5.2 Building Responses

A great number of modern RC buildings suffered from severe damage to collapse during the Chi-Chi Earthquake. Nevertheless, there are the buildings of same structure, even in the near sites with similar design and construction, remained undamaged or only sustained minor damage. It shall not be explained the different damage only in the point view of structure. However, the first one of the effective factors easy to find from relevant reports is the much higher PGA than design specifications (M.-S. Sheu, et al 2000). The design PGA of existing Taiwan Building Code is 0.23 G in disaster area (Taiwan MOI, 1997), while the near fault records in Sun-moon Lake was over 1.01G (Y.B. Tsai and M.W. Huang, 2000). A building shall not be expected to stand with the ground motions of high PGA that is three or four times to its designed load carrying capacity. Calculating the building response even would not be carried out because the response would be too large beyond the numerical process. In the other hand, the building data of Taiwan is not really opened for outside researchers for that there might be disputes over the responsibilities for the building damage and failure. Our interests then become to look at the responses of well seismic-designed building structure subjected to such intense ground motions. Therefore, a building designed and constructed according to the current Japanese design code is selected for the analytical study.

This section presents the analytical study on the responses of a high-rise frame-wall (HFW) RC apartment building subjected to the near fault acceleration records obtained from the Chi-Chi Earthquake. The building was designed and constructed in Kobe area in 1992 and sustained minor damage in the 1995 Hyogoken-Nanbu Earthquake. Therefore, the acceleration record of Sannomiya, Kobe is used to calculate the response as well and to compare the results. The HFW building is selected for the analytical study is not only for its interesting location but also for that the HFW apartment building has been adopted as earthquake-resistant structure and promoted in construction for urban residential buildings. Research and development by Japanese public and private organizations in collaboration have been carried out on the HFW building to ensure its seismic performance and load carrying capacity.

The building is idealized by three-dimensional frame model, considering the nonlinear behavior of individual structural members in sophisticated member analysis model, the multi-spring model. The results indicated that the building has moment-resistant mechanism in frame direction while shear failure in wall direction. It meets the design requirements in the load carrying capacity and behaves satisfactorily against the ground motion with PGA lower than 0.7G. However it may result in severe damage to the ground motion with PGA ranging 0.7~0.8G, and may collapse to the PGA over 1 G.

5.2.1 The Building Outline

The existing HFW building locates in Kobe city on mountainside near or in the severe damaged area (Figure 5.2.1). There are two adjacent apartment buildings, number 1 with beam-column frame in East-West direction and number 2 in North-South direction. Different from the number 2 building suffered major damage, the number 1 building sustained only minor damage. The damage investigation and preliminary analysis had been carried out soon after the 1995 earthquake (M. Hirosawa and Y. Yamamoto, 1996).

In this report, the number 1 building is used in the analytical study. It was 10-story 28.5 m high with 2-story pent houses, and 14-span 78.5 m in longitudinal frame direction, and total 136 residential units. The typical floor plan and frame elevation are shown in Figure 5.2.2 and 5.2.3. The section data of beam, column and shear wall of the earthquake resistant frames are listed in Table 5.2.1 to 5.2.3, and typical beam-column sections shown in Figure 5.2.4. The footing beams are the same thickness with the upper floor beams but with the depth in 2.3 to 2.9 m high. Pile foundation was used to support the building. The pile was cast in place RC circular piles with diameter 1.2 m and 1.4 m for exterior columns and staircases, and 1.6 m for interior columns. The pile length was 7.65 m reached rock layer (below ground level -9.55 m). The soil penetration number was about 40 in average.

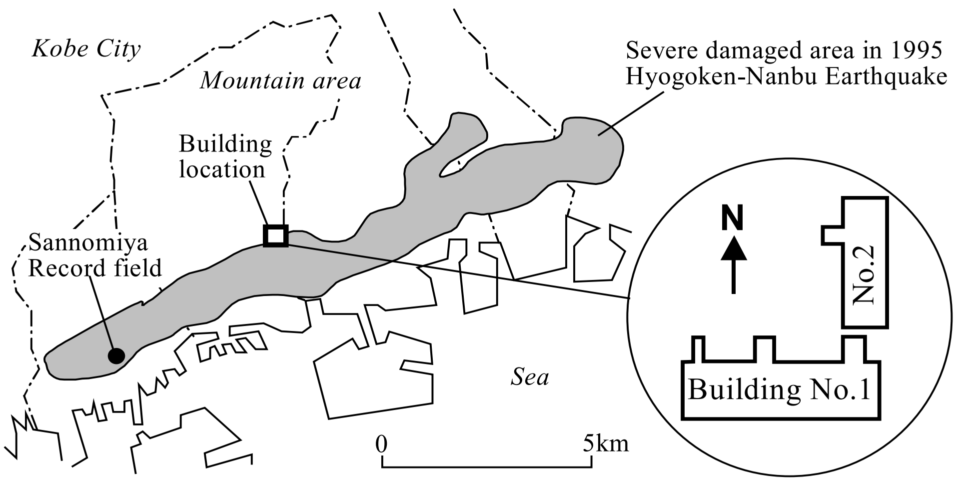


Figure 5.2.1. Location of the Building Used in the Analysis to Taiwan Chi-Chi Earthquake Records.

