Consideration of damage caused by recent large earthquakes and earthquake-proofing in Japan and the USA

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Abstract

Japan and the U.S.A. have suffered large earthquakes from 1989 to 1995. It is very important to carefully investigate the damage they caused and look into the effectiveness of our earthquake-proofing methods. The purpose of this study is to use site research and interviews with specialists to investigate and compare the earthquake damage, restoration efforts, progress with earthquake-proofing and new techniques requiring development. Research shows that further studies are needed on the following: 1) a reasonable level of seismic design criteria to prevent collapse of structures, 2) innovative, light weight, and cost effective methods of retrofitting existing structures, 3) design of supports and restrainers, seismic isolation and active controlling to prevent falling of girders, and 4) the flow taking into account the degree of deterioration, deterioration prediction, the life cycle cost, and so on.

Keyword: large earthquakes, earthquake-proofing

1. INTRODUCTION

Japan and the United States of America have recently suffered large earthquakes. Five major seismic events have recently caused death, injury, and property damage in Japan: the Kushiro-oki (1993), Hokkaido Nansei-oki (1993), Hokkaido Toho-oki (1994), Sanriku Haruka-oki (1994), and Hyogoken-Nanbu earthquakes (1995). In the U.S.A., on the other hand, two disastrous earthquakes have been experienced: the Loma Prieta (1989) and Northbridge (1994)

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earthquakes. An earthquake of similar scale in a densely populated metropolitan area, such as Tokyo or New York, could be expected to result in damage on a huge scale.

The different characteristics of each of these earthquakes had an influence on the resulting damage to concrete structures. It is very important to carefully investigate the damage they caused and look into the effectiveness of our earthquake-proofing methods. The purpose of this study is to use site research and interviews with specialists to investigate and compare the earthquake damage, restoration efforts, progress with earthquake-proofing, and new techniques requiring development.

2. METHOD

This investigation, which took place between 1993 and March 1998, focused on the damage caused by the recent large earthquakes as well as on the earthquake-proofing methods in use in Japan and the U.S.A. The methods used were site research, the collection of reports and data on the earthquakes, and interviews with specialists (mainly Americans) in the areas of earthquakes, earthquake-proofing, retrofitting RC structures, and structure maintenance and management.

3. INVESTIGATION RESULTS and CONSIDERATION

3.1 INVESTIGATION RESULTS

(1) OUTLINE OF LARGE EARTHQUAKES AND RESULTING DAMAGE

Outline of earthquakes is shown in Table 1. Outline of earthquake damage scale and level are shown in Tables 2 and 3. These earthquake characteristics were taken notice to study for structure design and maintenance.

1) Recent large earthquakes in Japan (1993 to 1994)

Four large earthquakes caused death, injury, and damage to civil property during this period: the Kushiro-oki (1993), Hokkaido Nansei-oki (1993), Hokkaido Toho-oki (1994), Sanriku Haruka-oki (1994) earthquakes. These earthquakes had magnitudes ranging from 7.2 to 8.2. All occurred between 20:06 and 22:23 in the evening, so they fortunately missed the critical periods during which people are traveling to work, at work or school, or cooking meals. The seismic centers were all near under the sea somewhat offshore. Large tsunami were caused by the Hokkaido Nansei-oki and Hokkaido Toho-oki earthquakes [9].

2) The Hyogoken-Nanbu earthquake (1995)

This was the most destructive earthquake to hit Japan since the 1923 Great Kanto earthquake. Damage to the infrastructure of the modern city of Kobe and Awaji Island, including damage to many concrete structures, was particularly great because the area, known as the Hanshin metropolitan area, is so highly populated. With a measured horizontal acceleration of over 800 gal at several points, there were many examples of the horizontal acceleration exceeding the design value by a large margin [9].

Table 1 Outline of Earthquake

Earthquake	In 1993 Kushiro-oki Earthquake	In 1993 Hokkaido Nansei-oki Earthquake	In 1994 Hokkaido Toho-oki Earthquake	In 1994 Sanriku Haruka-oki Earthquake	In 1995 Hyogoken-Nanbu Earthquake	In 1989 Loma Prieta Earthquake	In 1994 Northridge Earthquake
Time Occurred (local time)	20:06 15 January 1993	22:07 12 July 1993	22:23 4 October 1994	21:19 28 December 1994	05:16 17 January 1995	17:05 17 October 1989	04:31 17 January 1994
Location of seismic center	42.9° N 144.4° E	42.5° N 139.1° E	43.5° N 14.9° E	40.4° N 143.7° E	34.6° N 134.8° E	37.1° N 121.9° W	34.2° N 118.5° W
Focal Depth	107km	10km	50km	20km	10km	18km	18km
Magnitude	7.8	7.8	8.1	7.5	7.2	7.0 *	6.7 *

* The Richter scale

Table 2 Outline of Damage

Earthquake \ Damage type		In 1993 Kushiro-oki Earthquake	In 1993 Hokkaido Nansei-oki Earthquake	In 1994 Hokkaido Toho-oki *1 Earthquake	In 1994 Sanriku Haruka-oki Earthquake	In 1995 Hyogoken-Nanbu Earthquake	In 1989 Loma Prieta Earthquake	In 1994 Northridge Earthquake
Fatality	Dead Missing	1 —	202 29	— —	2 —	5462 2	67	65
	Seriously injured	61	81	—	—	35,080	65	over 5,000
	Slightly injured	677	240	227	285			
Damage to construction	Damage to road (DTR) Damage to bridge (DTB) Totally demolished house (TDH) Partially demolished house (PDH)	DTR: 994 TDH: 18 PDH: 182	DTR: 311 TDM:558 PDH:247 CC: 14	DTR: 44 PDM: 128 CC: 3	DTR: 28 PDH: 128 CC: 3	DTR: Seriously: 303 Slightly: 641 TDH & PDH 103__ DTR: 665 CC: 182	DTR: Seriously: 13 Slightly: 91	DTR: Seriously: 46 Collapsed: 7 Slightly: 240
Amount of damage		About: ¥10 billion	About ¥132.3 billion	About ¥9 billion	About ¥84.2 billion	About ¥10 billion	$6.7 billion (in 1989 dollars)	$15.3 billion preliminary damage estimate)

*1 Excepting the north territory of Japanese (Chishima island chain)
*2 Damage to piers of expressway

Table 3 Outline of Damage level and type of concrete structure

Earthquake / Level and type		In 1993 Kushiro-oki Earthquake	In 1993 Hokkaido Nansei-oki Earthquake	In 1994 Hokkaido Toho-oki *1 Earthquake	In 1994 Sanriku Haruka-oki Earthquake	In 1995 Hyogoken-Nanbu Earthquake	In 1989 Loma Prieta Earthquake	In 1994 Northridge Earthquake
Level	Ultimate strength limit level	△	△	△	△	◎	○	◎
	Service ability limited level	○	○	○	○	◎	○	◎
Type	Strong motion	○	○	○	○	◎	◎	◎
	Tsunami attack	—	◎	—	—	—	—	—
	Grand slide or collapse	○	○	△	△	○	—	—
	Fire	—	△	—	—	○	—	—

Note: Level ◎: remarkable
○: exist
△: very few
—: nothing or not clear
: excepting the north territory of Japan

3) The Loma Prieta earthquake

The Loma Prieta earthquake of October 17, 1989 occurred in a sparsely populated area in the Santa Cruz mountains. The earthquake caused more damage than any California earthquake except the Northridge earthquake of 1994.

