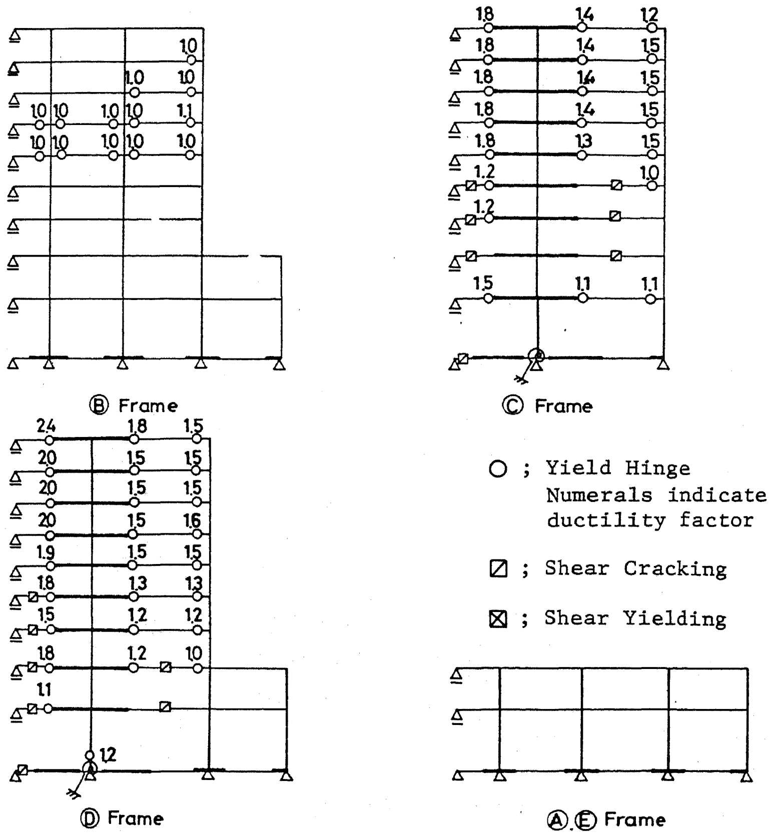


(b) Longitudinal Direction

Fig. 8 Response Accelerations to
The Earthquake of June, 1978



(a) Transverse Frames



(b) Longitudinal Frames

Fig. 9 Calculated Structural Damage Distribution to The Earthquake of June 12, 1978