THE HYOGO-KEN NANBU EARTHQUAKE (THE GREAT HANSHIN EARTHQUAKE) OF 17 JANUARY 1995

Report of the NZNSEE Reconnaissance Team

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ABSTRACT

This report describes the observations and preliminary assessments of the members of the Reconnaissance Team of the New Zealand National Society for Earthquake Engineering which visited Kobe, Japan and the surrounding areas following the Hyogo-ken Nanbu earthquake of 17 January 1995. The report covers aspects of the effects of the earthquake on the ground, lifelines, buildings, bridges and other structures, and the community. Lessons for New Zealand are discussed.

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1 INTRODUCTION

At 5.46 am on Tuesday 17 January 1995 an earthquake of Richter magnitude 7.2 occurred with epicentre located about 20 km to the southwest of the coastal part of Kobe. The epicentre was located just off the northeast end of Awaji Island (see Figure 1.1). The depth of the source of the earthquake was 16 km. As at 1 March 1995 the death toll was 5,452 people (The Japan Times, 2 March 1995). More than 33,000 people were injured.

The earthquake has become known in the popular press in Japan as the Great Hanshin Earthquake, but the formal name given by the Meteorological Agency of Japan is the Hyogo-ken Nanbu Earthquake. The population of the Hyogo Prefecture is 5.5 million and of the Osaka Prefecture is 8.5 million. The Hanshin district is the area of those prefectures which includes Kobe city, Osaka city and the cities between them located beside the Inland Sea (see Figure 1.2). The Hanshin district is the second most important industrial commercial area in Japan after the Tokyo-Yokohama area.



FIGURE 1.1 Japan Showing the Epicentre of the Hyogo-ken Nanbu Earthquake of 17 January 1995

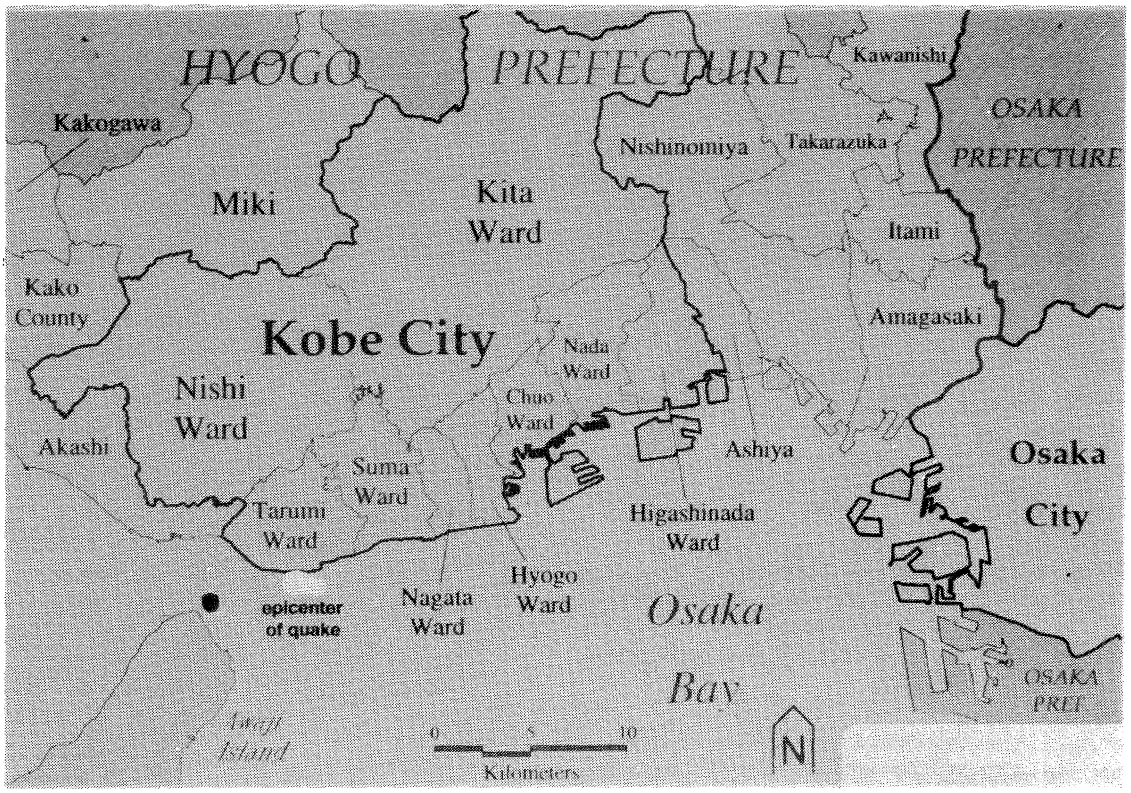


FIGURE 1.2 Kobe City and the Surrounding Areas



FIGURE 1.3 Kobe City Between the Rokko Mountains and the Inland Sea

The areas worst hit by the Hyogo-ken Nanbu Earthquake were the urban areas of Kobe, Ashiya, Nishinomiya and Takarazuka, and also the northern part of Awaji Island.

The city of Kobe is situated beside the Inland Sea of Japan (see Figure 1.3), nestled each side of the Rokko Mountains. The city has a population of 1.5 million. It has very important port facilities, being the sixth largest port in the world in terms of tonnage throughout in 1994, and handling for many years the greatest volume of containerised cargo of any port in Japan. The part of Kobe city worst affected was Sannomiya in the Chuo Ward. Bridges, roads, railways, the port facilities of Kobe and the utilities were badly damaged. Fires followed the earthquake in many areas. The death toll is the greatest due to an earthquake in Japan since the Great Kanto Earthquake in 1923 in Tokyo, which caused 140,000 deaths. Estimates are that the direct rebuilding costs as a result of the earthquake damage will be in the order of NZ$200 billion.

The severe ground motions measured in Kobe lasted about 20 seconds. The maximum peak horizontal ground acceleration reported to date is 0.85g at Fukiai very close to Sannomiya. The ground motion record there showed about two or three major cycles of horizontal motion.

The main party of the Reconnaissance Team of 13 members from the New Zealand National Society for Earthquake Engineering arrived in Japan six days after the earthquake on Monday 23 January and left on Thursday 2 February. The team spent 10 nights in Japan: 5 in a hotel in Kyoto and 5 in a hotel in Osaka. Travel to and from Kobe was generally either by train or ferry boat, sometimes combined with bus. The transportation was only to the edge of Kobe due to rail services in Kobe being out of action. The reconnaissance into the city was by foot. The team logged up many kilometres of walking to the various areas of interest in Kobe and the surrounding areas.

This report of the Reconnaissance Team covers aspects of the effects of the earthquake on the ground, lifelines, buildings, bridges and other structures, and the community. Lessons for New Zealand are discussed.

2 SEISMICITY, GEOLOGY AND TECTONICS

2.1 Historical Seismicity

Kobe city and its environs are regarded as areas of relatively low seismicity for Japan. Historical records suggest that there have been about 25 earthquakes larger than Richter magnitude 6 within 100 km of Kobe since year 827 AD, and 4 larger than Richter magnitude 7. In terms of the Smith and Berryman seismicity model for New Zealand, and bearing in mind that the catalogue is unlikely to be complete, the above numbers suggest an a_6 value of approximately 0.3, where a_6 is the number of earthquakes with Richter magnitude greater than or equal to 6 per year per 100,000 km^2. In comparison, the main seismic regions of New Zealand have a_6 values in the range of 0.2 to 0.4 and regions of low seismicity have a_6 values below 0.1 [Smith and Berryman, 1986 and 1992].

Four of the historical earthquakes were of Richter magnitude 7 or above. The dates, Richter magnitudes, and epicentral distances from Kobe for them, and the Hyogo-ken Nanbu earthquake, were:

1596	7.3	56 km
1662	7.8	98
1854	7.6	77
1852	7.0	61
1995	7.2	20

In 1916 there was a Richter magnitude 6 event at the same location as the current Hyogo-ken Nanbu earthquake. The fault involved, the Nojima fault located at the northern end of Awaji Island, is not known to have produced any other significant earthquakes during the preceding 1000 years.

2.2 Geology and Tectonics

The basement rocks of the Osaka Bay area (Figure 2.1) are highly metamorphosed sediment of Palaeozoic age (> 230 million years). They have been folded and faulted during at least orogenies during the Mesozoic era, and in the area affected by the earthquake were almost completely melted and recrystallised into granites. Younger granites were also intruded during the Cretaceous period (about 150-70 million years ago, Minato et al, 1965).

During the Tertiary Period, the area became inundated during the middle Miocene (about 15 million years ago), and again during latest Pliocene-earliest Pleistocene times (7-3 million years ago), when shallow marine sediments and onshore terrace gravels were deposited.

The weak blue-grey siltstones of Miocene age seen on Awaji Island are similar to New Zealand's Upper Tertiary age sediments ("Papa"). Such sediments are also apparently present beneath the Holocene alluvial and beach deposits of the Osaka Bay lowlands.

The rise in sea level after the last glaciation (ending about 10,000 years ago with sea level rising to about 5 m higher than the present level in this area by 6,000 years ago) inundated all areas with a present altitude of less than 10 m, and extensive estuarine and alluvial deposits accumulated [Ota, 1981]. Tectonic uplift subsequently formed a shoreline in the order of a kilometre inland from the present shoreline, which itself has been formed by major reclamations in the entire bay area, including many large artificial islands (for example, Port Island of 436 ha and currently being extended to twice this size, and Rokko Island of 580 ha).

Deep chemical weathering of the granite bedrock is apparent in the foothills of the Rokko Mountains on the edge of Kobe city, where the rock is reduced to a fine to coarse sand composed of mostly quartz and feldspar, the main constituent mineral crystals of the granite. Surprisingly there is little clay development, and the in-situ weathered rock is loose and friable with little cohesion. This is apparently the source for the Holocene sands of the bay area which have liquefied in the earthquake.

Liquefiable sands and sandy silts are present under all the reclamations and former beach areas. These are post-glacial (< 10,000 years old) shallow water and beach deposits about 5 to 10 m deep laid down over the last 2,000 years. Earlier Holocene marine silt, clay and sand up to 20 m deep overlie the upper Pleistocene gravels which in turn overlie stiffer Plio-Pleistocene and mid-Miocene marine sediments.

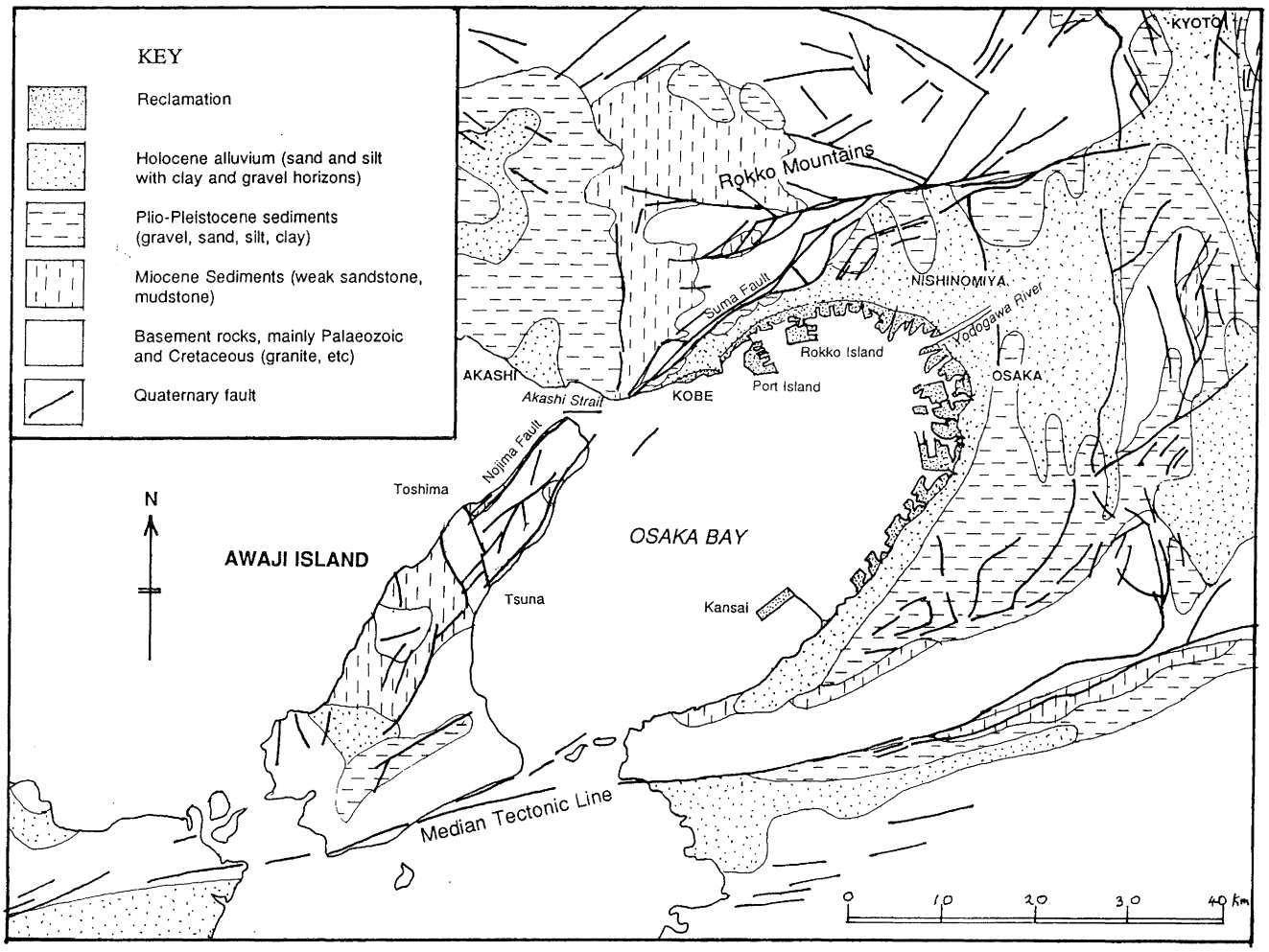


FIGURE 2.1 Generalised Geology and Faulting of Osaka Bay Area

Reclamation materials revealed by ground damage in the dockside areas are silty gravelly sands to sandy silty gravels. Similar material, which was being used to reinstate collapsed dockside areas, was observed on barges from quarries on Awaji Island, and similar materials are apparent in quarries in the Rokko Mountains. Lower quality materials from dredging (sand and silt) are present below the compacted capping of the reclamations. The total thickness of reclamation fill is up to 15 m.

The contiguous cities of Kobe, Ashiya, and Nishinomiya are located on the north-western side of Osaka Bay on a 2 to 3 km wide coastal plain. Immediately to the northwest of the cities is the Rokko mountain range which has as its highest point the 900 m Mt Rokko. During the last 1 million years the Osaka Bay region has been undergoing compressive deformation which, over the last 300,000 years, has resulted in the uplift of Mt Rokko and a 900 m downwarping of Osaka Bay on the south-eastern side of Kobe city. The rate of uplift of Mt Rokko, 3 mm/year, is approximately one tenth that of the Southern Alps in New Zealand.

A major, active, transcurrent fault called the Median Tectonic Line runs roughly east-northeast to the south of Osaka Bay, along the south coast of Awaji Island. Branching away from this are northeast to east-northeast trending faults which run through Awaji Island and across the Akashi Strait along the foot of the Rokko Mountains, forming the northwest margin of the Osaka Bay lowlands. The Nojima Fault on Awaji Island and the Suma fault, which were the source of the Hyogo-ken Nanbu earthquake, are not part of the main zone of active faults previously recognised in the region. The Nojima Fault was mapped as having less than 1 m displacement per 1000 years, while parts of the Suma Fault to the north of Kobe are recognised as undergoing more than 1 m of displacement per 1000 years.

The contiguous cities of Kobe, Ashiya and Nishinomiya are located on the north-western side of Osaka Bay on a 2 to 3 km wide coastal plain. Immediately to the northwest of the cities is the Rokko mountain range which has as its highest point the 900 m Mt Rokko. During the last 1 million years the Osaka Bay region has been undergoing compressive deformation which, over the last 300,000 years, has resulted in the uplift of Mt Rokko and a 900 m downwarping of Osaka Bay on the south-eastern side of Kobe city. The rate of uplift of Mt Rokko, 3 mm/year, is approximately one tenth that of the Southern Alps in New Zealand.

In the last 10,000 years there has been little or no uplift in the Osaka Bay area (the upper altitude limit of Holocene marine deposits ranges between 2 and -2 metres above present sea level). This means that there is no significant vertical component of movement on local active faults over this time.

2.3 The Hyogo-ken Nanbu Earthquake

Seismological details for the Hyogo-ken Nanbu earthquake of 20h 46m 51.0s UT, 16 January 1995 (5h 46m 51.0s local time on 17 January 1995) are as follows:

M_L (Local or Richter magnitude)	7.2
M_w (moment magnitude)	6.9
M_s (surface wave magnitude)	6.9
Epicentre location	34.527N, 135.005E
Hypocentral Depth	16 km

At least two faults are thought to have ruptured during the mainshock, the Nojima Fault on Awaji Island and the Suma Fault which lies beneath Kobe city (see Figure 2.1). Ground surface rupture has been observed on the Nojima Fault from Toshima village to the northern tip of Awaji Island, very close to the epicentre. Displacements of up to 1.5 m right-lateral horizontal and 0.5 m vertical were found. The continuation of this rupture has not been found across the Akashi Strait towards Kobe, but it is reported that a vertical offset on a fault was found by sidescan sonar in the strait a few kilometres east of the projection of the Nojima Fault. The piers of the Akashi Strait bridge, which is under construction, have been measured to be about 1 m further apart after the earthquake than before, consistent with a right-lateral fault displacement occurring between them. The Suma Fault runs through Kobe, Ashiya and Nishinomiya along the zone of greatest damage, but no ground surface rupture has been observed. Ground surface ruptures observed in Ashiya and Nishinomiya do not appear to be primary seismogenic ground ruptures, but are thought to be secondary features such as incipient landsliding or ridge-rents. Displacements here are mainly vertical, and there are signs of spreading, with inconsistent up and downthrown sides. However, they do seem to be aligned along a major plane of weakness, probably a fault.

Aftershocks recorded from 2 to 10 days after the mainshock (see Figure 2.2) define a 60 km long by 20 km deep linear zone extending from the south-western end of the Nojima Fault, along the north-western side of Kobe city, and into Nishinomiya city. The zone approximately follows the Nojima and Suma Faults.

A narrow zone of intensive damage to houses, commercial buildings, and the ground, has been delineated through Kobe, Ashiya and Nishinomiya cities (see Figure 2.3). The zone, which was also associated with most of the deaths caused by the earthquake, has a width of 2 to 3 km and extends roughly parallel to the coast for a distance of nearly 30 km. It has been called the "fault zone" on the grounds that it could be the surface expression of a concealed seismogenic fault, but it also coincides with a mid-Holocene shoreline, is close to the line of contact between the Quaternary basin of Osaka Bay and the granitic bedrock mountains that rise to the west of Kobe, and it does not coincide exactly with the rupture zone defined by aftershock locations. Whereas on Awaji Island there is a 10 km long surface rupture, no such rupture has been found in the cities of Kobe, Ashiya and Nishinomiya. On the basis of the observed damage and recorded ground motions an intensity of 7, on the Japanese 7 point JMA scale, has been assigned to the zone.

There was little evidence of liquefaction inside the "fault zone", mainly because it was mostly located on slightly elevated land which had some relief. The extensive areas of liquefaction were located on low lying land and reclamations on the seaward side of the zone.

2.4 Recorded Ground Motions

Several organisations manage networks of strong-motion accelerographs in Japan, and in due course possibly as many as 200 recordings of ground motions and structural responses for the Hyogo-ken Nanbu earthquake will become available. It is significant that 12 of the recordings reported to date were from sites within 10 km of the "fault zone", because world wide there is still very little strong-motion data from locations where the shaking intensity was MM10 (Modified Mercalli scale). The highest intensity for the 1994 Northridge earthquake was only MM9, yet PGA (peak ground acceleration) values recorded near the epicentre in that earthquake were similar, in some cases larger, than those recorded near Kobe.

The strongest PGAs reported to date range from about 0.5g to 0.85g. The most significant are listed in Table 2.1 and along with the more distant recordings, are compared with selected attenuation models in Figure 2.4. Two main points arise from the plot. Firstly, all four models fit the recorded data relatively well up to a distance of 30 km. From 30 to 300 km the models sourced from New Zealand and Japan [McVerry, Fukushima and Tanaka] fit the data better than the other two which are sourced from the western USA, though it should be borne in mind that the range of validity claimed for the Boore Joyner and Fumal model is up to 100 km. Secondly, the recorded PGA's seem consistently larger than predicted by any of the attenuation models at distances from 80 to 200 km. A similar phenomenon has been observed in other earthquakes and has been attributed to the arrival of S waves that have been reflected from the base of the crust [Somerville and Yoshimura, 1990].

At the time of writing very little accelerogram data was available in a form suitable for reproduction. However, accelerograms recorded within a few km of the "fault zone" seemed to be dominated by a few large cycles of motion in a time span of 5 to 6 seconds (see Figure 2.5a), which were followed by (relatively) lower amplitude motions for a further 10 to 20 seconds. At sites 20 to 30 km from the "fault zone" the strongest motions lasted for about 10 seconds, and were followed by 50 seconds of lower amplitude motions that may in

Table 2.1 Peak Ground Accelerations from Sites Within 10 km of the Zone of Intense Destruction, the "Fault Zone", of the Hyogo-ken Nanbu Earthquake

Site Name	PGA (g)	Estimated distance from "fault zone" (km)
Nishinomiya	0.81	0
Takatori Station	0.63	0
Kobe Motoyama	0.79	≤ 1
Shin-Kobe Station	0.57	1
Kobe Port Construction Office	> 0.5	1
Fukiai	0.85	2
Kobe University	0.31	2
Amagasaki	0.33	3
Amagasaki (port)	0.48	4
Takarazuka Station	0.61	6
Nishi-Akashi Station	0.49	8
Shin-Osaka Station	0.25	10

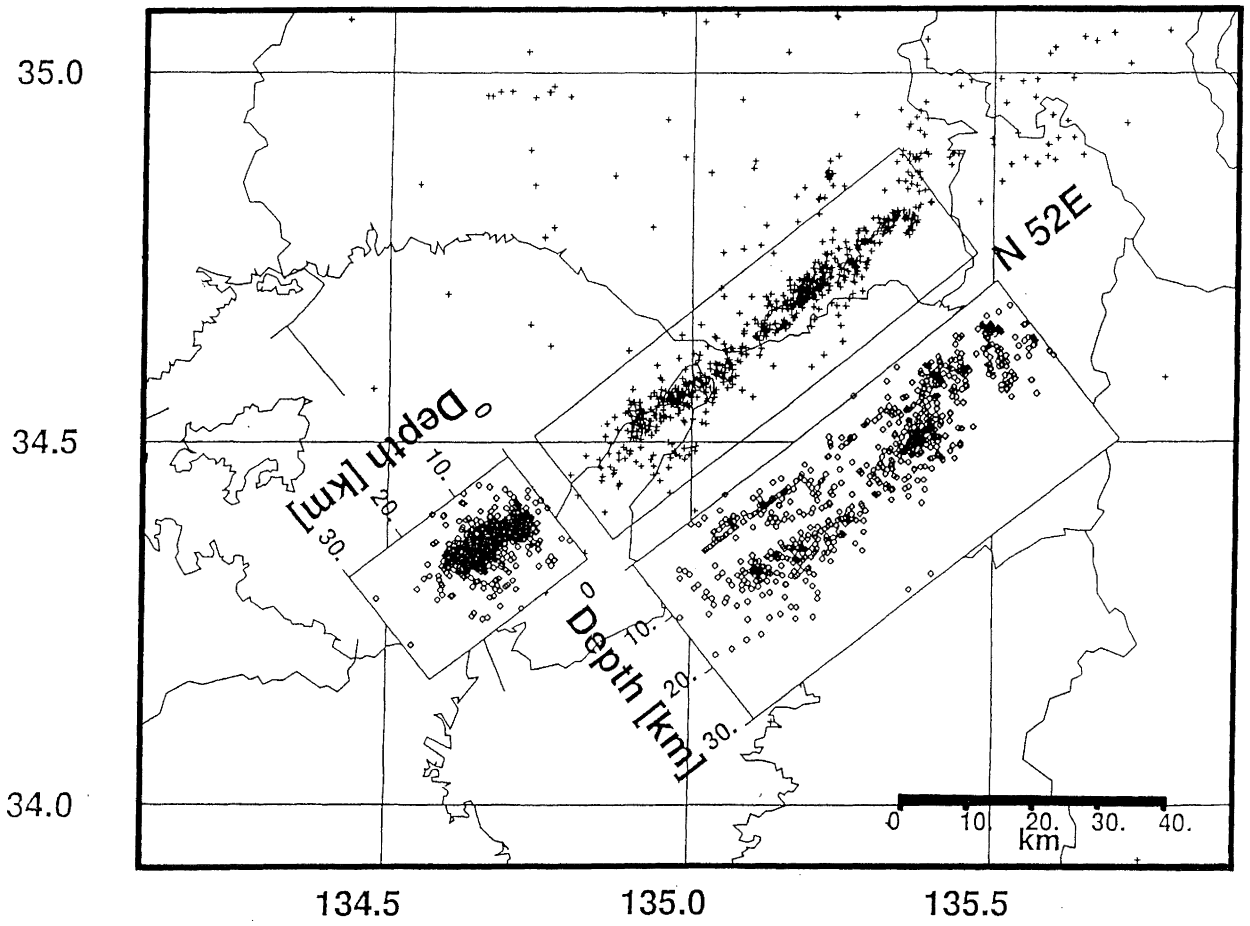


FIGURE 2.2 Locations of Aftershocks Recorded from 19 to 27 January, 1995; Rectangular Plan View and Vertical Projections

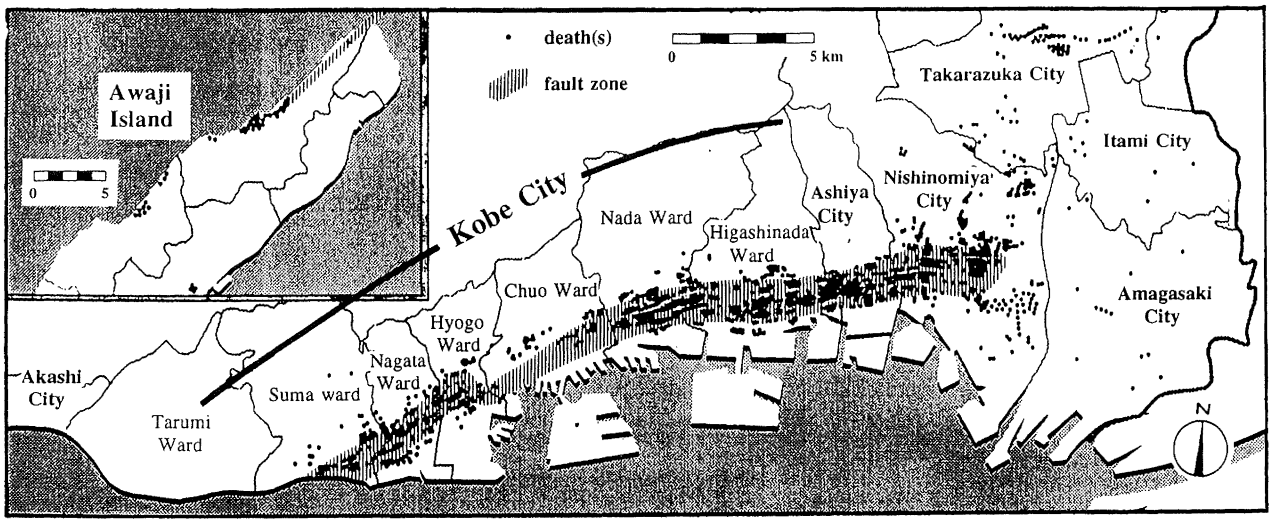


FIGURE 2.3 Location of "Fault Zones" and Places Where Deaths Occurred

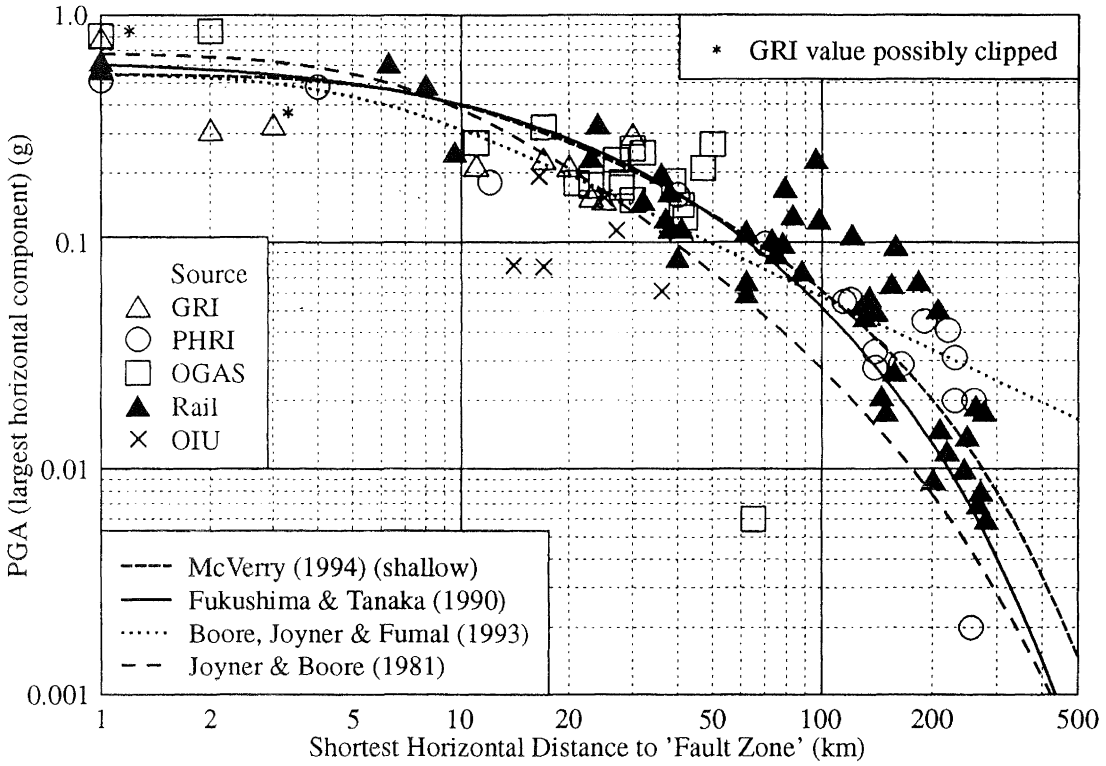


FIGURE 2.4 Peak Ground Accelerations Recorded During the Hyogo-ken Nanbu Earthquake. (Sources of data are: "GRI" Geological Research Institute, Osaka, "PHRI" Port and Harbour Research Institute, "OGAS" Osaka Gas, "Rail" Railway Technical Research Institute, and "OIU" Osaka Industrial University. Four attenuation models are also plotted).

some cases have represented a site response. Two response spectra, one from a site in Kobe itself (the site that yielded the PGA of 0.79g) and one from a site south of Osaka at a distance of 27 km from the "fault zone" are reproduced in Figure 2.5b-c.

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3 GROUND DAMAGE

3.1 Slope Stability

Given the steep topography surrounding Kobe city there was little evidence of slope instability. Minor surficial slope failures were observed in many parts of the Rokko Mountains and the

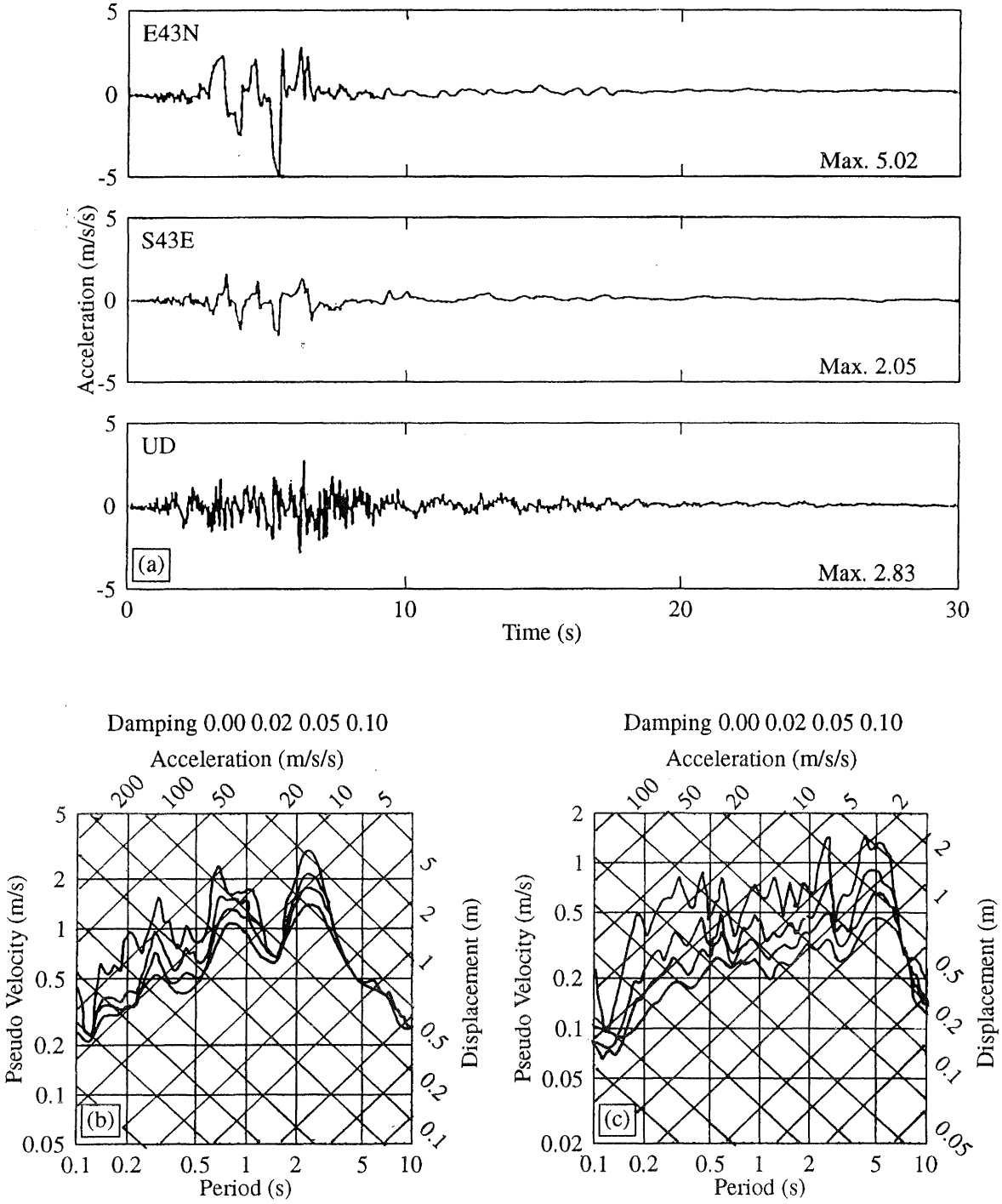


FIGURE 2.5 Representative Strong-Motion Data from the Hyogo-ken Nanbu Mainshock:
(a) (top) Accelerogram recorded at the Kobe Port Construction Office (note that the E43N component was clipped at 5.02 m/s/s),
(b) (bottom left) Response spectrum for site Kobe Motoyama, and
(c) (bottom right) Response spectrum for site Sakai
(Both of the Kobe sites were less than 1 km from the "fault zone", while the Sakai site was south of Osaka and 27 km from the "fault zone").

northern area of Awaji Island (minor rockfalls). It was reported that the road crossing the northern end of Awaji Island was blocked by landslides.

In the Kobe area two significant landslides at Nigawa and Okamoto resulted in damage to houses and many deaths. A 15 m high landslide dam was formed in the Ashiya river.

The Nigawa landslide, initiated by the earthquake, involved a modified slope below a water treatment station. It appears that fill material had been placed on the slope associated with the establishment of the station platform. The landslide debris appeared to be relatively dry, yellowish-brown weathered granite. The slope height was some 40 metres and had an overall slope of some 20 to 30 degrees. The area below the slope was densely developed with residential housing.

The relatively small amount of slope damage is attributed to a number of aspects:

- the earthquake occurred during the driest month of the year.
- the weathered granitic rocks which form the mountains are massive and relatively free of defects (bedding planes, persistent closely spaced joints, clay coated joint and fault surfaces, etc) as compared to New Zealand rocks (for example, greywacke). Also the weathered granite has a very granular texture and is relatively free draining.
- the absence of weak regolith material accumulation on many slopes may be a result of frequent typhoons in the area washing such material away. It is likely that the typhoons have a more detrimental effect on the stability of these slopes than the earthquake did. Some areas of elaborate reinforced concrete grillage slope protection to mitigate typhoon-induced instability were observed (see Figure 3.10).
- Displacements on faults in the Kobe area have not produced very high, steep fault line scarps such as those seen on the Wellington and Wairarapa faults. Also, rapid regional uplift which causes rapid downcutting by streams with the resultant formation of oversteepened, unstable slopes is not so prevalent in the Kobe area as in New Zealand.

3.2 Liquefaction, Lateral Spreading and Settlement

Liquefaction as demonstrated by the ejection of sand was found to be pervasive through the Kobe area extending from Suma north to the Yodogawa River (see Figure 2.1 for location) and there was evidence of liquefaction at Osaka-ko (Port Osaka). Liquefaction in this form was generally confined to the coastal strip (reclaimed land) and the multitude of reclaimed islands (see Figure 3.1).

Severe lateral spreading was associated with ground surface evidence of liquefaction and often extended some 100 m behind port breast works. Moderate ground damage often extended up

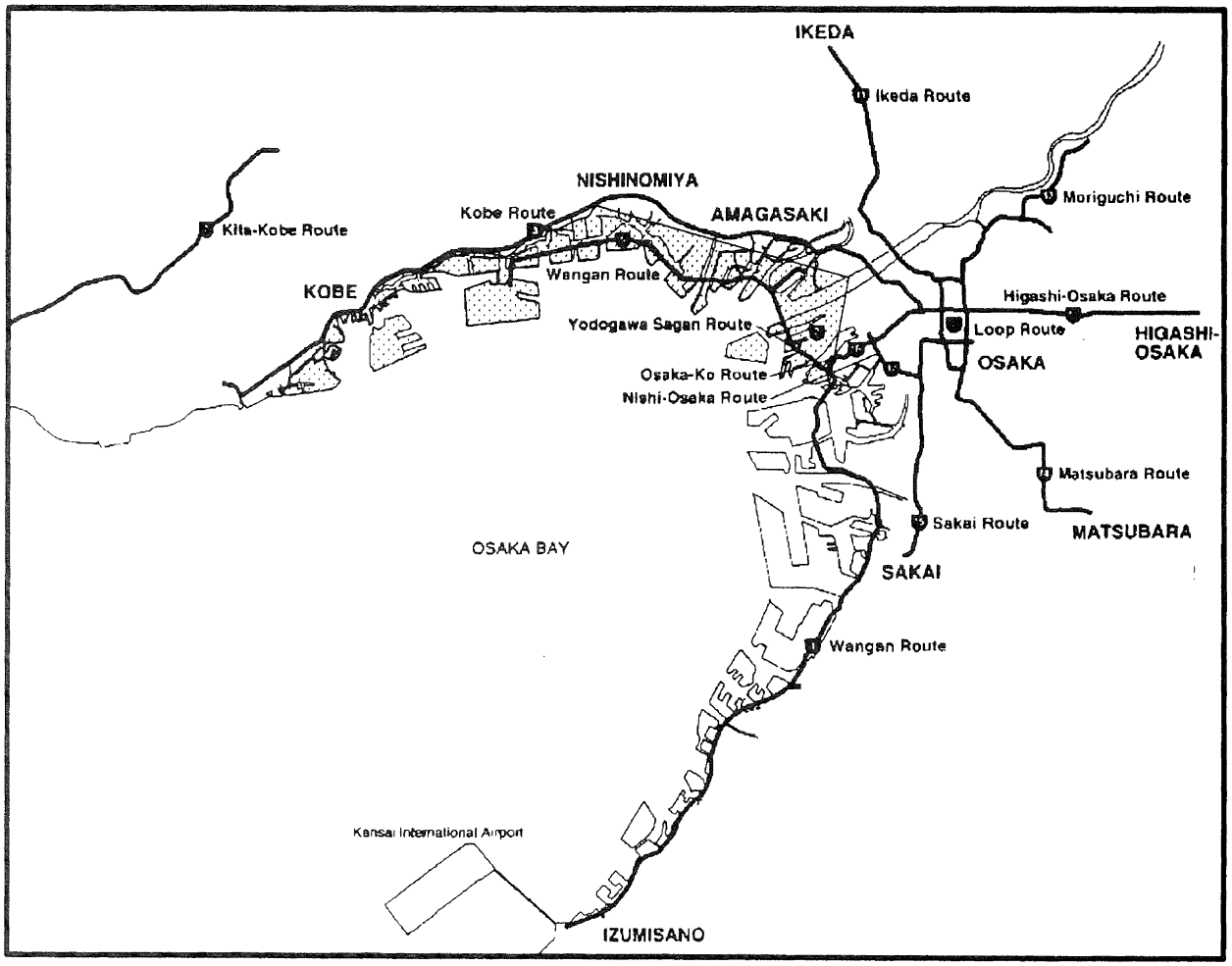


FIGURE 3.1 Osaka Bay. Area of Liquefaction Occurrence as Demonstrated by Ejection of Sand (shown shaded)

to 150 m and was observed in the vicinity of some rivers and in localised areas. Often areas of moderate ground damage extended inland from the coastal margin. Minor ground damage was observed in many areas and was usually apparent in the form of damaged footpaths and settlement adjacent to buildings. These descriptions of ground damage are typically as follows

Minor	$\Delta H < 50$ mm	$\Delta V < 50$ mm
Moderate	$\Delta H \leq 200$ mm	$\Delta V \leq 200$ mm
Severe	$\Delta H > 200$ mm	$\Delta V > 200$ mm, ejected sand

where ΔH = horizontal movement and ΔV = vertical movement.

Examples of the different levels of ground damage are presented in Figures 3.2 to 3.6.

On first observation it is difficult to ascertain any pattern to the ground damage apart from the obvious concentration along the coastal margin and the reclaimed areas. In and around the built up area from south of Kobe to Nishinomiya there were local areas of minor ground damage which are presumed to be related to variability in the colluvial/alluvial founding soils. Commonly building damage was observed to be more severe in these areas.

In areas of severe liquefaction, 2 to 4 storey concrete buildings were observed to settle and tilt (see Figure 3.7) but generally damage was light as multi-storey structures are typically piled. There was structural damage/failure of structures directly attributable to liquefaction/lateral spreading, particularly along the margins of the port structures.



FIGURE 3.2 Hanshin Expressway, Ogi Area. Minor Ground Damage - Note Kerbline Rotation



FIGURE 3.3 Sannomiya, Near Flower Road. Minor Damage to Street Paving

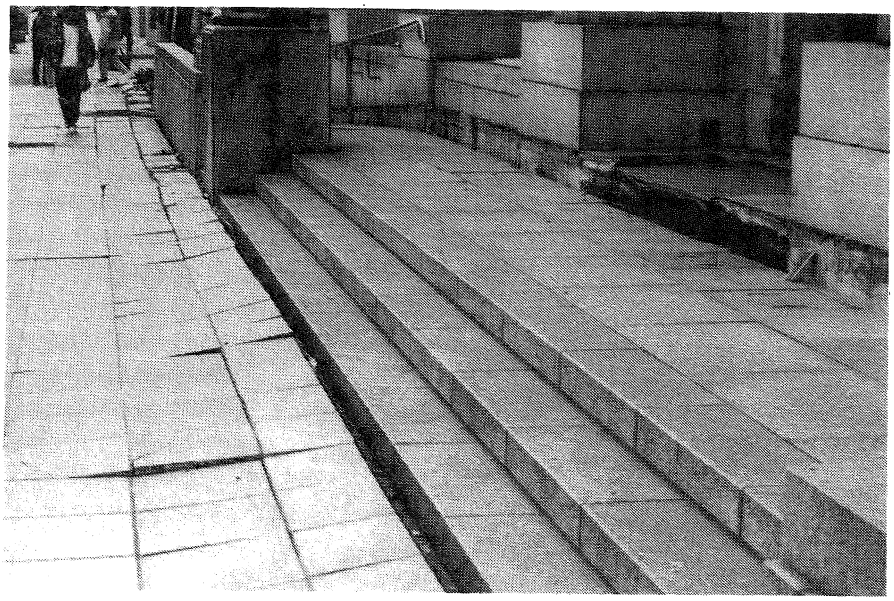


FIGURE 3.4 Kobe Museum, Sannomiya. Moderate Damage to Steps. Settlement Approximately 200 mm

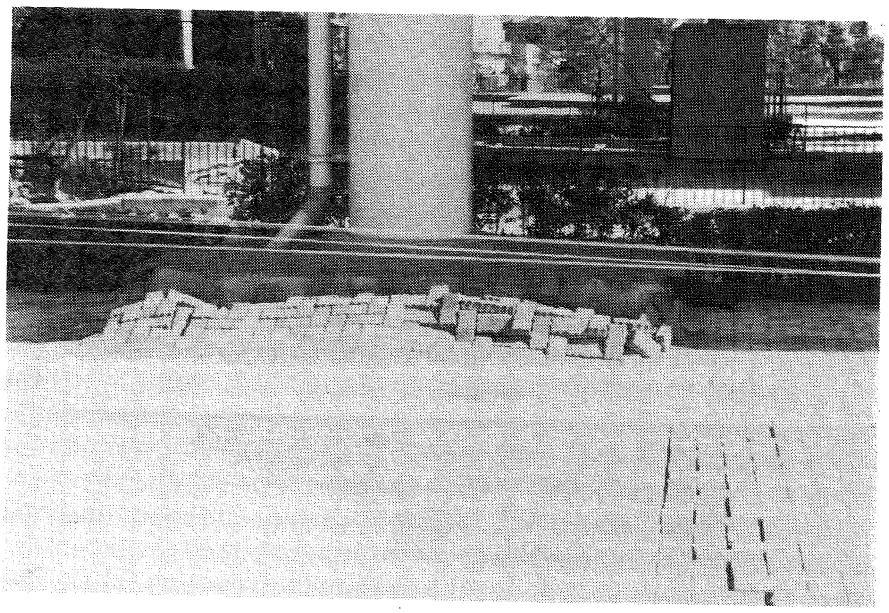


FIGURE 3.5 Rokko Island. Severe Ground Damage. Settlement Approximately 400 mm.

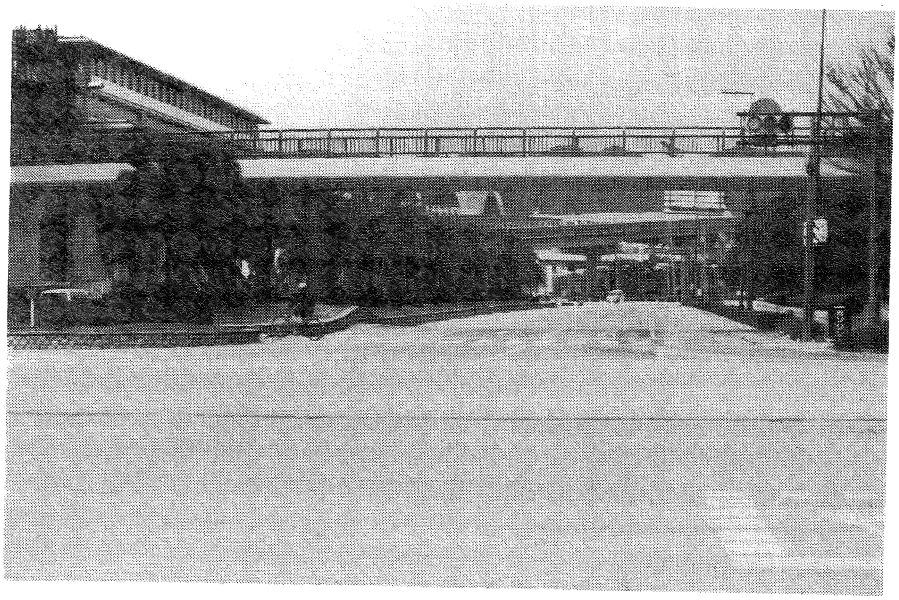


FIGURE 3.6 Port Island, Near Sannomiya. Severe Ground Damage. Note Hump at Bridge Pier and Sand on Street but Little Evidence of Disruption/ Deformation of Pavement



FIGURE 3.7 Higashi Yuenchi Park, Sannomiya. Small Concrete Building Has Settled > 200 mm and Tilted. Ejected Sand was Visible at Front of Building

The reclaimed islands within the port area have experienced major damage associated with displacement and rotation of the gravity caisson wall structures which form the perimeter. This displacement, estimated at 2 to 4 m, and the associated rotation, has resulted in severe damage to container cranes and adjacent port structures and sheds. While liquefaction, with associated settlements, was clearly evident in the central area of the reclaimed islands piled structures were largely unaffected while roads settled (with little uneven deformation) and buried services were severely distorted and ruptured. Further discussion of port facilities is presented in Section 8.

Liquefiable sands and sandy silts are present under the coastal reclamations and former beach areas. There are post-glacial shallow water and beach deposits about 5 metres deep laid down over the past 2000 years. Earlier Holocene marine silt, clay and sand up to 20 m deep overly the upper Pleistocene gravels which in turn overly stiffer Plio-Pleistocene marine sediments at unknown depth.

Reclamation materials exposed by ground damage in the portside areas are silty gravelly sands to sandy silty gravels. Similar material (weathered granite) was observed on barges from quarries on Awaji Island, which was being used to reinstate collapsed portside areas, and similar materials are apparent in quarries in the Rokko Mountains.

At the time of the visit of the reconnaissance team no detailed information was available regarding ground profiles and soil conditions associated with the different areas of ground damage.

The most distant occurrence of liquefaction observed by the reconnaissance team was at Osaka-ko (Port Osaka) some 40 km from the epicentre. This is well within the 80 km radius predicted by the formula of Kuribaysahi and Taksuoka, 1975:

$$\text{Log } R = 0.77M - 3.6$$

where

R = distance from epicentre (= 50 km)
M = Richter magnitude (= 7.2)

3.3 Retaining Walls and Slope Protection

Mosaic mortared stone walls are common in the Kobe area with heights up to 5 m and face slopes of between 0.25 and 0.5 horizontal to 1.0 vertical. Generally walls have performed very well (see Figure 3.8) with only isolated failures and obvious distortion. Dry stone walls were loosened and deformed by shaking (see Figure 3.9). No damage was observed to what appeared to be reinforced concrete gravity walls in the urban and residential areas (not influenced by water or submergence).

In several areas a grillage system (see Figure 3.10) as been used as cut slope protection. This reinforced concrete grillage is anchored at the node points with grouted anchor bars. As with other cut slopes, these slopes were undamaged.

3.4 Excavations

There was little evidence of open construction excavations during the visit of the reconnaissance team. Where they were observed they usually involved heavy sheet pile/H-pile support and there was no obvious distress.

3.5 Buried Structures

Some 3 m of subsidence was observed above the underground Daikai Station (see Figure 3.11) which apparently resulted from the failure of a central row of columns. Unfortunately the reconnaissance team was not able to inspect any of the buried railway station structures in Kobe apart from the undamaged Shin-Kobe station, situated on the margin of the mountains (see Figure 3.12).

Conventional tunnels in the Rokko Mountains performed well. Detailed inspection of a road tunnel behind Kobe did not reveal any cracking or distress (see Figure 3.10). However, there are reports that more than 100 cracks were found in the concrete linings of both the Rokko (19,250 m) and Kobe (7,970 m) tunnels of the J R Sanyo Shinkansen Line, some of them several tens of metres long. The cracks are repairable according to J R West Nippon [Asahi, 23 January, 1995].

