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Behavior of Reinforced Concrete Buildings during the Hyougoken–Nanbu Earthquake

Fumio Watanabe

Department of Architecture and Architectural Systems, Graduate School of Engineering, Kyoto University, Sakyo-ku, Kyoto, 606, Japan

Abstract

The Hyougoken–Nanbu earthquake caused serious damage to reinforced concrete building structures. Investigation results indicated that most buildings which suffered serious structural damage were built before 1981, i.e. new modern buildings behaved well, because the current seismic design provisions were enforced by the ministry of construction in 1981. Typical damage patterns of reinforced concrete building structures were: (1) collapse of soft first story construction; (2) collapse of a mid-height story; (3) torsional failure resulting from eccentricities of stiffness and mass; (4) inadequate spacing and anchorage of transverse reinforcement; (5) shear failure of short columns; (6) fracture of re-bar splices by gas-pressure welding; (7) damage to separation joints and elevated corridors; and (8) damage to non-structural members (Preliminary Reconnaissance Report of the 1995 Hyougoken–Nanbu earthquake, Architectural Institute of Japan, 1995 (English version)). © 1997 Published by Elsevier Science Ltd. All rights reserved.

INTRODUCTION

The Hyougoken–Nanbu earthquake caused serious damage to several types of structures such as buildings, bridges and highway viaducts. Over 6300 people were killed, due to building collapse resulting from the severe shaking and fire of traditional timber structures. Several task groups such as the reconnaissance teams of the Architectural Institute of Japan (hereafter abbreviated as AIJ)[1] and the Building Research Institute investigated the damage to building structures. Investigation results indicated that almost all of the buildings that suffered serious structural damage were built before 1981, i.e. new, modern buildings behaved well because the current seismic design provisions were enforced by the ministry of construction in 1981. However, some modern buildings suffered serious damage mainly to the soft first story construction. A prominent mode of failure of reinforced concrete buildings in this earthquake involved the story collapse of an upper floor level. Such a damage was hardly observed in past earthquakes. Damage to non-structural elements, such as partition walls and elevated corridors, was also observed in several buildings. This report deals with the characteristics of the earthquake, typical damage patterns of reinforced concrete buildings and future perspective.

CHARACTERISTICS OF THE EARTHQUAKE

The Hyougoken–Nanbu earthquake caused severe damage to the southern region of the Hyougo Prefecture, particularly to the city of Kobe. The severely damaged areas are located in a narrow band extending from the northern part of the Awaji island to Takarazuka city and includes the cities of Kobe, Ashiya and Nishinomiya. The most severely damaged areas, where the seismic intensity was 7 on the Japanese seven-stage scale (Shindo 7), is indicated in Fig. 1. The recorded maximum horizontal and vertical accelerations at several points are also indicated. The earthquake occurred at 5:46 a.m. local time on 17 January 1995. The Richter magnitude of the earthquake was estimated to be 7.2 and the focal depth was approximately

