SPECIAL REPORT

Lessons Learned from the Kobe Earthquake — A Japanese Perspective

Hiroshi Muguruma Ph.D.
Professor Emeritus
Department of
Architectural Engineering
Kyoto University
Kyoto, Japan

Minehiro Nishiyama Ph.D.
Department of
Architectural Engineering
Kyoto University
Kyoto, Japan

Fumio Watanabe, Ph.D.
Professor
Department of
Architectural Engineering
Kyoto University
Kyoto, Japan

This report presents an overview of the performance of reinforced and precast, prestressed concrete buildings during the Hyogoken-Nanbu earthquake (also known as the Great Hanshin earthquake) of January 17, 1995, situated in and around the city of Kobe, Japan. The performance of pile foundations is also examined. Highway bridges, rapid transit structures, and other special structures are covered elsewhere. The assessment of damage is related to the evolution of design code provisions for concrete building structures in Japan. Preliminary reports indicate that precast, prestressed concrete structures performed remarkably well during the earthquake, especially those designed with recent seismic code provisions. The probable causes of the damage are examined, although it should be emphasized that several investigations are currently being carried out to determine more comprehensive causes of structural failures by many researchers, engineers, the Architectural Institute of Japan (AIJ), the Japan Prestressed Concrete Engineering Association (JPCEA), and other organizations.

At precisely 5:46 a.m. in the early morning of January 17, 1995, a devastating earthquake struck Japan, imparting a trail of destruction across a narrow band extending from northern Awaji Island through the cities of Kobe, Ashiya, Nishinomiya and Takarazuka (see Fig. 1). The 7.2 Richter magnitude registered was one of the strongest earthquakes ever recorded in Japan.

Initially, Sumoto City on Awaji Island and Kobe City were assigned a Shindo 6 intensity. However, later the Japan Meteorological Agency (JMA) revised the Shindo intensity level from 6 to 7 for parts of the cities of Kobe, Ashiya, Nishinomiya and Takarazuka, and parts of northern Awaji Island. The Shindo intensities 1 to 7 correspond to the Modified Mercali Intensity Scale of I to II, II to IV, IV to V, V to VII, VII to VIII, VIII to IX, and IX to XII, respectively.

The Hyogoken-Nanbu earthquake (also called the Great Hanshin earthquake) will hereafter be referred to as the Kobe earthquake.

The earthquake resulted in 5502 deaths. More than 24,000 people were injured in the Hyogo Prefecture alone. As of June 1, about 40,000 people still live in temporary shelters. The estimated property damage ranges from \$95 to \$140 billion.

The earthquake caused significant damage not only to old buildings designed according to former design codes but also to modern buildings that conformed to current design codes and regulations. The performance of building structures during the earthquake has been studied by numerous researchers, engineers, and organizations. At this time, several clues to the causes of the devastating damage have been found.[1-5]

In this report, the damage to reinforced concrete buildings and precast, prestressed concrete buildings is assessed. Several possible causes for typical damage are presented. Also, the performance of precast, prestressed concrete piles is discussed. This report is limited to building structures. Special structures, such as highway bridges and rapid transit structures, are covered elsewhere by other researchers or engineers in the civil engineering field.

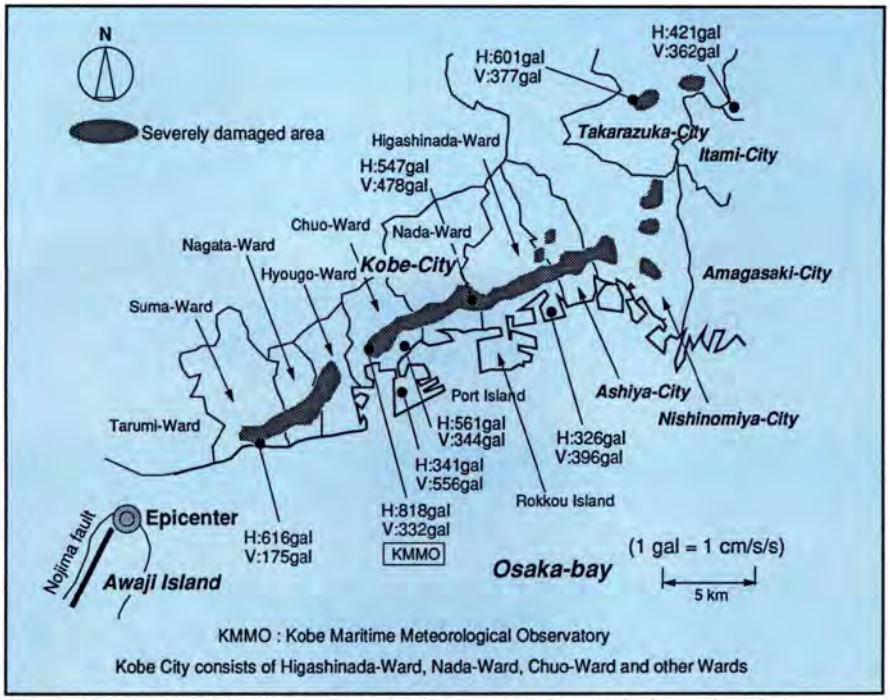


Fig. 1. Area map of severe earthquake damage and recorded accelerations.

GROUND MOTIONS

The focal depth of the earthquake was approximately 14 km (8.6 miles). The epicenter was located at 34° 36.4′ north latitude, 135° 2.6′ east longitude. Three faults are believed to have ruptured during the main shock. A horizontal displacement of up to 1.6 m (5.25 ft) was found at the Nojima Fault on Awaji Island. The severely damaged area consists of a narrow band from the northern part of Awaji Island to the city of Takarazuka. The cities of Kobe, Ashiya, and Nishinomiya are included in this region (see Fig. 1).

The Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake published by the Architectural Institute of Japan (AIJ)[1] states that the characteristics of the ground motions recorded may be summarized as follows:

1. Peak ground accelerations were large in both the horizontal and vertical directions. The peak accelerations observed at several sites are summarized in Table 1 and in Fig. 1.

2. Fig. 2 shows the velocity spectrum recorded by the Kobe Maritime Meteorological Observatory.

3. The duration of strong shaking was 10 to 15 seconds.

4. The predominant period was 0.8 to 1.5 seconds. A second predominant period was, at times, observed to be around 0.25 to 0.4 seconds.

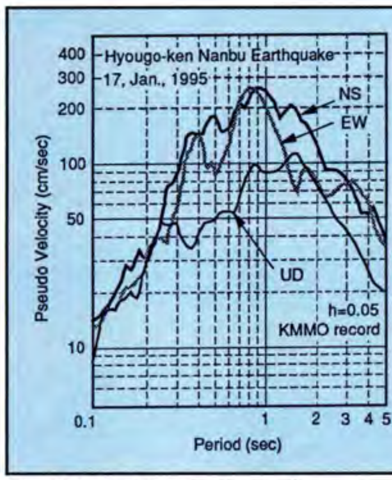


Fig. 2. Velocity spectrum of earthquake recorded by Kobe Maritime Meteorological Observatory.

5. Ground motions were affected by local soil conditions and topography. The preliminary reconnaissance report of AIJ states that ground motions were most likely amplified in the plains near the mountains between the cities of Kobe and Nishinomiya.

GEOLOGICAL ASPECTS

Because the geological aspects of the region are described elsewhere by experts in the field, this section will only briefly mention the highlights of the AIJ report.[1] The area of Shindo 7 intensity is approximately 20 km (12.3

Table 1. Peak accelerations and soil conditions (Ref. 1).

