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Restoration of Hanshin Expressway after Kobe/Awaji Earthquake – challenge of 623 days before opening

Hidenao Hayashi *, Tamotsu Marui, Nobuhiko Taniguchi, Shigeru Kayano

Hanshin Expressway Public Corporation, 4-1-3 Kyutaro-machi Chuo-ku, Osaka, Japan

Abstract

Kobe/Awaji Earthquake in 1995, called 'The Great Hanshin Earthquake', occurred in heavily populated area of Kobe City and took away the lives of more than 6000 people. Hanshin Expressway Route 3 running through this area was also damaged hideously and forced out of service. The strategy of its restoration was not only to increase the seismic structural safety but also to improve the environmental safety along the route. As it is a lifeline route, the restoration was demanded to complete as soon as possible to save the economy of Kobe City. The route of 27.7 km was back into full-service in 623 days after the earthquake with new structural designs. In this report, the outline of the restoration work is described focusing on some newly adopted structures such as hybrid pier and environmental measures of low-noise pavement or new noise barriers.

Keywords: Hanshin Expressway; Rehabilitation; Seismic safety; Design; Analysis; Environment

1. Damages from the earthquake

Kobe/Awaji Earthquake in 1995 struck the Kobe City and Awaji Island severely, and Hanshin Expressway Route 3 was also suffered a significant damage during the earthquake. One 635-meter elevated section was toppled in Higashi–Nada ward (see Fig. 1), and four other sections along Route 3 were collapsed. An emergency inspection was done immediately after the earthquake. It was found that 604 of the 1106 piers and 551 of the 1304 elevated sections were distressed in the 27.7 km stretch between Mukogawa and Tsukimiyama in Hyogo Prefecture as shown in the map of Fig. 2. Damaged structures were ranked on a scale between As, A, B, C, and D, according to the criteria specified in the Handbook of Earthquake Countermeasures for Roads (see Figs. 3–5).

15% of the 943 concrete piers were judged As or A that means functional damage, and 35% were judged B or C meaning relatively light damage. Besides, the number of severely affected steel piers were relatively low at 11, buckling or cracking around accessholes and peeling of coatings on welds were observed in about 80% of steel piers.

As for the superstructure, Route 3 consisted of 1143 spans of steel girder bridges and 161 spans of concrete girder bridges. 75 spans of steel girders were ranked As and A, and they were replaced. On the other hand, all the concrete girders, except 26 spans toppled at Higashi–Nada ward, were ranked as B, C, or D, and the existing girders were found to be reusable.

Fig 1. Toppled columns at Fukae Section.

* Corresponding author.

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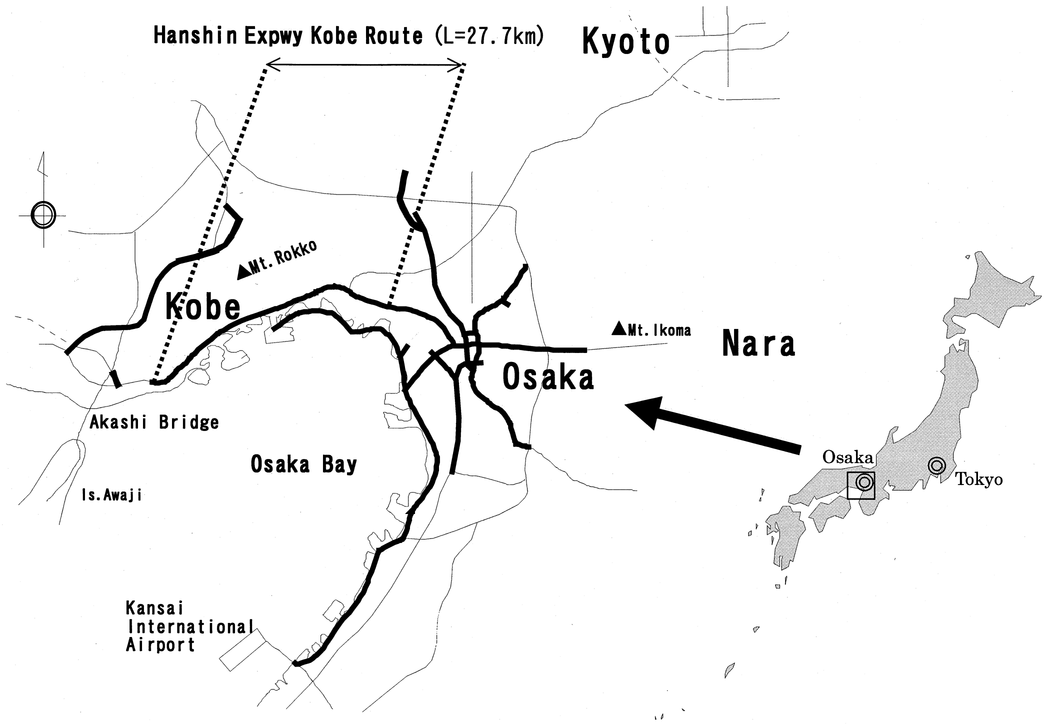

Fig 2. Location of Hanshin Expressway.

