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

A FIELD REPORT BY EEFIT

ACKNOWLEDGEMENTS

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The report was edited by Dr Adrian Chandler, of University College London and Antonios Pomonis of Cambridge Architectural Research Ltd.

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SUMMARY OF FINDINGS

1. General observations on the effects of the earthquake

(a) The Hyogo-ken Nanbu earthquake is the second worst in loss of life after the 1923 Great Kanto earthquake that destroyed large parts of Tokyo and Yokohama. The part of the fault system that moved during this earthquake is not known to have moved during historic times and no earthquake of $M \geq 7$ had occurred within a 50 km radius from Kobe in historic times.

(b) The direct monetary losses of the earthquake were estimated at around US$ 100 billion. Damage to the building infrastructure was the biggest contributor to this loss (59%). In absolute terms this is the costliest natural disaster in world history. However some of the costs are artificially inflated by the unusually high price of land in Japan, that drives the unit cost of infrastructure to much higher levels. In relative terms these losses amount to 2.2% of Japan's GDP and are less than Japan's 1994 trade surplus (US$ 121 billion). The cost of business interruption could be at least as much as the direct losses.

(c) The worst affected areas were 6 wards of Kobe city and the towns of Ashiya and Nishinomiya (immediately east of Kobe city). A 0.7 - 1.2 km band of very severe damage was observed and the Modified Mercalli intensity in this band has been estimated as X or somewhat higher. Peak ground accelerations and velocities in these areas ranged from 837 to 564 cm/sec^2 and between 55 and 30 cm/sec, respectively.

(d) The population density in the worst affected wards and towns was between 6,500 and 4,300 people per square kilometre. Some of the worst affected neighbourhoods were densely built-up with old traditional Japanese low-rise timber frame dwellings, with an average per hectare population density roughly double that commonly used in UK new town planning.

(e) In Kobe city it was estimated that almost half the population lived in multi-storey apartment buildings predominantly made from reinforced concrete structures. The worst affected areas had a high proportion of pre-1960 wooden dwellings, ranging from 38% in Nagata-ku to 14% in Suma-ku. Other areas of Japan, most notably Tokyo, have a much lower proportion of such old and seismically vulnerable dwellings. The proportion of people made homeless by the earthquake exceeded 20% of the population in five wards of Kobe and in Ashiya city.

2. Seismological aspects

(a) The Hyogoken Nanbu Earthquake was caused by a right-lateral strike slip movement of the Rokko-Arima-Takatsuki fault system in Kobe and the Nojima fault in Awaji Island. Significant surface rupture was clearly observed along the 11 km long Nojima fault. In Kobe, ground surface deformation such as pavement cracks was found at various locations along the Rokko fault system. However, it was difficult to determine these deformation features as surface rupture due to fault movement.

(b) Seismological analyses by several sources indicate that the earthquake had a moment magnitude of $M_w = 6.7$-6.9 and a focal depth of 14.3 km. Four foreshocks of a magnitude between 1 and 4 were recorded 24 hours before the earthquake in the region. The distribution of aftershocks makes a 50 km strip in the Osaka-Kobe region and the depth is between 5 km and 15 km.

(c) The maximum accelerations were 500-800 gal in Kobe, Nishinomiya and Takarazuka, 300-600 gal in Amagasaki and Takatsuki, and 150-300 gal in Osaka and Kyoto. The near-field motions were intense pulses, whereas the more distant motions had longer duration of shaking, the longer period resulting from surface waves and site resonance. Large vertical accelerations were also recorded at locations close to the epicentre. Ground motions varied significantly in the epicentral region by a complex interaction of important effects such as local geology, topography and the directivity of wave propagation. The severely damaged area was concentrated in a strip of 25 km long and 0.7-1.2 km wide between Hankyu Railway line and Route 43.

(d) The vertical array data recorded by the city of Kobe at the Northwest corner of Port Island provided valuable site response. The dramatic change in strong motion records at different sites was probably caused by the softening of soils by liquefaction and by non-linear response of the soft alluvial layer.

3. Building damage and human casualties

(a) Damage surveys in the worst affected area have shown that the collapse ratio of mid-rise pre-1981 RC and steel frame buildings was unexpectedly high. In central Kobe as much as 15% of pre-1981 RC buildings collapsed. An analysis of 36 collapsed structures has shown that the average volume loss associated with these buildings was 18 to 22%, depending on the collapse type. This is much lower than that experienced in collapsed RC buildings in earthquakes from other parts of the world.

(b) It was found that the severity of ground shaking attenuated rapidly in the N-S direction.

(c) Unlike the 1923 Great Kanto earthquake fire did not affect a large part of the earthquake area. Only Nagata ward experienced a large conflagration that burned about 5% of its land area. The loss of life attributable to fire was less than 10%. The wind in the morning of the earthquake was very light. Had the earthquake occurred during the typhoon season (July to November) the potential for urban conflagrations would have been much greater.

(d) Around 61% of Hyogo prefecture's 5.4 million people lived in the worst affected areas. The population of the wards and towns that experienced intensity JMA7 was 701,000 people in total, and around 250,000 actually lived in the high damage areas. It was found that in Kobe's six worst affected wards and in Ashiya city around 20% of the surface area experienced such an extreme intensity of ground motion. Around 80% of the fatalities were in these areas. Around 90% of the loss of life was associated with the collapse of pre-1971 low-rise traditional Japanese houses.

(e) There has not been an earthquake in recent decades to have affected a heavily urbanised area inhabited by around 2 million people with intensity X or higher on the Modified Mercalli scale. It was estimated that among the quarter million people living in the areas of JMA intensity 7 the loss of life ranged between 2 and 4%, with the exception of Chuo ward in downtown Kobe, where this ratio was around 1%. The earthquake affected disproportionately the elderly population, because they tended to occupy the older vulnerable low-rise timber houses. It is likely that the loss of life would have been greater had the earthquake occurred 2 or 4 hours later, during the morning commuter rush hour or during office hours.

4. Insurance

(a) The losses suffered by the insurance industry were lighter than originally predicted. Only about 3% of residential and 30% of commercial property in Kobe area, was insured for earthquake damage. The premium rates for such cover are quite high in Japan. The insurance losses have been estimated at around US$ 1.25 billion, or around 1% of the direct damage. In the 1994 Northridge earthquake in the Los Angeles area, around 40% of the direct losses were passed on to insurance companies. As a result, the Japanese government and the affected people will have to bear most of the damage costs.

5. Reinforced concrete design/detailing

(a) The hydraulic pressure analogy used in the derivation of the rules for the design of column links in Eurocode has been justified by observation of the ovulation of links in rectangular columns.

(b) Cross ties are needed in rectangular RC columns, though with closely spaced links the need for lateral confinement to alternate bars as required in Eurocode 8 may be excessive.

(c) Joints become critical in RC members when beams and columns are properly designed.
(d) Short welded splices on links are failure prone, though whether this is due to shock, unzipping, defective or insufficient welding remains to be established.

6. Steel construction

(a) Steel structures are generally more flexible than those of reinforced concrete.
(b) The high strength, cold formed, tubular steel sections used in Japan are non-ductile.
(c) Certain types of concentric bracing with the weak axis perpendicular to the plane of the frame are highly ductile.
(d) Steel base plates tied down with holding down bolts on plinths tend to crush the plinths, whereas base plates tied down into massive foundations perform much better.

7. SRC construction

(a) Present day SRC performed well in all.
(b) Combining EEFIT observations with those of others it appears that earlier forms of SRC, with structural steel components of low ductility, perform better, but not very much better, than non-ductile RC construction.
(c) SRC casings are insufficient to ensure the good performance of plinth mounted base plates.

8. Failure modes in frames

(a) The surprisingly high number of mid-height single storey failure mechanisms is attributable to:
  (i) A change in the medium of construction above the level of the mechanism, and/or
  (ii) The distribution of storey shear used in the design of buildings sometimes is unconservative to that given by spectral analysis. This is relevant to buildings up to about 10 storeys in post 1981 designed buildings, for which the distribution is the more unsafe the lower is the building, and to buildings of any height in buildings designed before 1981.
(b) Strong beam/weak column characteristics are not synonymous with single storey failure. In fact, when the columns are all of the same size and all nominally reinforced, column hinges are likely to be distributed over more than half the height of the building. This distributed inelasticity is the consequence of the absence of hinges in the corner columns.
(c) In buildings with single storey failures (or more complex modes with similar characteristics as in (d), the parts of the building above the collapsed storey were invariably less damaged than comparable buildings without single storey failure.
(d) A number of buildings have been noted with partial collapse of a single storey, a mechanism comprising a partial storey failure and a partial bay failure. In most of the cases observed there was no obvious vertical element adjacent to the failed bay to induce this form of failure. This needs further investigation. It could be that, due to the generally large columns in Japanese buildings, the beam and column strengths are very similar despite the considerable depth of the spandrel beams. In this situation a modest stairwell might be sufficient to induce this mode of failure.

9. Frame/shear wall structures

(a) Where shear walls and framing members are employed in the same building to satisfy the minimum area requirements in the 1981 code, it is unsafe to assume the walls and columns carry shear pro-rata to their cross-sectional areas. The walls being stiffer and generally more lightly reinforced would be expected to suffer damage before the columns.

(b) In particular the less severe damage to internal shear walls is not necessarily reflected in columns within the façade frames parallel to the internal walls. However when the façade columns were damaged, damage to internal walls at the same level was likely.

10. Taller buildings

(a) The generally better performance of buildings exceeding a height of 31m (8 to 11 storeys) calls into question the Ultimate Limit State design exemptions for lower buildings, in terms of the omission of checks at Ultimate Limit State. The earthquake provisions for the lower buildings rely upon a Serviceability Limit State check for the moderate earthquake (0.20g) and nominal requirements as to the minimum cross sectional area of vertical members. Only four taller buildings were found that were badly damaged.

(b) The greatest spectral magnification nevertheless occurred in the taller buildings, being in 14 storey buildings on moderately firm ground and 19 to 22 storey buildings on soft ground.

(c) On the supposition that the regulations had been strictly enforced, all the buildings taller than 31m (8 to 11 storeys) would have been of post 1981 construction. However two of the four buildings of this height found to be damaged, by their appearance, are considered to have been of earlier construction; so the question arises of how rigorously the earlier regulations were enforced.

11. Multi-storey car parks

(a) Excellent performance can be obtained from multi-storey car parks using steel and SRC construction.

(b) Excellent performance also can be obtained from very light and flexible construction, which can be exploited more in this class of urban structure than in others.

12. Link bridges between buildings

(a) Link bridges may be sufficiently rigid to cause damaging impact between buildings on closure of the joints.

(b) Nominal fixings of the top booms of trusses into buildings above movement joints in the deck (possibly a measure to simplify the cladding at the joint), increase the severity of its impact, cause local spalling damage and may even damage the bearings.

(c) More attention generally is needed in the design of the bearings.

(d) From the number of fallen link bridges reports, particularly in the higher stories (only one is considered in this report which failed for other reasons) the bearings should be designed for larger movements.

13. Damage prone construction

(a) The method of anchoring the plank-like façade units on steel frames is prone to failure.

(b) Precast concrete facing panels are a source of weakness as they add mass but have little effect upon the strength.

(c) Landing beams should, where possible, be tied into columns or walls in line with the beam.

(d) Buildings in the order of 120m long without seismic joints may be expected to suffer damage over the middle third.

14. Foundations and site effects

(a) Landslips apart there were no catastrophic building failures attributable to the failure of piled foundations.
(b) Whilst there are some reports of pile fracture it is considered that in general the ubiquitous piled foundations performed well. An indication of the likelihood of damage may be obtainable from the ratio of the permanent set of the ground to the pile diameter.
(c) Where liquefaction occurred the ground subsided around the buildings. In one, the subsidence was sufficient to expose the piles, which were undamaged.
(d) Corner sites at road intersections are damage prone needing special design consideration.

15. Joints

(a) For the present earthquake and ground conditions in Kobe expansion joint widths and separations between buildings of 50mm are adequate only for rigid shear wall structures of low to moderate heights on a single foundation.
(b) Where foundations are discontinuous, as between buildings, larger joints are necessary.
(c) Largest separations are needed between buildings on corner sites.

16. Base isolation

(a) Evidence has been presented of the ability of base-isolation techniques to successfully mitigate the effects of this strong earthquake both in terms of people safety and the integrity of the structure and its contents.
(b) Base-isolation is an effective means for enhancing the seismic performance of hospitals and other buildings housing essential services.

17. Repair and strengthening

(a) There is an urgent need to establish reliable and cost-effective techniques to evaluate, repair, strengthen/retrofit existing structures and buildings in Japan that suffered minor or localised damage. Where there is suitable access to below ground structure or where the cost of excavation can be justified, base-isolation may be considered as an alternative technique for retrofitting.

18. Damage to bridge structures

(a) The impact of the earthquake on bridge structures was substantial, and the severed transport arteries added greatly to the economic loss and suffering. The large number of single column supports to elevated roads and railway led to more collapses than otherwise would have been the case. Generally the worst affected structures were the older ones, as codes improved, so did the performance of the bridges. However, there were failures of modern structures, and there are still lessons to be learned.

(b) Piled foundations performed well, but movements of substructures were large in areas of liquefaction, and the span losses serve to re-emphasise the need for a fail-safe approach to avoid spans being dislodged.

19. Damage to industrial facilities

(a) The great majority of damage to industrial facilities resulted from differential displacements, imposed by soil movement. The response of piping to reasonable levels of displacement was generally good, provided the supports allowed flexibility and did not force the displacement to be accommodated in a short length. In many cases, the movements could have been allowed for, if this failure mode had been considered in design. In the case of gross displacements, such as several metres, piping generally failed, as would be expected. Such failures could have been avoided only by fundamental changes in foundation philosophy, wharf construction or soil compaction.

(b) Failures also occurred as a result of items of plant not being bolted down adequately. This type of failure is not uncommon, but would be relatively inexpensive to prevent. Damage to industrial buildings, and hence their contents, resulted from poor (non-ductile) RC details. Steel buildings generally avoided collapse, and remained serviceable, although long term damage may have occurred to welded connections, not visible to the investigation team. Many small industrial units were destroyed by fire.

(c) Damage to the lifelines has been quite heavy and severe disruption was experienced in all utility services. Electricity was restored relatively fast but fires and explosions occurred when restored electricity combined with leaking gas pipes. The worst lifeline problems were experienced in water supply. Around 85% of the affected population lost their water supply and complete restoration took five weeks.

20. Geotechnical aspects

(a) The most significant geotechnical aspect of the Kobe earthquake was soil liquefaction. The effects of liquefaction were seen in ground settlements, sand boils deformation and tilting of structures. They were widespread along the shoreline and in reclaimed lands and extend as far as 3km inland from the waterfront in residential and industrial areas of Kobe, Ashiya and Nishinomiya. Liquefied sand erupted and flooded many places in Port and Rokko Islands. The interior of islands subsided 20 - 50 cm by liquefaction settlement. On the other hand, the improved ground sites by vibro-rod compaction or sand compaction piles sustained less deformation and damage than did the adjacent ground.

(b) The lateral spreads caused many concrete caisson quay walls in the region to move 2-5 m into the sea. This significant displacement was possibly caused by inertia of the heavy concrete caissons, large dynamic lateral pressures created by liquefaction of the backfill materials, and the reduction of bearing capacity of replaced sands under the caissons. The east section of the Maya Futo No. 1 pier was designed to resist large earthquakes by installing strong pile foundations or gravity type structures. Apparently, they survived the earthquake with almost no lateral and vertical movement.

(c) In the areas affected by lateral spreads, some piles under the buildings and bridges were found to be broken. Most of the pile supported structures in the interior of Port Island remained in place and the surrounding ground subsided around the structures. The caisson foundations of the Hanshin Bayside Expressway were pushed toward the sea by the force of lateral spreads, causing a girder to drop to the ground.

(d) Many tunnels and underground structures in Kobe survived the earthquake fairly well. The rock tunnels through Rokko Mountains were reportedly less damaged even by the movement of active faults in the mountains. However, some of the shallow underground structures of the subway systems in Kobe were severely damaged. The underground cut-and-cover concrete structure of the Daikai station collapsed by shear failure of the central reinforced concrete columns. The collapsed section was approximately 120 m long.

(e) Several gravity retaining walls along the Hanshin Railway line and the Japan Railway line failed by tilting and sliding of the walls due to the increase in lateral pressure and inertia force. On the other hand, the geogrid reinforced walls in the region survived the earthquake with minor damage of small hairline cracks, even where severe damage in the surrounding residential areas was observed.

(f) The dams in the region performed well with no major damage for protective measures (PRWL 1995). The only exception was the failure of the Niteko dam in Nishinomiya possibly due to soil liquefaction. River dykes alone the Yodo river in Osaka were also severely damaged by the lateral spreading.

(g) Several landslides and rock falls occurred on slopes in the Rokko Mountains. The landslide at Nikawa was the most disastrous one, resulting in thirty four fatalities. The size of the landslide was about 250 m long and 50 m wide.

1.0 INTRODUCTION

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1.1 Background to the EEFIT Field Investigation

EEFIT is a group of engineers, architects and scientists with an interest in earthquakes. It was founded in 1982 with the aim of reporting to the UK and international engineering communities the lessons to be learnt from damaging earthquakes. EEFIT has organised a series of field investigations, each of which has resulted in the publication of a report to disseminate its findings further. The Kobe Investigation provided a wealth of experience for the EEFIT team and the earthquake engineering community world-wide. This report is a distillation of the team's findings.

The Hyogo-ken Nanbu (Kobe) Earthquake of 17 January 1995 was (in terms of economic losses) one of the most costly natural disasters in history.

Although the official name of the earthquake was the Hyogo-ken Nanbu Earthquake, it was also known by various names including:
The Kobe Earthquake
The Southern Hyogo Prefecture Earthquake
The Hanshin Awaji Earthquake
The Great Hanshin Earthquake

1.2 The Field Investigation Team

The EEFIT Team consisted of:

Joseph Barr, Rendel Palmer Tritton
Chris Bolton, BNFL Engineering Ltd.
Adam Crewe, University of Bristol
Philip Esper, University of Westminster
Daman Lee, Ove Arup & Partners, Hong Kong Ltd.
Peter Merriman, British Nuclear Fuels Ltd.
Ted Piepenbrock, Ove Arup & Partners
Antonios Pomonis, Cambridge Architectural Research Ltd.
David Smith, Scott Wilson Kirkpatrick & Partners
Kenichi Soga, University of Cambridge

Antonios Pomonis preceded the main EEFIT mission, and was in the field just three days after the earthquake, from 20 to 28 January 1995. The main EEFIT mission had 8 members lead by Ted Piepenbrock, and was in the field from 2 to 10 February 1995. The affected area was re-visited by Kenichi Soga, who was this time accompanied by Philip Esper from 5 to 12 May 1995.

In addition various EEFIT members assisted or contributed to the efforts of other Field Investigation Teams world-wide. Notably, Kenichi Soga was part of the US National Science Foundation-sponsored Geotechnical Team, he was also funded by the EPSRC and Sir Alexander Gibb and Partners. Adam

Crewe assisted the ECOEST (European Consortium of Earthquake Shaking Tables) mission. Antonios Pomonis was sponsored by CARtograph Ltd. a company specialising in natural hazard risk assessment systems for the international insurance industry.

1.3 Contents of the Report

Chapter 2 includes a brief review of Japan's and the affected region's seismotectonics, a socio-economic profile of the earthquake affected region with emphasis on the residential building stock and a summary of the life and economic losses of the earthquake.

Chapter 3 gives details of the seismological, tectonic environment and the regional seismicity of the Kobe earthquake, and analyses the principal features of the strong ground motion recordings.

Chapter 4 describes the typology and history of Japanese buildings and earthquake codes. It also includes a summary of the damage forms observed in the field, presented through many building failure case studies and includes many photos of characteristic damage. It includes a review of structural assessments by others, including comments by EEFIT on an exceedingly well illustrated Japanese report on damage to structural steelwork.

Chapter 5 summarises the results of two damage surveys and compares them to other published damage statistics in order to obtain a preliminary assessment of the overall performance of the building stock, by age, structural type and other characteristics. It also includes an analysis of the causes and locations of human casualties as well as brief sections on the post-earthquake fires, damage to the lifelines and insurance losses.

Chapter 6 is a brief summary of the performance of bridge structures. It contains separate sections for elevated road and rail structures, including damage to concrete piers, steel columns and foundations. It also contains a brief summary on the effects of the earthquake on the Awaji-Kaikyo suspension bridge, as well as a section on the historical development of design loads and codes for bridges in Japan.

Chapter 7 assesses the performance of industrial facilities, including ports and harbours, power stations, tanks and vessels, piping, plant and machinery.

Chapter 8 covers a range of geotechnical aspects of the earthquake including a detailed assessment of liquefaction, ground settlement and related effects. Brief consideration is also given to the damage caused to tunnels and underground structures and retaining walls. Finally, the damage effects for dams, embankments and from earthquake-induced landslides are reviewed.

1.4 Presentations

Except from compiling this report, members of the EEFIT team have contributed to the wide distribution of their findings through a large number of presentations. Many of the internal lectures and presentations are not included in this short summary of some of the most well attended events.

An evening of presentations on the findings of the EEFIT team took place on 22 March 1995 at the Institution of Civil Engineers in London. Chris Bolton was awarded the Lancashire & Cheshire IStructE Branch Prize for a presentation on 5 April 1995. Antonios Pomonis has reported his initial findings in a presentation to the London Institute of Underwriters on 9 March 1995 and to a special session on the Kobe earthquake in the 11th World Conference on Earthquake Engineering in Acapulco, Mexico in June 1996. David Smith has made three presentations to local associations of the ICE and IStuctE.

2.0 THE EARTHQUAKE AFFECTED REGION

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2.1 Seismotectonics of Japan

Japan lies in one of the world's most seismically active and complex zones, where the Eurasian plate meets with the Pacific and Philippine plates. The constant west-north-westwards movement of the Pacific plate and the northwards movement of the Philippine plate occurs against the east-south-eastwards movement of the denser continental Eurasian plate. As a result of the converging movement of these tectonic plates, the lighter oceanic plates sink underneath the Eurasian plate forming two subduction zones east of Japan's Pacific coast. These are known as the Japan trench and the Nankai trough respectively. Great earthquakes occur in these areas frequently, but only the ones that are sufficiently close to the Japanese islands are destructive.

Due to these great tectonic movements the main Japanese island of Honshu is undergoing severe deformations such as bending and the creation of an area of dense and complex surface faulting in its centre. In recent years extensive studies have culminated in mapping the inland active faults of Japan (Research Group for Active Faults in Japan, 1991). Special attention has been paid to the study of the central Japan seismic zone, because the area has potential for large magnitude earthquakes with immense destructive power. Seven seismic gaps have been identified in various sections of the numerous active faults of this area (Kanaori, 1991, 1992a, 1992b) and the temporal relationship with the occurrence of great earthquakes on the Nankai trough has been investigated (UNCRD, 1995).

Finally another distinct area of seismic activity in Japan is a large zone that covers the interior of Northern Honshu and Hokkaido islands including the Japanese Alps that cover the area between Tokyo and Nagoya. Large earthquakes occur often on the Japan Sea side of this zone, and that is why it is often called the Japan Sea seismic zone.

Summarily the occurrence pattern of Japanese earthquakes leads to the definition of four seismic zones as follows:

A. Japan Trench Seismic Zone
B. Nankai Trough Seismic Zone
C. Japan Sea Seismic Zone
D. Central Japan Inland Seismic Zone

These can be seen in Figure 2.1. Significant earthquakes in each of these zones are discussed briefly in the following sub-sections. The seismicity of Japan is highly complex and this summary is only intended as a brief introduction to the subject (for the interested reader, key references are given in the text).

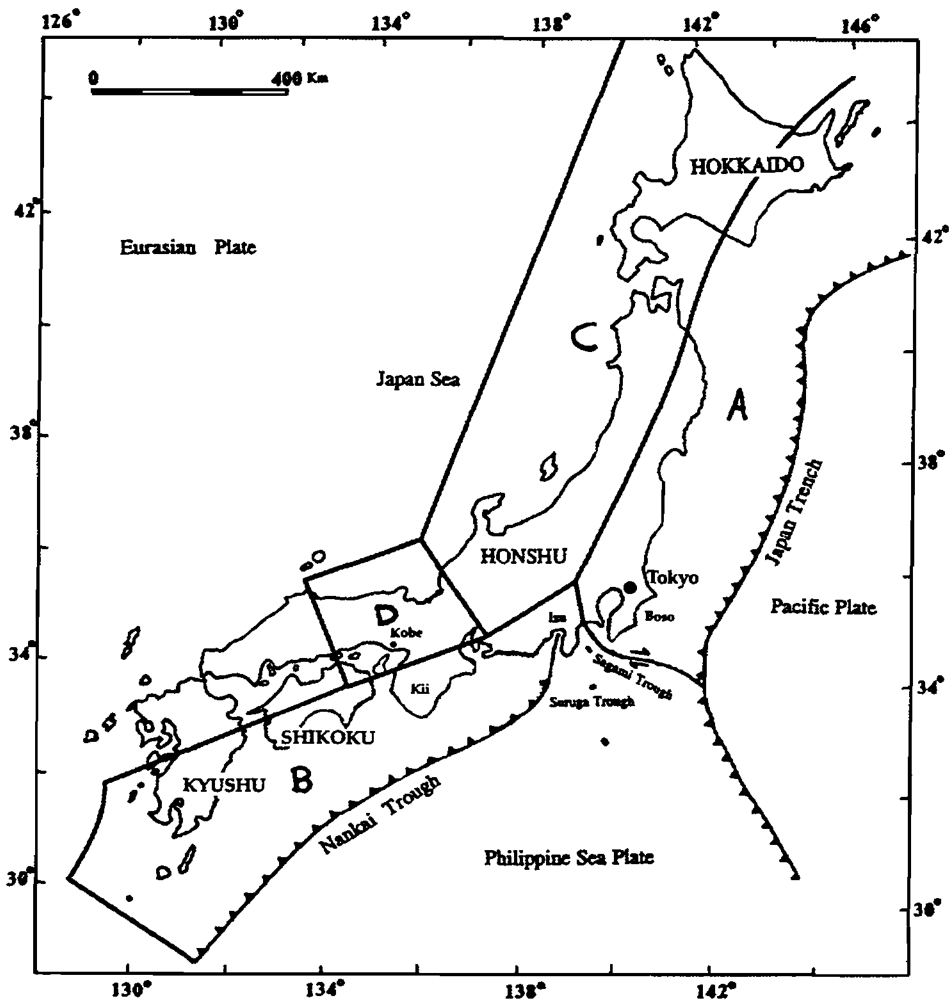

Figure 2.1: Major tectonic features of the Japanese islands.

A. Japan Trench Seismic Zone

This is the tectonic border line between the Eurasian and the Pacific plate extending southwards from the Kurile islands, to the east of the island of Hokkaido and continuing as far as the Sagami trough[1]. The seismicity zone is roughly parallel to the Pacific coasts of the islands of Hokkaido and northern Honshu. Most of the earthquakes in this zone occur around 30-km or more offshore, with the majority occurring at 20 to 70 km below sea level. However in the vicinity of the Kanto plain (the area that includes Tokyo) shallow earthquakes occur. The frequency of occurrence of large earthquakes (magnitude ≥ 7) is about one in every 10 to 20 years and the maximum magnitude expected is 8.5. A magnitude return period relationship based on an expression given by Okamoto (1973) is shown in Figure 2.2.

The most important earthquakes in this zone during this century were: the M8.1 Great Kanto earthquake of 1 September 1923, the M8 Off-Tokachi earthquake of 16 May 1968 and the M7.5 Off Miyagi-ken earthquake of 12 June 1978. These three earthquakes have triggered major progress in Japanese earthquake engineering and caused the 1924, 1971 and 1981 revisions of the Japanese Earthquake Code. Recent seismic activity of this zone has been in the northern part where two large earthquakes of M7.6 and 7.2 occurred in January 1993 and December 1994, off the eastern coast of Hokkaido.

[1] According to some seismologists the northern part of Honshu and the whole of Hokkaido are part of the North American plate and the extensive faulting in Central Honshu is interpreted as a collision zone between the Eurasian and the North American plate (EERI, 1995).

The Great Kanto earthquake destroyed most of Tokyo and Yokohama through a combination of shaking and post-earthquake fires and caused the loss of about 142,000 lives in Japan's worst natural disaster. The Miyagi-ken earthquake of 1978 seriously affected the town of Sendai in Northern Honshu and was the last earthquake in Japan to have caused serious damage to a large number of engineered structures, until the 1995 earthquake in Kobe. A repeat of the Great Kanto earthquake is being used in risk assessment scenarios for the prefectures of Tokyo, Kanagawa and Saitama, because it is considered as the event capable of causing the maximum probable loss in these areas (RMS, 1995). However, seismicity studies suggest that the recurrence of great earthquakes in the Sagami trough (source zone of the 1923 earthquake) is around 200 years (Annaka, et. al. 1988).

B. Nankai Trough Seismic Zone

This is the tectonic border line between the Eurasian plate and the Philippine plate running parallel to the Pacific coast of the islands of Kyushu, Shikoku, and western Honshu. The zone ends at the Suruga trough just west of the Izu peninsula. Earthquakes occur closer to the coasts, in the area between the Kii and Izu peninsulas. The frequency of occurrence of large earthquakes is somewhat lower compared with that of the Japan trench zone, namely once in every 30 to 50 years (see Figure 2.2).

The seismic risk in this zone is considerable since the earthquakes occur closer to the coast and they tend to be shallower. A large part of the Japanese industrial and urban development is concentrated in this area (Shizuoka and Aichi prefectures). The most important earthquakes in this zone during the 20th century were: the M7.9 Tonankai earthquake of 7 December 1944, the M6.8 Mikawa earthquake of 13 January 1945 and the M8.2 Nankai earthquake of 21 December 1946. The 1944 and 1946 earthquakes were off the coast of Shikoku island, while the 1945 earthquake was close to Nagoya city.

The Tokai region lies near the Suruga trough at the edge of this seismic zone. In December 1854 on two consecutive days (23rd and 24th), two earthquakes of M8-8.4 occurred in this area. It has been established that the chnaces of another great earthquake in this area are quite high (EERI, 1984). This is called the great Tokai earthquake and its occurrence is expected to cause severe damage in Shizuoka, Aichi, Kanagawa and Tokyo prefectures.

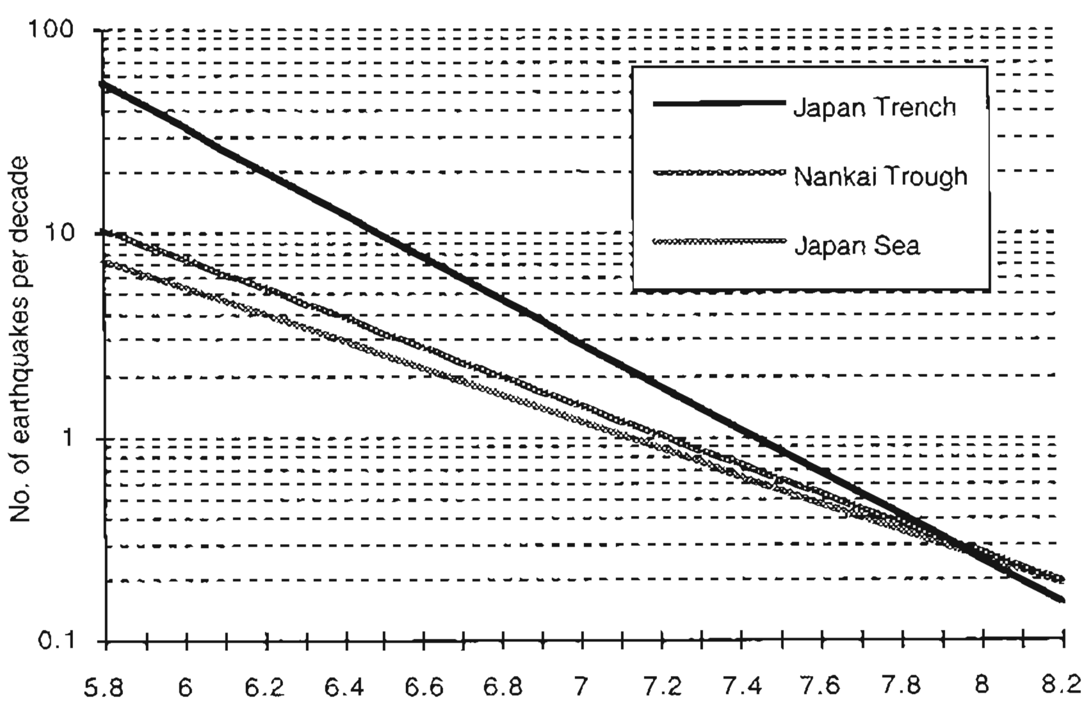

Figure 2.2: Magnitude and frequency of occurrence relationship in Japan seismicity zones

C. Japan Sea Seismic Zone
Earthquakes in this zone occur mostly along the coastline of the Japan sea. The earthquake occurrence pattern forms a clear belt of seismicity, starting inland from the Nagano and Maebashi area (Central Japanese Alps, site of the 1998 Winter Olympics) and continuing north-eastwards to the town of Niigata on the Japan sea coast. The belt of seismicity then continues very near the Northeast coast of Honshu. The sea of Japan in this area is quite shallow, suggesting a connection with the Eurasian plate. The zone continues further north, off the eastern coast of Hokkaido island. There the sea is much deeper, suggesting oceanic characteristics. The area is renowned for its harsh winter climate and extremely heavy snowfalls (annual depth of snow in mid-winter in Niigata is around 3 metres). Due to its remote geographical location and harsh climate, fewer important cities are situated on or near this zone of seismicity, with the exception of Sapporo (capital of Hokkaido).

The frequency of occurrence of earthquakes in this zone is lower than the previous two zones (see Figure 2.2). However large earthquakes do occur, at times in the magnitude range of 7.5 to 7.9. As the earthquakes are shallow and the epicentres are inland or quite near the coastal areas of Japan sea, damage from these earthquake is often severe. Tsunami occurrence is also commonplace in the Japan sea earthquakes that exceed magnitude 7.3. The fact that these earthquakes occur very near the coasts means that there is much less time available for evacuation or warning, when compared to the earthquakes occurring on the Pacific side.

The most important earthquakes in this zone during the 20th century were: the M7.5 Niigata earthquake of 7 May 1964, the M7.7 Japan sea earthquake of 26 May 1983 and the M7.8 Off Nansei earthquake of 12 July 1993. All three have caused severe damage due to ground shaking, liquefaction, landslides and the latter two triggered devastating tsunamis.

D. Central Japan Inland Seismic Zone
This is the seismic source zone of the 17 January 1995 Hyogo-ken Nanbu (Kobe) earthquake. It is an area of inland seismicity in the centre of Honshu island, running from the Ise bay in Nagoya to the Fukui prefecture on the Japan sea coast and extending westwards to include Hyogo, Tottori and Okayama prefectures in western Honshu. This small zone is of particular importance because it includes major urban, historical and industrial centres such as Nagoya, Osaka, Kobe, Nara, Kyoto, Gifu, Kanagawa, Toyota and Himeji. It is also known in the literature as the Ise-Echizen Seismic Zone or the Inner-Belt of Central Japan. The hypocentral depths of earthquakes in this zone are less than 50 km, with those of 20 to 30 km being most common. Seismicity is mostly inland or near-shore, thus having high potential for causing damage or destruction .

The Midori fault rupture caused by the great M8.1 Nobi earthquake of 28 October 1891, is in this zone. The earthquake caused a vertical uplift of up to 6 metres in this area and damage was extremely severe (more than 142,000 houses collapsed and about 7,300 people were killed). During the 20th century, seismicity in this zone has been mostly in its northern part, close to the Japan sea coasts. The most important earthquakes were: the M7.5 earthquake of 7 March 1927 in northern Hyogo prefecture (3,017 people killed), the M7.2 earthquake of 10 September 1943 in northern Tottori prefecture (1,100 people killed) and the M7.2 earthquake of 28 June 1948 in Fukui (approx. 5,000 people killed).

No other large earthquake occurred in this zone for the following 46.5 years, until the morning of 17 January 1995 when the Hyogo-ken Nanbu earthquake shook violently the city of Kobe and its immediate neighbouring towns of Ashiya and Nishinomiya.

An earthquake catalogue of Japan, that includes all fatal events in this century, regardless of magnitude is included in Appendix A of this chapter. The last earthquake-with great loss of life, before the Kobe earthquake, was the 1948 Fukui earthquake. The fatalities in the first half of the century exceeded 160,000, but this was reduced to only 824 in the second half of the century, prior to the earthquake in Kobe.

2.2 Seismotectonics of the Hanshin area

The Hyogo-ken Nanbu earthquake of 17 January 1995, affected most seriously the Hanshin region (the earthquake is also called the Great Hanshin earthquake or simply, as herein, the Kobe earthquake). The Hanshin region is a general name for the heavily populated south of Hyogo prefecture and Osaka plain

(Osaka city constitutes one of the 47 Japanese prefectures, the equivalent of the English counties or shires). This area is part of the Central Japan seismic zone, but has been active in historic times. During this century there has only been a moderate earthquake of magnitude 6.1 near Kobe (26 November 1916). In historic times the greatest earthquake is that of 1596 that is estimated to have magnitude of M7.2-7.5 and was located at the Osaka plain (more information on these in given in Chapter 3).

The main tectonic features of the Hanshin area are:

- the Arima-Takatsuki fault system and
- the Hanaore-Kongo fault system.

The 17 January 1995 earthquake was centred roughly in the middle of the Arima-Takatsuki fault system. This system consists of several active faults, on the island of Awaji and in the Rokko mountains. The system extends from Kyoto city through Awaji island to the Median Tectonic Line, that crosses through the island of Shikoku and Honshu. The fault that triggered this earthquake was the Nojima fault in the north of the island of Awaji. According to the earthquake records of Japan, historic seismicity on this system of faults has been limited (except for the 1916 earthquake). The only other significant earthquakes (M7-7.5) in the area are:

- in 8 July 868 in Himeji (probably Yamazaki fault); ≈ 40 km west of Kobe;
- in 5 September 1596 in the Osaka plain on the Hanaore-Kongo fault system; ≈ 40 km east of Kobe

No other earthquake of large magnitude (M≥7) has occurred within a 50 km radius from Kobe city in historic times. An earthquake of M6.8 occurred about 40 km to the east on 21 September 1510, but it is estimated to have caused less than intensity VII (on the Modified Mercalli seismic intensity scale) in the Kobe area.

It is therefore not surprising that the occurrence of the Hyogo-ken Nanbu earthquake was not anticipated. For more details on regional tectonics, seismicity and the geological setting of the Hanshin area, refer to Chapter 3 of this report.

Recent investigations suggest a temporal link between the occurrence of earthquakes in the Hanshin area and those in the Nankai trough. It has therefore been suggested that a great offshore earthquake (M≥8) is likely to occur on the Nankai trough in the period between 2003 and 2040 (UNCRD, 1995).

2.3 The Kobe earthquake and its seismic intensity

The Kobe earthquake occurred at 5:46:52 am local time, on 17 January 1995. It was a large earthquake of local magnitude ML7.2 (defined by the JMA) and moment magnitudeM_W 6.9. The fault rupture started on the Nojima fault in the north of Awaji island and extended north-eastwards, along the Arima-Takatsuki fault system, associated with the Rokko mountains. This fault rupture was inferred by the pattern of aftershock occurrence that stretched from Awaji island to Kawanishi-shi on the border of Hyogo-ken with Osaka-fu. The focal depth was calculated at 14.3 kilometres (UNCRD, 1995). The worst affected area was the heavily populated southern part of the Hyogo prefecture, and hence the name Hyogo-ken Nanbu jishin (which means the southern Hyogo prefecture).

The towns and wards that were located across the inferred fault rupture experienced ground shaking of intensity VII to X or more on the Modified Mercalli seismic intensity scale (hereafter also referred as MM scale). The worst affected areas in Honshu island stretched for a distance of approximately 25-km, starting from Suma-ku in western Kobe, as far as the eastern side of Nishinomiya city. All these areas can be seen on the map of Figure 2.3. Kobe consists of 7 wards that are on the sea front and two large wards, Nishi-ku and Kita-ku that are located on higher ground. The effect of the earthquake on these latter two wards was limited and mostly related to slope instabilities or local amplification effects attributed to the topography. In addition the effects on Tarumi-ku (despite its proximity to the epicentre) were also limited, mainly because of geological conditions (see Chapter 3). The remaining six wards of Kobe city and the towns of Ashiya, Nishinomiya and a small part of Takarazuka experienced the worst damage. The worst affected areas in these wards and towns were assigned intensity 7 by the Japan Meteorological Agency. For further description and mapping of the worst affected areas refer also to Chapter 3.

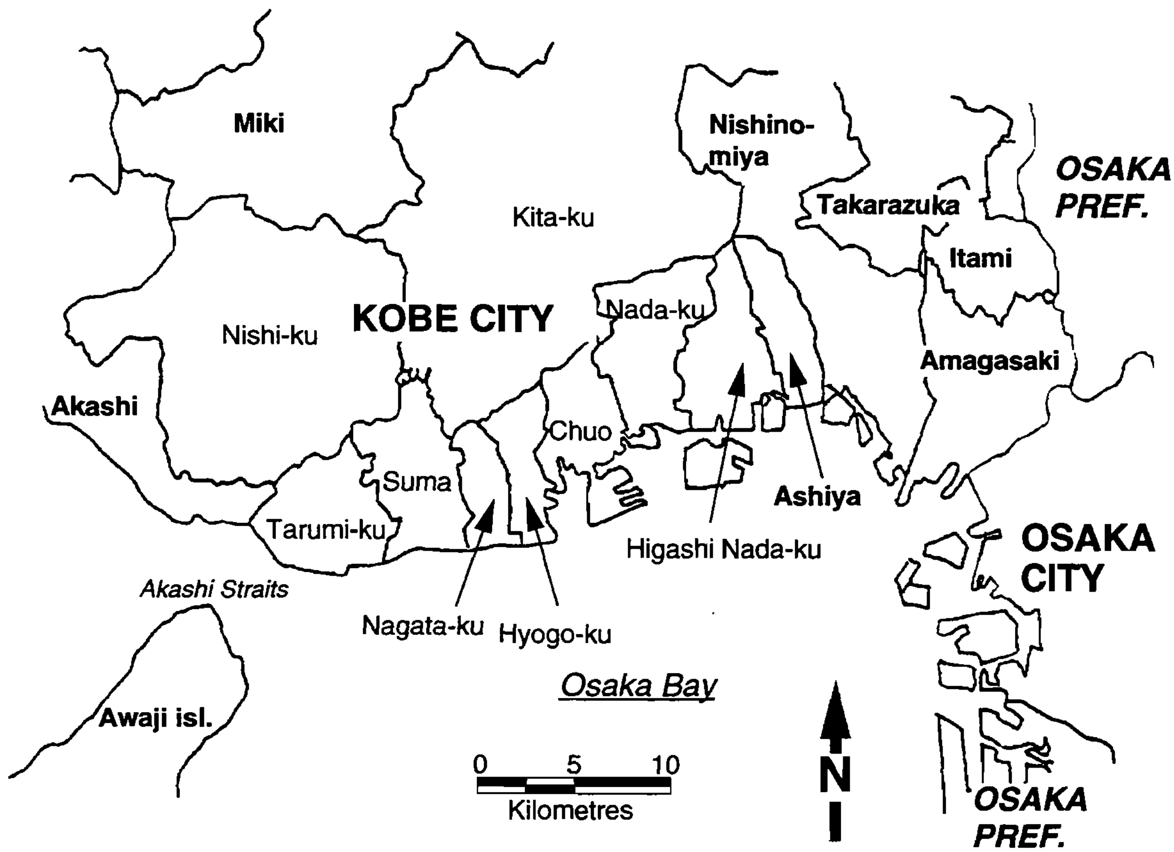

Figure 2.3: Map of the southern Hyogo prefecture, with administrative boundaries of the towns and wards affected (maps of location of surface rupture, epicentre and worst affected areas can be seen in Chapter 3).

The equation commonly used to convert between JMA and MM intensity scales, is as follows:

$$I_{MM} = 1.5 * I_{JMA} + 0.5 \text{ (Okamoto, 1973).}$$

This would mean that the ground shaking intensity in the worst affected areas was equivalent to XI on the MM intensity scale. It is very difficult to comment on the level of intensity on the MM scale, or even the MSK or EMS scales often used in Europe, because the building types described in these scales are very different in characteristics and resistance to those found in Hyogo prefecture. In this report it is proposed that the intensity was rather closer to X than XI on the MM scale. This is substantiated by damage surveys and analysis presented in Chapter 5.

The earthquake produced a 9-km surface rupture only in the northern part of Awaji island, where an average horizontal displacement of 1-1.5 metres was observed on the Nojima fault (EERI, 1995). The fault extended under the sea of the Akashi straits, for 300 metres. Two more submarine fault ruptures were found about 5-km east of the Nojima fault, that stretched for about 7-km and reached very close to the shores of Suma-ku in the western part of Kobe city. The strike-slip mechanism of the earthquake is in accordance with fault mechanisms of other significant Central Japan earthquakes (EERI, 1995). For a detailed description of the source mechanism of the earthquake, refer to Chapter 3.

Strong motion recordings were obtained by many different organizations including: Japan Railways, Japan Highways, JMA, Osaka Gas, Hankyu Railroads, Kobe University, Kyoto University, Osaka University, and the Kansai Earthquake Observation Committee. In the central wards of Kobe peak ground accelerations recorded on stations near ground level and close to the areas of the worst damage ranged from 564 to 837 cm/sec^2, while peak ground velocities ranged from 30 to 55 cm/sec. Chapter 3 contains detailed interpretation of the strong motion records and their relation to local geology.

The attenuation of shaking was particularly rapid in the NS direction (i.e. perpendicular to the high damage area). This could be noticed by the remarkable decrease in damage but also it was recorded by some strong motion instruments. North of the high intensity band the mean altitude rises rapidly as the terraces rise towards the Rokko mountains and damage to old timber buildings there was much lighter. Further up the slopes of the Rokko mountains, local pockets of high intensity were also observed, that were attributed mostly to slope instability and topographic amplification effects. For a detailed description of the attenuation of shaking across the worst affected areas read section 5.2.4 in Chapter 5.

2.4 The profile of the earthquake affected region

2.4.1 Administrative system of Japan

Japan is divided into 47 prefectures, mostly designated by the suffix -ken but sometimes the suffix -fu or -doh or -to are also used (these apply in case of prefectures that consist entirely of urban areas, like: Tokyo-to, Osaka-fu and Kyoto-fu). Prefectures are divided into areas associated with major cities, designated by the suffix -shi. There are 651 such city divisions in the whole of Japan. Eleven of these cities have population greater than 1 million and are subdivided into wards, designated by the suffix -ku. These are: Sapporo, Tokyo, Yokohama, Kawasaki, Nagoya, Osaka, Kyoto, Kobe, Hiroshima, Kitakyushu and Fukuoka. All the other cities have only one ward. Cities and (or) their wards are further subdivided into smaller administrative units designated by the suffix -cho, -machi or -gun (in rural areas). There are around 3,500 small administrative units in the whole of Japan. All of these have elected mayors and councils.

2.4.2 Main features of the affected area

Kobe and the adjacent towns of Ashiya and Nishinomiya are located on a narrow strip of bay front alluvial and colluvial flat land and the adjoining lower slopes of the Rokko mountains. The width of this heavily developed area is only 5-km on average, and is around 25-km long. Kobe is Japan's sixth largest city and is famous for its distinct environment, its amphitheatric setting ideally situated to overlook the Akashi straits and Osaka bay, as well as its old pre-war monumental buildings. These assets combined with a bustling manufacturing industry, highly developed urban infrastructure and central location in the island of Honshu, made Kobe one of Japan's favoured cities. Furthermore the perception of very low seismicity in the area was an additional attraction.

Until the beginning of the 20th century, the Kobe waterfront was situated about 1 kilometre from the present waterfront. Shortage of flat land and rapid industrialisation forced the city to reclaim the sea shores. Reclamation started in 1907 and at present 14% of downtown Kobe (excluding Kita-ku and Nishi-ku), or approximately 22.5 km^2 is on reclaimed land[2]. Port and Rokko islands were constructed between 1967 and 1992. Further plans to build the Kobe Airport on an artificial island were approved in the summer of 1995, but its completion date is expected to be delayed due to the earthquake.

The population density of Kobe city is 2,800 people per square kilometre but 73% of the population and almost all the commercial and industrial activity is carried out in the 7 wards on the bay front (see Figure 2.3). The population density in this area is actually 2.3 times higher, at around 6,530 people per square kilometre. In contrast the population density in Nishi-ku and Kita-ku is only 1,020 people per square kilometre. These two suburban wards are mainly inhabited by high income families. The profile of the seven downtown wards is quite different and varied. Some, like Tarumi, Suma and Nagata are predominantly industrial and occupied by low-income families, living in low-rise traditional Japanese neighbourhoods, while other like Nada and Higashi-Nada are predominantly occupied by middle-income families living in multi-storey apartment buildings. Hyogo and Chuo wards are predominantly commercial.

Most of Kobe as it was in the morning of the earthquake was built in the post-war years. Land shortage, characteristic of most major Japanese cities, forced the land value to sky-high prices that only stopped rising in the recent recession that started in 1991 and is to some extent still continuing.

[2] Geological evidence suggests that at the end of the last Ice Age and as early as 6,000 years ago the water level in the Kobe area was around 5 metres above that at present.

Because of the high land costs, commercial areas are remarkable for narrow-fronted multi-storey buildings (EERI, 1995) and narrow back streets for servicing. Low-rise residential areas are also densely built-up. A study of a typical such area in Osaka showed that the density was close to 100 people per acre (EERI, 1995). Narrow streets often no more than 3 metres wide, prevail in such areas. Meanwhile in most cities these neighbourhoods are slowly replaced by multi-storey apartment buildings. In Japan the most modern and luxurious of these are called mansions, indicative of the low and middle-income class craving to move away from the old traditional housing to the perceived luxury of apartments.

Replacing an old wooden house or re-developing an old neighbourhood can be quite difficult. Consensus is needed among the numerous occupants and the high taxation when selling a house prevents people from venturing to re-development.

The adjacent towns of Ashiya and Nishinomiya have also very high density (4,280 people per square kilometre). Ashiya is renowned as a high income area. Nishinomiya has large areas of traditional low-rise Japanese neighbourhoods.

2.4.3 Economic profile of the affected area

Hyogo prefecture with a population of 5.4 million people, contributes US$152 billion to the Japanese economy (UNCRD, 1995). This is roughly equivalent to 3.5% of Japan's GDP, or 0.6% of the global GDP. Around 18,000 manufacturing facilities are based in the prefecture, that are responsible for 5% of Japan's mineral and industrial production. Some sectors of the Japanese industry were heavily concentrated in Hyogo prefecture, with the most extreme example being that of synthetic rubber, 80% of which is produced there. This industry is the main supplier for car tyres, car brakes and shoe soles and suffered heavily in the earthquake. Production of steel is another main activity, contributing 10% to the national production. Main steel producers like Nishin Steel, Kobe Steel and Kawasaki Steel are all based in the prefecture. In addition shipbuilding is still active in the Kobe area, with Mitsubishi Heavy Industries as the main company. Other main manufacturing activities are: garment industry (38 factories), shoe industry (192 factories) and sake production.

About 45% of Hyogo prefecture's manufacturing product is generated in Kobe, the prefecture's capital, where 27% of its population lives. Kobe's manufacturing industry as a whole is estimated to contribute around 10% of the prefecture's GDP. The construction industry is also significant, being valued at 5 billion US$. The tertiary industry of Kobe consists of: transport, communications, trade and services, wholesale and retail trading and is the largest sector contributing 30% to the GDP of Hyogo prefecture.

One of the main assets of the city of Kobe is its harbour, that is the world's sixth busiest in terms of tonnage, Japan's first in terms of container traffic, handling 30% of the total container shipment, and second after Yokohama in terms of export volume. The harbour of Kobe handled import-export trade with a total value of US$71 billion in 1994. This was equivalent to 10.6% of Japan's trade, which totalled 396 billion US$ in exports and US$264 billion in imports in 1994. The harbour prior to the 1995 earthquake had 155 berths belonging to the public sector and scores more private ones. Export trade was handled through 22 international shipping berths. Mitsubishi Motors exported most of its cars and Daihatsu Motors most of its parts to supply other manufacturers around the world, through the harbour in Kobe.

Finally around 120 multinational companies had offices in the city, including Nestlé, Wellcome and Procter and Gamble. The latter company's Japan headquarters were in Kobe city.

2.4.4 Population and housing profile of the affected area

The population and housing statistics of the most affected cities and wards are summarised in Table 1. All of these belong to Hyogo prefecture. The city of Kobe is the main urban area, and is located closest to the epicentre of this shallow earthquake. Kobe is divided into 9 wards seen with the suffix -ku in the table. Table 1 includes some information on the proportions of detached dwellings (mostly made out of modern timber frame with plywood sheathing), dwellings in buildings made out of reinforced concrete block masonry, reinforced concrete frame and shear wall construction, steel frame or composite steel-reinforced concrete construction. Finally the proportion of dwellings that were built before 1945 is also given. The proportion of pre-1945 dwellings in the affected area as a whole was around 6%, which is quite high when compared with other areas of Japan, especially prefectures northwards of Central Japan, including Tokyo.

The total population of the worst affected areas is 3,315,000 people (61% of the total population of Hyogo prefecture). In Kobe city the population density of has increased from 2,500 people per square kilometre in 1973 to about 2,800 at the time of the 1995 earthquake. However the actual density where most of Kobe's population is based (i.e. outside Nishi and Kita-ku) exceeds 6,000 people per square kilometre.

Table 1: Population (1995) and housing statistics (1988) of the most affected areas.

City/Ward	Population (1995)	Occupied Dwellings (1988)	Pre-1945 Wooden Dwell. (%)	Detached Wooden (%)	Non-wooden Dwell. (i.e. RC etc.) (%)
Amagasaki-shi	488,000	167,480	7.4	30.3	32.5
Itami-shi	184,000	57,420	5	41.1	32.7
Takarazuka-shi	203,000	62,390	3.4	51.9	39.5
Kawanishi-shi	142,000	41,120	5	71.6	17.9
Nishinomiya-shi	402,000	139,550	4.4	37.3	48.5
Ashiya-shi	85,000	29,520	5.7	35.2	58.3
Akashi-shi	278,000	81,880	4.4	53.2	35.8
Higashi Nada-ku	188,000	66,430	5.7	30.8	51.7
Nada-ku	123,000	49,110	10.8	31.1	42.1
Hyogo-ku	121,000	48,830	10.9	26.4	41.6
Nagata-ku	127,000	50,790	19.2	28.6	26.3
Suma-ku	186,000	57,760	5.7	33.2	51.4
Tarumi-ku	236,000	73,680	2.2	41.7	50.3
Kita-ku	210,000	54,740	3.1	51.6	48.8
Chuo-ku	105,000	45,810	2.7	20.1	61.8
Nishi-ku	181,000	35,310	3.3	63.9	42.5
TOTAL	**3,259,000**	**1,061,820**	**6.1**	**38.9**	**42**

Note: Awaji island with 56,000 people is not included in these statistics

An analysis of the 6 wards of Kobe city and the three cities that have experienced in part intensity 7 on the JMA scale is shown in Table 2. It is seen that around 20% of the land area in the Kobe wards was inside the high intensity zone. This is an approximate estimation based on published maps of the worst affected areas (see Chapter 3). Nagata-ku is the most densely inhabited ward, with the largest proportion of old wooden houses. The worst fires in the aftermath of the earthquake occurred in this ward.

Table 2: Population density, fatalities and approximate proportion of area in MM10 in the worst affected wards and towns (fatalities as accounted in late January 1995).

TOWN/Ward	Population	Density (p/km^2)	JMA 7 (% area)	Fatalities
Nagata-ku	126,439	10,985	20	709
Hyogo-ku	119,032	8,255	12	390
Suma-ku	184,692	6,503	10	329
Higashi Nada-ku	187,193	6,356	20	1,202
Chuo-ku	104,331	4,801	22	211
Nada-ku	121,937	3,907	15	817
MMI 10 WARDS	**843,624**	**6,170**	**≈21**	**3,658**
Ashiya	85,736	4,976	17	400
Nishinomiya	412,463	4,163	9	884
Takarazuka	203,105	1,996	6	85
MMI 10 TOWNS	**701,304**	**3,216**	**8.3**	**1,369**

A major factor in the extent of damage and loss of life in this earthquake was the traditional Japanese timber frame dwellings predominantly built before 1971, when new regulations for low-rise dwellings were introduced (for a detailed description of Japanese building types and the evolution of Japanese building codes, refer to Chapter 4). As seen in Table 1, the proportion of dwellings that were built before 1945 was only 6.1% in 1988, and probably closer to 5% at the time of the earthquake. Housing survey statistics from the last 25 years reveal the changing character of the dwellings in Kobe city and the rest of Hyogo prefecture. These are summarised in Table 3. The changes are also illustrated in Figure 2.4.

Table 3: Dwellings by type and number of storeys in Kobe city and the rest of Hyogo prefecture.

	Kobe city (1988)	Kobe city (1968)	Rest of Hyogo Prefect. (1988)	Rest of Hyogo Prefect. (1968)
No. of Dwellings	**482,440**	**320,000**	**1,149,860**	**803,700**
Detached 1Floor (%)	4.5	16.3	10.9	28.8
Detached ≥2Floors (%)	31.3	18.8	49.9	29.7
Tenement 1Floor (%)[3]	2.4	12.9	2.9	11.6
Tenement ≥2Floors (%)	7.6	9.2	6.7	6.3
Apartments 2Floors (%)	12.9	30.9	9.1	16.5
Apartments ≥3Floors (%)	40.7	11.1	20.2	6.4
Others (%)	0.6	0.9	0.4	0.5

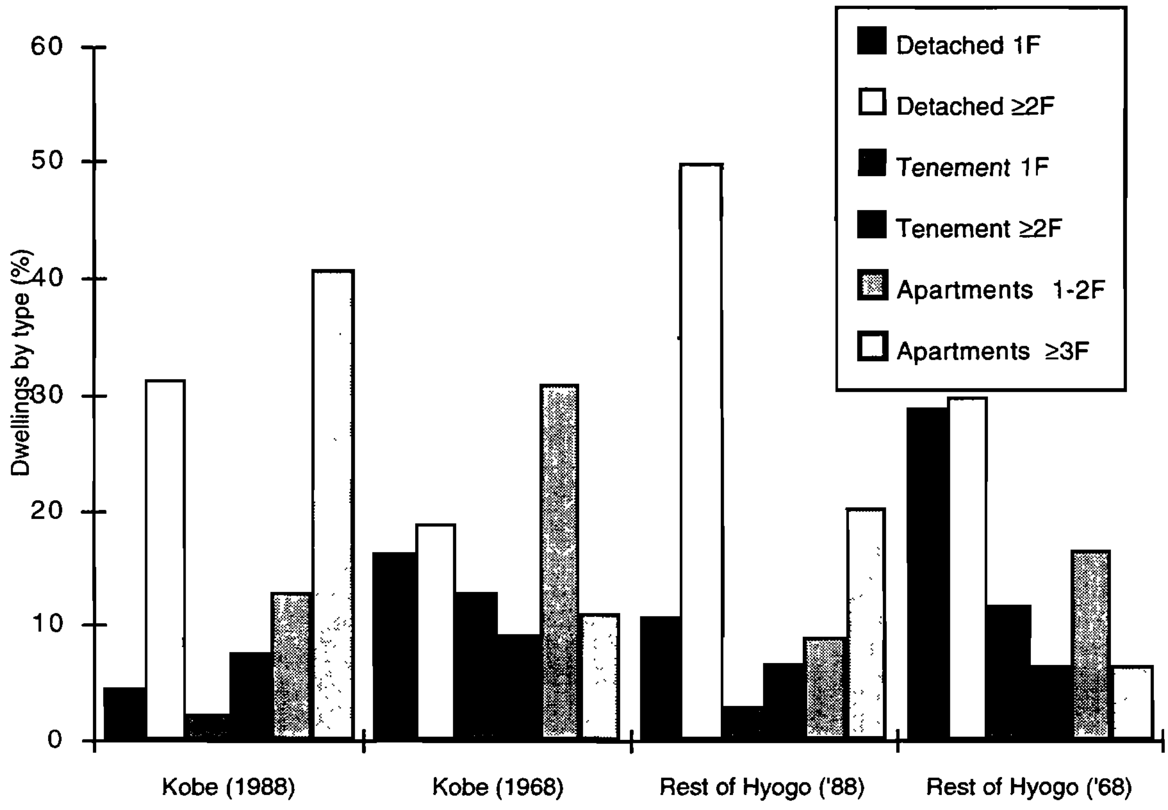

Figure 2.4: The changing character of dwellings in Kobe city and the rest of Hyogo Prefecture. (Source of data: 1988 Japan Housing Survey).

[3] Tenement houses are buildings which consist of two or more dwelling units connected to each other by walls but each having an independent entrance to the street (Japan Building Survey, 1986).

In Kobe city the proportion of dwellings in apartment buildings of 3 or more storeys, has almost quadrupled in the space of twenty years and it is estimated that almost half the population lived in multi-storey apartment buildings predominantly comprising reinforced concrete structures (see also Table 1). The proportion of people living in detached dwellings has remained almost unchanged (around 35%), but many single storey dwellings have been replaced by two or three storey detached dwellings. About 80% of the single storey detached dwellings are made from timber frame with the *shinkabe*, *okabe* or prefabricated system (for description of these building types see Chapter 4). This proportion is only about 50% in the case of the detached dwellings with 2 or more storeys, the rest being mainly from reinforced concrete (RC). The proportion of people living in tenement accommodation has been halved (down to 10% in 1988). About 45% of the tenement dwellings are made from reinforced concrete or light metal frame structures (Housing Survey of Japan, 1988).

In the rest of Hyogo prefecture the proportion of dwellings in apartment buildings of 3 or more storeys, has tripled, and it is estimated that about 25% of the population (prior to the earthquake) lived in multi-storey apartment buildings. The proportion of people living in detached dwellings has remained almost unchanged (around 60%), but many single storey dwellings have been replaced by two or three storey dwellings.

Figure 2.5 shows the proportion of wooden and non-wooden dwellings in Kobe and the rest of Hyogo prefecture in 1968 and 1988 respectively. It is seen that outside Kobe city, the proportion of wooden dwellings is much higher (around 70%). This trend is followed in the rest of the Hyogo prefecture, but at a much slower pace.

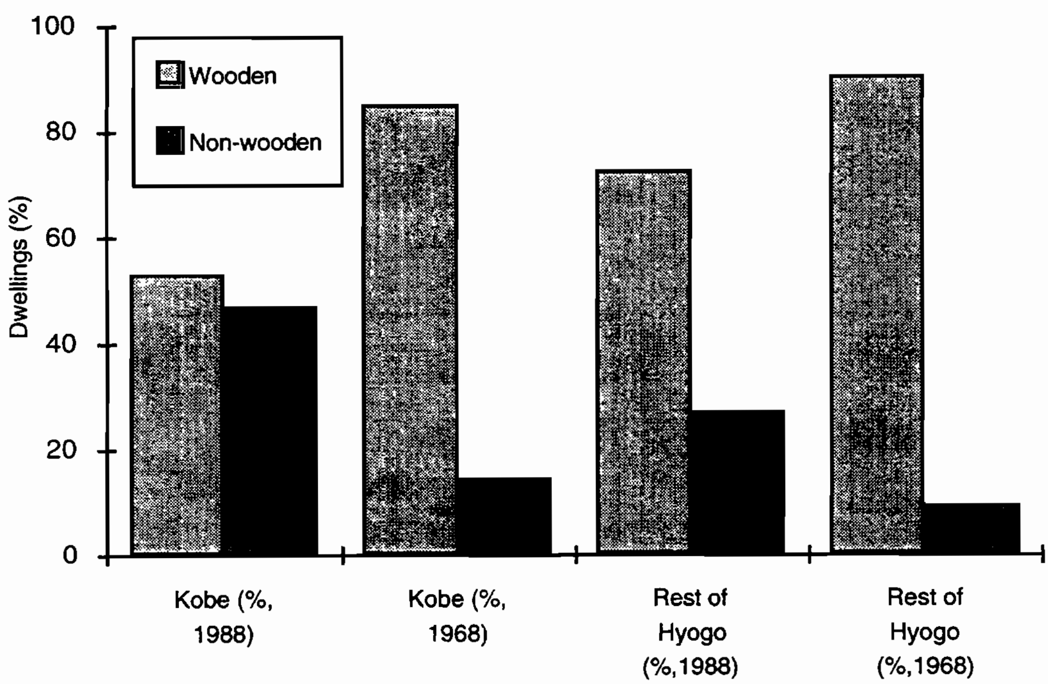

Figure 2.5: The proportion of non-wooden dwellings has increased dramatically in Kobe city. (Source of data: 1988 Japan Housing Survey)

Therefore the two main on-going changes in Kobe city and the rest of Hyogo prefecture in terms of the distribution and typology of dwelling types are:

- more and more people live in multi-storey RC apartment buildings and
- 1 storey wooden houses are being replaced by 2-3 storey timber or RC dwellings.

There are earthquake safety implications in these changes. Damage experience has shown that in the case of timber dwellings, the post-1971 wooden houses performed better, largely because prefabricated wooden houses with plywood sheathing and diagonal bracing, became common after the introduction of the 1971 Building Standard Law. Among the wooden houses built in the traditional style of shinkabe or okabe, the single storey ones performed slightly better that the two-storey. Many of the two storey dwellings are of mixed use, with the ground floor devoted to commercial purposes, having large openings (hence causing a soft-storey effect).

In the case of the multi-storey apartment buildings the damage experience showed that there are 3 distinct age groups: pre-1971, 1971-81, post-1981. The vulnerability increased with age (for details see Chapter 5).

2.5 The effects of the earthquake

The effects of the earthquake on buildings and lifelines will be described in some detail in Chapters 4 and 5, while the performance of bridges and industrial facilities are discussed in Chapters 6 and 7. This paragraph therefore summarises the most important aspects of the event, including human casualties, number of destroyed buildings, other damage and the estimated monetary cost of the earthquake.

Based on final statistics published in May 1995 the earthquake caused 5,502 deaths and around 41,500 injuries of which only 1,819 required hospitalization (UNCRD, 1995). A summary of damage and fatality statistics in the affected area is shown in Table 4. It is seen that the proportion of pre-1960 buildings largely corresponds with the fatality rate. The proportion of homeless exceeded 20% of the population in five wards of Kobe city and in Ashiya city.

Table 4: Statistics in the worst affected areas.

TOWN/Ward	Population	Households (H)	People Killed	Fatal. per 100 H	Homeless (%)	Pre-1960 bldgs (%)
Higashi-Nada	187193	75511	1202	1.6	21.4	18.6
Nada	121937	53409	817	1.5	24.9	29.1
Hyogo	119032	54679	390	0.7		19.9
Nagata	126439	53347	709	1.3	35.9	38.1
Suma	184692	65276	329	0.5	10.7	14.5
Chuo	104331	50423	211	0.4	32.3	20.8
NISHINOMIYA	412463	158857	884	0.6	9.6	16.2
ASHIYA	85736	33277	400	1.2	24.5	16.5
TAKARAZUKA	203105	67992	85	0.1		31.6

Source of Housing Statistics: Japan Housing Survey of 1988

Damage to buildings, particularly old wooden houses and pre-1981 mid-rise office and other commercial buildings was particularly severe. The total number of damaged buildings reached 394,440, which includes 100,209 completely collapsed, 107,074 partially collapsed, 183,436 damaged and 5,864 burned down buildings (UNCRD, 1995). As a result building losses were the biggest contributor to the cost of direct damage, a breakdown of which is shown in Table 5, by asset category. The direct damage has been estimated close to 100 billion US$. Therefore in relative terms the cost of direct damage was a staggering 2.2% of Japan's 1994 GDP.

As a result of the damage to buildings and road infrastructure, the worst affected areas were very difficult to approach for weeks after the event. Railway lines were however restored quite rapidly, and sooner than originally estimated. The costs of the disruption to economic activities must be quite severe. According to RMS (1995) business interruption costs will cost at least as much as the direct losses. Damage to the industrial infrastructure was particularly severe. Some examples are quoted below:

- 45% of garment factories in Hyogo prefecture suffered serious damage
- 73% of shoe factories in Kobe city either collapsed or burned down
- 500 offices of banks, and other financial institutions were closed
- 78% of publicly managed berths and 100% of berths designated for export were destroyed

Table 5: Direct economic losses, by asset category.

Asset Category	billion US$
Buildings	58
Railways	3.44
Highways	5.5
Other Public Infrastructure	2.78
Harbour Facilities	10.1
Educational Facilities	3.42
Damage to Primary Industries	1.18
Health & Welfare Facilities	1.73
Lifelines (Water, Gas, Electr., Telecom.)	5.76
Loss of Stock & Business (machin., equipm.)	6.3
Public Buildings	0.75
TOTAL	98.96

Source: UNCRD (1995)

2.6 Conclusions

1. The Hyogo-ken Nanbu earthquake is the worst to affect Japan since 1948 and the second worst in loss of life after the 1923 Great Kanto earthquake. The earthquake was centred roughly in the middle of the Arima-Takatsuki fault system that extends from Kyoto city through Awaji island to the Median Tectonic Line. The part of the fault system that moved during this earthquake is not known to have moved during historic times and no earthquake of $M \geq 7$ had occurred within a 50 km radius from Kobe in historic times.

2. The worst affected areas were 6 wards of Kobe city facing Osaka bay and the towns of Ashiya and Nishinomiya (immediately east of Kobe city). The inferred fault rupture extended for about 25-km eastwards from the Akashi straits and stopped just short of Osaka prefecture in Kawanishi town. A 1-km band of very severe damage was delineated for a large part of this distance. Recorded peak ground horizontal accelerations and velocities in these areas of Kobe ranged from 564 to 837 cm/sec^2 and from 30 to 55 cm/sec, respectively.

3. The population density in the worst affected wards and towns was between 6,500 and 4,300 people per square kilometre. Some of the worst affected neighbourhoods were densely built-up with old traditional Japanese low-rise timber frame dwellings.

4. About 60% of Hyogo prefecture's 5.4 million people lived in the worst affected area and 701,000 people lived in the areas that experienced intensity JMA7 (proposed MM intensity X or somewhat higher). It was found that in Kobe's six worst affected wards and in Ashiya city around 20% of the surface area experienced such an extreme intensity of ground motion. Around 80% of the fatalities were in these areas. Around 90% of the loss of life was associated with the collapse of pre-1971 low-rise traditional Japanese houses. However, in Kobe city it was estimated that almost half the population lived in multi-storey apartment buildings predominantly made from reinforced concrete structures. On the other hand outside Kobe city, the proportion of wooden dwellings is much higher (around 70%). It was also found that the worst affected areas had a high proportion of pre-1960 wooden dwellings, ranging from 38% in Nagata-ku to 14% in Suma-ku. Other areas of Japan, most notably Tokyo, have a much lower proportion of such old and seismically vulnerable dwellings. As a result the proportion of homeless exceeded 20% of the population in five wards of Kobe and in Ashiya city.

5. The direct monetary losses of the Hyogo-ken Nanbu earthquake were estimated at around 100 billion US$. Damage to the building infrastructure was the biggest contributor to this loss (59%). In absolute terms this is the costliest natural disaster in world history. However some of the costs are artificially inflated by the unusually high price of land in Japan, that drives the unit cost of infrastructure to much higher levels. In relative terms these losses amount to 2.2% of Japan's GDP and are less than Japan's

1994 trade surplus (121 billion US$). The cost of business interruption will be at least as much as the direct losses.

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Appendix A: Damaging Earthquakes in Japan in the 1900 - 1995 period

Year	Mon	Day	Hr	Min	Lat(⁰N)	Long(⁰E)	Depth	Mag	Dead	Injured	Damage
1900	5	12	2	23	38.7	141.1		7	Some	some	
1901	2	15			26	100.1		6	Many		Moderate
1901	8	9	18	23	40.5	142		7.5	18	some	Moderate
1902	1	30	23	1	40.5	141.3	Shall.	7	1	2	
1905	6	2	14	39	34	132	100	7.9	11	177	Limited
1909	8	14	15	31	35.5	136.3		6.8	41	784	Moderate
1909	8	29	19	27	26	128	Shall.	6.2	1	10	Moderate
1909	11	10	15	13	32.3	131.1	150	7.6	2	several	Considerable
1911	6	15	23	26	29	129	160	8.1	12	26	Moderate
1914	1	12	18	28	31.6	130.6	Shall.	7.1	35	112	Severe
1914	3	15	4	59	39.5	140.4	Shall.	7.1	94	324	Severe
1915	3	18	3	45	42.1	143.6	Shall.	7	2	0	Considerable
1916	11	26	14	8	34.6	135	Shall.	6.1	1	5	Considerable
1917	5	18	4	7	35	138.1	Shall.	6.3	2	6	Some
1922	4	26	10	11	35.2	139.8	Shall.	6.8	2	23	Considerable
1922	12	7	16	50	32.7	130.3			27	few	Moderate
1923	9	1	11	59	35.3	139.5		8.3	142807	103733	Extreme
1924	1	15	5	50	35.5	139.2	Shall.	7.3	19	638	Considerable
1925	5	23	11	10	35.7	135		6.8	428	834	Severe
1927	3	7	18	28	35.7	135		7.6	3017	7806	Extreme
1930	10	17	6	36	36.3	136.3	0	6.3	1	0	Limited
1930	11	25	4	3	35.1	139.1	0	7.3	272	572	Extreme
1930	12	25	19	3	35.1	133	5	7	272		Severe
1931	11	2	19	3	32.3	132.6	40	7.6	1	29	Severe
1931	11	21	2	20	36.1	139.2	20	7	16		Moderate
1933	3	3	2	31	39.1	144.7	10	8.3	3064	1092	Extreme
1933	9	21	12	14	37.1	136.8	30	6	3	57	Considerable
1935	7	11	17	24	35	138.4	10	6.4	13	299	Considerable
1936	2	21	10	7	34.6	135.7	0	6.4	9	59	Considerable
1936	7	4	6	2	35.2	127.6		5	Several		Limited
1936	12	27	9	14	34.4	139	0	6.3	3	70	Moderate
1938	5	29	1	42	43.6	144.5		6.1	1	0	Moderate
1938	11	5	17	43	37.3	142.2	30	7.5	1	9	Moderate
1939	3	20	12	22	32.3	132	20	6.5	1	1	Limited
1939	5	1	14	58	40	139.8		6.8	27	52	Moderate
1940	8	2	0	8	44.3	139.5	10	7.5	10	0	Considerable
1941	7	15	23	45	36.7	138.2	0	6.1	5	18	Considerable
1941	11	19	1	46	32	132.1	0	7.2	2	8	Considerable
1943	9	10	17	36	35.3	134		7.2	1100	3259	Extreme
1943	10	13	14	43	36.8	138.1	0	5.9	1	14	Moderate
1944	12	7	4	36	33.7	136.2	30	7.9	1223	2864	Extreme
1945	1	13	3	38	34.7	137.1	0	6.8	2306	3866	Severe
1945	2	10	13	57	41	142.1	20	7.1	2		Moderate
1946	12	21	4	19	32.5	134.5		8.2	1350	2632	Extreme
1948	6	15	20	44	33.8	135.4	10	6.7	2	33	Considerable
1948	6	28	16	13	36.5	136		7.2	3769	22203	Extreme
1949	7	12	1	10	34.1	132.7	40	6.2	2	2	Considerable

cont.

cont.

Year	Mon	Day	Hr	Min	Lat(^{0}N)	Long(^{0}E)	Depth	Mag	Dead	Injured	Damage
1949	12	26	8	24	36.7	139.7		6.4	10	163	Considerable
1952	3	4	10	22	41.8	144.1		8.3	33	287	Severe
1952	3	7	16	32	36.5	136.2	0	6.5	7	8	Considerable
1952	7	18	1	9	34.4	135.8	65	6.8	9	136	Moderate
1955	7	27	10	20	33.7	134.3	10	6.4	1	8	Some
1956	9	30	6	20	38	140.6	20	6	1	1	Some
1958	3	11	9	26	24.6	124.2	77	7.5	1	1	Damage
1961	2	2	3	39	37.4	138.4	20	5.2	5	30	Moderate
1961	2	27	3	10	31.6	131.8	40	7.1	2	7	Moderate
1961	8	19	14	33	36	136.7		7.3	10	43	Moderate
1962	4	30	11	26	38.7	141.1	0	6.4	3	272	Severe
1964	6	16	13	1	38.3	139.1	40	7.5	26	447	
1965	4	20	8	42	34.9	138.3	20	6.1	2	4	
1968	2	21	8	51	32	130.6	3	6.2	3	0	Moderate
1968	4	1	9	42	32.5	132.2	33	7.6	1	15	Moderate
1968	5	16	9	48	40.8	143.2	7	8	52	330	Severe
1969	9	9	14	15	35.8	137.1	0	6.6	1	10	Moderate
1974	5	9	8	33	34.5	138.7	10	6.9	29	102	Considerable
1978	1	14	12	24	34.7	139.4	23	6.5	25	211	Severe
1978	6	12	17	14	38.4	141.9	42	7.5	28	1325	Severe
1980	9	24	4	10	36	139.8	80	5.4	1	5	Some
1980	9	25	2	54	35.5	140.2	80	6	2	73	Some
1983	3	16	2	27	34.8	137.6	40	5.7	1	3	
1983	5	26	11	59	40.4	139.1	14	7.7	104	163	Severe
1983	8	8	12	47	35.5	139	22	5.8	1	28	Limited
1984	3	6	11	17	29.3	138.9	454	7.9	1	1	
1984	9	14	8	48	35.8	137.6	2	6.8	29	10	Severe
1987	3	18	13	36	32	132.1	48	6.6	2	6	Moderate
1987	12	17	11	8	35.4	140.5	58	6.7	2	138	Considerable
1989	2	19	12	27	35.96	139.79	60	5.7	1	2	
1993	1	15	20	06	43.4	143.26	107	7.6	2	614	Considerable
1993	7	12	22	17	42.8	139.4	34	7.8	236	667	Severe
1993	10	11			32	317.85	365	6.5	1	4	Limited
1994	12	28	21	19	40.45	143.5	33	7.2	3	783	Considerable
1995	1	17	5	46	34.55	135	16	6.9	5466	34568	Extreme

Note 1: Times of Occurrence are Local

Note 2: In addition the great 1960 earthquake that occurred in Chile, caused 203 deaths in Japan due to tsunami impact.

3.0 SEISMOLOGICAL ASPECTS

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3.1 Tectonics and regional seismicity

The islands of Japan are situated on or close to four tectonic plates: the Eurasian plate, the North American plate, the Pacific plate, and the Philippine Sea plate. As shown in Figure 3.1, the Eurasian plate is pushed from the east by the Pacific plate and from the south by the Philippine Sea plate. These movements cause interplate earthquakes to occur in 100-200 year intervals at the subduction zones. For example, the Tokyo metropolitan area is located where the Eurasian plate, the North American plate and the Philippine Sea plate collide together, creating an extremely high probability of earthquake occurrence in the area. On the other hand, the Osaka-Kobe region is located on the Eurasian plate. The risk of interplate earthquake in this region is governed by the Nankai trough, where the Philippine Sea plate collides with the Eurasian plate. Thus, the risk of an interplate earthquake occurring in the Osaka-Kobe region is considered to be less than in the Tokyo region.

The Hyogoken Nanbu earthquake was an intraplate earthquake. This type of earthquake is mainly caused by forces applied to a tectonic plate at subduction zones. The stresses in the tectonic plate concentrate in the thin hard crust of the earth, which in turn creates many active faults in the plate. The present stress measured in the Osaka-Kobe region is compression as the major stress along east-west direction (Fujita, 1985). Due to this compressional stress, many active faults in the region are strike and/or reversed faults.

The Hyogoken Nanbu earthquake occurred along active faults that were previously identified. In the Kobe region, the Rokko fault system runs through the city of Kobe and it divides the city into Rokko mountains and the alluvial deposits (Figure 3.2). This system changes its direction slightly eastward from the east of Kobe and Ashiya and joins with the Arima-Takatsuki fault system at Takarazuka. Many faults in the Rokko-Arima-Takatsuki fault system are reversed faults, of which the northwest section has upheaved. This upheaval movement created the present shape of the Rokko Mountains. Some faults also have a right lateral strike slip component. All these active faults in the system are classified individually as Class B faults, whose displacement rate is between 10 cm and 100 cm per 1000 years, while the whole fault system is ranked as a Class A active fault. The displacement rate of the Class A active fault is between 1 m and 10 m per 1000 years.

At the northern part of Awaji Island, several active faults were identified before the earthquake, as shown in Figure 3.2. After the earthquake, a significant surface rupture feature was found along the Nojima fault. This fault is approximately 7 km long and runs from Nojima to Hokutan-cho Ezaki, the northwestern shore of the island. Its strike direction is N60E. Mizuno *et al.* (1990) report the average displacement rate of this fault is 0.4~0.5 m/1000 years vertically and 0.9~1.0 m/1000 years laterally to the right.

In the Akashi channel between the Rokko fault system and the Nojima fault, the Japan Maritime Safety Agency found a 300 metre long off-shore extension of the Nojima fault by marine seismic survey after the earthquake. In addition, they located two fault ruptures of 7 km long, which ran parallel to the Nojima fault but offset by 5 km to the east. Therefore, the earth movement at the Nojima fault possibly shifted slightly to the northeast, continuing to the Rokko fault system. Iwasaki (1995b) mapped some faults that were identified in Osaka Bay, as shown in Figure 3.2.

The Kobe-Osaka region was considered by the public to be less affected by earthquakes in the past. However, according to the historical records, the region had quite a high risk of earthquakes. Figure 3.3 shows the locations of the epicenters of past earthquakes of greater than magnitude M6. In 868, an M7 earthquake occurred in Harima, west of Kobe, possibly caused by the Yamasaki fault (see Figure 3.2). Also, in 1916, an M6.1 earthquake was recorded at the location similar to the Hyogoken Nanbu Earthquake. More recently, the 1946 Nankai earthquake was an M8 interplate earthquake that occurred along the Nankai trough (see Figure 3.1).

3.2 Geological setting

A panoramic view of the Kobe region is shown in Plate 3.1. The geographical feature of the region can be divided into four sections: (1) the Rokko mountains area, (2) several levels of terraces, (3) alluvium and (4) reclaimed lands. As shown by a schematic diagram in Figure 3.4, these geographic features are the consequences of uplift mechanisms of the granitic rock base. The distance from the foot of the Rokko Mountains to the shore line is approximately 10 km. The average slope angle is about 4% at the foot of the mountain and decreases to 1% in downtown Kobe and along the shore line.

The geology of the Kobe region consists of (1) granitic rocks of the Mesozoic Cretaceous period, (2) sedimentary rocks of the Tertiary period and of the beginning of the Quaternary period, (3) Pleistocene terrace deposits, (4) Holocene alluvial deposits, and (5) reclaimed fill. The surface geology of the region is shown in Figure 3.5 (Public Works Research Institute, 1995).

The Rokko mountain area consists mainly of granitic rock called the Rokko granite formation. The surface of this granitic rock is severely weathered as potassium feldspar becomes clay minerals. This weathered granite is called the "Masa" soil, which is commonly used as a fill material in Kobe. Diorite can also be found in the south of the mountains. The Arima formation and the Kobe Group formation cover the northwest side of the Rokko Mountains. The Arima Formation consists mainly of pyroclastic rocks such as tuff, whereas the Kobe Group formation consists of sedimentary rocks of sandstone and mudstone with tuff deposits as interlayers.

To the south of the Rokko Mountains towards the shore line, three levels of terrace deposits exist. These deposits are mainly well compacted gravel and sand of the Pleistocene age. The water table is generally low.

Holocene alluvial deposits and reclaimed lands cover the shore line of Kobe and form relatively leveled land which contains most of the development in the Kobe area. Alluvium is mainly sand deposits originating from bars. Clay deposits can also be found from ancient back marsh. Land reclamation started in the 1860s, but the main construction has occurred since the 1960s and was initiated by the city of Kobe. The reclaimed materials were either hydraulically dredged fill material or decomposed granite "Masa" soils borrowed from the Rokko Mountains.

Another significant geographical feature of Kobe is two man-made islands: Port Island (826 ha) and Rokko Island (580 ha). The thickness of the fill is approximately 16~24 m on Port Island and 18~21 m on Rokko Island. The fill materials are either Masa soil or decomposed or crushed mudstones of the Kobe group formation.

A subsurface cross-section of Kobe is shown in Figure 3.6. The thickness of the alluvial layer is approximately 20 m along the shore line of Kobe. The alluvial deposits are underlain by the Osaka group formation, which consists of gravel, sand, silt and clay layers of the Pleistocene period. The surface exposure of this formation is not common in Kobe, but it covers the main part of Osaka Bay.

In the Osaka group formation, twelve marine clay layers are identified with sand and gravel interlayers. The total thickness of this formation is approximately 300~400 metres at the terrace, 1,000 metres at the shore and 1,700 metres in the Bay (Iwasaki, 1995c). The slope angle of the alluvial deposits and the Osaka group formation is between 1% and 5%. It is suspected that this rather dramatic change in the thickness of the soil layers influenced the large variation in the strong ground motion recorded during the earthquake (see Section 3.4).

Awaji Island has a steep mountain range (see Plate 3.2), which runs in the direction northeast to southwest. The northeast part of the island is mainly outcropped by the Ryoke granitic formation of the Cretaceous period. Granodiorite is exposed at the surface along the Nojima fault. At the narrow shorelines, the Osaka Group formation and the Kobe Group formation comprise the surface geology.

3.3 Source characteristics and aftershock

The Japan Meteorological Agency (JMA) reported that the epicentre of the earthquake was 34.60°N, 135.00°E, and the focal depth was 14.3 km. The epicentre is at the north tip of Awaji Island, which is approximately 20 km southeast of downtown Kobe (see Figure 3.2). The mechanism of the earthquake was right-lateral strike-slip faulting in a nearly vertical fault striking slightly east of northeast. The direction of the movement was parallel to the strike of many faults in the region. Seismological analyses by several sources (e.g. JMA, USGS, Harvard, DPRI) indicate right-lateral movement with a seismic moment of 1~2 x 1026 dyne-cm, which corresponds to a moment magnitude of Mw = 6.7~6.9. This is slightly smaller than the magnitude of the Northridge Earthquake, which occurred in Los Angeles exactly one year earlier.

The inversion analysis by Kikuchi (1995) shows that the earthquake consisted of three subevents within 10 seconds movement. The epicenters of these subevents moved northeast ward from the original epicenter as shown in Figure 3.7. The first event took place at a depth of 15 km below the Akashi channel, which is at the north edge of Awaji Island. The fault ruptured in both northeast and southwest directions with a moment magnitude of Mw = 6.8. If a uniform displacement of the fault plane is assumed, the displacement is approximately 1.5~2.5 metres. The rise time is estimated to be approximately 1~2 seconds.

The rupture continued into the city of Kobe by the next two subevents, which propagated along the Rokko fault system at depths of 6~8 km. Four seconds after the first event, the second event occurred at the mountainside of Nagata and propagated for about 5 seconds in a north-northeast direction. The moment magnitude was estimated as Mw = 6.3 and the displacement was 2.6 m in right lateral direction and 1.7 m in south side down vertical direction with an indication of thrust motion. The last event took place at 6 seconds after the first event and the location was 15 km northeast from the epicentre of the first event. The moment magnitude was Mw=6.4.

Four foreshocks were recorded 24 hours before the earthquake in the region (DPRI, 1995a). They are an M3.6 earthquake at the Akashi channel at 18:26, an M2.5 at 18:49, an M1.5 at 18:55 and an M2.1 at 23:49. For comparison, the number of earthquakes of magnitude larger than 1.5 was 78 in the past sixteen years. Thus, the occurrence of four earthquakes in 24 hours was quite unusual.

The distribution of aftershocks makes a strip in the Osaka-Kobe region, as shown in Figure 3.8 (DPRI, 1995b). The largest aftershock was an M4.9 that occurred 2 hours after the main shock. The plane of aftershock is perpendicular to the surface plane, and the depth is between 5 km and 15 km. A strip of 50 km long starts from Ichinomiya-cho in Awaji Island and runs through the Rokko fault system along the mountain side of Kobe. The northeast section extends up to Takarazuka. In Awaji Island, the aftershocks match not only the Nojima fault but also the Kariya fault, which runs parallel to the Nojima fault to the east.

3.4 Strong ground motions

3.4.1 General

Strong ground motions were recorded at various sites by several organizations, and the locations and their maximum accelerations are summarized in Figure 3.9. In general, the maximum accelerations were 500~800 gal in Kobe, Nishinomiya and Takarazuka, 300~600 gal in Amagasaki and Takatsuki, and 150~300 gal in Osaka and Kyoto (1 g = 980 gal). As shown by the attenuation characteristic in Figure 3.10, the acceleration decreases as the distance between the fault line and the recorded sites increases (Ejiri et al. (1996)). The data in Figure 3.10 were compared with the empirical relation obtained by Joyner and Boore (1981). The data matches their attenuation curve quite well for rock and stiff soil.

One of the records near the epicentre was obtained at the JMA Kobe Meteorological Observatory. This observatory is located on the high level terrace along the Rokko Mountains. The Egeyama fault of the Rokko fault system runs near the site. The strong motion records of this site are shown in Figure 3.11-(1). The maximum acceleration, velocity and displacement were 818 gal, 91 kine (1 kine = 1 cm/sec) and 20.8 cm, respectively. The P and S waves arrived at 4 seconds and 6 seconds after the rupture at the epicenter, indicating the site is very close to the epicenter.

Both vertical and horizontal motions reach their maximum at a similar time of 9 seconds. The total duration of the earthquake was about 20 seconds with main shaking of 10 seconds. The records indicate two main shocks : (a) 7~9 seconds, and (b) 9~13 seconds. The dominant frequency of these shocks is about 1.2 Hz, but the first shock had a slightly longer period than the other. The Fourier spectra of the acceleration records are shown in Figure 3.11-(2). The dominant frequencies are 1~2 Hz for both NS and EW horizontal motions and about 1 Hz for vertical motion.

Ground motions varied significantly in the epicentral region by a complex interaction of important effects such as local geology, topography, and the directivity of wave propagation. Records on a rock site were obtained at Rokkodai in Nada-ward, Kobe, which can be compared to the records on soil sites. The maximum acceleration and velocity at the rock site were 305 gal and 55 kine, respectively. This strong motion is smaller than at soil sites; for example, 833 gal at Fukiai, 616 gal at Takatori, and 792 gal at Nishinomiya Imazu. Thus, in the epicentral region, the recorded maximum accelerations at soil sites were two to three times higher than those at rock sites. The effect of local site conditions will be discussed more in Section 3.4.4.

The near-field motions were intense pulses, whereas the more distant motions had longer duration of shaking with a longer period resulting from surface waves and site resonance. The response spectra of horizontal motion for the JMA Kobe site, the Rokkodai site, and the Osaka Morikawachi site are shown in Figure 3.12 (CEORKA, 1995). At the Rokkodai site, the dominant period is 1~1.5 seconds, which is similar to the dominant period at sites close to the epicentre (Iwasaki, 1995b). At the JMA Kobe site, the range of the dominant period is wider between 0.3 and 1.5 seconds, and the pseudo velocity amplified to more than 200 kine. It is also important to note that the initial arrival of the vertical motion at the JMA Kobe site is approximately the same as the S-wave motion. As recorded sites move away from the epicenter, the dominant period becomes longer as shown by the records at the Osaka Morikawachi site. The range of dominant period of this site was in between 0.5 and 5 seconds. These response spectra depending on site locations are a very important feature for determining the dynamic response of structures.

Large vertical accelerations were also recorded from this earthquake. The ratio of the vertical to the horizontal maximum accelerations is shown in Figure 3.13 The ratio increases as a recorded site becomes close to the epicenter. These are the characteristics of the near epicenter region. For the sites where the horizontal acceleration is more than 300 gal, the ratio is approximately two-thirds.

3.4.2 Vertical array records on Port Island

Vertical array data was recorded by the city of Kobe at the northwest corner of Port Island providing valuable site response data. Strong motion instruments were installed at four different depths as shown in Figure 3.14 (CEORKA, 1995). The soil profile is also shown in the figure along with SPT N values and P-S velocities. At this location, the fill material of 18 m deep overlies the alluvial clay layer of 10 metres. The fill material is the Masa soil loosely compacted with an SPT N-value of about 5~7. Between 30 m and 60 m depth, gravel layers are interlayered by clayey sand layers. The SPT N values of these layers are between 10 and 50, and they are commonly used as supports for deep pile foundations. The deepest seismograph was located at 83.5 m below the ground surface, which is at the bottom of a Pleistocene clay layer (Ma12). The thickness of Quaternary deposits is more than 1000 metres at this site.

At each depth, acceleration-time histories were recorded in the NS, EW and UD directions as shown in Figure 3.15 (CEORKA, 1995; Iwasaki and Tai, 1996). The maximum acceleration values at the surface were 341.2 gal (NS), 284.4 gal (EW) and 555.9 gal (UD), whereas those at the depth of -83.5 m were 678.8 gal (NS), 302.6 gal (EW) and 186.6 gal (UD). The horizontal acceleration at the surface is less than that at the other recorded sites in the region. However, since the horizontal acceleration at 83.5 m depth is about the same order of magnitude as the other recorded sites, large attenuation occurred between the surface and 16 m depth, due to liquefaction of the loosely dumped granular fill. In terms of velocity, the maximum velocity in the NS direction amplified from 66.6 kine at 83.5 m depth to 89.8 kine at the surface. Therefore, this attenuation in acceleration and amplification in velocity is due to the large attenuation of the high frequency component of the waves in both the NS and EW directions (Iwasaki, 1995b). The softening of the soils by liquefaction and non-linear response of the soft alluvial soil layer are the causes of this dramatic change in strong motion records at different depths.

The ground motions in horizontal plane are shown in Figure 3.16 for 83.5 m depth and the surface (Madabhushi, 1995). For both records in the first 2 seconds, the motion is in the direction of northwest and southeast. However, the acceleration field is polarizing in the northwest and southeast directions, as the stress waves are propagating towards the surface. The loss of stiffness by liquefaction in local regions may render the site anisotropy, causing the strong motion to polarize in one direction (Madabhushi, 1995). Moreover, this direction is almost perpendicular to the direction of the fault movement, indicating that the directionality of the motions was strike slip movement. Iwasaki (1995) reports that a strike slip earthquake usually creates larger ground motion in the direction perpendicular to the fault than in the direction parallel to the fault.

The vertical motion at the surface was relatively large compared with the horizontal components. Vertical motions can be created by either P-wave or vertically propagating SV-wave. The acceleration records in Figure 3.15 indicate that a large amplification ratio is given at surface fill layer structure before the S-wave arrives at 13.5 seconds. From the measured P-wave velocities, the top portion of the fill was found to be not fully saturated. Thus, at Port Island, the magnitude of the P-wave traveling in the saturated soil probably amplified as it traveled through the unsaturated zone without filtering out the high frequency components (Iwasaki, 1995b). Further investigation is needed.