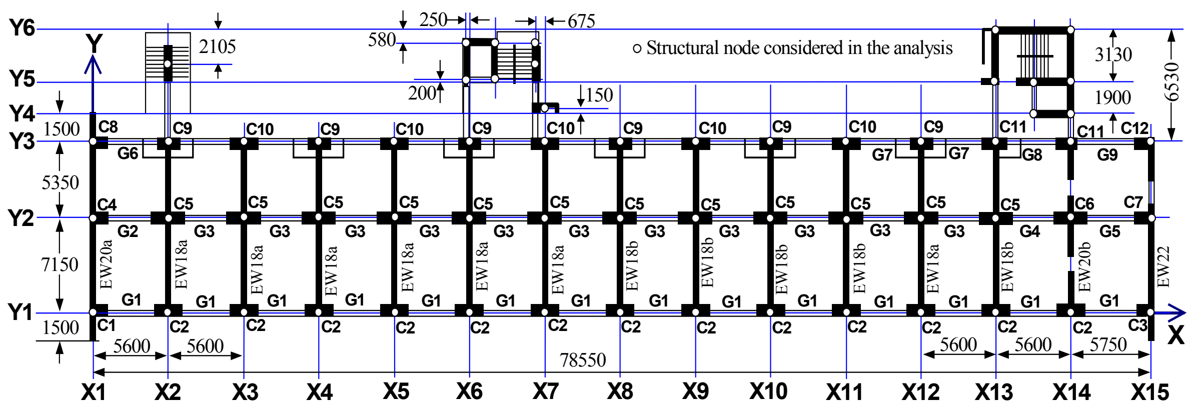


Figure 5.2.2. Typical Floor Plan and Structural Model (Unit in mm).

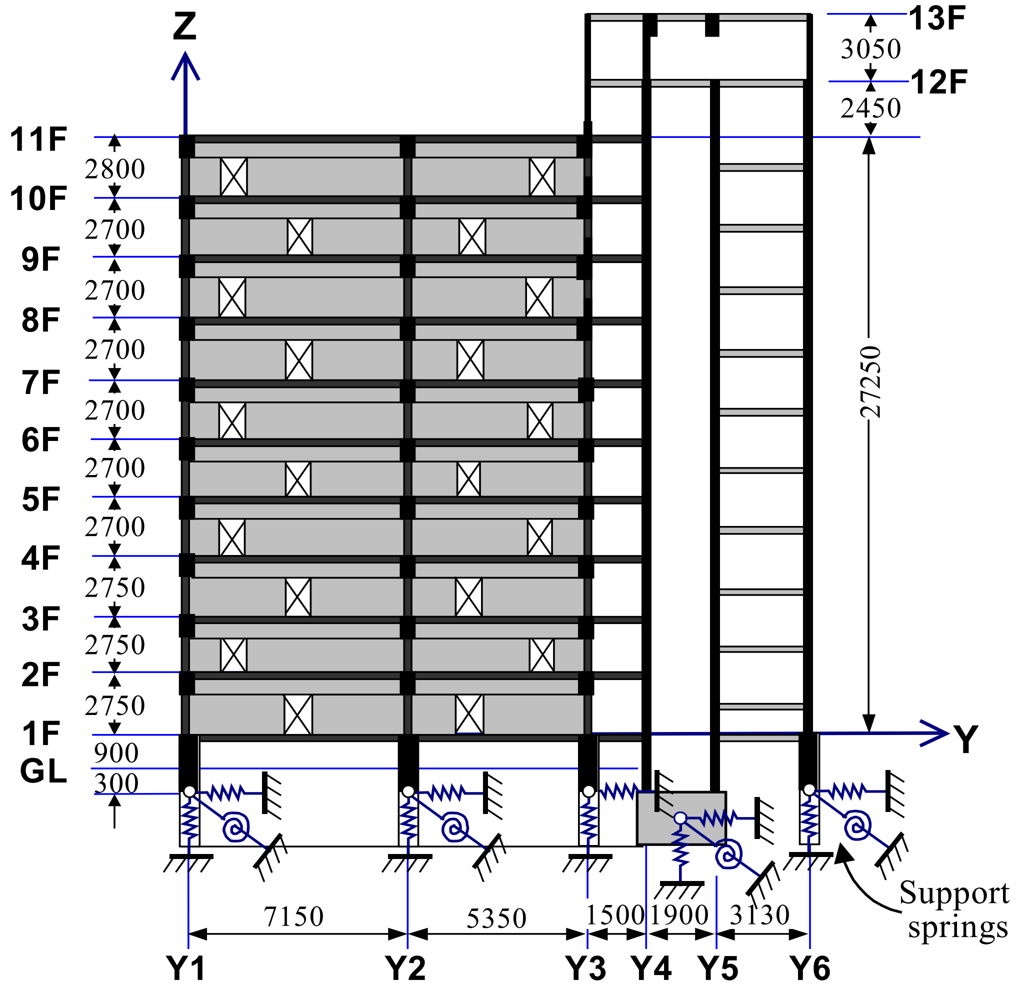


Figure 5.2.3. Typical Frame and Modeling of the Foundation Support (Unit in mm).

Table 5.2.1. List of Sections of Beam Members.