Fig 3. Damaged RC pier (ranked A).

Fig 4. Damaged RC pier (ranked B).

Fig 5. Damaged steel pier.

2. Superstructure

2.1. Repair of superstructure

On steel girder bridges, many buckling damages were found at the end of the main girder where end cross-beams had knee braces (see Fig. 6). It was considered that they were due to less transverse stiffness of grid, so that a full-web structure was employed for the end cross beam to reinforce the transverse stiffness of superstructure.

Most of the survived steel girders were repaired. Basically the buckled members were replaced with new members. When the damage was relatively light, buckled members were heated or welded as they were. And repaired girders were connected to each other as much as possible not to fall down with the structure as shown in Fig. 7. The continuation of girders is also effective for the environment by reducing the noise from expansion joints. When the girders could not be connected, seismic restrainers or displacement stoppers were installed to prevent the bridges from falling down (see Fig. 8).

Steel bearings were replaced with isolation rubber bearings so that horizontal forces could be dispersed and dumped. In case a bearing might take an uplift force, new steel bearing with bearing-plate was used. Most of the damages to concrete girders were due to displacement caused by the impaired end cross beams and bearings. Therefore, main girders were reused while the end cross beams were strengthened.

Fig 6. Buckling of main girder.

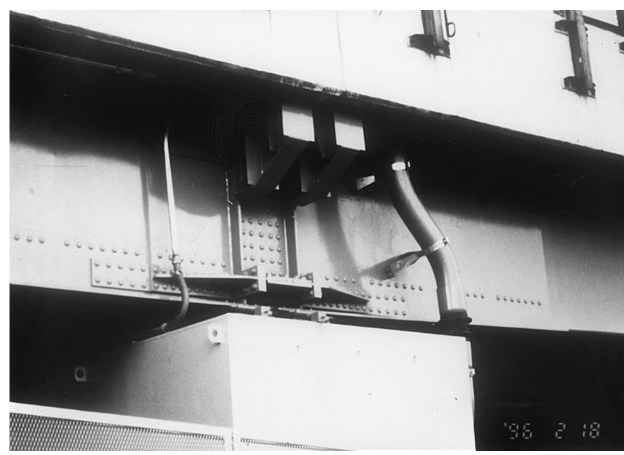

Fig 7. Connection of simple girders.


Fig 8. Seismic restrainer.

2.2. Reconstruction of superstructure

2.2.1. General

Severely damaged steel girders were reconstructed and their concrete decks were replaced by steel decks in order to decrease the dead load. In addition, isolation rubber bearings were installed to reduce the earthquake force. The designs of isolation rubber bearings were standardized for some supporting capacities, and adopted to reconstruction and repair of superstructures.

2.2.2. Fukae section

Fukae Section is located in Higashi–Nada ward, eastern part of Kobe City. There had been concrete bridges with mushroom-shaped piers ('pilz' type) and they were toppled during the earthquake as shown in Fig. 1.

The structures above ground level were removed totally within ten days after the earthquake to keep the safety of National Highway Route 43 running under the Hanshin Expressway Route 3.

The superstructures of toppled 18 spans were rebuilt as two 9-span continuous steel box girder bridges. The representative cross section is shown in Fig. 9. Seismic base isolation was considered in the design of the bridges at Fukae (see Table 1).

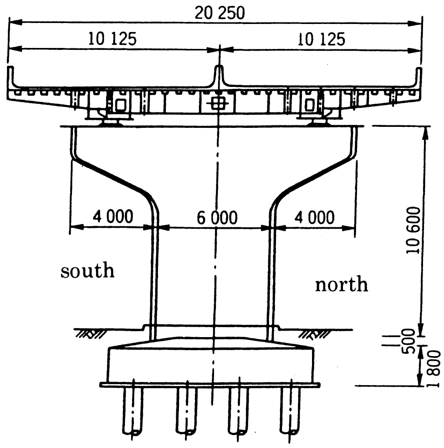

Fig 9. Cross section of new Fukae section.

Natural period of this bridge was analyzed with an entire model of 9-span continuous base isolating bridge in order to confirm the effects of seismic isolators under earthquake loading. The results are shown in Table 2. Case 1 means a case without base isolation and case 2 means with seismic isolators. The natural period of this bridge was found to become approximately twice using seismic isolation bearings.

