Volume 19 No. 3 1997

ISSN: 0958–9465

Cement & Concrete Composites

Special Issue on
KOBE EARTHQUAKE

Editor R.N. Swamy
Associate Editor A.E. Naaman

ELSEVIER

Cement and Concrete Composites 19 (1997) 193–201

0958-9465/97/$17.00

PII:S0958-9465(97)00017-6

Study on the Design Earthquake Resistance and Degree of Earthquake Damage of Reinforced Concrete Viaducts

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Abstract

The design earthquake resistance and actual damage caused by the Hyogoken–Nambu earthquake were studied and compared for reinforced concrete rigid-frame viaducts between Sumiyoshi and Nada on the JR Tokaido Line, which sustained the heaviest damage of all railway structures during the Hanshin-awaji earthquake disaster in January 1995 in order to find the design earthquake resistance required to withstand this earthquake. As a result of using the elastic response acceleration converted according to the law of constant energy as an index of earthquake resistance, it was found that structures with an earthquake resistance of 1000–1200 gal suffered column deformation and settlement but did not collapse, and structures with an earthquake resistance of 1200 gal or more experienced only light damage such as cover concrete peeling or less. *© 1997 Elsevier Science Ltd. All rights reserved.*

INTRODUCTION

An inland type earthquake registering a magnitude of 7.2 struck the Kinki district at 5.46 a.m. on 17 January 1995, causing vast numbers of casualties and severe damage in all areas. The epicenter of this earthquake was located in the northern part of Awaji Island (lat.: 34.6°N; long.: 135.0°E; depth: approximately 20 km).

This earthquake caused severe damage to harbors, roads, lifeline facilities and other civil structures.

The earthquake also damaged railway structures, with reinforced concrete (RC) rigid-frame viaducts in particular suffering heavy damage such as fallen bridges, collapse, settlement and major deformation. Major damage to railway structures is shown in Fig. 1.

The authors studied and compared the earthquake resistance and damage conditions of RC rigid-frame viaducts between Sumiyoshi and Nada on the JR Tokaido Line, which sustained the heaviest damage of all railway structures, including the Shinkansen, conventional lines and subways in order to find the design earthquake resistance required to withstand this earthquake. This paper is a report of these results.

Shinkansen viaducts were also severely damaged in a number of locations. However, most of this damage was due to column shearing failure, making these structures inappropriate for the purpose of finding the required earthquake resistance. Therefore, Shinkansen viaducts were excluded from this study.

TRANSITIONS IN EARTHQUAKE RESISTANT DESIGN REGULATIONS FOR CONCRETE RAILWAY STRUCTURES [1,2]

Regulations concerning the design of concrete railway structures date back to the 'July 14, 1914 Government Notice No. 684, Guidelines for the Design of Reinforced Concrete Bridges'. However, these guidelines did not include regulations regarding earthquakes.

Bridge pier design data noted in the 'June 12, 1919 Government Notice No. 541, Guidelines for Using Abutment and Bridge Pier Standards for Incidental Roller Compacted and Steel Beams' contains a description of obtaining the earthquake acceleration by calculating backwards from the allowable tensile stress of non-reinforced concrete as the allowable earthquake acceleration. According to the calculation example, this value is 0.12–0.14 G (G = gravitational acceleration).

The '1928 Ministry of Railways Notice No. 158, Steel Railway Bridge Design Guidelines' stipulates that 'Earthquake motion should be taken into account when designing attachments between bridge beams and lower structures'. However, clear figures for earthquake motion are not provided.

The Bridge Standard Designs enacted by the Construction Bureau in 1930 clearly state that a horizontal seismic intensity of 0.2 should be taken into account with respect to dead load and earth pressure. These are the first standards for railway structures that give a specific design seismic intensity.

The first earthquake resistant design regulations following the establishment of the Japan National Railways were the 'Design Standards for Non-reinforced and Reinforced Concrete Civil Structures'[3] enacted in 1955. These standards set the horizontal seismic intensity from 0.15 to 0.3 by district in consideration of the effects of earthquakes on dead load, earth pressure and water pressure.

