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STUDY ON SHEAR FAILURE AND REPAIR METHOD OF REINFORCED CONCRETE BEAM

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SYNOPSIS

Reinforced concrete viaducts of the Tohoku Shinkansen suffered considerable damage especially in mid-height beams due to the Miyagi-ken-oki Earthquake (1978), and there were quite a number that would not have been able to withstand normal use without remedy. In this study the strength-deformation characteristics of 1/2- or 1/3-scale model specimens of rigid-frame viaduct mid-height beams are obtained experimentally, and the bending shear characteristics of mid-height beams are examined. Further, various repair methods are applied to specimens loaded to different levels with crack width or displacement as the yardstick, and the effects are investigated. As a result, it is confirmed that member yielding load, resistance, deformation performance, etc., are not less than those of the original member if repaired properly by injecting epoxy resin in cracked parts.

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1. INTRODUCTION

Reinforced concrete viaducts of the Tohoku Bullet Train Line (Shinkansen) suffered considerable damage, especially in the mid-height beams due to the Miyagi-ken-oki Earthquake (1978), and there were a number of such beams which could no longer bear service loads without remedy[1]. Fig. 1 shows the route of Tohoku Shinkansen throuth Miyagi Prefecture and classification of ground.

Therefore, carrying out appropriate repairs on the damaged concrete viaducts in order to ensure safety was an important matter prior to the start of the commercial operation of the Bullet Train Line (Shinkansen).

Accordingly, we carried out experimental studies of the cracking features of reinforced concrete members. Furthermore, we determined strength-deformation characteristics of the mid-height beams of the actual rigid-frame viaducts, and studied methods of repairing them. Particularly, the effect of epoxy resin injection repairs on bending-shear failure of reinforced concrete members were experimentally investigated.

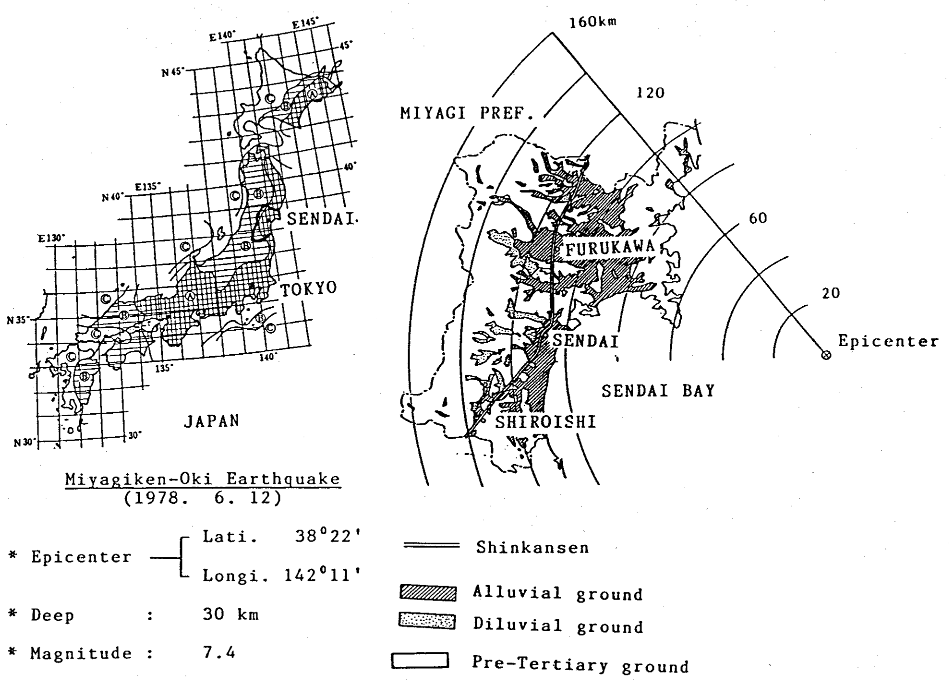

Fig.1 The route of Tohoku Shinkansen through Miyagi Prefecture and classification of ground

2. EXAMINATION OF EARTHQUAKE DAMAGE TO CONCRETE VIADUCT

2.1 Structural Properties of Members

Representative cracking patterns formed in the viaducts are shown in Fig. 2. The main structural features are as follows. For a mid-height beam the joint rotation angles at the two ends are antisymmetric and for practical purposes almost no rotation angle is produced. This member is estimated to have been subjected to a bending shear with a "shear span ratio" (shear span, a/ effective depth, d) of about 2.5 to 3.0 judging from the condition of the cracks. Furthermore, in order to reduce bending moments at the top and bottom ends of columns, the dimensions of this member are selected to provide a large bending stiffness (I/l). As a result, in light of the bending moment, the cross section is under reinforced, while web reinforcement ratio is close to the minimum value given by the Japanese National Railways' Design Standard, and therefore, extremely low. The axial force is negligible for that size of concrete cross section.

A column has a comparatively high axial reinforcement ratio. For practical purposes, it may be considered that the column is fixed at the top and bottom ends and that the so-called shear-type response takes place during horizontal seismic load. Shear span ratios are estimated to be 4 to 5 in case of a single-story viaduct and 2 to 3 in a two-story viaduct unless mid-height beams are damaged. Axial force is small for the concrete cross-sectional area, and the average compressive stress of concrete is about 1 N/mm^2 .