Section B×D (mm)		Longitudinal steel bar (top/bottom)	
Upper floor 5F~11F		4D25/3D25	10~11F G1~G9
G1	450×700	5D25/3D25	9F G1~G9
G2,G3,G4,G5	430×700	5D25/4D25	8F G1~G9
G6,G7,G8,G9	450×670	6D25/4D25	7F G1~G9
Lower floor 2F~4F		6D25/5D25	6F G1 G6~G9
G1	450×750	7D25/5D25	6F G2~G5, 5F G1 G6~G9
G2,G3,G4,G5	430×750	7D25/6D25	5F G2~G5, 4F G1 G6~G9
G6,G7,G8,G9	450×720	8D25/6D25	4F G2~G5, 2~3F G1~G9

Hoop and stirrups	
2D13@200	9~11F G1 G6, 11F G5 G8, 10~11F G7, 7~11F G9
2D13@150	7~8F G1, 11F G2 G3, 10F G5 G8, 8F G6, 9F G7, 6F G9
2D13@100	All others
3D13@100	2~4F G1 G6, 7F G2, 7~9F G3, 8~9F G4, 6~7F G5, 2~5F G7, 4~6F G8
4D13@100	6F G2 G3, 2~5F G5, 2~3F G8
3D16@100	2~5F G2 G3, 2~6F G4

Table 5.2.2. List of Sections of Column Members.

Section Size B×D (mm)							
C1	450×800	C4	430×1000	C7	430×1020	C10	450×1600
C2	450×1400	C5	430×1800	C8	450×900	C11	450×1300
C3	450×820	C6	430×1800	C9	450×1000	C12	450×920

Longitudinal bar D25 (middle bar 8D16, 12D16, 16D16, default 4D16)					
C1	C3	C4	C5(12D16)	C7	C9
10~11F 8D25	10~11F 8D25	10~11F 8D25	10~11F 10D25	10~11F 8D25	10~11F 12D25
8~10F 12D25	4~10F 12D25	9~10F 10D25	1~10F 14D25	7~10F 10D25	7~10F 24D25
4~8F 14D25	1~4F 16D25	4~9F 12D25		3~7F 12D25	6~7F 20D25
1~4F 18D25		3~4F 14D25	C6(12D16)	1~3F 20D25	1~6F 16D25
C2 (8D16)	C11(8D16)	2~3F 16D25	10~11F 10D25	C8, C12	C10(8D16)
10~11F 10D25	10~11F 12D25	1~2F 18D25	4~10F 14D25	5~11F 10D25	9~11F 12D25
1~10F 14D25	1~10F 16D25		1~4F 16D25	1~5F 16D25	1~9F 14D25

Hoop and stirrups			
3D13@100	2~3F C2 C5 C6 C11, 1~2F C10	4D13@100	1~2F C2 C5
3D16@100	2~3F C9, 1~2F C6 C9 C11	2D13@100	All others

Table 5.2.3. Shear Wall Sections (Unit in mm, H=horizontal bar, V=vertical bar).

Wall	Thickness	Steel bar (without indication horizontal/vertical bar D10@200 double)	
EW18a	180	Horizontal	1~2F D13@200, 2~4F D10/D13@200
		Vertical	1~4F D10/D13@200
EW18b	180	Horizontal	1~2F D13@150, 2~3F D13@200, 3~5F D10/D13@200
		Vertical	1~5F D10/D13@200
EW20a	200	Horizontal	1~2F D13@150, 2~4F D10/D13@200
		Vertical	1~4F D10/D13@200
EW20b	200	Horizontal	1~3F D13@100, 3~4F D13@200, 4~5F D10/D13@200
		Vertical	1~5F D10/D13@200
EW22	220	Horizontal	1~3F D13@100, 3~4F D13@150, 4~6F D10/D13@200
		Vertical	1~6F D10/D13@200

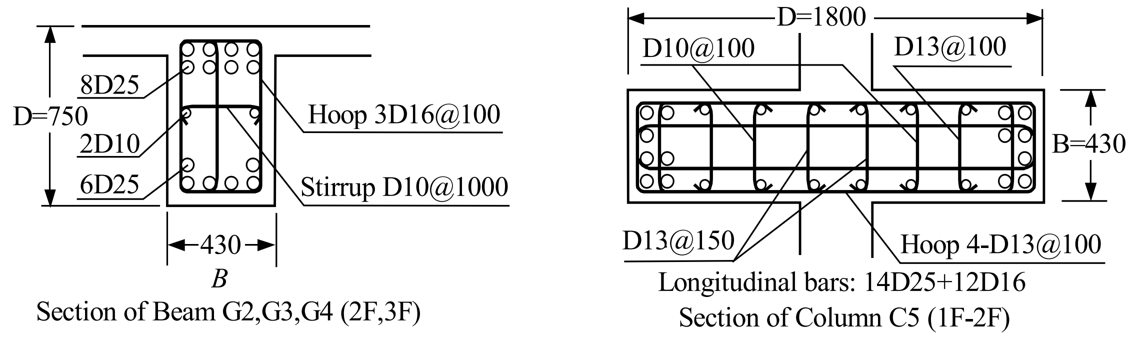


Figure 5.2.4. Typical Member Sections.

Table 5.2.4. Structural Weight (kN).