Recorded point	Location	Soil condition	Measured level	Peak ground acceleration (gal)		
				North-south	East-west	Up-down
JMA – Kobe	Chou Ward, Kobe City	Diluvial	1F	818	617	332
JMA – Osaka	Chou Ward, Osaka City	Diluvial	B3F	81	66	65
MTRC	Kita Ward, Kobe City	Rock	foundation	272	265	232
		Rock	GL - 15 m	208	213	116
A Building	Chuo Ward, Kobe City	Diluvial	B3F	223	208	292
B Building	Kita Ward, Osaka City	Alluvial	GL	182	267	302
C Building	Kita Ward, Osaka City	Diluvial	B4F	155	157	193
Site T	Minamikawachi District	Diluvial	GL	> 52	49	46
		Diluvial	GL - 100 m	23	–	16
Site Y	Kita Ward, Osaka City	Diluvial	GL	43	50	49
		Diluvial	GL - 60 m	24	49	–
Obayashi building	Chou Ward, Osaka City	Diluvial	B2F	X: 139	Y: 87	Z: 210
M apartment house	Miyakojima Ward Osaka City	Alluvial	1F	X: 60	Y: 86	Z: 42
Osaka mechanical material center	Taisyo Ward Osaka City	Alluvial	GL	195	140	122
Abiko apartment house	Sumiyoshi Ward Osaka City	Alluvial	1F	107	115	92
Takami Tall residence	Konohana Ward Osaka City	Alluvial	1F	156	178	176
		Alluvial	GL	222	267	255
Takatsuki Campus of Kansai University	Reizenjicho Takatsuki City	Fill-in ground	GL	117	85	53
		Sandstone	GL	67	61	36
		Sandstone	GL - 13 m	66	49	39
Point A	Abeno Ward Osaka City	Diluvial	GL - 3 m	76	–	26
Point D	Asahi Ward, Osaka City	Alluvial	1F	129	103	91
		Alluvial	GL	189	155	126
		Diluvial	GL - 25 m	129	113	81

Note: 1 m = 3.28 ft; 1 gal = 1 cm/s/s; 1 cm = 0.39 in.

miles) long and reaches from Kobe to Nishinomiya. Mount Rokko lies north of Kobe, extending in an east-west direction. The plains are within a narrow band of land between Osaka Bay and the mountains.

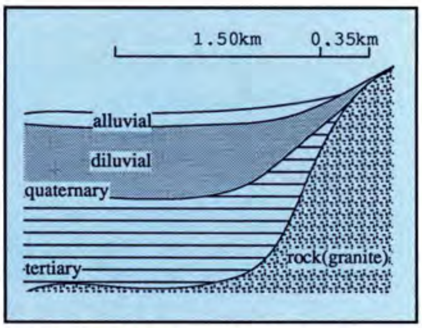


Fig. 3. Simplified profile of ground cross section in Kobe City.

Mount Rokko consists primarily of granite and is crossed by many faults. The southern side of the mountain has step-like slopes, consistent with downward displacements at the fault scarps relative to the north. Near the ground surface, the granite has weathered into decomposed granite soil. A simplified profile of the ground cross section in the north-south direction in Kobe City is illustrated in Fig. 3.

EVOLUTION OF DESIGN CODES

The damage to buildings caused by the Kobe earthquake is closely related to the design methods adopted. The first seismic design provisions were established just after the Great Kanto earthquake in 1923. The seismic design procedures in Japan have been revised every time a significant earthquake occurs and causes severe damage. The evolution of the seismic design codes is described below.

Historical Review of Seismic Design Provisions for Reinforced Concrete Buildings in Japan

In the seismic provisions of the Building Standards Law of 1950, the seismic design load applied to each floor, V_i, was calculated by the following equation:

$$V_i = [0.2 + 0.01(H_i - 16)/4]w_i \quad (1)$$

where

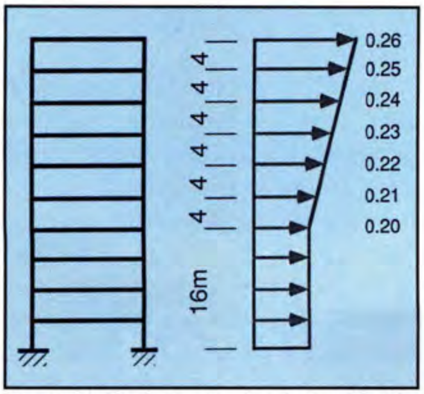


Fig. 4. Seismic design load specified in the Building Standards Law of 1950.

w_i = weight of ith story

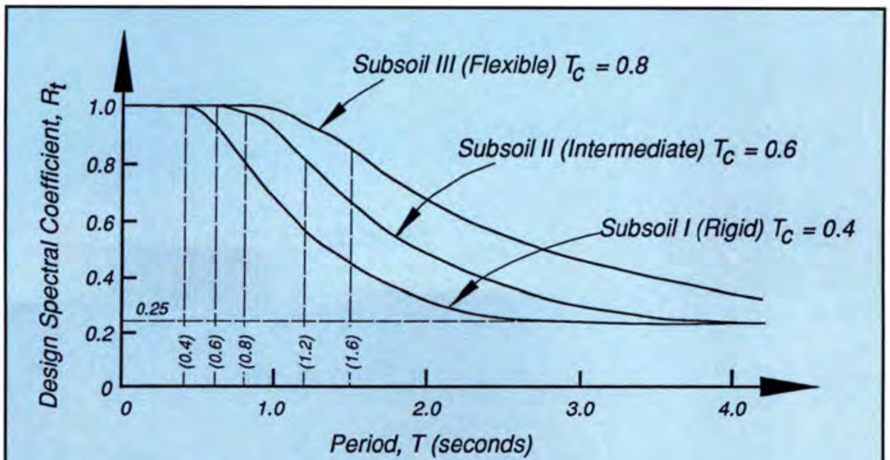
H_i = height of ith story from ground level in meters

If $H_i \leq 16$, then H_i = 16 m (52 ft) and therefore $V_i = 0.2w_i$. The seismic design load is illustrated in Fig. 4 as an example for a ten-story building.

The allowable stress design (working stress method) was conducted for design stresses calculated by linear elastic analysis. The combination of design stresses was $D + L + E$, where D, L, and E are stresses resulting from dead load, live load and seismic design load specified by Eq. (1), respectively. Buildings should be less than or equal to 31 m (102 ft) high.

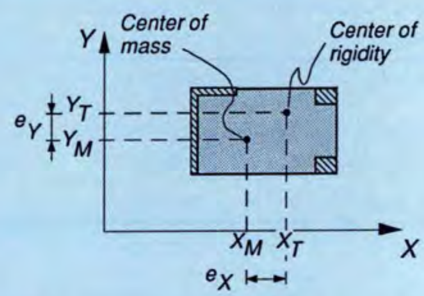
The Tokachi-oki earthquake of 1968 caused a significant number of columns to fail in shear. Extensive research on the shear resistance of columns started. This resulted in changes in the requirements of transverse steel reinforcement in 1971: the maximum spacing of transverse reinforcement was specified to be 10 cm (3.9 in.) at the ends of columns and 15 cm (5.9 in.) elsewhere.