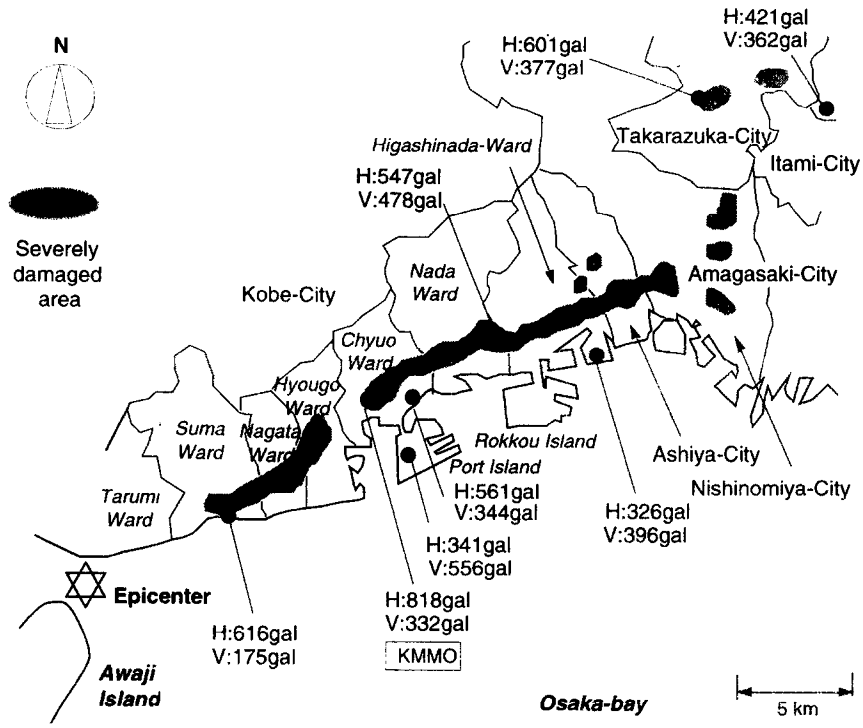

Fig. 1. Severely damaged area and acceleration records.

14.3 km. The epicenter was located at 34°36.4′ North latitude 135°2.6′ East longitude. With respect to the typical ground motion record, peak ground accelerations at Kobe Maritime Meteorological Observatory (KMMO) were 818 gal in the north–south direction, 617 gal in the east–west direction and 332 gal in the vertical direction. The duration of the strong shaking of the earthquake was less than 10 s. For the KMMO records, the predominant period of the horizontal ground motion is approximately 0.8–1.0 s. Two predominant periods of the vertical motion are observed: one at approximately 0.9–1.5 s and the other at approximately 0.25 s. A pseudo-velocity spectrum for KMMO records, calculated by Dr Okawa at the Building Research Institute of Japan, is shown in Fig. 3.

The cities of Kobe, Ashiya and Nishinomiya are located between the Rokko mountains (north) and Osaka Bay (south). It is believed that ground motions were amplified in the

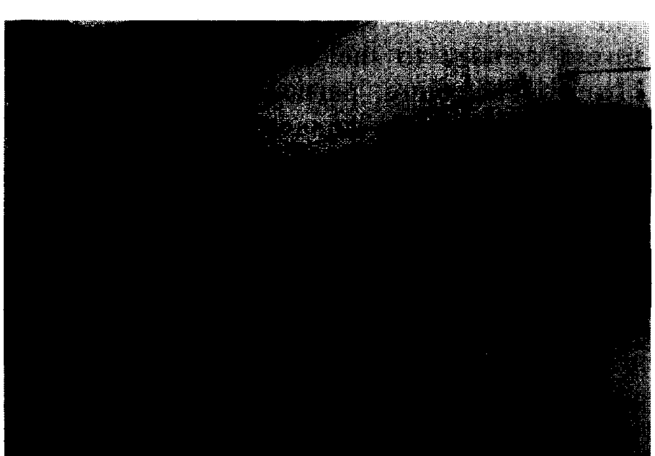

Fig. 2. Damage to soft first story of the 7 storied condominium building.

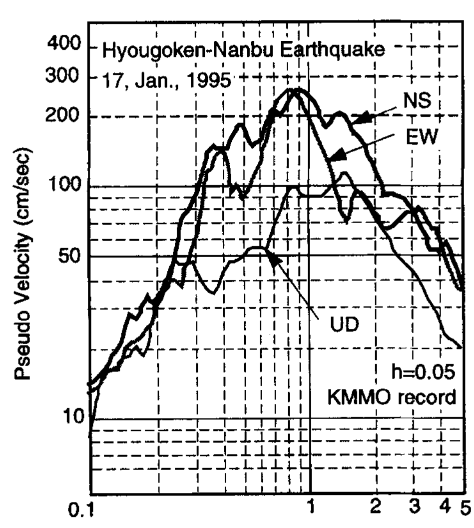

Fig. 3. Velocity spectrum.

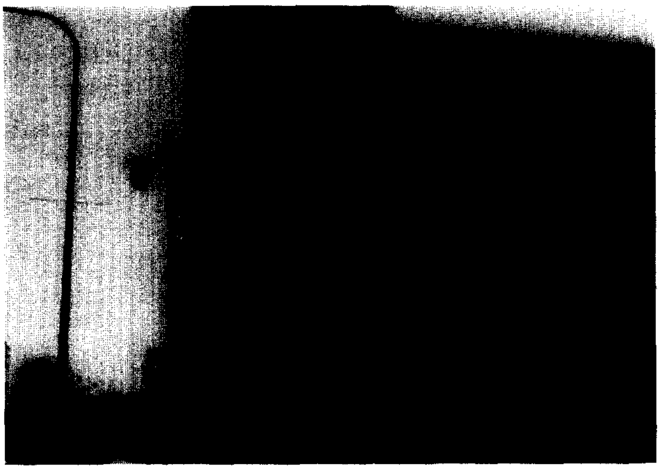

Fig. 4. Kobe city office building collapsed at the 6th story.