Floor level	Penthouse	11F	10F	5~9F	4F	3F	2F	ΣW_i
W_i	2983	13265	16140	16093	16232	16279	16506	161870

5.2.2 Analysis Model and Method

Building model

The building is idealized with limited number of displacement degrees of freedom at rigid structural nodes, which are placed at the intersection of each two axial lines X1 to X15 and Y1 to Y6 (as shown in Fig. 5.2.2 in white circles). The axial lines are considered coincident with shear wall's centerline. The structural nodes of the first floor level (1F) are located at the center of footing beams.

Each structural node has five displacement degrees of freedom: three translational displacements along the global axes X, Y and Z, and two rotations in the vertical planes (X-Z plane and Y-Z plane). The torsional stiffness of individual vertical members is ignored. Therefore, the rotation of individual structural nodes in horizontal X-Y plane is not included. However, floor slabs are treated as rigid diaphragm in the floor plane and have three displacement degrees of freedom, the two horizontal translations and the rotation in horizontal plane, that govern the structural nodes' horizontal displacements.

The weight of the building used for mass matrix in the analysis is calculated based on the structural self-weight plus 20 kN/m^2 live load per floor area. The results are given in Table 5.2.4.

Member model

Columns and shear walls are connected to relevant structural nodes at the member-end or through rigid zone or rigid bars, and are represented by line element located at the axial line of the member. The

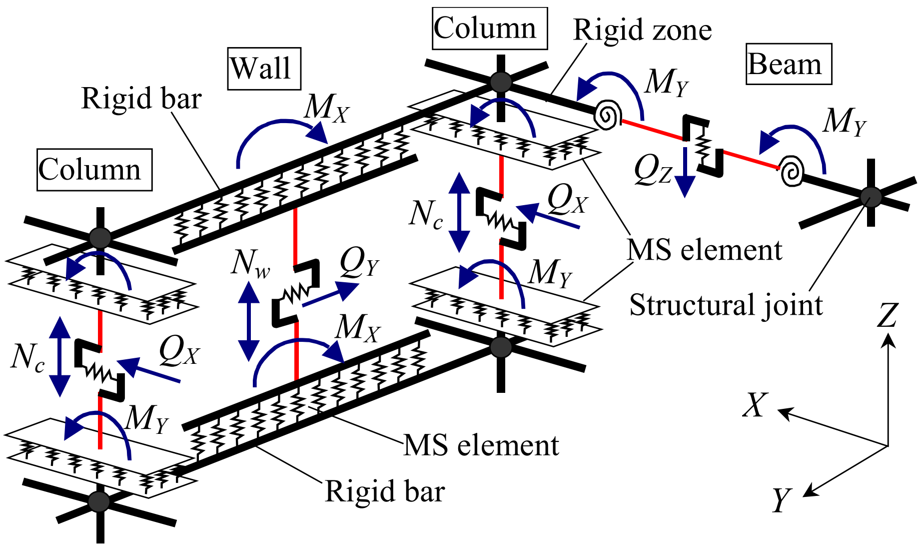


Figure 5.2.5. Illustration of the Analysis Model for Column, Wall and Beam Members.

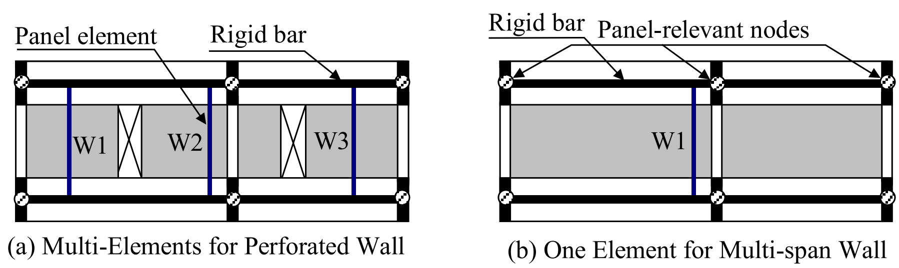


Figure 5.2.6. Modeling of Perforated Shear Walls and Multi-Span Shear Walls.

offset of the line elements to the structural nodes is taken in account in the computation. The analysis models for beam, column and shear wall are shown in Figure 5.2.5. Beam elements are considered to contribute bending and shear stiffness to the frame plane only (uniaxial flexural member), and idealized by rotational spring and shear spring. Column elements have axial tension and compression together with the flexural and shear contribution in the frame plane. The axial force of the column elements contributed to the resistance in the shear-wall direction as well. Panel element is used to idealize the shear walls. Panel element is treated similar to column element, i.e., it has flexural and shear contribution in the wall plane as well as axial tension compression. The panel element is associated with four or more structural nodes through the rigid bars on the wall base and topside. The rigid bar represents the assumption of plane section deformation of the panel element, keeps the column and panel element displacement compatible, and makes the column axial force contribute to lateral resistance in the transverse direction together with the panel element. Perforated shear walls are idealized by multiple panel elements that co-work through the rigid bars on the panel base and topside. While a single panel element is used to represent the shear wall extending into two or more spans (Figure 5.2.6). The middle nodes on the panel base and topside are kept in displacement compatible with the panel corner nodes through the rigid bar.

Multi-spring model is used to represent the flexural bending and axial tension/compression of column and panel elements. It takes into account the couple effect between the axial force and bending moment. Method of determining the properties of the multi-spring model and the hysteresis rules used for the springs of the beam, column and panel elements are given in the references (Li et al, 1999).

