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# Invited Paper

# IMPACT OF HANSHIN/AWAJI EARTHQUAKE ON SEISMIC DESIGN AND SEISMIC STRENGTHENING OF HIGHWAY BRIDGES

Kazuhiko KAWASHIMA[1] and Shigeki UNJOH[2]

[1] Fellow Member of JSCE, Dr. of Eng., Professor, Department of Civil Engineering, Tokyo Institute of Technology O-okayama, Meguro-ku, Tokyo, 152,Japan

[2] Member of JSCE, Dr. of Eng.,Senior Research Engineer, Earthquake Engineering Division, Public Works Research Institute, Ministry of Construction, Tsukuba Science city, Ibaraki-ken, 305 ,Japan

The Hanshin/Awaji Earthquake (Hyogo-ken Nanbu Earthquake) of January 17, 1995 caused destructive damage to highway bridges. Obviously this was the first experience to suffer such destructive damage since the 1923 Great Kanto Earthquake. Various tentative measures were taken for seismic design of repair and reconstruction of highway bridges which suffered damage due to the earthquake. This paper summarizes the damage feature of highway bridges and a series of actions taken for seismic design and seismic strengthening of highway bridges in about half year since the earthquake.

*Key Words : Hanshin/Awaji Earthquake, Seismic Damage, Highway Bridges, Seismic Design, Seismic Retrofiting*

## 1. INTRODUCTION

Highway bridges in Japan had been considered safe even against extreme earthquakes such as the Great Kanto Earthquake (M7.9) in 1923, because various past bitter experiences have been accumulated to formulate the seismic design method. Large lateral force coefficient ranging from 0.2g to 0.3g has been adopted in the allowable design approach. Various provisions for preventing damage due to instability of soils such as soil liquefaction have been used. Furthermore, design details including the falling-down prevention devices have been adopted.

In fact, reflecting those provisions, numbers of highway bridges which caused complete collapse of the superstructures was only 15 since the 1923 Great Kanto Earthquake. Based on this evidence, it has been regarded by the engineers and researchers in bridge seismic engineering that the seismic damage of highway bridges had been decreasing in recent year (2).

The Hanshin/Awaji Earthquake took place at Kobe and Awaji Island on January 17, 1995, exactly one year after the Northridge, California, USA, Earthquake, and it caused destructive damage to highway bridges. Falling-down and nearly falling-down of superstructures occurred at 9 sites, and other destructive damage occurred at 16 sites. Destructive earthquakes have not occurred at urban areas in recent years, and the earthquake revealed that there are various critical issues in seismic design and seismic strengthening of bridges in urban areas.

After the earthquake, the "Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake (Hanshin/Awaji Earthquake)" was formulated in the Ministry of Construction to survey the damage and clarify the factors which contributed to the damage. The Committee was chaired by T. Iwasaki, Executive Director, Civil Engineering Research Laboratory.

On February 27, 1995, the Committee approved the "Guide Specifications for Reconstruction and Repair of Highway Bridges Which Suffered Damage due to the Hyogo-ken Nanbu Earthquake," (11) and the Ministry of Construction noticed on the same day that the reconstruction and repair of the highway bridges

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which suffered damage by the H/A Earthquake shall be made by the Guide Specifications. On March 30, 1995 , the Committee compiled the "Interim Report on the Damage of Highway Bridges by the Hyogo-ken Nanbu Earthquake." (13)

Although when the Guide Specifications was made, there was a plan to prepare a tentative seismic design method which applies to new construction and seismic strengthening of existing highway bridges until the Design Specifications of Highway Bridges (3) is revised, it was decided by the Ministry of Construction on May 25, 1995 that the Guide Specifications shall be tentatively used as an emergency measure for seismic design of new highway bridges and seismic strengthening of existing highway bridges until the Design Specifications of Highway Bridges is revised. Tentative measures provided in the Guide Specifications were considered appropriate for new construction and seismic strengthening.

In May, 1995, the "Special Sub-Committee for Seismic Countermeasures for Highway Bridges" was formulated in the "Bridge Committee" of the Japan Road Association. The Sub-Committee is chaired by K. Kawashima, Tokyo Institute of Technology, and its main role is to draft the revision of the Design Specifications of Highway Bridges. It is planned to complete the revision by late fall of 1996. Because there were various inquiries from field engineers on the application of the Guide Specifications to new construction and seismic strengthening, the Sub-Committee released on June 30, 1995 the "Reference for Applying the Guide Specifications to New Bridges and Seismic Strengthening." (4) It included several supplements on the way of application of the Guide Specifications as well as several design examples.

The Committee issued the "Report on the Damage of Highway Bridges by the Hyogo-ken Nanbu Earthquake" in December 1995 (14).

This paper summarizes the damage feature of highway bridges and the impact of H/A Earthquake on the seismic design and seismic strengthening.

## 2. REVIEW OF PAST SEISMIC DESIGN METHODS AND SEISMIC STRENGTHENING

The first seismic provision for highway bridges was introduced in 1926 after experiencing destructive damage by the 1923 Great Kanto Earthquake. Since the first stipulations, the seismic regulations have been reviewed and amended several times as shown in Table 1. The Design Specifications of Steel Highway Bridges was issued in 1939 and was revised in 1956 and 1964. Because few concrete bridges were constructed at those days, stipulations were provided only for steel bridges. Only the seismic lateral force of 20% of the gravity force was stipulated, and no other seismic design related provisions were presented in these Specifications. The 20% gravity force has been used as a basic design force since then.

The first comprehensive seismic design provisions were issued by the Ministry of Construction in 1971 in a form of the "Guide Specifications for Seismic Design of Highway Bridges." The 1964 Niigata Earthquake triggered to develop the Guide Specifications. It was described in the Guide Specifications that the lateral force shall be determined depending on zone, importance and ground condition in the static lateral force method (seismic coefficient method) and structural response shall be further considered in the modified static lateral force method (modified seismic coefficient method). Evaluation of soil liquefaction was incorporated in view of the damage caused by the 1964 Niigata Earthquake. Design detailings to increase the seismic safety such as the devices for preventing falling-down of a superstructure from substructures were introduced. Design methods for substructures were also unified between 1964 and 1971 in a form of the "Guide Specifications of Substructures." Therefore, it is considered that seismic safety was considerably increased in the highway bridges designed after 1971. The year of 1971 was important not only in US but in Japan from upgrading point of view for seismic design of highway bridges.

The 1971 Guide Specifications of Substructures and the 1971 Guide Specifications for Seismic Design were revised in 1980 in a form of the "Part IV Substructures" and "Part V Seismic Design" of the "Design Specifications of Highway Bridges". The Part V was essentially the same with the 1971 Guide Specifications for Seismic Design, but an updated evaluation method for predicting soil liquefaction as well as a practical design method for foundations in liquefying sands was included in the Part V.

The latest Design Specifications was issued in 1990 in a form of the "Part V Seismic Design" of the "Design Specifications of Highway Bridges"

Table 1 History of Design Methods for Highway Bridges

| Provisions | | 1926 1939 1956 1964(1) 1964(2) 1966 1968 1970 1971 1972 1975 1980 1990 |
|---|---|---|
| Seismic Coefficient Method | Seismic Coefficient | kh=0.2; kh=0.1- 0.35 (kh=0.2 in most bridges); kh=0.1- 0.3; MSCM was introduced; kh=0.1- 0.3; SCM and MSCM were unified; depend on zone; depend on zone and soil condition; depend on zone, importance, structural response and site condition |
| | Dynamic Earth Pressure | Mononobe-Okabe foumula; First provision was introduced |
| | Dynamic Hydraulic Pressure | Provision for hydraulic pressure; Provision for dynamic hydraulic pressure |
| Check of Dynamic Strength/Ductility of RC Piers | | kh=0.7-1.0 depending on zone, importance, structural response and site condition |
| RC Piers | Design for Flexure | Similar design with the current method had been adopted; First provision was introduced |
| | Design for Shear | Similar design with the current method had been adopted; First provision was introduced; Allowable stress was decreased |
| | Design for Termination of Main Reinforcement at Mid-height | Ancholing length was increased |
| Foundations | Footings | First provision was introduced; Effective width and check for shear |
| | Piles | Check for vertical strength; First provision was introduced; Check for vertical and lateral strength; Details for pile head; Check for lateral movement |
| | Direct Foundations | Similar design with the current method had been adopted; First provision was introduced; Check for bearing capacity, overturning and sliding |
| | Caisson Foundations | Similar design with the current method had been adopted; First provision was introduced |
| Soil Liquefaction | | First provision was introduced; Introduction of FL method; Treatment for Liquefaction |
| Bearing Supports | | Provisions for steel bearings; Details to supprot lateral force |
| Falling-down Prevention Devices | | Bearing Seat length S; Seat length S; Falling-down prevention devices; Seat length SE; Seat length SE |

1926: Design Details of Road Structures, Road Raw, 1939: Design Specs. of Steel Highway Bridges, 1956: Design Spec. of Steel Highway Bridges, 1964(1): Guide Specs. of Substructures (Pile Foundations), 1964(2): Design Specs. of Steel Highway Bridges, 1966: Guide Specs. of Substructures (Investigation and Design), 1970: Guide Specs of Substructures (Caisson Foundations), 1971: Guide Specs. of Seismic Design of Highway Bridges, 1972: Guide Spcs. of Substructures (Cast-in-place Piles), 1975: Guide Specs. of Substructures (Pile Foundations), 1980: Part V Seismic Design , 1990: Part V Seismic Design

(3,9). Various major revisions were included in the Part V. The first was a unification of the static lateral force method (seismic coefficient method) and the modified static lateral force method (modified seismic coefficient method). This included the revision of the lateral force coefficient. The second was the introduction of the check of dynamic strength and ductility for reinforced concrete piers. Depending on the failure mechanism, dynamic strength of reinforced concrete piers has been checked based on the ductility. This was the first practice to check the nonlinear behavior of bridges after yielding of structural members. Although this provision has not been mandatory for all reinforced concrete piers, this has been used to increase the ductility of piers. The third was the introduction of the static frame method to accurately evaluate lateral force of multi-span continuous bridges. This has enabled to consider three dimensional behavior of bridges in the equivalent static analysis. The fourth was the provisions for design response spectra for dynamic response analysis.

The Ministry of Construction made 5 nation-wide seismic inspections for existing highway bridges (2,7). The first seismic inspection was made in 1971 to detect deterioration such as cracks of reinforced concrete structures, tilting, sliding, settlement and scouring of foundations. Following the first seismic inspection, it was subsequently made in 1976, 1979, 1986 and 1991, by gradually expanding the items of inspection to detect from deterioration to vulnerability to cause failure during earthquakes. The highway bridges with span longer than or equal to 15m on all sections of national expressways, national highways and principal local highways, and overpass bridges were inspected. The items inspected included deterioration, devices for preventing falling-down of superstructure, strength of substructures and stability of foundations.

Evaluation for shear failure of reinforced concrete piers due to inadequate anchoring length of main reinforcements at mid-height was included since 1986 seismic evaluation. As will be presented in Section 4.2, this was one of the most important design practices which contributed to the damage. An evaluation method and seismic strengthening methods for this have been developed and upgraded at the Public Works Research Institute since the 1982 Earthquake. A series of dynamic loading tests were made to verify the seismic strengthening methods including the steel jacketing (7). The steel jacketing was implemented to strengthen the terminated points of reinforced concrete piers at the Metropolitan Expressway and Hanshin Expressway (7).

## 3. DAMAGE FEATURE

Damage was developed at highway bridges on Routes 2, 43, 171 and 176 of the National Highway, Route 3 (Kobe Line) and Route 5 (Bay Shore Line) of the Hanshin Expressway, and Meishin Expressway and Chugoku Expressway of Japan Highway Public Corporation. The damage of highway bridges was surveyed in 7 cities including Kobe city to clarify the general feature of damage. The number of piers surveyed reached 3,396.

Table 2 shows the Design Specifications referred to in design of bridges that suffered damage. It is classified in terms of the number of piers. It should be noted in Table 1 that most piers (bridges) which suffered damage were designed according to the 1964 Specifications or older Design Specifications. Only the Route 5, Bay Shore Line, of the Hanshin Expressway was designed by the 1980 Design Specifications (substructures) and 1990 Design Specifications (superstructures). As described in previous chapter, the seismic design methods have been improved and amended several times since 1926 based on the past damage experience and the progress of bridge earthquake engineering. However in the 1964 Specifications or older specifications, only a requirement for lateral force was described.

Table 3 shows the classification of damage of piers on the Route 3, Kobe Line and Route 5, Bay Shore Line of the Hanshin Expressway. It is classified in terms of the materials of piers (reinforced concrete piers or steel piers), pier type (single column or other types) and the damage degree as defined in Table 4(a). It is apparent that about 14% of the piers on Route 3 suffered the damage which was classified as the damage degree of $A_S$ and A, while no such destructive damage was developed in the piers on the Route 5. It should be noted here that at short natural period the intensity of ground shaking in terms of response spectra was smaller at the Bay Area than the narrow rectangular area where JMA Seismic Intensity was VII. It is seen that single columns suffered more damage than other types of piers in the Route 3, but no clear difference is seen in the Route 5.