3.4.3 Effect of local site condition

The JMA uses an earthquake intensity index based on the ground motion level felt by humans, the proportion of damaged houses, and the degree of geological hazards. The scale ranges from 1 to 7, where the degree of damage increases with the numbers. Intensity 6 corresponds to the ratio of housing collapse of less than 30%, many landslides and surface rupture, whereas Intensity 7 corresponds to the ratio of housing collapse of more than 30%. Approximate accelerations for Intensity 6 and 7 are 250~400 gal and more than 400 gal, respectively.

The shaded area in Figure 3.17 was categorized as Intensity 7 from the earthquake reported by JMA. The severely damaged area concentrated in a strip of 0.7~1.2 km wide between Hankyu Railway line and Route 43. This 25 km long strip starts from Suma ward in Kobe and continues to Shukugawa in Nishinomiya. The Rokko fault system does not exist under this strip, but it runs parallel to the strip on the alluvial plane. The strip breaks at Hyogo ward. This may be due to the local geology of the site consisting of granitic low level terrace deposits found close to the shore line.

Iwata *et al.* (1995) investigated the effects of surface geology on seismic records by installing four temporary stations after the earthquake in Higashi Nada ward between the JR line and Route 43, where many wooden houses were severely damaged (Figure 3.18). The aftershock seismic records show that the maximum horizontal accelerations at the soil sites located on a thin layer of soft alluvium (10~15 metres thick) were larger than those at the rock site by factor of 3 to 10. These records suggested that the severely damaged area was greatly affected by local geological conditions. The focusing phenomena of seismic waves, which propagate from the underneath and north side of the sedimentary deposit (see Figure 3.4), may also have influenced the strong ground motion at this location (Nakagawa et al., 1996).

3.5 Surface rupture

If the distribution of the aftershock is assumed to be the rupture length, the area of the fault becomes 750 km^2. From this, the average displacement along the fault is estimated to be approximately 1.3 m. During the site investigation in Awaji Island, surface rupture features of this order of magnitude were observed at various locations.

Significant surface rupture was clearly observed along the 11 km long Nojima fault of Awaji Island. At Okura, the offsets were measured to be approximately 1.2 m right lateral and 0.4 m dip-slip as shown in Plate 3.3. The house shown in Plate 3.4 was nearly offset by the surface rupture, but it appeared to be only slightly damaged by the ground shaking, possibly due to its new construction. It was found that many old houses around this area were severely damaged.

The surface expression of the faulting was more difficult to find along the base of the mountains because the width of the rupture became fairly narrow. Plate 3.5 shows the surface rupture at Ezaki in Hokutancho. This is the location where rock stairs lead to a light house, and a rupture of 0.3 m gap with 1.5 m deep was found.

Kurita *et al.* (1995) mapped the displacements along the Nojima fault as shown in Figure 3.19. The measured displacements were in the range of 1 to 2 metres. The movement was predominantly right lateral with some dip-slip of east side up. The vertical displacement varies considerably with an average of 0.5 to 1 m and a maximum displacement of 1.3 m. This variation is possibly due to the change in the thickness of soil deposits on the Nojima fault.

Ground surface deformation such as pavement cracks was found at various locations along the Rokko fault system in Kobe. However, it was difficult to determine these deformation features as surface rupture due to fault movement. It can be created by other sources such as strong ground motion, the influence of building shaking, and lateral spreads. No significant surface rupture that seems to be parallel to the faults was found in Kobe during the site investigation.

3.6 Conclusions

The Hyogoken Nanbu Earthquake was caused by a right-lateral strike slip movement of the Rokko-Arima-Takatsuki fault system in Kobe and the Nojima fault in Awaji Island. Significant surface rupture was clearly observed along the 11 km long Nojima fault. In Kobe, ground surface deformation such as pavements cracks was found at various locations along the Rokko fault system. However, it was difficult to determine these deformation features as surface rupture due to fault movement.

Seismological analyses by several sources indicate that the earthquake had a moment magnitude of Mw = 6.7-6.9 and a focal depth of 14.3 km. Four foreshocks of a magnitude between 1 and 4 were recorded 24 hours before the earthquake in the region. The distribution of aftershocks makes a 50 km strip in the Osaka-Kobe region and the depth is between 5 km and 15 km.

The maximum accelerations were 500-800 gal in Kobe, Nishinomiya and Takarazuka, 300-600 gal in Amagasaki and Takatsuki, and 150-300 gal in Osaka and Kyoto. The near-field motions were intense pulses, whereas the more distant motions had longer duration of shaking a longer period resulting from surface waves and site resonance. Large vertical accelerations were also recorded at locations close to the epicentre. Ground motions varied significantly in the epicentral region by a complex interaction of important effects such as local geology, topography and the directivity of wave propagation. The severely damaged area concentrated in a strip of 25 km long and 0.7-1.2 km wide between Hankyu Railway line and Route 43. The Rokko fault system does not exist under this strip by it runs parallel to the strip on the alluvial plane.

Vertical array data was recorded by the city of Kobe at the northwest corner of Port Island providing valuable site response. The dramatic change in strong motion records at different sites was probably caused by the softening of soils by liquefaction and non-linear response of the soft alluvial layer.

3.7 Acknowledgments

The author is grateful to Mr. Y. Iwasaki (Georesearch Institute) for providing useful information including the strong motion records obtained by CEORKA (Committee of Earthquake Observation and Research in the Kansai Area), Professor K. Nakagawa (Osaka City University) and the people of Disaster Prevention Institute of Kyoto University.

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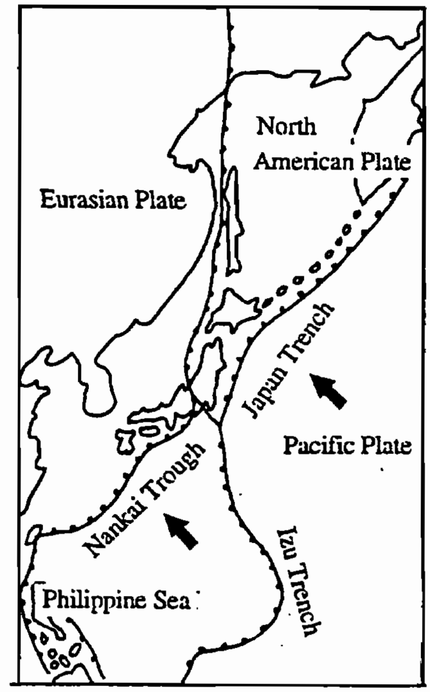


Figure 3.1 The islands of Japan and four techtonic plates

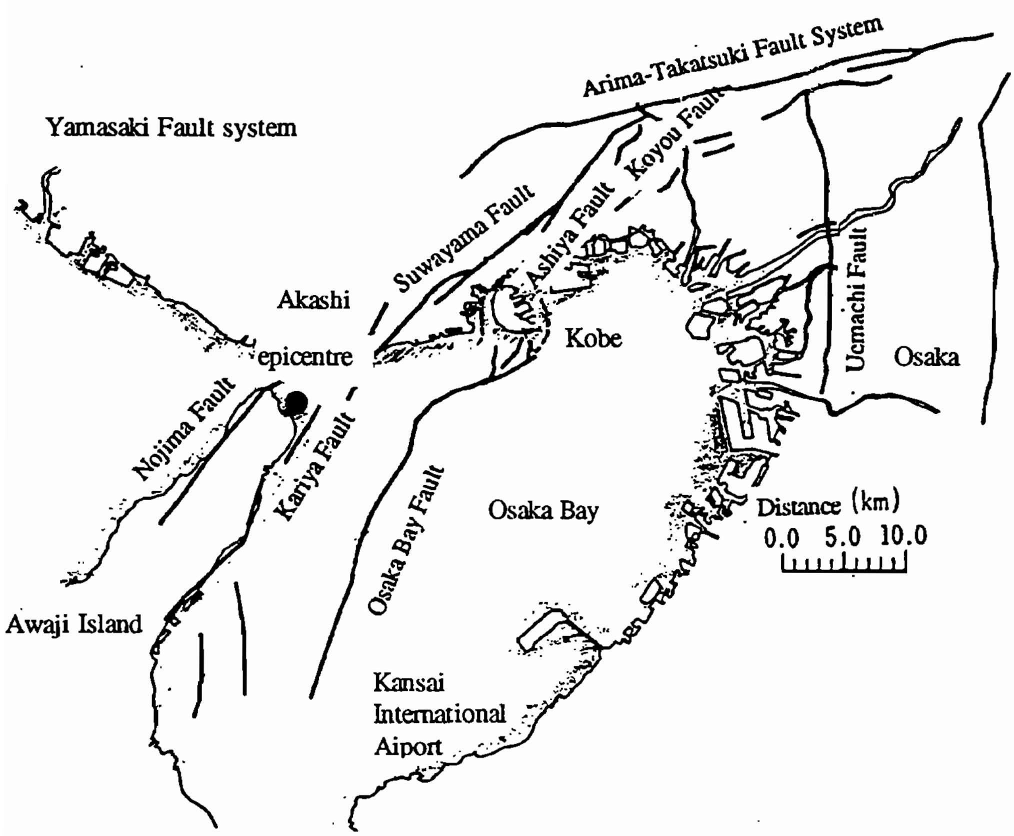


Figure 3.2 Active faults in the Kobe region (after Iwasaki (1995))

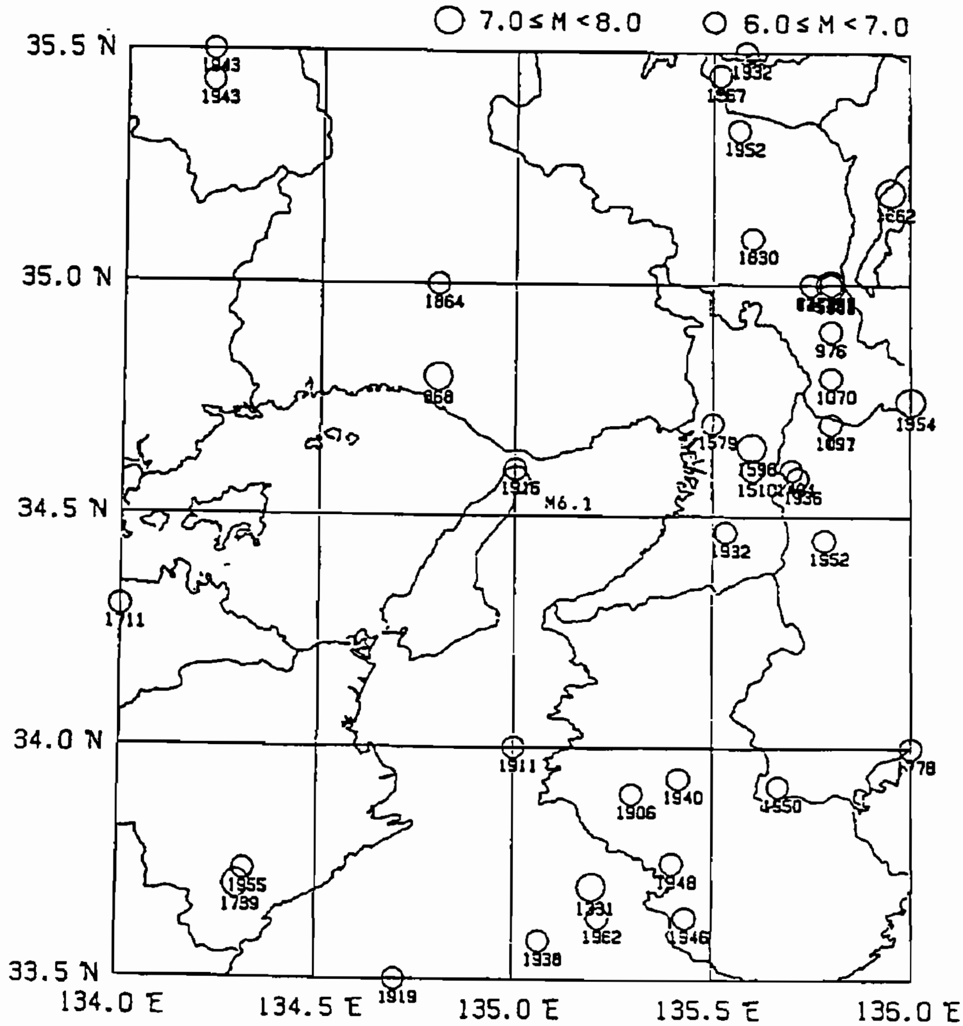


Figure 3.3 Locations of the epicntres of past earthquakes of greater than magnitude M6 (after Shimizu Construction, 1995)

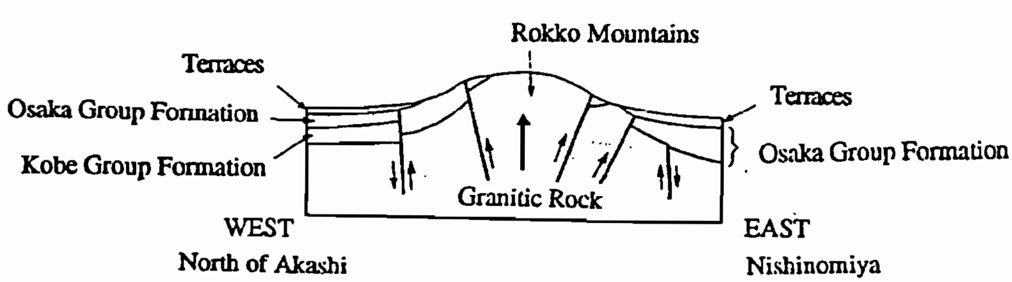


Figure 3.4 Schematic diagram of geographic features of the Kobe region

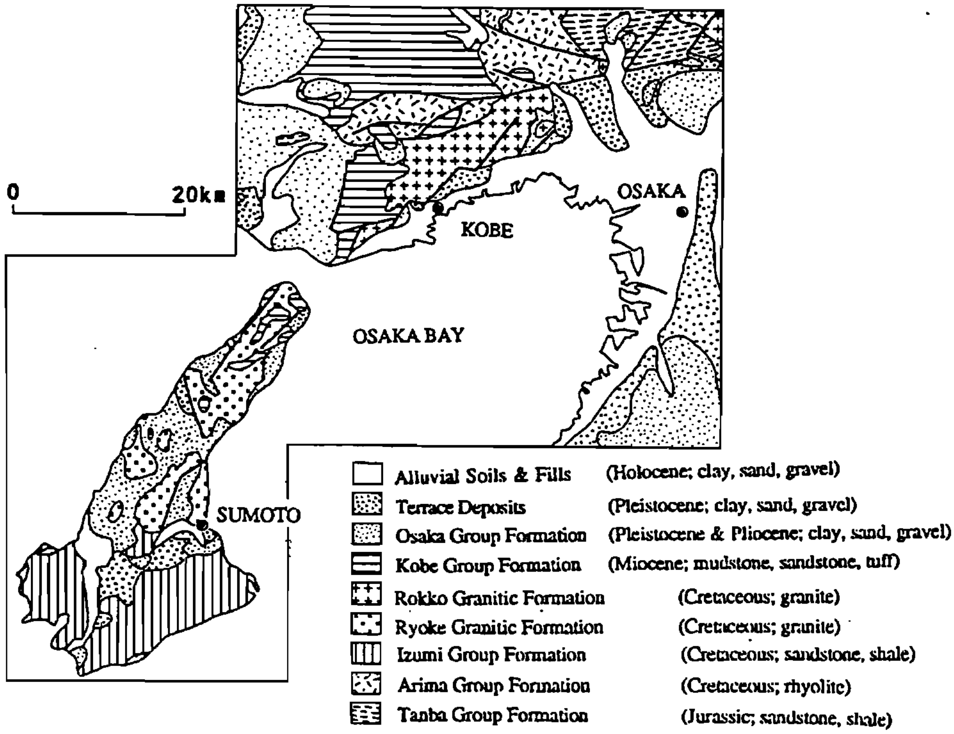


Figure 3.5 Surface geology of the Kobe region and Awaji Island (after PWRI, 1995)

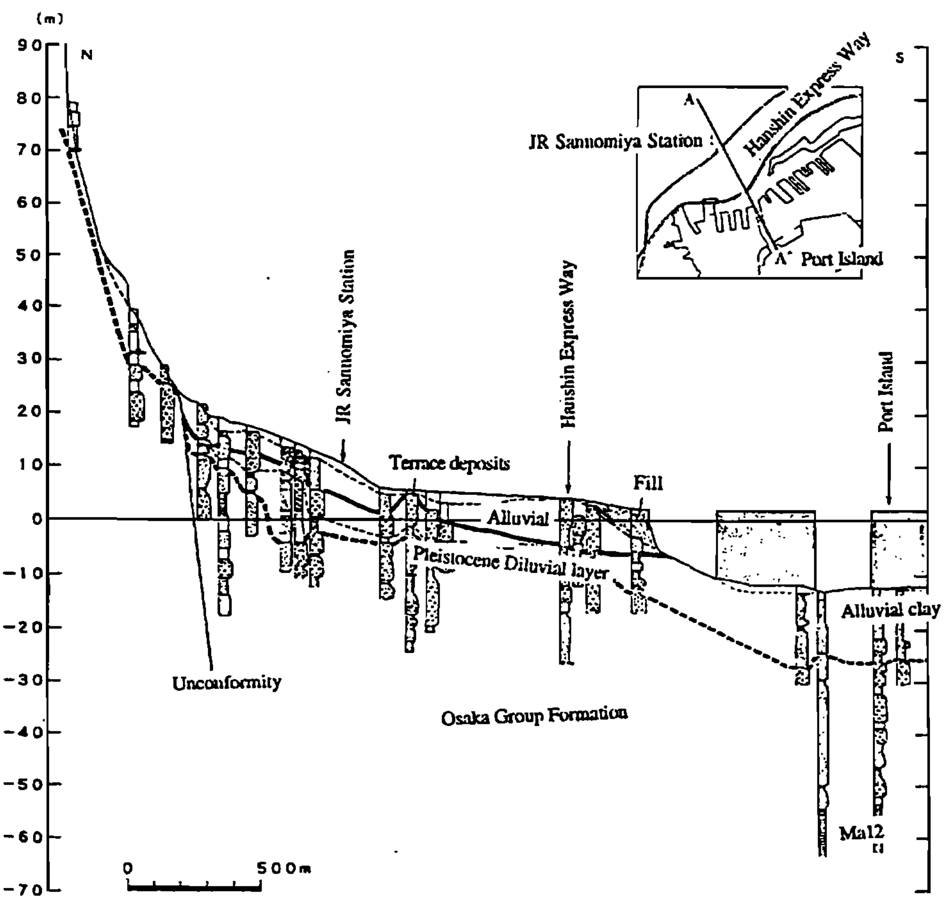


Figure 3.6 Subsurface section of Kobe (after Ishihara, et al. (1995))

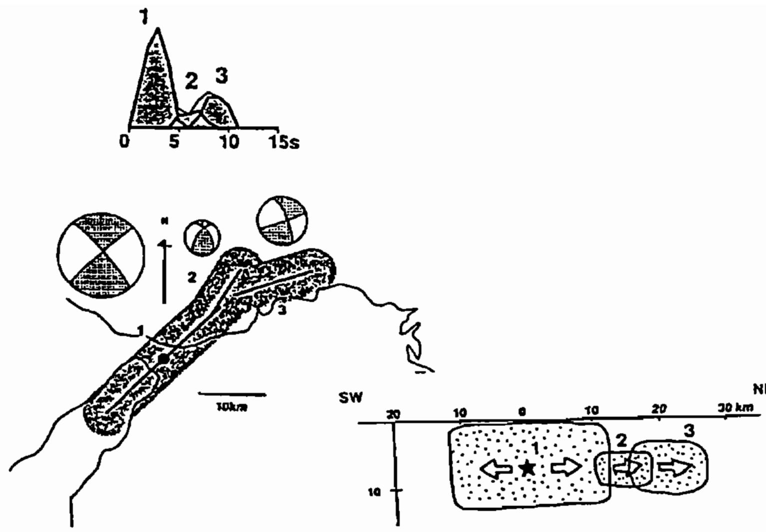


Figure 3.7 Three subevents of the Hyogoken Nanbu Earthquake reported by Kikuchi (1995)

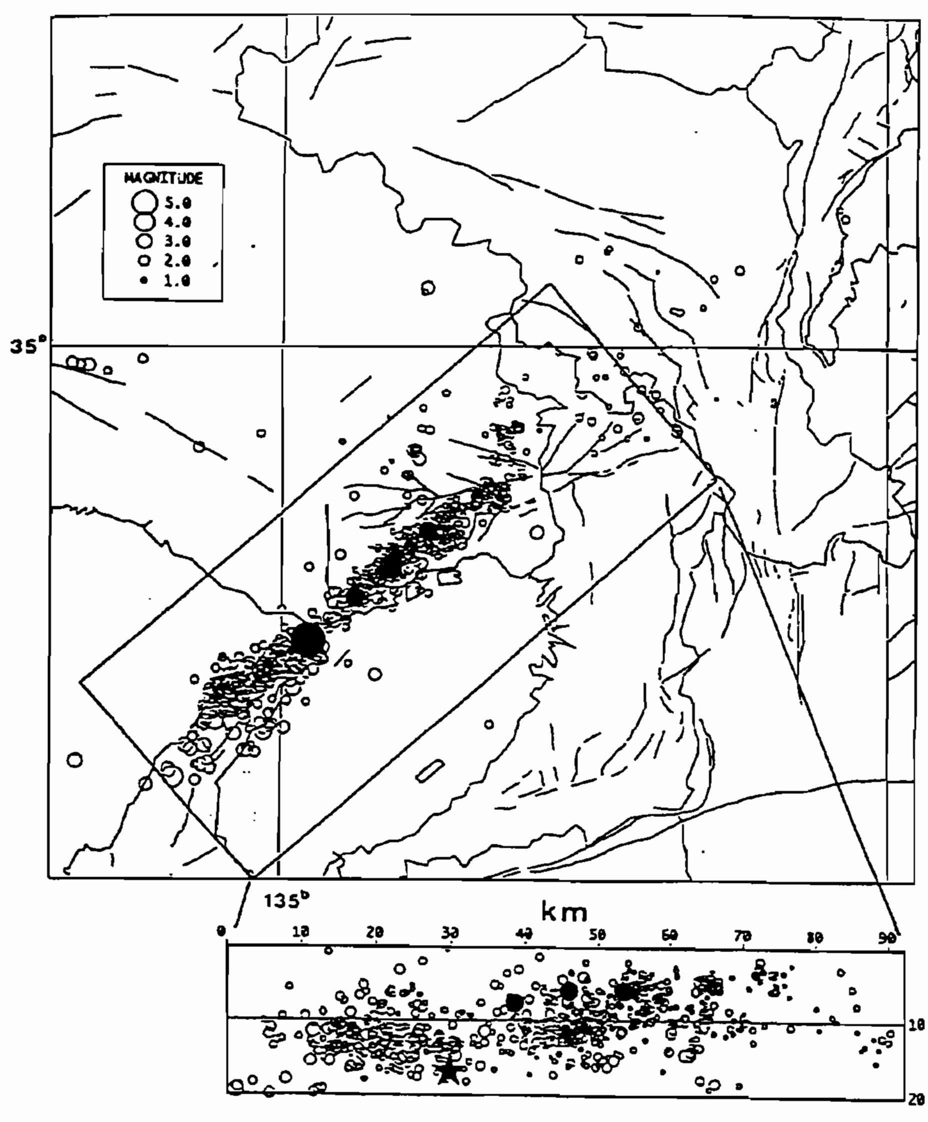


Figure 3.8 The distribution of aftershocks in the Osaka-Kobe region (DPRI, 1995)

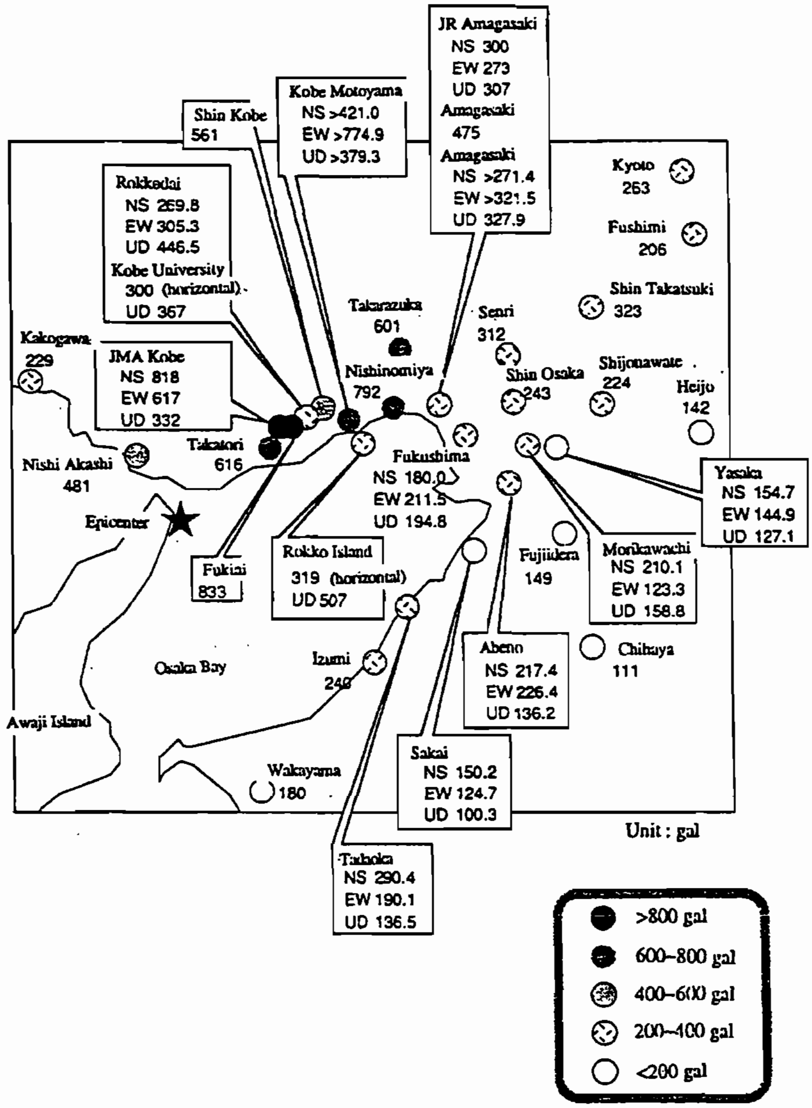


Figure 3.9 Recorded maximum accelerations (after Taisei Construction, 1995)

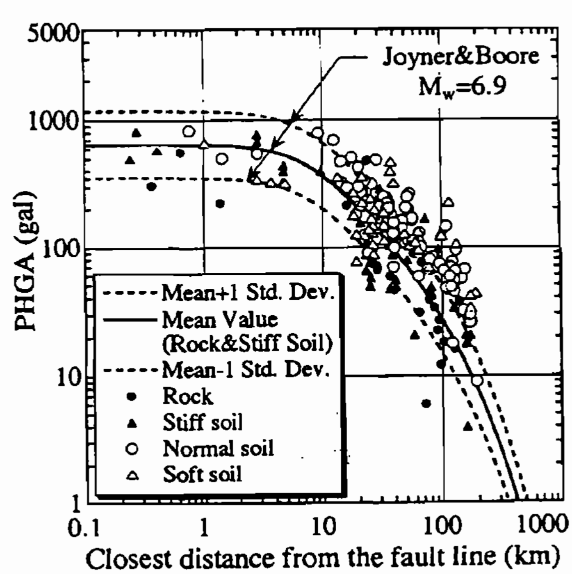


Figure 3.10 Attenuation of peak horizontal ground acceleration (Ejiri et al., 1996)

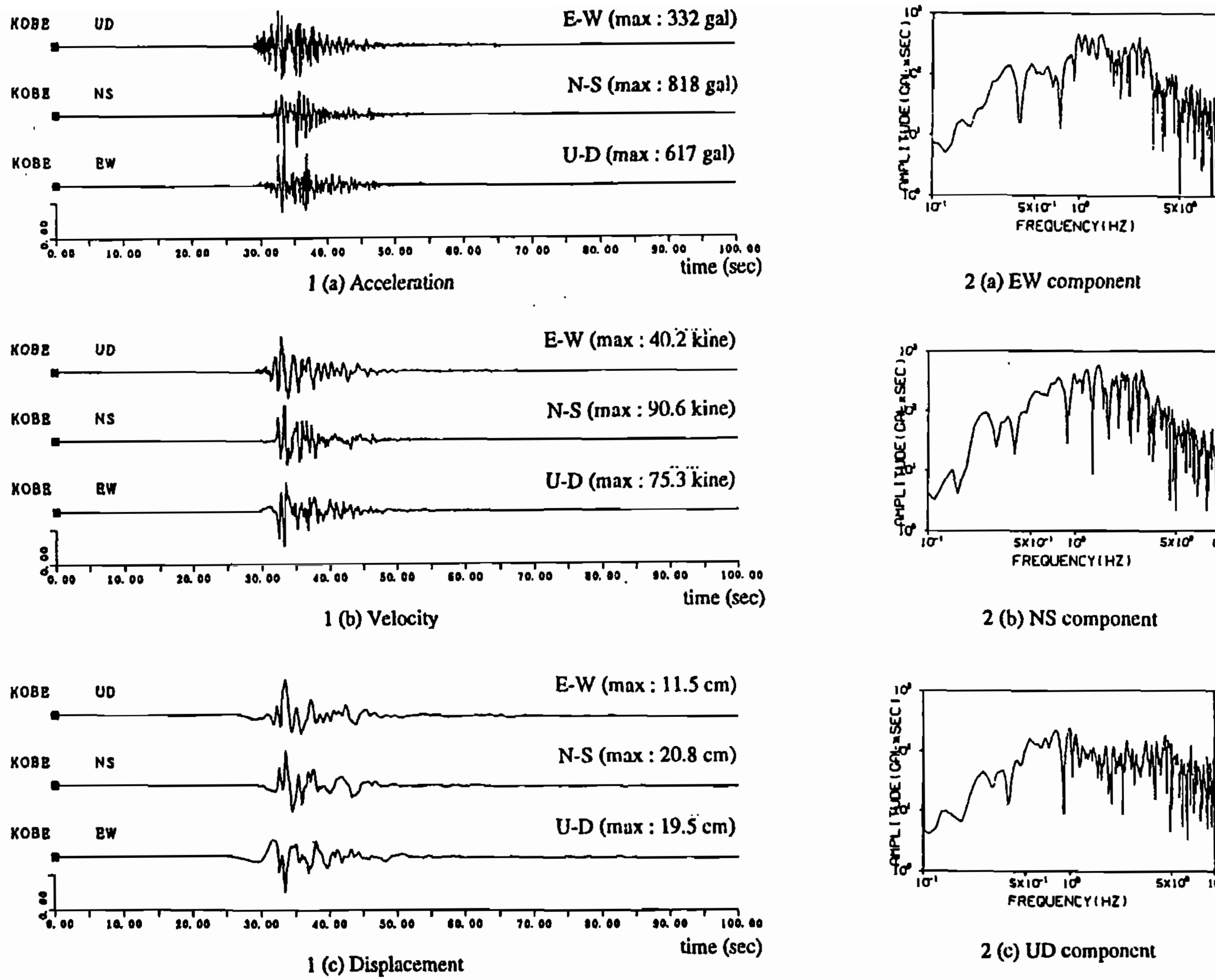
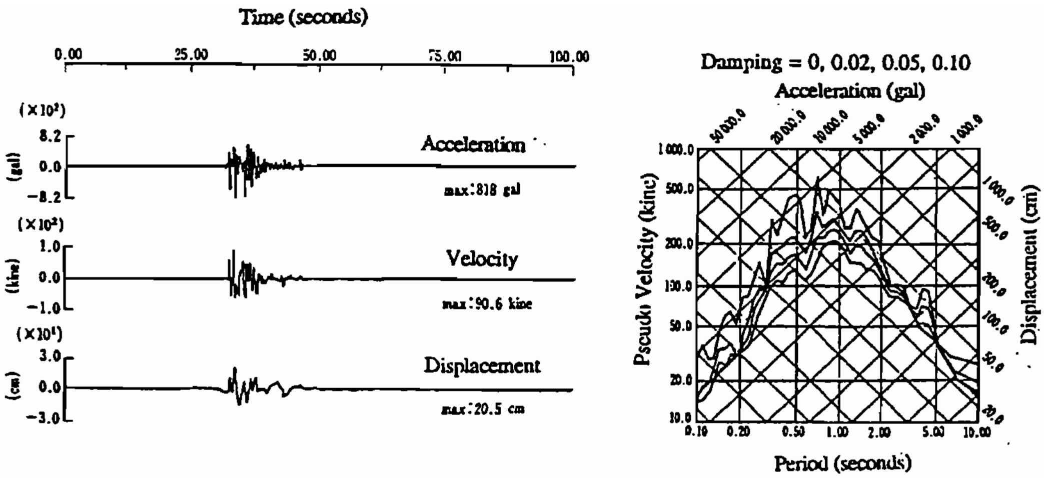
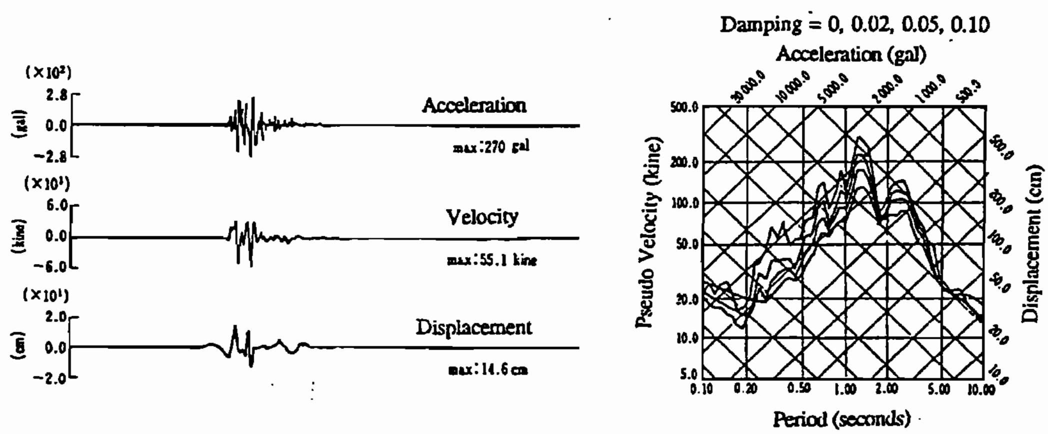


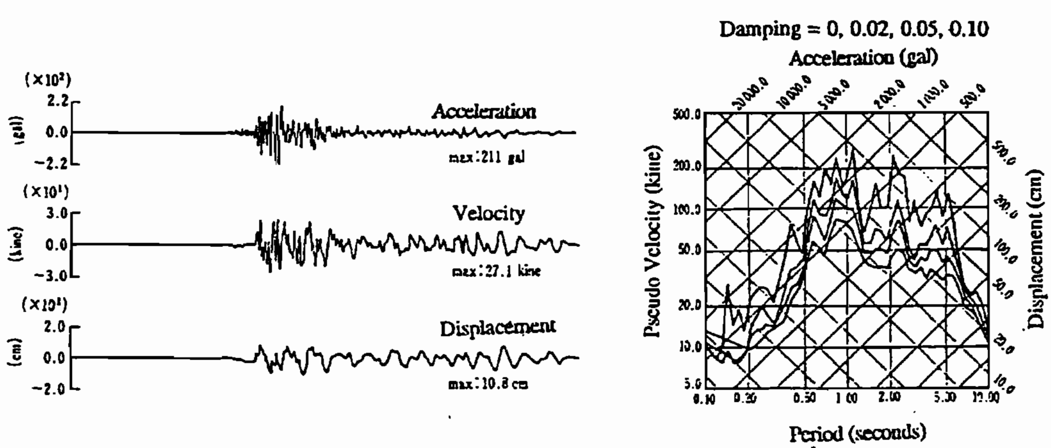
Figure 3.11 Strong ground motion records obtained at the JMA Kobe Meteorological Observatory



(A) JMA Kobe site (N-S component)



(B) Rokkodai site (N-S component)



(C) Osaka Morikawachi site (N-S component)

Figure 3.12 Strong ground motion records at the JMA Kobe site, Rokkodai site and Osaka Morikawachi site (after Iwasaki, 1995)

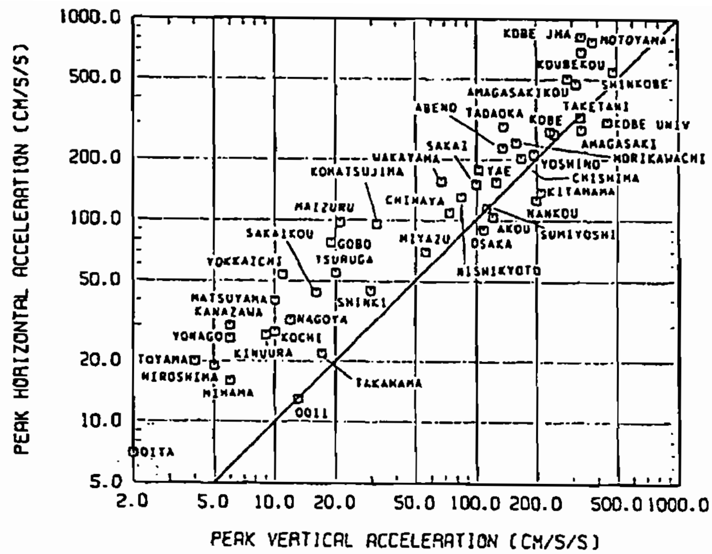


Figure 3.13 Vertical acceleration versus horizontal acceleration (after Shimizu Construction, 1995)

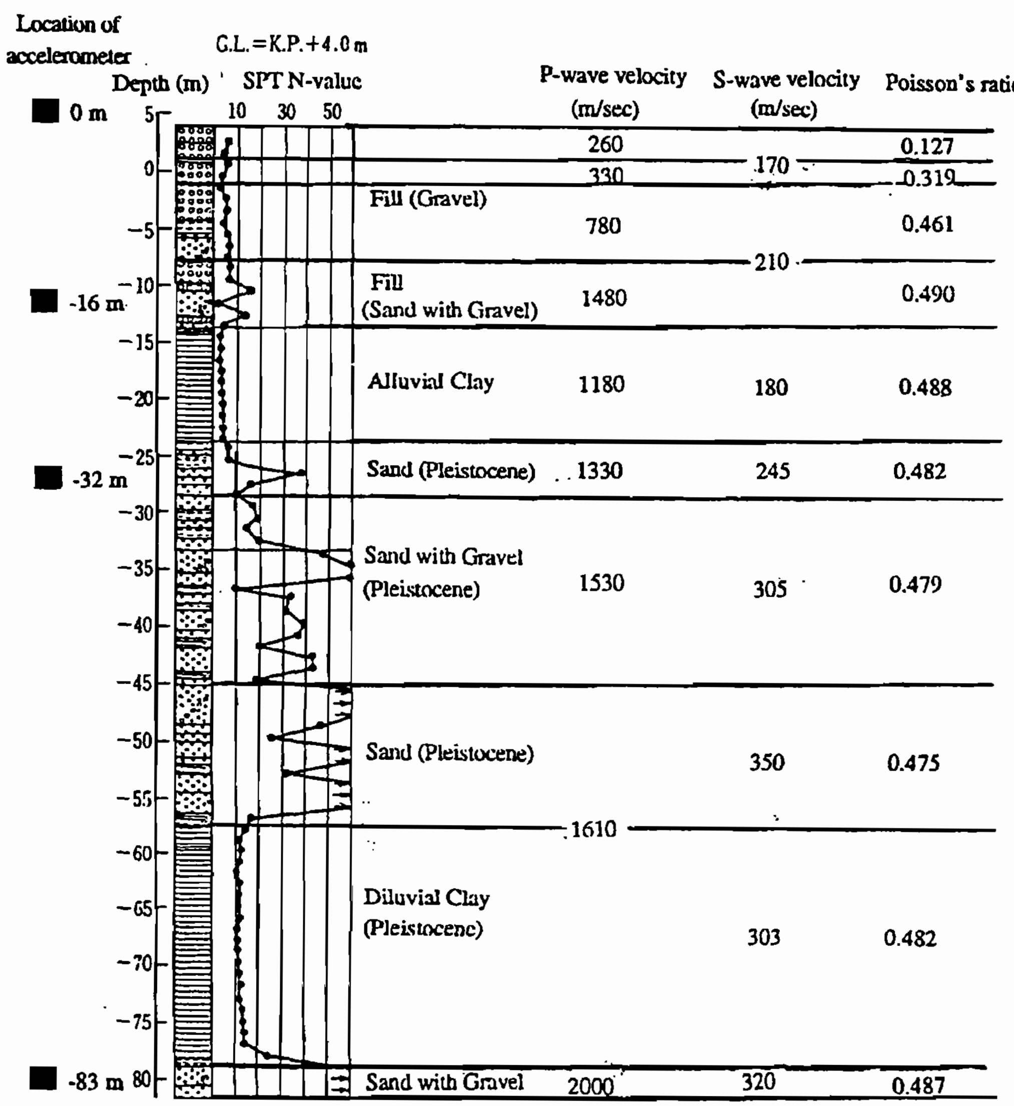


Figure 3.14 The locations of strong motion instruments at the northwest corner of Port Island

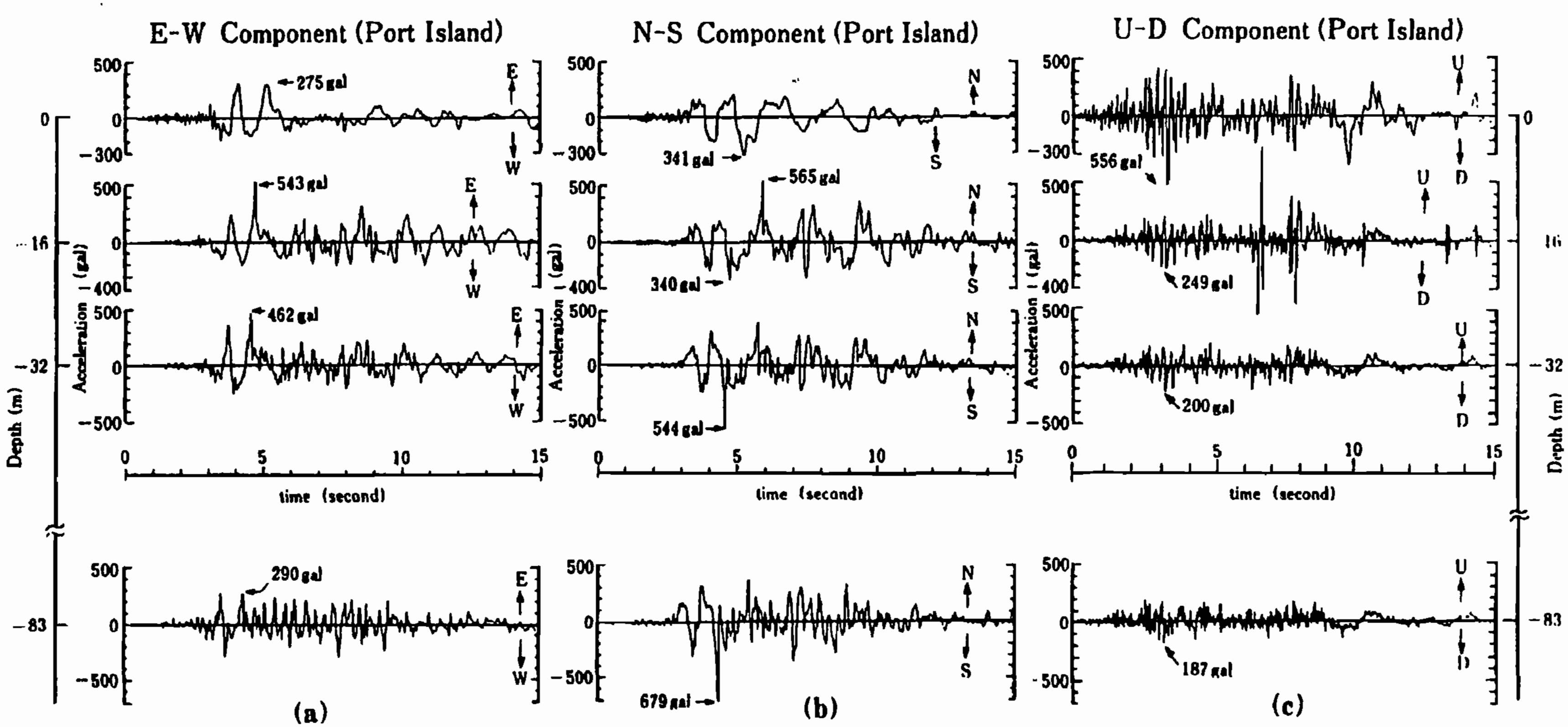
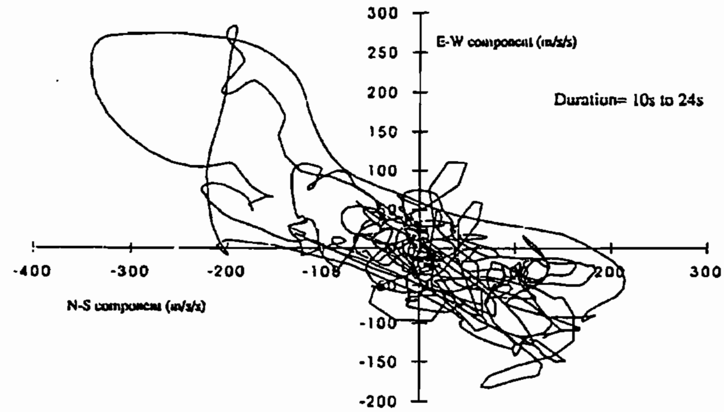
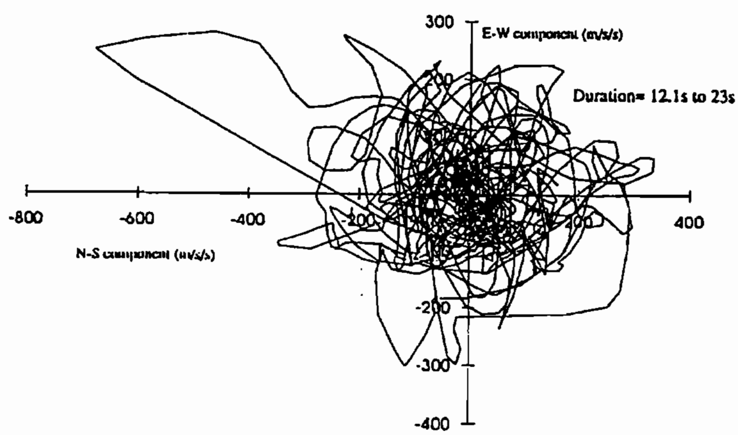


Figure 3.15 Vertical array record at Port Island (Iwasaki, 1995)



(A) At surface



(B) 83 m depth

Figure 3.16 Port Island ground motion in horizontal planes (Madabhushi, 1995)

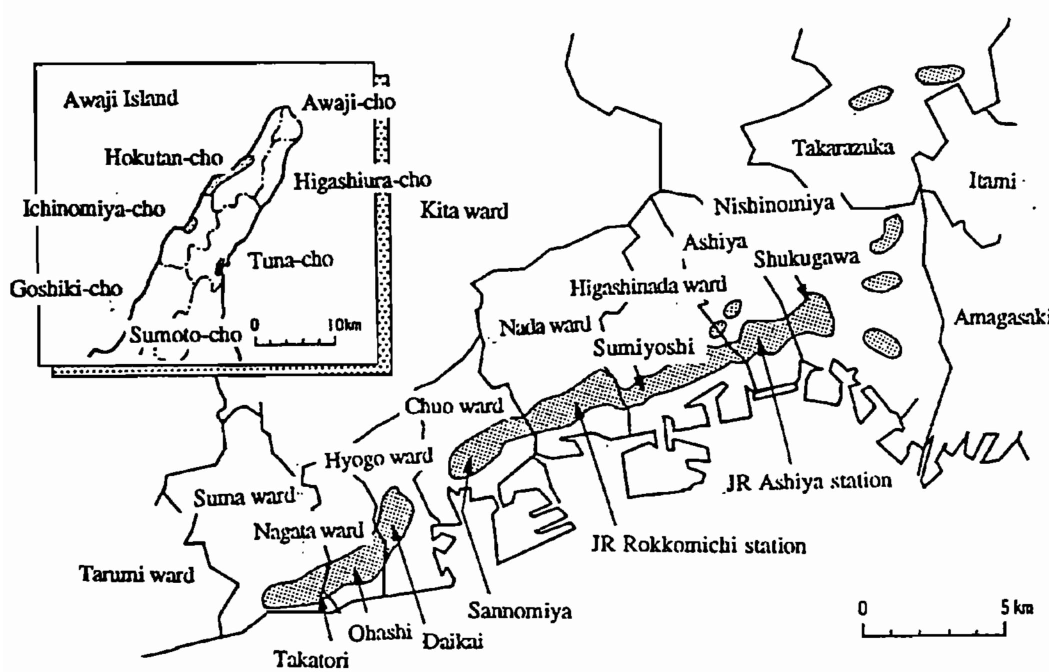


Figure 3.17 The area categorized as Intensity 7 by JMA

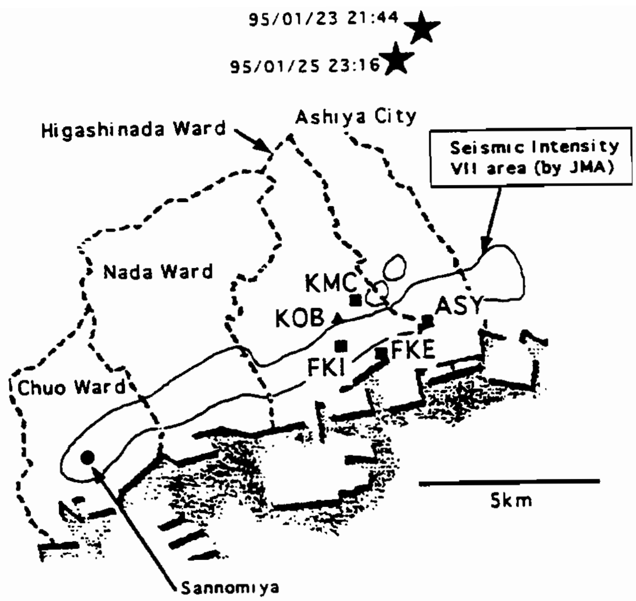


Figure 3.18 The locations of strong motion instruments installed by Iwate et al. (1995)

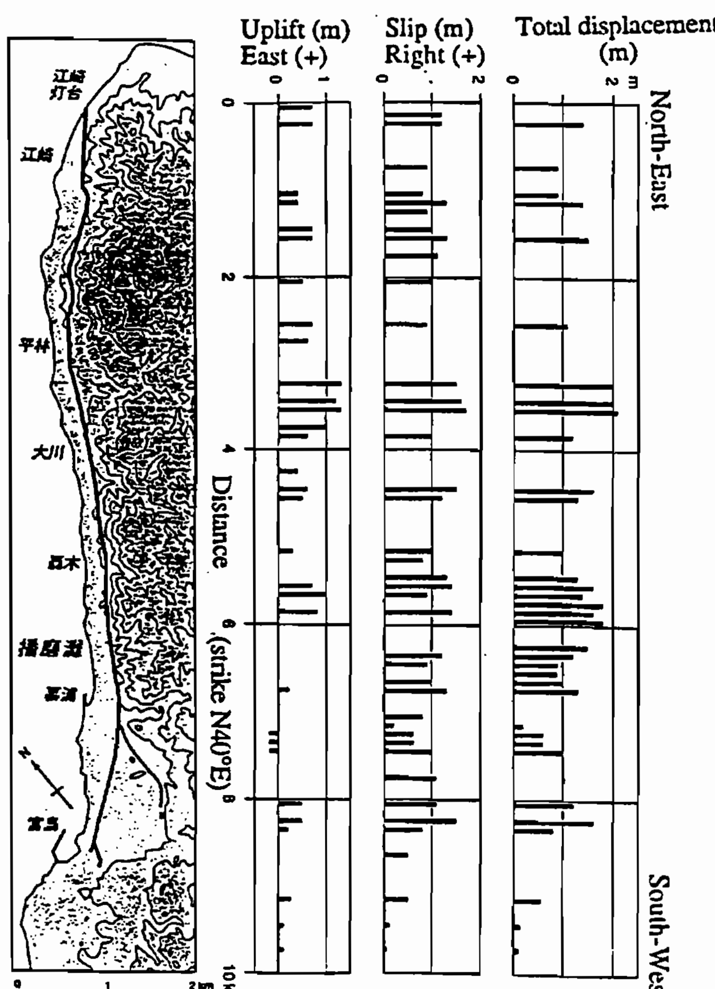


Figure 3.19 Surface displacements along the Nojima fault measured by Kurita et al. (1995)

Plate 3.1: A panoramic view of the Kobe region

Plate 3.2: The mountain range of Awaji Island looking toward Kobe.

Plate 3.3: Surface rupture of the Nojima fault observed at Okura, Awaji Island
(The rupture is covered by plastic sheets)

Plate 3.4: Pavement rupture due to the movement of the Nojima fault at Okura, Awaji Island

Plate 3.5: Surface rupture of the Nojima fault observed at Ezaki, Awaji Island

4.0 PERFORMANCE OF BUILDINGS

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Adam J Crewe, University of Bristol
Antonios Pomonis, Cambridge Architectural Research Ltd
Peter Merriman, British Nuclear Fuels Ltd
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The assistance is acknowledged of:
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Roy Page of Worthing

4.1 Introduction

The Kobe earthquake has been by far the most devastating earthquake yet in terms of the damage to a predominantly well built buildings stock of predominantly post-1950 construction, designed at least according to regulations for appreciable lateral forces and constructed generally to a high standard. It is nevertheless considered that some of the damaged buildings in Kobe had been designed for smaller forces than those required by the regulations. Perhaps this reflects the fact that Kobe is towards the southerly end of one of the two Japanese zones subject to the greatest risk and this applies particularly to some of the steel buildings built during the early years of Kobe's expansion.

Due to the quality of construction in Kobe, the common attribution of the damage in earthquake stricken regions to shoddy construction or poor materials is inappropriate. There is cause for concern that this was an earthquake of only moderate magnitude, yet many buildings were subjected to lateral forces between 150 and 250% of their ultimate limit state resistance (allowing for inelasticity). The damage to many of the older buildings without pronounced irregularities roughly corresponded to these load levels.[4.1] The effect of a really large earthquake in one of the five or so really large cities in Japan or California could be expected to be yet more devastating. Japanese design and construction practice before the earthquake is extensively reviewed, using information mainly from References 2 to 11.

This earthquake is notable for the widespread collapse of traditional housing, the surprisingly high number of mid-height collapses, mainly in commercial and residential properties but also in industrial structures, and the damage to steel buildings. The earthquake also highlighted the excellent performance of multi-storey car parks and the suspected excellent performance of piled building foundations.

The strategy for the inspection, which evolved during the visit, comprised an extensive examination of building exteriors, supplemented by telephoto enlargements of damaged areas. More details are given in Appendix 4D. When the opportunity arose selected buildings were also given an internal inspection. Multi-storey car parks were selected for thorough examination on account of their generally excellent performance, which contrasted markedly from that of this class of structure in the Northridge Earthquake.[4.1] Special studies were also made of industrial buildings and of the few base isolated structures. Some consideration was also given to the repairs.

There is insufficient information available to give a clear indication of the variation of the excitation in the areas of the greatest damage, as comparable excitations were recorded in central Kobe and Nishinomiya. The large majority of buildings considered in this chapter are from this area and for this reason their precise location is not identified.

In Kobe the maximum ground acceleration at various sites probably varied from 0.50g to 0.90g. Also in a 1km wide strip of mixed industrial, commercial and residential buildings between Kobe and Nishinomiya, and roughly centred on National Highway Route No 2, many buildings were badly damaged. At the Kobe end the peak ground acceleration (pga) was 0.80g perpendicular to the coast and 0.78g parallel to it. On the coast the greatest acceleration was perpendicular to it and possibly about 0.50g (there were two pgas of 0.50g and 0.45g). To the north of this the pga, in places, seems to have been up to 0.50g to 0.60g for about 1.5km.[(4.12)]

There are no records indicating the response of actual structures in the heavily damaged areas, though there are some from further afield, principally in Osaka. These are not discussed.

4.2 Building code requirements

The first earthquake resistant regulations in Japan were introduced in 1895 after the 1891 Nobi earthquake. They applied only to wooden buildings and were not compulsory.

Following the disastrous Great Kanto Earthquake of 1923, the Urban Building Law was introduced in 1924[(4.2)] and established allowable stress design procedures with a lateral force of 0.10W, where W is the weight of the structure. It was compulsory in Tokyo, Yokohama, Osaka, Nagoya, Kyoto and Kobe and a few other specially designated districts.

This was superseded in 1950 by the Building Standard Law,[(4.3)] in which the internal force was doubled to 0.20W, though with a corresponding increase in the allowable stresses.

As the requirements stood in 1970[(4.4)] just before many of the reinforced concrete buildings in Kobe were constructed, buildings taller than 9m to the eaves or 13m overall were not permitted in either timber or unreinforced masonry. Additionally there were height restrictions of 20m in residential districts and 31m (8 to 12 storeys) elsewhere. The design force was also increased from 0.20W to 0.24W (where W is the weight of the building) as the height increased from 20 to 31m. Higher values were employed for wooden buildings on soft ground, water tanks and domestic chimneys.

The present requirements, essentially those of the 1981 Code[(4.5)] (but reissued with very minor alterations in 1987) took into account the analysis of damage in the Off Miyagi earthquake in 1978 and the approach is quite different from the contemporary Uniform Building Code[(4.6)] used in California. The essential features of the (current) Japanese design method for buildings less than 60m high (16-22 storeys in most countries but sometimes 14 storeys in Japan) in Japan are[(4.5, 4.7)]:

(1) Elastic analysis with a static equivalent force method of assessing seismic effects and working stress design at the serviceability limit state (SLS).

(2) A basic lateral force of 0.2W for the 'moderate' earthquake, which is used for all checks at SLS in all regions. Reductions are made for longer period structures depending on the soil type, corresponding to elastic design with a peak ground acceleration (pga) of 0.067 to 0.083g (derived from Japanese spectra used elsewhere in earthquake resistant design).

    The distribution of force over the height of the building varies with the period, with equal weighting of masses over the height for very short periods.

(3) Where checks are required at the ultimate limit state (ULS) in regions subject to severe earthquakes (but see (5)) the basic lateral force is 5 times that in (2), which corresponds to elastic design with a pga of 0.33 to 0.42g.

(4) Checks are made on the storey sway in terms of the minimum sway angle compared to the average for all storeys and a maximum sway angle. The first of these criteria effectively enforces regular changes of cross-section with height.

(5) For regular buildings up to a height of 31m (8 to 11 storeys) no checks are required under the severe earthquake at ULS, subject to the following checks.

A *For steel buildings:*

(i) an increase in the design force of up to 50% is required if all the lateral load is carried by bracing.

(ii) for braced frames the strength of the connection must be at least 1.2 times that of the member, and for moment resisting frames a comparable check is required that the beam column connection is stronger than either the beams or the columns.

(iii) for moment resisting frames there are restrictions on the cross section slenderness.

B *For reinforced concrete buildings, including buildings with some or all the columns of steel reinforced concrete (SRC - see later):*

(i) there are minimum areas of vertical load bearing elements as a proportion of the floor area, which is expressed as the sum of the weighted areas of RC columns, SRC columns and RC walls.

(ii) for moment resisting frames the shear strength of members must exceed the flexural strength, which effectively requires an overstrength factor of one at SLS, but more at ULS.

(6) For buildings exceeding a height of 31 metres and for irregular buildings up to 31 metres, a check is required on the ultimate shear strength of each storey Q_{ULS}, which can be expressed in terms of the shear force at SLS as:

$Q_{ULS} = 5K_1 K_2 Q_{SLS}$, where

K_1 is a factor of between 0.25 and 0.55 depending on the material ductility and K_2 is a factor of between 1 and 2.55 depending upon the degree of irregularity.

The factor 5 represents the five fold increase in the lateral force discussed in (3).

The ratio Q_{ULS}/Q_{SLS} varies from 1.25 to 2.50 for most buildings, but could reach 6.19.

Storey drift is limited to 0.5% of storey height based on the damage surveys of the 1978 earthquake in Sendai which indicated that damage initiated when the storey drift exceeded 0.4% of the storey height. This is similar to the US limit. The storey drift is allowed to increase to up to 0.8% if there is little likelihood of severe damage to the non-structural elements (e.g. infill masonry, curtain walling, glass cladding etc). There are also stiffness and eccentricity limitations.

Appendix 4A gives the actual expression for horizontal force in the Japanese code and compares it with that in the UBC, which is similar to that in other US codes.

The code has been widely regarded as the most advanced in the world, principally due to the avoidance of unrealistic inelastic corrections and the introduction of inelastic time history analysis for very tall buildings. It has nevertheless been criticised in Japan as follows[4.7]:

- in order to satisfy the shape factor requirements, engineers are often tempted to use less than the required amount of shear walls and bracing needed to provide an adequate lateral load resisting system;
- in order to satisfy the stiffness and lateral eccentricity limitations, engineers are often tempted unnecessarily to increase the size of the columns;
- the structural coefficient in the ultimate shear strength calculations, is estimated for the whole structure rather than for each storey;
- the seismic zonation of the country is too crude;
- importance factors are not used;
- there are few regulations for the sub-structure;
- there are insufficient provisions for the prevention of nonstructural damage and equipment anchoring etc;
- non-linear behaviour after yielding is insufficiently addressed; and
- the seismic zonation and requirement for seismic resistant design is not enforced.

4.3 Japanese building stock

4.3.1 Composition

Studies of the building stock profile in Osaka in the late seventies (408,000 buildings) reveal that 80% were timber and 90% of the buildings were up to three storeys high. The remaining 10% are medium-rise buildings, generally of 4 to 15 storeys. Less than 0.2 of a percent of the building stock were more than 10 storeys high[4.8]. Very similar statistics apply for the city of Kobe and the townships of Ashiya and Nishinomiya at the time of the earthquake.

In recent decades more medium and high-rise apartment buildings of reinforced concrete, steel, steel reinforced concrete (see 4.3.8) and mixed construction have been erected in most Japanese cities. High-rise office and other commercial buildings are also commonplace and are mostly of steel or composite steel and RC construction. Old masonry buildings still exist but relatively few new masonry buildings have been constructed in recent decades. Industrial and agro-industrial structures are mostly of steel or RC. Steel and RC have a long history of construction in Japan and have been built in significant numbers since the early years of the 20th Century. According to the annual construction statistics new steel frame buildings overtook reinforced concrete buildings, in terms of floor area, in 1967 and since then have become progressively more common[4.9]. Reference 4.9 gives data on the structural systems employed in new buildings by structural type in Japan for the period 1960-1978. This data is summarised in Figure 4.1 for the years 1960 and 1978 respectively.

The next sections outline Japanese building and construction practices and where appropriate make comparisons with corresponding practice in the United States and other industrialised countries.

4.3.2 Old traditional shinkabe/okabe wooden buildings

The typical wooden low-rise buildings in Japan did not change very much until the seventies. There are two main types called "shinkabe" and "okabe".

Scawthorn[4.8] describes "shinkabe" as a timber post and beam vertical load carrying system with lateral resistance provided by infills comprising mud-filled two-way bamboo latticed walls. Characteristics affecting its earthquake vulnerability are:

- the wide-spread use of heavy tile roofs;
- internal walls of light construction with very wide sliding paper partitions called shoji (and usually devoid of strength);
- they rarely exceed 3 storeys.

Foundations in old buildings consist of isolated stone or concrete footings which have been largely replaced in newer buildings by perimeter concrete walls.

"Okabe" is the more recent variant with timber lath - stucco infills.[4.8]

Exploded views of these buildings from a different source and with some additional distinctions are shown in Figure 4.2.

The terrace version of shinkabe is "machiya", and many of these houses had shop fronts with soft storey characteristics.

Okabe and even shinkabe buildings are still built in Japan, but they have become less popular since 1971 with the introduction of more standardised prefabricated timber housing in all the major urban areas of Japan. Furthermore the newer shinkabe and okabe buildings tend to have more diagonal bracing than the older ones. Nowadays most construction of even these is largely confined in the areas of lesser seismic hazard (eg southern part of Japan).

4.3.3 Modern Japanese wooden construction

In recent decades and especially during the post-1971 period the nature of the traditional Japanese domestic dwelling has altered. Western engineering and architectural principles are mixed with traditional Japanese architecture and wood joinery. The typical new timber house is now closer to that in California, although, in Japan, the two-storey variety is rather more common than the single-storey variety due to the higher cost of land. The main changes that took place and contributed to the reduction of seismic vulnerability (along with the code developments) are:

- Introduction of veneer board planks. These replaced the traditional shoji or sliding paper walls, thus increasing the amount of wall area.

- Increase in the number of private rooms: social changes and urbanisation contributed to the demand for houses with private rooms, thus increasing the proportion of interior wall area.

- Replacement of the traditional mud or plaster external walls by cement hollow block masonry thereby the mass of buildings has been reduced considerably.

Prefabricated timber housing has been used in Japan for the last two decades and its popularity is increasing. About 30% of new houses are now prefabricated. Plan shapes are standard with little variety and tend to be fairly symmetrical in layout; so one would expect good seismic performance. The wall perforations are less than in "shinkabe" houses and substantial bracing and sheathing covers the whole building. However, timber buildings with brick or other veneer are uncommon in Japan. The sill plate is always bolted to the concrete foundation and cripple walls are rare. Otherwise, and apart for the greater symmetry of the Japanese houses, these prefabricated wooden buildings are of comparable construction to Californian houses.

The risk of post-earthquake conflagration due to the high density of most cities in Japan is quite serious, and fires in wooden houses are an additional risk factor to be taken into consideration. Nevertheless a two storey restriction on timber buildings has recently been so relaxed to three storeys.

4.3.4 Unreinforced masonry

There are few unreinforced masonry residential buildings in Japan. In many areas its construction was made illegal after the 1891 Nobi earthquake. Some unreinforced masonry buildings existed until the sixties, but either due to the increase in number of buildings or due to natural selection (demolition due to ageing or earthquake damage) their proportion has decreased substantially. Many of the remaining have also been reinforced. Whilst they do seem similar to those in other countries, the most common type is brick load-bearing masonry which is considered by Japanese researchers to be weaker than the equivalent Californian unreinforced masonry[4.10]. In many areas cut volcanic stone is also used as a material for load-bearing masonry, mostly used in barns and warehouses.

4.3.5 Reinforced masonry

There is a wide variety of reinforced masonry buildings in Japan, as shown in Table 4.1, but the most common in recent decades is the reinforced hollow concrete block variety. They are usually the double void type with vertical steel reinforcing bars at close intervals. Solid brick masonry walls are also used. Masonry columns with internal void spaces for vertical steel bars and concrete grouting are used in corners and near openings. In the more recent buildings hollow clay brick blocks are used with horizontal steel bars every three masonry courses. The typical building in this category is low-rise (up to 3 storeys) and the foundation usually consists of continuous inverted T reinforced concrete spread footings. The reinforcing bars are all tied together by means of hooks Reinforced concrete ring beams are also widely used in this form of construction.

4.3.6 Old Japanese moment-resistant reinforced concrete frames

There are still an appreciable number of old RC frame structures usually without shear walls which were built before the introduction of the 1950 seismic provisions. Reinforced concrete construction predating 1950 is of a generally poor standard and would be expected to have poor earthquake resistance.

4.3.7 Moment-resistant reinforced concrete frame with shear walls

This is the most common type of reinforced concrete buildings in Japan. Most are 4 to 10 storeys high. Collapse of post-1950 reinforced concrete buildings solely attributed to ground shaking was very rare in Japan until this earthquake. In the 1964 Niigata earthquake several buildings of this type collapsed or tilted, but this was due to the soil liquefaction rather than ground shaking. In the 1978 Off Miyagi earthquake, 5 reinforced concrete buildings partially collapsed in the city of Sendai, including the severely damaged Nagaoka hospital.

4.3.8 Steel reinforced concrete composite (SRC)

This is one of the most common structural systems in Japan for buildings between 8 and 20 storeys and many of the office and commercial buildings are built using this system. In this structural system (commonly referred as "SRC"), steel rolled sections are encased in reinforced concrete containing appreciable more reinforcement than in composite construction, the corresponding form of construction used in Europe and in North America. The beams may be cased or uncased (see 4.6.3). The most common steel sections in SRC are the I-beams (often fabricated into crosses), while T-beams are preferred for perimeter and corner columns. Typical details are shown in Figure 4.3. The concrete cover of the steel section is at least 60mm. Circular or hollow square steel sections are also used in columns. Floor slabs are either insitu reinforced concrete or precast slabs.

Before 1970 both RC and SRC had been used for buildings up to the limiting height of 30m, then equivalent to 10 or 11 storeys. Due to the generally better performance of SRC in the Tochachi earthquake of 1968 measures were introduced in 1971 following recommendations by the AIJ [4.11] by which for buildings of 7 to 14 storeys the lowest 5 to 7 were to be of SRC. Nevertheless post-earthquake surveys have shown that RC moment resistant frames continued to be used for buildings up to 10 storey [4.13] with the overall position being that for buildings over 7 storeys built since about 1970 many have SRC in the lower part.

More recently lightweight concrete has been introduced for the floors, particularly in buildings of 9 or more storeys, achieving a reduction in total building weight of 15 to 20%.

4.3.9 Precast reinforced concrete

There is a variety of precast reinforced concrete buildings in Japan, with various levels of prefabrication. The most common element are the precast RC hollow slabs. They are generally covered by a reinforced insitu topping about 50mm thick. There are a few entirely precast buildings with reinforced concrete shear walls and without frames.

4.3.10 Steel frames

Steel frames are the preferred structural system for high rise office or commercial buildings (20 storeys or more), but not for industrial buildings or other special building structures. Their design is similar to their Californian counterparts. Various types of bracings are being used, but the most common are the concentric ones, although chevron or inverted V-braces are also used and some have eccentric connections. The use of steel frames in residential buildings has increased recently. In a recent survey of nearly 1000 damaged steel buildings, excluding the older "light gauge" (lattice) construction, 70% of the minority of buildings for which the structural system could be determined had sway frames in both directions, with the remaining having braced frames at least in one direction[4.14]. In sway frames, heavy I section columns were somewhat more numerous than cold formed box columns, which have been popular since about 1980. Prior to this, box columns were made out of plated I beams. It is presumed that box columns are popular in Japan due to their bi-directional role, enabling all columns to contribute to the seismic resistance in both directions. Column-to-column connections are mostly welded, beam-to-beam connections mostly bolted with high strength bolts, and beam-to-column connections are nearly all shop fillet welded. In the older beam to column connections with wide flanged beams, stiffening plates (continuity plates) were often omitted and the webs were often bolted (the latter as in former US practice).

4.3.11 Light metal frames

Light metal frames systems are most common in industrial buildings and the two most common varieties are shown in Table 4.1. The majority of small and narrow 3-5 storey commercial buildings of mixed use which are so common in business districts of Japanese cities have light metal frames.

4.3.12 Mixed structural systems

An increasing number of modern major buildings have mixed structural systems with reinforced concrete, steel and SRC frame systems in a single building. Common combinations are:

- RC frame on the sides and SRC in the centre (usually in low-rise buildings).
- SRC for lower floors and RC frame for top floors (usually in medium-rise buildings).
- Steel frame with SRC and RC frame in the basement (in high-rise buildings).
- SRC columns linked with steel beams and RC in the basement (in medium and high-rise buildings).

4.3.13 Identification of structural systems

The Japanese code encourages designers to construct all walls in RC, so pure forms of RC frames are scarcer here than elsewhere and their identification can be difficult. Further, the general mix of construction types described in 4.3.12 makes it exceeding difficult to distinguish between SRC and RC construction, though SRC columns can be inferred with reasonable certainty where lengths of steel beams are visible.

4.4 Performance of reinforced concrete buildings up to a height of 60 metres

4.4.1 General

In Kobe many reinforced concrete buildings either had relatively light cladding systems, sometimes with deep aluminium mullions or external screens, or alternatively had masonry clad reinforced concrete external walls, sometimes constituting perforated shear wall. In both cases the internal structure, and sometimes even the external structure, was of mixed wall and column construction.

4.4.2 Older masonry-clad framed buildings

4.4.2.1 Kobe Immigration Office

The 8 Storey Kobe Immigration Office, constructed a 1972 is a typical weak column strong beam facade construction. On the facade the damage was confined to the short face on the west side, (see plates 4.1 and 4.2) and was most severe over the central bays which lacked restraint from the glazing. The damage in the third storey was reminiscent of Champagne Towers in Santa Monica[4.1]. In one mullion column the concrete core had fallen out, revealing small corner bars and widely spaced links (plate 4.2).

Internally intermittent longitudinal walls either side of the central corridor had diagonal X cracks, and these walls, being stiff, had protected the columns on the long facades. The wide chevron corner columns contributed to the protection of the mullions on all sides. Due to the large offices there were insufficient cross walls to protect the columns on the short facades from severe diagonal cracking, which was more noticeable on the unclad internal face (plate 4.3).

In the basement car park there was predominantly one-way diagonal cracking in the internal walls.

4.4.2.2 9-storey building to north of German Consulate

This interesting building of similar age and with similar perforated masonry clad RC perimeter walls to that in 4.4.2.1, had suffered severe flexural shear cracking in most of the columns on the facade except the corner columns, with the result that the inelasticity had been distributed over many storeys despite the strong beam/weak column configuration (plate 4.4). The avoidance of a single storey collapse appears to have been attributable to the substantial corner mullions which were of the same size and doubtlessly similarly reinforced to the other mullions. This mechanism is one which might be expected from an elastic analysis of frames with rigid beams and flexible columns, where the design for lateral loading assumed points of contraflexure at the centre of the columns. This was a common assumption in the design of frames in which the beams were stiffer than the columns before the widespread use of computers.

If in the design, shear had been assumed to have been shared equally between all the columns, or alternatively if all the columns had the same cross sectional areas and similar reinforcement and hence identical strength, then the first occurrence of damage can be forecast according to the distribution of moments, and shear as follows.

a) With the beams much stiffer than the columns:

external columns attract the same moment as internal columns,

external beams attract 2 times the moment of internal beams, and

external beams attract 1.5 times the shear of internal beams.

b) On the other hand with columns much stiffer than the beams:

external column attract half the moment of internal columns and

external beams attract the same moments and shears as internal beams.

Therefore, with columns stiffer than the beams, the overstrength of the corner columns deters single storey collapse, so inducing inelasticity at other levels. This matter is considered further in Appendix 4B in relation to some sample structures.

4.4.2.3 Two 8-storey buildings opposite the German Consulate

A variant of the type of buildings considered in 4.4.2.1 and 4.4.2.2 is provided by two buildings to the east of the German Consulate with exposed rectangular columns at typical bay spacing (every 6 windows) at ground floor level, but mostly obscured behind masonry-clad RC walling in the storeys above (plate 4.5). Internal columns were visible through broken windows (plate 4.6). The two buildings were similar except for the fact that the more northerly was monolithic with an unperforated masonry clad tower at the North end. The more northerly building had dropped one storey relative to its neighbour with the total collapse of the lower storey, both in the main building and the tower, and this building had moved in a westerly direction (away from the street) by about 300mm, so that it was no longer aligned with its more southerly neighbour.

Damage in the more southerly building was quite different, taking the form of flexural cracking in most of the mullions, being least in the top two storeys and most in the third to fifth storeys. The damage had occurred only on the N/S facade and the similar E/W facade on the south side was undamaged. The internal columns had suffered a comparable amount of damage. The widespread distribution of damage is attributable to the strong corner columns, as in 4.4.2.2.

The striking differences in the performance of the two buildings were in the direction of movement, the storey collapse in the more northerly, the excellent condition of the mullions elsewhere in that building, as opposed to the generally cracked condition on the more southerly building and the 15% loss of glazing panels in the more northerly building, compared to 70% in the more southerly. Despite the high breakage of glass much it was held by the glass retention/sealing system. The AIJ Report[4.15] notes that in buildings suffering failure at ground floor the damage in the upper storeys was minimal.

4.4.2.4 9-storey building to the NW of the German Consulate

This building with masonry clad RC perimeter walls with window perforations such as to form strong beam/weak column construction had suffered collapse on the third storey (plate 4.7), which had caused the upper part of the building to lurch forwards in southerly direction on the east side (plates 4.8 and 4.9), where the masonry wall was imperforate. The failure mode is likely to have been initiated by load attracted to the stiff side wall and it is interesting that the greater movement occurred on this side rather than on the side opposite, where elastic or inelastic analysis would have predicted the greater movements.

4.4.3 Soft storey collapses in framed buildings with continuous fenestration

4.4.3.1 12-storey building opposite the Town Hall

This irregular 12-storey building to the east of the town hall had RC shear walls on 3 sides but the fourth side along Flower Street had curtain walling with deep aluminium mullions. A collapse in the fourth storey on the street side had caused the building to tilt forward at that level (plates 4.10 and 4.11). The structural system was considered to have been of mixed construction with RC shear walls perpendicular to the street (plate 4.12).

4.4.3.2 7-storey building

A typical failure of a building with continuous fenestration is provided by the building shown in plate 4.13. This was a strong beam/very weak column structure which had collapsed in the 2nd storey.

4.4.3.3 9-storey Santica Building

This pre-1981 building had three bays over the lower two storeys with the columns exposed at ground floor level (plate 4.14). Above this the central columns continued but the end columns were replaced or supplemented by external shear walls. The structure had failed in the 5th storey and partially in the 4th storey with the upper part of the building fallen sideways.

4.4.3.4 Irregular office building of 5/6 storeys

In a highly irregular five to six storey structure with continuous fenestration it appeared that an oblique corner shear wall had sheared in the lowest storey, followed by the collapse of that storey and the storey above (plate 4.15). The building crushed several cars that were in the ground floor garage. The building had shear walls on opposite sides but there was no obvious signs of bracing in the longitudinal direction. A soft storey failure occurred at ground floor level probably due to the lack of stiffness in that direction. The most heavily damaged beams were in the bay adjacent to the shear wall, reflecting the damage pattern noted in Section 6.3.3. of Reference 4.1. The internal structure in this building was obscure.

4.4.3.5 5-storey commercial building

In this five storey building with a flexible but strong ground floor, failure had occurred in the first storey, where the building fell through the height of the windows, causing impact between the RC fascia beams of the first and second storeys, which suffered extensive vertical cracking, but did not disintegrate (plate 4.16). At ground floor level the shop fronts were damaged, but not the structure. The damage may be attributable to the change in shape of some of the columns which were circular in at ground floor level, but square above.

4.4.3.6 6-storey wide building

A six storey eight bay office building suffered spectacular collapse in the 2nd and 3rd storeys (plate 4.17).

4.4.4 Partial storey collapses in framed buildings

4.4.4.1 School building, Kobe

Partial storey collapses are very interesting because the question arises how the part of the building remaining intact had the capacity to resist the inevitable sway forces imposed on it as the attached part subsided. A spectacular instance of this is in a 5-storey 8-bay RC school building in which a ground crack, about 0.5m wide with an 0.5m step (Plate 4.18), pulled the foundation and ground floor beam apart by 200mm (plate 4.18). This caused an unusual collapse mechanism in which the end two bays above the ground floor lurched towards the remainder of the building causing failure of the columns. Plate 4.19 shows a close up of the back of the building. The three end bays of the buildings subsequently collapsed. This is the type of damage that foundation tie beams are provided to prevent.

This is effectively a single storey failure in which the sway moments failed the beam at the first floor level in an isolated bay inducing the failure of the beams above it. This structure therefore represents the transition between strong beam/weak column and weak beam/strong column behaviour. The change of direction of the failure plane occurred in a bay with no obvious distinguishing features, though there could have been an internal stairwell. The reinforced concrete design in this building represents the transition between ductile and non-ductile construction. It has the benefits of numbers of small bars with transverse reinforcement at a spacing less than that evident in most failed structures, possibly 150mm, and possibly conforming to the requirements for the intermediate ductility class in Eurocode 8.

Failure was the result of:

- the crack in the ground,
- liquefaction of the soil surrounding the building, which may have caused foundation damage and contributed to the damage in the fascia beam at ground floor level,
- lack of a tie beam, the effect of which is further discussed in 4.14, and
- the depth of the fascia beams constructed monolithic with the columns, which rendered the columns too stiff.

4.4.4.2 Old ten-storey building

In an old ten storey apartment building with external air conditioning units (unusual in Kobe), failure had occurred over part of the second storey as shown in plate 4.20.

The failure appears to have initiated one end (see 4.16.1), proceeded to the centre and propagated upwards, shearing the deep and slender masonry faced fascia units:

It is possible that the failure had been initiated by impact against the neighbouring building and the orthogonal motion had thrown the upper storeys streetwise by 500mm or more, with a shear failure mechanism characteristic of RC construction with inadequate shear reinforcement.

4.4.4.3 Mixed-usage building

Another partial storey collapse occurred in a long ten storey 3 bay building with apartments above three floors of office accommodation (shown in Plate 4.21). The failure was initiated by the out-of-plane failure of a three storey high external shear wall at the end of the facade (constituting a severe vertical irregularity), which had probably failed in-plane having first attracted more load than it could carry. This failure was unlike the previous two described in that it had occurred progressively over three bays. In the first bay adjacent to the uncollapsed part all seven fascia beams to the top of the building had suffered extensive plastic rotation, but had not failed. In the second bay the plastic rotation was confined to the two beams above the collapsed floor and in the third bay to just one beam above it. In these bays the beams in two/three storeys above were damaged, though there was no obvious rotation.

It appeared a three storey shear wall had prevented the collapse extending over the full length of the building by redirecting the failure plane upwards.

The AIJ[4.15] report indicates that the lower floors in this building are of SRC construction, though the failed 3rd storey columns were clearly of RC. They attributed failure in part to the U shape of the building and in part to the variation in column flexibility with height.

There was no evidence of transverse reinforcement in the failed columns.

4.4.5 Dual and mixed frame/shear wall structures

4.4.5.1 9-storey framed building with end shear walls behind the German Consulate

The unusual features of this mixed frame/shear wall structure, which may have had an SRC frame, are a 2-storey deep virtually windowless wall all around the top and on two opposite faces there were walls over the end bays above the ground floor. The main damage was:

- loss of cladding on the ground floor columns, which were otherwise undamaged
- cracking in the beam under the side walls, with loss of cladding locally
- spectacular delamination of the masonry at the corner of the walls, yet little masonry had fallen
- horizontal cracking and cracking at 30° to the horizontal through another side wall.

Plates 4.22 and 4.23 illustrate mixed frame/shear wall construction an example which shows the incompatibility between the two even when the frame is strong. The concrete-backed masonry veneer is delaminating at the corners revealing closely spaced column reinforcement (Plate 4.22) (at 75-100mm centres) and elsewhere the shear wall (Plate 4.23). The delamination is attributable to out-of-plane deformation of the shear wall due to the absence of orthogonal shear walls. Not only is it extremely hazardous to pedestrians, in loosing the cover concrete the effective areas of the shear walls and hence their strength have been significantly reduced. There was no damage in the ground floor soft storey other than the loss of cladding.

4.4.5.2 8-storey Building Near Car Park

In this classical failure of the ground floor storey of a framed building with end shear walls and with the frames resisting orthogonal motion the shear walls had failed out of plane, with the building moving both sideways and towards the street (plate 4.24). In this case the direction of failure appeared to have been influenced by buffeting from the neighbouring building as there was impact damage at first floor level (far right of plate). Buildings without longitudinal shear walls are especially vulnerable to such damage in end shear walls.

4.4.5.3 11-storey shear wall apartment building

Along the coastal highway in Kobe was a pair of relatively new blocks of apartments (Plate 4.25) considered to have been constructed in the mid 1980s, which had suffered a 200mm permanent relative displacement in the North-South direction (perpendicular to the coastal highway). This displacement (plate 4.26) was attributable to the difference in the vibrational characteristics, due to the difference in height and type of construction.

The construction of the taller building comprised the typical strong beam/weak column frames on the long facades, with most of the shear in that direction carried by shear walls either side of the central corridor. The transverse structure comprised mixed construction with coupled shear walls and some frames. The frames were on the eastern half. The coupled shear wall at the east end had suffered damage on the inside face of the return wall, above and below the stocky coupling beams over the entire height of the building, exposing the reinforcement. This damage was most severe at the second and third floors from the top of the building (plate 4.27). More severe damage, although only at the second floor, was a tension/compression failure on the Northern extremity of the wall which revealed inadequate (for earthquake prone regions) vertical and confining reinforcement, with the vertical reinforcement buckled (plate 4.28). The most interesting aspect of the damage is that the failure zone had propagated horizontally over two-thirds of the width of the wall in the form of 2-way diagonal cracks, and at the middle third position the damage was as severe as at the end. Such damage can be attributed to the combined effects of shear and flexure.

4.4.6 Perforated shear walls

4.4.6.1 The red building

In the most heavily damaged area of Central Kobe a substantial perforated shear wall building with small well separated windows received circular strike marks as the neighbouring building rotated and fell across the street (plate 4.29).

4.4.6.2 7-storey Dunlop Building

The older of the two Dunlop buildings in Kobe (Dunlop is now owned by the Sumitomo Bank) had a substantial walled ground floor storey, above which were three storeys with perimeter RC shear walls perforated by small windows. These were surmounted by three storeys of less rigid construction (plate 4.30). Severe X-cracking had occurred in the perforated shear wall. Vertical cracking had occurred below the end windows, protecting the outer wall section, and down the centre of the building. At the centre some of the concrete had disintegrated and fallen away, leaving the reinforcement intact.

4.4.6.3 Tilted building with one-sided collapse

This 6-7 storey infilled framed perforated shear wall building on a corner site had a soft storey at ground floor level and the frames above on 3½ sides infilled with masonry. The ground floor soft storey suffered complete collapse one side and a ¼ storey collapse on the other; collapsing into the side street side (plate 4.31). The superstructure underwent rigid body rotation and at the time of the visit was the second most heavily tilted building in Kobe, though others, like the most tilted discussed in 4.5.6 with less robust superstructures and in danger of further disintegration, had been removed by the time of the visit. There was clear evidence of impact damage, which would have contributed to the collapse.

4.4.6.4 14-storey shear wall structure

In the shear wall one end of a block of flats probably built about 1980 the shear wall had failed in compression one side at first floor level and a crack propagated diagonally upwards, through small perforations. Where it reached the central perforation it propagated vertically up the axis of the wall. The damage was attributable to lack of confining reinforcement (plate 4.32).

4.4.6.5 5- storey totally collapsed building

A masonry-clad mixed frame/perforated shear wall structure suffered a specular collapse, whereby each successive storey was thrown in front of the storey below when the column failed at the top of the ground floor storey. All the beam/column connections above also failed and the walls on each floor failed out of plane (plate 4.33).

4.4.6.6 2-storey building with separated shear walls

A 2-storey building had collapsed on separation of orthogonal shear walls (plate 4.34).

4.4.7 Irregular buildings

4.4.7.1 The Sanwa Bank

This 10-storey irregular building of mixed construction around a 12-storey tower was considered to have been constructed between 1960 and 1980, with the tower a more recent addition (plate 4.35). The main external damage was on the rear elevation where lightly reinforced connecting beams between two halves of a shear wall had been shattered right through without the normal X-cracking (plate 4.36). The more substantial and longer beam at 1st floor level was sagging, which is likely to have aggravated the damage above.

Internally all the shear walls examined were finely crazed with 2-way diagonal cracking at the pitch of the main reinforcement, and two walls examined had shattered along isolated cracks (plate 4.37). Exposed electrical ducts were found to contain live wires (plate 4.38).

4.4.7.2 The German Consulate

The German Consulate, built in 1955, is an 8-storey block with a 11-storey tower one side running the full length, with a lower 5-storey structure on the East (plate 4.39). It is predominantly a shear wall structure.

The only collapse was in the seventh floor columns of the eight storey block, but it affected the tower (see below). This appeared to have occurred progressively from the west end, with the total destruction of the columns over their height between the fascia beams. The seventh floor on this elevation constituted a weak storey as the storey height was greater here than in the two floors below.

The tower which on the elevation comprised coupled shear walls with an unusually long and deep coupling beam at each floor level had survived above the level of the 8-storey structure. However the ends of the coupling beams had failed at the two levels below the roof level of the lower structure, presumably as a result of frame/shear wall interaction. At the opposite end of the tower there was a uniform shear wall perforated only by small windows. Here the tower section could be seen to be breaking away from the lower part of the building on a diagonal shear zone (plate 4.40). The wall was clad with square precast concrete panels which were well tied to the insitu concrete behind, nevertheless many of the panels detached but were prevented from falling by tie bars (plate 4.41).

Around the stairwell by the entrance the usual orthogonal reinforcing mesh in the wall was supplemented by a one-way system of diagonal bars, which had performed well. Cracks nevertheless developed parallel to these bars and a 300mm square of wall concrete was missing (plate 4.42). Higher up a landing beam had punched through the stair well wall where the beam was offset from the wall behind (plate 4.43). Elsewhere in the lift shaft one of the walls had failed out of plane and had slumped inwards, but had been retained by the reinforcement. That this could happen in what was effectively the outstand in a 3-sided shaft, which had undoubtedly been designed to carry lateral loading, provides a warning to engineers on the care needed in the design of shear walls in lift shafts and on the need for out-of-plane restraint where there are no transverse walls near the tip of the outstand.

The large and modern-looking auditorium on the North East side, which was separated from the main building by an expansion joint, had performed well, though a small part of the roof had fallen through the ceiling. Spalled concrete and debris indicated that severe impact had occurred between the two parts of the building.

4.4.7.3 5-storey heavily tapered building

This RC building had suffered shear failure in the shear wall at the narrower end in and just above the ground floor storey, presumably due to the shear wall being stiffer and weaker than the rest of the vertical structure (plate 4.44).

4.4.7.4 No 69

This building with arched lintels on a corner site by an alley had a rigid superstructure and a ground floor soft storey. The isolated column on the more flexible corner had suffered spalling damage on opposite faces top and bottom, where rotation damage had occurred (plate 4.45). The damage could be attributed to asymmetry, the large opening in the lowest storey and buffeting from its neighbour, which undoubtedly influenced the direction of the drift, which was into the alley. The damage at the top was on a 45° plane, suggesting the influence of shear, but the most damage was associated with buckling of the longitudinal bars where the link spacing had been locally increased from a 75mm to 200mm. The bars between the beam and column were not visible and the anchorage to the top beam bars at least is considered to have been effective.

4.4.7.5 4-storey building on Flower Street

The "Kwong on Tai Hong" building on the west side of Flower Street was supported on three columns on the street side, with the middle column at the middle third position (plates 4.46). The middle column being of larger cross section that the others had suffered a spectacular shear failure, with the springing apart of the widely spaced links over a distance of 1.5m (plate 4.47). Over this length the core concrete had split into three unreinforced boulders such that they carried no load. The buckled longitudinal bars suggested the column had shortened by 75 - 100mm, with the load presumably having been redistributed by the hidden fascia beam to the edge columns.

4.4.7.6 Irregular 7-storey building

This building with windows confined to one half of the facade comprised an haphazard mixture of RC shear walls, columns, thin fin like masonry walls, a soft ground floor storey, a substantial corner column supporting the shear walls on the shorter side, and an extended wall down the longer side of the tapering corner site, supporting balconies (plate 4.48). The joint at the top of the ground floor corner column had exploded, presumably under the combined effects of torsion, vertical acceleration, and impact from its neighbour, revealing a few widely spaced links of very small diameter and an absence of horizontal bars tying the columns to the beams (plate 4.49). The building had sheared right through on the first and second storeys and had dropped slightly on the face opposite the long wall, where it had failed out-of-plane.

4.4.7.7 4/6-storey older building

This building with 4 storeys on the street and 6 behind had suffered severe cracking at ground floor level on the street side, partial collapse on the first storey at the rear, but there was no damage to the central columns. In the front was a cantilevered RC facade in which there were vertical splits at the ends of the beams (plate 4.50).

4.4.7.8 15-storey flats, north of Takenaka Complex

This very large block of flats considered to have been built in it 1980s had a configuration between that of an L and a T (plate 4.51). The stalk was 120m long, yet it had no movement joint. Damage took the form of severe X-cracking between the doors and the windows over the middle section of the building, over at least the lowest five floors.

4.4.7.9 15-storey flats

This large and quite new block of flats (plate 4.52) was divided by a wide seismic joint as shown in Figure 4.4 such that one part formed a U-section. To prevent in-plane distortion of the base of the U, which comprised only a corridor, four seismic props had been provided on the elevation (plate 4.53). Damage to the side walls had occurred in the form of horizontal cracking in the longer walls at 1st floor level and in the 5th storey (plate 4.52) and X-cracking in the walls between windows elsewhere at intermediate levels. The parallel shear walls on the inside of the U had suffered severe X-cracking between the windows over the 5th to 7th storeys (plate 4.54) and had sheared right through on a horizontal plane at the 8th. It is considered that damage (confined to the weaker half of the building) was caused by impact across the seismic joint, which was originally about 50mm wide but was between 100 and 120mm wide above the damaged part of the wall.

The spiral staircase supported vertically by a cantilevered spine wall had broken free from beams across the stairwell, provided presumably to provide the staircase with lateral stability (plate 4.55).

The reinforcement tying them together however was still intact, providing adequate restraint during the earthquake.

4.4.7.10 4-storey apartment building with open ground floor plan

This 1980's building with an open car park at ground floor level was supported by two rows of seven columns (plates 4.56 and 4.57). Masonry cladding on the storey above effectively rendered the superstructure rigid. Lateral stability to the ground floor storey was provided by a shear wall on one side (plate 4.58) and a car park ramp on the other (plate 4.57). The longitudinal movement had fractured the shear wall, which was only nominally reinforced. It also snapped the 6mm bars in the ramp, the only bars found in Kobe which had failed in tension (plate 4.61), other than at butt welds (see chapter 6). Rocking about the minor axis is considered to have contributed to the damage, as discussed in section 4.17.

The tops of all the columns in this storey were damaged, with the damage increasing in severity from the ramp end, where the longitudinal movement was about 40mm. The centre part away from that end of the building had dropped (plates 4.58 and 4.59). The links in the columns were 10mm high yield bars at 75mm pitch with mid-side restraining links in opposite directions in alternate layers. Failure of the columns had occurred due to failure of the small fillet welds at the side laps of the links, just visible where the links had sprung (plate 4.60).

4.4.7.11 Building behind Sanwa Bank

A joint in this building with an irregular frame had suffered classical shear failure and lateral distortion where the beam had punched into the column, buckling the longitudinal bars (plate 4.62). The long fracture zone at the top of the joint suggests the top beam bar, which was not visible, had pulled out.

