Hyogo-Ken Nanbu Earthquake of January 17, 1995:

A Post-Earthquake Reconnaissance of Port Facilities

Committee on Ports and Harbors Lifelines

Edited by Stuart D. Werner and Stephen E. Dickenson
Hyogo-Ken Nanbu Earthquake of January 17, 1995:

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Committee on Ports and Harbors Lifelines of the Technical Council on Lifeline Earthquake Engineering of the American Society of Civil Engineers

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ABSTRACT:

The objective of this committee report, Hyogo-Ken Nanbu Earthquake of January 17, 1995: A Post-Earthquake Reconnaissance of Port Facilities, was to observe and evaluate the seismic performance of ports in the Osaka Bay region of Japan. In addition to the actual observation and evaluation of the seismic performance of the port facilities, this scrutiny included numerous data-gathering meetings with representatives from cognizant port authorities, engineering consulting firms, construction companies, universities and private research organizations in Japan. The investigation was carried out over a 10-day period from February 18-27, 1995 and focused primarily on the seismic performance of the Port of Kobe, the Port of Osaka and, to a lesser degree, the Kansai Airport. This first-hand documentation of perishable data will enhance future engineering and research work at these and other ports around the world.

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CHAPTER 1
INTRODUCTION

1.1 Background

Past experience has shown that port facilities can be susceptible to severe damage from earthquake ground shaking and associated phenomena (e.g., ground deformation, liquefaction, submarine slope failures), and that such damage can result in significant economic losses to the port authority and to industries dependent on marine commerce. In recognition of this, the Ports and Harbors Committee of the ASCE-Technical Council for Lifeline Earthquake Engineering (TCLEE) is developing seismic guidelines for hazard evaluation, design, analysis, and emergency response/recovery for port waterfront, cargo handling/storage, and infrastructure components (Werner et. al., 1995). This Committee is comprised of experienced professionals from port authorities, consulting engineering firms, government, and universities.

An important source of information for these guidelines is the documented performance of ports during past earthquakes. Toward this end, the Committee has been actively compiling data on the performance of ports during historic earthquakes worldwide. It was felt that the Committee's data compilation efforts and overall seismic guidelines document would be greatly enhanced by an in-depth reconnaissance of ports in the region of moderate to strong ground shaking from the Hyogo-Ken Nanbu Earthquake. Furthermore, it was anticipated that the dissemination of this information to the ports community would foster an increased awareness of: (a) the potential vulnerability of ports to moderate to strong ground motions; (b) seismic performance characteristics of quay walls, fills, and other port components, and how this performance may be affected by the strength of the shaking and by design and fill placement procedures; (c) the potential risks to post-earthquake operations and repair/reconstruction efforts at ports due to damage to supporting utility and transportation lifelines; and (d) the potential effects of port damage on local, regional and international commerce and economies.

1.2 Reconnaissance Objective and Scope

The objective of this reconnaissance following the Hyogo-Ken Nanbu Earthquake was to observe and evaluate the seismic performance of ports in the Osaka Bay region of Japan (Fig. 1-1). The reconnaissance focused on first-hand documentation of perishable data for enhancing future engineering and research work at these and other ports in the United States and Japan, as well as the ASCE-TCLEE Ports and Harbors Committee's seismic guidelines.

This reconnaissance was carried out by Stuart D. Werner of Dames & Moore and Stephen E. Dickenson of Oregon State University, who are Chairman and Vice-Chairman, respectively, of the Ports and Harbors Committee. In addition to the actual observation and evaluation of the seismic performance of the port facilities, the reconnaissance included numerous data-gathering meetings with representatives from cognizant port authorities, engineering consulting firms, construction companies, universities and private research organizations in Japan.
The information gathered during our visit continues to be augmented with additional data provided by our colleagues in Japan and the United States who have also visited the ports after the earthquake, and also with data gleaned from the earthquake engineering literature from these countries. This information is being synthesized into reports that should contribute to an increased awareness in the ports community of the potential seismic risks to port facilities during moderate to large earthquakes. Relevant information and selected slides from this reconnaissance will be made available not only to the Ports and Harbors Committee, but also to the ports community with facilities located in regions of the United States that have a potential for damaging earthquakes.

The reconnaissance was carried out over a 10-day time period that extended from February 18-27, 1995. The investigations focused primarily on the seismic performance of the Port of Kobe and the Port of Osaka (Fig. 1-2). The reconnaissance was initiated with orientation and data-compilation meetings with persons in Tokyo, Kyoto and Osaka who have extensive experience with the ports. These meetings provided valuable information on seismic design procedures, construction practices, soil conditions, and soil improvement techniques employed at various portions of the port facilities. The data and guidance provided at this early stage of the reconnaissance enhanced subsequent field investigation efforts, which consisted of both walking and boat tours of the ports. Several of the tours were guided and arranged by port personnel, and numerous unaccompanied inspections were also conducted. In addition, we briefly met with personnel from the Kansai Airport, a major airport recently constructed on a 509 ha man-made island in Osaka Bay west of Osaka, in order to compile readily available information on the seismic performance of its island fills and quay wall components.

1.3 **Report Organization**

The remainder of this report is organized into five main chapters. Chapter 2 provides an overview of the seismological aspects of the Hyogo-Ken Nanbu Earthquake, and Chapters 3 through 5 describe our observations pertaining to the seismic performance of the Port of Kobe, the Port of Osaka, and the Kansai Airport. Chapter 6 contains concluding comments from the reconnaissance.
FIGURE 1-1
MAP OF JAPAN
(JTNO, 1986)
FIGURE 1-2
OSAKA BAY REGION
(EERI, 1995b)
CHAPTER 2
EARTHQUAKE OVERVIEW

2.1 Fault Rupture

The Hyogo-Ken Nanbu Earthquake ($M_w = 6.9$) occurred at 5:46 am (local time) on January 17, 1995. The rupture was initiated at a depth of approximately 10 km below the northeastern tip of Awaji Island (Figure 2-1a). In plan, this location is roughly 20 km southwest of downtown Kobe. Ground surface rupture has been mapped along the Nojima Fault located in the northwestern portion of Awaji Island. It is inferred from aftershock patterns (Figure 2-1b) that the strike-slip rupture propagated bi-laterally from the hypocenter, with the rupture to the east extending directly beneath the city of Kobe, a densely developed city with a population of approximately 1.5 million people. To the east of Awaji Island, a complex system of faults has been mapped on the alluvial plain between the base of the Rokko Mountains and the northern margins of Osaka Bay (RGAFF, 1991). The general geometry of these features is shown in Figure 2-2. In this area, the rupture is surmised by many to have occurred along the Ashiya Fault. To date, no evidence of surface rupture has been reported in the Kobe area. The total rupture length has been estimated from aftershock patterns to have been about 30-to-50 km.