plains near the Rokko mountains as a consequence of topographic irregularities. The characteristics of the ground motions recorded in the damaged area may be summarized as follows:

(1) Peak ground accelerations were large in both the horizontal and vertical directions.
(2) The duration of strong shaking was 10–15 s.
(3) The predominant period was 0.8–1.5 s. A second predominant period was sometimes observed at around 0.25–0.4 s.
(4) Ground motions were affected by local soil conditions and topography. It is believed that ground motions were amplified in the plains near the mountains between the cities of Kobe and Nishinomiya.

DAMAGE TO REINFORCED CONCRETE BUILDINGS

Overview of damage and historical review

An unprecedented number of reinforced concrete buildings were damaged during the Hyougoken–Nanbu earthquake, outnumbering those damaged in previous earthquakes. The damage was much more severe in buildings designed and built before the 1981 codes took effect. Most of them were designed according to the 1950 codes, where the structural calculation was based on the allowable stresses of materials, for example, allowable compressive stress of concrete was 2/3 times the specified compressive strength and allowable stress of flexural reinforcement was 1.0 times the specified yield strength.

Figure 5 shows a typical example of the correlation between the damage level and the year of construction of reinforced concrete buildings (some of them are steel encased reinforced concrete) in severely damaged areas (7 in Japanese seven-stage scale), such as the Nada-ward and the Higashinada-ward of Kobe city (see Fig. 1). This data was offered by the reconnaissance team formed at the AIJ Kinki Chapter. The damage statistics for general buildings and for buildings with a soft first story are separately indicated in Fig. 3. This figure tells us that the damage to buildings built after 1972 reduces. The reason is that the Building Standard Law Enforcement Order was partly amended in 1971, i.e. the requirement for the detailing of shear reinforcement was intensified as follows:

(1) The diameter of transverse reinforcement must be 6 mm or more;
(2) the spacing of column hoops must be 15 cm or less, and less than 15 times the diameter of longitudinal reinforcement; and
(3) the spacing of hoops must be 10 cm or less at the ends of columns within a range of two times the column section height.

This revision was due to the fact that many reinforced concrete columns failed in shear during the Tokachi–Oki earthquake in 1968.

It is also seen that the damage to buildings with a soft first story was much more than that for general buildings. It should be emphasized that the serious damage to buildings with a soft first story did not reduce after 1972, because the 1971 revision (mentioned earlier) was only for the requirement of transverse reinforcement. Again in 1978, many reinforced concrete buildings suffered severe structural damage during the Miyagiken–Oki earthquake ($M = 7.4$), mainly due to the structural irregularity. Therefore, extensive research was conducted to change the seismic design provisions in those days (the 1950 Codes). As a result of the research, a new seismic design provision of the Building Standard Law Enforcement Order was

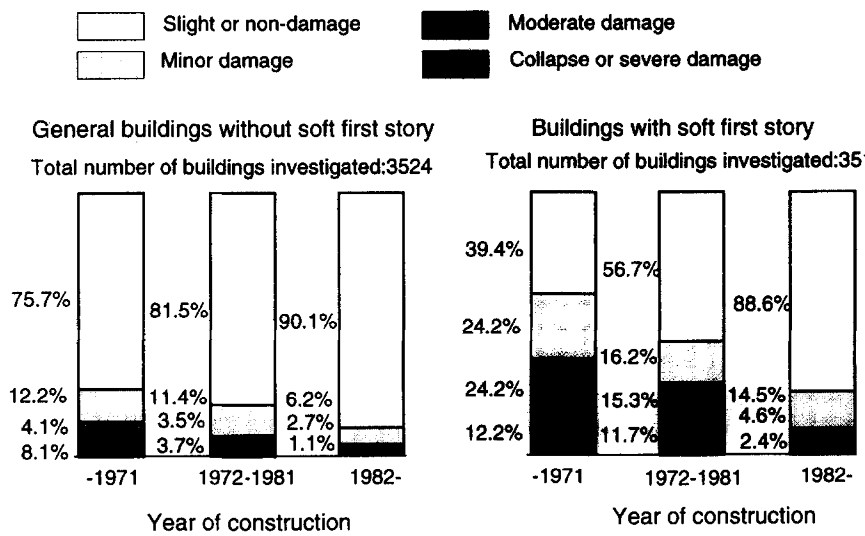

Fig. 5. Damage statistics in severely damaged.