The Miyagiken-oki earthquake of 1978 claimed 27 casualties. Several building structures suffered similar damage to that found in the Tokachi-oki earthquake. Five reinforced concrete buildings collapsed and more than ten structures were severely damaged. The extensive damage prompted researchers and structural engineers to investigate torsional failure resulting from eccentricities of stiffness and mass as well as collapse of soft first story construction. This research led to a drastic revision of the reinforcement regulations of the Building Standards Law.



$$R_t \begin{cases} 1 & (T < T_c) \\ 1 - 0.2(T/T_c - 1)^2 & (T_c \le T \le 2T_c) \\ 1.6T_c/T & (2T_c \le T) \end{cases}$$

Fig. 5. Design spectral coefficient, R_t.



$$e_x = [X_M - X_T] \qquad e_y = [Y_M - Y_T]$$

$$R_{ex} = \frac{e_x}{\sqrt{K_t / K_{hx}}} \qquad R_{ey} = \frac{e_y}{\sqrt{K_t / K_{hy}}}$$

$$K_{hx} = \sum_i J_{xi} \qquad K_{hy} = \sum_i J_{yi}$$

$$K_t = \sum_i J_{xi} \cdot y_i^2 + \sum_i J_{yi} \cdot x_i^2$$

Note:

J_{xi}, J_{yi} : Lateral stiffnesses of vertical structural element i in X and Y directions, respectively.

x_i, y_i : Coordinates of i-th element measured from the center of torsion.

R_{ex}, R_{ey} : Eccentricity ratios in X and Y directions, respectively.

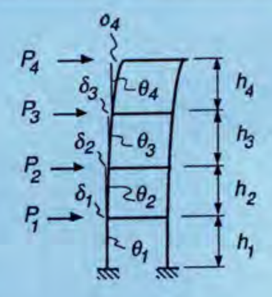
Fig. 6. Eccentricity ratio, R_e.

Three years after the Miyagiken-oki earthquake, the reinforcement regulations of the Building Standards Law were extensively revised and they were enforced in 1982. The seismic design consists of two phases: the first phase is design against moderate and small earthquakes; the second phase is design for severe earthquakes.

A moderate earthquake is defined as an earthquake that is assumed to occur a few times within the service life of a building. Here, buildings are expected to respond to an earthquake in an elastic manner and not be damaged. A severe earthquake is defined as a devastating earthquake that is assumed to possibly occur once in the service life of a building. In this case, buildings are expected not to collapse but to possibly undergo some structural and non-structural damage.

First Phase Design

The load combination to be considered is $D + L + E$, where E denotes the seismic design load due to lateral



$$\theta_j = \frac{\delta_j}{h_j}$$

$$R_{sj} = \frac{\dfrac{1}{\theta_j}}{\dfrac{1}{n}\left[\sum_{i=1}^{n} \dfrac{1}{\theta_i}\right]}$$

Note:

θ_j : Interstory drift of j-th story under seismic design load of the first phase design.

δ_j : Interstory displacement.

h_j : Story height.

R_{sj} : Stiffness ratio.

n : Number of stories.

Fig. 7. Stiffness ratio, R_s.

Table 2. Coefficients F_e and F_s with regard to the eccentricity ratio R_e and stiffness ratio R_s, respectively.

R_e	F_e	R_s	F_s
≤ 0.15	1.0	≥ 0.60	1.0
0.15-0.30	Linear interpolation	0.30-0.60	Linear interpolation
≥ 0.30	1.5	≤ 0.30	1.5

shear force Q_i given by Eq. (2). The interstory drift of each layer obtained by linear elastic analysis under the above load combination shall be less than or equal to $^1/_{200}$:

$$Q_i = ZR_tA_iC_ow_i \qquad (2)$$

where C_o is the basic seismic coefficient of 0.2. The symbol Z denotes the seismic hazard zoning coefficient and varies between 0.8 and 1.0. The second symbol, R_t, is the design spectral coefficient, which depends on the subsoil profile and the natural period of vibration of a building and is given by the following equation:

$$R_t = \begin{bmatrix} 1 & T < T_c \\ 1-0.2(T/T_c-1)^2 & T_c \le T \le 2T_c \\ 1.6T_c/T & 2T_c < T \end{bmatrix} \qquad (3)$$

where T is a period of the first mode and T_c is a factor with respect to a subsoil profile. T_c = 0.4, 0.6 and 0.8 for rigid, intermediate, and flexible subsoils, respectively. The coefficient R_t ranges between 1.0 and 0.25 and is expressed schematically in Fig. 5. The term A_i is the distribution factor of lateral shear forces along the height of the building and is given by Eq. (4):

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T} \qquad (4)$$

where α_i is the ratio of the gravity load above the ith layer to the total gravity load above the level of imposed lateral ground restraint.

Second Phase Design

The lateral load resistance in each story is calculated using inelastic analysis or a virtual work method based on the overstrengths of materials. The building to be designed is required to have a lateral shear strength greater than the shear force obtained from the load combination of $U = D + L + F_{es}E'$ at each story. The factor E' is due to seismic story shear Q_i', which is given by:

$$Q_i' = D_sw_iZR_tA_iC_o \qquad (5)$$

where C_o is the standard base shear coefficient and for the second phase design C_o = 1.0. The term D_s is a reduction factor that depends on the type and the ductility of the structure. The factor D_s ranges from 0.3 for ductile frames to 0.55 for buildings in which a large portion of the lateral load is assigned to the walls and braces. This factor is primarily based on the equal energy concept in which the energy absorbed by a building that yields with elasto-plastic characteristics is assumed to be equal to that of a building that is strong enough to respond elastically.

The other parameters are given in the first phase design:

$$F_{es} = F_eF_s \qquad (6)$$

where F_e is a coefficient that is related to the eccentricity ratio R_e in each story and ranges between 1.0 and 1.5 (see Fig. 6). The term F_s is a coefficient that is dependent on the stiffness ratio R_s in each story and ranges between 1.0 and 1.5 (see Fig. 7). Table 2 summarizes these coefficients, F_e, F_s and F_{es}. Therefore, F_{es} varies between 1.0 and 2.25. The factor F_{es} was introduced to provide an extra strength in the case of buildings with unsymmetrical arrangements of the seismic load resisting elements and/or with extremely flexible stories, compared to the other stories.

Historical Review of Seismic Design Provisions for Prestressed Concrete Buildings in Japan

The design procedure for prestressed concrete structures was first issued in 1960 when the Standard for Structural Design and Construction of Prestressed Concrete Structures was published by the AIJ. The structural design of prestressed concrete buildings has been based on the strength design method since it was first established, while the design of reinforced concrete buildings had been based on the allowable stress design until the drastic revision in 1981. All structural members of pre-