Table 2 Design Specifications Referred to in Design (number of piers)

| Routes | | 1964 Specs. or Older | 1971 Specs. | 1980 Spec. | 1990 Specs. | Total |
|---|---|---|---|---|---|---|
| National Highways | Route. 2 | 43 (37%) | 72 (63%) | | | 115 (100%) |
| | Route 43 | 152(100%) | | | | 152 (100%) |
| | Route. 171 | 158 (100%) | | | | 158 (100%) |
| | Route. 176 | 13 (65%) | 2 (10%) | 5 (25%) | | 20 (100%) |
| | Sub. Total | 366 (82%) | 74 (17%) | 5 (1%) | | 445 (100%) |
| Hanshin | Route 3 | 890 (80%) | 216 (20%) | | | 1,106 (100%) |
| | Route 5 | | | 289 (84%) | 56 (16%) | 345 (100%) |
| | Sub. Total | 890 (61%) | 216 (15%) | 289 (20%) | 56 (4%) | 1,451 (100%) |
| JH | Meishin | 1,039 (100%) | | | | 1,039 (100%) |
| | Chugoku | 461 (100%) | | | | 461 (100%) |
| | Sub. Total | 1,500(100%) | | | | 1,500(100%) |
| Total | | 2,756 (81%) | 290 (9%) | 294 (9%) | 56 (2%) | 3,396(100%) |

Table 3 Classification of Damage of Piers of Hanshin Expressway
(number of piers)

(a) Route 3, Kobe Line

| Type of Piers | | Damage Degree | | | | | Total |
|---|---|---|---|---|---|---|---|
| | | As | A | B | C | D | |
| Steel | Single Columns | 2 (4%) | 8 (15%) | 3 (6%) | 32 (60%) | 8 (15%) | 53 (100%) |
| | Other Types | 1 (1%) | 0 (0%) | 9 (8%) | 80 (73%) | 20 (18%) | 110 (100%) |
| | Sub. Total | 3 (2%) | 8 (15%) | 12 (7%) | 112 (69%) | 28 (17%) | 163 (100%) |
| Reinforced Concrete Piers | Single Columns | 50 (7%) | 69 (9%) | 85 (12%) | 199 (27%) | 329 (45%) | 732 (100%) |
| | Other Types | 14 (7%) | 9 (4%) | 17 (8%) | 26 (12%) | 145 (69%) | 211 (100%) |
| | Sub. Total | 64 (7%) | 78 (8%) | 102 (11%) | 225 (24%) | 474 (50%) | 943 (100%) |
| Total | Single Columns | 52 (7%) | 77 (10%) | 88 (11%) | 231 (29%) | 337 (43%) | 785 (100%) |
| | Other Types | 15 (5%) | 9 (3%) | 26 (8%) | 106 (33%) | 165 (51%) | 321 (100%) |
| | Sub. Total | 67 (6%) | 86 (8%) | 114 (10%) | 337 (30%) | 502 (45%) | 1,106 (100%) |

(b) Route 5, Bay Shore Line

| Type of Piers | | Damage Degree | | | | | Total |
|---|---|---|---|---|---|---|---|
| | | As | A | B | C | D | |
| Steel Piers | Single Columns | | | | | 6 (100%) | 6 (100%) |
| | Other Types | | | 13 (9%) | 21 (15%) | 103 (75%) | 137 (100%) |
| | Sub. Total | | | 13 (9%) | 21 (15%) | 109 (76%) | 143 (100%) |
| Reinforced Concrete Piers | Single Columns | | | 1 (1%) | 2 (2%) | 93 (97%) | 96 (100%) |
| | Other Types | | | | 20 (19%) | 86 (81%) | 106 (100%) |
| | Sub. Total | | | 1 (1%) | 22 (10%) | 179 (89%) | 202 (100%) |
| Total | Single Columns | | | 1 (1%) | 2 (2%) | 99 (97%) | 102 (100%) |
| | Other Types | | | 13 (5%) | 41 (17%) | 189 (78%) | 243 (100%) |
| | Sub Total | | | 14 (4%) | 43 (12%) | 288 (83%) | 345 (100%) |

Table 4 Definition of Damage Degree

(a) Piers

| Damage Degree | Definition |
|---|---|
| As | Collapse, Extensive damage to lose bearing capacity |
| A | Extensive cracks, Rupture and large outward buckling of main reinforcements |
| B | Local outward buckling of main reinforcements and large cracks of concrete, Local buckling of webs and flanges |
| C | Drop-off of cover concrete and slight cracks, Residual deformation of webs and flanges |
| D | No damage or minor damage |

(b) Foundations

| Damage Degree | Definition |
|---|---|
| a | Large lateral movement as well as large settlement |
| b | Large lateral movement, or some flexural cracks on piles |
| c | small flexural cracks on piles |
| d | No damage or minor damage |

(c) Superstructure

| Damage Degree | Definition |
|---|---|
| As | Falling-down of superstructure |
| A | Major damage of main members to cause loss of bearing capacity such as rupture of lower flange in steel girders and extensive drop-off of concrete in concrete girders |
| B | Moderate damage of main members to cause loss of bearing capacity such as deformation of lower flange of steel bridges and large cracks of concrete in concrete girders |
| C | Damage of secondary members |
| D | No damage or minor damage |

(d) Bearing Supports

| Damage Degree | Definition |
|---|---|
| A | Rupture of set bolts or anchor bolts, Large failure of sole plates or boss, Failure of crest of piers/abutments where bearings are placed |
| B | Rupture of pins or stoppers at upper bearings, Pulling out of rollers or anchor bolts, Failure of stoppers, Failure of mortar placed underneath bearings |
| C | Large deformation of upper or lower bearings, Set bolts which came loose, Deformation of stoppers, Cracks on mortar and concrete of crest of piers/abutments |
| D | No damage or minor damage |

Totally 340 foundations were surveyed as shown in Table 5 in the bridges on Routes 3 and 5 of Hanshin Expressway, Hamate By-pass of Highway 2, and Meishin and Chugoku Expressways. Foundations surveyed were carefully selected based on the type of foundations and bridges, damage degree of piers and superstructures, and locations. In spread (direct) foundations, damage was directly checked by excavating soils up to the surface of footings. Because direct excavation was difficult for pile foundations, caisson foundations and wall foundations, bore-hole camera systems were used. After boring from the surface of footing to piles, a small bore-hole camera was inserted to monitor damage around the boring hole inpiles.

Because damage degree of foundations was much less than other structural components, the damage was classified as shown in Table 4 (b) by small letters. There were no foundations which corresponded to damage degree of "a."

Table 6 shows the damage degree of foundations. In spite of destructive damage of piers and superstructures, the damage of foundations was small at the Route 5, Bay shone Line and Route3, Kobe Line of Hanshin Expressway. This was true for not only foundations supporting piers which suffered destructive damage but also foundations

Table 5 Survey of Damage of Foundations (number of foundations)

| | | Direct Foundations | Caisson Foundations | Pile Foundations | Wall Foundations | Total |
|---|---|---|---|---|---|---|
| Route 3 Kobe Line | Sub-Total | 133 | 44 | 929 | 0 | 1,106 |
| | Surveyed | 12 | 2 | 109 | 0 | 123 |
| Route 5 Bay Sore Line | Sub-Total | 0 | 52 | 280 | 13 | 345 |
| | Surveyed | 0 | 8 | 153 | 5 | 166 |
| National Highway 2 Hamate By-pass | Sub-Total | 0 | 15 | 57 | 0 | 72 |
| | Surveyed | 0 | 5 | 20 | 0 | 25 |
| Meishin and Chugoku Expressways | Sub-Total | 152 | 0 | 532 | 0 | 684 |
| | Surveyed | 5 | 0 | 21 | 0 | 26 |
| Total | Sub-Total | 285 | 111 | 1,798 | 13 | 2,207 |
| | Surveyed | 17 | 15 | 303 | 5 | 340 |

Table 6 Damage of Foundations (number of foundations)

| Routes | Damage Degree | | | | |
|---|---|---|---|---|---|
| | a | b | c | d | e |
| Route 3, Kobe Line | 0 (0%) | 0 (0%) | 17 (16%) | 92 (84%) | 109 (100%) |
| Route 5, Bay Shore Line | 0 (0%) | 17 (11%) | 57 (37%) | 79 (52%) | 153 (100%) |
| National Highway 2, Hamate By-pass | 0 (0%) | 0 (0%) | 10 (50%) | 10 (50%) | 20 (100%) |
| Meishin / Chugoku Expressways | 0 (0%) | 0 (0%) | 0 (0%) | 21 (100%) | 21 (100%) |
| Total | 0 (0%) | 17 (5%) | 84 (28%) | 202 (67%) | 303 (100%) |

supporting piers which suffered minor damage. At the Route 5, Bay Shore Line, about 11% of the surveyed foundations suffered the damage of rank "b," and this was larger than the Route 3, Kobe Line. This was due to large lateral spreading of soils associated with soil liquefaction (14). Foundations consisting of cast-in-place piles of 1.5 m in diameter moved as large as about 1 m at the Route 5. However even at such sites, damage of piles was only some residual cracks with as wide as 4 mm. Flexural cracks did not concentrate at top, but rather distributed at upper parts and the haigh where soil stiffness changed.

Table 7 shows the damage of superstructures on the Route 3 and 5 of the Hanshin Expressway. It was classified in terms of the number of spans. Therefore, if a bridge was of three spans, it was counted as three in Table 7. The damage degree was defined as shown in Table 4(c). It is seen in Table 7 that most of the significant damage of superstructure developed around the bearing supports. The falling-down prevention devices suffered damage not only in the devices but at the portions of decks where the devices were connected to.

Table 8 shows the damage of bearing supports. They were counted by number of *support lines*. When a deck is supported by 4 bearings at one end, those 4 bearings were counted as 1 support line. If the damage degree was different in bearings in a support line, the most highest damage degree was assigned to this support line. In 4,773 steel bearing support lines, 986 bearing support lines suffered damage classified as the damage degree of A. Damage of elastomeric bearing was much less. Table 9 classifies the damage of 4,773 steel bearing support lines in terms of their types. It is apparent that pin/ roller type bearings suffered the most destructive damage. Damage of bearing plate type and pivot type was also significant.

It is interesting to note that how the damage of bearings was coupled with the damage of piers. It is often pointed out that bearing may be a kind of "fuse" to limit excessive lateral force from superstructures to substructures. In fact, there were several sites where this seemed to be true. If this was true, destructive damage of piers should not be developed at the piers where bearings failed, i.e., the more destructively damaged bearings were, the smaller the damage of piers should be. Table 10 compares the damage degree of piers and bearings. It is seen that the damage degree of piers was almost independent of the damage degree of bearings. This means that the "fuse" assumption

Table 7 Damage of Superstructures of Hanshin Expressway (number of spans)

| Part of Superstructure | Damage Degree | | | | |
|---|---|---|---|---|---|
| | A | B | C | D | Total |
| Surroundings of Bearings | 112 (7%) | 142 (8%) | 62 (4%) | 1,364 (81%) | 1,680 (100%) |
| Portions where Falling-down Prevention Devices are Connected | 4 (0%) | 24 (2%) | 38 (3%) | 1,317 (95%) | 1,382 (100%) |
| Deck Itself | 1 (0%) | 5 (0%) | 2 (0%) | 1,672 (100%) | 1,680 (100%) |
| Others | 26 (14%) | 29 (16%) | 12 (6%) | 120 (64%) | 187 (100%) |
| Total | 143 (3%) | 200 (4%) | 114 (2%) | 4,473 (91%) | 4,930 (100%) |

Table 8 Damage of Bearings (number of support lines)

| Type | Damage Degree | | | | Total |
|---|---|---|---|---|---|
| | A | B | C | D | |
| Steel Bearings | 986 (21%) | 603 (13%) | 681 (14%) | 2,503 (52%) | 4,773 (100%) |
| Elastomeric Bearings | 0 (0%) | 6 (2%) | 19 (8%) | 219 (90%) | 244 (100%) |
| Total | 986 (20%) | 609 (12%) | 700 (14%) | 2,722 (54%) | 5,017 (100%) |

Table 9 Damage of Steel Bearings (number of support lines)

(a) Fixed Bearings

| Type | Damage Degree | | | | Total |
|---|---|---|---|---|---|
| | A | B | C | D | |
| Bearing Plate | 133 (13%) | 146 (14%) | 126 (12%) | 634 (61%) | 1,039 (100%) |
| Pin | 86 (34%) | 28 (11%) | 23 (9%) | 116 (46%) | 253 (100%) |
| Pivot | 12 (12%) | 9 (9%) | 45 (44%) | 36 (35%) | 102 (100%) |
| Line | 32 (9%) | 17 (5%) | 55 (15%) | 269 (72%) | 373 (100%) |
| Total | 263 (15%) | 200 (11%) | 249 (14%) | 1,055 (60%) | 1,767 (100%) |

(b) Movable Bearings

| Type | Damage Degree | | | | Total |
|---|---|---|---|---|---|
| | A | B | C | D | |
| Bearing Plate | 479 (26%) | 239 (13%) | 224 (12%) | 909 (49%) | 1,851 (100%) |
| Roller | 238 (30%) | 141 (18%) | 152 (19%) | 257 (33%) | 788 (100%) |
| Line | 6 (2%) | 23 (6%) | 56 (15%) | 282 (77%) | 367 (100%) |
| Total | 723 (24%) | 403 (13%) | 432 (14%) | 1,448 (48%) | 3,006 (100%) |

Table 10 Comparison of Damage Degree between Piers and Bearings

| | | Damage Degree of Piers | | | | | Total |
|---|---|---|---|---|---|---|---|
| | | AS | A | B | C | D | |
| Damage Degree of Bearings | A | 10 (3%) | 34 (10%) | 48(14%) | 157(44%) | 106(30%) | 355(100%) |
| | B | 7(3%) | 21(8%) | 33(12%) | 106(39%) | 104(38%) | 271(100%) |
| | C | 5(1%) | 21(6%) | 22(6%) | 148(39%) | 183(48%) | 379(100%) |
| | D | 48(5%) | 82(8%) | 110(10%) | 324(31%) | 490(46%) | 1,054(100%) |
| | Total | 70(3%) | 158(8%) | 213(10%) | 735(36%) | 883(43%) | 2,059(100%) |

cannot be adopted. Actual interaction mechanism of bearing failure and pier failure seems to be more complex. Collapse of a bearing support line would cause an increase of lateral force to adjacent piers, and thus successive failure would be developed. Dislodgment of decks from their supports after bearings failed would cause extremely large lateral force from the decks to piers, because the failed surface of bearings and the failed surface of decks sometimes rocked. Therefore it is required to design bearings keeping in mind that bearing is one of the important structural components of bridges. Because lateral force tends to build up in steel bearings until their failure, it is preferable to adopt elastomeric bearings and Menshin bearings. It is important to allow relative displacement to occur between piers and superstructures.