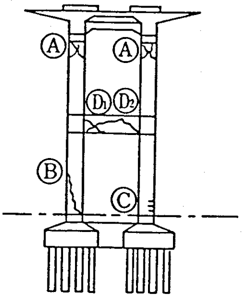

Fig.2 Cracking patterns formed in the viaduct

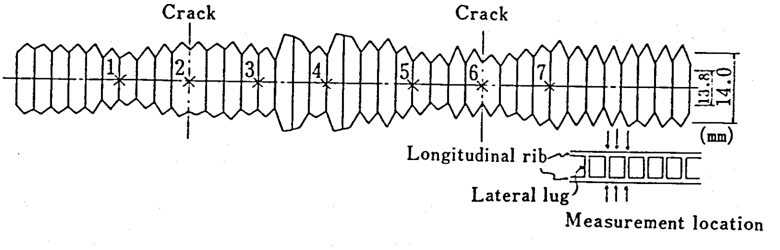

Fig.3 Example of diameter measurement of a bar

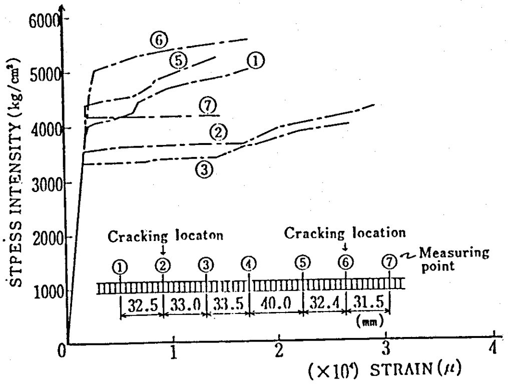

Fig.4 Examples of the stress-strain curves of bar

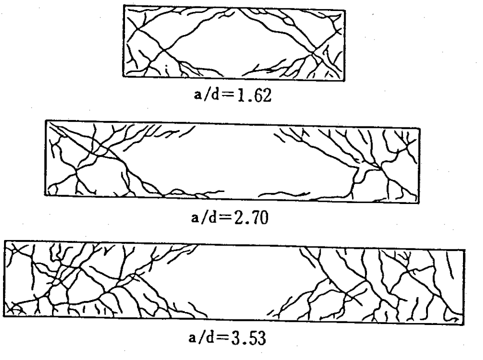

Fig.5 Examples of cracking patterns

2.2 Properties of Reinforcing Steel at Crack Locations

In order to investigate the degree of damage to the reinforcing steel in the damaged viaducts, reinforcing bars intersecting large cracks were taken out and subjected to tension tests. Fig. 3 shows an example of diameter measurements of a bar before tension tests were performed. Ther are complex diameter reductions induced in the bar, and it can be seen that plastic elongation strains scattered in a complicated manner in the axial direction were sustained during the earthquake. Fig. 4 shows examples of the stress-strain curves of such reinforcing bars. There is a tendency of greater strain hardening where there is a large reducation in diameter, with this tendency generally being most pronounced at the sites of large cracks.

2.3 Cracking of Mid-Height Beams of Rigid-Frame Viaducts

Tests were performed to ascertain the shear span ratio of load action to which mid-height beams were subjected during the earthquake. The specimens were Nos. 1 to 6 of series I as shown in Table 1, while Fig. 5 shows examples of cracking patterns.

A beam with a shear span ratio of 1.62 resulted in prominent diagonal cracks running from the fixed end directly to the beam center, in other words, to the loading point, and a beam with a shear span ratio of 3.53 produced two or three large diagonal cracks. In the latter case, each crack pattern is considerably different from the cracks observed in the actual mid-height beam. In contrast, the cracking pattern for a shear span ratio of 2.7 is quite similar to the cracking pattern produced in the actual mid-height beam.

3. OUTLINE OF EXPERIMENTAL STUDY ON THE REPAIR METHOD

Table 1 gives a list of the specimens. The experiments are classified into two series, I and II.

In series I, static alternating cyclic loads were applied to beam specimens Nos. 1 to 6 with 3 different shear span ratios to determine which specimens and shear span ratios resulted in cracking patterns similar to that of the actual mid-height beam. After shear span ratio was determined by this experiment, beam specimens (Nos. 7 to 10) were subjected to alternating cyclic load until crack width at the concrete surface reached the prescribed values or failure occurred. Then epoxy resin was injected into the cracks and alternating cyclic load was again applied until failure occurred in order to investigate the degree to which the strength and stiffness of the member had been recovered. For specimens Nos. 11 to 16, after injection of epoxy resin into the cracks, web portions of beams were further reinforced with steel plates to investigate the effects of such a repair. No. 17 is a specimen having web reinforcement four times that which had been used conventionally in mid-height beams of the Tohoku Bullet Train Line in the past.

Specimens of series II had a shear span ratio of 2.5 and were 1/3-scale models of actual mid-height beams. Nos. 1 to 7 had variable main axial reinforcement and web reinforcement and were subjected to one-way loading. For Nos. 8 to 14, the member yielding displacement was used as a standard during the loading to examine the bending shear characteristics of members subjected to alternating cyclic loads up to the prescribed displacement. These specimens were repaired with epoxy resin injection after which they were again loaded in the same pattern to investigate the effectiveness of the repairs.