The jurisdiction of Caltrans (California Department of Transportation) District 4 approximately matched the area of greatest damage in this earthquake; the department was responsible for 1,896 state bridges of which 91 (4.8%) suffered some degree of damage (mostly minor) in the earthquake. Structural damage or potential threats to public safety were sufficiently serious in the case of 13 of these state bridges to close them to traffic for some time [3].

4) The Northridge earthquake

The Northridge earthquake of January 17, 1994 occurred in a highly populated urban area. The most significant of the strong motion records were a 1.83G peak acceleration at a box girder bridge near the west abutment of the I-10/I-405 Interstate interchange. The earthquake caused serious damage to critical transportation systems: the collapse and partial collapse of bridges. Because the Northridge earthquake struck an urban area containing structures of many types, it provided the first real test for many modern seismic-design structures (Photo 1). Recently constructed bridges and post-1987 retrofitted reinforced concrete bridges appeared to perform reasonably well [3].

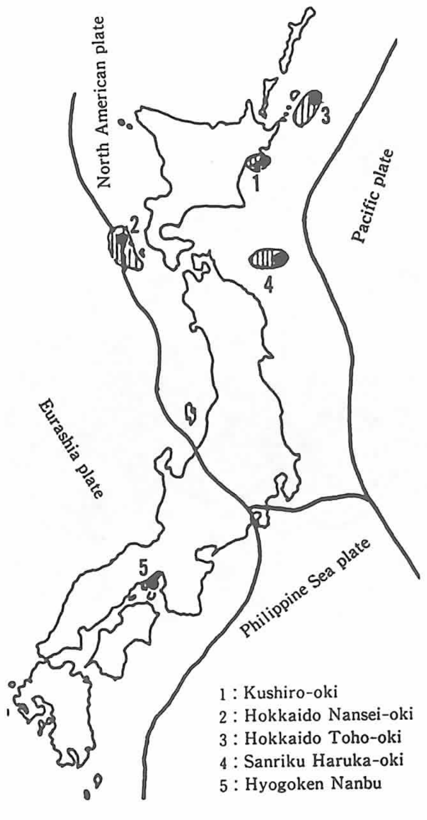


Fig. 1 Location of earthquake in Japan

(2) TYPES OF DAMAGE TO CONCRETE STRUCTURES

1) Earthquakes in Japan from 1993 to 1994

i) The Kushiro-oki earthquake

There was a great deal of significant damage to the supports of structures and deep piers (Photos 1 and 2). Particularly striking was the damage to relatively heavy T-shaped concrete girders. There was obvious damage due to displacement in the direction of the earthquake motion (Photos 4, 5, and 6). Cracks and other signs of damage were observed at the termination points of main reinforcing steel (Photo 3). This pointed to a clear need to further study the shear strength of columnar piers, since the maximum acceleration measured by strong motion recorders at the Kushiro Weather Observatory exceeded 900 gal [9].

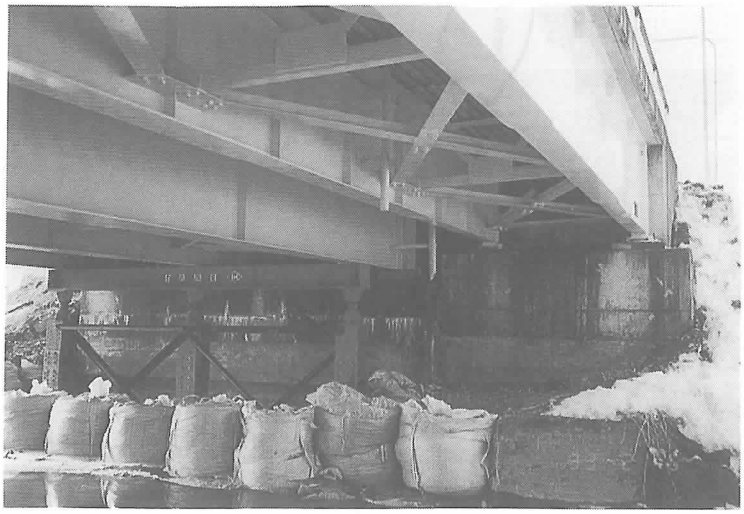


Photo-1 Movement of the abutment pier and shore protection (compound beam bridge, constructed in 1966)

Photo-2 Crack and inclination of abutment

Photo-3 Crack at 50 cm above reinforcing steel girder, between Chokubetsu steel section decreases point on and Atsunai abutment (T-beam of RC, between Ikeda and Shoei)

Photo-4 Broken shoe and concrete girder bridge (PCT: T beam of prestressed concrete bridge, between Hobetsu and Ikeda)

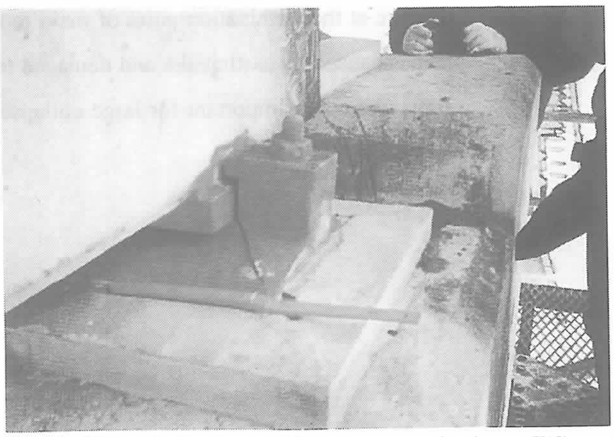


Photo-5 Broken side stopper of shoe (PC girder bridge, between Ineshi and Makubetsu)

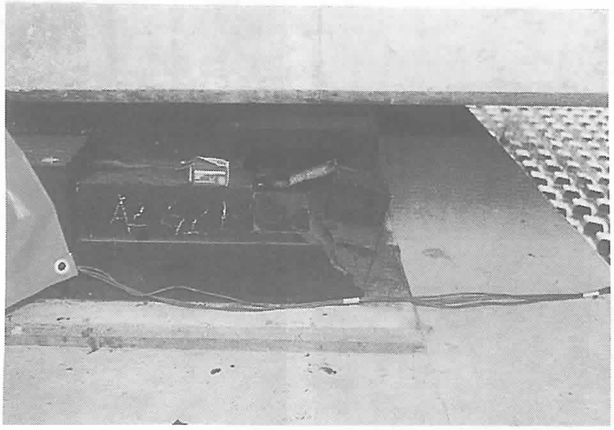


Photo-6 Broken shoe (PC box girder bridge, constructed in 1991, Urahoro)

ii) The Hokkaido Nansei-oki earthquake

Many road retaining walls and similar structures collapsed, and the tsunami caused displacement and movement of the abutment protection at a road bridge on Okushiri Island and Hokkaido mainland near the seismic center, destroying it (Photos 7-10, 12-13). The base walls of PC arch snow shelters and the superstructure of PC arches collapsed near the seismic center on Okushiri Island (Photo 11). Many damaged structures on the Hokkaido mainland were on the soft peaty, sandy soil of the low lying Kuromatsunai area in the south of the island. Cracks opened up in road surfaces as the filling behind abutments and bridge pier bases slipped away. Three bridges suffered damage at the termination point of main reinforcing steel in the RC piers [9]. Falling stones caused by earthquake and damaged to retaining wall (Photo 14 and 15). These signs were very important for large collapse of bedrock to tunnel accident in 1996 and 1997.