put into operation in 1981 (the 1981 codes,[2]). Code Provisions 1981 are quite different from the 1950 codes and the main points, newly introduced, are:

(1) additional lateral strength to compensate the damage concentration due to the structural irregularity, i.e. structural eccentricity and non-uniform distribution of lateral stiffness along a building height;
(2) new design seismic load distribution along a building height using the base shear coefficient;
(3) adoption of a procedure to examine the lateral story capacity at the formation of the collapse mechanism where the specified compressive strength of concrete and 1.1 times the specified yield strength of longitudinal reinforcement are used for the calculation of flexural strength of beams and columns, where the required lateral story capacity is specified based on the ductility;
(4) shear reinforcement ratio must be larger than 0.2% for beams, column and structural walls; and
(5) design and construction of buildings taller than 60 m need to be approved by the Minister of Construction.

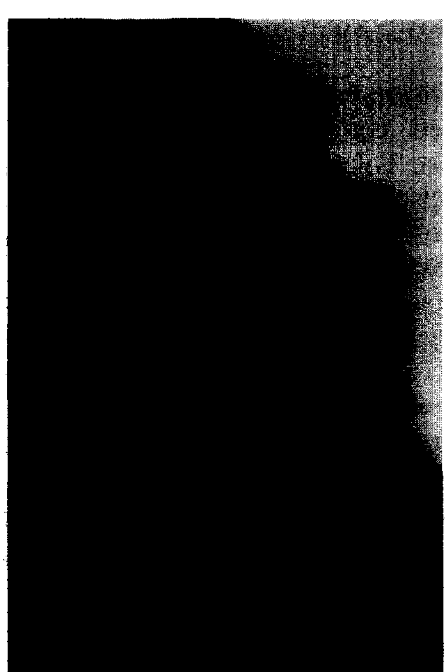

Fig. 6. Damage to elevated corridors.

However, some new modern buildings, built after the 1981 Codes, sustained significant damage. Whether these buildings conformed with the 1981 Codes requirements needs to be determined. While the 1981 Codes were successful in preventing damage in the vast majority of modern buildings, some changes may be warranted if the few modern buildings that suffered substantial damage are found to be in conformance with the 1981 Codes. Typical damage to reinforced concrete buildings in this earthquake is described in the following sections and some possible reasons for the damage are suggested.

Typical damage patterns

Collapse of a soft first story construction

Many of the multistory buildings that collapsed were constructed with open retail space or parking on the first floor. In these buildings, the lateral stiffness of the first story was smaller than that of other stories. Often damage to the structure concentrated in this story. Figure 2 shows a typical example of the damage to a soft first story of the seven storied condominium building with parking space on the first floor. The reason for such damage may be due to the lack of lateral strength and ductility of columns on the first story because this building was built before 1981. Similar damage was observed in new modern buildings designed according to the 1981 Codes. Whether the lateral story stiffness for each story was adequately estimated for this building's height, and a larger lateral strength was provided for the first story columns, is to be examined.

After the earthquake, several researchers investigated the dynamic behavior of such buildings with a soft first story and indicated that the additional lateral strength for such buildings required in the 1981 Codes is not enough. It was also found out that in some cases the lateral story stiffness of walls in overlying stories was underestimated using a very small stiffness reduction factor, for example, 5% of elastic stiffness and then not any additional lateral strength was provided to the soft first story. A more realistic estimation of wall stiffness is needed. From these lessons, the seismic design rules of the Building Standard Law Enforcement Order was intensified in October 1995.[3]

Alternatively, the stiffnesses of the overlying stories may be reduced to avoid the structural irregularity. This can sometimes be achieved by isolating the stiff non-structural partitions from the frame so that the stiffness, strength and framework of the structure are continuous from one floor to another.