4.4.7.12 Side street building

In this small 7-storey side street building, with irregular internal layout on a corner site, a perimeter column at cill level had been severely overloaded as a result of load redistribution in the superstructure, possibly associated with foundation movement. The rectangular column displayed classical behaviour with ovulation of the cross section, as anticipated by the design model in Eurocode 8[4.16] used for determining the links (plate 4.63 and 4.64). The links, at 75mm spacing, performed by pulling in the corner bars, whilst allowing the side bars to extend. The performance of this detail demonstrates the need for bars through the centre of rectangular sections to prevent this otherwise inevitable change in geometry. Plates 4.63 and plate 4.64 are the same view. Plate 4.63 is a ½ second exposure and Plate 4.64 is a flash photo. These demonstrate how the apparent damage is minimised when the flash gun is mounted on the camera.

4.5 Performance of Steel Buildings

4.5.1 General

Most of the non-industrial buildings examined except multi-storey car parks (see 4.9) were considered generally to be older than the majority of the reinforced concrete buildings reviewed in the previous section. It was noted however that there was a considerable number of steel framed buildings, of roughly the same age as the reinforced concrete buildings, on which the cladding had suffered serious damage. It is likely that these latter buildings were mostly sway frames (see 4.3.10) and in general somewhat more flexible than the reinforced concrete buildings, and too flexible for the cladding. As this form of damage is closely related to the type of fixings used or to damaged structural connections, and details of either feature may not be easy to determine, a very limited sample of this damage is included here. The performance of other, predominantly new, steel buildings is considered in 4.8 and 4.9.

4.5.2 Cladding failures on steel buildings

Typical of many buildings is a 7-storey block of flats along the Nishinomiya Road in which the precast concrete fascia panels (effectively planks) span vertically between beams. At the 2nd and 3rd storey one side, the end planks were missing (plate 4.65) revealing the usual bright red moment resisting frame, which was undamaged and not obviously distorted. The corner fixings on a number of panels still in place were badly damaged. The splices in the beams were at the eighth span position, indicating that the beam/column connections were factory assembled. The column splices were not visible.

Sway frames employing the minor axis flexural resistance of the columns by use of substantial knee braces (a device used to overcome the problem of accommodating windows in frames with CBF bracing) performed well structurally but tended to be too flexible for the cladding. In one building of this type all the cladding had fallen off the frames so braced (plate 4.66). Flexible angle cladding rails (rather than Ts) hung from the third points had swayed in the plane of the facade where they were welded to windows, contributing to the failure of cladding panels.

4.5.3 3-storey single bay frame

A building in an area devastated by fire had survived having shed its cladding and its floors. An exaggerated sway mechanism had developed with hinges either end of the drop-in spans, which were supported only by web connections (plate 4.67).

4.5.4 Old 7-storey braced building

In an old building with steel I sections for beams and columns, light strap bracing had been introduced in the bays transverse to the street, which was the direction of minor axis column bending. The bracing had snapped, buckled and otherwise distorted at all levels and the cumulative effect of the storey sway deflections at the top of the building was remarkable (plate 4.68). The large joint eccentricities of the gusset plates connecting the bracing to the columns had contributed to the failure of bolted column/beam connections, permitting large joint rotations (plate 4.69). Orthogonally the building was unbraced and in that direction the building had performed well. The buildings on either side however would have deterred along street collapse, at least in the lower stories.

4.5.5 Old 7-storey unbraced building

In an old building with similar member sizes to that in 4.5.4, but with welded and plated connections, and with the columns turned round and without bracing, one side of the building had drifted streetwards by 1.5 metres, and along the street by 0.4 metres towards an alley. Most of the movement was attributable to buckling of the ground floor column (plate 4.70). The curious positioning of an external staircase along most of the facade had resulted in partially braced frames along the street and large joint eccentricities about the minor column axis. The building was well separated from its neighbour so buffeting did not contribute to the damage.

4.5.6 6-storey apartment block

This 6-storey apartment block on a corner site in central Kobe (plate 4.71) had remarkably slender columns and very little structural bracing. However, the collapse occurred because the holding-down bolts at the base of two of the columns had failed. The bases of the columns had been encased in concrete, but when the building had toppled over the column had been pulled out of the concrete exposing the base-plate to foundation connection detail.

There were only two M16 bolt holes in each column base plate and the bolts appeared to have failed in tension, having bent the base plate (just visible in the detail) and elongated the holes in it.

4.5.7 8-storey U-shaped building

This building of narrow frontage compared to its depth behind the street, was unusual in that it lacked internal columns and moreover there were no columns on the front facade (plates 4.72 and 4.73). The back wall was of reinforced concrete construction with two vertical rows of small windows. On the sides were braced frames which appeared to comprise I-section columns (connected by bolted splice plates just above floor level), light lattice beams and bracing (either of rectangular sections or I section) with its minor axis perpendicular to the frame. A remarkable form of in-plane deformation had occurred, with in-plane buckling of the bracing taking place well away from the joints in the four storeys which had lost their cladding (plate 4.73). In the lowest of the storeys, where storey height was appreciably greater, the braces had failed at the central gusset. The opposite face had suffered little distortion, with only one cladding sheet badly damaged (plate 4.74). The building was well separated from its neighbour so buffeting had not occurred.

The difference in the distortions on the opposite faces, taking into account the similar fundamental frequencies in the two principal directions and in torsion, together with the fact that the maximum excitation in the two directions occurred over the same 1-2 second period, is significant. It raises the possibility that the maximum displacements in the two direction occurred simultaneously, with one face suffering enhanced displacements due to movement in the orthogonal direction, whilst the opposite face remained substantially stationary (Figure 4.5). Furthermore this situation could have existed for a substantial portion of the period of major shaking. It is felt the orthogonal motion combination rules in codes may not take adequate account of this situation.

4.5.8 Older framed building

An old style building with deep beams and columns fabricated from I and T sections had failed at the beam to column welds (plate 4.75). There was no evidence of bracing.

4.5.9 Takenaka Complex (Ashiya-hama Seaside Town)

By far the most extensive residential development in the Ashiya-hama seaside town, built by Takenaka in the early 1980s, comprises a series of blocks with individual towers up to 24 storeys (plate 4.76). The structural system employed the popular Japanese steel "mega frame" system for the most part, which comprised braced vertical girders outside the partially open stair wells, joined by storey height horizontal braced girders at every fifth floor, which were exposed (plate 4.77). Transverse bracing was contained within the walls of the staircases except at the open floors, where it was extended to the exterior. Precast concrete accommodation units were stacked, four units deep, on the transverse girders.

The vertical girders had been fabricated as 2 and 3 storey height units, with in-situ butt welds in the fabricated box columns. There were bolted splices in the bracing at either end of an infill section, introduced no doubt to accommodate tolerances. At the bottom of the building there were 4 x 6 bolt flange connections, but these reduced with height to 4 x 4 bolt and 4 x 2 bolt connections.

Most of the columns in the most westerly block of the development had fractured, either at the lowest storey (plate 4.78) or in the third or fourth storey. Most of the failures were at column welds though some failures occurred through the parent metal in heat affected zones and a number of failures occurred at mid height between floors. A few of the diagonal bracing members had also fractured and the AIJ[(4.15)] noted that some had buckled. Nowhere had they found any plastic deformation though some paint had flaked off, particularly from the diagonals. The column fractures, up to 10mm wide, had been welded together and tapered fins had been added to transmit the load past the damaged sections.

It has been suggested that this was the only block running exactly North-South and may have experienced larger in-plane ground accelerations than the other blocks as the accelerations were somewhat larger in that direction (see 4.1); nevertheless some of the other blocks would appear to have sustained similar, though less extensive, damage.[(4.15)] Other possible explanations are variation in the ground conditions or local variation in the excitation.
It is noted that the AIJ Report attributes the differing amount of damage in similar high rise apartments (of unstated construction) in Eastern Higashi-nada to likely but unobserved variation in the site conditions. See also 4.11.

Elsewhere, damage noted was spalling in the landing concrete (revealing it to be of lightweight concrete), cracking in the concrete surrounding the columns at ground level and impact damage on the sides of the precast accommodation units (plate 4.79), mostly where the balcony parapets met the side walls.

Around one building circumferential steps had occurred in the ground indicating general settlement of the surrounding ground relative to the building, which was undoubtedly piled (plate 4.80).

4.5.10 7-storey single bay building

This narrow single bay building (plate 4.81) had tilted sideways, revealing on one side a RHS column with the base plate mounted on a plinth without any holding down bolts (plate 4.82). The plinth concrete had crushed.

4.5.11 Modern 5-storey block

This was a relatively modern 5-storey apartment block with cold formed tubular steel columns and I beams (plate 4.83), typical of modern steel buildings in Japan. There were plates through the columns at the levels of both beam flanges and there were splices in the beams at the quarter span positions, with welded flanges and bolted webs. One of the mid side columns had failed through the lower weld at first floor level and the column to beam connections had also failed, although on one side the top flange weld and the upper part of the web to column weld were still intact (plate 4.84). At the outer column the beam to column weld had torn and the column had moved out by 40mm and was inclined (plate 4.85). A recent paper by Kuwamura and Akiyama [(4.17)] indicates the Japanese had become concerned by the lack of ductility in such construction before Kobe and their studies indicated failure commences in the heat affected zone in the corners of the cold formed section and the crack propagates across into the weld. The use of hot rolled sections, as manufactured by British Steel, avoids this type of failure.

4.5.12 The New City Hall

The newer part of the City Hall (see 4.6.2) shown in Plate 4.86 is a 32 storey building rising 132m above street level, with three basements below ground. It is of RC construction over the lowest two storeys and of structural steelwork above, in the form of steel plate shear walls with interconnecting steel plate girders in the 26th storey. The lowest three storeys of the steelwork are concrete encased to form a transition zone of SRC. The lower part of the building was undamaged but the shear walls had buckled by up to 20mm just below the plate girder and the building had moved 200mm northwards[(4.18)].

4.6 Performance of buildings of SRC and heterogeneous construction

4.6.1 General

As a rule where there are steel beams and the columns appear to be of concrete the columns are in fact of SRC construction. Some buildings are of SRC construction in the lower levels and become steel construction above, and in some the upper part is of reinforced concrete construction. No details of the connection between the SRC and RC column sections have been encountered and, noting the uniformity in the construction of some of the buildings considered in 4.5, it may be that the main reinforcement is not increased at the transition between the two forms of construction. It is understood that for a period between 1950 and 1981 (the exact period is not known) SRC was used for the lowest 20m (or 5 to 7 storeys) of 30m tall buildings. In general however materials in buildings are more mixed than in Europe and America, possibility on account of the code design criteria discussed in (5)B(i) of section 4.2, which introduces the possibility of damage between the two forms of construction[4.1 and 4.19].
The effects of this are most pronounced where the buildings are also irregular. However in the two documented cases of mid height failure in SRC buildings encountered (see 4.6.4 and 4.13) the collapsed floor did not occur at the level of the change in construction.

4.6.2 The Old Town Hall

The City Hall or "Municipal Buildings" comprises two interconnected blocks, the taller (see 4.5.12) built in the 1980s and an older one, the former City Hall (plate 4.87) of 8 storeys, built in 1957. The older building was of SRC in the form of embedded steel lattice column construction over the first four storeys and of reinforced concrete construction above. The older block (plate 4.87) however had suffered soft storey failure of the 6th storey and severe structural damage in the 5th. Three bridges connected the two blocks, at the 3rd, 5th and 8th storeys. The 8th had collapsed on the 5th, severely damaging it (see plate 4.88 which was taken after a removal of the fallen bridge). The differential movement of the two buildings put the connecting bridges under extreme loading and ultimately caused the failure in the old City Hall due to the increased horizontal load applied on the 8th floor. It is likely however that the building had been underdesigned in the 5th and 6th storeys[4.20]. The clock on the 7th storey records the time of the earthquake having stopped presumably due to the severing of the electrical supply (plate 4.87).

4.6.3 8-storey building

This predominantly 8-storey building was of heterogeneous construction, with RC and SRC construction in the lower 4 floors. Steel brackets supported a covered walkway along the facade at 2nd floor level, the superstructure of which had collapsed under impact from the adjoining pedestrian bridge (plate 4.89). The columns above 4th floor level were of tubular steel construction, the front row of which were supported by RC beams. Above this level the facade was supported by cantilevers. The change in stiffness occurring at the change in construction had caused collapse of the RC facade panels in the fifth and sixth storeys.

4.6.4 Irregular 8-storey building

This predominantly 6-storey building with a two storey penthouse at one end and with a heavily bevelled corner (plate 4.90) had suffered complete collapse on one side of the fourth storey and partial collapse on the other. On the partially collapsed side remains of the columns were visible and of particular interest was a beam which had fractured at the face of the column. This raises the question of whether similar damage had occurred at the collapsed end of the floor above. The staircase was trimmed by steel members, with steel stringers. The staircase was supported on structural steelwork and there was a considerable amount of mangled steel in the vicinity of the stairs. Nevertheless it is likely it was predominantly a building of RC construction.

4.6.5 The Matsushita building

This long ten storey building, constructed before 1981, extended the full length of Kobe's main square and had lost most of its glass (plate 4.91). On the top is a rigid off-centre penthouse, two thirds the height of the building, which caused most of the damage to be at one end of the building. Due to its obvious importance it is expected that this building was of SRC construction.

4.6.6 7-storey SRC building

In this building the SRC columns with thin base plates, only slightly larger than the RHS core, had jumped out of their plinths. There were only two small holding down bolts, one at the centre of two opposite sides of each column. Plate 4.92 shows a column that moved off the plinth and came to rest over 0.5m from its original position. A column nearby had fractured its holding down bolts and pulled up through the concrete plinth enclosing the base of the column (plate 4.93). The overall overturning moment on the building may have contributed to this failure.

4.6.7 3-storey building on soft ground

This recently completed building on soft ground near the docks, with much of the ground floor occupied by an open car park and undoubtedly piled, had been severely shaken and the foundations of a light external staircase had failed. The columns and external beams were of SRC construction. The feet of the corner and adjacent column had moved outwards, suggesting the absence of tie beams between pile caps (plate 4.94). The rigid body rotation of the column had cracked the first floor perimeter beam and had caused failure in the column to internal steel beam connection (at the eighth span position) at the next column, where the movement was greatest. The web plates had sheared between the bolts and the lower flanges had sprung apart (plate 4.95).

4.7 Performance of domestic houses

Of the approximately 800,000 buildings that collapsed during this earthquake, 70-80% of these were non-engineered residential buildings. It had also been established that of the 5,000 casualties more than 50% were caused by the collapse of timber framed buildings (plates 4.96 and 4.97). Of those left standing many were severely damaged and most had some damage to the stucco (Plate 4.98). The types of timber buildings that performed least well were those with traditional heavy ceramic roof tiles, where the tiles are fastened in place using cementitious soils. These roofs are popular in Japan because of their inherent wind resistance against typhoons and for their heat-insulation properties. Unfortunately, their large mass generates large inertial forces during an earthquake, which can cause timber framed buildings to fail. In plate 4.100 typical ceramic tiled roofs have caused extensive cracking of the mortar joints in a masonry house. Several timber frame houses collapsed completely; in many cases the walls collapsed simultaneously, leaving the roof undeformed. Such failure modes are indicative of minimal lateral resistance (plate 4.99), reminiscent of the damage in the Manjil Iran earthquake [4.41]. Housing with light roofing systems such as light metal sheets or asphalt tiles generally performed much better during this earthquake. Recently methods of constructing lightweight housing, similar to those used in California, have been introduced in Kobe and buildings constructed in this manner performed significantly better than those constructed using traditional Japanese methods.

The AIJ Report[4.15] attributes the high proportion of timber houses near rivers which were severely damaged to the local ground motion. The peak response would have been at a period of about 1 second close to the fundamental period of timber houses.

Older steel framed houses had light columns, truss floors and wire bracing in very narrow bays with aspect ratios typically of 3 to 1. The bracing failed throughout (plate 4.101).

Many timber houses suffered soft-storey failures where the ground floor of the house had been opened to form space for a garage. The form of construction was such that there was very little bracing in the structure, with many of the connections only lightly pinned with large tolerance connections, which were prone to fracture. Some more recent steel frame houses also suffered soft storey failures where the ground floor accommodated a garage (plate 4.102).

In some areas of Kobe prefabricated buildings were used for housing and light commercial buildings (plate 4.103). The bracing that provided lateral stability was provided by tie bars. Often these were inadequately connected to the columns and became detached during the earthquake as in plates 4.101 and 4.103, in which the buildings show significant lateral sway. Other similar buildings collapsed completely. In previous Japanese earthquakes these buildings had performed adequately despite some snapped bracing members, due to the high degree of redundancy in the bracing[4.25]. However, this provided insufficient protection in this earthquake.

There was no damage whatsoever in this three storey domestic house with an open ground floor, forming a car park (plate 4.104). The structure at ground floor comprised a moment resisting frame of rectangular steel columns with a shallow stiffened girder supporting the upper floors.

A similar building in the Loma Prieta earthquake[4.19] in a suburb of heavily damaged domestic structures in San Francisco had survived undamaged, confirming the robustness of this solution to the problem of accommodating cars.

4.8 Performance of industrial buildings

4.8.1 General

Being an important industrial region and a port, many large industrial structures were affected by the earthquake and these form a structural class on their own. They include heavy warehouse construction, low rise heavily built buildings capable of supporting light to medium industrial processes on all floors and others purpose built around specific activities. They inevitably converge on the more extensive type of low rise building. Another warehouse affected by buffeting is considered in 4.16.3.

4.8.2 Irregular 5-storey building

This irregular massively built RC building with a seemingly random arrangement of wall perforations, some very large, had failed on an erratic path, connecting the perforations at a level roughly at mid height (plate 4.105). The damage is likely to have commenced at the lower of three storey height penetrations near the centre of the building.

4.8.3 4-storey brewery

A building in the dockland area, undoubtedly piled, rather like that in 4.8.2 but with smaller windows, and with a soft storey one side of a movement joint, had survived undamaged except for a crumple section between the two halves of the building on the remote side from the soft storey (plate 4.106). This side had been sufficiently strong to resist the entire horizontal force parallel to the plane of the wall and sufficiently stiff to prevent impact across the joint, which was about 50mm wide. The movement of about half the joint width had resulted in the disintegration of the crumple section. The substantial columns forming the soft storey provide a reminder that there is no reason why columns in soft storeys need not be sufficiently strong to resist severe earthquakes.

4.8.4 Clerestory window building

This building on the waterfront and undoubtedly a piled structure on soft ground, contravenes the design principles in Eurocode 8 in having clerestory windows at all levels. It had survived undamaged (plate 4.107).

4.8.5 Older 3-storey building

This long low industrial building failed in the lowest storey at one end and progressive collapse had occurred in eight of the eleven bays (plate 4.108).

4.8.6 Older 4-storey building

This older building with rigid RC walls and small windows with flats at the higher level would have been expected to survive most earthquakes (plate 4.109). The building failed through the windows at the first floor level where one window was twice as long as at the level above. It had also failed in compression at the other end of the building. Differential foundation conditions associated with a sloping site may have caused the damage. The RC chimney on the roof had been badly damaged.

4.8.7 4-storey precast concrete building

This four storey building with 3 storeys of offices above ground and one storey in a well, had suffered collapse of the second storey and partial collapse in lowest storey (plate 4.110). The collapse appeared to have been promoted by the hard point where a slab in front of the main entrance bridged the well.

4.8.8 Irregular building with domed roof

Another building with a cantilevered extension in the front had fared less well. Though the cantilevers had held, the first storey columns in the cantilevered section had collapsed along the facade (plate 4.111). The construction in this one storey extension appeared very frail.

4.8.9 Garage workshop

A well designed holding down bolt arrangement, with provision to resist uplift, had failed due to crushing of the plinth on which it was founded (plate 4.112). The failure was mainly on one side, where the shafts of the holding down bolts were exposed.

4.8.10 Steel portal shed

The middle of the long side of a steel portal shed near the coastal highway moved towards the sea, which could be attributed to inadequate foundations generally and the absence of mid side tie beams (plate 4.113).

4.8.11 Traditional Japanese shed

A traditional Japanese portal of bamboo construction with substantial knee bracing, but with exceedingly thin columns and braced only at one end, had survived intact (plate 4.114). The flimsiness of the construction was clearly matched by its lack of weight.

4.8.12 3-storey warehouse

A large 3-storey warehouse in the port area with few windows but with large sliding doors at all levels, leading onto deep cantilevered balconies, had survived undamaged, but the ground in front had subsided (Plate 4.115).

4.8.13 Irregular modern building

A good example demonstrating that highly irregular buildings can be designed to resist strong ground shaking is provided by this heavily braced steel framed building with a two storey projection one end, carried on slender circular columns (plate 4.116). It would appear that, despite the appearance of the floor, the projection continues into the main building.

4.8.14 Japanese framed-wall building

An exceptionally robust form of Japanese construction is a frame in which the individual bays are completely infilled by reinforced concrete panels [4.39]. It is a superior variation on the confined masonry concept in Eurocode 8[4.16], in which strong bands of masonry act as internal ties and provide boundary confinement. Only one example of this type of industrial construction was encountered (plate 4.117) and it had survived undamaged, despite a soft storey one side and ground subsidence around the piles at the rear, exposing the top of the piles. The piles were clean and appeared undamaged. Buildings of this form of construction are difficult to identify as the wall parts may, as here, be covered by sheeting by way of decoration.

A remarkable feature of the design was that the columns reduced in size in every storey.

4.9 Performance of multi-storey car parks

4.9.1 General

Multi-storey car parks offer the designer the opportunity to develop, largely unhindered by architectural niceties and service considerations, structural systems to best suit the environmental conditions. In particular more flexible construction is admissible than for clad buildings. In Kobe car parks are of steel, SRC and RC. SRC is identified mainly by whether or not there is concrete encased bracing, as no car parks visited were sway frames.

4.9.2 11-storey car park by ferry

By far the largest car park in Kobe is an enormous 10 to 11-storey structure probably on reclaimed land, with link bridges to buildings on three sides (plate 4.118). The design philosophy for this building is unclear, but there was sufficient structure in the form of internal braced bents and perimeter shear walls (the latter at least in both directions) to resist the loading. Internally the floor system comprised RC floors on permanent metal formwork, supported by steel beams, the span of which had been reduced by RHS braces (plate 4.119). Intended or not this provided the structure with a moment resisting frame, albeit only in one direction. The gusset plates beneath the beam were unstiffened and under the combined effect of compression and lateral distortion had bent sideways the stiffened end fins on braces, breaking the weld between the stiffeners and the tube (plates 4.120 and 4.121). The 50mm lateral distortion was surprising in view of the rigidity and good condition of the remaining structure. The steelwork was protected by rigid cladding, which had been shed around the damage. At the time of the visit much of this had been replaced, apparently with no attempt to straighten or strengthen the steel.

Around the perimeter (plate 4.122), diagonal cracks ran into the shear walls above and below the connecting beams (plate 4.123). These indicate insufficient confined vertical reinforcement trimming the end of the walls.

Diagonal X-cracking had occurred in the external columns between the shear walls (just visible in plate 4.122), a possible consequence of their having been designed only for gravity loading. Despite deep walls at ground level such shear walls are known to be capable of rotating and imposing distortion on adjoining frames, which are prone to the observed type of damage when there are short lengths of column between deep parapets[4.1]. The damage to these columns was most severe internally, where the car park was weakened by the well within the spiral ramp (plates 4.124). The columns at the ends of this well, also between deep parapets, had undergone severe distortion and loss of concrete cover (plate 4.125). Despite the absence of through bars the closely spaced links had worked well, adequately confining the core concrete. It is likely that the columns elsewhere (noting the beams were of steel) were of SRC.

4.9.3 9-storey car park near Kobe Immigration Office

This car park is similar in concept to that considered in 4.9.2 except that longitudinal loads were resisted by a spine wall down the full length of the building (plate 4.126) and transverse loads by transverse walls either end of the spine walls, around which went the ramps. There were further transverse walls externally. The 'H' configuration of walls, being monolithic, was more robust than the more dispersed shear wall system in 4.9.2. The floors were of RC on permanent metal formwork supported by unprotected steel beams connected to fins out of the SRC columns.

The unprotected bracing members in this car park were of circular tubes rigidly connected at the lower end to substantial plinths (plate 4.126) but, despite the absence of stiffening at the top connection (plate 4.127), these were undamaged, probably due to the protection to them afforded by the shear walls. The good performance would appear to justify the curious stress relieving detail (plate 4.127), probably incorporated to overcome problems with earlier variations of the detail.

The cross walls however were damaged from the 2nd to the 5th floor, sometimes in the form of horizontal cracks (plate 4.128) and sometimes by diagonal cracks from the outstands of steel I beams (plate 4.129). All had been repaired when inspected.

4.9.4 7-storey Diamaru car park

A number of the multi-storey car parks in Kobe were of SRC construction ranging from the 7 storey Diamaru (plate 4.130) in the central area to 3 storey variations in surrounding areas (as that in plate 4.131).

The Dimaru with heavily protected steel beams, SRC columns and SRC cross-bracing (plate 4.132), had performed well, but one isolated braced bay by the entrance had suffered cracking in the column and cracking in the concrete around the central gusset plate of the bracing (plate 4.133). The cracks were too narrow to expose either the steel or the reinforcement.

In the three storey car park, also with SRC columns and SRC cross-bracing, cracks had occurred at the upper end of the bracing. In yet another, similar but with simplified bracing, reminiscent of the inverted V-bracing used in shear hinges, much larger cracks had developed at the upper end, either side of the shear hinge (plate 4.134).

4.9.5 Debs Park

This impressively strong and new looking 7-storey car park of SRC, with heavy dark blue columns and external bracing, was completely undamaged on the exterior. It was not examined internally.

4.9.6 4-storey steel car park

This car park has purpose-designed members and connections (plate 4.135), all of the most flimsy and inexpensive construction (if made on large enough scale!) with beams and columns of unprotected I sections (plate 4.136). The lateral resistance was provided by small RHS bracing to the third points in short central bays and around the perimeter. Some of the bays (including those in the centre) appeared to be of long shear hinge construction. Unlike the other multi-storey car parks this one had an external ramp. The usual column base plates were replaced by an intriguing stiffened open-sided crate, a system which accommodated and protected the holding down bolts and was exceedingly neat. The construction reduced at successive floor levels and at the roof it was virtually a tent (plate 4.137). The light flexible and undoubtedly long period structure was completely undamaged and it is considered ideal for regions subject to strong earthquakes.

4.9.7 Three-storey car park in the dockland area

This very economical car park, while having substantial RHS columns, had a very economical open grid floor with the floor beams so curtailed that the backs of vehicles overhung the building (plates 4.138 and 4.139). Due to the absence of obvious ramps and the curious parking of vehicles internally, mechanical parking is most likely, a factor associated with high lateral flexibility. The building had been too flexible for the cladding to the ground floor offices, which was badly damaged.

4.10 Performance of link bridges

Link bridges between adjacent buildings are high risk features in regions of high seismicity and their performance is always of interest. Serious damage to buildings attributable to link bridges is described in 4.6.2, 4.6.3 and 4.9.2.

On the south side of the car park considered in 4.9.2 there were two skewed link bridges (plates 4.140 and 4.141). There were a skewed movement joint at the south end of each bridge at deck level (plate 4.142). The bridge pulled away from the building and rotated, opening the movement joint up to 150mm.

The top boom of the truss above it, which had been dowelled into the adjoining structure (plate 4.143), separated from the building, pulling out the dowels. Impact debris in the form of broken glass was strewn around the movement joint.

On the west side a bridge had moved away from the car park, pulling away a large section of the wall beneath (plate 4.144). The extent of the damage had not at the time of the inspection been assessed by the owners.

Sometimes, as in the City Hall (see 4.6.2) failure occurred where a link bridge connected two dissimilar buildings. Plate 4.145 shows another instance in which (as with the building next to the car park discussed above) the buildings themselves were only damaged at the point of their connection to the bridge. The two buildings in this case were recently completed and strong enough to withstand the additional loads applied by the bridge.

The AIJ report notes a link bridge which fell due to a relative ground displacement of 200mm between the school buildings it connected. The buildings, built in 1989, suffered little damage.

Whilst in principle link bridges can be designed to have no effect on the buildings either side they require the use of bridge technology, taking into account not only the effects of the movements considered in 4.16 but also the effects of damage to either building and collapse of the bridge.

4.11 Performance of very tall buildings and effects of height

Very tall buildings are classified as those exceeding a height of 60m for which special design is required (see 4.2). This is presumably why there are an appreciable number of buildings in Kobe up to about 14 storeys, but few much higher. There are very few buildings in Kobe much taller than the 21 storey Sakura Card building. Those observed were clearly of recent construction for which non-linear analysis and special approval would have been required (see 4.2). Those noted were:-

the 32-storey City Hall (see 4.5.12),
the 33-storey Dunlop Building and
the 41-storey Okura Hotel.

No damage was reported in the Dunlop building and only minor damage to the City Hall (see 4.5.12), but the Okura Hotel at the time of writing is still not opened to custom and internal shear walls are believed to be damaged.

The effect of building height on the damage is well illustrated by a study of the performance of apartments of various height built on a site between 1991 and 1994 subjected to an estimated ground acceleration of 0.25g[4.21] where the ground was somewhat firmer than in the areas of more heavily damaged buildings to the south. The lower buildings (4 to 13 storeys) were of RC and were ground bearing and the taller buildings (14 and 25 storeys) were of SRC and piled. Most were of frame/shear wall construction.

The 14 storey buildings which had the same period as the ground (of 0.8s period) suffered the most damage, and far more than the 25 storey buildings (of 1.3s period). The damage was minimal in buildings of 7 storeys and less.

Structural damage was confined to cracking in the stiffer buildings but the non-structural damage was very much greater in the more flexible buildings.

Comparable observations have been made (by Professor Wada) on the complex described in 4.5.9 where the buildings were of 17, 19, 22 and 24 storeys and were of very similar construction. The damage, which was appreciable, was confined to the buildings of 19 and 22 storeys. It is likely that the period of the ground here exceeded the critical 0.8s noted above, so the vulnerable height range would have been greater.

4.12 Performance of base isolated structures

4.12.1 General

Due to the perceived lower risk of earthquakes in Kobe than in Tokyo before the earthquake only two buildings in this area were constructed on base-isolators [4.22]. These are the Laboratory Building of Matsumuragumi Technical Research Centre (MTRC) (Plate 4.146), which is located approximately 35km from the epicentre, and the Computer Centre of the Ministry of Post and Communications (CCMPC) (plate 4.147), which is about 2 km away from the MTRC.

4.12.2 Layout of the laboratory building of MTRC

This was the first base-isolated building in the Kansai area and was completed in April 1994. It is about 10km north-west of the Rokko Mountains and about 35km from the epicentre of the earthquake. The main structural parameters are (see also Figures 4.6a and 4.6b):

Structure area:	160m^2
Length of building (X-direction):	16m

Width of Building (Y-direction):	10m
No. of storeys above ground:	3 storeys
Eaves height:	12m
Total height:	12.50m
Standard floor height:	3.90m
Foundation bed level:	2.80m below ground
Type of construction:	Rigid reinforced concrete frame structure
Isolator type:	High damping rubber bearing (HDRB)

The building was designed according to the Building Standard Law (BSL) using ordinary earthquake-resistant design for a shear coefficient C_0 of 0.2. It is separated from the adjacent steel framed office building by expansion joints. The latter building is not seismically isolated and has approximately the same natural period of vibration as the isolated building.

Figure 4.7 shows the soil profile of the site. The foundation of the building is bearing on mudstone at a depth of 2.80m. The allowable bearing capacity of the soil for permanent and temporary loadings are $50kN/m^2$ and $100kN/m^2$ respectively. For these checks in Japan permanent and temporary loads are not combined.

A high-damping type rubber bearing (MRB-LHD) was used with two diameters, 600 mm (LHD-600) and 700mm (LHD-700). The bearings were arranged so that the LHD-600 isolators are under the corner columns, and the LHD-700 are under the internal columns, as shown in Figure 4.8. The specifications of the two rubber bearings are shown in Table 4.2.

4.12.3 Outline of construction work and maintenance plans

Temporary steel blocks having the same size and shape as those of the isolators were installed on the foundation. The usual construction work of the building was carried out with the building thus supported. After completion of construction work the whole building was jacked up to replace the temporary steel blocks with the isolators. Two sets of hydraulic jacks of lifting capacity 1000kN/set were used.

The whole building was lifted momentarily, maintaining small gaps sufficient to replace successively all the isolators. After completion of the replacement process the building was jacked back down and the anchor bolts were tightened. The replacement work and installation of the isolators did not cause any cracking to the structure (plates 4.148 and 4.149).

A seismic gap of 400mm was provided between the isolated building and the adjacent office building so that a relative displacement of this magnitude might occur in all horizontal directions. In the vertical direction, provision was made for a displacement of about 50mm.

In addition to the extensive check that would be carried out on the building after the occurrence of earthquakes, typhoons, heavy snowfall, etc., a check is carried out routinely every six months. This is in order to ensure that the building will continue to function as intended. If any serious deformation or damage was to be found during the regular checks, more extensive checks would be carried out.

4.12.4 Earthquake response

Figure 4.9 shows ground motions record and velocity response spectra at the MTRC building. At foundation level, peak ground accelerations were 272 gal (0.28g) in the N-S direction, 265 gal (0.27g) in the E-W direction, and 232 gal (0.24g) in the UD (vertical) direction. This shows that the peak vertical acceleration was approximately 0.9 times the peak horizontal acceleration.

The earthquake velocity input into the building was as follows:

0.205 m/s in the NS-direction
0.322 m/s in the EW-direction
0.087 m/s in the UD-direction

Although it is evident that the earthquake was not severe at the location of this base-isolated building, the isolators performed as designed and there was no report of any damage inside the building. The building was reported to be fully functional after the earthquake and the isolators undamaged, although there was a small residual displacement approximately 1mm[4.22].

Based on the above and other information provided by MTRC, a 3-D dynamic finite element analysis on the building has been carried out in order to achieve a good understanding of the seismic behaviour of the structure and to investigate the degree of correlation between the FE analysis and the real behaviour of the base-isolated structure.

4.13 Performance of hospitals and schools

The West Kobe General Hospital was a 9-storey SRC/RC building of SRC to the seventh floor with crossed I section cores. The top two storeys were of RC. It was built in the early 1970s. It suffered a mid-storey collapse in the 5th storey of one wing of the hospital.

The NHK television news reports[4.23] showed victims being carried away from a damaged hospital after the earthquake. It is essential that structures that are required to provide essential services after disasters, such as hospitals and fire stations, must be designed with greater resistance to seismic forces than ordinary structures, such as by the techniques considered in 4.12.

Many of the schools in central Kobe were damaged. Those that were undamaged were often used as refuge centres following the earthquake. Many of the other public buildings that were undamaged were also used to shelter the homeless after the earthquake.

Within one week of the earthquake 325 out of the 591 schools in Kobe and the Hanshin area had re-opened.

4.14 Performance of foundations

It was generally very difficult to find information on the types of foundations used in many of the buildings in Kobe though it is understood most buildings of any significance had been piled. The majority of the shoreline in Kobe had been reclaimed at some point and it is presumed that most of the buildings on the waterfront are on piled foundations. This supposition is confirmed by the performance of buildings along the waterfront. These buildings generally showed only minor damage but significant ground settlement was evident around them.

On the islands, particularly Port Island, which have all been reclaimed, the difference in performance of pile and raft foundations was more evident. The islands suffered severe settlement (see Chapter 8) and those buildings on piles performed well except where the tops of the piles were not connected with ground beams. There were several instances where single piles had moved laterally as the ground settled (see Chapter 7) and this movement was a potential source of damage to the buildings (see 4.6.7). However, the large settlements on Port and Rokko Islands had effectively severed the underground service connections to the majority of buildings which are on piles. The buildings on raft foundations generally performed less well as they settled with the soil, but many did not appear to have suffered adversely from the large settlements, probably because the settlement was very uniform over large areas of the islands away from the sea walls. On the mainland the settlements were not as severe and no instances of differential settlement across a raft that caused building collapse were observed.

Ground movements caused damage in the form of settlement and differential horizontal movement causing partial collapse of the ground floor storey at one end of the building on the slopes above Kobe discussed in 4.4.4.1, the failure in the long side of a steel shed discussed in 4.8.10 and damage to a new building near the ferry discussed in 4.6.7. This is the type of damage that foundation tie beams are provided to prevent.

4.15 Performance of ancient structures

The few very old buildings affected by earthquake were temples and monuments. All examined had fared badly, suffering column collapse (plate 4.150) or severe racking (Plate 4.151). In cemeteries sculptures and headstones toppled (Plates 4.152 and 4.153).

4.16 Impact damage and separation distances

4.16.1 Impact damage between buildings

There were a number of cases where buffeting (impact) had occurred between neighbouring buildings of different height, but this earthquake has produced evidence of serious damage to neighbouring buildings of the same height and construction. In two cases observed the main movement had occurred transversely and in one of these the two-way inclination of damaged structure indicated the relative movement had been in both directions (Plate 4.154).

In an old 9 storey corner office building with continuous fenestration and brickwork faced lintels, bevelled in plan at the street corner, severe impact damage had occurred against a much newer building of similar height (plate 4.155). It appeared to have suffered a considerable streetwise displacement, with severe damage to the column adjacent the adjoining building. The second floor had collapsed completely over the height of the windows.

Serious damage was generally in the lowest storey whilst the actual impact, as evidenced by the local spalling, had occurred higher up. For example the building in plate 4.156 (which relates to the building discussed in 4.4.4.2), in which a wide gap opened between the building.

An interesting buffeting failure had occurred between two buildings of equal height (plate 4.157) in which the top corner columns of the older RC building had fallen, leaving the top floor and roof supported by the cladding.

Many of the damaged buildings considered in this report are on corner sites at road intersections and bi-directional buffeting is likely to have contributed to their condition. Where wide separations are not practical consideration should be given to designing such buildings for higher forces. It has previously been noted that when the buildings are not separated the buildings on corner sites, whilst sustaining substantial damage themselves, actually stabilise the terrace houses between[4.19]. In this situation therefore these buildings would need to be designed for a substantial portion of the mass laterally supported.

It is of course impossible, due to destruction or invisibility of the evidence, to be certain in all cases that the damage was preceded by the buffeting, though the fact that the weaker building is usually thrown away from the stronger provides circumstantial evidence. However the many instances of less seriously damaged buildings in which the damage was largely confined to the adjacent edges (not specifically considered in this report as is was so common) suggest buffeting is likely to have contributed significantly to the damage in the more damaged buildings.

Damage from buffeting was generally most severe in corner properties and some collapsed, as in the Merina district of San Francisco in the Loma Prieta earthquake[4.19]. In Kobe these buildings had usually collapsed across the street and the debris cleared soon after the earthquake. Other examples are provided by the buildings shown in plates 4.31 and 4.71. Penelis[4.40], by reference to a series of lollypop oscillators, demonstrates how buildings at the end of streets absorb more energy than those elsewhere. Added to this is the additional factor that impact is always on the same side, so damage is cumulative in one direction.

4.16.2 Impact between buildings and viaducts

The high density of development in Japan has resulted in little or no separation between buildings and highway and railway structures, and cases were found when these had collided. Plate 4.158 shows an industrial structure (the electrical substation discussed in Chapter 7) fallen against a steel bridge. Sometimes the buildings were built around the columns of the highway structures. When the building is full height the highway survives, but adjacent spans supported on columns are liable to collapse (plate 4.159). When there is a short length of exposed column above the roof of the building this part of the column is liable to suffer severe damage (plate 4.160). In Kobe such damage was caused by less substantial buildings than those in Northridge under the Santa Monica Highway, where similar damage occurred.[4.1]

A long 4-bay wide predominantly one storey warehouse, though higher along the road, lay alongside a railway viaduct and had railway sidings over part of the roof. Impact had occurred between the warehouse and the viaduct causing disintegration of the drop-in slab between them (plate 4.161), causing severe transverse damage to the columns in the lowest storey. Longitudinally impact had occurred across the joint between the single and multi-storey structures and the single storey structures on either side, and was the main cause of damage in the upper storeys (plate 4.162). The distribution and extent of the damage is discussed in Appendix 4C.

4.16.3 Separation distances and widths of expansion joints

From the buildings examined it is possible to make a first order judgement on the separation distances between buildings and widths of expansion joints to avoid impact damage from earthquakes of the magnitude of the Great Hanshin earthquake on relatively soft soils. For rigid structures 50mm would appear to be adequate (see 4.8.3) where the foundations are continuous across the joint, but for medium rise buildings of very different height and construction and where the adjacent foundations are not interconnected 200mm or more may be necessary (see 4.4.5.3).

Interpolating one might conclude 75-100mm to be appropriate for medium rise buildings of moderate flexibility of the same height, proportions and construction, but slightly more when the proportions differ as in 4.4.7.9 where the final open joint is shown in Figure 4.163. For a moderate earthquake calculations show that on average about 200mm separation is needed between typical adjacent medium rise buildings of about 10 storeys and 15 storeys, but 100mm may be sufficient when the lower buildings is about 6 storeys[4.24]. However these distances need to be increased by about 50% to include for one standard deviation of possible responses. Clearly where the foundations either side of an expansion joint are separated wider gaps are required than when the foundations are on a common foundation. For a common foundation the calculated deflection can be used as a guide in determining the width of the joint.

4.17 Effect of vertical acceleration

This earthquake has stimulated an interesting debate on the effect of vertical acceleration on the damage. Damage due to vertical acceleration was noted on a column in a building in Santa Monica after the Northridge earthquake[4.1], but it was attributed to internal effects within the structure. The same general effect would be expected in the peripheral columns of braced shear wall buildings, which tend to rock under lateral loading subjecting columns to alternating compression and tension.

In Kobe vertical accelerations were comparable to the horizontal accelerations, with one measured value nearly 50% greater (at 0.45g), though they were generally less.

It is considered that, if high enough, vertical accelerations could not only cause both compression and tensile failure in the columns but also induce damaging vibration in the beams. However the worst effect is likely to be after failure when the superstructure could rise and fall at frequencies of a hydraulic breaker, but with a far greater load and with amplitudes smaller, but nevertheless in excess of 25mm. Reference 4.25 makes a plausible case for attributing much of the damage in columns to high vertical acceleration, but studies examining the effect of lower vertical acceleration (about 40% of the horizontal acceleration which was more typical in Kobe) found it to have a small influence on the forces and storey sways, and possibly too small to justify the inclusion in design[4.26].

A characteristic of Japanese buildings, well represented amongst the buildings considered in this chapter, is that many are only one or two bays deep whilst the buildings are somewhat taller in Japan than in most other countries, a consequence of the high density of population in the habitable areas. As a result the aspect ratio (height/minimum horizontal dimension or H/B) tends to the rather higher in Japan than in less densely populated countries. In comparison with the USA this applies particularly to the more suburban areas such as Northridge, San Fernando and most of Santa Monica, which could be regarded as suburbs of Los Angeles.

The high aspect ratios, substantial panel infills, deep fascia beams and the code-encouraged use of frame/shear wall structures (which restrains frame action in the lowest stories) cause buildings to rock, placing the columns on one external face in uniform compression and the remote face in uniform tension.

When the infills and walls are continuous through the ground floor storey the column moments may be no greater than those in infilled frame construction. The external columns are then subject to alternating tension and compression in phase with the horizontal forces. For H/B ratios above 2 this produces higher vertical accelerations than the vertical component of the excitation (which as noted above is usually, but not always, less than the horizontal component). Axial loads are therefore considered to have contributed to the failure modes observed in a number of reinforced concrete columns, an occasional steel column (evidenced by the failure modes in columns in 4.5.9 and E7) and to the failure of base plate/holding down bolt assemblies.

When there are shop fronts or car ports in the ground floor storey the axial forces are the same as in a building one storey lower. However, the columns in that storey are subject to similar moments as in buildings with sway frames only at the lower level. The combined flexural/rocking mode is well illustrated by the building considered in 4.4.7.10.

In order to assess the relative contribution of rocking and the vertical excitation to the damage an extensive survey was made of column failure modes in a wide, low and very heavy buildings, which indicates the modes occurring when rocking is substantially eliminated. This is described in Appendix 4C, from which it is inferred that in the buildings of high aspect ratio more damage was caused by rocking than by the vertical excitation.

4.18 Fire damage

Land space in Kobe, as in all of Japan, is at a premium and hence the outskirts of Kobe are very densely populated with only very narrow streets or alleys in between buildings. The older buildings are wooden frame and in most areas there is a mix of light industrial and residential buildings (plates 4.164 and 4.165). Timber buildings are particularly vulnerable as most Japanese homes do not have central heating and rely on kerosene, gas or electric heaters for warmth. These heaters are easily toppled in earthquakes and readily start fires. The combination of a large potential source of combustible material and small scale industry with unprotected gas tanks and domestic users with heaters has always had the potential for disaster.

The damage caused by fires after an earthquake can be as great as that caused by the earthquake itself and the risk in Kobe had been recognised by the authorities. Many specially designed underground water cisterns had been constructed and these were intended to ease the pressure that would be put on the cities main water supply lines in an event such as this. Unfortunately the scale of many of the fires and inaccessibility of many of the areas meant that these reserves were insufficient to cope with this disaster. The fire services were unable to reach the affected areas of the city quickly because many of the major roads and most of the freeway system had been badly damaged. Once in the region of the fires, access was further restricted as many of the smaller streets were completely blocked with rubble. The main water supply lines were also badly damaged during the earthquake, further reducing the ability of the fire services to control the outbreaks.

Fortunately at the time of the earthquake only a light wind was blowing but even so many of the small fires that started after the earthquake quickly became large fire storms. Because there was little space between the buildings there were no effective fire breaks that could stop the spread of fire. It has been estimated that over 1.3km^2 of the city was destroyed by fire and most of the areas destroyed were residential. Over 50% of this destruction occurred in Nagata Ward. It is unclear as to the extent of the damage to the houses before the fires started, although there is no reason to suppose that the houses in these areas performed any better than elsewhere. If not then many people would still have been trapped in the remains of their houses as the fires spread through the City.

The commercial properties interdispersed between the timber houses were badly damaged by the flames (plates 4.164 and 4.165). The areas affected in total amounted to about 70% of that affected by the Great Fire of London.

4.19 Repairs

Building repairs observed during the visit were limited. Clearly first priority had been given to clearing debris and there was no obvious strategy to repair. Possible strategies are given in Eurocode 8 Part 1.4[4.27] and it assumed engineers were for the most part engaged in deciding:

(1) Whether buildings were worth repairing (or effectively removing the orange tagging) and
(2) Deciding on repair priorities.

Small waterproofing tasks were in progress in a number of locations, but little reglazing. It was possible however that glazing measurements had already been taken and manufacture was already in progress. Well advanced however was epoxy grouting of cracked shear walls. One such building was subject to detailed inspection. Here cracks had been sealed on the outside and injection nozzles installed roughly at 300mm centres, for injection with resin from cartridges (plate 4.166), as in the UK. Different from UK practice however was the introduction of reservoir tubes on the remote face from that being injected, clearly intended as indicators as to the penetration of the epoxy (plate 4.167).

4.20 New construction

On a new building site the column bars for the ground floor lift has been assembled into cages. All the reinforcement was of deformed steel. The longitudinal bars were butt welded. There was 500mm stagger between the level of the welds in adjacent bars. The horizontal bars were U shaped, with ample laps, but they were not welded and there were no bars through the middle of the columns (plate 4.168).

Plate 4.169 shows one of the new steel buildings been developed on the city's waterfront.

The building, presumably designed to modern codes, had a substantial steel frame and no damage was apparent. Plate 4.170 shows the apparently brittle glass cladding on a column in a modern 3 storey commercial building. The building suffered no external damage and neither the glass panels on the column nor the glazing between the columns was damaged.

4.21 Comment on assessments by others

4.21.1 A broader perspective

As the scale of the damage exceeds the amount which could be assessed by a visiting team the perspective in this report has been broadened to include the initial observations of others.

4.21.2 AIJ Preliminary Report[4.15]

Interesting observations made on the basis of the buildings reviewed in the AIJ Preliminary Report are included in Appendix 4E. The conclusions in that report are not included in ours except as indicated.

4.21.3 Report of the Disaster Prevention Research Institute of Kyoto University[4.28]

Interesting observations besides those covered elsewhere follow:

(1) In many houses columns and beams have simple mortice and tenon connections, though there were a few, but generally insufficient, nails in the collapsed houses. The plank facing was too thin and too poorly connected to the frame to provide much restraint and the nailed connections tended to split or pull-out enabling the frames to deform to a parallelogram. In new houses extensive nailing is required and now the minimum walling is related to the floor area. Houses conforming with these requirements performed well.

(2) Emphasis in Japanese design is now placed on the need to distribute ductility throughout the storeys.

(3) This is the first earthquake in which storey collapse has occurred in SRC construction at positions of discontinuity of the embedded steel frame.

(4) Shear walls with boundary elements which should possess at least nominal ductility have suffered a considerable amount of damage.

(5) No shear wall buildings collapsed.

(6) Due to steel shortages in Japan in the early 1960s and the high price of steel, buildings were designed and constructed "without much regard to seismic considerations". Whether this implies disregard of the lateral forces needs to be ascertained.

(7) The cold formed steel sections used for columns generally buckle before achieving their plastic capacity.

(8) Even buildings with excessively slender braces usually survived, which is attributable to the degree of redundancy employed in Japanese steelwork construction.

(9) Most of the few moment resistant frames were found to have suffered damage at the beam to column connections and could be attributed to weld failures.

(10) Structural steelwork is hidden behind both fire protection and architectural finishes and damage is more difficult to detect than in RC building.

(11) It is stated that most connections in Japan (both formerly and now) have been designed to permit shop welding and field bolting, so a prime objective is to identify the cause of damage at beam/column connections.

(12) It is claimed that in newer buildings most damage in steel buildings exhibits ductility.

4.21.4 Observations of the NZEERE(4.29)

Additional points made by the New Zealand team were as follows:

(1) The few western style houses scattered throughout the areas of maximum damage survived.

(2) There are a significant number of new high glass clad buildings in Kobe which performed well.

(3) The brittle fracture in the cold formed columns of box sections was aggravated by the low temperature at the time of the earthquake of about 0°C.

(4) Recently buildings of 15 storeys and taller have tended to be of steel due to the speed of erection.

(5) In steel buildings the high slenderness ratio of plate elements in the columns caused severe local buckling.

(6) Steel moment resisting frames generally performed well as the welded beam to column connection was invariably stronger than the beam, though failure was noted about the column minor axis where the tips of beam end plates had been welded to the column and the welds failed.

(7) However, there was no deliberate attempt to design connections to provide an overstrength margin with the result that there were many failures at bolted connections, especially in bracing.

(8) In welded RHS construction a number of cases were noted in which the column welds were grossly undersized, with throat thicknesses from 4 to 5mm.

(9) In concentric braced frames (CBFs) compression buckling was noted in gusset plates, which was considered to have implications on New Zealand practice.

(10) Mid height single storey collapses were noted on older SRC buildings of between 5 and 10 storeys, attributed to the changes of cross sections and inadequate confinement. The important point is made (not noted elsewhere) that over the lower 3 to 5 storeys a solid web I section (not H section) was used for both beams and columns, but the steel section in the columns was then reduced to battened construction (in the form of two pairs of angles connected by horizontal battens). A failed mid-height weak storey exposing the battened construction is illustrated.

Failure occurred in the storey above the transition and was attributable to the sudden decrease in stiffness and strength of the columns.

(11) An RC building is illustrated with joint failure in the top two floors of an external column. The damage appears to have been assisted with pounding against its neighbour.

4.22 Conclusions

The Kobe Earthquake has potentially provided structural engineers with more valuable information than any previous earthquake, but the value of the experience depends upon the depth of the investigations that are undertaken relating to the performance with the construction. As a result of the fundamental differences in design philosophy between the Japanese codes (discussed in 4.2) and Eurocode 8 or the US codes, the above observations do not necessarily provide a good indication of the likely performance of buildings designed to these other codes. Nevertheless many useful indicators have emerged both supporting and occasionally calling into question aspects of these codes. It appears that until this earthquake the Japanese have used predominantly elastic design at the serviceability limit state with some checks at the ultimate limit state. The Japanese now have draft codes for full limit state design in both reinforced concrete[4.30] and steel[4.31], but it is not known whether or not these have been applied in the design of any of the newer buildings.

The present Japanese code effectively allows dual frame/shear-wall structures of mixed construction materials (RC and SRC) without restriction on the structural configuration or the method of design, other than a correction for elastic eccentrically in the calculation.

As a result there is a very heterogeneous mixture of buildings and variability in structural form, which is undoubtedly reflected in the very variable performance observed. It is unclear, at the time of writing, whether dual structures were designed allowing for the compatibility of displacement at each floor level. In a frame/shear-wall structure the shear wall at the base attracts most of the horizontal load, with little of the lateral load carried by the frame at that level. However when the shear wall is damaged or rotates on its foundation, appreciable lateral loading is thrown into the frame at this level and severe damage may occur. This situation is implicitly taken into account in the rules of other codes, such as Eurocode 8 and the UBC.

The lack of information on the distribution of the excitation has not permitted pgas to be estimated at the sites of the various structures discussed. The most typical value is considered to be 0.60g.

The conclusions from the observations of the EEFIT team supplemented by Japanese research but excluding those by other reconnaissance teams (whose reports are reviewed in Section 4.21) include:

(1) Reinforced concrete design/detailing

(a) The hydraulic pressure analogy used in the derivation of the rules for the design of column links in EC8[4.32] has been justified by observation of the ovulation of links in rectangular columns.

(b) Cross ties are needed in rectangular RC columns, though with closely spaced links the need for lateral confinement to alternate bars as required in Eurocode 8 may be excessive.

(c) Joints become critical in RC members when beams and columns are properly designed.

(d) Short welded splices on links are failure prone, though whether this is due to shock, unzipping, defective or insufficient welding remains to be established.

(2) Steel construction

(a) Steel structures are generally more flexible than those of reinforced concrete, of which there is some previous evidence(4.33).

(b) The high strength, cold formed, tubular steel sections used in Japan are non-ductile.

(c) Certain types of concentric bracing with the weak axis perpendicular to the plane of the frame are highly ductile.

(d) Steel base plates tied down with holding down bolts on plinths tend to crush the plinths, whereas base plates tied down into massive foundations perform much better(4.33).

(3) SRC construction

a) Present day SRC performed well.

b) Combining our observations with those of other it appears that earlier forms of SRC, with structural steel components of low ductility, perform better, but not very much better, than non-ductile RC construction.

c) SRC casings are insufficient to ensure the good performance of plinth mounted base plates.

(4) Failure modes in frames

(a) The surprisingly high number of mid-height single storey failure mechanisms is attributable to:

 (i) A change in the medium of construction above the level of the mechanism, and/or

 (ii) The distribution of storey shear used in the design of buildings sometimes is unconservative compared to that given by spectral analysis. This is relevant to buildings up to about 10 storeys in post 1981 designed buildings, for which the distribution is the more unsafe the lower is the building as shown in Figure 4.10, and to buildings of any height in buildings designed before 1981.

(b) Strong beam/weak column characteristics are not synonymous with single storey failure. In fact, when the columns are of all the same size and all nominally reinforced, column hinges are likely to be distributed over more than half the height of the building. This distributed inelasticity is the consequence of the absence of hinges in the corner columns, as discussed in 4.4.3, 4.4.4 and Appendix 4B.

(c) In buildings with single storey failures (or more complex modes with similar characteristics as in 'd'), the parts of the building above the collapsed storey were invariably less damaged than comparable buildings without single storey failure.

(d) A number of buildings have been noted with partial collapse of a single storey, a mechanism comprising a partial storey failure and a partial bay failure. In most of the cases observed there was no obvious vertical element adjacent to the failed bay to induce this form of failure. This needs further investigation. It could be that, due to the generally large columns in Japanese buildings, the beam and column strengths are very similar despite the considerable depth of the spandrel beams. In this situation a modest stairwell might be sufficient to redirect the mechanism.

(e) In one instance of the failure mode in (d) collapse had been induced by a ground crack which tore the building apart. It is doubtful whether this damage would have been prevented with normal tie reinforcement.

(f) In Appendix 4C frame mechanisms observed (excluding that in (d) which is confined to long buildings) are compared with the theoretical mechanisms assumed in codes.

(g) An attempt has been made to give an indication of the relative occurrence of causes and modes of failure and the members most affected in Table 4.3.

(5) Frame/shear wall structures

(a) Where shear walls and framing members are employed in the same building to satisfy the minimum area requirements in the 1981 code, it is unsafe to assume the walls and columns carry shear pro-rata their cross-sectional areas. The walls being stiffer and generally more lightly reinforced would be expected to suffer damage before the columns.

(b) In particular the less severe damage to internal shear walls is not necessarily reflected in columns within the facade frames parallel to the internal walls. However when the facade columns were damaged, damage to internal walls at the same level was likely.

(c) The partial storey failure of one building with a wall in one bay of a 5 bay structure was attributed to shear failure of non-ductile columns in that storey followed by collapse of the wall in the same storey[4.35].

(6) Low-rise buildings

(a) According to the 1981 provisions, buildings not exceeding a height of 31m (8 to 11 storeys) are designed at SLS for the moderate earthquake (0.20g) and nominal requirements as to the minimum cross sectional area of vertical members, but with no checks at ULS.

(b) It would be expected that buildings so designed would have performed very poorly because:

(1) The above force corresponds to only about one fifth of that considered at ULS (see 4.2(3)), and

(2) The distribution of the horizontal forces is sometimes unsafe (see conclusion 4a(ii)).

The fact that the buildings so designed performed better than somewhat taller buildings (see section 4.11), it attributable:

- mostly to the ample cross sectional area of vertical elements specified, and

- partly to their period of vibration being appreciably shorter than that of the structures suffering the greatest spectral amplification.

It is noted that walls contributing to the vertical cross-sectional area appeared not to be of ductile construction.

(7) *Taller buildings*

(a) The greatest spectral amplification nevertheless occurred in the taller buildings, being in 14 storey buildings on moderately firm ground and 19 to 22 storey buildings on soft ground, but nevertheless only four taller buildings were found that were badly damaged (See 4.4.1.3, 4.4.5.3 and 4.4.6.1 and 4.5.3).

(b) On the supposition that the regulations had been strictly enforced, all the buildings taller than 31m (8 to 11 storeys) would have been of post 1981 construction. However two of the four buildings of this height found to be damaged, by their appearance, are considered to have been of earlier construction. This supports the view of others that the earlier regulations were not enforced.

(c) There is some evidence from this earthquake supporting the premise in Reference 4.1 that pgas at the base of heavy buildings is less than that in the surrounding ground.

(8) *Multi-storey car parks*

(a) Modern multi-storey car parks were predominantly of braced steel and SRC construction. They performed excellently, though some damage was caused by link bridges (see 9).

(b) Excellent performance also can be obtained from very light and flexible construction, which can be exploited more in this class of urban structure than in others.

(9) *Link bridges between buildings*

(a) Link bridges may be sufficiently rigid to cause damaging impact between buildings on closure of the joints.

(b) Nominal fixings of the top booms of trusses into buildings above movement joints in the deck (possibly a measure to simplify the cladding at the joint), increase the severity of its impact, cause local spalling damage and may even damage the bearings.

(c) More attention generally is needed in the design of the bearings.

(d) From the number of fallen link bridges reported, particularly in the higher stories (only one is considered in this report which failed for other reasons) the bearings should be designed for larger movements.

(10) *Damage prone construction*

(a) The method of anchoring the plank-like facade units on steel frames is prone to failure.

(b) Precast concrete facing panels are a source of weakness as they add mass but have little effect upon the strength.

(c) Landing beams should, where possible, be tied into columns or walls in line with the beam.

(d) Buildings in the order of 120m long without seismic joints, at least on soils such as that in Kobe, may be expected to suffer damage over the middle third.

(11) Foundations and site effects

(a) Landslips apart (see Chapter 8), there were no catastrophic building failures attributable to the failure of piled foundations.

(b) Whilst there are some reports of pile fracture it is considered that in general the ubiquitous piled foundations performed well. An indication of the likelihood of damage may be obtainable from the ratio of the permanent set of the ground to the pile diameter.

(c) Where liquefaction occurred the ground subsided around the buildings. In one (see 4.8.14), the subsidence was sufficient to expose the piles, which were undamaged.

(d) The effect of site period is considered in section 4.11.

(e) Corner sites at road intersections are damage prone situations needing special design consideration. Large corner bevels constitute a severe plan irregularity, exacerbating the vulnerability of the situation (see Plates 4.5, 4.13 and 4.90 and Reference 4.29 for even more extreme examples).

(f) Where there are a large number of similar well built structures a small number may suffer serious damage to the main structure while the rest may be virtually undamaged. Whether this is attributable to foundations, the ground, the structure or the random nature of earthquake excitation needs to be established.

(g) Taking into account the information from Northridge earthquake there is some evidence (though not much) for the considerable variation in pga between sites which are quite close, such that one might have twice the pga of the other. Taking into account the non-linear relationship between damage and pga (see Table 6.1 of Reference 4.1) it would be possible to explain the striking variation in damage between buildings in one area. Chapter 5 discusses local variation in the ground conditions. There is some circumstantial evidence for suspecting the existence, even for a short period during an earthquake, of partial standing waves, with local points of high and low intensity.

Such standing waves could be caused by:

i) a mixed frequency excitation from a single source;
ii) well separated sources with a single frequency excitation;
iii) a single frequency excitation from a single source, with reflection off surface and sub-surface features.

or any combination.

(h) There is in the text above (except in 4.5.9), no comment on the effect of directionality of the excitation, but the orientation of most of the buildings was easily established and it is found that, for buildings at least, most of the damage where pounding and other strongly directional effects were absent, was in a north-south direction. This might have been expected from the accelerations in central Kobe, but these are probably not the explanation for the damage in the heavily damaged strip to the east of Kobe (see 4.1.1). There the buildings tended to be much longer in the direction perpendicular to National Highway Route No 2 (or north-south) and it is considered that differential foundation movements contributed to the damage.

(12) *Joints*

(a) For the present earthquake and ground conditions in Kobe expansion joint widths and separations between buildings of 50mm are adequate only for rigid shear wall structures of low to moderate heights on a single foundation.

(b) Where foundations are discontinuous, as between buildings, larger joints are necessary.

(c) Largest separations are needed between buildings on corner sites.

(13) *Base isolation*

(a) Evidence has been presented of the ability of base-isolation techniques to successfully mitigate the effects of this strong earthquake both in terms of people safety and the integrity of the structure and its contents.

(b) Methods such as base-isolation, should be taken more seriously as an effective means for enhancing the seismic performance of hospitals and other buildings housing essential services structures.

(14) *Repair and strengthening*

There is an urgent need to establish reliable and cost-effective techniques to evaluate, repair, strengthen/retrofit existing structures and buildings in Japan that suffered minor or localised damage. Where there is suitable access to below ground structure or where the cost of excavation can be justified, base-isolation may be considered as an alternative technique for retrofitting.

(15) *The excitation*

(a) There is a strong possibility that the rules for coincidental orthogonal motions in earthquake design codes do not reflect the conditions that occurred during this earthquake (see 4.5.7).

(b) There were few clear indications that vertical excitation contributed to the damage, mainly due to the absence generally of columns supported on beams and the scarcity of long cantilevers. Where there is evidence of high vertical accelerations it is likely that overall rocking of the building contributed more to the damage than the vertical excitation.

(16) *Damage in relation to ground acceleration*

Prior to the Northridge earthquake there was generally a depth of reliable information on the intensity of the excitation in the most heavily devastated areas, resulting from the low area density of seismographs. Because of this the closest recorded excitations are generally from outside these areas. The few exceptions tended to be regarded as special cases or even anomalous. For example the isolated horizontal pga in Santa Monica (0.93g) in the Northridge earthquake did not reek the devastation suggested by Table 6.1 of Reference 4.1, and this may be because the source material for this table assumed 0.7g only occurred in Magnitude 8 events.

It is now clear that with a sufficiently dense distribution of seismographs horizontal pgas of 0.80g to 0.90g will be recorded and it is likely that pgas of this order existed in previous earthquakes in the most devastated areas, in which there were generally no seismographs. As a result it may be that Table 6.1 of Reference 4.1 is pessimistic with the possible exception of unreinforced masonry, for damage from an earthquake of Richter Magnitude 7. For an earthquake of Magnitude 8 or more however Table 6.1 (Reference 4.1) may be proved correct or optimistic. This will only become clear when Japan, California or some other region employing equivalent earthquake resistant design rules, and with a sufficient area density of seismographs, experiences such an earthquake. It is to be expected that the pgas in the most devastated areas will be found to exceed 1.0g, possibly in all directions simultaneously.

(17) Outstanding queries

Finally of particular interest in view of the history of code regulations in Japan, but beyond the scope of a field investigation team, are the issues of whether:

(a) the area of load bearing walls was proportionally less in the many buildings which suffered single storey collapse than in buildings which escaped this fate, and

(b) damage was less in SRC columns than in contemporary steel and RC columns of comparable flexural and shear resistance.

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Table 4.1 Japanese Building Stock

Material	Structural System	Building Classification
1. Wood	Shinkabe	Shinkabe (1-2 floors)
	Okabe	Okabe (1-2 floors)
	Four by two system	
	Prefabricated	Prefabricated (1-3 floors)
2. Masonry	Unreinforced masonry	Solid brick
		Hollow concrete/Clay block
	Reinforced masonry	Cut stone
		Solid brick
		Hollow concrete/Clay block
3. Reinforced concrete	Concrete shear wall	ALC panel
		Precast RC panel
		RC shear wall (cast-in-situ)
	Reinforced concrete frame	RC frame (pre-1950)
		RC frame and RC wall (ductile)
		Braced RC frame and RC wall
		Precast RC frame
		Precast RC frame with shear walls
4. Steel	Steel frame	Unbraced steel frame
		Braced steel frame
5. Light metal	Light metal frame	Light metal frame with steel slates
		Light metal frame with ALC panels
6. Steel reinforced concrete (SRC)	SRC frame	SRC with infall masonry
		SRC with RC shear walls
	SRC frame & RC frame	RC at the basement
		RC frame at the top or sides
7. Speciality buildings	Long span	
	Base location	
	Buildings on piles	

Note 1: ALC=aerated lightweight concrete   Note 2: Pre-1950 RC frames are mostly non-ductile

Table 4.2 Specification for Rubber Bearings in MTRC Laboratory

SPECIFICATIONS	LHD-600	LHD-700
Rubber isolator diameter (mm)	600	700
Rubber layer thickness (mm)	6.5	7.5
Number of rubber layers	21	18
Thickness of external rubber layer (mm)	8	8
Thickness of internal steel plate (mm)	2.2	2.2
Number of internal steel plates	20	17
Thickness of external steel plates (mm)	25	25

Table 4.3 Incidence of Modes and Causes of Failure and Members Effected

Causes & Mode of Failure and Element Affected	Local Failures	Single Storey Failures	More General Failures
Underdesigned	Very many	Very many	Very many
Soft storey		A few	
Weak storey		Many	
Corner site	Some	Some	Some
Reduction in section	A few	One	
Change in construction	Some	Some	
Strong beam/weak column	Very many	Very many	Very many
Corner induced	Few		
Pounding induced	Many	Many	
Irregularly induced	Many	Some	Some
Column	Many		
Beam	Many		Many
Connection	Many	Some	Some
Panel zone	A few		
Brace	Very many	Very many	Very many

APPENDIX 4A

COMPARISON OF SEISMIC CODES OF USA AND JAPAN

The first important difference between the two countries is the fact that in Japan there is only one set of earthquake design code provisions, that of the Building Design Law of Japan published in 1981; whilst in the USA there are several (with the date of the most recent issue in brackets).

- Uniform Building Code (1994)
- Structural Engineers Association of California SEAOC (1990)
- Building Seismic Safety Council BSSC (NEHRP, 1991)
- American Society of Civil Engineers ASCE (1993)
- Defence sector, Tri-Services Manual (1986)

The first earthquake resistant rules for structures were issued in the USA after the 1906 earthquake in San Francisco. According to those provisions buildings had to be designed to resist a wind pressure of 1.30kN/m^2, roughly equivalent to weak cyclones.

However the first seismic code provisions in a modern sense were only introduced in 1923 in Japan and 1933 in USA as a reaction to the earthquakes of Kanto (Tokyo) and Long Beach (Los Angeles). According to those provisions buildings had to be designed with a base shear coefficient equivalent to 10 and 8% of the weight of the structure respectively.

Since the 1933 Long Beach earthquake, a number of revisions have been adopted by the USA seismic code namely: 1943, 1952, 1959, 1976, 1988 and 1994. Each code revision has introduced one or more new parameters, like the period of the building and the effective peak acceleration or velocity. However despite all these revisions, the basic requirement for base shear between the 1933 and 1988 seismic provisions remains almost unchanged (Bertero, 1992).

The next major improvement in the US structural codes is to be the adoption of more detailed zonation maps, that take into account the effects of soil, other associated hazards and microzonation studies.

The main difference of the current (1981) Japanese code is that it has introduced a 2-level design procedure that specifically requires that designers consider both the serviceability and ultimate limit states. A somewhat similar procedure was adopted in the Tri-Services Manual of US Army in 1986, but these do not apply to the main bulk of buildings in the USA. Further points of difference between the USA and Japan code are:

- The US code does not (with some exceptions) include the live load in the building mass.
- The base shear forces are substantially higher in Japan (at least 0.20g against of 0.14g in USA, which is about 40 per cent difference). This is likely to result in larger and stronger columns and braces.
- The vertical distribution of forces is different (no concentrated force at the top of long period buildings in Japan).

- Similar storey drift limits are applied in both countries, but because of the difference in input design levels the results are not directly comparable.

- The way of calculating a building's natural period is different.

- The US code has more detailed provisions regarding ductility requirements and columns supporting discontinuous shear walls.

- The minimum earthquake forces required in the two codes are as follows:

USA (UBC, 1988):

$$V = (ZIC/R_W)W$$

where:

V = base shear shore; Z = zone coefficient; I = importance factor;
C = numerical coefficient depending on the dynamic characteristics soil and structure;
R_W = numerical coefficient expressing the capacity of energy absorption of the structural system in the inelastic range;
$C/R_W \geq 0.075$ and
W = dead load of the structure, including of any permanent heavy equipment as well as $50 kg/m^2$ for the infill and partition walls.

Japan (BSL, 1981)

$$V = C_i W = Z R_t A_i C_o W$$

where:

C_i = lateral seismic shear coefficient of the i^{th} storey
W = weight of the building above the i^{th} storey; including part of the live load ($0.80 kN/m^2$ for office buildings)
Z = seismic hazard zoning coefficient
R_t = design spectral coefficient (corresponds to C in US code, but its values are much higher)
A_i = lateral shear distribution factor, determined by the fundamental natural period and the weight distribution of the building
C_o = standard shear coefficient (at least 0.2 for moderate and at least 1.0 for severe earthquake motions)

APPENDIX 4B

FAILURE MECHANISMS OF FRAMES WITH NOMINALLY REINFORCED RC COLUMNS OF CONSTANT SECTION

This Appendix contains a study of a typical RC buildings in Kobe, assessing the likely distribution of damage in columns in regular buildings subject to seismic loading (from spectral analysis) with uniform member cross sections and uniform reinforcement throughout, but with the axial load increasing uniformly with height. In Figure 4.10 the amount of reinforcement required is compared to that provided, for columns designed such that the two amounts are the same in the lowest storey of nine storey buildings with beams and columns of constant width and similarly reinforced, with the beam/column depth ratios of 1.0, 1.5 and 3.0 and with 0%, 1% and 2% reinforcement. It is seen that with 1% reinforcement, the minimum acceptable in column design in UK codes (and presumably similar to the minimum in Japanese codes), there is a remarkably uniform distribution of the ratio of the action effects/resistance over the face of the building, with this ratio being lower for the corner columns then the others on account of the lower moments and axial loads.

As the amount of column reinforcement is increased the uniformity of the relationship still holds, but over a reduced number of storeys. This also appears to be the case with less reinforcement, though in reality, as the amount of column reinforcement is reduced the performance becomes so erratic that a more random response would be expected.

APPENDIX 4C

FRAME FAILURE MECHANISMS

In the design of frames in earthquake prone areas the `design' model is that in Figure 4.11(a), in which hinges occur at the ends of beams, with the implicit assumption of a hinge at the feet of the columns or column base rotation, although to have a well developed hinge may be embarrassing. It would have been expected that amongst the very large number of structures seen, at least some structures would have displayed the required mechanism. In fact none were found, and whist strong beam/weak column structures may have been the norm, some inevitably would have been weak beam/strong column structures.

Single storey failure mechanisms with hinges on the columns were common (Figure 4.11(b), but also to be expected, an account of the generally low strength of joints compared to adjoining members (if properly designed) a shear hinge local mechanism, as shown in Figure 4.11(c), would be expected. Also none were found.

Failure mechanisms of stocky structures encountered in fact may be more aptly summarised by Figures 4.11(d) and 4.11(g), in which obvious joint failure is replaced by fallen beam mechanisms 4.11(d), or failures where total disintegration and local joint collapse has occurred Figure 4.11(g). This can result in apparent rigid body rotation of narrow buildings or partial soft storey collapse in longer ones. Partial storey failures have been a particularly interesting feature of this earthquake and the various forms observed are described in sections 4.4.4 and 4.8.5.

In more slender construction, very exaggerated curved deformation shapes results as in 4.5.5 and very large distortions may occur at the lower levels, as shown in Figure 4.12(b). When connections fail large distortions may occur higher up as in Figure 4.12(c), and the most extreme cases are in braced bents where the bracing fails as in section 4.5.4. These compare with the straight-line deformed shape in Figure 4.12(a) which is the design condition.

This earthquake has provided the opportunity:

(a) to study column hinge formation at foundations, and

(b) gain some insight into the sequence of events in soft storey collapse.

There is a striking lack of evidence to support the formation of a plastic hinge in the columns in Figure. 4.11(a)[4.33] and in Kobe, particularly in steel structures, rotation occurred at this position due to failure of the base plates or due to effectively pinned connections (see plate 4.171), causing the moments at the top of the ground storey columns to deform excessively leading to large storey drifts (in Plate 4.171 this was caused by buffeting), often followed by collapse. In an old shed an old form of construction was encountered, noted in other countries, in which large triangular gusset plates substantially reduced the forces on the holding down bolts, protecting the base plates and inducing plastic hinges in the columns (Plate 4.172). The columns however were clearly undersized in respect of their minor axis flexural resistances.

An indication of how well bond can anchor columns if the concrete is sufficiently thick was provided by the old steelworks noted in section 7.7 (see plate 4.173) in which the lower part of heavy rivetted lattice columns were encastre in tall plinths. Despite suffering substantial spalling, partly induced by corrosion of the reinforcement, this detail proved most effective and the superstructure was undamaged (plates 4.174 and 4.175). This arrangement, with a substantial encasement reliant largely on mechanical bond, is considered preferable to encased base plates of the type considered in 4.6.5 in which the base plate constitutes a discontinuity[4.33]. Encased base plates were the standard detail in SRC buildings and these performed indifferently in this earthquake.

Some insight into the sequence of events in soft storey failure is provided by the damage to the warehouse discussed in 4.16.2 in which the beams were stiffened and protected from damage by deep haunches. Large torsional rotations of the floor diaphragms had occurred, with the displacements increasing from the transverse shear walls towards the central movement joint (plates 4.177 and 4.179). The displacements were accommodated by the hinges at the top of the columns, with markedly different displacements between construction of similar height (Plates 4.176 and 4.180) being greater in parts of the building with fewer transverse infill walls. The graduation of displacement (see plates 4.176 to 4.180) in the lowest storey of the warehouse provided an excellent opportunity for studying the development of this type of damage. For most part the hinges were at the top of the columns, which were presumably in this position due to:

- flexible foundation strips in the transverse direction;
- by the high shears in bays with some (but not enough) infill walls, in which despite the beam haunches;
- the shear carried by the compression diagonal in the walls in transferred to the tope of the adjacent columns
- the discontinuities which can occur infills with strong beam/weak column construction due to the high tensions induced in the columns by infilled frame action

The damage was initiated by flexural compression failure on bursting of the ties, as seen by the least damaged hinge where the links split (right hand column in plate 4.177). As the damaged increased concrete monoliths formed inside the longitudinal reinforcement (plate 4.181), and these rocked providing the structure with surprising flexibility until either:

(a) stability between the monoliths was lost (as in plate 4.182) or;

(b) the monoliths, acting as anvils under the vertical excitation, caused the disintegration of the concrete beneath.

The motion in (b) might be expected to cause progressive collapse of the core concrete and destrigging of the links from the vertical bars.

Longitudinally there were continuous RC members at the foot of some of the columns. These induced shear hinges at lower end of the columns, as in plate 4.184. As these hinges developed the well centralised anvils, sharper and more symmetrical than those in the flexural hinges on account of the way in which they were formed, split the concrete beneath causing symmetrical buckling of the reinforcement on either side, as in plate 4.183. Subsequent collapse of a column originally suffering this form of damage is shown in Plate 4.184 and it is interesting to note that collapse occurred without significant plasticity at the top of the column. It is also interesting to note that there were no comparable failures of columns with initial hinge formation at the top.

APPENDIX 4D

INVESTIGATION STRATEGIES AND DAMAGE CATEGORISATION

Often the structural form was unclear from the road as expensive buildings often had curtain walling, sometimes with external shades. The interior was often obscured from view by hanging blinds, or temporary weather protection, or where glass had been broken. Where this was not the case telephoto lenses enabled failed internal columns to be recorded.

Where buildings had collapsed completely diagnosis of the cause was particularly difficult because in a heap of rubble little is clearly identifiable. Attempts were made to identify failed connections and (in tilted or partially demolished buildings) foundation failure. The main problem however was the uncertainty of whether apparent failures were the result of partial demolition, necessary to ensure the safety of passers-by.

Engineers are mainly interested in the initial failures which induce local or overall collapse. Cracking in RC and SRC construction and local buckling in steel are generally necessary for the appreciable absorption of inelastic energy and in well designed buildings these are not indicators of incipient failure as they may be poorly designed buildings. In apparently well designed buildings however inelastic damage in local members is nevertheless cause for concern since damage should be redistributed. Local damage may render a building severely irregular and so induce unpredictable excitations.

APPENDIX 4E

THE JAPANESE PERSPECTIVE

4E1. Introduction

In the AIJ preliminary report[4.15] comprehensive studies from the affected areas have been combined in such a way to enable a very large number of interesting case studies to be included superficially. This was facilitated by the fortunate adoption of the same system of making photographic records as has been adopted for this report. This is the systematic use of overall building views and close-ups of the damage. Despite the small size of the photographs it has been possible to make additional observations to those by the authors of the report.

4E2. Wooden houses

The AIJ report does not differentiate between okabe and shinkabe and it is assumed in Kobe houses were generally shinkabe. The collapse of so many wooden houses was attributed to heavy roofs, insufficient bracing or none, inadequate connections, particularly of the exterior walls to the frames, too few walls, poor layout, lack of ties to foundations and to deterioration. Buildings with adequate timber bracing merely lost the cladding. Some bracing however was inadequately nailed and broke loose. Buildings with light roofs and without bracing collapsed. It was evident that the ends of timbers had sometimes deteriorated badly and mortar contributed to the deterioration. The Report, acknowledging the benefits from heavy roofs in other circumstances, rather than recommending their replacement suggests the provision of more bracing.

4E3. Reinforced concrete structures

(1) *Code requirements for RC structures*

Buildings designed to the present code were badly damaged but did not collapse. In the older buildings column reinforcement buckled because before 1971 the transverse reinforcement (sometimes of rather small diameter) was at a spacing of 200 to 300mm. Further it was not well anchored into the column core as the requirements were for 90° hooks which were provided with extensions of 50 to 100mm, compared to the present requirement of 135° hooks and longer extensions. Some of the reinforcement was not even hooked.

Since 1970 deformed longitudinal reinforcement has been mandatory and deformed transverse reinforcement has been used since about 1977. Crushed stone aggregate has been used since 1975.

The gas pressured method of butt welding reinforcement in which bars are fused together under pressure (causing mushrooming at the splice) has been shown to produce connections which are prone to fracture.

(2) *Performance of newer RC buildings*

Buildings designed to the present code were badly damaged but did not collapse. A building with an open ground floor and with external links at 100mm pitch suffered flexural failure at the base (probably aggravated by axial forces) and the links opened. However in a four-storey apartment block with columns in an open ground floor storey collapsed despite external links at a pitch of 110mm.

In one relatively new high rise apartment, shear and bond failure occurred in the ground floor columns, with shear cracks and concrete crushing in the beam/column joint.

In a notable case with columns reinforced by external and internal (diamond) links at 100mm pitch, which was monolithic with a wall one side, the columns suffered severe shear failure over a length of 2.5 times its breadth and contained the concrete within the link cage. The shear crack in the wall propagated through the column, which was unable to bend and relieve the shear. In terms of EC8 the column could be regarded as a shear wall boundary element.

In one old six bay by four storey building in Konan University, all the ground storey columns except the corner column had one-way shear cracks and there was similar damage but in fewer bays in the next two storeys.

(3) *Joint and beam damage*

Few beam and joint failures were observed. This was partly because columns bore the brunt of the damage and partly because there was very little internal inspection.

In one building less than 10 years old, the joint was damaged either side of the column with clear signs of moment reversal. Whilst there were shear cracks in the column they were not pronounced. In another building bond failure of the longitudinal reinforcement in the beams was noted.

In one ward it is recorded there was no significant damage to beams, though in another shear compression cracks were noted in beams in a number of RC and SRC apartment buildings. In a gymnasium pure shear cracks had developed at the very ends of beams, suggesting low concrete strength, inadequate transverse reinforcement or direct tension. The last is considered the most likely.

4E4. Steel buildings

(1) The older steel buildings were predominantly of one or two bays wide and three or four storeys high. Braces often failed in the longitudinal direction which was attributed to the small number of walls in that direction.

(2) A number of buildings with cold formed light gauge lattice columns failed in the ground storey by flexure, overall buckling, buckling or fracture of the component rods (or angles) and fracture of the welded joints (perhaps the most common failure mode).

(3) Often the lattice beams and columns were connected by large imperforate end plates, when failure was commonly due to failure of the bracing, after which storey sways of 15° or more might occur or they collapsed. In one three storey braced building with lattice beams small H columns buckled in the ground storey and partially collapsed.

(4) Bracing often yields before the frame and this may lead to buckling or fracture. Cases were noted with H section X-bracing in which the bracing buckled, broke at the bolt holes at the central gusset and buckled unstiffened gussets welded to the columns which induced weld failure.

(5) Flats which were often connected by single bolt connections tore at the bolt holes adjacent to the frame or at the central gusset.

(6) In a number of apartment and office buildings deep I section beams were butt welded to H section or RHS columns by short web plates (called "diaphragm plates") and the flanges were unattached. The welds often failed and sometimes the bolts.

(7) Five to seven storey apartment buildings with H section beams and columns, with stubs welded to the columns so that bolted connections were well away from the joint, suffered large inter-storey drift on the lowest storey which sometimes collapsed. The failure mechanism is unclear.

(8) In fully welded construction with I beams and RHS columns the beam flanges were often welded to the flange plates through the columns. These failed at the butt weld between the beam flange and the flange plate and also at the welds between the web plate and the column.

(9) Weld failure sometimes resulted in large interstorey drifts and in one case the upper part of a building overturned.

(10) In one building the tubular columns fractured at welds at first floor level and the structure above collapsed.

(11) All the floors in one steel building of H section beams and SRC columns collapsed about the minor column axis which was partly attributable to fracture of the bracing bolts.

(12) In a modern 14 storey (60m) steel building residual storey drifts of 1% occurred in the middle and upper storeys and precast concrete panels on the perimeter walls had cracked.

(13) In one building the columns broke at the lower flange plate weld and moved by a metre or more without causing collapse of the upper floors. Such failures were only visible when displacements were sufficient to dislodge the finishes.

(14) In another building buckling of an RHS column occurred at the base of a tall open ground floor where the moments were high.

(15) Where the welds did not fail the lower flange of the I beam suffered local buckling.

(16) In one area low rise buildings suffered on account of the small size of the columns. In one building there were 100 x 100 H section columns.

(17) In truss type structures diagonals buckled and were dislocated.

(18) In a long span (230m span) triangulated grandstand roof of CHS tubes, some tubes had buckled, some had pulled off the supports (and were hanging) and fractures had occurred where they were welded at internal nodes.

(19) In one building with RHS columns there were clear signs of uplift where the plinth had failed.

(20) In another building anchor bolts had pulled out by 100mm but the columns had not been displaced.

(21) In Nada ward some relatively new buildings collapsed due to insufficient anchorage of base plates to the foundations and some anchors pulled out.

(22) In braced two storey apartment buildings columns had lifted off their foundations due to lack of anchors.

4E5 SRC buildings

(1) This is the first earthquake in which SRC buildings have been damaged. There were many failures, particularly of SRC members with built up steel sections fabricated from angle lattices or angle ladders, which was attributed to the low shear strength of such members. In 1975 full webs were required in the design rules and have been used since 1980.

(2) One unidentified SRC building complying with the latest requirements developed plastic hinges and large interstorey drift.

(3) In one ward the only damage in an SRC building was broken windows and cracks in the walls of the first storey.

(4) A three year old 13 storey SRC apartment building in E Nada ward with shops at the two lowest levels was severely damaged.

(5) In some SRC members the steel section fractured and in others diagonal shear cracks and spalling occurred at joints.

(6) Severe damage of non-structural partitions and jamming of doors occurred in many buildings.

(7) In SRC construction with RHS bracing failure occurred at the end gussets with X bracing and at the beam gusset with chevron (inverted K) bracing, when the lower flanges were distorted. SRC bracing also buckled on loosing the casing due to lack of reinforcement.

(8) "Non-embedment type" steel bases performed badly by crushing of concrete or pull-out of the bolts and the design criteria need to be reviewed. One building shifted sideways. In 1987 an alternative detail with the steel section anchored within the foundation was recommended.

(9) In an 11 storey SRC building the concrete had crushed beneath the base plate and the links had split despite ample longitudinal reinforcement ,which tied the column to the plinth beneath the base plate, had buckled.

4E6 Geometrical effects

Some of these observations relate only to RC and SRC buildings.

(1) The high proportion of buildings failing in the lowest storey was attributed to the absence of shear walls in that storey.

(2) However a building with a masonry clad perforated shear wall suffered X cracking in the columns at all the levels above the undamaged ground floor, where the storey height was appreciably greater. The damage was described as non-structural.

(3) Short columns at a mezzanine floor in apartments above an open ground floor suffered shear cracks. In another apartment building with clerestory windows in the storeys above a normal ground floor the columns failed in the second storey.

(4) Many low rise buildings which were long, but only one or two bays wide, had adequate transverse walls but failed longitudinally due to the inadequacy of walls in the direction.

(5) Long narrow buildings tended to collapse in the middle.

(6) Corner columns at the intersection of streets suffered particularly badly due to the L shaped layout of the adjoining buildings.

(7) A building on sloping ground failed. The columns in the lowest storey had been tapered to accommodate the fall and the shortest columns failed in shear.

(8) In a one year old nine storey apartment building with an eccentric wall layout the isolated columns failed.

(9) In Chuo ward in two buildings with elevated corridors between blocks at all levels, those at the higher levels only collapsed. The roof of a mall between two rows of buildings collapsed.

(10) In one ward a number of RC buildings, in the vicinity of a hospital with a sixth storey collapse, suffered collapses at the higher levels.

(11) Damage associated with setbacks was noted as follows:

   (a) In an SRC building the bottom of a corner column above a setback was damaged, though the links at 200-300mm pitch did not buckle.

   (b) In another building damage occurred to the joint at the floor above the setback revealing a lack of transverse reinforcement.

   (c) In a large 24 storey retail/apartment building, which appeared undamaged on the outside, walls had sheared and doors jammed in the fifth storey setback.

   (d) In another building column failure occurred at a setback above the third storey.

(12) Damage at roof level is caused by pounding against adjacent buildings of different height.

(13) Damage associated with external staircases was noted as follows:

   (a) In one block the external staircase separated from the building and the two structures pounded against each other causing shear cracks in the building columns.

   (b) Severe horizontal cracks occurred in the external lift shaft of a pre-1981 building where it was tied into the floors.

   (c) An emergency steel staircase had separated from an apartment block due to failure of the anchors.

(14) Mid height collapses were caused by changes in construction (SRC to RC), the incorrect distribution of inertial forces in older design codes and vertical acceleration which reduced both ductility and a shear resistance. Resonance between the ground and the structure, large storey drifts and local softening all may have contributed to

this failure mode. Often the shear resistance does not match the shear, which is aggravated by variation in storey heights and plastic hinging.

(15) Deep spandrel beams caused single storey shear failures.

(16) Isolation of non-structural partitions is recommended to achieve a more uniform distribution of response from floor to floor, but this may be beneficial only in ductile construction.

4E7 Details of failures in steel buildings

A deeper insight into the performance of steel buildings is provided by Reference. 4.36, from which the failure of members and connections are scrutinised.

From photographs of the failed wide flange beam to box column connections it would appear failure in or near the beam weld was the most common failure mode with failure generally initiated in the lower flange, presumably due to incidental composite action with the slab protecting the top flange. In a few beams the lower flange had began to deform plastically sufficiently to flake the paint for a distance of about 150mm from the column, indicating a degree of overstrength in the weld. The rotation for a hinge of this length however is unlikely to absorb much energy. On one beam however significant plasticity over a length of 250mm or more had occurred with the lower flange width reducing by 15% over 75mm at the column face. Local flange buckling was common and was often accompanied by significant plasticity. The second most common failure mode was failure beneath the through column diaphragms on the underside of the joint. A few cases were noted of the column flange cracking on a rectangular perimeter around the beam.

Sometimes the RHS columns buckled. The worst cases were generally in plated I section box columns in which the plates delaminated along the vertical welds and folded, as in Mexico City in 1985[(4.37)] and there was a notable buckling of an entire box section in which two opposite sides buckled inward and the remaining sides buckled outwards without any cracks.

Of particular interest to European engineers is the failures between wide flange columns and wide flange beams in which the connections were fully welded. The incidence of beam column weld failure, beam-to-beam bolted splice failure (near point of contraflexure) and buckling of relatively thin column flanges beneath the joint were comparable and one case is included of the shearing of a panel zone. Occasionally the column lacked stiffeners (continuity plates) in line with the beam flanges and in one case the web buckled pulling in the column flanges. Beam splice plates tore in the centre and many high strength bolts failed at both column plate and span connections.

To overcome the low flexural resistance about the minor axis of the columns, frames in the orthogonal direction were commonly braced and no instances are evident in Kobe of the direction of the major axis being changed to achieve moment resistance frames in both directions. Interesting failure modes included out of plane distortion of the gusset plates (as noted in 4.9.2) and two cases of web buckling of the beams at the connection of chevron bracing in eccentric braced frames, lacking, or lacking adequate, web stiffeners. The one with stiffeners nevertheless appeared to have undergone significant plastic deformation.

There was no evidence of significant plasticity on the columns above the base plates, though in one case cited flange buckling was probably associated with incipient hinge formation. A few cases were noted of pull out of foundation bolts including in one complete pull out and collapse of the column (as that in 4.5.6)

4E8 A Probable new direction

The papers given at a symposium on a new direction in seismic design[4.36], whilst including the use of better steels in Japan and damage controlled design with brace dampers, were mainly concerned with base isolation. This emphasis is also reflected in Reference 4.18 and it is therefore considered likely that most of the more important building in Japan will in future be base isolated.

