EARTHQUAKE DAMAGE TO CONTINUOUS GIRDER BRIDGES
WITH DAMPERS INSTALLED AT THE SUPPORT

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SUMMARY

This paper relates to the damage to the prestressed concrete continuous girder railway bridges with the dampers (1), installed at the support of the girders, during Miyagi Earthquake in 1978 in Japan.

The main purpose of the installation of these dampers was to distribute the seismic force acting longitudinally on the girders to the movable supports and avoid a concentration of the load to the fixed end support, which would often result in unfeasible design. The damage on the piers showed that the above-mentioned distribution was effective during the occurrence of earthquake.

1. DAMAGE TO NATORIGAWA BRIDGE

Natorigawa Bridge is situated at the southern part of Sendai City about 115 km. from the epicenter of Miyagi Earthquake (Fig.1). The bridge consists of 2-three spans and 1-four spans continuous, prestressed concrete, box girders as shown in Fig.2.

The constitution of the damper is shown in Fig.3. The upper part of a rectangular steel pipe filled with concrete, is fixed in the lateral beam of the superstructure (Fig.4) and the lower part of the pipe fits in a steel box embedded at the top of the substructure. The box is filled with viscous materials, composed of a mixture of asphalt and polymersolution and have a viscosity of 7000 poise at 20 °C. Those dampers are designed to resist a horizontal movement, but they do not carry a vertical load. No fixed end shoe is used at any substructure, but at P1, P5 and P9, springs are installed inside the damper as shown in Fig.3, to keep the girder in a neutral position after the earthquake. The resistance-velocity curve of a unit of the damper at a temperature of 20 °C is shown in Fig.5.

In Fig.4 the cracks in the lateral beam at A2 due to earthquake are shown, and Fig.6 shows typical crack patterns of the substructures due to the earthquake. The cracks of A1 and P1 concentrate at the lower part like A2 (Fig.6). On the other hand, at P7 and P3 the predominant crack starts at the upper cutoff point of the vertical reinforcement. At P2, P5 and P8 there are cracks also at the upper cutoff point, but more cracks are observed in the lower part like P4 (Fig.6).

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The difference of the crack patterns is influenced by the vertical bar arrangement. At A1, A2 and P1, all reinforcements go up to the top of the substructure. The difference of the crack pattern between P2, P4 group and P3, P7 group could be explained by Fig.7, which shows the distribution of the moment My, at which the yielding of the reinforcement starts, at P2 group and P3 group. From Fig.7 we recognize that at P3 the yielding of the bar starts at the upper cutoff section but at P2 at the bottom of the pier. It is known that at such a bar cutoff section moment cracks seem to start under a somewhat smaller load than would otherwise expected (2), and cutting off flexural bars in tension zones substancially reduces the shear strength of beams (3). From Fig.4 and Fig.6 we recongnize that the major cracks both in substructures and lateral beam are due to the longitudinal horizontal force acting to the substructures from north to south.

No structural damage was observed at the dampers and shoes, but some leakage of the viscous materials due to overpressure was observed (Fig.8) at 13 dampers out of 40 dampers. The percentage of the leaked volume to the whole volume used in a damper was 4.0 to 12.3 %. According to the experiment no substantially reduction of the resistance of a damper due to such amount of leakage was observed. After the earthquake all the leaked dampers were refilled with new material.

2. SEISMIC ANALYSIS OF NATORIGAWA BRIDGE

Earthquake response analysis was carried out both in the longitudinal and transverse direction to the bridge, which corresponds to N-S and E-W direction respectively. As an input earthquake the record obtained at the basement of Sendai Railway Building in Sendai City (Fig.9) was used. The damper is assumed as a dash pot, considering the non-linear characteristic (Fig.5). The frictional resistance of the roller shoe is neglected, and the damping ratios are assumed to be 3 % for concrete members and 5 % and 10 % for the foundations. The mathematical model used for the analysis is shown in Fig.10.