After that, the 'Structural Design Standards'[4] enacted in 1970 and later revised in 1974 were also based on the concept of design according to seismic intensity and set the horizontal seismic intensity from 0.12 to 0.24 by region and type of ground. Designs continued to be based on the seismic intensity method up until the 'Earthquake Resistant Design Guidelines and Commentary'[5] enacted in 1979 in reflection of the experiences from the Miyagiken-oki Earthquake that occurred in 1978.

The above-mentioned 'Earthquake Resistant Design Guidelines and Commentary' enacted in 1979 stipulated that design should be performed using the seismic intensity, revised seismic

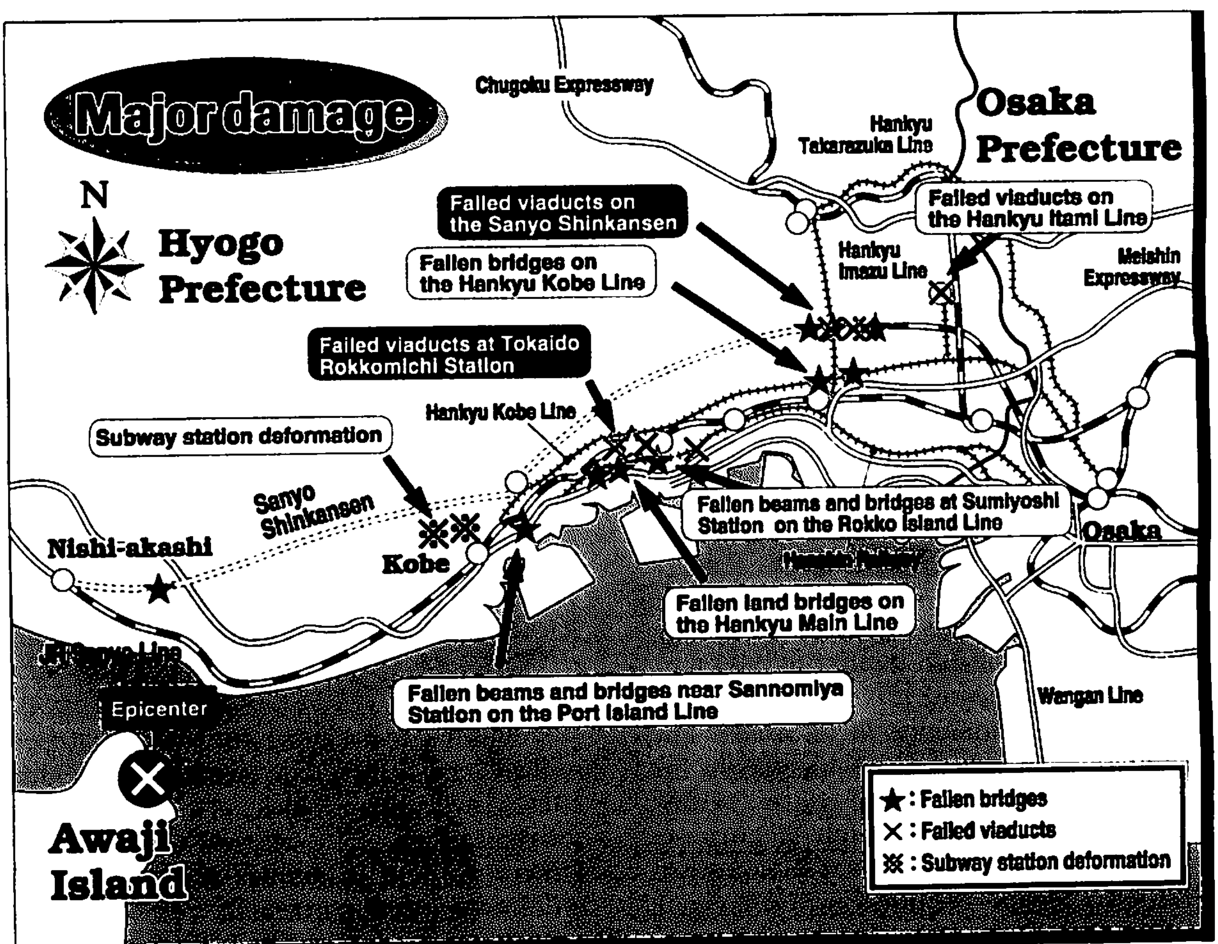


Fig. 1. Major damage to railway structures.