A dynamic analysis was carried out using the actual wave of Kobe/Awaji Earthquake and it was found that the shear deflection of isolator was 238% while the design capacity was 250% with a static horizontal force of 0.68 G.

Table 2
Natural period of Fukae bridge

	Case 1		Case 2	
	Natural period	Mode	Natural period	Mode
1st	0.496	Vertical 1st	1.158	Transverse sway 1st
2nd	0.477	Vertical 2nd	1.105	Transverse sway 2nd
3rd	0.470	Vertical 3rd	1.092	Longitudinal sway 1st
4th	0.446	Vertical 4th	1.014	Transverse sway 3rd
5th	0.423	Vertical 5th	0.664	Transverse sway 4th
6th	0.408	Vertical 6th	0.578	Vertical 1st

3. Substructure

3.1. Repair and retrofit of pier

3.1.1. Concrete pier

The damaged piers ranked greater than A, or residual displacement of which was more than 15 cm, were determined to be replaced. The rest of the piers were repaired and retrofitted.

Damage on the piers from this earthquake drew our attention on providing shear strength and ductility to the columns. Therefore, the combined use of reinforced concrete and steel jackets was selected as the standard method for strengthening existing concrete piers.

The steel jackets are intended to increase the ductility and shear capacity of the pier, so they were not anchored to the footing. Details of the strengthening are shown in Fig. 10. The steel jacket was designed as tie bars. The reinforced concrete jackets were intended to increase the flexural capacity of the pier in order to give an appropriate ductility for it, and designed to have a thickness of at least 30 cm from the surface of existing concrete. The reinforcing bars were fixed to the footing by chemical anchorage.

Those retrofits were conducted after the repair work of injecting epoxy resin into the cracks, removing the weakened concrete, and replacing the buckled or failed reinforcing bars.

Table 1
Analytical condition for Fukae bridge

	Analytical model	Type of analysis	Conditions for bearing	RC pier	Pier bottom end
Case 1	Three-dimensional model	Natural frequency analysis	Pin	Elastic anchored	Anchored
Case 2			Longitudinal: equivalent striffness Transverse: equivalent stiffness	Elastic anchored	Anchored
Case 3		Time history response analysis	Equivalent stiffness	Elastic anchored	Anchored
Case 4			Bi-linear model	Elastic anchored	Anchored