The analysis was performed, assuming that the concrete members and foundations remain elastic, and the maximum response of the bending moment Mr at substructure was compared with My at the same section (Table 1). We see here that in the longitudinal direction at all substructures Mr   My even in the case where the damping ratio h for the foundations is assumed to be 10 %, but in the transverse direction only in the case of h = 5 % at some piers Mr become larger than My. As shown in Fig.11 the piers suffered a large displacement to the south at the commencement of the earthquake. Those results of the analysis coincide with the crack patterns of the substructures, that is, at all substructures cracks remained open after the earthquake with a maximum width wider than 0.35 mm and the major bending cracks are located at the northern side of the substructures.

The longitudinal horizontal force Hy, acting at the top of the substructure and corresponding to My, is approximately 700 ton at P2, P4, P5, P6 and P8, 600 ton at P3 and P7 and 910 ton at P1 and P9. As it has been found that at all substructures H was larger than Hy, we recognized that all substructures cooperated each other to resist the longitudinal earthquake force acting on the girder.

3. EAIGAWA BRIDGE

Eaigawa Bridge, located about 50 km north from Sendai, has a distance of 120 km to the epicenter and consists of a four spans continuous pre-stressed concrete box girder as shown in Fig.12. To reduce the level of the attack in the bridge not only in longitudinal but also in transverse direction, omnidirectional dampers were installed at the three intermediate support, which have an intermediate steel box between the rectangular pipe and outer box as ilustrated in Fig.13. The two uniaxial dampers at the end supports have the same as Fig.3 in constitution.

The damage to the Eaigawa Bridge was slight compared to the Natorigawa Bridge. The percentage of the leakage of the viscous material was under 1 %, and only at P3 cracks beyond 0.2 mm width remain open along the con-struction joint as shown in Fig.14.

Earthquake response analysis was carried out with the similar method as Natorigawa Bridge, assuming a dash pot also in lateral direction at the three intermediate supports. As an earthquake record was not obtained in the vicinity, the earthquake record obtained at Kinuura in 1969, where the underground condition is similar to the site of Eaigawa, was used with an amplification to have a peak acceleration of 250 gals. From the damage to the tombstones in the vicinity, the ground acceleration in E-W direction is supposed to be larger than the acceleration in N-S direction, but as there was no quantative record, the same record was used both in transverse (E-W) and longitudinal (N-S) direction. Damping ratio was assumed to be 10 % for the foundation and 3 % for the concrete members.

The maximum response of the horizontal force Hr acting at the top of substructures and displacement of the girder r relative to the support are shown in Table 2 and compared to the design value Hd. The lateral displace-ment at the P3 exceeds the actual gap of 5 cm at the damper, that means the actual reaction of the damper probably exceeds 999 ton (Table 2). So the results of the analysis shows the ratio Hr/Hd has a maximum value at P3 in transverse direction, which coincide with the tendencies of the damage to the piers. We learned from this study that the width of the gap equal to 5 cm was not enough to fulfill the function as a damper at P3.

At the design of Eaigawa Bridge the rates of the distribution of the longitudinal seismic force Hg, acting on the girder, to the substractures was taken without any calculation as 0.2 Hg for P1 and P5 and 0.3 Hg for P2, P3 and P4. But a comparative study between dynamic analysis and statical calculations showed that we can get more reasonable value for the rates of distribution of Hg, by statical calculation replacing the dash pot (Fig.10) with an equivalent spring, at which the spring constant k is determined by the equation

$$k = \omega c$$

wherein c is the viscous damping coefficient of the damper, and ω is the natural circular frequency of the first mode of the bridge in the longitudi-nal direction replacing the damper by hinged connection.

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(2) P. M. Ferguson & F. N. Matloob, Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams, ACI Journal, Proceedings V. 56, No. 7, July 1959, PP. 5 - 23.