intensity or dynamic analysis method according to the dynamic response of the structure. However, these guidelines still did not deviate from the basic course of design using the conventional seismic intensity method, and the horizontal seismic intensity calculated using the seismic intensity or revised seismic intensity method ranged from 0.084 to 0.25 by region, type of ground and response characteristics.

The 'Structural Design Standards (Reinforced and Non-reinforced Concrete Structures)'[6] revised in 1983 were the last design standards to be based on the allowable stress method. However, these standards followed the above-mentioned 'Earthquake Resistant Design Guidelines and Commentary', and performed design using the seismic intensity method for structures with a characteristic cycle of less than 0.3 s or the revised seismic intensity method for a characteristic cycle of 0.3 s or more in consideration of ground characteristics. In addition, structures with a characteristic cycle of 2 s or more were analyzed using the dynamic analysis method. The design horizontal seismic intensity is the same except for the fact that the standard horizontal seismic intensity in the above guidelines is renamed the reference horizontal seismic intensity. However, these standards investigate the safety with respect to material section failure during earthquakes using a load combination of '1.0 × (dead load) + 1.0 × (train load) + 1.5 × (effects of the earthquake)' for bridge piers, rigid-frame bridges, arches and flat slab structures. In addition, the structural specifications for earthquake resistance stipulate that designs should ensure a ductility of about 4 or more based on past cases of damage and experimental results. Ductility here refers to the value obtained by dividing the limit displacement where the material strength does not fall below the yielding strength when subjected to around ten cyclic loads by the yielding displacement. These standards are both compatible with conventional designs, and at the same time aim to ensure structural ductility in the event of major deformation. The horizontal seismic intensity assuming that the structure exhibits an elastic response is thought to be 1.0. Although somewhat hidden in form, this is clearly a safety standard for concrete structures with respect to so-called major earthquakes, and is a design method that aims to ensure earthquake resistance for an elastic response during major earthquakes of about 1 G by ensuring sufficient structural strength and deformation performance.

The current 'Design Standards and Commentary for Railway and Other Structures (Concrete Structures)'[7] enacted in October 1992 conform to world trends and are design standards based on the limit states design method. Although the basic concept for the earthquake resistant design method is the same as that for the 1983 design standards, these standards are the first to give positive form to a design concept for major earthquakes.

These standards assume the elastic response during earthquakes, which is taken into account for design to be 1 G, and adopt the method where the safety is investigated with respect to strength and ductility in order to confirm that the design allowable ductility falls within the ductility of the structure. In addition to investigating structural safety, train running safety is also investigated separately for medium scale earthquakes (about 0.2 G), which are anticipated to occur a number of times during the design service life of the structure from the viewpoint of ensuring train running safety during general earthquakes.

VIADUCTS BETWEEN SUMIYOSHI AND NADA ON THE JR TOKAIDO LINE

The viaducts studied are located in the approximately 2.2-km-long section extending from Sumiyoshi Station through Rokkomichi Station to Nada Station. This section has a quadruple-track structure comprising two parallel double-track viaducts, and straddles roads in various locations with over-road bridges centering on RC rigid-frame viaducts.

Structurally, this section consists of parallel Tokyo- and Kobe-bound 2-line, 2-column, beam-slab type rigid-frame viaducts with a 3-span standard. The ground is alluvial sandy and gravely soil with an N value of 3–50 deposited approximately 2–10 m from the ground surface. This layer is used as the bearing layer by the viaduct foundations, of which 86% are direct foundations and the remainder are group-pile foundations consisting of 24 cast-in-place piles per footing. Cast-in-place piles are 1.1–4.0 m long, 30 cm in diameter and have H-beam cores.