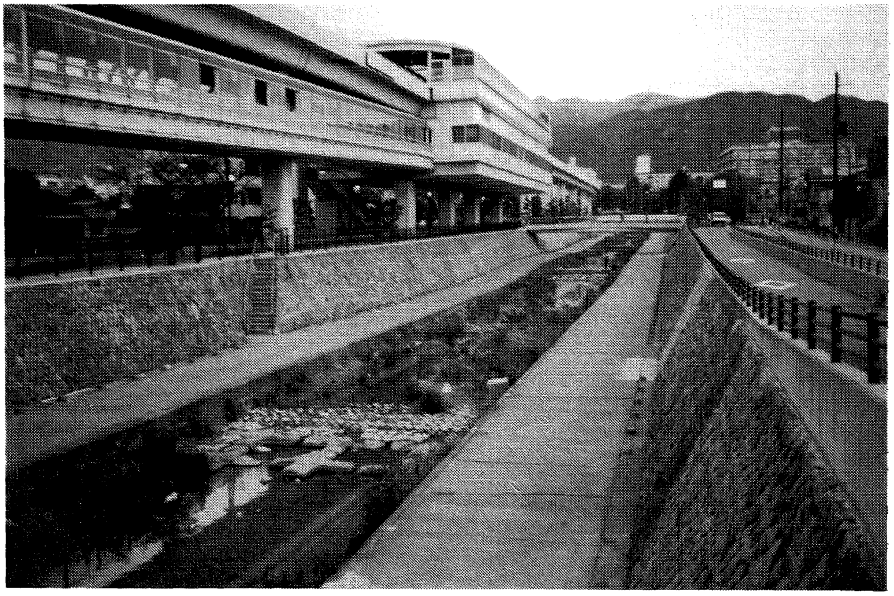


FIGURE 3.8 Sumiyoshi Station (Rokko Light Rail). Retaining Walls Show No Damage



FIGURE 3.9 Sannomiya, Near Shin-Kobe. Damage to Dry Stone Block Wall

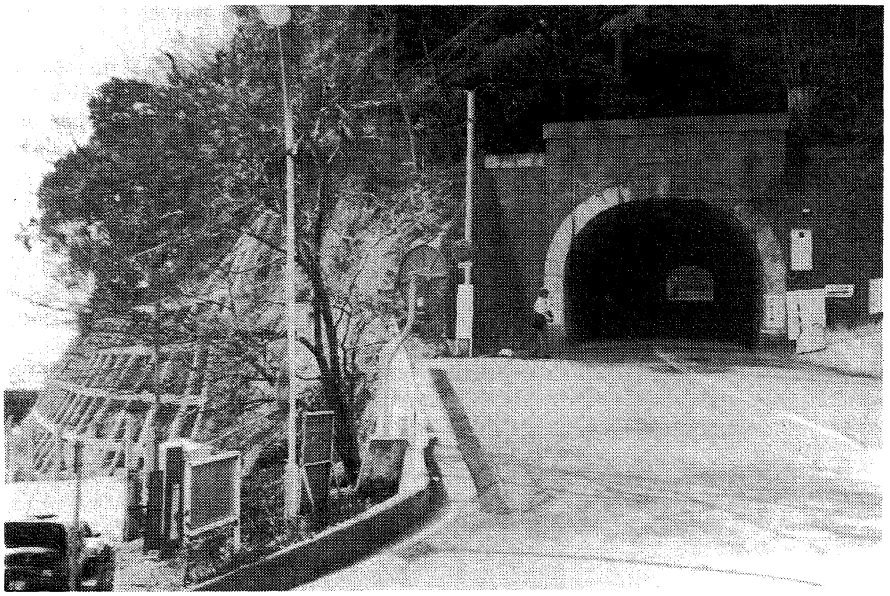


FIGURE 3.10 Futabi Driveway, Near Shin-Kobe. Retaining Wall, Slope Protection and Tunnel Undamaged

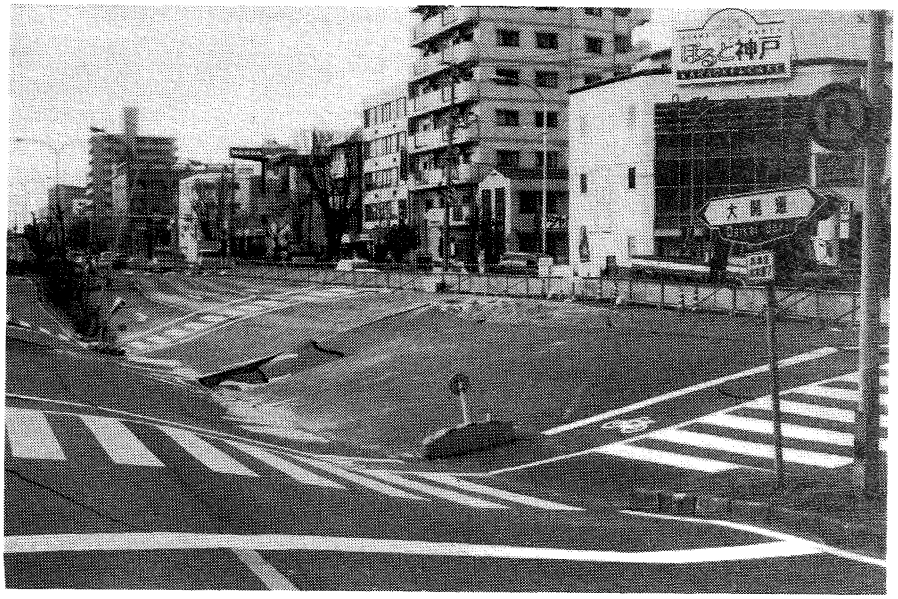


FIGURE 3.11 Kobe, Daikai Station. Street Subsidence Associated with Failure of Columns in Underground Station



FIGURE 3.12 Shin-Kobe Station. No Evidence of Any Earthquake Damage



FIGURE 3.13 Wangan Expressway. Bridge Ramp Settlement. Note Ejected Sand and Pavement Damage Adjacent

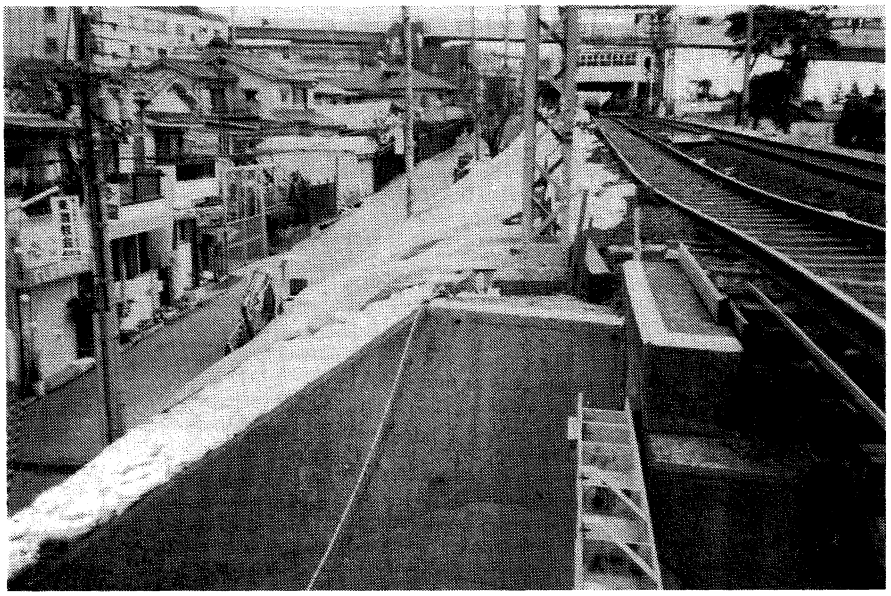


FIGURE 3.14 Hanshin Railway, Near Ogi Station. Embankment Settlement and Associated Rail Deformation

3.6 Embankments

The most common embankments were those associated with rail lines and bridge approaches. Bridge approaches commonly experienced settlement. This was more obvious in the areas of severe liquefaction with settlements of 200 to 500 mm common (see Figure 3.13).

Some rail embankments experienced settlements of up to 500 mm. They were generally very old and the deformation had the appearance of densification of the embankment fill with associated deformation of the rail alignment where it crossed structures (see Figure 3.14).

3.7 Buried Pipelines

There is little factual information available on buried pipelines. In some areas repair crews were active. It is known that in areas of moderate to severe ground damage service connections to buildings were usually damaged.

3.8 Ground Seismic Wave Displacements

In some areas of ground damage there were occasional features that indicated ground compression; for example buckled paving and kerbs. However these were generally of a minor nature. At the time of the visit of the reconnaissance team it was not known if there was associated compression damage to buried pipelines.

A number of the long elevated road structures experienced dropped spans. This indicates extension of the bridge structures associated with the ground motions.

3.9 References

Kuribayashi and Tatsuoka. 1975. Brief Review of Liquefaction during Earthquakes in Japan, *Soils and Foundations*, JSSMFE, 15(4).

4 LIFELINES (EXCEPT BRIDGES)

4.1 Introduction

Lifelines, such as road, rail, telephone, electricity, gas, sewerage and stormwater disposal are essential to the functioning of modern urban and industrialised societies. As in past events, several important lifelines were severely disrupted during this earthquake. The damage, particularly to gas, water and transportation systems had major impacts on emergency and recovery operations, and even on the extent of damage.

Unfortunately, information on the construction and performance of some utility lifelines was been very difficult to obtain at the time of the reconnaissance visit. All supply authorities were working under emergency conditions to restore services and were in no position to allocate resources to collating and distributing detailed information. As a result it was not possible to arrange meetings with any of the lifelines utilities.

The information reported in this section comes from observations, from subsequent contacts with some utility companies, and from a variety of published sources, including information issued by the utility companies that was reported in the media (mainly from The Japan Times and The Daily Yomiuri). The assistance of the City of Kobe's Sewage Works Bureau is particularly acknowledged. Some statistics for the gas and water supplies were also provided by Dr Nojima of the Hiroshima Institute of Technology (personal communication). Information has also been provided by Dr S Kuraoka of the National Research Council of Canada (personal communication), and by Mr N Coad of Transpower New Zealand Ltd (personal communication).

4.2 Transportation - Road and Rail

The main urban areas of Kobe, Ashiya and Nishinomiya lie in a narrow 30 km long east-west strip of land between the Rokko Mountains and Osaka Bay. The developed area is 2-4 km wide, and is the northern leg of a continuous crescent shaped area of urban development around the perimeter of Osaka Bay.

Eight parallel east-west arterial transportation routes pass through this narrow coastal strip:

- Three separate railway lines - the JR (Japan Rail) Tokaido line and the privately owned Hanshin and Hankyu lines.
- The JR Sanyo Shinkansen (bullet train) line along the northern perimeter of the urban area, which is tunnelled under the lower slopes of the Rokko Mountains for much of its length.
- Two multi-lane elevated expressways operated by the Hanshin Expressway Corporation-National Highway Route 3 (commonly referred to as the Hanshin Expressway or the Kobe Route) which runs approximately along the original coast line, and the newer Route 5 (Wangan Route) closer to the present foreshore. The Wangan Route is built entirely over reclaimed land and currently extends from Osaka only as far as Rokko Island in the south-west.
- National Highway Routes 2 and 43, which are multi-lane surface roads with traffic-light controlled level intersections at cross roads. Route 43 mostly follows the line of, and often under, the Hanshin Expressway (Kobe Route). It merges with Route 2 west of Nishi-Nada in the Nada Ward of Kobe. East of Nishi-Nada, Route 2 is 0.5-2 km inland (north) of Route 43.

All eight routes are important commuter arterials connecting Kobe with Osaka and beyond, as well as providing internal commuter transport within Kobe. The expressways and JR Tokaido line are also important land freight routes for the city and port.

In addition, there are several "north-south" arterial rail and expressway feeder routes linking with the rapidly developing, large-scale new towns to the west and north of the Rokko Mountains. About a quarter of the city's population lives in the new towns and consequently, there is heavy commuter traffic between these areas and central Kobe. There are also several "north-south" arterial routes serving cities to the north-east of Nishinomiya.

Despite the multiplicity of arterial routes, the city's road and rail transportation system was brought to a stand-still by the earthquake, with 7 of the 8 main east-west routes and several of the north-south rail and expressway feeder routes made impassable (see Figures 4.1 and 4.2). Most of the damage was due to collapsed spans and pier failures in elevated bridge structures (refer Section 7). Several railway stations were also severely damaged and even the Shinkansen track was seriously damaged over a 10 km length, with collapsed girders at seven or eight locations and about 440 piers damaged.

Sections of the Chugoku Expressway running east-west, inland of the Rokko Mountains were also closed for 10 days following the earthquake. Even after being re-opened, speed limits of 20 kph to 40 kph were imposed on some sections, pending further strengthening work. Several of the "north-south" rail and expressway routes were also severely damaged.

During the first few days following the earthquake, the only main east-west arterial route reported as remaining passable was Route 2. Route 43 was initially blocked by collapsed sections of the Kobe Expressway, but re-opened by the end of January. Parts of Route 2, were subsequently closed in late January. The reason for the closure is not known. There was no obvious damage to the road surface, although in some areas there were a lot of severely damaged low and medium rise buildings along both sides of the road.

Most local surface roads remained, or were quickly made operational, except for narrow side streets and lanes blocked by debris from collapsed houses and commercial buildings.

The closures of the Hanshin and Chugoku Expressways (and the JR Tokaido line) caused traffic chaos on these key routes linking the industrial west with the major urban areas in the Kansai (Osaka) and Tokyo regions. Some 22% of the total road freight in Japan normally passes through this narrow corridor.

Although Route 2 remained passable, it was incapable of carrying the additional traffic demand placed on it, making severe congestion inevitable during peak hours. This resulted in delays of 4 hours and longer during the first two weeks. From 19 January, Route 2 functioned as an emergency route, with one of its three lanes in each direction reserved for emergency vehicles (and motorbikes). As widely reported, failure to exert control earlier significantly hampered the initial delivery of emergency relief.

During the second week following the earthquake, sections of the main road and rail routes south of the Rokko Mountains started to re-open. However, critical sections of the two expressways and three of the four railway lines still remain closed and are not expected to re-open fully until about 6 months after the earthquake (tentative estimate). The JR Sanyo Shinkansen line is expected to re-open in early May. Other than by hydrofoil or direct ferry (see Section 4.3), the commuter travel time between central Kobe and Osaka during peak hours was of the order of 3 to 4 hours. In normal times, commuting between Kobe and Osaka can take as little as 40 minutes on a limited express train.

By early February, buses were running more freely in separate emergency lanes along Routes 2 and 43, and the total travel time between Kobe and Osaka by bus and train had reduced to about 1½ hours (plus up to 2 hours waiting in the bus queue during the evening peak time). The lanes used by private cars and commercial vehicles were, however, still heavily congested.

Commuter services from the new towns north of the Rokko Mountains were less disrupted, with the train service through the Rokko Mountains from Kita-ku to Shin-Kobe station (1 km north of Sannomiya) re-opening about one week after the earthquake. The final section of metropolitan subway line to Nishi-ku re-opened on 16 February, 30 days after the earthquake.

Road and rail transportation in the wider area were also temporarily disrupted. All railway services and highways in the Kansai area were closed a quarter of an hour after the earthquake. Railway services were also suspended on Shikoku Island, which is the fourth main island of Japan. The highway bridge to Kansai International Airport re-opened after 1 hour, but many railway lines in the Osaka area, including the two lines to Kansai International Airport, remained closed for 9 to 10 hours. The Shinkansen line between Nagoya and Kyoto re-opened about 12½ hours after the earthquake. In most instances these closures were precautionary measures to allow preliminary inspections to be completed.

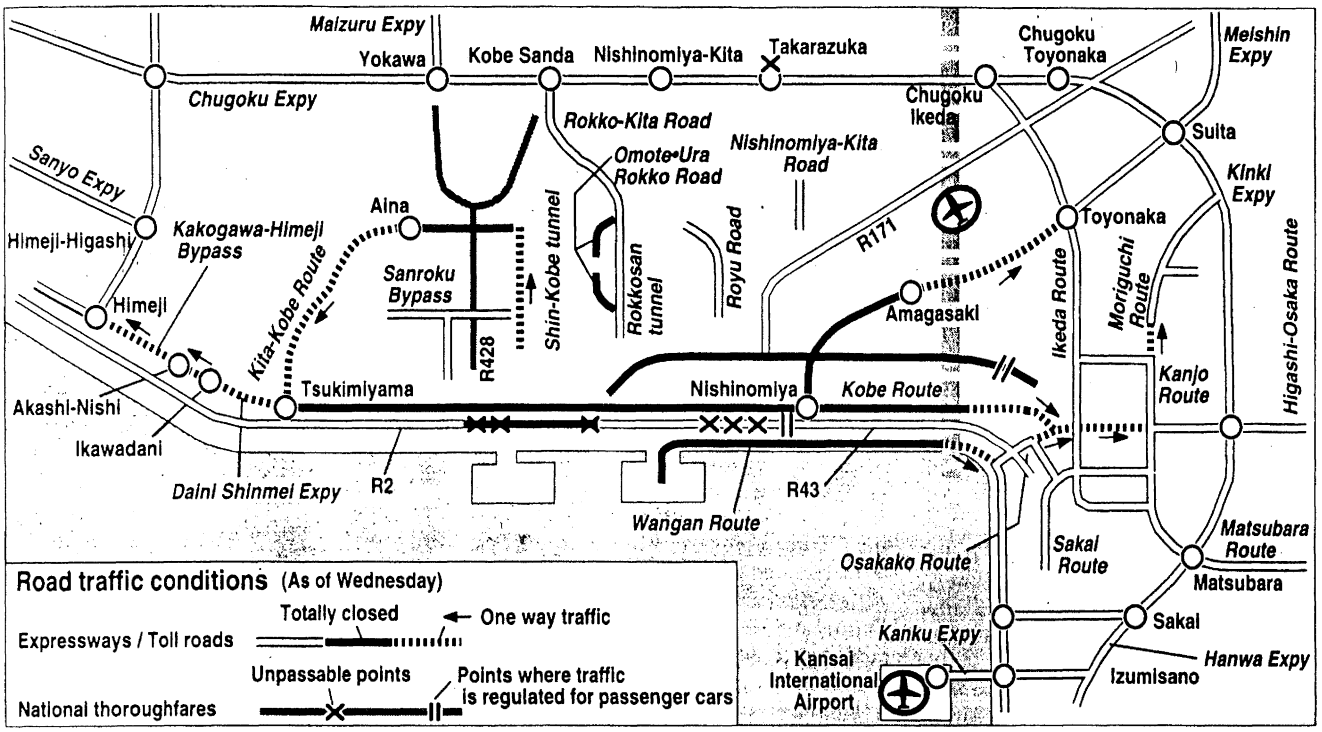


FIGURE 4.1 Road Traffic Conditions as at 18 January 1995

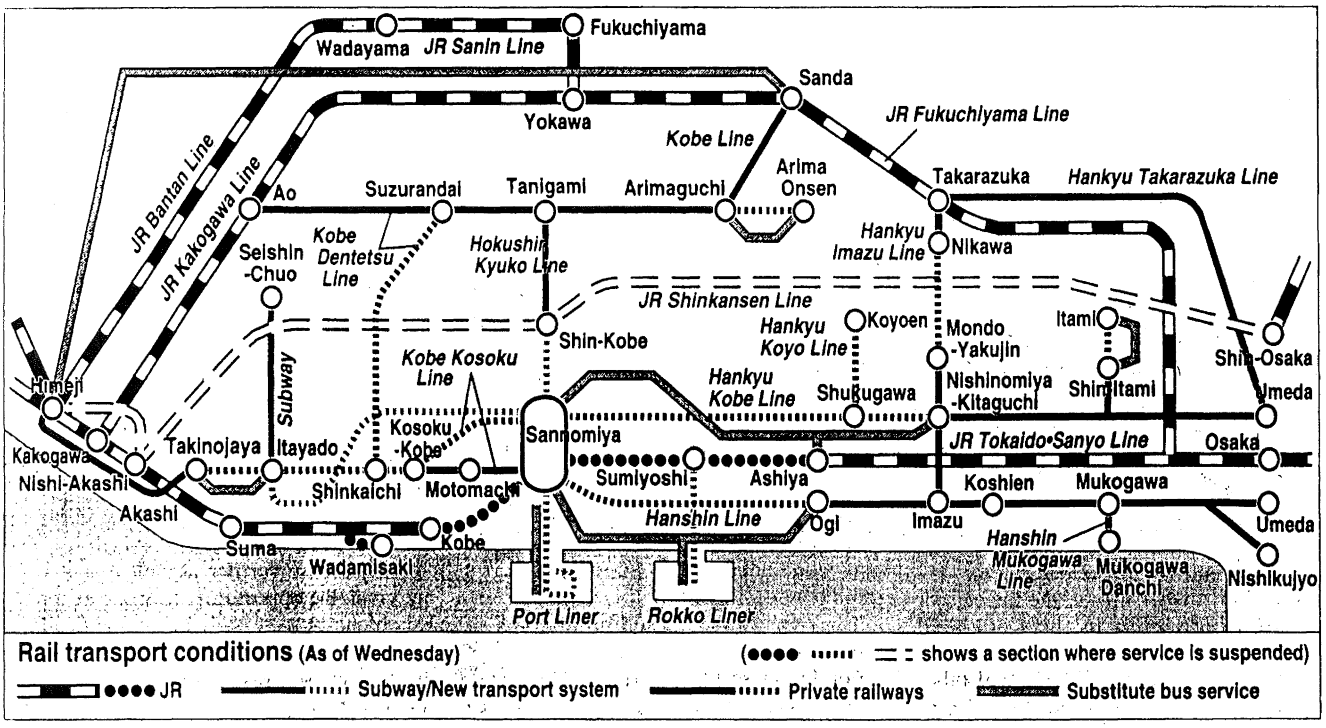


FIGURE 4.2 Rail Transport Conditions as at 18 January 1995

From observation of the repairs being undertaken on railway viaduct piers, priority appears to have been given to re-establishing services, if necessary by initially providing only temporary propping or reinforcing. In some cases, temporary tracks have been layed to detour around badly damaged lines or stations.

4.3 Transportation - Sea and Air

With 190 public wharves, the port of Kobe is the sixth largest port in the world by cargo volume, and the largest container port in Japan. The port is one of the city's proudest achievements and the source of much of its prosperity. Prior to the earthquake, about 39% of the city's gross industrial product was derived from port-related activities, which employed 17% of the city's total work force. The port is also vital to the national economy, handling 12% of Japan's total exports. Figures for 1990 show that the total volume of cargo handled was more than three times the combined total volume handled by all ports in New Zealand.

In about twenty seconds of strong shaking, all the container berths and the large majority of other cargo berths were rendered inoperable. Ten's of kilometres of seawall wharf frontage displaced and tilted to varying degrees, including the entire perimeters of the two large reclaimed islands. Settlement and lateral spreading of the filled ground behind the displaced seawall caissons, toppled or distorted many of the port's large gantry cranes and left the port crippled (see Sections 3.2 and 8.3). Three wharves that did survive with minimal damage are currently being used for unloading emergency supplies.

The direct damage to the port has been compounded by the rail and expressway failures, including serious damage to the link routes connecting the two main port islands to the mainland. Even if some port facilities are reinstated in the near future, overland transhipment of cargo to outside areas will remain restricted until critical sections of the expressways and railway lines re-open.

In the long-term, the damage to the port is likely to be the single largest item of cost to the Kobe economy. Although the national government is likely to contribute significantly to the physical reconstruction, the cost of short and long-term loss of trade is likely to be substantial. Some of the container berths are expected to temporarily resume operations by the end of June, but it is expected to be at least April 1997 before the full capacity of the port is restored. The total cost of restoration has been estimated at NZ $12 billion.

Completion of the second phase of the Port Island development will also be delayed by 6 months because of severe damage to the 14 km long conveyer used to transport rock and sand from the mountains to a jetty in Suma-ku.

Several commuter ferry services have continued to operate across Osaka Bay following the earthquake, with the hydrofoil service providing the fastest means of commuting to and from Osaka during the first few weeks. However, because of the high fare premium and very limited capacity relative to normal rail and expressway traffic volumes, these services have had little impact on the level of road congestion during peak hours.

There are two main civilian airports in the region. Itami Airport (formerly Osaka International Airport) is located just to the east of the heavily damaged area. The airport is now used only for domestic flights. The new Kansai International Airport is located on a purpose reclaimed island on the southern side of Osaka Bay, about 27 km south of the zone of heavy damage in Kobe. In addition, there is a large mixed civilian/military helicopter facility at Yao Airport just to the south of Osaka, about 30 km from the damaged area. Flight schedules at the Domestic and International airports were disrupted on the day of the earthquake, due mainly to road and rail closures (see Section 4.2). However, none of the airport runways or facilities were damaged.

4.4 Water Supply

About 75% of Kobe's potable water is supplied from the lower reaches of the Yodo River via two main pipelines operated by the Hanshin Water Supply Corporation (see Figure 4.3). This is a joint venture arrangement between Kobe, Ashiya, Nishinomiya, and Amagasaki Cities. The remaining 25% of Kobe's supply comes from three local reservoirs - two near the foot of the Rokko Mountains and one up in the mountains. Over half the water from the local reservoirs is allocated to the northern region of the city.

Details of the water distribution network have been obtained only for Kobe City. These indicate that in 1990, there were 3,765 km of water pipes in the city, 87.5% of which was classified as "earthquake resistant" and 12.5% "non-earthquake resistant" (see Table 4.1). Later data indicates that distribution network had increased to 4,000 km of pipe mains by the time of the earthquake, with a total of 119 distribution reservoirs. Emergency valves fitted to 21 of the distribution reservoirs are designed to activate if the ground accelerations exceed a preset level.

Both of the main supply lines were severed close to the Yodo River during the earthquake, leaving more than 1.5 million households without water supply. Six hundred and twenty thousand of these were in Kobe City (outage ratio of 95.3% for Kobe). Twenty three failures were found in one of the main lines (1.25 m diameter, apparently concrete pipe), while a pump station and treatment plant failed on the other line. The edge of another large water treatment works near Nigawa in Nishinomiya City was partly undermined by a large landslide (see Section 3.1), but it is not known to what extent function was lost at this plant.

As far as is known, none of the three local supply reservoirs were significantly damaged. But with less than half of the water from these reservoirs available to the southern part of the city, the downtown area was left with only 10% of its normal supply. Eighteen of the 21 automatic valves fitted to distribution reservoirs triggered during the earthquake, saving 33,000 m^3 of water [Dr Kuraoka, personal communication]. However, water left in the distribution reservoirs was depleted rapidly due to the lack of supply and leakage from the distribution pipes.

Overall damage statistics currently available for Kobe and Nishinomiya show that the average rate of breaks in the distribution network (2.6-3 per km) and service connections (0.06-0.075 per service) were similar in both cities (see Table 4.2). These are average pipe failure rates for both heavily and lightly damaged areas in both cities; for example, including Nishi-ku and Kita-ku in Kobe, where there was comparatively little damage. No details of the pattern of damage to the water distribution network, or of the performance of different pipe materials have been ascertained.

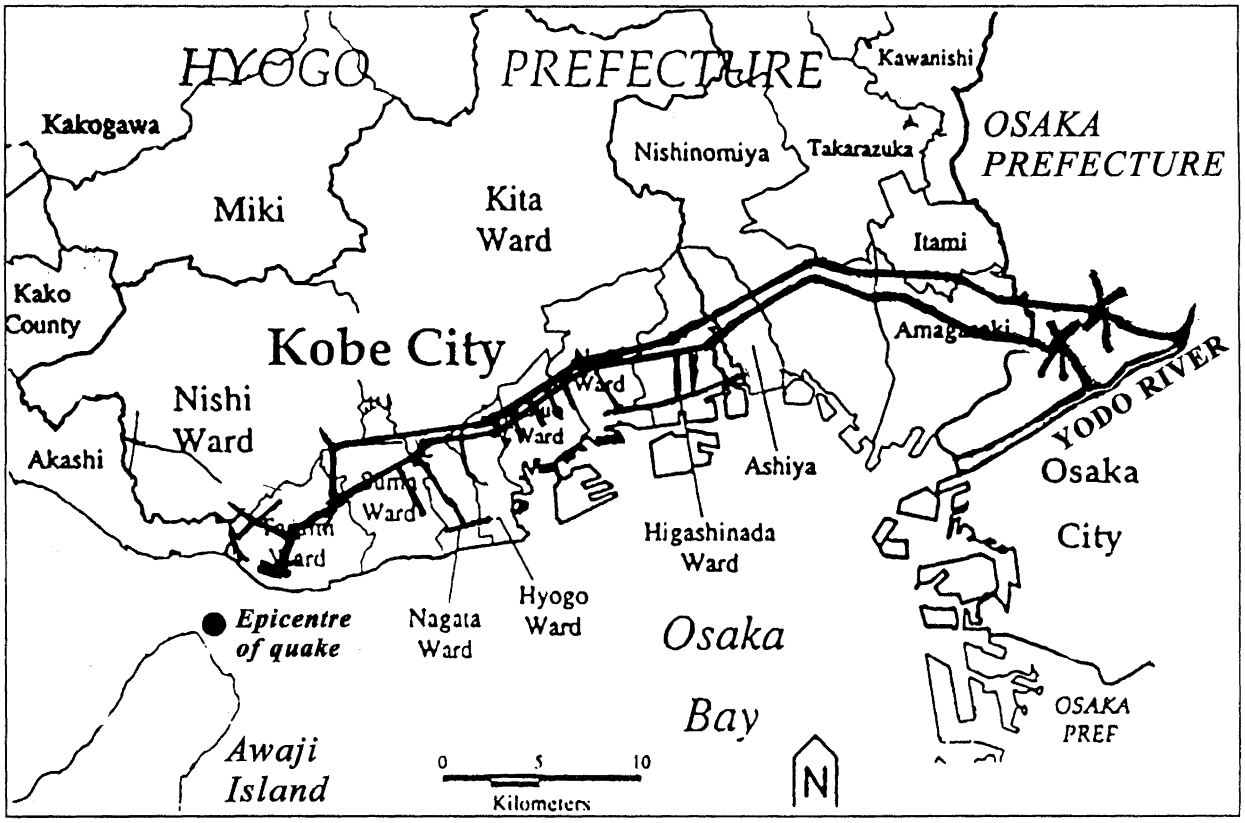


FIGURE 4.3 Water Mains from Yodo River

Table 4.1 Water Distribution Lines - Kobe City

	MATERIAL	JOINTS	LENGTH	
EARTHQUAKE RESISTANT	DCIP	Seismic pivot with lock ring	174 km	(4.6%)
	SP	Welded	98 km	(2.6%)
	DCIP	Mechanical or Tyton	3024 km	(80.3%)
	SUB TOTAL		3296 km	(87.5%)
NON - EARTHQUAKE RESISTANT	CIP	"Old fashioned"	17 km	(0.5%)
	CIP	Mechanical	333 km	(8.8%)
	VP		120 km	(3.2%)
	SUB TOTAL		470 km	(12.5%)

Note: DCIP = Ductile Cast Iron Pipe; SP = Steel Pipe; CIP = Cast Iron Pipe; VP - not known
"Old fashion" - probably spigot & socket with run lead ?
Data are for Kobe City only (including the new towns), as of 1990.

Table 4.2 Pipe Failure Rates for the Kobe and Nishinomiya Water Distribution Networks

	Number Services	Network			Sub-mains	
		Length (km)	Number Breaks	Breaks per km	Number Breaks	Breaks per Service
Kobe	650,000	4,000	10,430	2.61	40,100	0.062
Nishinomiya	147,600	966	2,863	2.96	11,100	0.075

(Details as at 6 March. Dr Kuraoka personal communication)

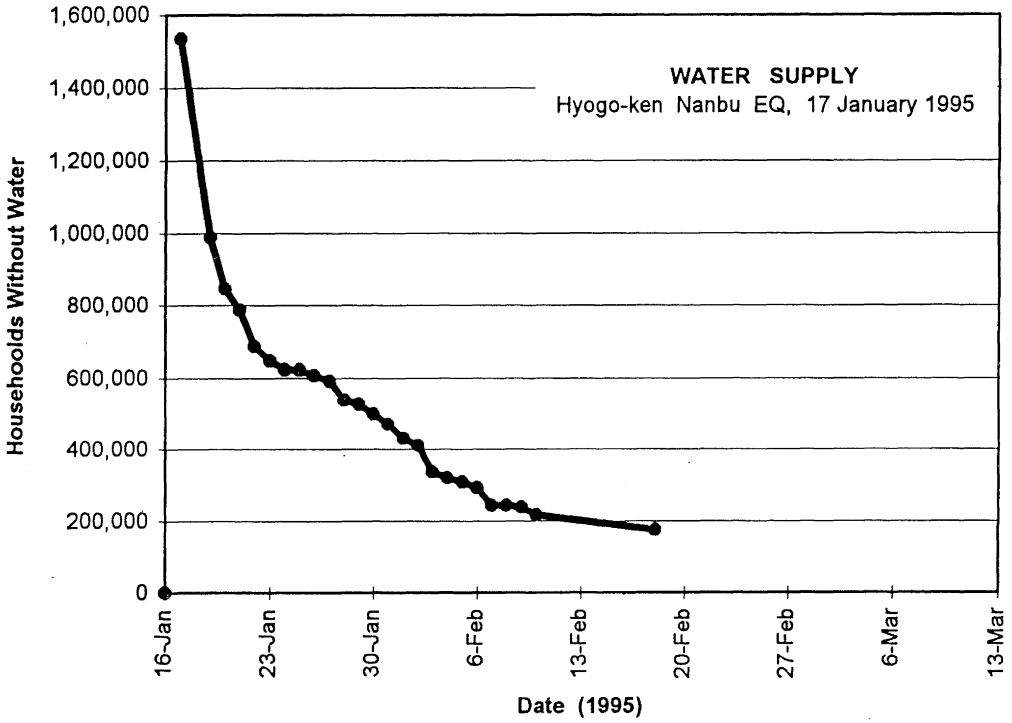


FIGURE 4.4 Restoration of Household Water Supplies

None of the seismic pivot joints failed, despite the fact that these joints were mostly used in poorer ground areas where significant movements were expected (see Table 4.1).

Although the main supply lines from the Yodo River were restored relatively quickly, "full" recovery of household and commercial supplies is likely to have taken at least six weeks. The latest data obtained showed that 175,100 households were still without water as at 18 February, a restoration ratio of about 88% (see Figure 4.4). Most of the households still without supply will have been located in heavily damaged areas.

The loss of water pressure considerably hampered firefighting efforts during the first few days following the earthquake, particularly in western wards such as Nagata-ku. The lack of water (and gas) has also had a longer term effect on hospitals, offices and industries in the area (see Section 12.7). Most hotels located in new highrise buildings sustained minimal damage, but had to remain closed until the beginning of March because of the lack of basic services.

Areas without reticulated tap water were supplied either by water tankers or by temporary street-side water stations located at intervals throughout the urban areas. These ranged from plastic water bottles distributed from the backs of trucks, to temporary self-service stations consisting of a horizontal pipe at waist height, with several taps arranged at each end. Several stations seen during the second week appeared to be drawing water from the street mains, even though no households were being supplied in the area.

For large commercial buildings, water was sometimes pumped directly from tanker trucks into the building's reticulation system (presumably only feasible for buildings with built-in water storage tanks). This was achieved by breaking open the water-main feeding the building.

There has been some criticism of the small number of water trucks available during the first few days after the earthquake. While better planning could have improved the situation, it was inevitable that there would be some delays given the unforeseen scale of disaster and the extent of disruption to the roading system.

4.5 Sewage and Stormwater Disposal

Both sewage and stormwater facilities and pipelines were heavily impacted by the earthquake.

Details of the location, age, capacity and damage to sewage treatment plants in Kobe city are presented in Table 4.3. The following points are noted:

- Three of the seven operating treatment plants suffered partial or full loss of function because of earthquake damage, as did the Tobu sludge incinerating facility. The Tobu facility incinerates the sludge from all treatment plants in Kobe.
- The treatment plants affected were three of the four largest plants in Kobe, providing 63% of the city's installed treatment capacity. The catchment for the three plants covered the entire urban strip from Nagata-ku to Higashinada-ku. The two largest plants (Higashinada and Seibu) were the most affected, losing 100% and 80% of function, respectively.
- All three treatment plants that lost function were located on the coastline, on reclaimed land (see Figure 4.5). The Tobu sludge incinerating plant (100% loss of function) was located on Rokko Island, which is the most recently reclaimed of the two large port islands.
- Two of the plants most affected were less than 10 years old (Seibu completed in April 1985, and Tobu in June 1986).
- Function has now been restored at all except the largest plant (Higashinada), which is currently expected to be restored by 1 May.
- The nature of damage which caused the losses of function has not been ascertained.
- Three of the four operating plants that did not suffer loss of function are located outside the main damage area (see Figure 4.5). The fourth is on Port Island where there was extensive liquefaction and ground settlement, but relatively little structural damage to buildings. The Myodani plant, which was being remodelled, was not damaged. This plant is located inland, 7 to 8 km west of the main damage area.
- In addition to the treatment plants, 6 out of 24 pumping stations lost function (see Table 4.4). These are a mixture of sewage and stormwater pumping stations. Details of the locations of the affected stations and the nature of damage are not available. All stations had been restored prior to 13 March 1995. A further 14 pumping stations sustained damage to some facilities, but the stations remained operable.

Table 4.4 Wastewater and Stormwater Pipe Damage - Kobe City

6 stations	**Pumping function lost**, some facilities damaged Function has now been restored at all stations
14 stations	Some facilities damaged, no loss of function
3 stations	No damage

Note: 12 are wastewater, 9 stormwater and 3 dual purpose pumping stations - other details not known. Status as at 13 March 1995

The effects of the earthquake on treatment plants in Ashiya, Nishinomiya and other cities are not known.

As at March 1994, there was about 3,800 km of sewer pipe under Kobe (aggregate for waste and stormwater). The largest pipe has an internal diameter of 3 m, and is part of the Shin-Ikuta sewer in Chuo-ku. Details of the pipe materials and jointing systems were not available.

Overall failure statistics for the sewage and stormwater pipeline networks are presented in Table 4.5. These data are only for the areas covered during the first stage of investigations. These were concentrated along the narrow coastal strip running from the southern tip of Suma-ku to Higashinada-ku, and have now been completed (see Figure 4.5). This area closely matches the areas where there was major building damage or extensive liquefaction. A second stage of investigation will cover adjacent, less heavily damaged areas, particularly in Nagata-ku and the southern part of Suma-ku, and an area at the western end of Tarumi-ku. The following points are noted:

- Within the first stage investigation area, about 3.5 % of the length of the sewer branch lines and 5% of the sewer truck lines investigated to date have been found to be damaged.
- The number of urgent repairs to sewer branch lines ranges from 2 per km length of pipe investigated in Kita-ku to 25 per km in Tarumi-ku, with typically between 4 and 17 per km in the main damage areas. The average for all wards was 9 urgent repairs per km length of pipe investigated. This corresponds to about 3 repairs per km, based on the total length of pipe in the city (compared with 2.6 breaks per km for the water pipe network). However, the above figure for the sewerage pipes includes only urgent repairs, and does not include any repairs that may be necessary in the second stage investigation areas.
- The data suggest that the stormwater pipes were less damaged, with an average failure rate of only 1.7%, and only 0.77 urgent repairs per km length of pipe investigated. The latter figure may simply be a matter of priority as it is currently the dry season and consequently the demand on the stormwater system is low. However the percentage length of damaged pipe (1.7%) is only half the rate for sewer branch lines. This suggests that the difference may be due to differences in the pipe details or materials.
- The investigation areas reflect the pattern of expected damage. Pipes in remaining urban areas not covered by the first and second stage investigations are unlikely to have been damaged significantly. These areas include much of the older residential area along the foot of the Rokko Mountains, between Hyogo-ku and Higashinada-ku, as well as all of Suma New Town, a large area of Tarumi-ku, and almost all of the Seishin (New Town) areas in Nishi-ku.
- The large scale New Town development in the northern half of Suma-ku was one of the main sources of fill for Port and Rokko Islands and is only about 5 km from the line of major damage and faulting. The lack of damage to underground pipes in this area is probably due to the competence of the ground (area of excavation) and to the fact that all the pipes

Table 4.3 Wastewater Treatment Plants - Kobe City

Name of Plant (Ward, year built)	Capacity (m^3/day)	Damage
Higashinada (Higashinada, 1962) Note 3	225,000	**Main function lost completely** Est. restoration date, 1 May
Port Island (Chuo, 1980)	20,300	Some facilities damaged, no loss of function
Suzurandai (Hyogo, 1968)	43,825	A few facilities damaged, no loss of function
Chubu (Hyogo, 1958), Note 1,3	77,900	**Lost 50% of capacity; restored 27 Jan.** Many facilities damaged
Seibu (Nagata, 1985), Note 1,3	161,500	**Lost 80% of capacity; restored 7 March.** Some facilities damaged
Tarumi (Tarumi, 1974), Note 2,3	133,890	Some facilities damaged, no loss of function
Tamatsu (Nishi, 1981)	75,000	Some facilities damaged, no loss of function
Myodani (Tarumi, 1975)	Being remodelled	No damage
Tobu Slude Incinerating Facility (Rokko Is., Higashinada, 1986)	600 t/day	**Incineration function lost** Restored 14 February 1995

(Information current at 13 March 1995)

Note: 1 Seibu linked to Chubu by pipe, to help treat waste from Chuo-ku.
2 Underground treatment facility.
3 Higashinada, Chubu, Seibu & Tarumi plants are located on the sea front.

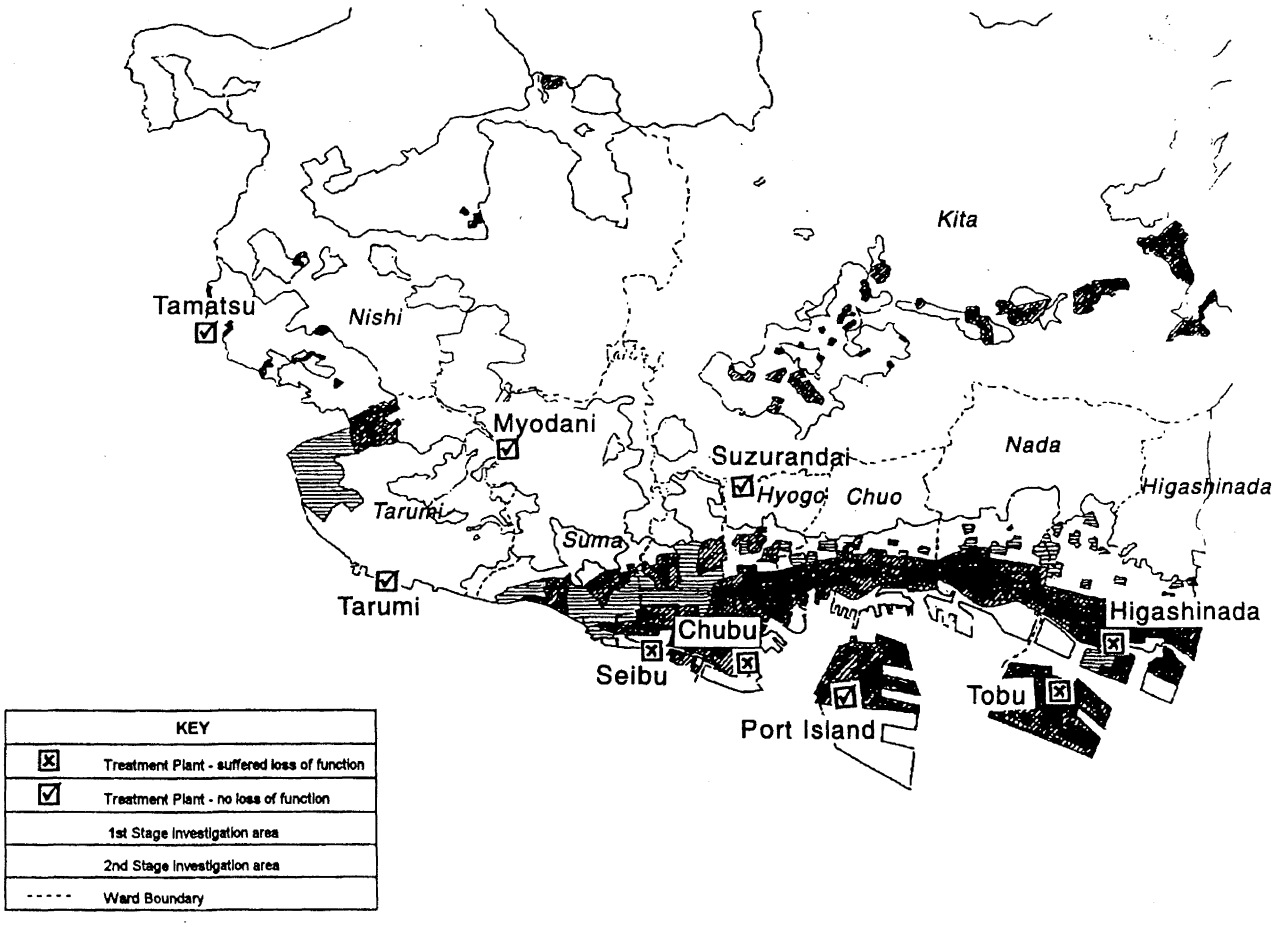


FIGURE 4.5 Location of Sewage Treatment Plants and Areas Investigated for Sewer Branch Pipe Damage (Kobe City)

would be less than about 20 years old. Similar comments apply to the Seishin areas in Nishi-ku which are 7 km-14 km west of the line of major damage. Only a few, isolated patches have been investigated in this ward. Larger areas were investigated in Kita-ku, but these were mainly in a river valley 7 km-10 km north of the zone of heavy damage.

- In reclaimed areas ground movements caused by liquefaction and settlements were observed to have caused extensive damage to sewers. Areas included downtown Kobe, port areas, the port islands and other reclaimed areas. Portable toilets were being provided in such areas.

Restoration of the sewerage system is essential before full use of water can be resumed. During the second week following the earthquake, the sewerage system was being used in many areas and was basically coping with the inflows. The demand on the system was however, well below normal as there was very little water for washing, or for flushing toilets. Damage to the treatment plants also meant that in at least one case, effluent had to be discharged into Osaka Bay without proper secondary treatment.

4.6 Gas Supply

Almost all houses in the urban areas damaged by the earthquake are connected to piped natural gas supplied by the Osaka Gas Company Ltd. Most households use gas for cooking and hot water, and sometimes for heating.

The Osaka Gas Company is a major gas utility, supplying some 5.7 million households in the Kansai area. The gas is imported as LNG through three terminals; Himeji about 50 km west of Sannomiya, and Senboku Terminals I and II on the south coast of Osaka Bay. The gas distribution network consists of 490 km of 600 mm diameter welded steel high pressure pipes, 4,687 km of medium pressure pipelines and 34,724 km of low pressure distribution pipes. There are 37 large diameter spherical gas holders at 19 supply stations distributed throughout the supply area. One of the supply stations with gas holders is located at Fukiai which is very close to Sannomiya, and another, at Nishinomiya. Both stations are equipped with accelerometers mounted on the ground. As indicated in Table 4.6, the medium pressure mains are principally steel (87%) or ductile cast iron (12%), while the low pressure distribution pipes are steel (61.6%), ductile cast iron (29%), cast iron (5.8%), or polyethylene (3.6%). Most of the low pressure metal pipes

Table 4.5 Wastewater and Stormwater Pipe Damage - Kobe City

Sewer Branch Pipes:

Locality	Length Investigated	Length Damaged		Urgent Repairs	
Higashinada	159.4 km	5.68 km	3.6 %	2,751	17.3 / km
Nada	108.7 km	0.77 km	0.7 %	756	7.0 / km
Chuo	110.8 km	10.96 km	9.9 %	1,046	9.4 / km
Hyogo	142.6 km	5.73 km	4.0 %	579	4.1 / km
Nagata	148.8 km	4.07 km	2.7 %	1,146	7.7 / km
Suma	73.9 km	0.72 km	1.0 %	681	9.2 / km
Tarumi	23.4 km	1.00 km	4.3 %	578	24.7 / km
Nishi	6.1 km	0.32 km	5.2 %	20	3.3 / km
Kita	104.3 km	2.16 km	2.1 %	218	2.1 / km
Kobe Total	877.9 km	31.41 km	3.5%	7,775	8.9 / km

Sewer Trunk:

Kobe City	80.0 km	4.27 km	5.3%	N.A.	

Stormwater Pipes:

Kobe City	377.6 km	6.31 km	1.7%	289	0.77 / km

Status as at 9 March 1995 - first stage investigations completed. Investigation of sewer truck incomplete.
Rate data relate only to the length of pipe investigated.
N.A. - data not available.

Table 4.6 Gas Distribution Lines - Osaka Gas Co.

	Steel	Cast Iron	Ductile Iron	Polyethylene
High Pressure (> 1000 kPa)	490 km			
Medium Pressure A (300 ~ 1000 kPa)	1,245 km			
Medium Pressure B (100 ~ 300 kPa)	2,836 km	58 km	547 km	
Low Pressure Main	427 km	2,014 km	10,073 km	
Low Pressure Branch (1 ~ 2.5 kPa)	20,954 km			1,254 km

Note: The above statistics are for the whole of the Kansai area supplied by Osaka Gas, as advised March 1995. They are not necessarily representative of the proportions of different types of gas pipe in Kobe, Ashiya and Nishinomiya.

have screw joints. Two near parallel medium pressure A pipelines run the full length of the Kobe-Nishinomiya urban strip, within about 1 km of the original coastline.

The distribution area is divided into 8 "super blocks" and 55 "middle blocks" that can be separately isolated. Only gas supplies to the 8 "super blocks" can be remotely controlled, but 74% of customers are equipped with micro-chip controlled meters set to automatically cut the gas supply as soon as strong vibration or a significant pressure drop is detected.

Approximately 6 minutes after the earthquake, the Osaka Gas Company established an Emergency Operations Headquarters after finding gas leaks. Six hours later, the supply to one middle block was manually interrupted and by the late evening, the gas supply to four other middle blocks in the heavy damage area had been cut. By 19 January, some 857,000 customers in Kobe, Ashiya, Nishinomiya and parts of the surrounding area were without gas.

Details of damage sustained by the gas facilities and distribution system are as follows:

- LNG Terminals: no damage [0.19g pga (peak ground acceleration) recorded at Himeji and 0.24g at Senboku II].
- Gas holders: no damage [0.85g pga recorded at Fukiai and 0.82g at Nishinomiya].

- High pressure trunklines: no damage (no lines within the heavy damage area).
- Medium pressure A pipelines: relatively minor damage, mostly to dresser couplings.
- Medium pressure B pipelines: relatively minor damage, mostly to dresser couplings.
- Low pressure pipelines: extensive damage, mainly to screw joints.

Restoration of the medium pressure A and B gas lines was completed on 31 January and 8 February, respectively. The low pressure lines were still under repair as at 10 March.

As shown in Figure 4.6, restoration of customer supplies commenced on 27 January and has proceeded more slowly than the recovery of other utilities such as the water supply. The latest information available is that by 10 March, almost two months after the earthquake, gas had been restored to 66% of customers who lost supply, with approximately 15% still to be restored. The residual 19% (an estimated 163,000 customers) relate to supplies where there is serious building damage or collapse, where resumption of supply cannot be anticipated. As shown in Figure 4.7, almost all supplies still to be restored as at 10 March were in the narrow urban strip between Suma-ku (in Kobe) and Nishinomiya. This closely reflects the distribution of heavy damage, as indicated by the percentages of customers for which reconnection is not currently anticipated (see Figure 4.7). It is also noted that the percentage of supplies not expected to be restored exceeds 30% in each of the five wards between Higashinada-ku and Nagata-ku.

Restoration of the gas supply on this scale is a major task and has involved up to 9,800 personnel working for over 2 months. Details of the emergency organisation and recovery procedure are presented in Table 4.7. Restoration of the gas supply has been carried out on an area by area basis, with most of the early reconnections in lightly damaged areas where there was minimal or no damage to the gas distribution lines. Resumption of supply in these areas still requires full testing and verification of the customer connections as per Table 4.7. The total cost of restoration is estimated to be about NZ $3.2 billion, of which $3 billion is for restoration of pipelines.

