

BULLETIN OF PUBLIC WORKS RESEARCH INSTITUTE

VOLUME 29

EARTHQUAKE-RESISTANT DESIGN
OF BRIDGES IN JAPAN

BY

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EARTHQUAKE ENGINEERING RESEARCH SECTION
STRUCTURE AND BRIDGE DIVISION

MAY 1973

**PUBLIC WORKS RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION**

2-28-32 HONKOMAGOME, BUNKYO-KU, TOKYO, JAPAN

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土木研究所彙報第29号

日本における橋梁の耐震設計(英文)

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要 旨 : この彙報は橋梁の耐震性に関する研究成果を報告したものであり、日本における橋梁の被害例ならびに関連する既往の研究成果を網羅するとともに、我国の橋梁の現行耐震基準を要約し、諸外国の基準との比較を行っている。

この調査の結論として、道路橋の耐震安全性を確保するための現耐震基準の有効性を論じ、耐震設計上考慮すべき事項を明らかにし、さらにより合理的な耐震設計法を確立する上に必要とされる研究課題を指摘している。

EARTHQUAKE-RESISTANT DESIGN OF BRIDGES IN JAPAN

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FOREWORD

The bulletin is published as a formal report of the author's mission at the University of California, Berkeley, California, U.S.A. during the period of July 1st through October 9th, 1971.

Most of the work presented in this bulletin was originally prepared by the author while he was participating in a research project "Seismic Effects on Bridges" at the University, through a grant from the Japanese Ministry of Construction. The author's participation was recognized as an activity of the Panel on Wind and Seismic Effects of the U.S.-Japan Cooperative Program in Natural Resources (UJNR), and therefore during the above period he was appointed to be a temporary member of the UJNR Panel.

Professor Joseph Penzien, Director of the Earthquake Engineering Research Center at the University of California, Berkeley acts as the principal investigator of the abovementioned research project, which is sponsored by the Federal Highway Administration, U.S. Department of Transportation.

ABSTRACT

This report is a complete review of the literature on seismic effects on bridges. The literature survey was conducted with specific attention to Japanese reports which document seismic damage to bridge structures and present the results of investigations on the subject. The report, furthermore, summarizes several existing specifications for earthquake-resistant design of bridges in Japan and compares them with earthquake regulations of structures in various countries.

From the extensive study, conclusions are drawn on effectiveness of current Japanese specifications for highway bridges, essential matters to which particular attention should be paid in the aseismic design of bridges, and research subjects on which further investigations are needed to establish reasonable design methodology in providing adequate protection against earthquake damage.

ACKNOWLEDGMENTS

The author is very grateful to Professor Joseph Penzien for his guidance and many helpful discussions in the preparation of this report.

The author wants to thank the executive staff at the Japanese Ministry of Construction, for their grant for the author to participate in the research project, and wants to thank Dr. E. O. Pfrang and Dr. S. Ibukiyama, the Chairmen of the Panel on Wind and Seismic Effects of the U.S.-Japan Cooperative Program in Natural Resources (UJNR) operated by the Governments of the two countries, for their cooperation through the UJNR activities. He also wants to thank Dr. T. Okubo and Mr. E. Kuribayashi at the Public Works Research Institute, Ministry of Construction, for their efforts in completing the project successfully.

The author wishes to express his appreciation and sincere gratitude to Mrs. Elenora D. Utter and Mr. William E. Wagy at the Earthquake Engineering Research Center of the University of California, Berkeley, for their help and encouragement during the course of this work.

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CHAPTER 1 INTRODUCTION

The Earthquake Engineering Research Center, University of California, Berkeley is conducting a comprehensive research project on seismic effects on bridges, which is sponsored by the U.S. Federal Highway Administration. The principal purpose of the project is to investigate the effectiveness of existing highway bridges design methodology in providing adequate structural resistance to seismic disturbances and to give recommendations for changes in design specifications and methodology to provide more adequate protection against future earthquake damage. To complete the project starting in 1971, it is expected to take three years.

The first phase of this project is to make a thorough review of the literature on seismic effects on bridges and related structures and then to define a research program which would be carried out in the subsequent phases. The literature survey has been completed and is presented in this report, with specific attention to those reports documenting the experience and investigations of bridge engineers in Japan where the incidence of damage to modern structures due to earthquakes has been considerable.

The report consists of five chapters and three appendices as follows:

- Chapter 1 Introduction
- Chapter 2 Seismic Damage to Bridge Structures
- Chapter 3 Research Activities on Seismic Effects on Bridge Structures
- Chapter 4 Earthquake-Resistant Design Criteria for Bridges
- Chapter 5 Summary and Conclusions
- Appendix A Bibliography on Seismic Effects on Bridges
- Appendix B Briefing of Seismic Design Codes in Various Countries
- Appendix C "Specifications for Earthquake-Resistant Design of Highway Bridges," Japan Road Association, January, 1971

Chapter 2 provides the experienced instances of seismic damage to bridge structures due to nine major earthquakes occurred in Japan since 1923, and summarizes general damage features. Chapter 3 briefs research activities which are related to seismic effects on bridges, and de-

scribes fruitful results obtained. Chapter 4 summarizes several existing specifications for earthquake-resistant design of bridges in Japan. Chapter 5 indicates the summary and the conclusions of the extensive literature survey.

Furthermore, Appendix A is a list of bibliography, primarily Japanese literature, on seismic effects on bridges. Appendix B summarizes, in a prescribed form, earthquake regulations in various countries, including those for bridges in the U.S. and Japan. And finally, Appendix C presents a complete translation of "Specifications for Earthquake-Resistant Design of Highway Bridges," issued in January, 1971, by the Japan Road Association.

CHAPTER 2 SEISMIC DAMAGE TO BRIDGE STRUCTURES

For getting better understanding of seismic effects on any structures and for pursuing a reasonable design method for seismic forces, it seems very important to investigate seismic damage to similar existing structures due to earthquakes previously experienced. This chapter deals with the characteristics of seismic damage to bridge structures including superstructures, substructures, and surrounding soils on the basis of the actual damage experiences during the past earthquakes in Japan.

Fig. 2.1 provides the brief information of nine major earthquakes which caused comparatively severe damage to modern engineering structures since 1923^{51), 210)*}. In the following sections (2.1 through 2.9) the damage characteristics of bridge structures caused by the nine earthquakes will be discussed individually, and in 2.10 general features of seismic damage to bridge structures will be summarized.

2.1 Kanto Earthquake of 1923²³⁾

A severe earthquake occurred about 10 km south off the coast of Sagami Bay in southern Kanto area which includes Tokyo, Yokohama and other major cities of central part of Japan on September 1, 1923. It was the first attack on modern facilities in Japan. The earthquake counted a magnitude of 7.9 on the Richter scale. Substantial damage to bridge structures as well as other engineering structures was caused throughout the southern Kanto area, especially in Tokyo, Yokohama and in the vicinity of the epicenter. Although the damage due to numerous

* Numerals indicate the numbers of references listed in Appendix A.

NO	Date	Name	M
1	Sept 1, 1923	Kanto	7.9
2	Dec. 21, 1946	Nankai	8.1
3	Jun. 28, 1948	Fukui	7.3
4	Dec. 26, 1949	Imajiri	6.4
5	Mar. 4, 1952	Tokachi-oki	8.1
6	Apr. 30, 1962	Northern Miyagi	6.5
7	Jun. 16, 1964	Niigata	7.5
8	Feb. 21, 1968	Ebino	6.1
9	May 16, 1968	Tokachi-oki	7.9

(*) Magnitudes are on the Richter scale, after Rika Nenpyo (Annual Report of Science) (1971).

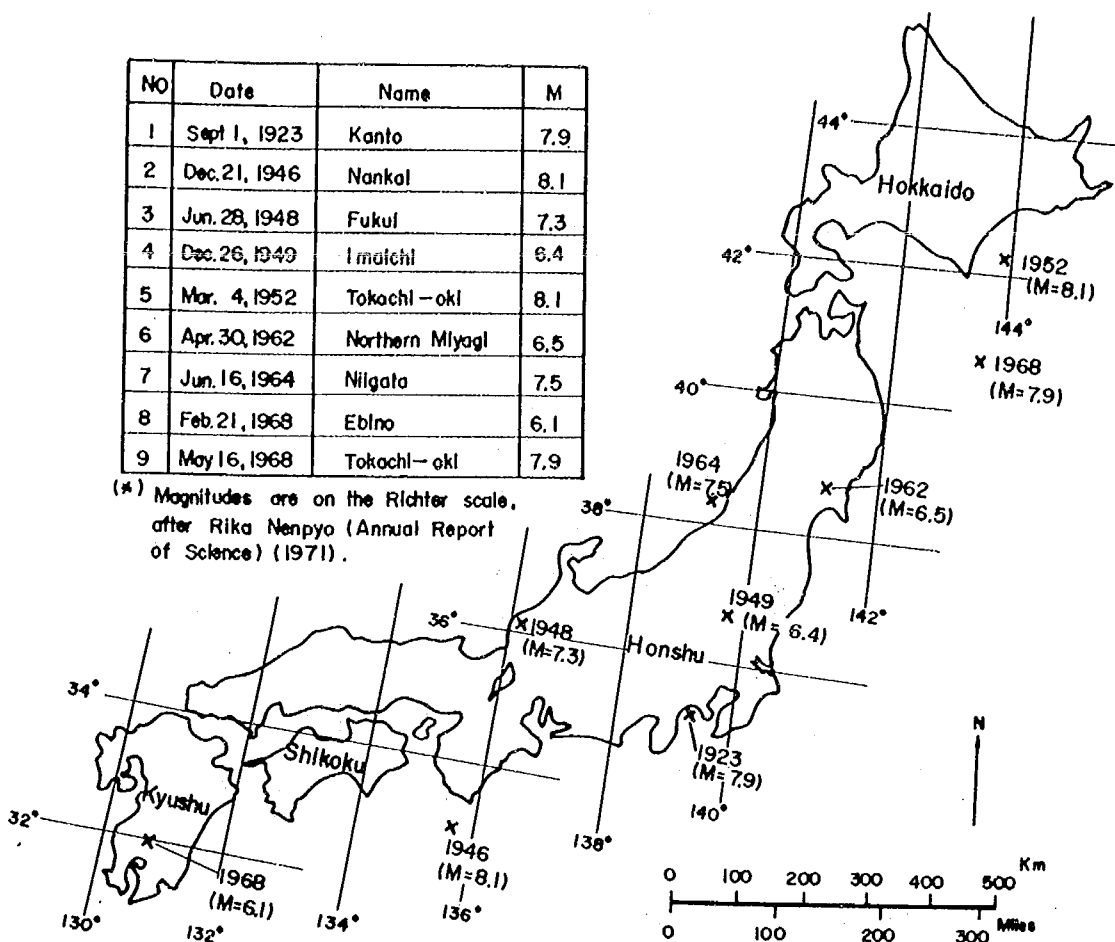


Fig. 2.1 Epicenters of nine earthquakes which caused comparatively severe damage to modern engineering structures in Japan

fires prevailed in the huge cities, the vibrational effects of the earthquake on engineering structures were also extensive..

