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Seismic retrofit of existing highway bridges in Japan

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Abstract

This paper presents the current technical developments for seismic design and seismic retrofit of existing highway bridges in Japan. At first, the histories of the past seismic design codes, past seismic evaluation investigation, and past seismic retrofit practices for highway bridges are described. Then the damage caused by the 1995 Hyogo-ken Nanbu Earthquake and the lessons learned from the earthquake are briefly described. Finally, the seismic retrofit program after the Hyogo-ken Nanbu Earthquake is described with emphasis on research and development as well as field practice of the seismic retrofit of existing reinforced concrete highway bridges such as single column bent, wall-type pier, and multicolumn bents.

Keywords: Seismic retrofit; Sesimic accessment; Exsisting bridge; Prioritization; Steel jacket; Concrete jacket; Carbon fiber jacket

1. Introduction

Japan is one of the most seismically disastrous countries in the world and has often suffered significant damage from large earthquakes. More than 3000 highway bridges suffered damage in the past earthquakes since the 1923 Kanto Earthquake. The earthquake disaster prevention technology for highway bridges had been developed based on such bitter damage experiences. Various provisions for preventing damage due to instability of soils such as soil liquefaction have been adopted. Furthermore, design detailing that includes the unseating prevention devices have been implemented. With progress of the improvement of the seismic design provisions, the damage to highway bridges by the earthquakes had been decreasing in recent years.

However, the Hyogo-ken Nanbu Earthquake of 17 January 1995, caused destructive damage to highway bridges. Collapse and nearly collapse of superstructures occurred at nine sites, and other destructive damage occurred at 16 sites [1]. The earthquake revealed that there are a number of critical issues to be revised in the seismic design and seismic retrofit of bridges.

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

On 27 February 1995, the Committee approved the "Guide Specifications for Reconstruction and Repair of Highway Bridges that suffered damage due to the Hyogo-ken Nanbu Earthquake" [2]. The Ministry of Construction noticed on the same day that the reconstruction and repair of the highway bridges that suffered damage during the Hyogo-ken Nanbu Earthquake should be made according to the guide specifications. It was also decided by the Ministry of Construction on 25 May 1995 that the guide specifications should be tentatively used in all sections of Japan as emergency measures for seismic design of new highway bridges and seismic retrofit of existing highway bridges until the Design Specifications of Highway Bridges would be revised. The Design Specifications [3] have been revised in November 1996 based on the guide specifications and further research and development that were made after the Hyogo-ken Nanbu Earthquake.

This paper summarizes the current technical developments for seismic retrofit of existing highway bridges in Japan as well as the past seismic retrofit practices.

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2. Histories of past seismic design codes and seismic retrofit practices before the Hyogo-Ken Nanbu Earthquake

2.1. History of past seismic design codes for highway bridges

One year after the 1923 Kanto Earthquake, it was initiated to consider the seismic effect in the design of highway bridges. The Civil Engineering Bureau of the Ministry of Interior notified "the method of seismic design of abutments and piers" in 1924. The seismic design method has been developed and improved through bitter experiences in a number of past earthquakes and with progress of technical developments in earthquake engineering. Table 1 summarizes the history of provisions in seismic design for highway bridges.

In particular, the seismic design method was integrated and upgraded by compiling the "Specifications for Seismic Design of Highway Bridges" in 1971, which exclusively provided issues related to seismic design. The design methods for soil liquefaction and unseating prevention devices were introduced in the specifications. It was revised in 1980 and integrated as "Part V: Seismic Design" in "Design Specifications of Highway Bridges". The primitive verification methods for ductility of reinforced concrete piers were included in the reference of the specifications. It was further revised in 1990 and ductility verification of reinforced concrete piers, soil liquefaction, dynamic response analysis, and design detailing were prescribed. It should be noted here that the detailed ductility verification method for reinforced concrete piers was firstly introduced in the 1990 specifications.

2.2. History of seismic vulnerability evaluation and retrofit of highway bridges

The Ministry of Construction made seismic evaluation investigation of highway bridges five times throughout the country since 1971 as a part of the comprehensive earthquake disaster prevention measures for highway facilities. Seismic retrofit for vulnerable highway bridges had been successively made based on these seismic evaluations. Table 2 shows the history of past seismic evaluation investigations [4–11].

The first seismic evaluation was made in 1971 to promote earthquake disaster prevention measures for highway facilities. The significant damage of highway bridges caused by the San Fernando Earthquake, USA in February 1971 triggered the seismic evaluation. Highway bridges with span length longer than or equal to 5 m on all sections of national expressways and national highways, and sections of the others were evaluated. Attention was paid to detect deteriorations such as cracks of reinforced concrete structures, tilting, sliding, settlement and scouring of foundations. Approximately 18 000 highway bridges in total were evaluated and approximately 3200 bridges were found to require retrofit.

Following the first seismic evaluation, it had been subsequently made in 1976, 1979, 1986 and 1991 with gradually expanding highways and evaluation items. The seismic evaluation in 1986 was made with the increase of social needs to insure seismic safety of highway traffic after the damage caused by the Urakawa-oki Earthquake in 1982 and the Nihon-kai-chubu Earthquake in 1983. The highway bridges with span length longer than or equal to 15 m on all sections of national expressways, national highways and principal local highways, and sections of the others, and overpasses were evaluated. The evaluation items included deterioration, unseating prevention devices, strength of substructures and stability of foundations. Approximately 40 000 bridges in total were evaluated and approximately 11 800 bridges were found to require retrofit. The latest seismic evaluation was made in 1991. The highways to be evaluated were expanding from the evaluation in 1986. Approximately 60 000 bridges in total were evaluated and approximately 18 000 bridges were found to require retrofit. Through a series of seismic retrofit works, approximately 32 000 bridges were retrofitted by the end of 1994.

In the seismic evaluation in 1986 and 1991, the evaluation was made based on a statistical analysis of bridges damaged and undamaged in the past earthquakes [12]. Factors that affect seismic vulnerability were detected as shown in Table 3. Table 4 shows the inspection sheet proposed to evaluate the seismic vulnerability.

Because collapse of bridges tends to be developed due to the excessive relative movement between the superstructure and the substructures, and failure of substructures associated with inadequate strength, the evaluation is made in Table 4 based on both the relative movement and the strength of substructure. Emphasis had been placed to install the unseating prevention devices in the past seismic retrofit. Because the installation of the unseating prevention devices was being completed, it had become important to promote the strengthening of substructures with inadequate strength, lateral stiffness and ductility.