(3) A. M. Kao & R. E. Untrauer, Shear Strength of Reinforced Concrete Beams with Bars Terminated in Tension Zones, ACI Journal December 1975, PP. 720 - 722

Table 1. The results of the analysis (Natorigawa Bridge)

damping ratio for foundation: h	5 %	10 %
longitudinal direction	at all piers & abutments Mr > My	at all piers & abutments Mr > My
transverse direction	at abutments & Pl, P9 Mr < My at other piers Mr > My	at all piers & abutments Mr < My

Mr: max response of bending moment
My: see Fig.7

Table 2. The results of the analysis (Eaigawa Bridge)

		Pl, P5	P2, P4	P3
longitudinal directon	Hd (ton)	655	983	983
	Hr (ton)	325	628	794
	r (cm)	1.1	2.4	0.3
transverse direction	Hd (ton)	506	1080	971
	Hr (ton)	612	999	999
	r (cm)	0.0	3.7	5.1
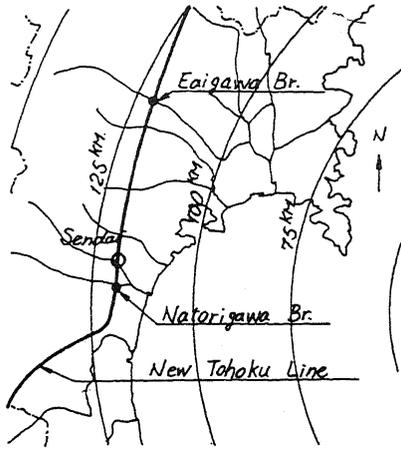
Fig.1
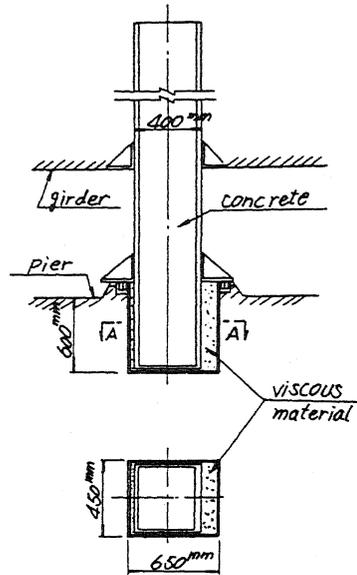
Section A-A

Fig.3 Damper
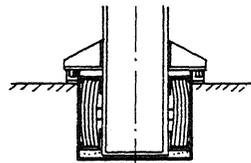
Damper with springs.

Fig.2 Natorigawa Bridge

girder

Damper

Damper

Crack at A2

Lateral Beam

Section A-A
at Pier

Section A-A
at Abutment
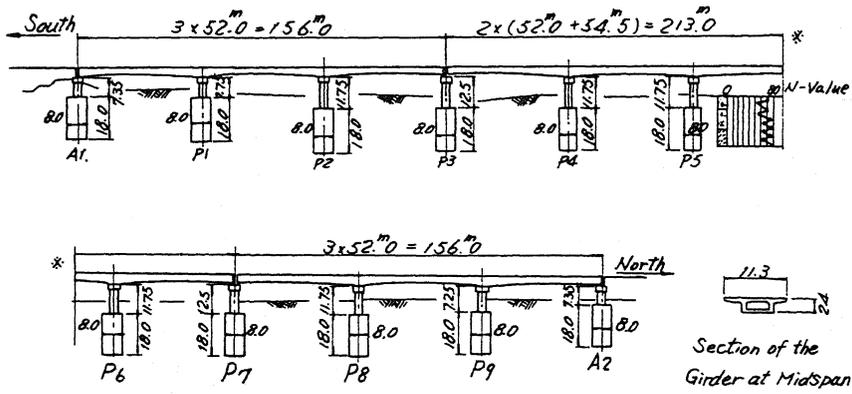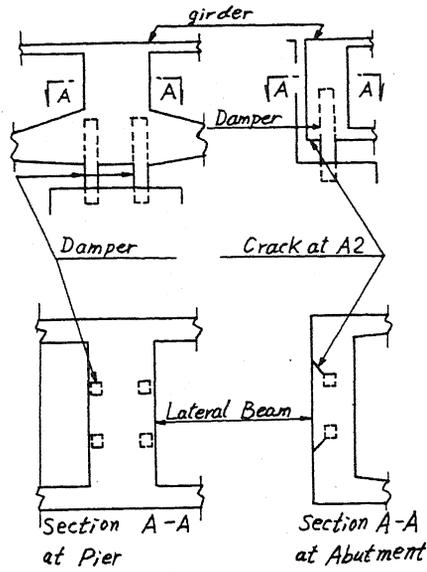
Fig.4 The Arrangement
of the Dampers
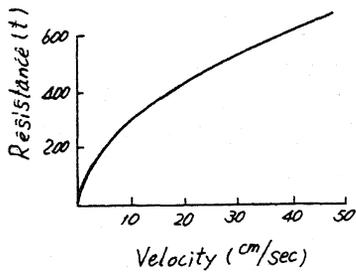
Fig.5 Resistance – Velocity
Curve of a Damper