It seems that the Kanto Earthquake is one of the most significant earthquakes which affected the design method of structures against seismic forces in Japan. Although before the earthquake no distinct regulations existed for seismic forces and accordingly there were only a few structures in which seismic forces had been considered in the design, after that specifications for earthquake-resistant design of several kinds of structures were stipulated and seismic forces were taken into account in the design of the structures (see Chapter 4 on specifications for highway bridges).

The engineers made an extensive investigation on the damage to bridge structures as well as

other structures²³). The outline of the Kanto Earthquake is as follows⁵¹):

Name of Earthquake	Kanto Earthquake
Date of Occurrence	September 1, 1923
Epicenter	Southern Kanto (35.2°N, 139.3°E)
Depth of Hypocenter	0 - 20 km
Magnitude on the Richter Scale	7.9

The location of the epicenter is shown in Figs. 2.1 and 2.2. The JMA Intensity in the Fig. 2.2 indicates the seismic intensity at various places that was determined by the Japan Meteorological Agency (JMA). The definitions of the JMA Intensity are tabulated on Table 2.1. In the table the corresponding magnitudes of accelerations on the ground surface which was suggested

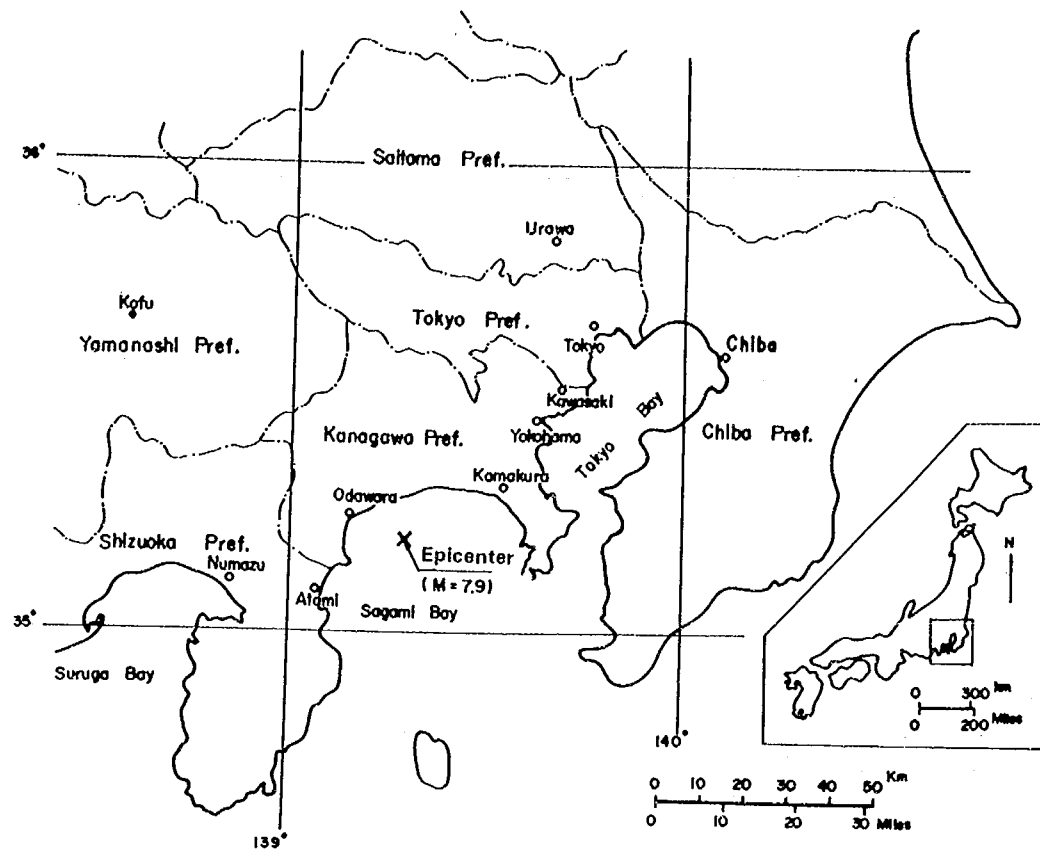


Fig. 2.2 The Kanto Earthquake of 1923

by H. Kawasumi⁸⁰⁾ are also shown.

Table 2.2 indicates the number of highway bridges sustained damage. From the table it is known that the damage to bridge structures due to the Kanto Earthquake was very extensive, and that about two thousand highway bridges suffered. The damage was severest in Kanagawa Prefecture to which the epicenter was very close. It is also remarkable that the damage in the city of Tokyo was mostly caused by fires rather than the vibrational effects of the earthquake. Although hundreds of bridges suffered by fires in the city of Tokyo, the number of bridges suffered by the vibrational effects was only 18.

On the other hand, the damage characteristics were different in the city of Yokohama, Kanagawa Prefecture, where the percentage of bridges suffered by the vibrational effects is very high. The following describes individually the characteristics of the damage caused by the vibrational effects on several bridges which suffered comparatively heavily.

Tsuruno-bashi Bridge (Fig. 2.3)

This bridge, completed in 1914, was on Bandaicho-Horaicho Road located in Yokohama, Kanagawa Prefecture about 40km northeast from the epicenter, and was across the Shin-yoshida River. The ground of the construction site was very soft alluvial layer. The both abutments were of brick masonry with the concrete foundations. Each of two piers was made of four spiral single-row cast-iron-pipe piles with some bracing as shown in Fig. 2.3. The superstructures were consisted of 3-span simple steel plate girders. The total length of the bridge was 33.6m (= 10.5m + 12.6m + 10.5m), and the effective width of the pavement was 7.3 m.

Due to the earthquake there were moderate damages to the bridge. The both abutments moved and tilted toward the center of the river, and two piers tilted considerably toward the left bank. The 3-span superstructure moved largely toward the left-bank (left when loading from the

Table 2.1 Definitions of JMA Seismic Intensity Scale

Scale	Definitions	Corresponding Magnitude of Accelerations*
0	No Feeling: Too weak to cause human feeling, to be registered only by seismographs.	0 – 0.8 gals
1	Slight: To be felt only feebly by persons at rest or by those who are observant to an earthquake.	0.8 – 2.5 gals
2	Weak: To be felt by most persons, causing slight shaking doors and Japanese latticed sliding doors (Shoji).	2.5 – 8 gals
3	Rather strong: To cause shaking of houses and buildings, heavy rattling of windows and Shoji, swinging of hanging objects, stopping sometimes pendulum clocks and moving liquid in vessels. Some persons are so frightened as to run out of doors.	8 – 25 gals
4	Strong: To cause strong shaking of houses and buildings, overturning of unstable objects, and spilling of liquid out of vessels.	25 – 80 gals
5	Very strong: To cause cracks in the brick and plaster walls, overturning of stone lanterns and grave stones etc. and damaging of chimneys and mud-and-plaster warehouses. Landslides in steep mountains are to be observed.	80 – 250 gals
6	Disastrous: To cause demolition of Japanese wooden houses less than 30%, intense landslides, fissures on the flat ground accompanied sometimes by spouting of mud and water in low fields.	250 – 400 gals
7	Ruinous: To cause demolition of houses more than 30%, large fissures and faults are to be observed.	400 gals or more

* After H. Kawasumi⁸⁰⁾

upstream side). The anchor bolts at the support on the left-bank abutment broke off. As a result of the breakage the connection between the left end of the superstructure and the crest of the abutment was disconnected. Therefore, the abutment could move freely to the center of the river, and the other portions of the bridge (the right-bank abutment, two piers and the superstructure) moved toward the left-bank according to the movement of the right-bank abutment. Since the bridge did not sustain fatal damage, it was capable of carrying the traffic even immediately after the earthquake.