First story collapse in regular buildings

Many buildings which had a uniform configuration suffered severe damage to or collapse of the first story as observed in past earthquakes, showing the shear failure of columns due to the insufficient amount of transverse reinforcement and the uncounted shortening of column length by drop walls from upper and lower beams.

Collapse of a mid-height story

A prominent mode of failure of reinforced concrete buildings in this earthquake involved the story collapse of an upper floor level. Several factors potentially responsible for these failures are described in the following.

(1) The design earthquake force distribution over the height specified in 1950 Codes is different from the one used now (1981 Codes). Figure 7 shows a comparison between 1950 and 1981 Codes with respect to story shear coefficient and story shear. It is seen from the figure that the proportion of design story shear was smaller above mid-stories in 1950 codes. On the other hand, in practical construction the minimum amount of longitudinal reinforcement, i.e. sectional area of longitudinal reinforcement should be greater than 0.8% of column sectional area, is arranged in columns, then the lateral strength of column at upper story easily exceeds the design story shear. Then a possibility of mid-height story collapse in buildings built before 1981 arises.
(2) If capacity design approache is not applied during design to prevent the development of story collapse mechanisms, damage may concentrate in any story. Especially when there is a possibility of column shear failure, a story collapse may suddenly occur at a certain story due to its very brittle behavior without any energy absorption.
(3) Damage may concentrate at any story for which the lateral strength and/or stiffness changes abruptly between adjacent stories, particularly if the strength distribution is such that a story collapse mechanism would occur. Some buildings collapsed at the story where the structural system changed from steel-encased reinforced concrete to reinforced concrete.
(4) Large vertical accelerations may have generated large compressive and tensile axial forces in the columns, causing

reductions in ductility and shear strength, respectively.

(5) Compressive failures of the columns at the mid-height of the building may have been induced by the vertical propagation of impulsive waves.

(6) Unexpectedly large stresses in the middle stories could have been generated by resonance between the ground motion and structure; large story drifts, reductions in lateral stiffness and second-order geometric effects (P-delta effects) may have been significant.

Figure 4 shows the Kobe city office building which collapsed at the sixth story. This building was constructed with steel encased reinforced concrete up to the fifth story and with reinforced concrete above, from the sixth to the eighth stories. Story collapse occurred at the story where the lateral strength of column changed. Dynamic time history analysis clearly revealed that the damage concentrated to the sixth story during the earthquake and structural elements failed in shear.

One steel encased reinforced concete building (12 stories) also showed the mid-height collapse. Story collapse was observed at the fifth story. There is a report on the damage to mid-height story. In the report, a similar model building was designed according to the 1950 codes. Each constituent member was dimensioned, based on the allowable stress design method. Three-dimensional time history analysis was conducted on this model structure. The fundamental period of the model building is 0.956 s. Original earthquake records (N-S component and U-D component) obtained at the Kobe Maritime Meteorological Observatory (KMMO) were used for time history analysis. The main results of analysis were:

(1) U-D component of the earthquake does not have a remarkable effect on the column response;

(2) above the fourth story, response shear becomes very close to or exceeds the column shear strength and, thus, the possibility of a collapse at mid-height story arises; and

(3) 1981 Codes require larger design shear above the third story. If the model building was designed according to the 1981 Codes the lateral story shear strength above the third story becomes larger than that of the model building, then such mid-height collapse may be prevented.

Column failure due to torsional response

Several buildings built before 1981 suffered serious damage due to the eccentricity of stiffness and mass resulting from the structural irregularity. For such buildings sufficient lateral strength and ductility of columns are required to compensate for the torsional response of the considered story as specified in the 1981 Codes. 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.

Damage to separation joints and elevated corridors

Many instances of damage to building separation joints and elevated corridors connecting adjacent buildings were observed as shown in Fig. 6. Differences in the response of the connected buildings can lead to damage to separation joints and corridors. Adequate separation and/or the use of flexibly connected structures should be considered to reduce such damage. When a corridor connecting two adjacent buildings is large, inertial response of the corridor's mass should be accounted for when detailing the connecting elements.

Damage to non-structural members

Significant damage was observed to non-structural partitions in many buildings in which little or no structural damage occurred. Such damage could be avoided if the non-structural partitions were isolated from the structural deformations. As an alternative, detailing that allows easy replacement is also effective.