Table 1 List of specimens

Series	Specimen		a/d	Reinforcing bar			Loading Pattern		Crack width (mm)	Epoxy resin injec-tion	Steel plate (mm)	Grout
	No.	Mark		Axial Type	Axial ρ_l(%)	Web ρ_w(%)	Before repair	After repair				
I	1		1.62	6D22	1.07	0.12				—	—	—
	2		〃	〃	〃	〃				—	—	—
	3		3.53	〃	〃	〃				—	—	—
	4		〃	〃	〃	〃				—	—	—
	5		2.70	〃	〃	〃				yes	1.8	re
	6		〃	〃	〃	〃				yes	1.8	CP
	7		2.70	5D22	0.89	0.12	I a	II	0.3	yes	—	—
	8		〃	〃	〃	〃	〃	〃	0.6	yes	—	—
	9		〃	〃	〃	〃	〃	〃	0.9	yes	—	—
	10		〃	〃	〃	〃	〃	〃		yes	—	—
	11		2.70	5D22	0.89	0.12	I a	II	1.0	yes	1.6	CP
	12		〃	〃	〃	〃	〃	〃	1.0	yes	3.2	CP
	13		〃	〃	〃	〃	〃	〃	2.0	yes	1.6	CP
	14		〃	〃	〃	〃	〃	〃	2.0	yes	3.2	CP
	15		〃	〃	〃	〃	〃	〃		yes	3.2	CP
	16		〃	〃	〃	〃	〃	〃		yes	1.6	CP
	17		2.70	5D22	0.89	0.48				—	—	—
II	1	A1	2.5	4D16	0.85	0.124	one-way			—	—	—
	2	A1	〃	〃	〃	〃	〃			—	—	—
	3	A2	〃	〃	〃	0.249	〃			—	—	—
	4	A3	〃	〃	〃	0.497	〃			—	—	—
	5	B1	〃	4D22	1.66	0.124	〃			—	—	—
	6	B2	〃	〃	〃	0.249	〃			—	—	—
	7	B3	〃	〃	〃	0.497	〃			—	—	—
	8	A1	2.5	4D16	0.85	0.124	III			—	—	—
	9	A1	〃	〃	〃	〃	〃			—	—	—
	10	A2	〃	〃	〃	0.249	〃			—	—	—
	11	A3	〃	〃	〃	0.497	〃			—	—	—
	12	B1	〃	4D22	1.66	0.124	〃			—	—	—
	13	B2	〃	〃	〃	0.249	〃			—	—	—
	14	B3	〃	〃	〃	0.497	〃			—	—	—
	15	A1	2.5	4D16	0.85	0.124	III-δy	III		yes	—	—
	16	A1	〃	〃	〃	〃	III-2δy	〃		yes	—	—
	17	A1	〃	〃	〃	〃	III-3δy	〃		yes	—	—
	18	B1	〃	4D22	1.66	0.124	III-2δy	〃		yes	—	—
	19	B2	〃	〃	〃	0.249	〃	〃		yes	—	—
	20	B3	〃	〃	〃	0.497	〃	〃		yes	—	—
	21	B3	〃	〃	〃	〃	III-4δy	〃		yes	—	—

(1) Concrete cross-sectional area
Series I : 425×550mm, d=510mm
Series II : 282×367mm, d=330mm

(2) Loading pattern (see Fig. 6), I_a and I_b indicate epoxy resin injection stage, one-way indicates one-way monotonic increase loading, III-δy, ete. indicate epoxy resin injection at displacement δy, etc. upon alternating cyclic loading

(3) Crack width : measured at concrete surface when loaded, and yardstick when injecting resin

(4) Grout : filling material between web and steel plate with re indicating epoxy resin and CP cement paste

(5) ρ_w varied by varying stirrup spacing

Examples of cross-sectional dimensions and arrangement of reinforcement of specimens of series I and II are shown in Fig. 6. Fig. 7 shows the loading patterns. Table 2 shows concrete mix proportion and material characteristics.

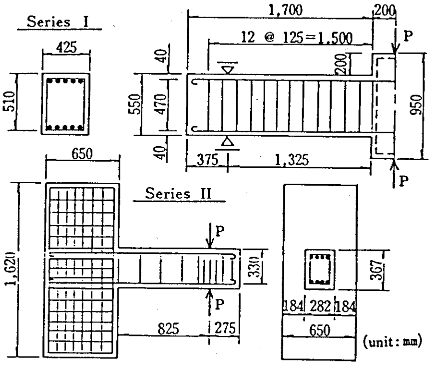

Fig.6 Cross-sectional dimensions and arrangement of reinforcement of specimens

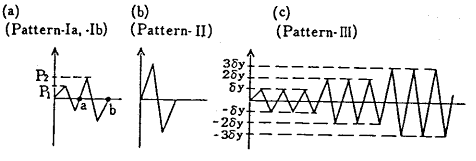

Fig.7 Loading patterns

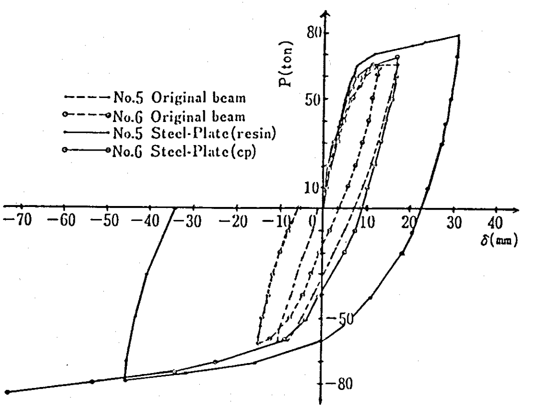

Fig.8 P-δ curves (No.5, No.6)

Table 2 Concrete mix proportion and material characteristics

Concrete	Series	Max size (mm)	Slump (cm)	Air (%)	W/C (%)	s/a (%)	Unit (kg/m³) W	C	S	G	Admix.
	I・II	25	12	4.5	48.5	45.6	162	334	822	1,016	Pozolith No5L

Reinforcing bar	Series		Grade	Dia.	fy (kg/cm²)
	I・II	Axial reinforcement	SD35	D22	4,709
				D16	4,550
		Web reinforcement	SD30	D 6	4,410

Epoxy resin	Series	Strength (kg/cm²)	
	I・II	Comp.	530
		Tens.	390

4. EFFECTS OF VARIOUS REPAIR METHODS (SERIES I)

4.1 Shear Span Ratio of Mid-Height Beam Based on Earthquake Damage (Nos.1 to 6)

As described before, the cracking patterns of beam specimens of a/d = 2.7 resembled the cracking pattern of actual mid-height beams most closely. Subsequently, the a/d of all specimens in series I were adjusted to 2.7.

4.2 Materials Effective for Bonding Steel Plate (Nos.5, 6)

Either epoxy resin or non-shrink cement paste was injected between specimen and steel plate (thinckness 1.8 mm) to examine which material was more effective in bonding the steel plate to concrete by use of spcimens No. 5 and No. 6.

Fig. 8 shows the load-displacement curves of No. 5 and No. 6. The quotient of load divided by its corresponding displacement is shown in Fig. 9. This figure gives secant stiffness equal to 1 when the load is 10 tons. Whichever grouting material was used the initial stiffness was approximately 10 percent lower compared with that of the original beam. Over time, however, in the case of non-shrink cement paste, the decrease in secant stiffness was roughly the same as that seen for the original beam, whereas in the case of epoxy resin, the rate of decline was slower compared with that of the original beam. Judging from basis of the P-δ curve after the repair, there is not much difference in strength between specimens treated with the two types of grouting material, but judging from the viewpoint of stiffness reduction, the use of epoxy resin is slightly more advantageous.