3. Lessons learned from the Hyogo-ken Nanbu Earthquake

Hyogo-ken Nanbu Earthquake was the first earthquake which hit an urban area in Japan since the 1948 Fukui Earthquake. Although the magnitude of the earthquake was moderate (M7.2), the ground motion

Table 1
Past seismic design methods for highway bridges

		1926 Details of Road Structure (draft), Road Law. MIA	1939 Design Specifications of Steel Highway Bridges(draft), MIA	1956 Design Specifications of Steel Highway Bridges, MOC	1964 Design Specifications of Substructures (Pile Foundations), MOC	1964 Design Specifications of Steel Highway Bridges, MOC	1966 Design Specifications of Substructures (Survey and Design), MOC	1968 Design Specifications of Substructures (Piers and Direct Foundations), MOC	1970 Design Specifications of Substructures (Caisson Foundations), MOC	1971 Specifications for Seismic Design of Highway Bridges, MOC	1972 Design Specifications of Substructures (Cast-in-Piles), MOC	1975 Design Specifications of Substructures (Pile Foundations), MOC	1980 Design Specifications of Highway Bridges, MOC	1990 Design Specifications of Highway Bridges, MOC
Seismic Loads	Seismic Coefficient	Largest Seismic Loads	kh=0.2 varied dependent on the site	kh=0.1 - 0.35 varied dependent on the site and ground condition						kh=0.1 - 0.3 Standardization of Seismic Coefficient Provision of Moditied Seismic Coefficient Method			Revision of Application range of Modified Seismic Coefficient Method	kh=0.1-0.3 Integration of Seismic Coefficient Method and Modified one
	Dynamic Earth Pressure	Epuations proposed by Mononobe and Okabe were supposed to be used.					Provision of Dynamic Earth Pressure							
	Dynamic Hydraulic Pressure	Less Effect on Plers except High Plers in Deep Water.						Provision of Hydraullc Pressure		Provision of Dynamic Hydraulic Pressure				
RC Column	Bending at Bottom	Supposed to be designed in a similar way provided in current Desigin Specifications						Provisions of Definite Design Method						
	Shear	Less Effect on RC Plers except those with Smaller section area such as RC Frame and Hollow Section						Check of Shear Strength					Provision of Definite Design Method, Decreasing of Allowable Shear Stress	
	Tremination of Main Rebar at Mid-Height											Elongation of Anchorage Lenght of Terminated Reinforcement at Mid-Height		
	Bearing Capacity for Lateral Fore							Less Effect on RC Piers with Larger Section Area					Ductility Check Check of Bearing Capacity for Lateral Force	
Footting							Provisions of Definite Design Method (Designed as a Cantilever Plate)						Provisions of Effective width and Check of Shear Strength	
Plle Foundation			Bearing Capacity in vertical direction was supposed to be checkd.		Provisions of Definite Design Method (Bearing Capacity in vertical and horizontal directions)					Special Condition(Foundation on Slope, Consolldation Settiement,Lateral Movement)		Provising of Design Delails for Pile Head		
Direct Foundation			Stability (Overturning and Sllp)was supposed to be checked.				Provisions of Definite Design Method (Bearing Capacity,Stability Analysis)							
Caisson Foundation					Supposed to be Designed in a similar way provided in Design Specification of Caisson Foundation of 1969				Provisions of Definite Design Method					
Soil Liquefaction									Provisions of Soil Layers of which Bearing Capacity shall be ignored in seismic design				Provisions of Evaluation Method of Soil Liquefaction and The Treatment in seismic design	Consideration of effect of fine sand content
Bearing Support	Bearing Support		Provisions of Design Method for Steel Bearing Supports(Bearing, Roller, Anchor Bolt)						Provision of Transmitting Method of Seismic Load at Bearing					
	Devices Preventing Falling-off of Superstructure							Provisions of Bearings Seat Length S	Provisions of Stopper at Movable Bearings,Devices for Preventing Superstructure from Falling(Seat Length S, Connection of Adjacent Decks)			Provisions of Stopper at Movable Bearings, Devices for Provention Superstructure from Falling(Seat Length S, Devices)		

Table 2
Past seismic evaluations of highway bridges[a]

Year	Highways inspected	Inspected items	Number of bridges		
			Inspected	Require strengthening	Strength-ened
1971	All sections of national expressways and national highways, and sections of the others (bridge length $\geqslant$ 5 m)	1. Deterioration 2. Bearing seat length S for bridges supported by bent piles	18 000	3200	1500
1976	All sections of national expressways and national highways, and sections of the others (bridge length $\geqslant$ 15 m or overpass bridges)	1. Deterioration of substructures, bearing supports and girders/slabs 2. Bearing seat length S and devices for preventing falling-off of superstructure	25 000	7000	2500
1979	All sections of national expressways, national highways and principal local highways, and sections of the others (bridge length $\geqslant$ 15 m or overpass bridges)	1. Deterioration of substructures and bearing supports 2. Devices for preventing falling-off of superstructure3. Effect of liquefaction 4. Bearing capacity of soils and piles 5. Strength of RC piers 6. Vulnerable foundations (bent pile and RC frame on two independent caisson foundations)	35 000	16 000	13 000
1986	All sections of national expressways, national highways and principal local highways, and sections of the others (bridge length $\geqslant$ 15 m or overpass bridges)	1. Deterioration of substructures, bearing supports and concrete girders 2. Devices for preventing falling-off of superstructure 3. Effect of soil liquefaction 4. Strength of RC piers (bottom of piers and termination zone of main reinforcement) 5. Bearing capacity of piles6. Vulnerable foundations (bent piles and RC frame on two independent caisson foundations	40 000	11 800	8000
1991	All sections of national expressways, national highways and principal local highways, and sections of the others (bridge length $\geqslant$ 15 m or overpass bridges)	1. Deterioration of substructures, bearing supports and concrete girders 2. Devices for preventing falling-off of superstructure 3. Effect of soil liquefaction 4. Strength of RC piers (piers and termination zone of main reinforcement) 5. Vulnerable foundations (bent piles and RC frame or two independent caisson foundations)	60 000	18 000	7000 (as of the end of 1994)

[a] *Note:* Number of bridges inspected, number of bridges that required strengthening and number of bridges strengthened are approximate numbers.

was much larger than anticipated in the codes. It occurred very close to the Kobe City with shallow focal depth.

Damage was developed at highway bridges on routes 2, 43, 171 and 176 of the National Highway, route 3 (Kobe Line) and route 5 (Wangan Shore Line) of the Hanshin Expressway, the Meishin and Chugoku Expressway. Damage was surveyed by the "Committee for Investigation on the Damage of Highway Bridges caused by the Hyogo-ken Nanbu Earthquake" for all bridges on National Highways, Hanshin Expressways, Meishin and Chugoku Expressways in the area where destructive damage occurred. Total number of piers surveyed reached 3396 [1].