Inadequate spacing and anchorage of transverse reinforcement

Most older buildings that were damaged had little transverse reinforcement in their columns as shown in Fig. 8. These details would not meet the requirements of the 1981 Codes. Many columns had transverse reinforcement spaced at 20–30 cm; since 1971, hoops had been required at a spacing of 10 cm or less in the

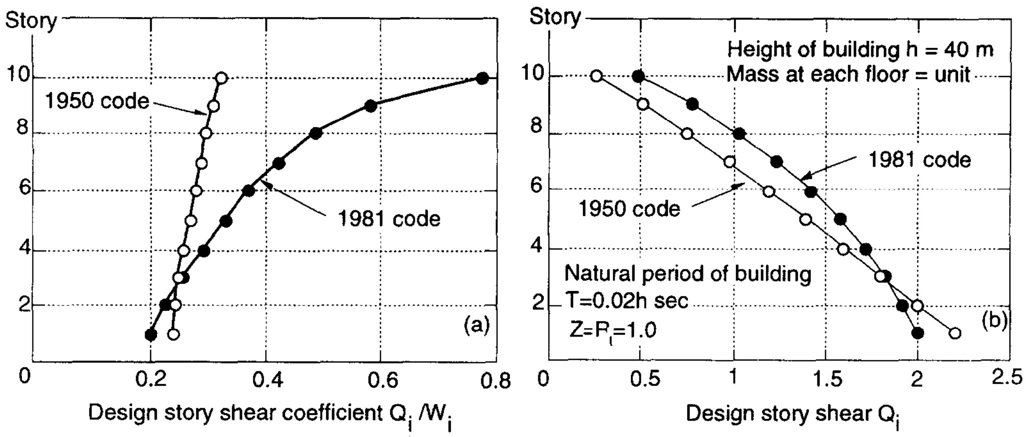

Fig. 7. Design seismic loads of the 1950 codes and the 1981 codes.

column end regions and 15 cm or less elsewhere. In columns that were damaged, most transverse reinforcement had 90° hooks and relatively short extensions. AIJ Standard for Structural Calculation of Reinforced Concrete Structures[4] specifies the anchor details of transverse reinforcement, where 135° hooks and 6 times bar diameter extension length are required. However, this specification was not followed in many old buildings, because this was not clearly indicated in the 1950 Codes. Therefore, the anchorage detail for transverse reinforcement was then clearly specified in October 1995.[2]

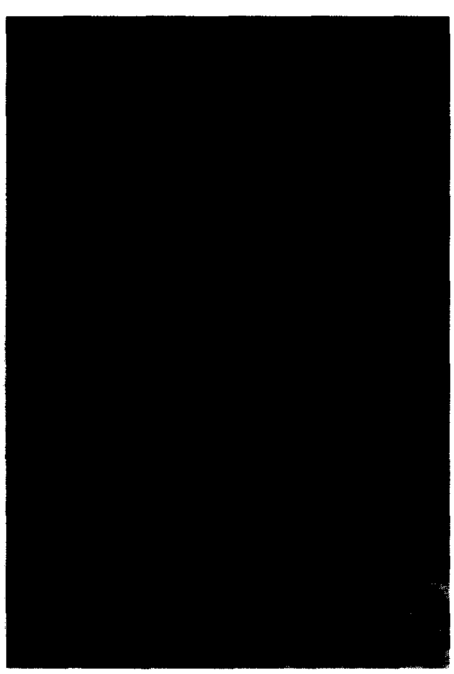

Fig. 8. Shear failure of column with inadequate transverse reinforcement.