The shear capacity of the elements is evaluated based on the section details using the BCJ recommended equations for RC member's ultimate strength (BCJ, 1997), while the flexural cracking and yielding strength of the beam elements are calculated by numerical method based on the material stress-strain relations and the assumption of plane section deformation. The material properties used in evaluating the element capacity, stiffness and spring properties are given in Table 5.2.5.

All the non-structural walls were connected by weakened section (intended as slit separation) to the adjacent structural members. Therefore, the stiffness of non-structural elements is ignored, only counting their weight into the response computation.

Modeling the foundation

The contribution of pile foundation is represented by translational and rotational support springs at the first floor level structural nodes. The spring stiffness is determined according to the AIJ recommendations (AIJ, 1988) and JRA specifications (JRA, 1996). The hysteresis model different in compression and in tension, as shown in Figure 5.2.7, is used for the vertical support spring to represent the uplift of the foundation under tension. Other support springs are treated as linear elastic spring.

Table 5.2.5 Material Properties (N/mm^2)

Concrete	1F~4F	4F~7F	7F~top
Fc	24.0	22.5	21.0
Ec	24500	23700	22900
Gc	9800	9500	9200

Steel bar (Es=200000)	σ_y
SD35 (D19 and above)	385
SD30 (D16 and below)	330

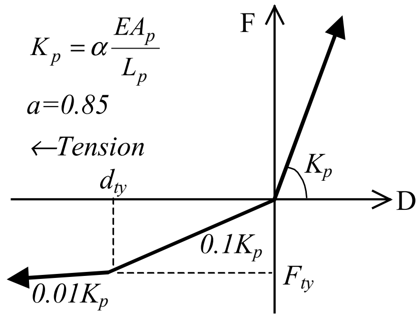


Fig.5.2.7 Pile Axial Stiffness Properties

Analysis method

The dynamic response analysis is carried out by CANNY program (Li, et al, 1999). The equation of motion is solved using step-by-step numerical integration method over a relatively small time interval of 1/500 second (about 1/240 of the elastic fundamental period of the building). In each time step, the stiffness of the structure and elements are treated as linear (piece-wise linear method). Iteration for equilibrium is not carried out but the overshooting due to stiffness change in the time step is brought into next time step to be corrected. Rayleigh's damping is used assuming mass and stiffness matrix proportional damping constant as 5 %.

Acceleration Records for the Input

Four Taiwanese records (Wufeng TCU065, Nantou TCU076, Sun-moon Lake TCU084 and Gukeng Chy028) near the Chelungpu Fault and near the epicenter of the 1999 Chi-Chi Earthquake (Figure 5.2.8) as well as the Kobe Sannomiya record of the 1995 Hyogoken-nanbu earthquake are used for the

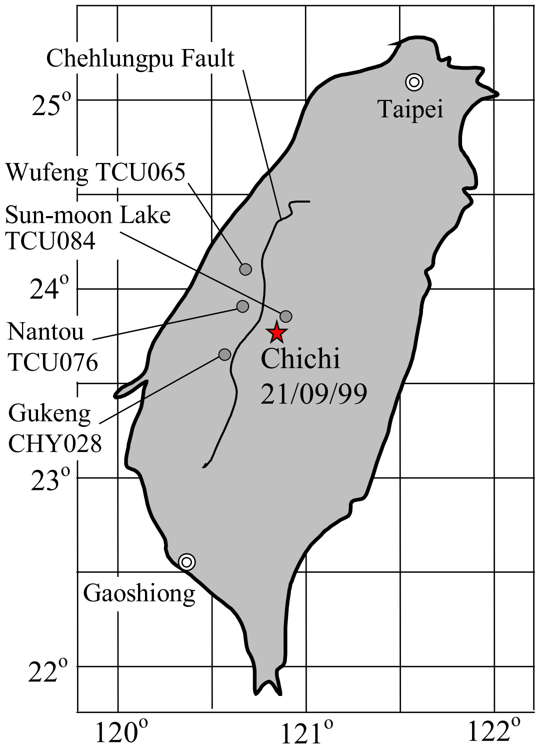
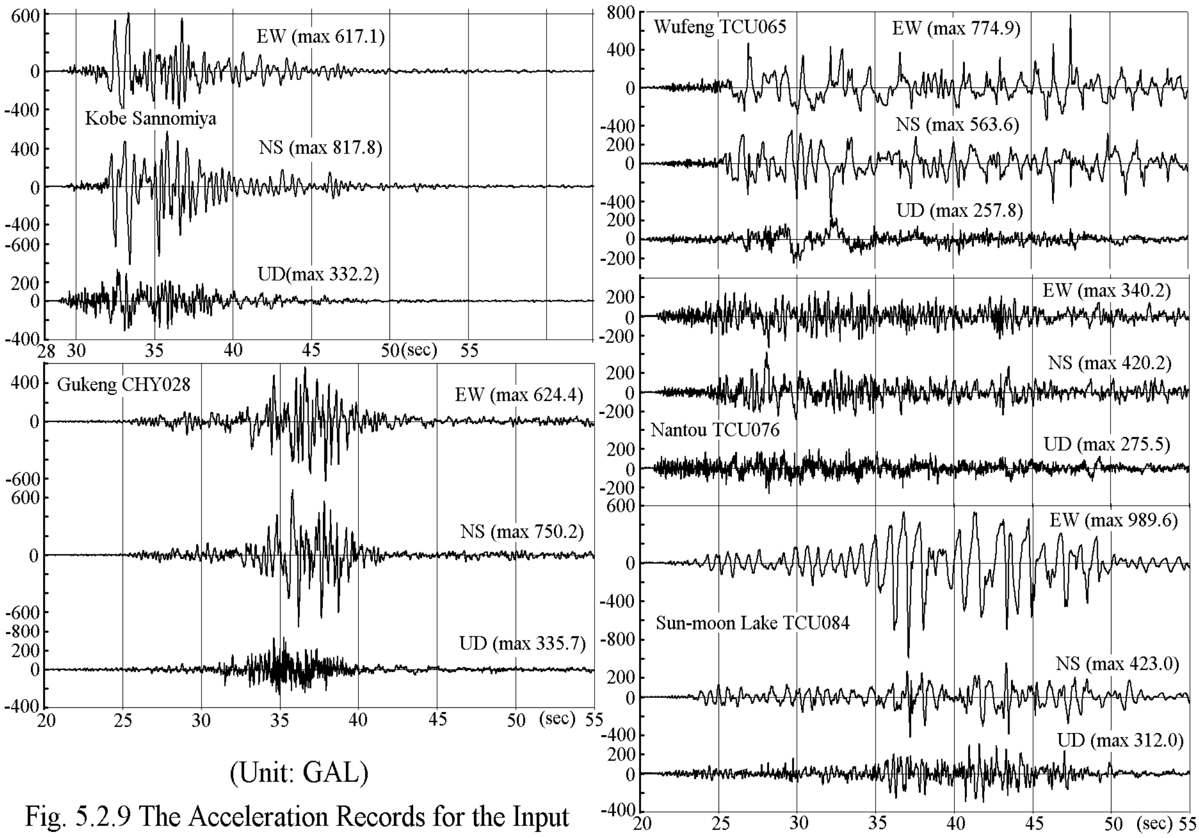