It was decided to use non-shrink cement paste as the grouting material in subsequent series I tests because (1) the strength and deformability of the original beam could be secured, and (2) it is economical for use in actual repairs.

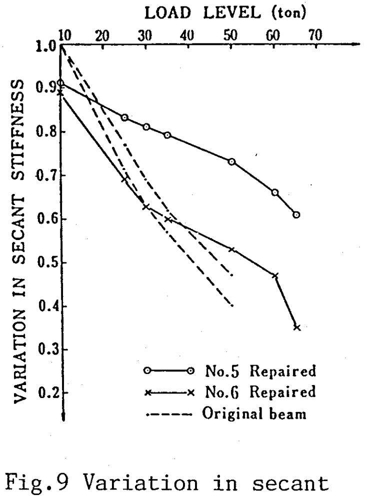

Fig.9 Variation in secant stiffness (No. 5, No. 6)

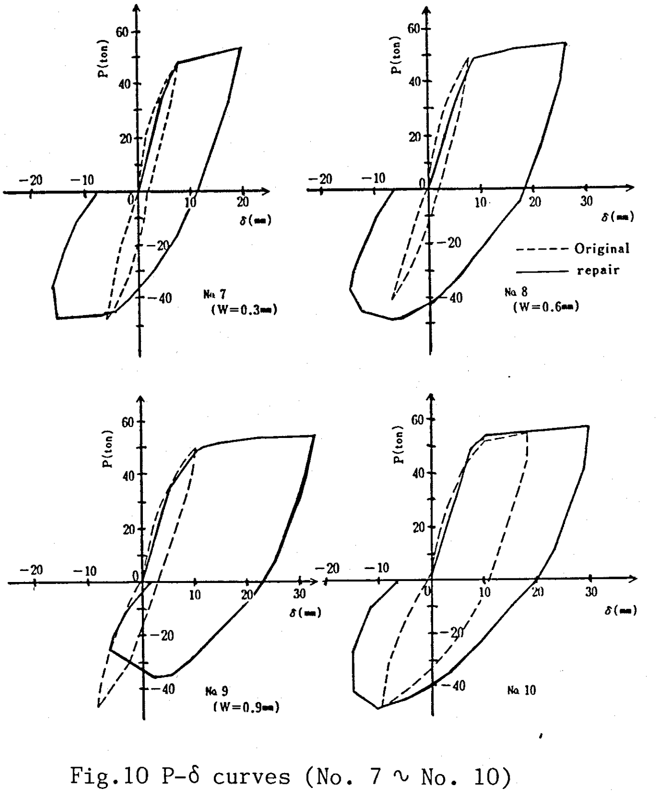

Fig.10 P-δ curves (No. 7 ∿ No. 10)

4.3 Repair Effect in Case of Using Crack Width as Parameter when Injecting Epoxy Resin (Nos. 7 to 10)

The P-δ curves of the original beams and repaired beams of No. 7 to No. 10 are shown in Fig. 10. In the original beams, loads to produce δ = 7 mm, 8 mm, 10 mm and 16 mm corresponding to No. 7, 8, 9, and 10, were applied as one alternate cycle.

The P-δ curve after repairs extends nearly linearly toward the member yield point of the original beam, and after yielding, the inclination of the curve is reduced to approximately one-tenth. While the stiffness at the time of load removal is roughly the same as the stiffness during loading, a hysteresis aimed at the previous maximum load point from a load of zero is seen during the negative loading.

Examinations of the variations in secant stiffness at various load levels show that the stiffness at P = 10 tons for a beam after repairs is reduced to approximately one-half compared with the initial stiffness of the original beam, but that the subsequent reduction in stiffness is extremely gradual and at the yielding load level it is roughly the same as the secant stiffness of the original beam (see Fig. 11). This can be said to be true regardless of the crack width of the beam before repairs. That the initial stiffness after repairs was low in this way is thought to have been due to the adopted resin injection method in which the grouting resin could not penetrate into cracks narrower than a certain limit thus leaving some cracks without grouting. It has been pointed out by some researchers [2] [3] [4].

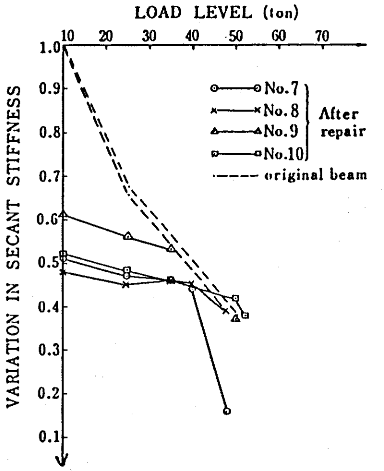

Fig.11 Variation in secant stiffness (No.7 ∿ No.10)

It can be said that there is a necessity for a method which uses low pressure and takes as much time as possible [5] [6] [7]. Specimen No. 10 was repaired after loading to 2δy, and it was confirmed that the strength was not lost even when subjected to repetitions of 3δy after repairs.

Fig. 12 shows strains of stirrups in specimen No. 10. In the case of the original beam, strains were initiated in the stirrups when the load became 25 tons (shear force 12.5 tons). This value was roughly equal to the load at which diagonal cracking was produced. After this point, only slight increments of load resulted in an abrupt increase in strains. In contrast, for the repaired beam, diagonal cracks exceeding 0.2 mm in width were produced at roughly P = 30 tons, with strains rapidly increasing thereafter. This proves that loads at which diagonal cracks were produced were not less than those for the original beam.