Table 3
Factors which affect seismic vulnerability of highway bridges

Items	Seismic vulnerability
1. Design specifications	Those designed in accordance with 1926 or 1939 specifications have higher vulnerability
2. Type of superstructure	• Gerber or simply supported girders or more spans have higher vulnerability • Arch. frame, continuous girders, cable-stayed bridges or suspension bridges have lower vulnerability
3. Shape of superstructure	Skewed or curved bridges do not necessarily have higher vulnerability than straight bridges
4. Materials of superstructure	Reinforced concrete bridges or prestressed concrete bridges have lower vulnerability than steel bridges although the difference is small
5. Slope in bridge axis	Bridges with slope in bridge axis have higher vulnerability
6. Device for preventing falling-off of superstructure	Bridges with devices for preventing falling-off of superstructure have lower vulnerability
7. Type of substructure	Bridges supported by single-line bent piles or by reinforced concrete frame placed on two separate caisson foundations have higher vulnerability
8. Height of piers	Bridges supported by higher piers have higher vulnerability
9. Ground condition	Bridges constructed on soft soil have higher vulnerability
10. Effect of soil liquefaction	Bridges constructed on sandy soil layers susceptible to liquefaction have higher vulnerability
11. Irregularity of supporting soil condition	Bridges constructed on soils with irregularity of supporting conditions have higher vulnerability
12. Effect of scouring	Bridges where the surface soils are scoured have higher vulnerability
13. Materials of substructures	Bridges supported by plane-concrete substructures designed in accordance with 1926 or 1939 specifications have higher vulnerability
14. Type of foundation	Bridges supported by timer, brics, masonry or other old unknown type substructures have higher vulnerability
15. Intensity of ground motion	Bridges subjected to higher intensity of ground acceleration have higher vulnerability. In particular, vulnerability becomes quite high when the bridges are subjected to peak ground acceleration larger than 400 gal (0.4 g)

The committee concluded the following based on the investigations of the damage to highway bridges:

1. Based on the strong motion records and earthquake response analyses of the ground, the effect of the horizontal ground motion by the earthquake on the structures was the largest after the Niigata Earthquake of 1964 when the strong motion observation was initiated. The level of the ground motion was larger than that considered in the practical design. The strong motion was also observed in the vertical direction.
2. There were reinforced concrete piers that were heavily damaged from flexure to shear at mid-height where some of the longitudinal re-bars were terminated without enough development length. Those piers were designed before 1980. These bridges were also damaged to the bottom of the piers. Based on the analysis of the relation between the design code and the damaged piers, 14% of the total piers were heavily damaged on route 3 (Kobe Route) of Hanshin Expressway which were designed according to 1964 and 1971 specifications. Heavy damage was not found on route 5 (Wanagn Route) of Hanshin Expressway which were designed according to 1980 and 1990 specifications.
3. There were steel bridge piers that suffered local buckling at the web and flange of rectangular section caused by the horizontal earthquake force. Then the fracture at the corner welding occurred and the deck was subsided by the decrease of vertical strength of piers.
4. Most of damages to superstructures were caused by the damage to bearing supports. In addition, there were some damages to fixing portion of the restrainers.
5. Some devices to connect adjacent girders were not effective to prevent unseating of superstructures.
6. Many damages such as fracture of set bolts, damage of bearing itself, dislodgment of roller and fracture of anchor bolts, were found at the steel bearings. Damage to rubber bearings was much smaller than that to steel bearings.
7. Further study should be made on the effect of ground flow on bridges. Ground with larger particles, such as gravel sand that is not required to check the liquefaction in the previous code, was liquefied. Liquefaction-induced ground flow was also found and some bridge foundations were affected by the ground flow.

4. Seismic retrofit program after the Hyogo-ken Nanbu Earthquake

4.1. Seismic design for reconstruction and repair

For seismic design of reconstruction of highway bridges that suffered damage due to the Hyogo-ken Nanbu Earthquake, the “Guide Specifications for

Table 4
Inspection sheet to evaluate seismic vulnerability of highway bridges

Point of inspection		Factors of inspection	Evaluation		
Inspection for vulnerability to develop excessive deformation	Inspection format (A)	(1) Design specifications	4.0: 1926 Specs. or 1939 Specs.	2.0: 1956 Specs. or 1964 Specs.	1.0: 1971 Specs. or 1980 Specs.
		(2) Superstructure type	3.0: Gerber girder or simply supported girders with two spans or more	1.5: Simply-supported girder of continuous girders consisting of two spans or more	1.0: Arch. flame continuous girder (one span), cable-stayed bridge, suspension bridge
	Deformation of super structure	(3) Shape of superstructure	1.2: Skewed or curved bridge	1.0: Straight bridge	
		(4) Materials of superstructure	1.2: RC or PC	1.0: Steel	
		(5) Gradient	1.2: 6% or Steeper	1.0: Less than 6%	
		(6) Unseating prevention device	2.0: None	1.0: One device	
		$P_A = (1) \times (2) \times (3) \times (4) \times (5) \times (6)$	$P_A =$		
	Inspection format (B)	(7) Type of sub structure	2.0: Single-line bent pile foundation	1.0: Others	
		(8) Height of pier H	2.0: $H \geqslant 10$ m 1.5: $5 \leqslant H < 10$ m1.0: $H < 5$ m		
		(9) Ground condition	5.0: Extremely soft in Group 4 2.5: Group 42.0: Group 31.2: Group 21.0 Group 1		
		(10) Effects of liquefaction	2.0: Liquefaction	1.0: Non-liquefiable	
	Inspection for deformation of substructure	(11) Supporting ground condition	1.2: Irregular	1.0: Almost uniform	
		(12) Scouring	1.5: Recognized	1.0: None	
		$P_B = (7) \times (8) \times (9) \times (10) \times (11) \times (12)$	$P_B =$		

Inspection for vulnerability to develop failure due to inadequate strength of substructure	Inspection format (C)	(13) Shear span ratio (h/D)	2.0: $1 < h/D < 4$	1.0: $h/D \geqslant 4$	0.5: $h/D \leqslant 1$
		(14) Tension cracks in flexure at terminated point of main reinforcement	2.0: Cracks will occur	1.0: Cracks will possibly occur	0.3: Cracks will not occur
	For strength of RC pier at termination of reinforcement	(15) Safety factor terminated(15)-1 Sfn	3.0: Sfn $\leqslant 1.1$	2.0: $1.1 <$ Sfn < 1.5	0.5: Sfn $\geqslant 1.5$
		Section of main reinforcement(15)-2 Smm	3.0: Smm $\leqslant 1.1$ 2.0: $1.0 <$ Smm $< 1.3$1.0: $1.3 <$ Smm $< 1.5$0.5: $S \geqslant 1.5$		
		(16) Shear Stress σ (tf/m^2)	3.0: $\sigma \geqslant 45$2.0: $30 \leqslant \sigma < 45$1.0: $15 \leqslant \sigma < 30$0.5: $\sigma < 15$		
		$P_C = (13) \times (14) \times (15)\text{-}1 \times (15)\text{-}2 \times (16)$	$P_C =$		
	Inspection format (D)	(17) Failure of fixed supports and proximity	5.0: Extensive failure	2.0: Small failure	1.0: None
		(18) Extraordinary damage of pier	5.0: Extensive damage	2.0: Small damage	1.0: None
		(19) Materials of substructure	2.0: Plane concrete older than 1926 excluding gravity-type abutment		1.0: Others
	Inspection for strength of substructure	(20) Construction method of foundation	2.0: Timer pile, masonry, bricks, other old construction methods	1.5: RC piles, pedestal piles, pier supported by two independent caissons	1.0: Foundation designed by 1971 specs and other later specs
		(21) Foundation type	1.5: RC flame supported by two independent caisson foundations		1.0: Others
		(22) Extraordinary failure of foundation	2.0: Recognized	1.0: None	
		$P_D = (17) \times (18) \times (19) \times (20) \times (21) \times (22)$	$P_D =$		
Evaluation of deformation and strength			$X = P_A \times P_B =$ and $Y = P_C \times P_D =$		