Shinminato Bridge (Photos. 2.1, 2.2; Figs. 2.4, 2.5)

This bridge, completed in 1910, was located between Kaigandori and Shinminatomachi, Yokohama, Kanagawa Prefecture, and was across a waterway in Yokohama Harbor. It was also about 40km northeast from the epicenter. The both abutments were of brick masonry with concrete spread footings rested on the ground of mudstone. One-span steel petit truss weighing 365t totally, of length of 36.6m and width of 13.4 m, was erected on the abutments. The

Table 2.2 Statistics on Highway Bridge Damage Due to the Kanto Earthquake of 1923

(A) Total Number of Bridges Damaged

Prefectures or Cities	Total Number of Bridges Surveyed	Number of Bridges Damaged due to Vibration and/or Fires	Percentages of Damage	Remarks
Tokyo	3,338	230	6.9%	Except city of Tokyo
City of Tokyo	675	358	53.0%	
Kanagawa	1,253	893	71.3%	Except city of Yokohama
City of Yokohama	108	91	84.2%	
Shizuoka	358	100	27.9%	Inside the affected area (Numazu or Northern area)
Saitama	1,313	27	2.1%	
Yamanashi	245	21	8.6%	Only wooden bridges suffered inside the affected area
Chiba	690	65	9.4%	
Total	7,980	1,785	22.4%	

(B) Damage Characteristics in the City of Tokyo

Type of Bridges	Total Number of Bridges Surveyed	Number of Bridges Damaged and Percentages		
		Caused by Vibration	Caused by Fires	Total
Wooden	420	6 (1.4%)	276 (65.7%)	282 (67.1%)
Steel	60	6 (10.0%)	49 (81.7%)	55 (91.7%)
Masonry	144	2 (1.4%)	5 (3.5%)	7 (4.9%)
Plain concrete	4	4 (100%)	0 (0%)	4 (100%)
Reinforced concrete	47	0 (0%)	10 (21.3%)	10 (21.3%)
Total	675	18 (2.7%)	340 (50.3%)	358 (53.0%)

(C) Damage Characteristics in the City of Yokohama

Type of Bridges	Total Number of Bridges Surveyed	Number of Bridges Damaged and Percentages			
		Caused by			Total
		Vibration + Fires	Vibration Only	Fires Only	
Wooden	75	26 (34.6%)	25 (33.4%)	8 (10.7%)	59 (78.7%)
Steel	31	11 (35.5%)	16 (51.6%)	3 (9.7%)	30 (96.8%)
Reinforced concrete	2	0 (0%)	2 (100%)	0 (0%)	2 (100%)
Total	108	37 (35.2%)	43 (39.8%)	11 (10.2%)	91 (84.2%)

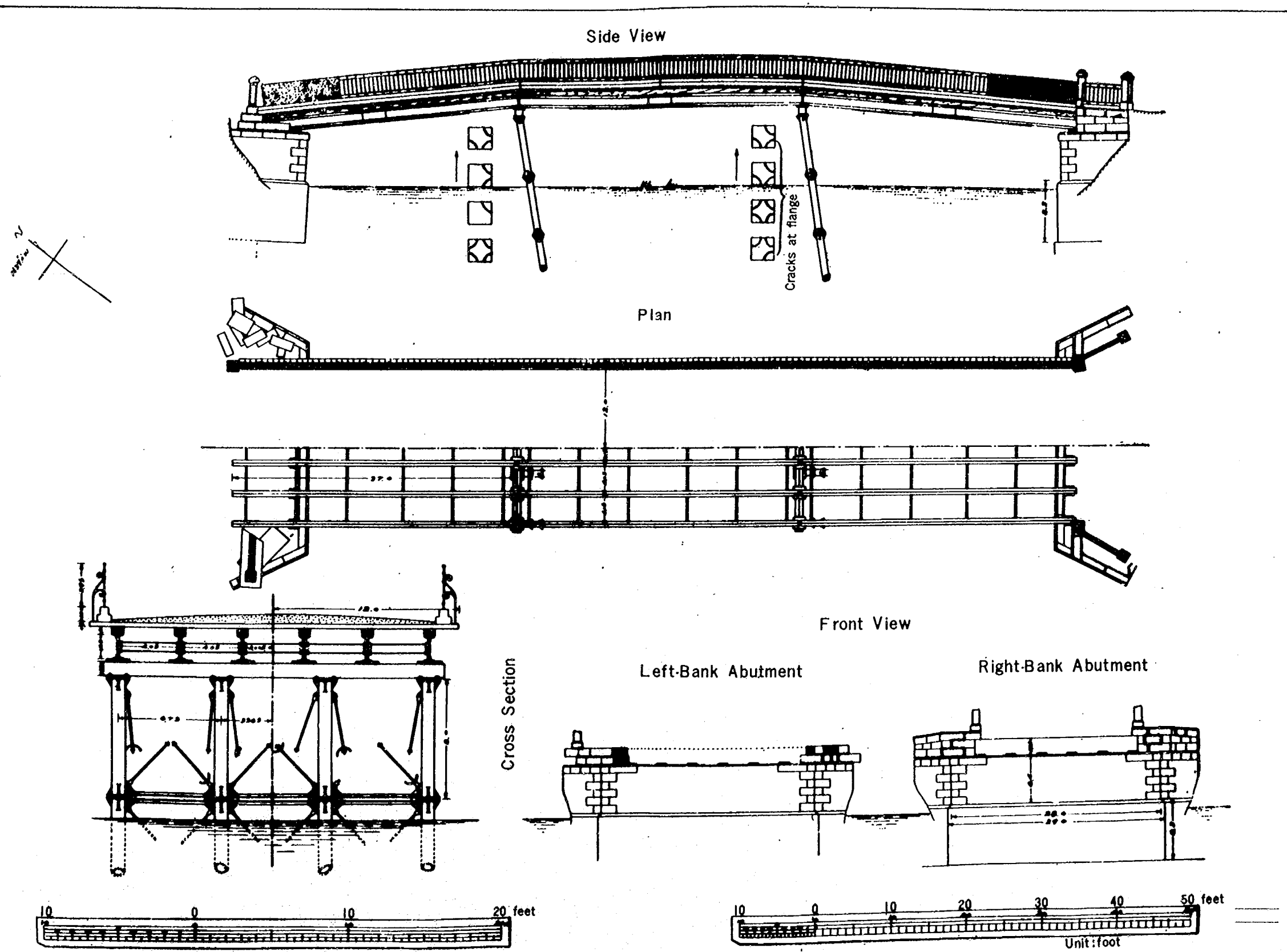


Fig. 2.3 Damage to the Tsuruno-bashi bridge

bridge carried both highway traffic and railway traffic.

Due to the earthquake it sustained heavy damage at both abutments, as shown in Figs. 24 and 25. The northern abutment moved considerably and tilted toward the center of the waterway. The movement reached 36cm at the maximum. Also were observed several heavy cracks at the masonry and dislocation occurred at the joint between the brick masonry and footing (Fig. 2.5). The southern abutment suffered cracks of width of 8cm at the maximum. Parapet walls at the both abutments, supported separately by the back fills, moved backwards seriously, probably due to the collision of the truss girder. No serious settlements were observed at the abutments themselves.

The truss did not sustain any significant damage. Due to the damage to the abutments, however, a large relative dislocation was observed between the truss and the substructures. The amount of relative dislocation reached 35cm at the supporting points. As a result of the dislocation the supporting shoes failed at both the movable and the fixed supports (see Photos. 2.1, 2.2).

Bankoku Bridge (Photos. 2.3, 2.4, 2.5; Fig. 2.6)

This bridge, completed in 1903, was very close to the Shinminato Bridge, and also across the same waterway. The abutments were of brick masonry with concrete block foundations rested on the ground of mudstone. One-span steel pony truss weighing 314t totally, of length of 36.6 m and width of 12.2 m, was constructed on the abutments (Photo. 2.3).

Due to the earthquake there were also severe damage to the abutments. The northern abutment slid horizontally toward the center of the waterway. The amount of the sliding was 1.2m at the maximum. This drastic sliding occurred at the joint between the upper block and the lower block of the foundation (Fig. 2.6). Also was observed a huge vertical breakage (width of opening was 2.7cm) at the abutment near the center line of the bridge axis. The two wing walls separated from the abutment.

The upper portion of the southern abutment also slid toward the center of the waterway. The amount of the sliding is about 30cm. The two wing walls also separated from the abutment. No significant damage was observed to the truss



Photo. 2.1 Damage to the East Support at the Southern Abutment of the Shinminato Bridge

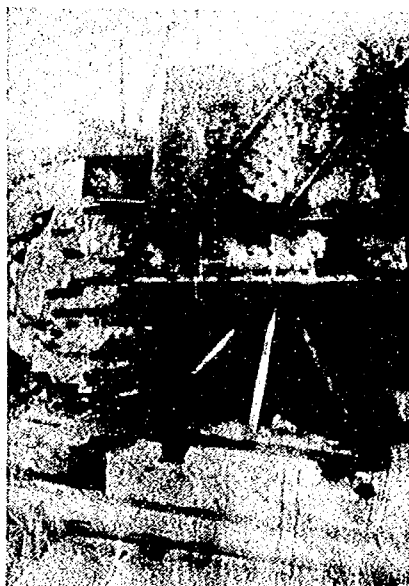


Photo. 2.2 Damage to the West Support at the Northern Abutment of the Shinminato Bridge

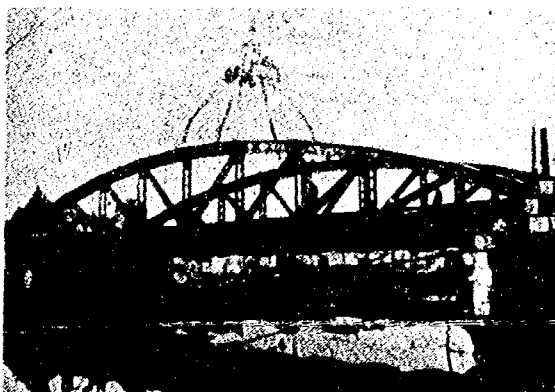


Photo. 2.3 Damage to the Bankoku Bridge (General View, Note Heavy Cracks at the Abutments)