Photo-7 Moving and collapsing of retaining wall due to retreating tsunami (concrete retaining wall, Matsumae in Okushiri)

Photo-8 Damage to parapet and abutment (concrete girder bridge, Monai in Okushiri)

Photo-9 Deformation of steel plate covers of water supply pipe underneath caused by tsunami (concrete girder bridge, Shimamaki)

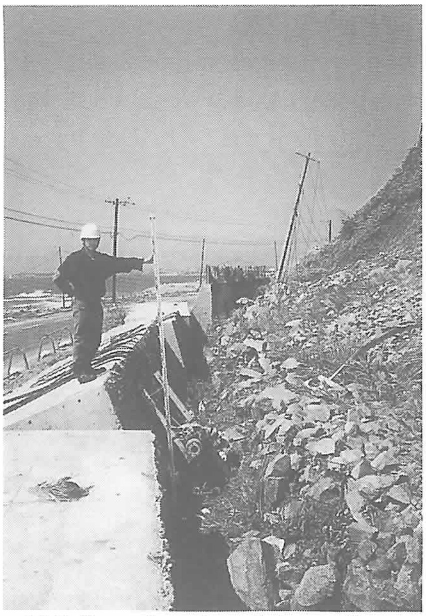


Photo-10 Collapsed road retaining wall caused by retreating tsunami (concrete retaining wall, Matsumae in Okushiri)

Photo-11 The collapsed snow shelter base wall of arch and the collapsed arch (PC arch structure Senjyozaka in Okushiri, just before completion in July 1993)

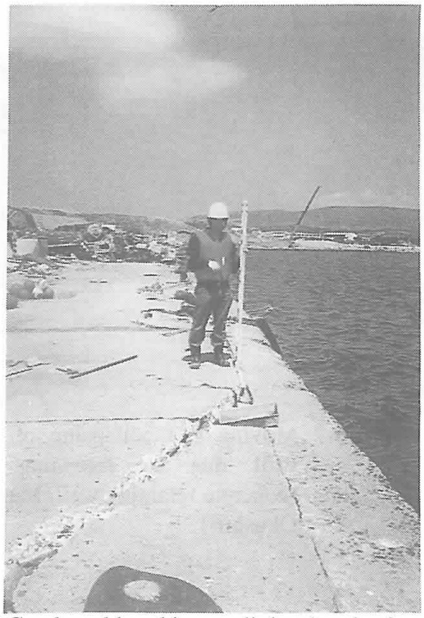


Photo-12 Crack and breaking on lighter's wharf by subsidence (concrete apron, Aonae in Okushiri)

Photo-13 Broken concrete of abutment (concrete girder bridge, Okushiri in Okushiri town)

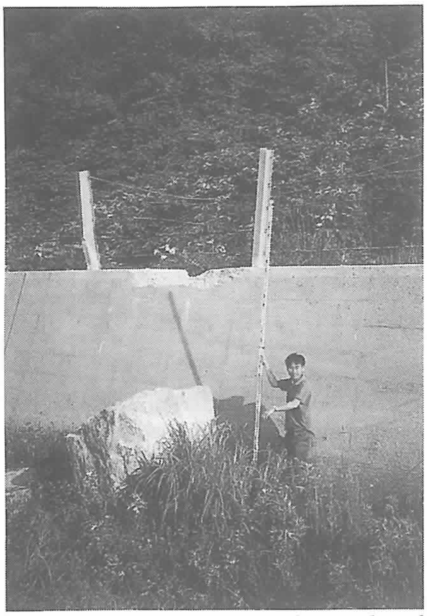


Photo-14 Falling stones caused by earthquake and damage to retaining wall (concrete retaining wall, Shimamaki)

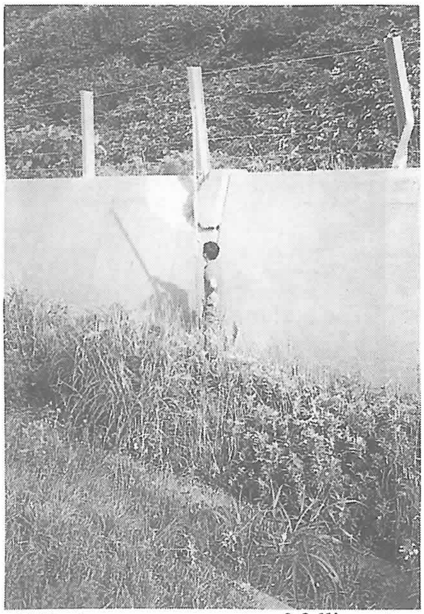


Photo-15 Broken root concrete of falling stone protection fall (concrete retaining wall, Shimamaki)

iii) The Hokkaido Toho-oki earthquake

Gaps opened up between the road surface level and the subbase course subsidence at bridge approaches and on bridge floor slabs. As a result of collapse of the subbase course, cracking, slipping, and bulging of sections of bridge abutments was noted in many places. The damage to a composite girder road bridge at Mannen in Bekkai Town was failure of the anchor support as well as cracking and scaling of the concrete abutment. In another case (a composite girder road bridge at Furen in Bekkai Town) failure and scaling of concrete, exposure of stirrup steel, and other damage was caused by collapse of the girder onto the abutment (Photo 16 and 17). In the case of a 3-span continuous box section road bridge with two main girders (at Midorimachi in Nakashibetsu town), failure of anchor supports, concrete cracking, and concrete scaling occurred (Photo 18).

A tidal wave protection wall suffered cracking at the concrete base for the gates as well as shear cracks in the wall itself (at the fishing port in Hanasaki, Nemuro City). This leads to questions about the wall's ability and harbor institutions to withstand a tsunami event [9] (Photo 19 and 20).

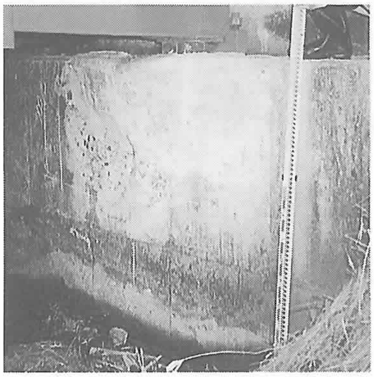


Photo-16 Failure of the anchor support, cracking, and scaling of the concrete abutment (road bridge, composite girder, Manmen in Bekkai town)

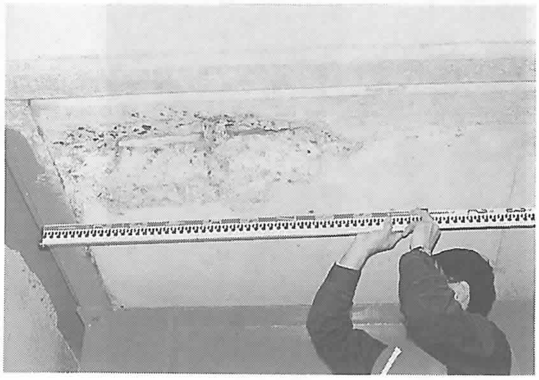


Photo-17 Failure and scaling of concrete, exposure of stirrup steel, and other damage were caused when the girder was crushed against the abutment (road bridge, composite girder, Furen in Bekkai town)

Photo-18 Failure of the anchor support, cracking, and scaling of the concrete abutment. The width of the abutment: 275 cm at the upper part and 460 cm at the lower part. (road bridge, 3 spans continuous, 2 main girders box section, Midorimachi in Nakashibetsu town)