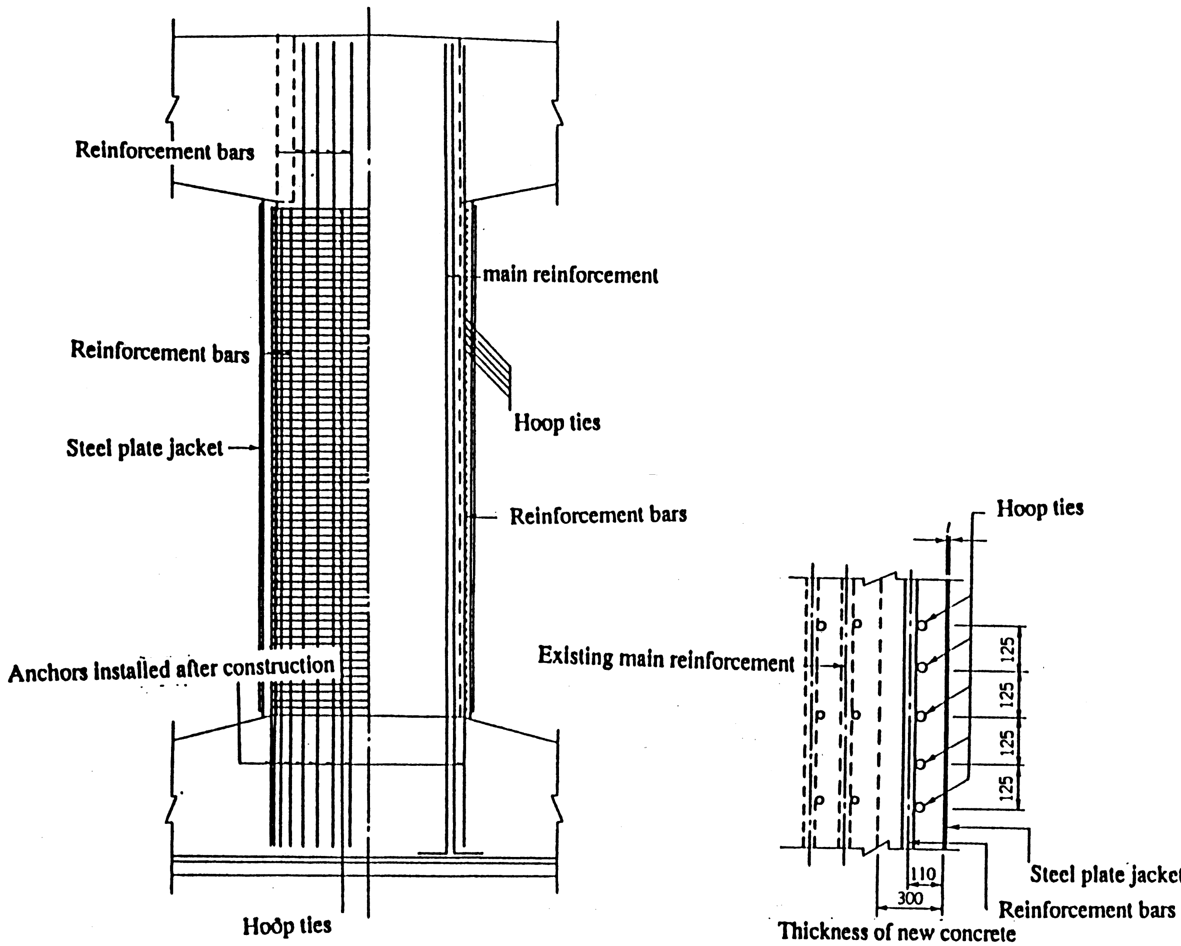

Fig 10. Repair and reinforcement of concrete pier.

3.1.2. Steel pier

Steel piers were removed and rebuilt when columns had been substantially buckled or cracks were observed around the circumference. Where repair work by replacing partial segments was judged possible, the superstructure was removed temporary, damage sections were cut out, and new segments were installed in their place. There are several measures to increase the lateral force bearing capacity of steel piers such as adding rib plates to the interior of a pier or filling concrete inside a pier. In the restoration of Route 3, the latter measure was employed according to the Restoration Specifications.

3.2. Hybrid pier

3.2.1. Structural features

A new type of structure, hybrid pier with an RC column and a steel beam was applied for the reconstruction of damaged concrete piers. The followings are the reasons for using the composite structure. By making use of steel box beam:

1. the whole pier becomes lighter.
2. the construction period at the site is minimized.
3. no form or scaffold is required, which minimizes necessity of traffic control.

The vertical main reinforcing bars for the RC column reconstructed by using the existing footing were inserted into the steel box beam and fixed by injected concrete. To composite the steel box beam and the concrete, the appropriate shear connectors were arranged on the inner surface of the steel box beam. Fig. 11 shows the structure of beam-column connection.

The stress in reinforcing bars and concrete at the connection can be calculated with a usual plain RC design method.

The connecting part of steel box beam is designed according to the method shown in the specification of steel piers. In this case, the stress delivered into the filling concrete can be neglected for safety.

Horizontal earthquake force can be transferred from beam to column by the studs installed underneath the lower flange of the steel box beam.

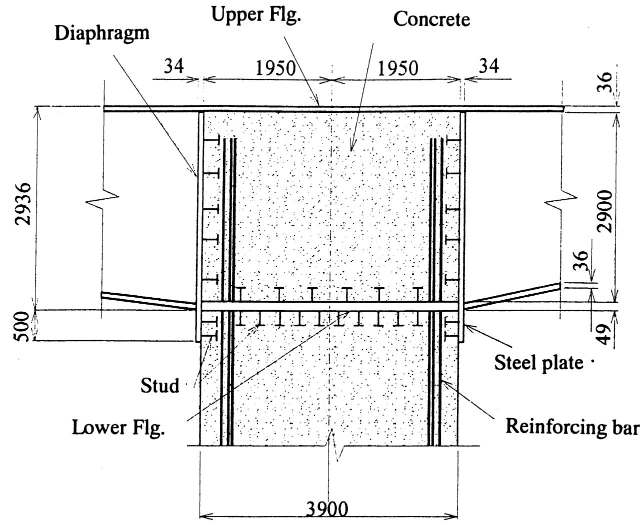

Fig 11. Structure of connection.

Fig 12. Before reconstruction.

Fig 13. After reconstruction (hybrid pier).

Prior to the practical application, the performance of this structure was ascertained by analysis and experiment using 1/5 scale models. More than 200 hybrid piers were constructed in the restoration of Route 3 (see Figs. 12 and 13).