Besides the above evidence, the followings were identified as the feature of the damage:

(1) Intensity of ground motions recorded was extensive, and was much larger than the seismic force anticipated in design.

(2) Shear failure associated with inadequate anchoring length of main reinforcements terminated at mid-height was one of the major types of damage of reinforced concrete piers. This will be described in more detail in the next chapter.

(3) Local bucklings of web and flange plates progressed to result in rupture of welding at corners in rectangular steel piers. This caused total loss of bearing capacity in vertical direction.

(4) Liquefaction occurred at coarse sand and gravel sites where potential to develop liquefaction has been considered limited. Lateral spreading was developed associated with soil liquefaction.

## 4. TYPICAL DAMAGE OF HIGHWAY BRIDGES

### 4.1 General

An "Interim Report on Damage of Highway Bridges by the H/A Earthquake" (13) was issued on March 30, 1995 by the "Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake." It was finalized in December 1995 in a form of "Report on Damage of Highway Bridges by the H/A Earthquake" (14). They showed the failure mechanism of highway bridges based on extensive field and laboratory tests, check and verification of original designs, and linear and nonlinear dynamic response analyses.

The table of contents of the Report are as;



The damage feature presented in Chapter 3 is from the Report. Some analyses on the failure mechanism are described in the followings.

### 4.2 Damage of Highway Bridges at Fukae, Route 3, Hanshin Expressway

The bridges collapsed in the most critical way as shown in Photos 1 and 2. There were 18 spans that totally collapsed. The bridges completed in 1969. They were designed by the 1964 Design Specifications of Steel Bridges, and 0.2 horizontal and ±0.1 vertical seismic coefficients were used

Photo 1 Collapse of 18-span Bridges from Kobe Side, Fukae, Route 3, Kobe Line, Hanshin Expressway

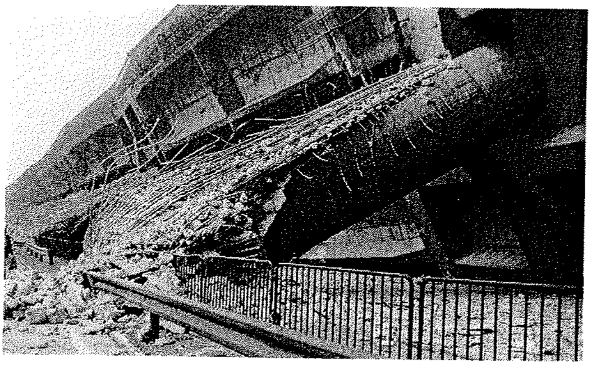


Photo 2 Collapse of 18-span Bridges from Osaka Side

based on the allowable design approach. At those days, only seismic coefficient was stipulated in the Design Specifications, and no other seismic design related issues were not presented. The bridges were of mushroom shaped slab bridges, i.e., the decks were of two hinged prestressed concrete decks with a span length of 22m, and the decks were supported by pilz-type columns as shown in Fig. 1. One side of the deck was fixed to a pilz column and the other side was supported by an adjacent pilz column allowing a relative displacement in longitudinal direction. At the joints, prestressed cables were provided to prevent excessive relative movement in longitudinal direction, while reinforced concrete shear connectors were provided to restrain relative movement in transverse direction. Because the same construction process could be used in the pilz type structures, it was beneficial for period and cost of construction.

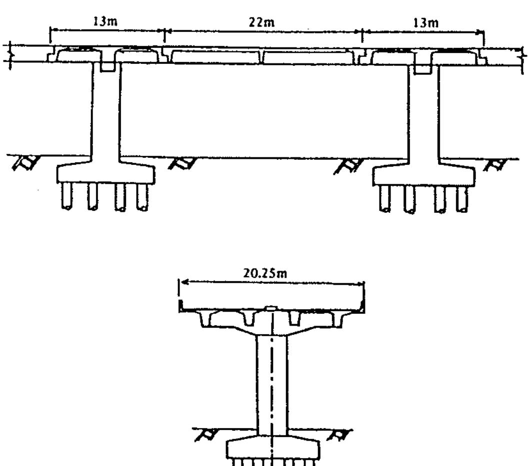



Fig. 1 18 Span Highway Bridges at Fukae, Route 3, Kobe Line of Hanshin Expressway

The columns had diameter of 3.1 m and 3.3 m with heights varied from 9.9 m (Kobe side) to 12.4 m (Osaka side). The design concrete strength of columns and footings was 270 $kgf/cm^2$. The deformed bars with diameter of 35 mm (SD30, D35) and the deformed bars with diameter of 16 mm (SD30, D16) were used for longitudinal and tie reinforcements in the columns. Number of longitudinal reinforcements was 180 at the bottom of columns, and it was reduced to 120 at 2.5 m from the bottom by terminating 60 reinforcements.

The footings were supported by 10-15 m long cast-in-place reinforced concrete piles with diameter of 1m. The soil was of sand and gravels, and was classified Type II ground condition (moderate) based on the current Design Specifications of Highway Bridges.

There were two important problems in such design. First was the allowable stress used in design. The allowable stress at those days for a load combination of seismic effect and dead weight was 135 $kgf/cm^2$ for concrete and 2,700 $kgf/cm^2$ for reinforcements. This has been used without change. The important point was the allowable stress for shear strength of concrete. Because reinforced concrete piers with large concrete sections had mostly been adopted, check for shear stress of concrete was not so important at those days. It was 1966 when the stipulations on the shear strength of concrete was incorporated at the first time in the Guide Specifications of Substructures. Prior to 1966, when the check of shear stress was required, it was made by referring to the Standard Specifications of Concrete by the Japan Society of Civil Engineers issued in 1931, 1940, 1949 and 1956 (5). In the JSCE Standard Specifications, the allowable shear strength of concrete was provided in two ways, i.e., if it is assumed that only concrete section resists for shear force, the allowable shear stress of concrete should be $\tau_{a1}$, and if it is assumed that both concrete and reinforcements resist for shear force, the allowable stress of concrete should be $\tau_{a2}$. The allowable stresses $\tau_{a1}$ and $\tau_{a2}$ depend on the design strength of concrete, and were 7 $kgf/cm^2$ and 20 $kgf/cm^2$, respectively, for concrete with design strength of 240 $kgf/cm^2$. Those values were used until JSCE Standard Specifications was revised in 1980, and $\tau_{a1}$ was reduced to from 7 $kgf/cm^2$ to 4.5 $kgf/cm^2$ for concrete with design strength of 240 $kgf/cm^2$. As described above, in the same year, the Design Specifications of Highway Bridges incorporated $\tau_{a1}$ and $\tau_{a2}$ by referring to the JSCE Standard Specifications. A little conservative allowable stresses $\tau_{a1}$ and $\tau_{a2}$ than those specified in the JSCE Standard Specifications were specified in the Design Specifications of Highway Bridges, and they were 3.9 $kgf/cm^2$ and 17 $kgf/cm^2$ for concrete with design strength of 240 $kgf/cm^2$, and 4.2 $kgf/cm^2$ and 18 $kgf/cm^2$ for concrete with design strength of 270 $kgf/cm^2$. This revision was made reflecting the evidence that significant damage was developed in reinforced concrete piers by the 1978 Miyagi-ken-oki Earthquake.

Table 11 shows the check calculation of a column (Column No. 138). It is seen that the shear stress was very large compared to the allowable shear stress specified in the current Design Specifications. Therefore, if the same column was designed in accordance with the current Design Specifications, the column should have larger diameter.

to the JSCE Standard Specifications of Concrete. It was stipulated in the JSCE Standard Specifications that reinforcements had to be anchored by either providing "enough" anchoring length or hocks/mechanical terminators. Because there was no stipulations on what was the "enough" anchoring length, the stipulations on splice length was alternatively used. The splice length depended on type and strength of reinforcements and bond stress, and was about 20 times diameter of reinforcements. Therefore, only 20 times diameter of longitudinal reinforcements had been adopted until 1980.

However, because reinforced concrete piers with large concrete sections were adopted in most cases, this did not become problem in those days. Such problem was first noticed in 1978 Miyagi-ken-oki Earthquake when several reinforced concrete piers suffered damage at the terminated points (18). More attention has been paid on this problem since 1982 when the Shizunai Bridge suffered significant damage at piers by the

Table 11 Check of Stress induced in a Column (Column No. 138) based on the Design Specifications Referred to in Original Design

| Check Point | | Bottom | | Terminated Point | |
|---|---|---|---|---|---|
| Horizontal Seismic Coefficient | | 0.2 | | | |
| Vertical Seismic Coefficient | | +0.1 | -0.1 | +0.1 | -0.1 |
| Force | Bending Moment (tfm) | 3,714 | 3,714 | 3,198 | 3,198 |
| | Axial Force (tf) | 1,619 | 1,325 | 1,588 | 1,300 |
| | Shear Force (tf) | 347 | 347 | 341 | 341 |
| Longitudinal Reinforcements | | D35x180 (1,722 $cm^2$) | | D35x120 (1,148 $cm^2$) | |
| Stress Induced ($kgf/cm^2$) | Concrete (Compression) | 123 | 122 | 125 | 125 |
| | Reinforcements | 1,827 | 1,990 | 2,014 | 2,246 |
| | Concrete (Shear) | 4.6 | 4.6 | 4.5 | 4.5 |
| Allowable Stress ($kgf/cm^2$) | Concrete (Compression) | 135 | | | |
| | Reinforcements | 2,700 | | | |
| | Concrete (shear) | 11.25 | | | |

Second was the anchoring length of longitudinal reinforcements of piers terminated at mid-height. Because bending moment of columns decreases with height, a part of longitudinal reinforcements were terminated at several heights to gradually decrease the number of reinforcements. They were terminated at 2.5 m from the bottom in the Column 138. A problem was the short anchoring length when the longitudinal reinforcements were terminated. Because there were no clear stipulations on the anchoring length in the Guide Specifications of Substructures until 1980, it was common to refer

Urakawa-oki Earthquake (17).

In the 1980 Design Specifications of Highway Bridges, it was stipulated that from the height where longitudinal reinforcements can be terminated based on a design calculation, longitudinal reinforcements had to be elongated with a length equivalent to an effective width of a column plus the splice length (about 20 x diameter of reinforcements). If this requirement was applied to the Column 138, the longitudinal reinforcements terminated at 2.5 m from the bottom had to be further elongated with a length of 3.1 m.

Table 12 Flexural and Shear Strength of Column (Column No. 138)

| Section | Py (tf) | uy (cm) | Pu (tf) | uu (cm) | $\mu$ | Ps (tf) |
|---|---|---|---|---|---|---|
| Terminated Point | 486 | | 661 | | | 780 |
| Bottom | 540 | 3.68 | 741 | 11.76 | 3.20 | 945 |

Extensive field study was made to investigate the strength of concrete, reinforcements and pressure weld. Test pieces were taken from both concrete surface of the damaged columns and concrete blocks which broke off on ground from the columns. The average strength of the concrete pieces directly taken from the surface of columns was 421 $kgf/cm^2$, while it was 327 $kgf/cm^2$ in the concrete pieces taken from the concrete blocks which broke off on ground. Because the concrete blocks which broke off on ground experienced high stress during the earthquake, their strength does not represent the original concrete strength of columns. Because the design strength of concrete was 270 $kgf/cm^2$, the concrete strength did not have a problem.

The tensile test for longitudinal and tie reinforcements was made. Because the most of reinforcements experienced yielding during the earthquake, clear yielding plateau was not detected. The averaged yield strength and ultimate strength was 3,590 $kgf/cm^2$ and 5,600 $kgf/cm^2$ in longitudinal reinforcements and 3,620 $kgf/cm^2$ and 5,793 $kgf/cm^2$ in tie bars, respectively, while the yielding and ultimate strength required in design was >3,000 $kgf/cm^2$ and 4,900-6,300 $kgf/cm^2$ for SD 35. Therefore, there were no problem in the strength of reinforcing bars.

Tensile test of longitudinal reinforcements, which were pressure welded and were not ruptured at the welded portion, was made for 32 specimens. They were taken from various collapsed columns. Because most of the reinforcements experienced yielding during the earthquake, clear yielding plateau was not detected. About half of the specimens ruptured at the reinforcing bars and the remaining about half ruptured at the welded points. However, the tensile strength was over the tensile strength required in design for SD 35 (4,900-6,300 $kgf/cm^2$) in 27 specimens among the 32. In the remaining 5 specimens, it was 70-89 % of the tensile strength required.