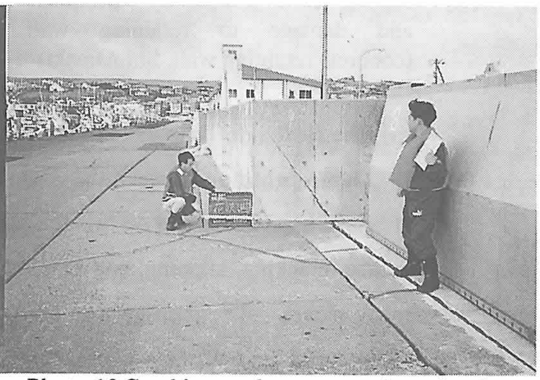


Photo-19 Cracking at the concrete base for the gates and shear cracks on the anti-tidal wave quaywall itself (Fishing port, Hanasaki in Nemuro city)

Photo-20 Wharf tilted and sank and broke the concrete plate of apron (fishing port, Habomai in Nemuro city)

iv) The Sanriku Haruka-oki earthquake

The most severe damage occurred characteristically along hide soft line which was northwest line from Matsugaoka to Ruike in Hachinohe City. The concrete piers of a steel road bridge built in 1931 (at Naganawashironai Funawatashi in Hachinohe City) were caused to tilt over by about 20 cm in the upstream direction, and the pier tops were crushed (Photo 21). Damage to the concrete piers and the bridge superstructure resulted from it coming into contact with a neighboring pedestrian bridge built in 1972. The motion of the bridge was restrained by the presence of the pedestrian bridge, leading to inclination of the bridge piers standing in the river channel.

The concrete structure of a high school building (the first floor of a 3-story high school building in Hachinohe City) was damaged by shearing of the pillars and walls where stirrups and longitudinal reinforcing steel were deformed to the extend that they reached yield. This structure had suffered from past large earthquakes: the Tokachi-oki earthquake (18th May, 1968; Magnitude 7.9) and the Kushiro-oki earthquake Several corroded sections of reinforcing steel were noted in sections damaged by the Sanriku Haruka-oki earthquake, indicating that cracks had been induced and reinforcing concrete weakened by the past large earthquakes. Some cracks certainly seemed to have been caused in earlier earthquakes [9] (Photo 22).

Institution of road and railroad were damaged on the northwest line (Photo 23). If the institutions such as transportation and lifeline construct crossing soft line, the carefulness designs are need.

Photo-21 Concrete pier of steel girder bridge tilted and damaged a part of concrete pier of pedestrian bridge by crashing onto each other (road bridge: constructed in 1931, pedestrian bridge: enlarge a construction in 1972. Kumanodou – Naganawa-shironai Funawatasi in Hachinohe city)

Photo-22 Concrete destroyed, broken stirrups, deformation and yield of longitudinal steel at recessed steel section past of RC pier (the first floor of 3-story high school building in Hachinohe city)

Photo-23 Collapse of a road embankment subbase caused an L-shaped concrete retaining wall to slide 3.5 m and subside by 2.5 m (at Matsugaoka in Hachinohe city)

2) The Hyogoken-Nanbu earthquake

Expressway piers and girders (of a PC bridge constructed in 1969 at Fukae Honjo-cho, Higashinada-ku, Kobe) failed and collapsed. Stirrups in RC columns failed, as did pressure welds in main reinforcing bars. The girders forming the superstructure were heavy, consisting of PC concrete, and the piers were high. It appeared that large perpendicular inertial forces resulted from the large amplitude of the earthquake motion. The bridge clearly reached its ultimate state (Photo 24).

Reinforced concrete rigid frame bridge piers of the Bullet Train trackbed (near the Mukogawa River in Nishinomiya City) suffered widespread and severe scaling of the overhead structure. Some piers were destroyed. Stirrups failed and the longitudinal reinforcing steel yielded and was deformed by the very strong inertial forces (Photo 25).

However, a nearby railroad bridge (a steel girder bridge over the Mukogawa River in Nishinomiya City) was scarcely damaged, despite having been in service for more than 60 years. This bridge was returned to service immediately after the earthquake. It had a short span and steel girder, and the low height of the piers means the inertial forces were probably small. Additionally, the girders were restrained in the direction of the bridge axis and perpendicular to it by the top of the concrete piers [9] (Photo 26).

Photo-24 Expressway piers and girders failed and collapsed (PC bridge constructed in 1969, at Fukae Bonjyochyo, Higashinada Ward in Kobe city)

Photo-25 Concrete destroyed, stirrups failed and the longitudinal reinforcing steel yielded and deformed at recessed part of RC pier (express railway bridge, PC girder, near Mukogawa river in Nishinomiya city)

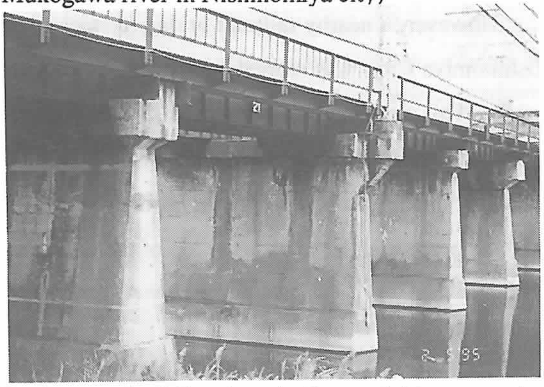


Photo-26 Undestroyed RC pier (railway bridge, steel girder, constructed in 1926, Mukogawa river in Nishinomiya city)

3) The Loma Prieta earthquake

This earthquake caused extensive damage to seven double-decker viaducts, including the collapse of the Cypress Street Viaduct (on I-880 in Oakland). Damage closed the San Francisco Oakland Bay Bridge for a month. Most of the bridges damaged in this earthquake were constructed before 1971, when construction standards were stiffened to reflect the lessons learned from the 1971 San Fernando earthquake. The greatest damage occurred to older structures on soft ground [3].

4) The Northridge earthquake

The seven major bridges that collapsed during the Northridge earthquake were all constructed to design standards much less stringent than those currently in use Caltrans. Many other bridges in the strong motion region sustained damage, but did not collapse; they remained in service, either full or limited. The damage ranged from minor cracking and spalling of the concrete to more severe damage that necessitated closing the bridge to traffic while repairs were made.

The damage to bridge piers at the I-10 La Cienega and Venice site, where cores completely disintegrated, hoop reinforcement ruptured, and longitudinal reinforcement buckled, suggests an explosive or brittle failure mode in some cases (Photo 2).

Almost vertical shear cracks and spalling of the cover concrete, as well as flexural cracks at column ends, in the case of the adjacent Fairfax and Washington ramps on I-10 clearly indicated the influence of column height and boundary conditions; the shorter columns fixed at both ends showed critical shear failure patterns.

Columns which had been retrofitted with circular steel jackets at the off ramp directly adjacent to Washington and Fairfax at Cadillac Avenue suffered only minor cracking and spalling in the joints between the steel jacket and the bottom soffit. These retrofitted columns survived the same ground accelerations that caused the above damage without sign of serious damage.

At the connector of interchange of the Sk-118 Mission and Gothic undercrossing, flexure/shear failure was visible in flared columns at the bottom of the flares. The flared columns of connector of interchange sustained ultimate state damage [3].

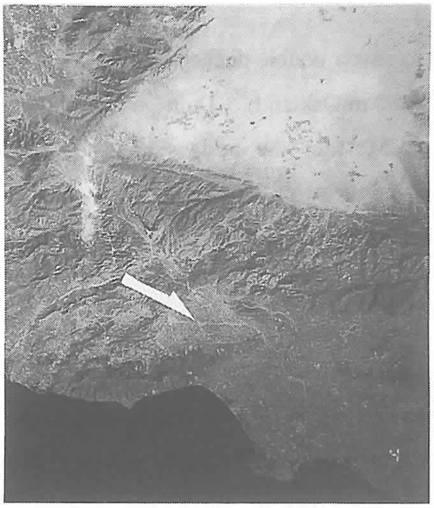


Photo-27 Epicenter of Northbridge earthquake in San Fernando Valley (the triangular basin at the lower center) in Pacific plate (mountainous part) and North American Plate (desert part)

Photo-28 Damaged bridge piers - the cores completely disintegrated at I-10 during the Northridge earthquake.