Shear failure of short columns

Brittle shear failures of short columns were prominent in the Tokachi–Oki earthquake of 1968. These failures were again prominent among older buildings damaged by the Hyougo-

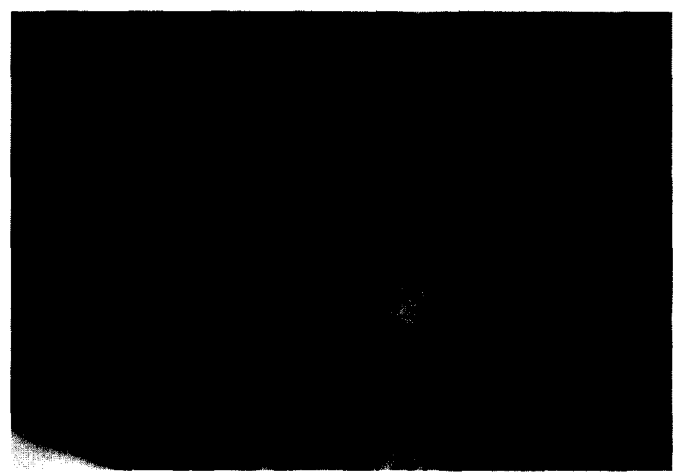

Fig. 9. Failure of ğas-pressure welded' reinforcement splices.

Table 1. Performance matrix

Repair level	Function level		
	A	B	C
A No			
B Slight			
C Medium			
D Heavy			
E Impossible			

ken–Nanbu earthquake. Deep spandrel beams at the building perimeter can create captive column conditions in which column shear failures are likely to occur. In such cases, separation joints must be provided to isolate the columns from the adjacent material, or the columns must be detailed to be ductile by providing large amounts of transverse reinforcement.

Failure of 'gas-pressure welded' reinforcement splices

In Japan, reinforcement splices in buildings and bridges in recent decades have been almost always made by a process known as 'gas-pressure welding', in which the bars to be joined are aligned and butted together: the bars are fused together by heat and pressure applied by mechanical devices, causing the bars to flare out or 'mushroom' at the splice. Pressure welded splices were observed to fracture in this earthquake as shown in Fig. 9. One possible reason of fracture may be poor workmanship at the construction site. The specifications for gas-pressure welding should be examined as well.

Damage to beam–column joints

Shear failures of beam–column joints were also observed in some buildings. These may have been accompanied by bond failure of beam longitudinal reinforcement. However, it should be noted that interior beam–column joints could not be observed by exterior inspection.

Several damage patterns were observed as mentioned above. Especially the story collapse was one of the most remarkable damage pattern to reinforced concrete buildings during the earthquake, including collapse at mid-height story. Such damage was observed even in buildings with regular planning. This may point out the importance of the care to final collapse mechanism of buildings during design process. If special care is taken to prevent the formation of a story sway due to the plastic hinging of columns, such failure could be prevented. AIJ Guidelines[5] and NZS Standard[6] recommend the capacity design to realize the beam hinging side-sway mechanism. One condominium building was designed to show the beam hinging side-sway collapse mechanism. This building was in the severely damaged area, however, it behaved well, showing beam hinging at each beam end section in x-direction.

CONCLUDING REMARKS

From the damage investigation, the following points can be stated for examination:

(1) Strength and ductility demand to first story columns for the construction of a soft first story.
(2) Realistic evaluation of lateral stiffness of structural walls.
(3) Design conditions for the building which shows column side-sway collapse mechanism.
(4) Detailing of transverse reinforcement.
(5) Assurance of good workmanship at the construction site.

This earthquake re-emphasizes the need to control the damage sustained by buildings in major earthquakes. Unquestionably, casualties caused by the collapse of buildings must be prevented. A performance criterion may also be needed in the design process to control damage in accordance with the type of buildings and their occupancy. Table 1 shows one idea of the damage controlled design criteria. The designer should choose the most appropriate levels for repair and function against the target earthquake. The level of the target earthquake should be discussed based on the correlation between the return period and the frequency during the life time of the building. One example of the levels of design earthquakes is indicated below.

Earthquake level

Level-1: minor earthquake (return period of 20–50 years).

Level-2: major earthquake (return period of 100–200 years).

Level-3: extreme earthquake (return period of 1000–2000 years).

The function level indicated in Table 1 is roughly determined as follows.Function level

Level A: the function of a building is maintained during and after the earthquake.

Level B: the function of a building is lost during the earthquake, but it can be recovered after the earthquake.

Level C: the function of a building is lost during the earthquake and cannot be recovered after the earthquake.

Engineering measures of function may be the maximum acceleration, velocity and drift at floors. The repair levels may be evaluated by predicting the residual crack width, the residual plastic strains of concrete and reinforcement and the permanent deformation of members and buildings. This repair level strongly relates to the repair expenses.

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