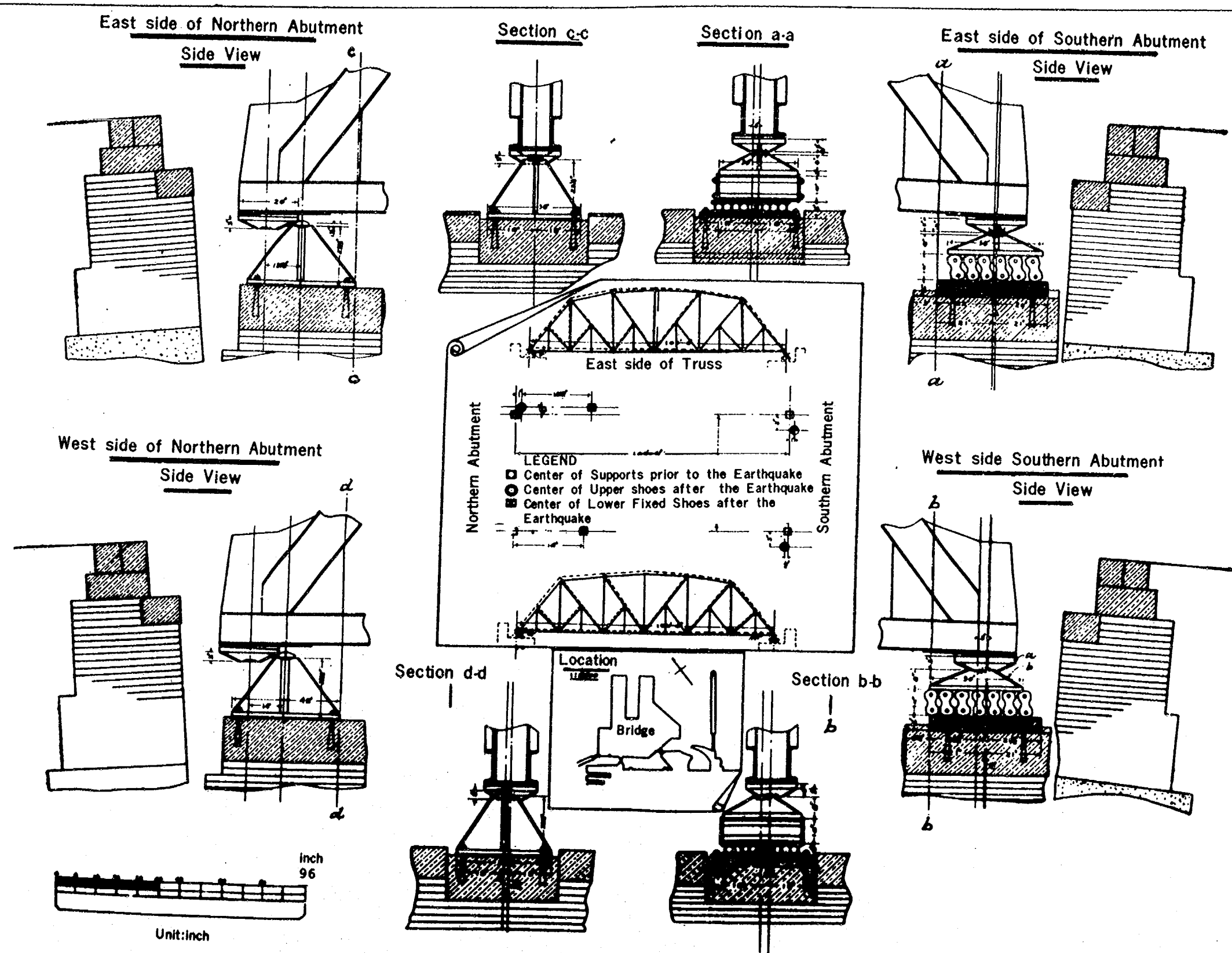


Fig. 2.4 Damage to the Shinminato bridge

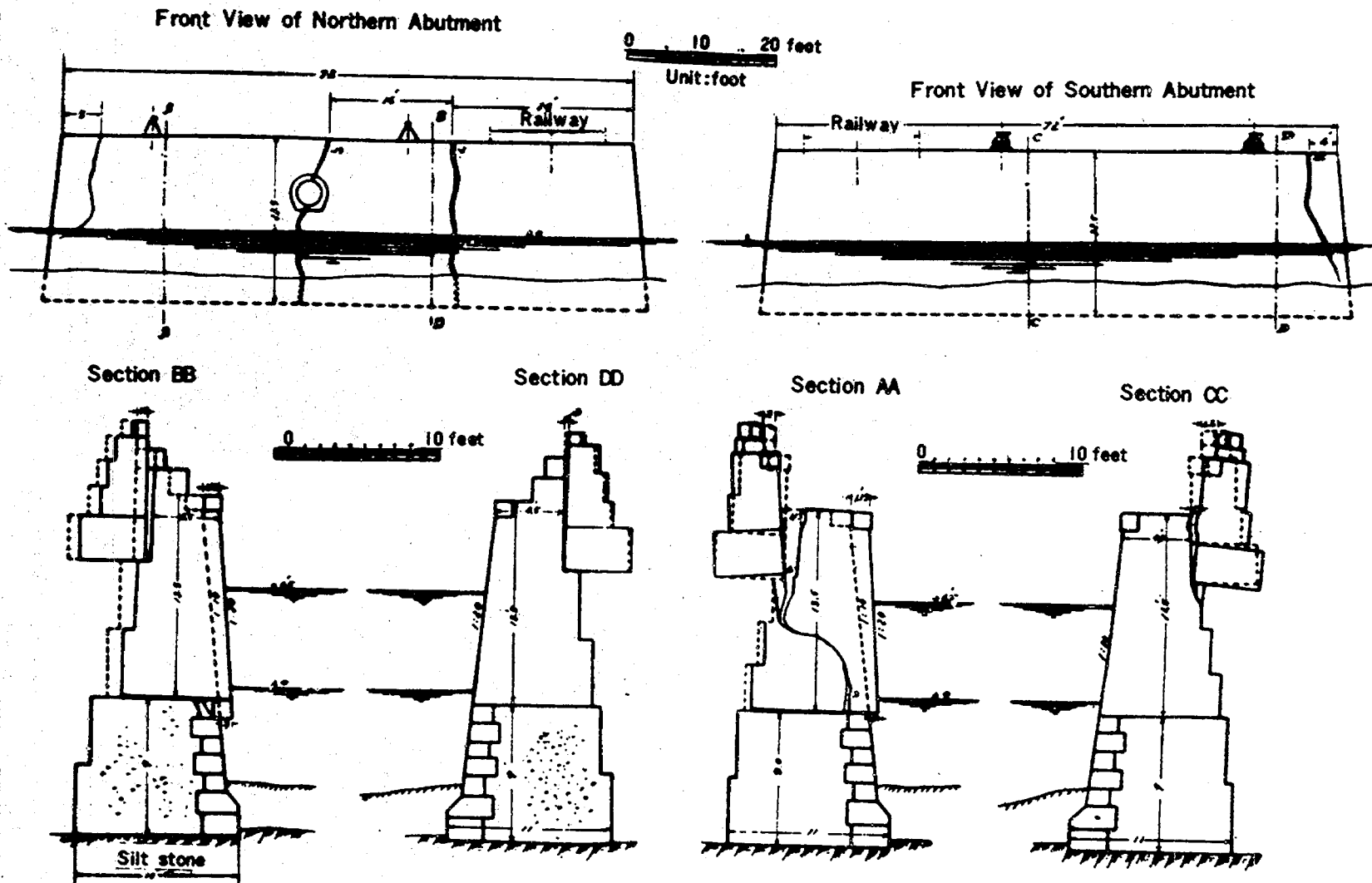


Fig. 2.5 Damage to the abutments of the Shinminato bridge

girder itself. However, several lattice beams attached to the lower chords of the truss were buckled.

The movable roller support on the northern abutment moved considerably (40cm at the east and 30cm at the west) to the south together with the abutment, and at the east side the sole plate attached to the end of the truss disconnected completely with the rollers (Photo. 2.4). The fixed support on the southern abutment also moved to the north (22 cm at the east and 34cm at the west) together with the abutment and all anchor bolts at both the east and the west sides were sheared off (Photo. 2.5).

From the damage to the bridge it is understood that block foundations like the case may sustain severe damage due to sliding between blocks.

Yamashita Bridge (Photo. 2.6)

This bridge, completed in September, 1922, was located on Kaigandori, Yokohama, Kanagawa Prefecture, and was across the Hori-kawa River. It was also about 40km northeast from the epicenter. The northern abutment (on the left-bank looking from upstream side) was made, from top to bottom, of place-in-concrete structure, 2-step concrete block, reinforced concrete placed in water, and wooden pile foundation in the soft soil layer. The southern abutment (on the right-bank) was made, from top to bottom, of place-in reinforced concrete structure, 3-step concrete blocks and sacked concrete on the mudstone ground. One-span steel Pratt truss of length of 52m and width of 14m was spanned on the abutments.

Due to the earthquake the southern abutment without pile foundation sustained severe damage. Heavy cracks and tilting were observed at the abutment (Photo. 2.6). No significant structural damage was observed to other portions of the bridge. However, most lattice beams of the truss were buckled.

It was reported that the earth pressures during the earthquake might have a significant effect on the southern abutment.

Hanazono Bridge (Fig. 2.7)

This bridge was located between Ogi-machi and Yamashita Park, Yokohama, Kanagawa Prefecture, and was across the Ooka River. It was

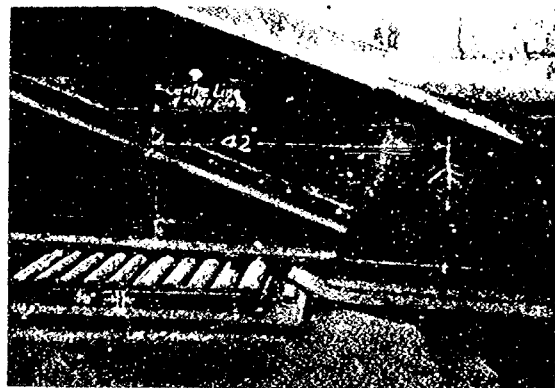


Photo. 2.4 Damage to the East Support at the Northern Abutment of the Bankoku Bridge (Note Large Dislocation)

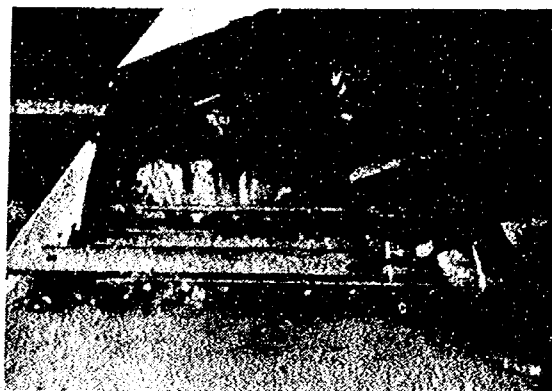


Photo. 2.5 Damage to the East Support at the Southern Abutment of the Bankoku Bridge (Note Anchor Bolts Sheared off)