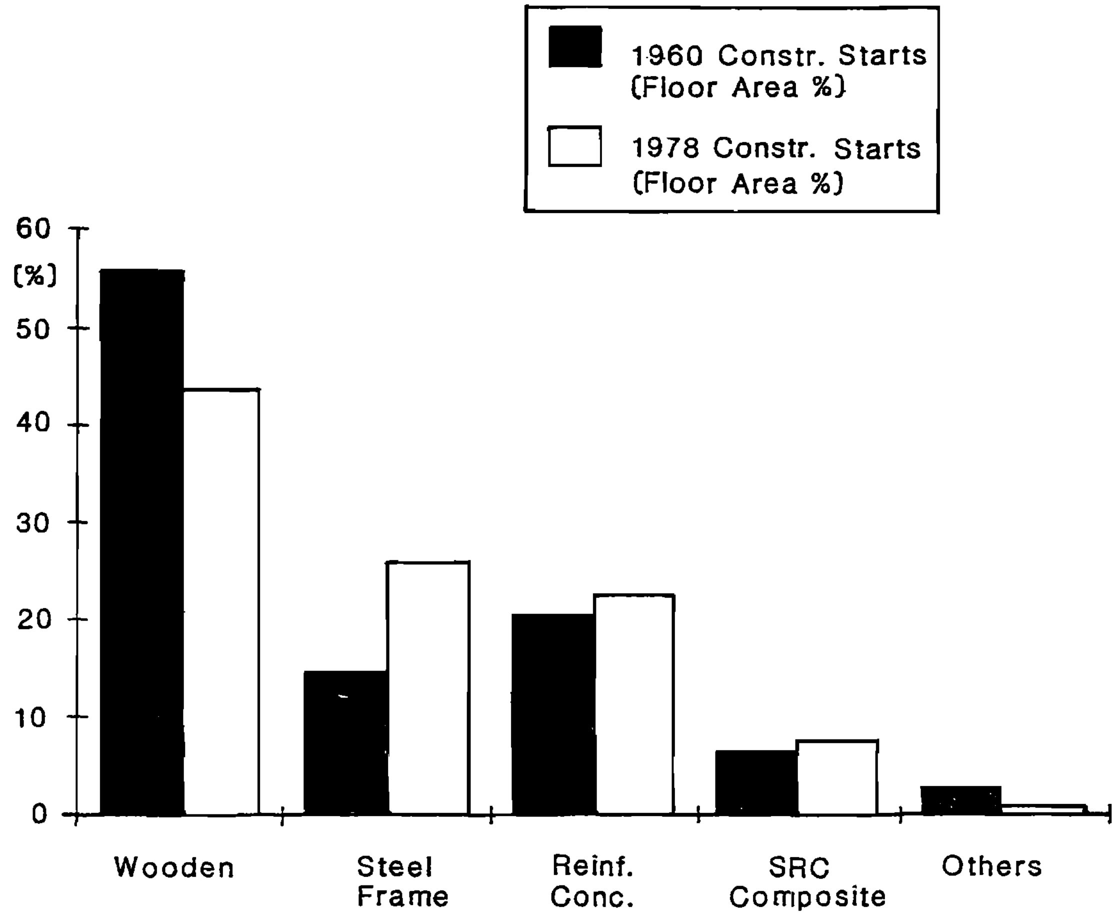
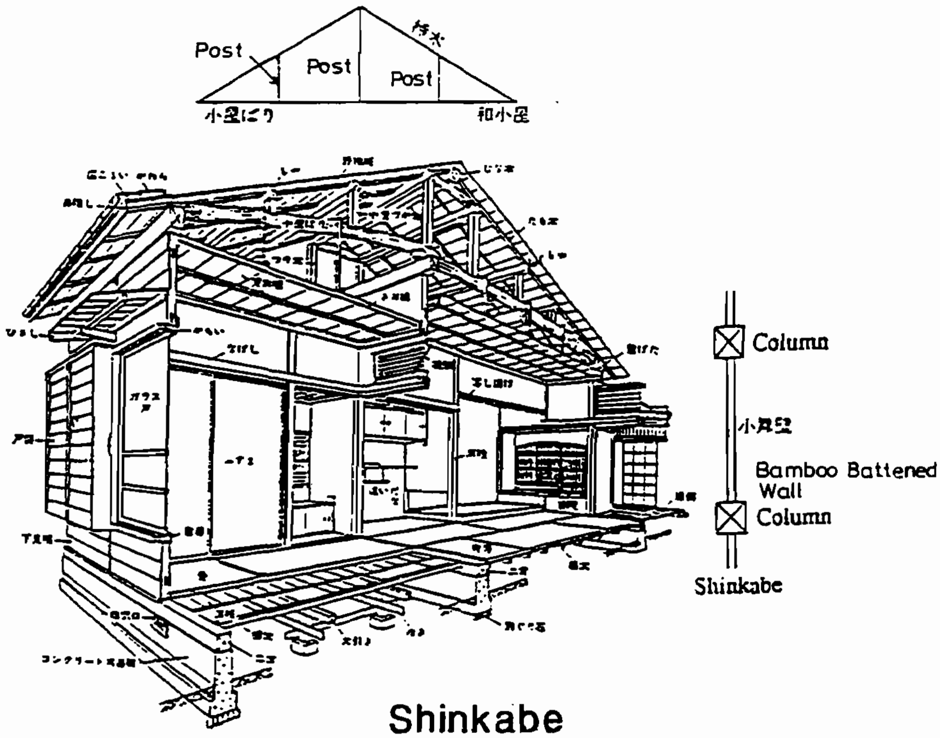
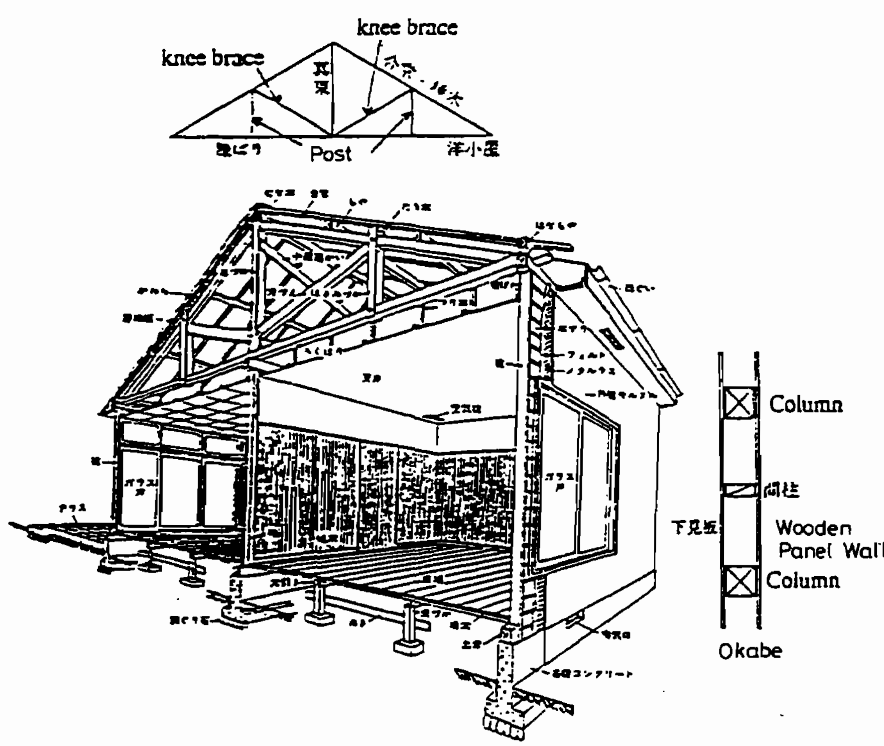


Figure 4.1 Proportion of building starts in Japan in 1960 and 1978, by structural type



Shinkabe



Okabe

Figure 4.2 Wooden Houses

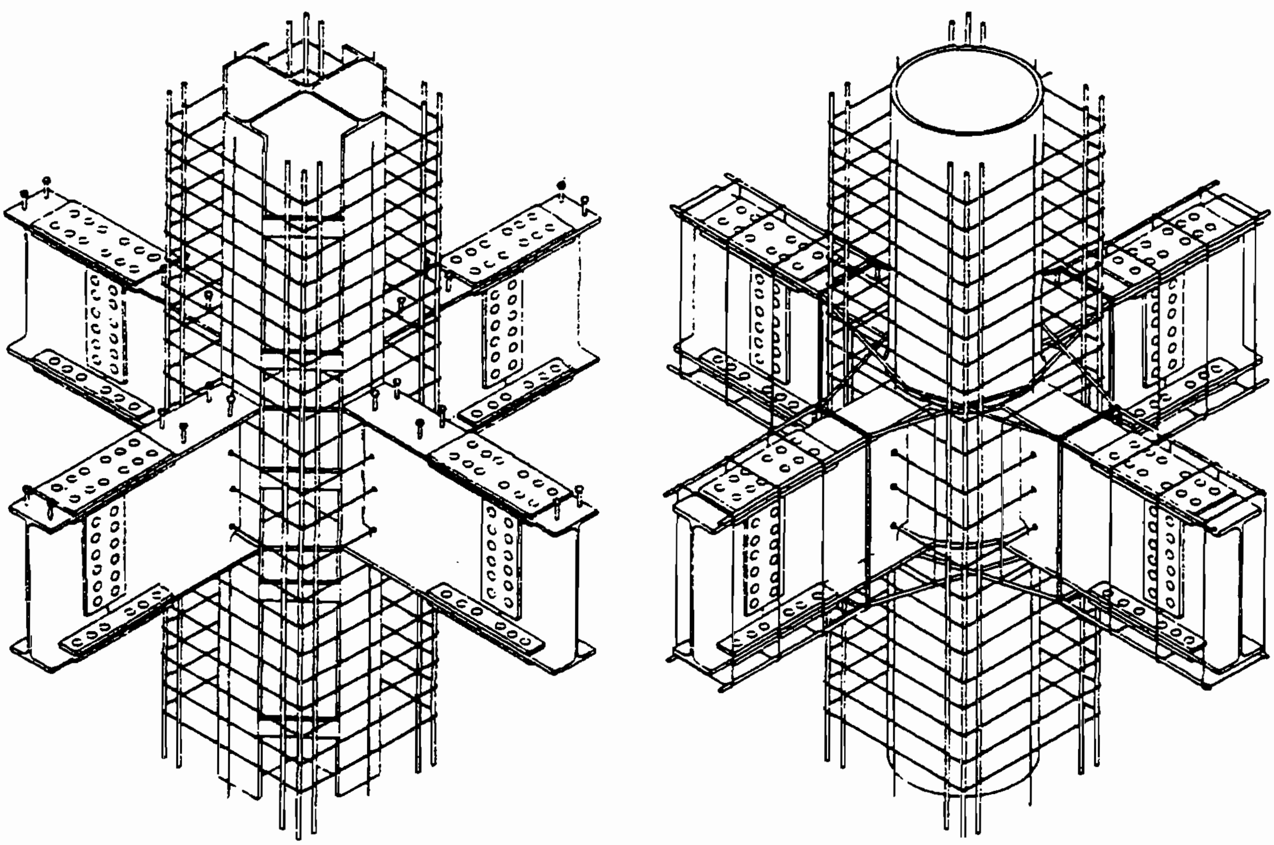


Figure 4.3 Typical beam/column joints in SRC construction

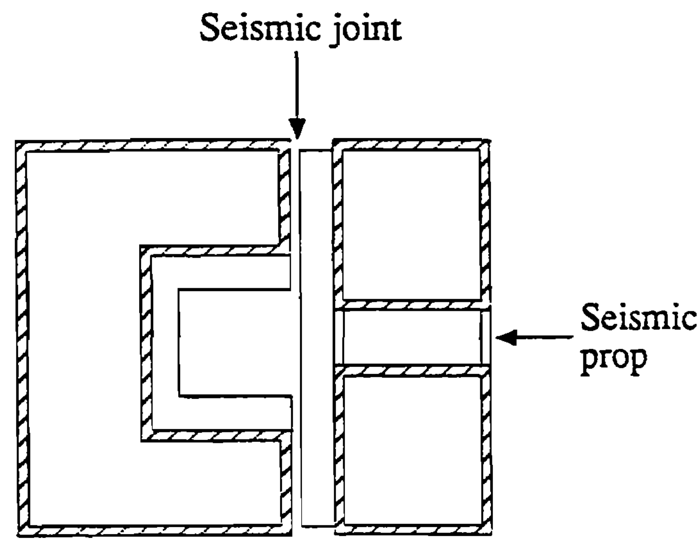


Figure 4.4 Masonry clad shear wall building

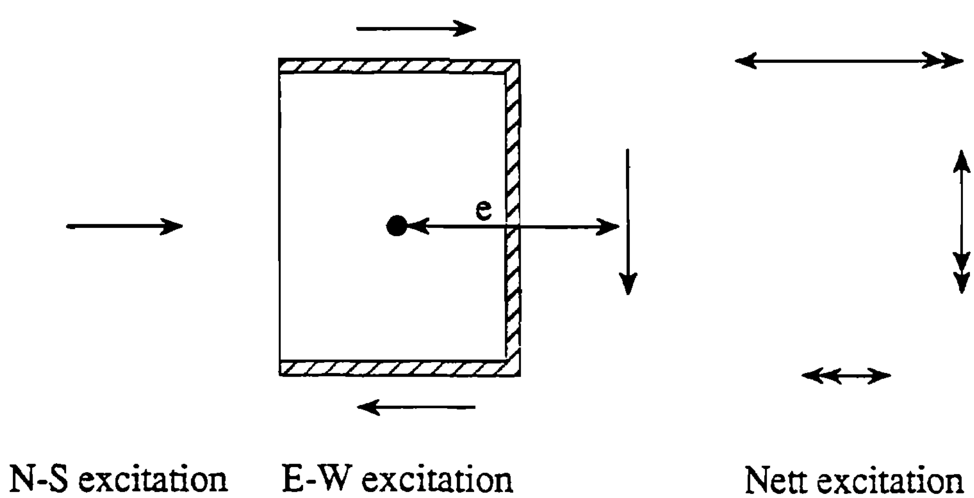


Figure 4.5 U-Shaped CBF framed building

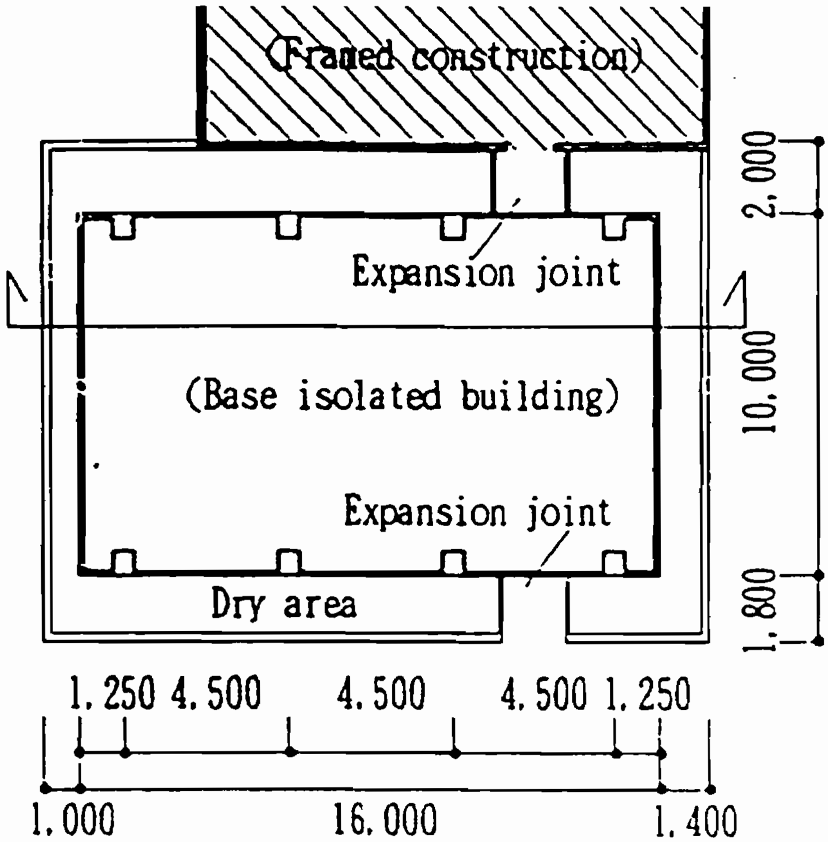


Figure 4.6a A plan of the first floor of the MTRC base-isolated building

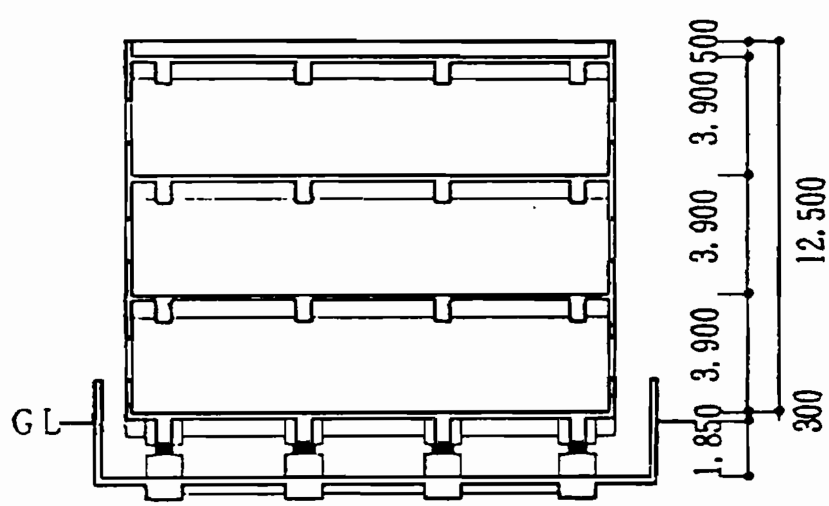


Figure 4.6b Cross-section through the MTRC base-isolated building

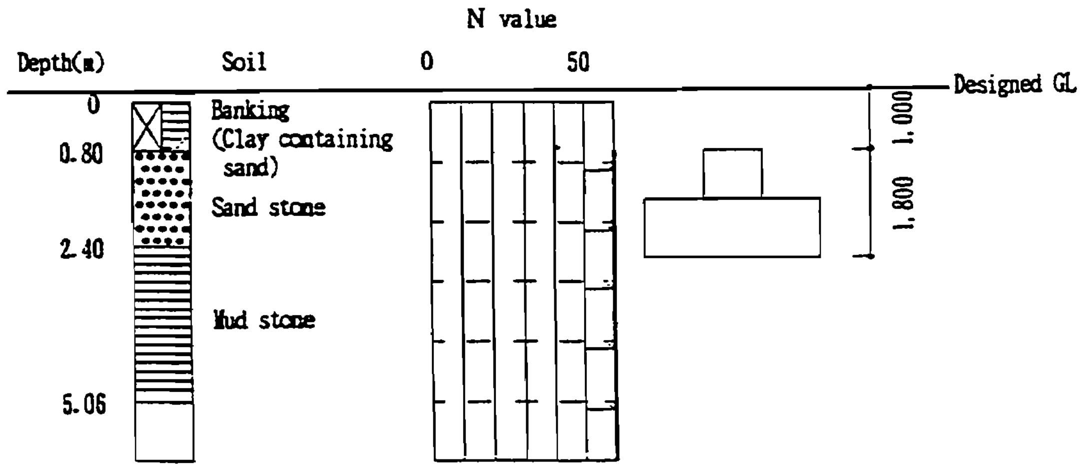


Figure 4.7 Soil profile of the MTRC building site

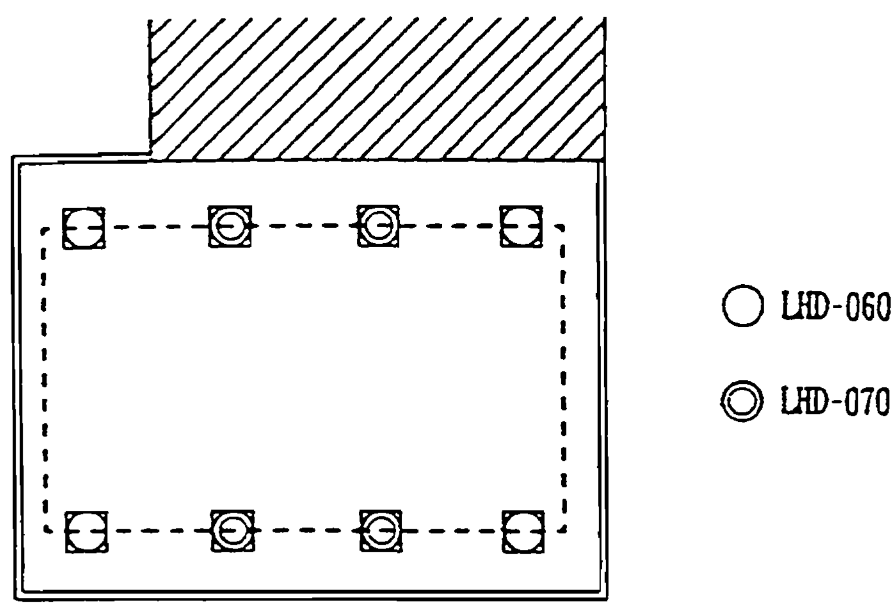


Figure 4.8 Arrangement of the isolators in the MTRC building

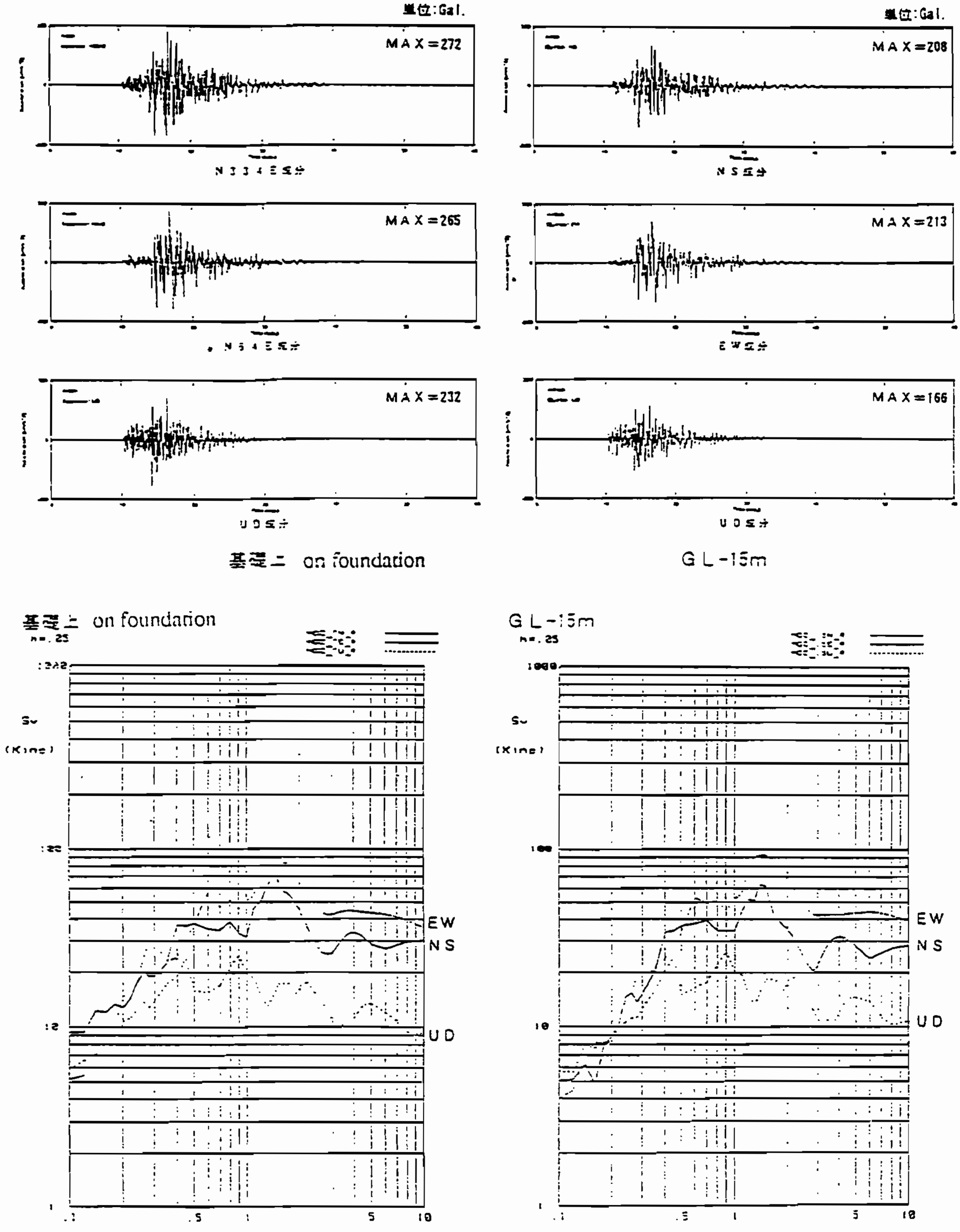


Figure 4.9 Ground motion records and velocity response spectra at MTRC building

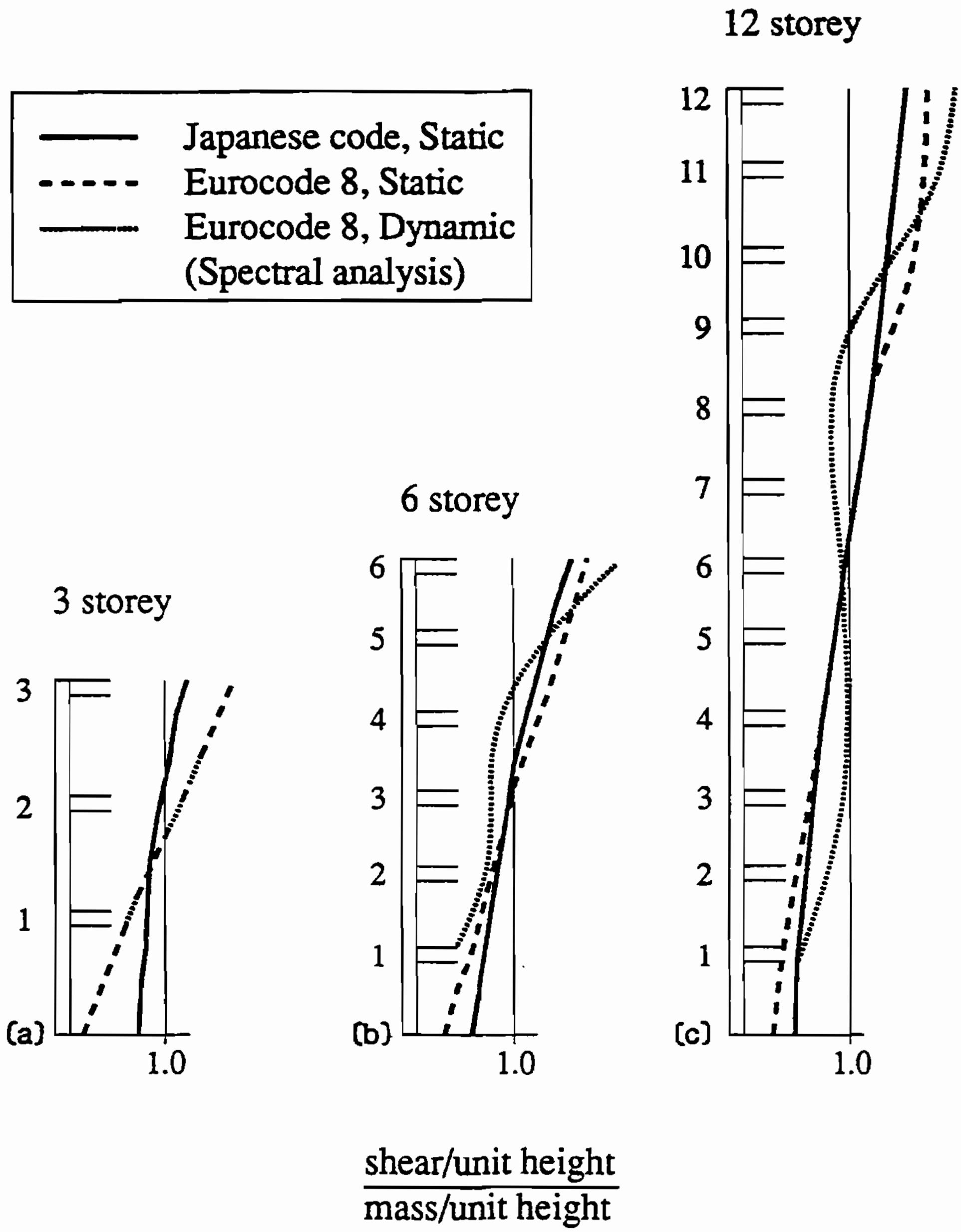
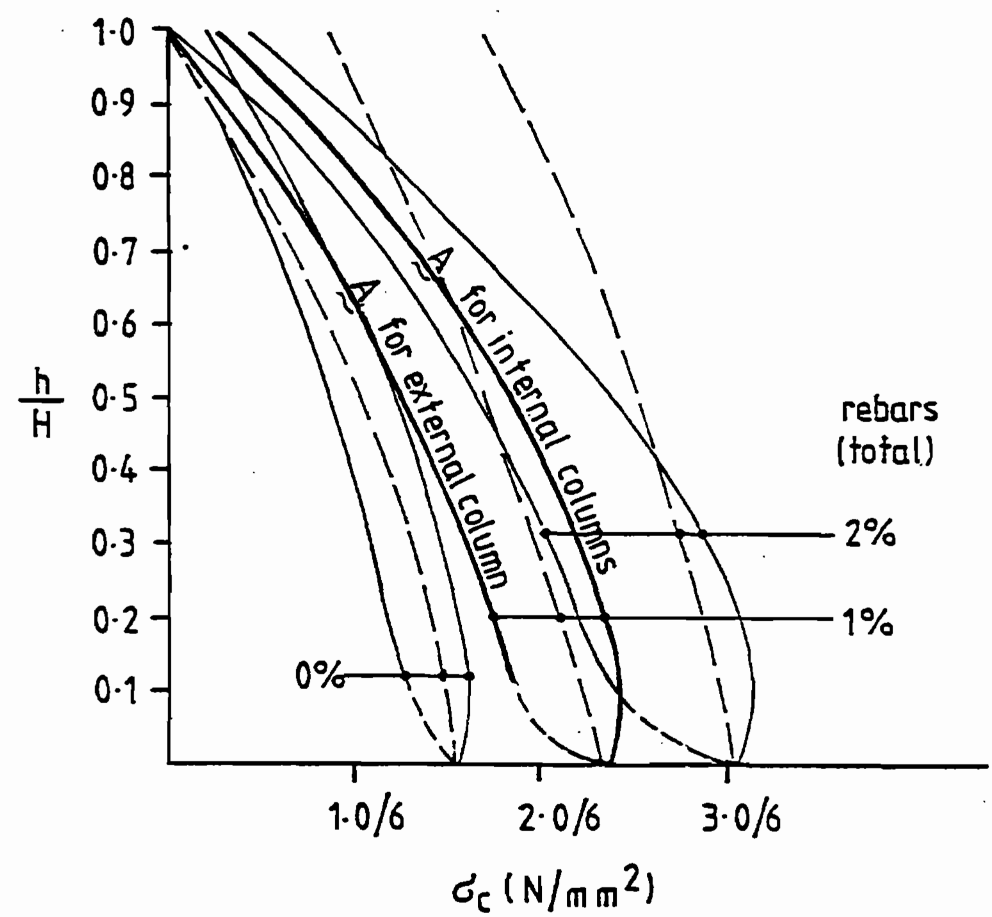


Figure 4.10 Comparison of distribution of lateral loading in present Japanese design and Eurocode 8



Flexural stress on gross section

Figure 4.11 Action effect & resistance distribution with columns of constant cross-section

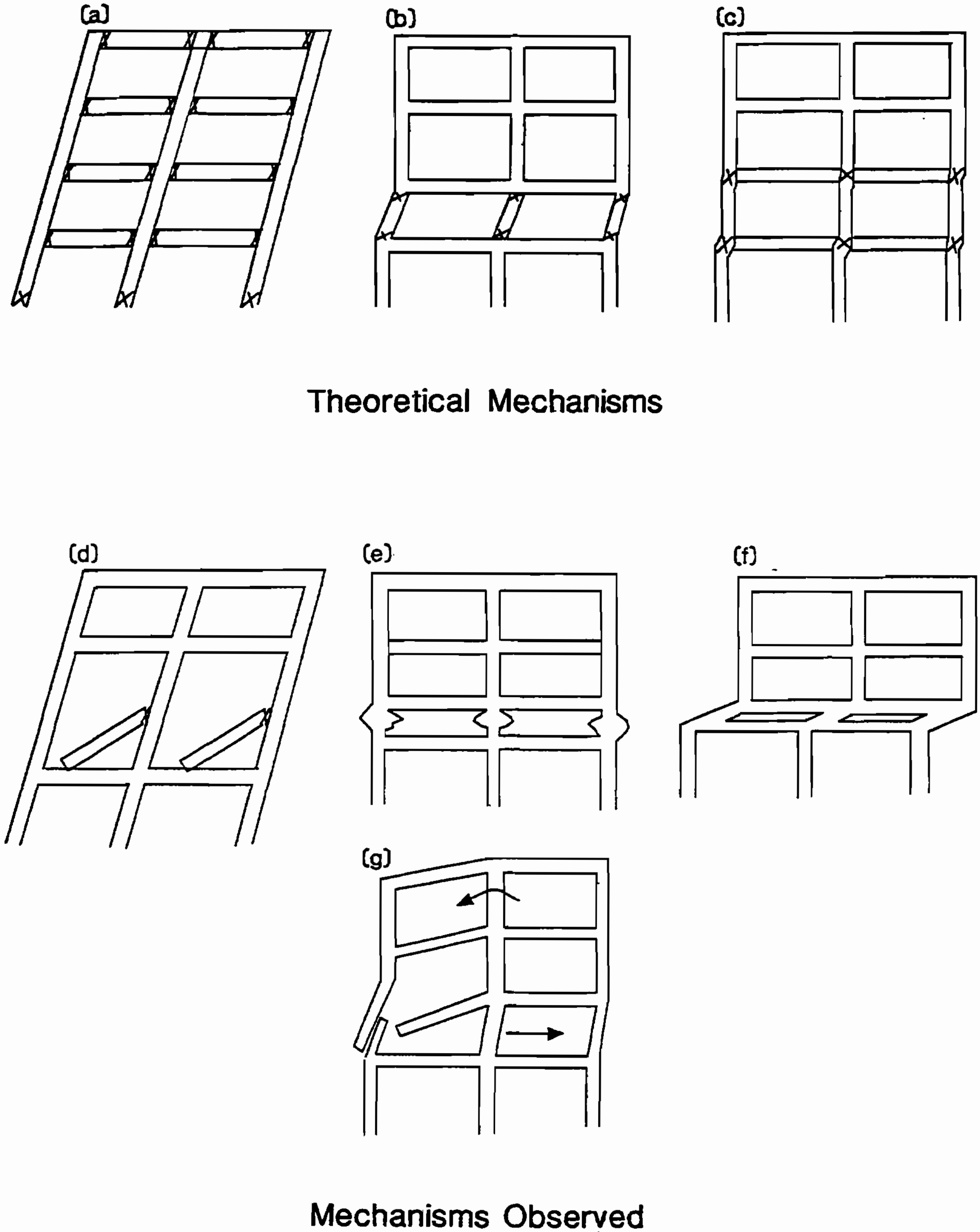


Figure 4.12 Theoretical frame mechanisms and mechanisms observed

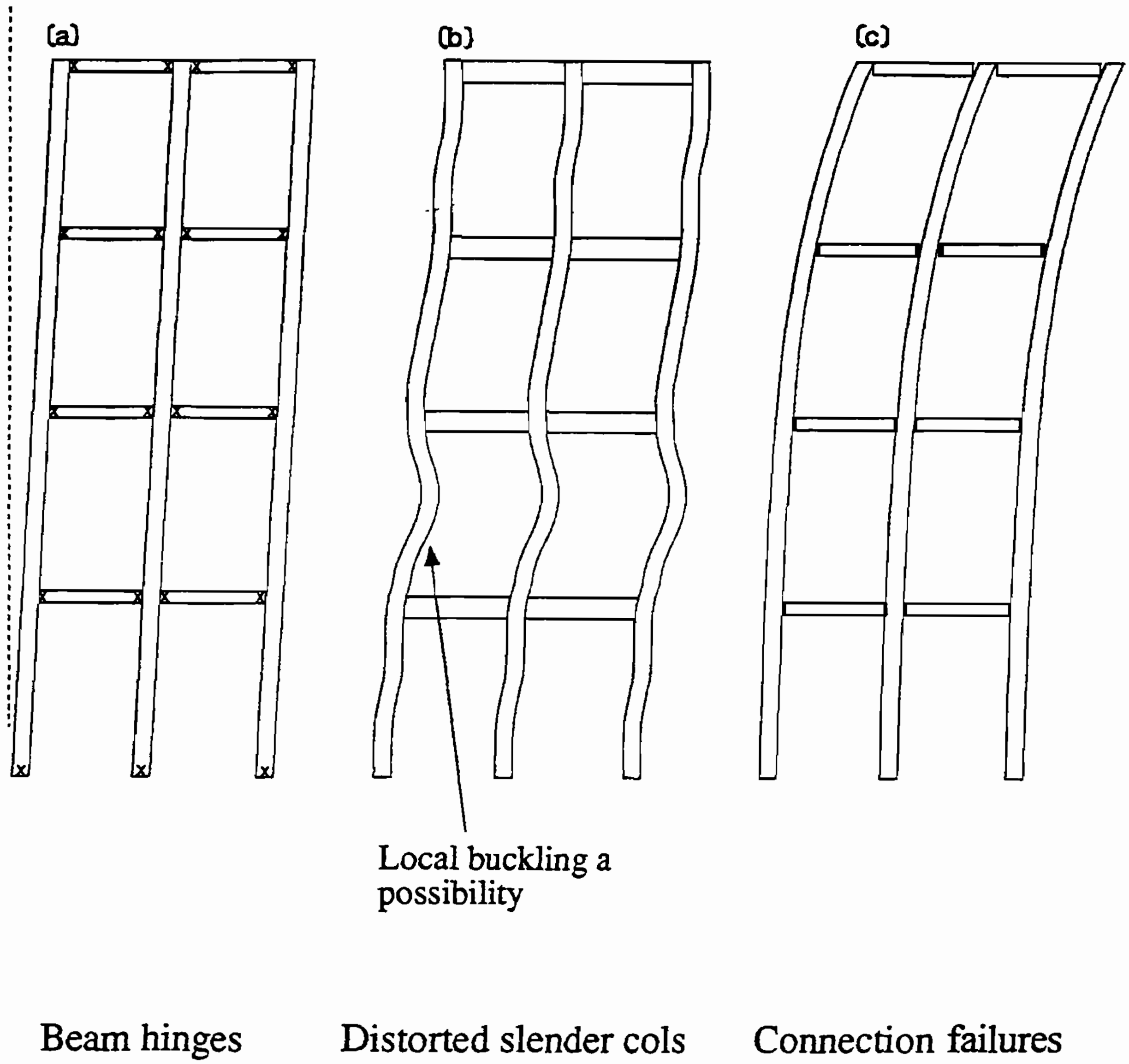


Figure 4.13 Deformed shapes of damaged buildings

Plate 4.1: Kobe Immigration Office

Plate 4.2: RC Mullion damage

Plate 4.3: Internal RC column damage

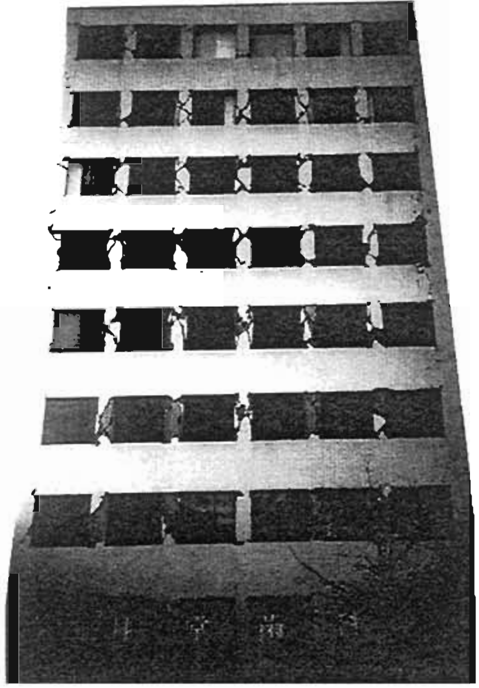


Plate 4.4: Distributed RC mullion damage due to minimal damage in corner mullions

Plate 4.5: Two RC buildings originally of same height, one with distributed damage and one with collapse in lowest storey

Plate 4.6: Detail of distributed mullion damage in Plate 4.5

Plate 4.7: RC Building with collapse in 3rd storey

Plate 4.8: Bulging facade at corner where building dropped

Plate 4.9: Streetwise displacement at adjacent corner

Plate 4.10: Streetwise tilt of 12 storey frame/shear wall structure

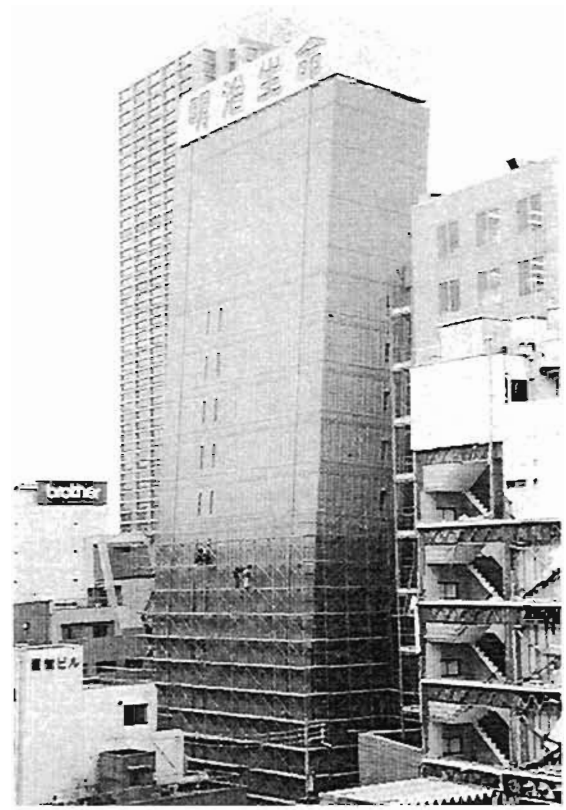


Plate 4.11: Rear view showing shear walls

Plate 4.12: Collapsed part of side shear wall

Detail of damage

Plate 4.13: 2nd Storey collapse in 7 storey corner building

Plate 4.14: 4th and 5th Storey collapse in 9 storey building

Plate 4.16: 1st Storey collapse due to change in column shape

Plate 4.15: 2 Storey collapse in irregular building

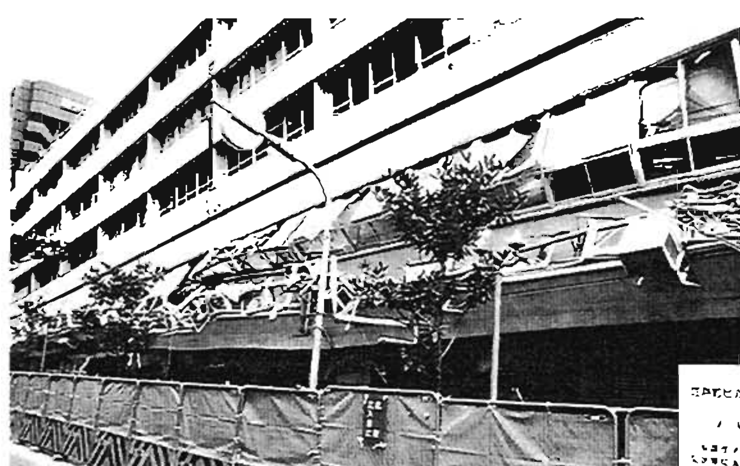


Plate 4.17: 2nd & 3rd Storey collapse in long building

Plate 4.18: Partial storey collapse in 2nd storey

Plate 4.19: Damage where building in 4.18 was torn apart

Plate 4.20: Partial storey collapse due to buffeting

Plate 4.21: Partial storey collapse due to shear wall failure one end of building

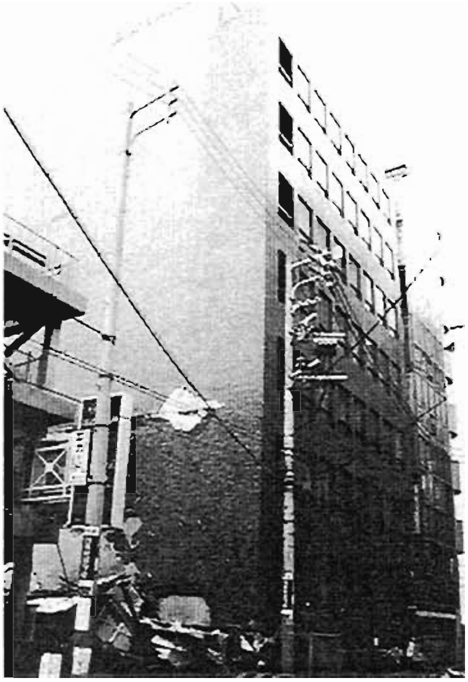


Plate 4.24:

2-way collapse following shear wall failure after buffetting and attributable to lack of longitudinal shear walls

Plate 4.22: Spalling attributable to lack of longitudinal shear walls

Damage at corner

Plate 4.23: Damage to shear walls from separated masonry

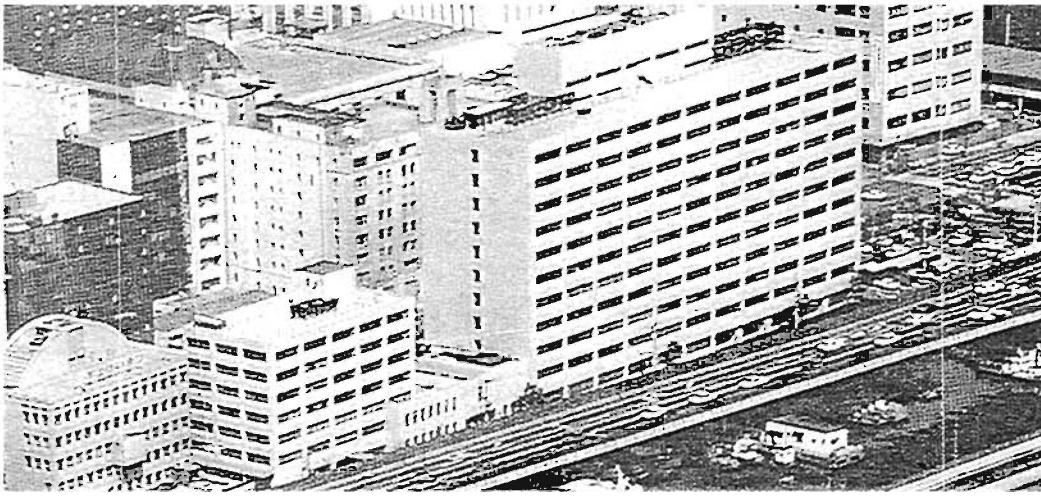


Plate 4.25: 2 relatively new blocks of apartments along the coastal road in Kobe

Plate 4.27: Coupled shear wall building

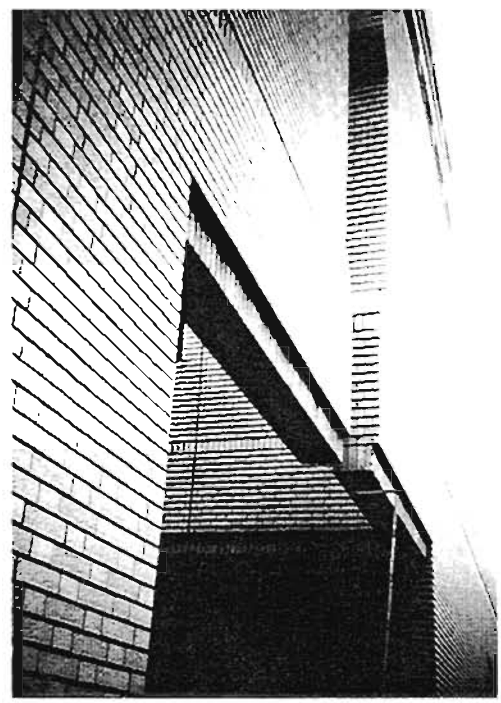


Plate 4.26: Relative displacement between buildings

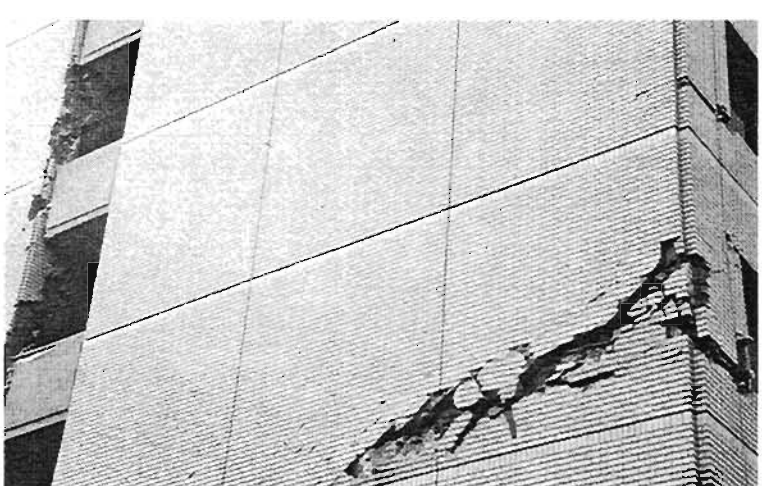


Plate 4.28: Flexural/shear failure in shear wall building

Buckled reinforcement

Plate 4.29: Substantial perforated shear wall building

Plate 4.30: Distributed inelasticity in shear wall structure

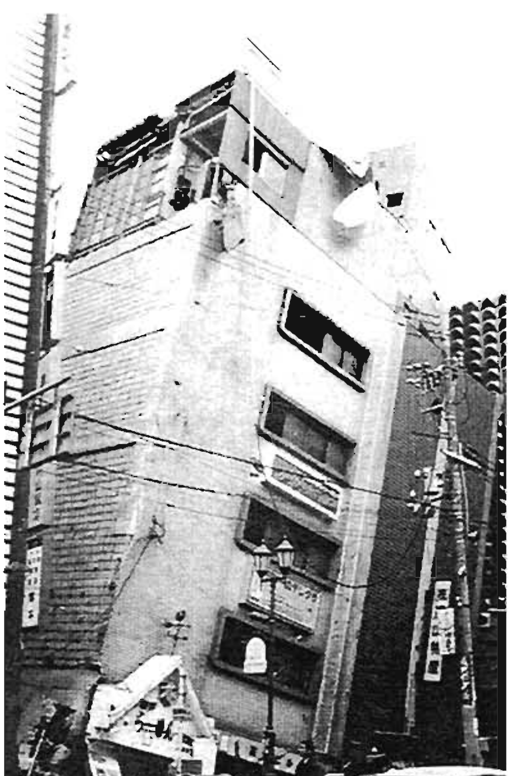


Plate 4.31: Rigid body rotation in one-sided collapse

Other side of building in plate 4.31

Plate 4.32: Shear wall compression failure with consequential shear fracture along vertical axis

Plate 4.33: Totally collapsed perforated shear wall building

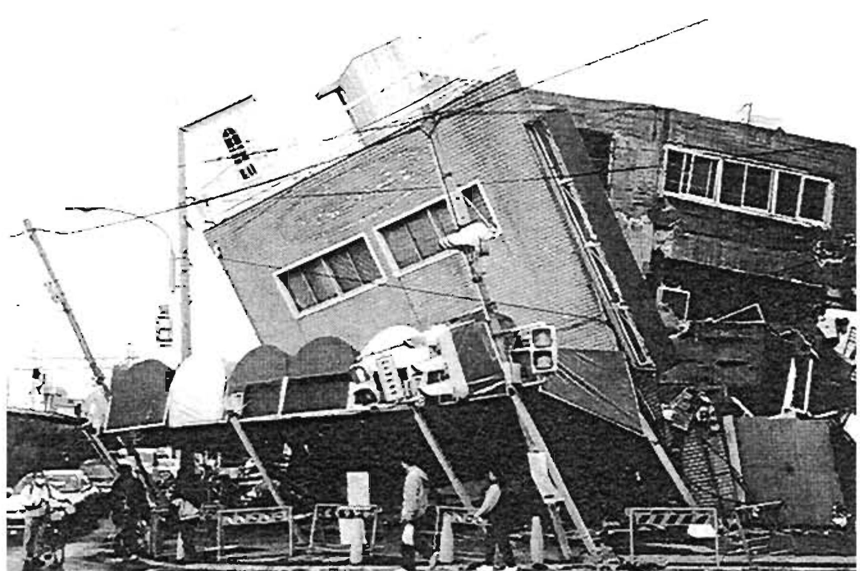


Plate 4.34: Collapsed building with separated shear walls

Plate 4.35: The Sumitomo Bank

Plate 4.36: Damaged connection beams

Plate 4.38: Live electrical duct in shear wall

Plate 4.37: Shattered shear wall

Plate 4.39: The German Consulate

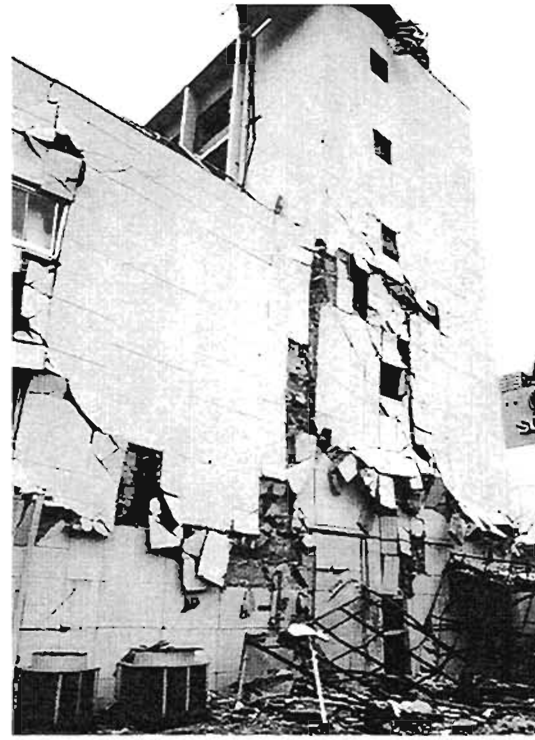


Plate 4.40: Damage at root of tower

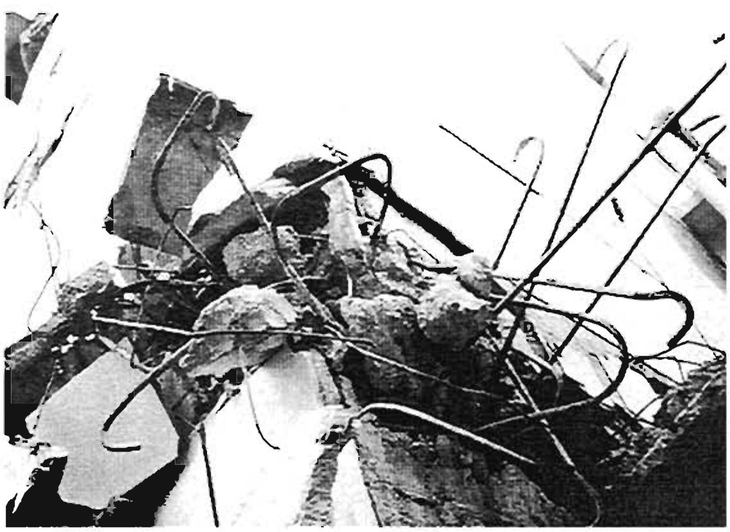


Plate 4.41: Disintegrated corner with reinforced fascia panels

Plate 4.42: Hole in asymmetrically reinforced shear wall

Plate 4.43: Landing beam punched through supporting wall

Plate 4.44: Damage at end of heavily tapered building

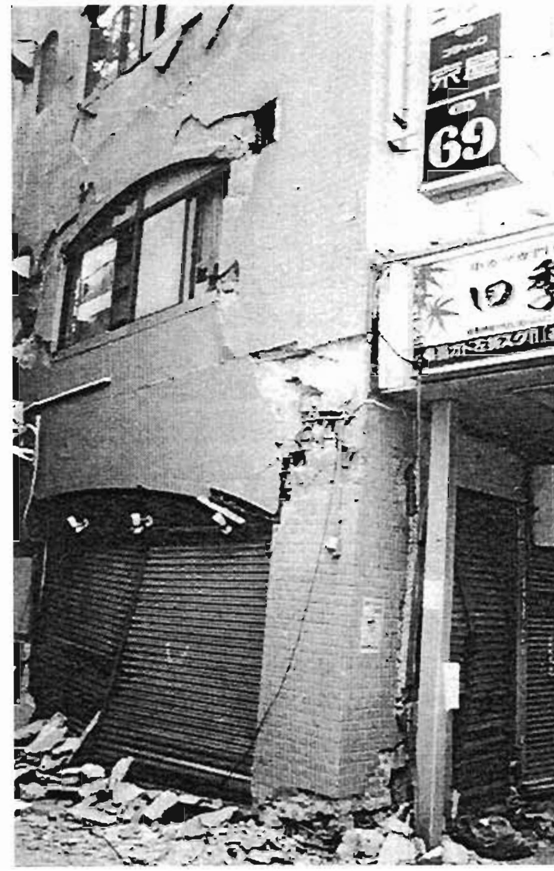


Plate 4.45: Corner joint damage in irregular building

Plate 4.46: The Kwong on Tai Hong Building

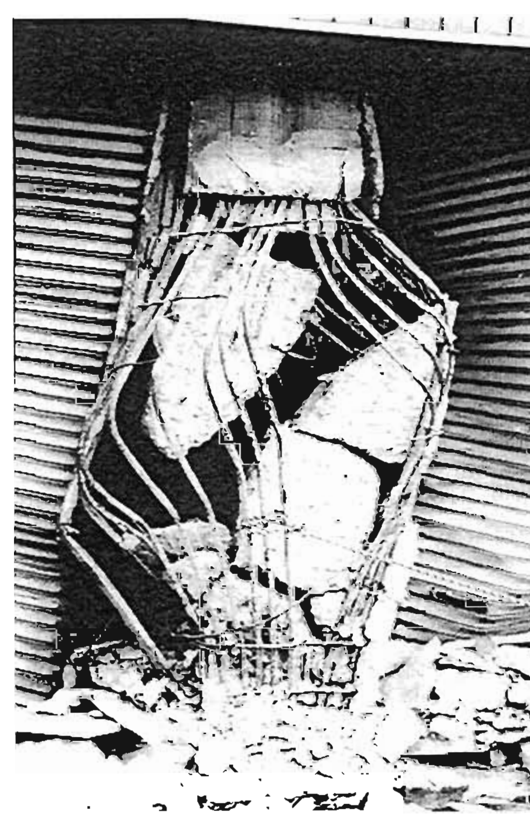
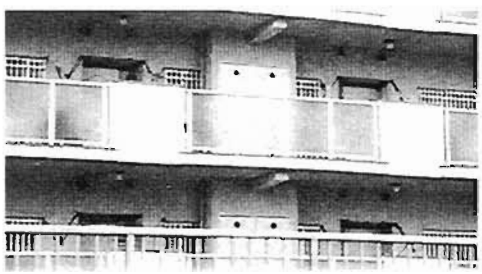


Plate 4.47: Failed off-centre central column

Plate 4.51: Long T-shaped apartments near Takanaka complex

Plate 4.50: Irregular building damaged front and back



Damage between doors and windows

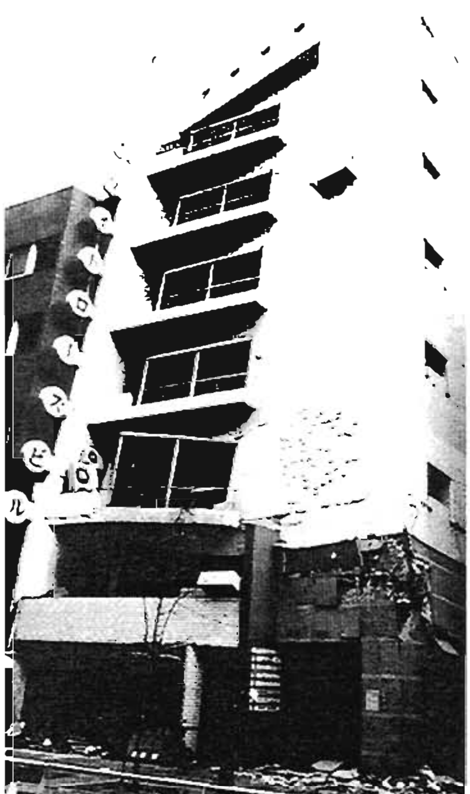


Plate 4.48: Irregular 7-storey building

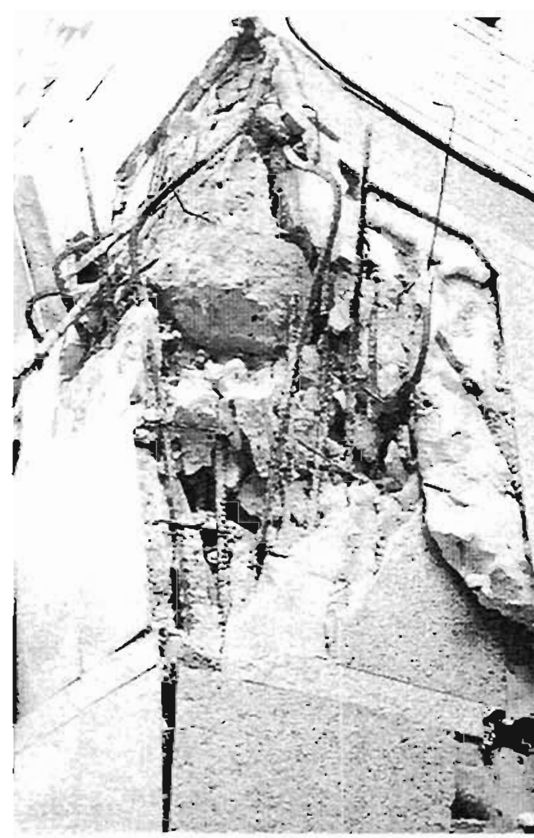


Plate 4.49: Exploded corner column joint

Plate 4.52: Damage in main shear wall of 12 storey flats

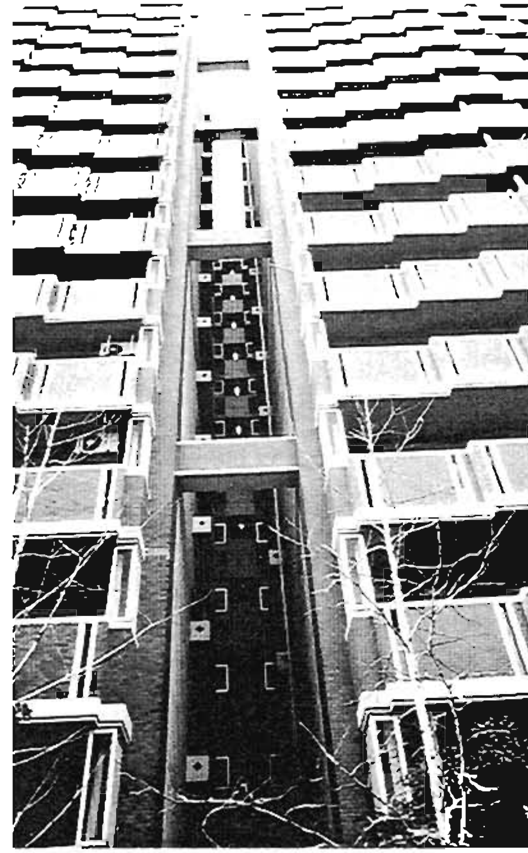


Plate 4.53: Seismic props to prevent diaphragm distortion

Plate 4.54: X-cracking in perforated shear walls

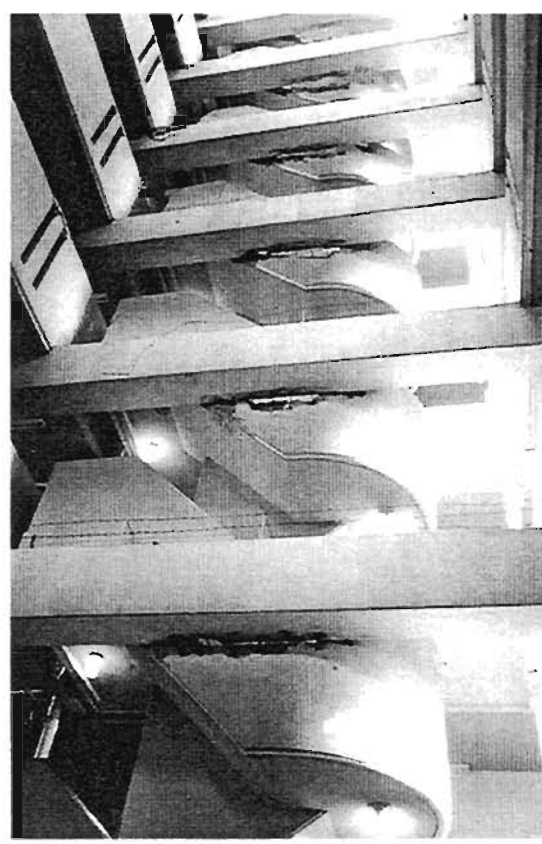


Plate 4.55: Spalling between staircase and stair well cross beam

Plate 4.56: Building with open ground floor

Plate 4.57: Anchorage of ramp to first floor

Plate 4.59: Exploded column top

Plate 4.58: Severed shear wall

Plate 4.60: Link weld failure

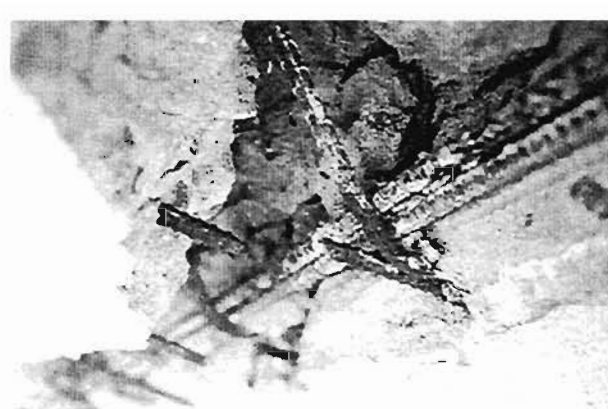


Plate 4.61: Snapped ramp tie

Plate 4.62

Shear crack in first floor joint

Plate 4.63:

Confined
column concrete
- ½ second exposure

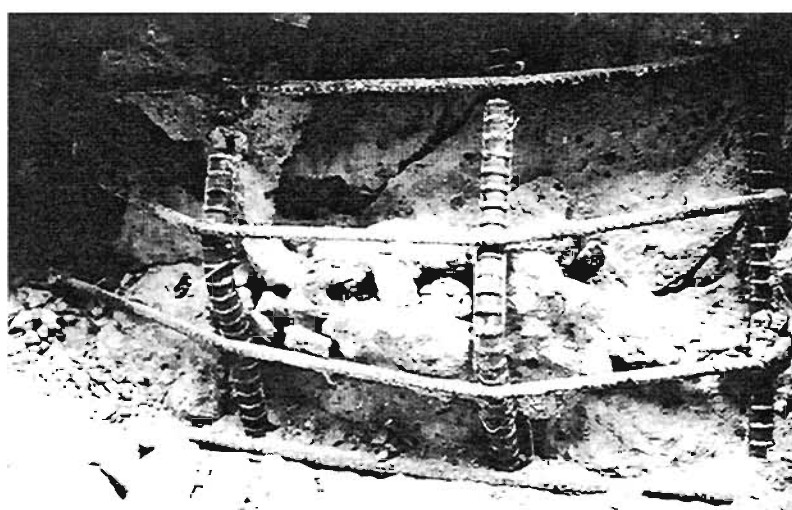


Plate 4.64:

Ditto but by flash

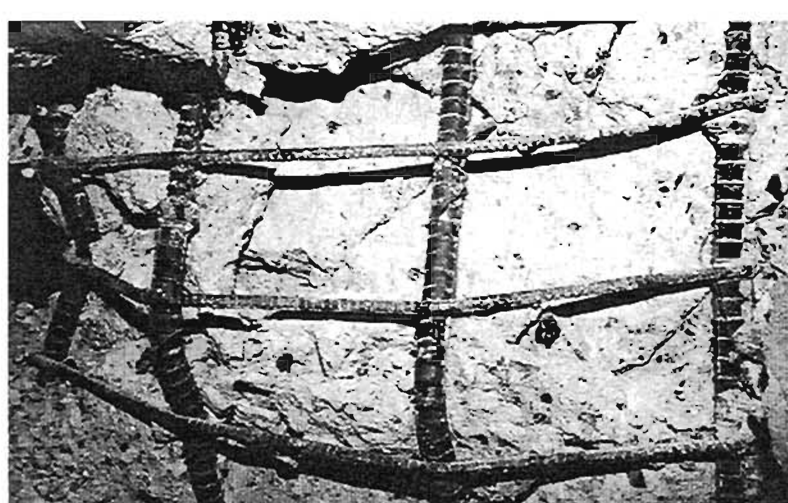


Plate 4.65:

Missing and damaged pre-cast cladding on steel framed building

Plate 4.66:

Missing cladding on older style knee-braced frame

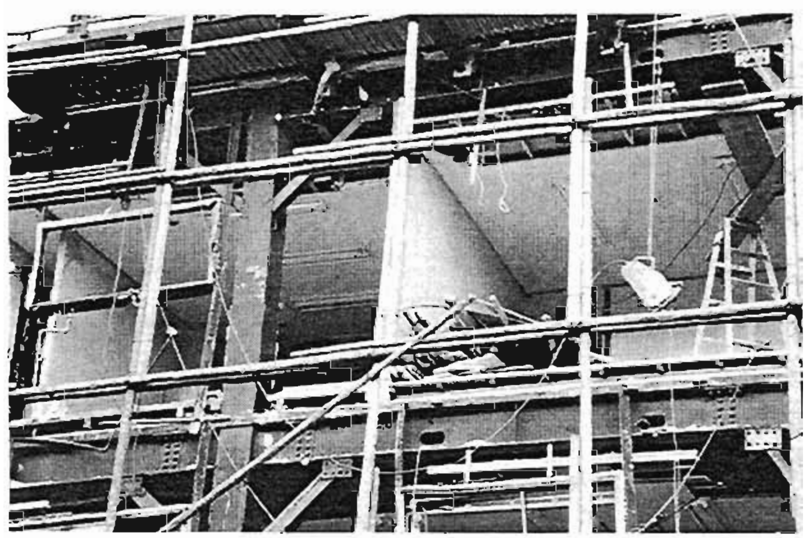


Plate 4.67:

Sway mechanism of bare frame with hinges at web-bolted connections

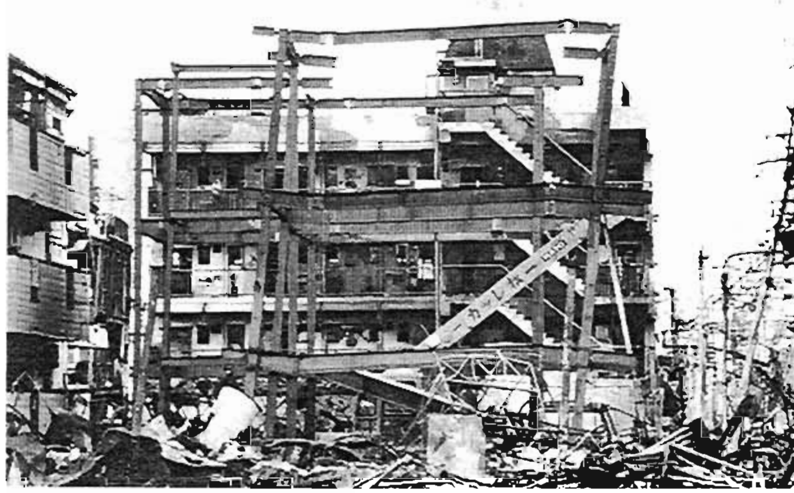


Plate 4.68: Sway after snapping of bracing flats in upper stories

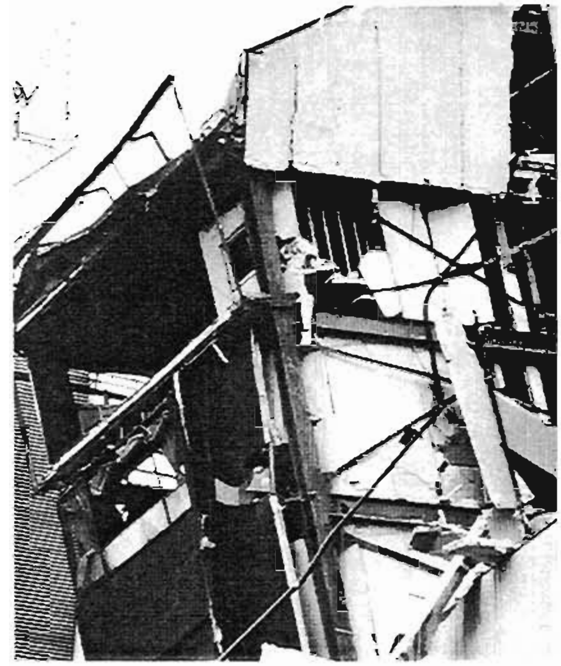


Plate 4.69: Snapped bracing

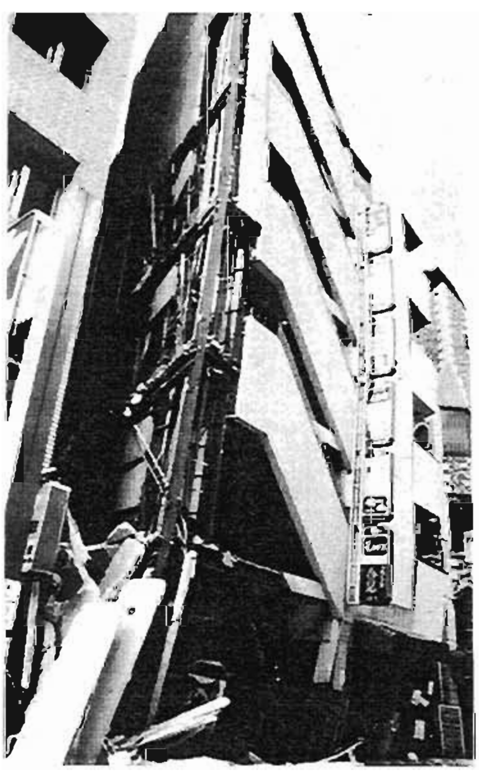
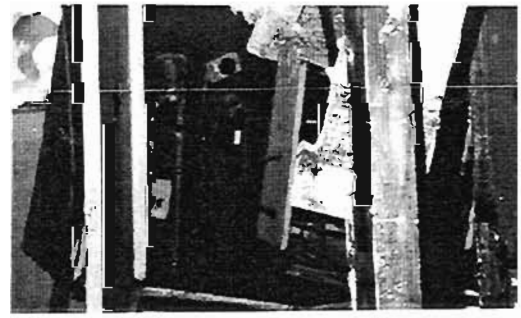


Plate 4.70: Sway after apparent failure

View along street

Plate 4.71: Overturning attributable to inadequate HD bolts



Detail of bolt failure in Plate 4.71

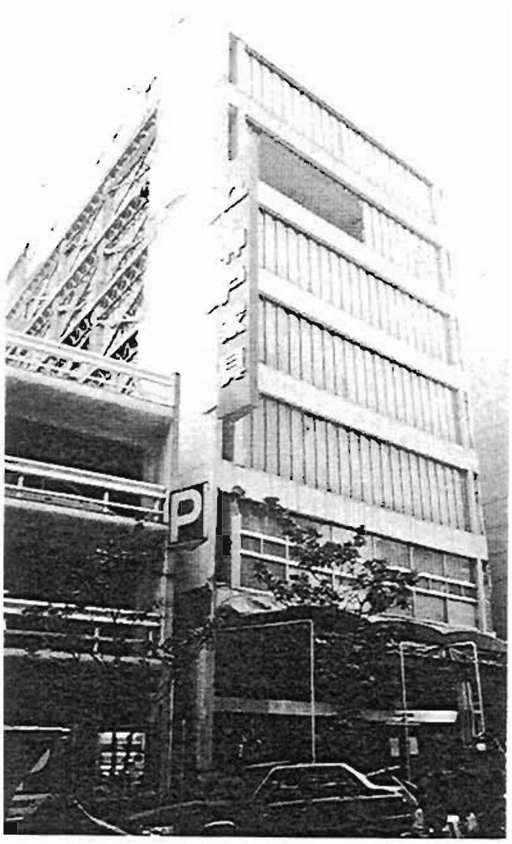


Plate 4.72: U-shaped CBF braced building

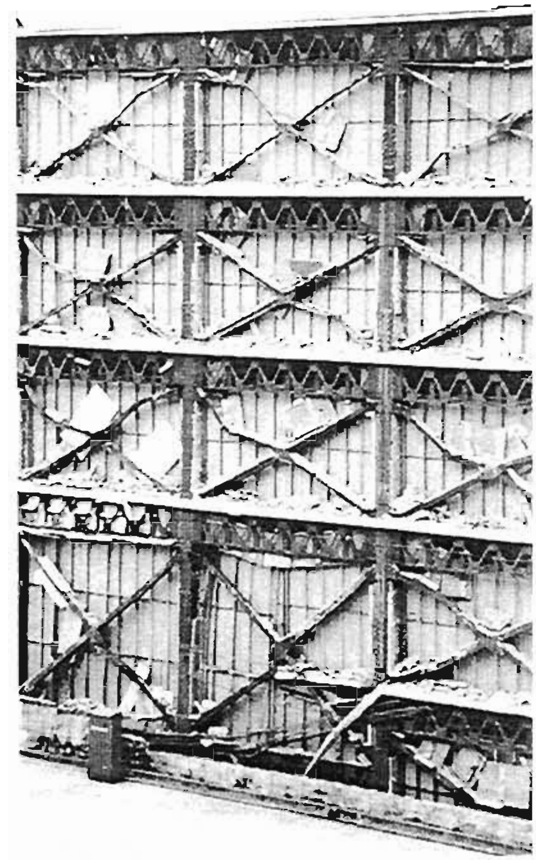


Plate 4.73: In-plane buckling of bracing

Plate 4.74: Damage to cladding on opposite face

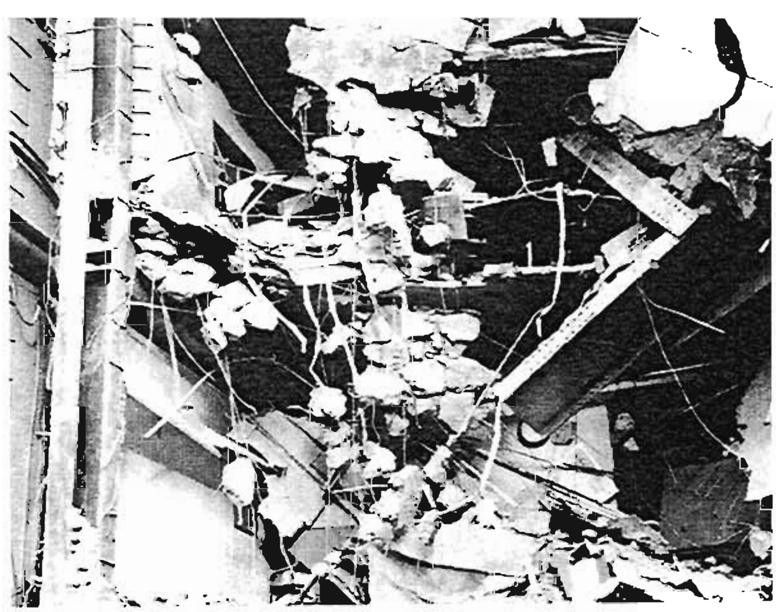


Plate 4.75: Building with failed welds at beam/column connections

More distant view

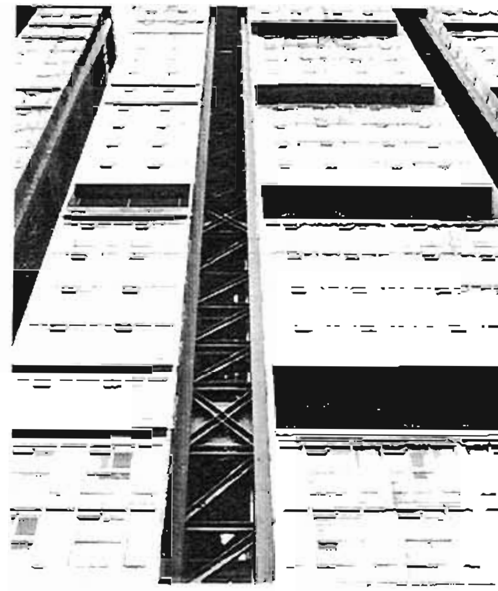


Plate 4.77: The "Megaframes"

Plate 4.76: The Takenaka complex

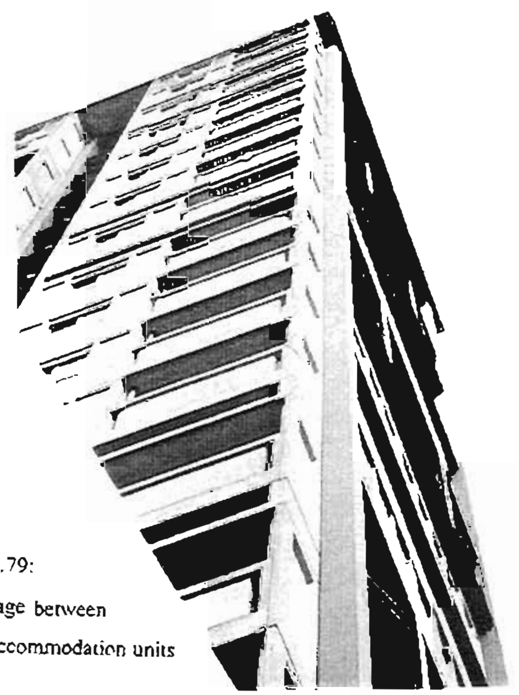


Plate 4.79:

Damage between

PC accommodation units

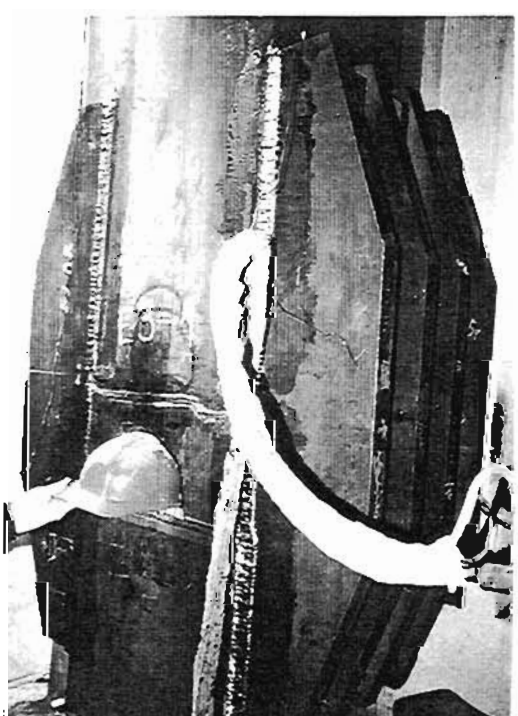


Plate 4.78: Fin strengthening at column fractures

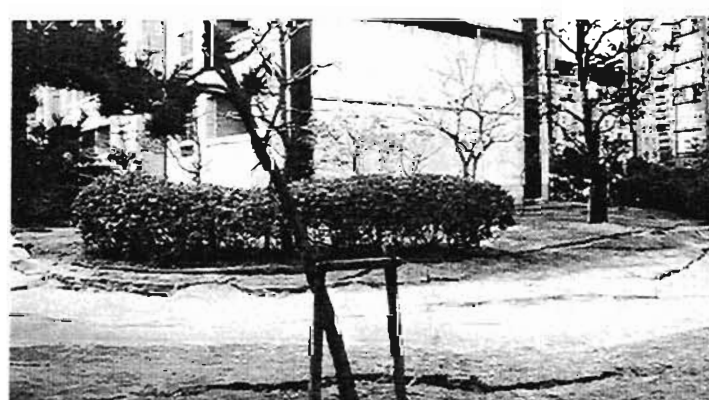


Plate 4.80: Settlement rings

Plate 4.81: High aspect ratio building with cold formed RHS columns

Plate 4.82: Disintegrated plinth of corner column

Plate 4.83: Apartments with RHS columns

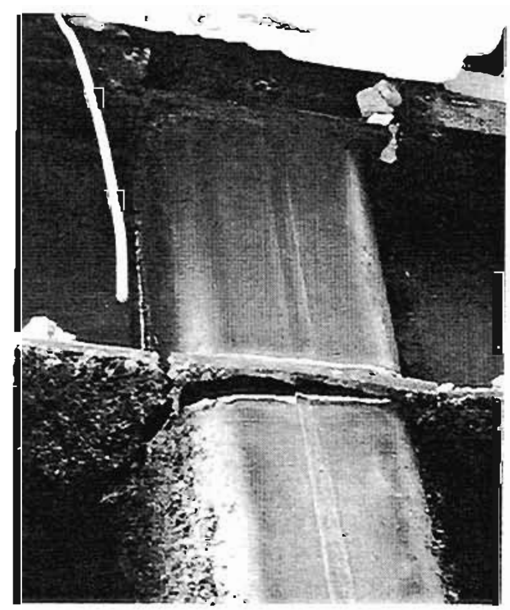


Plate 4.84: Failure at internal joint

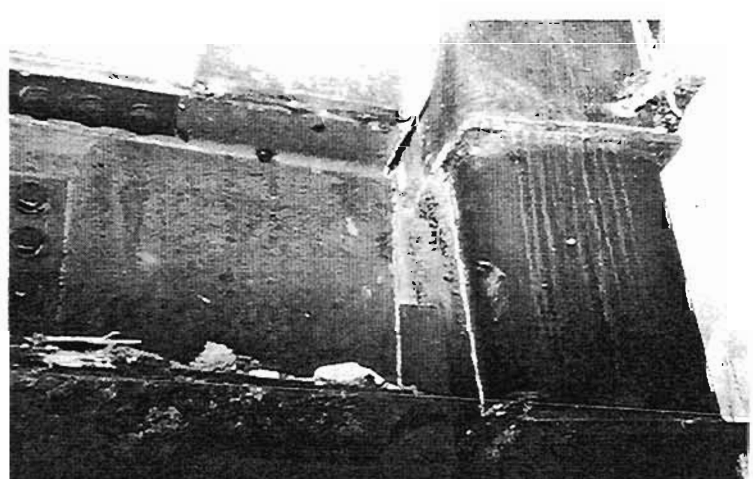


Plate 4.85:
Failure at external beam/column connection

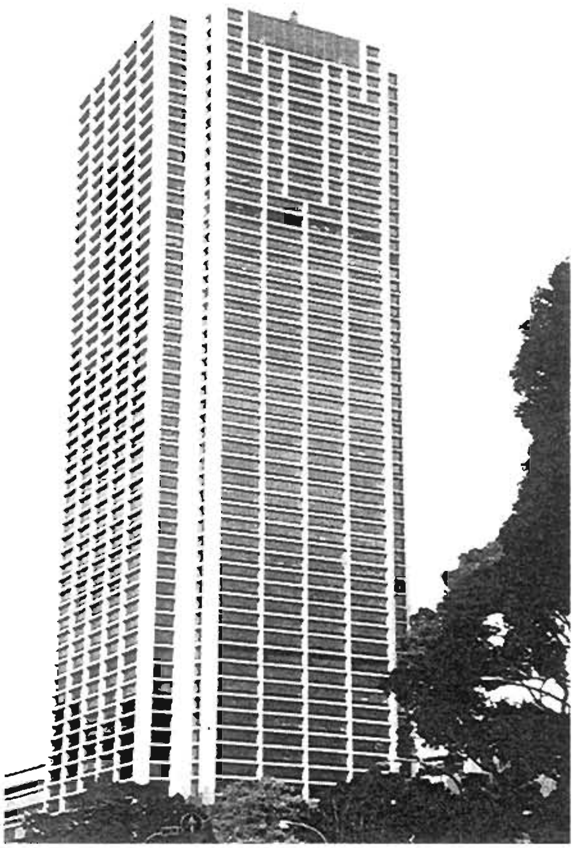


Plate 4.86: The 32 storey new town hall

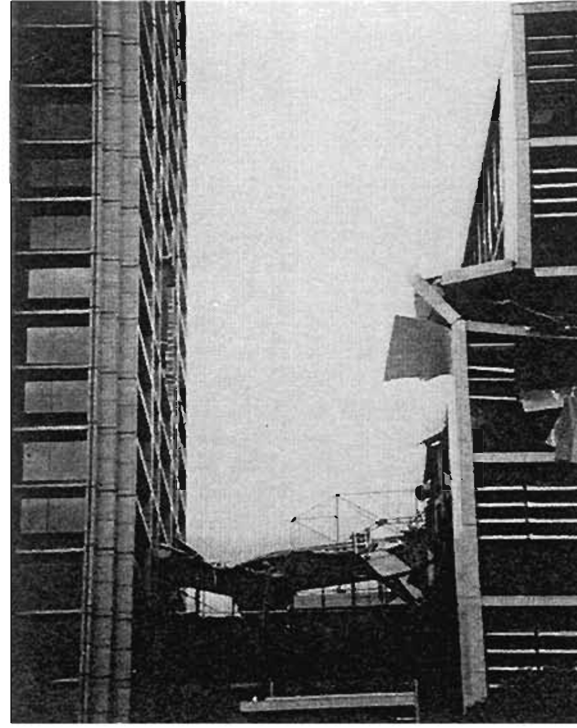
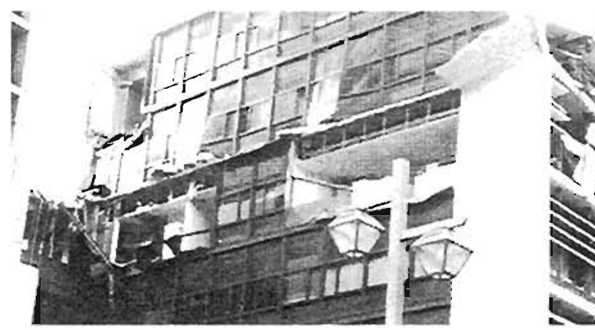


Plate 4.88: Collapsed and surviving link bridges



Close-up of damage

Plate 4.87: The old town hall with collapsed 6th storey

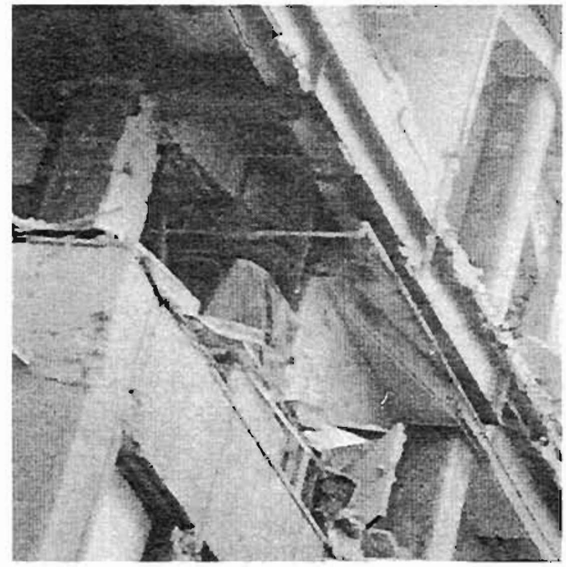
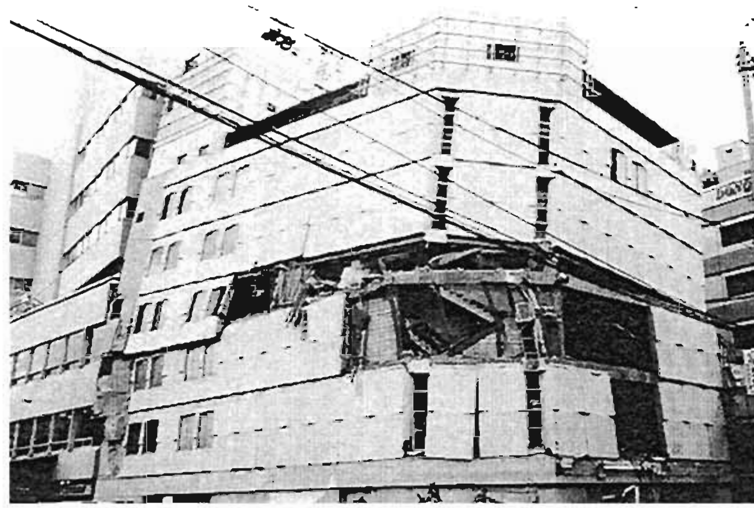


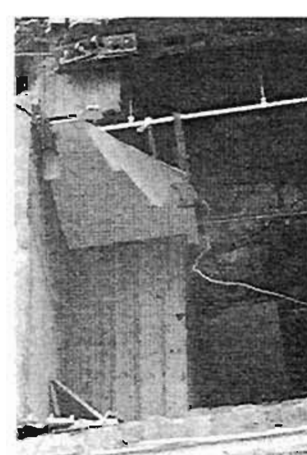
Plate 4.89: SRC building damaged by link bridge

SRC/steel transition

Plate 4.90: Damaged irregular corner building of SRC/mixed construction



Detail of beam damage in Plate 4.90



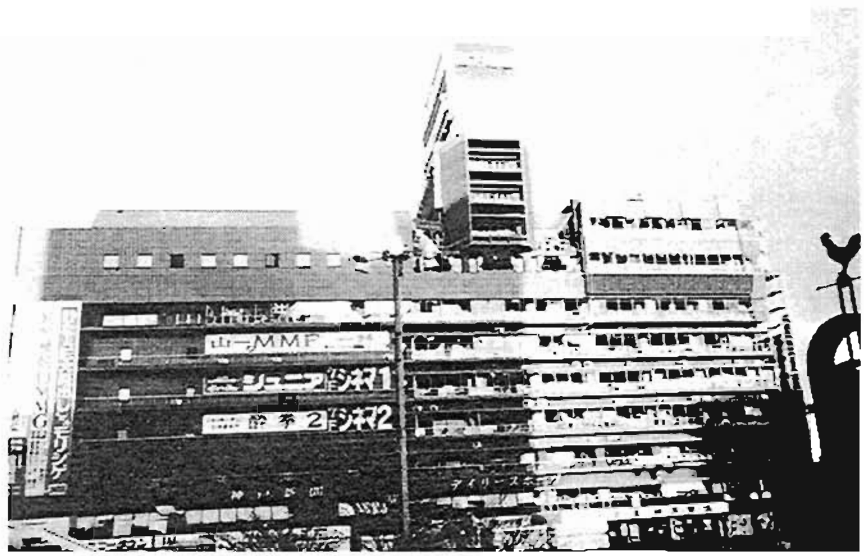


Plate 4.91: The Matsushita Building

Plate 4.92: Column moved 0.5m off plinth

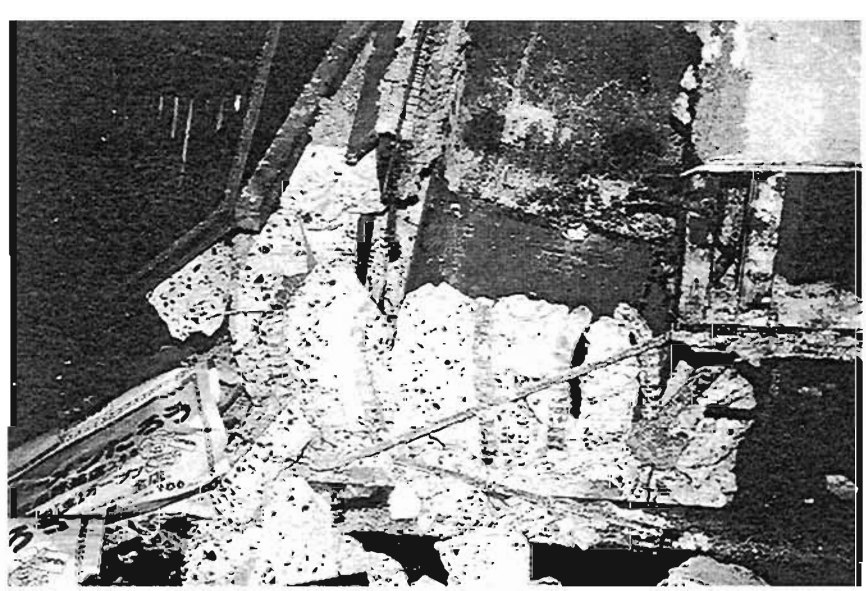


Plate 4.93: Column pulled out of plinth

Plate 4.94: Damage attributable to lack of foundation tie beams

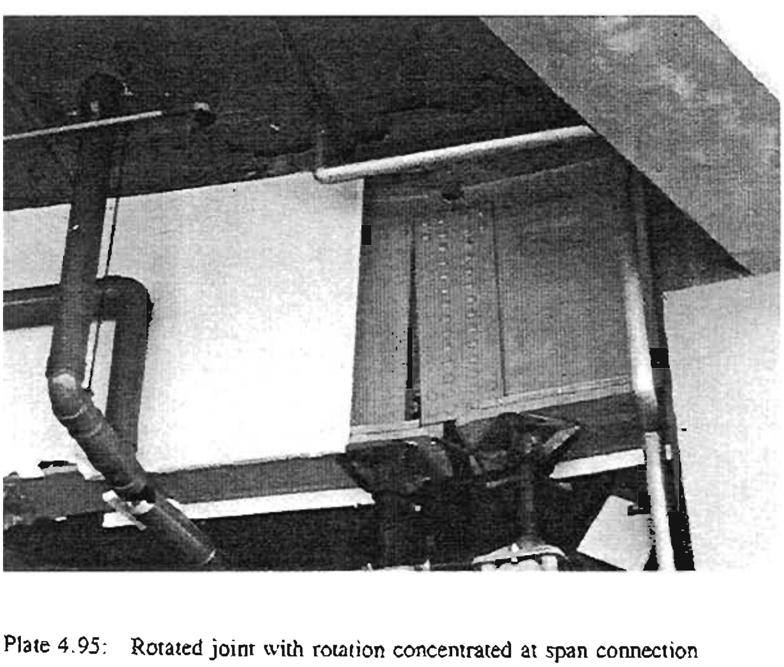
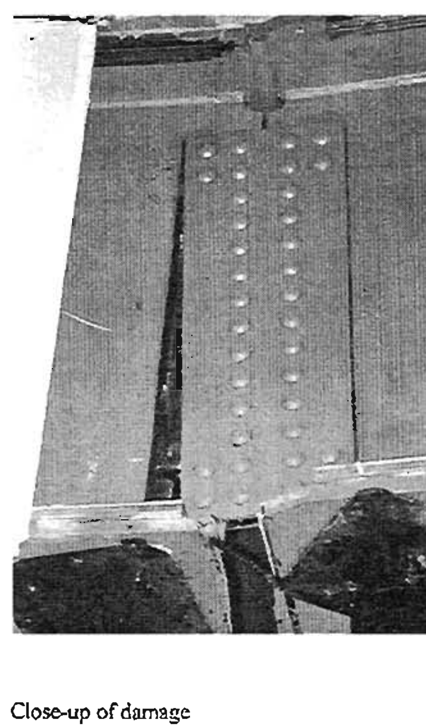