These rigid-frame viaducts were designed according to the 'Japan National Railways

Structural Design Standards' enacted in 1970. The Tokyo-bound line was completed in 1973, and the Kobe-bound line in 1976. Accordingly, these viaducts were designed using the seismic intensity method and have a design horizontal seismic intensity of 0.2.

The viaducts in this section suffered extremely severe damage including the collapse of Rokkomichi Station. The main types of damage were fallen bridges, collapse, settlement and major deformation due to damaged and failed columns of rigid-frame viaducts. Photograph 1 shows some typical examples of damage, and Fig. 2 provides an overview of the section. This section is approximately 2.2 km in length and contains 158 blocks of rigid-frame viaducts. The sections before and after the viaducts consist of embankments.

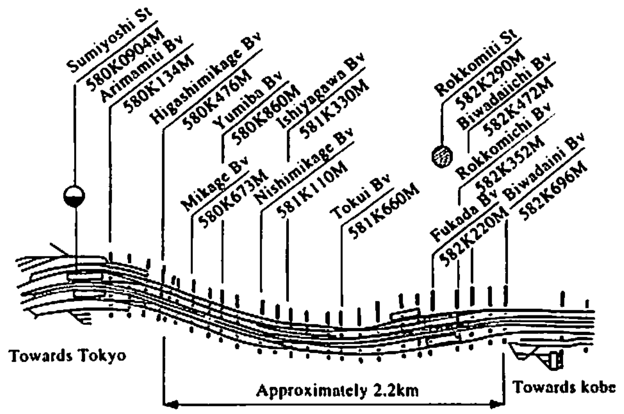


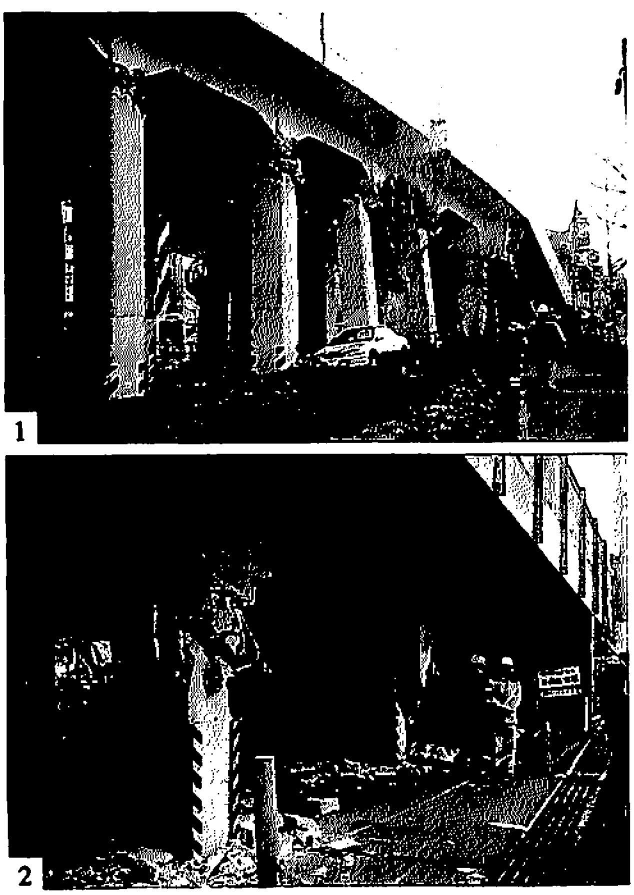
Fig. 2. Viaducts between Sumiyoshi and Nada on the JR Tokaido Main Line.

METHODS OF INVESTIGATING THE EARTHQUAKE RESISTANCE OF VIADUCTS

The earthquake resistance of viaducts was calculated using the formula for verifying deformation performance given in the 'Design Standards for Railway and Other Structures (Concrete Structures)' enacted in 1992 and used in the current design of RC railway structures. Essentially, the ductility* of viaduct columns was calculated using the formula for verifying deformation performance. Next, the horizontal seismic intensity $K_E = K_y \times (2\mu - 1)$, assuming that the structure exhibits an elastic response, was calculated from the ductility and the material yielding horizontal seismic intensity K_y based on N. M. Newmark's law of constant energy shown in Fig. 3. Then, the earthquake resistance of viaducts was set at $P_E = 1000 \times K_E$ (gal). Note that the original average formula was used to calculate the ductility instead of the design formula given in the design standards, which takes into account the safety ratio for verifying deformation performance.[8]

The conditions used to calculate the earthquake resistance of actual structures are listed below.