The tensile strength was also studied for the longitudinal reinforcements which failed at the pressure welded points. Reinforcements close to the failed welding points were used for the test. The averaged strength of reinforcements was 5,750 and they were over the required strength (4,900 $kgf/cm^2$-6,300 $kgf/cm^2$) in all the specimens. In the most of specimens, the yield plateau was not observed, and reduction of elastic modulus was observed due to Baushinger effect. From those evidence, although rupture was developed at pressure welded points, they performed well at least until the strength level required for reinforcements in design was reached.

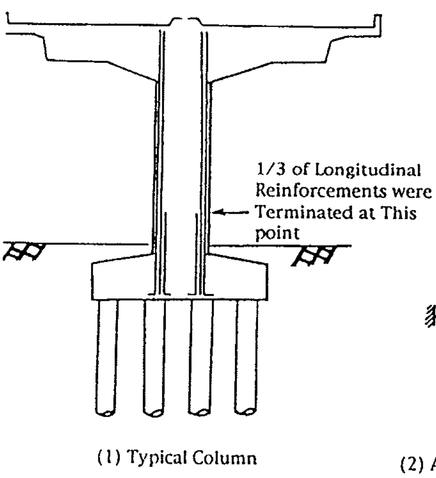



Fig. 2 Analytical Model of Pier 138, Fukae Bridges

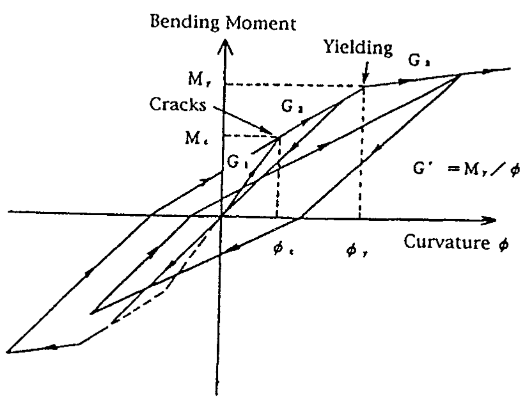



Fig. 3 Takeda Model for Nonlinear Hysteresis of Piers

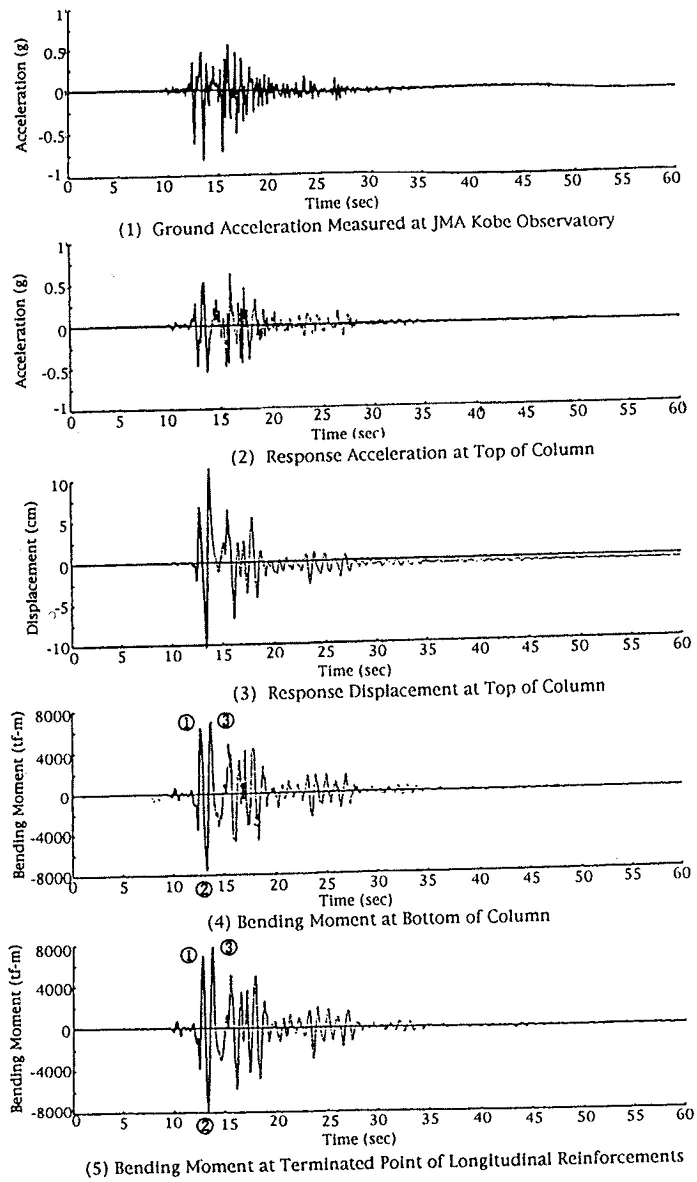



Fig. 4 Response of Pier 130 subjected to JMA Kobe Observatory Record

A series of equivalent linear and nonlinear analyses were made to clarify the response of the bridges. The yield strength $P_y$, yield displacement $u_y$, ultimate strength $P_u$, ultimate displacement $u_u$ and shear strength $P_s$ were evaluated based on the current "Part V Seismic Design" of the "Design Specifications of Highway Bridges" as shown in Table 12. It should be noted that $P_y$ and $P_u$ represent the lateral force at the center of gravity of a deck to develop the yielding and ultimate bending moment in a column. The strength and the shear strength of concrete was assumed 350 $kgf/cm^2$ and 8 $kgf/cm^2$, based on the measured data. Scattering of the measured data was taken into account by reducing one standard deviation from the mean value. The yielding strength of reinforcing bars was assumed 3,500 $kgf/cm^2$. Table 7 compares the strength of a typical column (Column No. 138) at the bottom and 2.5 m from the bottom where number of main reinforcements (SD30, D35mm) was reduced

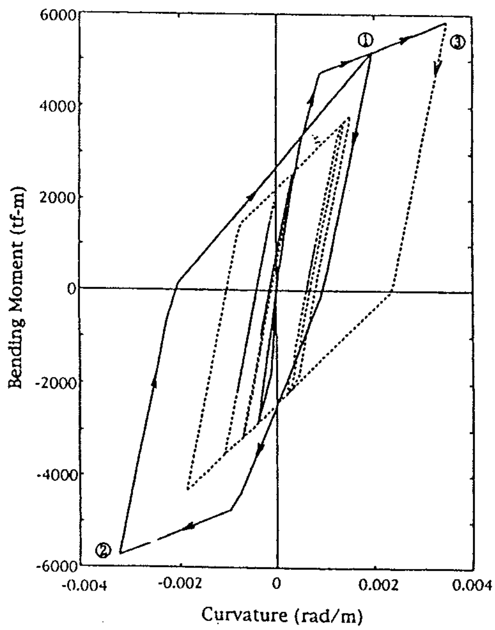



Fig. 5 Moment vs. Curvature Hysteresis of Pier 138

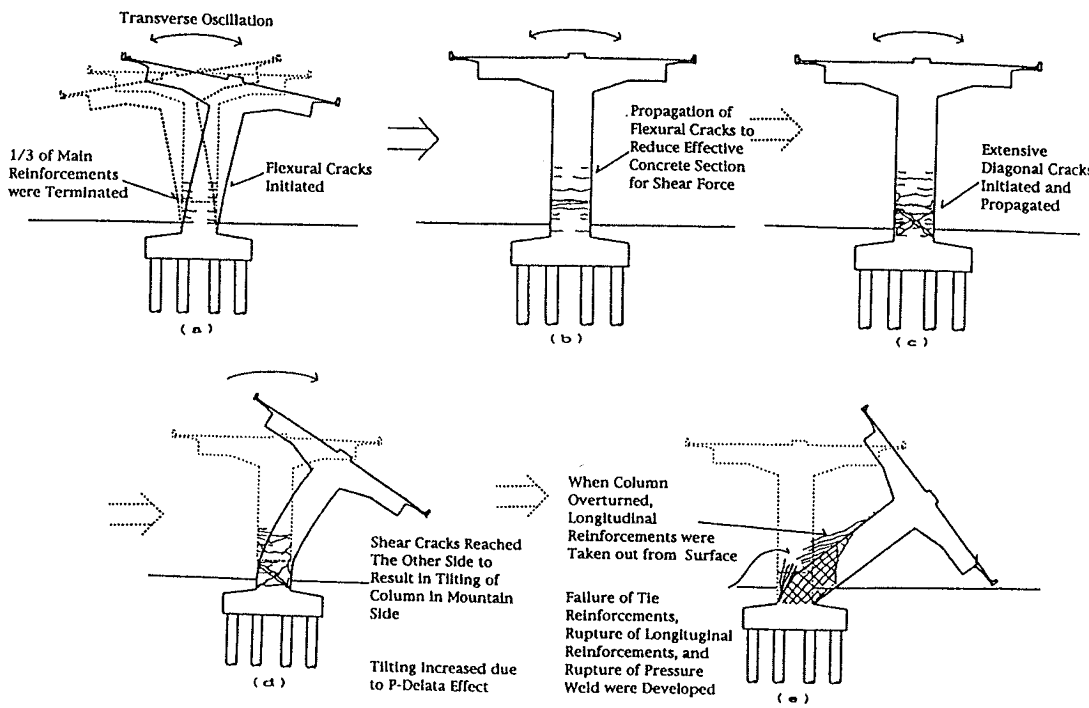



Fig. 6 Failure Mechanism of 18 Span Highway Bridges at Fukae, Route 3, Kobe Line of Hanshin Expressway

from 180 to 120. It is apparent that both the flexural strength and the shear strength are smaller at the terminated point than at the bottom.

From a linear analysis of 18 span bridges, it was found that the fundamental natural period was about 0.75 second.

Fig. 2 shows an analytical model of a typical column (Column No. 138). The effect of foundation was idealized by a set of equivalent linear springs. Takeda model (20) as shown in Fig. 3 was assumed to represent the nonlinear flexural hysteretic behavior of the column. Fig. 4 shows the response of the columns, and Fig. 5 shows the moment vs. curvature hysteresis. It is seen in Fig. 5 that at the height where 1/3 of longitudinal reinforcements were terminated, first yield was developed at about 13 second (point ①). Then unloading occurred to reach the point ② where curvature ductility reached about 3.2, and continued to move to point ③. It may be difficult to predict further response by the Takeda model because it represents flexural nonlinearity. Therefore, the response after the point ③ was plotted by dotted lines in Figs. 4 and 5. Based on the loading tests conducted at the Public Works Research Institute, it is known that shear failure tends to be triggered by the flexural cracks at the terminated points when the anchoring length was inadequate (7).

Based on the nonlinear analyses, it may be considered that the failure of the bridges proceeded as shown in Fig. 6. At first, the column caused extensive flexural cracks at 2.5 m above the footing where 1/3 of the total longitudinal reinforcements were terminated without enough anchoring length (Fig. 6 (a)). The flexural cracks first propagated horizontally (Fig. 6 (b)) and then they turned to diagonal cracks (Fig. 6 (c)). As the loading increased, concrete failed extensively along the diagonal cracks and the column initiated to tilt to the Rokko Mountain side (Fig. 6 (d)). Thus the P-delta effect became predominant and finally overturned in one side (Fig. 6 (e)). Rupture of tie bars and longitudinal reinforcements, and failure of pressure welding probably occurred during this process.

**4.3 Damage of Highway Bridges at Takashio, Route 3, Hanshin Expressway**

Two simply supported steel girder bridges were dislodged from their seats and fell down as shown in Photo 3. They had span length of 40 m and 30 m as shown in Fig. 7. The bridges completed in 1979. They were designed by the 1971 Guide Specifications for Seismic Design, and 0.23 horizontal and $\pm 0.11$ vertical seismic coefficients were used based on the allowable design approach.

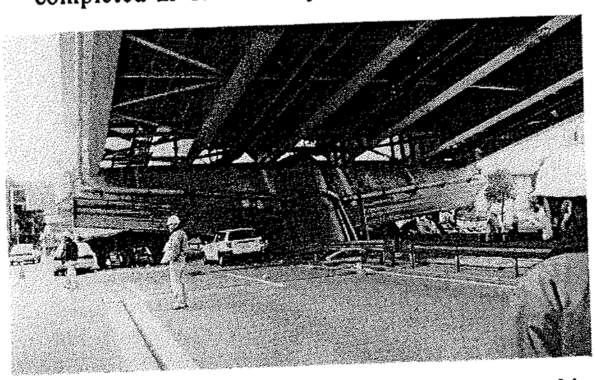


Photo 3 Failure of Reinforced Concrete Pier, Takashio

The decks were supported by single columns with a diameter of 2.8 m and 11.7 m high. The soil was of sand with gravels and silty material, and it was classified as Type II ground condition (moderate) based on the current Design Specifications of Highway Bridges. The design concrete strength of columns and footings was 270 $kgf/cm^2$. Deformed bars with diameter of 35 mm (SD30, D35) and 16 mm (SD30, D16) were used for longitudinal and tie reinforcements in the columns. Number of longitudinal reinforcements was 150 at the foot of the columns, and it was reduced to 120 and 60 at 3.3 m and 5.7 m from the foot, respectively. The anchoring length at the terminated longitudinal reinforcements met with the requirements by the 1971 Design Specifications, but it was not adequate. The footing was supported by cast-in-place concrete piles with diameter of 1 m and 14 m long.

Extensive analyses, similar to the Highway Bridges at Fukae, were made including nonlinear dynamic response analyses. Similar to the Bridges at Fukae, it was found from the field and laboratory tests that the strength of concrete, reinforcements and pressure welding met with the requirements by the 1968 Guide Specification on Design of Abutments and Piers. It was also found from a check calculation of the original design that the computed stresses of columns were less then the allowable stresses at those days, while redundancy of the shear stress was smaller by the current Design Specifications.