The loss of gas (and water) supplies throughout the earthquake damaged region has had a major impact on commerce and industry (see Section 12.7), as well as on the living conditions of all residents - displaced and non-displaced. January-February is the coldest part of winter, with day-time temperatures often little above freezing. The lack of gas and water apparently resulted in the closure of all hotels in Kobe, including several in modern high-rise buildings that were virtually undamaged. Some of the larger hotels re-opened on or about 1 March. They are currently almost fully occupied, partly due to the large number of gas and other utility repair staff brought into the

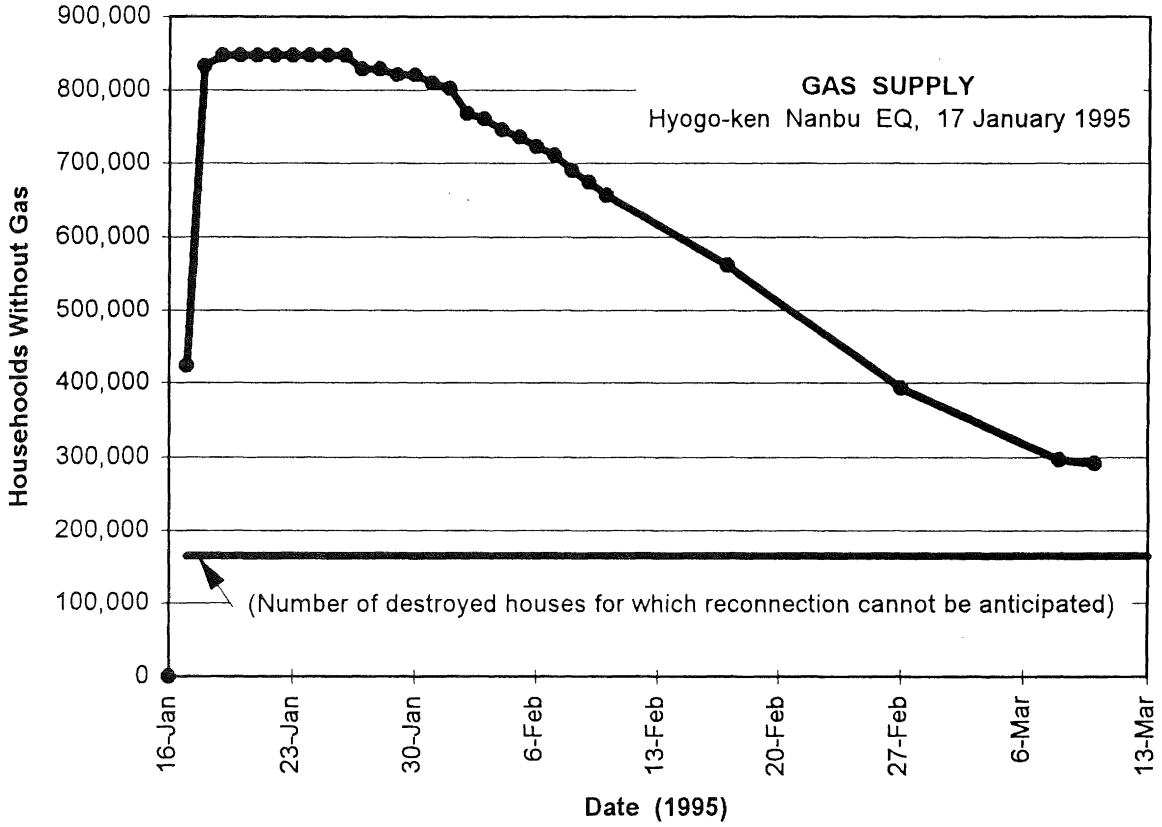


FIGURE 4.6 Restoration of Household Gas Supplies

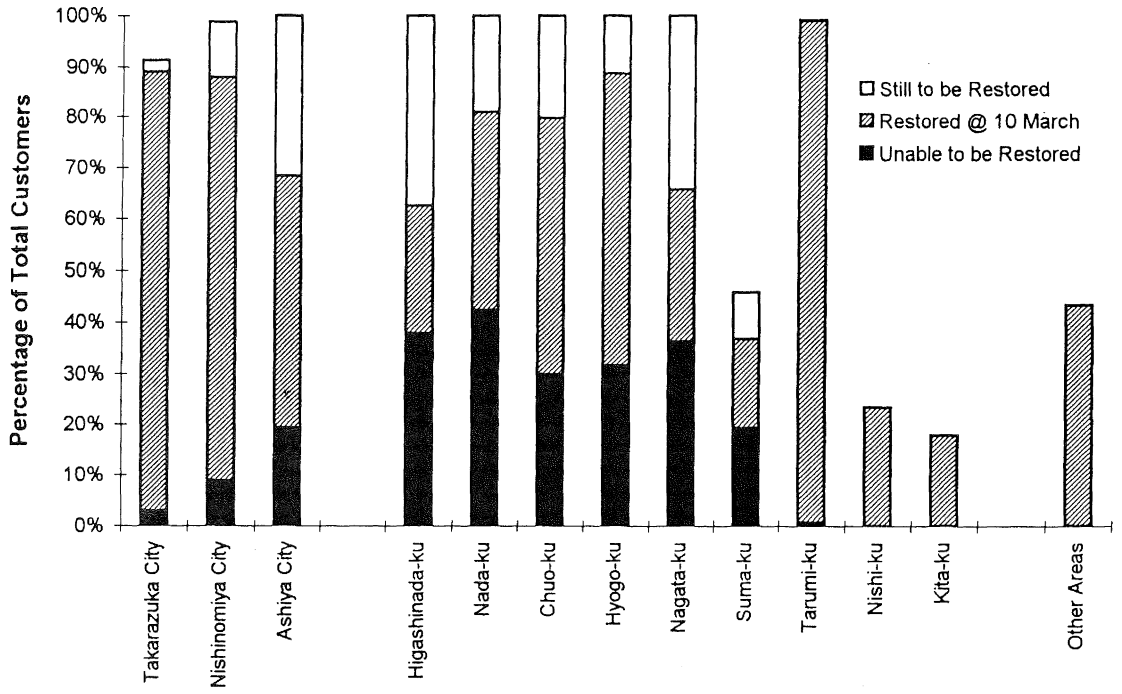


FIGURE 4.7 Restoration of Household Gas Supplies by Area

Table 4.7 Osaka Gas Emergency Operations

(1) Emergency Operation Headquarters

Administration Division	Gas Restoration Division	Public Relations Division
Customer Call Reception	LNG Terminal	
Information Systems	Trunkline	
Procurement Section	Distribution Pipeline	
Personnel Section	House Pipes	
General Affairs	Disconnection / Reconnection	
	Non-Residential Customers	
	Japan Gas Association	

Regional Divisions

(2) Restoration Operation

Basic Approach: Restoration commenced in areas of relatively minor damage. Priority to hospitals and facilities with higher public needs.

Restoration Process: Day 1: Separate block of houses & closing the meter valves.
Days 2 & 3: Leak survey of mains and branches, repair of leaks.
Day 4: Gauge test, repair of house pipes & opening meter valves.

Restoration Staff: 6000 Osaka Gas employees, plus 2,300 from other companies at the end of January, increasing to 3,800 as from 2 March (9,800 total).

Estimated Completion: Base on previous experience, initially estimated 1.5 months. Latest estimate is that all but the worst affected supplies will be restored by 31 March.

area. Hospitals were also significantly affected by the lack of gas and water.

Gas heaters and leaks have been reported as a factor in many of the fires that ignited following the earthquake. The Kobe Fire Department has confirmed some cases of fire erupting from broken gas pipes (The Japan Times, 24 February), but as yet (14 March) neither the Fire Department nor local authorities have issued any official comment as to the cause of fires. No consumers have contacted the gas company regarding gas as a source of fire ignition.

There has been some criticism in the news media of the time that was taken to cut the gas supply (for example, The Japan Times, 24 February), but the company President has responded that the company had to balance the great inconveniences caused by cutting gas to customers unaffected by the disaster and the prevention of secondary disaster, possibly caused by gas leaks. The intelligent gas meters with automatic cut-off valves were another factor and the company considers that these have proven effective in preventing gas related secondary disasters.

4.7 Electricity Supply

Electricity is supplied throughout the Kansai area by the Kansai Electric Power Company. Few details of the generation plant or transmission network were ascertained. Local distribution in the Kobe region is principally by low (100 volt household supply) and high voltage overhead lines supported on spun concrete poles. The high and low voltage lines are generally supported on the same poles, vertically separated. The low voltage lines appear to be insulated cables. Step-down transformers are attached to many of the poles. Underground wiring was only evident in the main central business district areas.

Forty eight of the company's 67 substations in the affected area are reported to have sustained damage in the earthquake, eight of which "blacked out". There was also comparatively minor damage to 25 transmission lines. By the following day, 46 power generating vehicles from 6 power companies had been brought in to supplement the electricity supply in the damage area.

Restoration of the electricity supply was very rapid. Two hours after the earthquake, about 1 million customers were without electricity supply (out of 11.7 million customers supplied by the company). Supply was restored to 50% of these within 8 hours, and to 90% after 3 days. Within 5 days, supplies were restored to virtually all customers requiring electricity.

The rapid restoration of electricity in the first few days following the earthquake has been blamed for starting some fires. There does not appear to have been close liaison with the gas company before electricity supplies were restored. However, the early resumption of supply must also have had considerable economic benefits, especially in the situation that existed with all gas supplies to the area being cut.

The full extent of recovery of the electricity supply was clearly evident during the reconnaissance team's visit. Even in heavily damaged areas, buildings with limited or no damage were seen with their lights on. In some cases, all adjacent buildings were heavily damaged or collapsed.

Damage to the distribution system appeared to be minimal, except for the region of heavy structural damage where numerous examples of the damage sustained by the local distribution system and repairs undertaken were seen. Overhead wires were mainly damaged by fires, collapsing buildings and poles that had tilted (ground failure) or fractured (pole failure), while the most common causes of poles fracturing were collapsing buildings and, less commonly, bending failures of poles with attached high level transformers. Damage was extensive in heavily damaged and burnt areas.

Although many damaged poles had been replaced, in many other cases the repairs undertaken appeared to have been short term measures directed at restoration of the power supply. These included cases where the damaged poles were left standing beside the replacement poles, propping leaning or cracked poles, and in some cases in heavily damaged residential areas, simply removing the damaged poles and allowing the wires to span between the poles on either side.

Officially released cost estimates put the cost of damage to "distribution and transmission lines" at ¥226 billion (NZ $3.9 billion), which is 20% higher than the estimated damage and restoration costs for gas. The reason for the high cost of damage to the electrical supply could not be ascertained. It is also unclear whether the quoted figure includes damage to substations and generation facilities.

4.8 Telecommunications

The telephone local access network in the Kansai area consists of a combination of overhead lines and ducted underground cables. About 20% of cables for local calls and 80% of the toll lines, are underground. All overhead lines appeared to be insulated and were carried on the same poles as the electric power lines.

Although there was inevitable heavy overloading of both local and international circuits during the first days following the earthquake, the telephone system as whole performed remarkably well. Shortly after the earthquake, local call volumes were 50 times normal but pay phones continued to function normally as they operate on separate circuits. These supplied a vital means of communication for survivors. In the days following the earthquake, NTT (Nippon Telegraph and Telephone) installed banks of public phones providing free domestic calls at many relief centres and other locations throughout the damage area. KDD also installed a limited number of telephone booths providing free local and international calls.

About 285,000 lines out of 1.5 million in the affected area were put out of action due to the loss of the electric power supply. Most of these lines would have gone back into service with the resumption of the electricity supply within a few days of the earthquake. A further 193,000 lines were cut by collapsing buildings, fires and damage to utility poles. Extensive damage to the overhead wires could still be seen in the heavy damage areas two weeks after the earthquake. However, many of the houses and commercial premises in these areas were destroyed or were uninhabitable and consequently restoration of service is not required in the near future.

By the end of January, only 10,000 viable lines remained to be reconnected[1]. It was expected that all of these would be restored within the following week.

Both copper wire and fibre-optic underground cables are understood to have survived largely unscathed, even in areas where there was ground disturbance or liquefaction. The use of ducts probably contributed significantly to the excellent performance of these cables.

All underground utility service chambers seen during the reconnaissance appeared to have survived without significant damage. No instances of chamber flotation due to soil liquefaction were seen in the areas visited (in most of these areas however, the water table was probably more than 2 m below ground level).

The performance of toll and local exchanges could not be ascertained. Given the scale of building damage throughout the Kobe - Nishinomiya area, it is likely that some exchange buildings and equipment were seriously damaged. At least one multi-storey NTT building seen had some damage to an exterior concrete wall. A 50 m communications tower on this building was also significantly distorted. However, the comparatively small number of lines out of service indicates that most exchange equipment either continued functioning or was readily bypassed.

During the time that the reconnaissance team was in the area (second week), the cellular phone system appeared to be functioning efficiently, despite cellular call volumes up to four times normal in the damage area (20-25% higher over the whole Kansai area). However the demand for rental phones considerably outstripped the supply, making it very difficult to find a phone available for hire.

Several dedicated emergency communication circuits did not fare so well. This included failure of two dedicated NTT circuits used for automatically transmitting seismograph readings. As a result, there was a delay of over half an hour before the seismograph recording from Kobe was transmitted (manually, by wireless) to the Osaka District Meteorological Observatory. Until then, the peak shaking intensity was thought to be only JMA 5, which is below the threshold for severe damage to buildings. The Hyogo Prefectural Government's ¥8 billion (NZ $125m) emergency satellite communications network also failed, cutting communications with central government and local municipal offices, as well as with the local emergency services. The systems stand-by generator was damaged during the earthquake, and although the back-up batteries worked, they lasted only two and a half hours.

JR West's communication system was another important emergency circuit that failed. Train crews normally communicate by radio with the stations, which are linked to head office by copper wire and fibre optic cables. All cables were severed by the earthquake and as a result, head office was unable to communicate with either the stations or any of the nine trains operating in the area at the time of the earthquake. Station staff took their own initiative and manually shifted signals to red immediately after the earthquake. Head office had attempted unsuccessfully to order the trains to halt.

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[1] That is, the number of lines for which there was an unfulfilled demand. The figures excludes most connections to the approximately 150,000 destroyed or severely damaged buildings, where restoration is impossible and service currently not required. The number of lines reported as having been reconnected at this time varied from about 45,000 to 80,000, in different newspapers.

4.9 Broadcasting and News Media

The extent of damage to local television and radio studios and transmission facilities was not ascertained. In practice, widespread networking and the multiplicity of services and facilities available in the neighbouring regions, means that any damage to local facilities would have limited impact on the availability of television and radio.

Regardless of damage, extensive television and radio coverage was maintained throughout the period following the earthquake. Even the local newspaper Kobe Shimbun, the only newspaper published in the damaged area, managed to go to press with a special edition only 14 hours after the earthquake, despite heavy damage to its offices. The publication facilities of other newspapers were not affected by the earthquake.

The news media, both Japanese and foreign language, have and are continuing to perform an important role in the emergency and recovery processes. While the reconnaissance team was not able to directly evaluate the Japanese language media, it was clear that there was extensive coverage of the earthquake both on television and in newspapers. Even the four English language newspapers provided detailed news, information and commentary on a wide range of issues related to the earthquake. Although they serve only a tiny portion of the population, all four papers provided comprehensive daily information on the locations and telephone contact numbers for relief food and shelter; public baths, showers and toilet facilities; housing and accommodation; counselling services; medical support; legal support; non-allergic food; help for pets; location of friends and relatives; mail services; temporary public telephone and fax services; support for minorities; foreign language information and news services; accommodation and insurance for relief providers; loan sources; immigration and alien registration; schooling; and a wide range of other relief and recovery services. Three of the papers also provided regular information on recovery of utility services and detailed daily coverage on the status of road and rail routes.

Similar comprehensive and probably more detailed information, news and commentary were provided in the Japanese language media. The English language news broadcasts on NHK television were more stilted and "news" orientated, but it was apparent that the main (Japanese) television services were providing detailed information. Similar information and assistance were broadcast on a number of radio stations.

The media also provided vital channels for public feedback and have not shied away from questioning the performance of official efforts during the emergency period, or other perceived bureaucratic impediments.

Both television and the newspapers were important sources of information for the reconnaissance team.

5 GENERAL BEHAVIOUR OF BUILDINGS

5.1 The Main Areas of Damage

The most severe damage to buildings appears to have occurred in a narrow band from about 0.5 km from the sea to the beginning of the lower slopes of Mount Rokko, a width generally of about 1 km (see Figures 2.3 and 5.1). In this area the intensity of surface ground motion appears to have been of the order of MM10. On the seaward side, it appears to have been about MM9, and to have been associated with intensive liquefaction causing significant ground settlement. On the landward side the intensity appears to have reduced rapidly to about MM6 on the lower slopes of Mount Rokko.

The Modified Mercalli (MM) and the Japan Meteorological Agency [JMA] intensity scales are compared in Table 5.1, along with the definitions of the JMA scale. It is of note that initially the Japanese authorities rated the worst hit area of Kobe as 6 on the JMA scale, but later increased that ranking to 7.

5.2 General Performance of Buildings

5.2.1 *Building Classes for Identification of Damage*

Six classes of buildings can be identified in terms of damage.

1 Traditional Japanese houses;

2 Western style houses;

3 Small traditional commercial buildings;

4 Older low rise commercial buildings

5 Older high density apartment buildings

6 New buildings

5.2.2 *Traditional Japanese Houses*

Traditional single family dwellings performed worse than any other class of building in the earthquake. They comprised the overwhelming majority of totally collapsed buildings, and those consumed by the devastating fires.

The traditional house is typically two stories in height of post and beam construction, normally of timber, with heavy tiled roofs bedded on clay, and heavy walls comprising clay and bamboo within timber lathes and an outer layer of stucco. The heavy brittle construction makes the structures particularly sensitive to earthquakes (see Section 9).

The range of damage, from light to severe, was dislodging of roof tiles, collapse of lower stories and total collapse.

The collapse of these buildings was a major cause of the fires.

5.2.3 *Western Style Houses*

Western style houses tend to be built using light frames with light roof and wall claddings and insulation. Most appear to be located on the higher sloping ground and were thus not exposed to the maximum shaking. However, there were a few scattered through the areas of maximum damage, standing alone undamaged amongst the ruins of traditional houses and older commercial buildings. No failures of this type of house were observed by the reconnaissance team.

5.2.4 *Small Traditional Commercial Buildings*

This type of structure is typically 3-4 stories high, narrow in the direction of the street, and extending back from the street 2-3 times its width. The bottom story is generally a shop with the upper floors being used as offices and for residential purposes. Construction is similar to traditional houses except the post and beams are replaced by frames, generally of structural steel, but

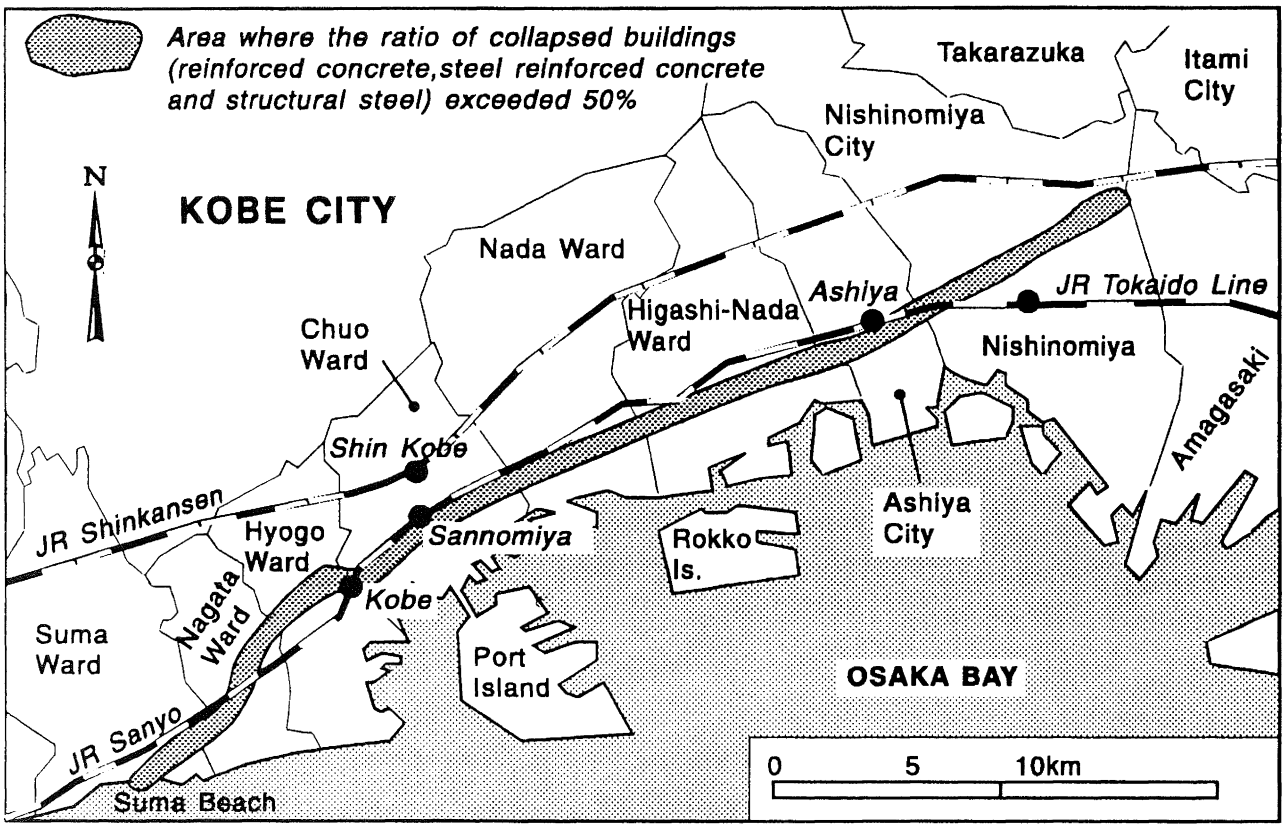


FIGURE 5.1 Area of Intense Damage to Buildings ("Collapse" Means Either Total Collapse or Loss of One or More Storeys Due to Damaged Columns and/or Walls)

Table 5.1 Comparison of the Japan Meteorological Agency (JMA) and the Modified Mercalli (MM) Intensity Scales

JMA	0	1	2		3		4	5		6	7	
MM	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII

The Japan Meteorological Agency Intensity Scale (JMA) Summary

0 = No feeling: registered by seismographs only
1 = Slight: felt by some individuals at rest
2 = Weak: felt by most people
3 = Rather strong: shaking of houses and buildings
4 = Strong: strong shaking of houses and buildings and unstable objects fall over
5 = Very strong: cracking of plaster walls, heavy objects such as tombstones and stone lanterns fall over, and damage to masonry chimneys and mud-plastered warehouses
6 = Disastrous: collapse of up to 30% of wooden houses, numerous landslides and embankment failures, fissures on flat ground
7 = Very disastrous: collapse of more than 30% of wooden houses, falling of objects, wavy deformation observed in horizon

Source of the Approximate Comparison of Earthquake Intensity Scales:

M D Trifunac and A G Brady. "The correlation of seismic intensity scales with the peaks of recorded strong ground motion", Bulletin of Seismological Society of America, Vol. 65, 1976.

also sometimes of timber or reinforced concrete. Structural engineering input, if any, was probably very small.

These buildings also performed poorly. The range of damage, from light to severe, was tile damage, lower storey cracking of brittle cladding materials, bottom storey collapse and total collapse. A major contributing factor to the damage was the failure of steel connections, both bolted and welded, and the lack of strength and stiffness of the seismic-resisting system.

These buildings also contributed to the fire damage.

5.2.5 *Older Low Rise Commercial Buildings*

These are typically 3-5 stories high with their width and depth similar or greater than their height. They are generally of frame construction, usually reinforced concrete, and were designed to the codes current at the time of construction. They often have relatively heavy concrete cladding, often precast.

The range of damage, from light to severe, was cracking near the base and bottom storey collapse.

5.2.6 *Older Medium and High Rise Commercial and Apartment Buildings*

These buildings, of which there are large numbers, are typically 10-20 stories high, and often long in length, and narrow in cross-section. Construction is similar to the previously mentioned low rise commercial buildings, except it is believed that the lower stories are often of steel reinforced concrete (structural steel embedded in reinforced concrete).

The performance of these buildings was similar to the commercial buildings except that there was more damage at higher levels with a significant number of collapses at intermediate - typically third to fifth - storey levels.

Some of these buildings also showed signs of significant rocking about their narrow axis on the soft soils, with some residual tilting evident.

5.2.7 *Modern Buildings*

Most modern (post-1981) buildings performed well. The good performance of these modern buildings is attributed to the introduction in Japan in 1981 of a new earthquake design standard which incorporated modern theories of earthquake resistant design.

Kobe has a significant number of new high rise glass clad buildings, and these appear to have suffered little if any building damage in general. Modern smaller commercial and apartment buildings also performed well, with only a few showing any signs of damage, generally in the lower storey, or where interconnection of structurally separated structures occurred.

However, not all modern (post-1981) buildings performed well. According to the newspaper Kyoto Shinbun, 10 February 1995, the Ohbayashi Corporation investigated 233 buildings and the percentages of them with significant damage were as follows:

Post-1981 buildings: 6% - danger, keep out
11% - caution, restricted use

Pre-1971 buildings: 36% - danger, keep out

Also the Taisei Corporation investigated 96 buildings and the percentages of them with significant damage were as follows:

Post-1981 buildings: 11% - caution, restricted use

Pre-1971 buildings: 42% - danger, keep out

A group of modern multistorey apartment buildings where some exposed high strength structural steel box columns suffered brittle failures, although the buildings remained in use. One of the contributing factors to these brittle failures may have the cold outside temperatures at the time of the earthquake (see the further details of these buildings in Section 6.5.7).

Where access was available, only limited internal architecture damage also appeared to have occurred.

5.3 **AIJ Statistics of Building Damage**

5.3.1 *General*

According to a preliminary report issued by the AIJ [Architectural Institute of Japan, 1995] the damage to residential housing, including apartment buildings, was as shown in Table 5.2.

The preliminary report also gives statistics on damage to buildings. These statistics were collected by a research group of the Kinki branch of the AIJ during an intensive survey of damage conducted in late January.

The research group of the AIJ Kinki Branch found that the area where the percentage of collapsed buildings (reinforced concrete, steel reinforced concrete or structural steel) exceeded 50% spread over 25 km from Suma beach to Nishinomiya city, forming a narrow band as shown in Figure 5.1. The width of the band is about 200 to 300 m. In the Nagata and Hyogo Wards, on the west side of Kobe city, about 200 buildings collapsed or were seriously damaged. The band of collapsed

Table 5.2 Number of Residential Houses Including Apartment Buildings Damaged in Prefectures

Prefecture	Date of Statistic 1995	Number of Total Collapses	Number of Half Collapses	Number Totally Burnt	Number Half Burnt
Hyogo	2 or 7 February	81,260	62,071	7,119	337
Osaka	8 February	708	3,515	-	-
Kyoto	3 February	3	3	-	-
Others	20 January	1	24	-	-

timber houses enveloped the above band with width of about 2 km, according to the Earthquake Research Institute, Tokyo University, as reported by the Asahi Shinbun Yukan (Asahi Evening News, 2 February 1995).

The damage statistics collected by the research group of the AIJ Kinki Branch for the different Wards of the damage zone are listed in Tables 5.3 to 5.6. The wards and cities referred to in the tables are shown in Figure 5.1

The damage categories are defined as follows:

- Collapse - total collapse or loss of one or more storeys due to damaged columns and/or walls
- Severe damage - damage severe enough for the possibility of collapse during aftershocks
- Medium damage - there are residual storey drifts.
- Small damage - no residual storey drifts but small cracks can be found in structural members and non-structural damage can be found.

The statistics in the tables is discussed in more detail below, which is a close translation of the AIJ report (Architectural Institute of Japan, 1995).

5.3.2 *Nishinomiya City, Amagasaki City, Itami City, and Takarazuka City*

Table 5.3 shows the statistical data.

(a) Reinforced Concrete and Steel Reinforced Concrete Buildings

Thirty reinforced concrete and steel reinforced concrete buildings collapsed due to column failure in the first storey of the 34 buildings that collapsed. Twenty-three of them had shopping stores in the first storey and apartments above, or office buildings, and had small wall areas at the ground floor level. Seven (the rest) consisted of three school buildings, two apartment buildings, one office building and one factory. Most of them were constructed from 1955 to the 1960s. In this area, there are a large number of four to five storeys apartment buildings with reinforced concrete wall system and they had slight or no damage.

(b) Structural Steel Buildings

Most of the structural steel buildings that were damaged or collapsed have three storeys or less.

5.3.3 *Ashiya City*

Table 5.4 shows the statistical data.

(a) Reinforced Concrete and Steel Reinforced Concrete Buildings

Most of the buildings which collapsed due to column failure in the first storey were designed according to the old design code. Those buildings had small wall areas at the ground floor levels. Most of the buildings that had been designed according to the current code (post-1981) survived although many shear cracks were found in the structural walls at various floor levels and also in the columns in the lower storey levels. However, a building near the JR Ashiya station, which is supposed to be designed according to the current code, because the column hoop spacing is about 100 mm, collapsed due to column failure at the ground level.

(b) Structural Steel Buildings

Most of the collapsed buildings are old apartment or office buildings with three storeys or less, which were constructed using light gauge steel. These buildings collapsed due to column failure mostly in the first storey while some of them collapsed in the second storey. In a group of apartment buildings with five to seven storeys, the cladding (precast concrete panels) fell off due to lack of stiffness of the frames since the sections of the beam and column members are small and no braces exist. With respect to the high-rise residential buildings in the Ashiyahama Seaside Town, fracture of steel box columns could be observed in the base metal part and in welded connections. The thickness of the steel walls of the columns at the base is 55 mm.

5.3.4 *Higashinada Ward and Nada Ward in Kobe City*

Statistical data for these areas is not given in this part of the AIJ report. However, most of the buildings which were constructed after the introduction of the current code in 1981 have little damage but some of them with inadequate structural details were damaged.

5.3.5 *Chuo Ward in Kobe City*

Table 5.5 shows the statistical data.

(a) Reinforced Concrete and Steel Reinforced Concrete Buildings

Most of the buildings which collapsed were designed according to the old (pre-1981) building code. The buildings with small wall areas at the ground floor level collapsed in the first storey. Some medium-rise buildings collapsed in an intermediate storey. Most of them are department or office buildings constructed using steel reinforced concrete.

(b) Structural Steel Buildings

Most of the collapsed buildings are old apartment or shopping store buildings with several storeys which were constructed using light gauge steel. Some of medium to high-rise buildings were also severely damaged.

5.3.6 *Hyogo Ward, Nagata Ward, and Suma Ward in Kobe City*

Table 5.6 shows the statistical data. Most of the collapsed buildings were constructed between 1945 to 1970. Some reinforced concrete buildings overturned apparently due to the columns parting from the foundations. Some buildings collapsed at an intermediate storey due to the discontinuity of the structural system; for example, the Kobe city Nishi municipal hospital building.

5.4 **Other Statistics of Building Damage**

Miki Shoji (a commercial firm) investigated the damage to 116 office buildings which are located in the Chuo Ward (JR Sannomiya Station area). The total floor area of each building is 1,980 m^2 or more. It was found that 28 buildings (24%) were to be demolished, about 90% of which were built more than 25 years ago. Twenty-five buildings (22%) had damage which required a repair time of more than one month. The number of buildings which could be restored within one month was 63 (54%). These findings were reported by the Asahi-shinbun (Asahi Daily News), 28 January 1995.

Table 5.3 Number of Damaged Buildings in Nishinomiya City, Amagasaki City, Itami City and Takarazuka City

Structure	Damage Category		
	Collapse or Severe	Medium	Small
Reinforced Concrete	34	15	50
Steel Reinforced Concrete	0	2	6
Structural Steel	22	23	21

Note: For definition of the damage category see the text.

Table 5.4 Number of Damaged Buildings in Ashiya City

Structure	Damage Category		
	Collapse or Severe	Medium	Small or Minor
Reinforced Concrete and Steel Reinforced Concrete	22	21	45
Structural Steel	38	27	37

Note: For definition of the damage category see the text.

Table 5.5 Number of Damaged Buildings in Chuoh Ward (in Kobe City)

Structure	Damage Category				
	Collapse	Severe	Medium	Small	Minor
Reinforced Concrete and Steel Reinforced Concrete	77	124	172	332	921
Structural Steel	55	93	177	293	403

Note: For definition of the damage category see the text.

Table 5.6 Number of Damaged Buildings in Hyogo Ward, Nagata Ward and a Part of Suma Ward (in Kobe City)

Structure	Damage Category		
	Collapse or Severe	Medium	Small or Minor
Reinforced Concrete and Steel Reinforced Concrete	146	43	71
Structural Steel	78	35	23

Note: The buildings are mainly houses and apartment buildings. Some are shopping stores and factories of medium or small size. Most were built during the period 1945 to 1965. For definition of damage categories see the text.

The disaster relief headquarters of Ashiya-city investigated 16,775 buildings and houses, and found that 2,543 of them (15.2%) had totally collapsed and 1,519 of them (9.1%) had half collapsed. 3,182 buildings and houses (18.9%) were partly damaged. Only 9,531 buildings and houses (56.8%) had no damage. They define the total collapse and the half collapse as the case when 70% of the total floor area is lost and when 20 to 70% of the total floor area is lost, respectively. No damage is defined as the case when no or minor damage is found. These findings were reported by Asahi-shinbun (Asahi Daily News), 29 January 1995.

5.5 References

Architectural Institute of Japan. 1995. *Investigation of the Disaster Due to the Hyogo-ken Nanbu Earthquake - The Urgent Meeting Report.*

6 ENGINEERED BUILDING STRUCTURES

6.1 Types of Building Construction in Japan

A paper by Aoyama, 1981 gives the following description of building construction in Japan:

"Figure 6.1 shows types of building construction and the commonly employed number of storeys for each type of construction. Traditionally, Japanese houses have been built in timber, and they are one or two storeys high. They are still very common. Masonry houses are scarce. Although provisions exist in Japan for the construction of reinforced concrete block masonry apartment houses up to three storeys high, this construction is not shown in Figure 6.1.

The more common type of construction for apartment houses uses reinforced concrete "wall" structures. For these the building code allows a much simpler method of design that for ordinary reinforced concrete frame construction. This wall construction is in the majority of low-rise apartment houses, ranging from three to five storeys. However, it may be used for lower or higher buildings up to eight storeys.

Ordinary reinforced concrete frame structures, with or without shear walls, represent the most common type of construction for various types of buildings, such as shops, offices, schools, and hospitals, ranging from three to seven storeys. It may also be used for lower or higher buildings. However, in ordinary cases Building Officials recommend the use of SRC construction for buildings taller than seven storeys.

SRC, an abbreviation for steel-reinforced-concrete, is a composite construction consisting of a structural steel frame encased in reinforced concrete. It has a long history in Japan, probably evolving from as early as 1920, when buildings were constructed with exterior steel frames encased in brick masonry and interior steel frames encased in concrete. SRC is generally believed to be more ductile and hence more earthquake-resistant than ordinary reinforced concrete. For buildings beyond seven storeys and up to about twenty storeys, or even higher in some cases, SRC is the most common type of construction in Japan.

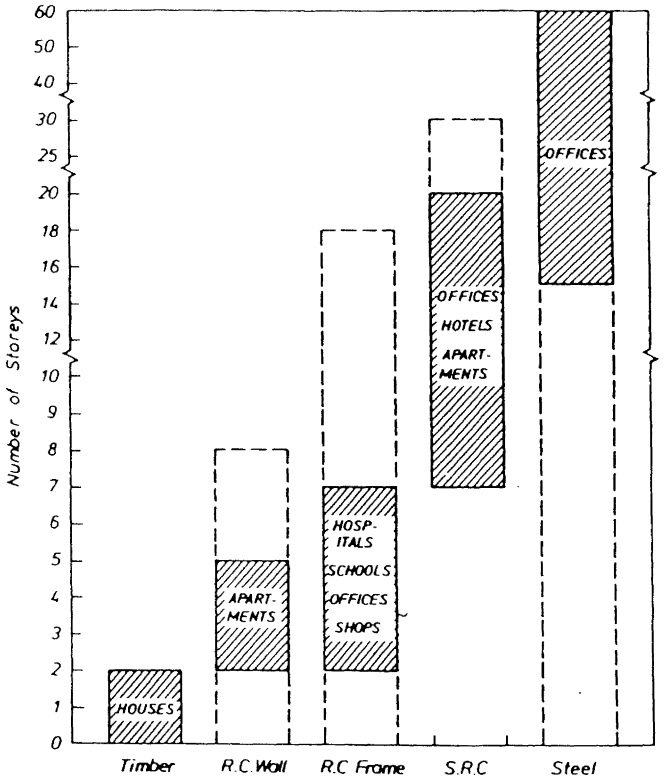


FIGURE 6.1 Types of Building Construction in Japan [Aoyama, 1981]

A steel structure is used for most high-rise construction in Japan ranging from about fifteen storeys up to the tallest buildings. Recently steel structures have become more popular in all ranges of buildings, mainly due to the rapid erection on the site. Some of these are frame structures but for lateral load resistance many of them rely, at least partly, on bracing".

6.2 Building Codes

6.2.1 *The Early Codes*

The paper by Aoyama, 1981 also traces the history of development of seismic design codes for buildings in Japan as follows:

"The history of seismic design in Japanese building code started in 1924 when the Urban Building Law was revised, as a consequence of the disaster of the Great Kanto earthquake of 1923. This adopted a set of structural provisions including a seismic coefficient of 0.1. After World War II the Building Standard Law replaced the Urban Building Law with much more elaborate provisions for various aspects of structural design. The standard value of seismic coefficient was raised to 0.2. The essential feature of seismic design was, however, unchanged. Allowable stress design based on elastic theory was used and this increase in seismic loading was accompanied by comparable increase in the allowable stresses for various materials.

Both the Urban Building Law and the Building Standard Law specified only loadings and allowable stresses, and

certain minimum requirements for the detailing of members. Details of structural design, such as methods of structural analysis and the proportioning of members, are specified in the Structural Standards issued by the Architectural Institute of Japan (AIJ). These Standards, prepared separately for each structural material, have served as the supplements to the Law. They have been revised more frequently to adapt new knowledge and to provide for new materials as they developed.

A particularly important event regarding seismic design was the 1968 Tokachi-oki earthquake which caused significant damage to modern buildings which were designed in accordance with building regulations then in force. Various actions were undertaken as a consequence of this event. A partial revision of the Building Standard Law, a large scale revision of AIJ Standards incorporating ultimate strength design in shear of reinforced concrete, the establishment of review procedure of existing buildings for seismic safety, were some of the changes. Also a five-year-project of the Ministry of Construction, conducted from 1972 to 1977, aimed at establishing a new and rational seismic design method. This was released in 1977, as a proposal for a new aseismic design method for buildings.

The following year, another important event took place, the 1978 Miyagiken-oki earthquake. Damage was as severe as in the 1968 Tokachi-oki earthquake. It also demonstrated the more complicated characteristics of urban disaster in the city of Sendai with more than 600,000 population. This earthquake gave a great momentum in the implementation of the Ministry of Construction proposal. In July 1980, a revision of the Enforcement Order of the Building Standard Law was released. It was also announced that this Order, together with the supplementary documents, would be enforced from the first of June, 1981".

6.2.2 *The 1981 Code*

The paper by Aoyma, 1981 describes the current Japanese seismic design code as follows:

"Therefore in 1981 the building code of Japan experienced its largest revision since its first version in 1924. The central feature of the revision is the introduction of a two-phase design for earthquakes. The traditional seismic design is retained, with some alteration, as the first phase design for earthquakes. This aims at the safety and serviceability of buildings during medium earthquake motions. The new, second phase design for earthquakes is being added to give safety against severe ground shaking.

Figure 6.2 shows the general flow of structural design stipulated by the 1981 revised Enforcement Order of the Building Standard Law. All buildings are first divided into four groups, mainly based on their heights. They are shown in the boxes marked as (1), (2), (3) and (4).

For buildings taller than 60 m (box 4) provisions of the Building Standard Law do not apply directly. These high-rise buildings are to be designed by the "special study", usually incorporating time-history non-linear response analyses. The design is then subjected to the technical review by the Highrise Building Structure Review Committee of the Building Centre of Japan. Upon its recommendation, a special approval of the structural design is issued by the Ministry of Construction.

For buildings, not exceeding 60 m in height, the basic intent of the general flow in Figure 6.2 is to make a two-phase design. This means that an additional design phase, hereafter called the second phase design for earthquakes, follows the conventional structural design, including the traditional seismic design, hereafter called the first phase design for earthquakes. The first phase design is for strong earthquakes which can occur several times during the life time of the building. The second phase design is intended mainly for severe or extraordinary earthquakes which could occur once in the life time of the building.

The application of the two-phase design is shown in Figure 6.2 for three different groups of buildings up to 60 m in height, in boxes (1), (2) and (3). For all of the buildings, conventional structural design is carried out first, box (5), including the first phase seismic design. This is an allowable stress design for permanent and temporary loadings, also taking ultimate strength into account. Major changes in the code relevant to this phase of the design is the method of seismic force evaluation.

Box (1) is the buildings of the most prevalent construction type in Japan. They include low-rise buildings of reinforced concrete with a generous amount of shear walls. For these buildings there is ample experience in Japan in seismic design and also evidence of seismic behaviour. The first phase design, basically unchanged from the rules of the former building code, should be successful in providing sufficient seismic resistance for these buildings to withstand severe earthquakes. Hence the second phase design need not be applied to these buildings.

For buildings of boxes (2) and (3), the second phase design follows. The most important step in the second phase design is the evaluation of ultimate capacity for lateral load in box (9). However, other considerations, listed in boxes (6), (7) and (8), are also included in the second phase design.

The evaluation of storey drift, box (6), is intended to eliminate flexible structures which might experience excessively large lateral deflection under the seismic loading. In the case of ordinary reinforced concrete buildings this check will never be critical. However, the results of calculations are needed later for items given in box (7) or (9).

For buildings whose height is up to 31 m, box (2), there is a choice of flow into boxes (7) and (8), or into box (9). Box (7) requires a check for the rigidity factor and the eccentricity. The rigidity factor refers to the vertical distribution of lateral stiffness. The purpose of this check is to eliminate building with one or more flexible storeys among the other storeys, such as the soft first storey. Checking for eccentricity is necessary to provide protection against excessive torsional deformation. These checks are followed by satisfying a set of additional minimum requirements specified by the Ministry of Construction, Box (8), to ensure certain levels of strength and ductility. Thus, the option of boxes (7) and (8) is intended to allow exemption from the evaluation of the ultimate capacity for lateral load for buildings up to 31 m in height, if they have a reasonably regular structural system conforming with the additional minimum requirements.

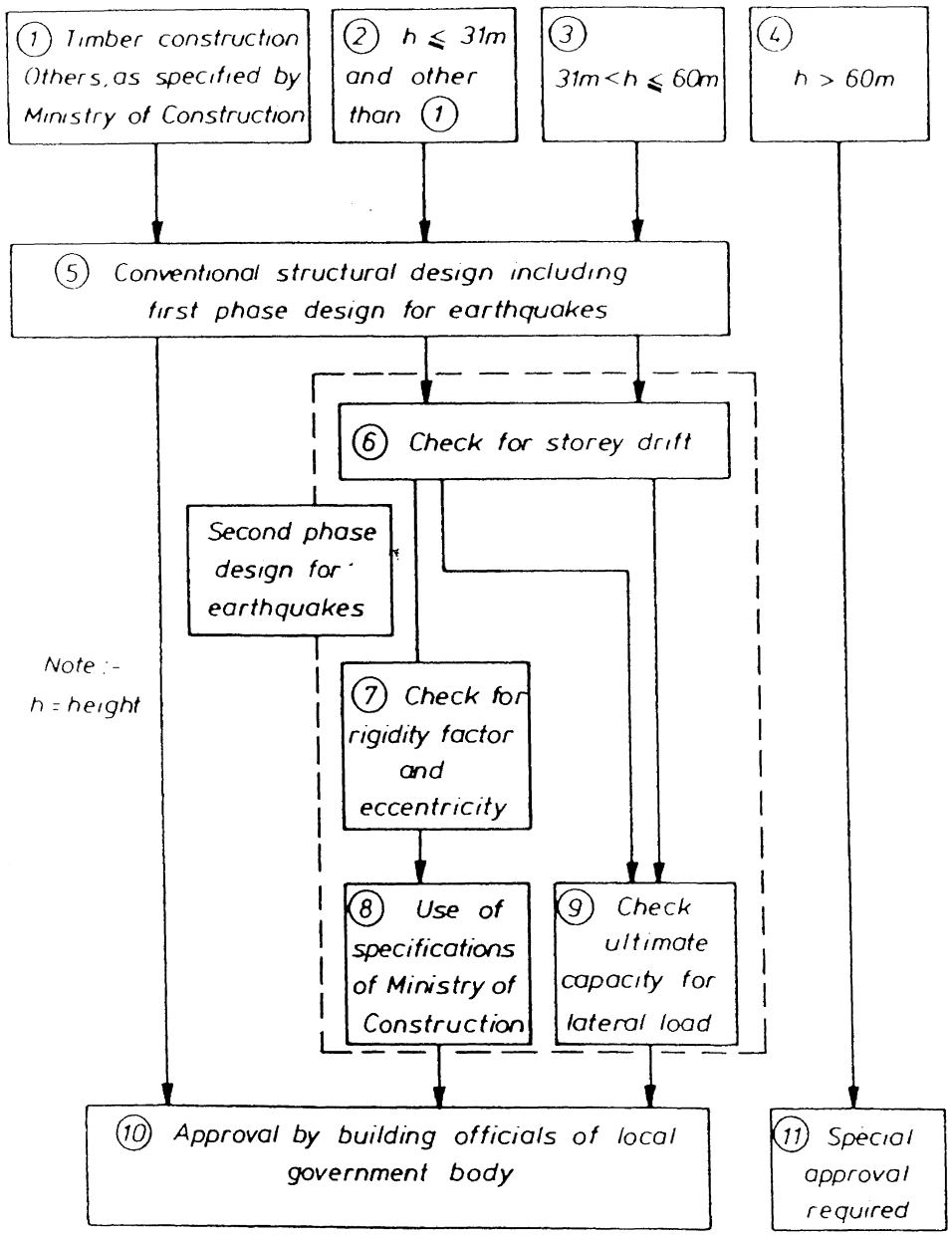


FIGURE 6.2 General Flow of Seismic Design [Aoyama, 1981]

However, there is no such choice for buildings exceeding 31 m in height, (box (3). These must be evaluated and checked for the ultimate lateral load carrying capacity. This route is the most straightforward application of the philosophy of the two-phase design.

The purpose of the check for ultimate capacity for lateral load is to evaluate the actual strength of the structure by means of limit analysis, and to ascertain that the response deformations would indeed lie within the ductility capacity provided.

The structural design and drawings must be presented to the local government body for approval by the Building Official of the city, town, or prefecture, as conforming to the requirements of the Building Standard Law".

6.3 General Types of Failure

6.3.1 *General Causes of Failure During the Hyogo-ken Nanbu Earthquake*

The AIJ report [Architectural Institute of Japan, 1995] gives damage statistics for the performance of reinforced concrete, steel reinforced concrete and structural steel buildings during the Hyogo-ken Nanbu Earthquake.

A general conclusion is that buildings designed to the current 1981 code performed very well on the whole. The damage was almost entirely to the older stock of buildings.

The poor performance of older (pre-1981) buildings which suffered damage was generally due to one or more of the following reasons:

- Inadequate strength of the overall seismic-resisting system.
- Inadequate stiffness of the overall seismic-resisting system.
- Poor distribution of strength/stiffness over successive storeys, leading to soft storey formation.
- Lack of provision of an adequate load path through the structure leading to partial or complete failure of the structure.
- Poor detailing of joints and connections leading to various types of non-ductile and/or buckling failures.

6.3.2 *General Types of Failure*

(a) Tilting or Overturning

Tilting of buildings (see Figure 6.3), and even overturning (see Figures 6.4 and 6.5) occurred in some cases in the most devastated areas of the city. The columns of the overturned buildings evidently parted from their foundations.

(b) Soft Storey Collapses

The predominant mode of collapse of pre-1981 buildings was of the soft storey type.

Many soft storey collapses were due to failure of the first storey columns (for example, see Figures 6.6 to 6.9). These failures occurred due to strong beam-weak column behaviour of moment resisting frames, lack of ductility of columns and failure of brace to frame connections in braced systems.

However, many soft storey collapses occurred due to the failure of columns in an intermediate storey up the height of the building (see Figure 6.10 to 6.13). For example, the Municipal Hospital in the west of Kobe had a soft storey failure in the 5th storey (49 people killed), as shown in Figure 6.13. These soft intermediate storey failures were evidently mainly due to:

1 The design horizontal earthquake forces for static load analysis, typically 0.2g in Kobe, for those early structures (for example, in the 1950s and 1960s) was applied uniformly up the height of the building. For the same imposed design static horizontal shear at the base of the frame, the shear in the columns at the mid height of a frame for horizontal loading distributed up the height in the shape of an inverted triangle (which is close to the loading due to the first mode of vibration) is 50% higher than for horizontal loading distributed uniformly up the height. Hence the shear forces and bending moments induced at the mid height of the frame due to earthquake shaking were underestimated.

2 Column sections were reduced significantly at intervals up the height of the building.

3 Steel reinforced concrete was often used in the lower storeys with only reinforced concrete used for the top storeys.

4 The columns were not adequately detailed for ductility and hence were incapable of undergoing significant inelastic rotation. For example, for structural steel columns severe local buckling may have occurred due to high slenderness ratios of plate elements in the column, and for reinforced concrete columns crushing of compressed concrete, buckling of longitudinal reinforcement or shear failure may have occurred due to lack of sufficient transverse reinforcement. The vertical spacing of hoops in the collapsed reinforced concrete columns of the Municipal hospital building was 300 mm.

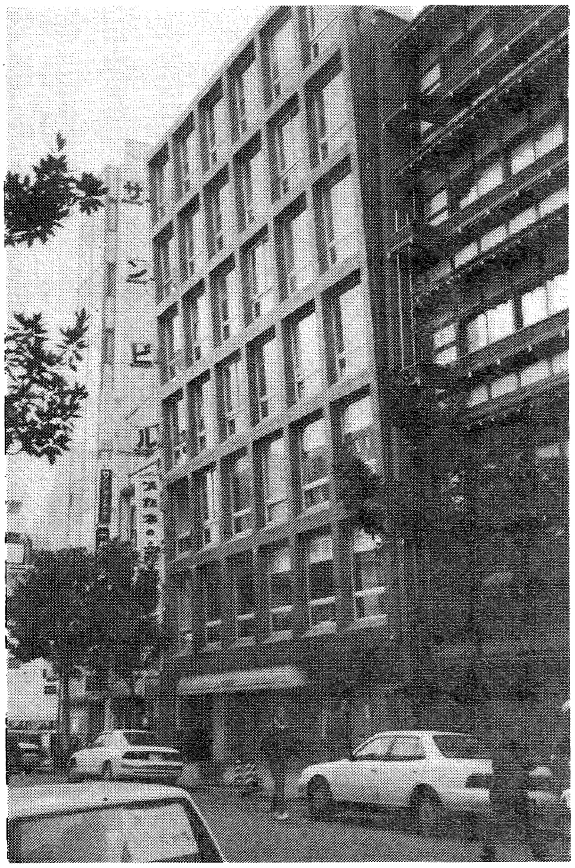


FIGURE 6.3 Tilting of Building

5 Failure of beam-column joint or connection regions.

Hence there was inadequate flexural strength of the columns in the upper stories of frames to cope with bending moments due to earthquake shaking, there were significant strength and stiffness reductions up the height of the frame (vertical irregularities), and the columns and joints were not detailed for ductility.

(c) Other Types of General Failure

Other general types of failure observed were due to torsion of structures on street corners which were unsymmetrical in plan (see Figures 6.14 and 6.15) and pounding of adjacent structures of different stiffnesses.

6.4 Behaviour of Reinforced Concrete Building Structures

6.4.1 *General Behaviour*

The modern reinforced concrete buildings (post-1981) appeared to behave well on the whole.