Plate 4.95: Rotated joint with rotation concentrated at span connection



Close-up of damage

Plate 4.96: Collapsed timber-framed houses

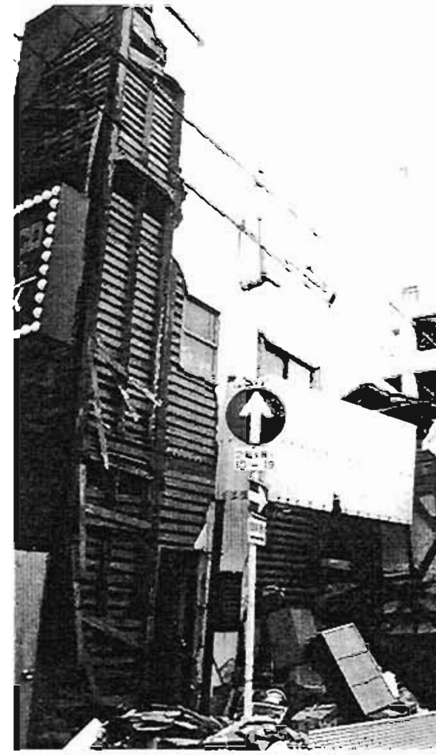


Plate 4.98: Timber-framed houses which shed their stucco

Plate 4.97: Collapsing house stabilised by pole

Plate 4.99: Collapse with roof intact

Plate 4.100: Severe cracking in masonry due to heavy roofs

Plate 4.101: Wire bracing in tall panels

Plate 4.102: Collapse of steel frame building with soft storey

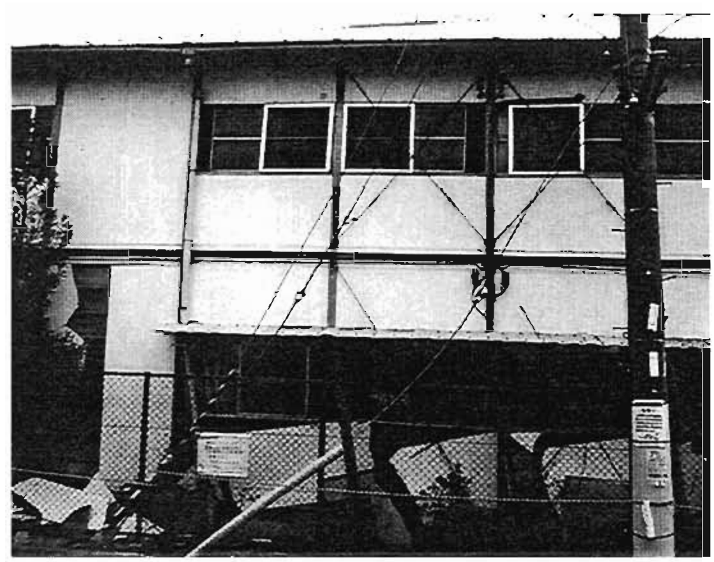
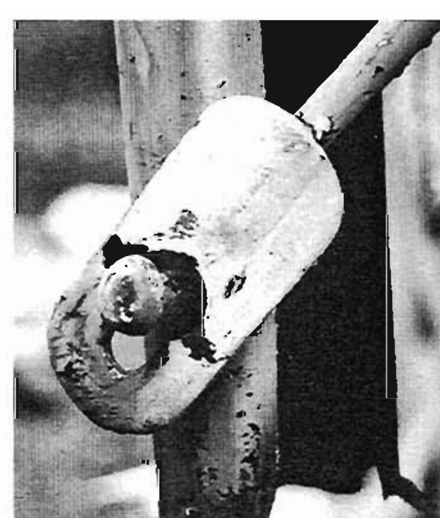


Plate 4.103: Prefabricated building with wire bracing

Plate 4.104: House with ground floor moment resisting frame



Detail of Plate 4.103

Detail of Plate 4.105

Plate 4.105: Mid-height failure of substantial irregular structure

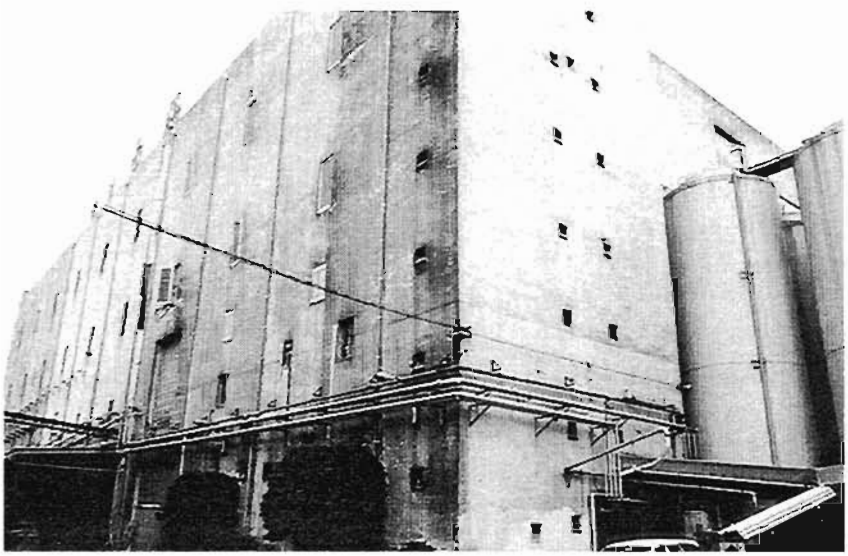


Plate 4.106: Unperforated shear wall of factory in port area

Detail of Plate 4.106 showing damage at expansion joint

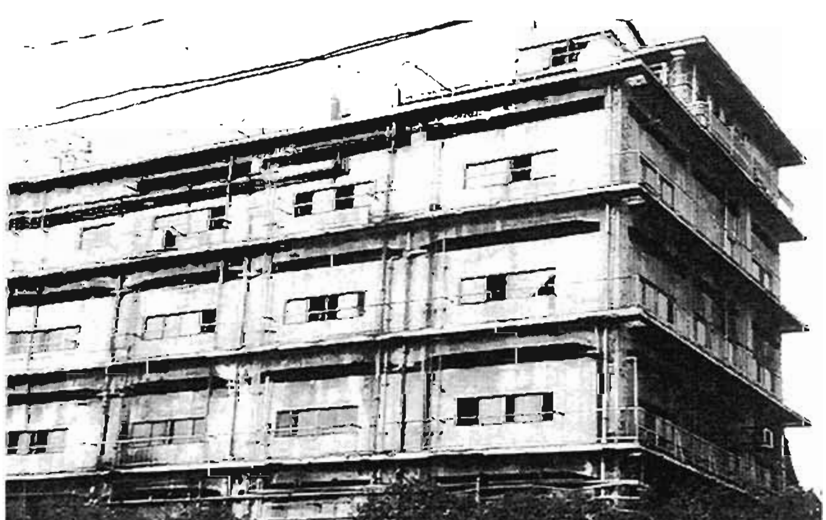


Plate 4.107: An undamaged clerestory windowed building on reclaimed land near the coastal highway

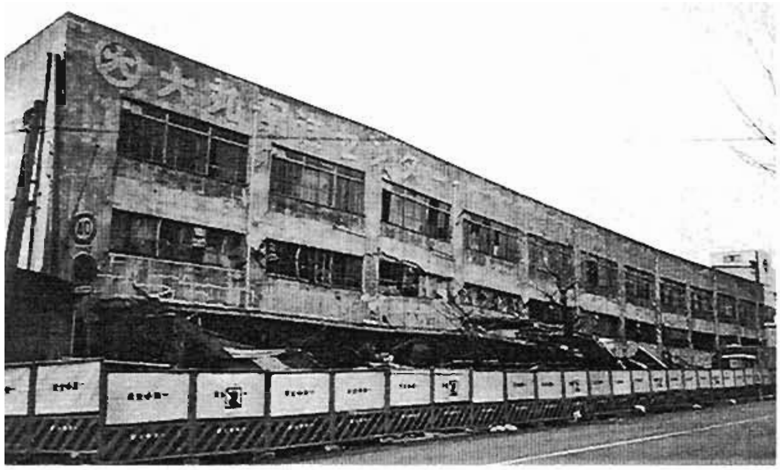


Plate 4.108: Progressive collapse in older 11 bay three storey building

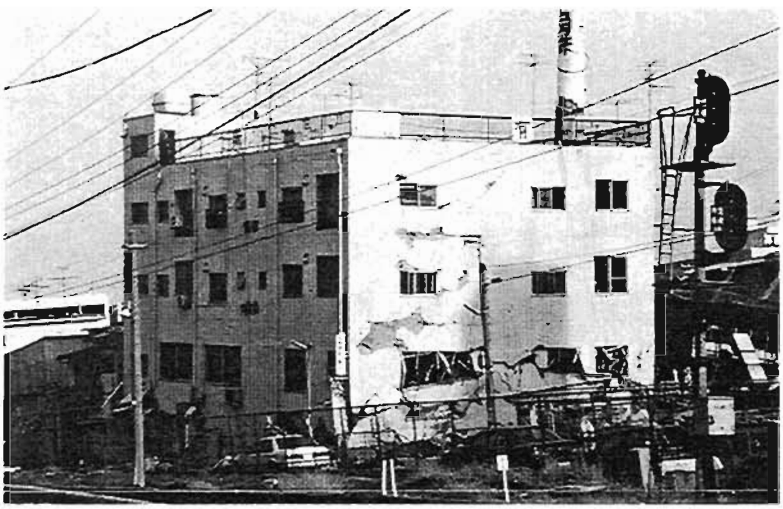


Plate 4.109: Local failure at long 1st floor window of building recessed into sloping site

Plate 4.110: Precast concrete building with partial collapse induced by link bridge

Details of precast monoliths

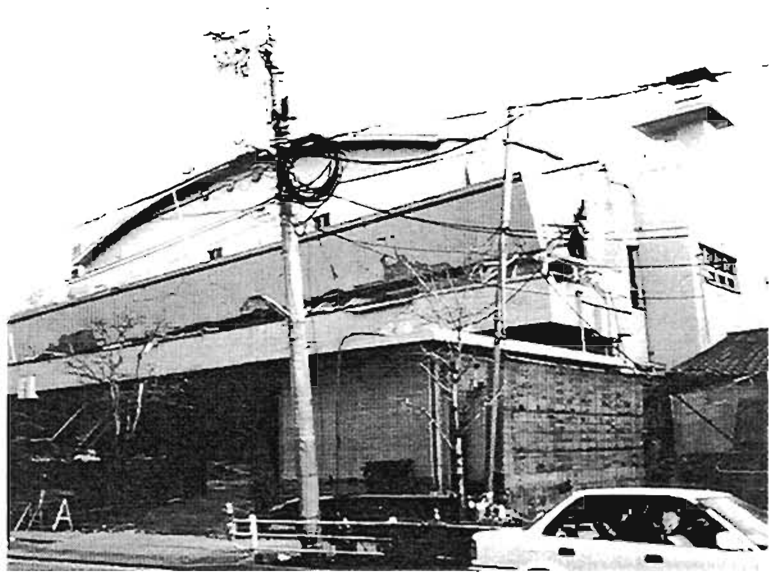


Plate 4.111: Soft storey collapse in domed-roof factory

Plate 4.112: A garage base plate which performed well despite the failure of the supporting plinth

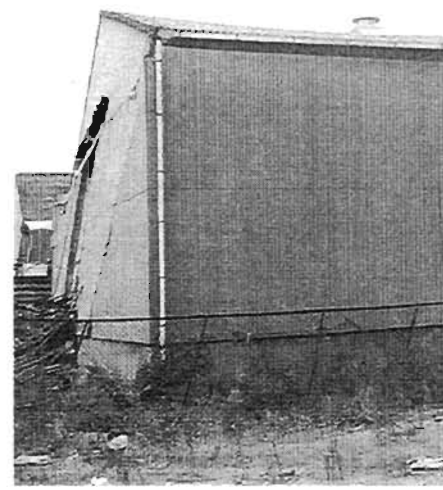


Plate 4.113: Mid-side failure of long steel frame shed

Plate 4.114: Undamaged bamboo portal garage

Plate 4.115: Undamaged 3-storey warehouse in the port

Plate 4.116: Undamaged well designed highly irregular industrial building

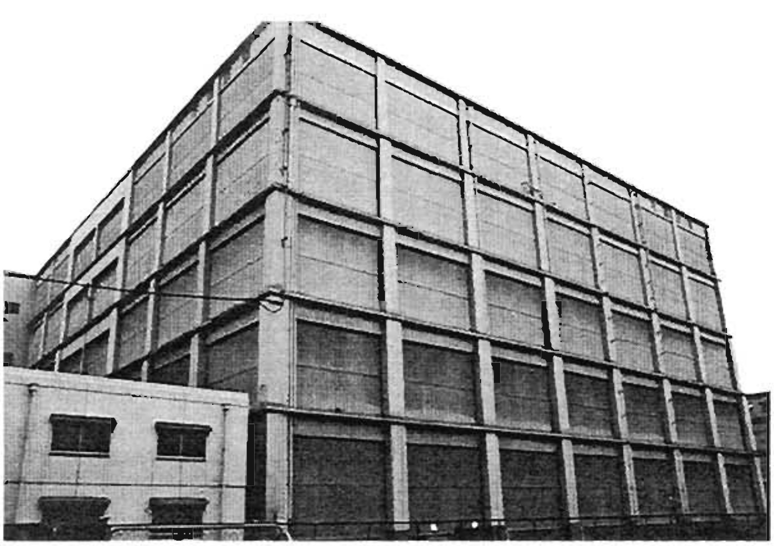
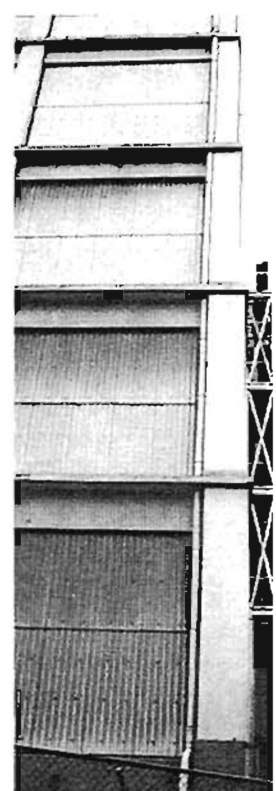


Plate 4.117: Undamaged Japanese framed-wall building



Close-up of corner

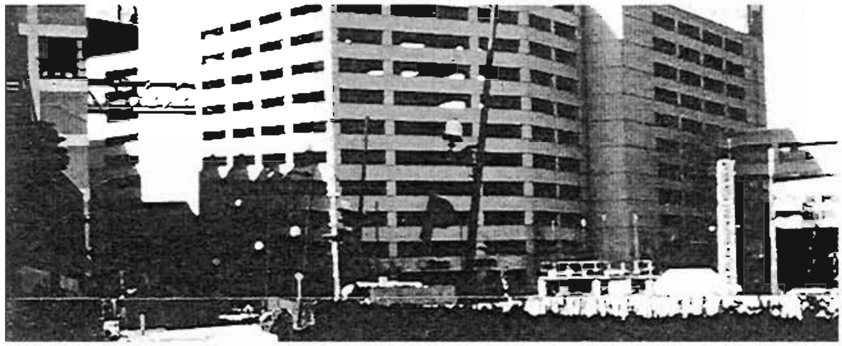


Plate 4.118: New large multi-storey car park near the ferry

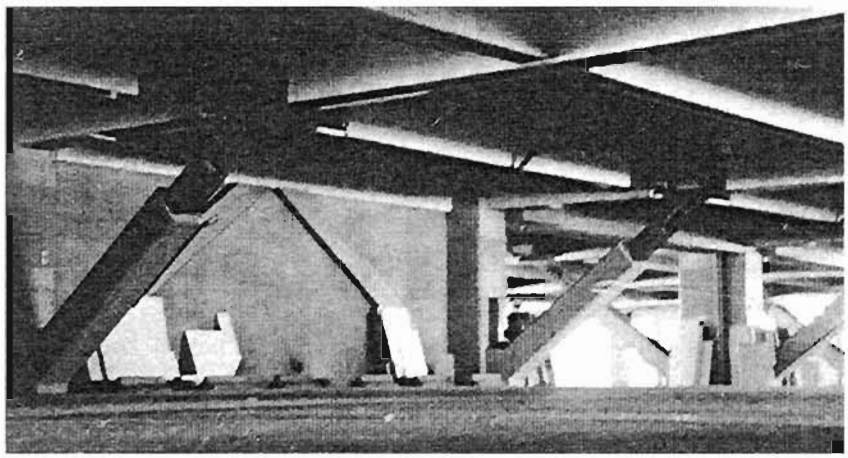


Plate 4.119: Internal bracing

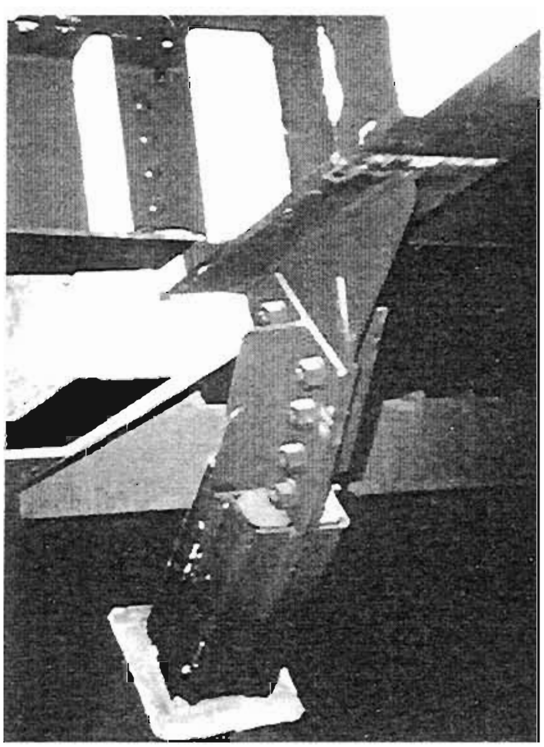


Plate 4.120: Upper end of brace

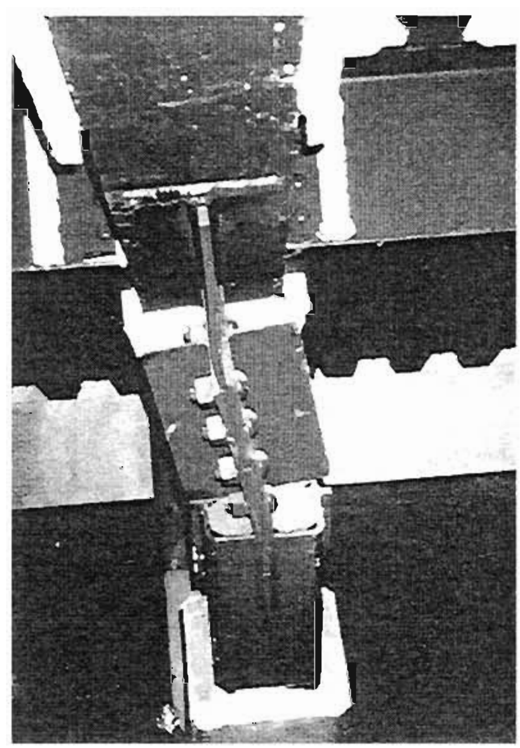


Plate 4.121: Buckling of gusset plate in Plate 4.1:

Plate 4.122: Perimeter structure with X-cracking in stub columns

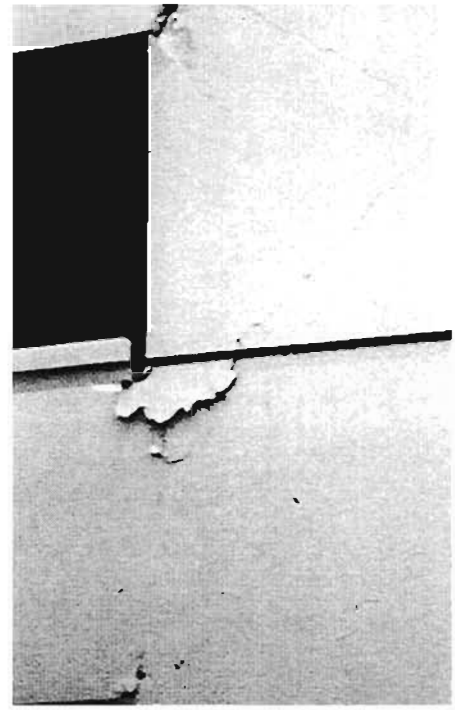


Plate 4.123: Cracking at ends of connecting

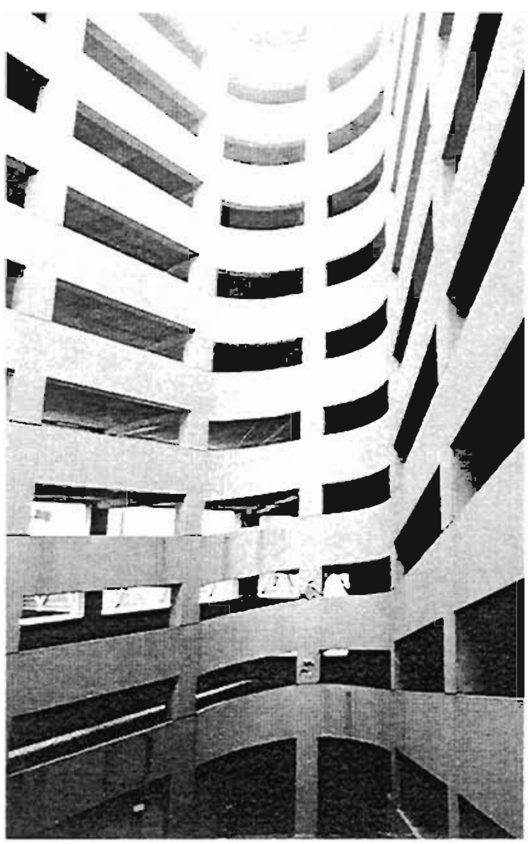


Plate 4.124: Well within spiral ramp

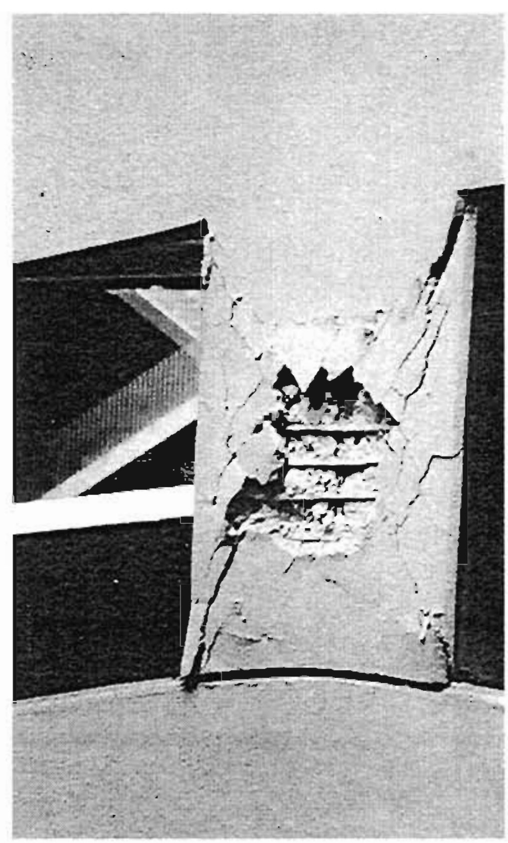


Plate 4.125: Damage to confined concrete at end

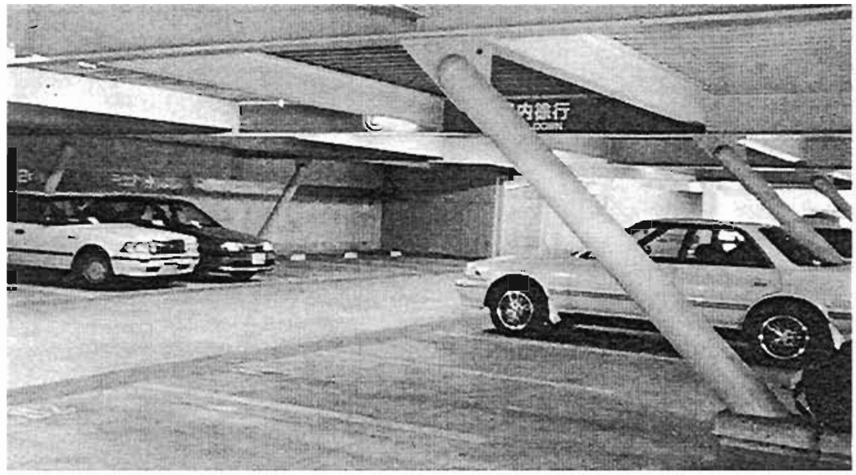


Plate 4.126: Internal structure of 9-storey car park showing internal walls

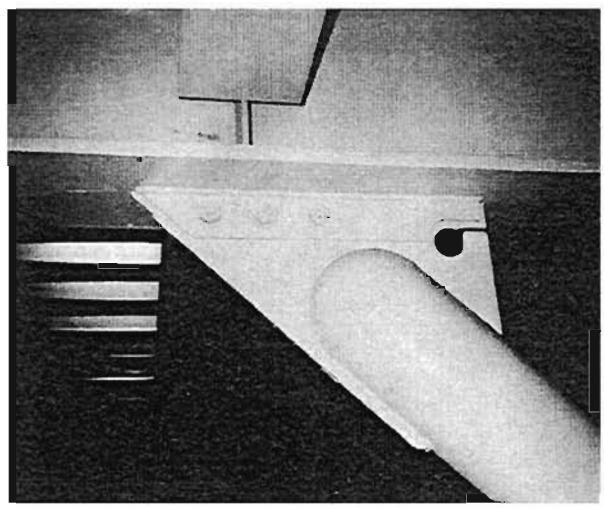


Plate 4.127: Brace connection - upper end

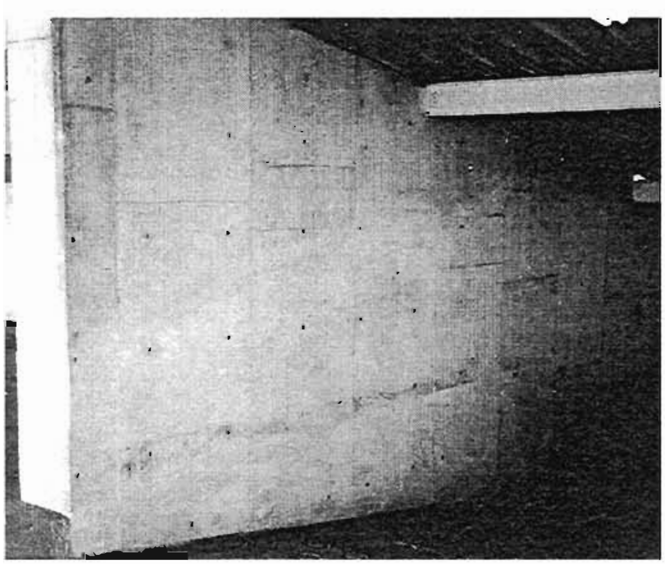


Plate 4.128: Repaired cracks in transverse wall

Plate 4.129: Repaired spine wall cracks at b

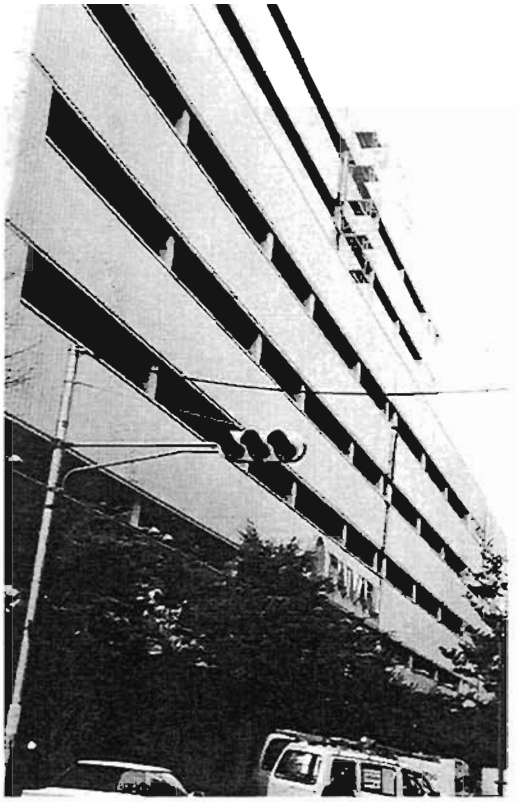


Plate 4.130: The 9-storey Diamaru car park

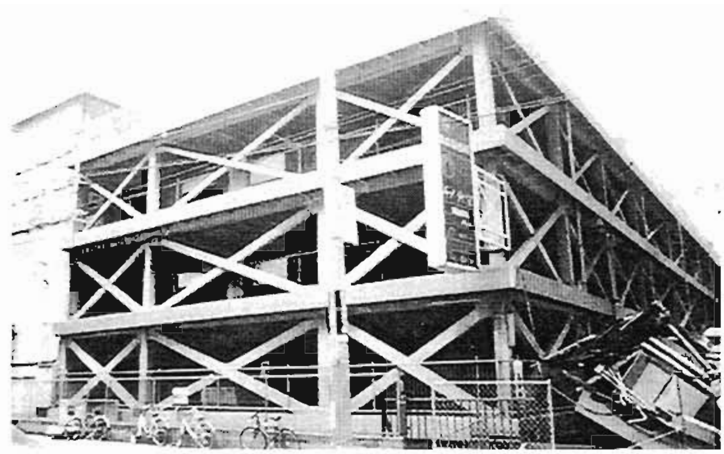


Plate 4.131: 3-storey SRC car park

Plate 4.132: Internal bracing of the Diamaru

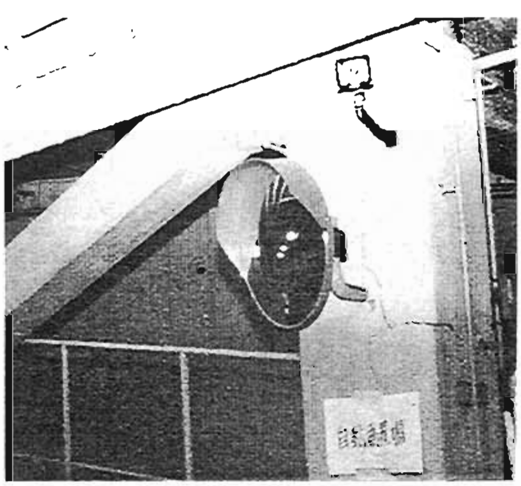


Plate 4.133: Column cracking in braced bay

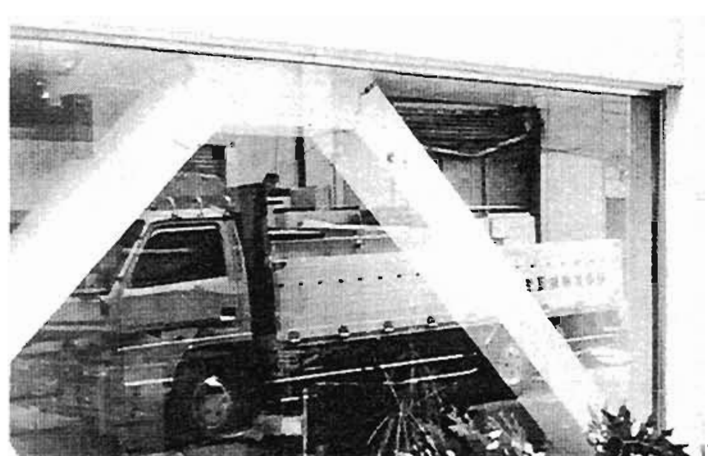


Plate 4.134: Cracking in SRC inverted-V bracing

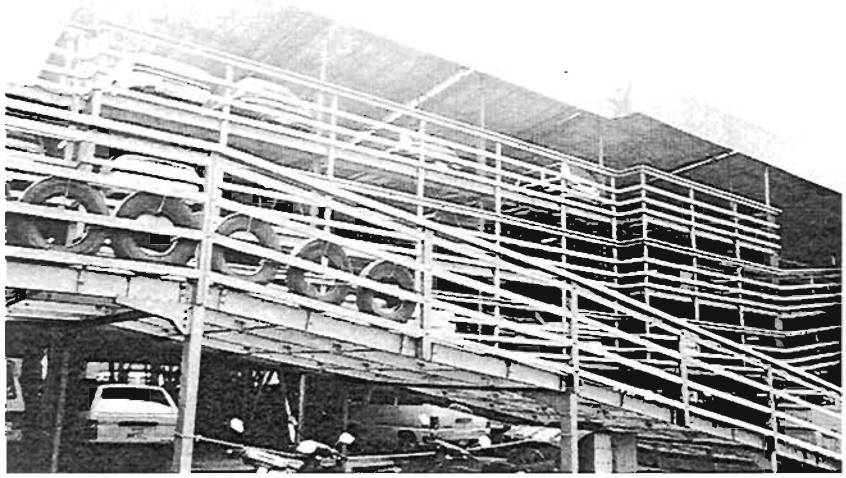


Plate 4.135: Undamaged flimsy and flexible car park

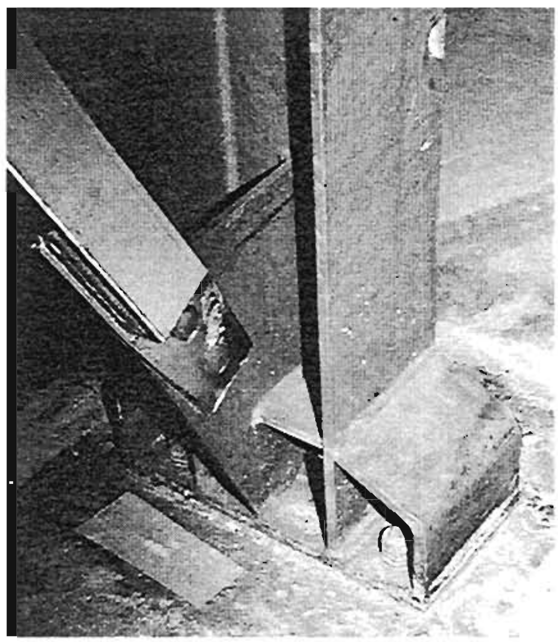


Plate 4.136: Column base/brace connection

Plate 4.137: "Tent" at roof level

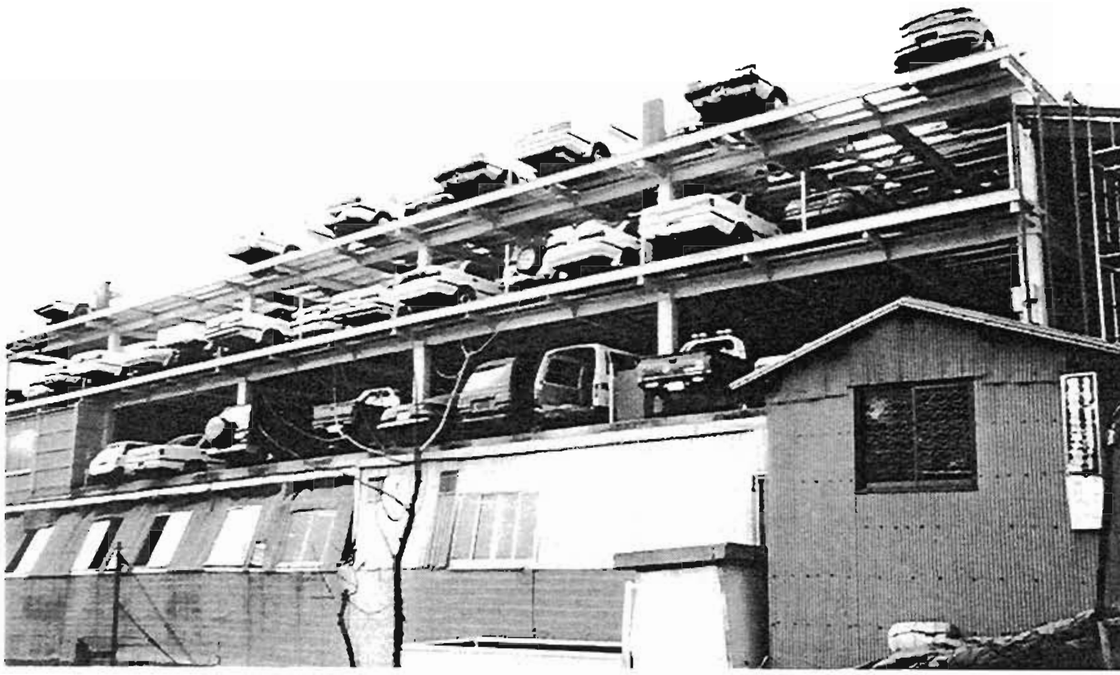


Plate 4.138: Flexible steel car park in port area above damaged ground floor office

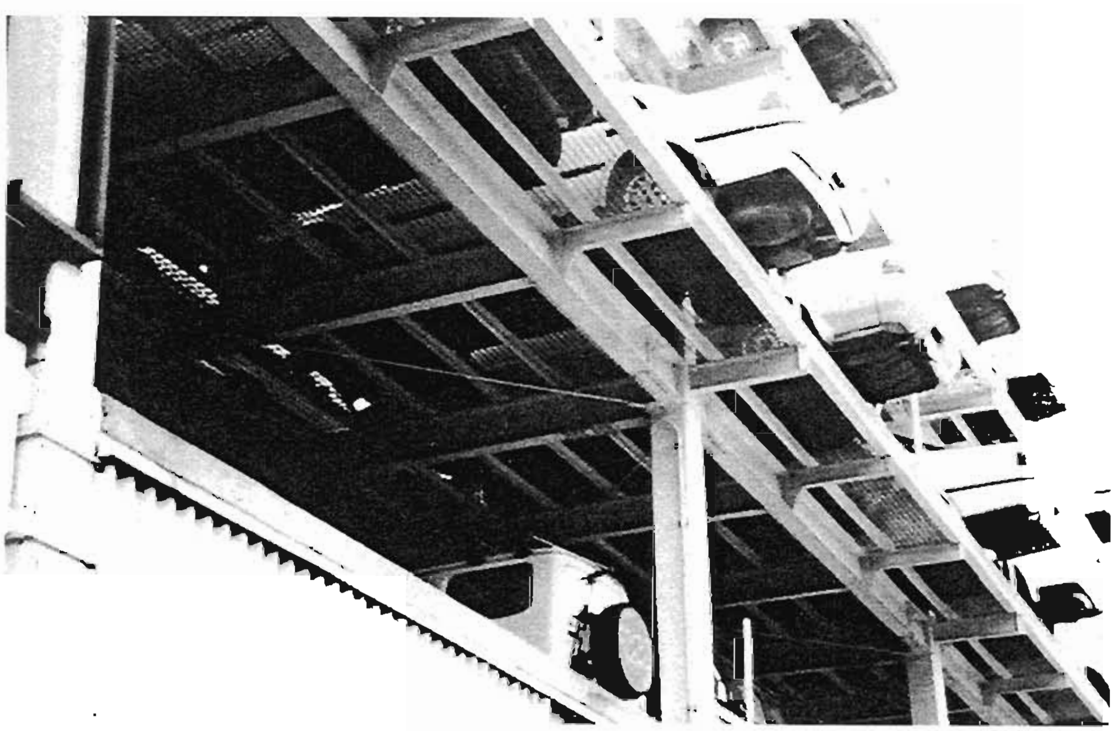


Plate 4.139: Detail of steelwork in Plate 4.138

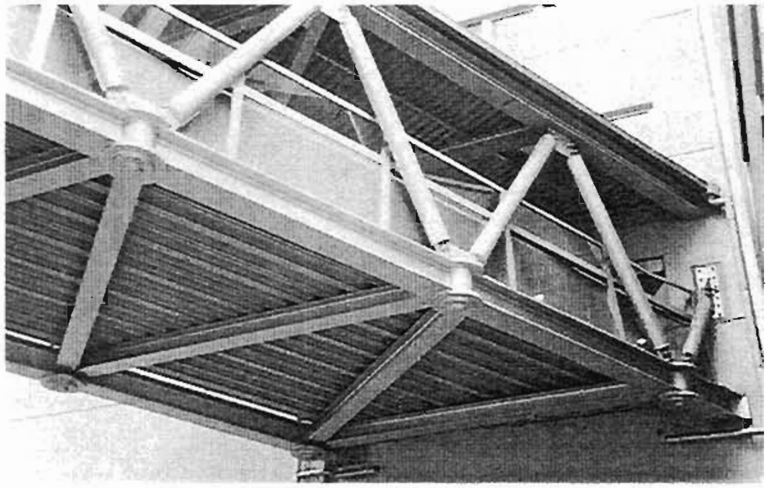


Plate 4.141: Underside of bridge

Plate 4.140: Link bridge to car park in Plate 4.118

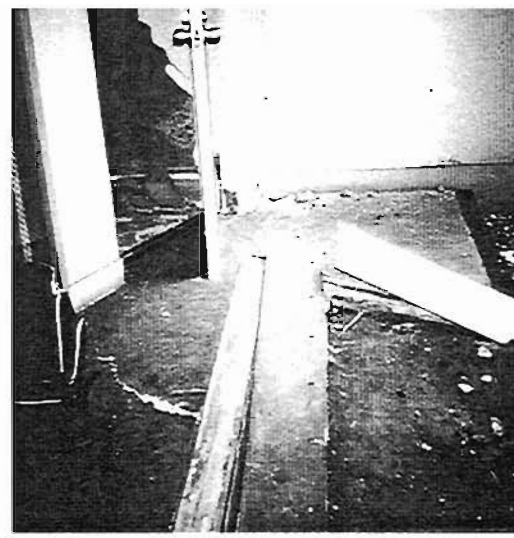
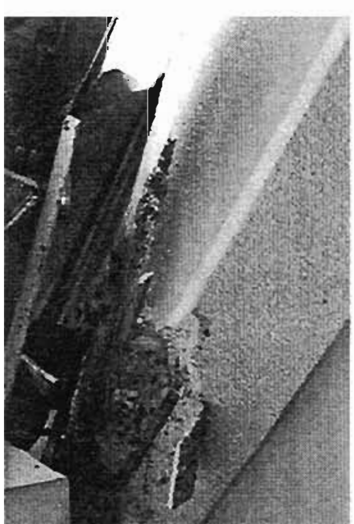


Plate 4.142: Movement joint



Anchor pull-out

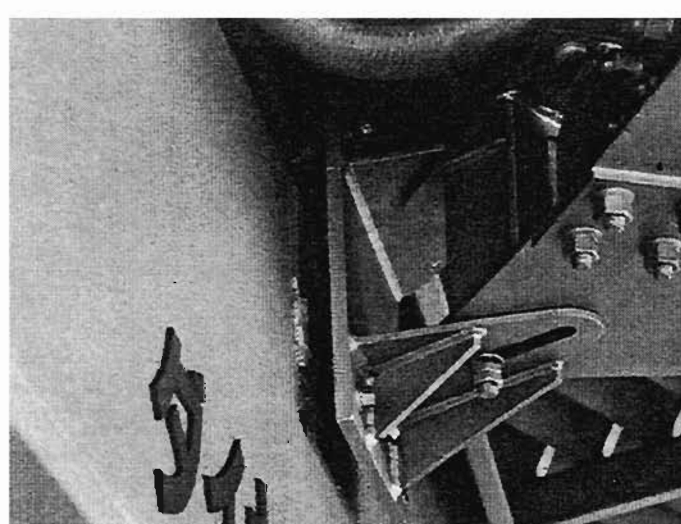


Plate 4.143: Failed top-beam connection

Plate 4.144: Damaged support beneath another link bridge to car park in Plate 4.118

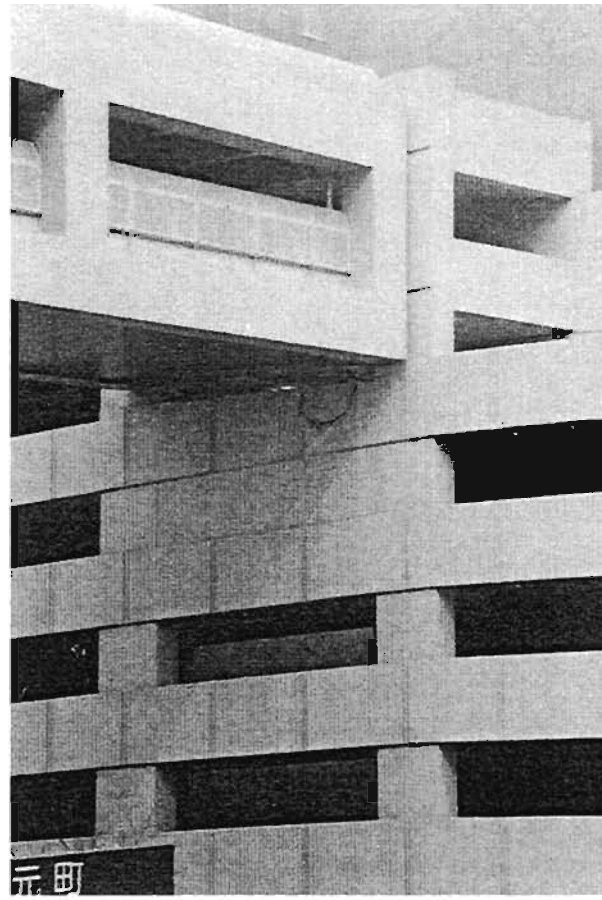


Plate 4.145: Connection damage between new buildings

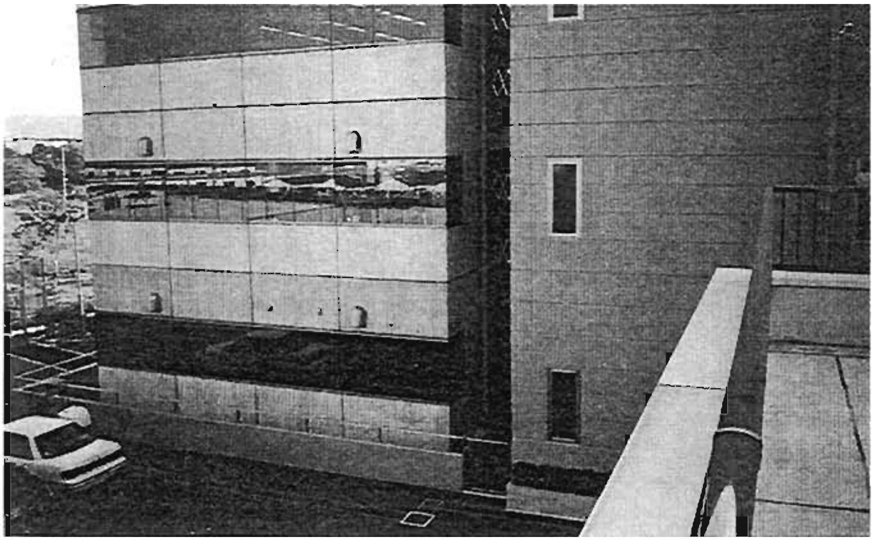


Plate 4.146: The MTRC
(base-isolated RC buildin
frame building on the rig
and expansion joints in be

Plate 4.148: Supports
in MTRC building

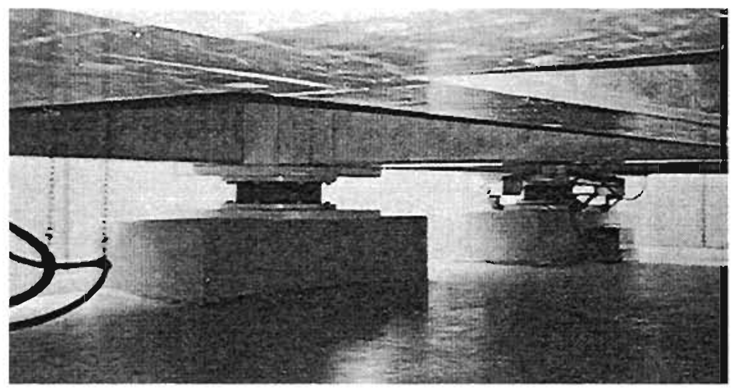


Plate 4.149: The base-isolators

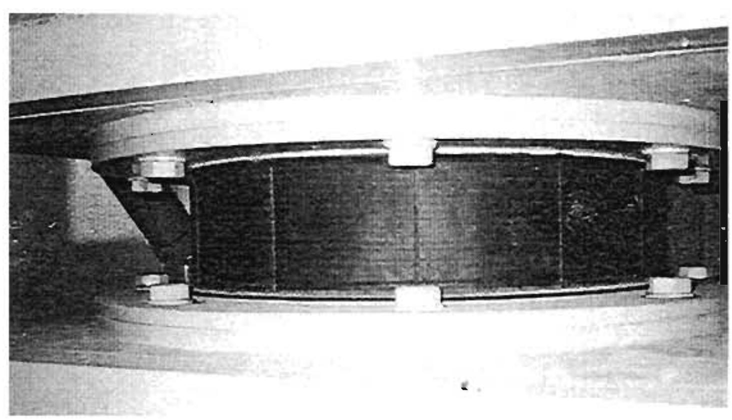


Plate 4.147: The Computer Centre
of Post and Communications (C
Building

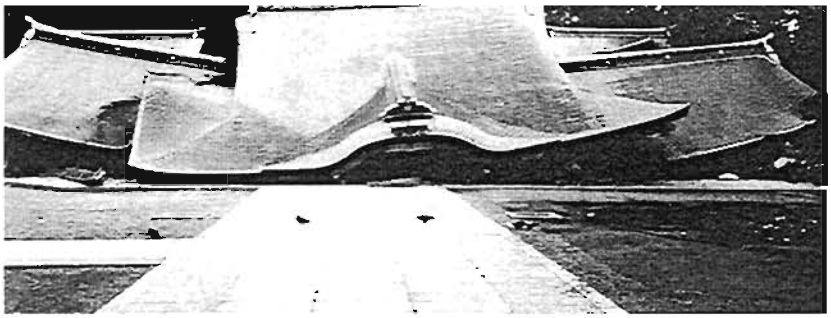


Plate 4.150: Collapsed temple

Plate 4.151: Severely racked shrine

Plate 4.152: Toppled sculpture

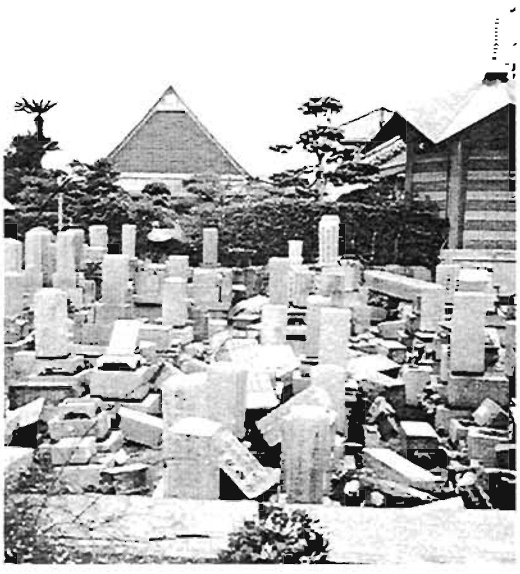


Plate 4.153: Collapsed headstones

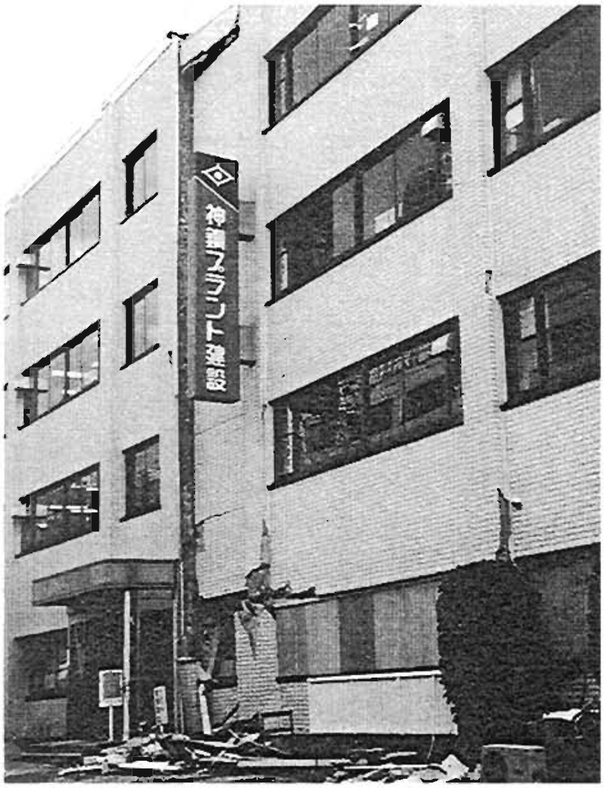


Plate 4.154: Impact damage between buildings of same height

Plate 4.155: Collapse of 3[rd] storey of corner building due to buffeting

Plate 4.156: Column damage due to buffetting

Plate 4.157: Buffetting damage in top storey

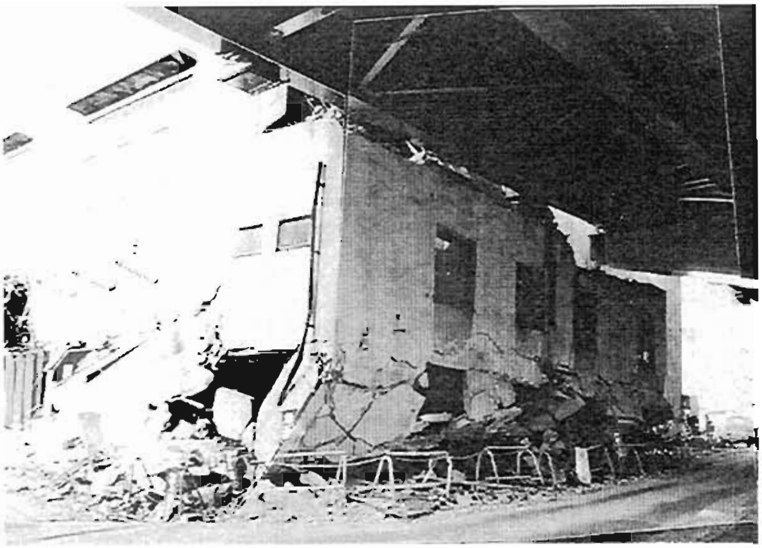


Plate 4.158: Substation collapsed against viaduct damaging viaduct

Plate 4.159: Collapse of viaduct span due to building restraint

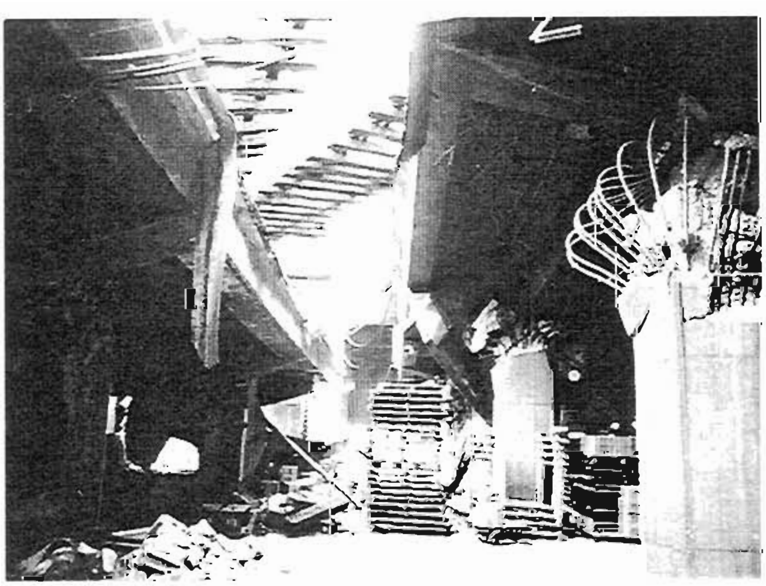


Plate 4.161: Collapsed infill slab due to buffeting between viaduct and warehouse

Plate 4.160: Damaged stub column under viaduct due to building restraint

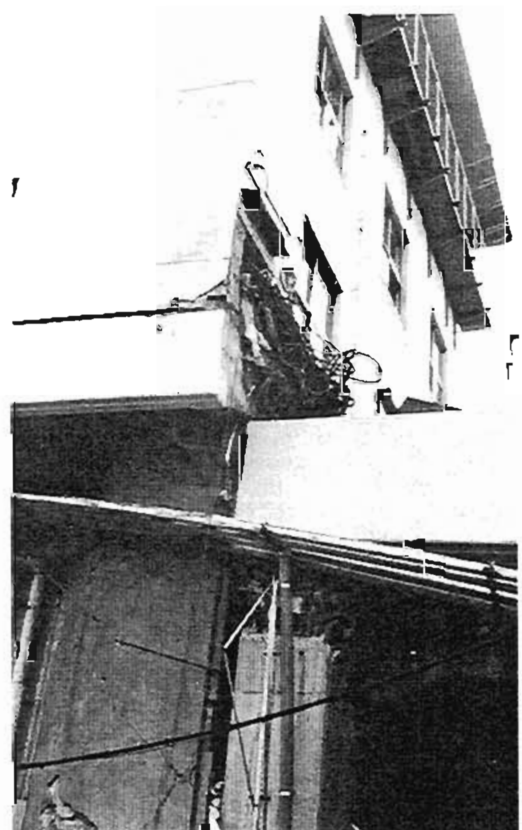


Plate 4.162: First floor column damage due to impact across expansion joint

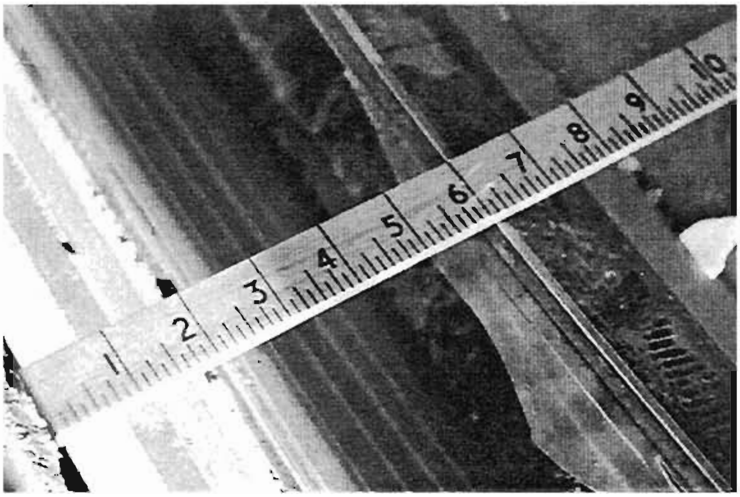


Plate 4.163: Width of seismic joint after earthquake

Plate 4.165: Combined fire and earthquake damaged building

Plate 4.164: Fire damaged commercial property in zone of collapsed housing

Plate 4.167: Close up of injector and reservoir caps

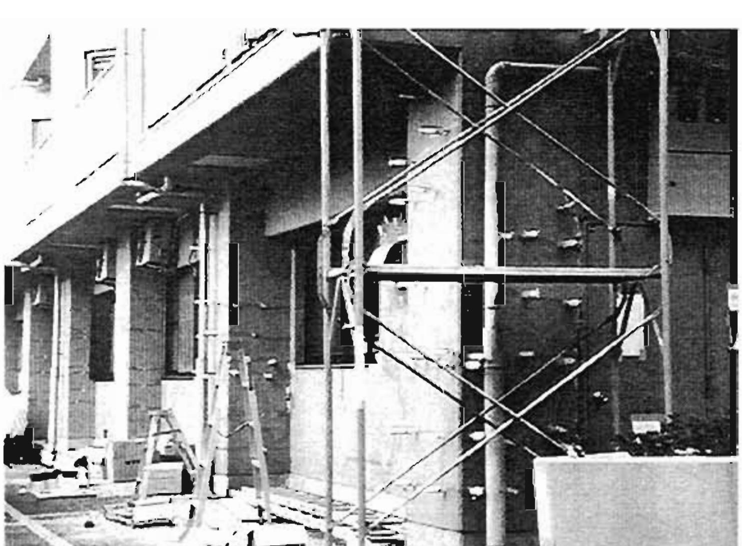


Plate 4.166: Epoxy grouting of columns in lowest floor

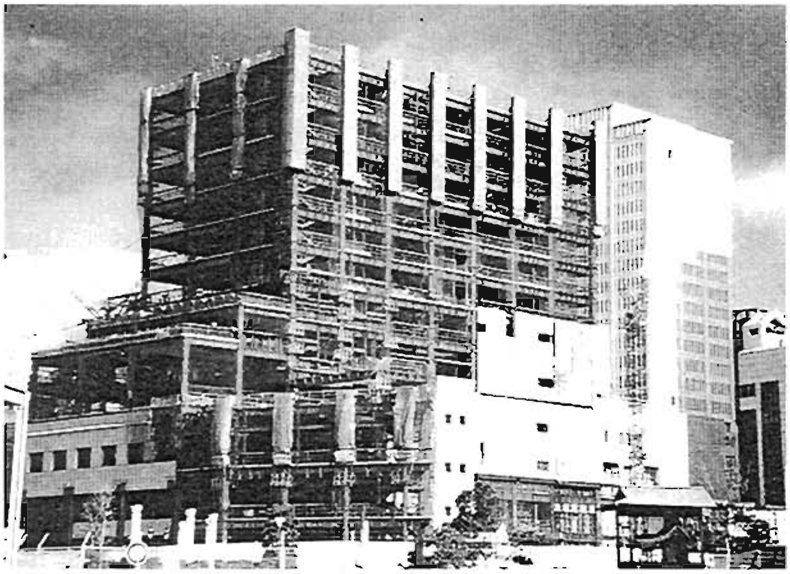


Plate 4.169:: New steel building on waterfront

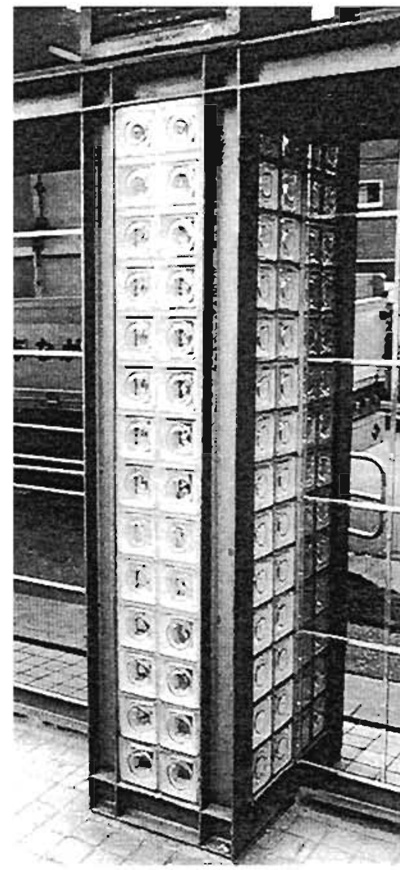


Plate 4.170: Undamaged glass column infill panels in new steel building

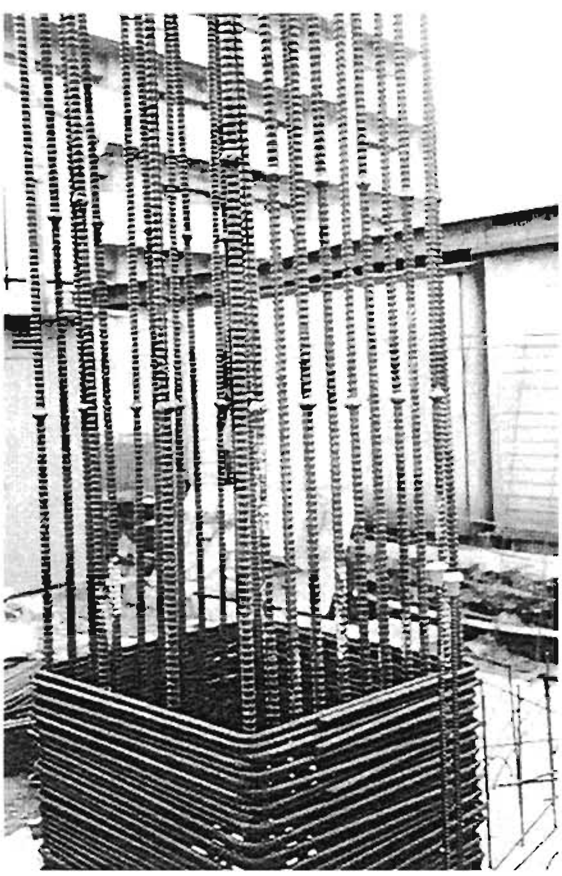
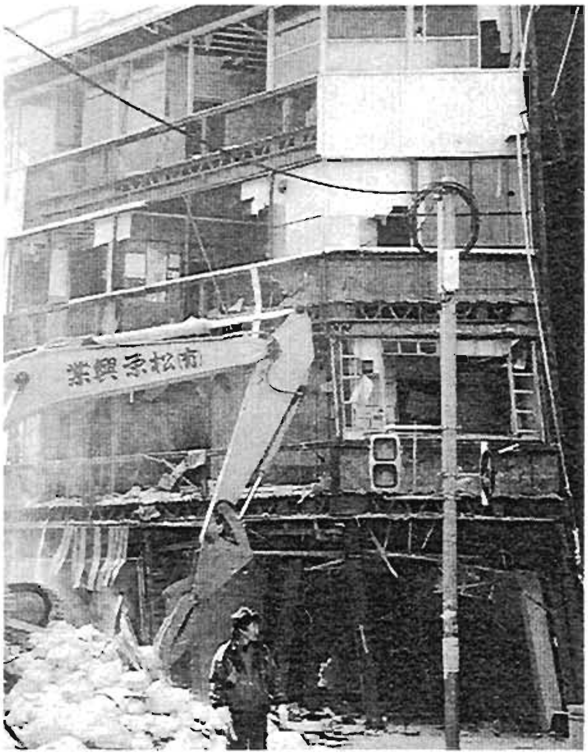


Plate 4.168: Column reinforcement in modern RC construction



Close-up of racked ground floor storey

Plate 4.171: 4 storey building with pinned feet

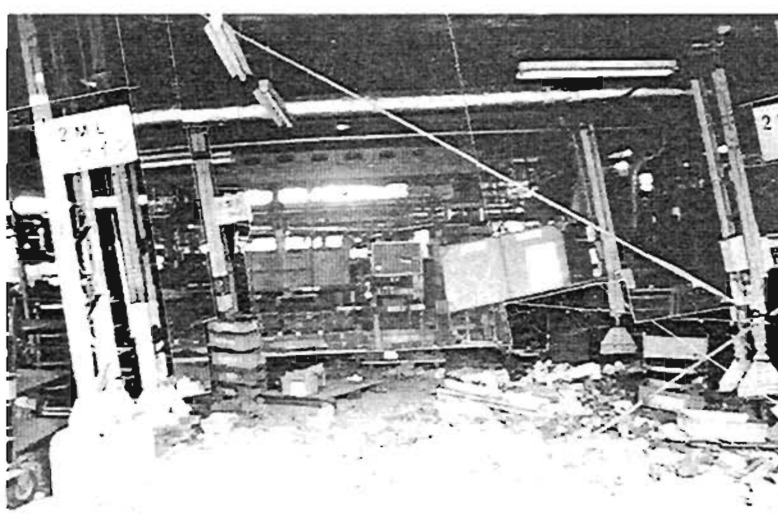


Plate 4.172: Full moment connection at column feet

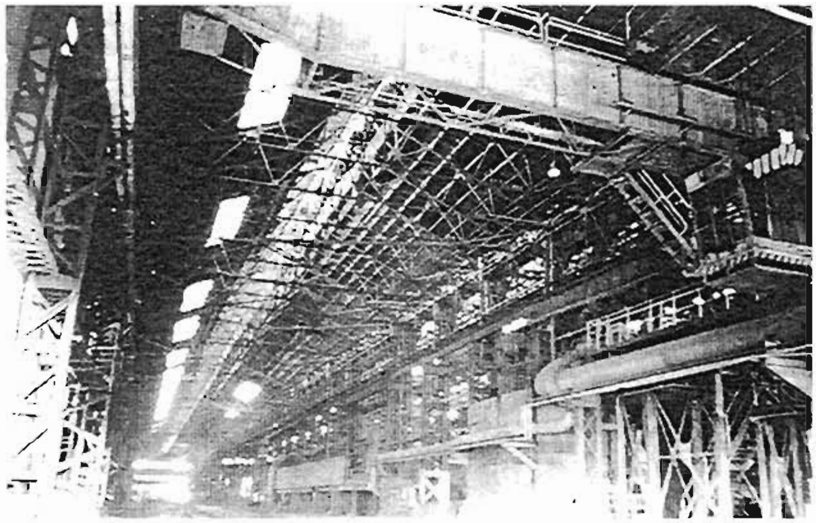


Plate 4.173: Industrial shed at old Kobe Steel Works

Plate 4.174: Cracked encased base of lattice column

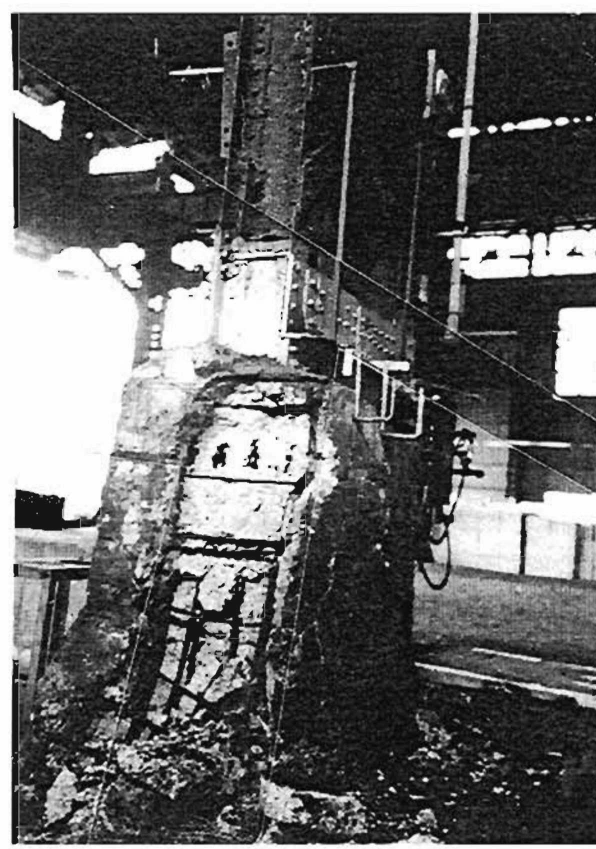


Plate 4.175: As 4.174 but severely corroded

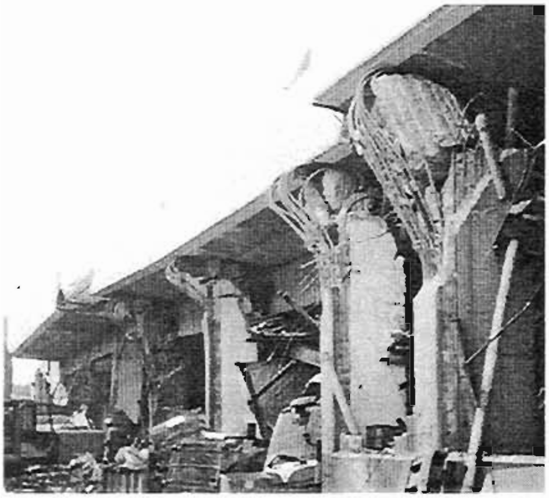


Plate 4.176: Differential drifts between transverse joints

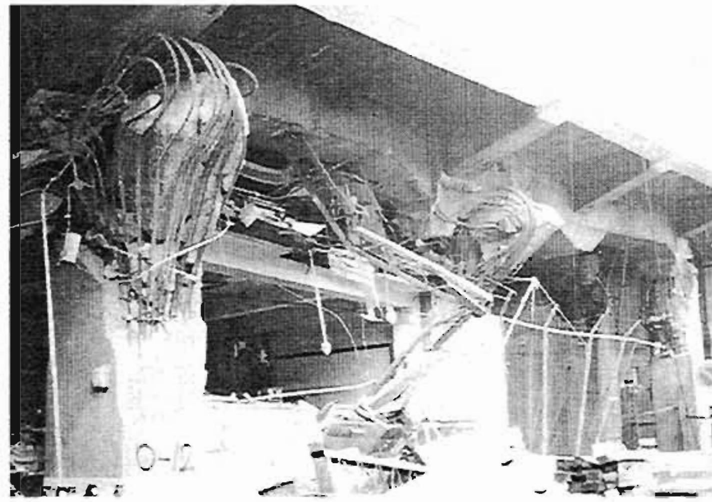


Plate 4.177: Incipient longitudinal hinge and more developed transverse hinges

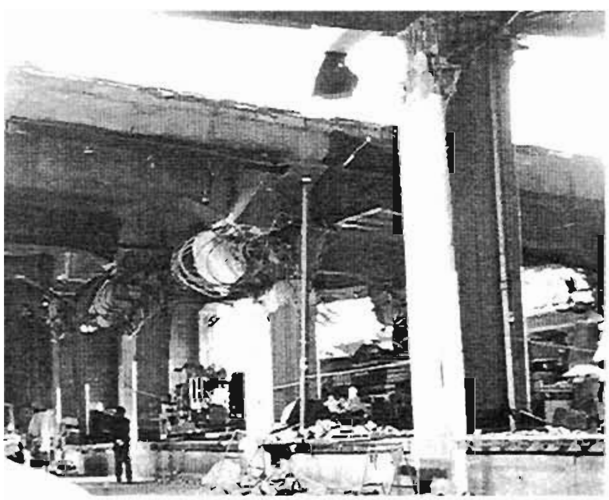
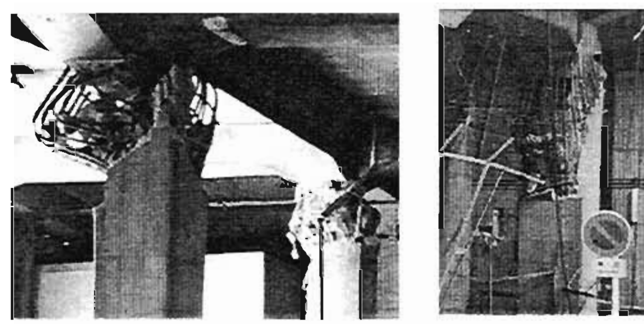


Plate 4.178: Incipient internal transverse hinge



Close-ups of hinges

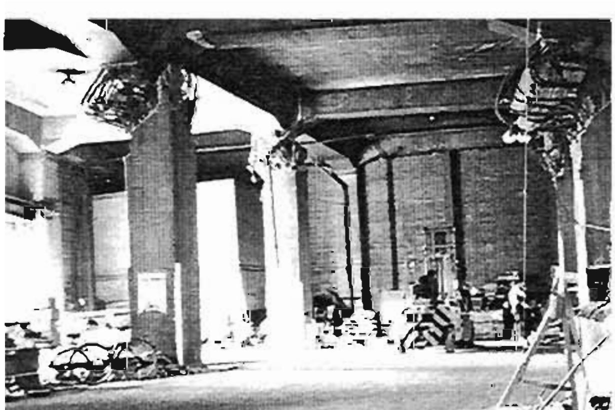


Plate 4.179: Graduated damage between wall and expansion joint in building

Plate 4.180: Incipient external transverse hinge

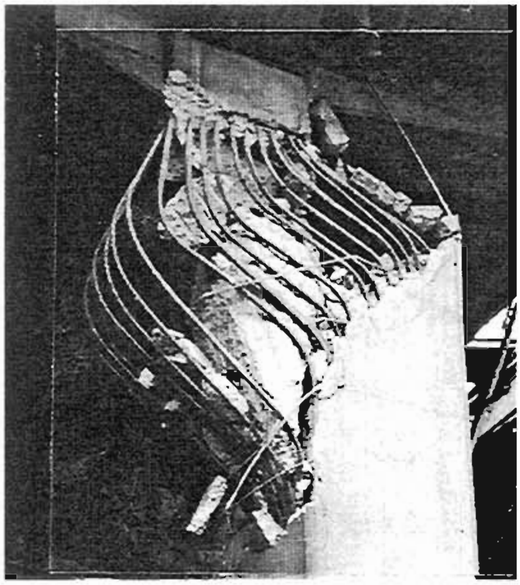


Plate 4.181: Anvil in flexural hinge

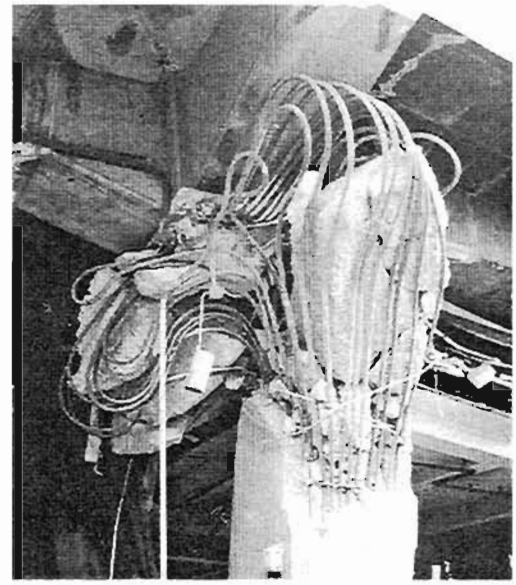


Plate 4.182: Collapsed anvil

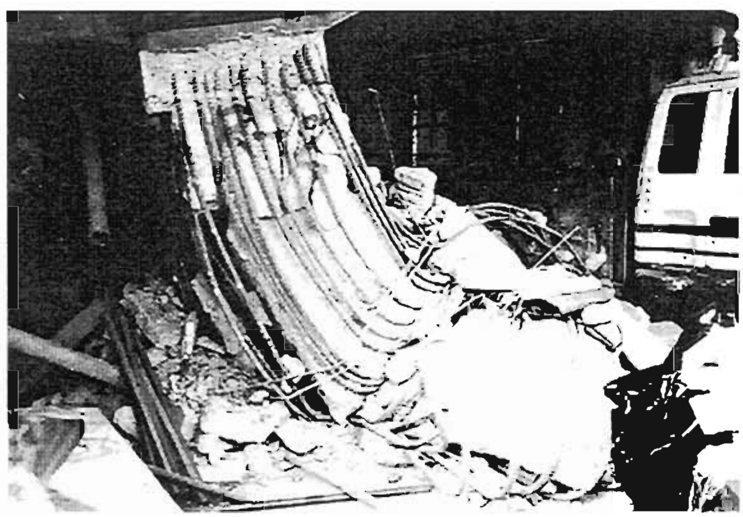


Plate 4.185: Collapsed column initiated by shear failure at base

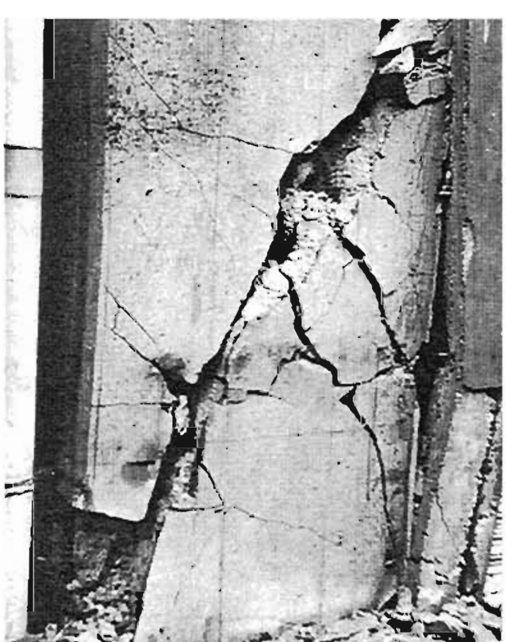


Plate 4.183: Incipient shear failure

Plate 4.184: Anvil in shear hinge and incipient spreading of column beneath

5.0 DAMAGE SURVEYS, HUMAN CASUALTIES AND SOCIO-ECONOMIC IMPLICATIONS

A Pomonis
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5.1 Introduction

The Hyogo-ken Nanbu (Kobe) earthquake was the most severe in post-war Japan. The number of lives lost and collapsed buildings, were the highest in this century with the exception of the Great Kanto earthquake on September 1, 1923. Since that terrible disaster the psychology of the whole Japanese nation in respect to natural disasters has been affected. Enormous efforts have been made to make this densely and highly developed country as strong as possible in the face of natural hazards. Arguably Japan is the world's leader in disaster mitigation and preparedness measures and the Japanese people are amongst the best educated in aspects of natural hazards, that continuously threaten and disrupt their busy and productive society.

Following the 1923 earthquake, disasters and high losses of life continued with 7 earthquakes in 1927, 1933, 1943, 1944, 1945, 1946 and 1948 (for information on these events see Appendix A of Chapter 2). These earthquakes caused loss of life between 1,000 and 5,000 each, and approximately 16,000 in total. Three of these events occurred in the Central Japan seismic zone (in 1927, 1943 and 1948) and three in the Nankai trough seismic zone (in 1944, 1945 and 1946). These disastrous earthquakes have affected the densely inhabited Central and Western Japan, in ways similar to the Hyogo-ken Nanbu earthquake.

Following the post-war reconstruction and rapid economic development of Japan, earthquake casualties have been reduced dramatically. Since the 1948 Fukui earthquake, Japan has not experienced any natural disaster that inflicted such heavy losses of life and property[1]. The loss of life from earthquakes in Japan in 1949-94 was around 850 or approximately 14 per annum. This is extremely low considering the frequency of large magnitude earthquakes and the high population density in Japan.

Similarly, damage to buildings in post-1948 earthquakes has been quite limited. In fact two-thirds of the loss of life in the 46 years between 1949 and 1994, was not due to building collapse or fire, but due to tsunamis and landslides in earthquakes that occurred in 1960, 1983, 1984 and 1993. The only earthquakes that seriously affected engineered structures were in 1964 in Niigata, in 1968 in Tokachi prefecture and in 1978 in Miyagi prefecture (Sendai city). These three events have triggered revisions of the Japanese seismic code in 1971 and 1981, that further contributed to the shielding of the country's building stock from strong ground shaking. A detailed description of Japanese building types and brief details on the historic evolution of Japan's seismic codes is presented in Chapter 4. As a result the structural engineering and building related community of Japan enjoyed a long period of confidence and justifiably it was quite proud of its contribution in reducing the loss of life due to structural failure in earthquakes, down to only 6 lives per annum.

In this context it is therefore not surprising that the devastation experienced in the cities of Kobe, Ashiya and Nishinomiya, shocked most people, including many in the earthquake engineering community around the world. Undoubtedly this is one of the most important earthquakes in modern

[1] The only exception was again in central Japan that suffered heavy losses during the September 1959 Ise-bay typhoon (near Nagoya in Aichi prefecture) that caused 5,100 deaths and destroyed 154,000 buildings during a devastating typhoon related storm surge.

Japan, and lessons learned will be of use to all civil, structural and earthquake engineers, practising in Japan or other seismic environments around the world.

More specifically in the field of seismic risk management the detailed analysis of the performance of the various building types, will contribute to a much better understanding of the vulnerability of Japan's building stock. This knowledge will be extremely useful in future risk assessment studies, especially in cities that might have similar building typologies and are expected to experience similar seismic intensities in the future.

5.2 The performance of buildings revealed by damage surveys

EEFIT carried out two surveys in the worst affected area, as a comparative tool to other more detailed damage surveys that will be published in the future. Two different areas were chosen, in order to cover a wide spectrum of building types found in the affected region. These were:

(a) Nishinomiya-Ashiya: this is a typical suburban area where a large number of low-rise timber frame residential and commercial buildings as well as many mainly commercial low-rise steel frame structures could be found, along with low and mid-rise reinforced concrete apartment buildings of varying age;

(b) Sannomiya district, Chuo ward (Kobe): the central business district of Kobe city, where a large number of mid and high-rise commercial structures of reinforced concrete (RC), steel-reinforced concrete composite (SRC) and steel construction are concentrated.

The survey areas were selected to be inside the Japan Meteorological Agency's JMA intensity zone 7 that was published several days after the earthquake as well as near strong-motion recording stations (for a brief description of the JMA intensity scale see Appendix A of this Chapter). The surveys took place between January 22 and 25, 1995, before any major demolition. Hereafter a brief qualitative summary of the author's findings is presented.

Japanese reports that contain further information on the subject are:

- Architectural Institute of Japan (AIJ, 1995);
- Disaster Prevention Research Institute of Kyoto University (DPRI, 1995);
- Japanese Building Research Institute (BRI, 1996).

5.2.1 Survey in Nishinomiya-Ashiya area

The survey took place in Nishinomiya and Ashiya along Route No. 2. In this survey every building (including buildings that were undamaged or slightly damaged) on either side of the road was photographed. Subsequently details about their construction type, age, damage level and estimated cost of repair were logged into a database. The survey started at the intersection of Route No. 2 with Road No. 82 in the centre of Nishinomiya city and continued westwards for about 6 kilometres, up to the Central Motoyama district in Higashi Nada ward of Kobe. An area of fire damage to a neighbourhood of old timber dwellings was not included because it was impossible to count the exact number of buildings.

Route No. 2 in this stretch runs almost parallel to the shoreline and lies approximately 1,000 metres north of the position of the seashore in the 19th century. During the 19th century the survey area consisted almost entirely of paddy fields. Several thousand years ago this area was actually on or very near the shore. The land between the old and 19th century shorelines is characterised by thick silty soil deposits (marine clays). Land reclamation started from the Meiji period (late 19th century) and accelerated during this century (Taiyo and Showa period). As a result this stretch of Route No. 2 now lies approx. 2,200 metres from the present-day shoreline. The area lies around 30 metres above sea level and the geology is a diluvium-alluvium transition area, with most of the 6km stretch lying on diluvium. Details on the geology of the area are given in BRI (1996).

The survey area lies mostly within the JMA intensity 7 zone. The level of ground motion experienced can be described by 5 strong motion records around the survey area. The strong motion data (peak acceleration and peak velocity), location, soil type and other relevant information from the five recording stations listed from east to west, are summarised in Table 1. Figure 5.1 shows the approximate location of the survey, as well as the location of the strong motion stations listed in Table 1, with the value of peak horizontal and peak vertical velocity of each record in brackets. Of these only one station (KOB Motoyama Primary School) lies very close to the survey area, (near the western end of the survey), but unfortunately it went off-scale in the earthquake (BRI, 1996).

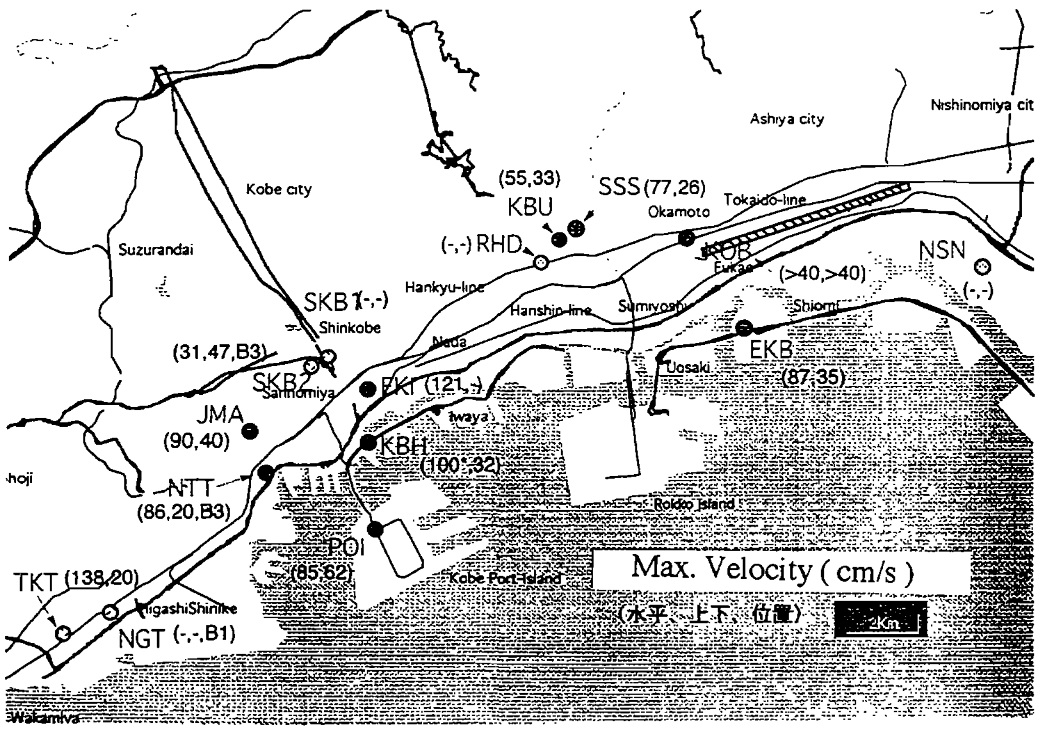

Figure 5.1: Location map of the strong motion recording stations in the affected area (values in brackets are peak horizontal and peak vertical velocity respectively; Source: BRI, 1996). Location of the Nishinomiya-Ashiya survey is also indicated.

Table 1: Strong motion data in the vicinity of the Nishinomiya-Ashiya EEFIT damage survey area.

Strong Motion Station	Compon.	Acceler. (cm/s2)	Velocity (cm/sec)	Location vs. Survey area	Soil Type (other information)
NSN Nishinomiya Supply Station	NS EW UD	 792 		2000m SE off the eastern start	made ground (fill) (1900-1940 reclamation)
EKB Higashi Kobe Bridge Ground Level (Ashiya)	S78W N12W UD	281 327 395	81 87 35	1500m S off the western end	made ground - offshore island (post-1950 reclamation)
KOB Motoyama Primary School (H. Nada; aka Fukae)	NS EW UD	421 775 379	 >40 	western end	diluvium & alluvium border (19th century paddy fields)
KBU Kobe University (H. Nada)	NS EW UD	270 305 430	55 31 	1500m W off the western end	hard ground (near Rokko outcrops)
SSS Shin-Kobe Substation (H. Nada)	NS EW UD	584 511 495	77 64 26	1000m W off the western end	weathered soils (19th century paddy fields)

Source: Compiled from EFTU (1995) and BRI (1996).
Note 1: it was not possible to obtain the time histories of the Nishinomiya record
Note 2: the Motoyama record is reported as being out-of-scale (BRI, 1996).

An important aspect of this survey was to establish an understanding of the comparative performance of the various building types, by load-bearing structure and wherever possible by age and height. This was easily achieved in the case of low-rise domestic timber frame buildings, but was more difficult in the case of reinforced concrete, steel and other structural types. In this quick street survey it was very difficult to differentiate between RC and composite steel reinforced concrete (SRC) structures[2]. In Japan most SRC structures are more than 8 storeys high, while in the survey area only recently built buildings were of this height. It is believed that among these recent RC buildings there were a few SRC structures as well as a few combined RC-SRC structures. The author has good knowledge of building typologies in Japan, so it was possible to assign an age estimate to each RC building in the survey, although it is expected that some errors are included in this judgement. Reinforced concrete buildings were grouped as pre and post-1981 structures, in order to take into consideration the effect of the major Japanese code revision of that year (see Chapter 4, for comments on the Japanese earthquake codes). Ideally pre-1981 reinforced concrete buildings should be further split into two age subgroups, i.e. pre-1971 and 1971-81, but this would introduce too many errors of judgement and was decided not to be included in the analysis. Steel and light metal frame structures were not differentiated by age. In the survey area many older commercial buildings with steel structure were found. In most cases these were 2 to 4 storeyed steel frames of smaller dimensions clearly based on older earthquake code requirements and built at times when steel was a very expensive commodity in Japan.

In order to estimate the likely losses by structural type, 10 damage levels were defined and a likely value of repair cost proposed as a percent of the building's value. The damage levels are described in Table 2. Damage levels and related repair costs were assigned after consultation with the MM, MSK, EMS intensity scales, as well as other work in this field (Scawthorn, et. al. , 1981; ATC, 1985). A range of costs per damage level was also assigned to account for the expected variation. Due to the narrow-long shape of the survey area, spatial damage severity variations were observed but as a whole these were never too great.

Table 2: Damage levels used in the damage surveys (for RC, SRC, combined RC-SRC, Steel structures).

Damage Level	Brief Damage Description (for RC and Steel structures)
0-1	No visible damage
1	Fine non-structural boundary cracks
1-2	Window and (or) limited cladding damage
2	Structural cracks (<10mm); extensive non-structural cracking
2-3	Local concrete spalling; Displacement of unanchored cladding
3	Significant damage to columns, beams, joints; Collapse/failure of unanchored cladding
3-4	Buckling of vertical members, extensive spalling
4	Partial or total collapse of a mid-floor
4-5	Total collapse of more than one mid-floor; Ground floor collapse
5	Complete collapse

The worst damage was found among the old low-rise timber frame dwellings. Japanese domestic timber dwellings are described in Chapter 4, along with a summary of the reasons for their poor performance. Significant damage was also observed in all of the older steel buildings, including severe damage to their cladding. These were mostly 2-4 storey pre-1970s commercial or mixed-use buildings. Post-1981 reinforced concrete buildings suffered much less damage than their older counterparts, and no collapse was seen in the survey area. The collapse ratio among pre-1981 RC buildings was close to 10%. The average loss of indoor space volume in these collapsed RC buildings was assessed at around 20%. The severity of damage as a whole was lower than that found in the central Kobe (Sannomiya) survey, briefly described in the following section.

In addition after the completion of the survey a smaller survey across the area of worst damage was carried out in order to investigate the attenuation of intensity across the fault rupture. This confirmed

2 For detailed descriptions of these building types see Chapter 4.

the findings of other groups (DPRI, 1995) that the severity of ground shaking attenuated rapidly in the N-S direction across the high damage areas that extended for about 20-km in the W-E direction. The worst damage area that was around 1 to 1.5-km wide, in the N-S direction.