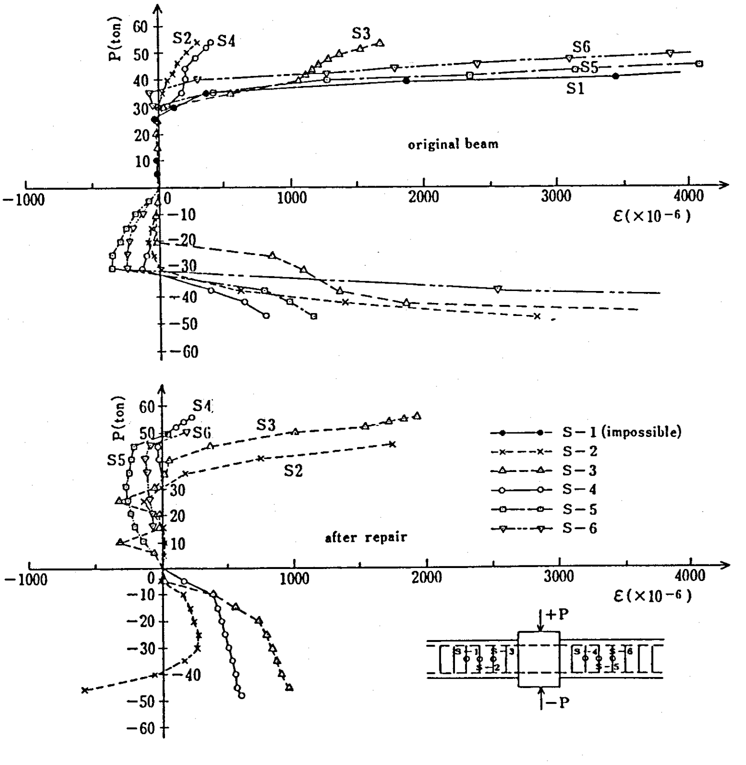

Fig.12 Stirrup strain hysteresis (No.10)

4.4 Repair Effect in Case of Injecting Epoxy Resin and Reinforcing with Steel Plate (Nos. 11 to 16)

Fig. 13 illustrates the P-δ relationships of Nos. 13 and 14. Yielding load after repair was approximately 15 percent higher than that of the original beam. Noting the fact that yielding load was not increased in case of repairs with epoxy resin only, it is judged that the increase in yielding load in this case was due to reinforcement of the web with steel plate. This P-δ curves after yielding of the members also do not depend on steel plate thickness and are roughly identical and practically the same as for the original beams.

As described above, when epoxy resin was injected and the web was reinforced with steel plate, although there was a difference in strains produced in stirrups and the steel plate depending on the thickness of the plate, it was found that there were not much differences in yielding loads of members and P-δ curves.

Although specimen No. 17 was repeatedly loaded at displacement four times the yielding displacement, capacity and deformability were extremely good.

In loading of series I, one or two cycles of loading were made at a specific displacement after member yielding [8].

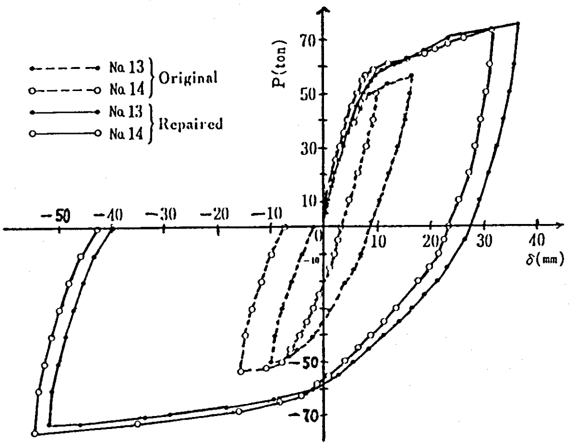

Fig.13 P–δ curves (No. 13, No. 14)

Table 3 Results of experiment in Series II

Specimen No.	Specimen Mark	Concrete strength (kg/cm²)		Bending Crack formation		Diagonal Crack formation		Member yielding			Ultimate			Max. displacement	Pu/Py	ductility	
		Comp.	Tens.	Pc (ton)	τc (kg/cm²)	Ps (ton)	τs (kg/cm²)	Py (ton)	τy (kg/cm²)	δy (mm)	Pu (ton)	τu (kg/cm²)	δu (mm)	δmax (mm)		δu/δy	δmax/δy
1	A1	251	25.9	6.0	6.4	9.0	9.6	11.3	12.1	6	12.7	13.6	11	15	1.12	1.83	2.50
2	A1	267	22.2	7.0	7.5	9.0	9.6	10.0	10.7	5.5	12.1	13.0	10.5	22	1.21	1.91	4.00
3	A2	312	29.4	5.0	5.4	9.0	9.6	10.9	11.7	7.5	13.3	14.2	60.5	71.5	1.22	8.07	9.53
4	A3	294	24.1	4.5	4.8	9.0	9.6	10.9	11.7	4.5	14.5 min	15.5 min	73 min	73 min	1.33 min	16 min	16 min
5	B1	292	27.4	8.0	8.6	12.0	12.8	15.6	16.7	5.5	15.6	16.7	9	9	1.00	1.64	1.64
6	B2	317	24.1	8.0	8.6	12.0	12.8	20.8	22.3	7	21.2	22.7	13	14	1.02	1.86	2.00
7	B3	264	19.6	8.0	8.6	12.0	12.8	20.4	21.8	6	25.8	27.6	45.5	65	1.26	7.58	10.8
8	A1	269	20.7	4.2	4.5	10.0	10.7	10.8	11.6	6	11.5	12.3	12	18	1.06	2	3
9	A1	251	25.9	–		–		11.6	12.4	5.5	12.4	13.3	11	16.5	1.07	2	3
10	A2	279	23.1	5.0	5.4	9.0	9.6	11.2	12.0	6.5	11.4	12.2	13	26.0	1.02	2	4
11	A3	279	23.1	5.0	5.4	9.0	9.6	10.7	11.5	4	13.4	14.3	28	32	1.25	7	8
12	B1	259	21.7	–		–		16.0	17.1	5.5	21.0	22.5	11	11	1.31	2	2
13	B2	186	17.3	–		–		19.0	20.3	8.5	20.3	21.7	17	17	1.07	2	2
14	B3	234	16.4	7.9	8.5	11.8	12.6	21.4	22.9	7	23.1	24.7	28	28	1.08	4	4
15	A1 − δy	282	19.4					11.3	12.1	5							
	repaired							12.0	12.8	5	12.0	12.8	10	20	1.00	2	4
16	A1 −2δy	240	20.9					11.4	12.2	5							
	repaired							11.5	12.3	5	12.8	13.7	10	20	1.11	2	4
17	A1 −3δy	240	18.2					10.4	11.1	5	12.6	13.5	10		1.21	2	
	repaired							10.8	11.6	5	11.9	12.7	15	20	1.10	3	4
18	B1 −2δy	221	17.2					15.5	16.6	5							
	repaired							17.1	18.3	5	22.0	23.6	10	15	1.29	2	3
19	B2 −2δy	271	20.6					20.2	21.6	6							
	repaired							18.4	19.7	6	22.2	23.8	18	24	1.21	3	4
20	B3 −2δy	277	26.2					21.0	22.5	6							
	repaired							19.9	21.3	6	25.0	26.8	24	30	1.26	4	5
21	B3 −4δy	247	22.1					19.4	20.8	6	22.1	23.7	12		1.14	2	
	repaired							17.2	18.4	6	25.7	27.5	12	42 min	1.49	2	7 min