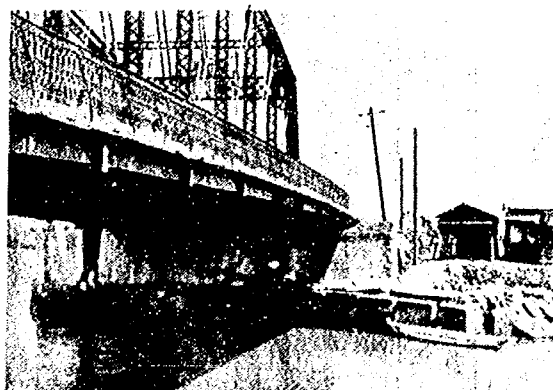
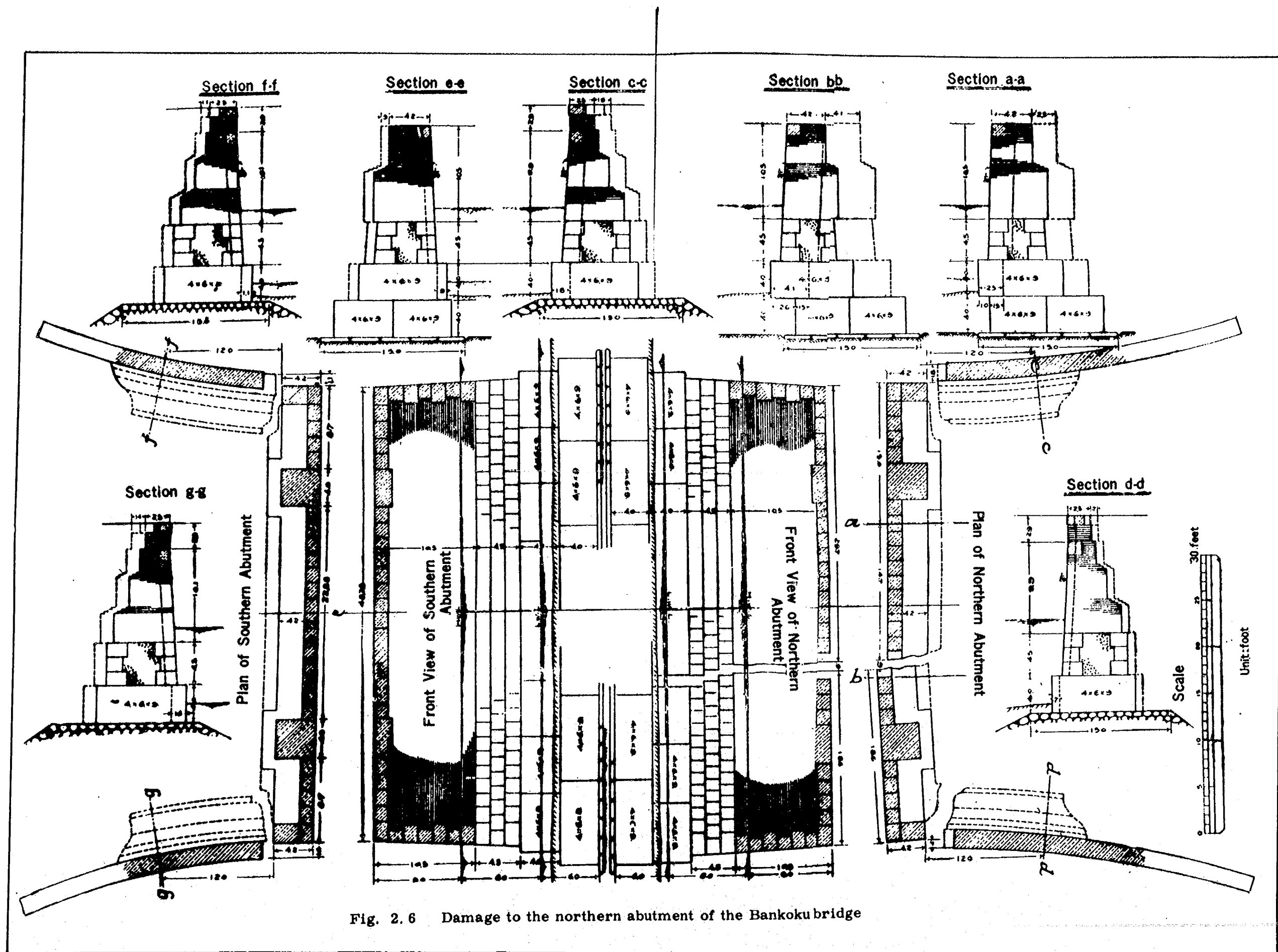


Photo. 2.6 Damage to the Yamashita Bridge (Failure at the Right-Bank Abutment)



also about 40km northeast from the epicenter. The two abutments were made of brick masonry structures in very soft soils. One-span steel pony truss having length of 50m and width of 7.6m was erected.

Due to the earthquake the abutments sustained moderate damage (Fig. 2.7). The right-bank abutment with the fixed support moved to the center of the river. A horizontal crack of width of 3 cm took place on the wall of the abutment near the top. The anchor bolts at the fixed support were sheared off. It was estimated that the crack occurred mainly due to the resistance of the superstructure against the movement of the abutment toward the center of the river.

The left-bank abutment with the roller support also moved toward the center of the river. The parapet wall collapsed due to the collision of the end of the truss girder. The bolts attached to the shoes broke off and the shoes moved by 24 cm to the left relatively to the abutment, and the sole plates attached at the end of the truss moved by 42cm to the left relatively to the shoes. Consequently the truss moved by 66cm to the left relatively to the abutment. No significant damage was observed to the superstructure. However, the camber of the truss girder disappeared completely, as a result of the large displacements of the abutments. The bridge also experienced a fire, which might have an effect on the disappearance of the camber.

It was understood that the earthpressures also might have a significant effect on the both abutments.

Toyokuni Bridge (Photos. 2.7, 2.8; Fig. 2.8)

This bridge was located between Horai-cho and Masago-cho, Yokohama, Kanagawa Prefecture, about 40km northeast from the epicenter, and was over the Ooka River. There had been light bridge superstructures of wooden-steel Pratt trusses. The new steel pony trusses were reconstructed in about 1897 without improving the substructures. The abutments and two piers were made of masonry structures with concrete fill inside, on the ground of very soft soils. No information is available about the foundations. The superstructures were of 3-span simple steel pony trusses having the total length of 47.5 m ($\approx 15.5\text{m} + 16.5\text{m} + 15.5\text{m}$) and the width of 6.7m. This was a skew bridge with an angle of some

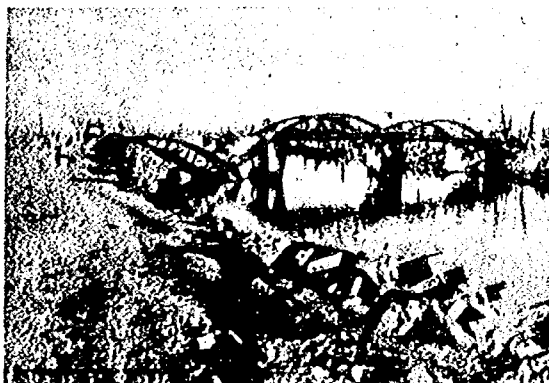


Photo. 2.7 Damage to the Toyokuni Bridge (General View of the Damage Bridge)

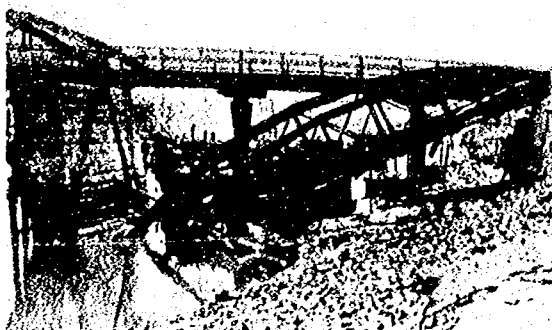


Photo. 2.8 Damage to the Pier and the Truss of the Toyokuni Bridge

20°, which is shown on the plan of Fig. 2.8.

Due to the earthquake the substructures moved considerably, and one end of a truss fell down into the river (Photos. 2.7 and 2.8). The both abutments moved toward the center of the river, and tilted in the direction of their back fill. No serious cracks were observed. The two piers tilted considerably toward the center of the river, with angles of inclination of $8^{\circ}41'$ at the northern pier and of 2° at the southern pier. The southern pier suffered a serious breakage between the base of the shaft and the top of the foundation. It was believed that the pier did not overturn only by the resistance of the superstructures. The superstructures moved drastically due to the movements of the substructures. On the northern pier having a fixed support made of a set of steel channels, two truss girders moved to the south together with the support. Although the support was almost dislodged from the top of