5.2.2 EEFIT survey in Sannomiya district

This survey took place in Sannomiya district in Chuo ward, where Kobe's Central Business District is situated. In this survey every building (incl. buildings that were undamaged or slightly damaged) was photographed and logged into a database. The survey covered the area to the south of the Japan Railways Sannomiya and Motomachi stations, until the sea front. The Kobe City Hall and the Flower Avenue lie roughly in the centre of the surveyed area. The exact location is shown in Figure 5.2.

Flower Avenue runs along the old banks of River Ikuta on an area of sandy alluvium. River Ikuta has been diverted eastwards to avoid flooding in central Kobe. The entire survey area is seaward of the Osaka bay shoreline as it was several thousand years ago. Since then conglomerate soils were washed down the steep slopes of the Rokko mountains, and transported by the Ikuta river and other streams and caused the land to expand southwards into the Osaka bay. The area lies around 15 metres above sea level and lies entirely within the alluvium zone. The northern limit of the survey area, i.e. Japan Railways Sannomiya and Motomachi stations lie almost exactly on the border between the alluvium and diluvium zones.

The level of ground motion experienced in the area can be described by 3 strong motion records near the survey area. The strong motion data (peak acceleration and peak velocity), location, soil type and other relevant information are summarized in Table 3. Figure 5.1 shows the location of these stations with the value of peak horizontal and peak vertical velocity of each record in brackets. The survey area lies entirely within the JMA intensity 7 zone.

Table 3: Strong motion data in the vicinity of the Sannomiya survey area.

Strong Motion Station	Compon.	Acceler. (cm/s2)	Velocity (cm/sec)	Location vs. Survey area	Soil Type (other information)
FKI Fukiai Supply Station	N120W N30W UD	687 802	57 121	1200m NE off the survey centre	made ground (fill) (1900-1940 reclamation)
JMA Kobe Kaiyo (Marine Meteorological Observatory)	EW NS UD	617 817 333	74 90 40	1500m NW off the survey centre	hilltop (moderate damage to old timber houses near the station
KBH Kobe Port Office	E43N N43W UD	205 502 283	34 100 32	1000m E off the survey centre	made ground (fill) (pre-1900 Meiji reclamation)

Source: Compiled from EFTU (1995) and BRI (1996).

It is seen that in the 2 stations most representative of the soil conditions in the survey area (Fukiai Supply Station and Kobe Kaiyo) the mean peak horizontal ground acceleration was 715 and 745 cm/sec^2, while the velocity was in the range of 80-90 cm/sec.

Damage in this survey was more severe, than in the Nishinomiya-Ashiya survey. There were many collapsed buildings and many more with severe structural damage. The large majority of the collapses were among the pre-1981 RC buildings. Only 2 steel frame buildings collapsed in the area, but this is largely because there are fewer old steel structures. The old commercial centre with large concentration of mid-rise steel and light metal frame structures was just north of the survey area (north of the two Japan Railway stations).

There were three collapse typologies:

- collapse of the ground floor, incl. overturning
- collapse of one or more middle floors
- collapse attributed to pounding or other causes

There were many mid-height collapses, of the type as yet only seen in Kobe (most mid-height collapses in the 1985 Mexico city were attributed to pounding with adjacent properties). This is a specific failure pattern in some cases clearly associated with combined RC-SRC structures, at the point where the structural type changes from SRC frame to RC frame (for further discussion on this subject read Chapter 4). However not all mid-level collapses were attributable to this cause. In at least three cases, the mid-level collapse was clearly due to changes in the vertical stiffness distribution of the structure, such as a large change in the amount of openings. The possibility of severe vertical shaking contributing to this failure pattern could be an additional factor.

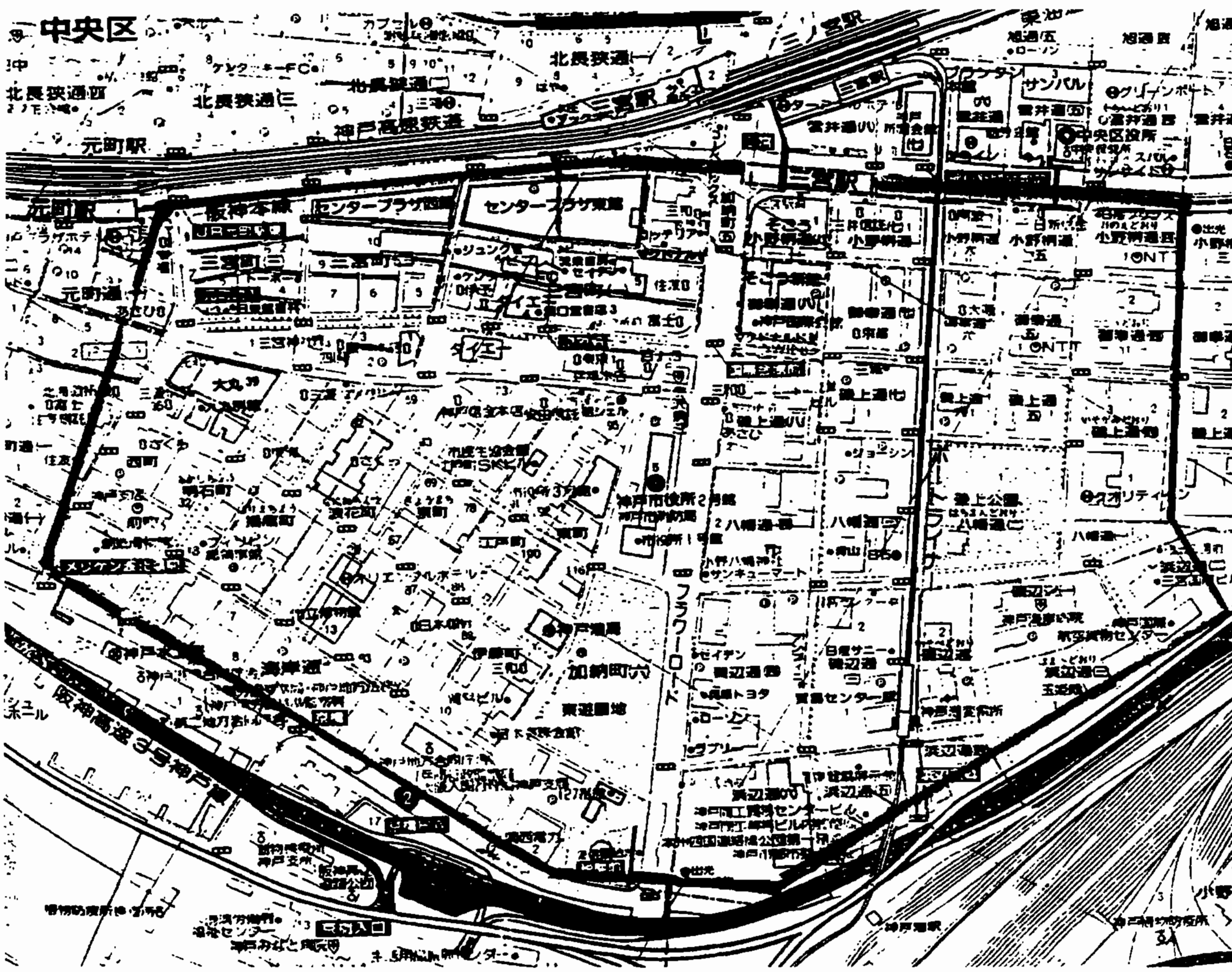

Figure 5.2: Location map of the damage survey in Sannomiya.

It was also found that the average loss of volume differed with the type of collapse. The highest volume loss was 22% and was associated with ground floor collapses. The mean volume loss in the 12 mid-height collapses was 18%, while in the other cases it was 10%. These volume losses compare favourably with experience from other RC collapses in other parts of the world, where the average volume loss usually ranges from 50 to 100% (pancake collapse pattern, often seen in S. Europe, Turkey, Armenia and Latin America). It means that if these buildings were fully occupied at the time of the earthquake the potential for loss of life would be lower, than commonly associated with collapsed RC structures.

Regarding non-domestic construction some of the features of the buildings in Kobe that caused many of the mid-storey collapses do not occur or are quite rare in other countries, i.e.

- sudden major changes in stiffness of vertical structure

- use of uniform distribution of horizontal loads up the buildings (not used in New Zealand and USA since 1965)
- butt-welding of vertical bars in RC columns
- lack of proper anchorage of ends of confinement steel in RC columns
- use of steel-reinforced concrete composite structures

The main reasons for the high collapse ratio of the buildings in Kobe can be briefly summarized as follows:

- soft storey effects (either at ground or at mid-floor level)
- short column effects
- insufficient connection of foundation to the superstructure
- irregularities in plan and elevation (e.g. setbacks, adjacent lower building portions)
- structural stiffness discontinuities (e.g. change in column cross-section, interruption of a shear wall along the height of a building, use of composite steel-reinforced concrete system only in the lower part of a building)
- contribution of high vertical accelerations to damage.

A detailed presentation of types of failures, case studies and photos is presented in Chapter 4.

5.3 Comparison with damage experience in other earthquakes

The amount of damage to reinforced concrete buildings in Kobe can be compared with that experienced in Mexico city during the September 1985 earthquake, Kalamata city (Southern Greece) during the September 1986 earthquake, Erzincan city (Eastern Turkey) during the March 1992 earthquake. The data for Mexico city comes from further analysis of the EEFIT mission findings (EEFIT, 1986) and is representative of zone IIIa of Mexico city, which was the area of highest damage during that event and has been delineated after detailed damage surveys (Seed, et. al., 1986). The data for Kalamata are obtained from the author's personal archives of detailed damage data by administrative subdivision. The intensity was estimated as VIII to IX and the damage statistics are representative of the area nearest to one of the town's strong motion recording stations (Messinia prefecture building). The data from Erzincan come from a detailed damage survey in the vicinity of the only strong ground motion station in Erzincan and are described in detail in the corresponding EEFIT report (EEFIT, 1993). The damage to RC buildings in that town as a whole was quite severe, but within the 500 metres radius of the recording station the damage was surprisingly lower. This was mainly due to the location of the station (local geology) and the type of RC buildings found near the station (low-rise 2 or 3 storeyed buildings), whereas all the collapsed buildings in other parts of Erzincan were larger, mid-rise apartment blocks, commercial and mixed-use buildings and hospitals (EEFIT, 1993).

The RC buildings in Kalamata and Erzincan are in-situ, often non-ductile and irregular frames with unreinforced infill masonry and were designed for base shear of 6 to 8%, similar to those in Mexico city, although smaller in size.

The proportion of partial and total collapse in Sannomiya was the highest, exceeding the respective ratios experienced by mid-rise RC frame buildings in the highest damage zone (IIIa) in Mexico city. The intensity of shaking in Sannomiya was of course more severe than in all the other events. The difference in ground motion severity is best seen when the acceleration response spectra from these events are compared. This is shown in Figure 5.3, with the JMA Kobe Kaiyo record thought as the nearest representation of the level of ground motion experienced.

The Motoyama Primary School record might not be representative of the true ground motion in the area, because it was reported afterwards that the instrument had been off-scale (BRI, 1996). It is quite likely that the Motoyama spectrum would be somewhat less severe than the Sannomiya, when the damage experience is taken into account. An averaged synthetic spectrum for Mexico city's zone IIIa is also shown, as presented by Seed et. al. (1986). In addition the spectra for the near-field shallow earthquakes of Kalamata and Erzincan are also shown. The Sannomiya record is clearly the strongest in the period range of 0.20-1.75 seconds. At longer periods the spectral accelerations experienced in Mexico city zone IIIa and in Erzincan were stronger. These data are part of the author's strong ground motion damage potential database, including an additional 15 strong records (representative of MM intensity VI to X+) in various soil profiles and at least 8 countries. For more details on this work refer to EEFIT (1993) and Spence et. al. (1992).

The main conclusion that can be drawn from this study is that the extremely severe ground motion similar to the one experienced in Sannomiya, is probably equivalent to intensity X or even higher in the Modified Mercalli scale. Under such ground motion severity, damage to older reinforced concrete and steel frame buildings can be severe and collapse rates can exceed 10%. Such high damage ratios to engineered buildings have not been experienced before in western industrialised countries. Around the world there are many cities that are likely to experience similar intensities in the future like: Tokyo, Yokohama, Nagoya, Los Angeles, San Francisco, Anchorage, Wellington, Concepcion, Valparaiso, Taipei and others. Seismic risk assessment in these areas must take into account the lessons learned from the earthquake in Kobe.

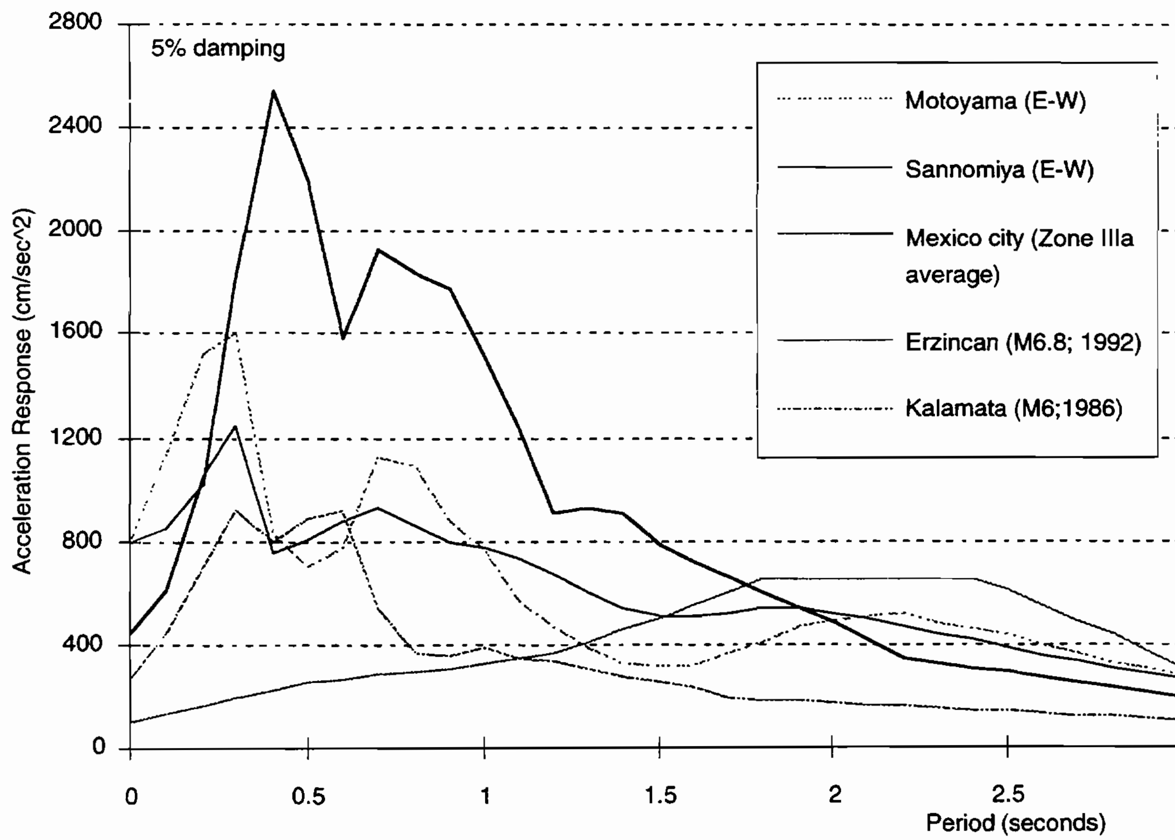

Figure 5.3: Response spectral accelerations (5% damping) in Sannomiya (JMA Kobe Kaiyo) and Motoyama (KOB Motoyama Primary School) compared to those of other recent destructive earthquakes. Their locations are shown in Figure 5.1.

5.4 Fire following earthquake

The earthquake caused 294 fires, destroying 66 million m^2 of residential and industrial area, 90% of which was in Hyogo prefecture (UNCRD, 1995). According to data presented by the New Zealand Reconnaissance Team (NZNSEE, 1995) about 40% of fires in a survey of fire causes were due to short circuits of electricity and about a third due to gas leaks. It was also noted that more than half of the fires occurred more than two hours after the earthquake. Building collapse causing fire has a smaller probability compared with fire being caused by ignition from naked flame sources. Figure 5.4, shows that the proportion of wooden dwellings that were fire proofed in Kobe city and the rest of Hyogo prefecture had increased in the last twenty years but still remained quite low, compared to other cities like Tokyo where around 75% of wooden dwellings are made with wood treated with fire retardants.

The largest conflagrations in the western wards of Kobe were in areas where most of the buildings were old traditional timber houses and three-storeyed light metal frame structures for commercial use, in neighbourhoods with narrow streets. The conflagrations occurred because there was a lack of water and fire fighting intervention. It is estimated that the lethality ratio amongst the residents of these unfortunate areas was much higher than in other parts of the affected area. People were trapped under collapsed houses while they were left to burn out. A brief summary of statistics about the areas where fire conflagrations occurred is shown in Table 4.

Table 4: The effect of fires on the worst affected wards of Kobe

Ward	Surface (km^2)	Popul. Density (p/km^2)	Fatality Ratio (/000)	Fatalities per 100 househs	Burned Area (km^2)	Burned (% area)
Higashi-Nada	29.45	6,356	6.42	1.59	0.039	0.13
Nada	31.21	3,907	6.70	1.53	0.028	0.09
Hyogo	14.42	8,255	3.28	0.71	0.012	0.08
Nagata	11.51	10,985	5.61	1.33	0.503	4.37
Suma	28.4	6,503	1.78	0.50	0.306	1.08

It is seen that the worst affected areas were Nagata and Suma ward. The total number of buildings that were affected by fires was 7,538, or about 5% of the buildings that collapsed completely or partially due to ground shaking. The fact that up to 70% of houses in Kobe had metered gas which was cut off automatically by the shock contributed to the limitation of fire breakouts.

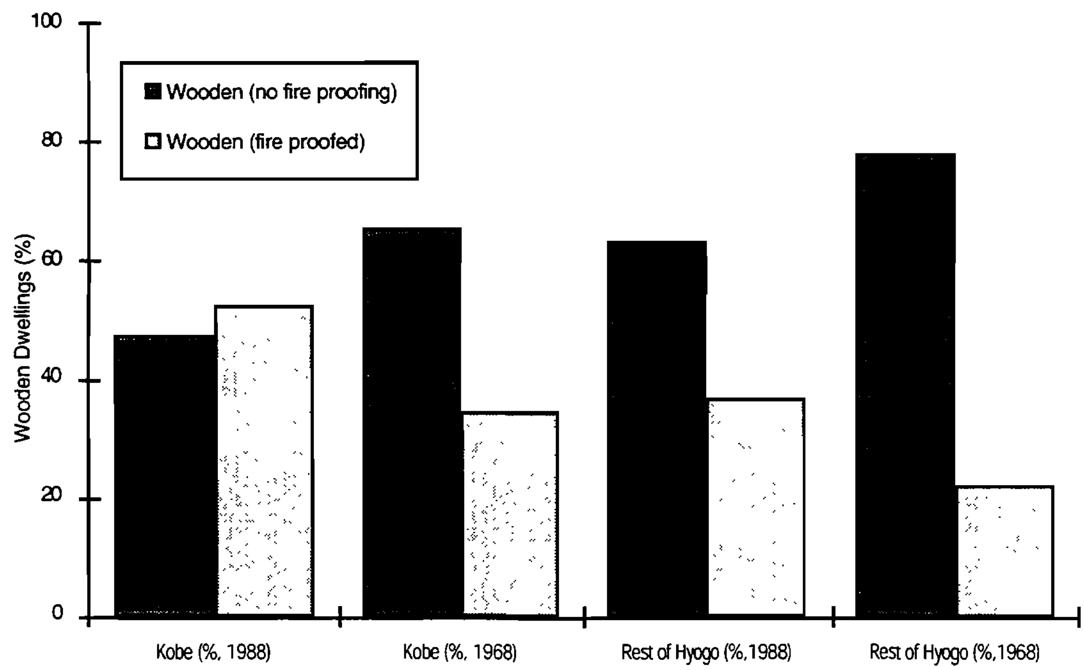

Figure 5.4: Proportion of fire-proofing wooden dwellings in Kobe city and the rest of Hyogo prefecture in 1988 and 1968 respectively.

5.5 Human casualties

According to casualty statistics published in late May 1995, the official number of deaths was 5,502 people. In addition 1,819 suffered serious injuries and 39,708 suffered lighter injuries (UNCRD, 1995). In this report we have attempted to estimate the fatality ratio in the areas of worst damage (areas of MM10 or more). Table 5 shows the summary of population, population density, life loss and fatality ratio (per thousand) in the administrative areas that experienced the worst damage.

Table 5: Loss of life in the worst affected wards of Kobe and other towns

TOWN/Ward	Population	Density (p/km2)	MMI10 (% area)	Fatalities	Fatality Ratio (/'000)
Nagata-ku	126,439	10,985	12	709	5.6
Hyogo-ku	119,032	8,255	12	390	3.3
Suma-ku	184,692	6,503	10	329	1.8
Higashi Nada-ku	187,193	6,356	20	1,202	6.4
Chuo-ku	104,331	4,801	22	211	2.0
Nada-ku	121,937	3,907	15	817	6.7
MMI 10 WARDS	843,624	6,170	15.6	3,658	4.3
Ashiya	85,736	4,976	17	400	4.7
Nishinomiya	412,463	4,163	9	884	2.1
Takarazuka	203,105	1,996	6	85	0.4

Note: The fatality ratio in MM 10 areas was obtained assuming uniform population distribution across the ward/town.

The fatalities for each area shown in Table 6 are those published in late January 1995, when the total life loss was estimated as 5,100. It is seen that about a sixth of the total area in the worst affected wards of Kobe city and in Ashiya city was in the JMA 7 zone. Assuming uniform distribution of the population across these administrative areas, it was estimated that around 146,000 people lived in these areas. It has been reported that around 90% of the loss of life occurred in collapsed old timber frame dwellings that were predominantly in the zone of extreme damage. It can therefore be commented that around 2-3% of the people in the areas of MMI10-10.5 were killed. In Nagata-ku were serious fire outbreaks occurred the loss of life in the area of extreme damage probably exceeded 4%. In the predominantly commercial Chuo ward this ratio was around 1%. The fatality ratio corresponds well with the proportion of pre-1960 dwellings (as shown in Table 4 of Chapter 2) in each ward.

Most of the early effort of Japanese research in this area was to find the proportion of fatalities by age. It was soon established that one of the characteristics of this earthquake was that it affected disproportionately the elderly population. More than half (53%) of the fatalities, were people aged over 59, but the proportion of people in this age group in Japan is 19.3%. Figure 5.5 shows the age distribution of Japan, based on 1993 data, derived from the 1991 population census[3] , against the age distribution of the earthquake fatalities.

The reason for this was that around 90% of the life loss occurred in old timber dwellings that collapsed or burned down. Most of these dwellings were occupied by elderly people, who slept on the ground floor and thus were trapped under the debris. The working population (20 to 59 years old) suffered 37% of the life loss in this earthquake, while the proportion of Japanese people in this age group is 57%.

It has also been established that only 10% of the life loss could be attributed to post-earthquake fires. Finally it was found that loss of life due to collapse of civil engineering structures like bridges and elevated railways and highways was quite small, possibly less than 5% of the total.

It has been speculated that if the earthquake occurred 2 to 4 hours later the loss of life would have been greater. The reasons for that are:

- hundreds of large occupancy non-domestic buildings collapsed, that were empty at the time;
- several kilometres of elevated motorway and railway lines collapsed just before the morning commuter rush time started;
- the response to the earthquake disaster was slow in the first three days and it would have been even more difficult if hundreds of search and rescue operations would have had to be set up in all the collapse sites of commercial and other multi-occupancy buildings

[3] The age distribution of the affected area is not significantly different to the national average.

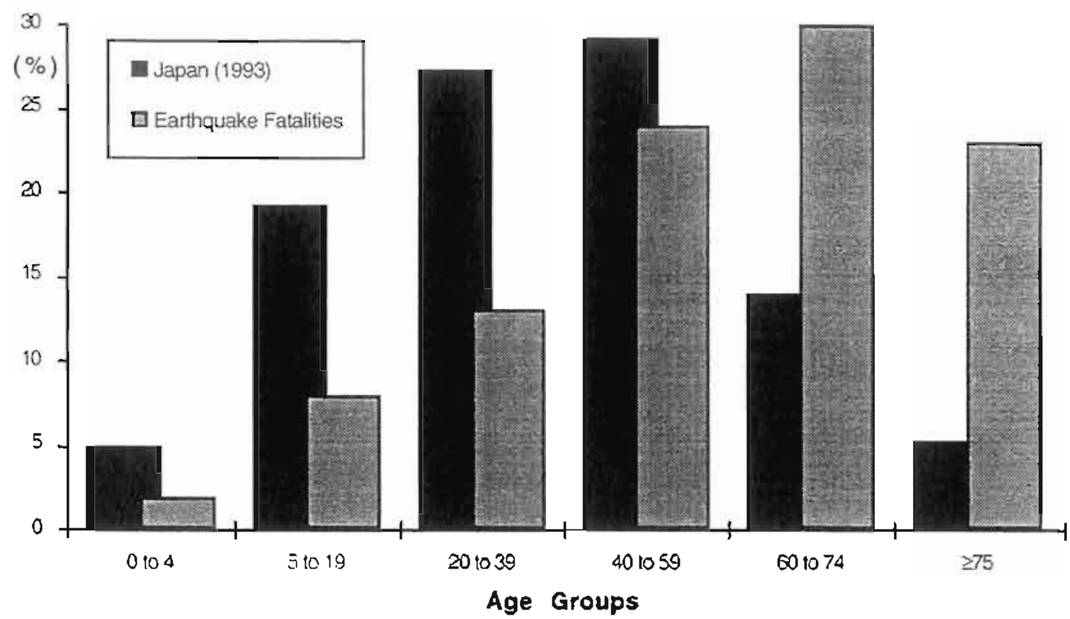

Figure 5.5: The elderly population was disproportionately affected.

Studies of collapsed RC buildings in previous earthquakes have shown that 20 to 90% of the occupants are known to have died. The collapse typology, volume loss and effectiveness of search and rescue activities were found to be the most important factors (Pomonis, et. al., 1991).

As presented in section 5.2.2 the loss of volume in the collapsed mid-rise RC or steel buildings was on average around 20%, not as high as commonly experienced in collapses in other parts of the world. Therefore the potential number of people trapped inside them would be lower. In order to estimate what the life loss would have been had the earthquake occurred during the morning commuting hours or during office hours, data on the occupancy of collapsed commercial buildings that were empty at the time of the earthquake will first need to be assembled.

This is an important area of research as proven by the large uncertainty associated with the predictions of casualties during a future earthquake in Tokyo. Studies on this subject predict loss of life ranging from 9,000 to 100,000 for daytime earthquake scenarios (RMS, 1995).

5.6 Lifelines

Damage to the lifeline network, including electricity, water and gas supply systems, was quite severe and the direct damage cost has been estimated at US$ 5 billion.

5.6.1 Electricity supply network

Electric power supply was interrupted throughout the affected area, including parts of Osaka-fu. The total damage was estimated at US$ 2.3 billion. The Kansai Electric Power Co., Inc. reported that about 1 million of its customers located mainly in Kobe and in the surrounding areas lost power supplies after the earthquake[4]. Power was restored to about half the affected customers within the first day, but it took around a week for a complete restoration. Six other regional electric utility companies provided assistance by sending 100 vehicles, including 46 mobile generator vehicles, and 295 personnel for restoration activities and other assistance. More than 4,700 men were mobilised for the

[4] Excluding buildings that collapsed completely or partially, burned down.

quick restoration of power. As a result only a limited number of customers was without supply by January 23rd, 1995 (KEPC, 1995).

Nine 275 kV and seven 154 kV substations were damaged, some of which blacked out. None of the 500 kV substations was affected. In addition thirty two 77 kV substations were damaged, but were all restored by January 18. The most common damages to the substations were:

- break of bus disconnecting switch
- damage to the pressure relief plate of transformers (oil leakage)
- bushing cracks to transformers
- overturned lightning rods

One third (12 of 36) of the units in fossil power stations that were on line before the earthquake were forced to trip. Seven of these units were restored by January 23rd, 1995. There was no damage reported to Nuclear Power Plants, but ten thermal power plants were damaged.

Damage to the 7,800 miles of transmission lines and 57,614 miles of distribution lines was also significant. The most common damages to these were:

- breakage of insulators
- opening of conductors
- collapse, leaning or subsidence of utility poles.

These were caused by: building collapse, ground subsidence or severe shaking.

The loss of electric power had follow-on effects on many aspects of urban life. One third of telephone lines did not function due to the cuts. Railways and subways were brought to a halt and the loss of traffic signals caused chaos on the roads.

5.6.2 Water supply network

Nine cities and five towns were affected in Hyogo prefecture, with a population of 3.4 million people. About 85% of these people lost their water supply. Thus about 54% of the Hyogo prefecture's population was without water. The loss of water caused severe problems to the fire fighting efforts in the first day after the earthquake. It was estimated that the frequency of damage, expressed in number of damage sites per kilometre length of network, was not unusually high. On average 0.23 damage locations per network kilometre were reported, with Ashiya city having the highest frequency (UNCRD, 1995). However the size of the affected network was very large (4,000 km in Kobe city) and as a result it took five weeks for a complete recovery[5].

The principal obstacles to the rapid restoration of water supply in the affected area were quoted as follows (UNCRD, 1995):

- collapse of the Kobe City Hall; the collapsed 6th floor was the headquarters of Kobe City's Water Supply and Sewerage Department, where all the data and drawings of the network were stored
- insufficient water pressure, for use in locating leakage points
- over dependency on water supplies from Yodo river (74%) affected by broken pipes

More than 60% of the damage occurred in joints, mostly by "pulling-out". A small proportion of joints was estimated to be of earthquake proof design. More than 90% of the pipelines were made from ductile steel.

There was sufficient water supplied by means of water tank rollers. Other regional water companies contributed 840 water tank rollers and more than 2,000 technicians to help the Hanshin Water Supply Cooperation in the period after the earthquake.

5.6.3 Gas supply network

Osaka Gas Company is the supplier of gas to 5.7 million customers in the Osaka-Kyoto-Kobe region. It was reported that 856,000 customers lost their supply of gas as a result of the earthquake. At least twenty fires were caused by gas leakage, some when electricity was re-installed. There were no gas explosions. There was no damage to gas production plants or high pressure pipelines. However

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[5] Excluding restoration of water supply to more than 100,000 collapsed buildings.

breakage occurred in middle and low pressure pipelines. Most of the leakage occurred in joints that did not allow for sufficient relative displacements. Polyethylene pipes performed quite well.

The restoration of gas supplies was a very difficult task and 8,300 personnel were involved in the operation. This is because the typical restoration procedure is complex and time consuming. The affected area had to be divided in small zones, where the supply was cut-off at every house and the whole zone was also isolated from the neighbouring zones. After that gas pipe breakage had to be located and repaired. Supply is re-installed after it is made certain that there are no leakage in the area.

5.6.4 Telecommunication network

On January 17 and 18, 1995 it was reported that the amount of telephone calls was about 20 to 50 times bigger than on a usual day (UNCRD, 1995). About 285,000 circuits were lost as a result of loss of power or due to damage to equipment and (or) cables. This was a loss rate of 20%. Nippon Telegraph and Telephone Company (NTT) suffered structural damage to 3 of its buildings, one being severe. The transmission towers on the roofs of these buildings were also damaged, and one had to be removed. The total damage to the NTT was reported at US$ 300 million. The restoration work was manned by 7,000 personnel, 4,000 on loan from other companies. Eleven portable and six mobile satellite stations were mobilised to help with the increased demand. Around 70% of the cut circuits were restored in the first day. The restoration then tackled the remaining most difficult cases, and continued for two weeks before completion[6]. The NTT supplied 2,255 free telephone lines and 361 free facsimile lines.

5.7 Effects on the insurance industry

5.7.1 Earthquake insurance options at the time of the earthquake

Insurance premiums represent 9-13% of Japan's GDP (depending on different accounts), a level exceeded only by the US and the UK. The average person in Japan has three times as much cover as an American. Insurance premiums in Japan amount to 22% of the global market, and the country's share of the world's life policies is 30% (Burke, 1996). Only 4% of this market is underwritten by foreign companies. The earthquake insurance laws of Japan are complicated and this section will only discuss the main aspects of it and summarise the latest knowledge on the losses to the insurance sector.

Property earthquake insurance in Japan is available since 1956 for commercial buildings and 1966 for residential buildings. Cover for fire following earthquake is only available since 1984. Home-owner earthquake insurance covered only 3% of households in Hyogo prefecture, as against 16% in Tokyo and 7.2% on a national average basis[7] (for comparison the household earthquake insurance cover in Los Angeles in 1994 was 35 to 40%). Premiums for domestic earthquake insurance in Hyogo prefecture were 0.14 or 0.3% of sum insured (depending on two building vulnerability classes). For non-residential buildings the premiums ranged from 0.3 to 1.4% (depending on five building vulnerability classes). Considerable increase in demand for home-owner earthquake cover was reported in the weeks after the earthquake. As a result the average national earthquake insurance take-up has reportedly increased to around 10%, being close to the largest ever recorded levels. It has been commented that this is the upper limit of earthquake insurance take-up for domestic property in Japan (CRN, 1996). Japan's Ministry of Finance is currently considering whether or not to make earthquake household insurance compulsory.

An additional earthquake insurance option is available to Japanese farmers through the National Mutual Insurance Federation of Agricultural Co-operatives (Zenkyoren). This is a mutual aid scheme that provides its members with earthquake coverage of 50% of the fire coverage up to a limit of ¥ 50 million (Alexander Howden Group Ltd., 1995).

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[6] About 38,000 cuts were not reinstalled associated with collapsed buildings etc.

[7] By contrast around 57% of Japan's residential buildings are insured for fire (in the UK 64% of households have property insurance offering cover against all natural hazards, theft and fire). However insurance cover for wind damage is far more common in Japan, with up to 90% of buildings having some kind of cover.

Foreign insurers mostly cover foreign owned properties, marine risks and foreign business interruption due to earthquake.

5.7.2 Estimation of insurance losses

According to the Marine and Fire Insurance Association of Japan (MFIAJ, 1995) immediately after the earthquake an emergency management centre for loss adjustment was established at its Osaka regional office. Most loss surveys on earthquake insurance policies were finished by the end of February 1995 and claims paid by the end of March 1995. The total amount of claims paid was between ¥120 and 130 billion (US$ 1.2 - 1.3 billion at 1995 rates)[8]. Classified by type of coverage, claims paid from the Earthquake Insurance on dwelling risks policies amounted to about ¥76 billion. This was the largest loss since the introduction of the earthquake insurance scheme 30 years ago. The remainder was covered by other policies such as Fire Insurance (Earthquake Fire Expenses Coverage), Earthquake Extended Coverage Endorsement on industrial risks, Marine (Hull and Cargo) Insurance and Aviation Insurance. An additional US$ 1 billion was expected to be paid by the Zenkyoren to its members.

Therefore the total estimated insurance loss is about 2% of the total direct losses[9] and 18 to 45% of the estimated underwritten earthquake insurance in Hyogo prefecture (estimates of this range between ¥ 500 and ¥ 1,200 billion).

The cost to the insurance sector was therefore limited as a proportion of the total losses attributed to the earthquake, but significant in terms of annual earthquake insurance premium income. In addition to the low take-up of earthquake insurance, further protection systems were set-up by the Japanese government in association with insurance companies in order to avoid excessive losses. For example earthquake insurance policies only cover up to 30% of a house's building sum insured for fire and up to 50% of its contents (In Tokyo and surrounding region the respective limits are 15 and 30%). Payouts are limited to ¥10 million for structural damage (around one quarter to a third of the average price of a typical house in Japan) and ¥5 million for contents losses. In the case of post-earthquake fire, payments are capped at ¥3 million or 5% of the total fire policy and a claim is only accepted if more than a fifth of the property has been burned. In the case of commercial properties cover is also limited to no more than 30% of the building sum insured for fire. Cover for post-earthquake fire is limited to ¥20 million.

The Japanese government provides settlement assistance of residential claims when these exceed ¥66 billion in one single event, through the Japan Earthquake Reinsurance Company (also formed in 1966). This is done by paying 50% of the layer of ¥ 66-400 billion and 95% of the layer of ¥ 400-1,800 billion. If claims still exceed ¥1,800 billion in any given year, the government is responsible for the whole of the remainder but the indemnities are reduced proportionately (Alexander Howden Group Ltd., 1995). The limit of ¥1,800 billion has been raised to ¥3,100 billion starting from 1996.

The total value of earthquake insurance in Japan is estimated at US$ 143 billion, 80% of which is reportedly reinsured abroad, a large part of it through the London reinsurance market. The total amount of reinsurance claims for the earthquake in Kobe is not yet known.

Life insurance payments were initially estimated at US$ 0.5 to 1.5 billion. In addition the insurance sector suffered a loss of about US$ 250 million, through direct damage to its offices in the area.

5.8 Conclusions

1. Japan has not experienced an earthquake disaster on such a scale for 46 years. The earthquake in Kobe came at the end of this period during which the mean annual loss of life due to building failures in earthquakes was reduced to 6 persons. As a result the extent of damage came as a shock to most people, including structural design engineers and risk assessment specialists.

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8 Initial estimates were as high as US$ 8-10 billion, later adjusted downwards to US$ 2-3 billion.

9 In the 17 January 1994, Northridge earthquake (Los Angeles, California) about 35% of the direct losses were passed on to the insurance industry.

2. Damage surveys in the worst affected area reviewed in this Chapter, have shown that the collapse ratio of mid-rise pre-1981 RC and steel frame buildings was unexpectedly high. In central Kobe as many as 15% of pre-1981 RC buildings collapsed. An analysis of 36 collapsed structures has shown that the average volume loss associated with these buildings was 18 to 22%, depending on the collapse type. This is much lower than that experienced in collapsed RC buildings in other parts of the world.

3. It was found that the severity of ground shaking attenuated rapidly in the N-S direction across the high damage areas that extended for about 20-km in the W-E direction. The worst damage area that was around 1 to 1.5-km wide, in the N-S direction.

4. Unlike the 1923 Great Kanto earthquake fire did not affect a large part of the earthquake area. Only Nagata ward experienced a large conflagration that burned about 5% of its land area. The loss of life attributable to fire was less than 10%. The wind in the morning of the earthquake was very light. Had the earthquake occurred during the typhoon season (July to November) the potential for urban conflagrations would have been much greater.

5. In recent decades there has not been an earthquake to have affected a modern urban area inhabited by 2 million people with intensity X or higher on the Modified Mercalli scale. It was estimated that around 250,000 people lived inside the areas of JMA intensity 7 where the loss of life ranged between 2 and 4%, with the exception of the predominantly commercial Chuo ward where this ratio was around 1%. It was highlighted that the earthquake affected disproportionately the elderly population, because they tended to occupy the older vulnerable low-rise timber houses. It is likely that the loss of life would have been greater had the earthquake occurred 2 or 4 hours later, during the morning commuter rush hour or during office hours.

6. Exactly one year before the earthquake in Kobe an earthquake of similar magnitude (M_S 6.8) struck the northern suburbs of Los Angeles city in California, USA. The worst affected area was Northridge in the San Fernando valley. The earthquake occurred at 4:31 am and caused the loss of 58 lives. The main reason for the large difference in human casualties was that the suburb of Northridge was an area that had been built in recent decades, predominantly with timber frame detached dwellings and apartments. These buildings performed much better and were rather similar to the new Japanese timber frame housing that also suffered limited damage in the Kobe earthquake.

7. Damage to the lifelines has been quite heavy and severe disruption were experienced in all utility services. Electricity was restored relatively fast but fires and explosions occurred when restored electricity combined with leaking gas pipes. The worst lifeline problems were experienced in water supply. Around 85% of the affected population lost their water supply and complete restoration took five weeks.

8. The losses suffered by the insurance industry were lighter than originally predicted. Only about 3% of residential and 30% of commercial property in Kobe area, was insured for earthquake damage. The premium rates for such cover are quite high in Japan. However the earthquake has caused a surge in earthquake insurance business in other parts of Japan. The insurance losses have been estimated at around US$ 1.25 billion, or around 1% of the direct damage. In the 1994 Northridge earthquake in the Los Angeles area, around 40% of the direct losses were passed on to insurance companies. As a result, the Japanese government and the affected people will have to bear most of the damage costs.

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Appendix A: Summary of the JMA Intensity Scale

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

6.0 PERFORMANCE OF BRIDGE STRUCTURES

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6.1 Introduction

The city of Kobe is confined between the sea to the south and a range of hills known as the Rokko Mountains to the north. As the city grew, the conflicting development demands for real estate and for improved transport were accommodated by elevating the trunk roads and railways which run through or close by the city. Figure 6.1 shows the primary transportation routes through Kobe. The transportation structures, mostly built in the 1960s to early 1980s, suffered considerable damage during the earthquake, and are the subject of the first section of this chapter.

This is far from a comprehensive review of the damage to bridge structures; the extent of damage throughout Kobe and the surrounding area, and the time available to the team, ruled out completeness. The text has been kept short, in the belief that illustrations with brief explanations will have more impact, and will be more useful to researchers or to those facing the challenge of designing or retrofitting bridges in seismic zones. The numerous examples of damage to elevated highways and bridges have allowed the authors to divide the coverage into three types; these are usually the main concerns for designers of new bridge structures:

- fallen or shifted decks;
- damage to columns; and
- foundation failure.

The next section in the chapter deals with the record-breaking Akashi-Kaikyo Bridge. The Akashi-Kaikyo Bridge will link Awaji Island to Kobe, and will be the longest suspension bridge in the world when it is completed in 1998. It is adjacent to the epicentre and straddles the fault whose rupture caused the earthquake.

From major to minor: observations of the damage to a selection of small bridges completes the description of damage. Every incidence of severe damage added to the disruption to life in and around Kobe. Apart from the loss of transport links, in some cases other crucial services, such as water supply, were also interrupted.

Generally, the structures which were worst affected are not modern, and to be properly understood the damage needs to be seen in the light of the state of knowledge at the time and the prevailing codes. A short section on Japanese bridge codes has been included to give an appreciation of how requirements have changed over the years.

6.2 The Road Network

The old roads through Kobe are National Highway Routes 2 and 43, and both were originally at ground level. To the east of Kobe, Route 2 is the main road connecting the string of wards and small cities between Kobe and Osaka, whilst Route 43 is the coastal route. To the west of central Kobe Route 43 disappears and Route 2 takes the coastal route.

The original Hanshin Expressway (sometimes called the Hanshin Expressway Route 1) is an elevated highway which runs along Route 2 to the west of Kobe and Route 43 to the east. In 1981 a new two-tier elevated section of Route 2 to the south of Kobe (known as the "Hamate Bypass") was constructed on the coastal side of the old route, parallel to the original Hanshin Expressway. Plates 6.1 and 6.2 show two views of the city while these structures were being built - the light coloured steel box piers can be seen. This new section converges with the old routes just after connecting to the Hanshin Bay Expressway. To the east there is the short Harbour Highway which from 1981 connected the string of berths between Port Island and the mainland, and which was subsequently extended to Rokko Island. The Hanshin Bay Expressway, constructed between 1991 and 1994, is the most recent addition to the network, and runs from Rokko Island to Kansai International Airport.

The Hanshin Expressway Public Corporation was established in 1962[6.1] to construct a series of toll roads and so alleviate the severe traffic congestion resulting from economic expansion. By 1987 138km of elevated highway had been constructed, and at the time of the earthquake this had increased to about 160km. Procurement was on a design/build basis, which explains the variety of structural forms within one route.

The Kobe-Osaka highway was one of the earliest sections of the network to be completed. The Kobe to Nishinomiya section is mainly of steel/concrete composite girder construction. The most common form of construction is I-girder deck, but there are also sections with twin and triple steel boxes and one section near the centre of Kobe of reinforced concrete construction. This latter form of construction is illustrated in Figure 6.2. This 635m long section near the centre of Kobe collapsed, and featured in the sensational early photographs; it is discussed further in Section 6.4.2 below.

6.3 The Rail Network

The railway system is complex because of the number of competing rail companies. Three elevated lines enter Kobe from the Osaka (east) side. Closest to the coast is the Hanshin Railway, parallel to this inland is the Japan Railways (JR) Tokaido Line, and further inland is the Hankyu Railway Kobe Line. Through central Kobe the three lines run side by side and change their names. To the west they cross over, with the JR Sanyo Line taking the coastal route.

Further inland still is the Japan Railways Shinkansen (or New Sanyo) Line which carries the bullet trains. This line is on elevated structure to the east, but bypasses Kobe in a 24km long tunnel which it enters near Nishinomiya, emerging well to the west of Kobe, almost due north of the new Akashi-Kaikyo suspension bridge.

All the lines mentioned above predate 1980, and only the spur line from the JR Sanyo Line onto Rokko Island has been built in recent years.

6.4 Damage to Elevated Road and Rail Structures

6.4.1 Fallen or shifted decks

Under this heading we will cover only those cases where primary failure stemmed from a span breaking free from its support and falling, not where column failure caused collapse. Fully continuous decks cannot collapse in this catastrophic mode, except at joints at abutments. However, full continuity is not feasible for very long viaducts because of the need to cater for thermal movements, although it is not uncommon for modern structures to extend for 500m without joints, and some decks have been made continuous for more than 2km. There are now techniques for sharing longitudinal inertia loads between supports, and loads can also be limited. However, in limiting loads there is usually a need to accommodate larger movements, but these can be moderated by appropriate damping.

Collapses

Given the many kilometres of elevated road and rail viaducts in and around Kobe this catastrophic mode of failure was not common for the regular spans of major elevated structures. In a number of cases fixed bearings had failed or movements at free bearings had been large. Despite the large longitudinal displacements, bearing shelves were generally sufficiently wide to prevent spans parting company completely from their supports. There were exceptions and they are discussed below.

The Hanshin Expressway comprises a variety of designs, but generally there was some limited continuity, say with joints every three or four spans, between movement joints. The Expressway lost spans in a few places due to loss of support at expansion joints (Plate 6.3). Early news photographs showed how fortunate the passengers in a tour bus had been as it came to a halt hanging over the chasm left by the missing span at this location.

Another spectacular example of this catastrophic failure mode occurred at the Nishinomiya Harbour Bridge on the Hanshin Bay Expressway linking Nishinomiya with Osaka. The main span of this major crossing is a bow-string arch of 252m span. A steel/concrete composite girder span immediately adjacent to the main span dropped off the main bridge end pier (Plates 6.4 and 6.5). Liquefaction of the reclaimed ground caused lateral spreading towards the water, shifting the sea wall and the bridge pier, allowing the approach span to fall.

The Shinkansen bullet train line suffered collapses at several locations along the elevated section to the north of Nishinomiya. One of these is pictured in Plate 6.6 taken shortly after the earthquake, and the same dropped span is shown in Plate 6.7 taken by the authors. It is unclear whether collapse was initiated by loss of bearing or by collapse of the reinforced concrete columns. This particular span was simply supported on relatively narrow bearing shelves, with joints at each end. The spans either side appear to have been strengthened by a very deep beam between the columns. This will have had the effect of reducing the length of the columns and thereby stiffening them longitudinally.

In a few cases, curved or straight ramp accesses onto elevated road structures failed by spans losing support, reminding us of the need for special consideration in design and retrofit where geometry or variation in stiffness of supports can focus force and displacement. Plate 6.8 shows the collapse of a curved steel box girder ramp on the Hanshin Expressway to the west of Kobe centre as it lost support at the abutment.

Decks which Survived Despite Large Shifts

Plates 6.9 and 6.10 show the dangerous shift of a curved box girder which forms part of the same interchange illustrated in Plate 6.8.

Plate 6.11 shows the effect of a transverse shift of about one metre of the first span of an on-ramp to the Hanshin Expressway in Kobe. Plate 6.12 shows how the steel girder was jolted off its bearing at the abutment. A little higher up the same ramp the twin girder spans were thrown off their circular column supports, leaving the deck to drop and come to rest with its concrete deck slab sitting on top of the columns (Plates 6.13 and 6.14). Even higher up the ramp as the piers become higher the column moments are such that the failure occurred in the reinforced concrete pier rather than the bearing, and one of these column failures is covered in Section 6.4.2 (and illustrated in Plates 6.27 and 28).

Total collapse due to transverse shifting of decks on their supports is unusual. Normally the transverse displacement compared with deck width means that the deck will not fall. However, in the case of a narrow rail deck the situation can be different, and Plate 6.15 shows an example where a twin I-girder deck has been left hanging from its portal frame pier. Perhaps the missing span collapsed as a result of shifting too far sideways. Robust shear blocks on the pier or lateral continuity of the deck slab would have avoided potential for this mode of collapse.

There were numerous examples where decks had shifted without collapse. Some of the most spectacular examples were on the Route 2 multi-level viaducts which run close to the shoreline near downtown Kobe (Plates 6.16 and 6.17). It was clear that some of the devices to prevent spans falling off their supports, introduced by code requirements (Section 6.7), were not designed for an event such as this. On the Hanshin Expressway plates which had been bolted across joints between steel girders had sheared the bolts when they reached the end of the movement capability of the slotted holes (Plate 6.18). On the bullet train elevated structure a steel fabrication intended to provide lateral restraint had been twisted out of position (Plate 6.19). The large loads and displacements damaged bearings, drainage pipes, other services, expansion joints, and parapets (Plates 6.20 to 6.23).

6.4.2 Damage to concrete piers

The majority of the older elevated road structures follow the line of ground-level highways, and to avoid the carriageway below the supports are usually large single column piers positioned in the central reserve of the road below. To achieve ductility in these massive single columns would have required a lot of transverse reinforcement, either continuous spiral or with legs properly anchored into the column core. Ductility requirements did not receive attention in Japanese bridge design codes prior to 1980 (Section 6.7). The majority of the badly affected structures predate 1980, and so it is not surprising that designers and constructors did not apply what would be considered good modern seismic detailing.

The best known collapse of elevated viaduct was the 635m long section of Kobe Route 3 on the Hanshin Expressway. The 4-lane Route 3 was supported above the 8-lane Route 43 by columns in the central reserve, and the deck collapsed onto the northern carriageway of Route 43. Design for this section was carried out in the early 1960's and construction commenced in 1969[6.1]. The 35m deck spans comprise table-top sections monolithic with the single column piers, and central deck sections spanning 22m between half-joints (Figure 6.2). Restrainers were fitted across the half-joints. This structural form has no redundancy, and the 635m of viaduct collapsed through flexure/shear failure of the brittle 3100mm diameter columns. Plate 6.24 taken shortly after the earthquake shows the mode of failure. Plate 6.25 shows a close-up of the first column of the all-concrete spans near the foreground of Plate 6.24. Reference 6.1 indicates that the longitudinal reinforcement in the columns consisted of three concentric rings of 60-D35 bars up to a height of 2.5m above the top of pilecap, at which point the inner ring was curtailed. The outer two rings of main bars had binding reinforcement of 16mm diameter bars at 200mm centres, while the inner ring had 16mm bars at 400mm. For a column of these proportions this amount of containment reinforcement is inadequate, and it could not sustain the shear loads nor confine the longitudinal rebar and concrete under the large inertia loads from the heavy deck. The flexure/shear failure initiated at the location of the curtailment of the inner ring of main bars.

Apart from the all-concrete section of the Hanshin Expressway, there were many other examples of non-ductile failures of concrete piers. Plates 6.26 to 6.37 is a selection of the many. Often failure occurred at the level of curtailment of main longitudinal reinforcement. In a number of cases the amount of reinforcement was so low that failure occurred along horizontal construction joints, probably where aggregate interlock had not been assured by removal of laitance. Links in failed piers were invariably small, widely spaced and their simple laps meant that they became ineffective as soon as the cover concrete split away.

The Maya Ohashi Bridge carries the Harbour Highway which connects various offshore berths on the sea side of Maya Harbour. Both of the wide main piers supporting the twin steel box girder superstructure were badly damaged. The east pier had failed completely at a change in section near mid-height, which probably also coincided with a construction joint (Plates 6.38 and 6.39). Sliding had occurred at this plane, leading to an estimated 350mm northward drift of the upper part. The longitudinal reinforcement was continuous, but there were no through-wall ties, simply transverse bars which were lost with the cover. The lateral movement on the east bank had occurred due to extensive liquefaction of the bank (Plate 6.40). The approach span pier two away from the east main pier, pictured in the centre of Plate 6.40, suffered severe damage to accommodate the distortion at the main bridge pier (Plate 6.41). At one end of this wide pier gas welds between vertical bars had failed (Plates 6.42 and 6.43). The piers either side of this failed pier were twin column portals, and being more flexible were undamaged.

The west main pier of the Maya Ohashi Bridge showed a shear split (Plate 6.44) as a result of the large transverse load transmitted to the pier from the bearing. To accommodate transverse thermal movements the designer probably restrained the deck transversely at one bearing only, hence all the inertia load would have come on that bearing. Given the magnitude of the forces, it is impressive that the bearings remained intact, and that the steel box girder appeared undamaged.

Alongside the Maya Ohashi box girder bridge there was a single tower cable-stayed bridge to carry local port traffic (Plate 6.45). This structure was undamaged apart from some disruption at the top of the steps.

6.4.3 Damage to steel columns

The older Route 3 of the Hanshin Expressway had both steel and concrete, and circular and rectangular pier columns. The design/build teams had the freedom to use the technology and materials which they favoured. There were examples of collapse or severe damage to both, and so no simplistic conclusions can be drawn. The twisted remains of steel and concrete in the centre of Route 43 just to the east of Kobe in Plate 6.46 bears witness to the demise of a stiffened steel box pier. The deck and pier debris had been cleared, but photographs in references 6.2 and 6.3 show that the pier crosshead came to rest on the concrete filling at its base, i.e. on top of the remains which can be seen in Plate 6.46. The first metre to a metre-and-a-half of the steel box columns were filled and cased with concrete to avoid vehicle collision damage, as can be seen in Plates 6.47 and 6.48 near the centre of Kobe. The damaged area of steelwork was conveniently highlighted by the flaking of the light grey top coat of paint exposing a bright orange undercoat/primer. The vertical stiffeners inside the column show up as dark lines below the orange paint. The extent of the buckle near the base of a column can be seen in Plate 6.49. The freshly flaked paint on the circular column in Plate 6.50 near Kobe centre indicates strong action reversals normal to the line of the viaduct.

The columns of the later and much taller double-decked Route 2 were of either steel box or steel tubular form. They seemed to fare reasonably well; but were designed to a more stringent code. Some distress was noted; for example the circular column in Plate 6.51 viewed along the line of the structure, and in close-up in Plate 6.52 viewed transverse to the line of the structure, shows a very symmetric buckle at a change in column wall thickness. Similarly, the buckling damage to the stiffened box column on the Route 2 viaduct a little closer to the centre of Kobe (Plate 6.53) is likely to be at a change in section at the first panel of thinner material.

To the west of Kobe centre the columns of a narrow ramp structure onto the Hanshin Expressway suffered interesting damage. The ramp was supported on single large diameter tubular steel columns, the tallest of which had buckled just above the reduction in wall thickness (Plate 6.54). The ratio of diameter to wall thickness of the thinner appeared to be at least 100, well above compact section proportions. Under earthquake cyclic action the steel wall had alternately stretched beyond yield then buckled then fractured (Plates 6.55 and 6.56). The pier above the damage was left leaning towards the north. Steel T-section splints had been welded on to temporarily stabilise the column.

Near the centre of Kobe the circular columns of a steel portal frame pier supporting the Hankyu Rail Line fractured with no apparent ductility (Plates 6.57 and 6.58). The brittle nature of the fracture gave the impression of cast iron, and it has been reported[6.2] that the columns were centrifugally cast; a somewhat unusual application of this material given the ductility demand of a portal frame subject to seismic effects.

6.4.4 Foundation Damage

Most bridge foundations were piled and when they were not surrounded by saturated fill performed well; hence there are only a few examples of foundation failure. Often there were signs of considerable movement having occurred around piers, for example Plate 6.59 shows settlement and separation around one of the pier columns of the Shinkansen bullet train viaduct near Nishinomiya.

More problems arose due to liquefaction and lateral spreading of ground around those bridges built along the coast on reclaimed ground or linking the man-made islands to the mainland. An example which has been discussed above is the collapse of the approach span to the Nishinomiya Harbour Bridge on the Hanshin Bay Expressway (Section 6.4.1 and Plate 6.5).

On Minatojima Island pronounced liquefaction-induced settlement occurred around some ten of the piers supporting the automatic rail line known as the "Portliner" linking the JR Line and Port Island (Plate 6.60). The reinforced concrete columns were undamaged, but blocks of soil above the pile caps can be seen projecting from the central reserve of the highway, showing the extent of the settlement. Elsewhere along this line it has been reported that hinges formed in the columns and four spans collapsed[6.4].

6.5 Akashi-Kaikyo Bridge

The Akashi-Kaikyo Bridge will be the world's longest suspension bridge when it is completed in 1998. It is the remaining link on the most easterly of three routes being constructed by the Honshu-Shikoku Bridge Authority (HSBA) to connect Shikoku Island to the mainland. It will carry a six lane highway from Kobe to Awaji Island over the Akashi Straits, straddling the fault whose rupture actually caused the earthquake. At the time of the rupture the anchorages and towers had been completed, and the main cables had been erected. No hangers or deck sections were in place.

The scale of the structure is impressive (Plate 6.61). The main span is 1900m (the Humber Bridge in the UK still holds the record at 1410m, although the Great Belt East Bridge in Denmark will have a main span of 1624m when it is completed in 1997), the end spans are 960m each, the cables are 1.1m in diameter, the steel truss deck girder is 14m deep and 35.5m wide, the steel towers stand 283m above the top of 80m diameter caissons, the tower caissons are 70m deep, the anchorage blocks are 84m long by 63m wide and the Kobe anchorage is 123m deep.

Dr Akiyama Haruki, HSBA's Chief Engineer responsible for Akashi-Kaikyo gave generously of his time to EEFIT members at his Tarumi office beside the north anchorage of the bridge.

The fault rupture had caused significant changes in dimensions and levels of the structure, and Dr Akiyama provided the survey data which has been sketched in Figure 6.3. Apart from a kink in plan at the Kobe tower, it will be seen that the overall length of the bridge between anchorages had increased by 1.1m to 3911.1m and the length of the main span had increased by 0.8m to 1990.8m. In elevation it appeared that the Kobe tower had lifted by 0.2m while the Awaji tower had settled by 0.3m.

Structural checks carried out following the earthquake have demonstrated that apart from geometric modifications in fabrication, the distortions to the intended geometry create no cause for concern.

In design, both response spectrum and time history analysis had been used. The response spectrum had been derived by enveloping the 150 year return response spectrum (obtained by statistical evaluation of the earthquakes exceeding magnitude 6 on the Richter scale which had occurred within a radius of 300km of the site), and the response spectrum for a major ocean floor earthquake of magnitude 8.5 at a distance of 150km. For time history analysis the strong motion record at the Kaihoku Bridge during the Off-Miyagi earthquake was modified to accord with the seismicity at the Akashi-Kaikyo Bridge site. Analysis took account of the travelling wave and soil-structure interaction, but did not take account of any tectonic or fault movement.

Aerodynamic considerations dominated superstructure design, and played an important part in design of the towers. Wind excitation of the towers during construction and in the permanent structure is controlled by an ingenious compact mass-damper system within the tower legs.

6.6 Damage to Small Bridges

Some small bridges suffered badly in areas affected by liquefaction. A road bridge and a footbridge over a canal near the coast in Kobe were affected by lateral spreading, effectively shortening the span. The expansion joints of the steel box girder road bridge jammed, and an abutment wall moved forward. The bearing remained undamaged, but the bottom flange and webs of the box buckled (Plate 6.62).

The small bridge over a stream in Plate 6.64 near Nishinomiya was severely damaged. The skew and the unusual mixture of columns and walls made it vulnerable to shaking and to the pressure from retained semi-liquefied soil. A large hole had appeared behind one of the abutments where the fill had settled.

Buckling of the tubular steelwork occurred on the footbridge in Plate 6.65. The bridge links two buildings in downtown Kobe, and out of phase movements caused the damage.

6.7 Developments of Japanese Bridge Codes

Seismic design requirements were first introduced into Japanese bridge design guides in 1926, following the Great Kanto Earthquake of 1923. Table 6.1 outlines the changes in force requirements for seismic design of highway bridges over the years.

Table 6.1 History of Seismic Design Loads for Highway Bridges in Japan[6.5,6 and 7]

Year	Name of Regulations	Seismic Design Method	Other Stipulations	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 $k_H = 0.15 \sim 0.4$ ($k_H \geq 0.3$ 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 ($k_H = 0.1 \sim 0.35$, $k_V = 0.1$)	Not considered	1952 Tokachi-oki (M8.2) 1964 Niigata (M7.5)
1967*	Specifications for Seismic Design of Honshu-Shikoku Bridges, JSCE	Modified Seismic Coefficient Method ($k_H = 0.2$)		1968 Tokachi-oki (M7.9)
1968*	Aseismic Design Standards, Hanshin Expressway Public Corporation	Seismic Coefficient Method ($k_H = 0.20 \sim 0.28$)		
1971	Specifications for Seismic Design of Highway Bridges, MOC	• Seismic Coefficient Method ($k_H = 0.1 \sim 0.24$) • Modified Seismic Coefficient Method ($k_H = 0.05 \sim 0.3$)	• Evaluation of Sandy Layers Vulnerable to Liquefaction • Device for Preventing Superstructure Falling off	
1980	Part V Seismic Design Specifications of Highway Bridges, MOC	• Seismic Coefficient Method ($k_H = 0.1 \sim 0.24$) • Modified Seismic Coefficient Method ($k_H = 0.05 \sim 0.3$) • Check of Deformation Capability of RC Piers • Dynamic Response Analysis	• F_L Method for Evaluation of Liquefaction • Device for Preventing Superstructure Falling off	1978 Miyagi-ken-oki (M7.4) 1982 Urakawa-oki (M7.1) 1983 Nihon-kai-chubu (M7.7)
1990	Part V Seismic Design Specifications of Highway Bridges, MOC	• Seismic Coefficient Method ($k_H = 0.1 \sim 0.3$) • Check of Bearing Capacity of RC Piers for Lateral Force • Check of Deformation Capability of RC Piers • Dynamic Response Analysis	• F_L Method for Evaluation of Liquefaction • Device for Preventing Superstructure Falling off	

* Specifications for highway bridges related to special large projects

6.8 Concluding Comments

The impact of the earthquake on bridge structures was substantial, and the severed transport arteries added greatly to the economic loss and suffering. The large number of single column supports to elevated roads and railway led to more collapses than otherwise would have been the case. Generally the worst affected structures were the older ones; as codes improved, so did the performance of the bridges. However, there were failures of modern structures, and there are still lessons to be learned. Piled foundations performed well, but movements of substructures were large in areas of liquefaction, and the span losses serve to re-emphasise the need for a failsafe approach to avoid spans being dislodged. Unfortunately there were no bridges with modern isolation systems in the affected area.

References:

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6.2 Horikawa Prof K, and Sakino Y, *"Damages to Steel Structures caused by the 1995 Kobe Earthquake"*, Structural Engineering International 3/96, published by IABSE.

6.3 The Hyogo-Ken Nanbu Earthquake, Great Hanshin Earthquake Disaster, January 17, 1995. *Preliminary Reconnaissance Report.* Earthquake Engineering Research Institute, February 1995.

6.4 Masanobu S., *"Summary of the Earthquake"*, special supplement to the January 1995 issue of the NCEER Bulletin.

6.5 *"The Hyogo-Ken Nanbu Earthquake of 17 January 1995".* Bulletin of the New Zealand National Society for Earthquake Engineering, Volume 28 No 1, March 1995.

6.6 Kuribayashi E, Iwasaki T and Ueda O, *New Specifications for Earthquake Resistant Design of Highway Bridges in Japan"*, Proceedings Second Joint US-Japan Workshop on Performance and Strengthening of Bridge Structures and Research Needs, San Francisco, USA, 1985.

6.7 Ichimasu H., Kodera J., and Kawashima K., *Design details for Highway Bridges in Japan,* 1991.

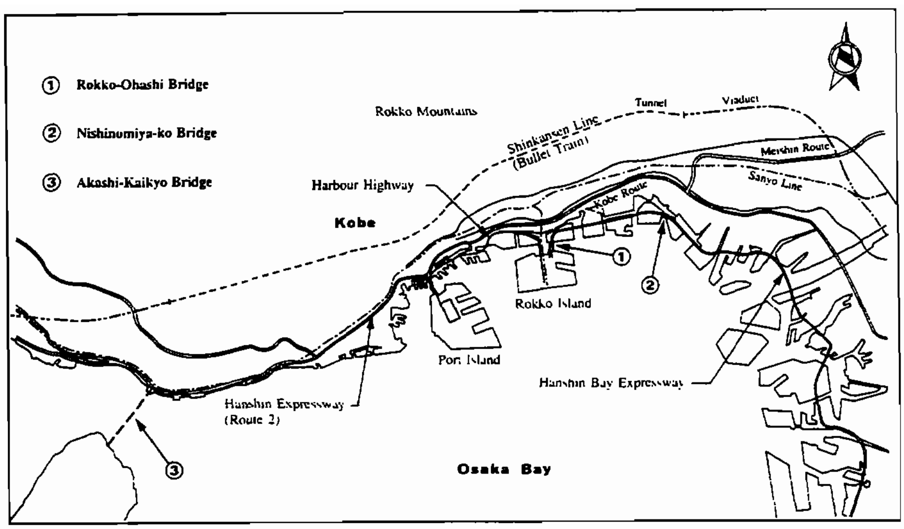


Figure 1 Primary Transportation Routes

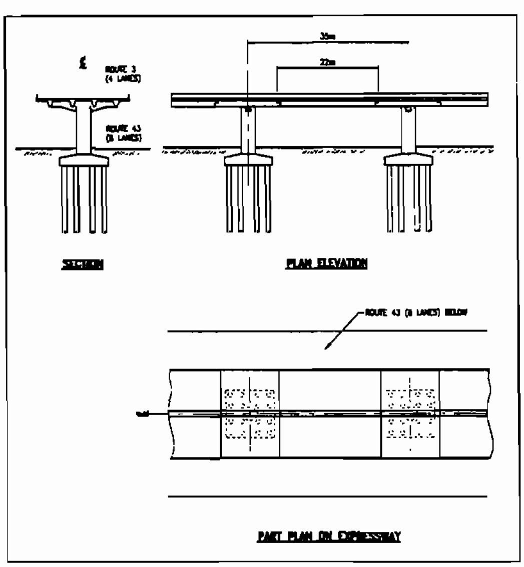


Figure 2 Typical Arrangement of the 635m Long All-Concrete Section of the Hanshin Expressway which Collapsed

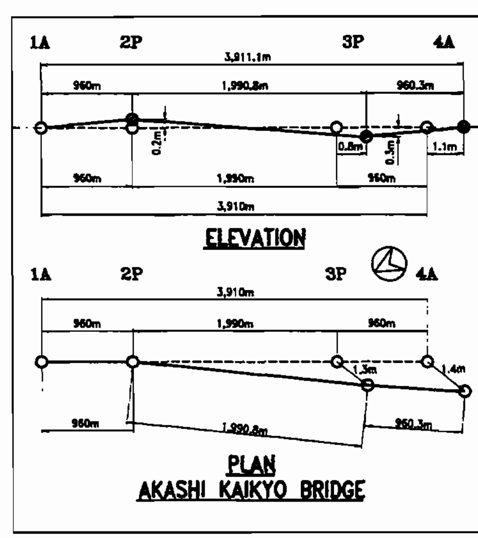


Figure 3 Survey of Akashi-Kaikyo Bridge after Earthquake

Plate 6.1 Kobe looking towards the east from Port Island: Photo taken about 1980 showing the piers of Route 2 (the "Hamate Bypass") prior to erection of the deck.

Plate 6.2 Kobe looking towards the north-west from Port Island: Photo taken about 1980 showing the piers of Route 2 (the "Hamate Bypass") prior to erection of the deck.

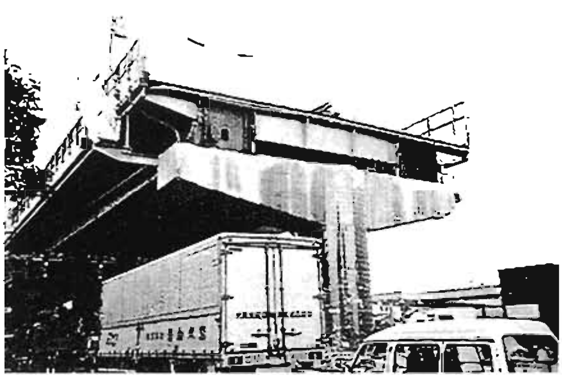


Plate 6.3 Hanshin Expressway near Nishinomiya: Large movement at joint caused span to slip off bearing shelf. It was here that a tour bus came to a halt overhanging the abyss left by the missing span.

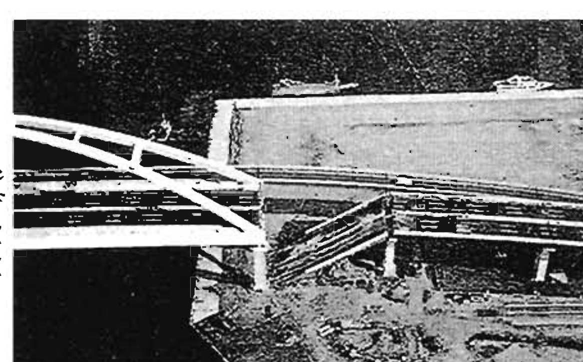
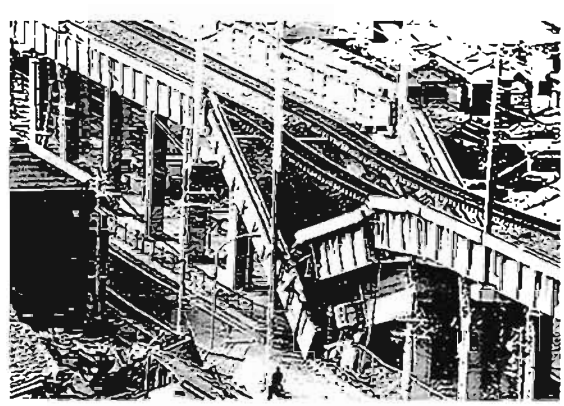


Plate 6.4 Nishinomiya Harbour Bridge on the Hanshin Bay Expressway: Aerial photograph of collapsed approach span. Liquefaction caused lateral spreading which moved sea wall and main pier towards the water.

Plate 6.5 Nishinomiya Harbour Bridge on the Hanshin Bay Expressway Approach span fell due to loss of support as main pier shifted towards main span.

Plate 6.6 Shinkansen Bullet Train Line near Nishinomiya:Collapse of simply supported span between two continuous sections of bridge. Note deep beam between columns in spans either side of collapse (photo taken immediately after earthquake);



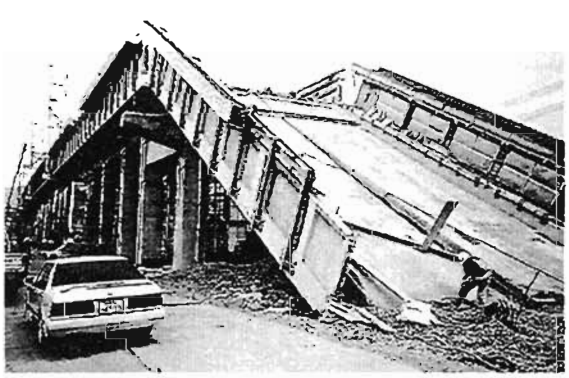


Plate 6.7 Shinkansen Bullet Train Line near Nishinomiya:as Plate 6.6, but photo by authors

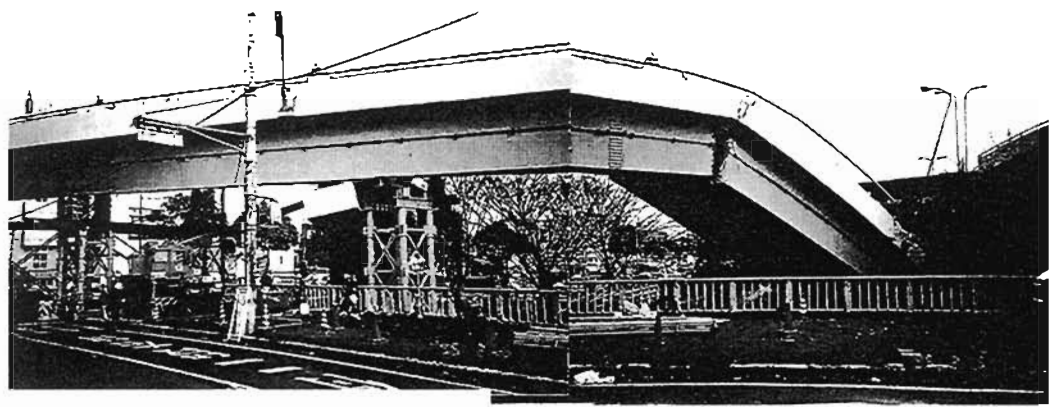


Plate 6.8 Hanshin Expressway to the west of Kobe centre:Collapse of curved steel box girder ramp due to loss of support at abutment.

Plate 6.9 Hanshin Expressway to the west of Kobe centre:Large sideways shift towards south of curved deck, same interchange as Plate 6.8

Plate 6.10 Hanshin Expressway to the west of Kobe centre:close-up of Plate 6.9 subject

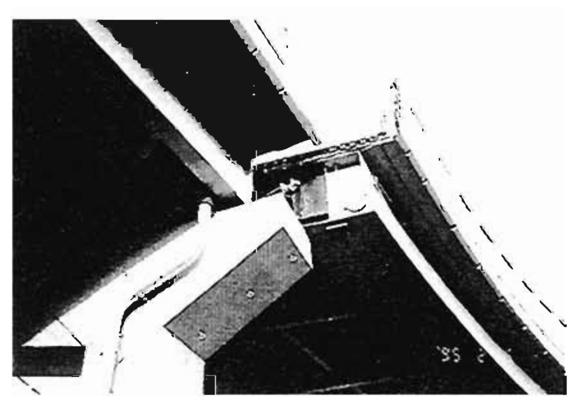
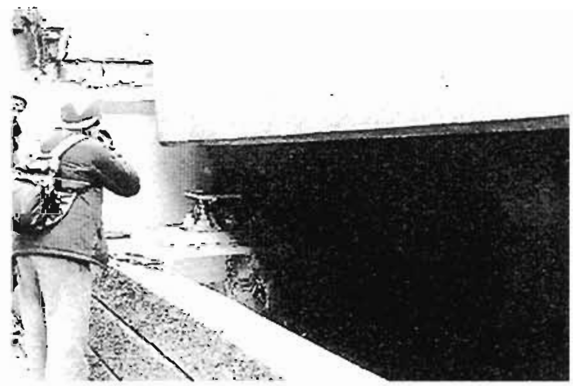


Plate 6.11 Ramp onto Hanshin Expressway near centre of Kobe: Approximate one metre shift of first ramp span towards the north

Plate 6.12 Hanshin Expressway near centre of Kobe: Same location as Plate 6.11, showing girder jolted off its bearing towards the north



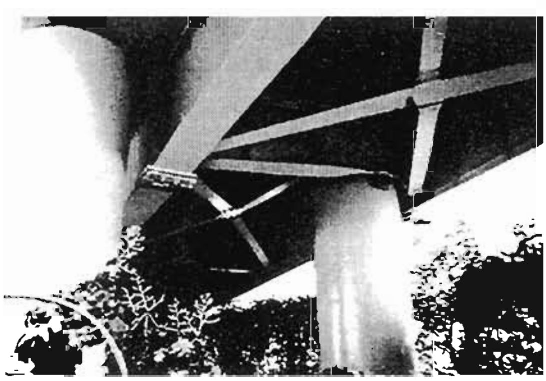
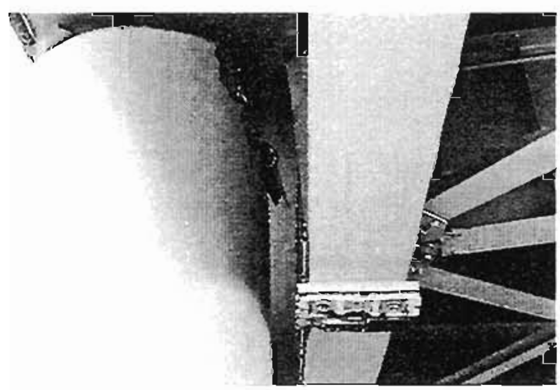


Plate 6.13 Hanshin Expressway near centre of Kobe: A little higher up same ramp as Plates 6.11 and 6.12, showing girders shifted off column supports and dropped onto deck slab

Plate 6.14 Hanshin Expressway near centre of Kobe: Close-up of left hand column in Plate 6.13



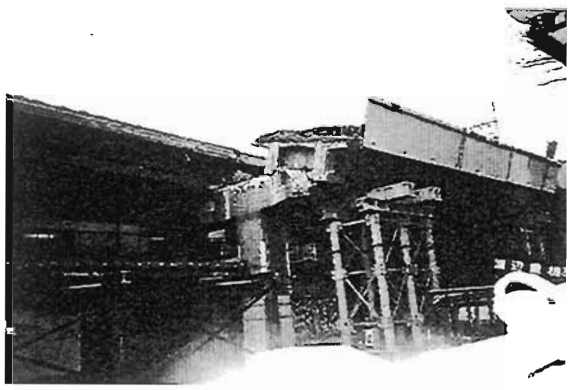
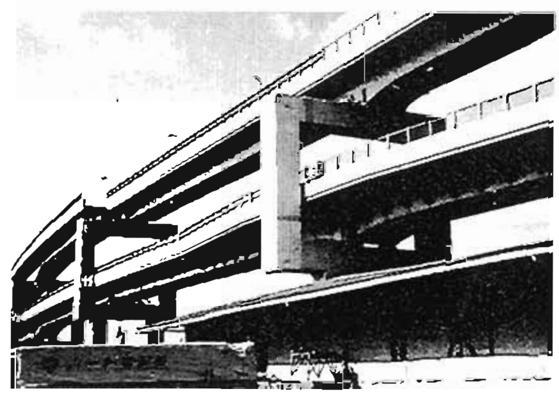


Plate 6.15 Train Line in Kobe: Narrow twin I-girder carrying one track almost fell off portal pier transversely - the missing one probably did fall off sideways.

Plate 6.16 Multi-level Section of Route 2 near downtown Kobe: large transverse displacement of upper deck at movement joint



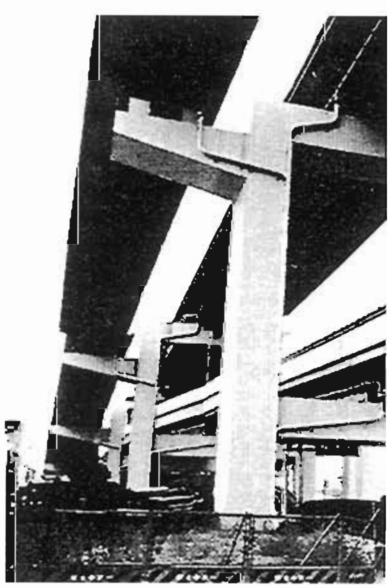
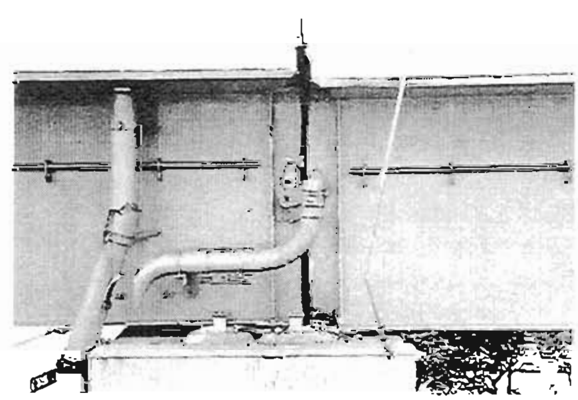


Plate 6.17 Route 2 near downtown Kobe: Large transverse displacement of deck at movement joint supported on cantilever from portal pier

Plate 6.18 Hanshin Expressway to the east of Kobe: Permanent longitudinal movement of about 400mm caused bolts to shear in bearings and in restrainer plate between girders. Drainage pipes also broken.



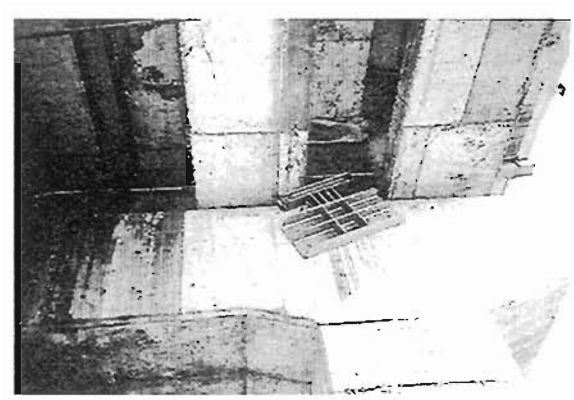


Plate 6.19 Shinkansen Bullet Train Line, Nishinomiya: Retrofit steel fabrication intended to provide lateral restraint to deck had been twisted out of position.

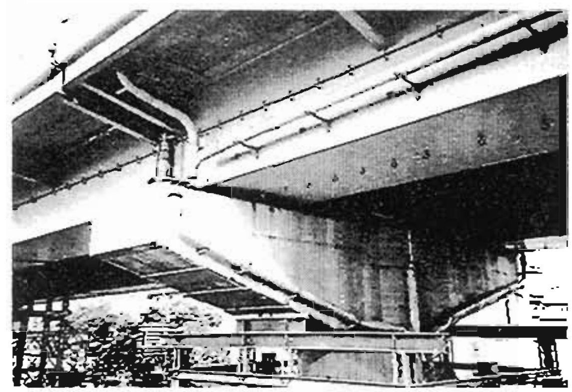


Plate 6.20 Hanshin Expressway to the east of Kobe: Transverse shift of deck has sheared bearing bolts and dislocated drainage

Plate 6.21 Hanshin Expressway to the east of Kobe: Longitudinal shift of deck has sheared bearing bolts and dislocated drainage

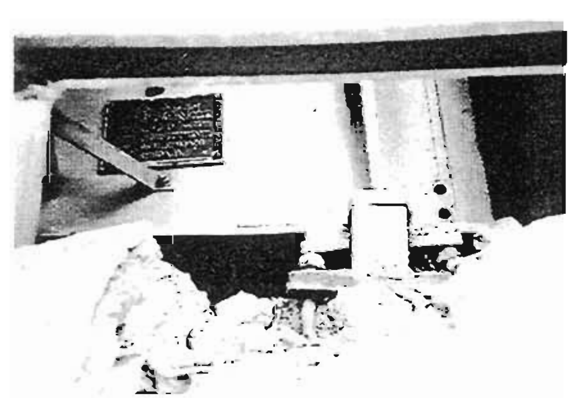


Plate 6.22 Hanshin Expressway to the east of Kobe: "Fixed" bearing at end of ramp stayed "fixed", but bolts and concrete damaged

Plate 6.23 Ramp onto Hanshin Expressway near centre of Kobe: Hammering at joint has damaged deck joint and parapets

Plate 6.24 Hanshin Expressway, Kobe Route 3, Fukae-Honcho Section toppled 635m of all-concrete viaduct due to flexure/shear of single column piers.

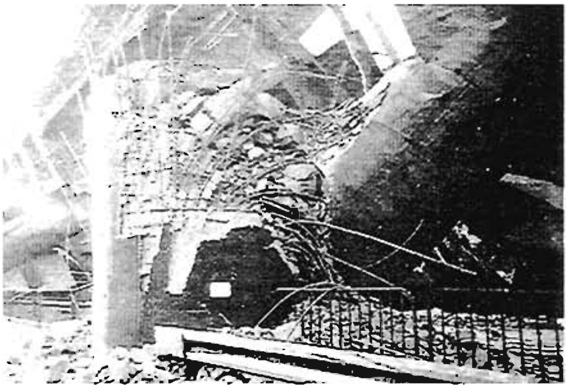
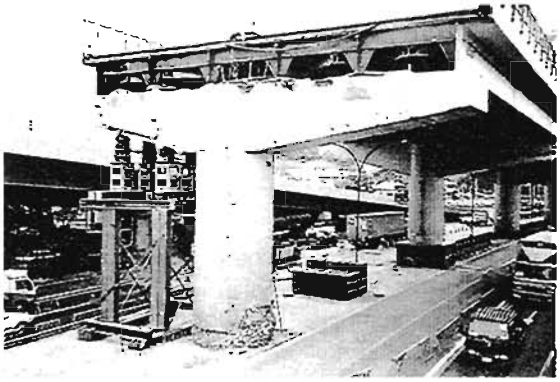


Plate 6.25 Hanshin Expressway, Kobe Route 3, Fukae-Honcho Section close-up of pier in foreground of Plate 6.22 showing small lateral bars and curtailed longitudinal bars.

Plate 6.26 Hanshin Expressway, Kobe: Failure of column due to inadequate confining reinforcement. Note steel box jacket to pier beyond to temporarily stabilise structure.



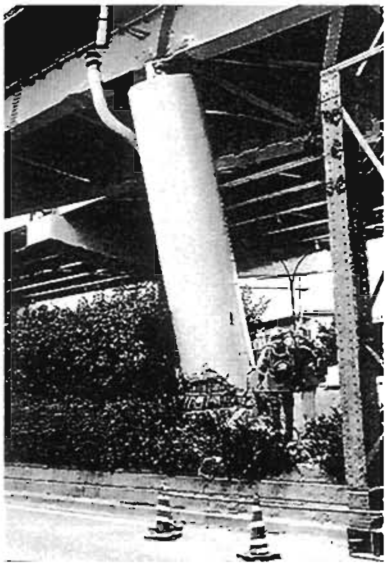


Plate 6.27 Ramp onto Hanshin Expressway, Kobe: Deck has shifted sideways and taken top of column with it.

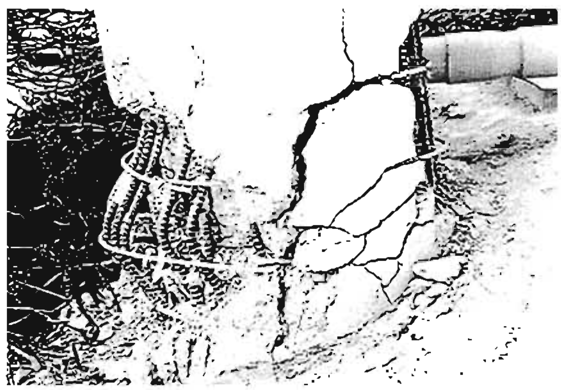


Plate 6.28 Ramp onto Hanshin Expressway, Kobe: Base of column in Plate 6.27: simple and almost non-existent laps offered no confinement. Failure is at location of curtailment of inner main bars.

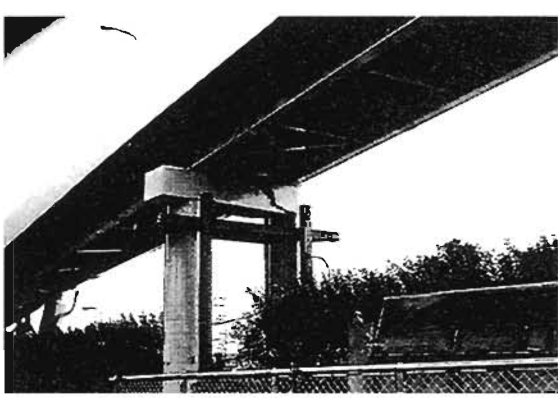


Plate 6.29 Ramp onto Hanshin Expressway, Kobe: Sideways shift of deck and brittle fracture of portal frame crosshead beam

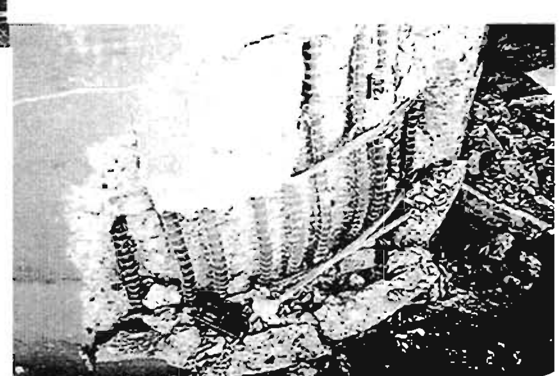
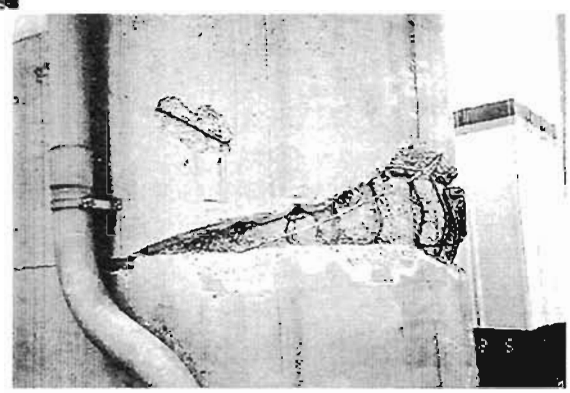


Plate 6.30 Hanshin Expressway, Kobe: Large diameter column with small confining bars. Ends of bars hooked, but with tails not long enough to anchor into core concrete.

Plate 6.31 Hanshin Expressway, Kobe: Damage to large pier at level of construction joint and/or curtailment of reinforcement.

Plate 6.32 Hanshin Expressway, Kobe: Close-up of damage shown in Plate 6.31.



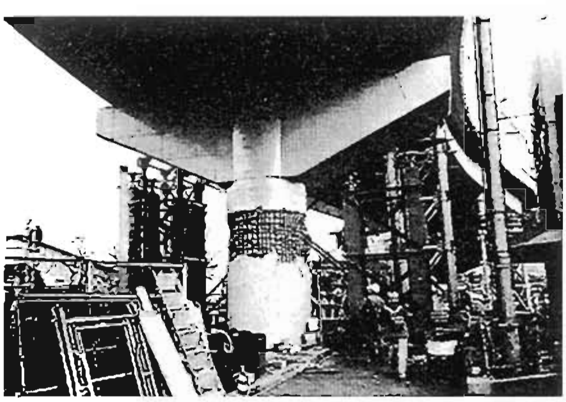
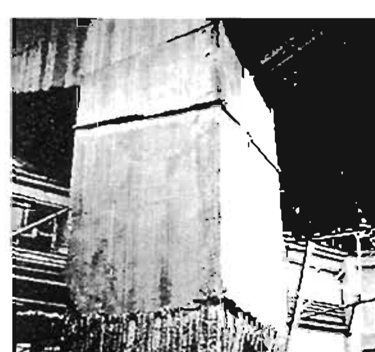


Plate 6.33 Hanshin Expressway, Kobe: Shear-flexure failure at mid-height where main reinforcement is curtailed

Plate 6.34 Hanshin Expressway, Kobe: Flexure/shear failure at laps in main bars - bond lost with cover concrete



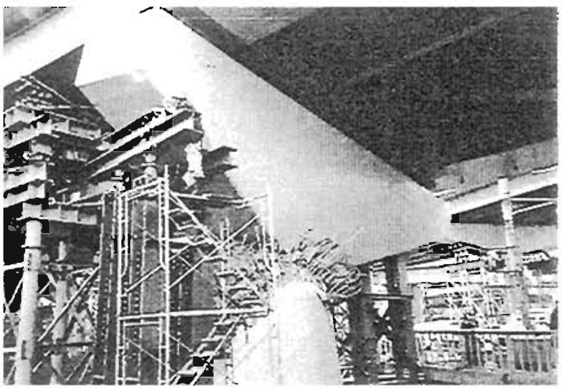


Plate 6.35 Hanshin Expressway to the west of Kobe centre: Shear-flexure failure at underside of column crosshead - lack of confinement

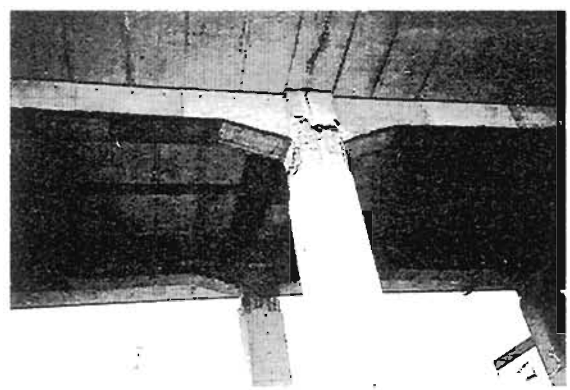


Plate 6.36 Shinkansen Bullet Train Line Viaduct, near Nishinimoya shear-flexure failure at beam/column connection - lack of confinement

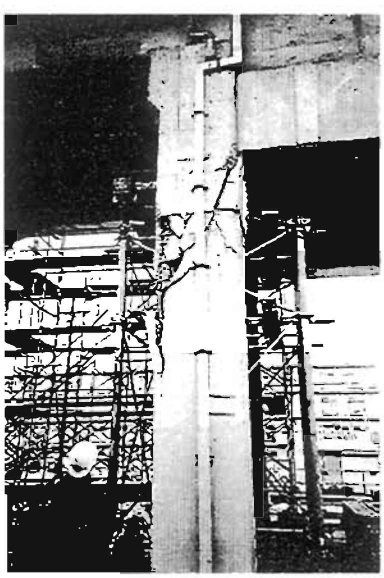


Plate 6.37 Shinkansen Bullet Train Line Viaduct, near Nishinimoya shear-flexure failure at beam/column connection

Plate 6.38 East Pier of the Maya Ohashi Bridge on the Harbour Highway failed completely at a change in section near mid-height, leading to a 350mm northward shift of the upper part

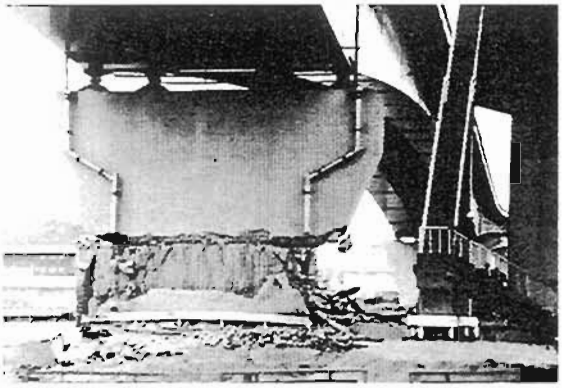


Plate 6.39 East Pier of the Maya Ohashi Bridge on the Harbour Highway no through-wall ties, simply transverse bars which were lost with the cover. At least the longitudinal reinforcement was continuous.