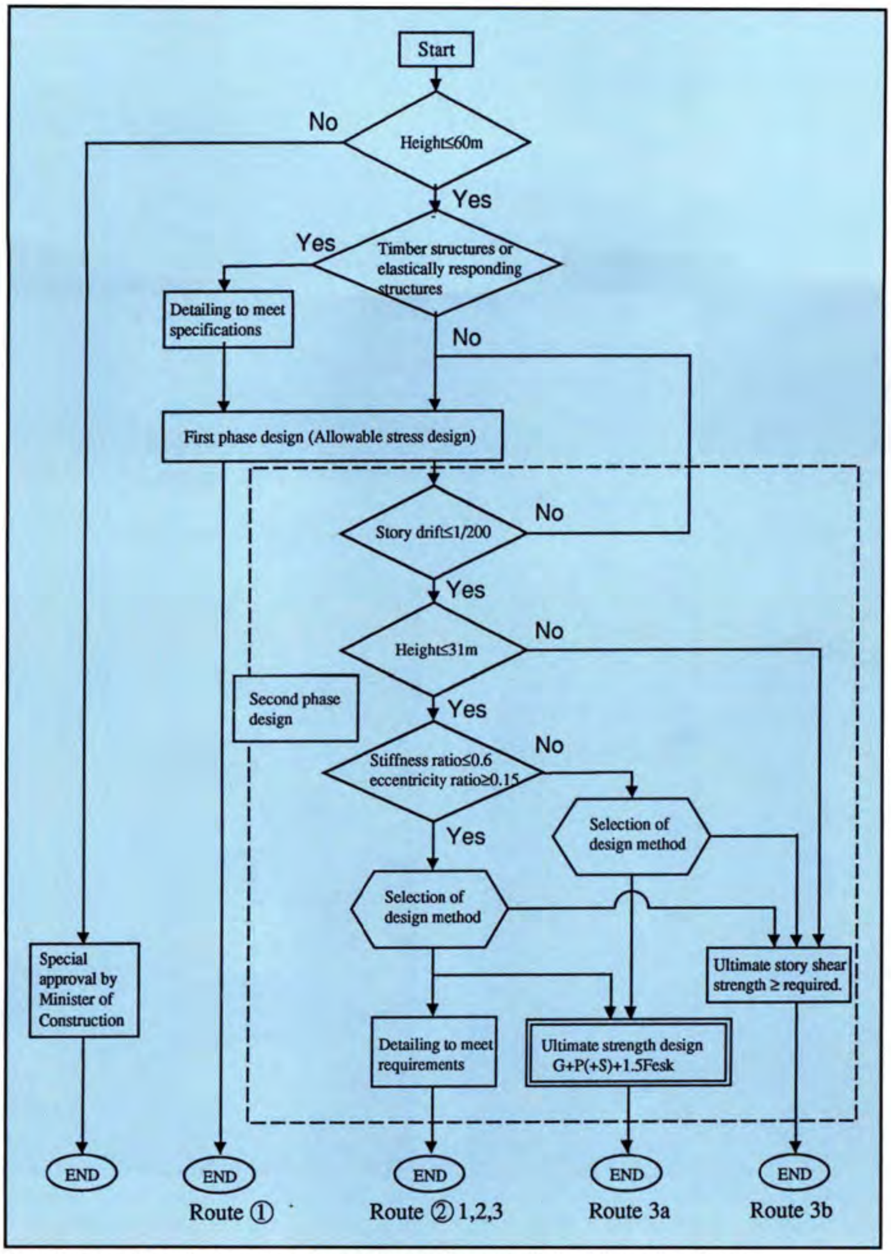


Fig. 8. Flowchart for design of prestressed concrete building structures.

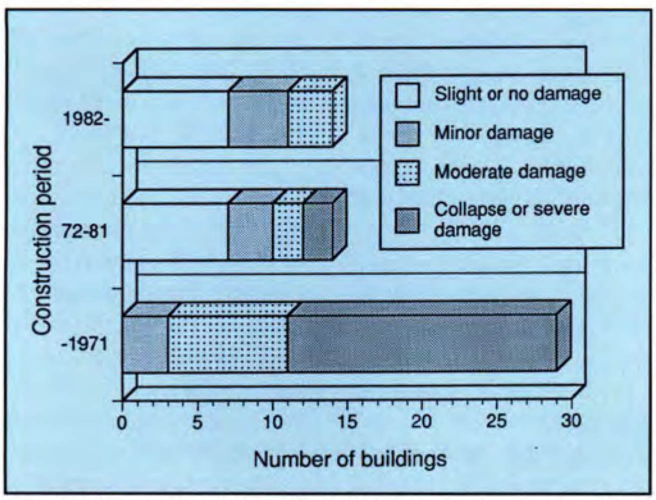


Fig. 9. Damage level of reinforced concrete structures with respect to the year of construction (Ref. 1).

stressed concrete buildings are not prestressed. For example, columns are usually not prestressed. They may be designed according to the codes for reinforced concrete.

When the design procedure was first introduced, the design seismic load applied to each floor was calculated using Eq. (1). The maximum height was four stories or 16 m (52 ft). The basic seismic coefficient, therefore, was 0.2 regardless of the height of the building. The load combination was $1.2(D + L) + 1.5E$.

In 1973, the height limitation for prestressed concrete building structures was extended to 31 m (102 ft), which was the same for buildings of other structural types. In addition, the design seismic load applied to each floor was calculated using Eq. (1). The load combination was $D + L + 1.5E$ for flexure by the strength design method and $D + L + 2.0E$ for shear by the allowable stress design method. The design stresses were calculated by linear elastic analysis.

The method of calculating the ultimate shear strength of prestressed and non-prestressed members had not been established at that time. Therefore, a relatively large design load combination was specified and the allowable stress design method was used. Reinforced concrete members should be designed to fail in flexure. The maximum spacing of transverse reinforcement was 10 cm (3.9 in.) at the ends of columns and 15 cm (5.9 in.) elsewhere.

After the 1981 code revisions came into force, prestressed concrete buildings could be designed according to either the pre-1981 design method described above or the new seismic design procedure aimed primarily for conventionally reinforced concrete buildings (see Fig. 8). The revised seismic design load distribution and intensity given by Eqs. (4) and (5), respectively, are used. However, buildings higher than 31 m (102 ft) and lower than or equal to 60 m (197 ft) must be designed according to the latter design method. In addition, the design of concrete buildings higher than 31 m (102 ft) requires approval by the Minister of Construction.

Fig. 10. A pre-1981 apartment building that collapsed at the soft first story.

Fig. 11. A pre-1981 apartment building that collapsed at the soft first story.

PERFORMANCE OF REINFORCED CONCRETE BUILDINGS

Damage to buildings by the earthquake was much more severe in buildings built before the 1971 code revision took effect. The investigation conducted by the AIJ Kinki Branch revealed that in the Chuo Ward of Kobe City, the center of Kobe, 18 reinforced or steel-encased reinforced concrete buildings constructed before 1971 collapsed or suffered severe damage (see Fig. 9). On the other hand, only two of those buildings built between 1971 and 1981 were found collapsed or severely damaged. No concrete buildings built after the 1981 revision collapsed.

Collapse of Soft First Story

Many buildings that were constructed with open retail space or parking on the first floor collapsed. In old buildings designed and constructed before the 1981 revisions, the collapse can be attributed to a more flexible and/or weaker story than the other ones, and inadequate transverse reinforcement in terms of its amount and detailing (see Figs. 10 and 11). Since 1981, an excessively flexible story, compared with the other stories in a building, has been restrained or has been required to have extra strength. This was realized by the introduction of the stiffness ratio, R_s. In addition, the detailing of transverse reinforcement has been improved.

However, several buildings that conform to the current design code requirements collapsed in the open first story (see Figs. 12 and 13). The collapse calls attention not only to a uniform distribution of story stiffness along the height of buildings but also to an excessively weak story, compared to the other stories, even if it has greater story shear strength than that specified by the code.

Fig. 12. A post-1981 apartment building that collapsed at the soft first story.

Fig. 13. A post-1981 apartment building that collapsed at the soft first story.