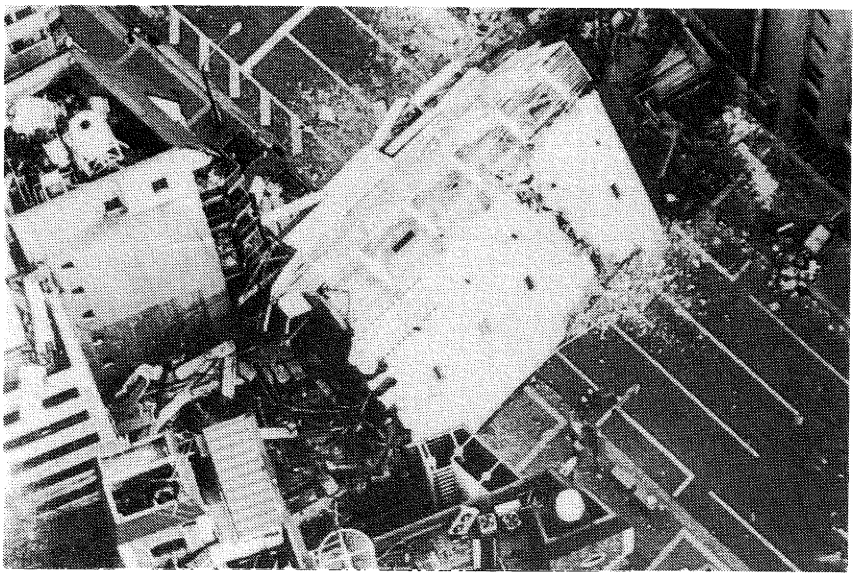


FIGURE 6.4 Overturned Building Lying Across a Street

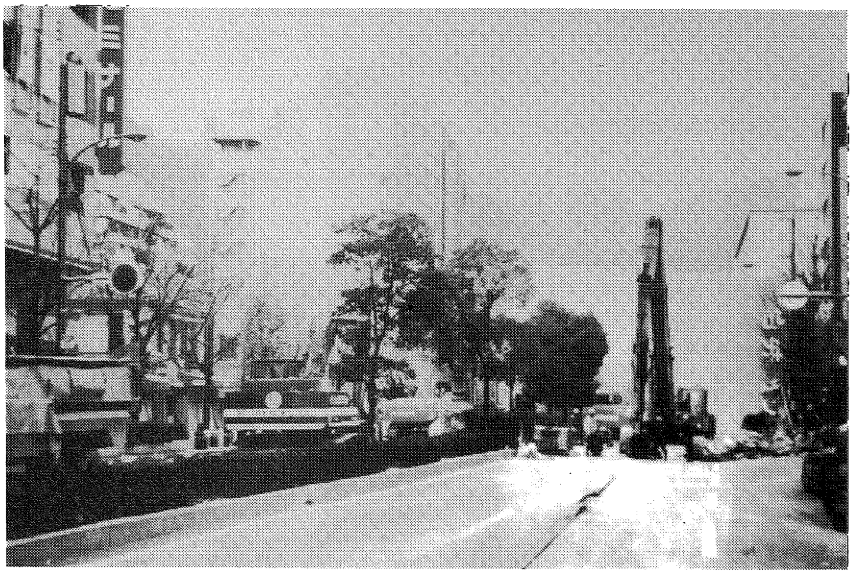


FIGURE 6.5 Overturned Building Lying Across a Street

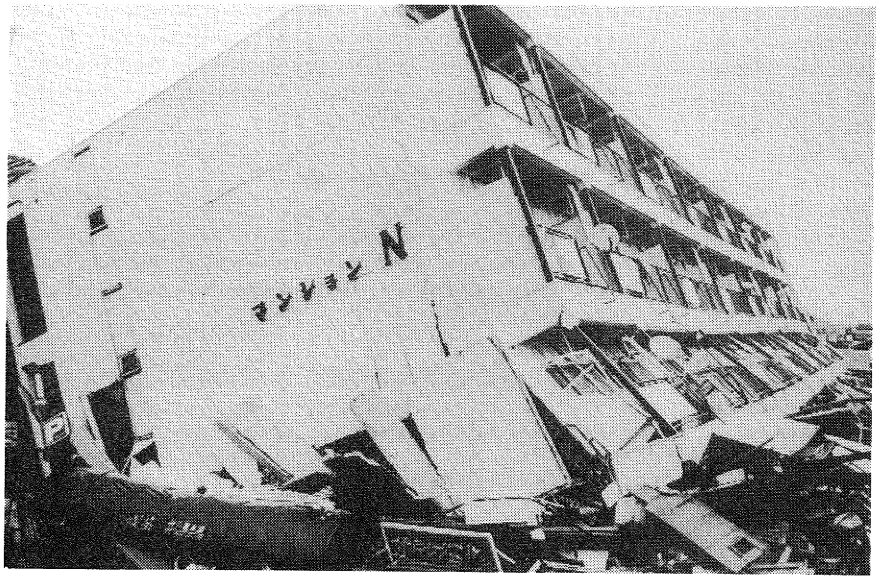


FIGURE 6.6 Soft Storey Collapse of the First Storey of an Apartment Building

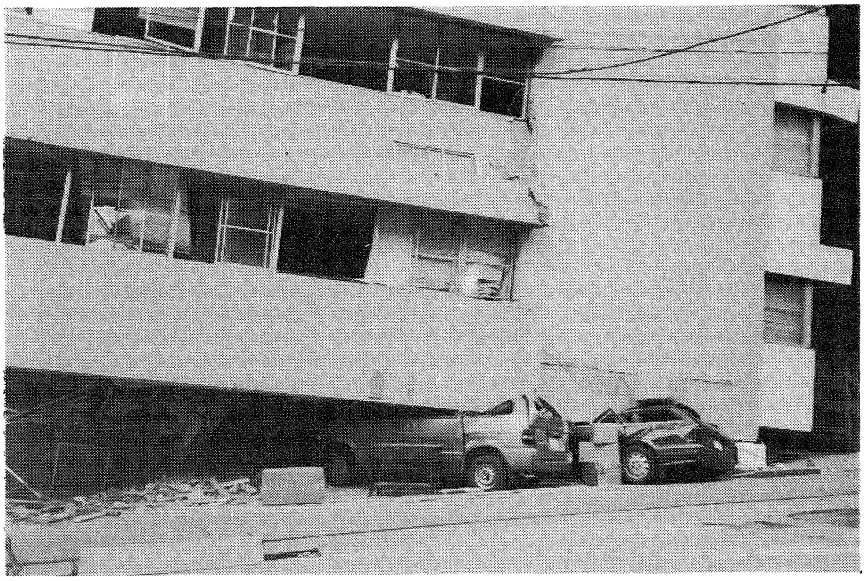


FIGURE 6.7 Soft Storey Collapse of the First Storey of an Apartment Building

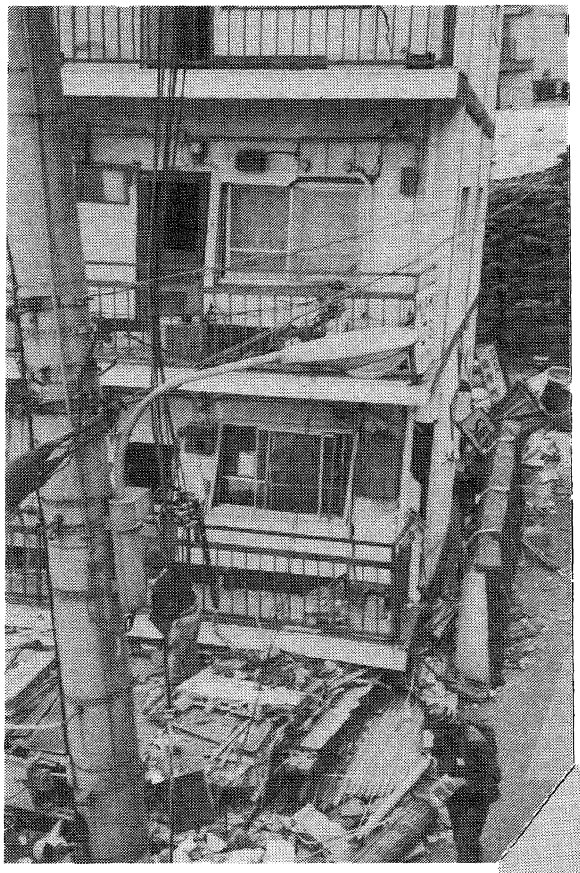
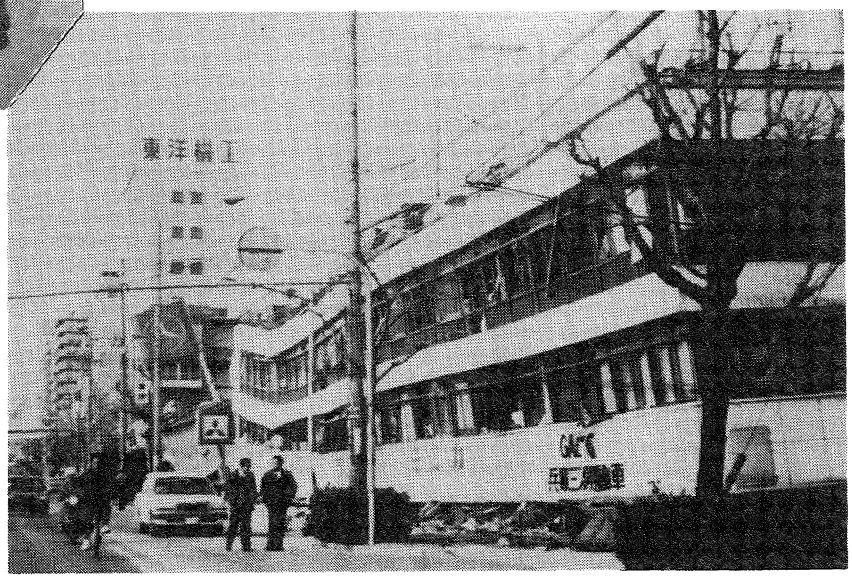


FIGURE 6.8 Soft Storey Collapse of the First Storey of an Apartment Building

FIGURE 6.9 Soft Storey Collapse of the First Storey of a Departmental Store



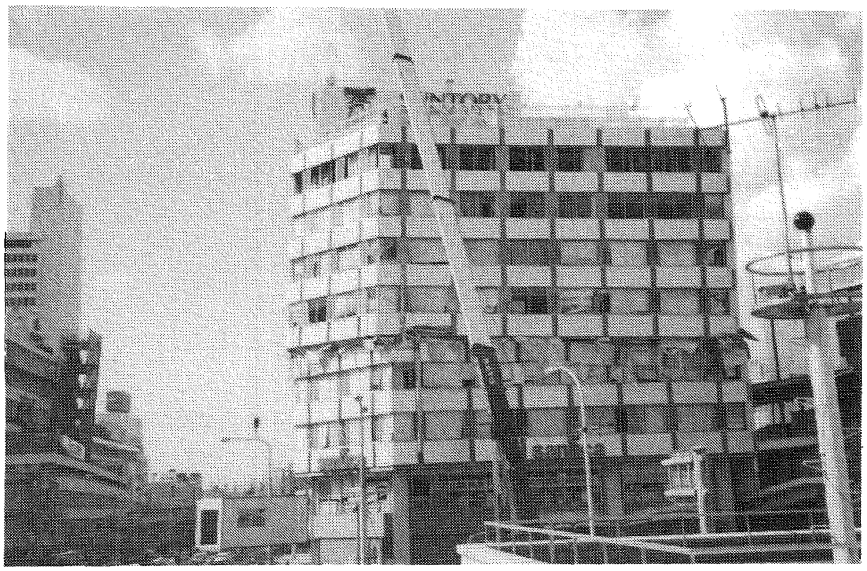


FIGURE 6.10 Soft Storey Collapse of an Intermediate Storey of a Building

FIGURE 6.11 Soft Storey Collapse of an Intermediate Storey of a Building

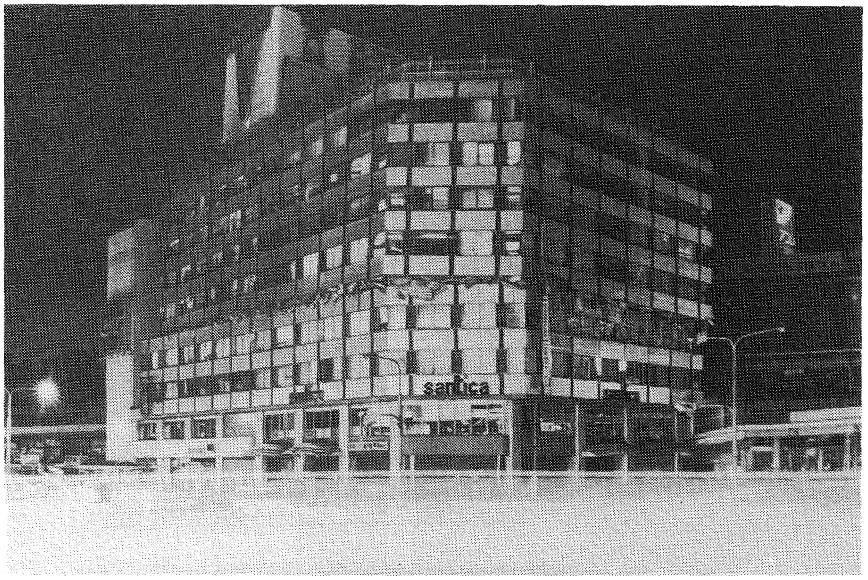




FIGURE 6.12 Soft Storey Collapse of an Intermediate Storey of the No. 2 Municipal Building, Sannomiya



FIGURE 6.13 Soft Storey Collapse of an Intermediate Storey of the West Kobe Hospital Building



FIGURE 6.14 Collapse of a Building With an Unsymmetrical Floor Plan

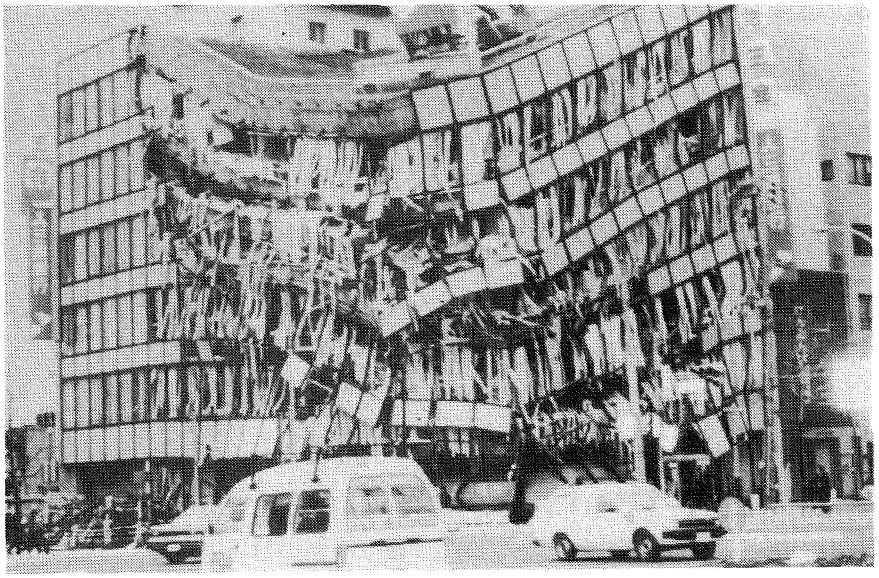


FIGURE 6.15 Collapse of a Building with an Unsymmetrical Floor Plan

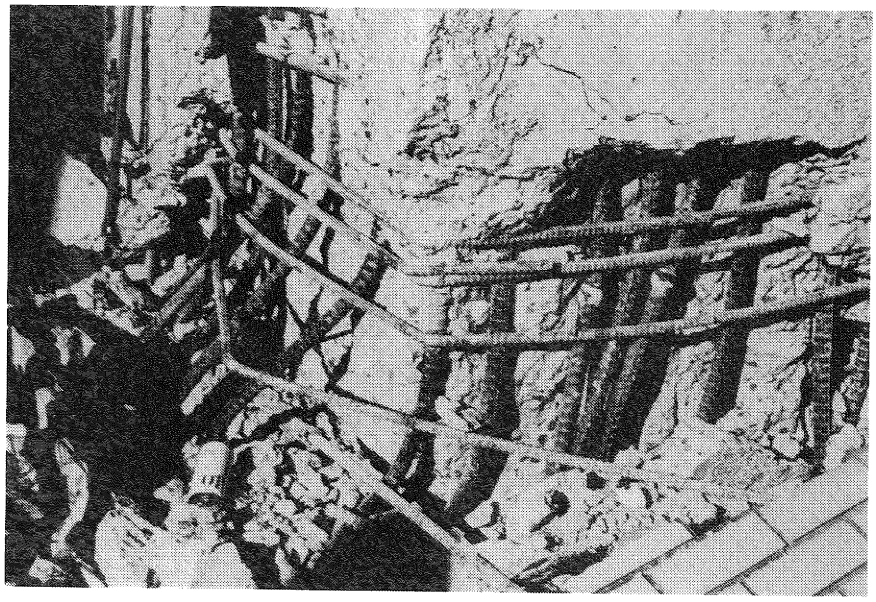


FIGURE 6.16 Poor Anchorage of Hoops of a Reinforced Concrete Column as a Result of the Use of 90 Degree Hooks

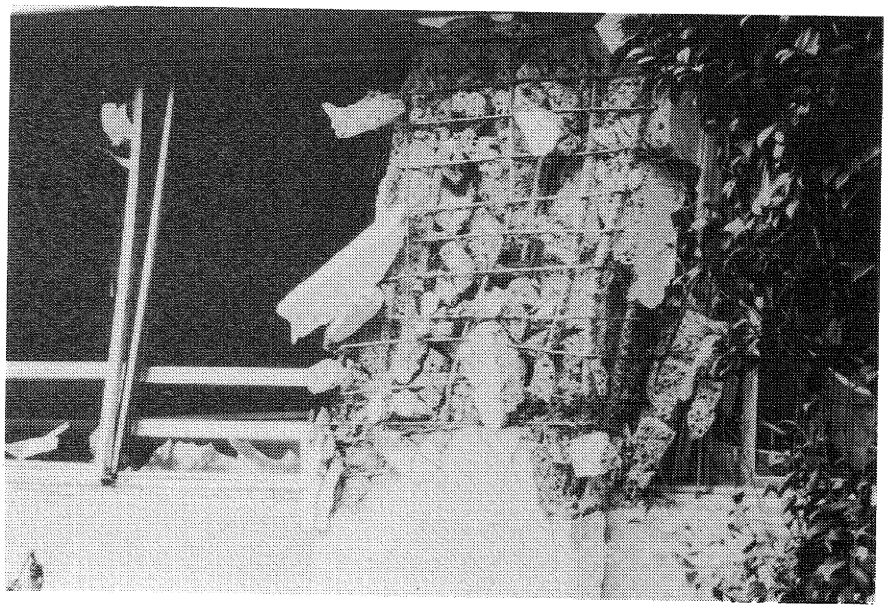


FIGURE 6.17 Shear Failure of a Reinforced Concrete Column

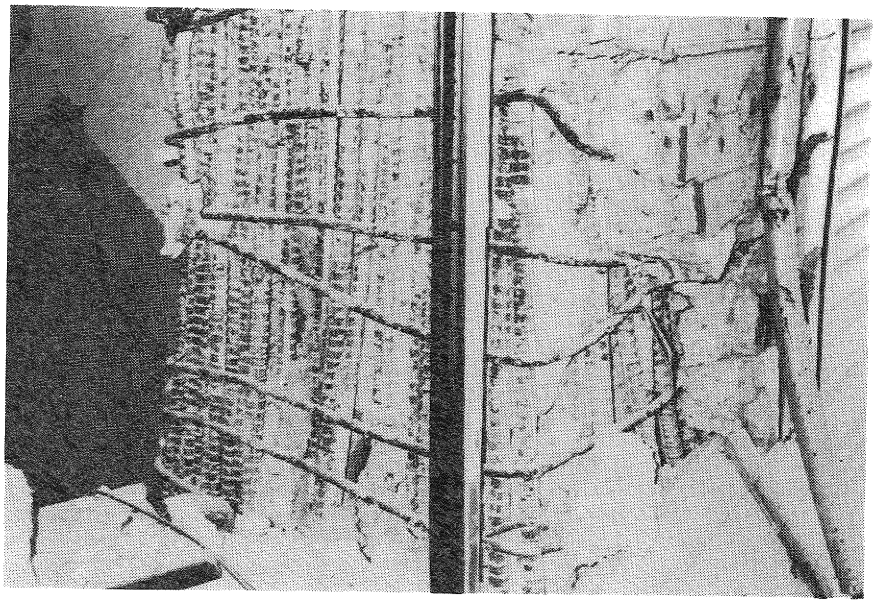


FIGURE 6.18 Congestion of Longitudinal Reinforcement in a Reinforced Concrete Column

The columns of older reinforced concrete buildings (pre-1981) proved to be very vulnerable, as has been found in past major earthquakes. As a result of the Tokachi-oki Earthquake in 1968, in 1971 the vertical spacing of transverse reinforcement in reinforced concrete columns was reduced from 300 mm to 100 mm. However, rules for cross ties in rectangular column sections were not introduced. That is, building columns of rectangular section were only required to have perimeter hoops.

6.4.2 *Types of Failure*

The particular problems for reinforced concrete structures were due to poor detailing of reinforcement leading to shear failures, non-ductile compression failures of concrete, buckling of compressed longitudinal reinforcement, and anchorage failure, and hence:

- Failure of columns due to a wide spacing of hoops and lack of supplementary cross ties (see Figure 6.13).
- Failure of columns due to the poor anchorage of the ends of hoop bars as a result of the use of 90° degree hooks and/or short end extensions (see Figure 6.16).
- Shear failure of columns because the transverse reinforcement was inadequate for the imposed shear force (see Figure 6.17). Some times the imposed shear force was unintentionally increased because the actual span length was shortened by the presence of side walls.
- Failure of columns due to congestion of longitudinal reinforcement (see Figure 6.18).
- Failure of columns due to the fracture of the longitudinal bars where butt welded by gas heated pressure welding.
- Failure in the beam-column joints (see Figure 6.19).
- Shear failure of non-structural wall elements (see Figures 6.20, 6.21 and 6.22).

These types of failure emphasise the need for seismic design procedures for reinforced concrete structures to incorporate:

(a) Capacity design in which it is ensured as far as possible that the balance of strength of structural elements is such that, in the event of a major earthquake, yielding (post-elastic deformation) occurs only in regions of the structure which are chosen by the designer, where it can be tolerated by the structure.

(b) Ductile detailing whereby the chosen regions where yielding is expected are carefully detailed by the designer to ensure that adequate ductility is available there.

Seismic design procedures involving capacity design and ductile detailing were introduced in New Zealand for reinforced concrete structures in NZS 4203:1976 and NZS 3101:1982.

6.5 Behaviour of Structural Steel and Steel Reinforced Concrete Building Structures

6.5.1 *Modern Buildings*

Modern (post-1981) Japanese steel seismic-resisting systems are fabricated and constructed using similar concepts and techniques to those in New Zealand. This especially relates to the welding of moment-resisting beam to column connections, in the fabricators shop, coupled with the use of site-bolted splices. These systems performed well, both in terms of performance of the structural system and of the non-structural components. Performance of the site bolted splices was especially good, even where these were subjected to severe and unexpected overloading due to, for example, impact from adjacent structures.

The combination site-welded, site-bolted moment-resisting frame beam-to-column connection favoured in North America does not appear to be used in Japan. This connection detail performed poorly in the January 1994 Northridge Earthquake (Los Angeles, USA). No examples of this type of failure were observed in the Hyogo-ken Nanbu earthquake.

6.5.2 *Older Medium-Rise Commercial and Residential Buildings*

This building group covers pre 1981 buildings of 3-10 storeys. Where seismic design was undertaken on these buildings, it probably involved use of a uniform 0.2g static lateral force. The resulting seismic-resisting systems had limited strength in the elastic range, especially in the upper storeys and were not designed or detailed for dependable inelastic response. An example of a collapsed structural steel building is shown in Figure 6.23.

The moment-resisting framed (MRF) beam-to-column connections were typically shop welded and those involving beams connected to the column major axis (that is, beams framing into the face of a column flange) performed well, as the connection was invariably stronger than the beam. However, a case of inadequate welding leading to fracture of the welds at the beam-column face connection is shown in Figure 6.24.

Another example of beam-column connection failure, due to inadequate provision of a load path through the connection, is shown in Figure 6.25. The seismic-resisting system comprised a two-way MRF, utilising universal column (UC) sections as column members. The beam to column connection about the column's weak axis comprised an endplate welded to the beam on one side and to the column flange tips on the other. The weak link of this beam-column subassemblage in bending was the endplate to column flange tip welds, which peeled off, as shown in Figure 6.25. This led to a sidesway mechanism developing in the four storey building, with partial ground floor collapse.

No attempt was made, to design connections stronger than the primary members they were connecting. In site-bolted concentrically braced framed (CBF) systems, this led to numerous instances of failure at the brace to frame connections, sometimes with graphic results, as shown in Figure 6.26.

Also, no attempt was made to develop an overall collapse mechanism in the seismic-resisting systems, with the result being examples of soft storey mechanisms forming, with partial or total collapse.

Many examples of mid-height soft storey collapse were observed in the older SRC buildings of between 5 and 10 storeys in height (for example, see Figure 6.27).

The most likely reason for these collapses lies in the combination of change to the form of encased steel column member, coupled with inadequate confinement of the concrete.



FIGURE 6.19 Failure of Reinforced Concrete Beam-Column Joint Regions

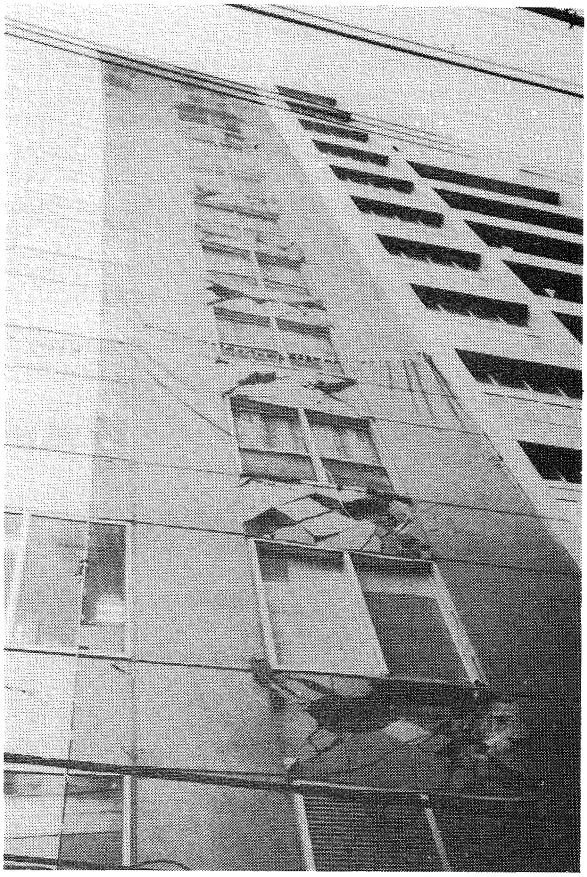


FIGURE 6.20 Shear Failure of Non-Structural Reinforced Concrete Beams in Wall Elements

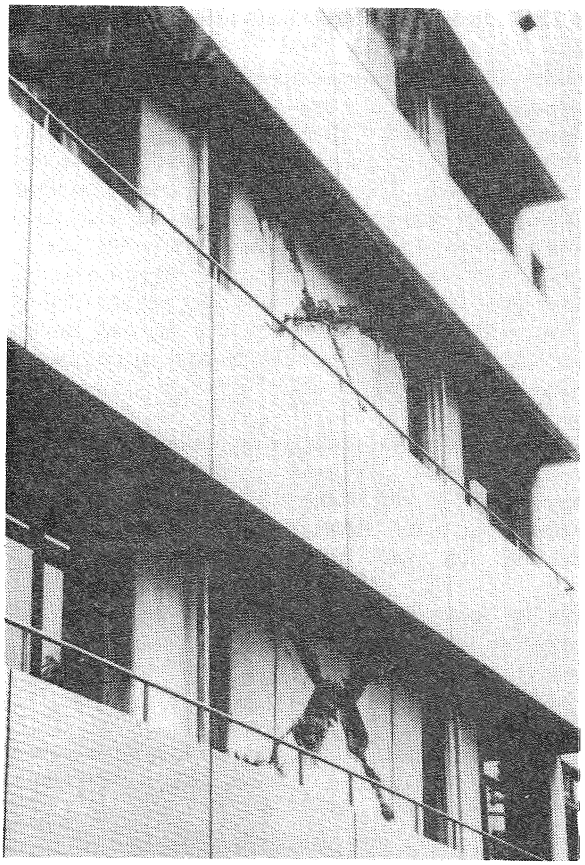


FIGURE 6.21 Shear Cracks in Non-Structural Reinforced Concrete Wall Elements

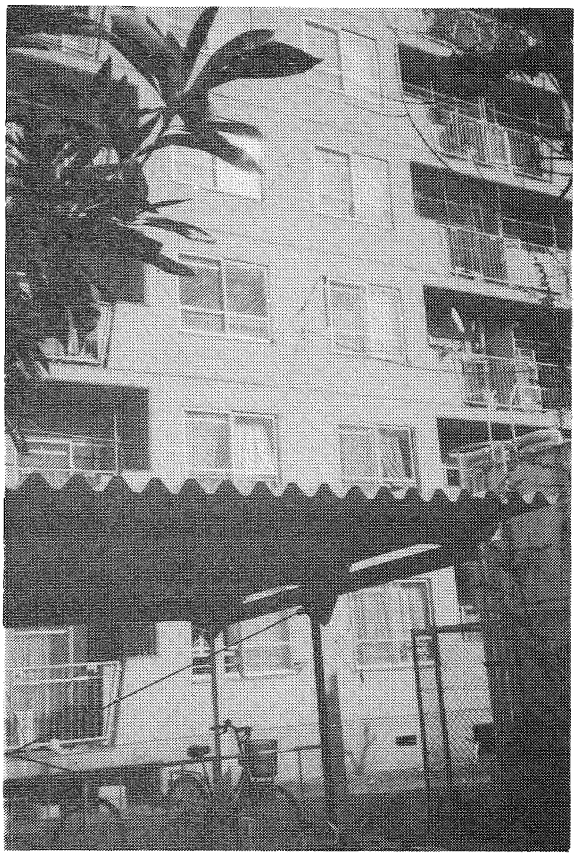


FIGURE 6.22 Shear Cracks in Non-Structural Reinforced Concrete Wall Elements

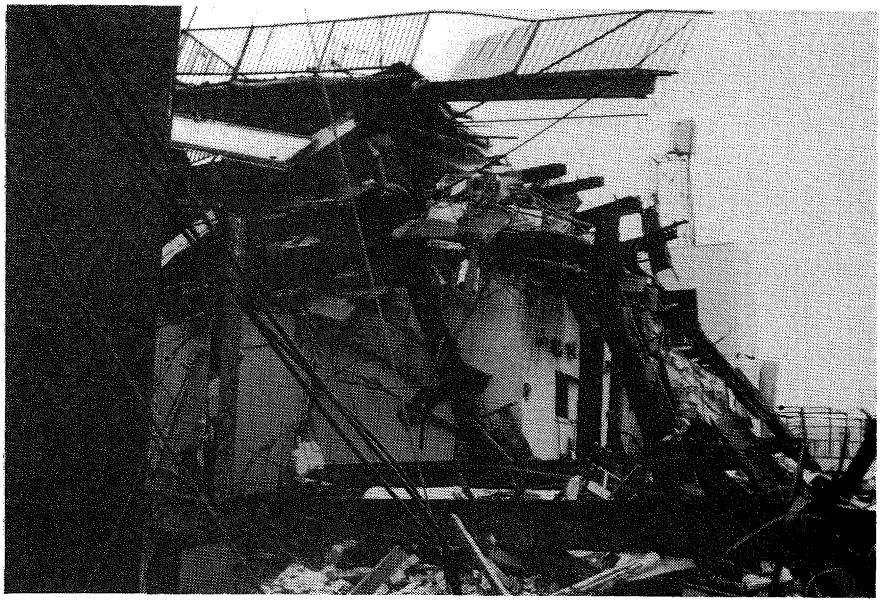


FIGURE 6.23 Failure of a Structural Steel Moment Resisting Frame



FIGURE 6.24 Failure of Welds at Beam-Column Face Connection

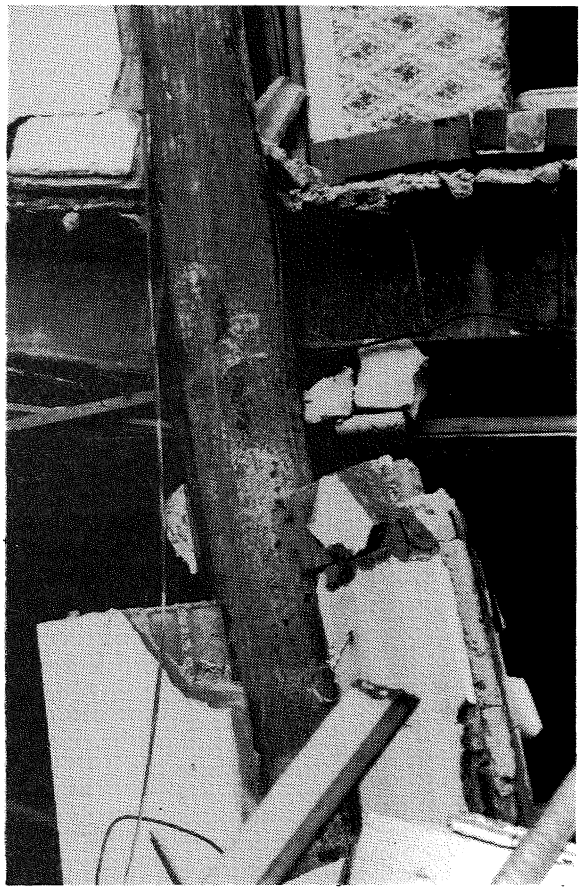


FIGURE 6.25 Connection Failure in Beam to Column Weak Axis "Rigid" Connection of a Pre-1981 Steel Moment-Resisting Frame



FIGURE 6.26 Failure of Brace to Frame Connections of a Concentrically Braced Steel Frame

Typically, the lower 3 to 5 storeys involved a solid I-section beam profile (ie, relatively deep and narrow flanged) for both beam and column members. Above this point, the encased column steel section then changed to a battened member, comprising two pairs of angles connected by horizontal battens to form an I-section with a discontinuous web. The confining concrete typically contained pairs of D20 longitudinal bars in the corners and at the middle of each face, with transverse reinforcement comprising a single R6 or R10 stirrup at around 300 mm centres.

During the earthquake, the loss of stiffness due to concrete cracking accentuated the large difference in stiffness and strength between the solid I-section below the transition point and the battened I-section above the transition point. Inelastic demand was rapidly concentrated into the latter column, which, once the encasing concrete was lost, would have been incapable of supporting even the gravity loads alone as a bare steel section. Column collapse soon followed, as shown in Figure 6.27.

Over 95% of the damaged/collapsed steel or SRC buildings were in this group of older, medium-rise commercial and residential buildings.

6.5.3 *Modern Medium-Rise Commercial and Residential Buildings*

This group covers post-1981 buildings in structural steel or SRC. The design loadings for these buildings specifies an inverse triangular static load and the materials code specifies a degree of strength hierarchy in connection design; for example, requiring connections to CBF braces to develop 1.2 times the nominal tensile strength of the brace. A detailed description of the current Japanese seismic design criteria for steel buildings is given by Kato, 1986. There is no requirement to develop an overall type mechanism of seismic-resisting system response when designing for anticipated post-elastic response. The typical level of structural ductility demand expected in design would lie between the New Zealand limited ductile and nominally elastic categories [NZS 3404, 1992].



FIGURE 6.27 Disintegration and Collapse of a Concrete Encased Steel Battened Column in a Medium-Rise SRC Moment Frame

This group of buildings generally performed well. Most exhibited no damage. Minor points of damage that were noted involved compression buckling of gusset plates in CBF brace to beam connections and minor hinging of beams in some MRF systems. One example of gross beam-to-column failure due to faulty welding was observed and five examples of columns with grossly undersized shop welds to column spices were noted. One of these was a two-way MRF four storey building which utilised 300 mm square rectangular hollow section (RHS) box columns of 12 mm wall thickness. These were welded to transfer plates at the connections with a fillet weld all round. The as-welded design throat thickness of this weld varied from 4 to 5 mm and the visually observed weld quality was poor. These welds had failed in the lower two storeys, leading to complete collapse of these levels.

6.5.4 ***High Rise Commercial and Residential Buildings***

With one exception, involving column brittle fracture and covered in Section 6.5.7, all these buildings performed very well. No structural damage was observed or reported and only minor instances of non-structural damage were noted. This non-structural damage involved minor glass cracking and cracking of tiled and cladding panels. All these buildings observed were in use by the time of the visit of the reconnaissance team, although most had suffered disruption of water services and gas supply due to ground movement in the region.

6.5.5 ***Industrial Buildings***

The industrial buildings within the strongly shaken area comprised a mix of storage buildings and buildings housing light- to medium- industry. The single storey buildings were typically of portal frame construction, with light-weight metal or fibre-cement roofs. Walls were typically light-weight metal cladding over a mix of light-weight metal cladding on the upper half height and concrete panel over the lower half height.

This group of buildings performed well. The few examples of damage observed were due to one of:

(1) Ground settlement and/or ground lateral spreading (see Figure 8.8).

(2) Impact of contents on building.

(3) Gusset-plate tearout of braces (one example noted in an older building).

6.5.6 ***Steel Lattice Towers***

The most common form of angle steel lattice tower were the high-tension power pylons. As far as could be observed, these had all performed well. A 50 m high steel lattice tower of similar construction sited on top of a 10 storey building suffered failure to some lower bracing members and permanent buckling of one of the four lattice legs. This tower supported an array of communications equipment.

6.5.7 ***Examples of Column Brittle Fracture***

An example of brittle failure of columns occurred in the lower levels of the box columns forming part of external CBF seismic-resisting systems in a complex of 50 high-rise apartments (see Figure 6.28). These apartments were built in 1980 and utilised 500 MPa yield strength steel box columns. The box columns varied from 500 mm square cross section with 55 mm wall thickness in the lowest levels to smaller cross section and 15 mm wall thickness in the upper levels. The lower levels, with wall thicknesses of 50 and 35 mm, had suffered brittle failure in around 10% of the columns. The form of this failure is shown in Figure 6.29, although the residual gap shown there (20 mm) was the largest recorded. In some of the CBF systems, this primary failure generated subsequent tearing and/or inelastic buckling of some braces at higher levels. One reason for these brittle failures may have been the members being below their transition temperature at the time of the earthquake.

The buildings looked outwardly undamaged and were occupied at the time of the reconnaissance team's visit, although lack of water services and gas supply had caused some occupants to temporarily vacate. Repairs to the damage were being effected, although the design rationale to these repairs was not apparent.

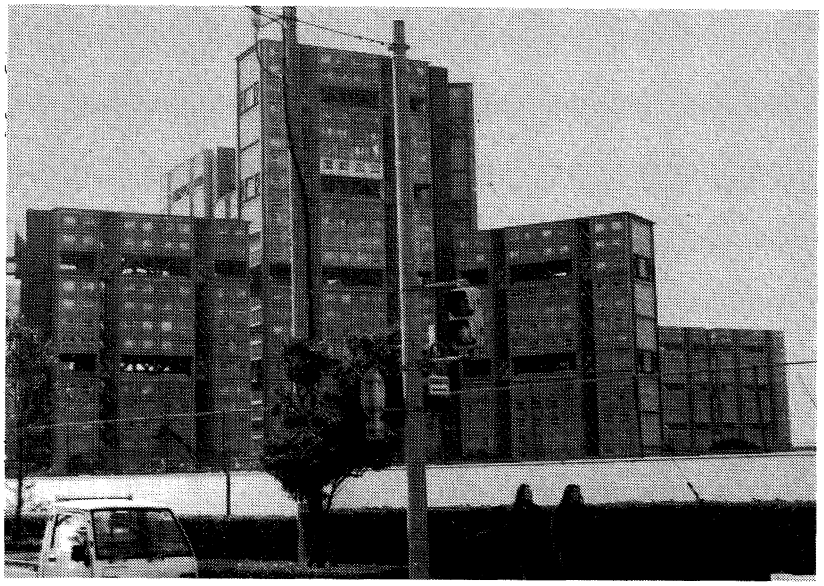


FIGURE 6.28 Fifty Building Apartment Complex Where Brittle Fracture of External Concentrically Braced Frame System Columns Occurred

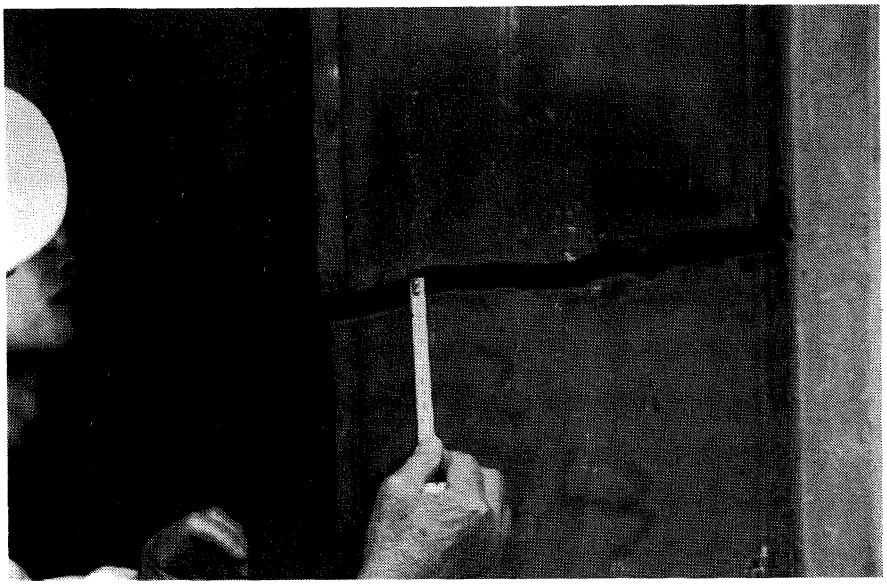


FIGURE 6.29 An Example of CBF Column Brittle Fracture Showing the Residual Gap

6.5.8 ***Performance of Specific Types of Structural Steel Seismic Resisting System***

The principal forms of structural steel seismic-resisting systems in the strongly shaken area were:

(a) MRFs, typically utilising box columns and I-section beams.

(b) CBFs, utilising x-braces with braces effective in tension.

(c) CBFs, utilising v-braces with braces effective in tension and compression.

There were no eccentrically braced frame (EBF) systems observed.

The construction of these was similar to New Zealand practice, using shop welded assemblages and fully tensioned high strength structural site bolted splices.

Well proportioned systems, where all connections between members maintained their integrity, performed very well, with no structural damage and negligible non-structural damage.

Well proportioned braced framed systems, but where some connections between members failed due to insufficient strength and ductility, still performed quite well. These buildings typically showed cracking of non-structural cladding, although many had been repaired by the time of the visit and were operational.

Poorly proportioned systems, which especially with inadequate column strength and storey stiffness, performed poorly. This was especially the case with tension braced CBF systems which also suffered brace connection failures.

6.5.9 ***Performance of Connections***

Shop welded connections which were proportioned to develop the design capacity of the weakest member or members performed well. The one exception was an MRF with gross lack of fusion in the welds between the beam flanges and the column flange. Shop welded connections which contained an (unintentional) weak link in the principal load path through the connection performed poorly.

One specific recommendation can be made for New Zealand relating to design of gusset plates in the brace to frame connection of concentrically braced steel frames (CBFs) where the braces are designed to be effective in compression as well as in tension. The recommendation is to ensure that the gusset place will not suffer compression buckling prior to transmitting the maximum anticipated compression load developed in the brace through to the surrounding frame. This step is assumed, but not explicitly stated, in the design requirements of NZS 3404:1992, whereas design of the connection to transmit the maximum anticipated tension load is an explicit requirement.

The recommendation arises from the observation of two medium-rise v-braced CBF systems where compression buckling in the gusset plate was the only form of structural damage observed. If the CBF system is expected to undergo inelastic action, than the compression buckling associated with this should occur in the braces, rather than the connection gusset plates.

Site bolted splices involved the use of high strength structural bolts (class 8.8) in the fully tensioned mode, typically in fully bolted splices. These splices, in modern structures, appeared to be similarly proportioned to those which would result from current New Zealand practice. Considerable use was made of the Torque Control (TC) bolt. This bolt has received very limited use in New Zealand. The performance of these splices was very good, even when they were situated immediately adjacent to or within yielding regions. The TC and conventional high strength structural bolts both performed equally well.

Some bolt shearing occurred in the flange splice plates of older seismic-resisting systems. However this only occurred in conjunction with more general structural distress.

6.6 References

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7 BRIDGES

7.1 Introduction

7.1.1 *General*

The primary transportation links for the Kobe area are shown in Figure 7.1. These links comprise a series of expressways and railways, generally supported on elevated bridge structures. The elevated bridges suffered severe damage and collapses occurred at many locations in the areas of intense shaking.

All the primary transportation links between Nishinomiya city and Kobe were cut, causing the remaining on-grade roads to become chaotically overloaded. Bridge collapses on the Shinkansen (bullet train) Line cut the system, isolating the northern and southern lines.

Considering the severe damage suffered to the bridges the death toll was relatively modest reflecting the early morning timing of the earthquake. On the Kobe route of the Hanshin Expressway 16 deaths occurred while on the Wangan Route there were 2 deaths.

7.1.2 *Bridge Codes*

As with the building codes, seismic design requirements for bridges in Japan were introduced as a result of the Great Kanto Earthquake of 1923. Seismic design requirements for highway bridges were introduced in 1926, and revised periodically in response to lessons learned from damaging earthquakes. This is illustrated in Table 7.1 which is taken from Ichimasu, Kodera and Kawashima, 1991.

Major revisions to the highway bridge design codes occurred in 1971, 1980 and 1990. Of particular relevance is the 1980 code. This contained significant provisions to improve the ductility of bridge piers and was introduced primarily as a result of the Miyagi-ken-oki Earthquake of 1978. An important provision was the requirement to extend the terminations of pier longitudinal reinforcing by the column dimension plus 20 bar diameters past the point theoretically needed for flexure.

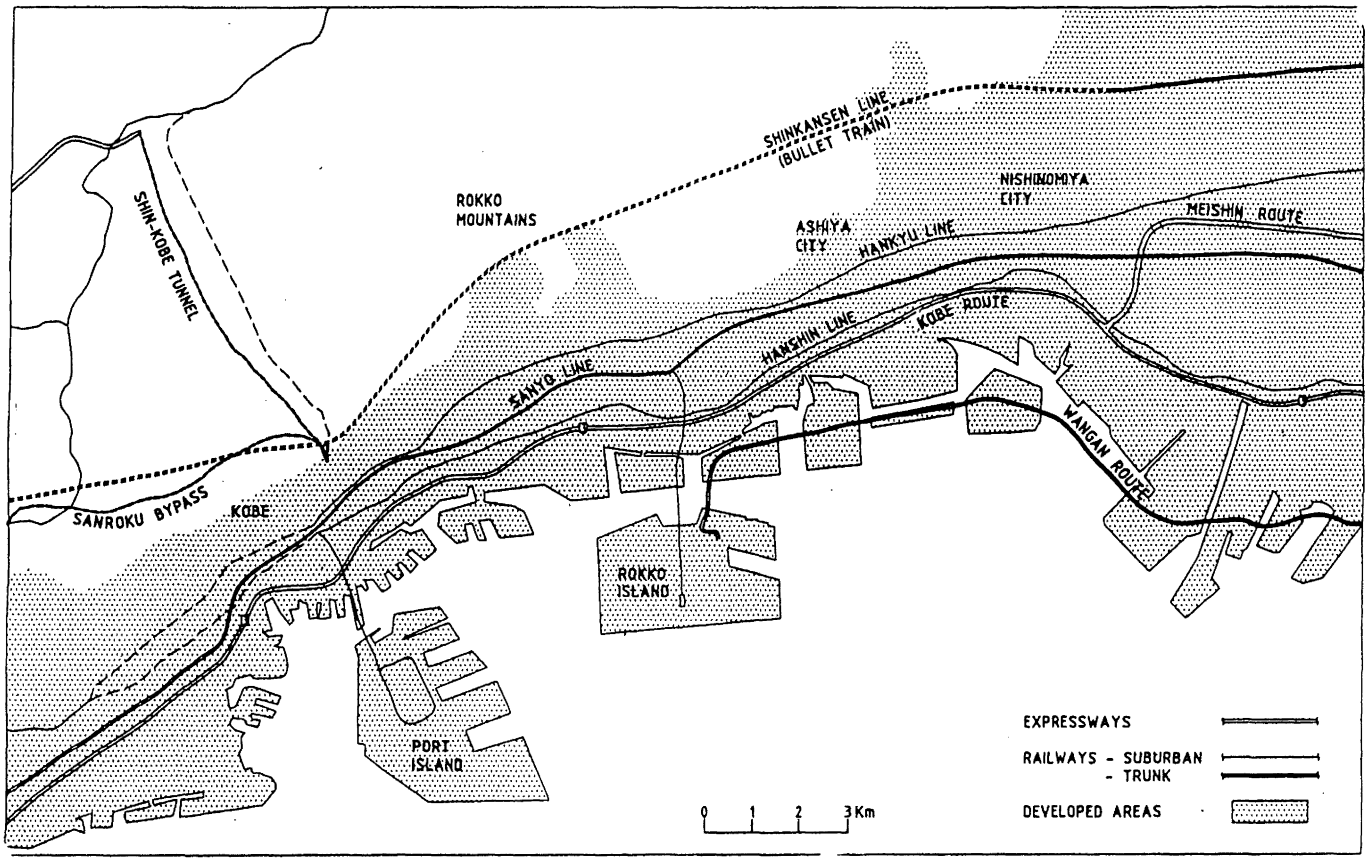


FIGURE 7.1 Primary Transportation Routes in the Kobe/Nishinomiya Area

Table 7.1 History of Seismic Design Loads for Highway Bridges in Japan

Year	Highways Inspected	Inspection Items	Number of Bridges		
			Inspected	Require Strengthening	Strengthened
1971	All Sections of National Expressways and National Highways, and Sections of the Others (Bridge Length ≧ 5m)	① Deterioration ② Bearing Seat Length S for Bridges supported by Bent Piles	18,000	3,200	1,500
1976	All Sections of National Expressways and National Highways, and Sections of the Others (Bridge Length ≧ 15m or Overpass Bridges)	① Deterioration of Substructures, Bearing Supports and Girders/Slabs ② Bearing Seat Length S and Devices for Preventing Falling-off of Superstructure	25,000	7,000	2,500
1979	All Sections of National Expressways, National Highways and Principal Local Highways, and Sections of the Others (Bridge Length ≧ 15m or Overpass Bridges)	① Deterioration of Substructures and Bearing Supports ② Devices for Preventing Falling-off of Superstructure ③ Effect of Soil Liquefaction ④ Bearing Capacity of Soils and Piles ⑤ Strength of RC Piers ⑥ Vulnerable Foundations (Bent Pile and RC Frame on Two Independent Caisson Foundation)	35,000	16,000	13,000
1986	All Sections of National Expressways, National Highways and Principal Local Highways, and Sections of the Others (Bridge Length ≧ 15m or Overpass Bridges)	① Deterioration of Substructures, Bearing Supports and Concrete Girders ② Devices for Preventing Falling-off of Superstructure ③ Effect of Soil Liquefaction ④ Strength of RC Piers (Bottom of Piers and Termination Zone of Main Reinforcement) ⑤ Bearing Capacity of Piles ⑥ Vulnerable Foundations (Bent Piles and RC Frame on Two Independent Caisson Foundation)	40,000	11,800	6,000 (As of the End of 1989 Fiscal Year)

Note) Number of bridges inspected, number of bridges that required strengthening and number of bridges strengthened are approximate numbers.

The requirements of the 1980 code are so significant, that is convenient to classify the performance of bridges into pre-1980 and post-1980.

7.1.3 *Bridge Seismic Inspection and Retrofitting*

In parallel with the progressive improvements to the bridge codes in Japan, the deficiencies inherent in existing structures have been recognised. For highway bridges, the sequence of inspection and seismic retrofitting is illustrated in Table 7.2 which is taken from Kawashima and Unjoh, 1990.

It is known that trial retrofitting using steel jackets on 20 piers on the Hanshin Expressway was implemented several years ago [Kawashima, Unjoh and Iida, 1992]. Had appropriate pier retrofits been applied to the bridges in the areas of intense shaking, the damage and the extent of collapse would have been reduced dramatically.

7.1.4 *General Performance*

Pre-1980 Bridges

In general the elevated bridge structures constructed prior to 1980 performed poorly. The most serious damage was to bridge piers which were heavily damaged and collapsed in a number of areas. The damage and collapse was primarily due to the inability of the columns to perform in a ductile manner. In concrete columns shear failures and lack of concrete confinement in the yielding regions were endemic.

Major damage also occurred to bridge bearings, seismic restrainers and road joints.