The bi-lateral mode of fault rupture experienced during this event is very similar to the rupture mechanism exhibited during the 1989 Loma Prieta Earthquake in the San Francisco Bay region. This type of faulting results in a duration of strong shaking that is about half of what would be considered characteristic for earthquakes of the same magnitude. The effects of this reduced duration were clearly overshadowed by the propagation of the fault rupture into the city of Kobe. This path of rupture propagation appears to have resulted in a directivity of seismic energy into the urban area of Kobe and into areas to the northeast, thereby enhancing the intensity of the ground motions adjacent to and northeast of the fault rupture (Somerville, 1995).

2.2 Ground Shaking

The combination of the directivity effects and the close proximity of the city of Kobe to the fault rupture resulted in extremely strong shaking, with peak horizontal ground accelerations recorded at strong motion accelerometer stations in Kobe that were often on the order of 500 to over 800 cm/sec² (Fig. 2-3). Peak vertical accelerations were generally about two-thirds of the peak horizontal accelerations. The duration of the strong shaking segment of these recorded motions was up to about 10 sec (Fig. 2-4). It is noted that the soil conditions at most of the accelerometer stations in Kobe consisted of alluvial deposits (predominantly older alluvial sediments in the downtown area, with a transition toward the bay to fill underlain by young marine deposits and by deep alluvium).

Along the margins of Osaka Bay and on the artificial islands of the Port of Kobe deep soil motions were modified by overlying strata of soft to medium stiff marine clay and loose sandy fill soils which have been placed since the late 1800s (Fig. 2-5). Strong motion records measured at the Port
of Kobe (including records from a down-hole array on a site on Port Island where the surface fill layer liquefied) are described in Section 3.2.

Figure 2-6 shows a return period vs. peak acceleration (PGA) relationship for the Kobe area that was provided to us during our visit to Japan. This figure shows estimated return periods for PGAs of 500-800 cm/sec² that range from about 500 years to over 1000 years. These values are corroborated by the occurrence of an estimated Magnitude 7.25 earthquake in Kobe in 1596 (Tsukuda, 1987). It is interesting to note that an independent study made in Japan over 40 years ago estimated that peak accelerations on firm ground of approximately 190 cm/sec², 300 cm/sec² and 420 cm/sec² are associated with return periods of 75, 100 and 200 years, respectively, in the Kobe region (Kawasumi, 1951). Maps of the accelerations and return periods produced by Kawasumi are shown in Figure 2-7 and, for the Kobe area, agree reasonably well with the data in Figure 2-6. The return periods for these acceleration levels are comparable to those estimated for similar levels of ground accelerations in highly seismic regions in California.

Figure 2-8 compares peak horizontal accelerations and velocities recorded at soil sites in Kobe to those predicted for a strike-slip earthquake using empirical attenuation relationships for soil sites that are based mainly on California data (Somerville, 1995). This figure shows that the peak accelerations recorded in Kobe are generally comparable to those predicted by the empirical relationships, whereas the peak velocities recorded in Kobe tend to be somewhat larger. It is noted that the peak near-field velocities recorded in Kobe are comparable to the largest velocities recorded in Northern Los Angeles during the 1994 Northridge Earthquake (EERI, 1995a).

2.3 Earthquake Effects

The earthquake effects on the City of Kobe and in the immediately adjacent areas were often devastating. Over 5,300 people in the area were killed, nearly 27,000 people were injured, and an estimated 300,000 people were homeless after the earthquake. This was due primarily due to collapses of older houses, built of post and beam construction techniques, that had only minimal lateral force resistance and supported roofs covered with heavy clay tiles. Older engineered building structures also suffered major damage that included first-story and mid-story collapses, leaning, and severe shear and flexural damage. Major fires occurred in several areas of Kobe, and many highway and railroad bridges collapsed or were severely damaged (Fig. 2-9). Water, wastewater, and natural gas systems and components in the area were also severely damaged. The Kobe Port suffered extensive damage due to liquefaction of the uncompacted fills throughout the port, and due to quay walls that were inadequate to resist the increased lateral pressure that resulted from the associated pore water pressure buildups. Earthquake-induced losses in Kobe have been estimated to be as high as $200 billion, which is about an order-of-magnitude larger than the estimated losses in the Los Angeles area due to the 1994 Northridge Earthquake. Further extensive description of the effects of the Hyogo-Ken Nanbu Earthquake on the buildings and lifelines in the area is contained in several reports on the earthquake that have been produced in Japan and the United States in recent months (e.g., DPRI, 1995; INCEDE, 1995; EERI, 1995b; SEAONC, 1995).
It should be noted that although this reconnaissance focused on the ports at Kobe and Osaka, several other ports in the region were affected by the Hyogo-Ken Nanbu Earthquake. The strong ground motions experienced along most of the margins of Osaka Bay and the northeastern portion of the Harima Sea (northwest of Awaji Island) exceeded 0.2 g (Fig. 2-3). The intensity of these ground motions corresponds to levels of shaking that have resulted in considerable damage to port facilities worldwide (Werner and Hung, 1982). An investigation of the seismic performance of other ports in the Osaka Bay area (e.g., Amagasaki-Nisinomiya-Ashiya, Sakai-Senboku, Hannan) and the waterfront regions northwest of Awaji Island (e.g., Takasago, Hirohata, and Abashi) is currently underway as a subsequent phase of this reconnaissance.
a) Main Shock on January 17, 1995 at 5:45 am (JTL, 1995)

b) Aftershocks (1/19/95 to 1/27/95) (DPRI, 1995a)

FIGURE 2-1
HYOGO-KEN NANBU EARTHQUAKE OF JANUARY 17, 1995
FIGURE 2-2
REGIONAL TECTONIC MAP (Sugiyama, 1994)
Peak Acceleration Measurements:

- Committee of Earthquake Observation and Research in the Kansai Area (Largest of Three Orthogonal Components)

- Osaka Gas (Vector Sum of Two Horizontal Components)

▲ JR (Vector Sum of Two Horizontal Components After They Have Been Lowpass Filtered at 5Hz)

FIGURE 2-3
PEAK GROUND ACCELERATIONS (GRI, 1995a)
FIGURE 2-4
GROUND MOTION RECORDED AT DOWNTOWN KOBE ACCELEROMETER STATION
(DPRI, 1995b)
FIGURE 2-5
RECLAMATION IN OSAKA BAY
(Mikasa and Ohnishl, 1981)
FIGURE 2-6
PROBABILISTIC ESTIMATES OF PEAK GROUND ACCELERATION
IN JAPAN AND KOBE AREA (PHRI, 1995)
FIGURE 2-7
PROBABILISTIC ESTIMATES OF GROUND SHAKING THROUGHOUT JAPAN
(Kawasumi, 1951)

2-10
FIGURE 2-8
COMPARISON OF PEAK ACCELERATIONS AND VELOCITIES
RECORDED IN KOBE TO THOSE PREDICTED USING EMPIRICAL
RELATIONSHIPS BASED MAINLY ON CALIFORNIA DATA
(SOMERVILLE, 1995)

2-11
a) Building Collapse

b) Major Fires

c) Rail Bridge Collapse

d) Subway Collapse

e) Highway Bridge Collapse

FIGURE 2-9
DAMAGE IN KOBE DUE TO HYOGO-KEN NANBU EARTHQUAKE (INCEDE, 1995)
CHAPTER 3
KOBE PORT

3.1 General Background

3.1.1 Port Description

The Kobe Port is Japan’s largest container port, handling about 30 percent of Japan’s container traffic, and currently ranks sixth worldwide in annual cargo throughput (53 million metric tons in 1993). The port contains approximately 64 km of total quay length, with about 9 km of this devoted to container quay length and about 9 km for break bulk wharf and warehousing. As of April 1994, it had 24 container berths with facilities for accommodating up to 250 large ships at any time.