Resistance (t)

600

400

200

10 20 30 40 50

Velocity (cm/sec)

Top of Pier

Bottom of Pier

P4

P7

A2

Vertical Bar Arrangement

Section
of Pier

3.5 m

6.0 m.

——— crack t > 0.35 mm
- - - - crack t < 0.35 mm
t: width of crack
N: North
S: South
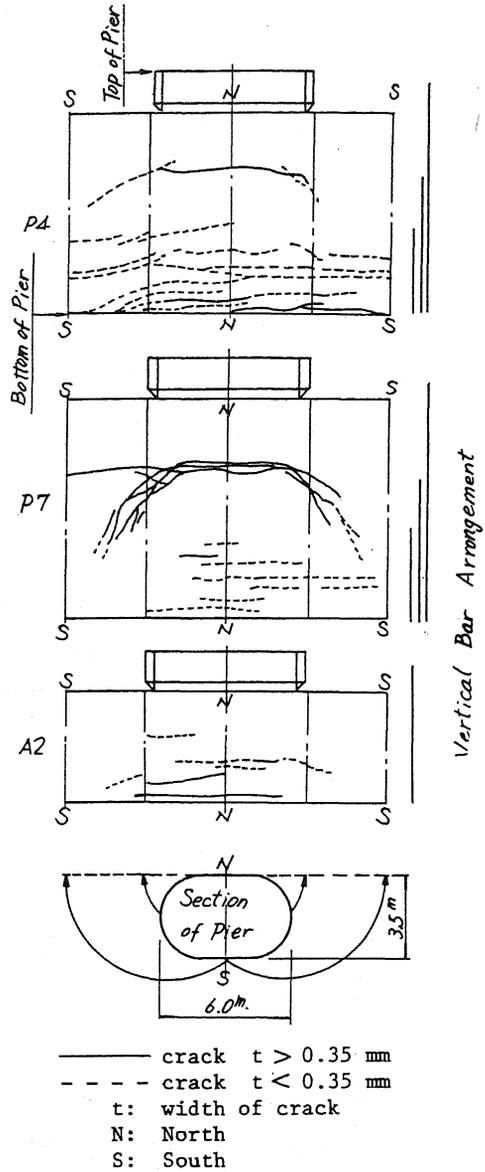
Fig.6 Crack Pattern
of Natorigawa Bridge