A significant number of bridge spans collapsed fully or partially.

Post-1980 Bridges

The elevated bridges performed quite well and little pier damage was observed. In two instances damage to pier foundations had resulted in loss of spans. This appeared to be primarily due to liquefaction and lateral spreading effects on piers located adjacent to waterways.

Damage to bridge bearings and road joints was extensive in the area of intense shaking.

7.2 General Behaviour

7.2.1 *Hanshin Expressway*

The Hanshin Expressway is the primary highway link through the Kobe area. The 4 lane section between Kobe and Nishinomiya was constructed between 1965 and 1970. The section east of Nishinomiya, linking Osaka, was constructed between 1981 and 1985. The section between Kobe and Nishinomiya is an elevated bridge structure. Collapses occurred at 14 locations and involved approximately 10% of this section of the expressway. Major damage occurred to a significant proportion of the bridge piers and damage was observed to bridge bearings and road joints at many locations.

A 600 metre section near Ashiya collapsed sideways (northwards) throwing traffic off the expressway and crushing vehicles beneath (see Figure 7.2). This expressway section was constructed with 36 m concrete spans, each span comprising a 25 m "drop-in" section and an 11 m hammerhead supported on reinforced concrete single column piers of 3.1 m diameter. The

Table 7.2 Details of the Past Seismic Inspections of Highway Bridges in Japan

Year	Name of Regulations	Seismic Design Methods	Other Stipulations for Seismic Effects	Major Earthquakes
1886	Order No. 13, Ministry of Internal Affairs	Not Considered	Not Considered	1881 Nohbi (M8.4) 1923 Kanto (M7.9)
1926	Details of Road Structures (Draft), Road Law, MIA	Seismic Coefficient Method k_H=0.15~0.4 ($k_H \geqq 0.3$ advised in Tokyo and Yokohama)	Not Considered	
1939	Design Specifications of Steel Highway Bridges (Draft), MIA	Seismic Coefficient Method (k_H=0.2, k_V=0.1)	Not Considered	1946 Nankai (M8.1) 1948 Fukui (M7.3)
1956 (and 1964)	Design Specifications of Steel Highway Bridges, Ministry of Construction	Seismic Coefficient Method (k_H=0.1~0.35)	Not Considered	1952 Tokachi-oki (M8.2) 1964 Niigata (M7.5)
1971	Specifications for Seismic Design of Highway Bridges, MOC	• Seismic Coefficient Method (k_H=0.1~0.24) • Modified Seismic Coefficient Method (k_H=0.05~0.3)	• Evaluation of Sandy Layers Vulnerable to Liquefaction • Device for Preventing Falling-off of Superstructure	1978 Miyagi-ken-oki (M7.4)
1980	Part V Seismic Design of Design Specifications of Highway Bridges, MOC	• Seismic Coefficient Method (k_H=0.1~0.24) • Modified Seismic Coefficient Method (k_H=0.05~0.3) • Check of Deformation Capability of RC Piers • Dynamic Response Analysis	• F_L-Method for Evaluation of Liquefaction • Device for Preventing Falling-off of Superstructure	1982 Urakawa-oki (M7.1) 1983 Nihon-kai-chubu (M7.7)
1990	Part V Seismic Design of Design Specifications of Highway Bridges, MOC	• Seismic Coefficient Method (k_H=0.1~0.3) • Check of Bearing Capacity of RC Piers for Lateral Force • Dynamic Response Analysis	• F_L-Method for Evaluation of Liquefaction • Device for Preventing Falling-off of Superstructure	

piers are supported on piled foundations. Typical elevation and cross section are shown in Figure 7.3.

In the earthquake the piers suffered catastrophic flexural-shear failures. The 16 mm diameter circular hoops in the columns, which were lapped in the cover concrete, progressively unravelled and disintegration of the column resulted. One third of the 35 mm diameter longitudinal pier reinforcement terminates at 2.5 m above the pilecaps and it is likely that the column failure was initiated in this region.

The remainder of the Hanshin Expressway has a steel superstructure with a concrete deck slab. The spans are predominately simply supported steel I-girders with steel box girders at crossings of major roads and the like. Most of the steel superstructure is supported on single column piers, mostly with circular reinforced concrete columns but in some areas with square reinforced concrete columns or steel box columns. A large number of these columns were damaged.

The vulnerability of many Japanese reinforced concrete bridge piers to shear failure near the termination of the longitudinal reinforcement is well documented. Measures introduced in the 1980 Highway Bridge Code to mitigate this included extension of the bar terminations by the pier dimension plus 20 bar diameters past the point theoretically needed for flexure. Retrofitting of 20 square reinforced concrete columns with 6 mm steel jackets was carried out on the Hanshin Expressway some years ago. Their performance in the earthquake is understood to be satisfactory. However, they were not in the area of intense seismic shaking.

In areas where the column damage was significant, some lengths of the expressway had a lean to the North as shown in Figure 7.4. Where damage was more severe, with shear failure and degradation in the columns more advanced, failures often appeared to have resulted from longitudinal seismic actions (that is, along the bridge).

The steel box columns supporting the expressway at a number of locations also suffered damage. Local buckling of the steel plates was observed on a number of columns. The damage was generally near the base of the columns just above the stiffening effects of the column base, or in some cases just above additional stiffening plates applied at the column base. At one location where a 3 storey building collapsed against the

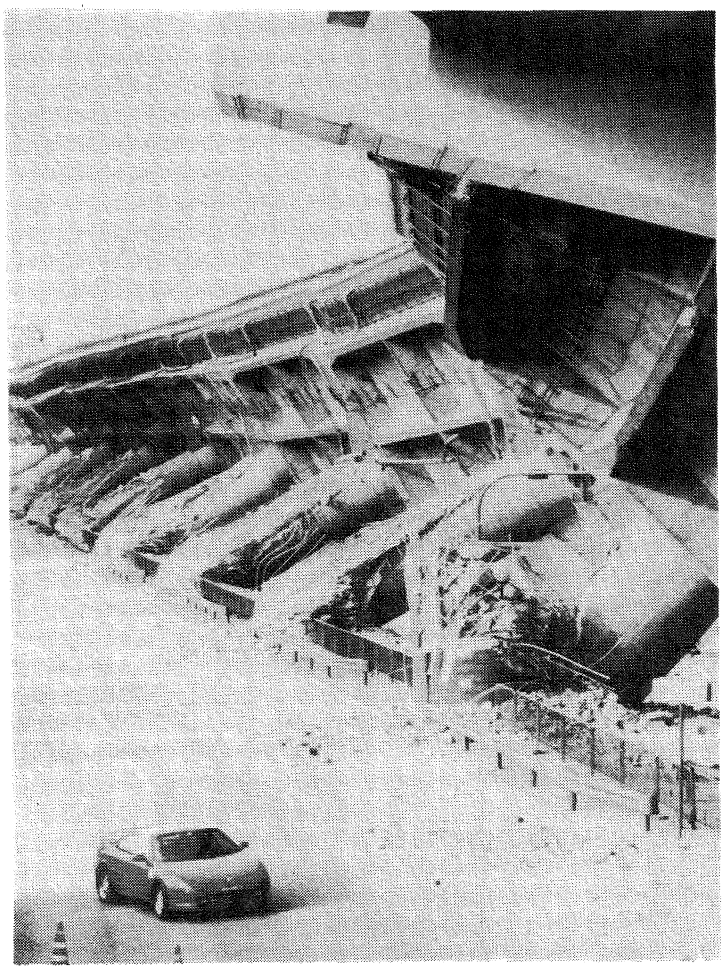


FIGURE 7.2 Part of the 600 m Collapsed Length of the Hanshin Expressway Near Ashiya

expressway, the external reinforcing plates split off the column. Two cases were observed where the steel box columns totally collapsed. One of these was a cantilever column supporting a hammer head. The other was in a pier comprising a steel box girder supported on two columns, one under the expressway and one on an outrigger column. The steel box column under the expressway collapsed, and the steel box girder was heavily damaged.

Major damage was sustained to the bearings, restrainers and road joints at many of the articulations in the bridges. The bridge bearings are fixed at selected piers with provision for small movements (due to temperature and shrinkage) at the adjacent piers. Restraint for lateral loads is provided by restraints incorporated within the bearings. At the movement bearings, many of which were steel/brass sliders, restraint was provided by steel plates around the bearings. Longitudinal steel links were also provided between the spans. In the areas of intense shaking the bridge came off the bearings at many piers, dropping 100 to 150 mm onto the pier cap. The longitudinal links also often sheared their connections. In general the bridge moved in a longitudinal direction when falling off the bearings, sometimes over many spans. A section near Nishinomiya was observed to have moved about 300 mm and is shown in Figure 7.5. Only in a few cases were significant transverse movements of spans observed, generally associated with ramp structures. The seating lengths provided on the piertops were relatively generous being in the order of 600 to 900 mm. This undoubtedly prevented much more extensive loss of spans. At one pier (see Figure 7.6) evidence of sliding movements between the girders and the pier cap of about 1 m could clearly be seen.

7.2.2 *Meishin Expressway*

The Meishin Expressway connects with the Hanshin Expressway at Nishinomiya, linking the Kobe area with Kyoto. The expressway was completed in 1972. A 3 km elevated section inspected near Nishinomiya comprised a continuous hollow slab concrete superstructure supported on reinforced concrete wall type piers and piled foundations. The wall piers were cantilevered, with bearings between the piertop and the superstructure. Span lengths were 21 m, with an expansion joint provided every 5 spans.

Damage to the structure was primarily shear failures of the wall piers, caused by longitudinal actions on the structure. The piers did not have any transverse steel and about half of the longitudinal reinforcement terminated in a line about two thirds of the way up the pier. The shear failures appeared to initiate at this point leading to splitting of the pier and the peeling back of the cover concrete and longitudinal reinforcement. Many piers had partially collapsed, in some cases by up to 1 m. The continuous superstructure prevented total collapse and major cracking in the concrete superstructure was observed at the settled piers. Details are shown in Figure 7.7.

The severe pier damage described was evident throughout the whole length of the structure observed.

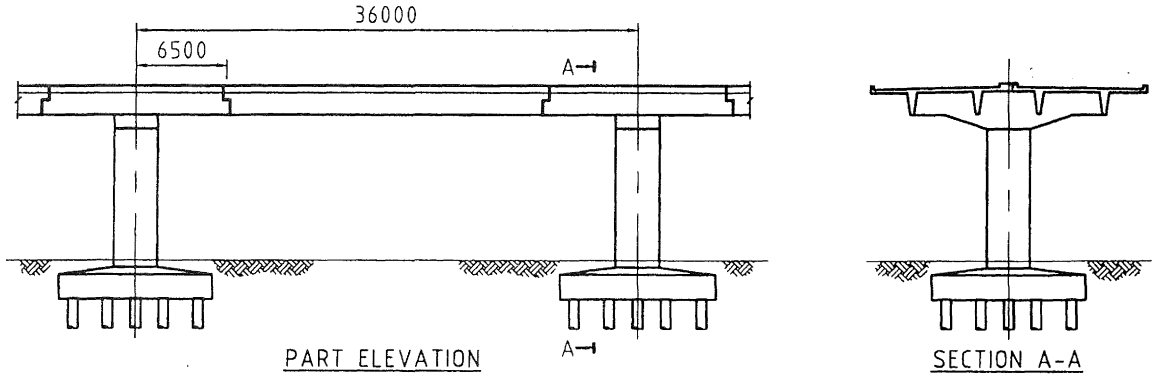


FIGURE 7.3 Elevation and Cross Section of the Hanshin Expressway Near Ashiya

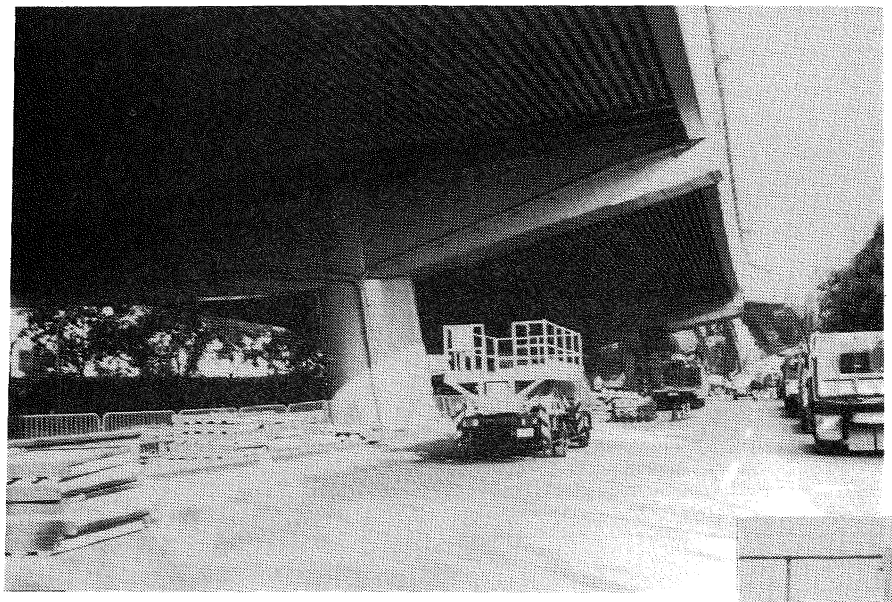


FIGURE 7.4 Example of Tilting of the Hanshin Expressway Due to Column Damage



FIGURE 7.5 Example of Longitudinal Displacement of the Hanshin Expressway

FIGURE 7.6 Example of Longitudinal Displacement of the Hanshin Expressway



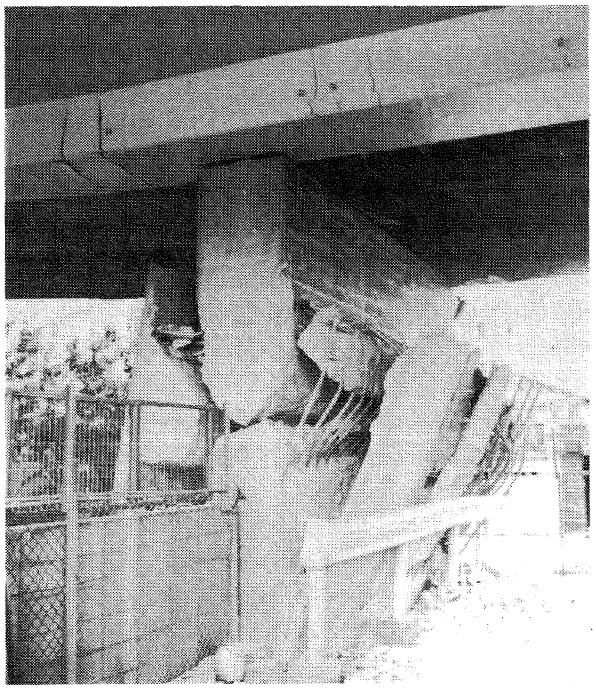


FIGURE 7.7 Example of Pier Damage of the Meishin Expressway

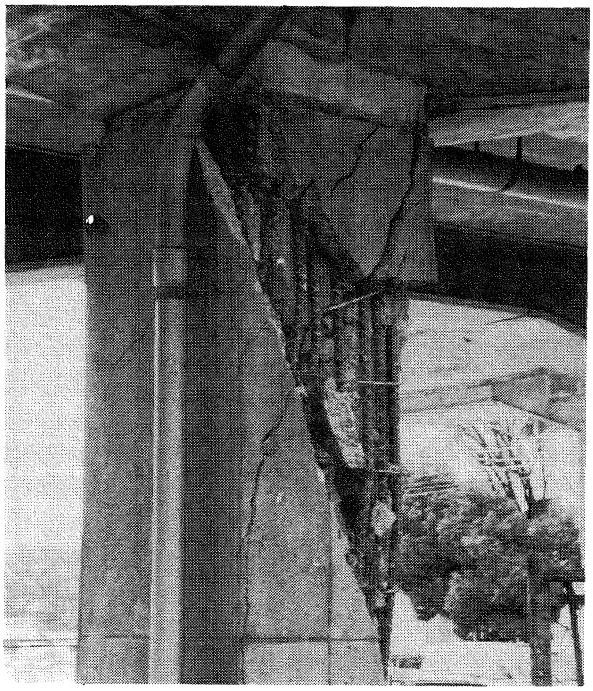


FIGURE 7.8 Example of Pier Damage of the Hankyo Rail Line

7.2.3 ***Hankyu, Hanshin and Sanyo Rail Lines***

Rail access to the area between Kobe and Nishinomiya is primarily provided by the government owned J R Tokaido Trunk Line and two private lines the Hankyu and Hanshin. Most of the lines are supported on elevated structures which suffered severe damage, cutting access between Kobe and Nishinomiya. The damage to the Hankyu line, which is typical of the damage suffered, is described below.

The Hankyu Line has two tracks supported on separate but abutting structures. Each bridge is supported by a continuous reinforced concrete girder haunched and monolithic at the columns. The 600 mm square columns are about 6 m high and spaced at about 6 m. Expansion joints are provided at every 7 to 8 bays. Three bridge sections (of 8 bays each) collapsed near Nishinomiya and several others collapsed elsewhere on the line. The collapses were either a gross sideways collapse of the bridge section or else rotation of the superstructure at the failed section at the top of the columns. The structure which remained standing suffered significant damage to the columns, primarily at their upper ends.

Observations indicate that the column damage primarily resulted from shear failures caused by longitudinal seismic actions on the structure. The longitudinal shears caused by the large flexural capacity of the columns, exceeded the capacity of the ties provided. The ties are light perimeter ties only and spaced at about 200 mm centres. The resulting lack of confinement led to rapid degradation at the tops of the columns, as shown in Figure 7.8.

On the J R Sanyo Line in the Kobe area a steel pier supporting a steel superstructure was observed to have failed. The pier comprised two circular steel columns approximately 500 mm diameter and 40 to 50 mm wall thickness, framing into a steel box cap beams. The cap beam had moved sideways (north) about 800 mm and the columns had fractured in a brittle manner just below the cap beam. Collapse was only prevented by the continuity of the superstructure. No other steel piers were observed under the rail bridges.

7.2.4 ***Shinkansen (Bullet Train) Line***

The Shinkansen runs in a tunnel through most of the Kobe area, emerging near Nishinomiya onto an elevated bridge. The structure was seriously damaged over about a 2 km length near the tunnel mouth. Collapses occurred at 5 places and many piers were damaged.

Except at river crossings, the two rail lines are carried on an monolithic reinforced concrete structure which comprises a concrete deck supported on two longitudinal girders haunched at the reinforced concrete columns. Transverse beams run between the two column lines at each bent. Joints are usually at midspan every 4 spans. The columns are about 600 mm square and at about 7 m centres.

Near the tunnel mouth the structure is supported on columns about 6 m high. Damage had occurred to the column tops, apparently due to longitudinal seismic actions and was similar to that described above for the Hankyu Line. Two of the three collapses on this section involved longer spans crossing roads and rail lines beneath. The piers supporting these longer spans were much stiffer and appeared to have attracted longitudinal seismic forces leading to their more rapid degradation and collapse.

About 1 km east of the tunnel mouth the piers became "two storey" with intermediate framing beams in both directions at the mid-height of the columns. A major collapse of several sections of the bridge had occurred when "soft storeys" developed as the columns failed in shear. Inspection of various elements indicated major shear cracks developed under longitudinal seismic actions, degradation of the column was then rapid as the very light column perimeter ties (at about 200 mm centres) lost their integrity. Details are shown in Figures 7.9 to 7.11.

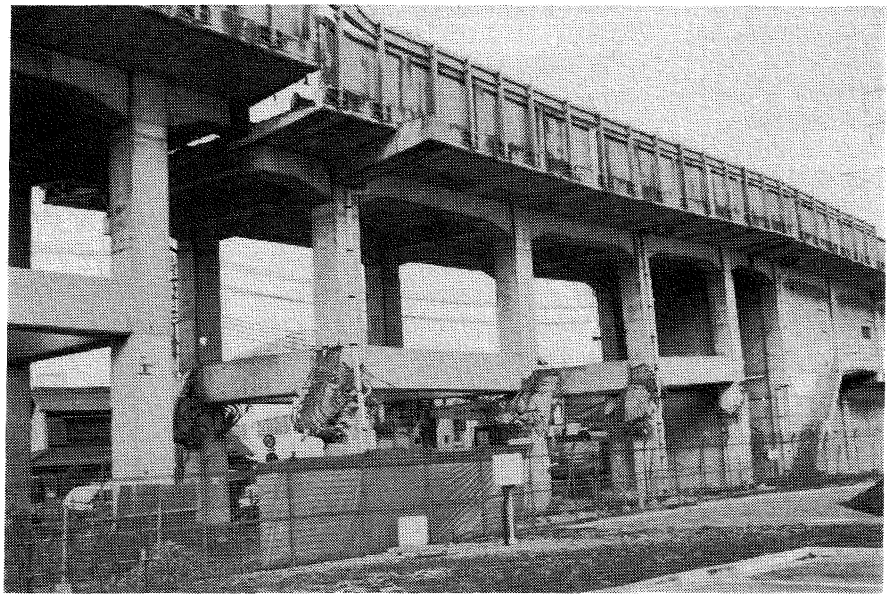


FIGURE 7.9 Example of Pier Damage of the Shinkansen Line

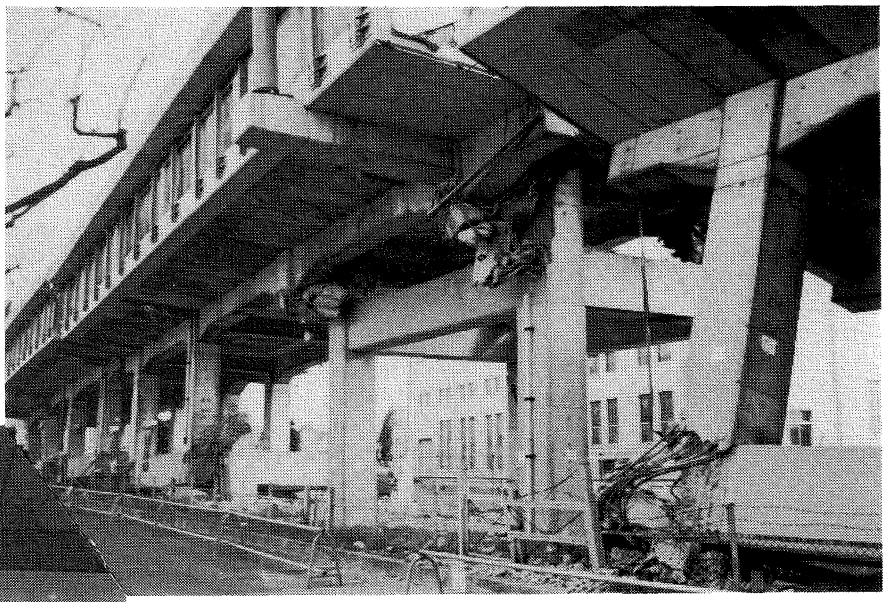


FIGURE 7.10 Example of Pier Damage of the Shinkansen Line



FIGURE 7.11 Example of Pier Damage of the Shinkansen Line

Further to the east at the Mukogawa River crossing the Shinkansen is supported on a single line of large reinforced concrete piers. These exhibited typical midspan flexural shear failures at the longitudinal steel terminations as shown in Figure 7.12. Although seriously damaged, they did not collapse.

7.2.5 *Wangan Expressway*

The Wangan Expressway was constructed between 1991 and April 1994 and links Rokko Island and the Southern Nishinomiya area with Osaka. This is a high elevated expressway with a number of major bridges spanning across shipping lanes beneath. The bridges in the area south and west of Nishinomiya included tied arch, steel box and cable stayed structures. The expressway traverses reclaimed land over all of its length.

The 5 lane Nishinomiya-Ko bridge is located on reclaimed land south of Nishinomiya. The eastern bridge pier supporting the 252 m tied arch main span and 52 m steel box girder approach span moved westwards at the top allowing the approach span to drop (see Figure 7.13). This effectively cut the route, but the Hanshin Expressway Public Corporation hopes to have the expressway operational again within 3 months. It was estimated the pier had moved about 400 mm at its top which was sufficient to allow the span to drop, even though a seating length of about 800 mm had been provided. The pier top movement was due to rotation and possibly some displacement of the pier apparently caused by the liquefaction and lateral spreading which occurred in the area (see Figure 7.14). The pier is located about 25 m from the reclamation edge. The edge of the reclamation suffered the significant lateral spreading movements and ground slumping similar to that observed throughout the port areas. Lateral spreading cracks extended back to the region of the pier. The reclamation in the area to the east showed evidence of liquefaction.

At the west end of the bridge the approach span had dropped about 75 mm below the main bridge span due to bearing damage sustained during the earthquake. At many bridges on the expressway in this area, settlements of 75 to 300 mm were observed where bearings had been damaged or lost and the bridge had dropped. Typical damage is shown in Figure 7.15. This generally severely damaged the road joints and would have hindered traffic on the expressway had it been operational.

A cable stay supporting the deck of the Nishinomiya-Ko Bridge from the arch above had failed at a lower anchorage during the earthquake but no other damage resulted.

The cable stayed Higashi-Kobe Bridge has three spans of 200/485/200 metres configuration, making it the 8th longest cable stayed bridge in the world. The two 3 lane decks are supported on a steel truss superstructure. The towers are steel boxes. The bridge and towers are instrumented with strong motion instruments and the data is awaited with interest. During the earthquake the links which hold the bridge vertically at the eastern end of the bridge (presumably against live load, wind or seismic uplift forces) fractured allowing the superstructure to rise about 0.6 m. This prevented access across the bridge.

One other noticeable area of damage was at the abutments of the on and off ramps where they met the reclaimed ground which had in many cases settled up to 300 mm. In most cases, as shown in Figure 7.16, access was still (with difficulty) possible, presumably due to the detailing of the abutments in these areas.

7.2.6 *Rokko-Ohashi Bridge*

The Rokko-Ohashi Bridge carries the main highway and the light rail system linking the industrial and residential areas on Rokko island with the Kobe area (see Figure 7.17). The bridge is of recent construction and has a cable stayed main span of about 250 m. The two 3 lane decks are supported on a steel truss superstructure. The towers are steel boxes supported on pinned connections at the piers. The approach spans are steel box girders with separate structures for the road and light rail lines.

The bridge suffered significant damage in the earthquake. At the southern pier of the main cable stayed span the bearing supporting the steel truss was seriously damaged due to differential movement between the truss superstructure and the pier (see Figure 7.18). The truss superstructure had moved about 250 mm to the north relative to the pier (along the bridge centreline) and the superstructure appeared to have settled by about the same amount due to the bearing damage sustained.

The cause of this movement and the state of the other bearings is not known at this stage. The differential movements did however seriously affect some of the services carried under the bridge (see Figure 7.17), causing their collapse into the sea below. One of these services is understood to be the watermain for Rokko Island.

At the Rokko Island (south) end of the crossing, a box girder span carrying the light rail collapsed severing the crossing between the mainland and the island (see Figure 7.19). The top of the steel box pier supporting the span appeared to have moved northwards from the reclamation. This pier was at the reclamation margin and appeared to have moved due to liquefaction and lateral spreading, in a similar fashion to the pier at the Nishinomiya-Ko Bridge. There was significant damage at the supports and joints of several other adjacent spans, apparently caused by the differential response of the widely varying structural systems used to support the spans. This may have contributed to the collapse noted.

7.2.7 *Port Island Light Rail System*

The Port Island light rail system sustained damage at the mainland side of the crossing. A significant section of the elevated line, carried on circular steel columns developed a lean to the west. The line runs on the east side of the reclamation which was observed to have spread laterally in the area. However the lean direction is inconsistent with the lateral spreading damage observed elsewhere. It is possible that failure of the piled foundation may have occurred.

Immediately adjacent is the Port Terminal station building and lateral spreading here, of about 0.5 m, caused the spans of the circular steel box girder access road bridge to collapse.

7.2.8 *Other Bridges*

Besides the elevated structure described above there are many smaller bridges in the area between Nishinomiya and Kobe. Many of these are single span or two span structures carrying the on grade roads in the area. The reconnaissance team found no instance where the bridges had suffered significant damage or become unoperational. In several cases movements of abutment side walls had occurred and some minor settlement of approach fills was observed.

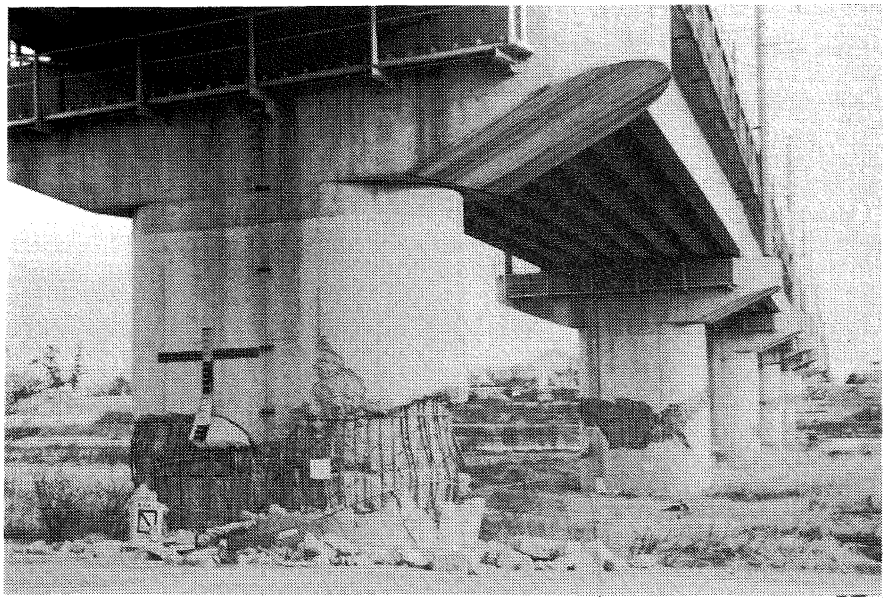


FIGURE 7.12 Example of Pier Damage of the Shinkansen Line

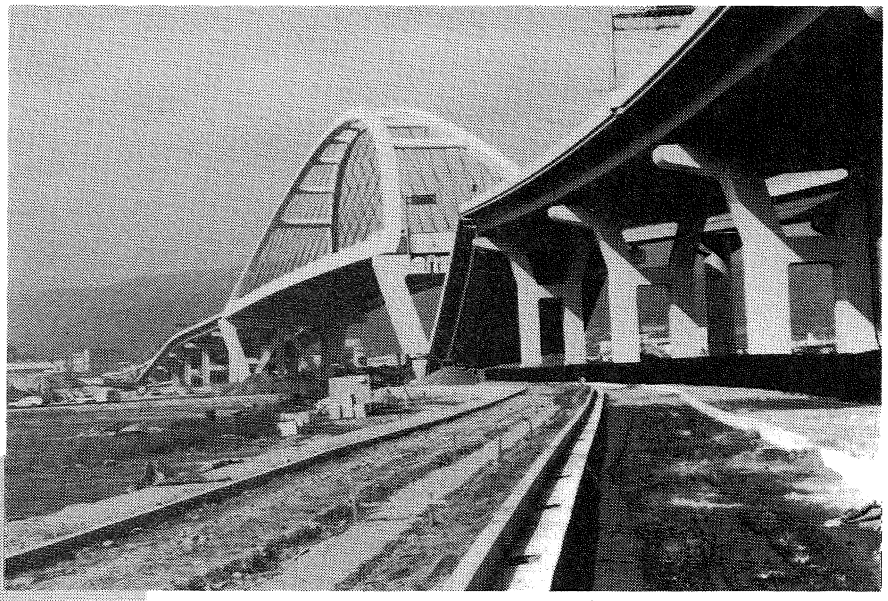


FIGURE 7.13 Loss of Approach Span of the Nishinomiya-ko Bridge

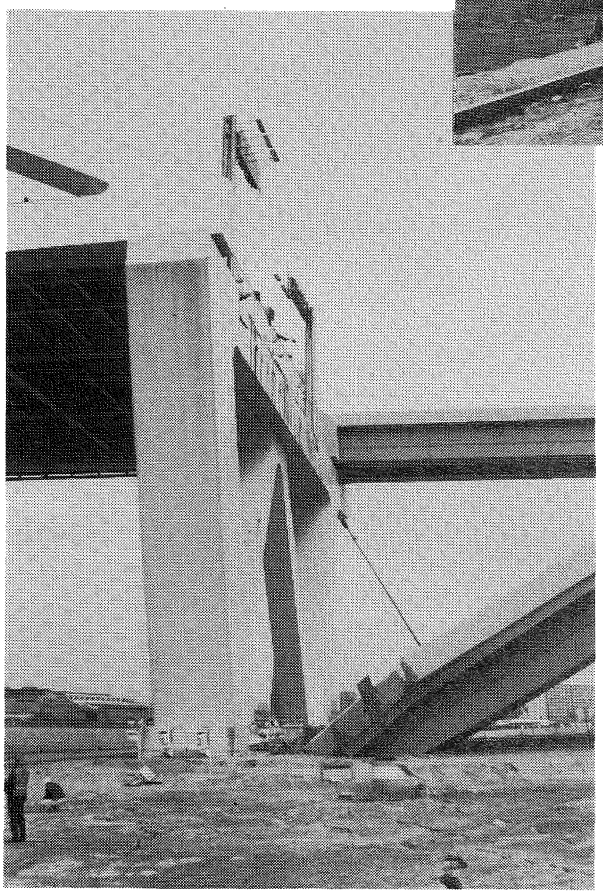


FIGURE 7.14 Pier Displacement of the Nishinomiya-Ko Bridge

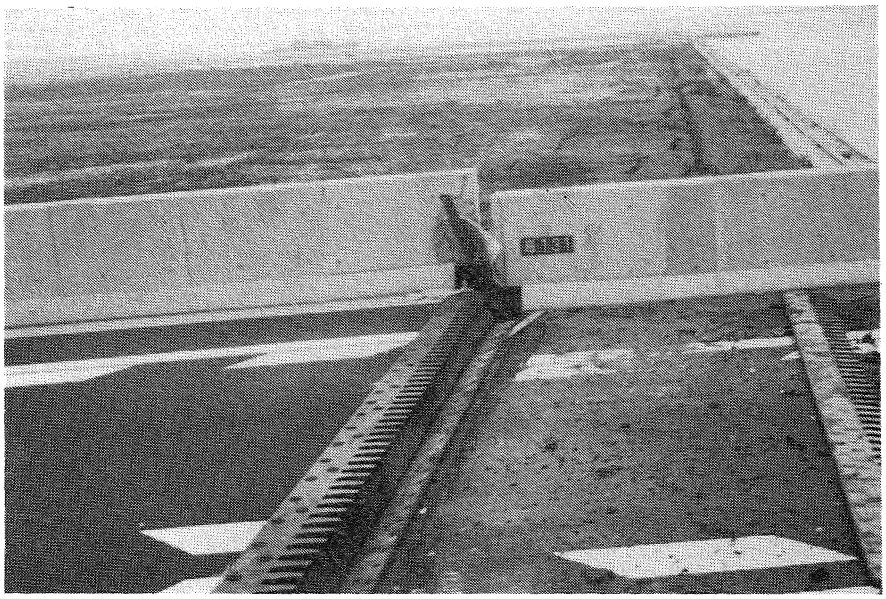


FIGURE 7.15 Example of Joint Damage of the Wangan Expressway

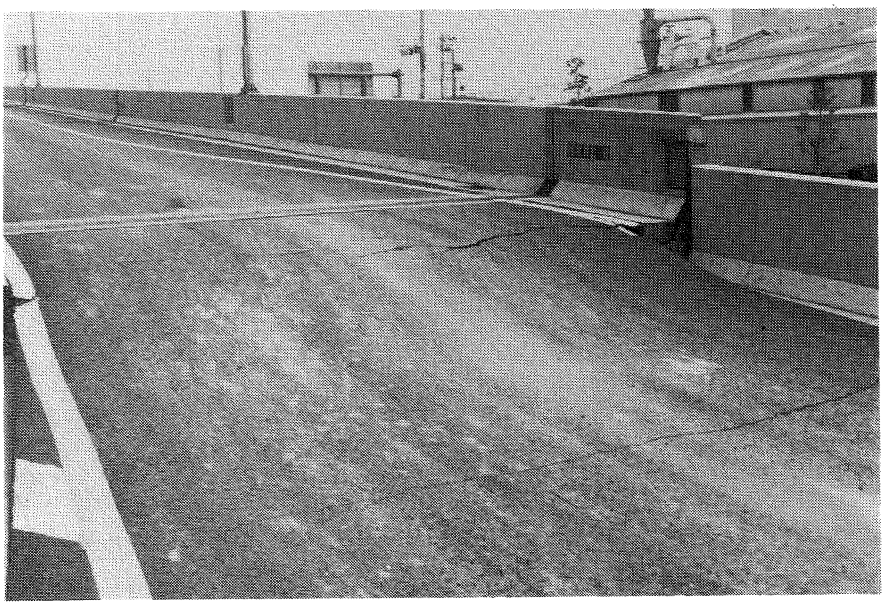


FIGURE 7.16 Example of Approach Settlement of the Wangan Expressway

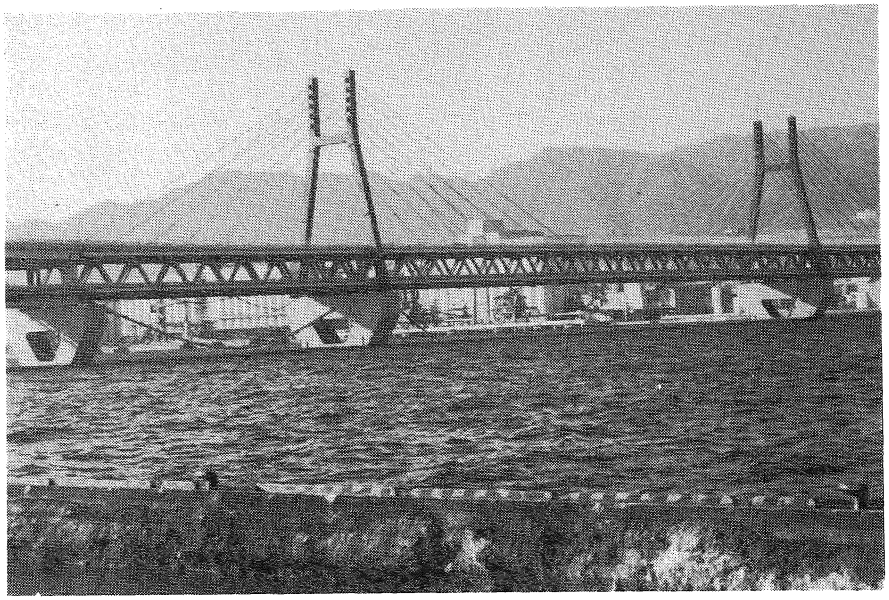


FIGURE 7.17 Rokko-Ohashi Bridge

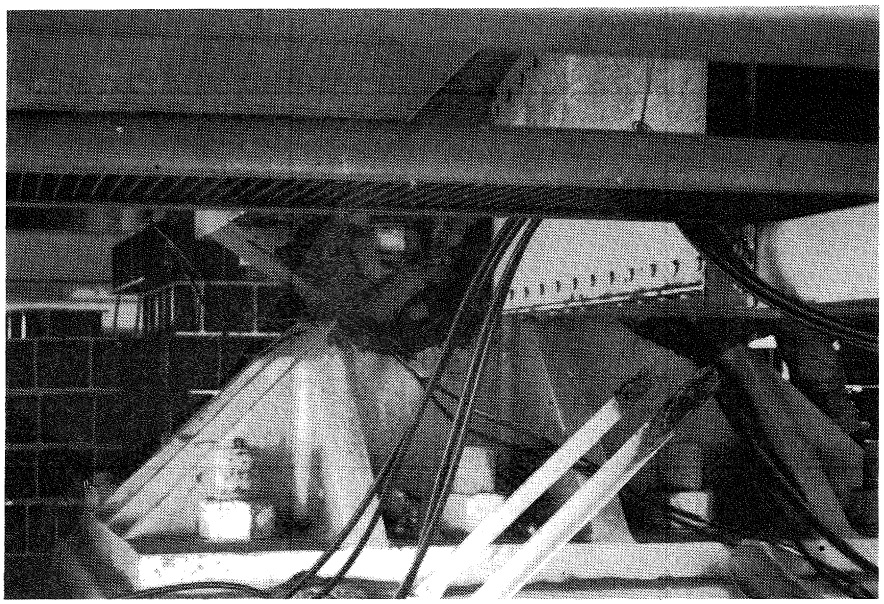


FIGURE 7.18 Bearing at South Pier of Rokko-Ohashi Bridge

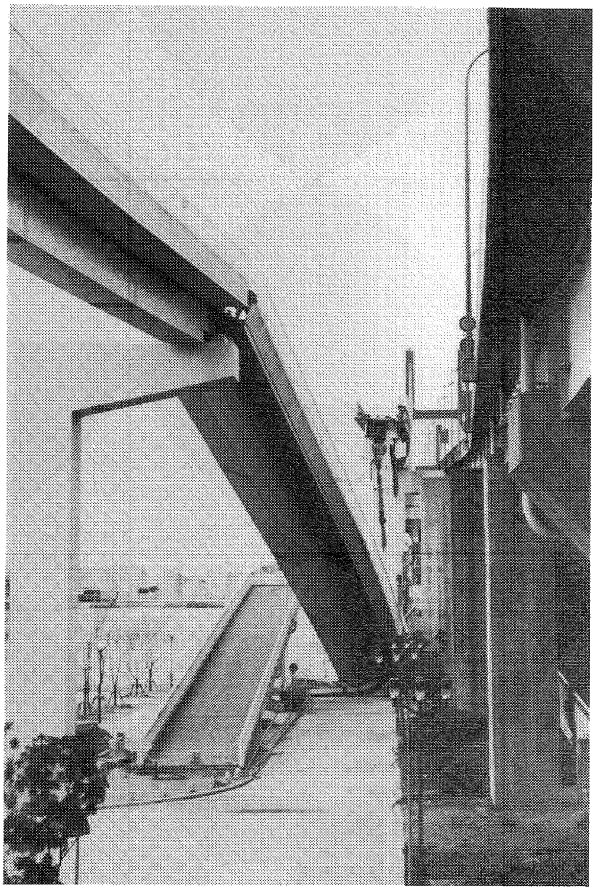


FIGURE 7.19 Span Loss of Rokko Island Light Rail, Rokko-Ohashi Bridge

7.3 Behaviour of Reinforced Concrete Piers

7.3.1 *Types of Pier Failure*

Several types of pier failure were observed, including:

(a) Tilting of piers due to movement in the foundations (see Figure 7.20a).

(b) Flexural failure of columns of piers at the base due to inadequate transverse reinforcement for concrete confinement and restraint of longitudinal bar against buckling (see Figure 7.20b).

(c) Flexural failure of columns of piers at section above the base due to longitudinal bar cut-off (see Figure 7.20c).

(d) Shear failure of pier columns due to inadequate transverse reinforcement (see Figures 20d and e).

(e) Shear failure of pier cap beams due to inadequate transverse reinforcement (see Figure 7.20f).

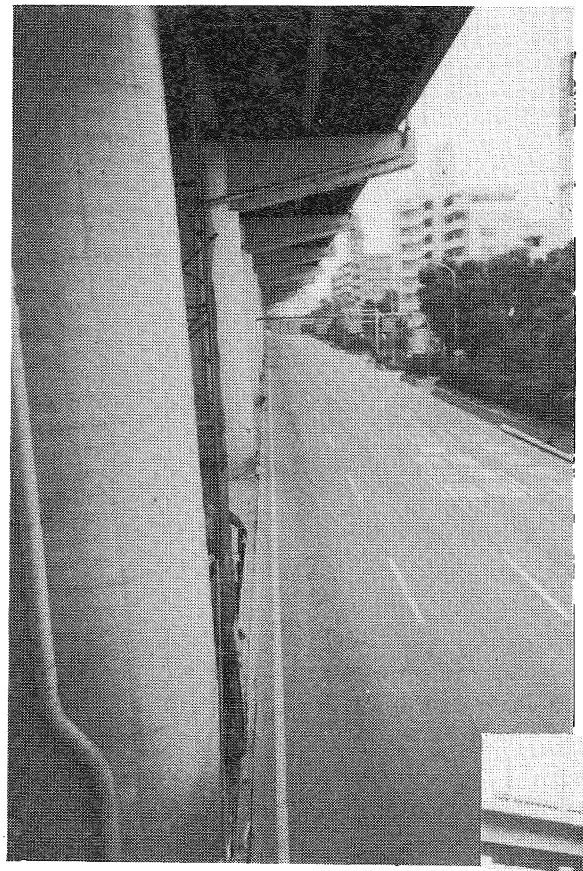
An example of cut-off of longitudinal bars is shown in Figure 7.21. Other causes of the above types of failure were the rupture of butt welded longitudinal bars at the weld (see Figure 7.22) and anchorage failure of transverse bars which were lapped without hooks or welding in the cover concrete (see Figure 7.23).

7.3.2 *Reinforced Concrete Piers of the Hanshin Expressway*

Of considerable interest was the collapse of a 600 m length of the Hanshin Expressway (see Figure 7.2). This length of expressway was constructed in the 1960s (opened in 1969) and consisted of single stem circular piers of 3.1 m diameter containing 35 mm diameter longitudinal bars in circular array in three concentric rings. The inner ring of longitudinal bars, containing one-third of the total area of longitudinal bars, were terminated (cut-off) at 2.5 m above the pile cap. The longitudinal bars in two outer rings extended the full height of the column. The 16 mm diameter circular hoops were at 400 mm vertical spacing around the inner ring. For each of the two outer rings the 16 mm diameter hoops were at 200 mm vertical spacing for the bottom 2.5 m height of column and 300 mm vertical spacing above, around each ring.

The piers had apparently been designed for horizontal seismic acceleration of 0.2g using working stress design with a permitted overstress of 50%. This means that the probable flexural strength of the pier would have been reached when the

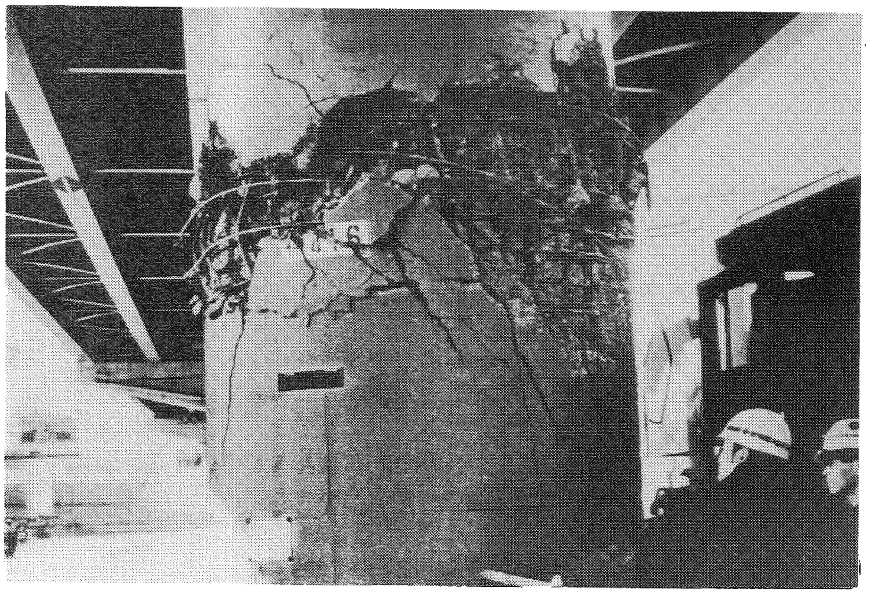
FIGURE 7.20 Types of Failure of Reinforced Concrete Piers



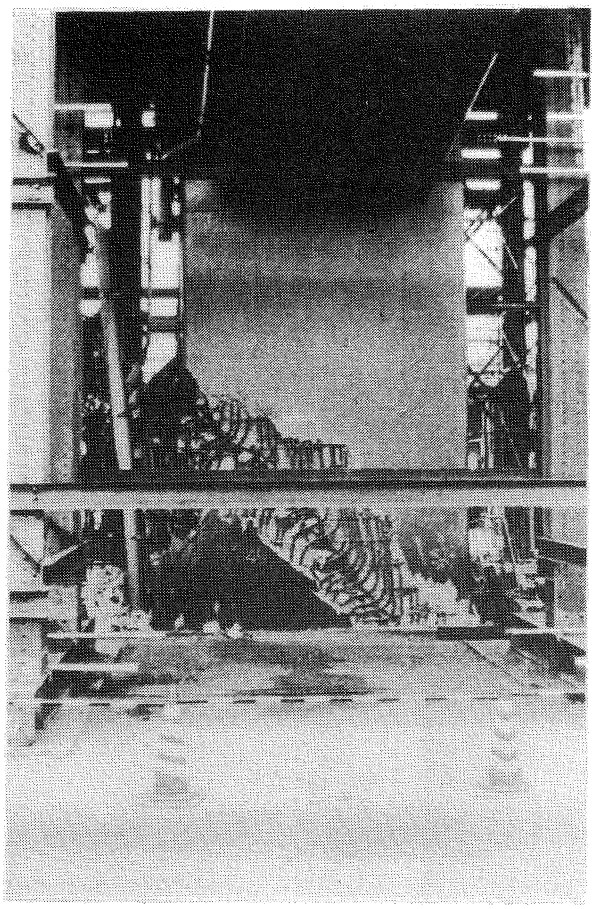
(a) Tilting of Piers due to Foundation Movement

(b) Flexural Failure of Column of Pier at Section of Maximum Bending Moment

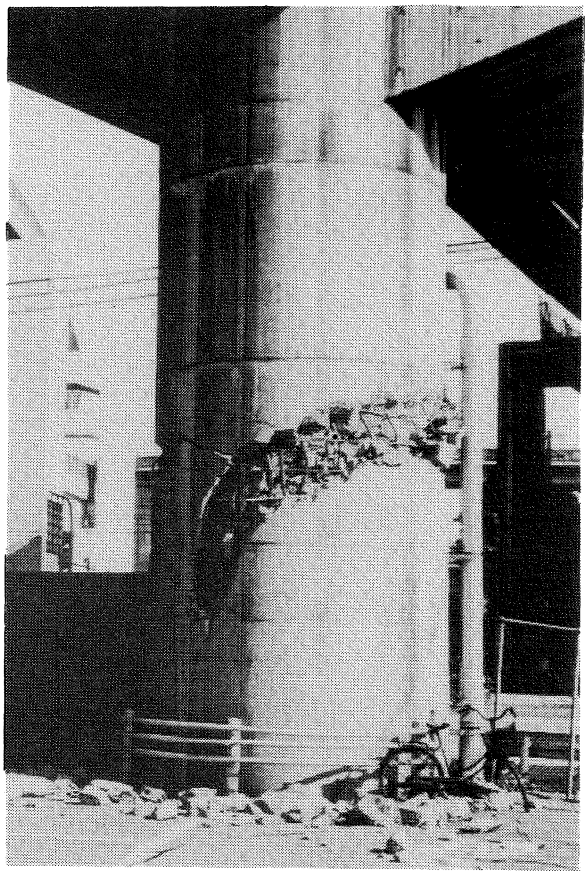




(c) Flexural Failure of Columns of Pier at Section of Cut-Off of Vertical Bars



(d) Shear Failure of Column in Region of Section of Maximum Bending Moment



(e) Shear Failure of Column of Pier at Region of Cut-off Of Vertical Bars



(f) Shear Failure of Cap Beam of Pier

FIGURE 7.20 Types of Failure of Reinforced Concrete Piers (continued)

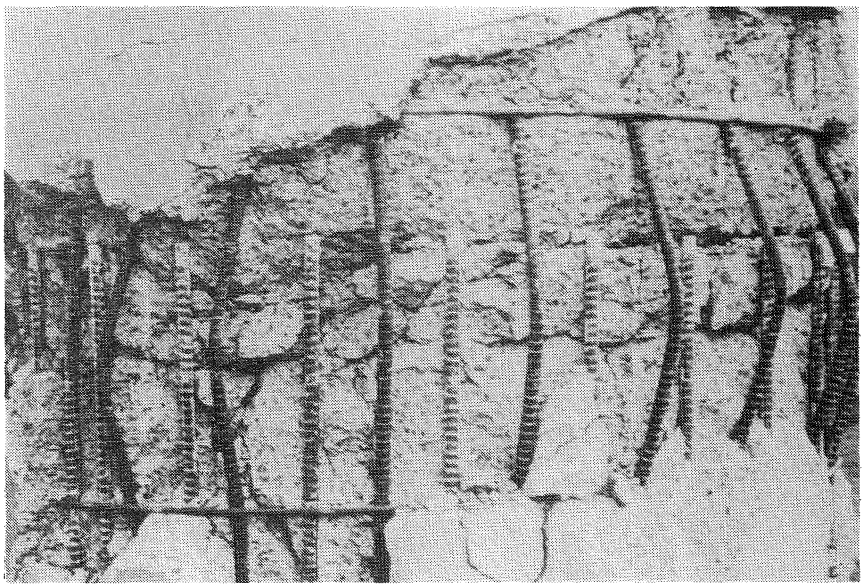


FIGURE 7.21 Cut-off of Longitudinal Bars

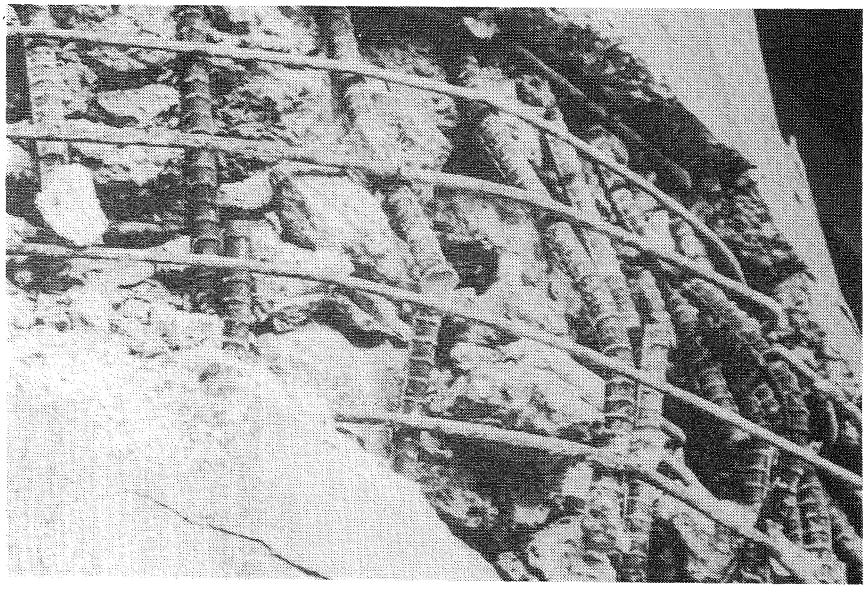


FIGURE 7.22 Rupture of Butt Welded Longitudinal Bars and Cut-Off of Longitudinal Bars

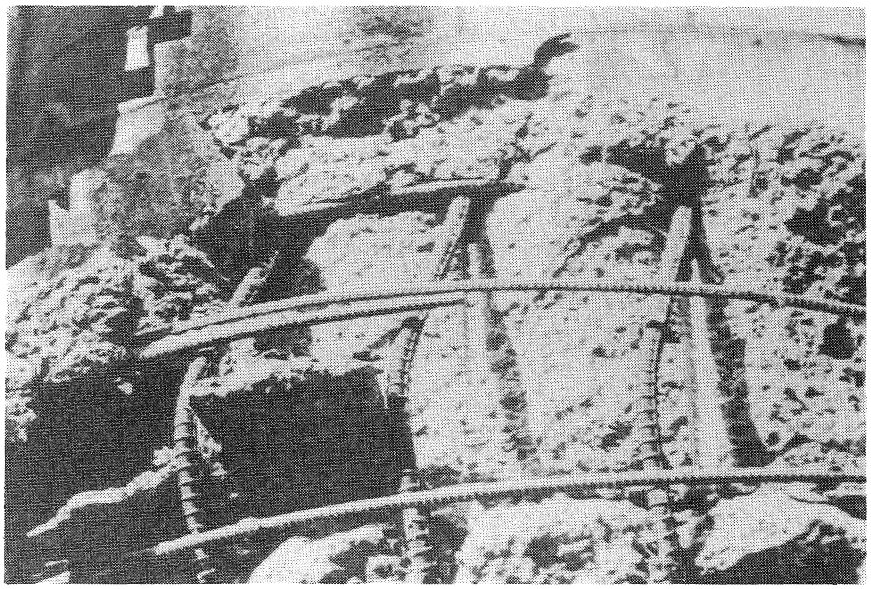


FIGURE 7.23 Transverse Hoops Bars Lapped in Cover Concrete

horizontal seismic acceleration was at least 0.3g. Hence the peak horizontal ground acceleration of about 0.8g was 0.8/0.3 = 2.7 times the available strength, thus requiring the structure to have an available structure ductility of at least 3.