Plate 6.40 East Bank at the Maya Ohashi Bridge on the Harbour Highway liquefaction caused extensive lateral spreading

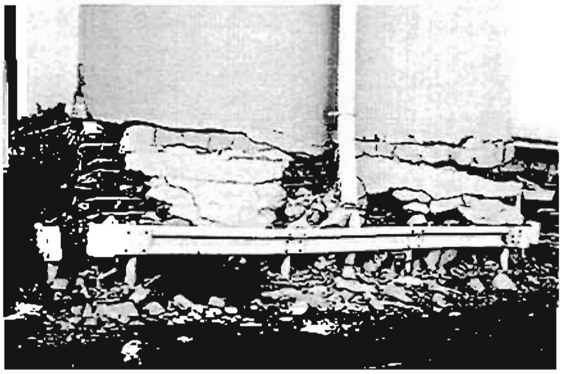


Plate 6.41 Pier on East Approach to the Maya Ohashi Bridge on the Harbour Highway close-up of pier in centre of Plate 6.40 - this is the second pier from the east main pier, and suffered severe damage in accommodating the distortion at the main bridge pier.

Plate 6.42 Pier on East Approach to the Maya Ohashi Bridge on the Harbour Highway damage at one corner of the pier in Plate 6.41 - note the failure of the welded bar in the centre of the Plate

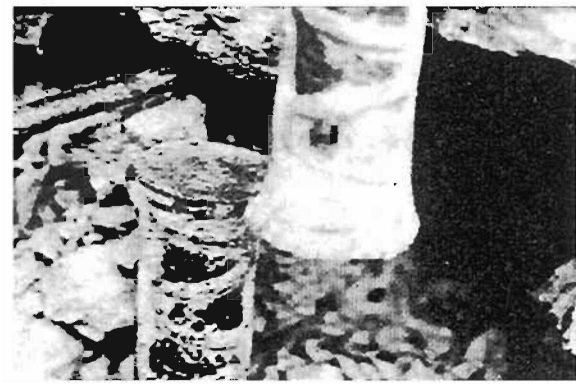


Plate 6.43 Pier on East Approach to the Maya Ohashi Bridge on the Harbour Highway close-up of failure of gas weld in Plate 6.42

Plate 6.44 West Main Pier of the Maya Ohashi Bridge on the Harbour Highway shear split as a result of large transverse load transmitted from the bearing

Plate 6.45 Cable-stayed Bridge alongside the Maya Ohashi Bridge structure was undamaged apart from some minor damage at the top of pedestrian access stair

Plate 6.46 Hanshin Expressway - the Remains of a Steel Stiffened Box Pier the column crosshead came to rest on at this level after collapse of the column walls

Plate 6.47 Hanshin Expressway - Damage to Rectangular Steel Pier Column stiffened walls buckled exposing orange coloured primer/undercoat

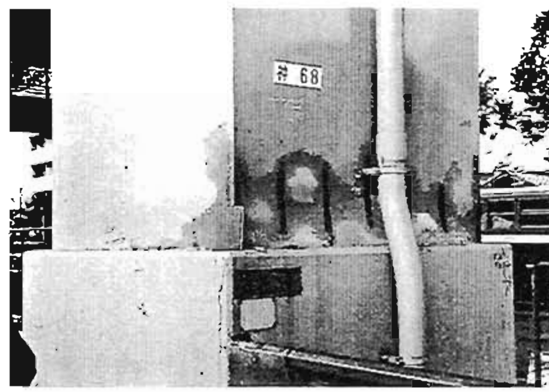


Plate 6.48 Hanshin Expressway - Damage to Rectangular Steel Pier Column close-up of buckled area of column in Plate 6.47 - internal stiffeners show up as dark lines on exposed orange paint

Plate 6.49 Hanshin Expressway - Damage to Rectangular Steel Pier Column extent of buckle indicated by Dr Peter Merriman

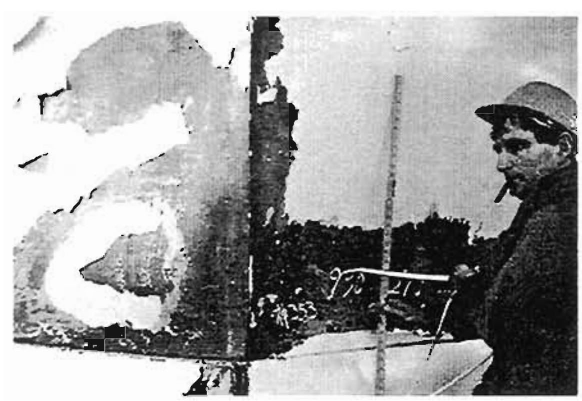


Plate 6.50 Hanshin Expressway - Damage to Circular Steel Pier Column freshly flaked paint on column near Kobe centre indicates strong north-south cyclic action

Plate 6.51 Hanshin Expressway Route 2 near the centre of Kobe- Damage to Circular Steel Pier Column viewed along the line of the structure shows a symmetric buckle at change in wall thickness

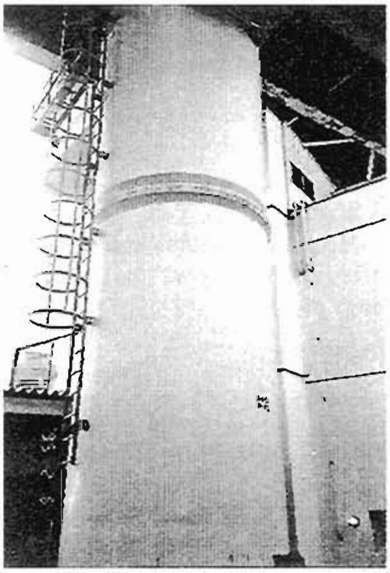


Plate 6.52 Hanshin Expressway Route 2 near the centre of Kobe- Damage to Circular Steel Pier Column viewed normal to the line of the structure shows symmetric buckle at change in wall thickness

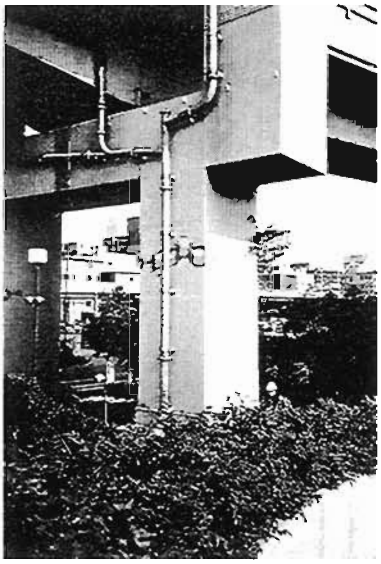


Plate 6.53 Hanshin Expressway Route 2 near the centre of Kobe- Damage to Rectangular Steel Pier Column change in wall thickness is likely to explain location of buckle in this stiffened box column

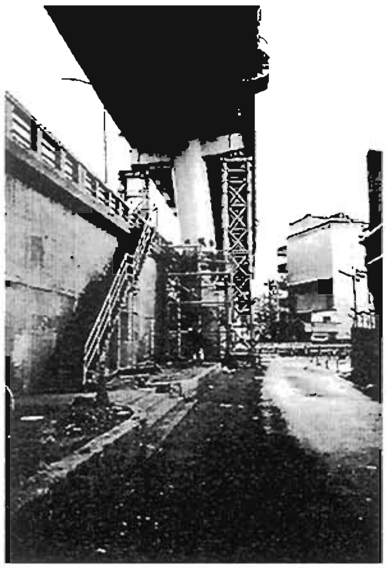


Plate 6.54 Ramp onto the Hanshin Expressway to the west of Kobe Steel column kinked at change in wall thickness near mid-height. Stabilised by welding on T-section "splints" .

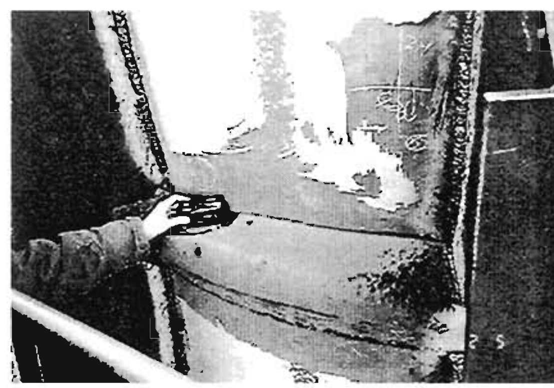


Plate 6.55 Ramp onto the Hanshin Expressway to the west of Kobe Close-up of damage to column in Plate 6.54 - extensive buckle and rupture of pier wall

Plate 6.56 Ramp onto the Hanshin Expressway to the west of Kobe Another view of the tear in the steel wall of the column in Plates 6.55 and 6.56

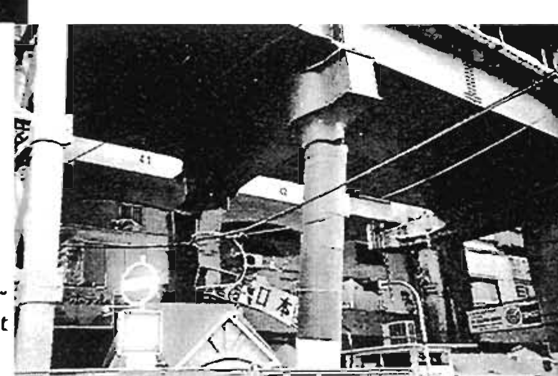


Plate 6.57 Railway near Centre of Kobe - Damage to Steel Portal Pier centrifugally cast columns lacked ductility

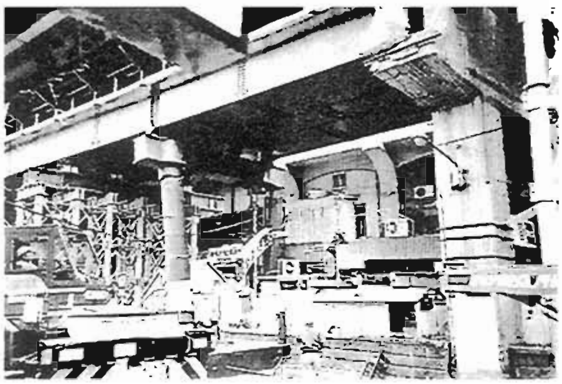


Plate 6.58 Railway near Centre of Kobe - Damage to Steel Portal Pier centrifugally cast columns lacked ductility

Plate 6.59 JR Shinkansen Bullet Train Viaduct near Nishinomiya signs of movement and settlement around pier column

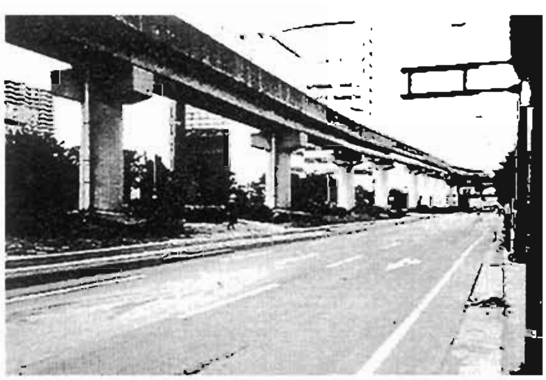


Plate 6.60 Automatic rail line linking the JR Line and Port Island softening and settlement of ground left blocks of soil above pilecaps projecting above median

Plate 6.61 The Akashi-Kaikyo Bridge Linking Awaji Island and the mainland with a main span of 1900m this will be the world's longest suspension bridge when it is completed in 1998. The span was almost one metre longer after the earthquake!

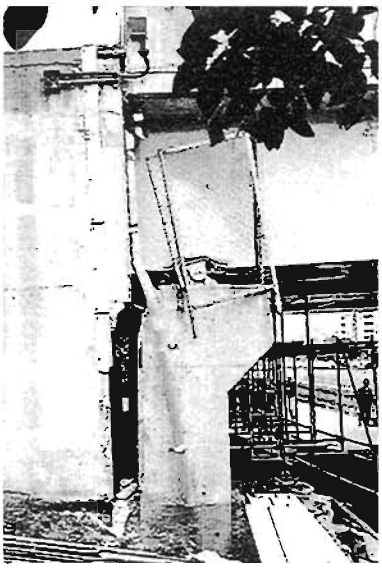


Plate 6.62 Steel Box Girder Road Bridge over Canal, Kobe Ground either side of canal moved towards canal shortening span. Abutment wall moved forward and buckled flange and web of box girder.

Plate 6.63 Footbridge over Canal, Kobe lateral spreading caused banks to move inwards, and girder sliced through roadway

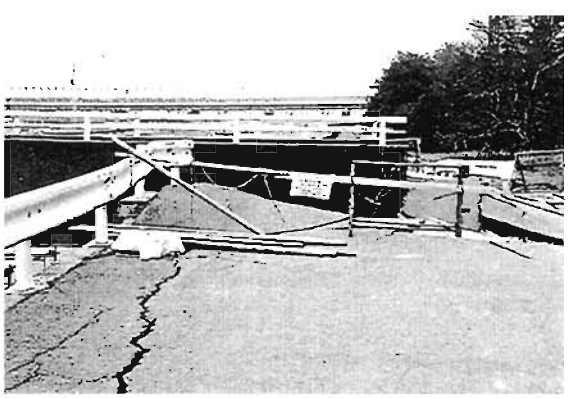


Plate 6.64 Footbridge over Canal, Kobe close-up of right-hand end of span in Plate 6.63

Plate 6.65 Single Span Bridge to the North of Nishinomiya Ground movements caused collapse of columns and loss of material from behind abutments

Plate 6.66 Footbridge Linking Buildings in Kobe Out of phase movements caused buckling of tubular steel members

7.0 INDUSTRIAL FACILITIES

C J Bolton
BNFL Engineering Ltd

P A Merriman
British Nuclear Fuels

7.1 Introduction

Kobe and the surrounding areas form a large, modern, manufacturing and industrial conurbation. Due to the shortage of flat land, many of the industrial plants are sited near the coast, often on reclaimed land. The band where most of the damage was concentrated is some distance inland, and thus the majority of the industrial plants were not subjected to the same intensity of shaking that affected the worst hit areas.

Although the intensity of the earthquake motion was reduced near the coast, significantly more soil failures occurred, due to the nature of the ground. The mechanisms of these soil failures are discussed in more detail in Chapter 8, while this Chapter reports the consequences for the structures.

The large areas affected, combined with the difficulty in moving around the area and in interpreting local maps, meant that many potentially interesting sites were not visited. Further, language difficulties and commercial confidentiality meant that access to the interior of facilities was often not available.

7.2 Ports and harbours

Prior to the Hyogoken-Nambu earthquake, Kobe was the largest cargo handling port in Japan, carrying a significant proportion of Japanese exports. As a result of the earthquake, all operations were shut down, and 90% of the 167 berths were damaged. An initial estimate is that it may take three years to return to full operation, at a cost of £5 billion. This may result in permanent loss of trade as shipping companies establish other routes for their cargoes.

Soil failure caused serious damage to quay walls over a large proportion of the port. The performance of container handling cranes should be noted; many being supported with one track on the caisson wall and the other on concrete piles or sand pile reinforced ground, inland. The outer track invariably moved seaward, separating the crane legs and causing buckling of the steel frame. In most cases, the crane remained standing, despite this distortion.

The Mitsubishi and Kawasaki shipyards appeared to have suffered significant damage, although since entry was not obtained, inspection was limited to what could be seen from ferries which passed close by. There were few inertial failures, mostly of crane jibs or lifting gantries, but substantial damage due to soil failure. (Plate 7.1)

Shipyard cranes in general stood up well. One failure at the slewing ring and one jib failure (Plate 7.3) were noted, and other wreckage may have been removed, but the number still standing suggested that at least 80% were undamaged.

Dry docks appeared to survive well, and were not flooded, with no obvious damage to caisson gates. Adjacent to one dry dock, a track mounted crane was leaning at about 10° off vertical. (Plate 7.2) This appeared to be due to settlement of the ground, while the dock remained in position or settled a small amount. It is assumed that the dock was supported on piles. Lettering on the dock caisson gates appeared partially submerged, suggesting that some settlement of the entire dock had occurred (Plate 7.4)

Some dry docks contained ships under construction, and these appeared to have remained stable (Plate 7.5). Submarines under construction were upright (Plate 7.6). There were unconfirmed reports of a ship having rolled over due to failure of the propping under one side, but the team were unable confirm the location or inspect it.

Built up areas near the coast are protected from inundation by tsunami using concrete sea walls. Some cracking of these occurred, due to lateral spreading of the foundations, but the major vulnerability arises from steel sliding caisson gates, which allow passage for vehicles, and so forth. Even minor settlement damage to the rails carrying the sliding gate prevents closure (Plate 7.7). Fortunately, there was no tsunami, but the port areas will remain vulnerable to such effects from a large offshore earthquake until repairs can be made.

7.3 Power Stations

The EEFIT team visited the Higashinada Gas Turbine station, commissioned in 1974. As a peak load station, it was off line at the time off the earthquake, but was put out of action. The generating plant appeared to be intact, although it was not examined in detail. (Plate 7.9) The station is on the coast, and settlement of the order of 0.5m occurred in the tank farm supplying the station, due to liquefaction and lateral spreading. Welded steel piping was distributed through the site in concrete pipe trenches, which were founded on piles where they crossed roads but bearing on the ground elsewhere. The differential settlement resulted in 0.5m steps in the road level, and similar abrupt steps in the ducts. (Plate 7.8) The pipe supports were generally flexible enough to allow the pipes to accommodate the movement over a sufficient length to avoid rupture, although there were some leaks.

According to information supplied by the regional generating company, 12 of the 36 fossil fuel generating units which were in operation were tripped by the earthquake. Of 63 units in total, 10 were put out of use, mainly due to damage to boiler tubes or settlement damage to services. Seven of the ten were back in service within 6 days.

No damage occurred to Hydroelectric stations. There were no nuclear generating stations in the epicentral area, the nearest being on the north coast some 120km from the epicentre. No damage was reported, although given the intensity at the sites, none should have been expected.

At the nuclear site, the peak horizontal and vertical peak accelerations measured were considerably less than those at neighbouring sites. (Table 7.1). This is due to the Japanese policy of constructing the Nuclear plants on bed rock and thereby avoiding the soil amplifications that occur on intermediate and soft sites. Similar ground amplifications were observed at the Loma Prieta earthquake (EEFIT Report, Ref 2). In the one region, 10 of 15 units which had been on line before the earthquake continued normal operations. The acceleration levels at these were well below the automatic trip points, set at 0.16g to 0.21g. None of these plants were damaged by the earthquake.

7.4 Tanks and Vessels

A visit was made to the Effluent Treatment Work at Uozaki. This is situated on an artificial island, just inland of Rokko Island, and the damage was typical of the area with large ground settlements up to 1 metre.

The majority of the Process Building at Uozaki remained undamaged, this being largely attributed to the piled foundations, which had been exposed by settlement in places. Cracking had occurred in some of the hollow precast concrete piles (Plate 7.11). In the harsh maritime environment, there may be long term integrity problems with the constant movement of saline groundwater. It was observed in several areas (such as the ferry terminal, Plate 38) that many of the piles were either lightly connected to the base above (Plate 7.12) or had no apparent connection at all (Plate 7.10).

The rectangular concrete tanks were almost entirely undamaged. These open topped tanks were an estimated 5m deep, and were founded at about 4m below ground level. At one end, relative settlement had occurred resulting in rotation of the end bay and consequent opening up of a concrete joint (Plate 7.13). These tanks had been drained.

Other large circular tanks were maintaining containment of the sewage but the roof structure above them had suffered minor damage. Some of the fibre glass panels suspended from the external steel frame had cracked and some areas had dropped out. (Plates 7.14 and 7.15) This may have been caused by the relative flexibility of the steel structure interacting with the more rigid shape of the sheeting. It is very unlikely that the Codes of Practice would recommend a dynamic analysis to investigate the interaction between the two components.

The most extensive damage to tank farms occurred when they were situated on filled ground, close to wharves and other retaining structures. The lateral spreading of the ground supporting the tanks led to severely tilted tanks as liquefaction occurred, with settlement up to 2 metres. Towards the higher ground inland, there was less ground movement and performance improved as the boundary conditions conformed more to design assumptions.

Near the centre of Kobe is a spherical gasholder containing some 17000 cubic metres of town gas, at a maximum pressure of 7 bar. (Plate 7.18) This was constructed in 1976, from 31mm plate, and appeared completely undamaged. Smaller tanks at other locations also appeared intact.

A water tank on the hillside above Nishinomiya toppled off the steel platform supporting it. This was not inspected closely, but reports suggest that the holding down bolts pulled out of the foundation (AIJ, Ref. 1).

A number of steel tank farms were inspected and generally no major damage was noted, although some movement was apparent from distortion of pipes. (Plate 7.24) and one of the taller tanks showed a typical buckling failure (Plate 7.16). In most cases, the reinforced concrete base slab supporting the tanks were supported on piles. At Higashi-Nada power station, excavations were in hand to determine the extent of damage to the piles (see 8.1.4 in Chapter 8).

From the investigation, it was evident that the major design of the tanks had been satisfactory. Improvement might be possible with regard to secondary effects. For example, damage was caused at a tank farm near Fukae as no movement provision had been made (Plate 7.17). There were connection failures between the tank and service pipework at ground level on another tank farm.

7.5 Piping

In the various tank farms which were inspected, steel piping was generally intact. Some failures at bolted flanges were evident where the strain imposed by soil movement was excessive. No damage to valves was noted, except relating to the flanged connections to them. In general, welded piping deformed by flexure of the bends, to meet the imposed displacements.

At the Uozaki effluent treatment plant, welded steel pipework between tanks again performed well, despite large deformations in areas where settlement had occurred.

The substantial clad pipebridge connecting the treatment works to the mainland suffered damage due to lateral displacement of a column foundation adjacent to the water's edge. The deck of the bridge remained in place, and the differential movement between it and the process building to which it led was small. An older open structure pipebridge adjacent to it suffered similar displacements, although some pipes were damaged due to their rising up the displaced column, not continuing to the building. (Plate 7.19) Many of the pipes did appear intact, although temporary flexible pipes were in evidence in the water below the bridge.

In areas of the plant unaffected by settlement, piping appeared completely intact (Plate 7.20). The arch pipebridge over the channel between the works and the mainland was undamaged, despite movement of the abutments due to lateral spreading (Plate 7.39). This form of construction allows deformation when subject to forced displacement.

The most interesting pipe was a large diameter main, with bolted flanges, originally supported on steel columns, founded on substantial independent concrete pads. These pads had settled about 1m, leaving the pipes alone to carry the weight of their supports (Plate 7.21).

At a small industrial facility near Fukae, polythene pipes were used extensively. These suffered considerable damage, in some case being fractured more than once in the same run (Plate 7.23). A small number of polythene pipes were also used in the Uozaki effluent plant, and these suffered serious damage.

Small diameter stainless steel pipework at an industrial plant (possibly a brewery) near Uozaki appeared generally intact, as did its supporting steelwork, although minor repair work was implied from the presence of a temporary access ladder (Plate 7.22).

7.6 Plant and machinery

Understandably, there was a marked reluctance to permit engineers into damaged buildings that were unoccupied either due to potential building failure or loss of services. A visit was made to a large shopping complex and conference area which included a 1800 seat auditorium, 2 cinemas and the German Consulate (See 4.4.4), with own electrical substation in the basement. There had been large settlement of the ground around the building foundation, which led to the loss of both water and gas services. The electricity service, however, was still operational after the event. The plant consisted of a combination of older equipment dating from the original construction in 1955 to more modern equipment. It was evident that the new equipment, such as standby batteries, water pumps (Plate 7.25) and a 1500kVA transformer had been designed seismically, with clear load systems and connections to the base. The main failures requiring remedial work were due either to the familiar sliding of plant on unconnected base plates or the inadequacy of the foundations under heavy plant (Plate 7.26). This led to a number of breakages of pipes which were sheared by lateral movement between heavy components. In the building, damage to services increased towards the sixth floor, where the entire floor had collapsed. Above this, roof mounted ventilation equipment with a light outside bracing system had survived the earthquake effects. (Plate 7.27)

A 3 storey RC factory near Ogi with large steel tanks and pipework, including some cyclones or similar plant on the roof, suffered no damage visible from outside. The concrete construction was unusual, consisting of an RC frame, with concrete infill panel extending 80% of the height of each storey, leaving only a slot at the top (Plate 7.28, shows end elevation without infill panels). This would have been expected to put high shear strain demands on the short lengths of free column, but no damage was apparent. The factory is near the coast, and is probably on filled ground, where attenuation of the high frequency may have occurred.

No damage to EOT cranes was noted, nor were any seen to have derailed. A typical light industrial fabrication shop (Plate 7.29) showed settlement damage to the floor, resulting in overturned machinery, but no observed damage to the building frame or the cranes.

7.7 Industrial structures

A group of Silos in Higashinada suffered buckling failures near the top. (Plate 7.30) This appeared to be due to the mass of steelwork supported over them. No other silo failures were seen by the team.

The current Kobe steelworks was inspected from outside the perimeter fence only, as access was not granted. Some damage to the tops of tall structures was evident, with buckling of columns to the charging structure above one furnace, and derailment of the charge hoist, although it remained in place (Plates 7.31). It is understood that serious damage occurred to the ore conveyors on the seaward side of the steelworks, due to soil movement, but these could not be inspected.

The conveyors from the wharf to the Kobe Onba concrete batching plant appeared almost intact. Some repair work was in hand to restore one section which had undergone a longitudinal displacement. The plant was in operation. Other conveyors were seriously affected by settlement of the foundations (Plates 36 and 37).

Immediately West of Mayo Port is a redundant steelworks, dating from the 1950s. Despite being in poor condition, with severe rusting, this suffered little damage, possibly as a result of the damping and inherent ductility of the riveted steel frame. There was limited cracking (Plate 7.32) in the concrete supports for the longitudinal steel bracing members just above ground level. Other factors that improve the building performance are the reduction in dead loads due to removal of services and the conservatism in design resulting from the need to support the EOT cranes in any position. There were several other open framed buildings in the area with rusting steelwork that also had little damage. As discussed in Chapter 3 (3.4.2), attenuation of acceleration occurred in deep saturated fills, but this was accompanied by amplification of velocity. The steel frame structures of the steelworks would generally have low natural frequencies, and so would not have benefited from this effect.

It was interesting to note that within 100m of this old steel frame, a modern RC substation had collapsed. This substation suffered from the non-ductile concrete details seen elsewhere; only 90 degree hooks on links, bobbed bars not necessarily meeting others. In addition, there may have been a difference in earthquake motion, due to a possible change of soil type from fill to natural alluvium.

A 40m high RC Chimney constructed in 30 pours each 1.35m deep had lost 10 metres of its original height (Plates 33, 34). At the failed section, there was limited vertical reinforcement, with only 10mm bars at 450mm centres, in a 150mm thick section with a 100mm brick lining. Steel bands had been placed around the chimney to strengthen it. It was uncertain whether these were required due to cracking at that height due to previous loading by wind, or most probably an earthquake, or whether inadequate details had led to spalling of the concrete surface and loss of connection strength at the joints between precast units. The chimney had failed at a section just below where the strengthening occurred, the refurbishment having moved the failure plane to the next weakest point.

A more interesting case was the identical failure of two modern reinforced concrete chimneys (Plate 7.35). Experience on chimneys for Nuclear Chemical Plants has illustrated the need to consider different vibration shapes (Merriman, Ref 2). Increased acceleration has occurred at the higher positions on chimneys, resulting from a different mode to the main side sway mode (Figure 7.1). The effect of this second mode is to increase the moments at the top of a 60m chimney considerably, while only increasing the moment at the base by a small percentage (Table 7.2.) Therefore the application of an equivalent lateral force may not be adequate for design. In practice, it is more likely that these stacks have resonated with the ground input motion and failed where there is either a marked change in vertical reinforcement or a lack of confinement of the concrete. If the chimney is supported on a slab foundation, the chimney may be sensitive to Soil Structure Interaction affects. Other reports (EQE, Ref 4) describe similar failures towards the top of chimneys, suggesting that some reappraisal in the Codes may be necessary.

7.8 Conclusions

The great majority of damage to industrial facilities resulted from differential displacements, imposed by soil movement. The response of piping to reasonable levels of displacement was generally good, provided the supports allowed flexibility and did not force the displacement to be accommodated in a short length. In many cases, the movements could have been allowed for, if this failure mode had been considered in design. In the case of gross displacements, such several metres, piping generally failed, as would be expected. Such failures could have been avoided only by fundamental changes in foundation philosophy, wharf construction or soil compaction (see Chapter 8)

Failures also occurred as a result of items of plant not being bolted down adequately. This type of failure is not uncommon, but would be relatively inexpensive to prevent. Damage to industrial buildings, and hence their contents, resulted from poor (non-ductile) RC details, as with RC construction generally. Steel buildings generally avoided collapse, and remained serviceable, although long term damage may have occurred to welded connections, not visible to the investigation team. Many small industrial units were destroyed by fire.

7.9 References

1. Preliminary Reconnaissance Report of the 1995 Hyogoken-Nambu Earthquake, English Edition, April 1995, The Architectural Institute of Japan.

2. The Loma Prieta, USA, Earthquake of 17 October 1985. A Field Report by EEFIT, Institution of Structural Engineers July 1993.

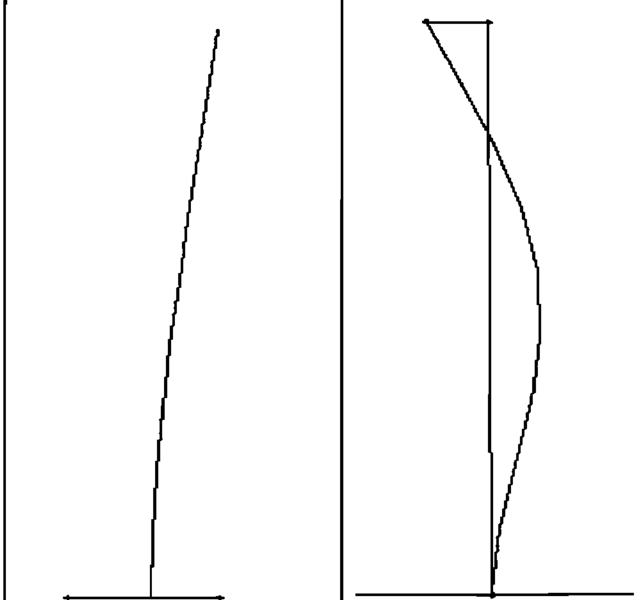
3. Plant Design for Seismic Conditions. P A Merriman, Institution of Mechanical Engineers, Nov 1991.

4. The January 17 1995 Kobe Earthquake, An EQE Summary Report, April 1995.

SITES	H	V
Nuclear	**(g)**	**(g)**
Mihama (3 Units)	.016	.006
Takahama (4 Units)	.022	.017
Ohi (4 Units)	.013	.013
Tsuruga (2 Units)	.011	.009
Non Nuclear		
Miyazu Energy Laboratory	.070	.056
Maizuru Ocean Observatory	.067	.039

Table 7.1: Acceleration levels related to Nuclear sites

MODE	1	2
FREQ. (Hz)	0.88	3.95



Acceleration mode shapes for stack

Figure 7.1

MODE	1	2	SRSS
Frequency (Hz)	0.88	3.95	
BM Base (KN.m)	28.239	25.909	39.436
Shear Base (KN)	565	1.452	1.810
BM 41.0m (KN.m)	6.781	10.765	12.922
Shear 41.0m (KN)	357	380	701
Deflection at Top (m)	0.088	0.013	0.089

Table 7.2

Typical RC Stack Design (response to horizontal earthquake motion)

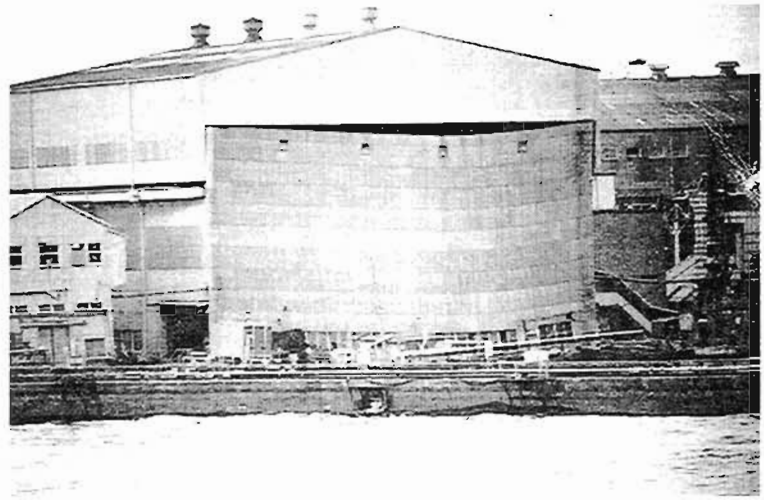


Plate 7.1 Settlement of gable posts at Mitsubishi shipyard

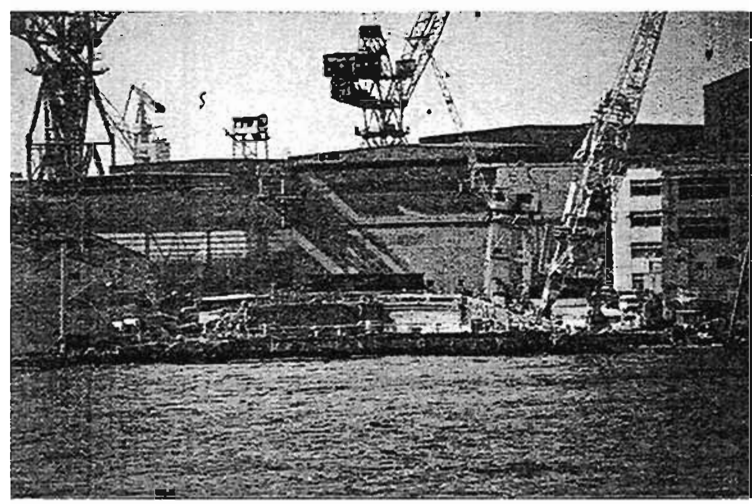
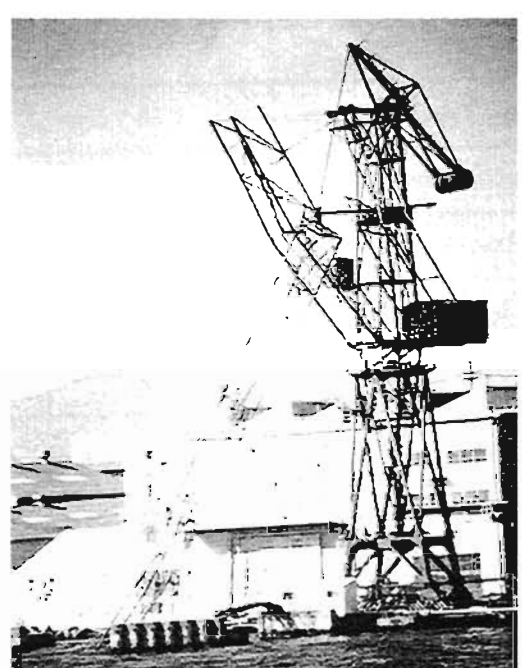


Plate 7.2 Settlement at Dry Dock, Kawasaki shipyard

Plate 7.3
Crane jib
Kawasaki shipyard



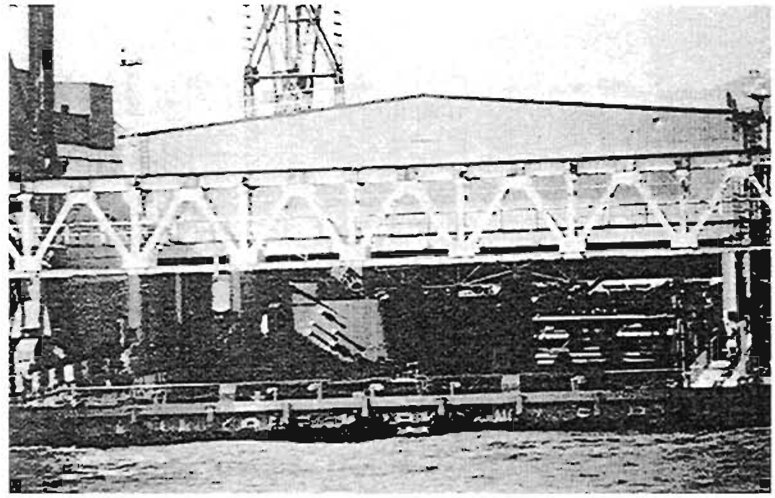


Plate 7.4 Dry Dock Gates at Mitsubishi shipyard

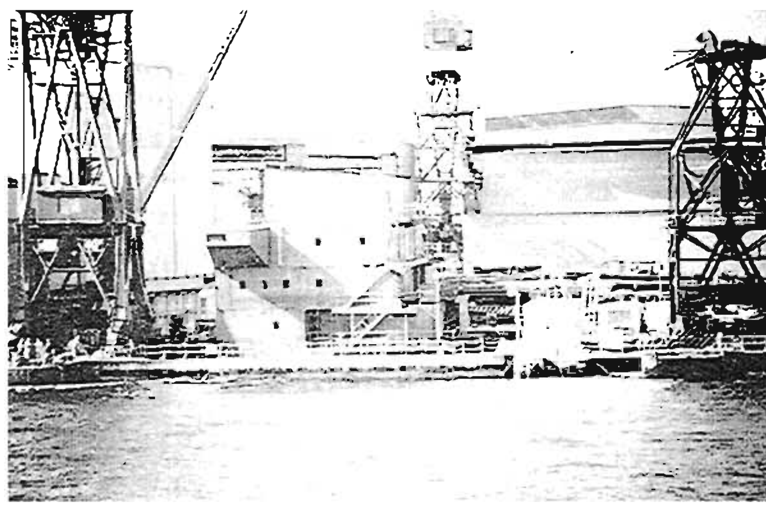


Plate 7.5 Ship under construction at Mitsubishi shipyard

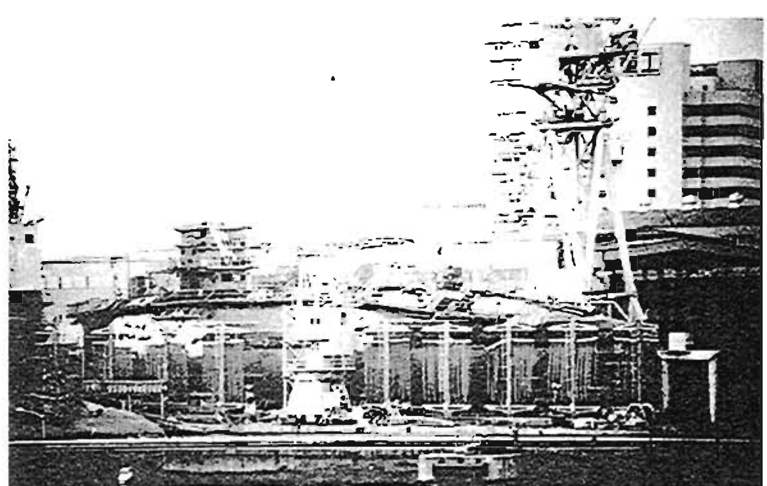


Plate 7.6 Submarine at Mitsubishi shipyard

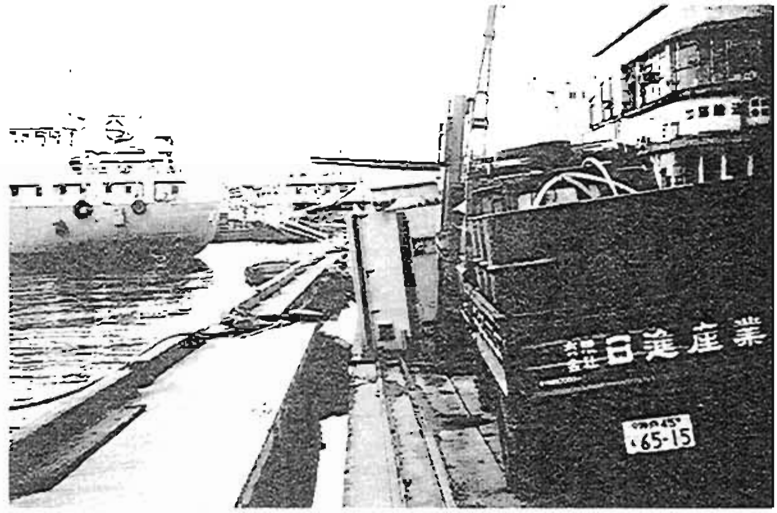


Plate 7.7 Flood prevention gate at harbour

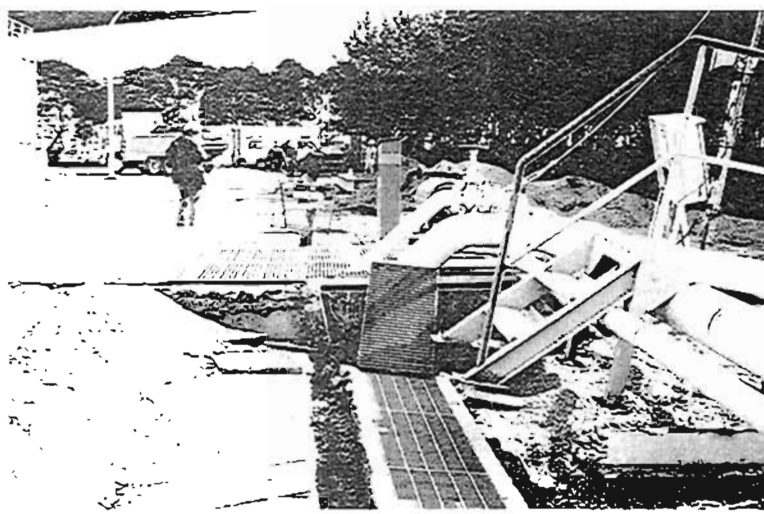
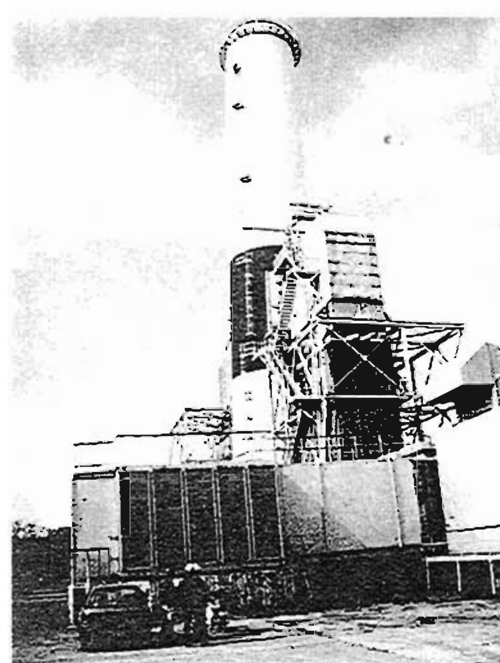


Plate 7. 2 Step in road, Higashi Nada GT Power Station

Plate 7 9
Higashi Nada
Gas Turbine
Power Station



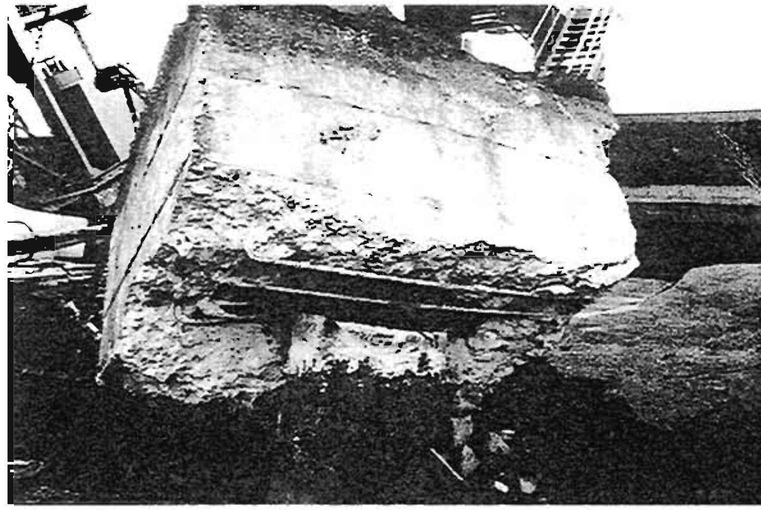


Plate 7.10 Steel pile unconnected to foundation

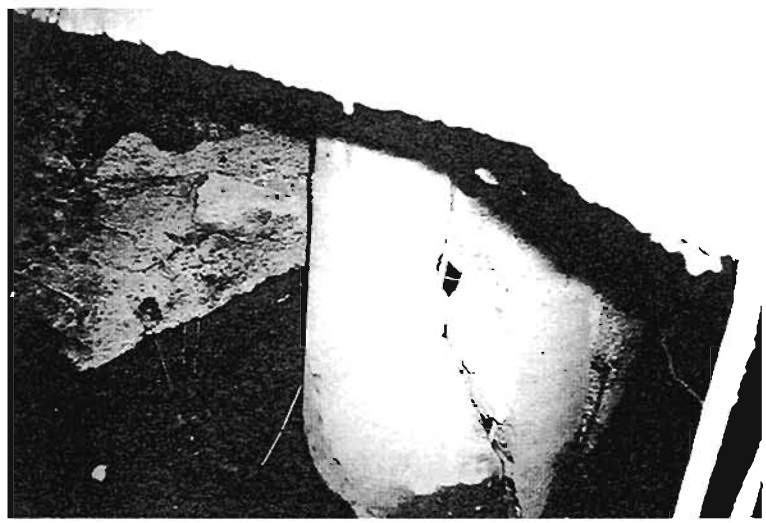


Plate 7. 11 Cracked RC hollow pile

Plate 7.12
Poorly connected
RC pile

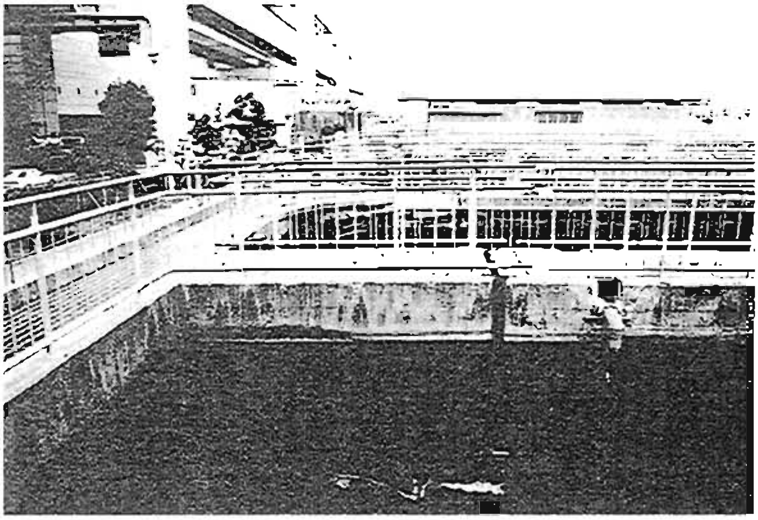


Plate 7.13 Rotation of end of RC tank

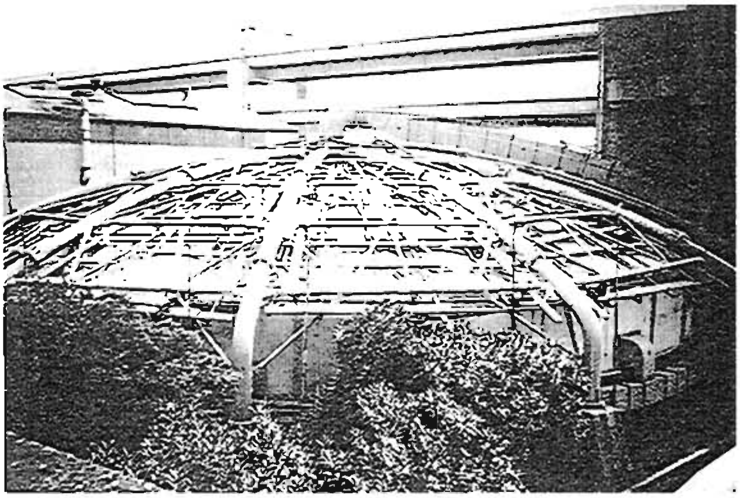


Plate 7.14 Glassfibre panel roof, with temporary supports

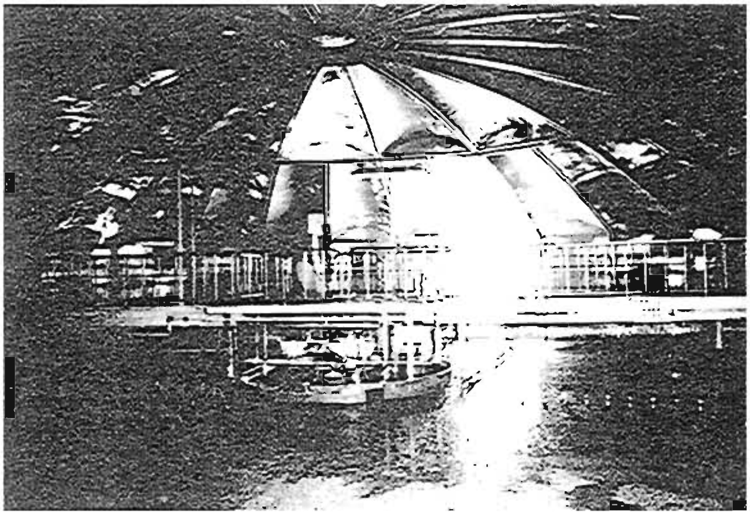


Plate 7.15 Glassfibre panel roof, internal view

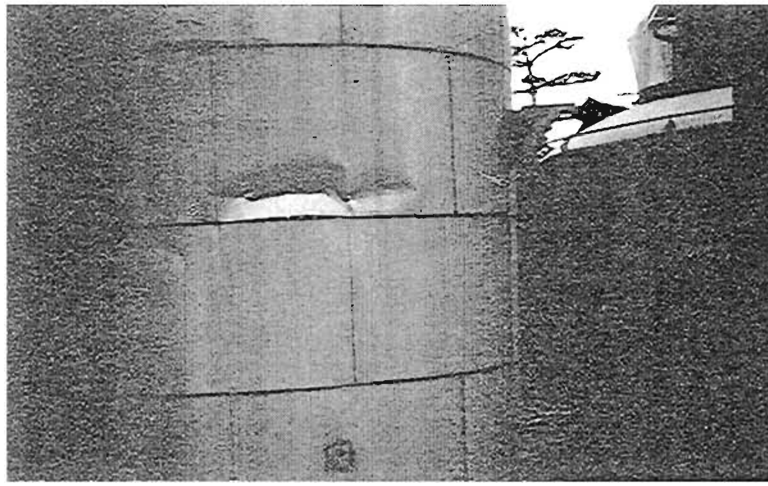


Plate 7.16 Buckling in tank wall

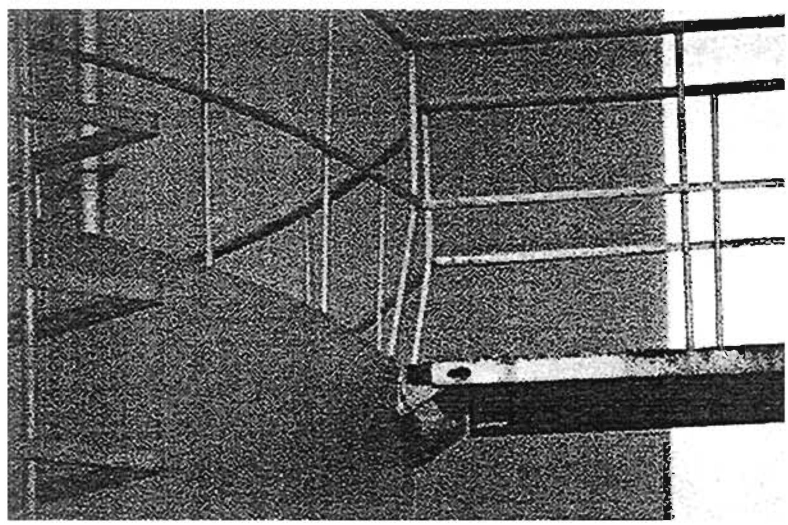


Plate 7 17 Effect of lack of movement joint

Plate 7.18
Intact
Steel Gasholder
Kobe Centre

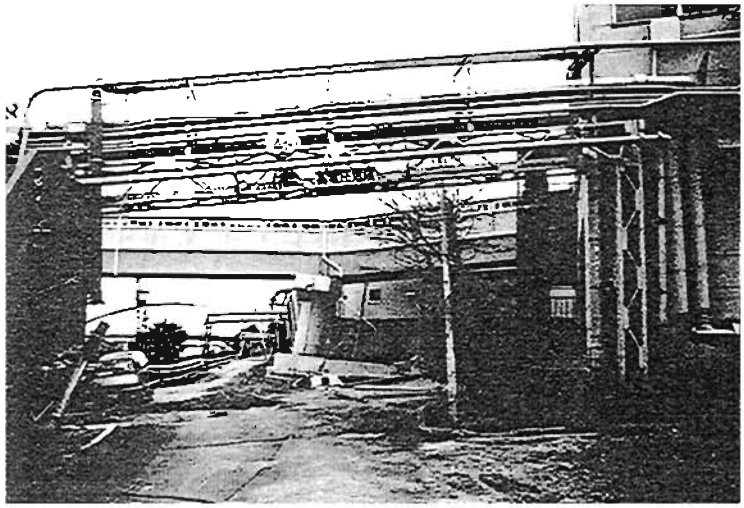


Plate 7.19 Pipebridge damaged by lateral ground spreading

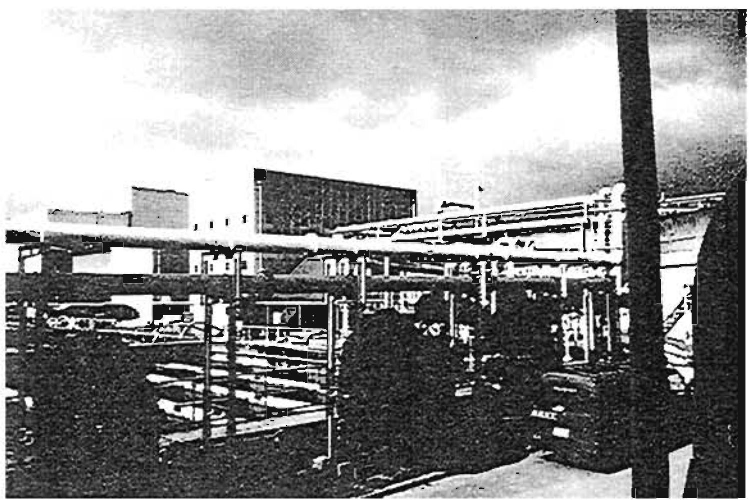


Plate 7.20 Nearby undamaged pipebridge on intact ground

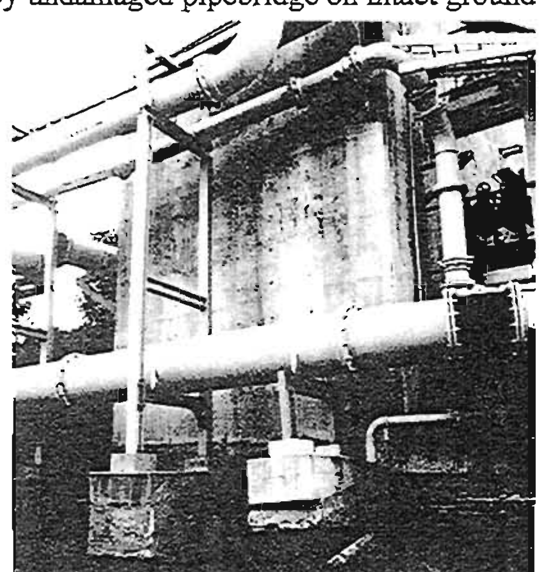


Plate 7.21 Pipes, now supporting their foundations

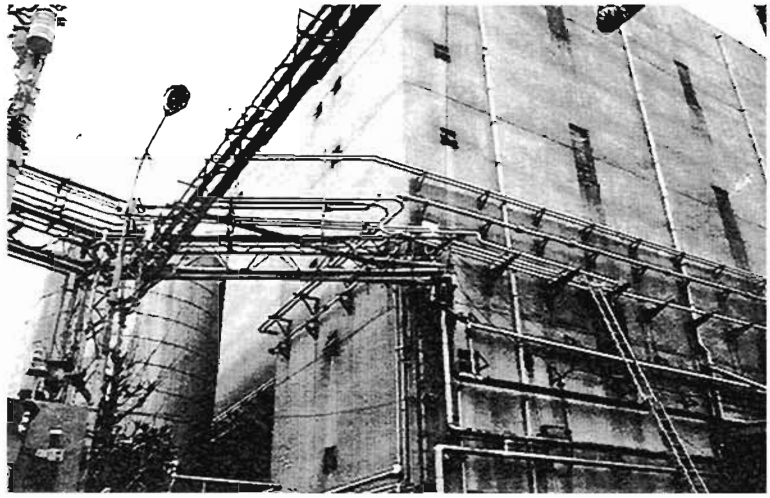


Plate 7.22 Pipework on wall of brewery

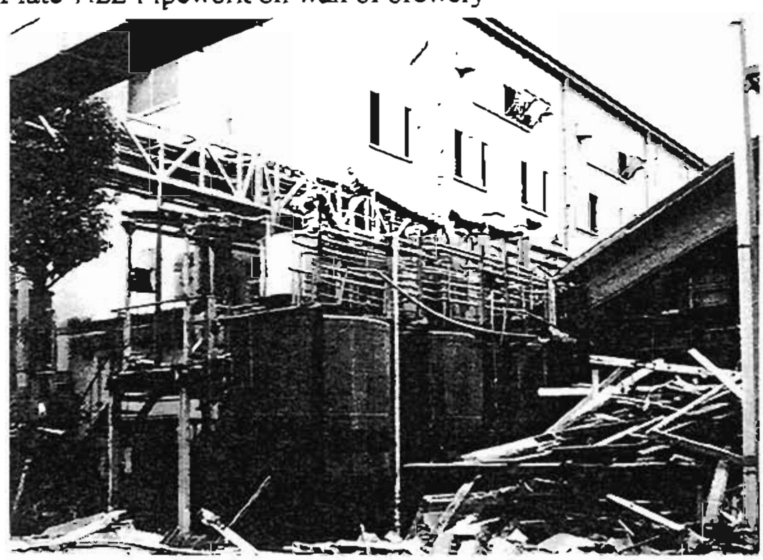


Plate 7.23 Plastic pipes, seriously damaged

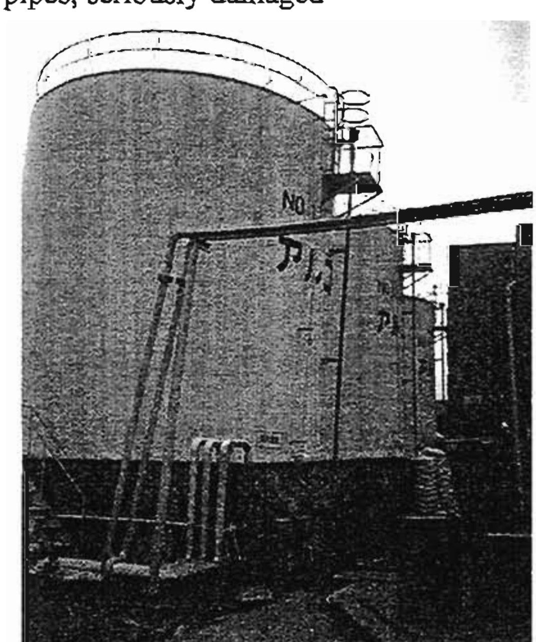


Plate 7.24
Lightly restrained pipes
deformed not fractured

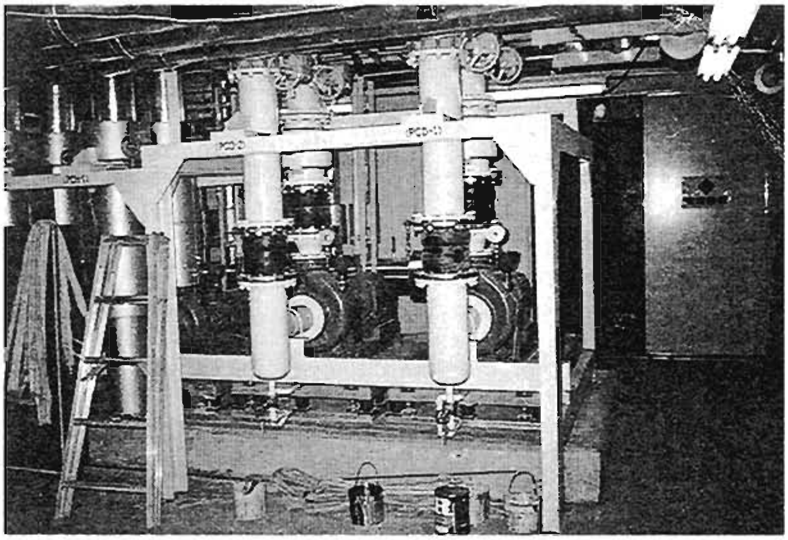


Plate 7.25 Pumps in basement; well anchored

Plate 7.26 Poorly anchored pump, sliding at base

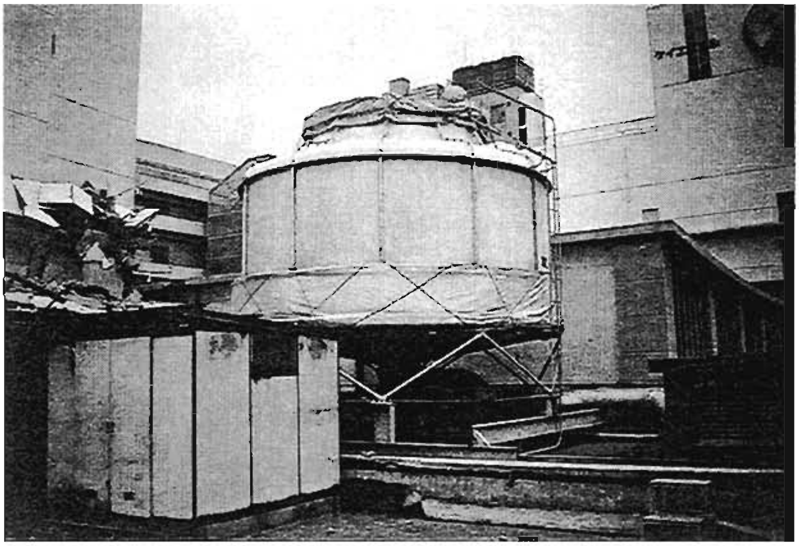


Plate 7.27 Lightly framed Air Handling Unit on roof.

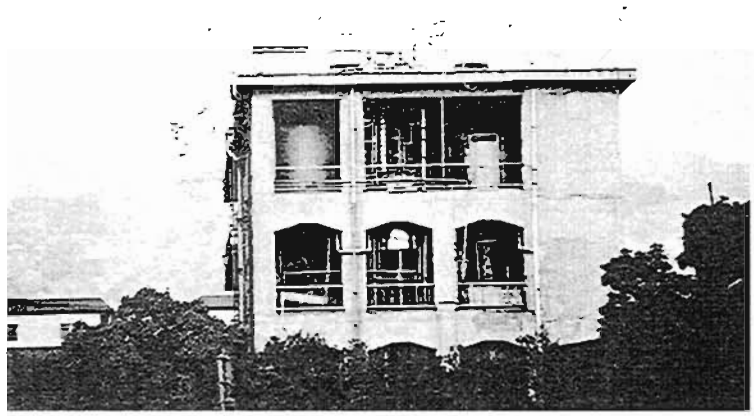


Plate 7.28 Unusual RC building with part filled panels

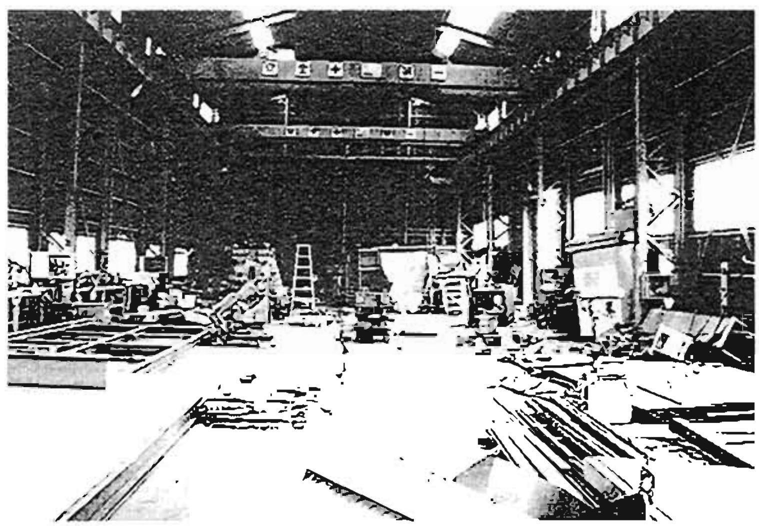


Plate 7.29 Machine shop, OHT crane, floor settlement

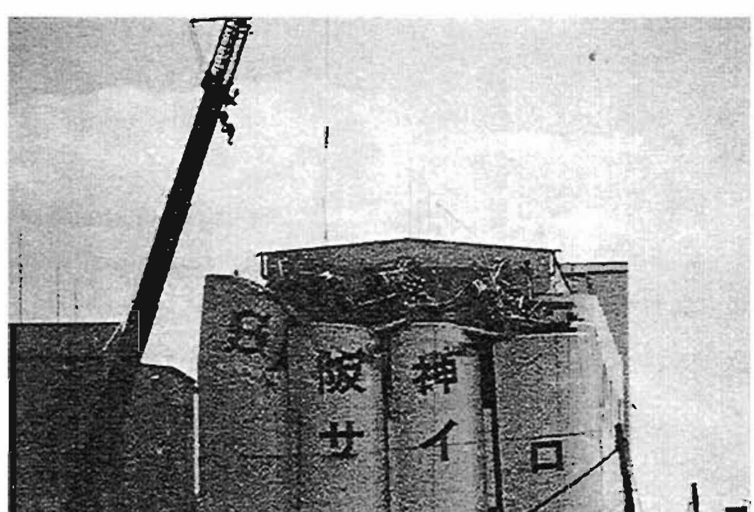


Plate 7.30 Silos with heavy plant above

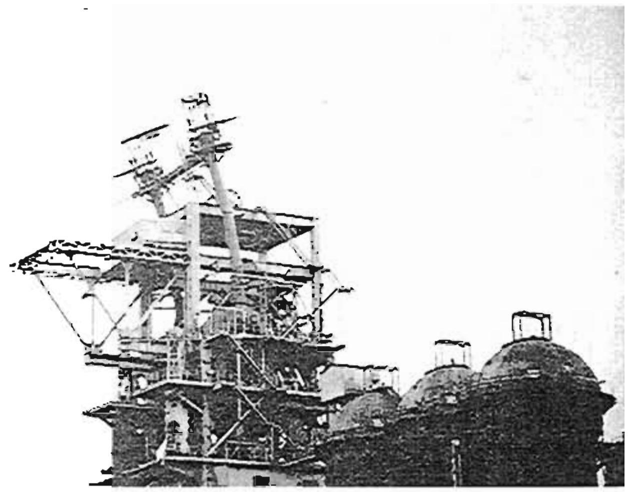


Plate 7.31 Kobe steel works

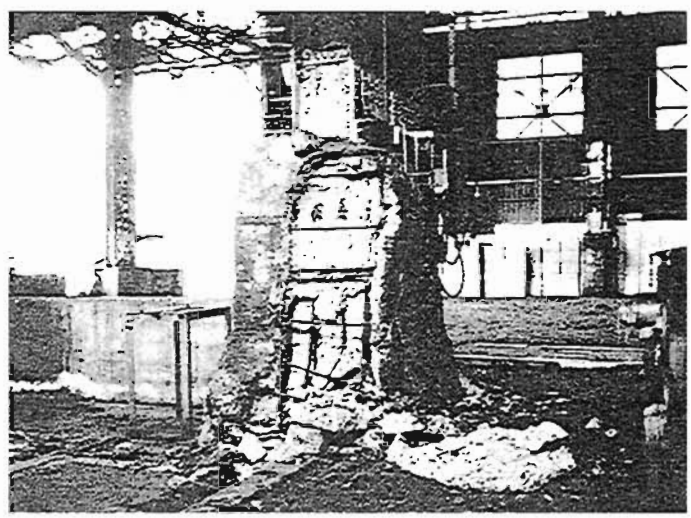


Plate 7.32 Old steelworks. Base of column

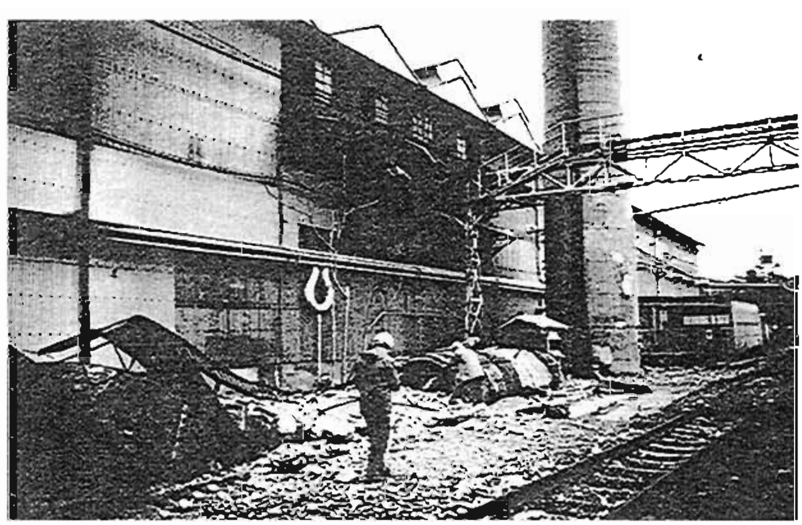


Plate 7.33 RC stack at old steelworks, top section fallen

Plate 7.34 Fallen section of RC stack

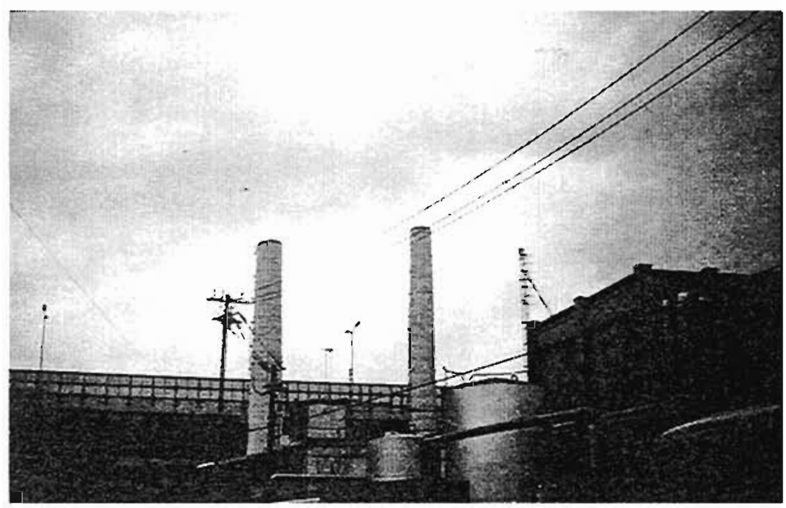


Plate 7.35 Two RC stacks, top sections fallen

Plate 7.36 Overhead Conveyor with ground settlement

Plate 7.37 Collapsed section of conveyor

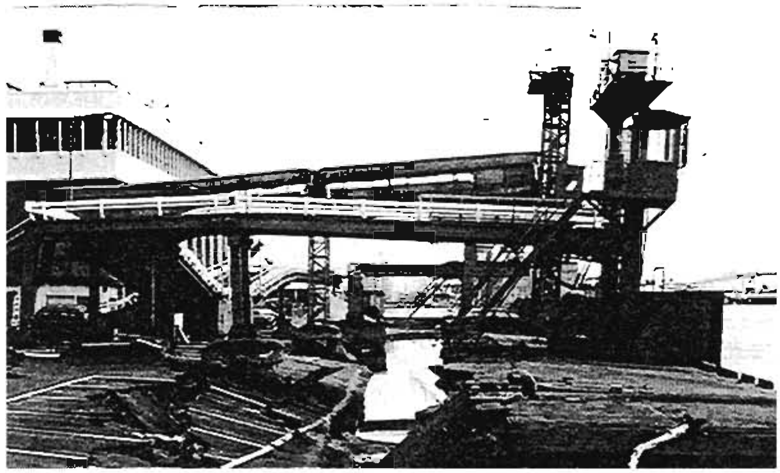


Plate 7.38 Settlement damage to ferry terminal

Plate 7.39
Arch pipebridge
Intact despite
support movement

8.0 GEOTECHNICAL ASPECTS

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8.1 Liquefaction and related effects

8.1.1 General

The most widely observed geotechnical phenomenon during the site investigation was liquefaction. The effects of liquefaction were seen in ground settlement, lateral spreads, sand boils, deformation and tilting of structures, fractured curbs and pavement fissures. The effects were widespread along the shoreline and in reclaimed lands and extended as far as 3 km inland from the waterfront in residential and industrial areas of Kobe, Ashiya and Nishinomiya as shown in Figure 8.1. The damage to underground utilities was also extensive throughout the region.

The most damaging effects caused by liquefaction were lateral spreads around the margins of Port Island, Rokko Island and mainland port areas. The lateral spreads caused many quay walls in Port Kobe to move 2-5 m into the sea. Piles under buildings and bridges were broken by lateral spreads. The Nishinomiya Bridge of the Hanshin Bay Expressway collapsed as the large lateral force induced by the liquefied soils pushed the caisson foundation. The interior of Port Island and Rokko Island subsided as much as 50 cm by liquefaction settlement.

In this section, firstly, the effects of liquefaction caused by the Hyogoken Nanbu Earthquake are described, based on the observation of ground settlement of reclaimed lands, waterfront structures affected by lateral spreads, and foundation failures of buildings and bridges. Secondly, ground improvement had been performed at several locations in Port and Rokko Islands prior to the earthquake. The performance of these improved ground sites is described. Finally, the liquefaction potential of fill materials used in Port and Rokko Island is discussed.

8.1.2 Ground settlement on reclaimed lands

Because of the strong ground motion, liquefied sand erupted and flooded many areas of the reclaimed lands in Kobe. As a consequence, large ground settlement was observed in many places in Port and Rokko Islands.

Port Island was constructed in two phases (see Figure 8.2). During the first phase, between 1966 and 1981, an area of 436 ha was reclaimed consuming 80 million m^3 of Masa soils. Since then, the phase 1 section has been used extensively for industrial and residential purposes including port facilities, wharves, high-rise apartments, an amusement park, and warehouses. The filling of the second phase, which started in 1987, created an additional 390 ha extending from the south side of the phase 1 section. A total volume of 92 million m^3 of fill material, which originated from mudstones of the Kobe Group formation, was used. At the time of writing (1995), the reclamation work in the phase 2 section has almost been completed.

Rokko Island, an area of 580 ha, was constructed between 1972 and 1992. Like the phase 2 section of Port Island, mudstone material of the Kobe Group formation borrowed from the Rokko Mountains was used as a fill material.

Typical subsurface sections of Port and Rokko Islands are shown in Figure 8.3. The depth of the fill is approximately 16~18 m in Port Island, whereas it is approximately 18~21 m in Rokko Island. The thickness of the fill increases from the shore line of Kobe into Osaka Bay. The SPT N values of the fill are approximately 5~10 in Port Island and 7~10 in Rokko Island.

Many pavement cracks and sand boils were found in both Port and Rokko Islands. For example, a manhole of approximately 60 kg was displaced by liquefied water pressure at the parking area near the KCAT Ferry Terminal in the phase 2 section of Port Island (Plate 8.1). Although most sand boil deposits were already cleaned away from the main roads at the time of the visit, there were several locations where the characteristics of erupted sands could be examined. For instance, a parking area in Minatojima Nakamachi 1-chome was covered by sand boil deposits as thick as 0.5 m (Plate 8.2). A thin layer of clayey soils was observed on top of the sand as well (Plate 8.3). This was presumably caused by the different sedimentation rates of different particle sizes contained in the erupted soils.

In Port and Rokko Islands, the elevation of structures supported by deep foundations provided a good reference in estimating the amount of ground settlement, because the deep foundations supported under the liquefied layer remained in place and the ground subsided around the building. For instance, piers supporting the elevated Portliner railways between Naka Futo Station and Kita Futo Station in Port Island rose relatively upwards by as much as 40~60 cm as shown in Plate 8.4. At a construction site in Minatojima Nakamachi, the ground where piles were installed remained in place as shown in Plate 8.5. It was found that the ground settlement in the interior of Port Island was relatively uniform with most of the roads and open spaces showing little evidence of severe deformation. Thus, the settlement of the phase 1 section of Port Island was estimated to be about 0.5 m.

The ground settlement in the phase 2 section seemed to be smaller than that in the phase 1 section. The KCAT Ferry Terminal building was the only reference point for estimating the ground settlement in the phase 2 section because it was the only building in this section supported by deep foundations. At this location, the ground settlement was estimated to be approximately 30 cm (Plate 8.6). The difference in settlement between the two sections in Port Island may be due to the difference in fill materials or the difference in the thickness of fill and clay layers. This needs further investigation.

At Rokko Island, the ground settlement was approximately 20~30 cm. Smaller ground settlement was reportedly found at the centre of the island possibly due to the improved ground. The performance of ground improved sites in Port and Rokko Islands is described in Section 8.1.5.

8.1.3 Damage to waterfront structures

Port Kobe is the second largest port in Japan after Yokohama and the port related industries generate about 39% of the city's gross industrial product. The earthquake damage resulted in total shutdown of the port facilities creating huge economic loss for the city.

Most of the waterfront structures in the port were built after 1906 (Figures 8.4). In the mainland port areas, the Shinko Piers Nos.1-6, the Hyogo Piers Nos.1 and 2, and the Naka Pier were built before World War II. Whereas, the Shinko Piers Nos.7 and 8, the Nada Futo Wharf, the Hyogo No. 3 Pier and the Maya Futo Wharf were constructed in the 1950s and 1960s. The waterfront structures in Port Island and Rokko Island were built since 1966.

Nearly all the quay walls in the port suffered from lateral spreads. Several different types of quay walls exist in the port, but about 90% of them are caisson walls. The other quay walls include sheet pile quay walls at Nada-hama Higashicho and pile supported walls at the Shinko Piers Nos. 7 and 8.

Port and Rokko Islands

A typical configuration of concrete caisson walls in Port and Rokko Islands is shown in Figure 8.5. The caisson wall consists of a reinforced concrete box of about 15 m wide and 20 m high (Plate 8.7). During the construction, it is towed to the site, submerged into position, and then backfilled with sand.

The structure supports the earth pressure acting on the wall by the friction at the bottom. The soft alluvial clay underneath the caisson is usually replaced by the same material used for back fills in order to provide sufficient bearing capacity in the foundation.

Typical damage of caisson walls observed during the site investigation is shown in a schematic diagram, Figure 8.6. Lateral spreads pushed the walls into the sea as much as several metres with an average lateral movement of 1-2 m (Plate 8.8). At the apron slabs behind the quay walls, grabens and depressions were formed 2-4 m deep and 10-20 m wide (Plate 8.9). Lateral spreads also created ground fissures parallel to the walls as far as 200 m inland. The concrete caissons moved either in rotation or sliding, indicating that the significant displacement was possibly caused by inertia of the heavy concrete caissons, large dynamic lateral pressures created by the liquefaction of back fill materials, and the reduction of bearing capacity of replaced sands under the caissons. However, the caissons themselves showed no damage.

The permanent displacement of Port and Rokko Islands measured by the GPS system is shown in Figure 8.7 (Iai, 1995). The horizontal displacement ranges from 2-6 m with an average of 3 m. The displacement vectors in the figure suggest the major movement of the caisson walls is in a north-south direction, which is similar to the direction of maximum acceleration described in Chapter 3.

Nearly all the major cranes in the islands suffered from the displacement of the quay walls supporting them. For example, some giant cranes in Rokko Island were pulled apart at the base (Plate 8.10). The forelegs placed on top of the caissons moved forward towards the sea due to lateral spreads. This movement caused buckling failure either at the foreleg supports or at the lateral supports. The hind legs were placed on concrete blocks supported by pile foundations. Some of these piles were exposed by the subsidence of the surrounding ground (Plate 8.11). The crane rails on the concrete blocks were displaced and deformed by the movement of the cranes (Plate 8.12).

The magnitude of lateral pressures that moved many quay walls can be examined from the liquefaction damage observed at the old sea walls, which exists along the border of Phase 1 and 2 sections of Port Island (see Figure 8.2). At present, the front of the walls is filled by the Phase 2 reclamation as shown by the cross-section of the walls in Figure 8.8 (Konoike construction, 1995). The pavement along the Phase 1 side was covered by thick deposits of erupted soils (Plate 8.13). Most of the soils were sand overlying a few millimeters of clay. Some large boulders, which originated from the backfill gravel, were also present on the surface. The differential settlement between the sea wall structures and the pavement was approximately 30~50 cm. At the toll gate next to the old sea walls, a dried water splash marker of liquefied soils was found at about 1 m above from the ground surface, suggesting that a large amount of water came out of the ground (Plate 8.14).

It is suspected that the extensive liquefaction observed at the old sea walls is due to the fact that the walls were not able to move in any direction since the front face was filled by new reclamation. In such a case, large pore pressure developed in the fill could only be released by water flowing upwards to the ground surface. As for the cases on caisson walls along the waterfront, the movement of the caissons into the sea created a crack from which the excess pore pressure dissipated. Consequently, few sand deposits were found behind the caissons. Nevertheless the extent of sand boils at the old sea walls mirrored the scale of lateral spreads that occurred along the caisson walls in Port Island.

Four months after the earthquake, another site visit was completed to investigate the repair work on the damaged quay walls in Port Island. As a temporary measure, the displaced caissons were aligned by very large on-sea cranes and the subsided section behind the caissons was backfilled (Plate 8.15). However, it is reported that new concrete caissons of 11.6 m wide and 12.5 m high will be installed in front of the old caissons for permanent measures (see Figure 8.9). In addition, the foundation under the caissons will be improved by the sand compaction pile method before putting the new caissons in place.

Mainland Port Areas

The quay walls along the mainland port were also greatly affected by lateral spreads (see Figures 8.1 and 8.2). In particular, the Shinko piers located north of Port Island were severely damaged. The quay walls located at the tip of the Shinko Nos. 5 and 6 Piers were block type gravity structure (Figure 8.10), and many of them moved toward the sea side (Plate 8.16). The Shinko No. 7 and 8 Piers were piled piers. The piles seemed to have survived the earthquake, but many columns of the warehouses located on the piers were torn apart (Plate 8.17). At the east bank of the Maya Futo wharves, severe lateral spreads and differential settlements were observed (Plate 8.18). The reinforced concrete piers of the Maya-ohashi bridge placed on this bank contained large shear cracks and differential settlements of about 1 m occurred in many locations.

The east section of the Maya Futo No.1 pier was designed to resist large earthquake loading by installing strong pile foundations or gravity type structures. Apparently, they survived the earthquake with almost no lateral movement and no vertical settlement at the apron slabs (Plate 8.19). The cross sections of these quay walls are shown in Figure 8.11. The two sections at the north side of the pier are composed of two structures: a steel cell of 15.5 m diameter for a counterweight and steel pipe sheet piles with inclined shafts (Figure 8.11 (a)). At the south side section of the pier, the structure is composed of steel cells on the land side and concrete caisson on the sea side (Figure 8.11 (b)). A seismic design coefficient of 0.25 was used for these quay walls. Whereas, most of the quay walls in Port Kobe were designed using a seismic design coefficient of 0.15 or 0.18.

8.1.4 Damage to Foundations

Pile foundations

The effect of liquefaction on pile foundation was very hard to discover because they needed to be excavated for inspection (see Plate 8.20). Not much damage to piles has been reported since the earthquake, but the number of piles suffering damage is expected to increase with time (e.g. Tokimatsu et al. (1996)).

In the interior of Port Island, most of the structures supported by pile foundations remained in place and the surrounding ground subsided around the structures, as described in Section 8.1.2. Most of the piles used in Port Island are either steel piles or precast concrete piles. The piles are approximately 40 m long and are placed into the sand gravel layers under the alluvial clay. For example, the foundation conditions under the Kobe Shimin Hospital are shown in Figure 8.12. The building was designed to withstand an earthquake of the same class as the Kanto Great Earthquake.

Many piers supporting the elevated Portliner railway on Port Island rose relatively upward as much as 50 cm. Steel piles were used for these piers as shown in Figure 8.13 (Watanabe, 1981). Skirt sheet piles were placed around the piles to prevent the surrounding soils flowing under the footing in case the surrounding ground subsides relative to the foundation and creates a void.

Some piles near the quay walls on Port Island were found to be broken by lateral spreads. At Minatojima 3-chome, a precast reinforced concrete pile of 400 mm diameter was displaced about 20 cm from the pile cap (Plate 8.21). The nominal reinforcement indicates that it was designed to resist vertical loads only. As a consequence of the pile damage, a warehouse founded on the broken piles tilted by approximately 1/500 radians.

Several tanks in Port Kobe also suffered from lateral spreads as well. A site visit was done at the Higashi-Nada Power Station, located north of Rokko Island. For one of the tanks in the station, an excavation has been made around the perimeter to inspect the pile damage (Plate 8.22). This tank is reportedly supported by 33 piles, each 33 meters long. Twelve of the 33 piles are arranged in an outer ring near the perimeter of the tank and were exposed for inspection. The piles were reinforced concrete piles with diameters of approximately 35 cm. The surrounding ground settled approximately 28 cm. One pile contained large cracks allowing large blocks of concrete to be broken out. The other piles had hairline cracks in the upper meter or two (Plate 8.23). However, damage to the piles appeared to be insignificant to the stability of the tank.

The pile foundations under some piers of the elevated Portliner Railway running in downtown Kobe were damaged by lateral spreads. The repair work was observed during the site visit six months after the earthquake. The pier was supported by four piles with diameters of approximately 60 cm. Cracks in the piles were reportedly found after the excavation of the surrounding ground. As part of the repair work, four additional piles were installed adjacent to the existing piles to resist lateral loads in future earthquakes (Plate 8.24).

Caisson foundations

Liquefaction and lateral spreads seemed to affect the collapse of a section of the Nishinomiya bridge of the Hanshin Bay Side Expressway. The foundation condition of the piers of this bridge is shown in Figure 8.14. It consists of a concrete caisson of 40 m long, 13 m wide and 23 m high. The pier is located approximately 20 m from the concrete caisson quay walls. The entire fill showed evidence of liquefaction (Plate 8.25). Many surface cracks were observed parallel to the shore line, and the extent of lateral spreading reached as much as 100 m inland into the caisson foundation (Plate 8.26). The total displacement of the lateral spreads was approximately 3 m. It was evident that the caisson foundations were pushed toward the sea by the force of lateral spreads (Plate 8.27). Consequently, the joint connectors between the spans broke and the girders supporting the expressway dropped to the ground.

Shallow foundations

The shallow foundations in reclaimed lands were also to greater or lesser degree affected by liquefaction. At the KCAT Ferry Terminal in the Phase 2 section of Port Island (see Figure 8.2), the damage to two buildings was investigated. The buildings were located adjacent to each other close to the water front, and fissures caused by lateral spreads intersected the foundations of each building. The larger terminal building was constructed on a pile-supported shaft foundation. In spite of extensive lateral spreading and liquefaction settlement of 30 cm, no significant damage was found in the exposed shafts or the building itself (Plate 8.28). On the other hand, a three storey office building which seemed to be on shallow foundations was greatly affected by lateral spreads. The building itself was not damaged, but the structure tilted as a block by about 2.5 degrees toward the sea side (Plate 8.29). The building was also slightly sheared as seen from observation of unfitted windows.

Despite extensive liquefaction in recently filled Ashiya Shiomi-cho, the shallow foundations of residential houses in the area seemed to perform relatively well. Although a few houses tilted slightly and need to be jacked back, none were severely damaged (Plate 8.30). It is suspected that the foundations of these houses are reinforced by tying the perimeter wall and footing together with interior cross walls (Plate 8-31) (Youd, 1995).

8.1.5 The effect of ground improvement on liquefaction settlement

The improved ground sites on Port and Rokko Islands sustained significantly less deformation and damage than did the adjacent ground. The locations of the improved sites are shown in Figure 8.15. The types of ground improvement are sand drains, preloading, vibro-rod compaction, and sand compaction piles. The performance of the improved sites showed the effectiveness of these ground improvement methods against liquefaction.

Most of the improved ground sites in the islands were treated by either sand drains or preloading, which are used primarily for settlement control of soft alluvial clay foundation under a high-rise building founded on a pile foundation. Although they are used to accelerate consolidation settlement, the observation of the sites showed that they were effective against liquefaction. For the sand drains method, the vibration while installing sand drains densified the fill material, thereby increasing its cyclic strength. It is reported that the installation of sand drains can give increase in the SPT value of the order of 2 to 3 blow counts (Fudo Construction, 1995). Preloading may also increase the cyclic strength of fills by increasing the overconsolidation ratio (OCR).

Among many methods to densify the ground to resist liquefaction, the vibro-rod method and the sand compaction pile method were used in several places on Port and Rokko Islands. In the vibro-rod method, a pile hammer is used to force a specially designed probe repeatedly into the ground (Figure 8.16). The rod will be pushed and pulled for about 1 m per cycle towards the ground surface from the base of the soil layer that needs to be densified. As the ground around the rod settles during the penetration, additional sand is added from the ground surface. A common spacing between columns is between 1.5 m and 3.0 m depending on the loading conditions and fines content of the soil being treated. It is reported that this method is not suitable for a soil that has fines content of more than 15%. Also, for a site that has clay seams in a sand layer, it becomes difficult to add sands from the surface into the ground during penetration.

On Port Island, the vibro-rod method was used at four locations : (1) a warehouses site at the northwest corner of the Island, (2) the Portpia Amusement park, (3) a buildings site in the central west section, and (4) a four storey building site at the central east section. Clear evidence of liquefaction, such as surface cracking and sand boils, was limited in these areas and the ground settlement was much less than in the adjacent untreated areas (Plate 8.32). The treatment depths of the sites are 12~19 m with column spacing of 2.4~2.6 m. The SPT N values before and after the treatment of these sites are shown in Figure 8.17 (Fudo Construction, 1995). In general, an increase of between 5 and 15 with an average of 10 was achieved using this method. At the four storey building site in the central east section, Tanahashi *et al.* (??) report that the wet density of the fill increased by approximately 6% after the treatment, resulting in ground settlement of approximately 50 cm. This magnitude of settlement is about the same as the liquefaction settlements observed at untreated sites on the Island.

The sand compaction pile methods were used at four locations on Rokko Island. In this method, first, a casing pipe is driven to the desired depth using a vibrator at the top (Figure 8.18). Then, sand is introduced into the pile and the pipe is withdrawn for a certain distance. In some cases, compressed air is forced into the pipe to hold the sand in place. Finally, the casing is vibrated downward to compact the sand, hence creating a sand pile. This process is repeated until the sand pile reaches the ground surface.

The sand piles used in Rokko Island were 0.7 m diameter installed with 2~2.25 m spacing. The treatment depth was 6~7 m at the Rokko Island East Parking site and the Nikko Driving School site and 15~16 m at the Rokko Island Train Inspection site and the Konan University Sports Facility site. The foundation condition at the Rokko Island Train Inspection site is shown in Figure 8.19 (Nakajima et al., 1992). At this site, the target N value was N = 14 at the centre of the building and N = 10 surrounding the building; an average of N = 8.1 before treatment. After treatment, average N-values of between 15.8 and 19.4 were achieved at the location between the piles. At the tip of the piles, the N values were between 22.6 and 24.8. The treated sites reportedly performed well during the earthquake compared to the adjacent untreated sites (Fudo Construction, 1995).

The effect of ground improvement techniques against the observed ground settlement in both Islands is shown in Figure 8.20 (Ishihara et al, 1995). For untreated sites, ground settlements of 40~50 cm were observed in many places. Considering the depth of fill, these settlement values correspond to 3-5% volumetric strain, which concurs well with common laboratory data obtained by cyclic triaxial tests. Application of preloading and sand drains reduced the liquefaction settlements to 15~25 cm. However, they cannot be considered as effective against liquefaction since the range of settlement is not acceptable for building foundations. Whereas, the sand compaction pile method and the rod compaction method were quite effective with almost zero displacement even by the strong shaking of the earthquake.

8.2 Damage to tunnels and underground structures

Many tunnels and underground structures in Kobe survived the earthquake fairly well compared with the damage observed in buildings and bridge piers on the ground surface. However, some of the shallow underground structures of the subway systems in Kobe were severely damaged. The most catastrophic damage was the collapse of the Daikai station of the Kobe Kousoku Railway.

There are two subway systems in Kobe: the Kobe Kousoku Railway and the Kobe Municipal Railway (see Figure 8.21). The Kobe Kousoku Railway is 7.6 km long with an underground section of 6.6 km made by the cut-and-cover method. The railway was constructed between 1963 and 1968. The Kobe Municipal Railway, which was constructed between 1977 and 1983, has an 11.3 km underground section of which 9.1 km was constructed by the cut-and-cover method, 0.4 km by the shield method and 1.8 km by the NATM method. There are 17 underground stations in total, which were all constructed by the cut-and-cover method.

Earthquake damage was reportedly found at the Daikai and Kousoku stations of the Kobe Kousoku Railway and at the Shin-Nagata, Kamisawa and Sannomiya stations of the Kobe Municipal Railway. Most of the damage was caused by the failure of reinforced concrete columns in the stations and running tunnels.

The underground concrete box structure of the Daikai station was found to be the most severely damaged of the underground structures. The station collapsed along a section of approximately 120 m (Plate 8.33). The cross-section of the station is shown in Figure 8.22 (The Japanese Society of Civil Engineers, 1995). The main part of the station consisted of a reinforced concrete box of 17.0 m wide and 7.17 m high. Thirty-five reinforced concrete columns were placed 3.5 m apart along the centre line of the station. The size of the reinforced columns was 1 m long x 4 m wide x 3.5 m high reinforced by steel bars of 30 mm diameter. Tie bars of 9 mm diameter were placed with 30 cm spacing. The average thickness of the overlying soils was 1.9 m at the first floor basement section and 4.8 m at the second floor basement section. The soils were mainly alluvial sand with a clay interlayer on the west side of the station as shown by the subsurface condition in Figure 8.23 (The Japanese Society of Civil Engineers, 1995). The SPT N values varied from 10 to more than 50, and the depth of the water table is approximately 3 m.

Thirty out of the thirty-five central columns in the station were sheared to failure with large bending of reinforced steel bars. As a consequence, the top slab collapsed into the tunnel as shown by the schematic diagram in Figure 8.24. The surface settlement of 2~3 m was measured along a section of the Daikai Road of 100 m long and 23 m wide. The side walls had some cracks at the corners.

The location of the shear failure in damaged columns was not in the middle section of the columns but about 1/3 of the column height from the top slabs or from the bottom slab (unfortunately, the photos taken during the inside visit did not show up clearly). Judging from the reinforcement in the columns, they were designed primarily to resist axial loading and not to resist the bending moment and shear forces of the earth pressures acting on the side walls or overburden pressures. It was found that the corners of the box yielded first because of the bending moment (Nikkei Construction, 1995). Subsequently, the shear force acting to the columns exceeded their shear resistance. As a consequence, the columns failed primarily due to shearing.