(3) SEISMIC DESIGN CRITERIA

In Japan, according to the specification of the Ministry of Construction, seismic safety in the event of ground motions equivalent to those developed by the Hyogoken-Nanbu earthquake is to be verified by dynamic response analysis, taking into account the nonlinearity of structural members, according to three ground types: stiff ground (I), moderate ground (II), and soft soil (III) [10]. In the provisional specifications drawn up by Japan Society of Civil Engineers [16], two types of earthquake motions were defined for use in earthquake resistant design. Level I earthquake motions would occur several times during the service life of a structure. The possibility of Level II earthquake motions occurring during the service life of a structure is few. Further, it is assumed that all three levels are adequate for earthquake-proof performance. The earthquake-proof performance for Level I is most excellent, in that function, soundness, and service without repair would be maintained.

In the U.S.A., after the San Fernando earthquake of February 9, 1971, Caltrans (in Sacramento) began comprehensively upgrading their Bridge Seismic Design Specifications and other seismic construction details. A major element of the improved seismic design specifications and procedures was the adoption of a site-specific approach. Maximum credible events on seismic faults throughout the State of California were used to define peak bedrock acceleration levels (Photo 3). This data is documented in CDMG (California Division of Mines and Geology) MAP Sheet 45.

Since the 1989 Loma Prieta earthquake, Caltrans has been implementing site-specific risk analysis to determine a probable design earthquake for major structures. This event is incorporated into the seismic design procedure along with the maximum credible event. Through a massive research program funded at $5.0 million annually the required confinement details have been developed to ensure ductile performance in a seismic event. These details have been tested with half-size laboratory models for confirmation of their ductile performance. They have also been introduced in newer and retrofitted bridges, and were field-tested in three moderate earthquakes (1992 Landers and Cape Mendocino; 1994 Northridge).

The excellent performance of bridges incorporating these newer design criteria and confinement details indicates to bridge designers that these structures can withstand larger earthquakes without collapse [7].

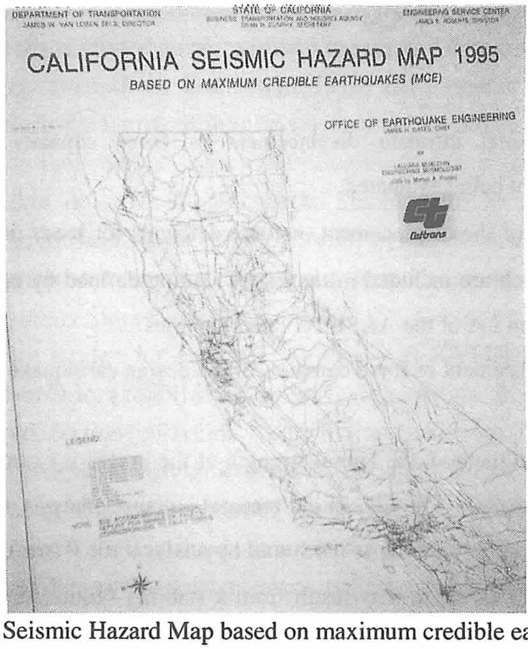


Photo-29 California Seismic Hazard Map based on maximum credible earthquakes by Caltrans

(4) EVALUATION METHOD

In Japan, the ductility (ability to displace) of the bridge as a whole is conventionally ensured by checking the lateral load sustained in an earthquake using the static lateral force (seismic coefficient) method. Ability to withstand the ground motions developed by the

Hyogoken-Nanbu earthquake must also be verified by carrying out a dynamic response analysis considering the nonlinear behavior of structural members. To check the dynamic strength and ductility of reinforced concrete piers, a new stress-strain relation for confined concrete has been introduced; the ultimate strain is defined as the strain at which the concrete strength decreases by 20% from its peak value [10, 13].

In USA, the seismic-retrofitting manual for highway bridges gives two methods for the detailed evaluation of existing bridges: the capacity/demand (C/D) ratio method and the lateral strength method.

In the C/D ratio method, a calculated C/D of less than 1.0 indicates the need for a retrofit. This procedure is thus based on an element-by-element evaluation rather than on the performance of a bridge as a single structural system.

The basic equation for determining the seismic C/D ratio, γ, is:

$$\gamma = \frac{R_c - \Sigma Q_f}{Q_{EQ}} \qquad \cdots\cdots \text{Eq. (1)}$$

where

R_c = The nominal ultimate displacement or force capacity for the structural component being evaluated.

ΣQ_1 = The sum of the displacement or force demands for loads other than earthquake loads which are included in the group loading defined by equations 6-1 and 7-1 of Division I-A of the AASHTO Specifications.

Q_{EQ} = The displacement or force demands of the design earthquake loading at the site

In the lateral strength method, the lateral strength of the bridge is examined as a system, or at least as individual segments. Through an incremental collapse analysis, the load-deformation characteristics of the bridge up to collapse are found by analysis for P (the column axial force) - $\triangle$ (displacement) effect. Collapse may result from a stability failure, such as when the P-$\triangle$ moment exceeds the residual capacity of the bridge columns.

The equation for determining the seismic risk from the limit-state spectral capacity, or spectral response, $k_{(r)}$, is:

$$k_{(r)} = \frac{S_a(V)}{S_a(\gamma)} \qquad \cdots\cdots \text{Equa. (2)}$$

where

$S\alpha(V)$: the spectral ordinate (base shear coefficient) corresponding to the equivalent elastic lateral strength Vs

$S\alpha(\gamma)$: the site-assessed response spectrum

(5) RETROFITTING METHOD

In Japan, the main purpose of seismic strengthening of reinforced concrete columns has been to increase shear capacity, especially in piers where longitudinal reinforcement is terminated with inadequate anchoring length under pre-1980 design specifications. The method suggested for doing this was seismic strengthening by adding steel jackets for a controlled increase in flexural strength. A steel jacket is added around the existing columns. Epoxy resin or non-shrinking concrete mortar is injected between the concrete surface and the steel jacket. A small gap is left at the pier base between the steel jacket and the top of the footing. Carbon fiber sheets are also effective where steel jackets cannot be placed due to limited pier spacing or the proximity of adjacent structures and where cranes cannot be used [11, 13].

In the U.S.A., the retrofitting of glass fiber, carbon fiber, and so on is being studied at the University of Southern California, the University of California at San Diego, and other places. Columns retrofitted with prefabricated glass fiber reinforced composites exhibit significantly improved seismic performance, and this validates the effectiveness of jacketing. Carbon is applied to bridge columns as a composite carbon shell system (Photos 4 and 5). The carbon shell system represents an innovative and cost-effective approach to bridge column design by taking advantage of the excellent mechanical properties of an advanced composite material, automated manufacturing processes, and fast field implementation [4] (Photos 6, 7, 8, and 9).