The failure of the reinforced concrete columns (see Figure 7.2) was evidently due to a combination of circumstances:

(a) Lack of adequate transverse reinforcement for confinement, for restraint of longitudinal buckling against buckling and for shear resistance due to:

- The small quantity of the 16 mm diameter circular hoop reinforcement present. The circular hoop reinforcement at the bottom of the column was not capable of resisting the applied shear force by a 45° truss mechanism when yielding, leaving a substantial shear force to be resisted by the concrete mechanisms of aggregate interlock, dowel action and across the compression zone.

- Poor anchorage of the circular hoop reinforcement. The circular hoop bars were lap spliced, without hooks or welding in the cover concrete, and anchorage would have been lost when the cover concrete spalled.

(b) Poor performance of the 35 mm diameter longitudinal reinforcement due to:

- Butt welds of the reinforcement which in many cases fractured in a brittle manner. The butt welds were observed to be not staggered but to be all at the same level.

- The termination of about one-third of the longitudinal reinforcement at about 2.5 m above the pile cap. This cut-off apparently followed the bending moment diagram without allowance for the spread of steel yielding up the column due to diagonal tension cracking (that is, the Tjd effect), leading to severe cracking due to yielding of steel at the cut-off section 2.5 m above the pile cap.

The failure of the columns would probably have involved the following sequence of events:

1 Flexural and diagonal tension cracking occurring during the acceleration pulses in each direction leading to crushing and spalling of cover concrete and to the spread of yielding of the longitudinal reinforcement up the column, and to degradation of the concrete shear resisting mechanisms, then to

2 Widening of diagonal tension cracks due to inadequate shear resistance from the hoops particularly when anchorage of the lap spliced hoops was lost due to crushing of cover concrete and to yielding at sections of cut-off of longitudinal bars, and finally to

3 Failure in shear made particularly brittle by the failure of the butt welds of the longitudinal bars at the top of the inclined crack (see Figure 7.2).

Other bridge structures (rail and road) also failed mainly as a result of inadequate transverse reinforcement in columns which resulted in shear failure.

Shear failure of the columns of the piers could have been prevented had the design shear force been calculated by capacity design as that associated with flexural overstrength of the column, and had the amount of shear reinforcement been calculated conservatively as that needed to resist most, if not all, of the applied shear force by a 45° truss mechanism, and had the circular hoops been properly anchored by hooks or welding.

7.4 **Behaviour of Structural Steel Piers**

7.4.1 *Nature of Construction*

The structural steel bridge piers observed consisted of a thin-plate box steel construction, with internal stiffeners. These sections were shop fabricated into assemblages, with site bolted splices, used to connect the subassemblagers into piers. The box steel piers were used in conjunction with a steel deck, either plate girder at box girder. An example of this single column box pier construction is shown in Figure 7.24.

The slenderness ratio of the plate elements used in the box column walls typically varied from 100 to 160.

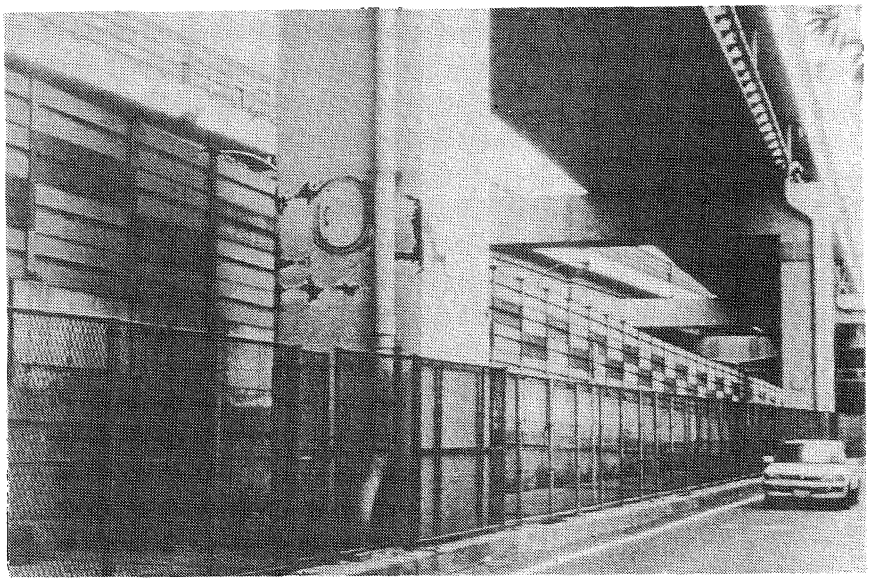


FIGURE 7.24 Local Buckling of Plate of Steel Column Near Koshien Station

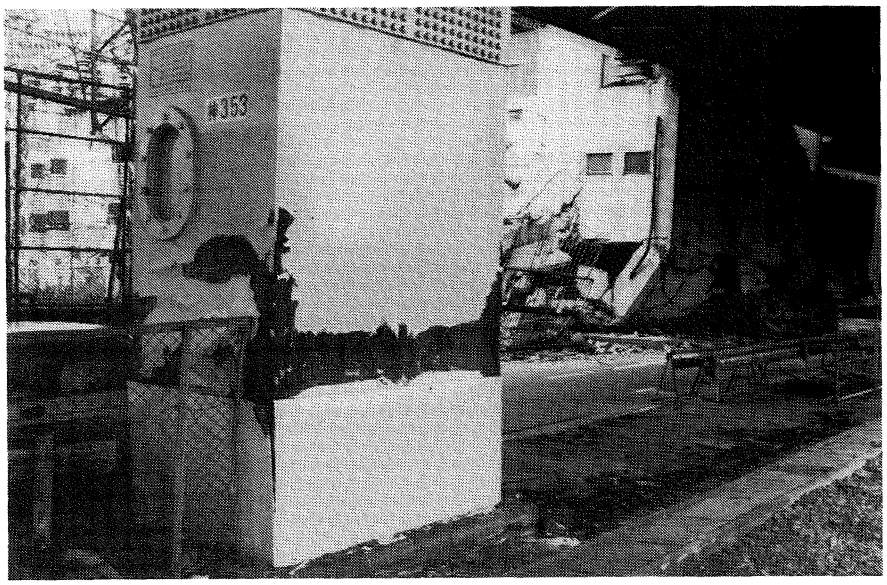


FIGURE7.25 Damaged Steel Box Column Pier Showing Partial Separation of Outer Plate Near Base

Both single and double column (H-type) piers were rigidly connected into a pier cap founded on piles. All structural plates up to 70 mm thick are hot rolled at a mill. A plate thickness of 15 mm was typically used in these steel box walls. When there is the possibility of traffic collision against the column a concrete barrier is placed in and around the column as an absorber of energy. However, when space is limited, a sacrificial steel plate about 9 mm thick is installed outside the column instead. An example of this is shown in Figure 7.25.

7.4.2 *General Behaviour*

The great majority of steel box column piers performed well. A few piers were found with minor local buckling at the points of maximum compression stress and maximum wall slenderness. This occurred typically above the junction of the main plate with the additional plate layer around the base of the pier.

Several piers on the Hanshin Expressway suffered greater damage, one of which is shown in Figure 7.25. The localised damage in this instance was due to the lateral load induced by the soft-storey collapse of an adjacent older reinforced concrete building into the deck of the expressway. The inelastic demand was sufficient to partly separate the outer plate from the main box pier construction.

Two steel box piers on the Hanshin expressway were observed which had suffered a complete collapse, presumably due to local buckling of the pier.

In the H-type piers of the Wangan Expressway at Port Island one example of local shear buckling in the web of a steel box intermediate beam was observed. This buckling was of visual but not structural significance. This type of box pier construction is being used in modern elevated road and rail system construction in the Kobe area. All the modern (post-1980) piers observed had performed well.

In all instances the site bolted splices performed very well, showing minor slip only in the cases of most significant overall damage to the older box piers.

As noted in Section 7.2.3 a brittle failure of two columns in a portal frame pier supporting the J R Sanyo railway was observed. This occurred in a transition section between the box beam and the circular hollow column. The section in question was probably a cast steel piece and it is possible that failure of this section was due to it being below the transition temperature at the time of the earthquake. The fracture caused the pier and deck system to move laterally over about 800 mm.

7.5 **Behaviour of Bearings, Restrainers and Joints**

A large amount of damage was sustained to the bridge bearings, seismic restrainers and road joints at articulations. The actual seating lengths provided at articulations were usually quite generous and although many spans fell off bearings, total collapse was prevented. All bearings observed had relatively small movement capabilities. However generally restrainers were provided to retain the bridge on the bearings. In general the restrainers were incorporated with the bearings and appeared quite robust. Nevertheless at a significant number of piers they were unable to cope with the seismic forces required to develop the ultimate capacity of the supporting piers beneath.

Longitudinal restrainers in the form of flat steel link bars with fixed or sliding pins were provided between spans on the elevated expressways. While many had survived, a significant number had broken, generally by shearing the connecting pins, again indicating the significant longitudinal actions occurring. As a result of bearing and restrainer damage, the road joints were seriously damaged at many locations.

7.6 **Liquefaction**

Along the coastal strip widespread evidence of liquefaction, under and adjacent to bridges was observed, from the area south of Kobe to south of Nishinomiya.

In general, in spite of the large areas of liquefaction relatively little damage to the bridge structures was observed. For bridge piers located at the margins of the reclamations, the lateral spreading and settlements in these areas appeared to have resulted in permanent seaward displacements of some bridge

piertops. The span collapses noted above on the Nishinomiya-Ko and Rokko-Ohashi bridges are believed to have been primarily caused by this effect. Detailed measurements will be needed to confirm this.

For bridge piers located well away from the margins of the reclamations on Port Island the ground had settled by 300 to 500 mm around the pilecaps of typical bridge piers without obvious distress to the bridge.

Ground damage associated with bridge structures commonly observed was that of settlement of approach embankments. In areas of reclaimed land settlements of up to 500 mm were observed with 300 mm being common (see Figure 7.16). While the settlements resulted in a step up to the bridge structure there was generally no evidence of significant structural damage. In many cases emergency ramps had been constructed to re-establish access over bridges. Earthquake induced bridge abutment settlement is of short term inconvenience but provided the overall integrity of the approach is retained the bridges remain trafficable.

7.7 Ground Displacement Effects

The behaviour exhibited by the long elevated structures indicates that longitudinal seismic actions played an important part in their performance. In the majority of cases where major damage occurred to long elevated bridges it appeared to be due to effects along the bridge. Examples are: longitudinal shear initiated failures of the concrete piers on all the rail bridges and on the Hanshin Expressway; longitudinal seismic damage to steel box piers on the Hanshin Expressway; and displacement of steel spans on the Hanshin Expressway by about 300 mm longitudinally at piers where bearings and restrainers had failed. In all cases the damage appeared consistent with displacements being applied which were much greater than the strength or displacement capabilities of the components. However similar levels of damage in the transverse direction were not observed. For example at the "two storey" Shinkansen structure shear failures were observed from longitudinal actions on the columns and longitudinal beams, but not from the transverse actions.

It can therefore be concluded that the peak forces on these long elevated bridges resulted from non synchronous longitudinal ground displacement effects rather than the synchronous response of the structure to ground shaking. That is, the longitudinal ground displacement effects are caused by out of phase displacements which occur as the seismic waves pass along the structure.

The relatively stiff piers on the rail bridges and Hanshin Expressway apparently did not have sufficient ductility to sustain these movements, even though they may have had sufficient capacity to carry the forces which would result from synchronous response (that is, all the piers in phase).

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8 INDUSTRIAL FACILITIES

8.1 General

8.1.1 *Introduction*

The Kobe area is an important industrial area for Japan. Industries were severely affected by the earthquake. A range of industries from heavy to light operated in the area including heavy steel industries, petrochemical refineries, motor vehicle parts production, electronics, synthetic shoes, sake brewing, clothing and pearl industries. From the area west of Kobe city through to Nishinomiya it appeared that most industries had ceased operation and many would be out of production for some time. The primary reasons for the disruption were:

- Outage of electricity, water, gas and sewerage systems
- Damage to plant, building, materials or product
- Unavailability of labour force due to domestic or transportation difficulties
- Major disruption of the land transportation system, which prevented movement of materials, products and people.
- Major damage to the port, cutting all sea transportation of goods

Except for the port areas, the reconnaissance team was generally unable to enter industries themselves. Information was obtained from local newspapers, local interviews, local contacts and "over-the-fence" inspections. Reported briefly, below are some typical examples of the earthquake effects noted.

8.1.2 *Steel Industries*

Kawasaki Steel, Kobe Steel and Mitsubishi Heavy Industries plants in the Kobe area were closed by the earthquake. Kawasaki Steel and Kobe Steel have older plants located in the port area south east and near to Kobe city. Damage observed to these plants included collapse and damage to old masonry buildings and masonry walls, damage to steel cross braced buildings and damage to services. Adjacent to Kobe Steel, a 3-storey electrical substation had suffered a soft storey collapse to the lower floor and collapsed against the adjacent expressway (see Figure 8.1). It was reported that it was uneconomical to reopen the plants and that they would be closed permanently and the valuable sites sold. Kobe Steel reported a week after the earthquake that out of 19,000 employees in the Kobe area 15,000 were affected and 8,000 had not yet made contact.

8.1.3 *Electronics*

Most Japanese-produced active displays for laptop computers originate in the Kobe area. Makes of displays for Fujitsu, Sanyo, Sharp, IBM and Toshiba were reported to have temporarily suspended production due to plant damage. Production of the screens requires pristine manufacturing conditions.

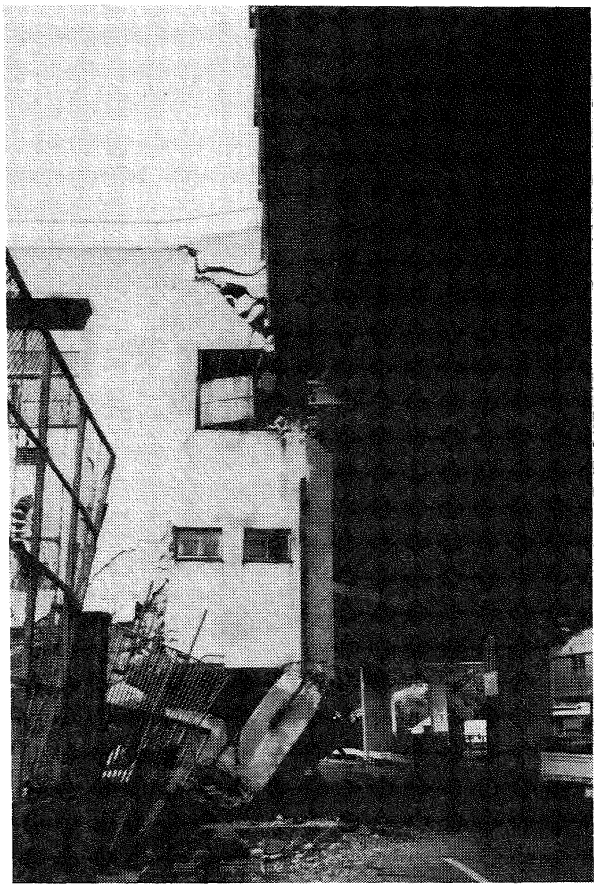


FIGURE 8.1 Near Kobe Port. Collapse of a Three Storey Substation Against Expressway

Hoshiden's Kobe liquid crystal display factory was reported closed until gas and water were restored.

8.1.4 *Synthetic Shoes*

85% of Japanese production of synthetic shoes is in the Kobe area. Many are small businesses located in the fire damaged areas west of Kobe city. Two thirds were severely damaged and 10 days after the earthquake the few who decided to restart reported that power was still out and their employees had not yet shown up.

8.1.5 *Sake*

Prior to the earthquake the Kobe area produced one third of Japanese sake, mostly in older traditional style wooden buildings with clay tile roofs. Of the 31 breweries, 17 were destroyed and the rest severely damaged and inoperable after the earthquake.

8.1.6 *Automotive Industries*

A substantial volume of automotive parts are produced in the Kobe/Osaka region, generally for assembly plants outside the region. Parts produced in Osaka/Kansai area would normally be transported through Kobe. The disruption to supply directly affected all the major automakers in Japan because of their "just-in-time" policy of keeping inventories to a minimum. Effects ranged from closure or major disruption in the week following the earthquake. For example Toyota reported plant closures due to parts procurement and distribution difficulties would result in a production shortfall of 20,000 vehicles. Mazda reported halting production at Hiroshima and Hofu plants due to severed supply routes. However, 10 days after the earthquake they reported supplies had resumed through alternative routes.

8.1.7 *Port Areas*

Damage to the port areas was particularly severe and is reported on in Section 8.2 below. Major damage to two large concrete chimneys was observed. One on Rokko Island was in danger of collapse (see Figure 8.2) and other near the Kobe Steel plant described above had collapsed. Both were built on reclaimed land. A number of tanks were observed and this is reported in Section 8.4.

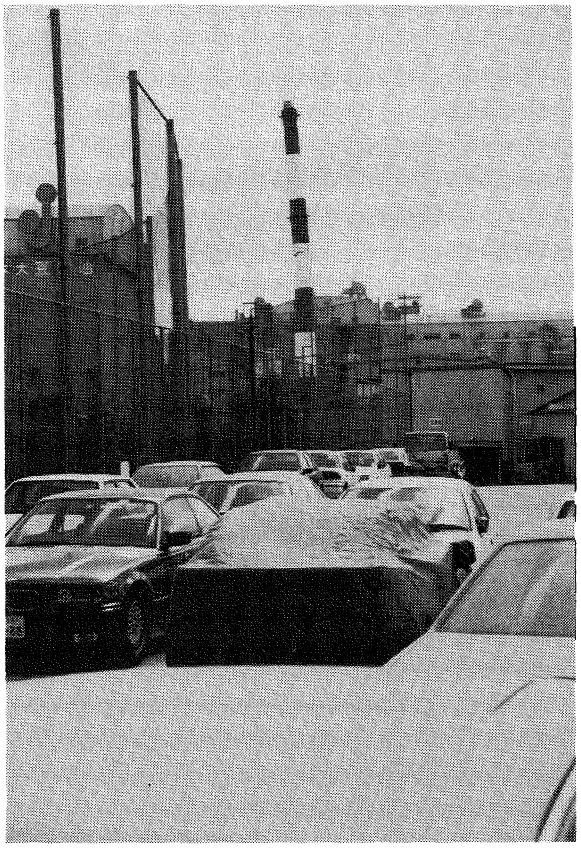


FIGURE 8.2 Rokko Island. Broken Chimney

8.2 Port Facilities

8.2.1 *Introduction*

The Port of Kobe is the sixth largest in the world and the second largest in Japan, handling 30% of Japanese trade volume. In 1990 171,465,111 tonnes of cargo was handled and in terms of containerised cargo the port is the leading Japanese port. Before the earthquake 20% of Kobe's workforce was employed at the port or its off shoots.

The earthquake caused massive damage to the port and to all facilities adjacent to the seafront, closing the port to all traffic except a small number of vessels bringing in relief supplies. The port damage will severely affect the Kobe area for some time to come. Further information on the Port is given in Section 4.3.

8.2.2 *Description of Facilities*

The Osaka Bay region is characterised by extensive areas of reclaimed land which flank the perimeter from Kobe in the west to Kansai in the east. In addition to the multitude of "smaller" reclaimed areas the bay is dominated on the western side by Port Island and Rokko Island, both of which are operated by the Port of Kobe, and by the new Kansai airport island on the eastern side

The scale of reclamations is huge. Port Island, some 436 ha, was completed in 1981, has a resident population of 20,000, and is presently being extended by a further 390 ha. Rokko Island, some 580 ha, was completed in 1992 and has a resident population potential of 30,000. Reclamations have involved excavation of fill from the mountains to the west of Kobe, to form housing estates, in a "mountains to the sea" project. Both are comprehensive developments which, while primarily created for port operations, include numerous apartment buildings, offices and hotels. The residential areas are located at the centre of the islands with the industrial and then port areas around the perimeter. Hospital facilities and amusement parks are also on the islands. A domestic airport is proposed as a third stage attached to Port Island.

The port of Kobe has continued to develop since it was formally reopened to international trade in 1868. The older reclamations and port areas abut the mainland while the newer reclamations comprise offshore islands. The port facilities for conventional or special cargo ships are generally on the mainland with the container terminals on Rokko and Port Islands.

Typically the areas of reclamation around the Osaka Bay area are contained within gravity retaining wall structures. Other than minor access jetties, no piled wharf structures were observed. The two recent Rokko and Port Island reclamations are contained within gravity caissons (see Figure 8.3). Soft marine clay sediments which underlie the bay sands have been excavated and replaced to avoid excessive consolidation settlements. No information was available at the time of the visit of the reconnaissance team to ascertain the actual construction procedure adopted or the nature of any ground improvement, if any. While the reclamation fill was sourced from the Rokko Mountains it is recognised that the weathered granite is not durable and during the construction process much of the material degraded to a sandy gravel/gravelly sand. It is understood that sand piles were adopted in the central Rokko Island area to accelerate consolidation and there was some densification of the overlying reclamation fill materials.

8.2.3 *Earthquake Damage*

In the port areas on the Kobe side of Osaka Bay, earthquake damage to the margins of the reclamations and anything located in these areas was especially severe. On the two reclaimed islands, 110 containers fell into the sea and all the container cranes could not be operated due to track misalignment, derailment, damage or in some cases collapse (see Section 8.3).

Typical damage at the margins of the reclamations was wide spread displacement and rotation of the perimeter retaining structures (see Figures 8.4 to 8.8). This damage was associated with widespread evidence of liquefaction in the form of ejected sand and lateral spreading. Typically at the older reclamations, retaining structure displacements of 1 to 3 m and settlements of the order of 1 m were observed, with well developed graben features formed behind them. Associated lateral spreading extended behind the wharf perimeter for 100 to 150 m usually resulting in severe damage to adjacent sheds and port facilities.

The main container port facilities are located on Port and Rokko Islands. The caissons which retain the islands typically experienced displacements of 2 to 4 m, settlements of 1 metre and rotations of 4 to 7 degrees (see Figure 8.4). Associated with the caisson movements were well developed graben features behind (1.5 to 3 m deep) and lateral spreading was severe 40 to 70 m beyond the face of the wharf (cracks up to 100 mm wide), with evidence of spreading displacements 100 to 150 m beyond the wharf face. In general displacements were quite uniform with the face alignment remaining relatively linear. No caisson structures were observed to have totally collapsed. Damage to a typical warehouse caused by lateral

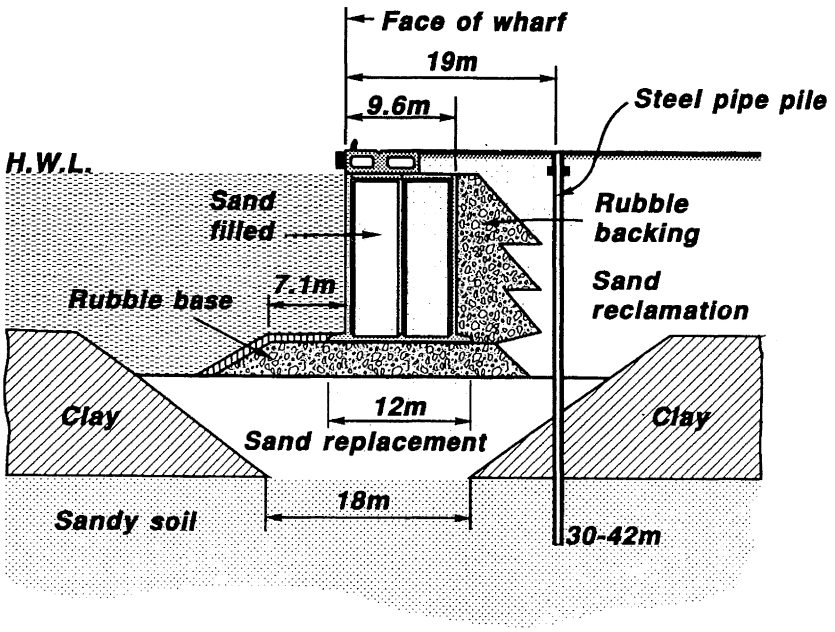


FIGURE 8.3 Port Island Caisson Arrangement. Standard Cross Section of the Container Wharf

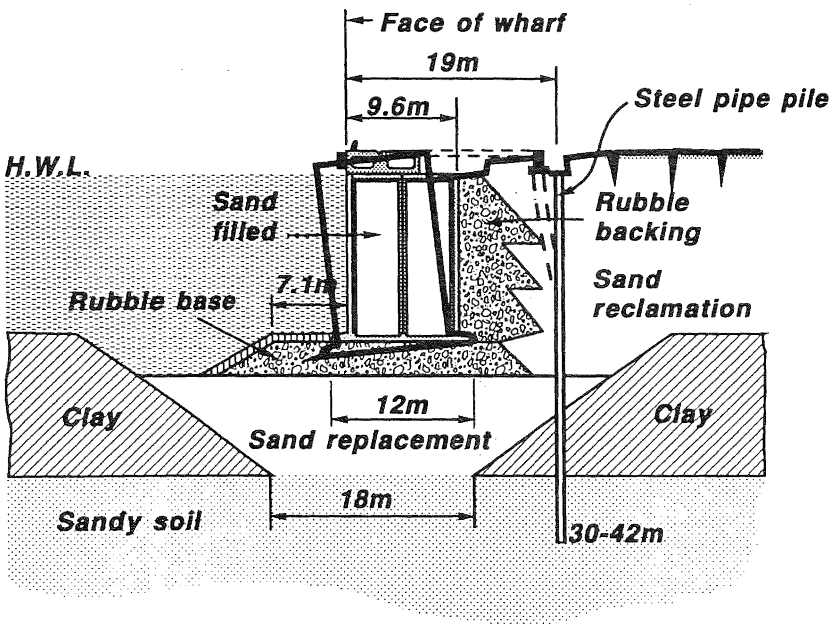


FIGURE 8.4 Port Island Caisson Cross Section. Post-Earthquake Displacement

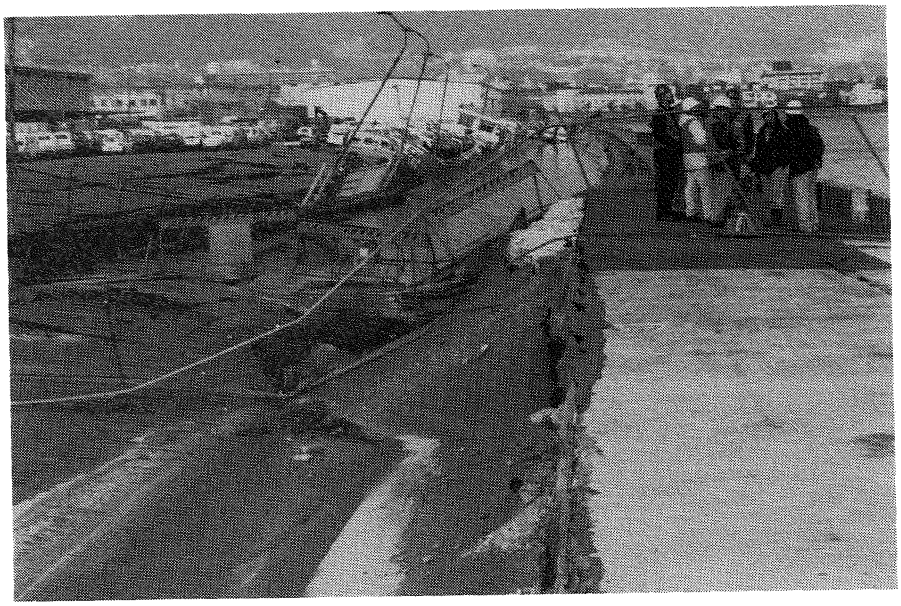


FIGURE 8.5 Near Higashi-Kobe Bridge, Wangan Route, Hanshin Expressway. Seawall Displacement and Associated Damage



FIGURE 8.6 Near Higashi-Kobe Bridge, Wangan Route, Hanshin Expressway. Ground Damage 100 m Behind Seawall of Figure 8.5

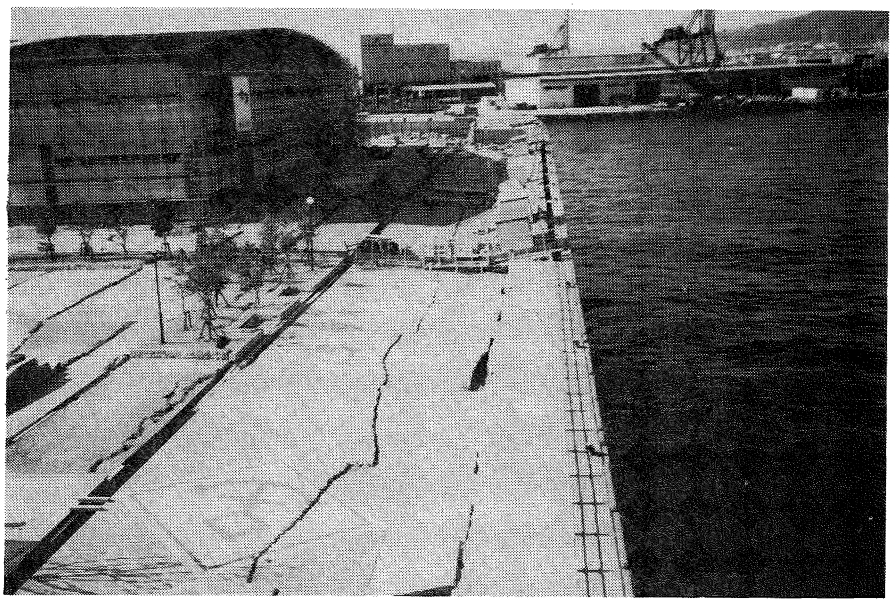


FIGURE 8.7 Near Rokko Island, End of Rokko-Ohashi Bridge. Seconday Wall Failure and Lateral Spreading

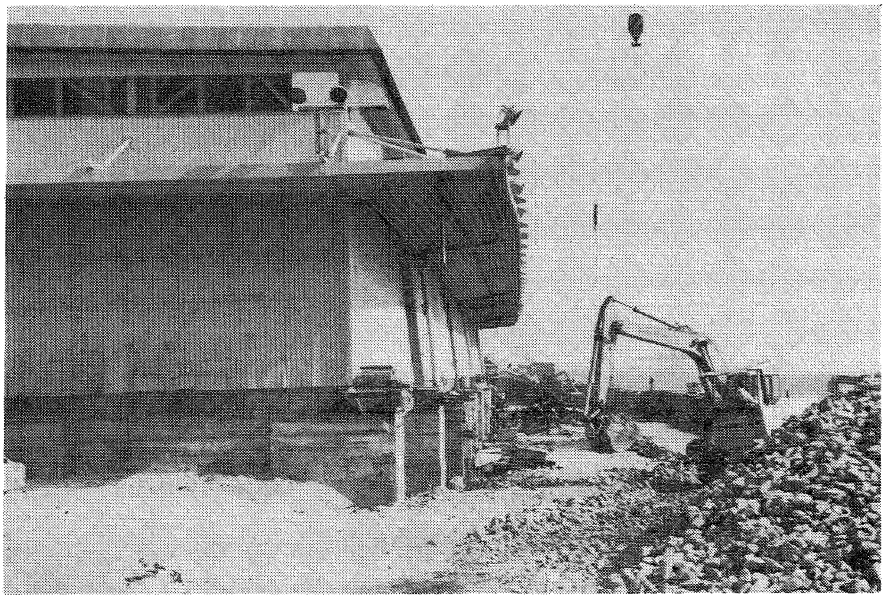


FIGURE 8.8 Port Island. Damage Due to Lateral Spreading

spreading is shown in Figure 8.8. Tilting damage to several buildings founded on spread footings was also observed. Damage to major bridges with piers at the reclamation margins is described in Section 7.

Settlements throughout Port Island and many of the perimeter areas of Rokko Island and the adjacent coastal reclamations were typically observed to be between 200 and 400 mm. This appeared to have little influence on the structural performance of buildings and bridges which were all supported on piled foundations. Relative settlement displacements resulted in widespread damage to building service connections and buried services within reclaimed land areas.

Central Rokko Island was remarkable for the absence of ground damage (extensive paved areas were undamaged) and this is attributed to the sand pile ground improvement in this area. It is also significant that there were no compressional damage features observed as might be expected in a soft ground situation.

It is reported that damage to earthquake resistant wharves was so light that the facilities could be used without difficulty [IAPH, 1995]. The reconnaissance team did not find any of these structures. Damage to the port facilities is so severe that the return to full operation will require an extended period and billions of dollars of repair and reconstruction. A container handling company whose facility was seriously damaged estimated that it would be a year before operations could resume. In the meantime cargo was being shipped through other Japanese ports including Osaka, Nagoya, Yokohama and Tokyo.

8.2.4 *Discussion of Damage*

Records for Kobe Port indicate that accelerations of 0.5 to 0.7g horizontally and 0.3g vertically were experienced [IAPH, 1995].

It appears that the design expectation for Kobe Port's structures was a_{max} = 0.2g with a 75 year recurrence interval [IAPH, 1995]. The recorded ground motions substantially exceed this.

The nature of the weathered granite fill is such that the reclamations are essentially sandy fills. The densities achieved during construction are not known but it was suggested by a major construction company that the as-placed fill materials may have SPT N values as low as 3 to 5. This being the situation the materials could be expected to be highly susceptible to liquefaction.

Port Island was reclaimed during the period from 1966 to 1981. The water depth was typically 15 m but was up to 25 m deep. The reclamation was constructed to a nominal 5 m above mean sea level. Settlements in excess of 3 m have been recorded as a result of consolidation of the alluvial clay deposits. Bore hole data within Port Island indicates that the reclaimed fill materials have SPT N values in the range of 10 to 30 [Histaoku et al, 1988]. The larger values may well be related to the rock origin of the sandy fill material.

Containment of reclaimed port areas was exclusively in the form of gravity, and specifically caisson, type structures. No tied back wall structures, or piled wharf structures with marginal spill through revetment slopes, were observed by the reconnaissance team. This latter form of construction is more common in New Zealand.

Extensive damage to cranes and adjacent structures was associated with the displacements experienced by the gravity caisson walls. During the reconnaissance visit works were well advanced in many areas to reinstate a working platform behind the displaced walls. While they had displaced, settled and rotated they obviously still provided a reasonable level of static stability thus enabling emergency reinstatement to be undertaken. This was also assisted by the existing seagoing reclamation barges which are being utilised to transport fill from Awaji Island to the south, thus not placing any demands on the road transportation network.

Sheds and adjacent portside facilities appeared to have performed well except where damaged by adjacent wharf displacements. It is concluded that a higher level of engineering in relation to the perimeter containment of wharf facilities relative to normal structure design standards is required to provide more robust operational performance. This observation needs to be investigated with respect to the actual design standards adopted for the Kobe port development.

8.2.5 *Port Seismic Design*

Seismic design for port and harbour facilities is based on the use of a design seismic coefficient [BPH, 1980]. The coefficient is determined on the basis of:

Design seismic coefficient = Regional seismic coefficient
x factor for subsoil condition
x coefficient of importance

The various factors are defined in Tables 8.1 to 8.4. Based on newspaper reports it appears that the port structures at Kobe adopted a design seismic coefficient of 0.15 (0.15 x 1.0 x 1.0). The Japanese press suggested that the standards were too low.

Liquefaction is required to be considered in design [BPD, 1980]. No guidelines are provided for design and the mitigation of liquefaction effects.

8.3 **Container Cranes**

The container cranes situated within the areas of severe shaking were those at the berths on Rokko Island and Port Island. These wharves which comprise the container port of Kobe have the ability under normal operating conditions to service 120 container ships at one time.

The structural system supporting each crane comprises a rigid steel box column and beam frame, with four legs. These legs sit on two rails, parallel to the water's edge. Distance between legs in plan is typically 20 to 30 m, with the cranes being some 40 m in height.

Table 8.1 Regional Seismic Coefficients

Region	District	Regional Seismic Coefficient
1st Region	Hokkaido (Nemuro, Kushiro, Tokachi & Hidaka) Kanto (Chiba, Tokyo & Kanagawa) Chubu (Shizuoka & Aichi) Kinki	0.15
2nd Region	Hokkaido (Ishikari, Iburi, Shiribeshi, Hiyama, Oshima, Rumoi) Tohoku Kanto (Ibaragi) Chubu (Niigata, Toyama, Ishikawa & Fukui) Shikoku Chugoku (Torrori, Okayama & Hiroshima) Kyushu (Oita & Miyazaki)	0.10
3rd Region	Hokkaido (Soya & Abashiri) Chugoku (Shimane & Yamaguchi) Kyushu & Okinawa (Fukuoka, Saga, Nagasaki, Kumamoto, Kagoshima & Okinawa)	0.05

Table 8.2 Factors for Subsoil Condition

Subsoil Class	Class 1 Subsoil	Class 2 Subsoil	Class 3 Subsoil
Factor	0.8	1.0	1.2

Table 8.3 Subsoil Classes

Thickness of Quaternary Deposits	Sand Gravel Layer	Ordinary Sand, Clay Subsoil	Poor Subsoil
Less than 5 m	Class 1	Class 1	Class 2
5-20 m	Class 1	Class 2	Class 3
More than 25 m	Class 2	Class 3	Class 3

Table 8.4 Coefficients of Importance

Classification of Structure	Character of Structure	Coefficient of Importance
Special Class	Structures of which the Characteristics 1 to 3 of Class A are strongly evident.	1.5
Class A	1 Structures tending to cause loss of life and property upon seismic damage. 2 Structures playing an important role in the recovery from earthquake disaster. 3 Structures handling hazardous materials and tending to cause serious damage to life or property upon seismic damage. 4 Structures causing serious influences on the economic and social activities of the areas concerned upon seismic damage. 5 Structures for which there would be considerable difficulty in recovery upon seismic damage.	1.2
Class B	Structures other than the Special Class, Class A and Class C	1.0
Class C	Small structures other than the Special Class and Class A, and permitting recovery with ease	0.5

The steel used in the supporting system is typically a medium-tensile carbon steel, with relatively high slenderness ratios for the individual wall elements. These high slenderness ratios make the crane elements prone to severe local buckling under accidental overload. However, the normal operating environment is such that this is not a constraint.

Various forms of damage were observed (see Figures 8.9 to 8.12). Many cranes had come off their tracks. In some cases there was clear evidence that the crane had rocked with the crane support bogey and wheels coming to rest up to 500 mm from the crane rail without evidence of any dragging or intermediate contact. Some of these derailed cranes had suffered structural damage in one or more legs. A significant number of cranes were seriously damaged due to the settlements and lateral spreading which occurred at the reclamation margins and which increased the gap between the supporting crane tracks by between 1.0 and 1.5 m. This, in turn, forced the pairs of crane legs apart and induced severe local buckling in the legs and/or intermediate supporting beams, as is illustrated in Figure 8.12.

In its severest form, the local buckling induced complete collapse of the crane. Two such examples were observed, one of which is shown in Figure 8.11.

8.4 Tanks

There are a significant number of tanks and tank farms in the Kobe area and these were subject to intense seismic shaking. Immediately after the earthquake an LPG tank was reported to be leaking seriously. No further information is available. Damage was reported to 10 of 61 tanks constructed on reclaimed ground in the Mitsubishi Kobe Terminal. Settlement



FIGURE 8.9 Rokko Island Container Terminal. Displaced/Rotated Caisson Wall and Graben Formation

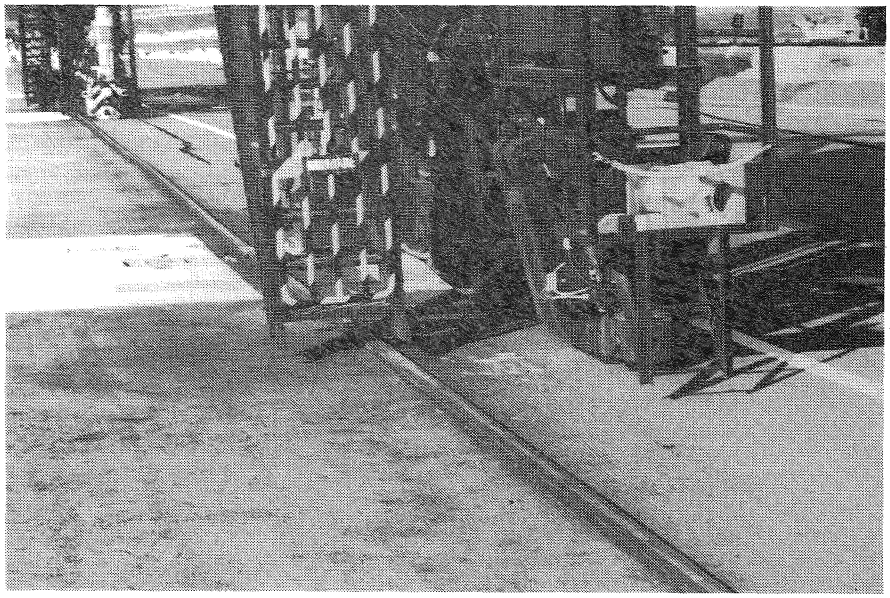


FIGURE 8.10 Rokko Island. Container Crane Displacement from Crane Rail

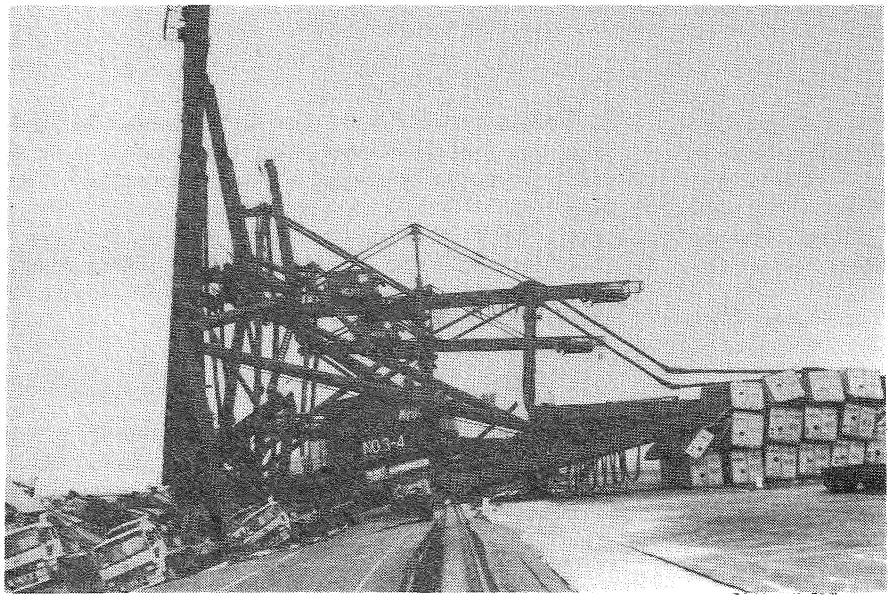


FIGURE 8.11 Rokko Island. Collapsed Container Crane



FIGURE 8.12 Rokko Island. View Along Wharf Showing Container Crane Line-up and Damage to Frame Caused by Lateral Ground Spreading

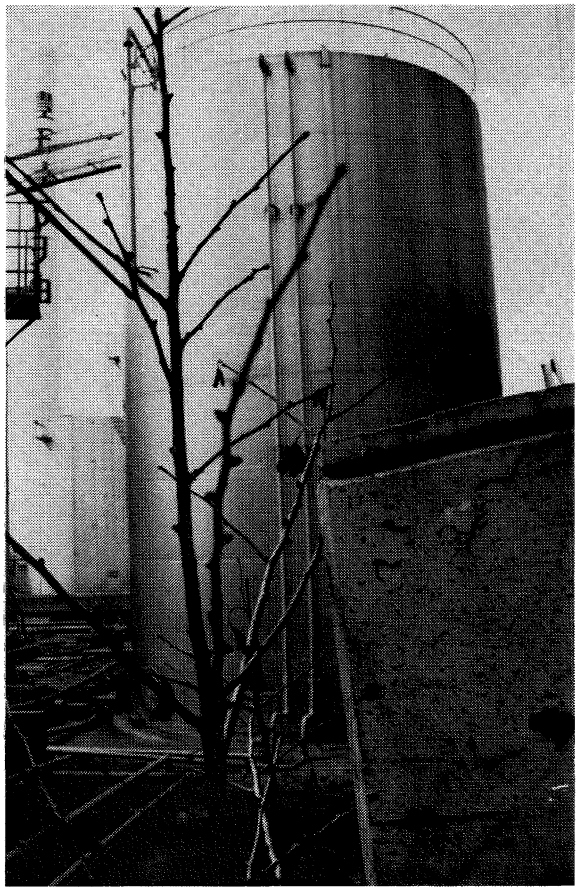


FIGURE 8.13 Kobe Foreshore, Adjacent to Port Island. Close-up of a Tall Tank and Pipework Facilities

of up to 700 mm occurred causing tank floors to incline up to 300 mm. The tanks were up to 15 m high and 20 m diameter and contained products such as styrene monomer and methanol. Cracks in pipes of up to 5 mm were reported. All 10 damaged tanks were emptied.

Tanks observed by the reconnaissance team had generally performed well. The tanks inspected had aspect ratios (height:diameter) varying from about 1:3 to 2:1 and were generally supported by spread foundations. On reclaimed land ground improvement using ground compaction methods is generally understood to be applied. Some tanks were, however, believed to be piled.

The pipework connected to these tanks, in all instances observed, had been well designed for seismic conditions, with flexible coupling and no screw joints situated on pipe bends.

All of these tanks had performed well with no outward sign of damage to tanks, foundations or piping. Minor tilting was observed to tanks in two instances (for a typical example, see Figure 8.13).

8.5 References

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9 HOUSES

9.1 Introduction

As of 2 February 1995, more than 56,000 residential houses had collapsed as the result of the main and aftershocks of the earthquake. As the main shock of this earthquake occurred at 5.46 am, failures of so many private residences contributed heavily to the confirmed 5,102 casualties and 26,803 injured people. As part of the visit of the reconnaissance team, houses were inspected in the Kansai region shortly after the earthquake. Damage to older traditional single-family Japanese houses was widespread in Higashinada and also in the epicentral area of Awaji Island. More modern western style housing construction was also surveyed in the residential area of Kitano-Cho. These dwellings performed better than the traditional Japanese houses, the damage being limited to unreinforced brick or stone chimney collapses and broken glass windows.

9.2 Performance of Traditional Japanese Houses

World War II created a housing shortage in the Kansai area.

Kobe city alone lost more than 70% of its residential houses as the result of heavy bombardments [Kawazoe, 1973]. Following the war, the government and public institutions had no clear policy for city planning and housing restoration. This created a post-war era of "self-construction" with each family building independently. These houses, inspired from the traditional Japanese style, performed poorly during the Hyogo-ken Nanbu earthquake.

One of the major problems with these traditional houses is that heavy tiles are placed on the roof. Mud is used also on the roof to keep the tiles in position, although they are often secured by wires, and to provide some form of insulation. The houses are thus top-heavy which, coupled with inadequate construction below it, created many soft storey failures. Figure 9.1 shows such a failure of a house in Higashinada.

The lack of shear stiffness and resistance of timber framing and infilled walls contributed also to many soft storey failures. Traditional houses use post and beam construction, so that when the partitions between the rooms are removed an open space is easily created. The posts and beams are held together by mortises and tenons, but no nails, as is illustrated in Figure 9.2. This system is unable to provide adequate bending moment resistance at the beam-post joint. Figure 9.3 shows an example of a simpler house built with a timber frame. A typical framing system is made up of 38 mm x 89 mm studs spaced at approximately 500 mm on centre. Horizontal boards made up of 5 to 8 mm thick by 40 to 100 mm wide sections are nailed to the studs. These horizontal boards are rough cut, rather than butt jointed, with gaps as large as 10 to 12 mm between them. In this respect the boards act more as lath rather than as a full stress skin that full boarding achieves. In some cases diagonal bracing members are present, although their nailed connections to the studs are inadequate to provide significant lateral stiffness and strength. Infilled walls are made of mud, reinforced with bamboo laces, and finished with sand or plaster on the surface. As is illustrated in Figure 9.4 for a damaged house in Higashinada, such wall construction is not adequate to resist the inertial shear stresses mobilized by the earthquake ground motions.