[Calculation conditions]



Photograph. 1. Typical examples of damage.

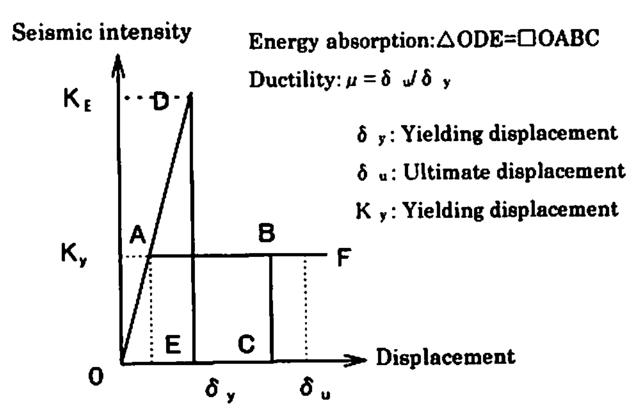


Fig. 3. N. M. Newmark's law of constant energy.

- Cross-section dimensions and bar arrangements are assumed to be as stated in the design drawings.
- Material strengths are assumed to be as follows.

Concrete compression strength
$f'_{ck} = 23.5$ MPa
Rebar yielding strength
$f_{sy} = 343.4$ MPa [SD345]
$f_{sy} = 294.3$ MPa [SD295]
$f_{sy} = 235.4$ MPa [SD235].

- Axial force is constant at an axial compressive stress of $\sigma_N = 0.98$ MPa
- Ductility is calculated using the formulas below based on past research:[8]

$$\mu = (\mu_0\delta_{y0}+\delta_{u1})/(\delta_{y0}+\delta_{y1}) \tag{1}$$

$$\mu_0 = -1.9+6.6V_{yd}\cdot l_a/M_{ud}+(13p_w-1.6)p_w, \tag{2}$$

where,
μ = ductility,
μ_0 = ductility of only the frame,
δ_{y0} = yielding frame displacement,
δ_{y1} = yielding rotational displacement due to axial rebars slipping out,
δ_{u1} = ultimate rotational displacement due to axial rebars slipping out,
V_{yd} = shearing strength,
l_a = shearing span,
M_{ud} = bending strength, and
p_w = tie hoop ratio.

Note that the formula for calculating ductility in the current design standards assumes eqn (2) above to be 80% reliable, and recalculates μ_0 using the formula below.

$$\mu_0 = -1.6+5.6V_{yd}\cdot l_a/M_{ud}+(11.4p_w-1.4)p_w. \tag{3}$$

- The horizontal seismic intensity when the materials reach the yielding point is calculated using the formula below.

$$K_y = 1.5K_h,$$

where:
K_y = horizontal seismic intensity at the material yielding point, and
K_h = design horizontal seismic intensity (0.2).

Note that while it is necessary to strictly calculate the load and material yielding horizontal seismic intensity, in this study, the horizontal seismic intensity at the material yielding point was set to $1.5 \times$ (horizontal seismic intensity) = 0.3. This was done in order to take into account the effects of side rebars, which were ignored in the designs at that time, and due to the fact that designs generally have some margin. In consideration of these factors, the yielding strength of the column materials varies slightly for each rigid-frame viaduct, but since most columns are thought to have a yielding strength of about 1.5 times the design horizontal seismic intensity, this value was applied universally in this study. Taking a column with a cross-section of 900 mm × 900 mm (Fig. 4) in these viaducts as a specific example, the bending yielding strength is 1091 KN. However, the design for these viaducts ignores the effects of side rebars and only takes the outermost axial rebars into account when investigating the allowable bending moment M_{ad}. This results in a value of 859 KN for a horizontal seismic intensity of 0.2. Therefore, the safety with respect to the bending yielding strength is as follows.