Fig. 8 shows the estimated failure mechanism from the nonlinear dynamic response analysis. It was similar to that at the Fukae Bridges.

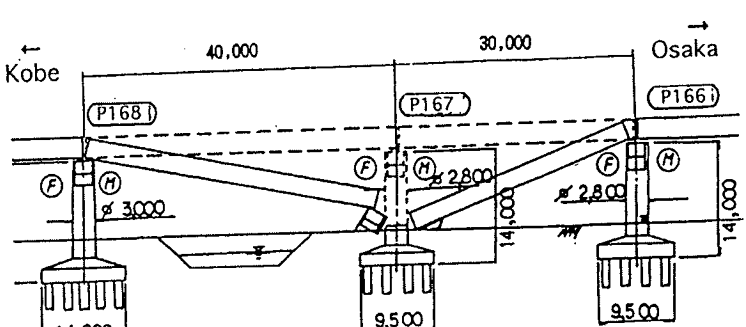



Fig. 7 Highway Bridges at Takashio, Route 3, Kobe Line of Hanshin Expressway

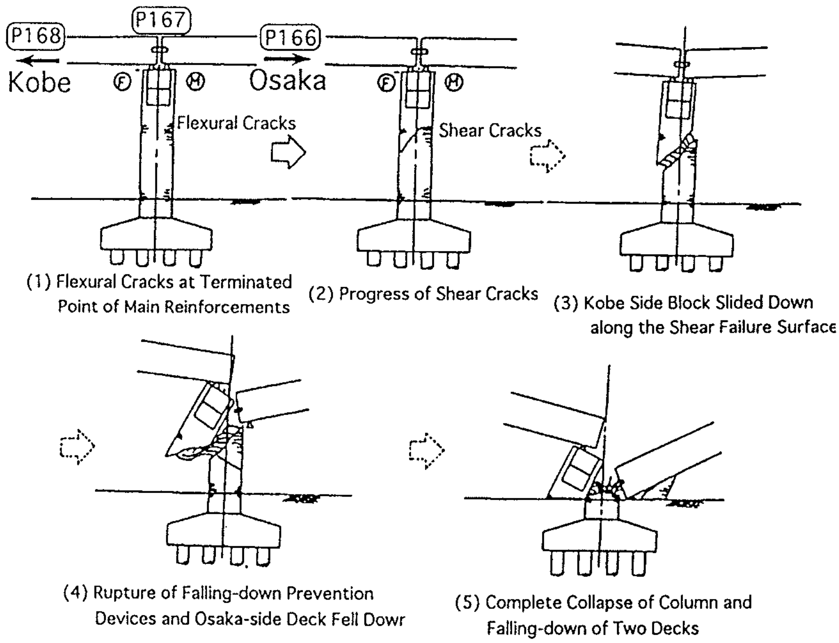



Fig. 8 Failure Mechanism of Highway Bridges at Takashio, Route 3, Kobe Line of Hanshin Expressway

**4.4 Damage of Highway Bridges at Tateishi Crossing**

Steel columns were first damaged by the H/A Earthquake as shown in Photo 4. Similar damage was developed at Iwaya Crossing of National Highway 43. A steel column supported two simply supported composite steel deck girders with a span length of 45 m and 70 m. The column had a section as shown in Fig. 9. The bridge completed in 1969. They were designed by the 1964 Design Specifications of Steel Highway Bridges, and 0.2 horizontal and ±0.1 vertical seismic coefficients were used based on the allowable design approach. The footings were supported by cast-in-place piles. Soil was of sand and gravels, and it was classified as Type II ground condition (moderate) based on the current Design Specifications of Highway Bridges.

The columns were constructed in the following two stages, i.e., the steel columns were first constructed to support a main deck at center, and later two reinforced concrete columns and two extended lateral beams were constructed to support the two side decks. The steel columns were designed considering the future increase of load by the two side decks.

The steel column failed as if it was crushed in vertical direction. The lateral beam buckled and settled about 6m down to the surface of infilled concrete in the steel column. The concrete was infilled for preventing failure due to collisions of automobiles. Welding was ruptured at corners, thus bearing capacity of the steel column was lost. Particular problems of strength and quality of steel plates (SM50) and the welding were not found for design and construction of the pier.

When the design was reviewed, about 78 % of the dead weight was supported by the steel column and the remaining 22 % was supported by the two reinforced concrete columns. On the other hand, about 66 % and 84 % of the lateral force was supported by the steel column in longitudinal and transverse directions, respectively. The thickness of flange plates (transverse direction) of the column varied from 28 mm at the corner with the lateral beam to 18 mm at the bottom, while the thickness of web plates (longitudinal direction)

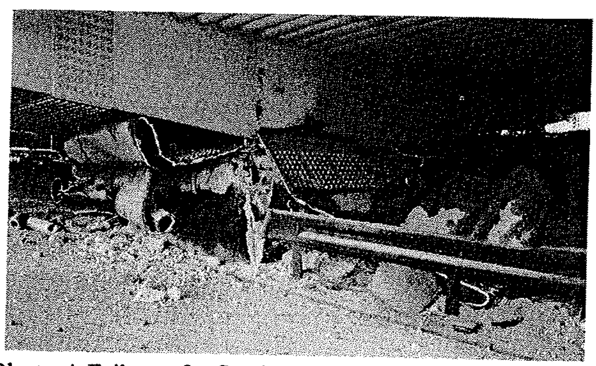


Photo 4 Failure of a Steel Pier, Tateishi, Route 3, Kobe Line, Hanshin Expressway

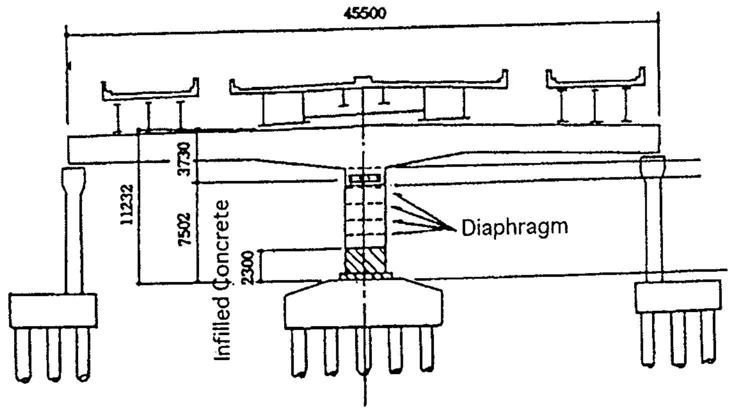



Fig. 9 Highway Bridge at Tateishi, Route 3, Kobe Line of Hanshin Expressway

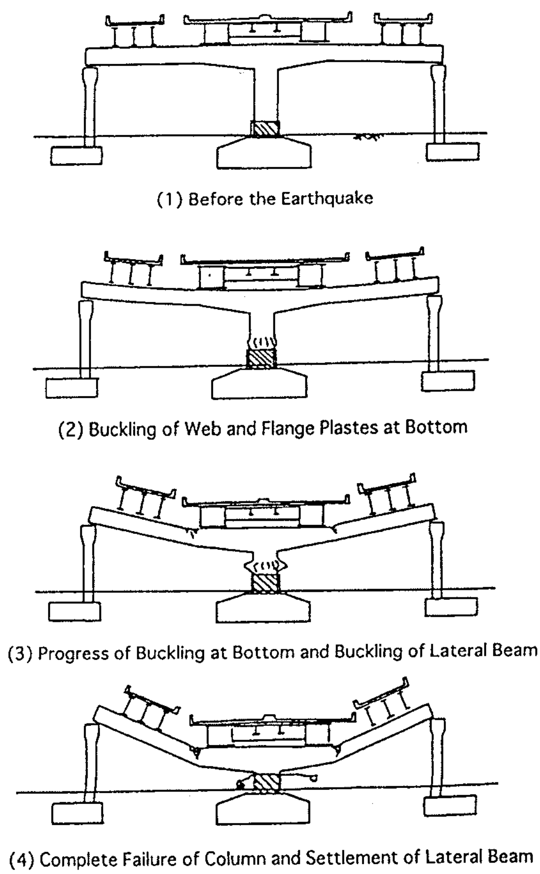



Fig. 10 Failure Mechanism of Highway Bridges at Tateishi, Route 3, Kobe Line of Hanshin Expressway

varied from 24 mm at the corner to 19 mm at mid-height and 21 mm at bottom. Stress induced in the web plates became the maximum at the bottom of the column, and it was 98 % of the allowable stress when a lateral force equivalent to 20 % of gravity force and the dead weight were applied. The contribution of dead weight to the stress induced in web plates was about 28 %.

Buckling load of column was 6,640 tf, while the axial load due to the dead weight of three decks was only 1,519tf. Therefore, 3.3 g acceleration was required to explain the damage by vertical excitation. It is reasonable to consider that the failure was developed due to local buckling of the web and flange plates when the column was subjected to a large ground acceleration in lateral direction. Failure mechanism may be explained as shown in Fig. 10. After the columns were subjected to a large ground motion, the steel column caused local buckling at the bottom This decreased the bearing capacity of column in vertical direction, and the column gradually settled down due to the dead weight of decks. This increased the bending moment in lateral beam, and developed buckling at both sides of the center deck. This further increased the axial load in the steel column, and progressed the buckling at the bottom and settlement. Finally when the corners of the column ruptured, the column totally lost the bearing capacity.

### 4.5 Nishinomiya Bridge, Route 5, Hanshin Expressway

An approaching girder to the Nishinomiya Bridge fell down from one of the supports as shown in Photo 5. The girder was 52 m long, and was supported by two steel frame piers with 24.7 m and 23.5 m in height as shown in Fig. 11. The bridge completed in 1993. The foundations were designed by the 1980 Design Specifications of Highway Bridges, and the superstructure and the steel piers were designed by the latest 1990 Design Specifications. The horizontal seismic coefficient of 0.3 was considered based on the allowable

Photo 5 Collapse of an Approaching Bridge to Nishinomiya Bridge, Route 5, Bay Shore Line, Hanshin Expressway

caisson developed a residual lateral movement of 9 cm in the channel side, and the pier tilted in the same direction resulting a residual lateral movement of 8 cm, relative to the top of a caisson, at its top. Thus, the top of Pier 99 caused a residual lateral movement of 17 cm in the channel side. On the other hand, at the Pier 100, the residual movement at the top of caisson was 3 cm in the channel side, and the pier tilted in the opposite direction resulting a lateral movement of 2 cm relative to the top of caisson. Thus, the top of Pier 100 caused a residual lateral movement of 1 cm in the channel side. Therefore, the distance between the Pier 99 and Pier 100 was reduced with an amount of 18 cm.

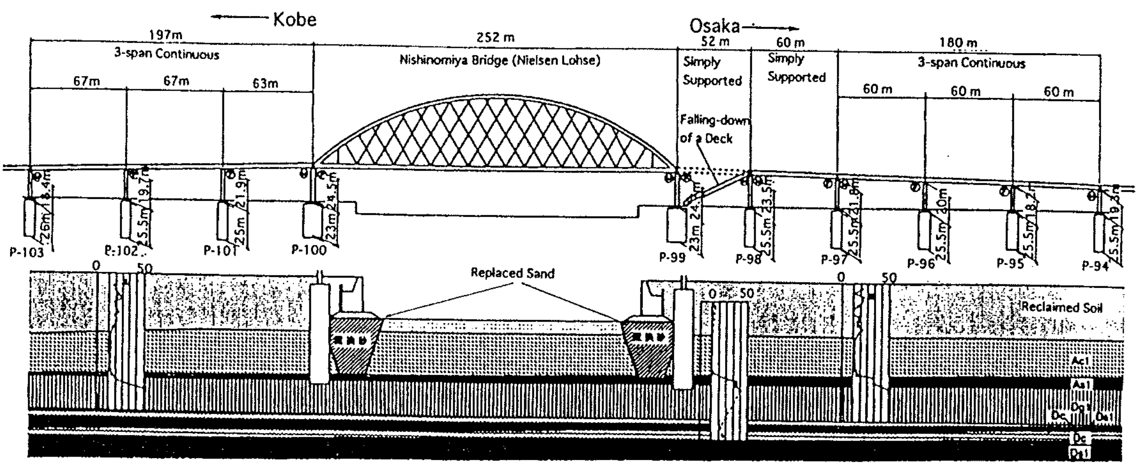



Fig. 11 Nishinomiya Bridge and Approaching Bridges

design approach. The girder was adjacent to a Nielsen Lohse bridge, which crosses the Nishinomiya Channel, with 252 m long. The foundations were of caissons. At Pier 99, where the approaching girder dislodged from its supports, the caisson foundation was 40m wide, 13 m long and 23 m high. The foundations were constructed in a soft reclaimed land, and were supported by a diluvium layer at their bottom. Coarse sandy materials with gravels from the Rokko Mountain, which had been considered stable against liquefaction, was used for the reclamation. Soil condition was classified as Type III ground condition (soft) based on the 1990 Design Specifications.