Reconstruction and Repair of Highway Bridges which suffered damage due to the Hyogo-ken Nanbu Earthquake" [2] was issued by the Ministry of Construction on 27 February 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 applied only for reconstruction and repair of the highway bridges that suffered damages due to the Hyogo-ken Nanbu Earthquake.

The bridges shall be designed so that they can resist with enough structural safety against the earthquake force developed during Hyogo-ken Nanbu Earthquake. To achieve this goal, the 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 verification 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 Hyogo-ken Nanbu 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 (Seismic Isolation) Design for distributing lateral force of superstructure to substructures. 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.
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 in-filled 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 further use rubber bearings because they absorb relative displacement 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 unseating of a superstructure from substructures shall be designed so that they can avoid unseating 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.

4.2. Reference for applying guide specifications to new highway bridges and seismic retrofit of existing highway bridges

For increasing seismic safety of the highway bridges that suffered damage by the Hyogo-ken Nanbu 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 Hyogo-ken Nanbu Earthquake. Although intensified review of design could be made when it was applied to the bridges only in the Osaka and Kobe area, it may not be so easy for practical design engineers to following up the new guide specifications when the guide specifications is used for seismic design of all new highway bridges and seismic strengthening of existing highway bridges. Based on such demand, the Reference for Applying the Guide Specifications to New Bridges and Seismic Strengthening of Existing Bridges [13] was issued on 30 June 1995 by the Sub-Committee for Seismic Countermeasures for Highway Bridges, Japan Road Association.

The reference classified the application of the guide specifications as shown in Table 5 based on the importance of the roads. All items of the guide specifications are applied for bridges on extremely important roads, while some items that prevent brittle failure of structural components are applied for bridges on important roads. For example, the items for Menshin design, tie reinforcements, termination of longitudinal reinforcements, type of bearings, unseating prevention devices and countermeasures for soil liquefaction are applied for bridges on the important roads, while the remaining items such as the design force, concrete in-filled steel bridges, and ductility check for foundations are not applied [14].

Table 5
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 expressway, Honshu-Shikoku bridges, designated national highways	Apply all items, in principle	Apply all items, in principle
Non-designated national highways, prefectural roads, city, town and village roads	Apply all items, in principle	Apply partially, in principle

Because damage concentrated to single reinforced concrete piers/columns with small concrete section, the seismic retrofit program has initiated for those columns, which were designed by the pre-1980 Design Specifications, at extremely important bridges such as bridges on expressways, urban expressways, and designated highway bridges, and also double-deckers and over-crossings, etc. which significantly affect highway functions once damaged. The program is a 3-year program since 1995 and approximately 30 000 piers were evaluated and retrofitted by the end of 1997 fiscal year. Unseating devices also should be installed for these extremely important bridges.

5. Seismic evaluation and retrofit of highway bridges

5.1. Prioritization concept for seismic evaluation

The 3-year retrofit program will be completed in 1997 fiscal year. In the program, the single reinforced concrete piers/columns with small concrete section that were designed by the pre-1980 Design Specifications on important highways have been evaluated and retrofitted. The other bridges with wall-type piers, steel piers, and framed piers and so on as well as the bridges on the other highways should be evaluated and retrofitted if required in the next retrofit program. Since there are approximately 200 000 piers, it is required to develop the prioritization methods and the evaluation methods of the vulnerability for the intentional retrofit program.

Fig. 1 shows the simple flow chart to give the prioritization of the retrofit works to bridges. The importance of highway, structural factor, member vulnerability (reinforced concrete piers, steel piers, unseating prevention devices, foundations) are the factors to be considered for the prioritization.

Priority R of each bridges may be evaluated by the following equation:

$$R = I \cdot S \cdot V_{\mathrm{T}} \cdot w_{\mathrm{v}} \cdot (f(V_{\mathrm{RP1}}, V_{\mathrm{RP2}}, V_{\mathrm{RP3}}), V_{\mathrm{MP}}, V_{\mathrm{FS}}, V_{\mathrm{F}}) \times 100, \tag{1}$$

$$f(V_{\mathrm{RP1}}, V_{\mathrm{RP2}}, V_{\mathrm{RP3}}) = V_{\mathrm{RP1}} \cdot V_{\mathrm{RP2}} \cdot V_{\mathrm{RP3}}, \tag{2}$$

in which R is the priority, I the importance factor, S the earthquake force, V_{T} the structural factor, w_{v} the weighting factor on structural members, V_{RP1} the design specification, V_{RP2} the pier structural factor, V_{RP3} the aspect ratio, V_{MP} the steel pier factor, V_{FS} the unseating device factor, and V_{F} is the foundation factor.

The each item and category with a weighting number is tentatively shown in Table 6. If applied this prioritization method to the bridges damaged during the Hyogo-ken-Nanbu Earthquake, the categorization number is given as shown in Table 6.

5.2. Seismic retrofit of single column bent

Main purpose of the seismic retrofit of reinforced concrete columns is to increase their shear strength, in particular in the piers with termination of longitudinal reinforcements at the mid height without enough development length. This enhances the ductility of columns because premature shear failure could be avoided.