The Kobe Port is located along the northern margins of Osaka Bay. It extends in a predominantly east-west direction from the Eastern Sea Construction Areas 1 through 4 to the Suma and Nagata Harbors (West Reclaimed Lands) -- a shoreline distance of about 24 km (see Fig. 3-1).

With the exception of the western-most 4 km of the shoreline, the entire waterfront area of the port has been extensively developed for commercial use. The facilities that comprise the port include commercial zones (i.e., facilities managed and operated by the City of Kobe Port and Harbour Bureau), industrial zones (i.e., privately-owned facilities) and a relatively small marine zone (Meriken Park). The bulk of the cargo handling and storage facilities are located in six areas within the harbor limits, of which the largest are at Port Island and Rokko Island -- two major offshore reclamation islands (Fig. 3-2). The other four major cargo handling and storage facilities are located on the mainland and on near-shore reclamation islands, and consist of the Hyogo Piers, Shinko Piers, Maya Piers and Container Terminal, and Fourth Reclamation Area.

3.1.2 Chronology of Port Development

A chronology of the development of the Kobe Port is provided in Table 3-1, Figure 3-3, and the following paragraphs. It represents useful reference information for evaluating how such factors as design standards and construction methods may have affected the seismic performance of the port facilities during the Hyogo-Ken Nanbu Earthquake.

The port has been operating as an international port since 1868 and, since that time, development of new facilities and modernization of the older facilities has continued virtually uninterrupted. A period of extensive construction that started in 1897 and continued into the early 1940's led to the development of many of the port's mainland facilities including the Hyogo Piers, Takanawa Wharf and the Shinko Piers (No's. 1 through 6). The post-World War II era (from about 1950 to 1970) saw the completion of the Shinko Piers (No's. 7 and 8), the Maya Piers, and reclamation islands to the east that comprise the Eastern Sea Construction Areas 1 through 4.
Since 1970, the City and Port of Kobe have focussed development efforts on the two major offshore port complexes -- Rokko Island and Port Island. The first stage of Port Island was completed in 1981 after 15 years of construction, and Rokko Island was completed in 1991 after 20 years of reclamation work. Several redevelopment projects have also been carried out since 1970 to modernize selected existing facilities in other areas of the port.

With the completion of Rokko Island, development of the Kobe Port area has focussed on the planning and construction of a transportation system between various portions of the port. A 390 ha second stage of Port Island is currently under development, and the construction of an extensive system of bridges and tunnels that link the islands to the mainland has been initiated. In addition, plans are well underway for development of an approximately 300 ha island off of the southern shore of Port Island that will contain a new Kobe domestic airport.

3.1.3 Soil Conditions

The city of Kobe is founded largely on competent alluvial soils transported from the Rokko Mountains. These deposits are broadly characterized as an interbedded sequence of dense sands and gravels, and stiff clayey soils. The depth to bedrock beneath Kobe increases dramatically toward Osaka Bay to the south, due to dip-slip offsets of bedrock across numerous faults near the base of the Rokko Mountains. The portions of Kobe that have been built outboard of the historic margins of Osaka Bay are underlain by variable thicknesses of a soft to medium stiff marine clay which is very similar in its engineering properties to San Francisco Bay mud.

The Port of Kobe has been built almost exclusively on reclaimed land. Sandy soil has been placed over the soft clay deposits resulting in significant settlement due to the consolidation of clays. The thicknesses of the sandy fill and marine clay generally increase with distance south of the historic shoreline. The soil profile is somewhat similar to that found along the margins of the San Francisco Bay at the Ports of Oakland and San Francisco. A generalized soil profile for Port Island is shown in Figure 3-4, together with grain size distributions of sands throughout the Port of Kobe that liquefied after the Hyogo-Ken Nanbu Earthquake.

Port Island was developed by barge-dumping granular soil onto a roughly 10-to-15 m thick layer of very soft to soft marine clay. The fill material is predominantly decomposed granite (masa soil) that ranges in classification from SP/SM to SW/SM. Early reports on the performance of the reclamation at Port Island indicate that the fill includes an appreciable content of boulder-sized material (Nakakita and Watanabe, 1981). The fine-grain portion of the soil observed in numerous sand boils has negligible plasticity. The loose nature of the fill (as indicated by the low penetration resistance, $N_{100} \approx 5-7$ blows/foot) resulted in an extremely high susceptibility to liquefaction, as will be discussed in a following section of the report. From the ground surface down, the soil profile at Port Island consists of approximately 15-to-20 m of loose sand underlain by 15 m of soft to medium stiff marine clay; 30-to-35 m of interlayered dense gravelly sand and stiff clay; 20 m of stiff marine clay; and interbedded very dense sand and stiff to hard clay to the maximum depth of the borings at 90 m.
Several methods of soil improvement have been utilized at selected sites on Port Island and Rokko Island. These techniques include preloading to minimize differential settlements under structures, sand drains, sand compaction piles and "composite" piles. The areas of the islands where soil improvement has been implemented are located primarily within the interior of the island (at areas of commercial development not associated with shipping and cargo handling). Figures 3-5 and 3-6 show that very little soil improvement has been performed at the shipping berths and wharves along the periphery of the islands.

The sand drains have been used primarily to expedite consolidation settlement of the marine clay that underlies the sandy fills. The sand compaction piles involve densification of the replacement sand, as well as, the soil adjacent to the piles. This soil improvement technique has been used to increase the bearing capacity and reduce the potential for earthquake-induced liquefaction of the sandy fill. Based on our visual inspection of several of the sites indicated as having received soil improvement, it appeared that the sand drains contributed negligible resistance to the development of excess pore pressures leading to liquefaction. This is not surprising in light of the placement method for the drains and the fact that densification of the soil is not provided by this method. Relatively few sites where sand compaction piles were used were accessible during our reconnaissance. Based on these limited observations, it appeared that the sites improved with sand compaction piles performed much better than adjacent sites which had not been improved. Further substantiation of the effectiveness of these soil improvement techniques for minimizing ground failures should await results of detailed instrumental surveys of the vertical and horizontal deformations at Port and Rokko Islands.