The structure of a traditional Japanese house is not connected to the foundation, but is weighed down by the heavy tiles on the roof. Because of the weight of the roof, the structure can withstand significant wind pressures. During the strong ground shaking generated by the Hyogo-ken Nanbu earthquake, however, the lateral inertia forces caused many houses to overturn sideways, as shown in Figure 9.5.

In most of the houses surveyed, the preservative treatment of the wood that serves as the main structure was clearly inadequate. Several structural elements, especially studs and posts, were in an advanced state of decay. The infilled mud walls restricted the air flow through the framing and contributed also to the wood degradation. The restriction in airflow was also compounded by the use of lathe and plaster on the outside. The poor quality of the wood contributed to several member failures in tension perpendicular to grain as shown in Figure 9.6. The decay seemed to be worst at the bottom of the studs.

Although most of the damaged or collapsed houses surveyed were of post-war construction, similar deficiencies were also detected in new Japanese style construction. Figure 9.7 shows, for example, a large two-story house under construction in the epicentral region of Awaji Island. The roof of this post and beam construction will host heavy tiles, shown piled up in front of the house in Figure 9.7. As shown in Figure 9.8, the sill plate is well anchored to the foundation by anchor bolts but, again, no hold-down connection is incorporated between the posts and the sill plate.

9.3 Performance of Western and Other Style Houses

Some western style houses were surveyed in the residential area of Kitano-Cho. These houses performed much better than the traditional Japanese houses. The western style of construction of these houses, even older ones, incorporated full weatherboard type skin on the outside which provided a larger shear strength than traditional Japanese houses. The damage to these houses was limited to tile roof failures, unreinforced brick or stone chimney collapses and broken glass windows, as illustrated in Figure 9.9. Where chimneys fell, they often damaged the surrounding roof structure. The glass damage was worst in smaller panes where lighter glass was used. Once again, the failure of plaster cladding was evident in these western style houses.

An important class of modern Japanese house is a typically 2 or 3 storey building with a light weight slate or iron roof and a "Hardiflex" type boarding for exterior wall cladding (see Figure 9.10). Plywood sheeting in generally present under the weather boards. These houses are common in the new housing areas. No damage to these houses could be observed from the outside, other than an occasional broken window.

It is also of note that none of the 30,000 buildings in Nishi-ku were severely damaged. Nishi-ku is one of the two main new towns and is the fastest growing area of Kobe. It extends to within only about 6 km of the epicentre and a similar distance from the worst damaged parts of Nagata-ku.

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10 URBAN PLANNING AND ARCHITECTURE

10.1 Urban Planning

The severity of the Hyogo-ken Nanbu earthquake and the extent of damage it created came as something of a shock to Japan and the world. This impact of a major earthquake on a large urban area and its effect must inevitably cause rethinking of inputs into urban planning policies and preparedness for major disasters. In some respects the extraordinary way in which the residents of Kobe reacted to this overwhelming disaster helped to mitigate the potential urban social disaster that would result under similar circumstances in many other parts of the world. There were no reports of looting or robbery and many survivors who had lost



FIGURE 9.1 Heavy Tiles and Mud on Roof of a Collapsed House in Higashinada

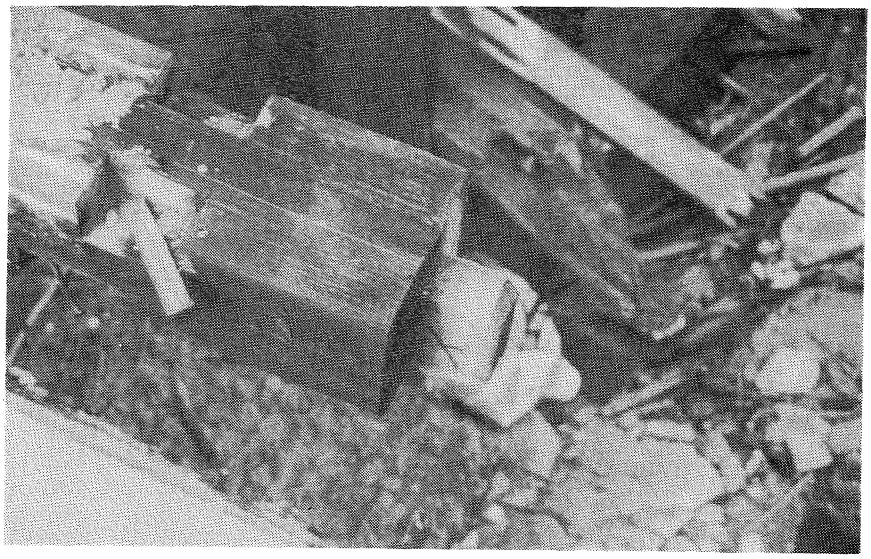


FIGURE 9.2 Tenon and Mortise Detail Between Post and Beam of a Collapsed House in Higashinada.

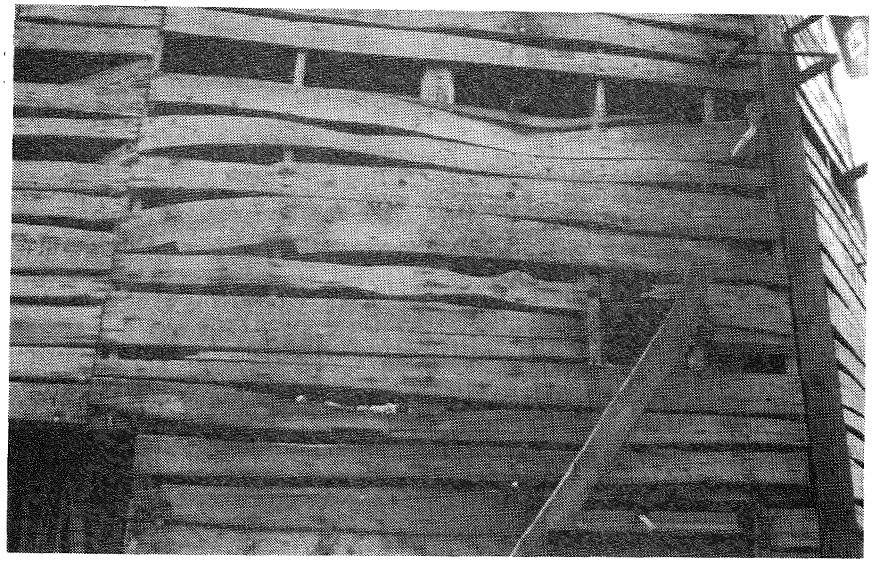


FIGURE 9.3 Timber Frame Detail of a Damaged House in Kitano-Cho

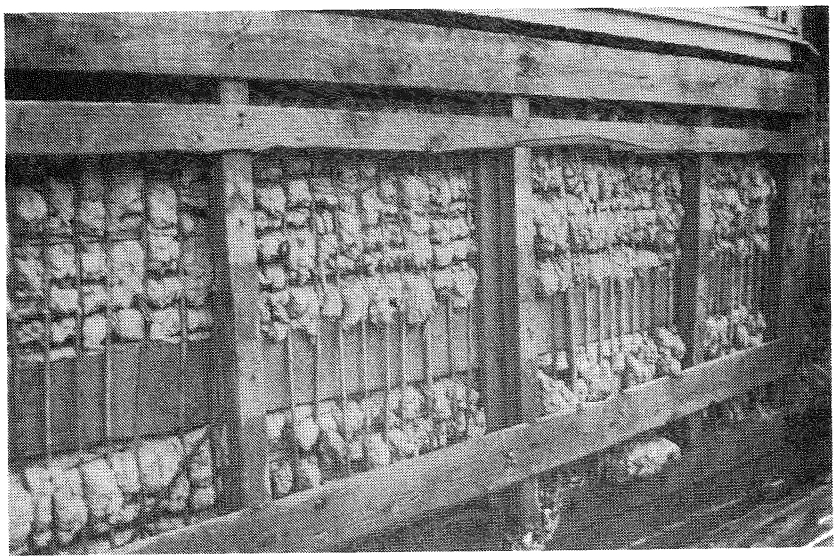


FIGURE 9.4 Infilled Mud Wall Reinforced With Bamboo Laces in a Damaged House in Higashinada

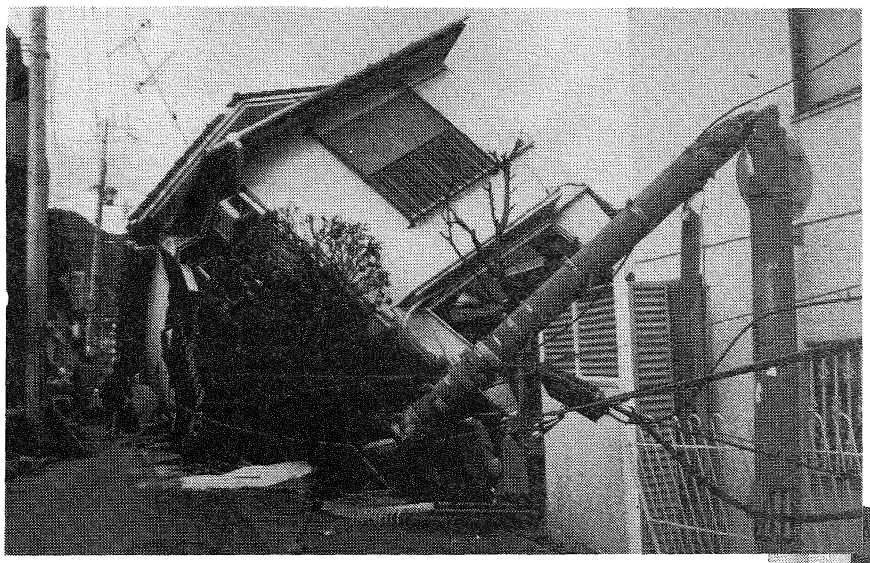


FIGURE 9.5 Overturned House in Higashinada



FIGURE 9.6 Poor Wood Condition of a Damaged House in Higashinada

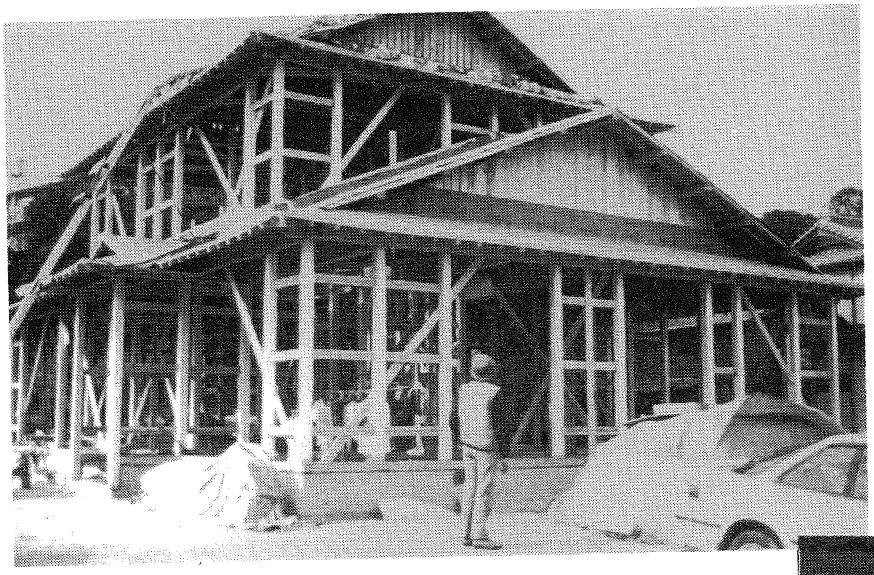


FIGURE 9.7 Two Storey House Under Construction on Awaji Island

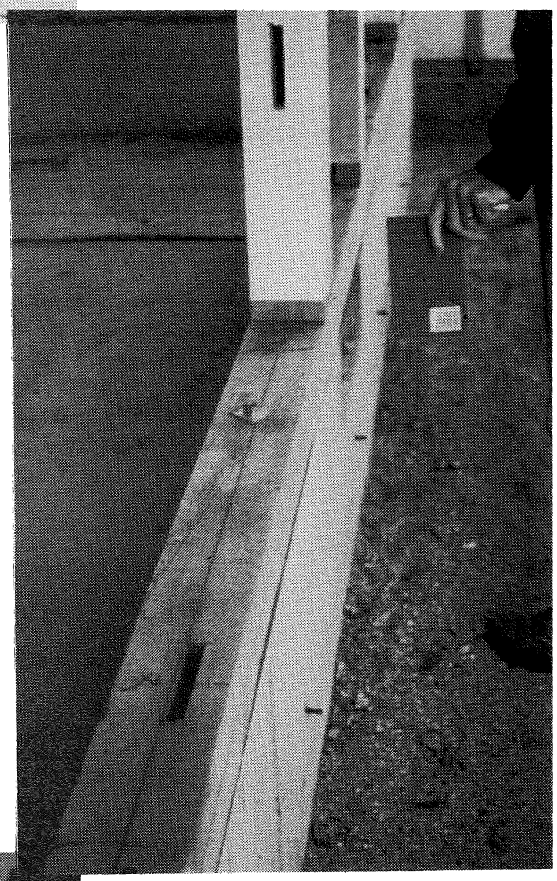


FIGURE 9.8 Base Detail of a House Under Construction on Awaji Island

FIGURE 9.9 Collapsed Chimney on a House in Kitano-Cho





FIGURE 9.10 Typical New Japanese Timber Housing (the Group on the left) of the Type that Survived with No Obvious Damage from the Outside

their homes and loved ones in a flash of awesome destruction stood in lines patiently for hours for water and slept in crowded public places. They suffered a lack of electricity, gas, communications, water and sewerage for days in a situation where rioting and public displays of fear and frustration would in other parts of the world be expected. Many observers could not help admiring their spirit of perseverance, solidarity, civility and above all voluntary observation of their civic duty and law and order.

Nevertheless the impact of the earthquake highlighted some fundamental issues that should be practical inputs into the urban planning process. These are important enough that they should have the same value placed on them as density, traffic movement, sunlight and shade, urban form, urban spaces, etc. They include:

(a) It is essential to establish traffic access routes in and out of the city that can accommodate emergency situations. Kobe was well served by highway, train and ferry services. The earthquake disrupted all of these - overhead highways were rendered unusable so that only one land based road was available for access; train lines were rendered unusable so that the only route was a torturous one through the mountains, and ferry terminals were displaced although these were relatively quickly re-established.

The consequence was that most traffic endeavouring to access Kobe for whatever reason had to traverse one single road. Legislation was in place that enabled the authorities to restrict traffic on this road to vehicles essential to the disaster management. In the event, however, the priorities placed on police staff and the difficulties in determining the status of vehicles meant that this road was consistently clogged, so that trips that would normally take 45 minutes to one hour were, in the extreme, taking eight hours. Within a relatively short time one lane was dedicated to emergency vehicles. However the difficulties experienced by the authorities highlight the need for alternative route management and capacity design of major arterial access routes.

(b) Open spaces, pocket parks and recreation areas take on a high value after a major earthquake. In Kobe these spaces were used for:

(i) Establishing temporary accommodation camps
(ii) Establishing military/disaster management bases
(iii) Establishing rubble dumps for demolition material - this is particularly important given the difficulty of transporting rubble out of the city as a result of the damage to major arterial access routes.

(c) Strategic buildings require multi-use capabilities. Municipal Halls and schools were particularly valuable in Kobe as temporary accommodation. This highlights the need for these buildings to have care exercised in their location, the open space around them and the servicing to them.

(d) Road widths become significant when demolitions of major buildings in an urban environment have to be undertaken. Because of the proximity of other buildings demolition by explosives is not possible. The buildings need to be scaffolded and demolished by using mechanical and hand breakers. Inevitably this requires a section of the road onto which the building is fronting to be dedicated to construction and demolition work. The demolition requirements tend to be greater than that of construction due to the nature of the machinery to be used (see Figure 10.1). It is desirable for roadways to be able to accommodate demolition/construction without unduly restricting traffic flows.

10.2 **Architecture**

The damage experienced at Kobe tended to reinforce all the lessons learned from previous earthquakes. Given the number of buildings destroyed and the extent of the damage it was possible to see virtually every type of building failure one could imagine. These included:

(a) Failure of buildings due to the "classic" soft storey syndrome (see Figure 10.2).

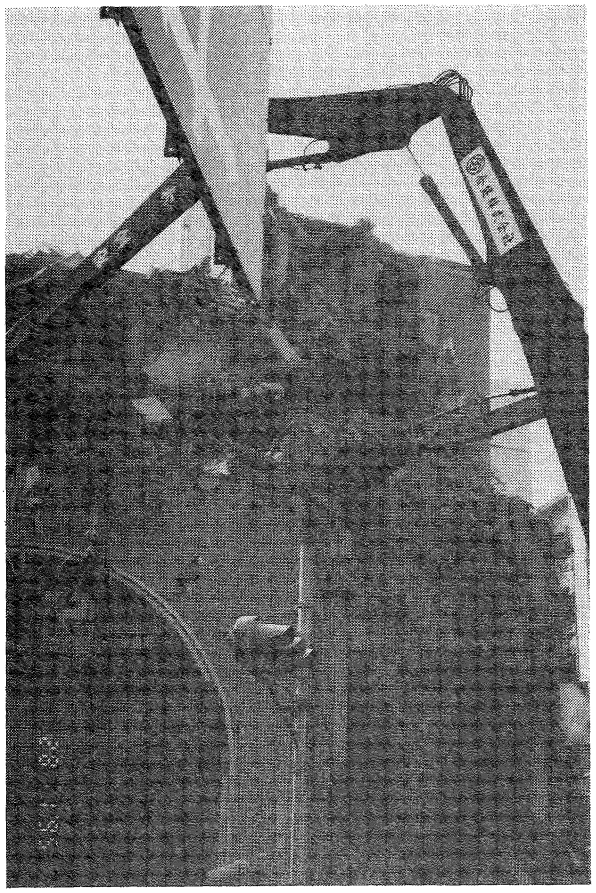


FIGURE 10.1 Demolition Machinery in Use

(b) Damage of buildings through pounding.

(c) Damage of buildings through the failure of adjacent buildings separated by a party wall (see Figure 10.3).

(d) Failure of key non-structural elements such as cladding, shopfronts, etc, (see Figure 10.4).

(e) Failure of masonry structures (see Figure 10.5).

(f) Damage or failure of architectural elements (see Figure 10.6).

Inspection of the failures highlighted the need for architectural conceptual work to have an inherent understanding of the seismic consequences of design decisions. It also highlighted the fact that appropriate architectural concepts combined with modern seismic structural and component design can produce buildings that will withstand an earthquake of the magnitude of the Hyogo-ken Nanbu earthquake. This was demonstrated by the modern buildings that were relatively undamaged even though their neighbours were severely damaged. However, movement of the ground around the perimeter due to liquefaction often affected entry areas and vehicular ramps (see Figure 10.7). So even though these buildings withstood the damage, they could be rendered unusable by the failure of incoming services due to ground movement and/or the dislocation of adjoining footpaths and roadways.

A characteristic of the damage to residential buildings in Kobe was the failure of tile roofs. In extreme conditions these contributed and were part of the total collapse of many

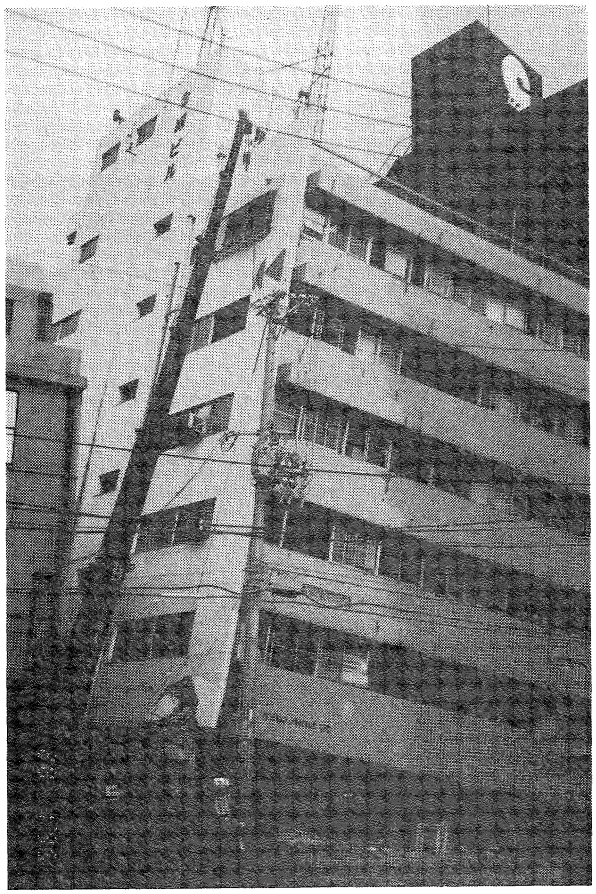


FIGURE 10.2 Failure of a Building Due to Soft Storey Behaviour

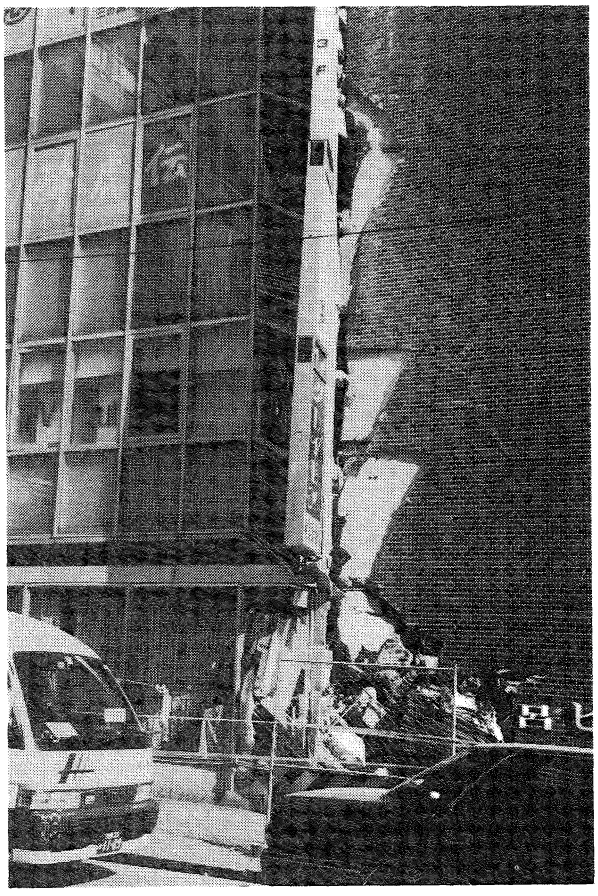


FIGURE 10.3 Damage of Buildings Through Failure of an Adjacent Building



FIGURE 10.4 Failure of Key Non-Structural Elements

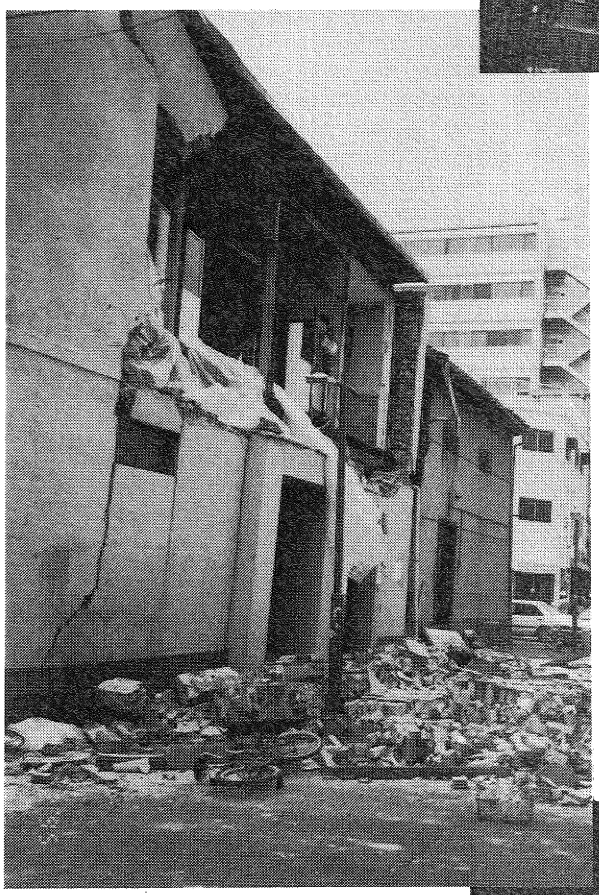


FIGURE 10.5 Failure of Masonry Structures

FIGURE 10.6 Damage of Architectural Elements



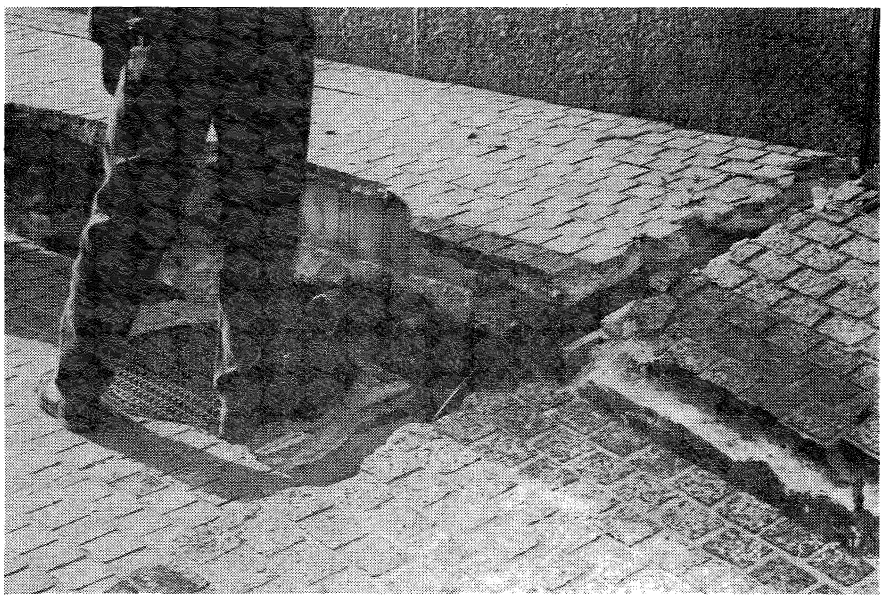


FIGURE 10.7 Ground Movement Around Perimeter of a Building



FIGURE 10.8 Failure of Tile Roofs

thousands of houses. Further from the major affected areas however the damage was still extreme (see Figure 10.8) and was demonstrated by the extensive use of blue tarpaulins on house roofs visible from trains and highways. Typical failures were along the ridge - and sometimes extending down into the slope of the roof. This would seem to highlight the need for reviewing the fixing of ridge tiles in some way more positive than simply mortaring the joint.

11 SECONDARY ELEMENTS OF BUILDINGS

11.1 Cladding

11.1.1 Introduction

In considering cladding systems it is appropriate to differentiate between low rise residential structures and commercial and high rise buildings.

11.1.2 *Low Rise Residential Houses*

The construction of houses has been described in Section 9. Typical cladding of the buildings was achieved by 10 to 25 mm thick plaster applied over a kind of lath created by 12 mm boarding. This boarding while cut true in the longer dimension was rough cut on the edges and often separated by 10-12 mm at the join (see Figure 11.1). Building paper was usually applied over the timber and chicken mesh fixed over that; usually it would appear without spacers. The plaster was then applied to this so that the mesh was close to the back surface. This meant that there was frequently corrosion evident in the mesh where failure was observed.

Failure of this plaster cladding was widespread. In some cases it would seem that the plaster had contributed to the structural instability of the timber framed wall by sealing the timber cavity and causing the timber framing to decay. This may well have

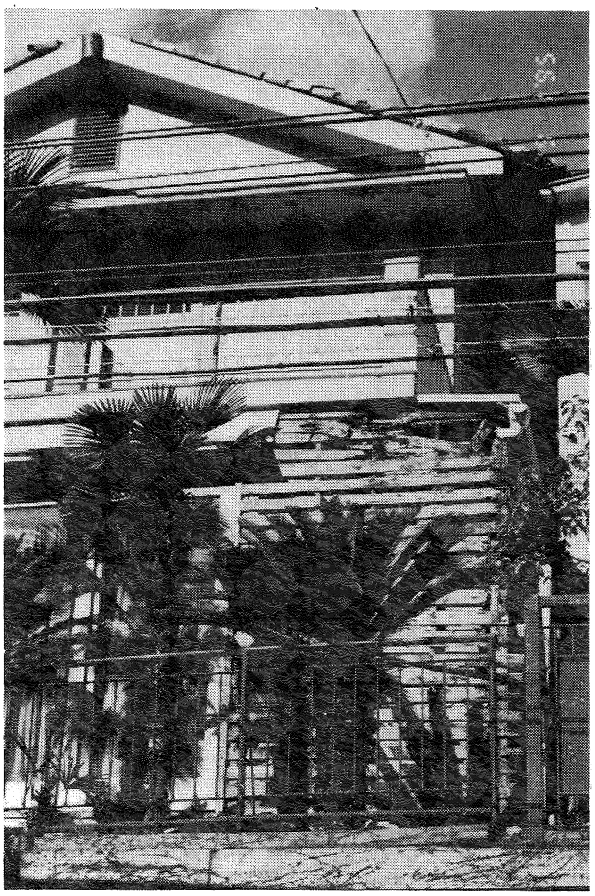


FIGURE 11.1 12 mm Thick Boarding Forming a Lath for a Wall

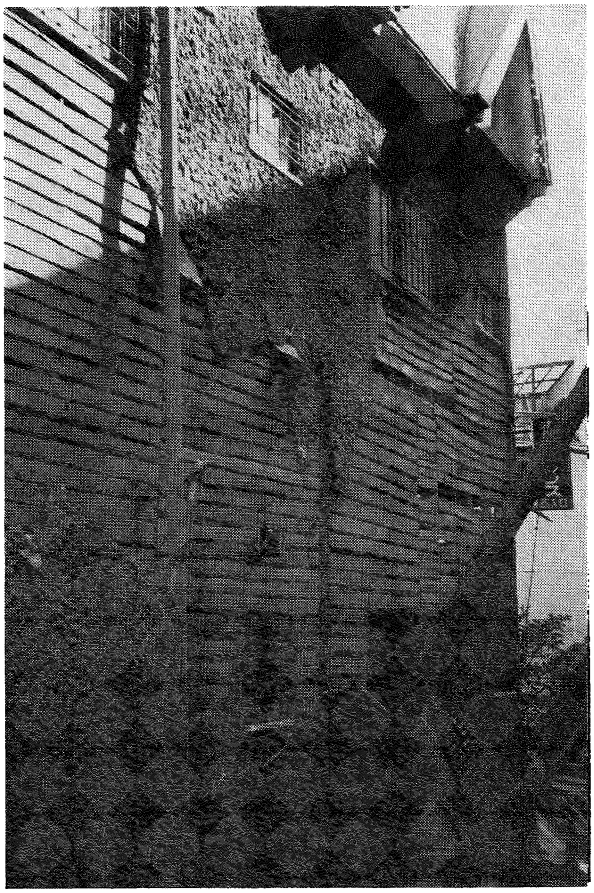


FIGURE 11.2 Failure of Plaster Cladding on House

contributed to the large number of total failures observed. There were no expansion joints evident in the plaster work.

In some cases where the building had remained intact, failure of the plaster cladding was observed (see Figure 11.2). There was consistent failure of this kind of cladding in both residential and commercial buildings.

11.1.3 *Commercial and High Rise Buildings*

Many commercial buildings used cladding similar to that described in Section 11.1.2. The total failure of plaster cladding on two or three storeyed commercial building was quite common. Once again this plaster would appear to have been applied to mesh fixed on building paper over a rough timber lath. The lack of spacers caused the mesh to finish up hard on the back face of the plaster. Subjected to the inevitable moisture it could well have contributed to the consistent failure of this cladding.

Precast panels were also used in cladding in some residential and commercial buildings. Extensive failure of these were evident. One building in particular had not only failed structurally but shed the majority of the precast panels that had clad it. These panels seem to have been constructed of lightweight aggregate and held top and bottom by steel angles and bolts. It was difficult to see how the bottom angles were fixed however it would appear that the panels were put in place and a levelling screed poured on the floors after they were fixed. In any event these panels had fallen onto the road and it was just as well the earthquake occurred early in the morning as they would have inevitably caused vehicle damage and death if it had been during a time in which this busy street was occupied (see Figure 11.3).

External tile cladding was not uncommon and localised failure, often in unpredictable areas, was quite common. In many situations however this would be relatively easy to repair. A number of buildings showed characteristic failure in the panels between the windows, even when there was no loss of glass (see Figure 11.4).

One of the most successful cladding systems would appear to be Alucabond or similar materials. This was often used in new buildings and providing the building remained intact appeared to suffer very little if any damage.

11.2 External Fittings and Fixings

11.2.1 *Canopies*

Examples of canopies collapsing were observed (see Figure 11.5). This would appear to be a problem of fixing. However it demonstrates the danger of even lightweight awnings being suspended over sidewalks and failing as a result of an earthquake.

11.2.2 *Shop fronts*

Damage to shop fronts was observed in two ways:

(a) Failure of glass shop fronts.
This was fairly common in many buildings and creates a security hazard for retail premises and display areas. In



FIGURE 11.3 Failure and Dropped Position of Precast Concrete Cladding Panels

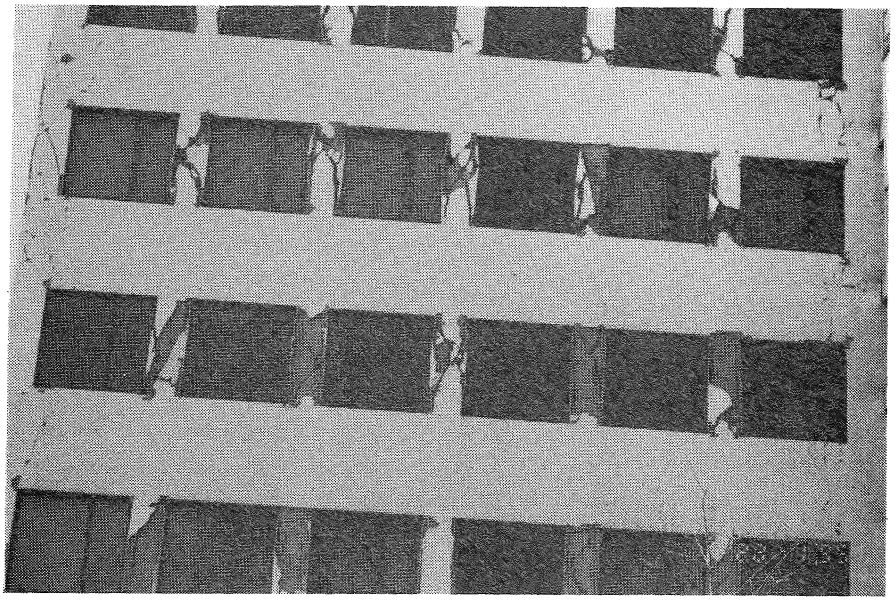


FIGURE 11.4 Failure of Tile Cladding Between Windows

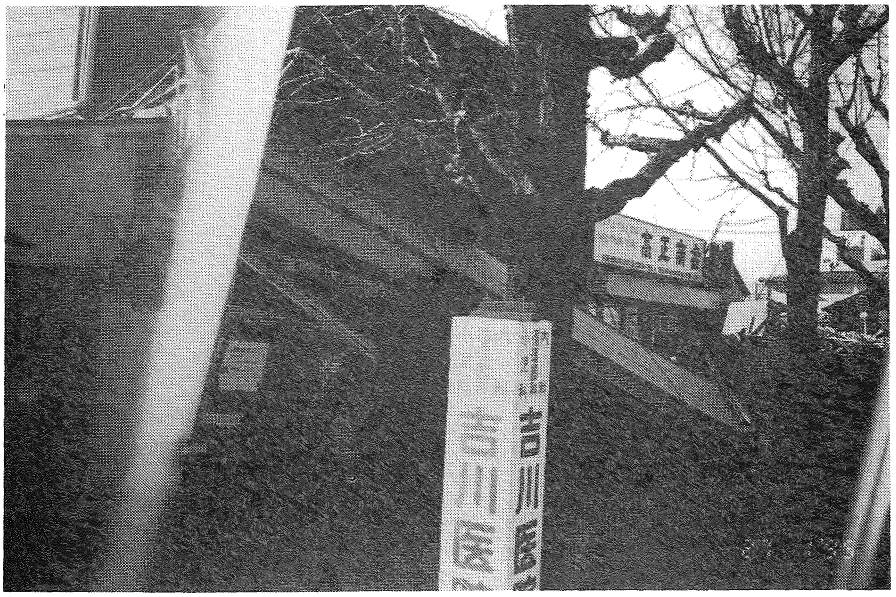


FIGURE 11.5 Collapse of a Canopy

most cases of failure no seismic separation was evident in the shop front detail design.

(b) Shop fronts falling out onto the sidewalk.
This was observed in the case of one wall of glass blocks (see Figure 11.6) and one feature opaque shop front wall (see Figure 11.7). In both cases it would appear that seismic fixing and/or seismic separation had not been considered in designing these decorative elements of the buildings. In both cases had people been on the busy sidewalks there could have been severe injury or death.

11.2.3 *Signs*

A characteristic of buildings that had failed was the stability of the signs fixed to them. Even where buildings had collapsed by one or more storeys it was usual to see the large illuminated and display signs still fixed to the face of the building. This was interesting given that these elements could well be extreme hazards in severe earthquakes (see Figure 11.8).

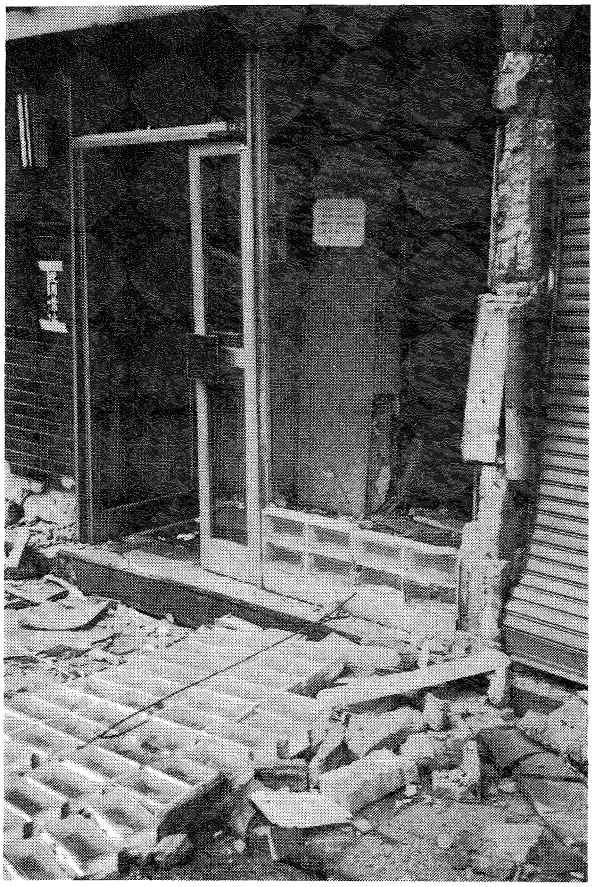


FIGURE 11.6 Failure of a Glass Block Shopfront Wall

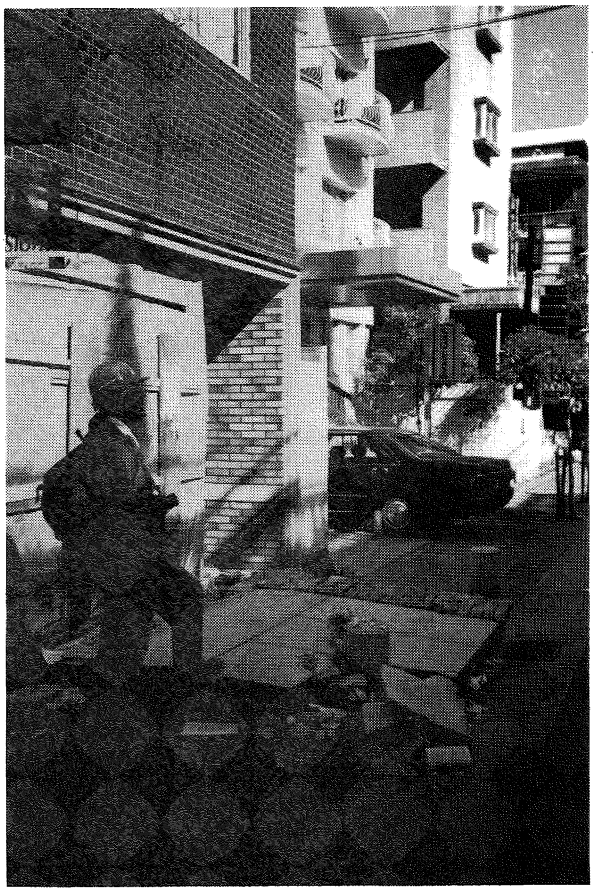


FIGURE 11.7 Failure of an Opaque Shopfront Wall

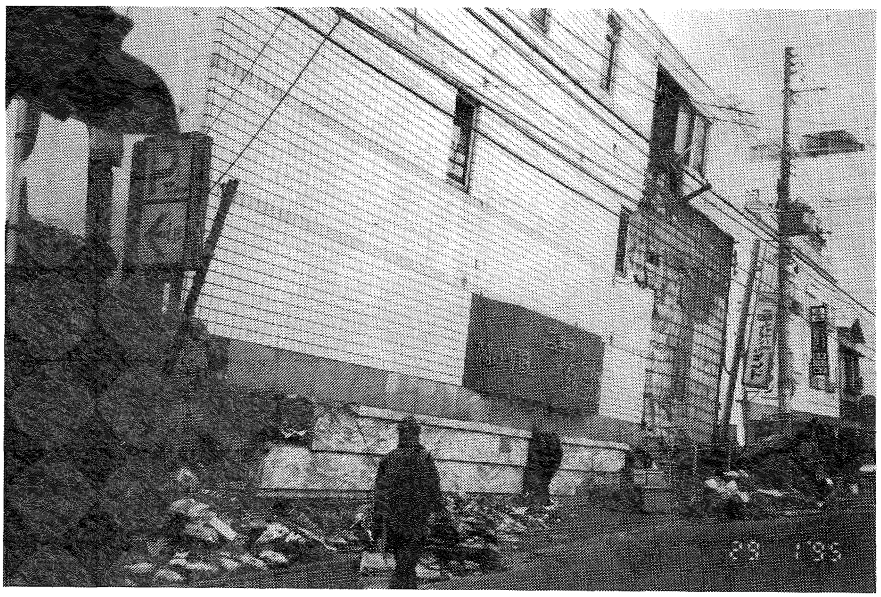


FIGURE 11.8 Sign Still Fixed to Face of Collapsed Building

11.2.4 *External Louvres*

Like signs, there was no real evidence of failure of external louvres. Even in buildings where severe failure had occurred, the louvres had stayed intact, even when they themselves were damaged by soft storey failure (see Figure 11.9).

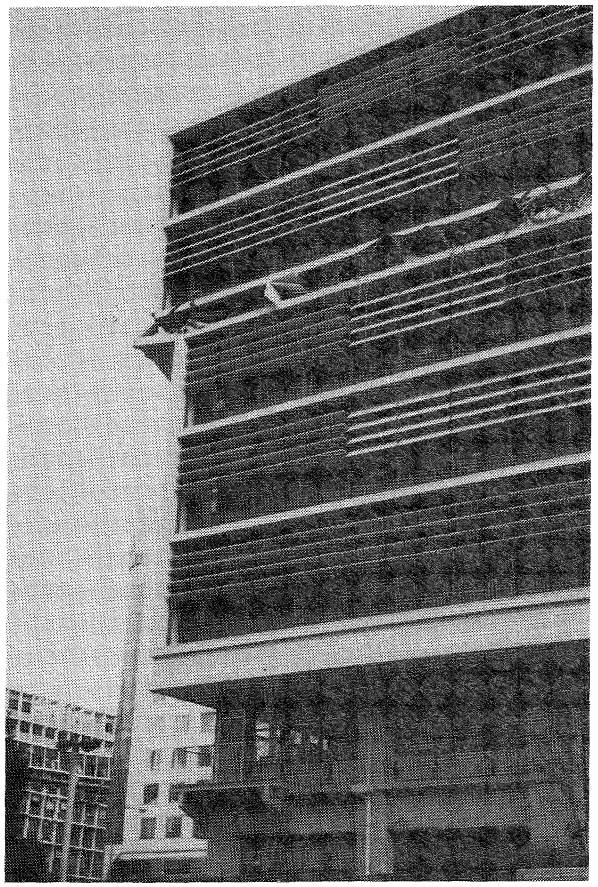


FIGURE 11.9 Louvres Remain in Place on Collapsed Structure

11.3 Internal Fittings and Fittings

11.3.1 *Suspended Ceilings*

In almost all buildings observed, even where there was severe failure, suspended ceilings appear to have remained intact. Members of the reconnaissance team were able to inspect the No. 2 Municipal Building which had suffered severe damage and yet no evidence of suspended ceiling damage was observed.

11.3.2 *Internal Partitions*

Members of the reconnaissance team were able to observe the effect of the earthquake on the internal partitions in the No. 2 Municipal Building. Generally fixed partitions had remained relatively intact. Demountable and partitions not full height had often suffered and moved considerably. It is not known whether seismic separation or fixing was incorporated into their design. Internal glazing had also broken.

11.3.3 *General Fittings and Fixtures*

The reconnaissance team members that inspected No. 2 Municipal Buildings reported that there was extreme chaos as a result of the effect of the earthquake on fittings and fixtures. Shelving had fallen over, files had distributed their contents widely, vending machines had moved and split, and plan cabinets spread their contents (see Figure 11.10). Basically the earthquake had created chaos inside the office as a result of fittings, fixtures and filing material being widely distributed through the floor. This was considered to be of such an extent that it could injure occupants and block egress routes if the building is occupied during an earthquake. Computers suffered like other office equipment in that they were flung around the floor. It was reported that in buildings where the computers had not been badly damaged the potential damage through dust was of real concern. One major building occupant reported that regular back up to their American Head Office would minimise danger. Their building had not suffered damage but was rendered unoccupiable by the lack of sewerage and water. For this reason they have not been able to determine whether their computer system had suffered.

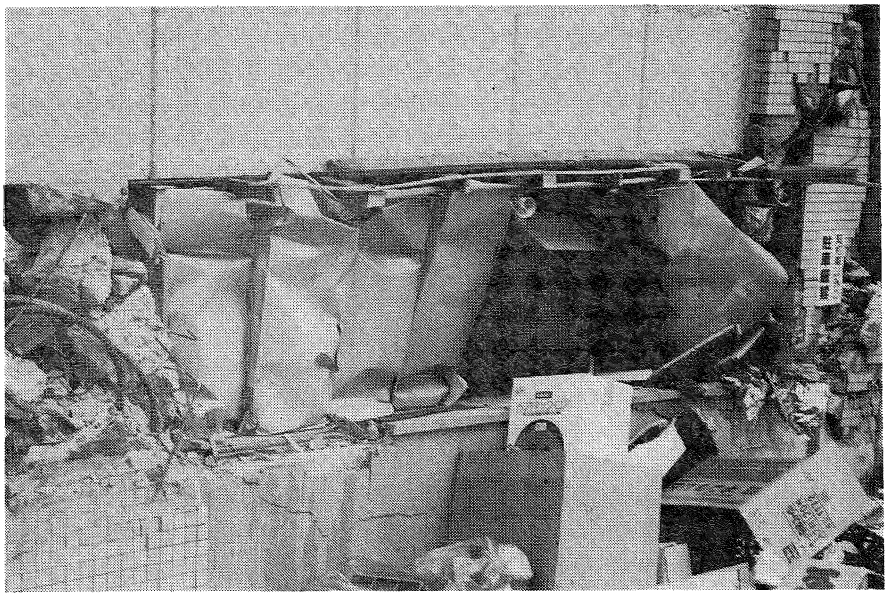


FIGURE 11.10 Filing Cabinets Inaccessible as a Result of Building Failure

11.4 Entries and Stairs

Generally external entries had remained intact although the access to them was affected by the movement of the ground and footpaths and roadways around them.

11.5 Bridge Links

Two bridge links were observed linking the No. 2 Municipal Building with the No. 1 Municipal Building. One glazed link had remained intact while the other had failed as a result of the building failure.

Bridge links between multi-storey residential buildings where one building had failed also failed dramatically. This was to be expected as a result of the extreme failure of one building (see Figure 11.11).

A bridge between two large residential buildings demonstrated the impact of pounding, though the bridge structure itself was relatively light (see Figure 11.12).

11.6 Glazing

A number of aspects of glazing and the impact of earthquakes on it were observed.

(a) As an overall impression the vast majority of window panes in the epicentral region of the Hyogo-ken Nanbu earthquake were not damaged, even in buildings with extensive areas of glazing and in the zones of most severe structural damage (see Figures 2.3 and 5.1). In only a small proportion of the commercial buildings inspected were most of the window panes broken, and in only a very few cases could there have been large scale shedding of glazing.

(b) In a building totally clad by curtain wall (see Figure 11.13) there was virtually no damage (two or three broken panes in the curtain wall) and yet close observation indicated that there was severe damage to the structural frame inside. This glazing comprised of 800 mm thick curtain wall sections with silicone join in the centre. There would appear to have been a reasonable degree of movement provided by the detail of the frame and the junction of the glass and the frame. In any event the building appeared to be intact on the outside even though there was severe failure of the structural lateral load resisting system on the inside.

(c) Many buildings suffered glazing failure apparently as a result of the lack of seismic separation. Characteristic failures were horizontal strip glazing fixed between two spandrels (see Figure 11.14). Also the impact of seismic separation in glazing was highlighted by a building in which every second glass pane had broken. The panes that had remained were in fact opening sashes and these would most likely have provided some seismic movement capability (see Figure 11.15).

(d) One particular failure of glass assembly in a corner building was observed. This was not a large building and it would appear that seismic separation had not been designed into the top and bottom fixings. As a result the glass had failed particularly in the corner. Other glass assemblies were observed that had not failed, particularly in the Sakura Bank which had an 8 m high glass assembly and granite panels above. This was a recently completed building designed to current codes and it exhibited no earthquake damage.

12 BUILDING SERVICES

12.1 Introduction

The ability of the reconnaissance team to observe the effect on building services was limited by the extent of damage. Particular points noted are outlined below:

12.2 Water Tanks

A number of examples of failed elevated water tanks were observed. These ranged from towers falling off (see Figure 12.1) to water tanks almost falling off. In one twelve storey apartment building the lift motor room had collapsed onto the penthouse level. This was apparently caused by the elevated water tanks. So, while the building was still occupied it had no water and the residents were obliged to fill buckets from a standpipe on the street outside and carry it upstairs. There was no elevator either. The reconnaissance team was advised by a resident that the company who had constructed the building had visited from Tokyo, checked the building out and confirmed that it was safe to occupy even though there were no services. The failure of the penthouse highlights the problem of locating heavy water tanks at this high level.

12.3 Electricity

No specific failures or damage to the main electrical distribution was observed. However in the case described of the twelve storey building where the lift motor room and lifts had collapsed, the structure came to rest very close to a distribution board. If this had fallen a little bit further there could have been severe damage.

12.4 Elevators

A number of elevators in damaged buildings were observed. In one twelve storey apartment building it appeared that the elevator was located at the ground floor (this would be expected at the time of the day the earthquake happened). However, apparently some heavy element had fallen down the shaft and it had almost broken through the lift car roof. In some residential apartments where buildings had been damaged the elevators had stopped. Characteristically these elevators were located half way between the ground and first floor. In no cases were the elevators operable.

12.5 Waste Services

It was difficult to see the impact on major sewer and water lines. In one block of eight three storey apartment buildings there had clearly been movement of the buildings relative to the in ground service lines. This had caused fracturing of the vertical surface mounted risers from the horizontal service lines, thereby rendering them inoperable. Additionally the risers had pulled away from the inground service line junctions (see Figure 12.2).

12.6 Fire Risers

In instances where soft storey collapses had occurred all risers including the fire risers were fractured. This would clearly create a problem should there be fire at the time when upper floors residents were being evacuated.