3.2.2. Construction

The process of construction for the hybrid pier is shown in Fig. 14. The damaged column was cut at the height of approximately 1.5 m above the footing (step 1). The vertical main reinforcing bars were welded with the existing ones (step 2). Additional reinforcements, having an increased tie bars, were arranged and the 1st concrete was cast to the height of 2.5 m below the beam (steps 3 and 4). The steel box beam was erected over the column whose reinforcing bars penetrated the lower flange of the beam (steps 5–7). After that, second concrete was cast between the beam and the 1st concrete column, and inside the beam-column connection (steps 8 and 9).

3.2.3. Static load test and analysis

In order to ascertain the safety of this structure and the design method, a loading test was carried out on three actual hybrid piers [1–4]. A general view of one of those piers is shown in Fig. 15.

The loading step was divided into two; step 1 with a dead load of superstructure and step 2 with a live load by eight test trucks. All the measurements were carried out four times for step 1, and three times for step 2. Under the step 2 load, 75% of design flexural moment and 76% of design axial force were anticipated to arise. Some cracks were also expected to occur at the connection part because the tensile stress of concrete below the lower flange of beam would exceed its capacity according to the result of full-section analysis using the actual elastic modulus.

Strain gauges were arranged on each member at the beam-column connection such as reinforcing bars, flanges, webs, diaphragms, and studs inside the steel shell. Displacement was measured at every corner of each web plate and also at the edge of the beam.

The behavior of the pier was analyzed with three-dimensional finite element method. The analysis model and boundary conditions are shown in Fig. 16 and Table 3, respectively. Since both edges of the beam were supposed to have less influence on transferring the stress, they were modeled in beam elements. The bottom of the column was treated as a fixed boundary. Two cases were compared, as shown in Table 3, in which different bonding conditions between the steel plate and concrete were assumed. Note, however, the concrete is fixed to the steel, under compressive state of stress in both cases.

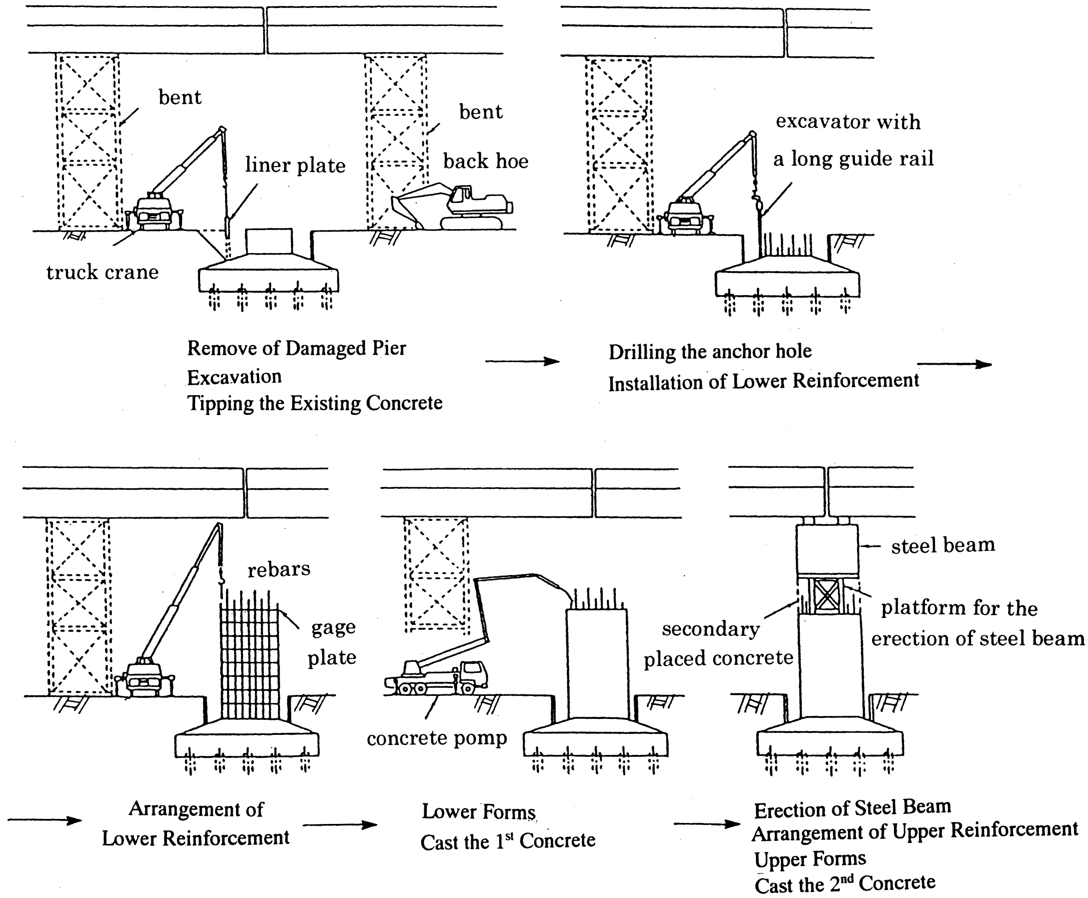

Fig 14. Process of reconstructing the hybrid pier.