The review of a limited number of soil boring logs and profiles at other piers indicates that the soil profile at Port Island is fairly representative of the conditions at the other portions of the Port of Kobe if an allowance is made for the distance bayward from the historic shoreline as previously noted. For example, the Fourth Reclamation Area (island) is located at the eastern-most edge of the harbor limits of the Port of Kobe. The southwest corner of this island is approximately 2 km northeast of the Rokko Island Ferry Terminal. Boring logs obtained at a site located in the southwest portion of the island demonstrate that the soil profile consists of 4 m of medium-dense to dense (N = 20 to 55 blows/ft.) sandy (mala) fill underlain by approximately 13 m of loose to medium dense sandy fill (N<sub>eq</sub> = 8-to-10 blows/ft), 11 m of soft to medium stiff marine clay, and an interbedded sequence of medium dense to very dense sands and clayey sands to the depth of the boring at 42 m.

Although vibro-methods of soil improvement were reportedly used in soils adjacent to the concrete caissons along the southwestern portion of the island, it is not presently known if the upper 4 m of fill at the site of the soil boring (an inland location) was intentionally densified by a soil improvement technique or perhaps by a fortuitous construction sequence which densified the upper zone. The ground water table is located at a depth of 3 m; therefore it can be surmised that this near surface soil was probably end-dumped and compacted by construction traffic.
One potentially significant difference in the soil profiles was reported by several of the engineers that we met with. It was noted that the fill soil used at Port Island was sandy masa soil excavated from the Rokko Mountains northwest of Kobe, and that some of the fill used in the inland portions of Rokko Island contained "a significant portion" of crushed mudstone and siltstone material. The geotechnical properties of the crushed sedimentary rock fill used at Rokko Island have not yet been ascertained (as of April 1995). It is interesting to note that the grain size characteristics of soils excavated from sand boils located along the perimeters of Port and Rokko islands are very similar.

Based on the geotechnical information available at this time, it appears that most of the areas within the Port of Kobe are founded on similar soils. Among the soil characteristics that these areas have in common are: (a) thick surficial layers of loose saturated sand; (b) fairly extensive deposits of soft to medium stiff marine clays; and (c) deep soil profiles over basement rock. The extensive occurrence of liquefaction in the sandy fill at almost every pier within the Port of Kobe has been well documented. In addition to this phenomena, the underlying soils also had a significant influence on the characteristics of the strong ground motions experienced throughout the port. Although very few acceleration response spectra are available at this time, it is anticipated that the deep soil deposits contributed to an enhanced intensity of strong motion at longer periods (on the order of 0.5 sec. to 1.0 sec or greater) and a reduced intensity of shaking at the shorter periods. These amplified longer period motions probably contributed to the damage that was observed at most of the bridges which connect the large reclamation islands.

3.1.4 Caisson Quay Wall Sections

Typical caisson quay wall cross-sections along Rokko Island and Port Island are shown in Figure 3-7. This figure shows that these quay walls consist of concrete caissons that are filled with sand and supported on a cobblestone foundation that rests on the underlying sandy fills. The backfill that has been placed along the landward side of the walls consists of gravelly material adjacent to the wall and sandy soil further inboard. Pile supports have not been provided for any of the quay walls at Port or Rokko islands. Piles were used only for the landward crane rail at the crane locations along the west and north faces of Port Island. None of the crane rails at Rokko Island are pile supported.

The quay walls along the smaller islands and the mainland also consist of concrete caissons whose cells are filled with sand and, in some cases, concrete. Unlike Port Island and Rokko Island, pile foundations have been utilized to support several sections of the mainland quay walls. Examples of these foundation types include the Naka Pier and Pier 6 of the Shinko Piers (Figure 3-8). Piles are also used to support piers at the Takahama Wharf and at Berth C, Pier 1 of the Maya Piers, as discussed in a subsequent section of this report. Steel pipe piles have apparently been used to support crane rails along the eastern portions of the Maya Piers (Berths Q, R and S).

3.1.5 Seismic Design Procedures for Caissons

The seismic design procedures used for caissons at Japanese ports have been based primarily on an evaluation of the stability of the walls when they are subjected to inertia forces that are defined
in terms of an equivalent seismic coefficient (i.e.; pseudo-static lateral force methods). Code-based methods of analysis did not address the effects of liquefaction of foundation and backfill soils on the seismic performance of the caissons and other waterfront retaining structures (i.e.; quay walls, piers, breakwaters, bulkheads) until the most recent revision of the design standards was adopted in 1989 (JPHA, 1989). To our knowledge, such assessments had not been incorporated into the prior design of any of the caissons within the Port of Kobe.

(a) Equivalent Seismic Coefficient

The caisson-type quay walls at the Port of Kobe have been designed to resist pseudo-static lateral forces computed using a variety of seismic coefficients. For example, seismic coefficients used in the design of the quay walls at Port Island and Rokko Island were 0.1 and 0.15, respectively. At the Maya Pier, most of the quay walls were designed using a seismic coefficient of 0.18, except along the west side of Pier No.1, where a seismic coefficient of 0.25 was used. Caissons at the southwest portion of the Fourth Reclamation Area were designed with a seismic coefficient of 0.15. Based on the information provided to us in Japan, it appears that the remaining quay walls along the mainland piers and wharves were designed using a seismic coefficient of 0.1.

In order to evaluate the design criteria utilized for waterfront retaining structures at various portions of the Port of Kobe, a comparison of the seismic design codes prescribed for port and harbor facilities in 1959 and 1978 is presented. The zonation maps which provide the seismic coefficients for Japan are illustrated in Figure 3-9. In both maps, Kobe and Osaka lie in the region rating the highest seismic coefficient. It is interesting to note that the range of seismic coefficients was reduced from 0.15-0.25 to a single value of 0.15 in the latter code. The sparse documentation discovered to date on the 1959 code (OCSWCEE, 1960) indicates that;

"The ranges of the seismic coefficient in each section (i.e., seismic zone) are legislated, and the final seismic coefficient used in the design of a structure is decided from the standard range of seismic coefficient, taking into consideration the kind and importance of the structure, and the condition of the foundation."

No information was available regarding the factors used in the code to represent the soil conditions and the importance of the structure.

The most recent code that is currently available (1989) for port facilities defines the design seismic coefficient \( (k_a) \) as:

\[
k_a = ZGI
\]

where \( Z \) is the seismic zone factor, \( G \) is the ground condition factor, and \( I \) is the importance factor. The \( Z \) factors used in this computation of \( k_a \) are shown in Figure 3-9b, and the \( G \) factors and \( I \) factors are given in Table 3-2. The factors of safety incorporated into the seismic design of the caissons are 1.0 and 1.1-to-1.2 for sliding and overturning respectively.
As of this writing, information describing the basis for specifying the seismic coefficients in areas such as the Maya Piers, where several different values have been used for the design of the quay walls and the pile supported berth has not been available. It is anticipated that such issues will be resolved in the near future.