5. BENDING SHEAR FAILURE AND CAPACITY (SERIES II, NOS. 1 TO 14)

Table 3 shows the results of experiments in series II.

The load at which the diagonal crack was formed was mainly affected by the quantity of main axial reinforcement. Loads were 9 and 12 tons for Series A and B, respectively. The larger the number of stirrups, the better is the dispersion of diagonal cracks. In addition, the angle of the diagonal crack to the axial line, θ, becomes larger.

The relationships of shear stress intensity and $\rho w/\rho l$ at the yielding point of the member and at its ultimate state are shown in Fig. 14. ρw and ρl mean web reinforcement ratio and tensile reinforcement ratio respectively. Fig. 15 shows the relationships of shear stress intensity and $\rho w/\rho l$ at yielding of reinforcement. The $\rho w/\rho l$ may be considered as a yardstick expressing the ratio between shear-resisting and flexure-resisting capacities of the member. Since yielding of stirrups and main tensile reinforcement occurred almost at the same time for specimen A1, the ratio of $\rho w/\rho l$ for case A1 is designated as 1 in these figures.

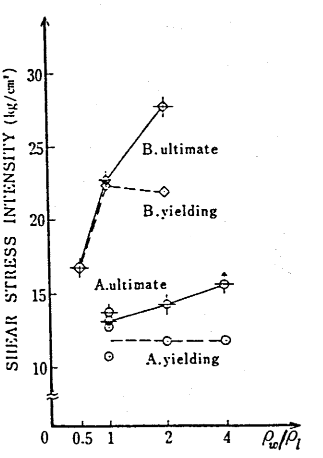

Fig.14 $\rho w/\rho l$ - τ

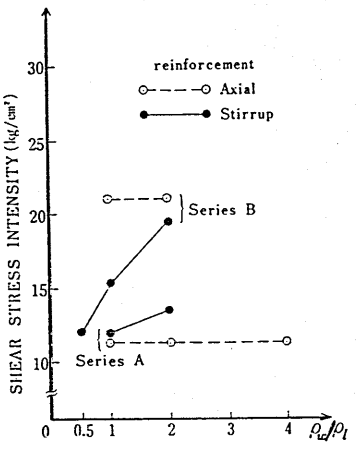

Fig.15 $\rho w/\rho l$ - τ

The specimens of series A show that yielding of the members begins with the yielding of main axial reinforcement and reaches the ultimate states when stirrups yield. In contrast, for B1 and B2, member yielding occurred with yielding of all stirrups intersecting diagonal cracks and ultimate states were reached with almost no increase in loads. The achieved capacity value agreed well with calculated values according to CEB [9]. In contrast, for B3, the first stirrup intersecting a diagonal crack yielded at S = 18.2 tons, and main axial reinforcement yielded with an approximately 2 tons increase in shear force to bring about the yielding of the member. Maximum capacity was indicated with yielding of all stirrups intersecting diagonal cracks.

The states at failure can be divided into the two categories. One is "shear tension failure" where the growth of diagonal cracking is accompanied by longitudinal cracks along main axial tension reinforcement to reach the ultimate state i.e. cases A1, B1, and B2. The other is "shear compression failure" where diagonal cracks grow and the ultimate state is reached due to the crushing of concrete in the compression zone, i.e. cases A2, A3, and B3. Since the principal cause of longitudinal cracks is the loss of bond between reinforcing