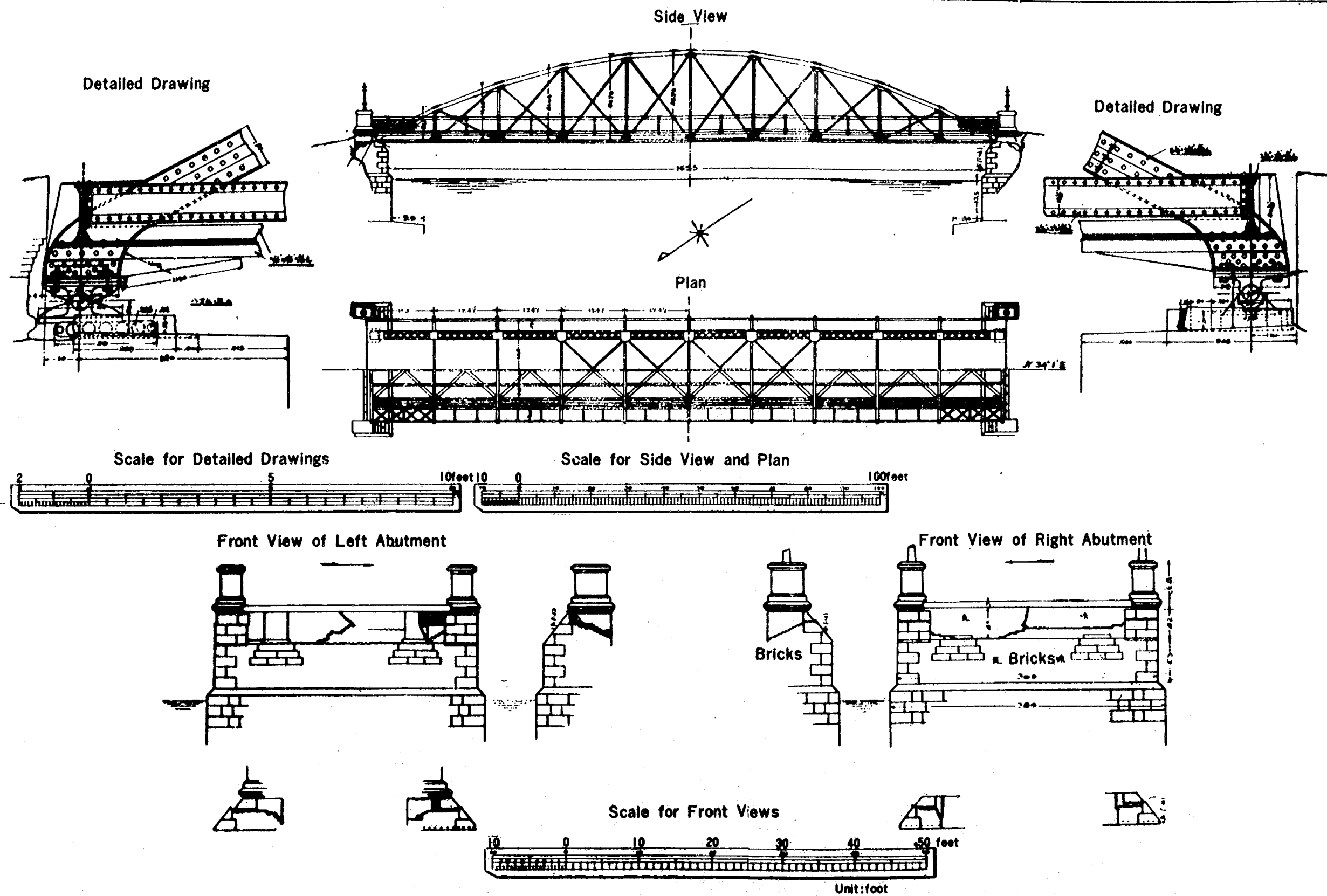


Fig. 2.7 Damage to the Hanazono bridge

the pier as seen on the detailed drawing in Fig. 2.8, the superstructure did not fall down from the pier. On the southern pier having a roller support, two truss-girders also moved to the south, and the end of the southern most girder fell down into the river eventually. No significant structural damage was observed to the two truss girders which did not fall.

It was understood that since the substructures had been designed and constructed for the older light superstructures, they were not sufficient to support the new superstructures safely during the earthquake. It was also noticeable that the end of a girder supported with a roller bearing fell down.

Sakawa Bridge (Photos. 2.9 – 2.11; Fig. 2.9)

This bridge was erected on the National Highway No. 1, located in Odawara, Kanagawa Prefecture, and was across the Sakawa River. The bridge, completed in July, 1923 only two months prior to the occurrence of the earthquake, was near the mouth of the river and was one of the highway bridges located very close to the epicenter (about 13km northwest from the epicenter). The abutments were made of gravity-type reinforced concrete structures in soft soils. The thirty two piers were of reinforced concrete rigid frame structures with concrete caisson foundations covered by wooden frames, also in soft soils. The superstructures were consisted of 33-span simple reinforced concrete T-type girders. The total length was 363m ($= 33 \times 11.0\text{m}$) and the width was 6.7m (Fig. 2.9).

Due to the earthquake the bridge sustained the severest damage among the highway bridges. The right abutment suffered a huge crack between the main structure and one of the wing walls. The left abutment tilted slightly, and the parapet wall broke off (Photo. 2.11). All the piers collapsed completely and also the 33-span girders collapsed and fell down into the river (Photo. 2.9 and 2.10).

It is reported that the fall of the superstructures initiated from the extremely left span and transmitted to the right direction one by one. From the fact that a heavily damaged pier remained between two sections of a fallen girder it is supposed that the girder moved very extensively before falling. The maximum distance between the ends of the fallen girders and the nearest pier was about 1.7m.

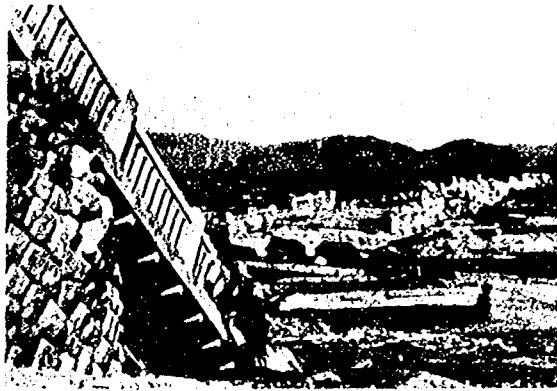


Photo. 2.9 Damage to the Sakawa Bridge (1) (General View of the Damaged Bridge)



Photo. 2.10 Damage to the Sakawa Bridge (2) (Damaged Pier and Fallen Girder)

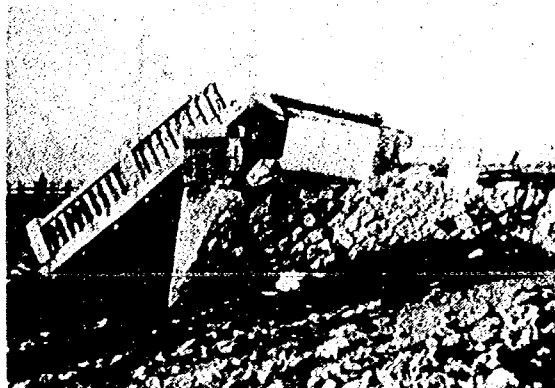


Photo. 2.11 Damage to the Sakawa Bridge (3) (Damage to the Left Abutment)

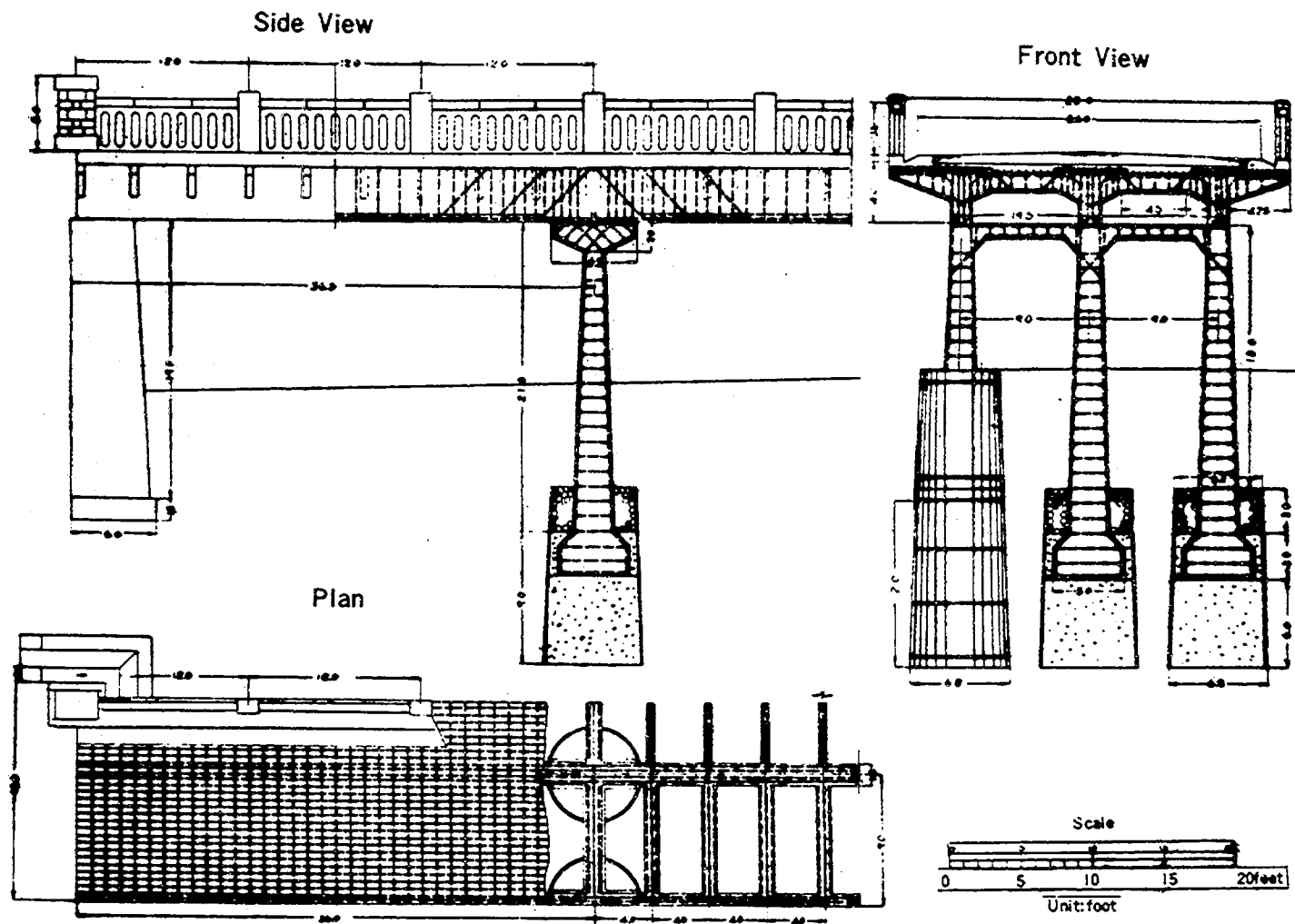


Fig. 2.9 Drawings of the Sakawa bridge

It was understood that the severest damage to the bridge was caused by the following three facts: (1) the location was very close to the epicenter (about 13km), (2) the soils at the site were very soft, and (3) the substructures were not sufficient to resist seismic forces from the rather heavy reinforced concrete girders.

Banyu Bridge (Photos. 2.12, 2.13; Fig. 2.10)

This bridge was also on the National Highway No. 1, located about 15m northeast from the epicenter and between Chigasaki and Hiratsuka, Kanagawa Prefecture, and across the Banyu River (presently the Sagami River). The bridge was under construction of the substructures at the time of the earthquake. The both abutments, already completed at that time, were of gravity-type reinforced concrete structures with pile foundations. The piers were of reinforced concrete rigid frame with concrete caisson foundations. Among fifty six piers, only six piers were completed near the left bank, and as for the remaining forty two piers the caisson foundations were completed or under construction. The superstructures, 57-span reinforced concrete T-shape simple girders of the total length of 620 m ($= 57 \times 10.9$ m) and the width of 7.3 m, were not erected at the time.