(Bending yielding strength M_{yd} = 1091 KN
(Allowable bending moment M_{ad}) = 859 KN
(Safety with respect to the bending yielding strength) = M_{yd}/M_{ad} = 1.27.

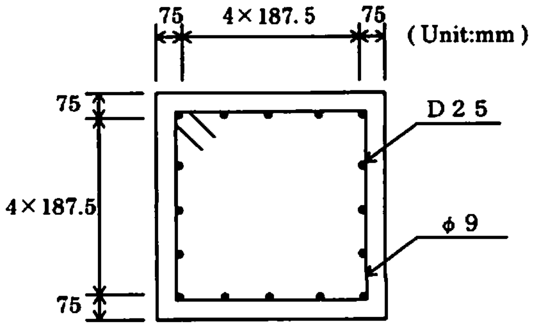


Fig. 4. RC viaduct column cross-section (900 × 900 mm).

Therefore, taking into account the design margin, the yielding horizontal seismic intensity of the materials is thought to be about 1.5 times the design horizontal seismic intensity.

STUDY AND COMPARISON OF EARTHQUAKE RESISTANCE AND DEGREE OF DAMAGE

Determining the degree of damage

The degree of damage for all 158 blocks of rigid-frame viaducts in the studied section was determined according to the damage conditions of these viaducts. Table 1 shows the table used to determine the degree of damage for actual bridges. The degree of damage was determined according to the most severely damaged column for each rigid-frame viaduct, and was classified in order of descending degree of damage into ranks A, B and C.

Here, rank A corresponds to fallen beams and slabs or collapsed structures due to column damage or failure. Rank B corresponds to settled beams and slabs or deformed (but not fallen or collapsed) structures due to partial column damage. Rank C corresponds to light damage such as shearing and bending cracking or cover concrete peeling.

Note that the damage was determined from photographs of damage conditions.

Earthquake resistance and degree of damage

Table 2 shows the results for the calculated earthquake resistance and the actual degree of damage for all 158 blocks of rigid-frame viaducts within the studied section.

Here, the calculated damage type was determined by the ratio of the material shearing and bending strengths. Viaducts were determined to suffer shearing first if this ratio was less than 0.9, or bending first if this ratio was 0.9 or more.

In the table, calculated earthquake resistance values in parentheses are for when the ductility is calculated using the design formula in the current design standards.

The earthquake resistance values in Table 2 are the smaller of the values in the directions of and perpendicular to the bridge axis. However, these values were roughly the same in both directions, and arranging the data in Table 2 by the directions of and perpendicular to the bridge axis, or by the larger of the earthquake resistance values, produced roughly the same results.

The following items were understood from Table 2.

(1) As the calculated earthquake resistance used in this study rose, the degree of damage became smaller.
(2) There was no major damage at an earthquake resistance of 1200 gal or more. (All damage was rank C or less.)
(3) The maximum damage was rank B and there was no rank A damage at an earthquake resistance of 1000–1200 gal.
(4) Rank A damage appeared at an earthquake resistance of 800–1000 gal.
(5) Many viaducts that experienced shearing damage first suffered severe rank A damage such as fallen beams and slabs or structural collapse.
(6) When the average formula was used for the yielding horizontal seismic intensity and ductility, which took into account the design margin, and a strength of 1.3 times the specification values and design standard strengths was used as the material strength, the earthquake resistance that resulted in C rank damage or lower was calculated to be about 1500 gal. Therefore, an earthquake resistance of roughly 1500 gal is thought to be sufficient to withstand even the maximum force acting on these viaducts assuming an elastic structural response.

Next, the relationship between the equivalent characteristic period, earthquake resistance and degree of damage for these viaducts is shown in Fig. 5. The values only for those viaducts which were calculated to suffer bending damage first are plotted in this figure. Also, viaducts with roughly the same earthquake resistance which experienced the same degree of damage are omitted.

Note that the material yielding rigidity was used to calculate the equivalent characteristic period, and a model was created that integrated the foundations and structures with a spring[7]. The design value is used as the spring constant[9].