(b) Assessment of Liquefaction Potential

The most recent (1989) Japanese code for port facilities now includes provisions for assessment of the liquefaction potential of saturated sandy soils during earthquakes. These provisions consist of the following steps (Tsuchida, 1990); (a) check the grain size distribution of the soil against critical ranges of grain sizes specified by Iai et. at. (1989) for liquefiable soils (Figure 3-10a); (b) compute equivalent penetration resistances (N-values) for each saturated sand layer by correcting N-values from SPT test results for fines content and to a reference vertical effective stress (0.66 kgf/cm²); (c) carry out an equivalent linear site response analysis, and obtain an effective acceleration for each saturated sand layer as described by Tsuchida; (d) for each layer, use the corrected N-value and the computed effective acceleration to enter a design chart (Figure 3-10b) in order to assess the potential for liquefaction of the soil at the site; and (e) if liquefaction is indicated, incorporate appropriate countermeasures to reduce liquefaction risks to the port. Because these liquefaction assessment procedures have been in effect in the codes for ports since only 1989, it is our understanding that site-specific liquefaction analyses have been carried out for only a very few locations (on Port Island only) prior to the Hyogo-Ken Nanbu Earthquake (Youd, 1995).

3.2 Earthquake Ground Motions

The general level of ground shaking at the Port of Kobe is indicated by the ground motions recorded at the Kobe Port Construction Office (on the mainland) and by a downhole array of accelerometers on Port Island. The accelerograms at the Kobe Port Construction Office (Figure 3-11) exhibit peak horizontal accelerations of 502 cm/sec² and 205 cm/sec², and a peak vertical acceleration of 283 cm/sec². The duration of the strong shaking segment of the recorded horizontal motions was about 5 sec.

The downhole array of accelerometers on Port Island are located at the northwest portion of the island, just south of the Kobe Bridge (Figure 3-12a). The array consists of instruments on the ground surface and at depths of about 17 m, 33 m, and 84 m; the uppermost two instruments are at the top and the base respectively of the loose fills, the third instrument is in an intermediate sand layer that is medium dense, and the deepest instrument is in a very stiff sand layer (Figure 3-12b). The horizontal motions recorded at each instrument in this array (Figure 3-13) were very strong, with peak accelerations that ranged from about 280-340 cm/sec² at the ground surface and larger values at various depths below the ground surface (about 540-680 cm/sec²). Vertical motions were also recorded at this array and had peak accelerations of 560 cm/sec² at the ground surface, 790 cm/sec² at the 17 m depth, and about 190 to 200 cm/sec² at lower depths. The accelerations at the top of the fill were smaller and had a much longer predominant period than the accelerations at the base of the fill. This reflects the effects of softening, and liquefaction of the upper fill layer.
Based on the pattern of peak accelerations throughout the city of Kobe, the relative distances to the rupture zone, and the similarities in the soil profiles, it is inferred that the range of strong ground motions were similar across Port and Rokko Islands.

3.3 Seismic Performance Overview

The Port of Kobe was severely damaged during the Hyogo-Ken Nanbu Earthquake. Virtually all of the 240 berths at the port were closed indefinitely after the earthquake (i.e., only 6 of the 240 berths were serviceable to any degree). Repair costs for the port have been estimated at about $10 billion, and repair times (to restore total operations at the port) have been estimated to be about 2 years from the time of the earthquake (Griffin, 1995). Probable causes of this severe damage and the types of damage that occurred at the Kobe Port are discussed in the remainder of this chapter.

3.4 Seismic Performance of Waterfront Retaining Structures and Cranes

The primary cause of the extensive damage at the port was ground failure due to widespread liquefaction of the fill materials (Fig. 3-14). This resulted in increased lateral pressures applied to the quay walls which, for the quay walls without pile supports, led to large seaward displacements of the walls (often on the order of several meters). As a result of these seaward movements, the fill soils behind the quay walls moved laterally and settled substantially (Fig. 3-15). The large lateral soil deformations in the waterfront areas extended into the backland areas distances of up to 75-to-100 m, as evidenced by tension cracks in A.C. pavements and movement of near surface soils away from pile-supported structures. Youd (1995) has reported that ground fissures oriented parallel to the quay walls formed as far inland as 200 m, indicating the extent of the lateral spreading.

3.4.1 Concrete Caisson Quay Walls

Most of the quay walls at the Kobe Port were concrete caissons (soil and concrete filled) without pile supports, as described in Section 3.1.4. The seismic performance of these types of quay walls in the presence of the liquefied fills and increased lateral pressures against the walls was poor. In addition, liquefaction of the underlying foundation soils may have contributed to the poor performance of these quay walls.

Based on information obtained to date, we are aware of only two locations where concrete caisson quay walls were supported on piles — the southern end of the Naka Pier and along the shallow-water sections of Shinko Pier No. 6. At each of these locations, foundation support is provided by tapered timber piles with a tip diameter of 0.21 m and lengths ranging from 6.3 m to 7.2 m. The ground deformations at these pile-supported caissons were slightly-to-significantly less than those observed at adjacent caisson quay walls without pile supports. It is noted that similar piles were placed beneath the concrete caissons at Hyogo Pier No. 2 although, based on the cross sections provided to us during our reconnaissance, it does not appear that the piles actually support the caissons at this pier. This is significant because the movement of the caissons at Hyogo Pier No. 2 was large, resulting in complete submergence of the end of the pier.
Regarding the effects of the seismic design coefficient \( k_h \) on the performance of these caisson quay walls, we were informed that the \( k_h \) values used in the design of the caissons at Port Island and Rokko Island were 0.1 and 0.15, respectively. These different \( k_h \) values are partially reflected by the smaller height-to-width (H:W) ratios for the caissons at Rokko Island. (The values of these ratios ranged from 1.21 to 1.32 at Port Island and from 0.97 to 1.19 at Rokko Island.) It is important to note that no form of soil improvement explicitly intended to densify the foundation pad or backfill soils adjacent to the caissons was performed at either Port or Rokko Islands. For the most part, the performance of the caissons during this earthquake complements an extensive list of case histories that demonstrate the deficiencies of seismic coefficient-based design methods for caissons in loose saturated sandy soils, since these methods do not account for the potential liquefaction of these soil materials (Werner and Hung, 1982).

### 3.4.2 Cranes

All of the gantry cranes at the Port of Kobe are rail mounted. Most of these cranes have a rail span of 30.5 m, and were built in the 1980s and 1990s. A few of the older cranes at Port Island (built in the early 1970s) have a rail span of 16 m. All bayward crane rails rest on the caisson. The landward rail for the cranes with a 30.5 m rail span were supported on engineered fill, and the landward rail for the cranes with a 16 m rail span were supported on piles. Most of the cranes were secured with the stowage pins in place (Liftech, 1995).