Due to the earthquake the substructure sustained extensive damage (Fig. 2.10). The right abutment tilted about 12° toward the center of the river. And the left abutment also tilted about 4° toward the center of the river. Heavy failures took place at the horizontal beams of the piers (Photo. 2.12). The effects of insufficient hardening of concrete should be considered for the damage. Drastic movements and floating of several caisson foundations were observed (Photo. 2.13). In view of the behavior of the foundation it is suggested that liquefaction of soils took place at the site.

Hayakawa Bridge (Photos. 2.14, 2.15; Fig. 2.11)

This bridge, completed in 1917, was on Odawara-Yugawara Route of Kanagawa prefectural roads, located in Odawara, Kanagawa Prefecture about 15 km west from the epicenter, and was across the Hayakawa River. Abutments and piers were of plain gravel concrete with corner quoin stones, and supported by caisson foundations in gravel layers. The superstructures,



Photo. 2.12 Damage to the Piers of the Banyu Bridge (under Construction)

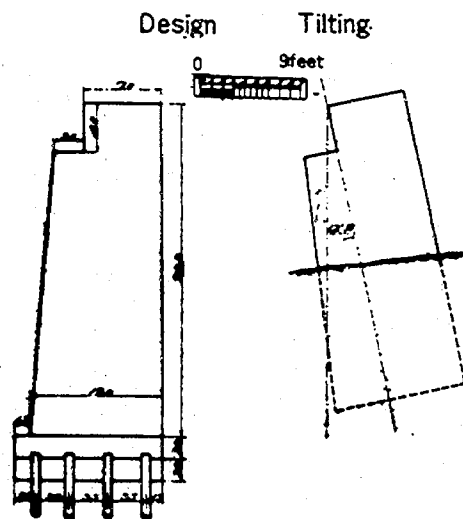


Photo. 2.13 Movements of the Caisson Foundations of the Banyu Bridge (under Construction)

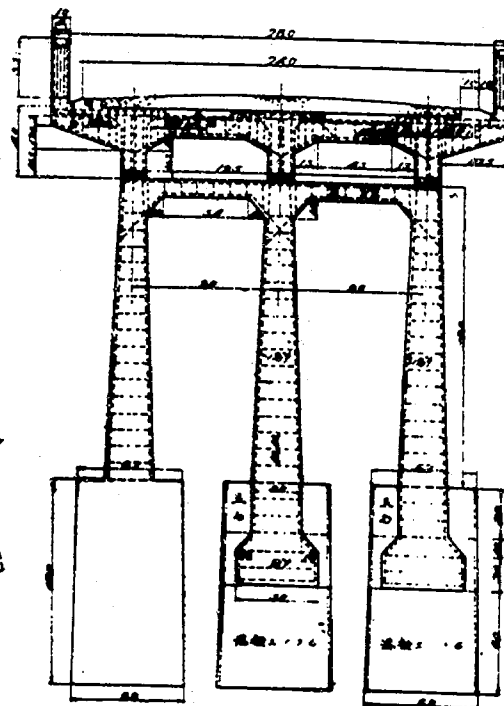
the length of 82m ($\div 6 \times 13.6$ m) and the width of 5m, were of 6-span reinforced concrete T-shape simple girders. The three southern spans had already deformed permanently even before the earthquake, because of the small cracks at the girders which were caused by the heavy traffic immediately after the completion. Some repairing work had been made by pouring cement milk into the openings.

Due to the earthquake serious damage including the fall of a girder was observed to the repaired girdes as well as the other portions (Photos. 2.14 and 2.15; Fig. 2.11). The northern abutment had cracks at the masonry, and some cornerstones fell down. No significant damage was observed to the southern abutment. Four piers except the central one suffered several cracks on the downstream side. The southernmost pier was pushed to the north by the fallen

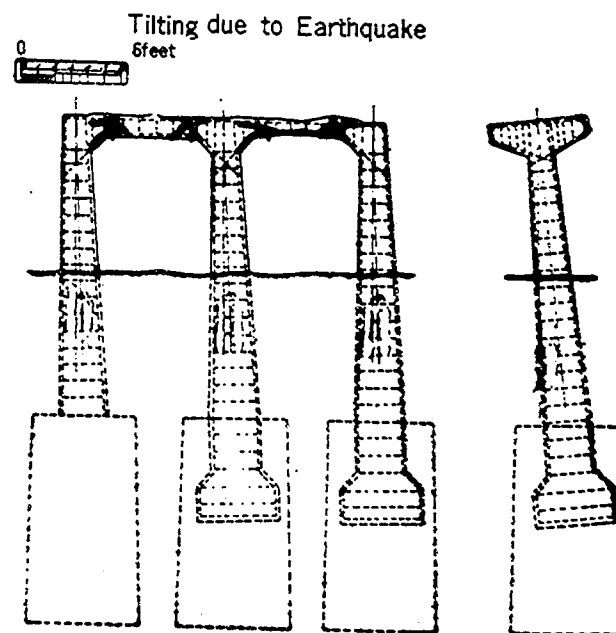
(2) Damage to Abutment
of Hiratsuka-side



Design



(1) Damage to Pier of
Chigasaki-side



Unit: foot

Fig. 2.10 Damage to the Banyu bridge

girder, inclined and suffered heavy cracks at the masonry. All the girders moved to the downstream side (or east) and to the right-bank side (or south). The maximum movement was about 50cm. The three southern girders which had had trouble before as mentioned above, suffered fatal cracks at their webs, and drooped. The southernmost span eventually fell down into the river because of the failure of the main girder. The bridge, which had a clear span of 13.6m, was one of the longest concrete bridges in those days. It was reported that the bridge was exerted into strong vertical motion by the earthquake.

It was peculiar that the damage to the bridge was primarily caused by the weakness of the superstructure, which had been recognized already before the earthquake.

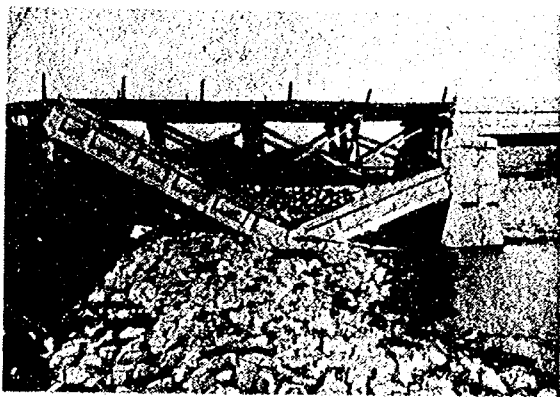


Photo. 2.14 Damage to the Hayakawa Bridge (1) (Fallen RC Girder)

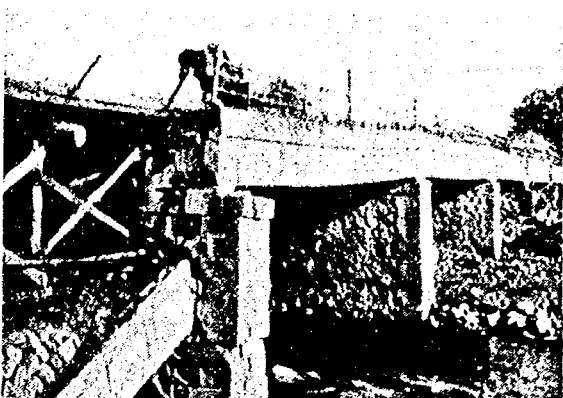


Photo. 2.15 Damage to the Hayakawa Bridge (2) (General View)

Takahata Bridge (Photos. 2.16 ~ 2.18; Fig. 2.12)

This bridge was on a Tokyo prefectural road, located in Hino, Tokyo about 53 km north from the epicenter, and was across the Asakawa River. The maximum acceleration near the site was estimated about 0.2g, much less than any sites described previously. At the site a gravel layer of depth of 3m was on a hard clayey layer. Near



Photo. 2.16 Damage to the Takahata Bridge (1) (Failure of a Pier)



Photo. 2.17 Damage to the Takahata Bridge (2) (Failure of the Fifth Pier)

the right bank, however, were soft soils. The both abutments were of gravity-type concrete structures. The right-bank abutment was supported by the pile foundation. The piers were of reinforced concrete rigid frame (3 columns and a beam) structures. The superstructures, the total length of 115m ($= 7.5\text{m} + 11 \times 9.1\text{m} + 7.5\text{m}$) and the width of 5.5m, were of 13-span I-shape steel plate simple girders.

Due to the earthquake severe damage was observed to the rightbank abutment and two piers (Photos. 2.16 to 2.18 and Fig. 2.12). The right abutment moved to the center of the river by the action of the earth pressures, and sustained serious damage due to the collision with the superstructure (Photo. 2.18). No significant damage was observed to the left abutment. Most of piers did not have severe damage except

two piers. Each of the fourth and fifth piers from the right bank sustained heavy damage near the upper joint at the central column of three columns, probably due to large bending moments exerted by the superstructures. Photo. 2.17 shows the failure at the fifth pier. No significant damage to the superstructures was observed.