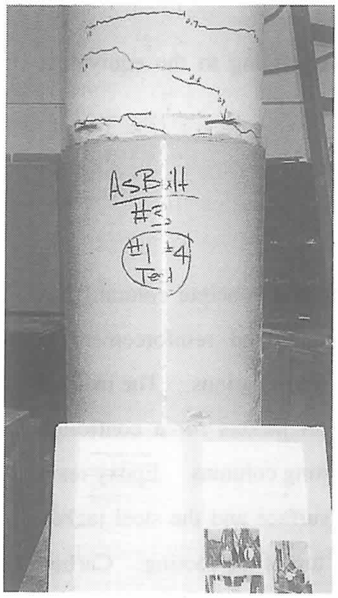
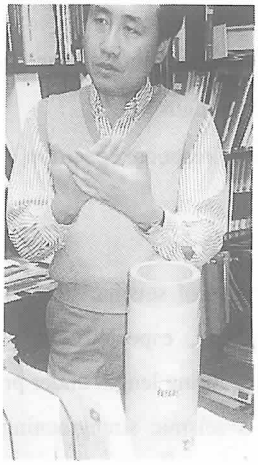


Photo-30 Retrofitting experiments in USC, adopting glass fiber by Professor Xiao



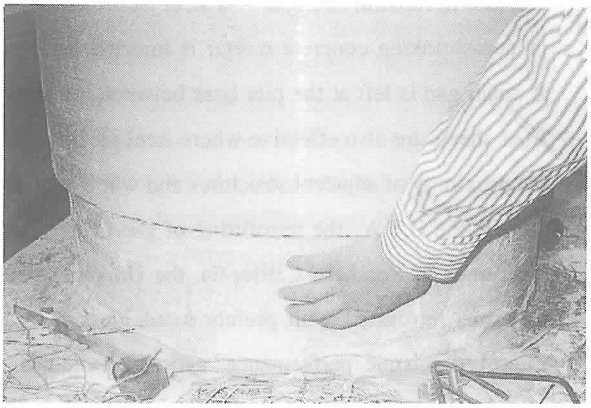


Photo-31 Retrofitting the part between the pier and base to retain original structural design

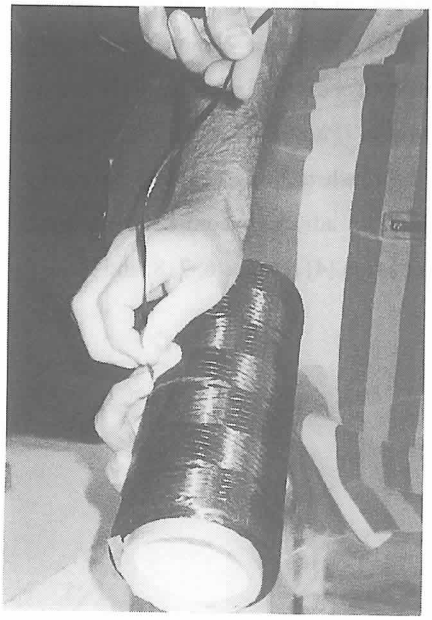


Photo-32 Carbon fiber material

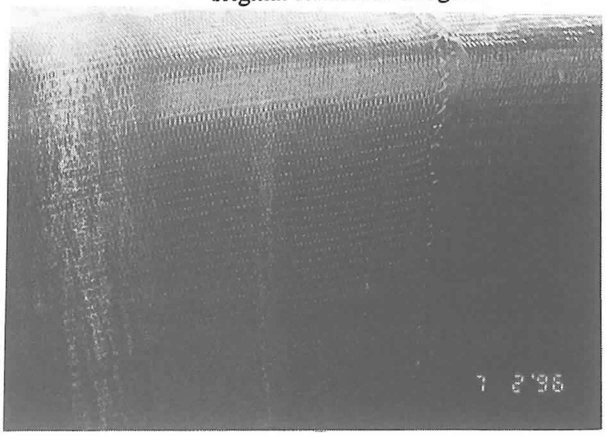


Photo-33 Weave-type reinforcement fiber

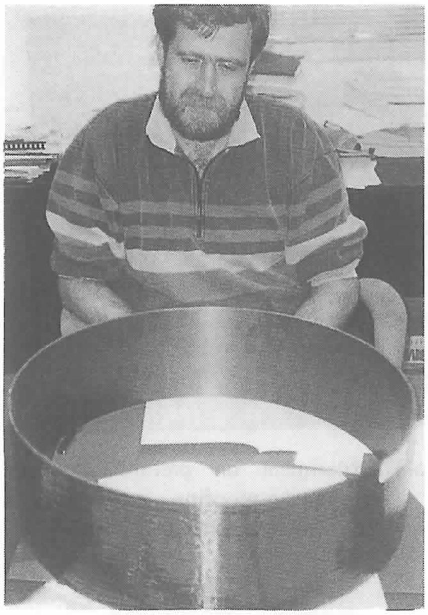


Photo-34 Section of carbon fiber shell jackets and Professor Seible of UCSD authority in retrofitting

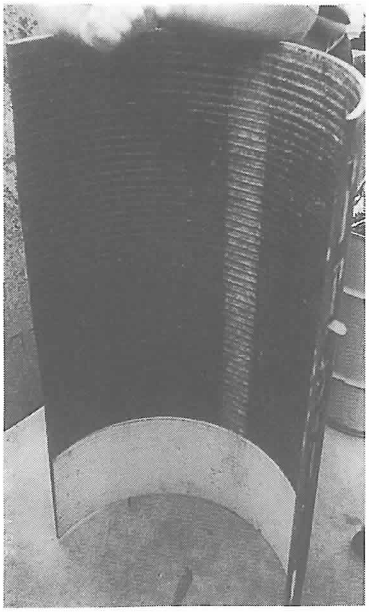


Photo-35 Shape of carbon fiber shell used to reinforce concrete pier

(6) EXPERIMENTS WITH LARGE SCALE SPECIMENS

The carbon fiber jacked retrofit system and its design methodology have been experimentally validated through large-scale (40% scale models of typical prototype columns) bridge column tests at the University of California, San Diego. A large-scale retrofit was tried on a shear bridge column with a steel reinforcement ratio of P_{as}=2.5% in an experiment directly analogous to a similar test conducted for Caltrans on "as-built" and steel-jacket retrofitted circular bridge column test specimens. The effectiveness of the design recommendations was verified through full-scale testing. Using the recommended design guidelines, it is possible to give the superstructure column connection the desired functionality as regards longitudinal seismic response without introducing inelastic behavior of the superstructure, which would require difficult post-earthquake repairs and resulting extended bridge closures [5] (Photos 10 and 11). Still more experiments on high-strength pier are being done in USC (Photo 12).

Photo-36 Magnified specimens of carbon fiber jacket retrofitting system used in USCD

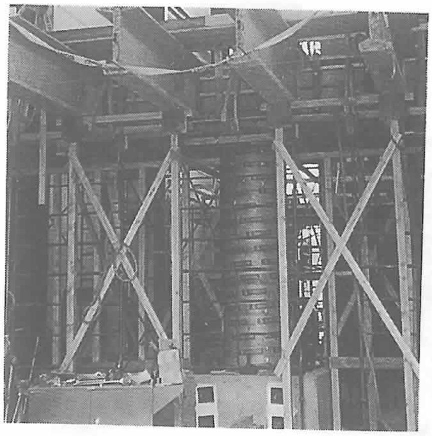


Photo-37 Experiment on prefabrication and earthquake-proofing system of highway structures in USCD