However, if only ductility of piers is enhanced, residual displacement 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. 2. Epoxy resin or shrinkage-compensation mortar is 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 excessive increase in

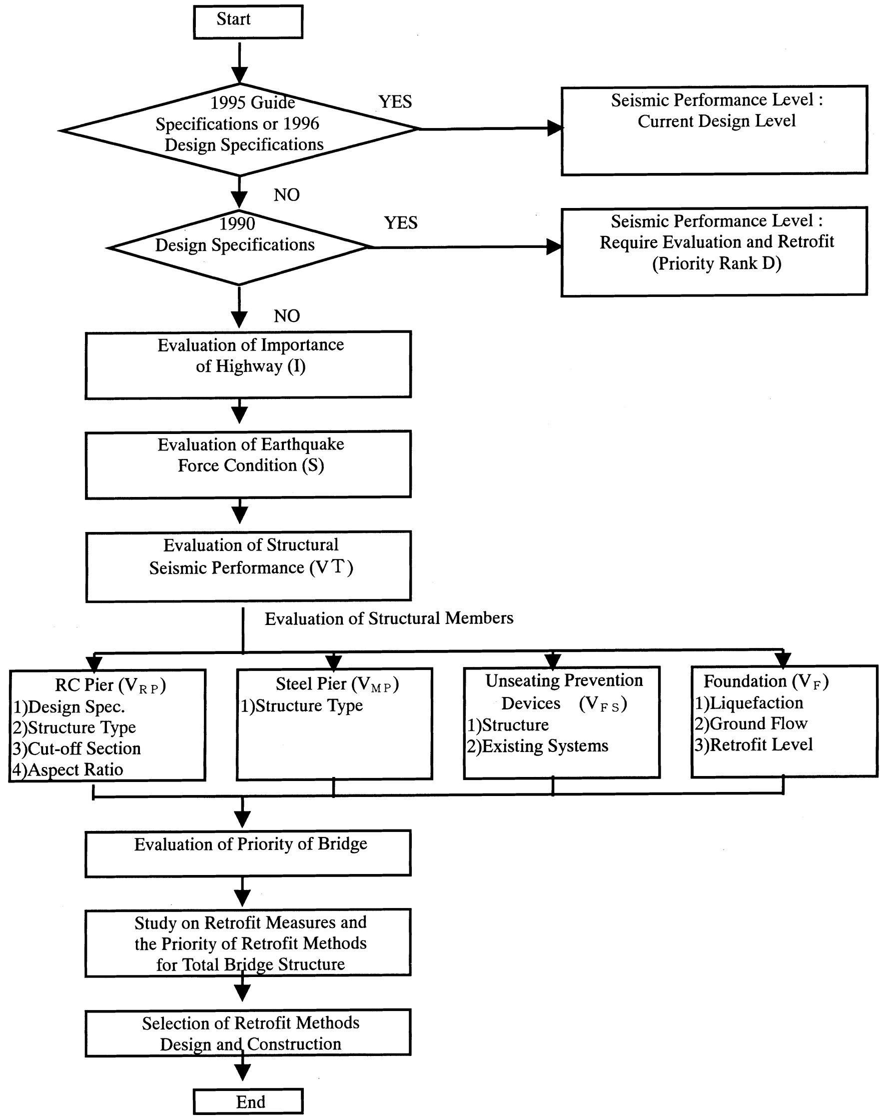

Fig. 1. Prioritization concept of seismic retrofit of highway bridges.

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. Piers with a rectangular section also have H-beams installed around them at the lower end of the jacket. This prevents the bulging of longitudinal bars and keeps the confining effect of the jacket.

In order to verify the effectiveness of this retrofit method, cyclic loading tests were carried out to examine the seismic behavior of as-built and retrofit reinforced concrete columns [15].

Fig. 3 shows the details of the tested specimen. The cross-section of the specimen was a square of 60 cm × 60 cm. The shear span ratio was 5.0. The reinforcing details are shown in Fig. 3. For the retrofit specimen, a thickness of 1.6 mm plate was installed with a vertical gap of 10 cm to the footing. In addition, H-beams were set to strengthen the lower end of the jacket. Epoxy resin was injected between the reinforced concrete pier and the steel jacket. Anchor bars were arranged to increase flexural strength of as-built specimen by 30%. The applied axial load was 150 N/cm^2. The test sequence consisted of three cycles of $1\delta y$, $2\delta y$, $3\delta y$ and so on in displacement control. The displacement was continued to increase until the test specimen caused the serious damage such as a fracture of the longitudinal reinforcement.

Fig. 4 compares the hysteresis loops of lateral load and displacement relation between the as-built specimen and the retrofitted. In the case of as-built specimen, the

Table 6
Example of prioritization factors for seismic retrofit of highway bridges

Item	Category	Evaluation point
Importance of highway (I)	(1) Emergency routes	1.0
	(2) Overcrossing with emergency routes	0.9
	(3) Others	0.6
Earthquake force (S)	(1) Ground condition type I	1.0
	(2) Ground condition type II	0.9
	(3) Ground condition type III	0.8
Structural factor (V_T)	(1) Viaducts	1.0
	(2) Supported by abutments at both ends	0.5
Weighting factor on structural members (V_T)	(1) Reinforced concrete pier	1.0
	(2) Steel pier	0.95
	(3) Unseating prevention devices	0.9
	(4) Foundation	0.8
Reinforced concrete pier (1), design specification (V_{RP1})	(1) Pre-1980 design specifications	1.0
	(2) Post-1980 design specifications	0.7
Reinforced concrete pier (2), pier structure (V_{RP2})	(1) Single column	1.0
	(2) Wall-type column	0.8
	(3) Two-column bent	0.7
Reinforced concrete pier (3), aspect ratio (V_{RP3})	(1) $h/D \leqslant 3$	1.0
	(2) $3 < h/D < 4$ with cut-off section	0.9
	(3) $H/D \geqslant 4$ with cut-off section	0.9
	(4) $3 < h/D < 4$ without cut-off section	0.7
	(5) $H/D \geqslant 4$ without cut-off section	0.7
Steel pier (V_{MP})	(1) Single column	1.0
	(2) Frame structure	0.8
Unseating prevention devices (V_{FS})	(1) Without unseating devices	1.0
	(2) With one device	0.9
	(3) With two devices	0.8
Foundations (V_F)	(1) Vulnerable to ground flow (without unseating devices)	1.0
	(2) Vulnerable to ground flow	0.9
	(3) Vulnerable to liquefaction (without unseating devices)	0.7
	(4) Vulnerable to liquefaction	0.6

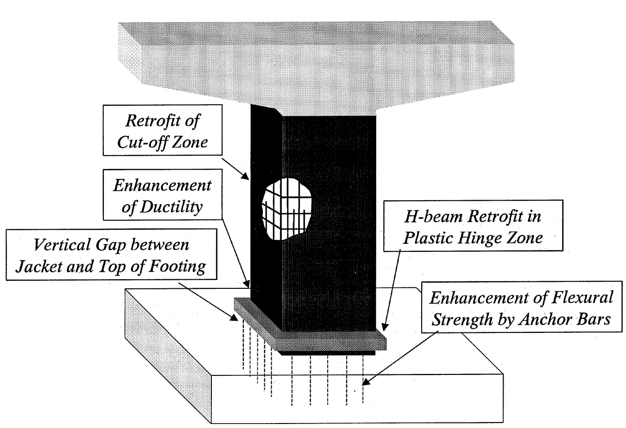

Fig. 2. Seismic retrofit of reinforced concrete piers by steel jacket with controlled increase of flexural strength method.

peak strength was 227 kN. This was maintained until $4\delta y$ was loaded, when its cover concrete started to spall off. In the stage of $5\delta y$ loading, the core concrete started to suffer damage, and the hysteresis loop began to become unstable.