The damage tends to concentrate into the weakest story, as shown in Fig. 14. The figure was obtained by dynamic response analyses on lumped multi-mass shear systems to observe how large a deformation was concentrated to weakest stories when a column sidesway mechanism formed. The systems consisted of eight masses. The yield capacities of the systems were calculated based on a different base shear coefficient but the elastic stiffness of the layer was the same. The A_i distribution was used as a shear force distribution over the height of the systems.

The envelope curve model for shear force-interstory drift of each layer in the case of the base shear coefficient of $C_o = 0.35$ is shown in Fig. 15 as an example. The standard El Centro NS 1940 earthquake wave record was used. It was amplified to the maximum velocity of 50 cm/s (20 in. per sec). Fig. 14 shows the maximum interstory drifts of the systems analyzed.

The results include the response of: (1) the linear elastic system; (2) the systems that were designed using base

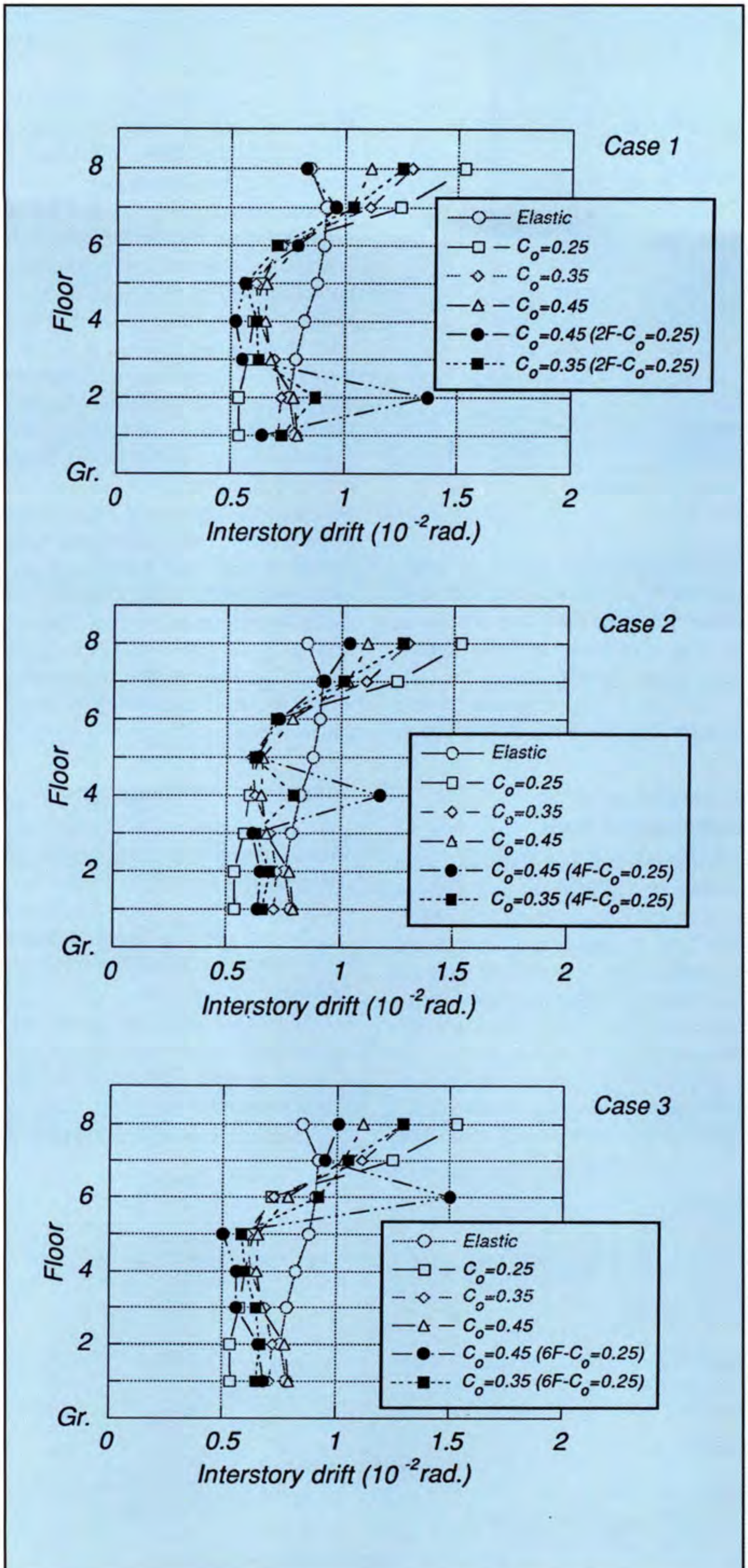


Fig. 14. Dynamic response of buildings with one story weaker than the other stories.

shear coefficients of 0.25, 0.35 and 0.45; and (3) the systems designed basically using base shear coefficients of 0.45 and 0.35, but the story shear capacity of the second (Case 1), the fourth (Case 2) or the sixth (Case 3) layer was provided from the base shear coefficient of 0.25.

Therefore, an interstory drift displacement was expected to concentrate into the weakest layer. If the shear capacity based on C_o = 0.25 is assumed to be required, the layers other than the weakest layer had reserve strength. The ratios of the provided strength to the required strength were 1.8(0.45/0.25) and 1.4(0.35/0.25), respectively.

As expected, the interstory drift concentrated in the weakest story. Table 3 summarizes the analytical results of the maximum interstory drift angles in 0.01 radian. The column of the weakest story of each system was surrounded by double lines and included the corresponding ductility ratio.

In the 1981 revisions, a stiffness factor was introduced to prevent an excessively flexible story. It is, however, based on the elastic stiffness. In order to avoid an excessively weak story, a distribution of story shear strength along the height of the frame should be considered.

A non-ductile frame that does not rely on plastic deformation of the members can be designed if D_s = 0.45 is used. The reduction factor D_s for a ductile frame is 0.3. Because the maximum value of F_s is 1.5, a non-ductile open first story that conforms to the current design requirements is realized if a story shear strength of 0.675W is provided with the story in which the provision for a stiffness ratio is not satisfied. The term W is the weight of the building. However, several post-1981 buildings that collapsed in the first story revealed that a story shear strength of that quantity may not be enough if one or more columns fail in a brittle manner before the shear strength of each column in the first story should develop.

In the first phase design of the current design code, the story shear force-interstory drift curve of each layer based on the elastic stiffness should pass over the coordinates [$^1/_{200}$ in interstory drift angle, $0.2ZR_tA_iw_i$ in story

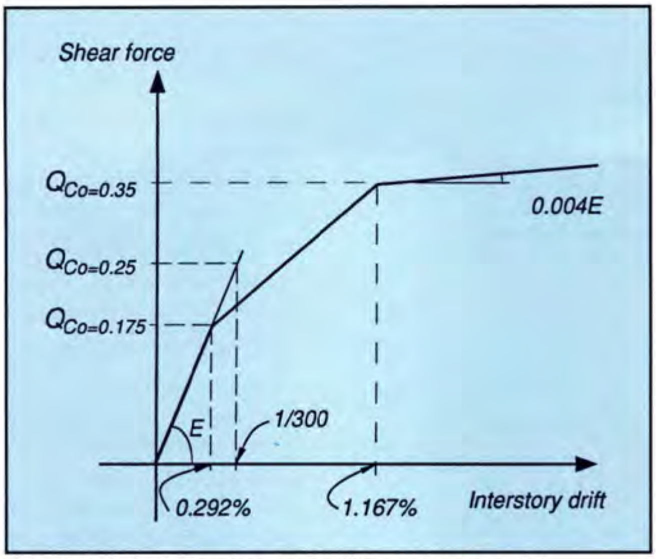


Fig. 15. Shear force-interstory drift envelope model.

shear] as shown in Fig. 16. Beyond this point, no consideration of displacement is required. Each layer of the building is required to have a story shear capacity greater than specified in the codes no matter how large interstory drift may be attained. If a story shear strength of $0.675W$ is attained at so large a displacement that a second-order geometric effect should be considered, the structure would become unstable, which would lead to collapse.