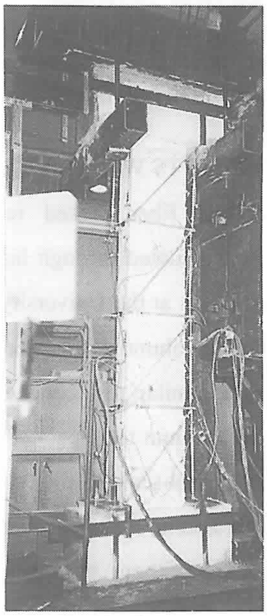


Photo-38 Experiment on high-strength concrete pier to USC

(7) SEISMIC ISOLATION AND ACTIVE CONTROL SYSTEMS

Isolation bearing systems can be expected to provide rigidity under service loadings, plus flexibility and damping under seismic loading. This usually means that the force-deflection characteristics are nonlinear. This is because, at small amplitudes, the isolators must be stiff to ensure that service loads are supported without the bridge being excessively "lively"; while at higher amplitudes, they soften to give the required flexibility to isolate the bridge during an earthquake. The systems currently being used for bridges and buildings include both rubber-based and friction-based systems. The majority of bridge applications are rubber-based, and the principal design being adopted in the U.S.A., Japan, and New Zealand is the lead-filled elastomer bearing. The selection of isolation hardware is an important decision, since both short-term and long-term performance is crucial. In the long term, reliability is essential [6]. The Menshin Design is close to a seismic isolation method, with the difference that energy-dissipating capacity is increased and lateral forces on the deck are distributed to the substructure [10, 13]. A test of its characteristics in cold conditions is currently underway at the University of California, Berkeley (Photos 13 and 14).

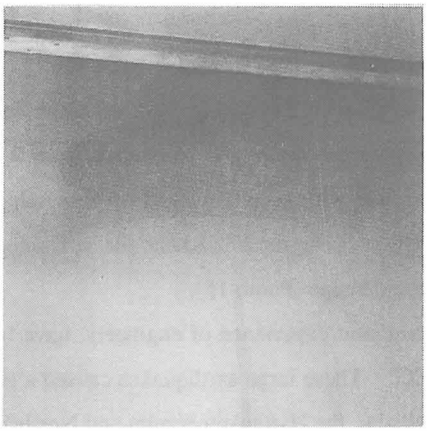


Photo-39 Cold condition testing of rubber bearing in UCA

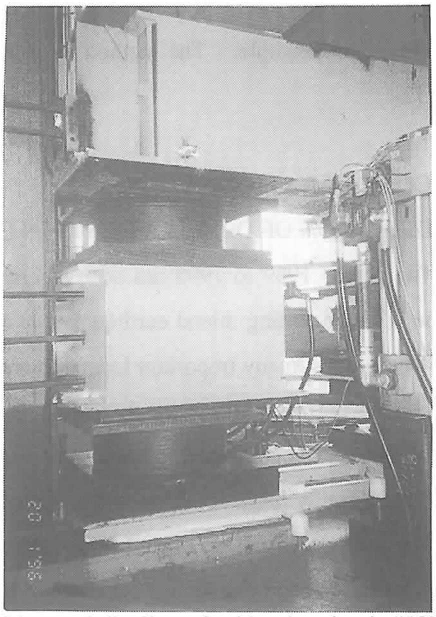


Photo-40 Cooling of rubber bearing in UCB

The experience of the Hyogoken-Nanbu earthquake has heightened interest in the development of active control systems that can also be effective during severe large-amplitude excitations. Active structural control has been successfully applied to commercial buildings as well as to ten bridges in Japan, but none in the U.S.A. so far. Yet even in Japan, few new projects which include active control systems are being initiated [8].

(8) APPLICATION OF RELIABILITY THEORY

It is important that methods of predicting earthquake risk using reliability theory take into account life-cycle cost, as pointed out by Professor M. Shinozuka of the University of Southern California. His study presents a framework for extending the traditional life-cycle cost analysis to include potential repair costs and associated user costs resulting from earthquake [1]. Though a large amount of data is needed to guarantee general applicability and reliability, analysis need to be applied in situations where the amount of data is really inadequate, as pointed out by Professor A. D. Kiureghian of University of California, Berkeley. His unique approach which accounts for all sources of model uncertainty and deals with the important issue of model-induced correlation has been presented. This therefore provides a reliable prediction for elastic concrete for an example. The method is general, and can be incorporated into building design codes [2].

3.2 CONSIDERATION

(1) DIRECTION OF DEVELOPMENTS IN EARTHQUAKE-PROOF TECHNIQUES

The period 1989 to 1995 has seen a series of large, record-breaking earthquakes in Japan and the USA, including inland earthquakes in urban areas and ranging in magnitude from 6.7 to 8.2. These days, many important locations are monitored with accelerometers for research into earthquake-proofing, so large peak ground accelerations were recorded near the epicenter of some of these earthquakes and in areas suffering severe damage (Photo 15).

Unexpectedly large accelerations, outside the common experience of engineers, have been recorded: peak accelerations ranged from 0.9G to 1.8G. These large earthquakes caused a great deal of severe damage to concrete structure. In particular, the Hyogoken-Nanbu and Northridge Earthquakes caused the failure and even collapse of many important structures such as elevated expressway structures, highway bridges, and others. The buildings that suffered the worst collapse are recognized as those built in Japan before 1980, when specifications were revised in response to the 1978 Miyagi-Oki Earthquake, and built before 1971 (and the specification revision occasioned by the San Fernando Earthquake) in the USA. However, structures built after these specification revisions also suffered damage - particularly the collapse of bridge girders due to failed supports or restraining devices, and due to liquefaction-induced ground displacement in reclaimed areas [14]. This type of disaster has caused unease in society as regards the safety and reliability of concrete structures. To improve confidence, the following investigations must be carried out:

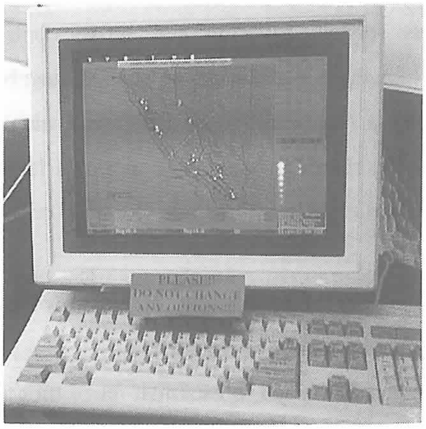


Photo-41 Setting points of accelerometer and the earthquake observation system (CUBE) in CALTECH

1) The maximum likely earthquake force assumed to act on a structure over its service life must be accurately worked out on the basis of the nearest faults. The input earthquake force must be considered at sufficient range, in view of the large acceleration which has been measured recently.
2) Recent results indicate that, as methods for evaluating the structural strength required to withstand earthquakes, the capacity/demand (C/D) ratio method for members, the lateral strength method, and the static lateral force (seismic coefficient) method with ductility (lateral displacement) which verifies dynamic response analysis considering nonlinear of structural members. Investigations are needed to compare the effectiveness of each method and to verify the structural strength to prevent collapse during large earthquake force reaching limited conditions.
3) Where retrofitting is indicated, the areas needing particular attention are where the main reinforcing steel terminates and where bridge piers have inadequate stirrups. As

a retrofitting method, steel jackets are commonly used. Another method is to use carbon fiber and glass fiber as reinforcing materials. In the present state, if there are subjects which are cost, characteristics of displacement and durability and so on, some type of retrofitting method must be developed to make use of each material characteristics and adaptation to site.