In the case of the retrofitted specimen, the yield displacement was smaller than that of the as-built one, because the flexural stiffness was increased by the steel jacket. The peak strength was 311 kN. This was maintained until $6\delta y$ was loaded. Its anchor bars started to buckle under the load of $4\delta y$. In the stage of the $6\delta y$ loading, some anchor bars were broken. At the same time, the hysteresis loop began to become unstable. According to these experimental results, it is verified that the retrofit method introduced here successfully enhances the flexural strength as well as the ductility of the specimen.

Table 7 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,

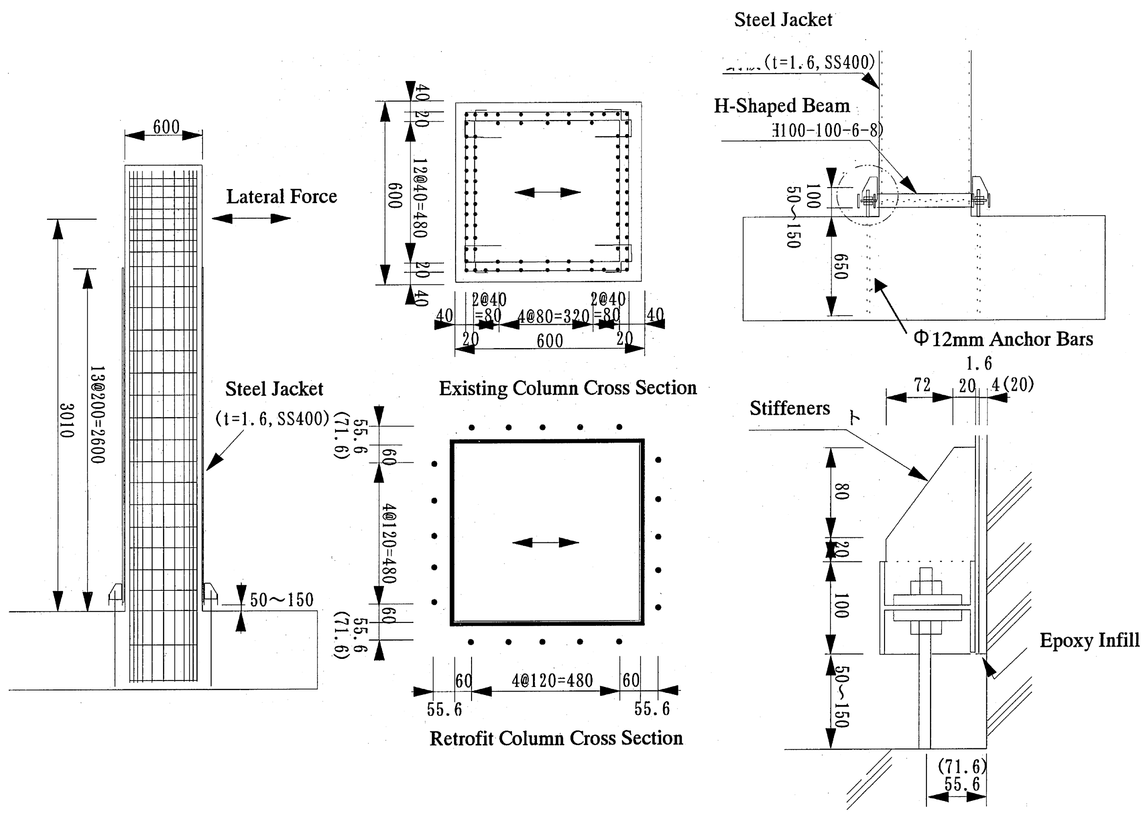

Fig. 3. Details of the cyclic loading test specimens for as-built and retrofit reinforced concrete piers.

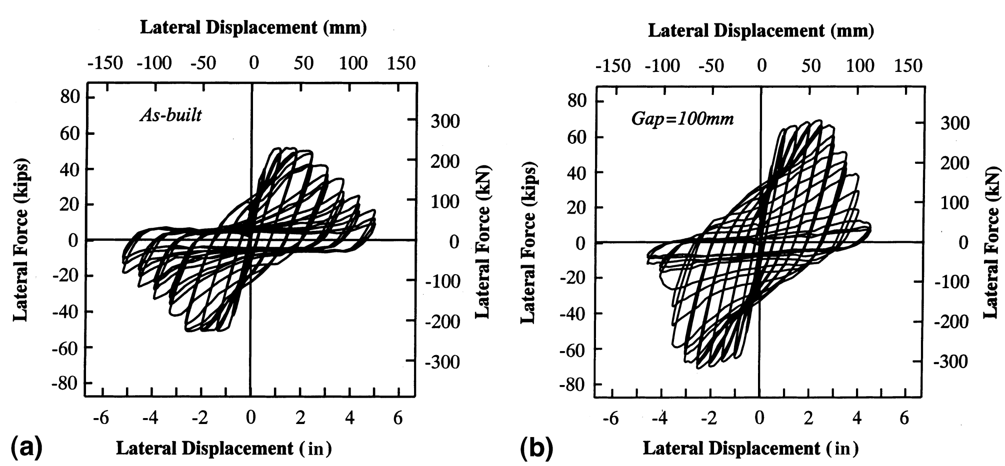

Fig. 4. Hysteresis loops of lateral load and displacement relation of as-built and retrofit piers: (a) as-built specimen and (b) retrofit specimen.

Table 7
Tentative retrofit method by steel jacketing

Column/piers	Steel jackets	Anchor bolts
$a/b \leqslant 2$	SM400, $t = 9$ mm	SD295, D35 ctc 250 mm
$2 < a/b \leqslant 3$	SM400, $t = 12$ mm	
Column supporting lateral force of a continuous girder through fixed bearing and with $a/b \leqslant 3$		

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 ratio of flexural strength of columns is less than about 2.

Conventional reinforced concrete jacketing methods are also suggested for the retrofit of reinforced concrete piers, especially for the piers that require the increase of strength. It should be noted here that the increase of the strength of the pier should carefully be designed in consideration with the strength of foundations and footings.

5.3. Seismic retrofit of wall-type piers

The steel jacketing method as described in the above was applied for reinforced concrete with circular section or rectangular section of $a/b < 3$. It is required to develop the seismic retrofit method for wall-type piers. The confinement of concrete was provided by a confinement beam such as H-shaped beam for rectangular piers. However, since the size of the confinement beam become very large, the confinement may be provided by other measures such as intermediate anchors for wall-type piers.