The quay wall displacements (translational and rotational) and the large differential soil movements described previously resulted in severe spreading and deformation of crane rails. This, in turn, led to buckling and yielding of the legs of many of the cranes, and complete collapse of the cranes in some cases (Figures 3-16). Although liquefaction and movement of the underlying soils was the primary cause of the observed crane damage, Liftech (1995) has observed that the gantry crane construction also influenced the degree and type of damage experienced by the cranes during the Hyogo-Ken Nanbu Earthquake. In particular, it was noted that cranes with weak portal beams and with non-ductile moment frames suffered significant buckling of the legs above the portal frames, whereas damage to cranes with strong portal beams forming ductile moment frames was restricted to small areas near the portal beams.

### 3.4.3 Steel Plate Cellular Bulkheads

Instead of the concrete caissons which are predominant throughout the port, steel plate cellular bulkheads were deployed at the Maya Piers during their original construction in the late 1950s and early 1960s. These caissons were fabricated out of 9 mm thick steel plate, and were formed to a diameter of 15.5 m and height of 14.0-to-16.5 m. The cells were filled with sandy soil and 0.4-to-0.5 m diameter steel pipe piles were driven through the interior fill to support crane rails. It is noted that, during the subsequent redevelopment of the Maya facilities for container handling and storage (1987-1991), concrete caissons were placed outboard of many of these older cells.

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Modes of failure of the steel cell caissons are similar to those outlined for concrete caissons (i.e., settlement due to densification and deformation of foundation soils, translation caused by increased earth pressures, and loss of ground behind the retaining structures due to the flow of liquefied soil through gaps between individual caissons). This is shown in Figure 3-17.

3.4.4 Concrete Block Quay Walls

In many of the shallow-water portions (i.e., inboard ends) of the piers at the Port of Kobe, retaining walls constructed out of stacked blocks of concrete are used. The blocks are generally 1.5 m in height and increase in width with each successive lower block (e.g.; at the Maya Piers, the width of the upper block is 1.5 m and the lower blocks measure 2.5, 3.5, 4.5, and 5.0 m; at the Naka and Shinko Piers, the blocks measure 1.32, 1.52, 1.82, and 2.93 m). The performance of these retaining structures is also linked to the extent of the liquefaction that occurred during the earthquake. We observed damage to these walls that varied from minimal to catastrophic, with greater damage experienced at the Naka and Shinko Piers.

3.4.5 Quay Walls at Maya Piers

The development of the Maya Piers, located near the mainland between Port and Rokko Islands (Fig. 3-18), was initiated with reclamation of the island in 1959. Construction was completed in 1967, and the facility originally consisted of four finger piers extending to the south (i.e., bayward) with additional shallow-water berths (4 m) along the northern perimeter of the island. A variety of waterfront retaining structures have been employed at the Maya Pier complex, providing an opportunity to evaluate the relative seismic performance of these structures under equivalent seismic loads.

An extensive collection of quay wall sections for the Kobe Port which we obtained during our reconnaissance (Iwasaki, 1995a; Tsuchida, 1995), indicates that the waterfront areas of the Maya Piers were originally developed as follows. In this, it is noted that the locations of the various berths denoted below are shown in Figure 3-18.

(a) At Berths D, G, I, K, and Q, steel plate cellular bulkheads (SPCBs), with a diameter of 15.5 m and a height of 16.5 m were deployed.

(b) Along the south-facing quay of Pier No. 2 and at Berth O, SPCBs were deployed that had a diameter of 15.5 m and a height of 14 m.

(c) At Berth A, SPCBs with a diameter of 15.5 m and a height of 15 m were deployed, together with an outboard wharf supported on batter piles.

(d) Along the south facing quay of Pier No. 1, soil-filled concrete caissons were deployed that had a width of 9.1 m and a height of 10.7 m.
Along many of the shallow-water (4 m) portions of the piers, concrete block walls were deployed.

SPCBs were also used along Berths B, C, E, H, J, R and S, although information on the exact dimensions of these cells was not available at the time this report was prepared.

In all cases, the SPCBs were founded on rubble (i.e., rock) fill and backfilled with sandy soil. Steel pipe piles (with a diameter of 40-to-50 cm, a wall thickness of 0.6 cm, and a length of 24-to-30 m) were driven through the interior fill to support crane rails. In most cases, a row of interlocking steel pipe piles (with a diameter of 40 cm, a wall thickness of 0.9 cm, and a length of 30 m) were driven adjacent to the bayward portion of the cells to form a continuous seawall.

In 1987, a four year redevelopment project was initiated to transform Piers No. 3 and 4 into a container handling facility - the Maya Container Terminal. At that time, the 9.5 ha area between the two piers was filled, and new concrete caissons were placed roughly 15 m bayward of the older SPCB wall at Berths O and P. The space between the caisson and the SPCB was filled with sandy soil. The soil was presumably end dumped, and it was not densified prior to the construction of the pavement section for the deck.

The facilities along the western side of Pier No. 1 were also expanded during this recent phase of development. At Berth A, the pile-supported wharf has been removed and replaced with 17.1 m by 15 m concrete, soil-filled caissons (Fig. 3-19a). The caissons were placed approximately 3 m outboard of the older SPCB on an asphalt mat underlain by rubble fill. The void between the retaining structures was filled with rubble. A similar procedure was followed at Berth B, where 14.4 m by 12 m concrete, soil-filled caissons were placed bayward of the original SPCB. Rubble fill was again used to fill the area between the retaining structures.

At Berth C, a pile-supported wharf was constructed in lieu of the concrete caissons used at Berths A and B (Fig. 3-19b). A 10 m wide wharf supported by 1.2 m steel pipe piles was constructed adjacent to the original SPCB. The inboard row of these piles includes batter piles, which have often been identified as particularly vulnerable to strong shaking during earthquakes (Werner and Hung, 1982; EERI, 1990). The outer row of piles consists of interlocking steel pipe piles which form a continuous wall (Figures 3-20k and l). The area between the interlocking piles and the original SPCB has been filled with rubble to a level that was below the tops of the piles (see Fig. 3-19b).

Based on the information currently available, it appears that the area between Piers No. 1 and 2, Berths D and E, and the southern edge of Pier No. 1 were not modified during redevelopment. In these areas the original quay walls, designed with static lateral force coefficients of 0.18, are still in place.

The new caissons and pile-supported wharf at Berths A, B, and C of Pier No. 1 have been designed using a seismic coefficient of 0.25. The use of this relatively high static lateral force factor

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has led local engineers to refer to this section of the Maya Piers as the "Aseismic-Reinforced Berths" or the "Earthquake Resistant Berths". In striking contrast to the poor performance of the other waterfront areas at the Maya Pier complex (as summarized below), the quay walls at Berths A, B and C exhibited very little damage during the earthquake.