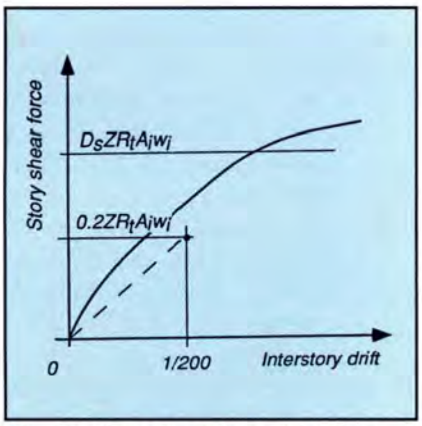


Fig. 16. Story shear force-interstory drift curve.

Shear Failure of Columns and Walls

Numerous columns and walls were observed to fail in shear. Such failures were pointed out in past earthquakes. This kind of damage can be attributed to short columns, insufficient shear reinforcement, no cross-tie or supplemental ties, and inadequate construction (see Figs. 17, 18 and 19).

Old buildings constructed before 1971 had relatively little transverse reinforcement in their columns. The transverse reinforcement of 9 to 10 mm (0.35 to 0.39 in.) diameter was provided in the spacing of 20 to 30 cm (7.8 to 11.8 in.). Due to the 1971 revisions, transverse reinforcement was required to have a spacing of 10 cm (3.9 in.) or less in the column end regions.

The observed damage to the columns in the Kobe earthquake and other past earthquakes indicated that a 90-degree hook followed by relatively short extensions cannot prevent columns and walls from failing in shear. A 135-degree hook of transverse reinforcement and sufficient extensions must be provided, as required in the current codes. Use of closed ties and cross-ties is recommended, especially when the column section is large and ductility demand is high. Even a 135-degree hook was found ineffective in some cases because of spalling of cover concrete.

Brittle fracture at the bent was observed in the transverse reinforcement. This may be due to the poor quality of 9 to 13 mm (0.35 to 0.51 in.) diameter bars.

Collapse of a Midheight Story

A conspicuous mode of failure of reinforced concrete buildings in the earthquake is the story collapse at a midheight story (see Figs. 20, 21 and 22). Several reasons described below are potentially responsible for these collapses:

1. Unless a building structure is designed so that a certain collapse mechanism is intentionally formed, damage may concentrate in any story.

2. Damage can concentrate at a

Table 3. Maximum interstory drift angle (0.01 radian).

Floor	Elastic response	$C_o = 0.25$	$C_o = 0.35$	$C_o = 0.45$	$C_o = 0.45$ (2F - C25)	$C_o = 0.35$ (2F - C25)	$C_o = 0.45$ (4F - C25)	$C_o = 0.35$ (4F - C25)	$C_o = 0.45$ (6F - C25)	$C_o = 0.35$ (6F - C25)
8	0.849	1.532	1.298	1.116	0.832	1.256	1.037	1.274	1.007	1.292
7	0.920	1.248	1.112	1.030	0.963	1.038	0.928	1.013	0.950	1.052
6	0.909	0.716	0.724	0.787	0.791	0.703	0.729	0.716	1.502 (1.81)	0.920 (1.109)
5	0.879	0.636	0.611	0.657	0.561	0.567	0.632	0.626	0.501	0.583
4	0.822	0.598	0.625	0.653	0.519	0.612	1.175 (1.41)	0.794 (0.965)	0.561	0.600
3	0.784	0.576	0.692	0.675	0.551	0.622	0.615	0.628	0.561	0.650
2	0.758	0.535	0.722	0.775	1.370 (1.65)	0.872 (1.05)	0.649	0.691	0.661	0.667
1	0.786	0.538	0.709	0.794	0.638	0.725	0.639	0.660	0.683	0.650

Fig. 17. Shear failure of short columns.

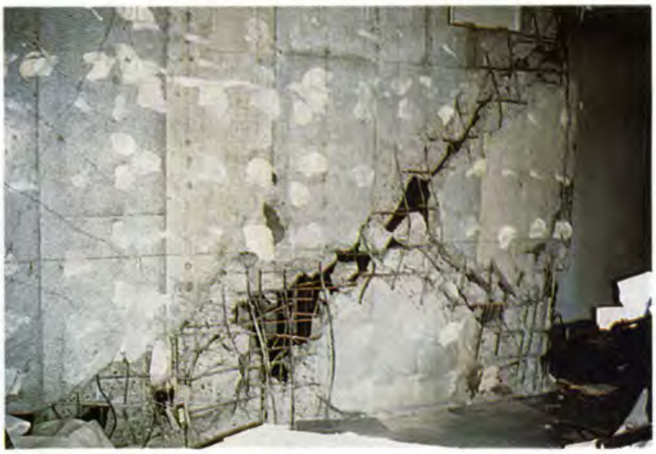


Fig. 18. Shear failure of walls.

story in which the story shear strength and/or stiffness changes abruptly between adjacent stories. Several buildings were found collapsed at the story where the structural system changed from steel-encased reinforced concrete (SRC) to reinforced concrete. In another case, the amount of structural walls in the collapsed story was found to be much less than the other stories.

3. The seismic design load distribution over the height used in the old design codes is different from current codes. Although the codes cannot be compared directly due to differences between the design procedures, the proportion of design story shear was smaller at the middle stories in the old codes than the current ones, as shown in Fig. 23.

4. Large vertical accelerations may have generated large compressive and tensile axial loads in the columns, which resulted in ductility and shear strength reductions. The interaction of horizontal and vertical accelerations may also be a reason.

Torsional Failure Resulting From Eccentricities of Stiffness and Mass

One building had a structural wall on one side of the perimeter in the first floor. The other three sides were open. The building sustained damage in columns on the open sides due to torsional response. Members susceptible to larger force and deformation demands due to plan eccentricity need to be designed recognizing their actual stiffness and strength properties and the impact of these properties on torsional response. Buildings should be as regular as possible.

Failure of Gas-Pressure Welded Reinforcement Splices

In Japan, reinforcement splices in buildings and bridges recently have almost always been made by a process known as gas-pressure welding. In this process, the bars to be joined are aligned and butted together; the bars are then fused together by heat and pressure applied by mechanical devices, causing the bars to flare out at the splice. Pressure welded splices were observed to


Fig. 19. Shear failure of a column due to inadequate transverse reinforcement.


Fig. 20. Kobe City Hall, which collapsed at the sixth floor.


Fig. 21. A hospital that collapsed at the fifth floor.

Fig. 22. An apartment building that collapsed at the third story due to torsion resulting from eccentricities of stiffness and mass.

fracture in the earthquake (see Fig. 24). Investigation is needed to identify the causes of the fracture.

Performance of High Rise Reinforced Concrete Buildings

No damage was found in high rise reinforced concrete buildings in the region of severe ground motions due to careful design and construction and use of high strength concrete. Acceleration responses recorded in a 31-story reinforced concrete building located about 43 km (26 miles) east of the epicenter indicated that the maximum accelerations in horizontal and vertical directions on the 31st floor are 1.14 and 1.7 times those on the ground, respectively.