4) The design of supports and restrainers must be revised, since lack of strength in these areas has caused girders to collapse in many cases.
5) The idea of actively controlling earthquake forces must be investigated, using techniques such as seismic isolation and active control systems. Further, the conditions under which these systems can be applied must be examined carefully.
6) The risk imposed by a large earthquake must be evaluated using reliability theory, taking into account the return period of the assumed large earthquake. Further, the life-cycle cost of a structure must include the cost of earthquake management and maintenance.
7) To improve the reliability of retrofitting, a method of evaluating the soundness of structural concrete members must be established.
8) Strong financial support is needed to establish retrofitting research that offer adequate earthquake-proofing.

(2) SYSTEMS OF MAINTENANCE, MANAGEMENT, AND EARTHQUAKE-PROOFING

Concrete structures have to withstand the non-stationary effects of external forces such as earthquakes, wind, and so on, as well as the stationary effects of the deterioration of external factor and internal factor. Though the stationary effects of deterioration have recently been a lively area of study [15], the safety of deteriorated structures during earthquakes in its service has not been studied. However, examinations which include durability and earthquake-proofing are essential for the proper and safe maintenance and management of concrete structures. A procedure for evaluating both deterioration and earthquake-proofing, and the problems with such an evaluation, are considered in this section.

Photo-42 Stationary effects of the deterioration by external factor and internal factor to concrete structure in service year

Current ideas on the evaluation of concrete structure soundness include a recent plan for maintenance with deterioration prediction based on a guideline for the maintenance of concrete structures [15]. Some performance criteria cannot be evaluated because the relationship between each type of deterioration and the various performance criteria has not been clarified by studies carried out so far. This evaluation of the guideline is based on using a non-dimensional deterioration index to weight the factors which influence overall deterioration of the structure in the carrying out of maintenance.

In the case of new structures and new constructions, safety is increased by measures to increase ductility. In the case of existing structures, structures which have suffered salt damage, the alkali-aggregate reaction, and past earthquake damage were recognized in the damage caused by Sanriku-Haruka-Oki earthquake and Hyogoken-Nanbu earthquake. There were problems of how to evaluate the soundness of existing, concrete structures, the safety to earthquake-proof reflecting its soundness, and safety in the face of deterioration after retrofitting. In particular, the soundness of a member and its interior concrete itself, deterioration prediction, and inspection are important.

Another problem is that the department involved in maintenance and earthquake-proofing work independently. It is important to unify their evaluation systems in future. A flowchart for unifying evaluation and maintenance is shown below as Fig. 2, which takes into account both deterioration and earthquake-proofing. In this flowchart, an attempt is made to take into account the degree of deterioration and deterioration prediction in evaluating earthquake-proofing measures.

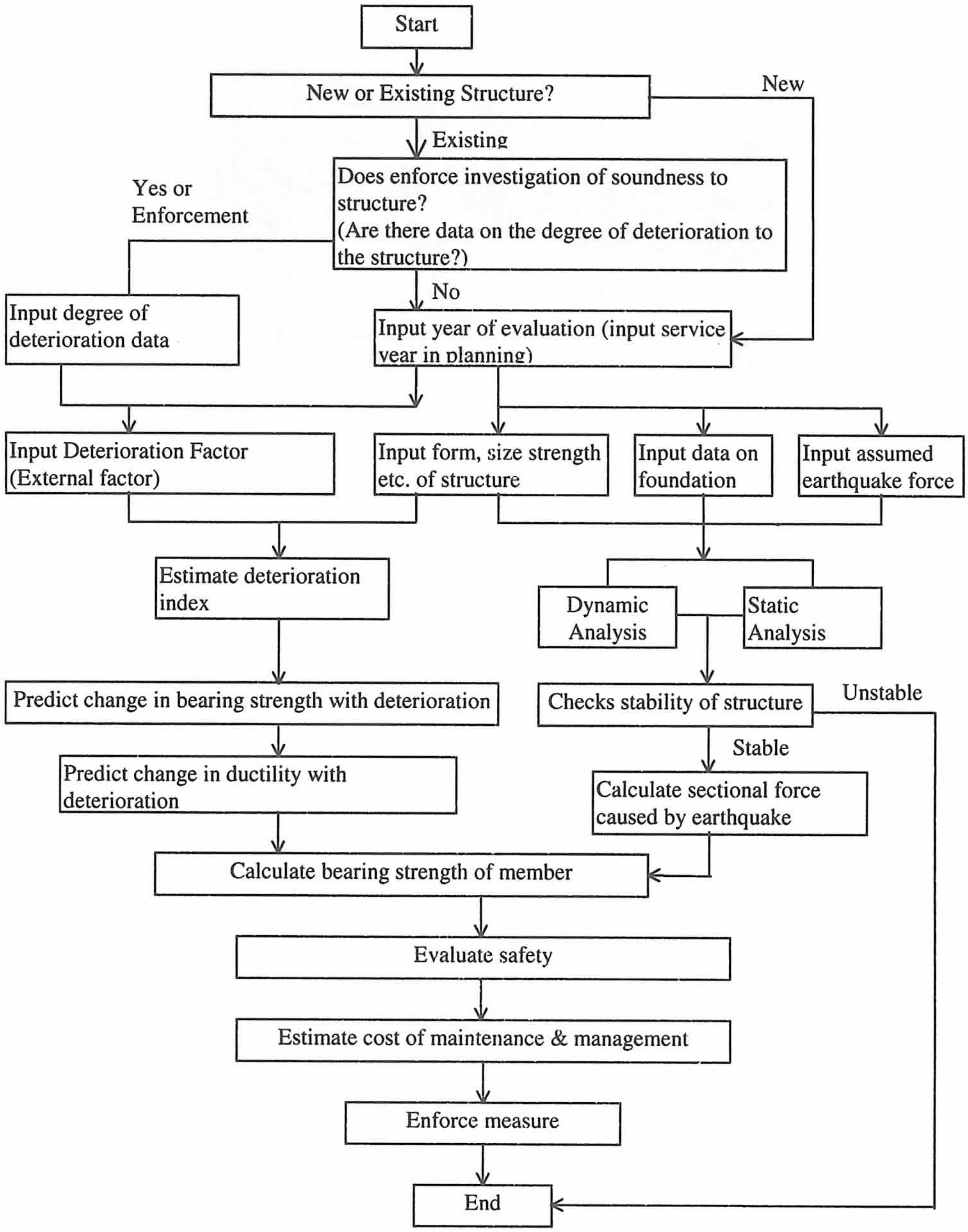


Fig. 2 Flowchart of evaluating for earthquake-proofing in consideration of structure.

4. CONCLUSION

The investigations of damage caused by the recent series of major earthquakes in Japan and the USA, and developments in technology to enhance the earthquake resistance of concrete structures, can be summarized as follows.

The recent major earthquakes - which are the Kushiro-Oki and Hokkaido Nansei-Oki in 1993, the Hokkaido Toho-Oki and the Sanriku Haruka-Oki in 1994, the Hyogoken-Nanbu in 1995, the Loma Prieta in 1989 and the Northridge in 1994 - are all large inland events in urban areas, and all were great disasters. They caused much damage to concrete structures.

Further studies are needed on the following:

1) Levels of seismic design criteria considering hazard and influence level during an earthquake and an evaluation method are needed to verify the structural strength to prevent collapse during large earthquake forces which exceed the limit condition of new and existing structures.
2) The method of retrofitting existing concrete structures and members by adopting innovative, lightweight and cost-effective materials such as glass fiber, carbon, etc.
3) The design of supports and restrainers to prevent girders from falling down should be revised. Additional seismic isolation and controls systems should be more actively applied to control earthquake forces.
4) The flow to take into account the degree of deterioration, deterioration prediction method, and the total life cycle cost of concrete structures in evaluating earthquake-proofing measures.

ACKNOWLEDGMENT

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