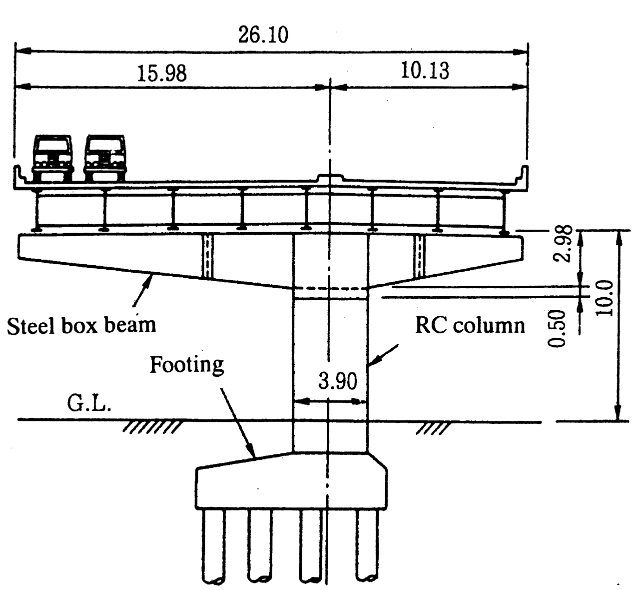

Fig 15. General view of test pier.

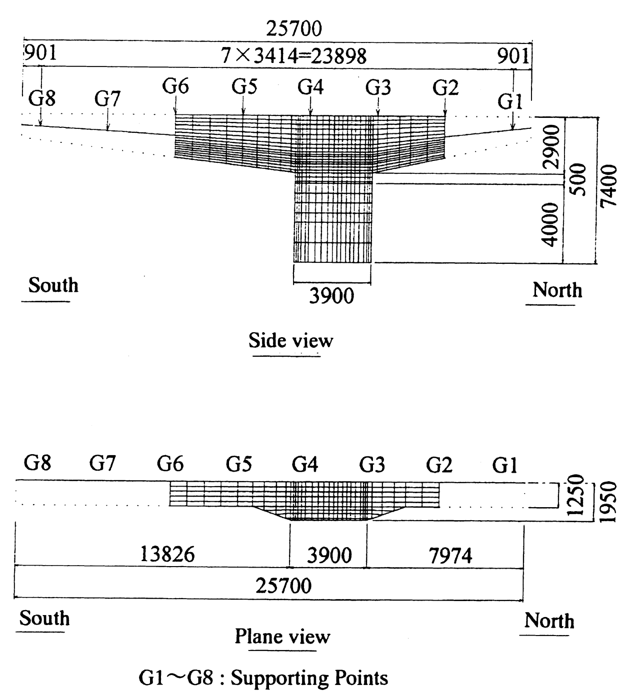

Fig 16. Analysis model.

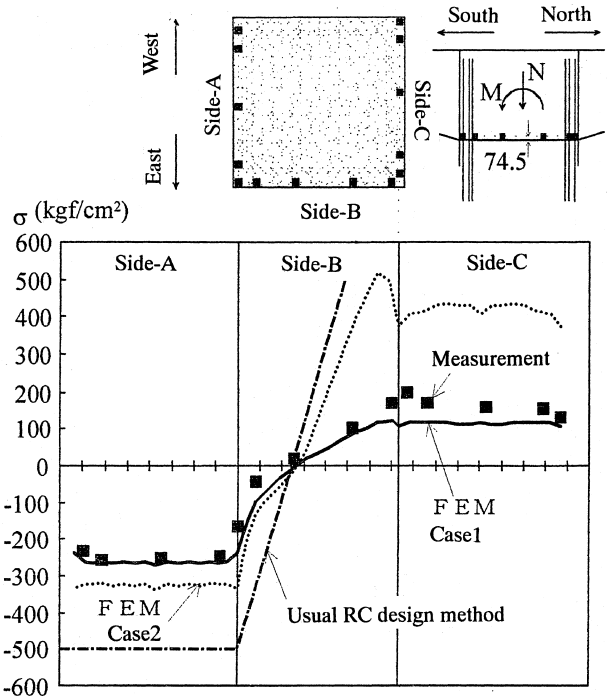

Fig 17. Distribution of axial stress in the boundary section.