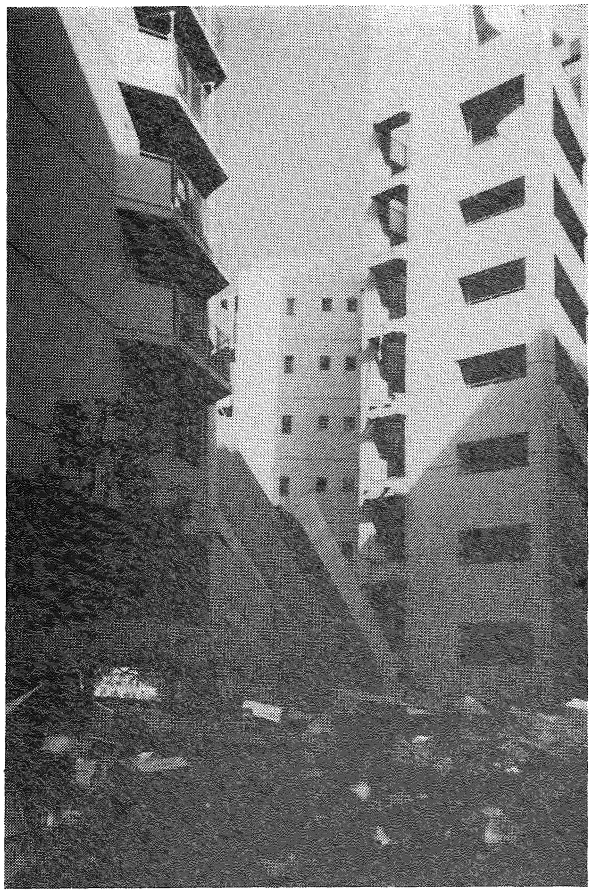


FIGURE 11.11 Performance of Bridge Link Between Residential Buildings

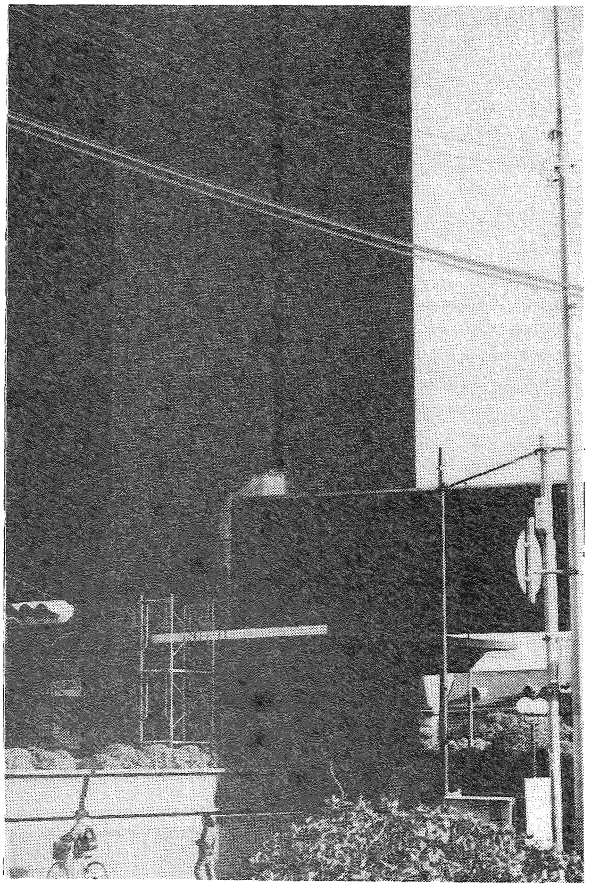


FIGURE 11.12 Effect of Pounding of Bridge Link

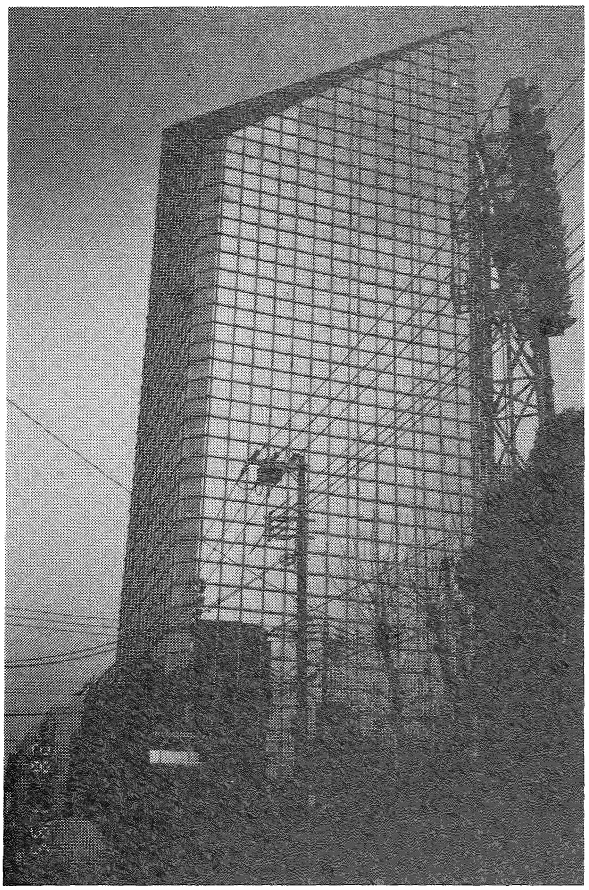


FIGURE 11.13 Undamaged Glass Curtain Wall With Damaged Structural Frame Behind

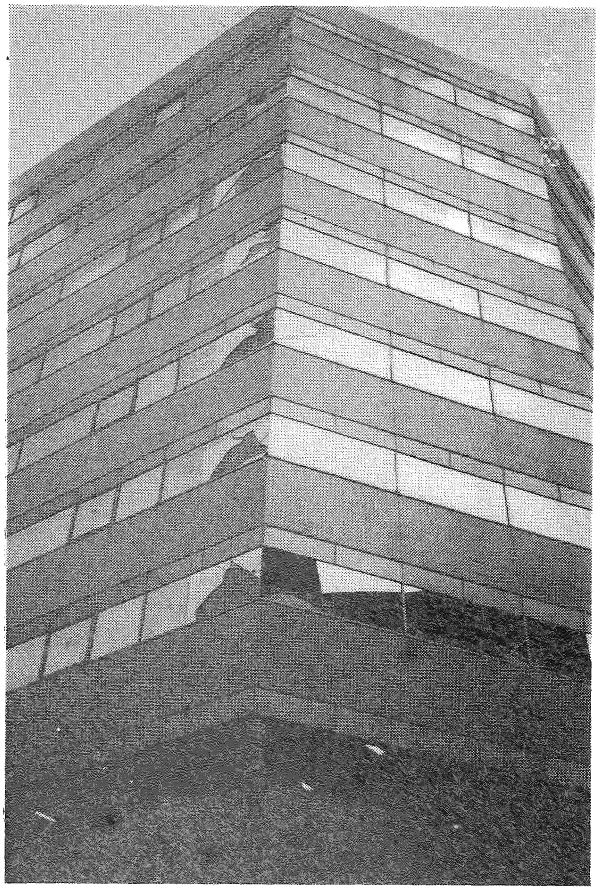


FIGURE 11.14 Failure of Strip Glazing Without Seismic Separation



FIGURE 11.15 Intact Glass Panels in Opening Sashes



FIGURE 12.1 Collapse of Tower Supporting a Water Tank

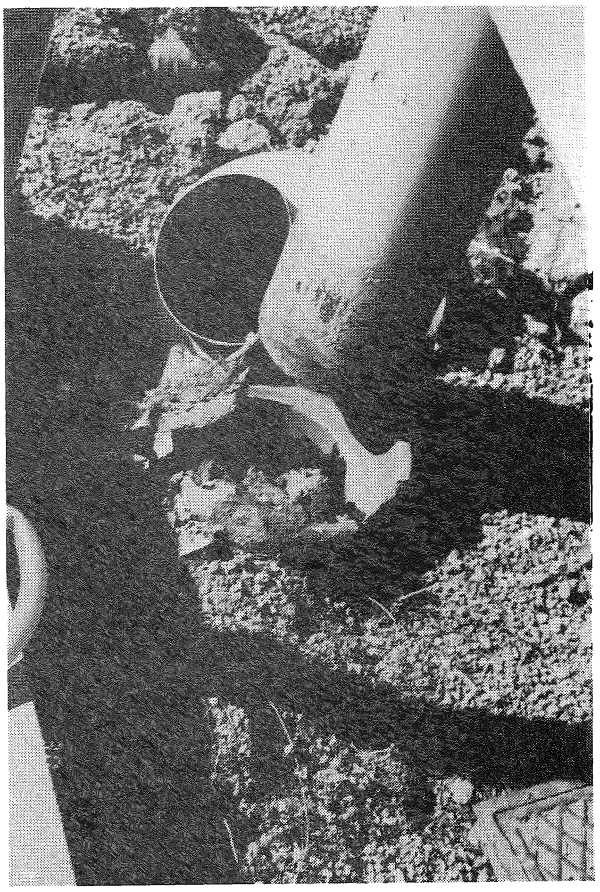


FIGURE 12.2 Fracture of Vertical Service Risers Due to Movement of Building Relative to Inground Service Lines

12.7 General

The failure of water, gas and sewerage services had a major impact on building occupancy. The main Kobe Hospital was still occupied, displaying damage only at the junction of large building blocks. These junctions had been temporarily repaired to make the building serviceable. The hospital had to reduce operation to emergencies only and was limiting patient numbers due to water and gas supply limitations. A large firm on Rokko Island was unable to occupy its 30 storey building due to lack of essential services, yet the building being relatively recent had not suffered damage.

13 PREPAREDNESS AND EMERGENCY RESPONSE

13.1 Preparedness of the Kobe Municipal Government

There is considerable public comment in Japan that suggests that the Kobe Municipal Government was ill prepared for the earthquake that struck in January 1995.

Seismic experts predicted that the areas most likely to experience a large earthquake were in the north-east (Tokyo, Yokohama and the island of Hokkaido), with the cities of Kobe and Osaka being assessed as having a much lower potential. It is fair to say that as a result of these predictions, the local governments of Kobe and Osaka limited their planning for such a disaster.

While the emergency services were obviously stretched by the number of calls for assistance, it was not necessarily their response or actions that attracted criticism. They were faced with an enormous task. Fires and rescues were widespread, they had access difficulties because of damage to the roading networks, and for at least 24 hours, they had no outside assistance.

Criticism was directed at authorities at all levels. However, it appeared that the problem originated from inefficiencies within the management of the municipal departments. An indication of this, and the degree of its relevance, was demonstrated by the manager of the city's water supplies, who committed suicide a few days after the earthquake.

Local government had allowed certain areas of the city to remain totally underdeveloped, co-ordination between individual city departments was at best disorganised, information networks were ineffective, their management structures were too rigid, and, and they were slow and bureaucratic in their reaction to the needs of the people.

13.2 Preparedness of the Community at Large

The general preparedness of the local government had a flow-on effect throughout the total community. There was a high expectation that the emergency services would come to the aid of everybody. This obviously did not happen, and it was this fact that lead to the general criticism of the emergency services. Clearly communities had no understanding of the extensive range of problems they would have to face alone.

Japanese people are very loyal employees. This was evidenced by the number of people attempting to get to work, regardless of how difficult that may have been. The desire to keep cottage industries running, even amongst the ruins, was witnessed in all parts of the city. Buildings had partially collapsed, services were clearly disrupted, but in the midst of it all were people working, sometimes with the assistance of portable generators set up on the footpath.

Business management is very structured. This approach is reflected in their preparedness for emergencies. The plans that had been prepared were predominantly ineffective, due to the lack of flexibility in both their content and application.

In some cases, modern business methods that were not supported by contingency plans resulted in the collapse of commerce and industry. Companies that operated "just in time" methods, suffered badly from lost production, due to the inability of suppliers to deliver inventory. Some may never recover.

13.3 The Role of the Emergency Services

Japan does not operate national emergency service structures. The previously mentioned inadequacies of the municipal government impacted on the efficiency of the emergency services. The Fire Department is the service responsible for fire suppression and civilian rescue, but is not equipped for heavy structural rescue, and neither is any other service. The Fire Department also provides paramedic and ambulance transport services alongside the hospital ambulance services. However the hospital service is primarily geared to patient transfer. The Police have a combined crime and traffic role.

The impression is that the Fire Department and the Police worked in isolation, as did the Self Defence Force (Army).

13.4 Co-ordination of Emergency Services

As indicated above, there was little indication of a cohesive and co-operative relationship between the services.

13.5 General

While not excusing the circumstance in Kobe, the unfortunate predicament with an event of this proportion, is the fact that the only method of determining how effective the planning has been, is to suffer the consequences of a disaster, and equally unfortunate is the fact that ...

> ... *those who have planned will be in a better situation than those who have not planned; however, this is still no guarantee that you will have planned appropriately* ...

In determining the necessity for planning there are traps. One is not to recognise the need; another is to plan unrealistically. For example, the Tokyo Metropolitan Government predicted three years ago that their worst case scenario will, among other predictions, leave 9,300 dead and 147,000 injured. They then take into account that maybe it will be worse if it occurs at 6.00 pm and there is a 6 metres per second wind blowing. The point being emphasised is that if you plan to just one scenario, how prepared are you if the situation is significantly worse, or significantly different?

There is no purpose in being overly critical of the situation in Kobe; on the contrary, there is plenty that New Zealand can learn from Japan in regard to earthquake preparedness. However, we will go on record as saying, that for any form of emergency management to be effective, we *must* plan, we must plan with interagency co-operation and co-ordination, we must

plan with flexibility and we must plan realistically. Additionally, and equally as important, the community has to be educated in the very important role they will have to play.

Individuals must take personal responsibility for an appropriate amount of preparedness for a major earthquake, and for their safety and well being immediately afterwards. It is totally unrealistic to expect emergency services to "make everything better" immediately or even soon after a large earthquake, because it simply will not happen, even if all emergency responses are activated rapidly and as planned. Individuals have to be ready to cope with loss of normal supplies of water, transport, food, power, telephone, gas and fire fighting assistance. In some locations, some services may be disrupted for days, weeks or even months. The larger the earthquake, and the larger the population centre affected, the longer will be the periods of disruption.

14 IMPACT OF FIRE

14.1 Causes of Fires

According to the newspaper Asahi-Shinbun, 7 February 1995, a study was conducted by Professor Murozaki of Kobe University and his group who interviewed 400 people about the causes of fire. The number of fires in question were 112 in Kobe city, 31 in Nishinomiya city, 14 in Ashiya city and 7 in Amagasaki city. The causes were confirmed for 69 fires.

Twenty-two fires were due to gas leaks. Eight of them started immediately after electric power supply was resumed, perhaps due to short-circuit. Nine of them occurred when people entered in a room, where gas was present, with a candle light. Five of them were from hot-water suppliers. In Ashiya city, there was a case where an automatic gas switch-off device (which is controlled by micro-computer) for a hot water supplier did not work and therefore the ignition fire (a small fire to ignite the gas heater) was not extinguished, and as a result fire occurred.

Thirty fires were due to short circuits of electricity, and eight of them were due to a combination with gas leaks. Nine of them started after the electric power supply had been resumed. Four of them were due to electric heaters falling over clothes or other items and igniting them.

It is noted that one-half of the fires occurred more than two hours after the earthquake. Sixteen fires occurred on 18 January and 12 fires occurred on 19 January 1995.

While Kobe experienced significant fire damage, it is apparent that this damage was limited by the design and construction method of traditional housing, and the way this housing collapsed. Traditional housing places mud between the roofing timbers and the ceramic tiles. When these houses collapsed the mud and tiles smothered the framing timbers and housing contents, establishing a barrier to substantial fire development. Modern timber frame housing did not collapse to the same degree, and had fires occurred within the modern housing settlements, it is very likely that Kobe would have experienced more fire damage.

14.2 Spread of Fire

The reconnaissance team estimates that only 2% of the area of most significant destruction from the ground shaking was subsequently involved in fire. The reasons for this unexpected and fortuitously low extent of destruction by fire are given in Section 14.1. This section briefly describes the barriers that suppressed further fire spread.

The two major areas affected by fire, in the western regions of Kobe, comprised around 18 hectares of mainly low-income suburbs. These contained blocks of older wooden houses and local light commercial buildings, separated by narrow lanes.

This type of suburban layout also suffered severe damage in the central and northern regions of Kobe and its environs. However, the central and northern regions suffered only small pockets of subsequent fire devastation. The reasons were that in the western areas of Kobe there was a lack of available water and a lack of Fire Service intervention. This left the only barriers to fire spread being main city streets, two lanes plus parking in width, or lines of modern buildings with fire-resistant external wall construction.

These wider streets were not complete barriers to fire spread. In some instances, fire spread across them. The mechanism of fire spread was not by radiation, but through embers carried on the wind across the street and dropped onto combustible material from earthquake-shattered buildings on the other side.

The water supply to the Kobe area comes from Osaka, to the north, and the main supply pipes were fractured at the northern end of the region of severe shaking (see Section 4.4 for more details). While this cut water supplies to the central and southern regions of Kobe, water was restored in the northern regions sufficiently quickly to assist in fire fighting. Isolated pockets of fire in these areas were therefore able to be extinguished through the use of water and Fire Service intervention.

14.3 Effect on Buildings

14.3.1 *Structural Fire Severity*

The reconnaissance team estimates that the general structural fire severity of the fires that devastated the 18 hectare region of Western Kobe was between 15 and 30 minutes time equivalent. (The time equivalent is the time under the standard ISO furnace test). Localised pockets of greater fire severity occurred.

This estimate is based on the effect of the fire on numerous low- and medium-rise steel framed buildings in the affected area.

14.3.2 *Effect on Structural Steel Buildings*

Two general structural steel gravity load-resisting systems were employed in the medium-rise steel framed buildings in the fire-affected area. The first comprised truss beams supporting a light concrete floor on steel deck and the second utilised I-section beams to support this concrete floor on steel deck.

The second form of construction suffered very little distortion in the fire, as shown in Figure 14.1. The first form suffered considerably more damage, typically with collapse of the roof and the upper level and distortion in the lower levels.

14.3.3 *Effect on Concrete Buildings*

The concrete buildings in the fire-affected areas of Western Kobe were of old stock and suffered severe damage in the earthquake. It appeared that the subsequent fire did not cause significant further damage to the structural frames.

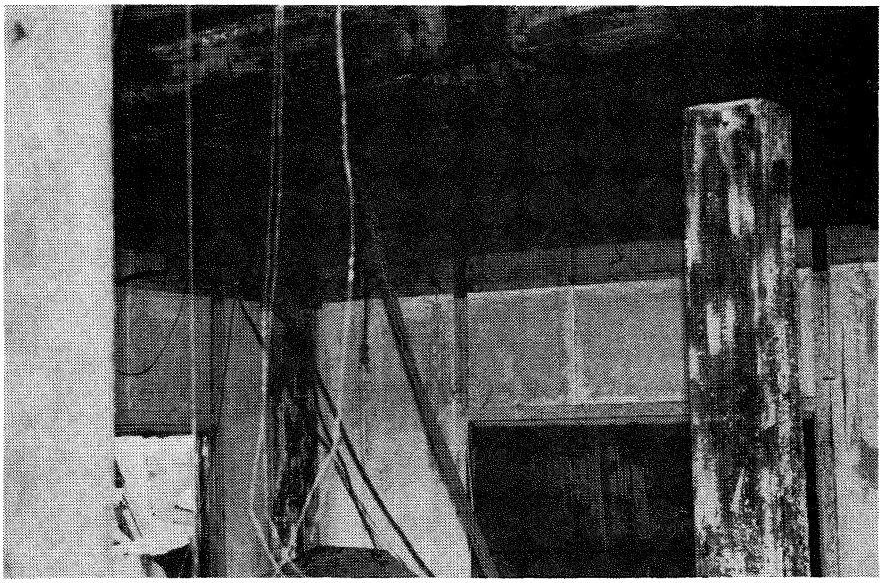


FIGURE 14.1 Three Storey Modern Steel Building, With Unprotected Steel Frame, After Fire Following Earthquake. Kyogo Prefecture.

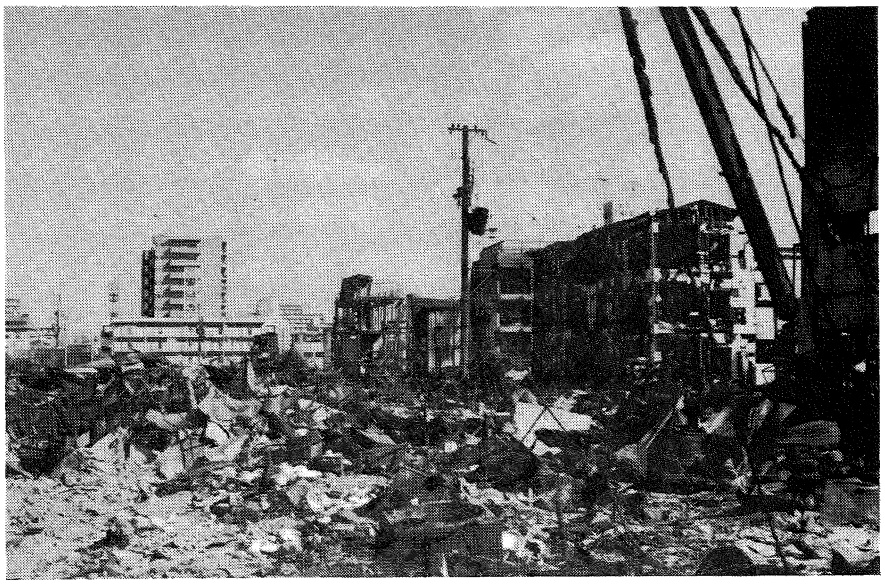


FIGURE 14.2 General Picture of Devastation, Fire Following Earthquake. Hyogo Prefecture.

14.3.4 *Effect on Timber Buildings*

Timber buildings suffered complete destruction in the earthquake plus fire, with very little of the timber frames remaining and combustible contents mostly consumed. An example of general destruction of terraced timber buildings is shown in Figure 14.2. The burned out frame of a multi-storey steel framed building is visible in the background.

15 LESSONS FOR NEW ZEALAND

15.1 Introduction

There are many lessons to be learned from observing the damage caused by a large earthquake. The proximity of the Hyogo-ken Nanbu earthquake to a major urban area make observations from it particularly valuable. Below is given a summary of lessons for New Zealand.

15.2 Specific Lessons

(a) *Seismicity, Geology and Tectonics*

The Kobe area is regarded as having relatively low seismicity for Japan, and the possibility of a major earthquake was much underrated. However, the seismicity of the area is within the range for New Zealand's main seismic regions and we in New Zealand are not likely to be as surprised by such an event as the Japanese were.

There was a long, narrow zone of severe damage in Kobe (see sections 2.4 and 5.1). A map published in a popular magazine about 10 days after the earthquake (see Figure 2.3) defined a

curved zone, about 2-3 km wide, that appeared to follow a prehistoric shoreline, and a second map produced about a week later by the AIJ (see Figure 5.1) showed a narrower zone, about 1 km wide. Our observation was that the level of damage decreased very rapidly with increasing distance from the zone. Moving to the northwest, the intensity of shaking appeared to decrease from about MM10 in the zone to about MM6-8 at the foot of the Rokko mountains, over a distance of 2-5 km. Moving to the southeast, the intensity decreased from about MM10 to about MM8-9 along the coast and in the large reclamations, over a similar distance.

In comparison with the Hyogo-ken Nanbu earthquake, the "Wellington fault" earthquake [Van Dissen and Berryman, 1991] is expected to be larger (Moment magnitude: 7.1-7.8 as against 6.9) and to have a longer fault rupture (75 km versus 60 km) that extends to the surface over much of its length.

If the zone of severe damage is fault controlled then, for the Wellington case, we could postulate such a zone extending 2 km either side of the Wellington fault and stretching from Cook Strait to near Kaitoke. Such a zone would encompass most of the central business district in Wellington and most of the Hutt Valley floor. An isoseismal map for a strike-slip earthquake of moment magnitude 7.5, generated by calculating isoseismal areas with the Dowrick [1992] formula and assuming effective fault lengths of 35 and 50 km, respectively, for the MM10 and MM9 isoseismal lines, is plotted in Figure 15.1.

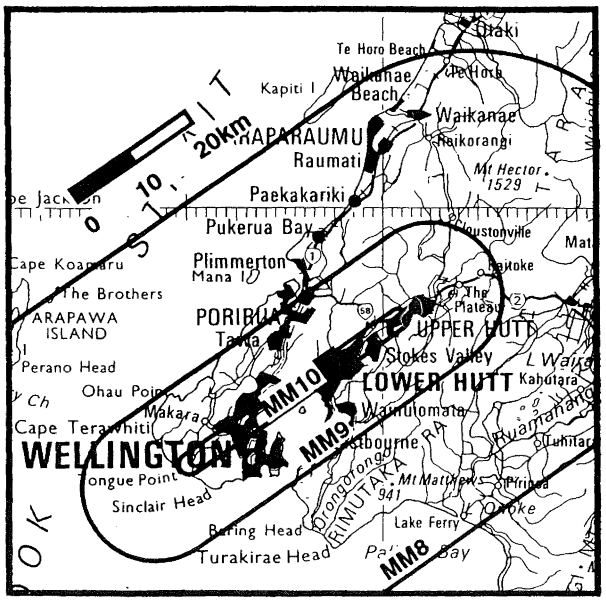


FIGURE 15.1 Isoseismal Map for a Moment Magnitude 7.5 Earthquake Located on the Wellington-Kaitoke Segment of the Wellington Fault.

If on the other hand the zone of severe damage is determined in some way by the width, thickness and composition of alluvial layers then, for the Wellington fault earthquake, we could possibly expect to see patches of severe damage to appropriate classes of buildings on alluvial soils at distances of up to 5-10 km from the Wellington fault. Wainuiomata and most of the Porirua basin lie within 10 km of the Wellington fault.

There were several recordings of peak ground accelerations (PGAs) in the range 0.5 to 0.8g, some from sites at nearly 10 km from the "fault zone". Similar accelerations were also recorded during the Northridge (California) earthquake of January 1994. Accelerations of these levels have been used in the design of some special purpose buildings in New Zealand, but the 450 year design spectra in the current New Zealand loadings standard, for the worst seismic zone, are based on a PGA of 0.4g. This loading level may well correctly reflect the 450 year hazard in most of the zone, but is likely to be an under-estimate of the true hazard close to those major active faults which rupture at intervals of less than about 500 years. The Wellington and Hope faults are two examples. Although there are very few such faults in New Zealand there is clearly a need for a special "near fault" category in the next revision of the New Zealand loadings standard.

(b) ***Ground Damage***

The widespread liquefaction as a result of the Hyogo-ken Nanbu earthquake which caused lateral spreading and ground subsidence throughout all the reclaimed areas is not likely to be so prevalent in the New Zealand situation, particularly Wellington, as truly liquefiable sediments have not been found to be as extensive or thick as in Osaka Bay.

Liquefaction induced damage to street and road pavements is likely to be limited to tension cracks (typically less than 50 mm) except near waterfront structures. Lateral spreading resulted in severe ground damage within 70 to 100 m of waterfront structures in Kobe.

The liquefaction did not appear to cause structural damage to well founded buildings, but settlement of the ground around such buildings appears likely to have caused multitudinous breakages to buried pipework. Above-ground services, such as electricity supply and the telephone system, appeared not to be greatly damaged by the liquefaction.

There has been some debate in New Zealand over whether a 20 m exclusion zone for structures in the vicinity of an active fault trace is necessary because it is suggested that the ground shaking alone would cause so much damage that ground rupture damage would be superfluous. Evidence at Toshima on Awaji Island and at Nishinomiya is that the ground rupture which passes through structures causes much greater damage than shaking alone. It is appropriate to include an exclusion zone, the width of which will depend on the nature of the fault rupture expected.

Landslides were much fewer in the Kobe area than would be expected in areas of New Zealand with similar topography for several reasons:

(i) It was the dry season.

(ii) The granitic rocks exposed in steep slopes in the mountains do not have the planes of weakness (bedding planes, clay-lined fault planes, persistent closely spaced joints, etc), that are typical of many New Zealand rocks (for example, greywacke). This means the rocks in the Kobe area are much more homogeneous and with better rock mass quality than most New Zealand rocks.

(iii) As a result of the rainfall during frequent typhoons in the area there is little unstable regolith is present on natural slopes, and where the slopes are modified by cutting for road, rail and buildings, very extensive slope protection is installed to a much more lavish level than seen in New Zealand. This protection is provided for the typhoon rainfall, and means that the slopes are well protected against earthquake-induced failure as well.

(iv) There are no major fault scarps in the Kobe area analogous to the Wellington and Wairarapa faults. This means that there are no obvious unstable fault line scarps, and less rapid river downcutting. That is, the Kobe area landscape has formed under a much less active regime of vertical deformation than is common in New Zealand.

The conclusions are therefore that New Zealand earthquakes have caused, and will in future cause, much more slope instability than observed in Kobe.

(c) ***Lifelines***

The earthquake was notable for the widespread damage to underground pipes and elevated transport structures. The extent of damage can in part be attributed to the extensive soft soils and reclamations, and to the urban area being concentrated along a 30 km length of faultline. This situation has close parallels in the Wellington-Hutt Valley area (except that the Wellington fault is larger and more active), but there are also many other towns and cities in New Zealand with extensive areas of soft soils.

While many newer buildings located on reclaimed land survived with little damage, underground pipes appear to have been relatively heavily damaged. Elevated roads and railways in these areas were also damaged, including the recently completed Wangan Expressway. The continuity of these lifelines over long distances makes them vulnerable to the large differential displacements and permanent ground deformations commonly experienced in soft soil areas. Provision of adequate axial displacement capacity and lateral flexibility is often more important than strength in these situations. Lifelines that cross boundaries between soft soils and rock, and buried services entering a building, are particularly vulnerable. However, the good performance of welded ductile steel pipelines and pipelines with flexible joints is encouraging and confirms that the pipeline materials now being used in New Zealand can be designed to survive significant ground deformations.

Ground displacements are also likely to have contributed to the damage sustained by the sewage treatment plants located on the Nagata-Higashinada coastline.

The extensive damage to gantry cranes and other port facilities was also primarily a result of large ground deformations. This type of damage has been observed in many past earthquakes. The vulnerability of these facilities needs further investigation. In Kobe, comparatively moderate expenditure on seismic upgrading could have prevented much of the damage to the port facilities. Again, potential ground displacements need to be carefully considered in the design and the effects minimised.

The multiple failures of expressways, rail lines, and water trunklines highlights the desirability of providing alternative lifeline routes that pass through geologically different and preferably less vulnerable areas. This particularly applies to existing lifelines that have not been designed to ensure ductile behaviour, or that are vulnerable to large ground displacements. The early restoration of rail services into the centre of Kobe via the Shin-Kobe tunnel under the Rokko mountains highlights the value of having completely different access routes. The extensive damage to elevated structures also showed the value of alternative arterial surface roads. The rapid establishment of emergency vehicle access also needs to be a priority.

Wellington is particularly vulnerable in respect to road access, with both main road routes into the city vulnerable to landslides and fault rupture. A secure alternative (tunnelled?) route would significantly assist recovery and restoration following a major earthquake.

Overhead electricity and telephone cables were extensively damaged in the worst affected areas due to fires, tilted and collapsed poles, and collapsed buildings. Collapsed buildings should be much less of a problem in New Zealand residential areas, but consideration needs to be given to the vulnerability of utility poles, especially those with attached transformers. As in many past earthquakes, substations were one of the most vulnerable components of the electricity supply.

In Kobe, truck mounted generation units were used to supplement the electric power supply during the restoration period. Other alternative sources of electric power generation such as ships could also assist in the restoration of supply.

The rapid recovery of the electricity supply, and extensive repairs evident nine days after the earthquake, show that overhead cables can be reinstated relatively quickly, but it needs to be recognised that similar levels of resources would not be available in the event of a major earthquake in New Zealand. Official figures also indicate that the cost of repairs the distribution system will still be substantial, despite the rapid recovery of supply. Strategies also need to be developed to minimise the risk of fires caused by the restoration of electricity (only restored to occupied houses?) and there needs to be close co-ordination with gas supply utilities.

In Kobe, many people did not return to collect possessions from damaged homes until 10 or more days after evacuating. Many of these left in the dark with personal possessions strewn about by the earthquake. Because there was no electricity at the time of evacuation, many heaters could have been left on. The importance of turning off the electricity at the building mains before evacuating in these circumstances needs to be emphasised.

Most New Zealand gas utilities have or are in the process of converting gas mains and branch lines to welded ductile steel or polyethylene pipes. Consequently the distribution systems are expected to survive a large earthquake with minimal damage. However, as in the Northridge and other earthquakes, house pipes remain a potential source of gas leakage and fires. A detailed review of the economics and performance of the intelligent meters used in the Kobe area is recommended.

The good performance of underground telecommunication cables demonstrates the value of ducting in providing protection against ground displacements, even though the ducts themselves may be damaged. The failure of several emergency communications systems is, however of concern and there appears to be a need to more closely examine potential weaknesses in emergency systems. Cellular phones could possibly be used as a back-up in many situations. Another feature of interest is that although the public switched network in the emergency area was heavily overloaded immediately following the earthquake, pay phones were not affected as they run on separate lines. This is believed to be not the case in New Zealand. Options ensuring that pay phones remain functional in cases of overloading should be examined, at least for local areas affected by an emergency. In Kobe, pay phones provided valuable means of communication for quake survivors.

The important role of the news media needs to be recognised.

The establishment of Earthquake Lifelines Groups in Wellington and Christchurch over the last few years means that utilities in these areas should be better prepared to cope with a Kobe type emergency. Particular encouraging is the degree of co-operation that has developed between the various utilities and the advance emergency planning that is now being undertaken.

(d) ***Behaviour of Engineered Building Structures***

Modern buildings in Kobe, which were designed and constructed in accordance with the most recent (1981) Japanese seismic code, performed well on the whole. Care is needed when extrapolating this finding to New Zealand structures. The design seismic forces used in Japan are higher than those used in New Zealand for ductile design, but nevertheless the use of capacity design in New Zealand, and the ductile detailing used in New Zealand for a greater expected ductility demand than in Japan, gives confidence that the current design standards in New Zealand are adequate. This emphasises the need in New Zealand to enforce our current building design and construction standards strictly, and justifies the considerable upgrading of our seismic standards which has occurred since the mid 1970s commencing with the general design and loadings standard NZS 4203:1976. This upgrading involved the introduction of capacity design and detailing for different levels of ductility demand.

Many older buildings in Kobe did not fare so well - typically buildings of the 1950s and 1960s which were designed to old (now sub-standard) standards. Many suffered failure of columns (reinforced concrete, steel reinforced concrete and structural steel) which caused soft storey collapses. Those buildings had been designed according to the standards of the day. This earthquake emphasises again that many of New Zealand's older buildings need to be assessed for seismic resistance in the light of current seismic design standards. Some older structures are inherently strong and have satisfactory earthquake resistance. Others are not strong and are not ductile and need to be retrofitted. It also has to be borne in mind, as is emphasised above, that the seismic forces used in design in Japan correspond to slightly greater than those associated with limited ductility design in New Zealand, and hence more ductility is required of New Zealand ductile structures to survive a severe earthquake similar to the Hyogo-ken Nanbu earthquake.

The lesson is that buildings which did not have good seismic detailing ensuring ductile behaviour in the inelastic range often performed poorly, with severe damage and collapse common in areas of high intensity shaking. Almost all the failures of medium rise buildings could be attributed to a lack of capacity design to prevent brittle failure mechanism and a lack of ductile detailing, plus some failures which could be attributed to inadequate lateral strength.

(e) ***Bridges***

The recently (post-1980) designed and constructed bridges performed quite well. Damage to bridge piers which caused loss of several spans was primarily due to liquefaction and lateral spreading effects. Damage to bridge bearings and road joints was however quite extensive. In applying these findings to modern New Zealand bridges, similar comments to those made for modern building structures in (d) above apply. Of particular note is the need to carefully consider the effect of seismic movements on bridge bearings and road joints at articulations.

Older elevated bridge structures (pre-1980) in general performed poorly with major damage occurring to many bridge piers and to bridge bearings, seismic restrainers and road joints. A significant number of spans collapsed fully or partially. Had the piers for these structures been retrofitted to improve the known deficiencies in their seismic performance, the major damage and collapse which occurred would have been greatly reduced if not avoided altogether. This emphasises the need for New Zealand to continue with the active seismic assessment and retrofitting of vulnerable bridges in the country. Older elevated structures with inadequate ductility or inadequate details to prevent span collapses proved especially vulnerable in Kobe and they need to be given priority in New Zealand.

The effect of ground movements on bridge piers at waterway margins was very significant in the reclaimed areas. This illustrates the need to properly consider these effects for both new and existing structures in such locations. Settlement of bridge approaches in areas of alluvial sediments which experienced liquefaction was common. Reinstatement of access on to bridges is readily achieved with temporary filling. Abutment settlement slabs provide an effective ramp transition.

Longitudinal seismic actions along long elevated bridge structures appeared to have been very significant in the Hyogo-Ken Nanbu earthquake This matter needs to be studied further when more data is available. Consideration of such effects, presumably, due to travelling ground waves, should be given when evaluating the seismic performance for long elevated structures.

Finally the similarities between the Kobe area and Wellington area are notable. Both contain long elevated structures, running beside active faults, and passing over reclaimed land. There are also similarities between the age and construction of some of the bridge structures. There are lessons for other areas also. Two major bridges with spans similar to the Auckland Harbour Bridge suffered significant damage, in one case rendering the structure inoperable.

(f) ***Industrial Facilities***

General

Industrial facilities suffered significant damage. Disruption of business occurred to almost all industries in the Kobe area, regardless of the type of industry. Industries in seismic areas need to understand and plan for such effects.

Industries located in lower seismicity areas with suppliers in active seismic areas need to consider the possible impact on their business. Major industrial facilities outside the Kobe area were disrupted quite severely because of the widespread use of the "Just-in-Time" philosophy of low or zero inventories and in some cases the use of sole suppliers.

Port Facilities

Almost all reclamations in Osaka Bay are retained by gravity caisson wall structures, none of which were lost completely, but which mainly tilted outwards with resulting ground subsidence, cracks and liquefaction sand boils behind. The gravity caisson wall structures which retained the reclaimed islands rotated and moved laterally but 2 to 3 m resulting in lateral spreading of the ground extending often as far as 100 to 150 m behind the port breast works. Severe damage was inflicted on container cranes and adjacent buildings. In contrast, most reclamations in New Zealand (Wellington at least) are retained by walls or sheet

piles, whose performance may not be analogous to those of Osaka Bay in an earthquake.

The earthquake damage which completely cut all container port operations for sometime illustrates the need to understand and make provision for the impact that a major earthquake will have on such a facility. It is unlikely that Kobe Port will regain its previous status for sometime, if ever.

A more conservative approach to the design of wharf structures is needed, given the serious effect of their damage on port operations. Reduced displacements would result in less damage to adjacent structures, buildings and cranes.

(g) ***Houses***

Most of the damage to housing was suffered by traditional Japanese houses consisting of heavy roofs on a post and beam construction tied together by tenons and mortices. This type of construction is seldom encountered in New Zealand. Nevertheless, the Hyogo-ken Nanbu earthquake has highlighted some seismic deficiencies which can be found in the construction practice of residential wood framed buildings in this country, and the need to incorporate specific detailing requirements to ensure ductile behaviour. In particular, the following problems should be recognised and urgently addressed.

Many older houses in New Zealand incorporate heavy clay or concrete tiles on their roofs. As demonstrated during the earthquake, these top heavy structures are susceptible to soft-storey collapse, particularly if adequate structural connection between the roof cladding, wall framing and bottom plates is lacking. Lighter galvanised metal tiles reduce the roof weight without disturbing the architectural attributes of a tile roof.

Many chimneys unsupported above the roof of single family residences in New Zealand are built of unreinforced brick or stone and can collapse or fall over during an earthquake. If the chimney comes apart, the brick or stone may fall, damaging houses and cars and injuring people. To secure the tie from the fireplace chimney to the structure, a steel bracket can be added externally at the roof line as illustrated in Figure 15.2. The roof and/or ceiling framing must be adequate to restrain the chimney from moving. If necessary, the existing board sheathing of the roof can be replaced by plywood sheathing behind the chimney to prevent injury or damage from toppled bricks. This procedure may involve the removal of roof linings for the insulation of the plywood as a safety measure. Such plywood should be extended along the rafters about 1.5 times the chimney height, but not less than 1.2 m [RLA, 1991]. Plywood could also be added inside the building, over the ceiling joists.

Where roof pitches are steep, the chimney may be quite high. In that case, it may be more appropriate to remove a brick chimney, rebuild it with wood framing and face it with a thin brick veneer. If steel braces are used for the retrofit of a long brick chimney, as shown in Figure 15.3, care must be taken to prevent leakage where the braces penetrate the roof. Alternatively, braces could be horizontal and fixed to the top plate of the roof. Finally, patios, children's play areas or parking spaces should not be located near a questionable chimney.

As demonstrated during the earthquake, a house can move off its foundation during severe ground excitation if it is not properly anchored. This movement can cause a fire from broken gas lines, and damage to the foundation, floors, walls, windows and other utility connections as well as to the contents of the home. It is very expensive to lift a house up, put it back on its foundation, and repair this damage.

To insure proper shear force and overturning moment transfer between the structure and its foundation, anchor bolts can be installed through the sill plate and into the foundation, as illustrated in Figure 15.4. For single family residences, 75 mm x 250 mm long wedge anchors are usually sufficient. When the foundation concrete is of poor quality, epoxied anchors can also be used. One bolt should be installed for each 25 m^2 of floor area on each side of the foundation wall [Building Education Centre, 1993]. Alternatively, external steel straps, connecting the foundation to the sill plate, can be installed as shown in Figure 15.5 [SSC, 1992]. Also, nailing plates as connectors between members and solid blocking between joists are of great value.

(h) ***Urban Planning, Architecture, Secondary Elements and Building Services of Buildings***

The urban planning process should consider the likely impact of a major earthquake when considering such issues as road design, placement of major open spaces, location of strategic buildings and facilities. Specifically, city planning should consider:

- the construction of underground water tanks as part of urban renewal;

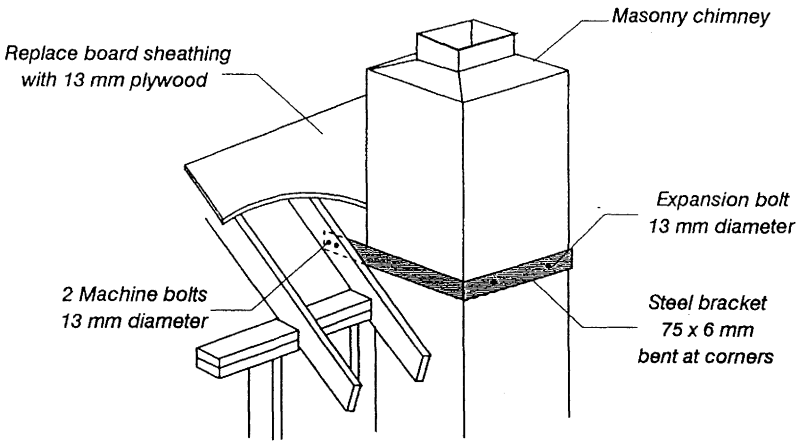


FIGURE 15.2 Steel Bracket Connecting Fireplace Chimney to Roof [RLA, 1991]

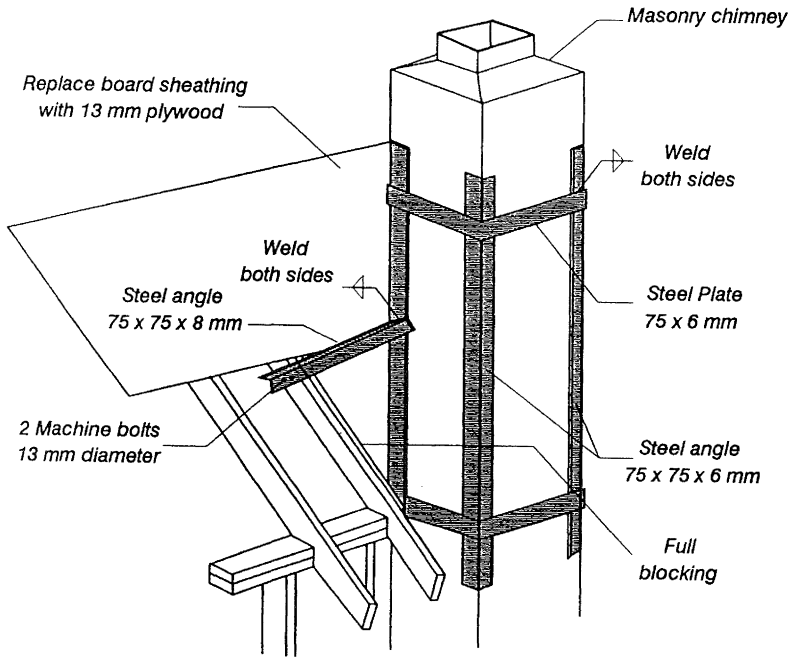


FIGURE 15.3 External Reinforcement for Existing Brick Chimney [RLA, 1991]

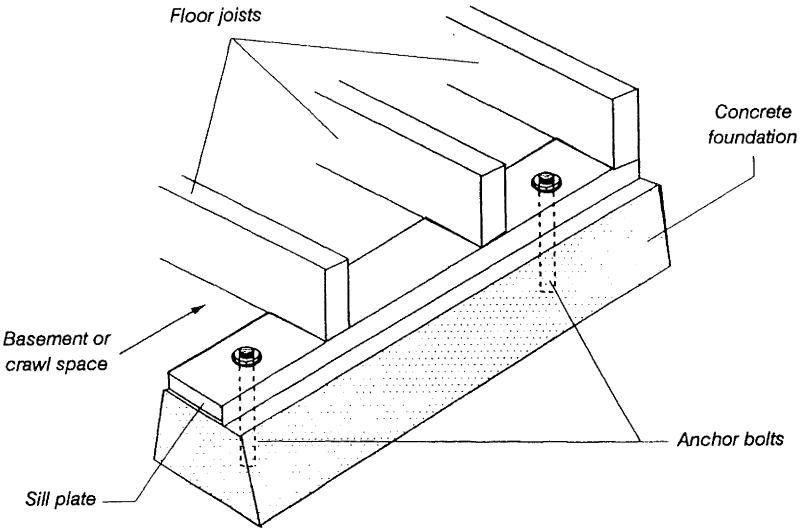


FIGURE 15.4 Anchor Bolts Through Sill Plate Into Foundation [SSC, 1992]

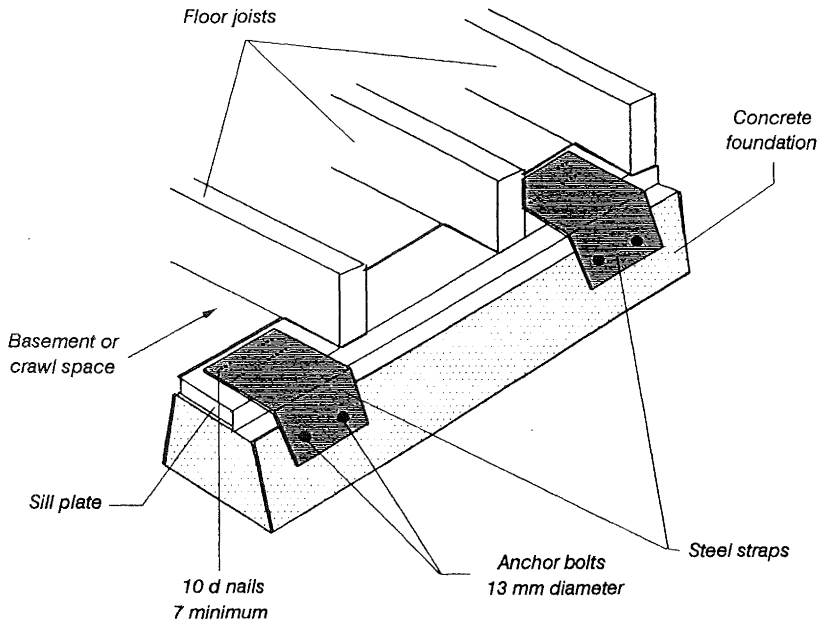


FIGURE 15.5 Foundation Connection Detail Using Exterior Metal Straps [Building Education Centre, 1993]

- the construction of some city streets wide enough for them to act as fire barriers;
- the development of urban roading considering alternate access routes;
- the development of sites within cities that can be dedicated as helicopter landing zones; for example, limitation to overhead electrical supply, limitation on the growing of trees, etc;
- the prevention of the construction of essential service buildings or complexes at the end of "dead end streets"; for example, the hospital on Rokko Island has only one form of access;
- the control of the growth of vegetation so as to limit or prevent accelerated fire spread, particularly in hill suburbs.

Architects should be aware of the impact of seismic activity on buildings as an input into the conceptual design process. As the design develops the requirements of seismic design in structure, claddings, services and secondary elements should be a major input than is instinctively understood by the Architect.

Earthquake audits of the structures of existing buildings could identify any needed structural upgrading work that would minimise the impact of an earthquake. This is well worth promoting.

Also, secondary elements in building services, building claddings, canopies and shop fronts must be designed with appropriate seismic separation and seismic fixing. The placement of major services such as water tanks should recognise the dangers of seismic activity on the building.

Organisations should also be encouraged to have an earthquake audit carried out on the interior of their premises recognising the chaos that an earthquake could inflict on the business. The need for filing cabinets, shelves, computer equipment and internal partitioning to be properly restrained needs to be recognised.

(j) ***Preparedness and Emergency Response***

The Hyogo-ken Nanbu earthquake will have a profound effect on Japan's earthquake preparedness. In the future more emphasis will be given to preparedness and emergency response. In the past Japan's disaster mitigation strategy has been heavily dependent on structural (building standard) measures. Criticism is being levelled at government in Japan for its slow response and inadequate preparedness for the effects of a major damaging earthquake.

Preparedness and emergency response for a major earthquake in Wellington or anywhere in New Zealand is also essential.

(k) ***Impact of Fire***

The impact of fire was less in Kobe than it could have been, even though some fires developed unchecked for some time in some areas, mainly due to lack of water and lack of fire service intervention. The fire spread rapidly through the closely spaced wooden houses.

The principal lesson for New Zealand is the impracticality of stopping fire spread without water. Given that reticulated water supplies are likely to be disrupted, the most practical means of getting water to the fire would be by air. Attacking fires by air would require rapid response and good co-ordination, in accordance with a previously prepared plan. Such a plan would need to consider the great variation in resources that would be required, depending on the location of the earthquake, the topography and the prevailing weather conditions.

In New Zealand aggressive fire attack strategies also need to be developed for those areas of urban development that are at risk from radiated heat transfer due to housing being constructed close together, or a combination of radiated and convected heat transfer due to housing being sited vertically close together in steep hill suburbs.

The built-up areas most at risk from widespread fire are the suburbs. The commercial business districts are likely to be least at risk, with the risk in industrial districts depending on the industry. Reinforced concrete and steel framed multi-storey buildings are likely to exhibit good post-earthquake stability under fire attack. However, fire will spread through such buildings unless active suppression systems remain operational after the earthquake. A holding tank supplying, for example, five sprinkler heads for 30 minutes operation would be of considerable assistance in multi-storey buildings. This should be located in the upper level, to allow gravity feed if need be.

15.3 General Conclusions

The 20 seconds of strong ground shaking of the Hyogo-ken Nanbu earthquake has provided many important lessons for urban areas of New Zealand. Of these several are paramount:

- New Zealanders must not be complacent in their consideration of earthquakes. The attitude or belief "that it will not happen to us" or "it will most likely only occur in Wellington" needs to be eradicated.
- Buildings and bridges designed and constructed according to modern seismic standards in general survived the earthquake well. This justifies the design and construction provisions of current standards (which are very much more severe than older standards) and emphasises the need to enforce current standards strictly.
- The earthquake hazard of older structures was demonstrated. Many mid 1970s and older buildings and bridges in New Zealand may need retrofitting.
- The need for lifelines of cities to have adequate seismic resistance was very apparent. A city will suffer severe economic loss and disruptions if the utilities are not operating and transport is not flowing freely after an earthquake.
- The importance of disaster preparedness was emphasised. The community at large must be made aware of the factors that will, in all probability, contribute to a reduced capability of the emergency services, and they must be made aware of the self reliance situation that they will inevitably face.
- The importance of disaster emergency response services was demonstrated. There will need to be significantly greater interagency coordination and cooperation. Planning will need to take on a far greater importance. These plans will have to be both realistic and flexible for them to have any degree of effectiveness.

15.4 References

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