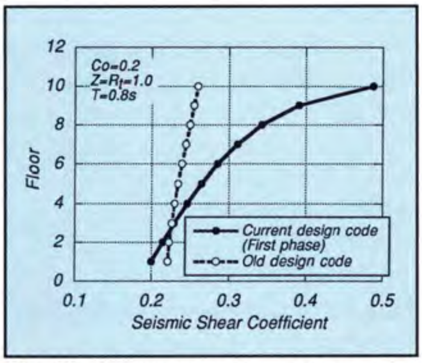


Fig. 23. Comparison of seismic design load between old and current design codes.

Collapse Mechanism

Several new buildings were designed such that a certain collapse mechanism, especially a beam sidesway mechanism, is intentionally formed. Fig. 25 shows a ten-story apartment building constructed in 1991. Fig. 26 shows the elevation of a structural frame and the plan. Plastic hinges at the ends of the beams in the second to seventh stories were found as intended in the design (see Fig. 27). The residual drift at the top of the building could not be observed. Because non-structural walls suffered damage, the building is scheduled for repair.

The typical damage described above was also observed in the 1968 Tokachioki earthquake and the 1978 Miyagikenoki earthquake. Old buildings designed according to the old design codes should be analyzed and strengthened.

Concrete of approximately 20 MPa (2.9 ksi) in design compressive strength is generally used in Japan. Higher strength concrete should be required. Higher strength concrete not only improves the seismic performance of buildings but also leads to better and more careful construction.

PERFORMANCE OF PRECAST, PRESTRESSED CONCRETE BUILDINGS

The inspection carried out by the Japan Prestressed Concrete Engineering Association (JPCEA)[2] reported that there are 163 precast, prestressed con-

Fig. 24. Failure of gas-pressure welded reinforcement splices.

Fig. 25. An apartment building designed such that a beam sidesway mechanism was intentionally formed.

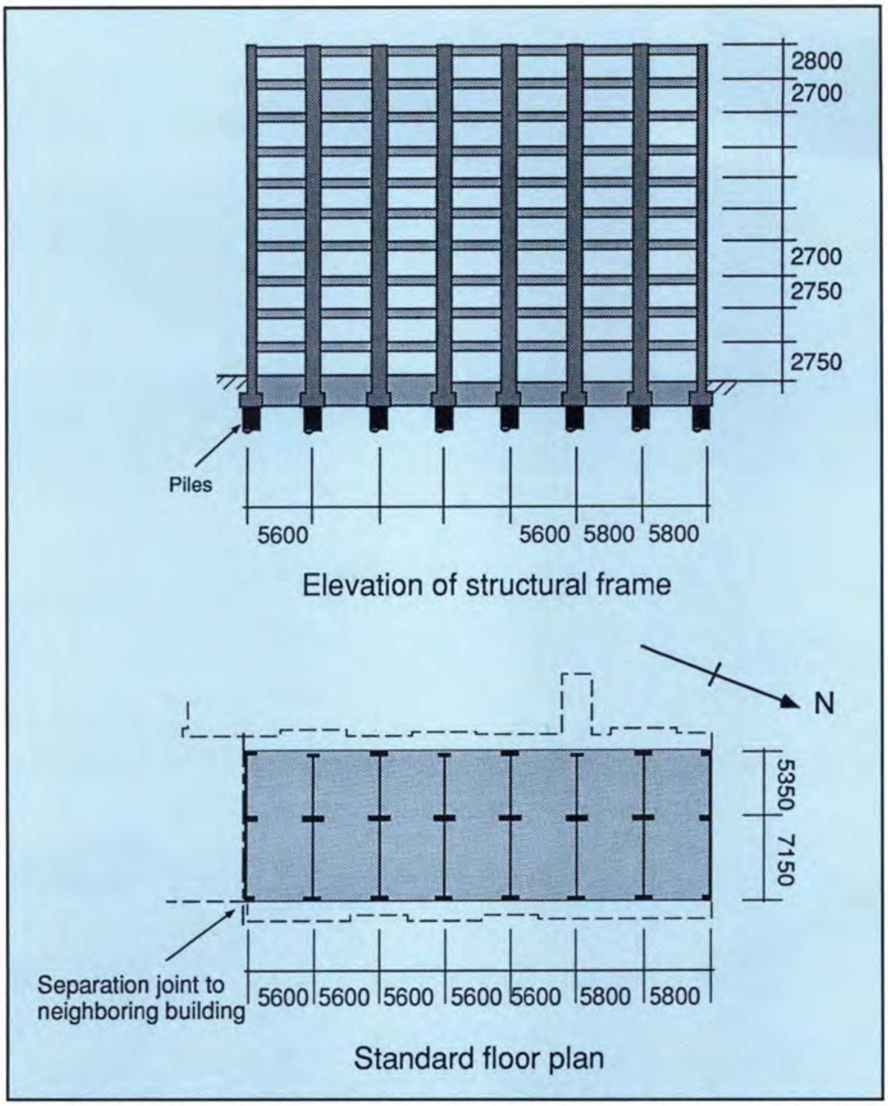


Fig. 26. Structural frame and plan of ten-story reinforced concrete apartment building pictured in Fig. 25.

crete buildings in the region of Shindo intensity of 6 and 7: Kobe, Ashiya, Nishinomiya, Takarazuka, Itami, Amagasaki and Kawanishi. On Awaji Island, which is one of the regions of Shindo intensity of more than 6, there are three prestressed concrete buildings. These buildings sustained no damage.

The number includes buildings that have some precast, prestressed concrete members. Eleven of these buildings are precast, prestressed concrete buildings, 49 buildings which had non-structural precast, prestressed members, 89 cast-in-place prestressed concrete buildings and 14 buildings which had non-structural cast-in-place prestressed members. Buildings with unbonded tendons are excluded. Use of unbonded tendons for primary earthquake resistant members is prohibited in Japan.

Most of the precast, prestressed concrete buildings performed remarkably well in the earthquake. The reasons why little damage was found in precast, prestressed concrete buildings are summarized below:

1. Seismic design loads assigned to precast, prestressed concrete buildings are larger than those of buildings of different structural types, although the design methods are different. Therefore, resistance to earthquake loads of precast, prestressed concrete buildings is expected to be higher than that of the other types of building structures.

2. Precast, prestressed concrete buildings are generally regular structures with a symmetrical shape in plan and a uniform distribution of masses and stiffening elements.

3. High strength and quality concrete is usually used, resulting in higher shear resistance and careful construction.

4. Precast, prestressed concrete buildings are relatively new.

Among these buildings, only three sustained severe structural damage.


Fig. 27. Cracks at the beam-to-column interface — an indication of the formation of beam sidesway mechanism (building pictured in Fig. 25).


Fig. 28. Prestressed concrete building with extension of steel tumbled down.

Fig. 29. Shear failure of a column in the second floor of the building shown in Fig. 28.

Fig. 30. Damage to a column that supported a prestressed concrete beam.

One building sustained architectural damage in its precast non-structural elements.

The most devastating damage of a concrete structure was found in a four-story bowling arena that was built in 1973. The building had an extension made of steel in the front, half of which completely tumbled down (see Fig. 28). The beams of the fourth story are 37.2 m (122 ft) cast-in-place prestressed concrete. The total beam depth was 2 m (6.6 ft). No damage was observed in the beams. Several small cracks were found but they could not be identified as cracks caused by the earthquake. They are considered to have been caused by the

Fig. 31. A gymnasium of an elementary school in which precast, prestressed concrete roof shells fell down.