The stress in the main reinforcing bars at the connection is shown in Fig. 17. And the states of the peeling of steel webs and flanges are shown in Fig. 18. The result of case 1 matches better to measured stress than case 2 and than that of the conventional RC design method. Although some cracks were expected to appear in the filling concrete before the test, they were not observed. It was supposed that few cracks would be developed due to the elastic deformation of steel beam that absorbs the tensile force on concrete.

The stress of concrete and that of steel web acted almost in the same direction as shown in Fig. 19. Approximately, 80% and 20% of the shear stress in the connection part were carried by concrete and steel web, respectively. The displacement under the step 2 load is shown in Fig. 20. The deformational behavior of connection was almost the same in the analysis of Cases 1 and 2. It also matched with the measurement that was within L/500, the allowable standard deflection of steel box beams. Through the static load test and analysis, the adequacy of using the method shown above was confirmed.

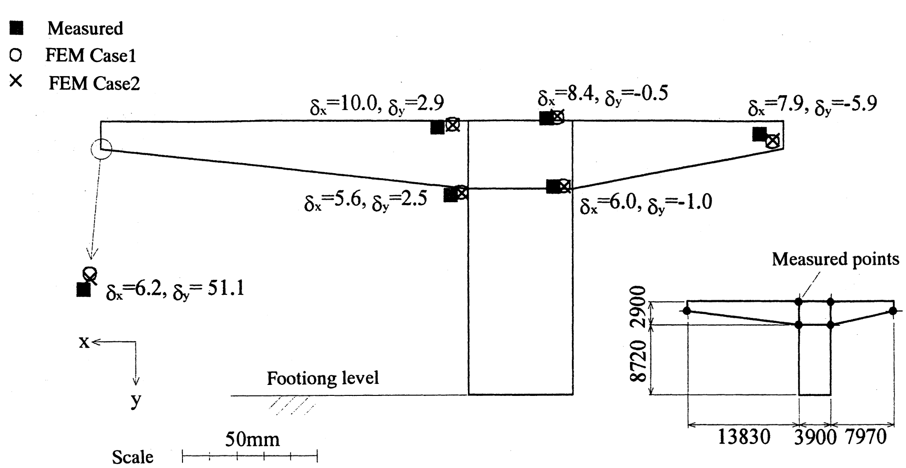

Fig 18. State of displacement.

Table 3
Boundary condition used in FEM analysis

	Relative displacement	Case 1	Case 2
Compression	Vertical direction	Restricted	Restricted
	Slide direction	Restricted	Restricted
Tension	Vertical direction	Tensile stress <27 kgf/cm^2: restricted Tensile stress >27 kgf/cm^2: not restricted	Not restricted
	Slide direction	Tensile stress <27 kgf/cm^2: restricted Tensile stress >27 kgf/cm^2: not restricted	Not restricted