The seismic retrofit concept for wall-type piers is the same as that for rectangular piers. It is important to increase the flexural strength and ductility capacity with the appropriate balance. Generally, the longitudinal reinforcement ratio is smaller than that for rectangular piers, therefore the flexural strength is smaller. Therefore, it is essential to increase the flexural strength appropriately. Since the longitudinal reinforcement was generally terminated at mid-height without appropriate anchorage length, it is also important to increase both of flexural and shear strength mid-height section.

Fig. 5 shows the suggested seismic retrofit method for wall-type piers. To increase the flexural strength, the additional reinforcement by re-bars or anchor bars are arranged and fixed to the footing. The number of reinforcement is designed to give required flexural strength. It should be noted here that anchoring of additional longitudinal reinforcement is controlled to develop plastic hinge to the bottom of pier rather than the mid-height section with termination of longitudinal reinforcement. And the increase of strength should be carefully designed considering the effect on the foundations and footings. The confinement in the plastic hinge zone is provided by PC bars or re-bars which were installed inside of the column section.

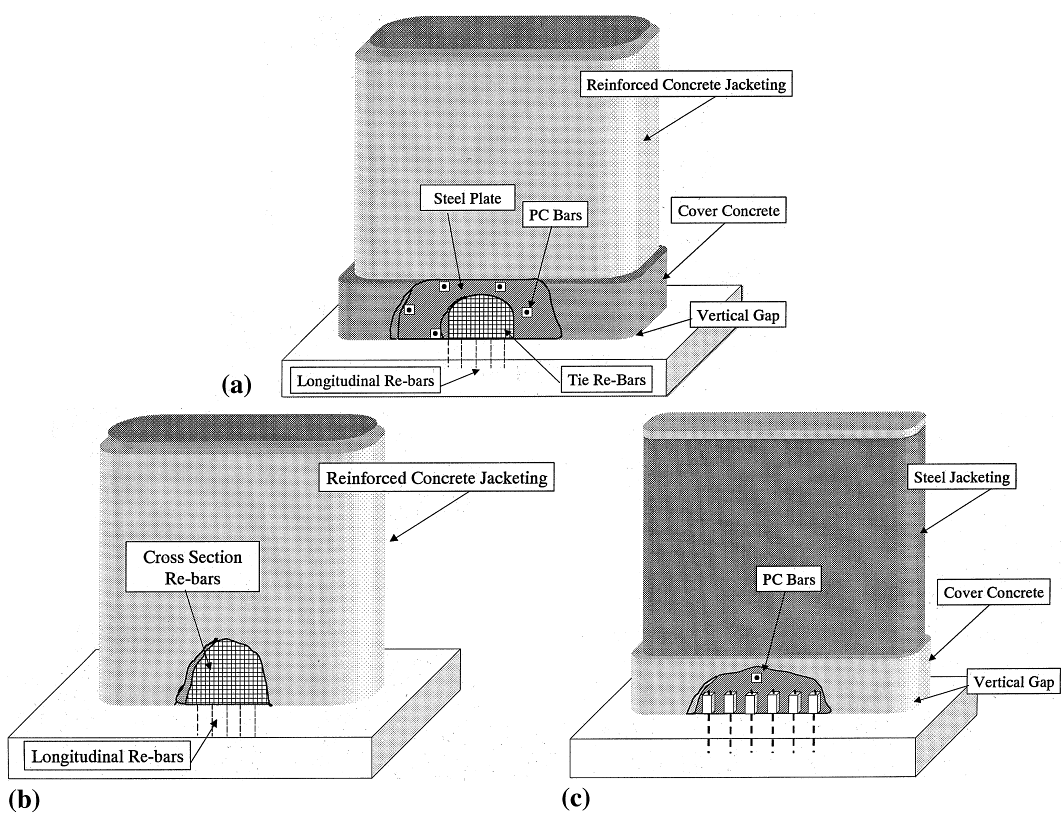

Fig. 5. Seismic retrofit of wall-type piers by concrete jacketing method with tie-bars: (a) integrated seismic retrofit method with reinforced concrete and steel jacketing, (b) reinforced concrete jacketing and (c) steel jacketing.

In order to verify the effectiveness of this retrofit method, cyclic loading tests were also carried out to examine the seismic behavior of as-built and retrofit specimen. Among the three methods shown in Fig. 5, the test results for the concrete jacketing method are shown below [16].

Fig. 6 shows the specimen details. The cross-section of the specimen was 40 cm × 188 cm. The height was 227 cm to the loading point. The reinforcing arrangement is also shown in Fig. 6. For the retrofit specimen, a thickness of 13 cm concrete jacket was provided and additional longitudinal reinforcement and hoop reinforcement were installed in the jacketed concrete. Tie bars were provided and penetrated into the pier to tie up the both sides of the each longitudinal reinforcing bar at the expected plastic hinge zone. The loading procedure is the same as the previous one.

Fig. 7 compares the hysteresis loops of lateral load and displacement relation between the as-built specimen and the retrofitted. Taking a look at the as-built specimen, spall-off of the cover was noticed at the bottom of the column at $5\delta y$. The cover concrete continued to spall-off at the bottom of the column and longitudinal reinforcement deformed and buckled at $6\delta y$. As for the retrofit specimen, spall-off of the cover concrete was noticed at $8\delta y$. The fracture of some of the longitudinal reinforcement was also observed at $8\delta y$. According to these results, the effectiveness of the ductility enhancement of the wall-type pier was verified.

5.4. Seismic retrofit of two-column bents

During the Hyogo-ken Nanbu Earthquake, some two-column bents were damaged in the longitudinal and transverse directions. The strength and ductility characteristics of the two-column bents have been studied and the analysis and design method was introduced in the 1996 Design Specifications.

The strength and ductility of existing two-column bents were studied both in the longitudinal and transverse directions. In the longitudinal direction, as the same as single columns, it is required to increase the flexural strength and ductility with appropriate balance. In the transverse direction, the shear strength of the columns or the cap beam is generally not enough in comparison with the flexural strength.

Fig. 8 shows the suggested possible seismic retrofit methods for two-column bents. The concept of the retrofit is to increase flexural strength and ductility as well as shear capacity for columns and cap beams. In the

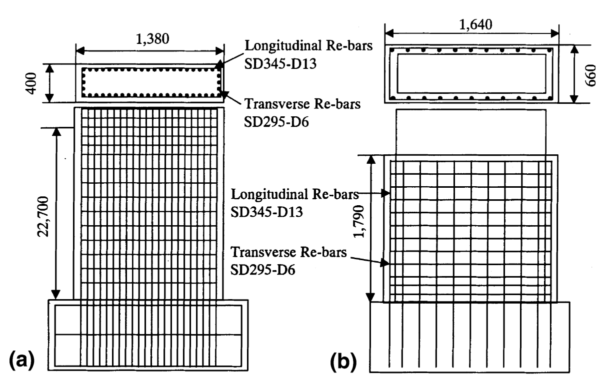

Fig. 6. Details of the cyclic loading test speciments for as-built and retrofit wall-type piers: (a) as-built specimen and (b) retrofit specimen.