Extensive liquefaction was developed around the bridges, and lateral spreading as large as 2 m was observed. It caused some residual lateral movement and tilting in the foundations. For example, at the Pier 99 (refer to Fig. 11), the

The failed approaching girder had a weight of about 1,900 tf, and was supported by three pivot (fixed) bearings at the Pier 99 and three movable bearings at the Pier 98. The Lohse girder had a weight of about 12,000 tf, and was supported by two pivot (fixed) bearings at the Pier 100 and two movable bearings at the Pier 99. One of the two pivot bearings (mountain side) at the Pier 100 completely failed at the upper bearing as shown in Photo 6, and the other bearing also suffered damage. The seat length at the Pier 99 was 110 cm in the approaching girder side and 420 cm in the Lohse girder side. The approaching girder and the Lohse girder were connected by six tie bar type steel restrainers with 30 mm thick and 230 mm wide. They were torn off at their connections to the Lohse girder. The approaching girder collided with the adjacent girder at the Pier 98, while there was no evidence that the Lohse girder and the approaching girder collided at the Pier 99.

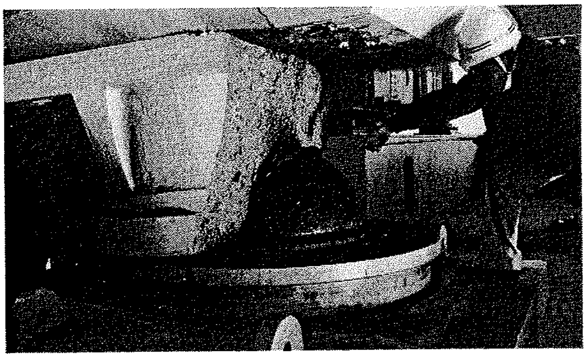


Photo 6 Failure of Fixed Pivot Bearing on Pier 100

Equivalent linear and nonlinear dynamic response analyses were made to clarify the dynamic response of bridges and possible failure mechanism. It was found from the analysis that the fundamental natural period of the Lohse girder and the Pier 100 structural system was 1.98 second. It was very close to the fundamental natural period of surrounding ground, while the fundamental natural period of the approaching girder and the Pier 99 structural system was about 0.98. It was also found from the analysis that the relative displacement developed at the Piers 99 and 100 easily reached 87 cm and 98 cm, respectively, with the peak response acceleration of about 1 g. The inertia force induced in the Lohse girder associated with the about 1 g response acceleration was sufficiently large to cause the failure at two pivot bearings on the Pier 99.

Based on the analyses, the failure of approaching girder was considered to be interacted by the response of Lohse girder, and may be described as shown in Fig. 12. Because the gap at the tie bar type restrainer was ±155 mm, a large inertia force in the Lohse girder was transferred to the approaching girder through the restrainers, when the Lohse girder pulled the approaching girder in the channel side. When this large inertia force failed the pivot bearings which supported the approaching girder at Pier 99, the approaching girder became free to move in longitudinal direction. During such large movements, the restrainers were ruptured. When the relative displacement developed in the approaching girder became larger than the seat length of 110 cm at the Pier 99, the girder fell down.

#### 4.6 Conclusions of the Report

Based on the various analyses and considerations, followings were presented in the

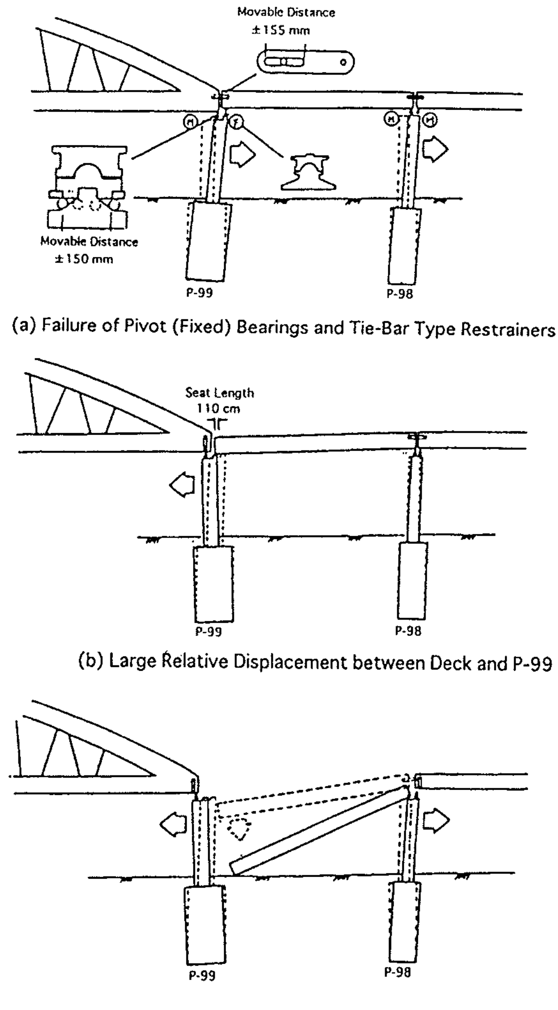



Fig. 12 Failure Mechanism of an Approaching Bridge to Nishinomiya Bridge, Route 5, Bay Shore Line of Hanshin Expressway

Report as the issues required to address further study:

(1) Evaluation of Design Seismic Force

Intensity of ground shaking during the Hanshin/Awaji Earthquake was extensive, and may be the largest ever experienced in the world in terms of response spectral value at natural period of 0.7-2 seconds. It may be required to re-evaluate design seismic force. In the re-evaluation, it is important to investigate what in the bridge response contributed to cause the destructive damage in structural members of bridges.

It is important to re-evaluate the seismic safety level required to bridges, from the view point of the importance of the bridges including the influence of loss of function, function required for restoration in the area, and economical and social requirements.

(2) Evaluation of dynamic strength and ductility of steel piers, foundations and bearings require more intensified attention and research. Evaluation of ductility of whole bridge system is also important.

(3) Dynamic response analysis has been regarded as a special analytical tool in the past, and therefore it has been used only for special bridges such as long-span bridges. However, it should be more actively used in seismic design for usual bridges. Development of user-friendly software for dynamic response analysis is essentially required.

(4) The Menshin Design should be more widely adopted.

(5) It has been long discussed whether bearings serve as a "fuse" to limit the lateral force from a superstructure to substructures, it is apparent through the evidence developed by the H/A Earthquake that the damage of bearings caused substantial damage to superstructures and interrupted traffic for long time. They should be designed to have more dynamic strength and ductility keeping the fact that the bearings are one of the major structural segments of bridges in mind.

(6) New devices to prevent falling-down of decks from substructures are required so that they can mitigate the shocks and dissipate energy. Re-evaluation of the seat length $S_E$ is required for the bridges on very soft soils, bridges with tall piers, skew bridges, and bridges on soils with high potential for liquefaction and lateral spreading.

(7) Liquefaction occurred even at the coarse sand and gravel sites where potential for liquefaction has been regarded limited. Effect of lateral spreading on the damage of foundations requires more thorough study.

(8) Evaluation of seismic safety of a whole bridge system is required against extreme earthquakes. More attentions need to be paid for selection of structural type.

## 5. SEISMIC DESIGN FOR RECONSTRUCTION AND REPAIR

### 5.1 General

For seismic design of reconstruction of highway bridges that suffered damage due to the Hanshin/Awaji Earthquake, the "Guide Specifications for Reconstruction and Repair of Highway Bridges Which Suffered Damage due to the Hyogo-ken Nanbu Earthquake" (11) was issued by the Ministry of Construction on February 27, 1995 upon approval by the "Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake."

The Guide Specifications was originally developed to be applied for reconstruction and repair of the highway bridges that suffered damage due to the H/A Earthquake, and it is currently being used for all new construction and seismic strengthening in all parts of Japan as an emergence measure.

The table of contents of the Guide Specifications for Reconstruction and Repair was as:

1. General
2. Basic Principle of Seismic Design
3. Dynamic Response Analysis
   - 3.1 General
   - 3.2 Analytical Methods and Analytical Models
   - 3.3 Input Ground Motions
4. Menshin Design
5. Check of Dynamic Strength and Ductility of Reinforced Concrete Piers
   - 5.1 General
   - 5.2 Stress-Strain Relation of Confined Concrete
   - 5.3 Hoops
   - 5.4 Treatment of Statically Eccentric Bending Moment
   - 5.5 Treatment of P-Delta Effect
   - 5.6 Termination of Main Reinforcements at Mid-height
6. Check of Dynamic Strength and Ductility of Concrete Infilled Steel Piers
   - 6.1 General
   - 6.2 Evaluation of Dynamic Strength and Ductility
   - 6.3 Treatment of P-Delta Effect
7. Check of Dynamic Strength and Deformation Capability of Foundations
   - 7.1 General
   - 7.2 Basic Principle for Check of Dynamic Strength and Deformation Capability
8. Bearing Supports and Surroundings
9. Falling-down Prevention Devices
10. Treatment for Lateral Spreading of Soils due to Soil Liquefaction

The following outlines the Guide Specifications.

### 5.2 Basic Principle of Seismic Design

The bridges shall be designed so that they can withstand with enough structural safety against the H/A Earthquake. To achieve this goal, following basic principles shall be considered.

(1) To increase the ductility of whole bridge systems, dynamic strength and ductility shall be assured for whole structural members in which seismic effect is predominant. Although the check of dynamic strength and ductility has been adopted for reinforced concrete piers since 1990, it has not been applied for other structural members such as steel piers and foundations.

(2) Seismic safety against the H/A Earthquake shall be verified by dynamic response analysis considering nonlinear behavior of structural members.

(3) In design of elevated continuous bridges, it is appropriate to adopt the Menshin Design for distributing lateral force of superstructure to as many substructures as possible. The Menshin Design is close to the seismic isolation, but the emphasis is placed to increase energy dissipating capability and to distribute lateral force of deck to substructures (8,12).

(4) Enough tie reinforcements to assure the ductility shall be provided in reinforced concrete piers, and the termination of main reinforcements at mid-height shall not be made.

(5) Concrete shall be filled in steel piers to assure dynamic strength and ductility. Steel piers designed by the current practice developed local bucking at web and flange plates although they were stiffened by longitudinal stiffeners and diaphragms. This tends to cause sudden decrease of bearing capacity in lateral direction after the peak strength and therefore less energy dissipation is anticipated. This subsequently deteriorates the bearing capacity of steel piers in vertical direction. Because it is now at the stage that technical developments are being made to avoid such behavior, it was decided to tentatively use steel piers with infilled concrete for reconstruction and repair.

(6) Foundations shall be designed so that they have enough dynamic strength and deformation capability for lateral force. The dynamic strength and deformation capability of foundations shall be larger than the flexural strength and ductility of piers to prevent damage at foundations.

(7) It is suggested to adopt rubber bearings more actively because they absorb relative displacements developed between a superstructure and substructures. In design of bearings, correct mechanism of force transfer from a superstructure to substructures shall be considered.

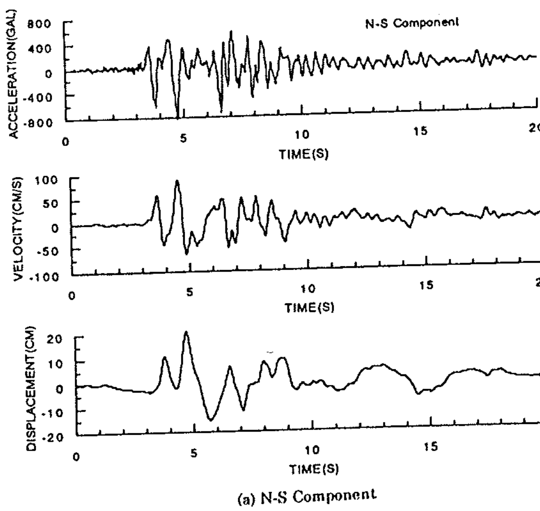
(8) The devices to prevent falling-down of a superstructure from substructures shall be designed so that they can assure falling-down of decks. Attention shall be paid so as to dissipate energy and to increase strength and deformation capability.

(9) At those sites where potential to cause lateral spreading associated with soil liquefaction is high, its effect shall be considered in design. Because technical information to evaluate earth pressure in laterally spreading soils is limited, it is important to recognize that such evidence exists and that countermeasures shall be taken in any possible ways.

### 5.3 Dynamic Response Analysis

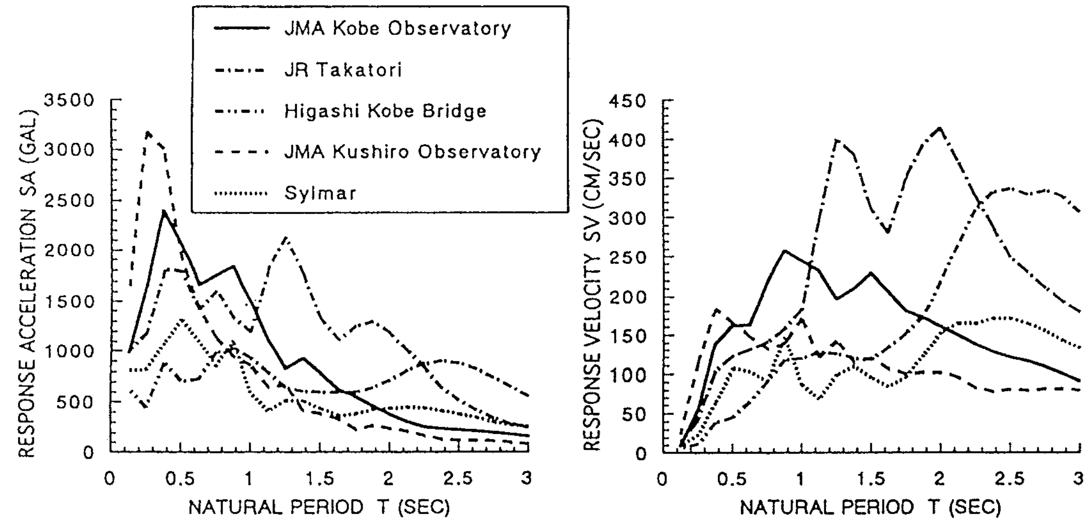
Besides the dynamic response analysis described in the "Part V Seismic Design" of the "1990 Design Specifications of Highway Bridges," dynamic behavior of bridges shall be carefully clarified by nonlinear dynamic response analysis, and result of the analysis shall be reflected in design.