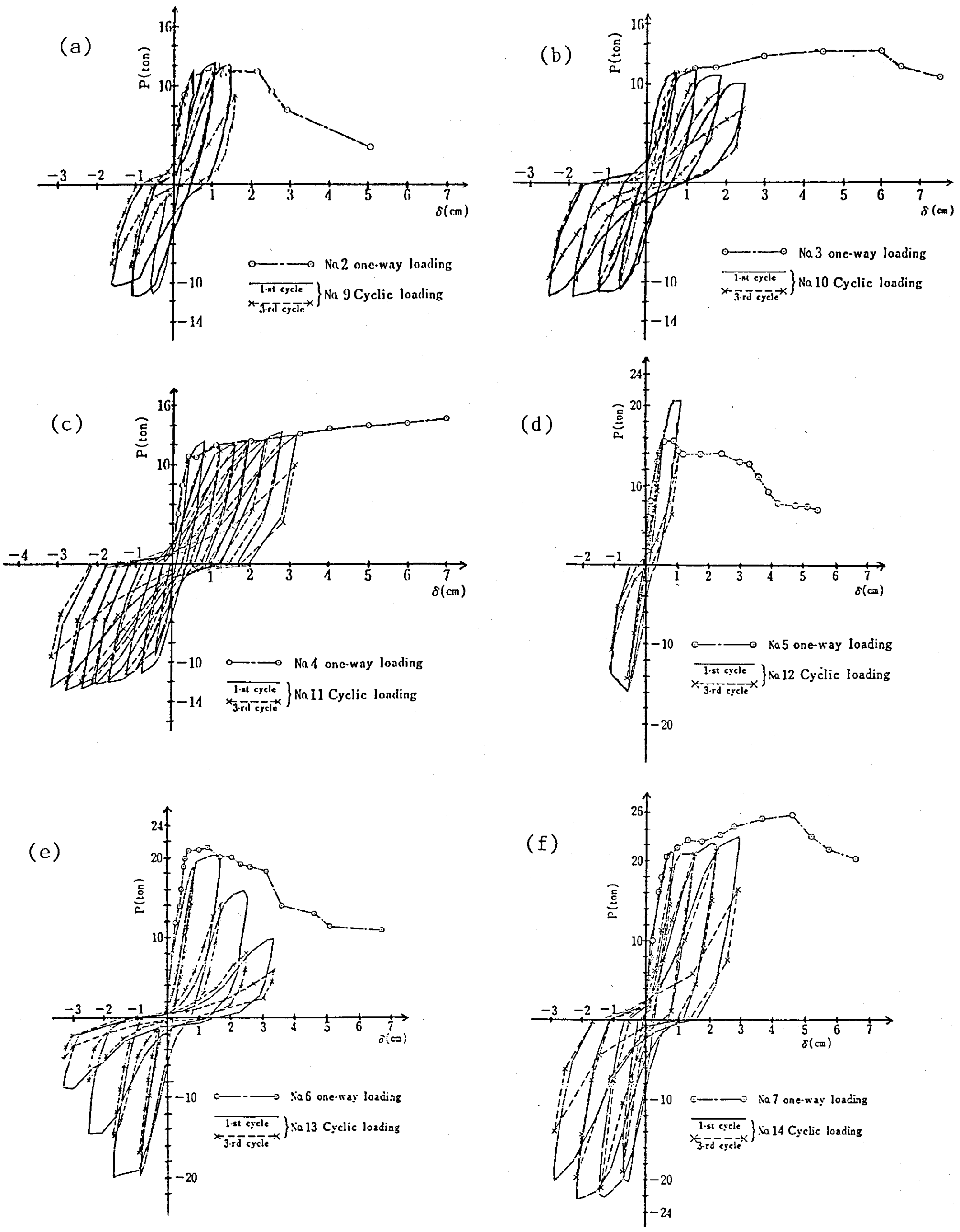

Fig.16 (a) ∿ (f) P-δ curves

steel and concrete, the quantities of main axial reinforcement and stirrups have much influence on the type of failure that actually results. In this experiment, bond failure occurred in cases in which the web reinforcement ratio was less than ρw = 0.249 %.

Fig. 16 shows P-δ curves which resulted when either one-way loading or cyclic loading were applied to specimens of identical specifications. For both types of loading the ultimate displacement depends on stirrup quantity. It is also seen that on being subjected to cyclic loading, the ultimate displacement is small compared with that of one-way loading. This seems understandable because through alternating loading, bond between reinforcing steel and concrete is gradually lost beginning with the stage when displacement is small, or the shear force shared by concrete is gradually decreased. However, deformability improves when stirrup quantity is increased. For example, in the case of No. 11 in which the stirrup amount is four times that of No. 9, there is no capacity reduction even though the P-δ curve begins to indicate an inverse S-shape.

Fig. 17 shows the relationship between ductility, $\delta u/\delta y$ or $\delta max/\delta y$, and $\rho w/\rho l$. δy means member yielding displacement at loading point, δu means displacement at ultimate state of member, and δmax means displacement corresponding to the yielding load in the decending branch in P-δ curve. In case of one-way loading, only very low ductilities are indicated when $\rho w/\rho l$ is equal to or less than 0.15, but with $\rho w/\rho l$ equal to 0.3, both $\delta u/\delta y$ and $\delta max/\delta y$ greatly increase. However, when subjected to cyclic loading, $\delta u/\delta y$ is extremely small with $\rho w/\rho l$ equal to or less than 0.3. For instance, ductility which was 8 for one-way loading becomes 2, which is extremely low. This is thought to be due to the fact that the bond between reinforcing steel and concrete is lost earlier than is the case with one-way loading, and because resisting capacity can not be maintained due to longitudinal cracks.

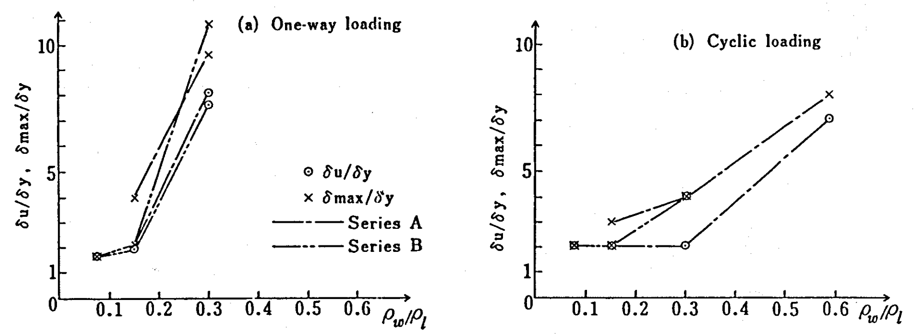

Fig.17 $\rho w/\rho l$ - ductility

6. EFFECT OF REPAIR BY EPOXY RESIN INJECTION (SERIES II, NOS. 15 TO 21)

Cracks produced by loading after epoxy resin injection are formed close to the locations of crack occurrence in the original beam (Fig. 18). When subjected to cyclic loads up to a large displacement extent compared with those of the original beam, since resin is injected to a wider and deeper extent, the formation and growth of cracks are hindered.