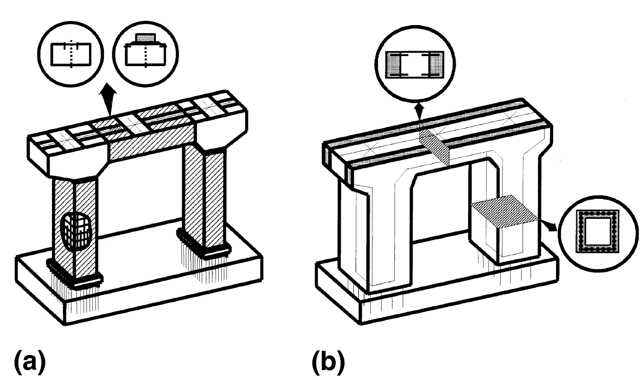

Fig. 8. Seismic retrofit of two-column bents with controlled increase of flexural strength method: (a) steel jacketing and (b) reinforced concrete jacketing.

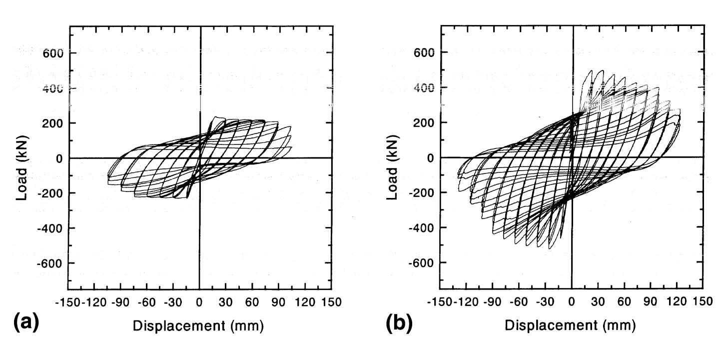

Fig. 7. Hysteresis loops of lateral load and displacement relation of as-built and retrofit wall-type piers: (a) as-built specimen and (b) retrofit specimen.

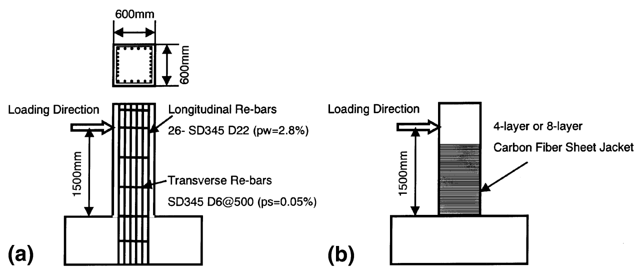

Fig. 9. Details of the cyclic loading test speciments for cap beams by carbon fiber sheets: (a) as-built specimen and (b) 4-layer or 8-layer retrofit specimen.

field practices, axial force in the cap beam is much smaller than that in the columns so that the enhancement of the shear capacity for the retrofit of the cap beam is more essential. It should be noted here that since the jacketing of a cap beam is difficult because of the existing bearing supports and construction space, it is required to develop much effective retrofit measures for cap beam such as application of jacketing by new materials with high elasticity and high strength and external-cable prestressing, etc. New materials such as carbon fiber sheets and aramid fiber sheets are attractive to be applied for the seismic retrofit of cap beams. Since new materials such as fiber sheets are very light so there is no need to use machines and its is easy to be constructed using glue bond such as epoxy resin.

In order to verify the effectiveness of shear strength enhancement of cap beams by carbon fiber sheets jacketing method, cyclic loading tests were also carried out [17].

Fig. 9 shows the details of the specimen. In order to perform cyclic loading test, just half-length of the cap beam was modeled. The cross-section of the specimen was 60 cm × 60 cm. The height was 150 cm to the loading point that is the half-length of the scaled cap beam. The reinforcing arrangement is also shown in Fig. 9. For the retrofit specimen, 4 or 8 layer carbon fiber sheets were provided and glued. The carbon fiber sheet used in this test has the properties as the unit weight is 175 g, the thickness is 0.0972 mm, and the tensile strength is 249 kN/cm^2.

Fig. 10 shows the hysteresis loops of lateral force and displacement relation between as-built specimen and two retrofitted specimens. Taking a look at the as-built specimen, shear failure was developed before the yielding of the longitudinal rebars at the bottom. Both 4 layered and 8 layered retrofitted specimens had enough shear strength and expected to be failed in flexture if 2/3 of the tensile strength of the carbon fiber sheet can corporate to resist against shear force. However, 4 layered retrofitted specimen was failed in shear eventually and the 8 layered retrofitted specimen was successfully failed in flexture. This is because since the elastic modulus of the carbon fiber is almost the same as that of the steel rebars the certain shear deformation was developed to achieve the full strength of the high strength materials. Therefore, it is essential to appropriately evaluate the contribution of the carbon fiber sheets to the shear strength enhancement. In this study, in order to transfer the failure mode from shear to flexture, the design effective strain of the carbon fiber sheets is evaluated as the value between 1673μ and 3413μ. Those values are almost 1/4 to 1/8 of the rupture stain of the sheet.

Based on those findings, retrofit using carbon fiber sheets now can be gradually seen in the field.

6. Conclusions

This paper presented seismic retrofit of existing highway bridges with emphasis on the program after the Hyogo-ken Nanbu Earthquake. Because most of the substructures designed and constructed before 1971 do not meet with the current seismic requirements, it is urgently needed to study the level of seismic vulnerability requiring the retrofit. Upgrading of the reliability to predict the possible failure modes in the future earthquakes is also very important. Since the seismic retrofit of substructures requires more cost, it is required to develop and implement the effective and inexpensive retrofit measures and the design methods to provide for next event.

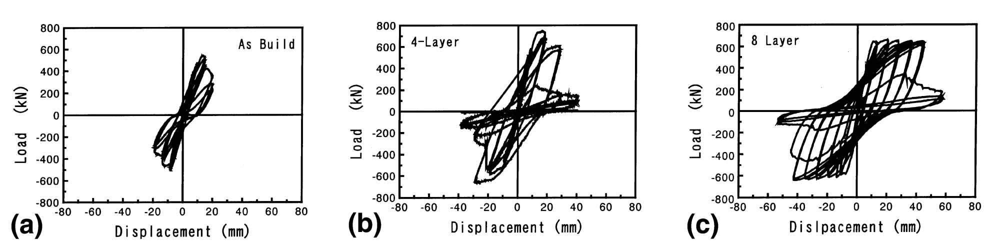

Fig. 10. Hysteresis loops of lateral load and displacement relation of as-built and retrofit calp beams: (a) as-built specimen, (b) 4-layer retrofit specimen and (c) 8-layer retrofit specimen.

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