Fig. 5.2.8 Field of Strong Motion Records Used in the Analysis



(Unit: GAL)

Fig. 5.2.9 The Acceleration Records for the Input

input to calculate the building responses. The selected acceleration records have the PGA of horizontal components ranging from 0.35 to 1.01 G (Figure 5.2.9).

The input is made at the first floor level (1F). All the three components (EW, NS and UD) of the records are inputted in the X, Y and Z direction, respectively. The Station TCU084 at Sun-moon Lake recorded the highest PGA at 1.01G but have very different PGA in the EW and NS components. Therefore, changing the input direction of horizontal acceleration components was considered: (1) input of the EW-component in X-direction and NS-component in Y-direction input; (2) input of the EW-component in minus Y-direction and NS-component in X-direction. That is, the acceleration component with higher PGA is made input each against the beam-column frame direction and the shear wall transverse direction.

5.2.3 Analysis Results

The responses of the lateral displacement at top floor level, the maximum inter-story displacement, the maximum story-shear forces and the maximum ductility factors of beam elements are shown in Figure 5.2.10 to 5.2.13. The responses of material (steel bar) extreme ductility factor of vertical members are listed in Table 5.2.6. Other results of shear failure indicated are as followings:

Kobe Sannomiya and Wufeng TCU065: Lower-story columns result in flexural damage. Shear failure is indicated in some lower-story exterior columns and in almost all lower-story walls.

S. Lake TCU084: When input the EW-component in X-direction, most of the lower story (1st and 2nd-story) walls as well as some middle-story walls are indicated shear failure, and some lower story (1st and 2nd story) columns of exterior frames (X1 and X15) also experience shear failure. When input the EW-component in Y-direction, almost shear walls (middle and lower stories) result in shear failure, and some exterior columns of first to fourth-story experience shear failure as well.

Nantou TCU076: All columns remain in elastic stage. Only shear failure indicated in some lower-story walls in the frame X1, X6, X7, X13, X14, and X15.

Gukeng CHY028: Columns almost remain in elastic stage, except some first-story columns having steel yielding. Shear failure is indicated in the lower-story walls of some frames (X1, X6, X7, X13, X14, X15).

5.2.4 Discussion on the Analysis Results

Generally, the responses results of the HFW building can be categorized in to three different levels. First, the responses to the records with PGA over 800 GAL (Kobe Sannomiya and Taiwan Sun-moon Lake records) obviously indicate severe damage. From the excessive responses of element and material ductility factors and the inter-story displacements the Sun-moon Lake record (PGA over 1 G) has no doubt resulting the building model in collapse. Second, the responses to the PGA ranging 0.7~0.8G, i.e. the Wufeng and Gukeng records, depend on the characters of the acceleration waves. Wufeng record makes the responses of maximum inter-story displacement over 1/100 in the frame direction and in the lower story of shear wall direction as well. The relative large inter-story displacement occurred in the shear wall direction is cased by the shear failure of the walls. While the responses to Gukeng record almost remain in elastic except damage indicated in some lower story shear walls where the stiffness is enhanced by the staircase or by thicker shear walls. Third, the response to the Nantou record (PGA 0.42 G) has almost no damage.