340
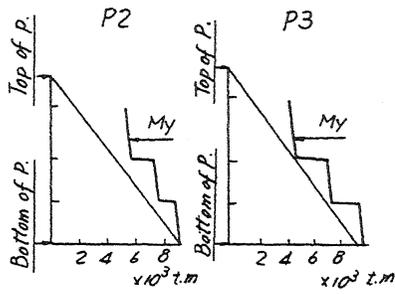
Fig.7 Distribution of My
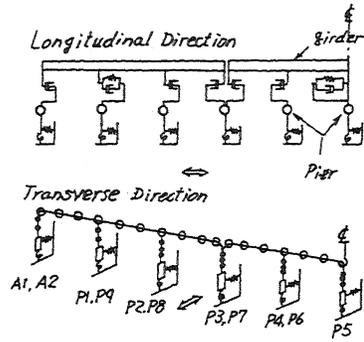
Fig.10 Mathematical Model
of Natorigawa Bridge
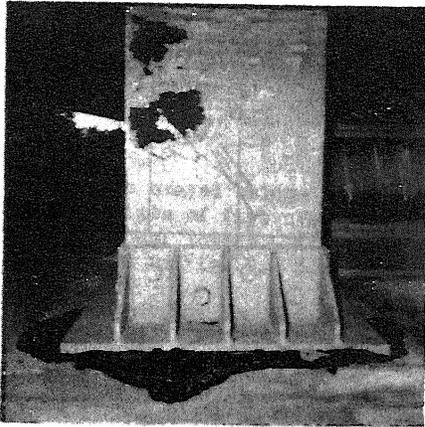
Fig.8 Leakage of Viscous
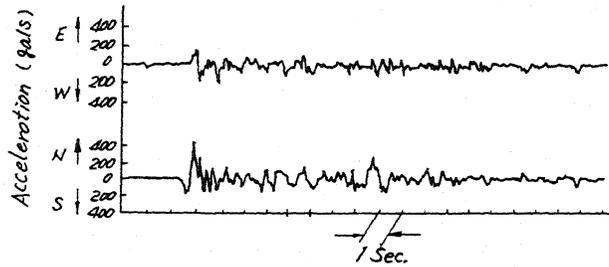Material
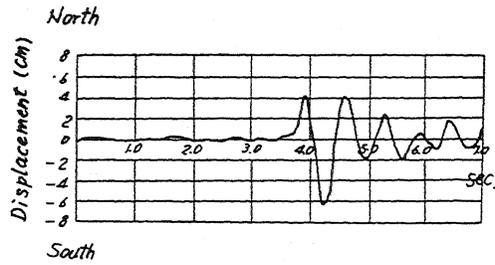
Fig.11 Horizontal Displacement
at the Top of P3

Fig.9 Earthquake Record Obtained at Sendai Railway Building
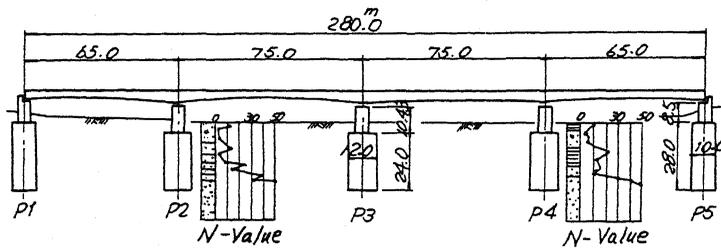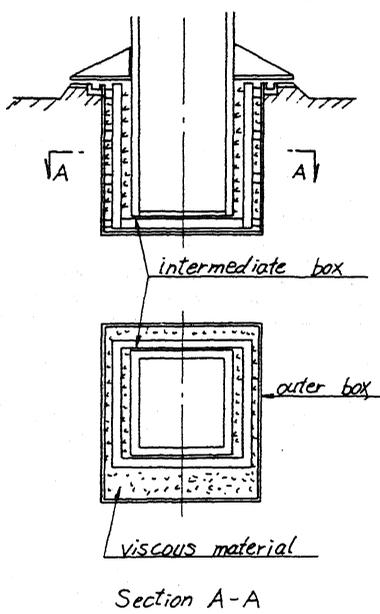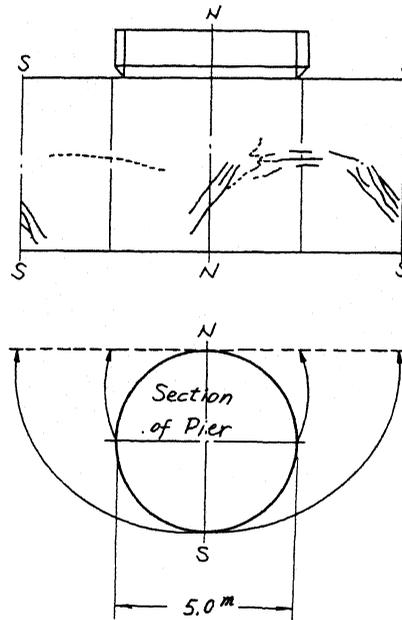
Fig.12 Eaigawa Bridge

Fig.13 Omnidirectional
Damper

──── crack   t > 0.35 mm

- - - - crack   t < 0.35 mm

Fig.14 Crack Pattern of P4