The photographs in Figure 3-20 provide an overview of the damage sustained by the various quay walls and pile-supported structures at the Maya Piers. With the exception of Berths A, B, and C, damage to quay walls was extensive throughout the Piers. At the north approach to the Piers, lateral movement of the quay walls resulted in damage to adjacent buildings and warehouse foundations (Figures 3-15f, 3-20a, b). Liquefaction-induced damage to the waterfront structures was also extensive along the northern perimeter of the island. The new concrete caissons along Berths O and P shifted dramatically toward the bay, resulting in minor warping of the legs of cranes working in this area (Fig. 3-20c). To the west, similar damage to concrete block retaining structures between Piers No. 1 and 2 was experienced (Fig. 3-20d).

Pier No. 1 provides a valuable opportunity to evaluate the effectiveness of various types of retaining structures and seismic design provisions for waterfront structures. The waterfront areas along the east and south-facing portions of the pier were not affected by the redevelopment. It is again noted that the seismic coefficients used in design of the original caissons and the newer retaining structures are 0.18 and 0.25, respectively. The quay wall at Berth E failed dramatically with the retaining structure translating and rotating into the bay. The adjacent SPCB at Berth D performed relatively well, although the backland soils liquefied and appear to have flowed into the bay through gaps between the cells. (Figs. 3-17a, b). The southern corner of the pier includes a pile supported wharf. Lateral ground deformations resulted in buckling of several of the steel pipe piles (with a diameter of 1.0 to 1.2 m) near the pile caps. The concrete caissons along the southern end of the pier moved toward the bay; this resulted in the formation of a deep graben (2-to-3 m deep) in the backland area, but surprisingly little damage to the adjacent warehouses (Figs. 3-20f, g). It is not until the quay wall at Berth A is approached that any appreciable difference in the seismic performance of the retaining structures is apparent.

As previously noted, the quay walls along the west side of Pier No. 1 have been reconfigured with the addition of concrete caissons or interlocking steel pipe piles. These retaining structures are founded on thin mats (~ 0.5 m) of rubble fill and dredge sand. The retaining structures at Berths A, B and C performed very well during the earthquake, with only minor deformations observed between Berths B and C (Figs. 3-20h-j). No damage of the interlocking pipe pile wall at Berth C was evident (Figs. 3-20k, l). The continuous nature of the waterfront wall at Berth C precluded our observation of the batter piles which support the inboard portion of the wharf.

The seismic resistance of the quay walls at Berths A, B and C provides one of the few examples of acceptable performance of waterfront retaining structures at the Kobe Port. The use of stringent pseudostatic design requirements is considered to be only partially responsible for this success. Liquefaction of the fill surrounding the warehouses adjacent to the quay walls was evidenced by sand boils and substantial settlement. The increased lateral earth pressures due to
liquefaction of the fill were resisted by both the SPCB and the newer retaining structures. It is surmised that the rubble fill immediately behind and beneath the quay walls was not susceptible to the generation of significant excess pore pressures. Therefore, the two retaining structures, acting together, provided adequate resistance to the increased lateral earth pressures.

An interesting comparison can be made between the performance of the new quay walls at Berths A, B and C and the new caissons at Berth O and P, where concrete caissons have also been placed outboard of the original SPCB. Based on information obtained during our reconnaissance, the seismic coefficient used for the new caissons at Berths O and P was 0.18. This may have contributed to the disparate performance of the caissons at this portion of the Maya Container Terminal. More important is the fact that the new concrete caissons at Berths O and P were backfilled with loose sand, not the coarse rubble used at Berths A, B and C. Liquefaction of the fill behind the caissons at Berths O and P clearly contributed to the failures that occurred.

3.5. Seismic Performance of Pile-Supported Structures

3.5.1 Pile-Supported Structures in Waterfront Areas

Based on available information at the time of this report preparation, nearly all of the port's wharves, warehouses, passenger ferry terminals, commercial buildings, and elevated highway and rail systems are supported by end-bearing piles. These piles extend through the loose sandy fill and underlying soft marine clay to depths of 20-to-30 m or greater where they are embedded in the dense older alluvium. The placement of the fill over the marine clay has resulted in considerable settlement due to consolidation of the clay (e.g.; 4 to 5 m at Port Island). Most structures are supported by deep foundations in order to alleviate potential damage due to differential settlements.

The extensive catalog of quay wall sections and a very limited number of foundation schematics for bridges, buildings and other structures, indicate that virtually all of the piles are vertically oriented. The only use of batter piles that has been documented to date is at Berth C, Pier 1 at the Maya Piers. These batter piles are isolated from view by a row of interlocking steel pipe piles; therefore, the condition of these piles and the pile cap had not been ascertained at the time of our reconnaissance.

The observed performance of piles and pile-supported structures can be related in most cases to the direction and extent of ground deformation which occurred at the site. As previously discussed, the soils adjacent to quay walls and extending significant distances inland exhibited pronounced lateral as well as vertical deformation. In these areas, piles were often subjected to large lateral loads due to the movement of the surrounding soils. Soil settlements facilitated our observation of numerous piles and pile-caps beneath structures located in waterfront areas. Common concrete pile types included roughly 30 and 40 cm diameter hollow cylinder piles (approx. 8 cm wall thickness), 30 and 40 cm diameter solid cylinder piles, and 55-to-70 cm diameter hollow cylinder piles (approx. 8 cm wall thickness). In most cases, steel reinforcement for both prestressed and conventionally reinforced concrete piles was minimal or nonexistent.
During our reconnaissance, we observed numerous examples of severe cracking or fracture of hollow concrete cylinder piles at or near their connection to the pile cap (Fig. 3-21b). In addition, we observed a number of solid concrete cylinder piles in the waterfront area that appeared to perform well in the presence of large lateral movements of the surrounding soils, with only negligible-to-minor hairline cracking near pile caps. This contrasting performance of the hollow concrete cylinder piles and the solid concrete cylinder piles has also been documented by members of other reconnaissance teams (e.g., Youd, 1995).

Steel pipe piles were also observed to have experienced damage. Underwater inspection of a pile-supported grain transport wharf at the southwestern margin of the Fourth Reclamation Area revealed moderate buckling to 1.2 m diameter (1.9 cm wall thickness) steel pipe piles (Fig. 3-21a). Damage at this wharf included the collapse of cargo cranes (presumably due to inertial effects as opposed to the ground failure-induced mode of failure common at Port and Rokko Islands), minor deformation of crane rails, and severe damage to the grain conveyor system. Damage to the wharf deck itself appeared to be minor. An example of apparently good performance of steel pipe piles is the vertical piles that support the Takahama Wharf, whose deck did not suffer any cracking or damage (Fig. 3-22).