The results of maximum inter-story displacement responses are almost even distributed over each story when the displacement is less or about 1/100. It means the building was proportionally designed with well-distributed stiffness along the vertical direction. From the results of maximum story-shear responses (Figure 5.2.12), the load carrying capacity is about 0.3W in the frame direction and over 0.5W in the wall direction. It meets the design requirements for load carrying capacity (see the JCB structural characteristics factor D_s-value, JCB, 1997).

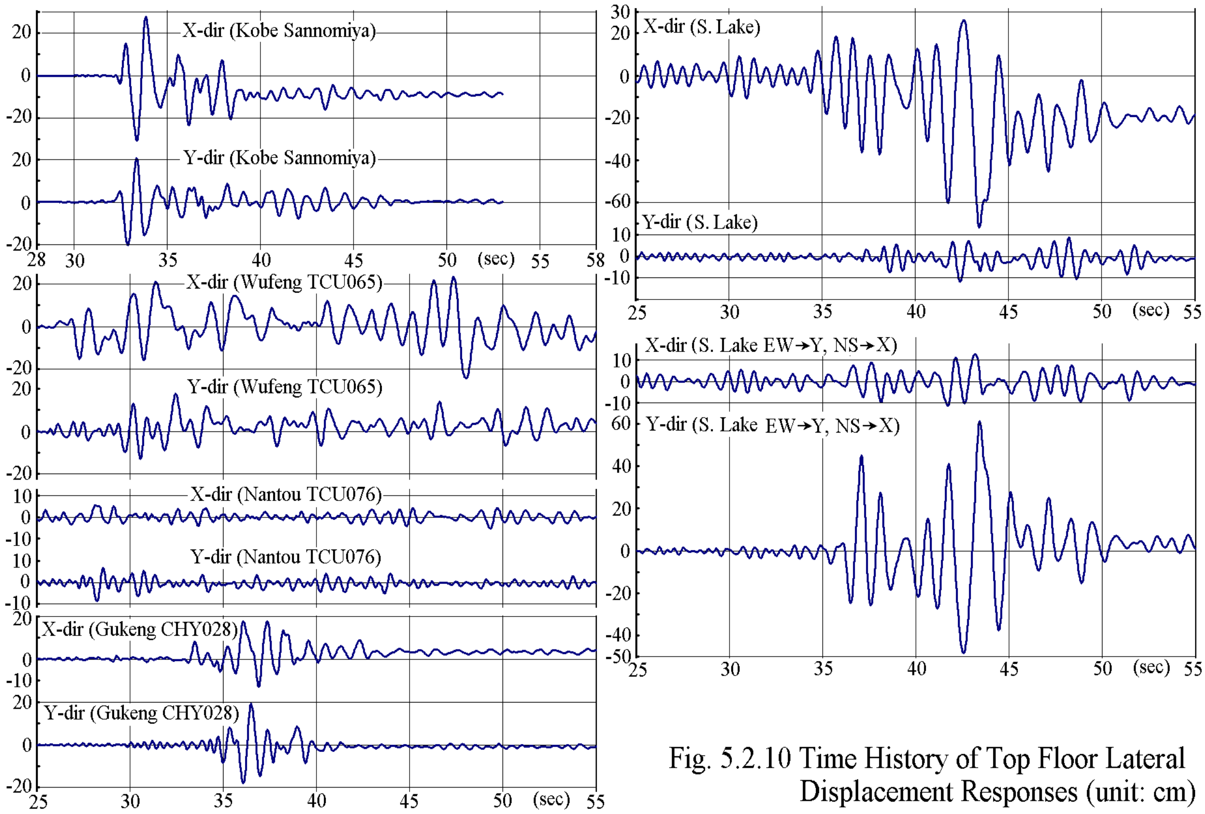


Fig. 5.2.10 Time History of Top Floor Lateral Displacement Responses (unit: cm)

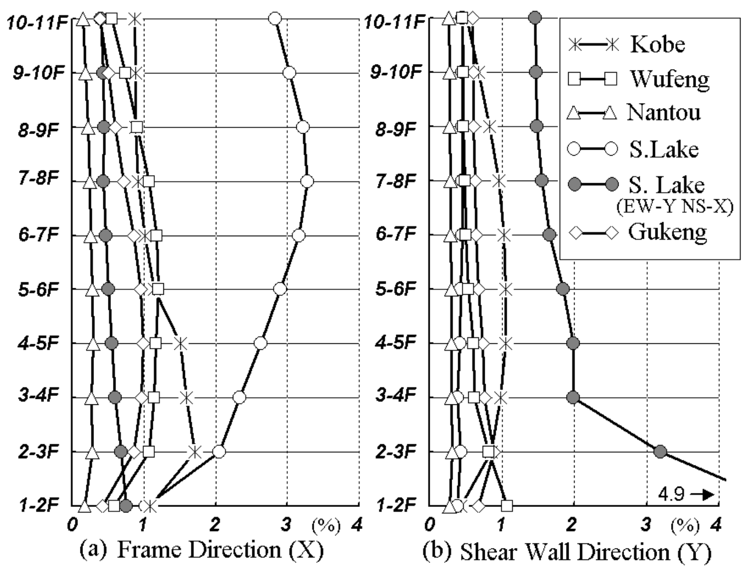


Fig. 5.2.11 Maximum Inter-Story Displacement Angle (%)

Table 5.2.6
Material Extreme Ductility Factor.