Fig. 32. Seven pieces of the precast, prestressed concrete roof shells fell down.

Fig. 33. Damage to a column of the gymnasium shown in Fig. 31.

Fig. 34. Damage to a pile.

introduction of prestressing.

Almost half the inner reinforced concrete columns and several peripheral columns in the second story failed in shear (see Fig. 29). The column section was 1000 x 1000 mm (39.4 x 39.4 in.) with a clear height of 4 m (13 ft). The third story had columns only on the peripheral frame. Therefore, the vertical load of the fourth story was mainly transmitted to the peripheral columns in the second story.

A small column axial load resulted in reduction of shear resistance. This, as well as insufficient transverse reinforcement, may be a reason for the column failure. Several severely damaged columns were observed in the first floor as well. The spacing of the transverse reinforcement of D13 of the columns was 150 mm (5.9 in.).

The most typical type of damage to prestressed concrete building structures was found in this building: a column failure prior to yielding of the prestressed concrete girders and beams that frame into the column. Fig. 30 shows the top of a column in the third floor that a prestressed concrete beam was framed into. Immediately below the beam, the column had its cover concrete spalled off.

Prestressing tendons are usually provided to cancel or reduce flexural moments due to dead and live loads. This results in much more beam strength than required for the actions due to design seismic loads. Plastic hinges are expected to form in the columns rather than the beams. In the worst case, this would result in story collapse. Structural designers should pay attention to these characteristics. The columns should be provided with sufficient transverse reinforcement and careful design is needed.

Another two buildings sustained the same structural damage: hyperbolic precast, prestressed concrete shell roof panels fell down on to the ground. They were gymnasiums designed and constructed in 1972 and 1974.

The newer one was a gymnasium of an elementary school (see Fig. 31). The building had 17 precast, prestressed concrete panels as the roof. The roof panels were supported by the pillow beams at both ends through a rubber plate. Steel bolts were used to fasten them. Seven pieces fell down on to the ground (see Fig. 32). In some cases, one end of the panel was left on the top peripheral beam. The columns on the third floor failed in flexure at the place where some of the longitudinal reinforcing bars terminated and the number of the bars was reduced (see Fig. 33).


Fig. 35. Another example of damage to a pile.

A relative movement between the panel and the pillow beam is considered to be about 100 to 150 cm (39 to 59 in.). The failure of the reinforced concrete columns would trigger off the drop of the roof panels. The precast panels were installed to absorb the distance change between the supports. However, the reinforced concrete frame that supported the panels was so flexible that the movement seemed to exceed the margin.

A similar type of collapse was found in the Northridge earthquake in 1994.[4] At least three precast concrete parking structures partially collapsed due to a lack of ties connecting precast floor elements. Another defect was that the combination of large lateral deformations and vertical load caused crushing in poorly confined columns that were not designed to be part of the lateral load resisting system.

PERFORMANCE OF PILE FOUNDATIONS

Damage to foundations is, in general, invisible. However, in the Miyagiken-oki earthquake of 1978, severe damage to precast, prestressed spun concrete piles was found under several reinforced concrete buildings. Six years after the earthquake, new seismic design provisions for foundation piles were mandated by the Ministry of Construction. In the code, piles are required to be designed to resist elastically the loads from the superstructure as large as $0.2W$, where W is the weight of the superstructure.

In Japan, three grades of precast, prestressed spun concrete piles are currently being produced with an average prestress of 4, 8, and 10 MPa (580, 1160, and 1450 psi), respectively. Until 1984, only piles with a prestress of 4 MPa (580 psi) were produced. However, since the seismic code provisions for piles were enforced in 1984, piles with a prestress of 8 and 10 MPa (1160 and 1450 psi) have mainly been produced.

In the Kobe earthquake, large ground settlements and landslides exposed damage to foundations and piles (see Figs. 34 and 35). An effort to identify the damage to piles has also been made. Excavations for the inspection of piles with 8 and 10 MPa (1160 and 1450 psi) prestress levels revealed shear failures at pile caps. Nondestructive examinations by impact wave propagation indicated that the failure or severe cracks probably occurred at the middle or tip portion of the piles. It is anticipated that piles with 4 MPa (580 psi) average prestress will reveal severe damage, although their inspection has not started as of this date.

Piles are currently designed to resist elastically the seismic design load of 20 percent of the total weight of the superstructure, while superstructures are required to be designed plastically against the seismic design load corresponding to a base shear coefficient of 0.3 if they are ductile frames. Therefore, piles should be designed in the same way. Presently, precast, prestressed spun concrete piles are not provided with transverse reinforcement for resisting shear and for confinement of compressed concrete. It is also recommended that ductile piles with transverse reinforcement be used.

CONCLUSIONS

The following conclusions can be drawn on the basis of the field observations and investigations resulting from the earthquake:

1. Damage to buildings in the Kobe earthquake was much more severe in buildings built before the 1971 revision of the reinforcement requirement of the Building Standards Law. Old buildings should have been strengthened.

2. The typical damage to reinforced concrete buildings was collapse of the soft first story, shear failure of columns and walls, collapse of a mid-height story, torsional failure due to eccentricities of stiffness and mass, or failure of gas-pressure welded reinforcement splices.

3. The majority of the precast, prestressed concrete buildings performed remarkably well in the earthquake because they are relatively regular and/or new structures with higher strength and quality concrete than ordinary reinforced concrete structures. Their resistance to earthquake loads is considered to be higher than that of the other structural types of concrete buildings.

4. Design procedures for piles should be revised in the same way as for superstructures. Use of ductile piles with transverse reinforcement is recommended.

5. One reinforced concrete building that had 37.2 m (122 ft) long cast-in-place prestressed concrete beams on the third floor suffered devastating damage in ordinary reinforced concrete columns on the second and first floors. No damage was found in the prestressed concrete beams.

6. Precast, prestressed concrete shell roof panels fell down in two gymnasium buildings. The cause of this failure was attributed to support frames that were too flexible and poor detailing to install the panels on to the support girders.

Lastly, the findings reported herein are a preliminary assessment of the causes of damage resulting from the Kobe earthquake. More detailed reports will be forthcoming after comprehensive investigations have been carried out by researchers, engineers, the AIJ, the JPCEA and other organizations.

ACKNOWLEDGMENT

The authors wish to express their appreciation to Dr. Y. Ohno of Osaka University and Dr. I. Takewaki of Kyoto University for permission to use their photographs in this article. The authors also wish to thank H. Sato for preparing several of the drawings.

REFERENCES

1. *Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake* (English Edition), Architectural Institute of Japan, April 1995.

2. *Preliminary Reconnaissance Report on Damages to Precast, Prestressed Concrete Structures of the Hyogoken-Nanbu Earthquake* (in Japanese), Japan Prestressed Concrete Engineering Association, April 1995.

3. *The Standard for Structural Design and Construction of Prestressed Concrete Structures* (in Japanese), Architectural Institute of Japan, 1987.

4. *Preliminary Reconnaissance Report on Northridge Earthquake, January 17, 1994*, Earthquake Engineering Research Institute, Oakland, CA, 1994.

5. Nishiyama, M., *Seismic Response and Seismic Design of Prestressed Concrete Building Structures*, Doctoral Thesis, Kyoto University, Kyoto, Japan, 1993.