During our reconnaissance, we observed that the lateral and vertical resistance provided by the piles invariably minimized the lateral and vertical displacements and deformations of waterfront buildings (e.g., Fig. 3-23c). Although damage to some of the pile-supported buildings due to ground shaking was observed, there was relatively small permanent displacement of the building structures even though the surrounding soils moved substantially. In waterfront areas, the lateral ground deformations decreased with increasing distance from the quay walls. Soils underlying waterfront structures subsided and settled substantially throughout the port facilities. In several cases, the pile foundations for large warehouses exhibited extensive damage on the waterfront side of the structure (Fig. 3-21b) and minimal damage on the inland side. In many instances, the warehouses performed very well despite the extensive deformation of foundation soils and complete separation from the pile foundations. This performance may also be attributed to the existence of well-reinforced floor slabs.

The influence of pile foundations on the extent of lateral movements was also observed at several large bridge foundations. Lateral ground deformations were extensive (greater than 1-to-3 m) adjacent to the foundations of many of the major bridges which link the port facilities. In every case, the extensive pile groups limited the deformation of the bridge piers (as discussed further in Section 3.7.2).

It should be noted that our observations regarding waterfront pile performance (particularly where the piles appeared to be undamaged) should be tempered by the fact that our observations were confined to the upper several feet of the piles, and that only very limited underwater or subsurface inspections of piles had been made at the time of our reconnaissance. In view of the large soil movements that occurred along the waterfront area of the Kobe Port, it is likely that many more examples of pile damage could be uncovered after more extensive subsurface inspections of the piles are carried out. The performance of the piles within the liquefied soils, at the interface between the
sandy fill and the soft marine clay, and at the interface between the marine clay and the older alluvium, are important issues that remain to be addressed. Assessment of these particular issues, together with the overall assessment of the seismic performance of the waterfront pile foundations, should also include the compilation of further information pertaining to the type of piles employed, the geometry of the pile groups, and the design and construction of the piles.

3.5.2 Pile-Supported Structures in Interior Areas

Ground deformations in the inland portions of the reclamation islands were predominantly vertical due to the densification of the sandy fill. In these areas, pile-supported structures appeared to perform very well, and their foundations did not exhibit any evidence of permanent deformations that could be related to pile failure. On several of the reclamation islands, pile-supported buildings appeared to remain at their design elevations despite significant settlement of surrounding soils. On Port Island, the relative vertical movement averaged roughly 0.5 m, with settlements in several portions of the island reaching as much as 1 m (Fig. 3-23a, b, and d). This relative movement between the pile caps and the ground surface is interpreted to indicate that the piles did not fail by buckling in the liquefied soils, nor did they settle appreciably in the dense bearing strata due to the loss of skin friction in the liquefied fill and subsequent down drag on the pile that may have occurred as the densified fill settled. The latter phenomenon is judged to be a minor effect, since excess pore pressures are still high as the sand settles and it is anticipated that gaps may form between the soil and the pile in the upper several diameters during strong ground shaking. Both of these factors would significantly reduce the downdrag due to skin friction on the pile immediately after the earthquake. From a practical perspective, an additional factor that would contribute to minimal earthquake-induced settlements of end bearing piles (which is the predominant pile type at the Kobe Port) is the relatively large factor of safety that is commonly used in the design of deep foundations. It is also common for geotechnical engineers to ignore the contribution of skin friction provided by loose soils, such as the sandy fill, to the load carrying capacity of the end bearing piles. These design methods would also result in a substantial reserve in the end bearing that could be provided by the dense clayey sands at depth. It is again noted that, at the time of our reconnaissance, extensive subsurface inspections of pile foundations had not been made. Such investigations may reveal pile damage that was not evident during our reconnaissance.

As a related issue, the settlements observed next to pile-supported structures provide a unique opportunity to evaluate current methods for estimating settlements of sandy soils due to earthquake shaking (e.g., Tokimatsu and Seed, 1987; Ishihara and Yoshimine, 1992). Given the penetration resistance of the sandy fill at two sites on Port Island (Figs. 3-4 and 3-12 show an average value of \( (N_d)_{so} = 7 \) blows/ft) and the accelerations recorded in the downhole array (Fig. 3-13), the Tokimatsu-Seed method predicts volumetric strains of the sand of about 3%, while the Ishihara-Yoshimine approach indicates a post-earthquake volume reduction of about 4%. For a fill thickness of 17 m, (as shown in Fig. 3-12), these strains correspond to ground settlements of 0.5-to-0.7 m. These estimated values agree well with observed settlements, indicating the utility of these methods for estimating earthquake-induced settlements in sandy soils.
In the inland areas of Port and Rokko Islands, newer multistory buildings (most likely designed to post-1980 structural codes) appeared to perform very well during the earthquake. Judging from the exterior appearance of the structures, architectural damage was minor and no evidence of permanent deformation was observed. In light of the substantial ground settlement which occurred next to these buildings, it is anticipated that underground utility lines and other appurtenances could have been damaged. We understand that the extent of such damage to structures and buried lifelines throughout the port complex is under continued investigation.

3.6 Seismic Performance of Buildings

3.6.1 Older Corrugated Metal, Wood, and Non-Ductile Concrete Frame Buildings

Older low-rise buildings (e.g., warehouses, etc.) within the port were subjected to varying degrees of damage due to the earthquake-induced inertia forces that greatly exceeded those considered during the design of these structures. The port buildings that fall in this category were constructed of corrugated metal, wood, or concrete shear wall or moment frame (probably non-ductile) elements. Damage to the corrugated metal and wood buildings took the form of connection failures, large racking deformation of the walls (which caused jamming of doors into the buildings), foundation damage due to large ground movement, and falling of contents of the buildings (Fig. 3-24).

Non-ductile concrete frame structures, such as the warehouse buildings at Piers 7 and 8 of the Shinko Piers, suffered severe damage and collapse (Figures 3-25 and 3-26). The long, two-story warehouses along Piers 8A and 8B had a soft first story, consisting of a lower story (for parking) whose lateral force resisting system consisted of flexible exterior frames and some interior walls, and an upper story (for storage) whose lateral force resisting system consisted of stiff shear walls. Because of this, virtually all of the columns along the lower level of the buildings either collapsed or were severely damaged, and cranes mounted on the buildings overturned. A strong motion accelerometer in the upper level of one of these buildings recorded very strong motions (with a peak acceleration of about 700 cm/sec²) whose duration of strong shaking was about 8 sec (Fig. 3-27). The durations of several of the peaks in the acceleration history were about 1 sec., indicating significant velocities and displacements of the structure.

3.6.2 Shear Wall Buildings and Newer Construction

Concrete shear wall warehouse buildings, such as those at Pier 6 of the Shinko Piers and at the Naka and Minami Wharves on Port Island, generally exhibited much better performance than did the above-indicated corrugated metal, wood, and non-ductile concrete frame buildings (although the interiors of these shear wall buildings were not accessible for observation) (Figure 3-28). In addition, modern buildings probably designed and detailed using more recent seismic design standards, such as the office buildings in the interior of Port Island and the passenger terminals at Port Island and Rokko Island, appeared to exhibit good seismic performance, even though there was often noticeable settlement of the soils along the exterior of the buildings (Fig. 3-22a). It is noted that the interiors