Table 4 shows how Py, δy, Pu, δu, and δmax have changed after repairs. For specimen A1, there is not much changes in the values of Py, δy, Pu, δu, and δmax, as compared with values for the original beam and repaired beam. For Nos. 18, 19, and 20 although differences could not be seen between the original beam and the repaired beam with regards to Py, δy, and Pu, there were differences seen in δu and δmax. Particularly, it was confirmed that δmax increased after repairs even when stirrup quantity was small. In the case where proper repairs had been made, it was ascertained that even for a member damaged to such an extent, the member capacity and ductility were not less than that of the original beam.

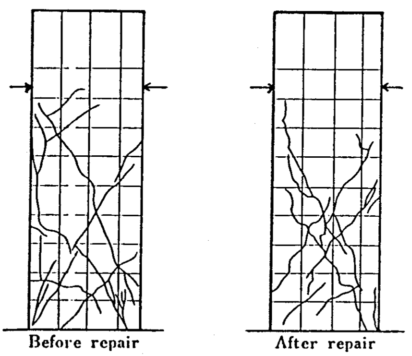

Fig.18 Cracking patterns

Table 4 Repair effect

Specimen No.		15 A1-δy	16 A1-2δy	17 A1-3δy	18 B1-2δy	19 B2-2δy	20 B3-2δy	21 B3-4δy
Py	B	11.3	11.4	10.4	15.5	20.2	21.0	19.4
	A	12.0	11.5	10.8	17.1	18.4	19.9	17.2
	R	1.06	1.01	1.04	1.10	0.91	0.95	0.89
δy	B	5	5	5	5	6	6	6
	A	5	5	5	5	6	6	6
	R	1	1	1	1	1	1	1
Pu	B	12.4	12.4	12.4	21.0	20.3	23.1	23.1
	A	12.0	12.8	11.9	22.0	22.2	25.0	25.7
	R	0.97	1.03	0.96	1.05	1.09	1.08	1.11
δu	B	11	11	11	11	17	28	28
	A	10	10	15	10	18	24	12
	R	0.91	0.91	1.36	0.91	1.06	0.86	0.43
δmax	B	16.5	16.5	16.5	11.0	17.0	28.0	28.0
	A	20.0	20.0	20.0	15.0	24.0	30.0	42.0^{min}
	R	1.21	1.21	1.21	1.36	1.41	1.07	1.50^{min}
Pu/Py	B	1.07	1.07	1.07	1.31	1.07	1.08	1.08
	A	1.00	1.11	1.10	1.29	1.21	1.26	1.49
δu/δy	B	2	2	2	2	2	4	4
	A	2	2	3	2	3	4	2
δmax/δy	B	3	3	3	2	2	4	4
	A	4	4	4	3	4	5	7^{min}

B : before repair, A : after repair, R : A/B
Units : P (ton), δ (mm)

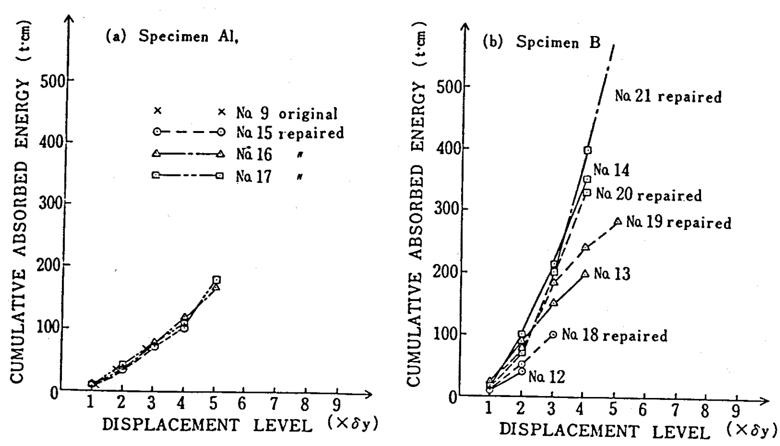

Fig.19 Cumulative absorbed energies

Table 4 gives the ductilities before and after repairs, and except for No. 21, the values of $\delta u/\delta y$ were roughly the same. As for $\delta max/\delta y$, its value increased from 1 to 2 in all specimens after repairs, and it was ascertained to be not less than the original ductility.

Fig. 19 gives comparisons of the cumulative absorbed energies before and after repairs of specimens of the same kinds. It is seen that the absorbed energy after repairs is roughly the same as for the case of the original beam regardless of the displacement level in loading of the original beam.

7. CONCLUDING REMARKS

In summary, by experimental studies of methods of repairing damaged mid-height beams of reinforced concrete viaducts, it was ascertained that if their cracks are repaired with injections of epoxy resin, even members that had just about reached failure could regain their original capacities, and that their ductilities could be recovered allowing them to become serviceable. Therefore, it can be said to be amply safe to use the values prior to damage in evaluating resisting capacity and deformability after repairs. Furthermore, the effect of repairs by applying steel plates to the web was also studied. However, it was shown that if the objective of repairs is only the recovery of the original capacity and deformability, such a repair method is extremely uneconomical, and adequate results can be attained just by injection of epoxy resin.

In the experiments reported here, it is thought epoxy resin was injected into cracks of width about 0.2 mm using the manual pumping method [10], but it cannot necessarily be said that injection had reached into cracks that were even finer. However, since in concrete structure it is the surface portion, that is, the portion at which crack width is comparatively large, where stress conditions are high, and even though resin may not be thoroughly injected so as to reach into minute cracks, member yielding load, resisting capacity, and ductility are recovered, it is thought ample repair effect can be obtained with the injection method adopted in these experiments. At present, it is possible for epoxy resin to be injected into minute cracks if pumped at low pressure over a long period of time (working time), but in injecting resin it will be necessary to proceed with the work carefully while confirming that the resin is being thoroughly injected [5] [7]. It is noteworthy that mid-height beams of actual reinforced concrete viaducts have been repaired injecting epoxy resin based on the results of these studies and these viaducts are presently in use.

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