The reconstruction of the Daikai station started in June, 1995 (Nikkei Construction, 1995). The collapsed section of 90 m long and 23 m wide will be excavated first. The bottom slab and the rails will be reused. For central columns, three sections of 45 cm square steel pipe filled with concrete will be jointed together to make a 450 x 1350 mm section. A total of 26 D32 reinforced bars will be placed in the concrete. The thickness of the steel pipe is 12 mm. For the upper slabs and wall sections, the thickness of concrete will be as same as before, but the diameter of the rebars will increase from 22 mm to 25 mm. Additional reinforcement will be installed at the connection between the slabs and the wall. In the new design, it is emphasized that the shear failure at the central columns will not occur before the bending moment failure.

The 1 km railway section between the Daikai and Kousoku Nagata Stations was less damaged than the collapsed station. However, it was reported that approximately 210 reinforced concrete columns in the section had some cracks. The south side wall moved into the tunnel as well. For reconstruction, the section will be reinforced by placing 6 mm thick steel around the middle columns, upper slabs and side walls.

Compared to the observed damage to the Kobe Kousoku Railway, the underground section of the Kobe Municipal Railway was less damaged. The cross-sections of the Kamisawa and Sannomiya Stations are shown in Figure 8.25 (PWRI, 1995). The damage is primarily to the reinforced concrete columns. In Sannomiya station, the central columns of the second and third floor basements were 550~650 mm diameter steel pipes. These columns showed no damage.

The tunnels through the Rokko Mountains were reportedly less damaged even by the movement of active faults in the mountains. In the Rokko Tunnel (16,250 m) and Kobe Tunnel (7,970 m), it is reported that more than 100 cracks were found in the concrete linings, but most of them were found to be easily repairable.

8.3 Damage to retaining walls

Failure of gravity retaining walls was observed along the Hanshin Railway line and the Japan Railway (JR) line. In many cases, the type of failure was tilting and sliding of the walls due to the increase in lateral earth pressure and inertia force. For instance, 5 m high concrete gravity walls overturned along the Hanshin Railway line (Plate 8.34). Large displacements of back fill material were observed in a collapsed masonry stone retaining wall (Plate 8.35).

According to Tatsuoka et al. (1995), there are four geogrid reinforced walls in the region affected by the strong shaking of the earthquake. They are : (1) the Amagasaki section (length : 1 km, average height : 5 m), (2) the Tanata section (length : 260 m, average height : 6 m), (3) the Amagasaki station (length : 400 m, height : 3 ~ 8 m), and (4) the Tarumi section (length : 200 m, average height : 5 m). Each of these walls was constructed between 1990 and 1994. The performance of these walls provided an excellent opportunity to investigate the seismic stability of the current state-of-the-art walls.

The retaining walls in Tanata, between the Ashiya and Setsumotoyama stations of the Japan Railway line, were subjected to very high ground acceleration of approximately 800 gal in the east-west direction (Figure 8.26 (a)). More than 80% of houses in this region collapsed (Plate 8.36). The geogrid walls are 6 m high with grid reinforcements approximately 2.5 m long(tensile strength = 3 tf/m) placed behind the cohesionless backfill as shown in Figure 8.26 (b). The wall is built on the ground without any deep foundation support. A seismic design coefficient of 0.2 g was used for the seismic stability. Compared to the severe damage of the surrounding area, the damage to the geogrid walls in the Tanata section was minor. The surface of the wall contained very small hairline cracks of 2 mm. The maximum lateral movements of the wall were 3 cm at the bottom and 8 cm at the top at the joint section with a reinforced concrete box structure, which is the underpass of the railway embankment (see Figure 8.26 (a) and Plate 8.37).

A reinforced concrete wall was located adjacent to the geogrid walls at the Tanata section. The wall was founded on piles of 1.2 m diameter and 6 m long with 3 m spacing as shown by the cross-section in Figure 8.26 (c). The wall also survived the earthquake with minor damage similar to the geogrid walls. The lateral movement was approximately 10 cm.

8.4 Damage to dams and embankments

There are numerous dams in the region affected by the earthquake. Public Research Work Institute (PRWI) (1995) conducted inspections on dams where the recorded acceleration was more than 25 gal. A total of 251 dams (101 in the Kansai region) were investigated by PRWI immediately after the earthquake. Most dams were at low reservoir water level at the time of the earthquake. In general, the dams performed well with no major damage for protective measures.

The only exception was the failure of the Niteko dam in Nishinomiya possibly due to soil liquefaction. The dam is located in a small valley and is primarily used for water supply and recreational uses. The total length of the dam is 450 m and the width is approximately 100 m as shown in Figure 8.27. The dam consists of three reservoirs (the north, central, and south) separated by embankments 5~6 m high. The dam was originally constructed more than 100 years ago with minor modifications in recent years. At the time of the earthquake, the dam was filled with water at 1/3 of the maximum reservoir height.

Due to the strong motion of the earthquake, the north and east sections of the bank protection and the two embankments collapsed (Figure 8.27 and Plate 8.38). The failure pattern appears to be deep sliding downstream. The subsurface soil is reported to be fine sands or sandy silts, strongly suggesting that soil liquefaction at the foundation caused the failure. Several sand boils were found in the residential area near the site.

River dikes along the Yodo river in Osaka were also severely damaged by the earthquake. The peak ground acceleration of the area was estimated to be 0.2 to 0.3 g. A section of the Osaka side of the river approximately 1.8 km long experienced major damage in the form of overturning of the parapet and the concrete slab protection wall. The embankment broke up into blocks and sank into the river possibly due to soil liquefaction as shown by a schematic diagram of Figure 8.28 (the Japanese Society of Civil Engineers, 1995). Many sand boils were found in the surrounding area. It has been reported that the subsurface soils consisted of a sand layer more than 12 m deep with SPT N values of 2 or 3. Due to the collapse, the dikes settled at a maximum of 3 m. This is almost half of its total height of 6~8 m. Thus, the flooding was a primary concern after the earthquake because the wall height was only 1.8 m higher than the high tide level.

8.5 Earthquake induced landslides

Several notable landslides occurred on slopes along the Gosukeyama and Ashiya faults in the Rokko Mountains. The landslide at Nikawa, north of Nishinomiya, which buried thirteen residential houses and killed thirty four people, was the most disastrous one (Plate 8.39). In addition, some landslides damaged houses at Okamoto in Higashi-Nada ward, Kobe. In most of the cases, slides occurred in the decomposed granitic Masa soil.

The size of the Nikawa landslide was about 250 m long and 50 m wide as shown in the topographic map of Figure 8.29. The head scarp was as high as 100 m. The debris material was decomposed granite and flowed into the Nikawa river, which is located at the toe of the hill. The slope of about 15 degrees angle seems to be a cut-and-fill portion of the natural hill. The upper portion of the hill was a wooded area, with residential houses built below. A filtration centre is located on top of the hill immediately adjacent to the landslide. The pile foundation of the two storey building for water treatment was exposed on the scarp of the slide, showing the presence of fill. It is reported that the slide will be stabilized by boring one drainage well and installing 140 steel pipe piles of 50 cm diameter and 20 m long (Nikkei Construction, 1995).

Several rock falls were observed along natural slopes of the Rokko Mountains. Huge rounded boulders, originating from natural weathered granitic rocks rolled downhill at a hill side in Nishinomiya. A boulder of approximately 2 m in diameter fell for a distance of 20 m and completely crashed a car and demolished a gate (Plate 8.40). The area was susceptible to rock falls even before the earthquake because deflection fences were found in some places.

8.6 Conclusions

The most significant geotechnical aspect of the Hygoken Nanbu Earthquake was liquefaction. The effects of liquefaction were seen in ground settlements, sand boils, deformation and tilting of structures. They were widespread along the shoreline and in reclaimed lands and extend as far as 3 km inland from the waterfront in residential and industrial areas of Kobe, Ashiya and Nishinomiya.

Liquefied sand erupted and flooded many places in Port and Rokko Islands. The interior of islands subsided 20 - 50 cm by liquefaction settlement. On the other hand, the improved ground sites by vibro-rod compaction or sand compaction piles sustained less deformation and damage than did the adjacent ground.

The lateral spreads caused many concrete caisson quay walls in the region to move 2-5 m into the sea. This significant displacements was possibly caused by inertia of the heavy concrete caissons, large dynamic lateral pressures created by liquefaction of the backfill materials, and the reduction of bearing capacity of replaced sands under the caissons. The east section of the Maya Futo No. 1 pier was designed to resist large earthquakes by installing strong pile foundations or gravity type structures. Apparently, they survived the earthquake with almost no lateral and vertical movement.

In the areas affected by lateral spreads, some piles under the buildings and bridges were found to be broken. Most of the pile supported structures in the interior of Port Island remained in place and the surrounding ground subsided around the structures. The caisson foundations of the Hanshin Bayside Expressway were pushed toward the sea by the force of laterals spreads, causing a girder to drop to the ground.

Many tunnels and underground structures in Kobe survived the earthquake fairly well. The rock tunnels through Rokko Mountains were reportedly less damaged even by the movement of active faults in the mountains. However, some of the shallow underground structures of the subway systems in Kobe were severely damaged. The underground cut-and-cover concrete structure of the Daikai station collapsed by shear failure of the central reinforced concrete columns. The collapsed section was approximately 120 m long.

Several gravity retaining walls along the Hanshin Railway line and the Japan Railway line failed by tilting and sliding of the walls due to the increase in lateral pressure and inertia force. On the other hand, the geogrid reinforced walls in the region survived the earthquake with minor damage of small hairline cracks, even where severe damage in the surrounding residential areas was observed.

The dams in the region performed well with no major damage for protective measures (PRWI, 1995). The only exception was the failure of the Niteko dam in Nishinomiya possibly due to soil liquefaction. River dykes along the Yodo river in Osaka were also severely damaged by the lateral spreading.

Several landslides and rock falls occurred on slopes in the Rokko Mountains. The landslide at Nikawa was the most disastrous one, resulting in thirty four fatalities. The size of the landslide was about 250 m long and 50 m wide.

8.7 Acknowledgments

The author would like to thank the following people who helped and guided his site investigation: Professor Emeritus K. Akai of Kyoto University (presently Georesearch Institute), Mr. Y. Iwasaki of Geo-research Institute, Professor K. Nakagawa of Osaka City University, Professor K. Ishihara of University of Tokyo (presently Tokyo Science University), Professor I. Towhata of University of Tokyo. Professor K. Tokimatsu of Tokyo Institute of Technology, Professor J. Koseki of University of Tokyo and Dr. Chris Bolton of BNFL kindly supplied some of their photos. Many useful information was provided the following research institutes and construction companies : Public Works and Research Institute of the Ministry of Construction, Osaka Geo-Research Institute, Disaster Prevention Research Institute of Kyoto University, Central Research Institute Electrical Power Industry, Railway Technical Research Institute, Kansai Electric Power Co., Shimizu Construction, Kajima Construction, Taisai Construction, Obayashi Construction, Fudo Construction, Konoike Construction, and Toyo Construction.

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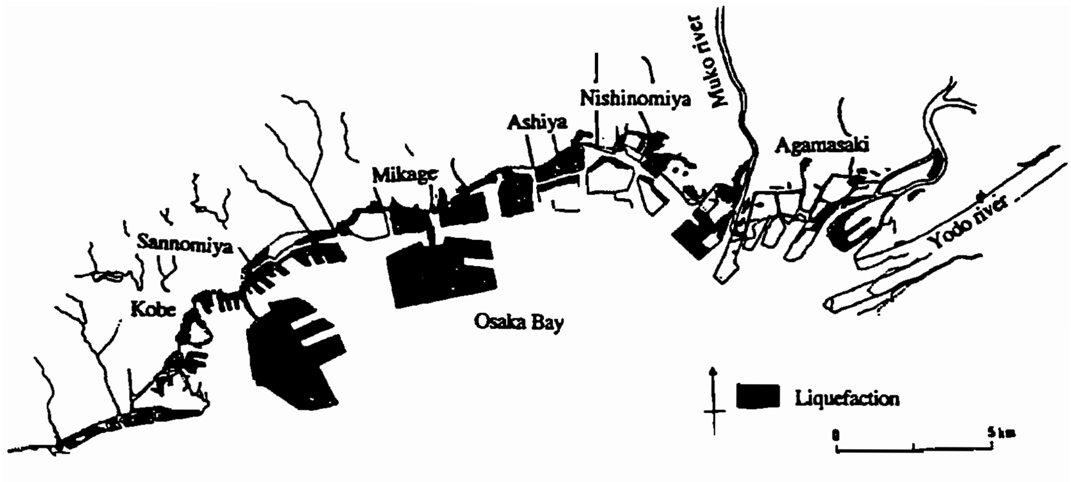


Figure 8.1 Area of liquefaction in the Osaka-Kobe region

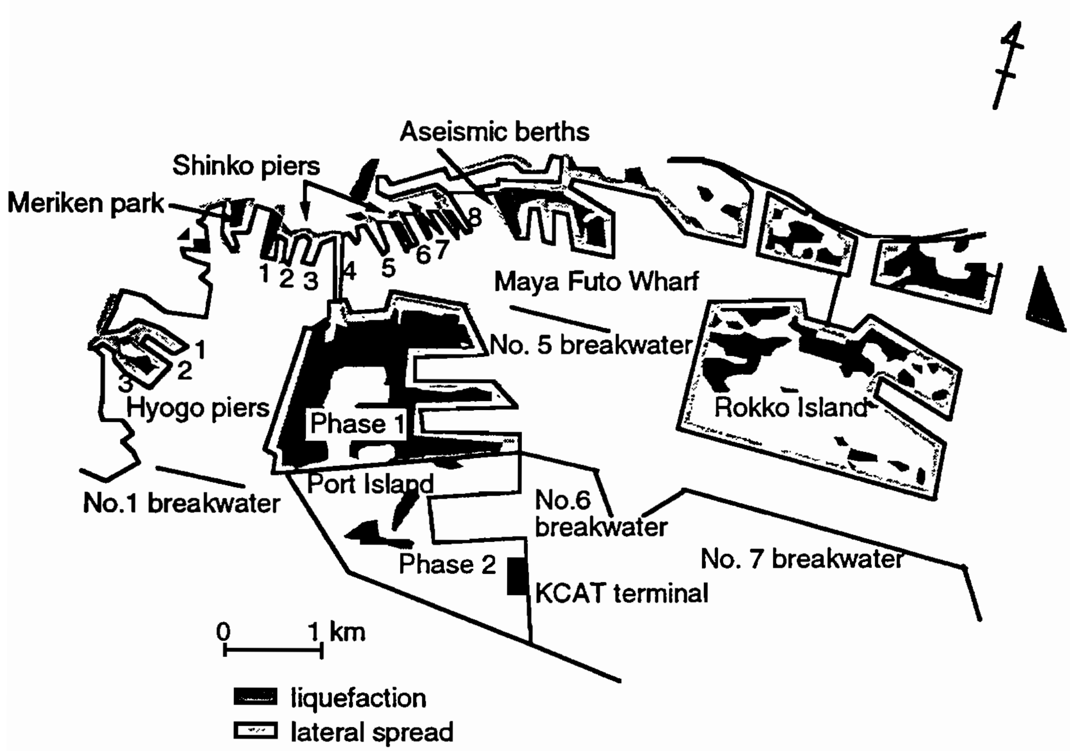


Figure 8.2 A map of Port Kobe

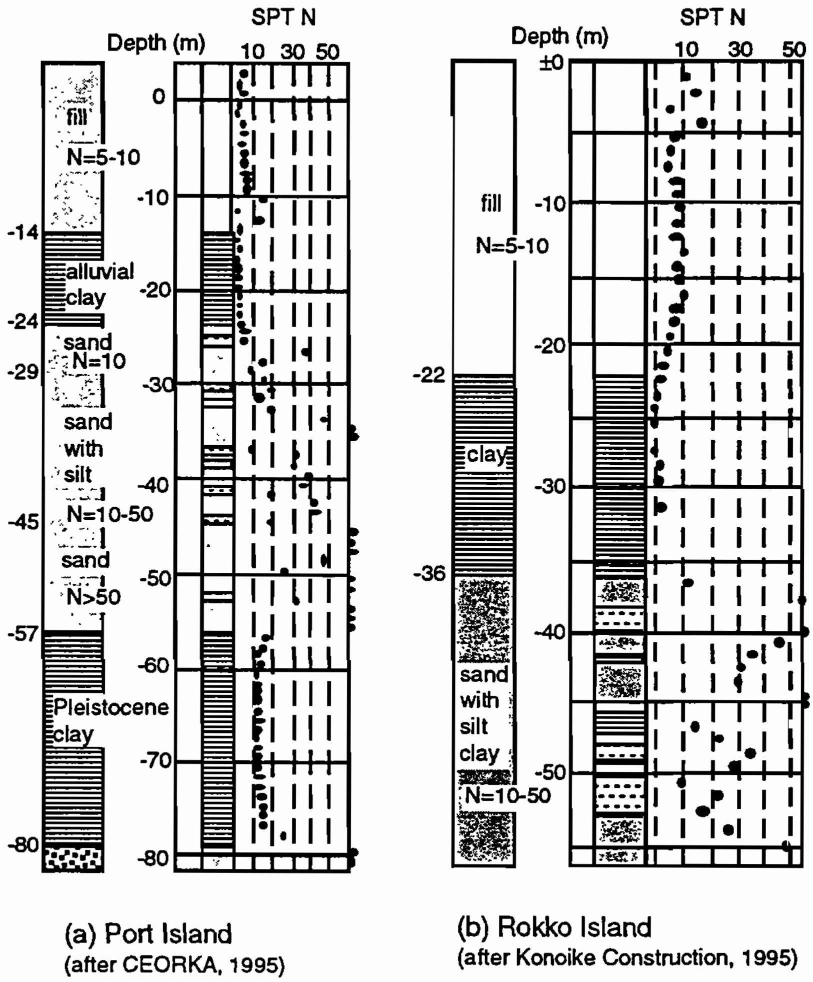


Figure 8.3 Subsurface sections of Port and Rokko Islands

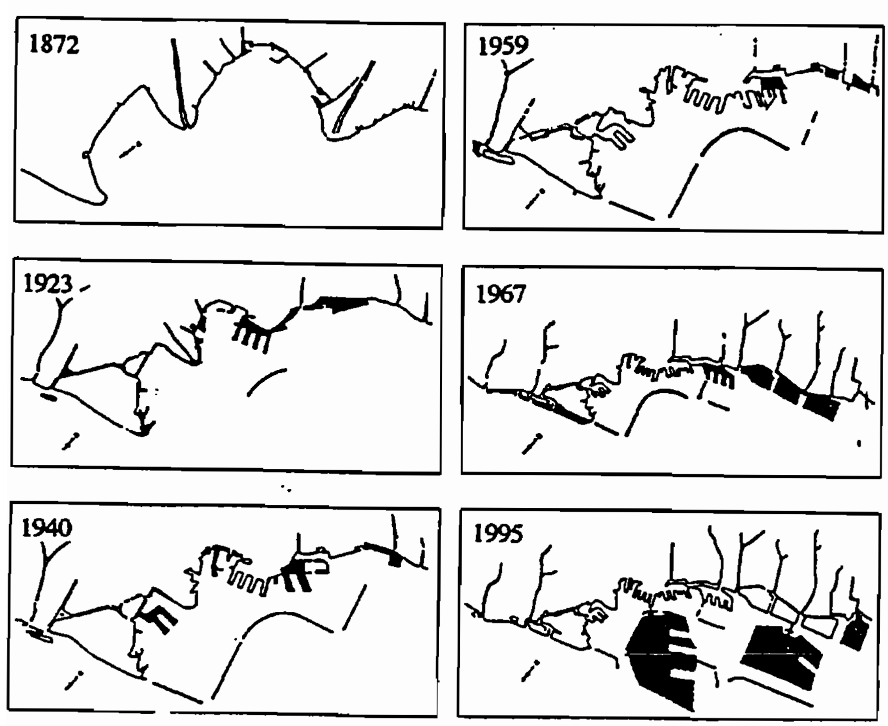


Figure 8.4 Historical development of Port Kobe

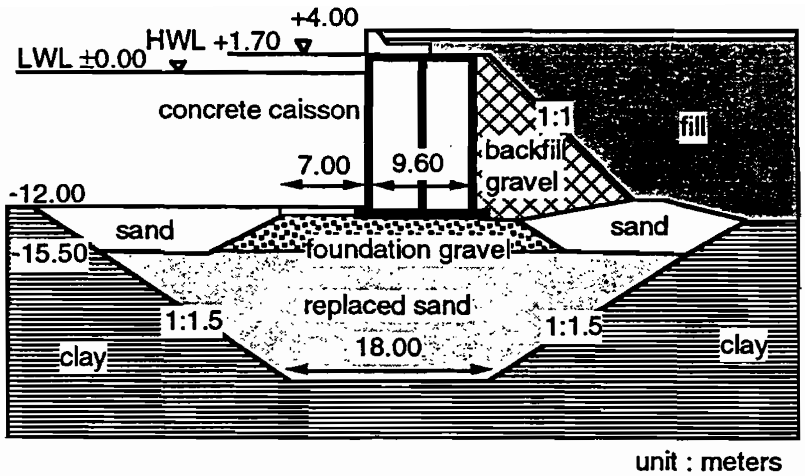


Figure 8.5 Concrete caisson wall at northeast section of Rokko Island

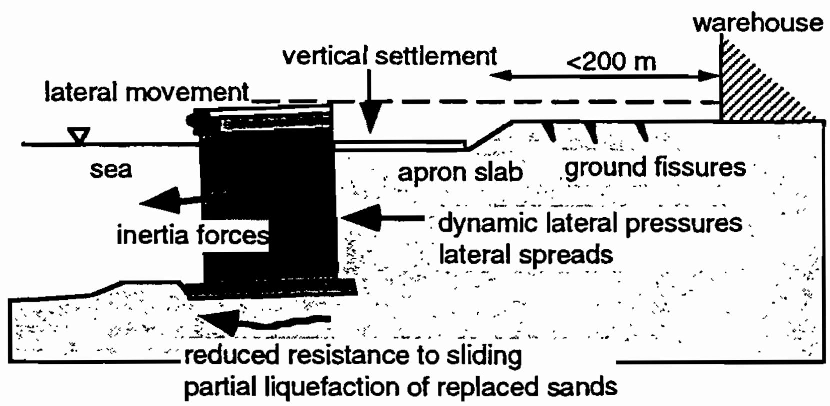


Figure 8.6 Typical damage of caisson walls

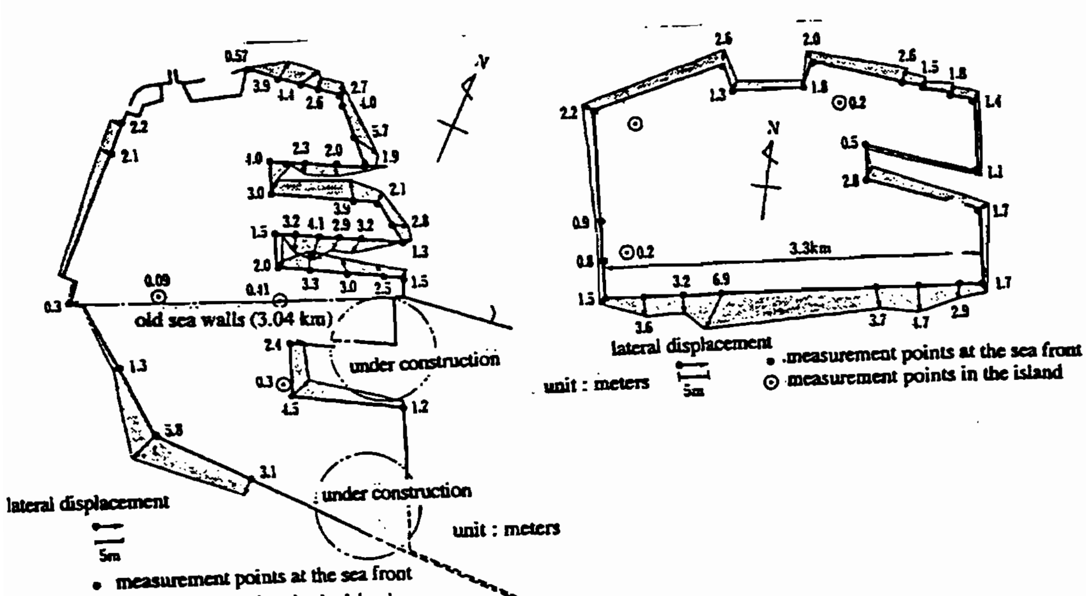


Figure 8.7 Permanent lateral displacements measured on Port and Rokko Islands (Iai, 1995)

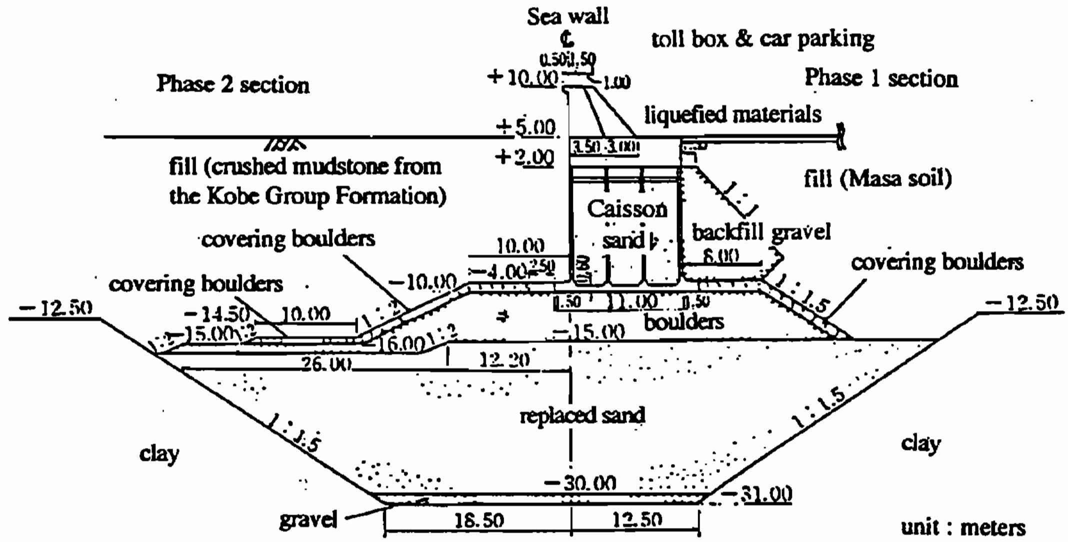


Figure 8.8 Cross-section of the caisson walls between Phase 1 and 2 of Port Island (Konoike, 1996)

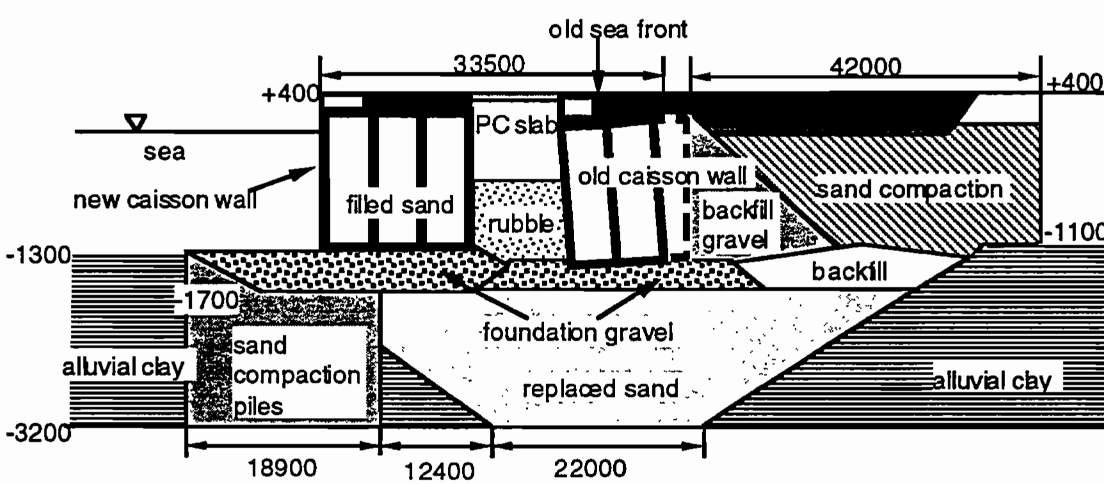


Figure 8.9 Cross-section of a new quay wall on the south section of Rokko Island (Wako, et al. 1996)

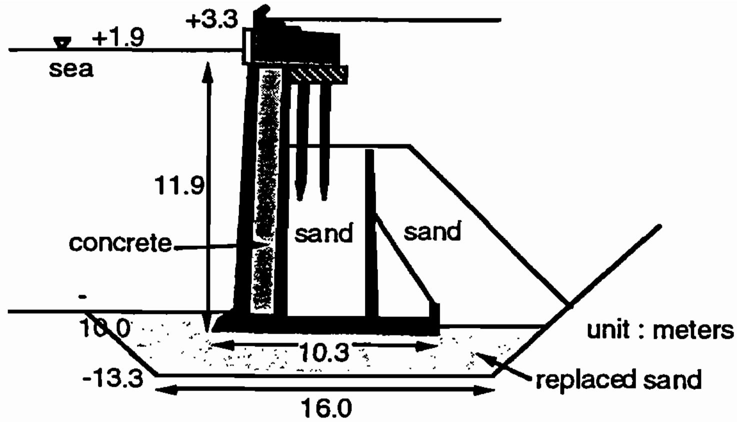
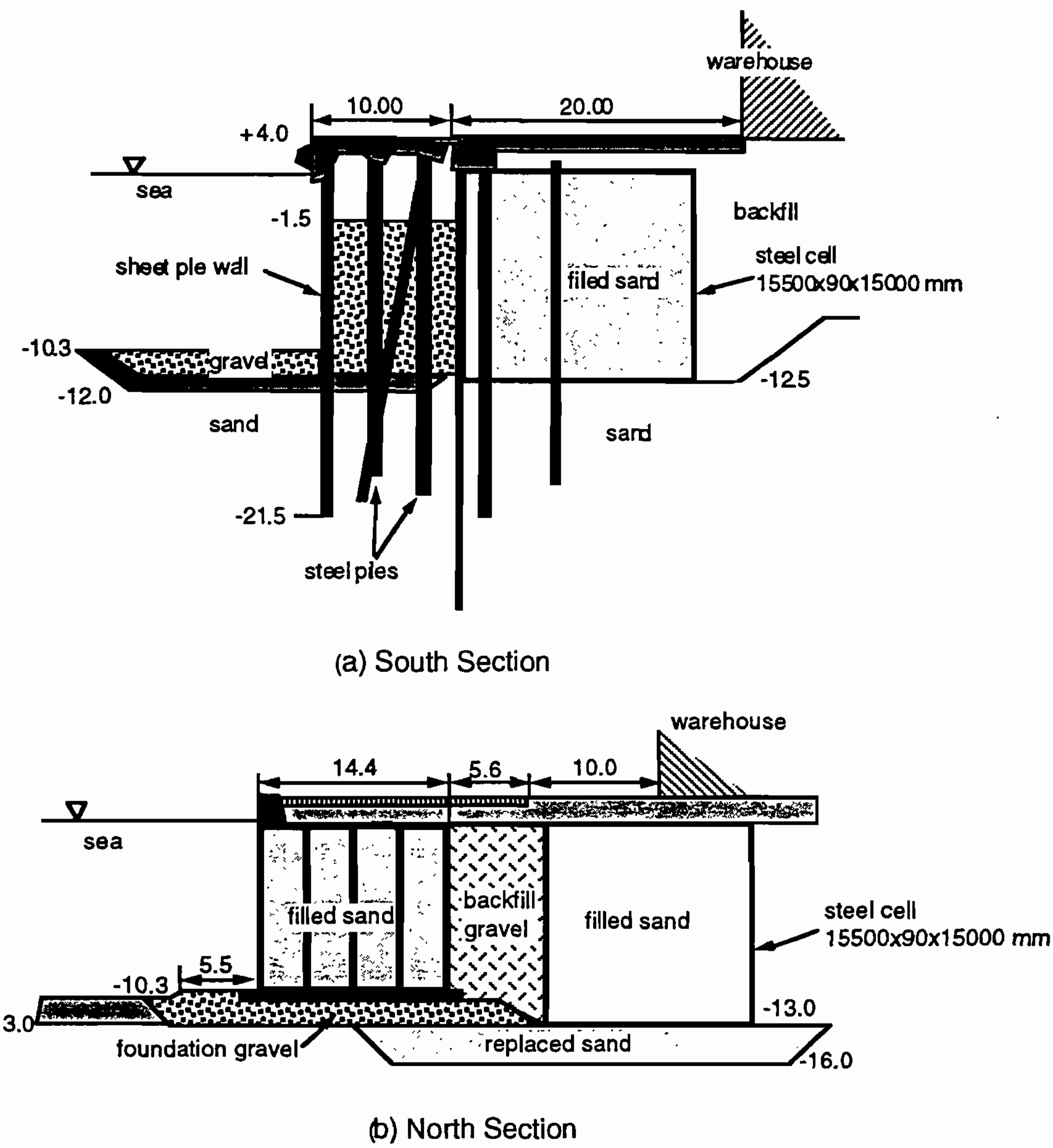


Figure 8.10 Cross-section of Shinko Pier No. 6



(a) South Section

(b) North Section

Figure 8.11 Cross-section of Maya Futo No.1

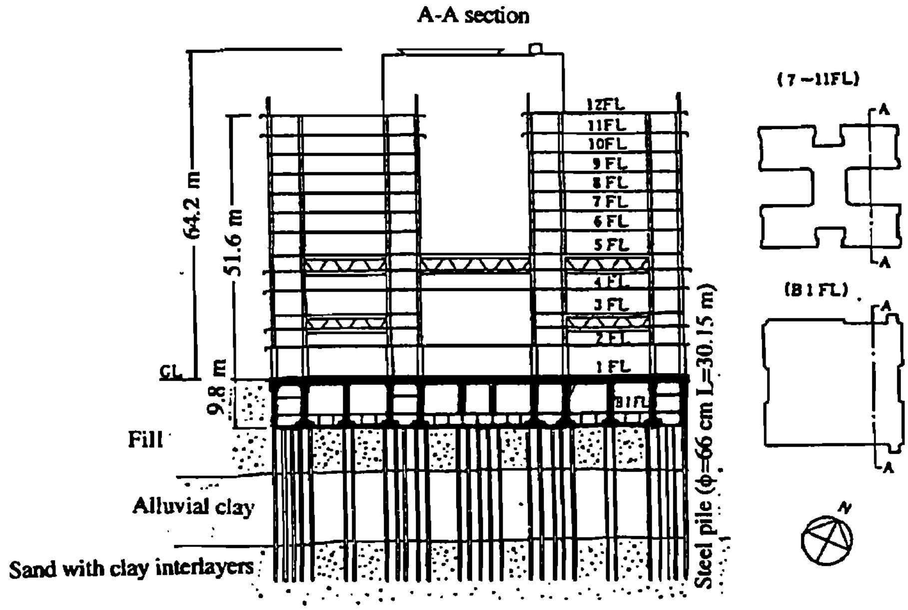


Figure 8.12 Pile foundation of the Kobe Shimin Hospital

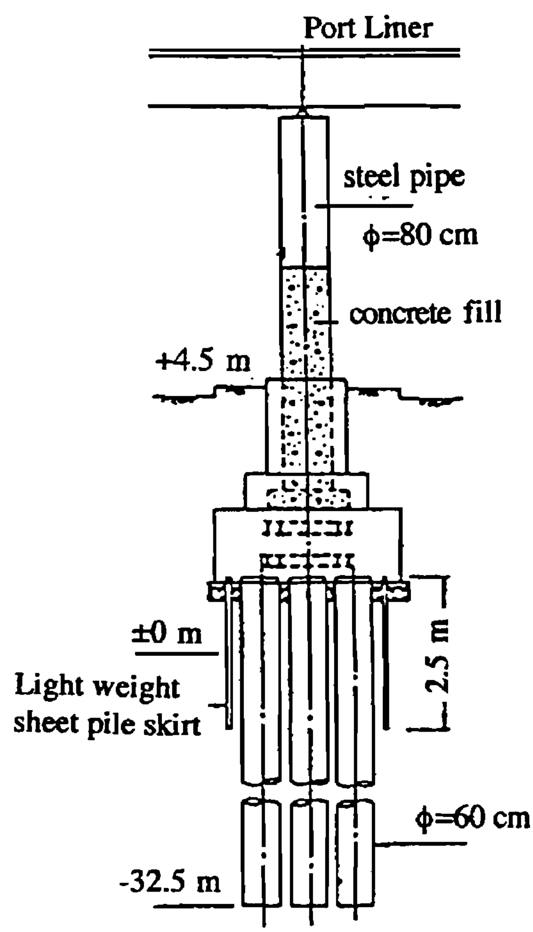


Figure 8.13 Pile foundation of the elevated Portliner Railway on Port Island (Watanabe, 1981)

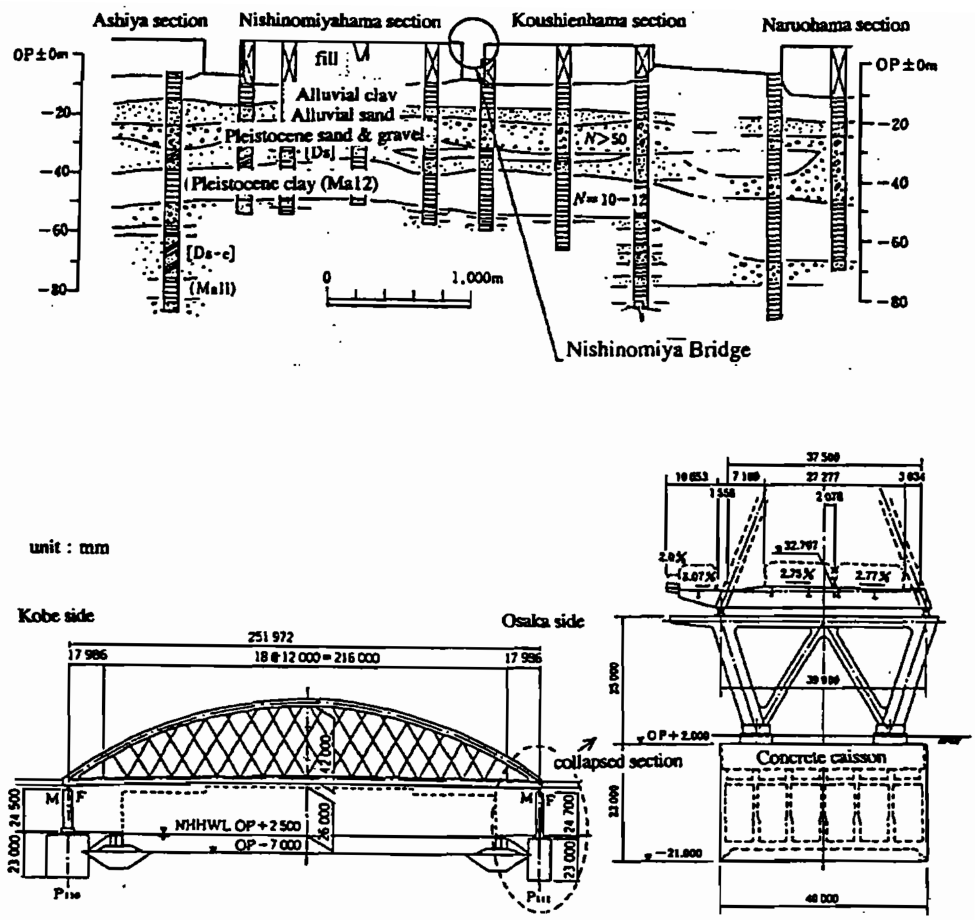


Figure 8.14 Cross-section of the Nishinomiya Bridge of the Hanshin Bayside Expressway (Konoike, 1996)

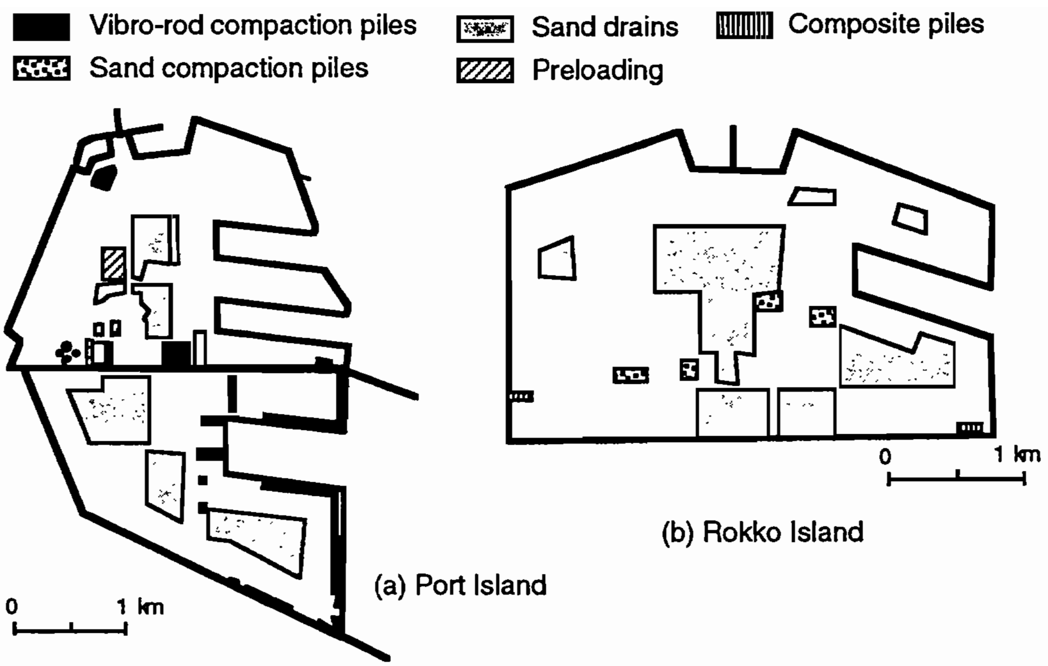


Figure 8.15 Locations of improved ground sites in Port and Rokko Islands
(after Fudo Construction Co. Ltd.(1995) and the Japanese Society of Civil Engineers(1995))

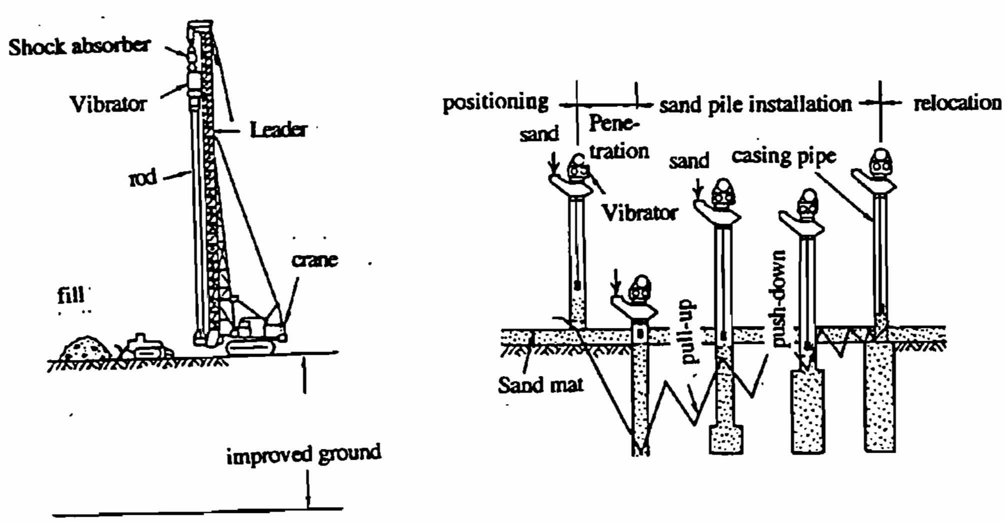


Figure 8.16 Ground improvement by the vibro-rod method

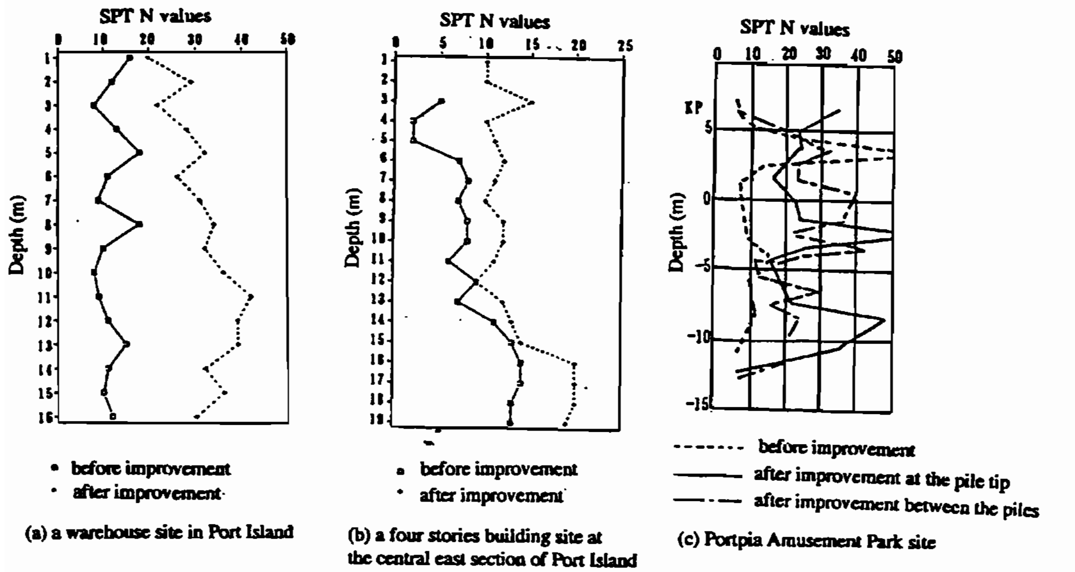


Figure 8.17 SPT N-values before and after the ground improvement (Fudo Construction, 1995)

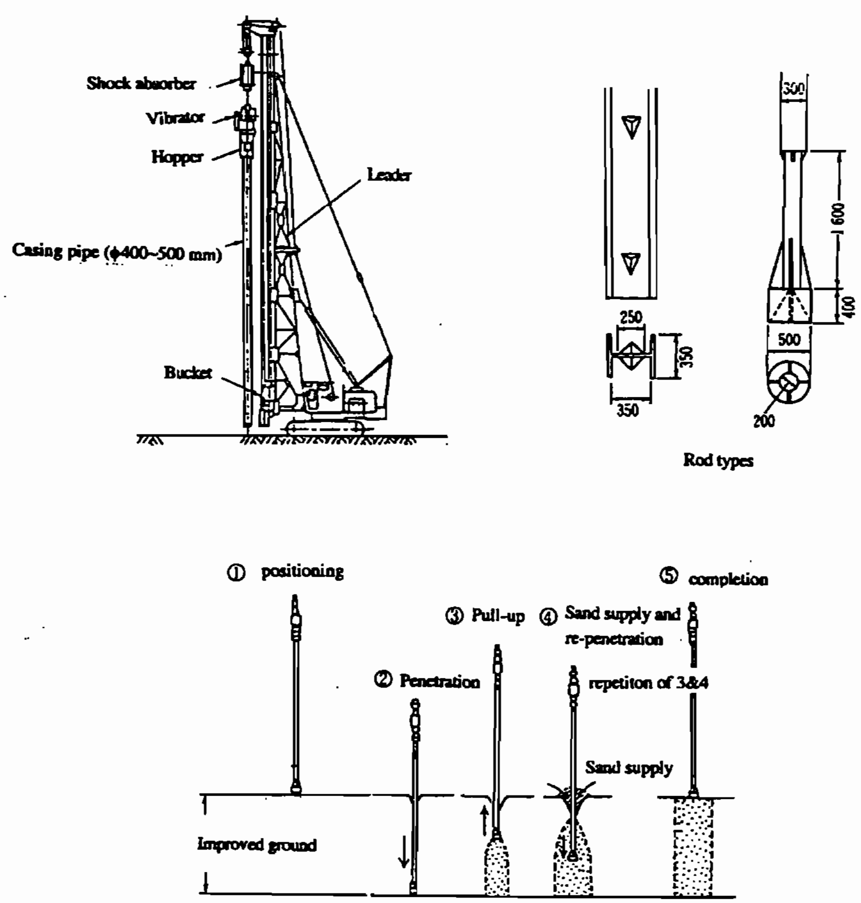


Figure 8.18 Ground improvement by the sand compaction pile method

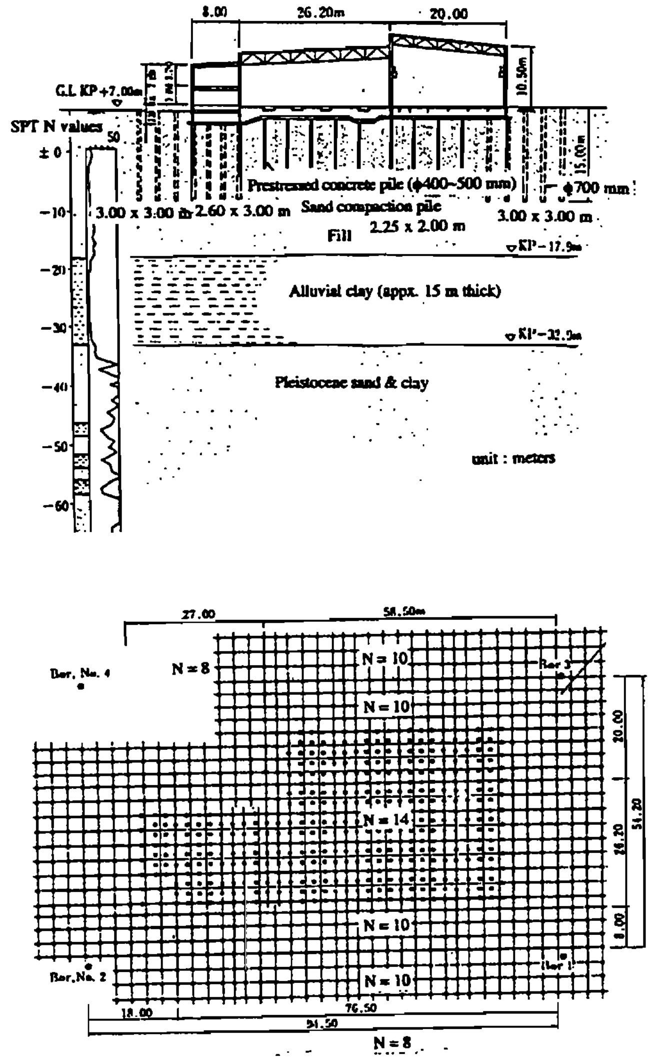


Figure 8.19 Ground improvement at the Rokko Island train inspection site (Nakajima et al, 1995)

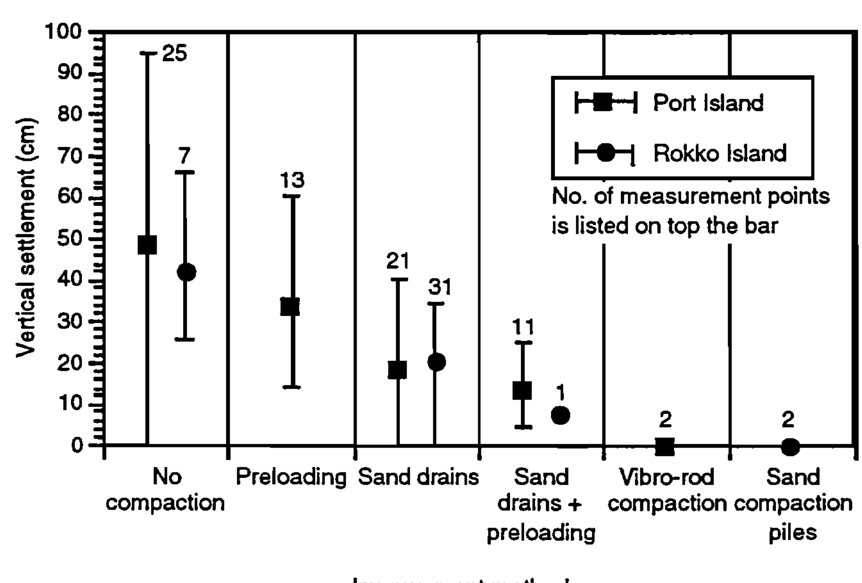


Figure 8.20 Measured vertical settlements at improved ground sites
(after Mitchell, et al., 1995; Ishihara et al., 1995)

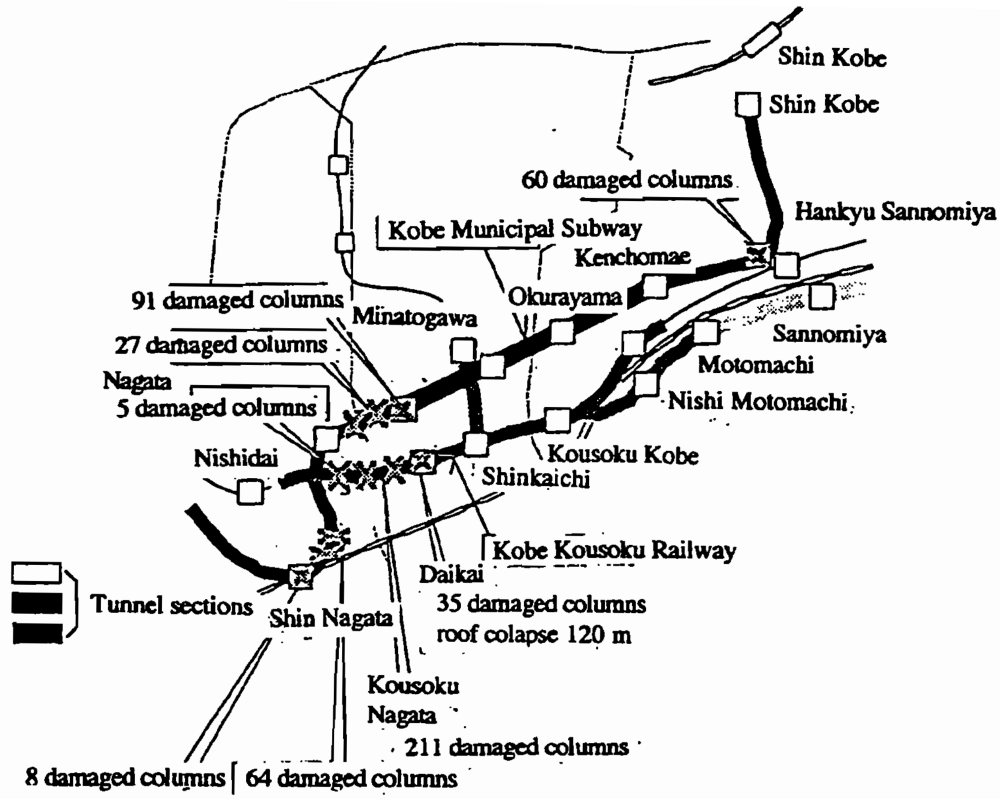


Figure 8.21 Subway systems in Kobe

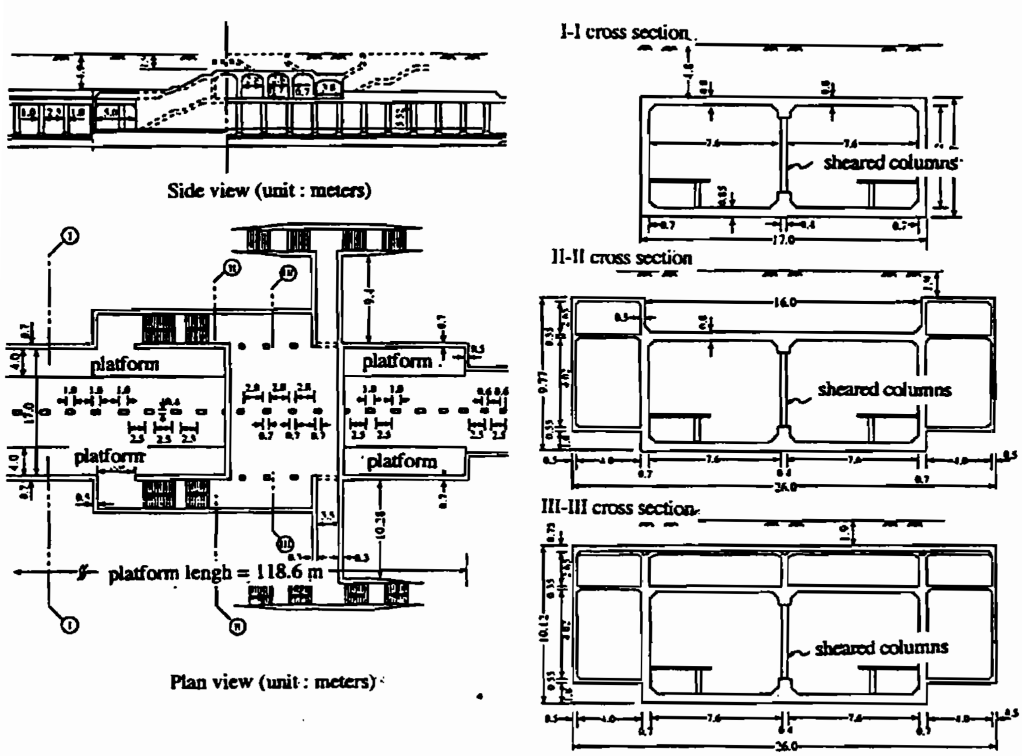


Figure 8.22 Cross-section of the Daikai station

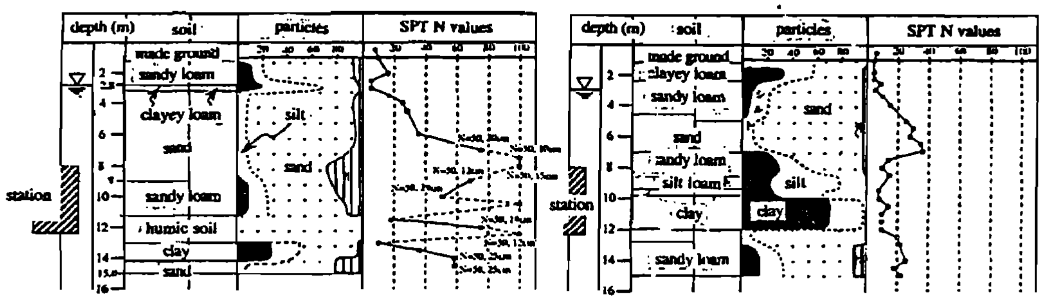


Figure 8.23 Subsurface condition of the Daikai station (JSCE, 1995)

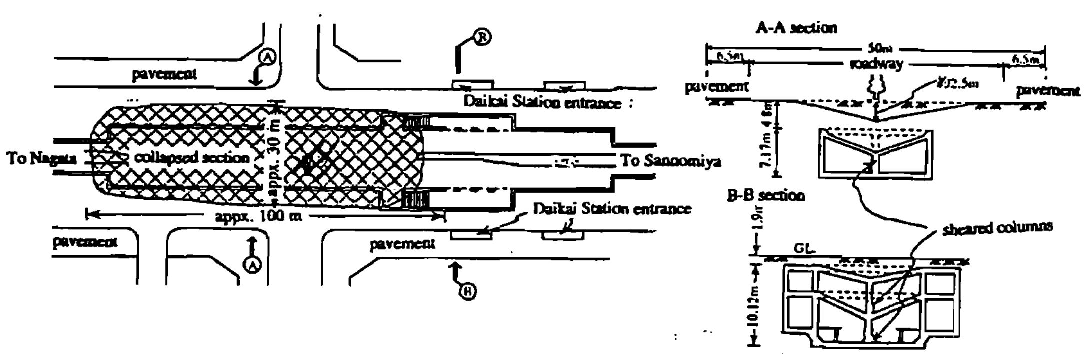


Figure 8.24 Collapsed section of the Daikai station (JSCE, 1995)

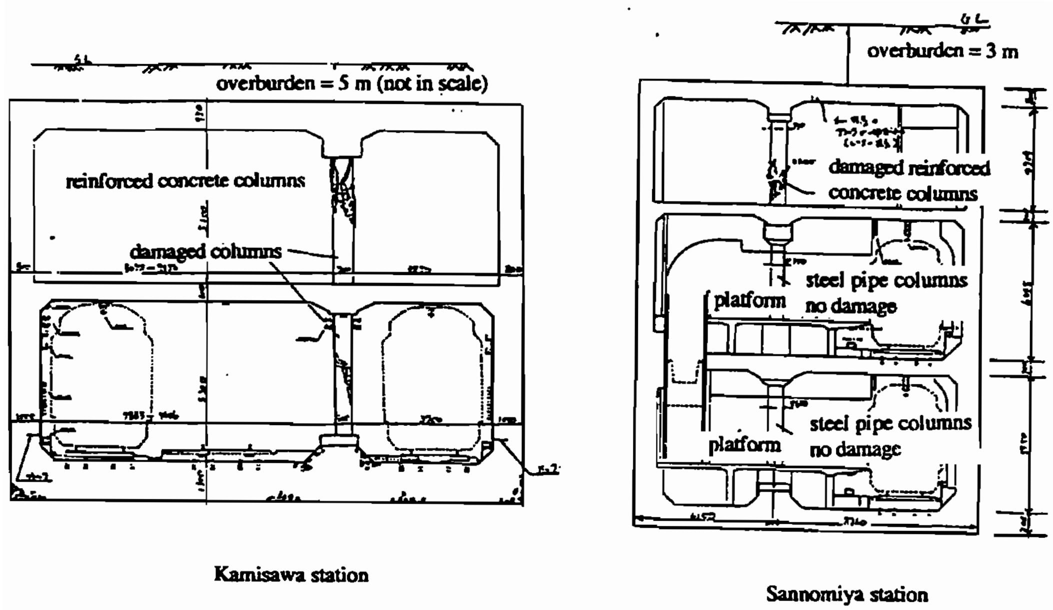


Figure 8.25 Cross-sections of the Kamisawa and Sannomiya stations (PWRI, 1995)

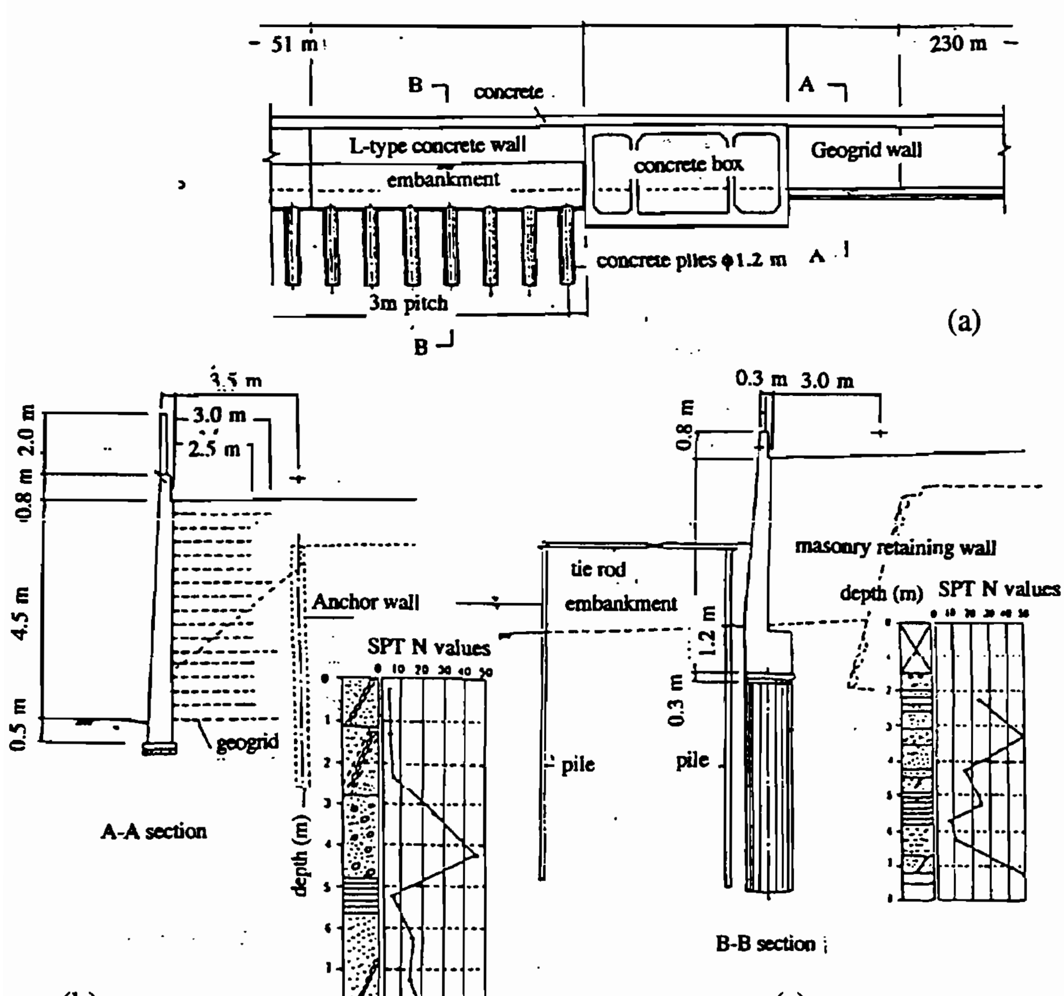


Figure 8.26 Cross-sections of the retaining walls at Tanata (Tatsuoka et al., 1995)

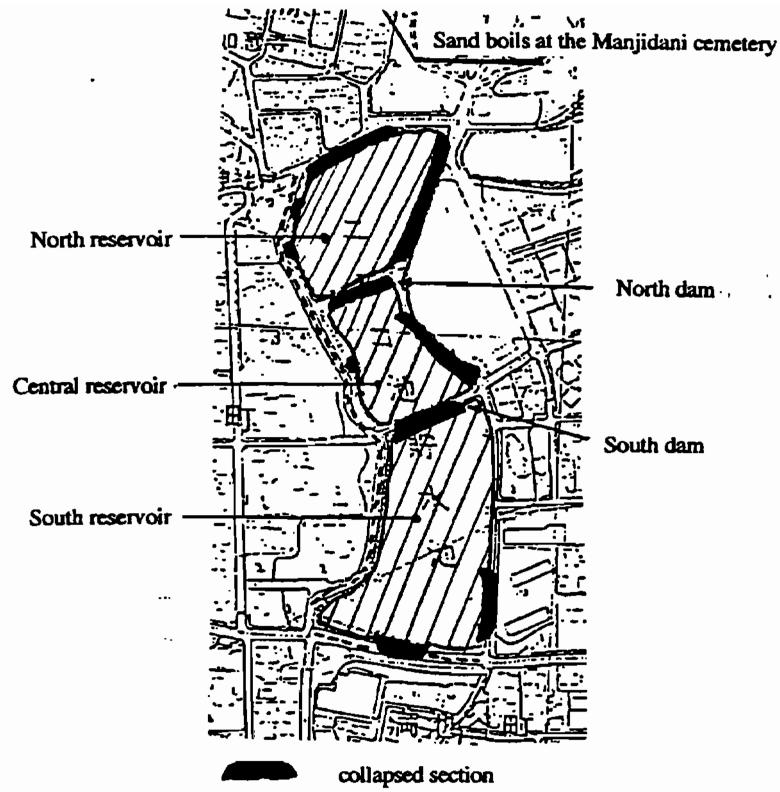


Figure 8.27 Failure of the Niteko dam caused by liquefaction effect

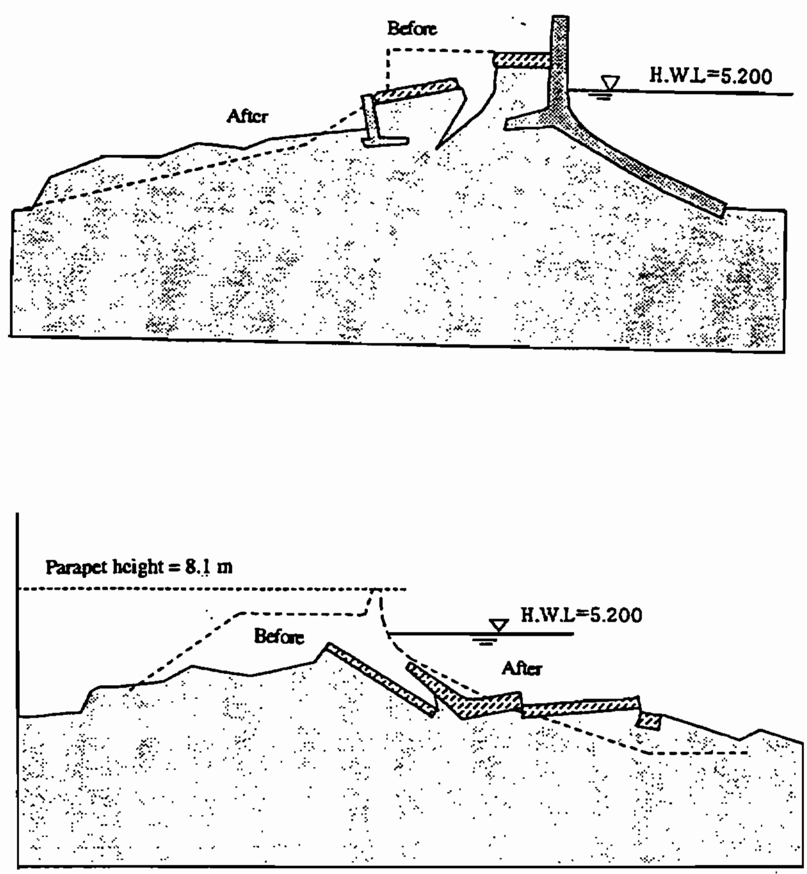


Figure 8.28 Failure of river dykes along the Yodo river (JSCE, 1995)

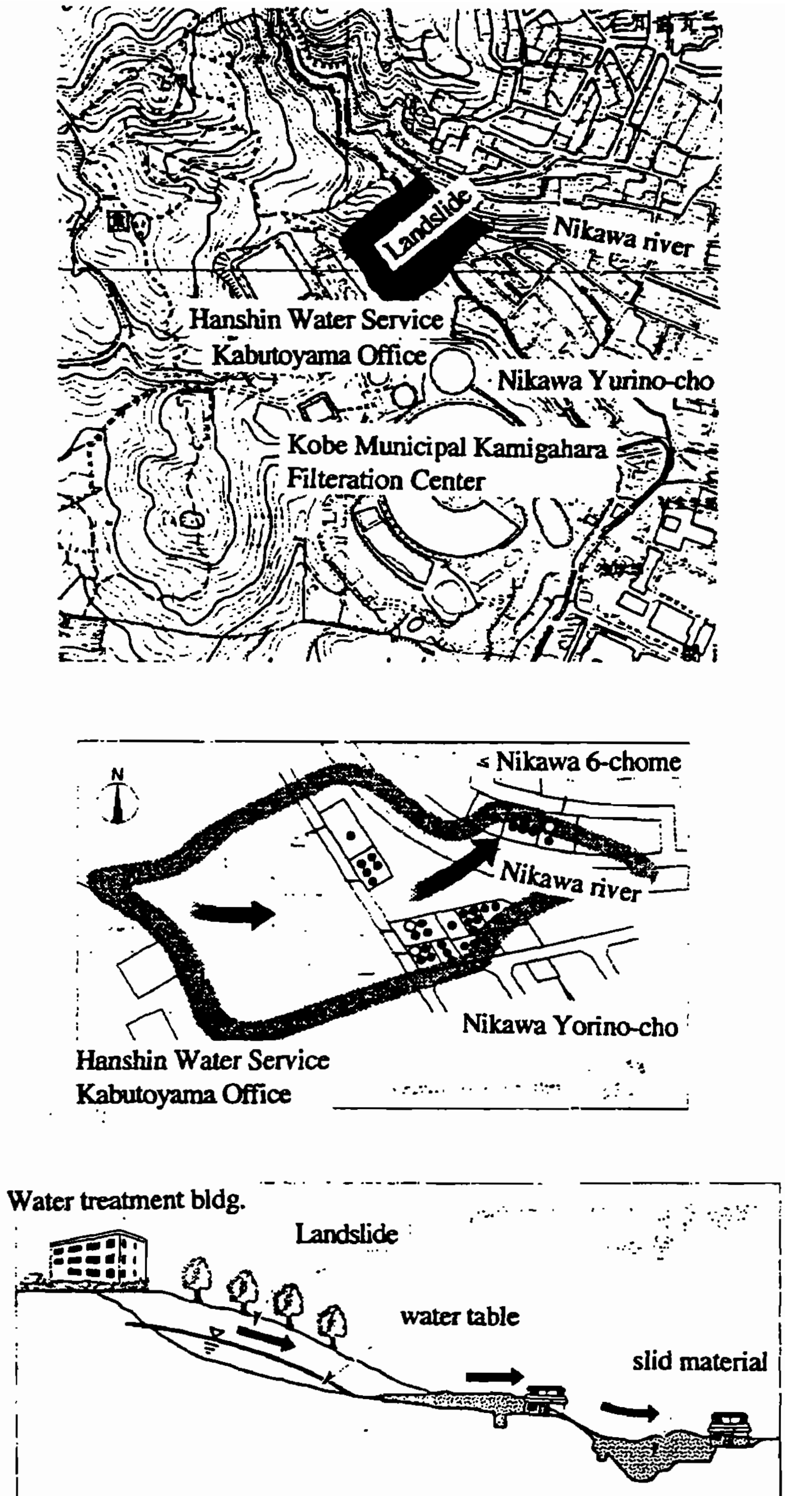


Figure 8.29 The landslide at Nikawa, north of Nishinomiya (after JSCE, 1995)

Plate 8.1: Liquefied soil deposits coming out from a manhole in Port Island

Plate 8.2: Liquefied sand deposits at a parking area at Minatojima, Port Island

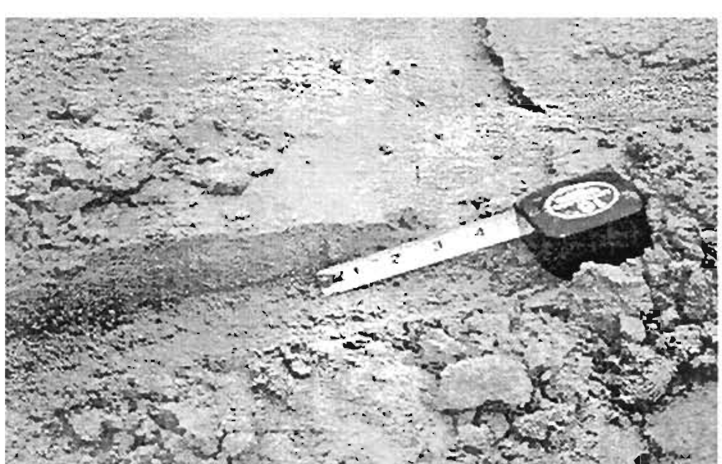


Plate 8.3: A layer of liquefied sands erupted from the ground

Plate 8.4: Liquefaction settlement observed at a pier of the elevated Portliner railway in Port Island

Plate 8.5 : A pile remained in place and the surronding ground liquefied and settled.

Plate 8.6: Liquefaction settlement observed at the KCAT Ferry Terminal, Port Island

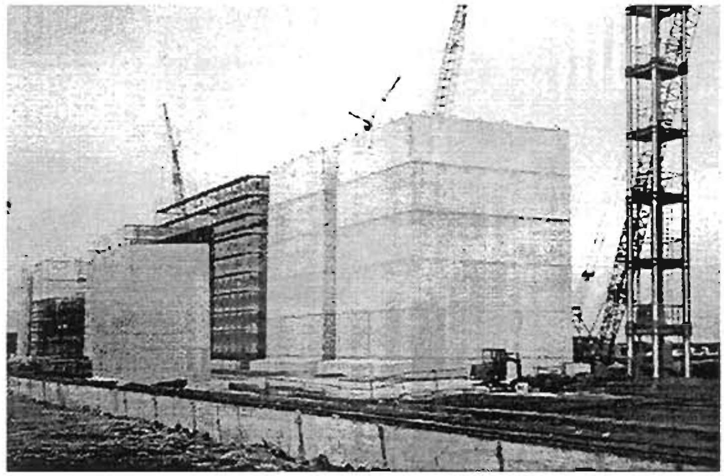


Plate 8.7: Typical concrete caisson walls used for Port Island

Plate 8.8: Lateral movement of concrete caisson walls in Rokko Island

Plate 8.9: Grabens and depressions at the apron slabs behind quay walls

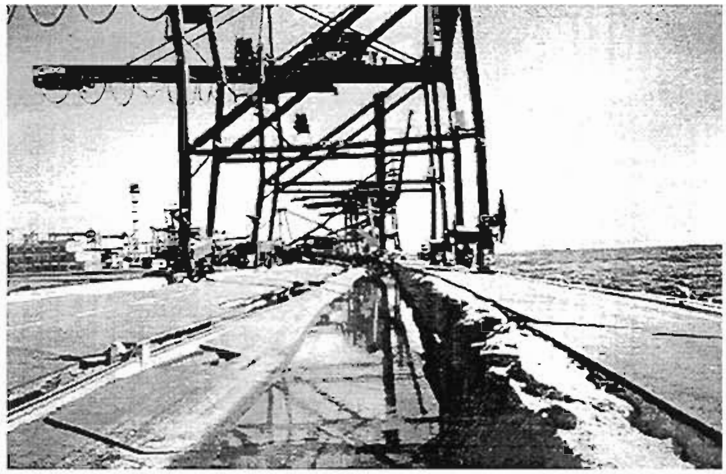


Plate 8.10: Cranes in Rokko Island were pulled apart by the lateral movement of concrete caisson walls

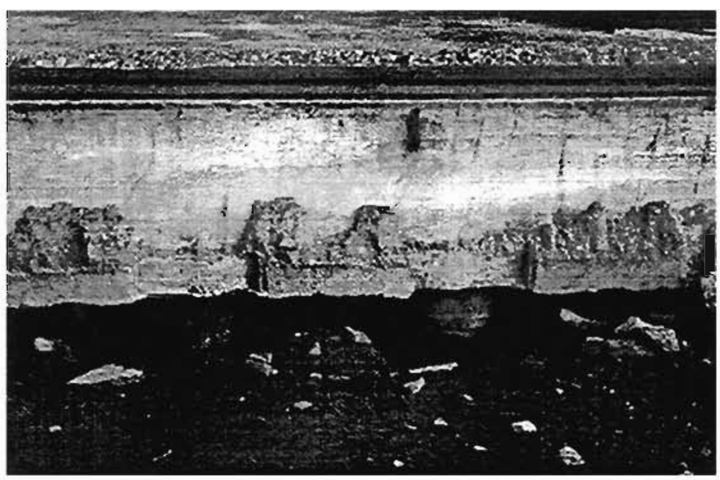


Plate 8.11: Exposed piles under the concrete block foundation of the cranes in Port Island

Plate 8.12: Displaced rail tracks by the movement of cranes observed on Port Island

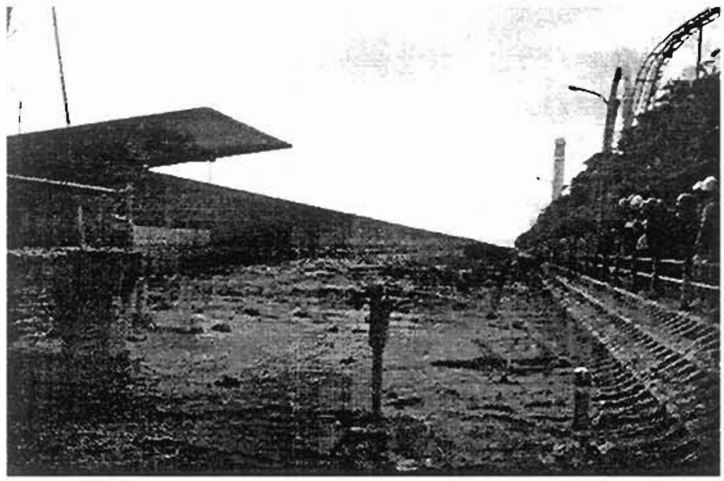


Plate 8.13: Erupted sand deposits found at the boundary between Phase 1 and 2 sections of Port Island

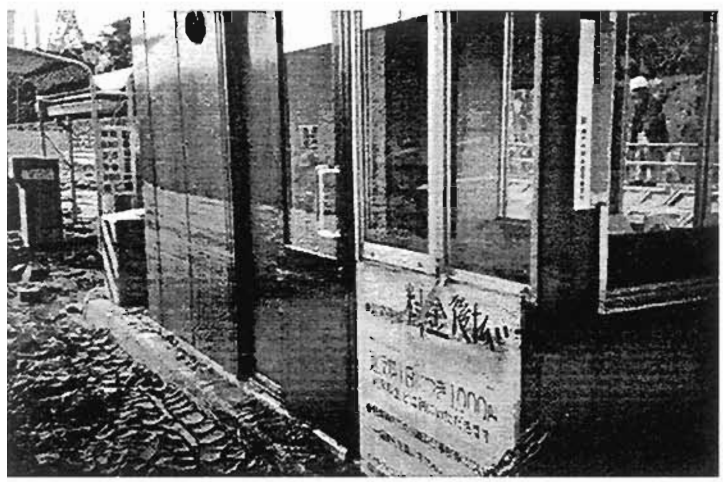


Plate 8.14: A dried water splash marker of liqufied soils found at the toll gate box on Port Island

Plate 8.15: Temporary backfilling works of damaged concrete caisson walls on Port Island

Plate 8.16: Lateral movement of concrete caisson walls at the Shinko Pier in Port Kobe

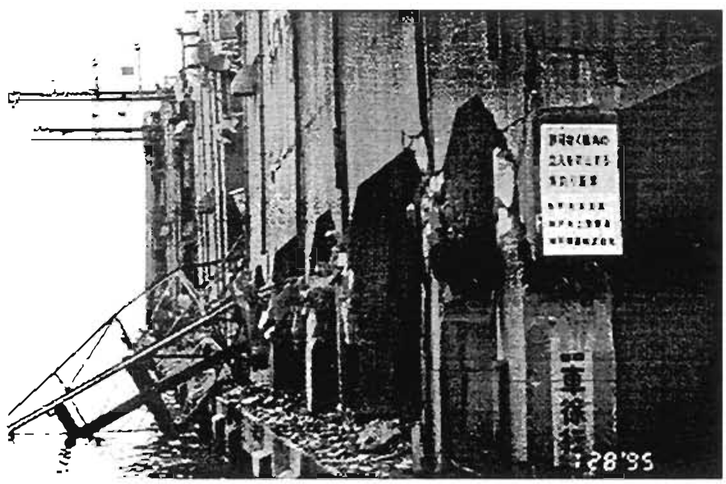


Plate 8.17: Damaged columns of warehouses at the Shinko Pier in Port Kobe

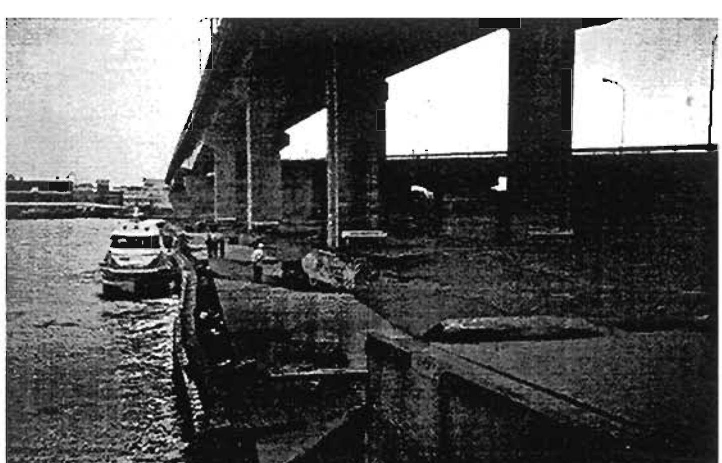


Plate 8.18 Extensive lateral movement of quay walls at the Maya Futo wharves

Plate 8.19: No lateral movement of the Maya Futo No.1 pier

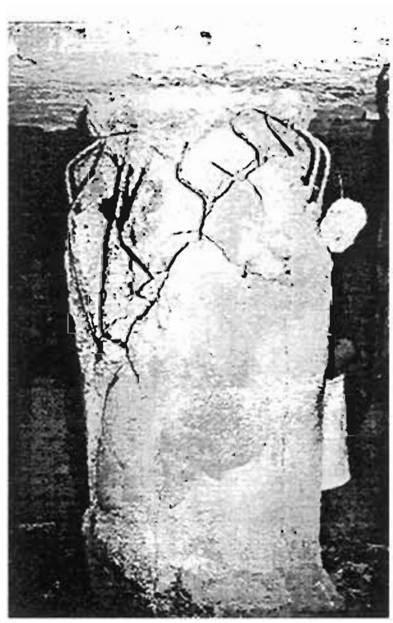


Plate 8.20: Damaged pile foundation due to soil liquefaction (photo by Prof. K. Tokimatsu)

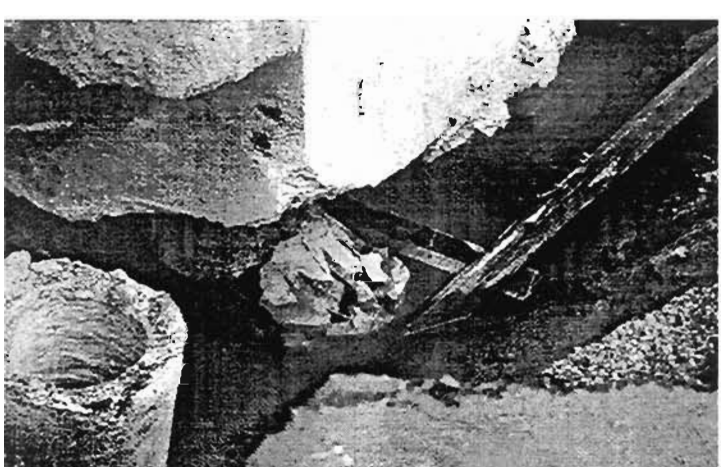


Plate 8.21: Damaged reinforced concrete piles found at Minatojima, Port Island

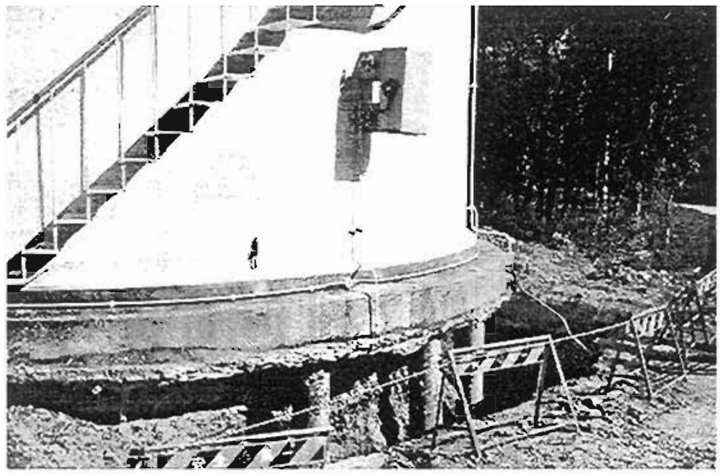


Plate 8.22: Excavated piles under a water tank at the Higashinada Power Station (Photo by C. Bolton)

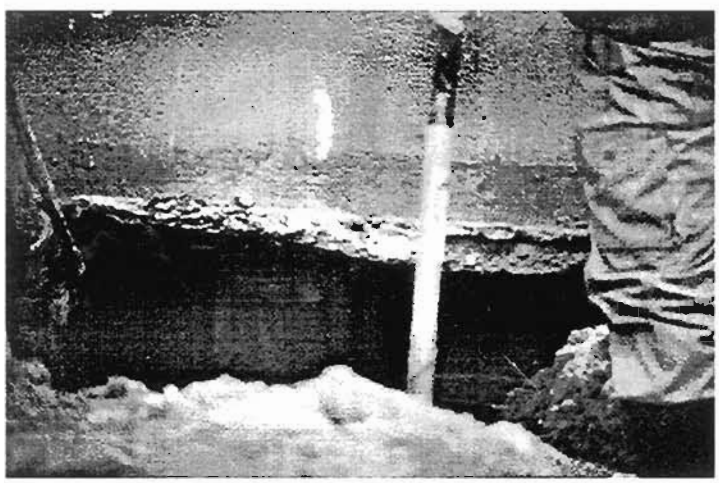


Plate 8.23: Hairline cracks observed in the piles of the water tank (photo by C. Bolton)

Plate 8.24: Repair works of a pile foundation of the elevated Portliner Railway in Kobe

Plate 8.25: Erupted soil deposits observed at the reclaimed land near the Nishinomiya Bridge

Plate 8.26: Lateral spreading of the ground observed from the Nishinomiya Bridge

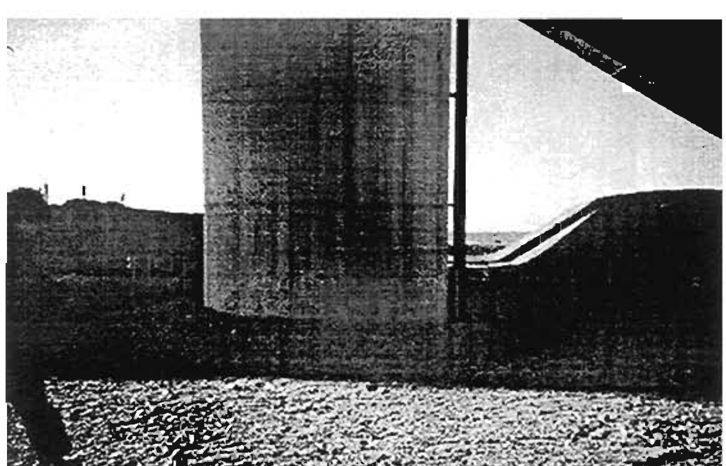


Plate 8.27: Lateral movement of the surface ground pushing the pier of the Nishinomiya Bridge

Plate 8.28: Exposed foundation of the KCAT Ferry Terminal building on Port Island

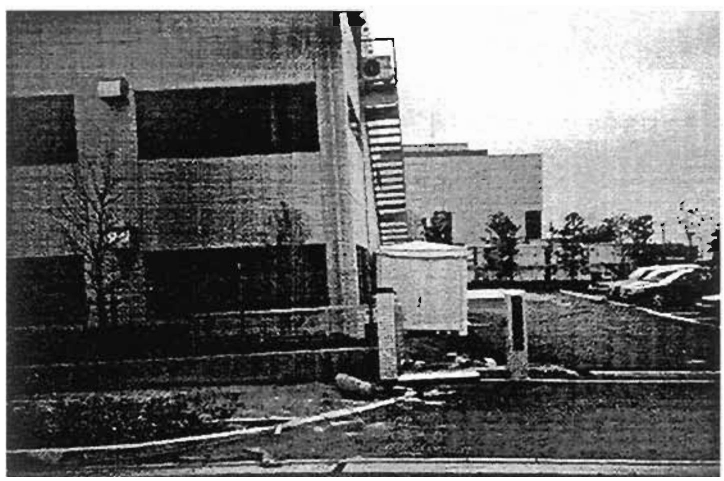


Plate 8.29: Tilting of a three story office building affected by lateral spreads

Plate 8.30: No apparent damage to the foundation of a house even large amount of erupted sands

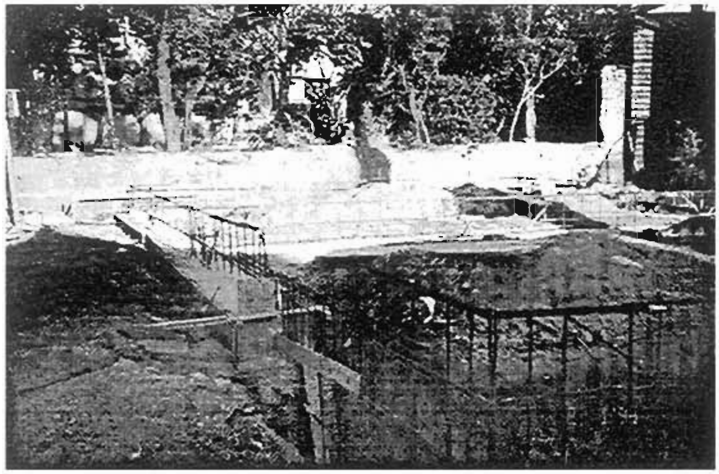


Plate 8.31: A house foundation with reinforcements found in Nishinomiya

Plate 8.32: Less liquefaction settlement found at an improved ground site on Port Island
(The foreground is an untreated site, whereas the background behind the car is a treated site)

Plate 8.33: Collapse of the Daikai Station

Plate 8.34: Tilt of a concrete gravity retaining wall (photo by Prof. J. Koseki)

Plate 8.35: Collapse of a masonry stone retaining wall (photo by Prof. J. Koseki)

Plate 8.36: Good performance of a reinforced geotextile wall at Tanata

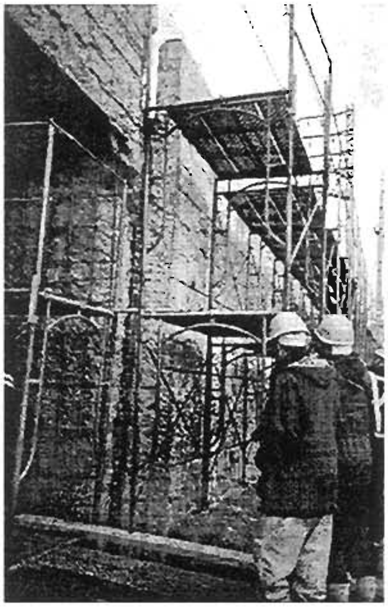


Plate 8.37: Lateral displacements observed at the geotextile reinforced wall in Tanata

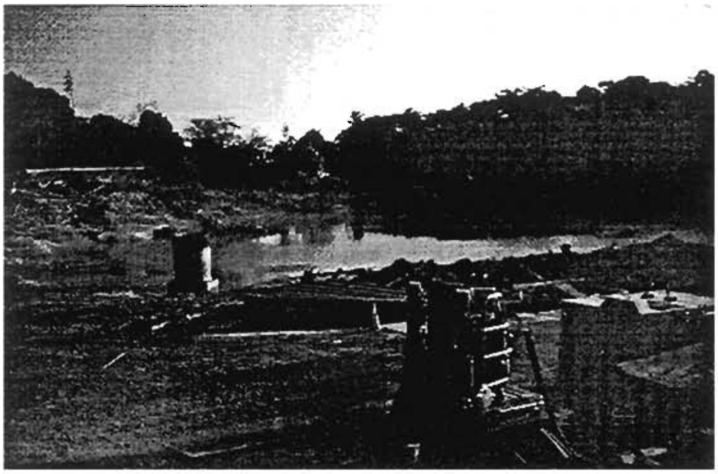


Plate 8.38: Liquefaction failure of the Niteko reservoir

Plate 8.39: Earthquake induced landslide observed at Nikawa, Nishinomiya

Plate 8.40: A falling rock crashing a car in Nishinomiya