The following items were understood from Fig. 5.

(7) The equivalent characteristic period of the studied viaducts ranged from about 0.4 to 0.9 s.

(8) There were no clear trends between earthquake resistance and degree of damage for the equivalent characteristic cycle range of 0.4–0.9 s.

SUMMARY

The items understood from the above study results are as follows.

(1) As a result of calculating the earthquake resistance by the methods currently used in the design of RC structures and com-

Table 1. Table for determining the degree of damage

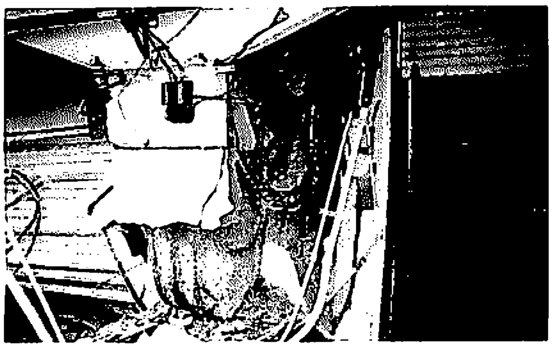
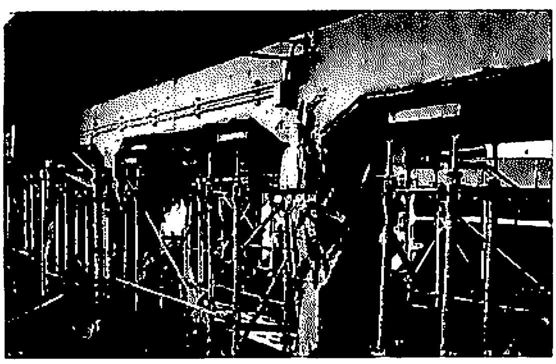
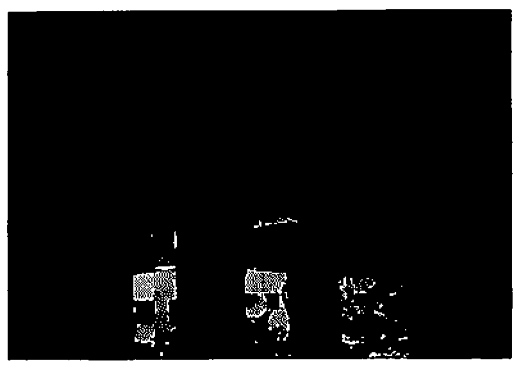
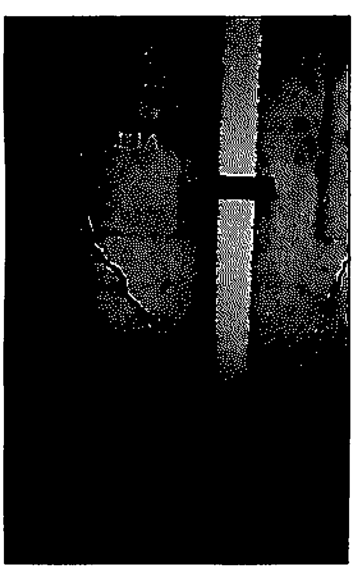
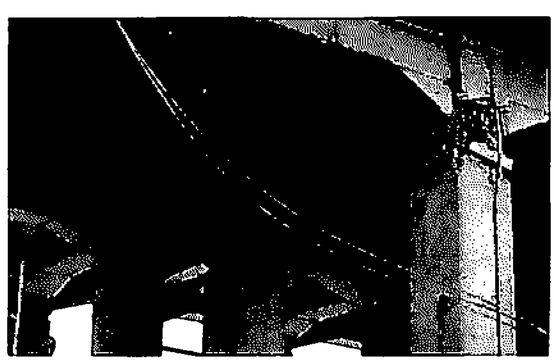
Rank	Photographs	
A		
	Fallen beams and slabs	Collapsed columns
B		
	Settled and deformed beams	Column cover concrete, peeling, exposed rebars
C		
	Settled and deformed beams and slabs	Column cover, concrete peeling, exposed rebars