Input	D_{st}	D_{sc}
Kobe Sannomiya	19.7	7.4
Wufeng TCU065	15.0	13.3
Nantou TCU076	5.4	—
S. Lake (EW→X)	30.0	5.2
S. Lake (EW→Y)	91.1	23.0
Gukeng CHY028	6.5	3.0

D_{st}= Ductility of steel bar in tension
D_{st}= Ductility of steel bar in compression

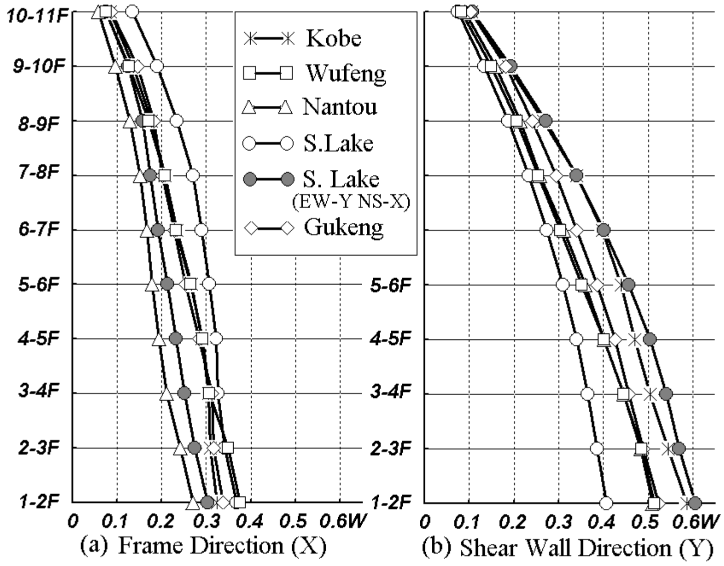


Fig. 5.2.12 Maximum Responses of Story Shear Force

Table 5.2.7
Fundamental Period T_p (sec).

Direction		X	Y
Elastic period		0.49	0.38
Kobe	T_p	0.90	0.81
Sannomiya	f_p/f_e	0.54	0.47
Wufeng	T_p	1.30	0.77
TCU065	f_p/f_e	0.37	0.50
Nantou	T_p	0.75	0.60
TCU076	f_p/f_e	0.65	0.63
S. Lake	T_p	1.34	0.71
(EW→X)	f_p/f_e	0.36	0.54
S. Lake	T_p	1.32	1.17
(EW→Y)	f_p/f_e	0.37	0.33
Gukeng	T_p	0.89	0.76
CHY028	f_p/f_e	0.55	0.50

f_p/f_e=plastic/elastic frequency ratio

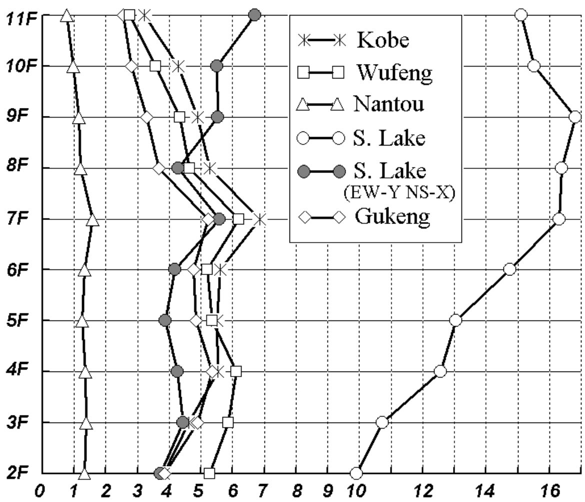


Fig. 5.2.13 Maximum Ductility Factor of Beams

The response results indicate moment-resistant mechanism in the frame direction. All beams result in flexural yielding, while columns keep strength and resistance thought shear failure might occur in some exterior columns. The cause of the column shear failure can be attributed to the increase of flexural strength when the column is under increasing axial compression due to over turning moment. The shear walls have shear failure mechanism. Lower story shear walls may have poor shear capacity.

Investigating the responses to the Sun-moon Lake record inputting the EW-components in the shear wall direction (Y-direction) and NS-component in the beam-column frame direction (X-direction), significant damage to the exterior columns (shear failure and the excessive material ductility factor) is indicated despite the lower PGA (0.43 G) of the NS-component inputted in the beam-column frame direction. This is explained as the attribution of the overturning moment in transverse shear wall direction caused more tension and compression in exterior columns compared with the input of the strong EW-component in longitudinal frame direction.

The fundamental period of the building before the earthquake response is 0.49 sec in the frame direction and 0.38 sec in the shear wall direction. After the response, the period is extended. That is attributed to the damage of the building. From the results given in Table 5.2.7, the ratio of the frequency before and after the responses may be used as a general measure of the structural damage.

5.2.5 Summary

The analytical study on a HFW apartment building is carried out to investigate the seismic behavior of building subjected to the input of near fault acceleration records obtained from the Chi-Chi, Taiwan Earthquake. Three-dimensional nonlinear model is in the responses analysis. The response results indicated that the building has moment-resistant mechanism in the frame direction while shear failure in the shear wall direction. The load carrying capacity meets the design requirements of Japanese design code. However, it may result in severe damage to the ground motion with PGA over 0.7 G. The results clearly indicate collapse of the building model to the very intense ground motion with PGA over one G.

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