In the dynamic response analysis, the ground motion that was recorded during the H/A Earthquake with the largest intensity of ground acceleration shall be used as an input to assure the seismic safety of bridges against the Hanshin/Awaji Earthquake. The acceleration recorded at JMA Kobe Observatory as shown in Figs. 13 and 14 may be regarded as the input




(a) N-S Component

Fig. 13 Strong Motion Accelerations Measured at JMA Kobe Observatory (NS Component) motion. Clarification of appropriate ground motions needs to be made based on future research. In Fig. 14, response spectra for ground motions at JR Takatori Station (16) and Higashi Kobe Bridge (Route 5, Bay Shore Line of the Hanshin Expressway) by the H/A Earthquake, JMA Kushiro Observatory by the 1993 Kushiro-oki Earthquake, and Sylmar Parking Lot by the 1994 Northridge Earthquake (1) are presented for comparison.



(a) Response Acceleration Spectra

(b) Response Velocity Spectra

Fig. 14 Response Spectra (h=0.05) at JMA Kobe Observatory (NS) in comparison with the Ground Accelerations Measured at JR Takatori Station and Higashi Kobe Bridge by the H/A Earthquake, JMA Kushiro Observatory by the 1993 Kushiro-oki Earthquake, and Sylmar Parking Lot by the 1994 Northridge Earthquake

### 5.4 Menshin Design

It is appropriate to adopt the Menshin Design to mitigate shocks associated with deck response and to reduce lateral force by increasing energy dissipation. Emphasis shall be placed to distribute lateral force to as many substructures as possible, and to increase energy dissipation capability. Excessive elongation of natural period to reduce lateral force shall not be made, because this increases response displacement of a deck. This causes various disadvantage such as an increase of noise and vibration associated with the adoption of large expansion joints. The Menshin Design shall not be used at the sites where potential for soil liquefaction and unstability of weak clayey materials is high. The "Manual for Menshin Design of Highway Bridges" (8,12) issued in 1992 as an outcome of the Joint Research between the Public Works Research Institute and private companies may be refereed to in the Menshin Design.

For introducing the Menshin Design for existing simply supported bridges, it is suggested to connect simply supported girders to make the superstructures continuous. The "Reference for Design and Construction to Eliminate Expansion Joints from Existing Bridges" (19) issued by the Road Maintenance Center in 1995 may be refereed to for this purpose.

### 5.5 Check of Dynamic Strength and Ductility of Reinforced Concrete Piers

The check of dynamic strength and ductility shall be made for reinforced concrete piers after they are designed by the static lateral force method. The current provisions for the check were expanded in the Guide Specifications as:

(1) For evaluating the strength and the ductility of piers, the effect of confinement of concrete by ties has not been precisely considered in the current Design Specifications. A new stress-strain relation of confined concrete as shown in Fig. 15 was introduced (6). The "ultimate strain" was defined as the strain where concrete strength decreases 20% from its peak value.

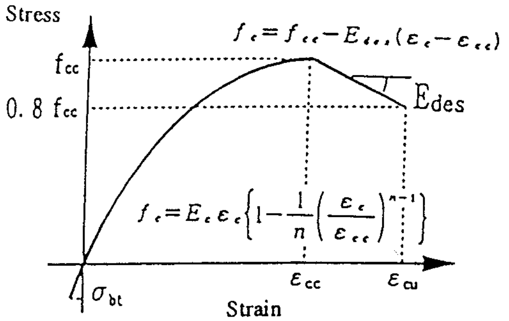



Fig. 15 Stress-Strain Relation for Concrete Confined by Tie Bars

(2) Enough ties shall be provided to assure ductility. Ties shall be of deformed bars with diameter larger than 13 mm. The minimum interval of ties was decreased from 30 cm to 15 cm to increase ductility and shear strength. The interval of ties shall not be suddenly changed but smoothly changed to prevent sudden decrease of strength along piers.

Intermediate ties were newly introduced in rectangular piers to assure the confinement. Diameter and material of the intermediate ties shall be the same with those of the ties, and shall be placed at each level where ties are placed. The spacing of intermediate ties in lateral direction shall be less than 100 cm.

(3) In the inverted "L" (or, "C") piers, eccentric bending moment is induced in piers due to the dead weight of decks. Such effect shall be considered in the check of dynamic strength and ductility.

(4) For slender piers, the P-delta effect shall be considered in the check of dynamic strength and ductility.

(6) Termination of main reinforcement at mid-height shall not be made, in principle, to prevent shear failure at the terminated points.

**5.6 Check of Dynamic Strength and Ductility of Concrete-Infilled Steel Piers**

Although various researches have been made for dynamic strength and ductility of steel piers, they are not still sufficient to provide general stipulations for the check of dynamic strength and ductility of steel piers. Therefore, it was suggested to tentatively infill concrete in steel piers for reconstruction and repair. The procedure for the check of dynamic strength and ductility of concrete infilled steel piers follows 5.5 with some appropriate modifications.

**5.7 Check of Dynamic Strength and Ductility of Foundations**

Because the allowable stress design approach has been used for foundations, appropriate design methods for the check of dynamic strength and ductility (deformation capability) have not yet developed for foundations. It was therefore proposed to check the safety of foundations tentatively assuming that the flexural strength of piers evaluated in 5.5 shall be applied to the footing as a seismic lateral force to the foundation. In this evaluation, the safety factor for the allowable stress design were increased.

**5.8 Design of Bearings and Surroundings**

Bearings have several functions, i.e., they need to support dead weight of a superstructure and live load, allow relative movement due to thermal movement of decks, support seismic lateral force, and prevent excessive lateralmovement and uplift during earthquakes. Because various functions concentrate at bearings, this makes the structure of bearings complicated. Thus, the bearings tend to be damaged during earthquakes. It was therefore suggested to provide separate structural components to share the function of bearings. This may be made by providing additional members to prevent the uplift or excessive relative movement.

Adoption of rubber bearings, in particular Menshin bearings that has energy dissipating capability, was recommended. Because the rubber bearings allow relative movement to occur in various directions, they are generally free from damage.

Bearings and surrounding parts of a superstructure shall be designed considering the mechanism of lateral force transfer from superstructure to substructures. For example, when tall bearings such as pin/roller type bearings are adopted, a bending moment is induced associated with the distance between the gravity center of a deck where lateral force is applied and the bottom of bearings where lateral force is supported. Although such bending moment has been ignored in the past, this shall be considered to properly design bearings and surroundings.

Bearings shall be designed so that the ultimate strength is larger than the force developed during the extreme earthquakes.

**5.9 Devices to Prevent Falling-down of Superstructure from Substructures**

In the current Design Specifications, it is stipulated that at both ends of superstructure, either the seat length $S_E$ or the installation of the devices for preventing falling-down of a superstructure from substructures shall be adopted. In the Guide Specifications, it was described to provide both the seat length and the installation of falling-down prevention devices. The falling-down prevention devices shall be so designed that they are safe against the lateral force not only in longitudinal direction but in transverse direction, and that the portion of the decks where the falling-down prevention devices are connected to should have enough strength to prevent rupture. It was suggested to adopt the falling-down prevention devices with energy dissipation capability.

### 5.10 Countermeasures against Lateral Spreading associated with Soil Liquefaction

At the sites where foundations are close to rivers and channels and where the potential of liquefaction is high and the liquefiable sandy layer is inclined, effect of lateral spreading shall be taken into account in design. Because it is difficult at this moment to properly evaluate the earth pressure due to the lateral spreading, general engineering judgment is required.

## 6. REFERENCE FOR APPLYING "GUIDE SPECIFICATIONS" TO NEW HIGHWAY BRIDGES AND SEISMIC STRENGTHENING

### 6.1 General

For increasing seismic safety of the highway bridges which suffered damage by the H/A Earthquake, various new drastic changes were tentatively introduced in the "Guide Specifications for Reconstruction and Repair of Highway Bridges Which Suffered Damage due to the H/A Earthquake." Although intensified review of design could be made when it was applied to the bridges only in the Hanshin area, it was not easy for field design engineers to follow up the new Guide Specifications when the Guide Specifications was decided to be used for seismic design of all new highway bridges and seismic strengthening of existing highway bridges in other areas. Based on such demand, the "Reference for Applying the Guide Specifications to New Highway Bridges and Seismic Strengthening" was issued on June 30, 1995 by the Special Sub-Committee for Seismic Countermeasures for Highway Bridges , Japan Road Association (4). Several educational seminars were held with use of the Reference at major cities including Tokyo, Osaka, Nagoya, Hiroshima, Fukuoka and Sapporo, with the participants over 3,000.

The table of contents of the Reference was as:



### 6.2 Application of the Guide Specifications

The Reference classified the application of the Guide Specifications as shown in Table 13 based on the importance of the roads. All items of the Guide Specifications have been applied for bridges on "extremely important roads," while some items which are required to prevent brittle failure of structural components have been applied for bridges on "important roads." For

Table 13 Application of the Guide Specifications

| Type of Roads and Bridges | Double Deckers, Overcrossings on Roads and Railways, Extremely Important Bridges from Disaster Prevention and Road Network | Others |
|---|---|---|
| Expressways, Urban Expressways, Designated Urban Expressways, Honshu Shikoku Bridges, Designated National Highways | Apply all items, in principle | Apply all items, in principle |
| Non-designated National Highways, Ken Roads, City, Town and Village Roads | Apply all items, in principle | Apply partially, in principle |

Table 14 Standard Coefficient $k_{hc0}$

| | | | |
|---|---|---|---|
| Group I | $T_{EQ} < 0.3$s<br>$k_{hc0} = 4.46 T_{EQ}^{2/3}$ | $0.3 \leq T_{EQ} \leq 0.7$ s<br>$k_{hc0} = 2.0$ | $0.7\text{s} < T_{EQ}$<br>$k_{hc0} = 1.24 T_{EQ}^{-4/3}$ |
| Group II | $T_{EQ} < 0.4$s<br>$k_{hc0} = 3.22 T_{EQ}^{2/3}$ | $0.4 \leq T_{EQ} \leq 1.2$ s<br>$k_{hc0} = 1.75$ | $1.2\text{s} < T_{EQ}$<br>$k_{hc0} = 2.23 T_{EQ}^{-4/3}$ |
| Group III | $T_{EQ} < 0.5$s<br>$k_{hc0} = 2.38 T_{EQ}^{2/3}$ | $0.5 \leq T_{EQ} \leq 1.5$ s<br>$k_{hc0} = 1.50$ | $1.5\text{s} < T_{EQ}$<br>$k_{hc0} = 2.57 T_{EQ}^{-4/3}$ |

example, for bridges on the important roads, the items for menshin design, tie reinforcements, termination of longitudinal reinforcements, type of bearings, falling-down prevention devices and countermeasures for soil liquefaction are applied, while the remaining items such as the design force, concrete infilled steel bridges, and ductility check for foundations are not applied.

### 6.3 Ground Accelerations Used for Nonlinear Dynamic Response Analysis and Seismic Coefficient for Check of Ductility and Dynamic Strength

As reference ground motions used for nonlinear dynamic response analysis, the strong ground motion accelerations recorded during the H/A Earthquake at JMA Kobe Observatory, JR Takatori Station (16), and Higashi-Kobe Bridge (Route 5, Bay Shore Line of the Hanshin Expressway) were suggested for analysis of bridges at the Type I, II and III soil condition, respectively.

In addition to the seismic coefficient specified in the current Design Specifications for the check of ductility and dynamic strength, it was also suggested to use the seismic coefficient as

$$k_{hc} = c_z \cdot k_{hc0} \tag{1}$$

where, $k_{hc}$ =seismic coefficient for check of ductility and dynamic strength, $k_{hc0}$=standard seismic coefficient and given by Table 14, and $c_z$=zoning coefficient.

The standard seismic coefficient $k_{hc0}$ in Table 14 was tentatively provided by taking envelopes of the acceleration response spectra (damping ratio of 0.05) of the records measured in the H/A Earthquake. The modification of the response spectra in terms of the damping ratio vs. natural period relation was incorporated (10). At mid-range natural period, $k_{hc0}$ was assumed to be larger at Type I ground condition (stiff sites) than Type II (moderate) and Type III (soft soil sites) ground conditions. This reflects the fact that surface ground accelerations were smaller than the accelerations in underground at soft soil sites.

### 6.4 Seismic Design of Foundations

The check of ductility and dynamic strength (deformation capability) of foundations was more quantitatively presented in the Reference. In pile foundations, the check is made as follows:

First, a foundation is analyzed by the "displacement method," in which a foundation is so idealized that a rigid footing is supported by piles which are elastically supported by soils. The flexural strength of a pier shall be applied as a seismic force to foundations at the bottom of the footing together with the dead weight of superstructure, pier and soils on the footing.

Safety of the foundation shall be checked so that 1) vertical load induced in piles shall be less than half of their ultimate bearing capacity, 2) lateral displacement of footing shall be less than about 30 mm, and 3) bending moment developed in piles shall be less than their yielding moment.