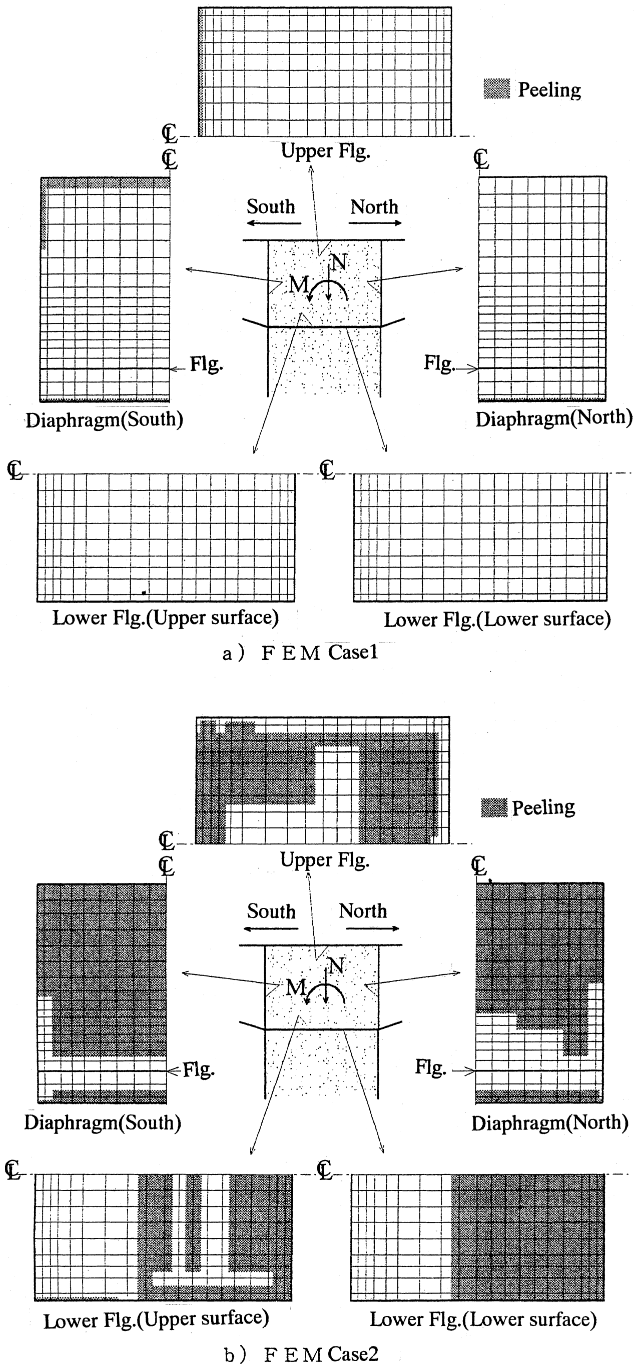

Fig 19. Boundary state of peeling between steel and concrete (analysis).

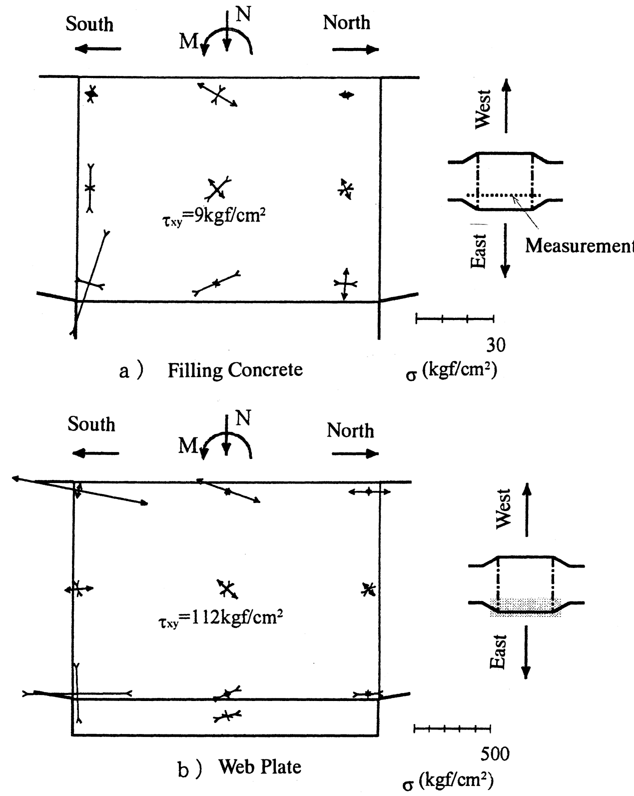

Fig 20. Stress in the connection (measured).

3.3. Repair and retrofit of foundation

Foundations were checked for damage over the full length of their piles by core boring. Although small cracks were found in some pile caps, the overall damage was light. Loading tests were carried out on the most severely damaged piles. The results suggested that their load bearing capacity had not been reduced. However, some of large cracks were repaired with injecting resin to protect the reinforcing bars from corrosion.

According to the Restoration Specifications, foundations were to bear the horizontal force equal to the bearing capacity of the piers on them. So, every foundations were checked to compare their bearing capacity with that of piers, and 13 out of all foundations along the Route 3 (except Benten Section, which is also a totally collapsed area) were decided to be strengthened with adding piles or increasing the thickness of the footing.

4. Environmental measures

4.1. Low-noise pavement

The low-noise pavement was adopted on the deck. It is a kind of high drainage asphalt concrete pavement whose surface is porous to reduce the noise arising from the traffic. The low-noise pavement had been developed to keep the safety of traffic with smooth drainage, and very few had actually been applied to the bridge decks. Therefore, its effects and durability have still been investigated continuously.

It is said that it can reduce about 3 dB of traffic noise when used on elevated bridges. At this moment it is

Fig 21. Tall noise barrier.

Fig 22. Strengthening of beams with external PC cables.

found to be exhibiting a sufficient performance from the continuous observation.

4.2. New noise barrier

4.2.1. Noise reducer

A new type of noise absorbing device, called 'noise reducer', was installed on top of noise barriers to perform the same effect as building normal noise barriers 1.5–2.0 m taller. An experiment was carried out to confirm the effects and it was found that the noise values can be decreased approximately 1–3 dB at the point 10 m away from the device.

4.2.2. Tall noise barrier

Tall noise barriers were also installed in front of tall residential buildings along the Route 3 as shown in Fig. 21. Although they are effective physically against the traffic noise, some other reinforcing works are needed to the existing structure to endure their adding loads. Fig. 22 shows an example of strengthening of existing prestressed concrete beam using additional external stressing tendons.

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