Table 2. Calculated earthquake resistance and degree of damage

Damage type and earthquake resistance (calculated)		*Total number*	*RankA*		*RankB*		*RankC*		*Minute cracking*	
			Absolute number	*Proportion (%)*	*Absolute number*	*Proportion (%)*	*Absolute number*	*Proportion (%)*	*Absolute number*	*Proportion (%)*
Bending first	1200 gal or more (1100 gal or more)	7	0	0	0	0	6	86	1	4
	1000–1200 gal (915–1100 gal)	16	0	0	4	25	10	63	2	12
	800–1000 gal (728–915 gal)	54	11	20	8	15	20	37	15	28
	800 gal or less (728 gal or less)	22	7	32	4	18	2	9	9	41
Shearing first	—	59	30	51	10	17	6	10	13	22

Viaducts are assumed to suffer shearing damage first if the shearing to bending strength ratio is less than 0.9, or bending damage first if this ratio is 0.9 or more.
Earthquake resistance values in parentheses are for when the ductility is calculated using the design formula in the current design standards.

paring these values with actual damage conditions, the degree of damage suffered by actual structures was found to decrease as the earthquake resistance increased.

(2) Viaducts with an earthquake resistance of 1200 gal or more did not exhibit Rank A or B damage, and suffered only relatively light damage.

(3) Viaducts with an earthquake resistance of 1000–1200 gal did not exhibit A rank damage. However, some of these structures suffered B rank damage, with columns settling or deforming slightly but not collapsing.

(4) Many viaducts that suffered shearing damage first exhibited A rank damage. A high proportion of these viaducts sus-

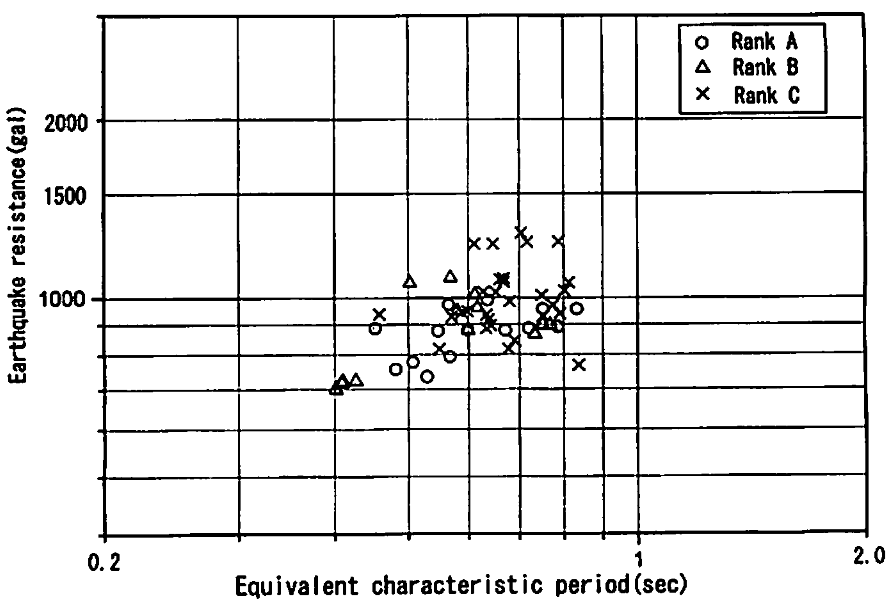


Fig. 5. Relationship between the equivalent characteristic period, earthquake.

tained heavy damage, with rank A and B damage accounting for 68%.

CONCLUSION

This study investigated the design earthquake resistance of actual damaged structures and RC railway structures. It is the authors' hope that the results of this study will aid in the preparation of new earthquake resistant design standards in the future.

ACKNOWLEDGEMENTS

The authors wish to express their deep thanks to all staffs at the Structural Technology Center of the Construction Division and the Construction Control Room of the Tokyo Construction Office of East Japan Railway Company for their immense cooperation in calculating the earthquake resistance in this study, and to all concerned parties at West Japan Railway Company for providing invaluable materials and data.

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