When unreasonable size was required in the foundation designed by the "displacement method," the "nonlinear method" shall be used. In the nonlinear method, a foundation is idealized as shown in Fig. 16. Nonlinear parameters for vertical strength, uplift, lateral movement and soil springs shall be evaluated according to the "Part IV Foundations" of the "Design Specifications of Highway Bridges." Nonlinearity of piles shall be evaluated in the same way to reinforced concrete piers.

be increased in design of falling-down prevention devices.

Because the lateral force equivalent to $R_d$ times twice of the seismic coefficient has been so far used, Eq. (2) provides almost twice as large lateral force than the previous design force. Because 50% increase of allowable stress which has been adopted so far is not made, actual design force for falling-down prevention devices was increased almost 3 times.

Rigid type restrainers should not be used to connect two adjacent girders if the size of the two girders is significantly different. The size of the two adjacent girders should be regarded significantly different when the reaction forces of the two girders are different by a factor of 2 or

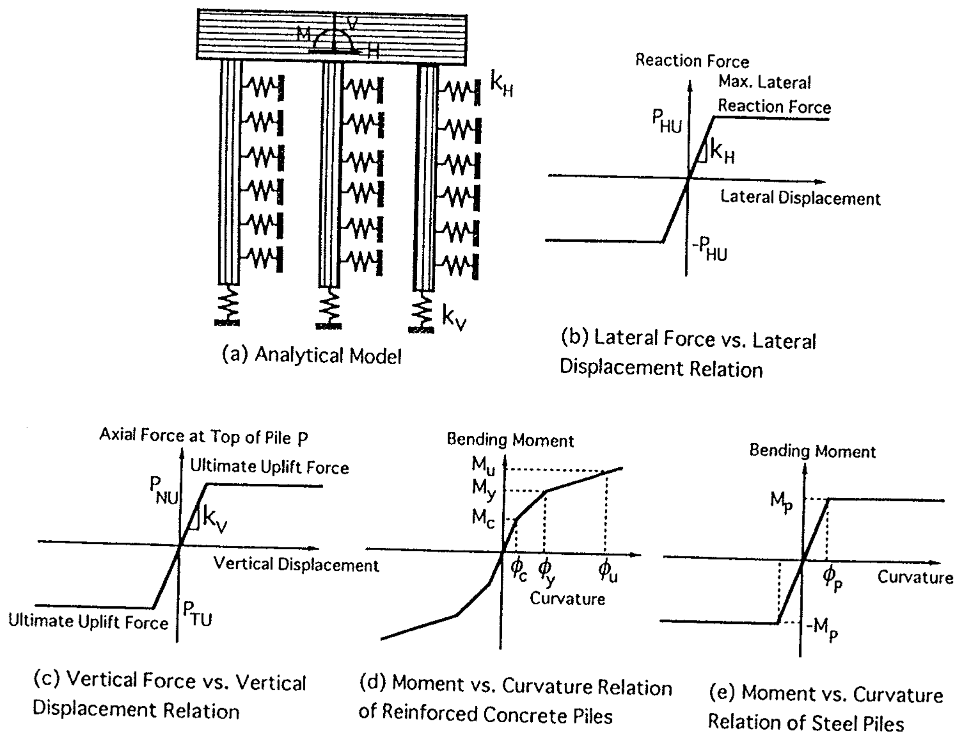



Fig. 16 Idealization of Pile Foundation in the Nonlinear Analysis

the fundamental natural periods of the two structural systems are different by a factor of 1.5.

**6.5 Design of Falling-down Prevention Devices**

It was tentatively suggested that the following force $P$ shall be used as a design force of falling-down prevention devices.

$$P = R_d \tag{2}$$

where $R_d$ represents a reaction force at the bearing. If the reaction force is different between the adjacent girders, the larger value shall be used in Eq. (2) for restrainers which tie together two approaching girders. Allowable stress should not

It was suggested to provide some redundancy in the seat length in the following conditions:

(1) skew bridge (tentatively, bridges with skew angle less than 60 degree) and curved bridges (tentatively, bridges with curvature less than 100m and angle formed by both ends less than 30 degree)
(2) bridges supported by slender piers with long natural period (tentatively, bridges with natural period longer than 2 second)

(3) bridges which tend to have large lateral displacement in foundations due to soilliquefaction/ lateral spreading
(4) bridges supported by slender columns with extremely short seat length

It was also suggested to provide falling-down prevention devices not only in longitudinal direction but also in transverse direction in the following bridges:

(1) skew bridge (tentatively, bridges with skew angle less than 60 degree) and curved bridges (tentatively, bridges with curvature less than 100m and angle formed by both ends less than 30 degree)
(2) bridges with Gerber joints
(3) bridges supported by slender columns with very short seat length

**6.6 Termination of Longitudinal Reinforcements**

It was described in the Guide Specifications that longitudinal reinforcements shall not be terminated at mid-heights. This was acceptable because there were few bridges which were supported by tall piers in the damaged area. However, when the Guide Specifications were used nationwide, there are bridges which are supported by tall piers. Therefore, the following minimum heights are suggested if the termination is inevitable.

$$h_i = H(1 - \frac{M_{yi}}{2M_{yB}}) + b \qquad (3)$$

where, $h_i$=i-th height from the bottom where longitudinal reinforcements are terminated, $H$=height of a pier, $M_{yi}$=yielding moment of a pier at i-th terminated point, $M_{yB}$=yielding bending moment of a pier at bottom, and $b$=smaller dimension between longitudinal and transverse widths of a pier.

Furthermore, it was suggested as follows:

(1) Longitudinal reinforcements should not be terminated between the bottom and 2 times b from it.
(2) Longitudinal reinforcements should not be at once reduced more that 1/3.
(3) Outside longitudinal reinforcements should not be terminated.

**6.7 Intermediate Ties for Circular Reinforced Concrete Columns**

It was suggested in the Guide Specifications that the intermediate ties are not required for circular reinforced concrete piers. However, they are effective to increase shear strength even in the circular piers, while confinement effect may be small. It was therefore suggested to tentatively provide the intermediate ties at every other ties in circular reinforced concrete piers. In the circular piers, the intermediate ties can be considered to contribute to shear strength, but it should not be considered to evaluate ductility in terms of the tie reinforcement ratio. It was described in the Guide Specifications that the intermediate ties can be considered to evaluate shear strength and the confinement effect in the rectangular piers.

**6.8 Seismic Strengthening of Existing Reinforced Concrete Columns**

Because damage concentrated to single reinforced concrete piers/columns with small concrete section, seismic strengthening is being initiated for those columns, which were designed by the pre-1980 Design Specifications, at extremely important bridges. Main purpose of the seismic strengthening of reinforced concrete columns is to increase their shear strength, in particular in the piers with termination of longitudinal reinforcements without enough anchoring length. This increases ductility of columns, because premature shear failure could be avoided.

However if only ductility of piers is increased, residual displacements developed at piers after an earthquake may increase. Therefore the flexural strength should also be increased. However the increase of flexural strength of piers tends to increase the seismic force transferred from the piers to the foundations. It was found from an analysis to various types of foundations that failure of the foundations by increasing the seismic force may not be significant if the increasing rate of the flexural strength of piers is less than 2. It is therefore suggested to increase the flexural strength of piers within this limit so that it does not cause serious damage to foundations.

For such requirements, seismic strengthening by "Steel Jackets with Controlled Increase of Flexural Strength" was suggested. This uses steel jacket surrounding the existing columns as shown in Fig. 17. Epoxy resin or non shrinkage concrete mortar are injected between the concrete surface and the steel jacket. A small gap is provided at the bottom of piers between the steel jacket and the top of footing. This prevents to excessively increase the flexural strength.

To increase the flexural strength of columns in a controlled manner, anchor bolts are provided at the bottom of the steel jacket. They are drilled into the footing. By selecting appropriate number and size of the anchor bolts, the degree of increase of the flexural strength of piers may be controlled. The gap is required to trigger the flexural failure at the bottom of columns. A series of loading tests are being conducted at the Public Works Research Institute to check the appropriate gap and number of anchor bolts. Table 15 shows a tentatively suggested thickness of steel jackets and size and number of anchor bolts. They are for reinforced concrete columns with a/b less than 3, in which a and b represent the width of column in transverse and longitudinal direction, respectively. The size and number of anchor bolts were evaluated so that the increasing rate of flexural strength of columns is less than about 2.

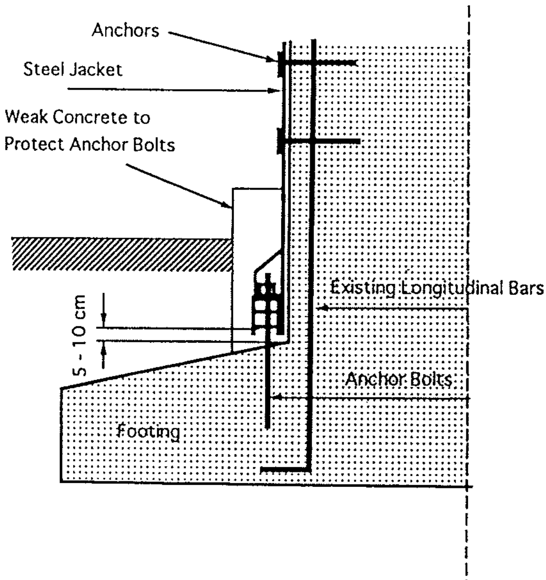



Fig. 17 Seismic Strengthening of Reinforced Concrete Piers by Steel Jacket with Controlled Increase of Flexural Strength

and revealed that various technical developments are required in both seismic design and seismic strengthening for mitigating the damage of highway bridges against extreme inland earthquakes. Based on the information available at this moment, the followings may be tentatively pointed out:

(1) Intensity of ground shaking during the earthquake was extensive, and may be the largest ever experienced in the world in terms of response spectral value at natural period of 0.7-2 seconds. It may be required to re-evaluate design seismic force. In the re-evaluation, it is important to investigate what in the bridge response contributed to cause the destructive damage in structural members of bridges. It is required to change to the direction in which "realistic ground shaking during extreme earthquakes," "realistic design calculation" and "realistic evaluation on dynamic strength and ductility of structural members" can be incorporated.

(2) Destructive damage occurred in the bridges designed by the pre-1971/1980 Design Specifications, and the bridges designed by the post-1980 Design Specifications suffered less damage such as the Route 5, Bay Shore Line of the Hanshin Expressway. However, even the Route 5, Bay Shore Line suffered considerable damage, and it was closed for about half year since the earthquake.

(3) Precise evaluation on how the Route 5, Bay Shore Line performed if it was located at the site of Route 3, Kobe Line, where ground shaking was the most destructive, requires more intensified and careful analysis to evaluate the seismic safety goal in the future revision of the Design Specifications of Highway Bridges.

(4) Damage was destructive in new types of bridges in urban area, which have not yet experienced seismic disturbances in the recent earthquakes, such as the elevated bridges supported by slender reinforced concrete columns/steel piers, and flexible bridges

Table 15 Steel Jacket and Anchor Bolts

| Columns/Piers | Steel Jackets | Anchor Bolts |
|---|---|---|
| a/b≦2 | SS 400, t= 9 mm | SD 295, D35 ctc 250 mm |
| 2<a/b≦3 | SS 400, t= 12 mm | |
| Columns supporting Lateral Force through Fixed Bearings and with a/b≦3 | | |

## 7. CONCLUDING REMARKS

The Hanshin/Awaji Earthquake was the first earthquake that hit an urban area in recent years, constructed on extremely soft soils vulnerable to liquefaction and lateral spreading.

(5) More clear difference on the importance of highway bridges may be required. From road administrative point of view, it is not easy to designate which are "important" and which are "less important." But because highway bridges in urban areas are obviously "lifelines" for life of peoples, more clear difference of investment on construction cost should be made.

(6) It has been said that bridges should be structurally safe against small to moderate earthquakes, and that they should not be collapsed against a destructive earthquake. However, it became apparent after the Hanshin/Awaji Earthquake that there were tremendous scatter in interpretation among administrators, researchers, engineers, and more importantly public on what was the tolerable damage level of urban highway bridges. A good example may be the Route 5, Bay Shore Line of the Hanshin Expressway. They did not collapse excluding an approaching girder to the Nishinomiya Bridge. However as described in (1), it could not be used for about half year.

It seems apparent from the earthquake that the urban area highly depended its existence on the highway systems, and therefore the urban highway systems needed to withstand with more controlled damage so that they could be reopened shortly after the earthquake. How "shortly" requires more discussion, but it should not be the half year. From such sense, even current seismic safety level specified in the latest Design Specifications may be required to be upgraded. Construction of new bridges with higher seismic safety could be made with limited increase of construction cost, because the increase of seismic safety requires increase of strength and ductility of only piers and foundations, and because land cost is extremely expensive in urban areas. However seismic strengthening to upgrade seismic safety to the current level is extremely expensive. More technical developments which enable not to require to strengthen foundations are required, because the strengthening of foundations is extremely costly.

## ACKNOWLEDGMENTS

The contents presented here were the works conducted at the "Committee for Investigation on the Damage of Highway Bridges Caused by the Hyogo-ken Nanbu Earthquake" and the "Special Sub-committee for Seismic Countermeasures for Highway Bridges." Hard working of the committees members and working group including N. Miyata, Y. Iwasaki and S. Kikuchi, Heard-quarters of the Ministry of Construction, and K. Nishikawa, M. Nakano, T. Terayama, Y. Kimura, J. Hoshikuma, and T. Honda, Public Works Research Institute, MOC is highly acknowledged.

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