2.2 Nankai Earthquake of 1946²⁴⁾

A great earthquake occurred about 60 km south off Shionomisaki (coast of the Kii Peninsula), Wakayama Prefecture of Honshu Island on December 22, 1946. The magnitude of the earthquake counted 8.1 on the Richter scale. It is one of the greatest earthquakes experienced in or near Japan. Since the earthquake broke off about a year after the end of the World War II, it seems

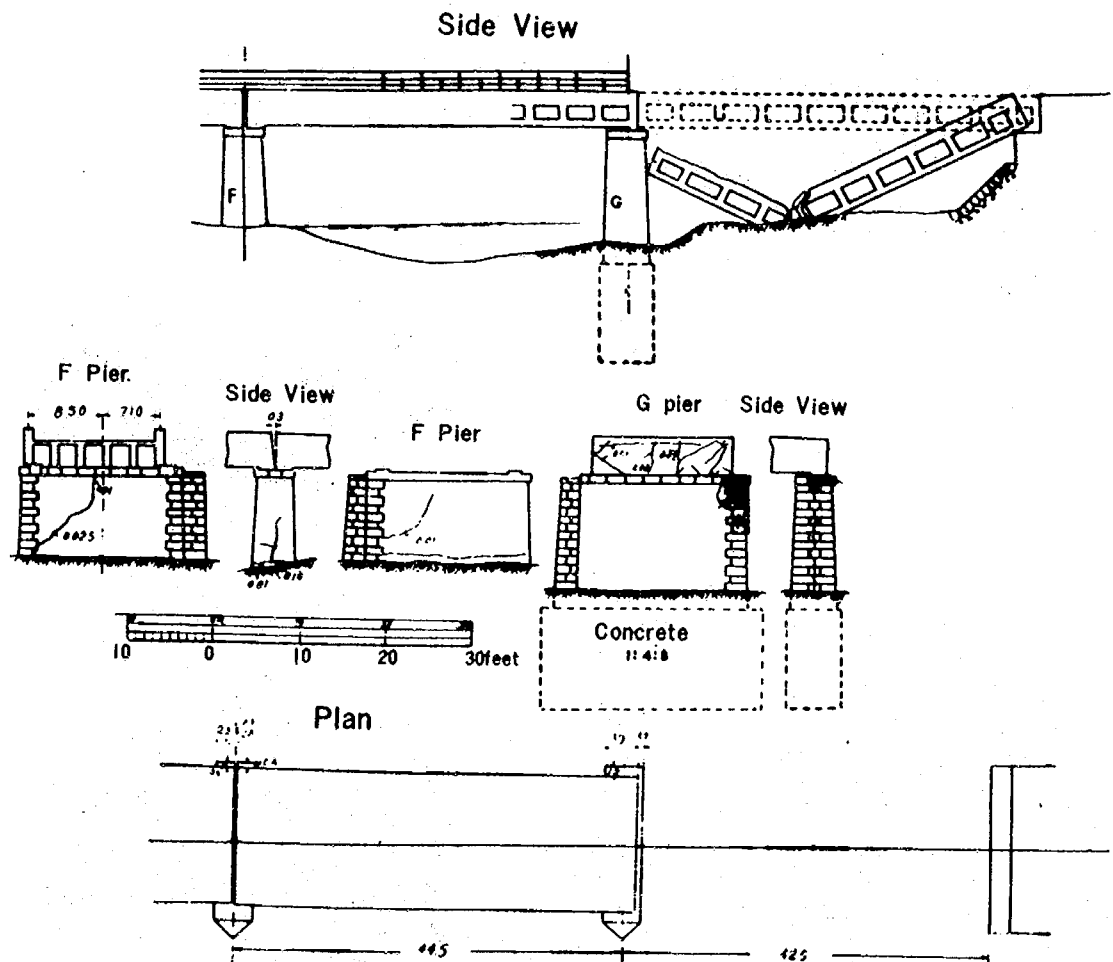


Fig. 2.11 Damage to the Hayakawa bridge

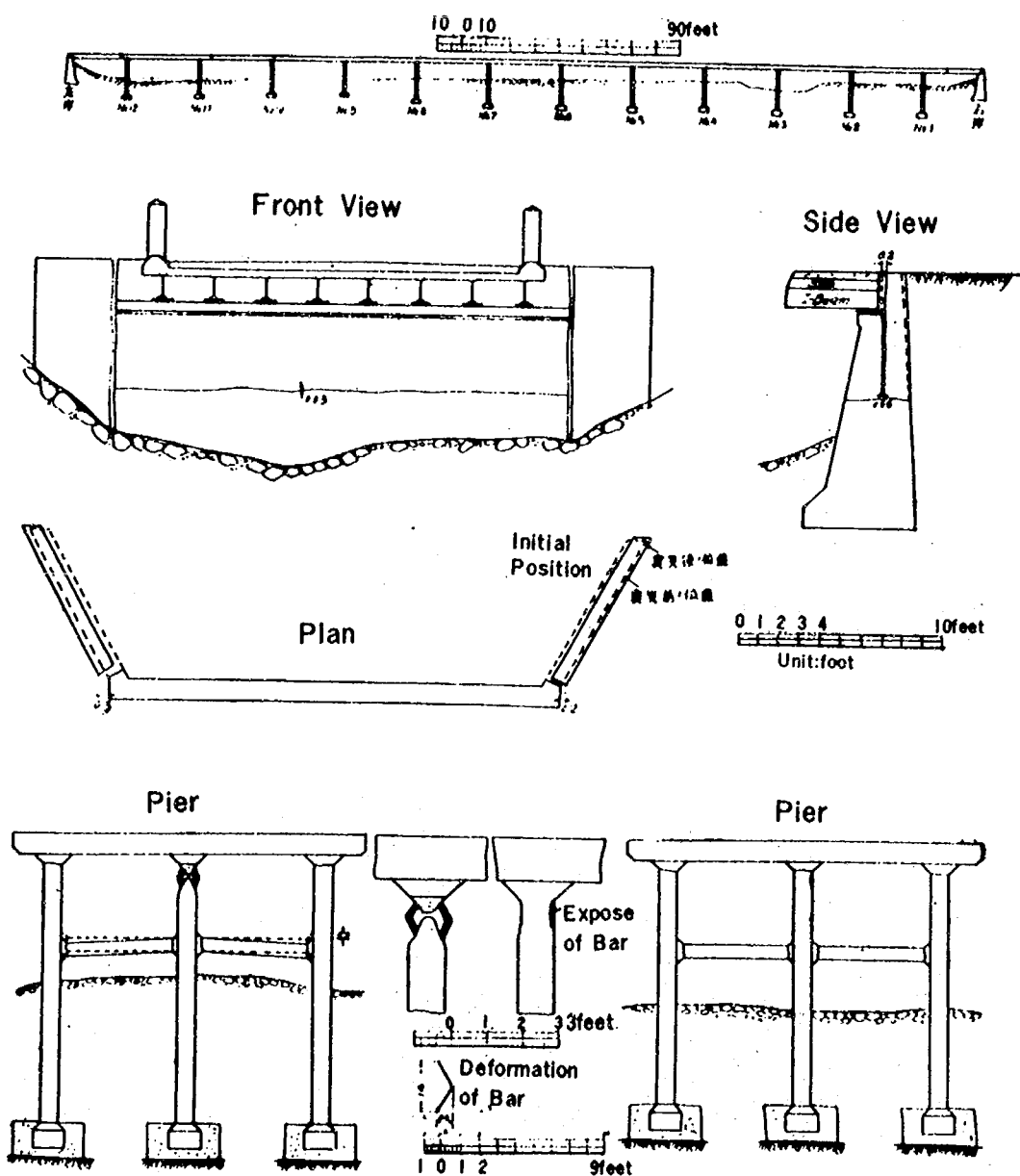


Fig. 2. 12 Damage to the Takahata bridge



Photo. 2.18 Damage to the Takahata Bridge (3) (Damage to the Right Abutment)

that there were great difficulties to grasp the damage exactly and to repair the damaged structures promptly. Nevertheless, engineers and seismologists made a great effort in surveying the damage characteristics and in reconstructing the damaged structures. Several reports are available

on the earthquake and the damage²⁴⁾, 210). The following description is a summary of the damage to bridge structures by referring to these reports. The outline of the Nankai Earthquake is as follows⁵¹⁾:

Name of Earthquake:	Nankai Earthquake
Date of Occurrence:	December 21, 1946.
Epicenter:	Off Shionomisaki, Wakayama (33.0°N, 135.6°E)
Depth of Hypocenter:	30 km
Magnitude on the Richter Scale:	8.1

The location of the epicenter is shown in Fig. 2.13 as well as in Fig. 2.1. JMA seismic intensities at various places are also indicated in Fig. 2.13. The number of highway bridges damaged by the earthquake was over three hundreds, as tabulated on Table 2.3. The damaged bridges were observed throughout the southern Japan. The heavily damaged ones were in Kochi Prefecture of Shikoku Island and Wakayama Prefecture of Honshu Island.

Next, the individual damage characteristics are described for two bridges damaged comparatively severely.

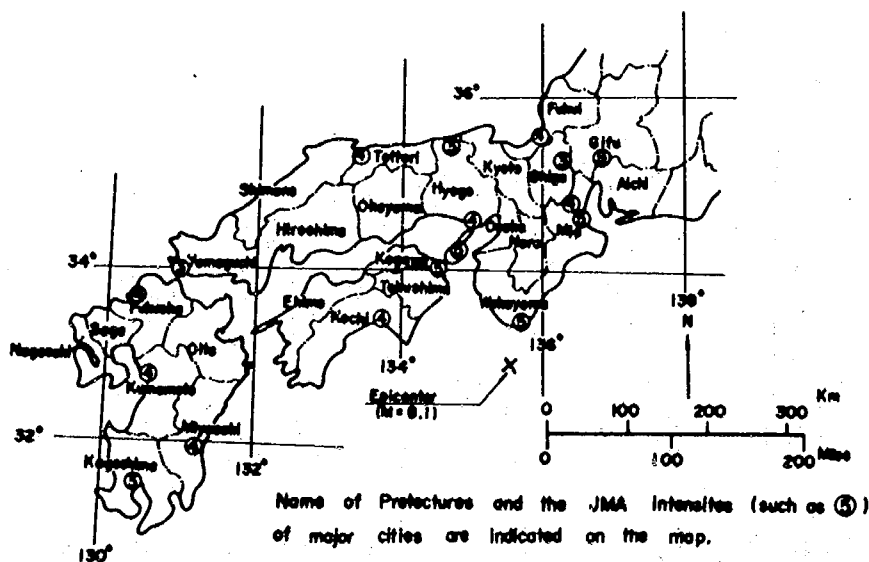


Fig. 2.13 The Nankai Earthquake of 1946

Table 2.3 Statistics on Damage to Highway Bridges due to the Nankai Earthquake of 1946

Prefectures	Number of Bridges Damaged	Amount of Loss in Bridges	Amount of Loss in Highways Except Bridges
Honshu Island	(179)	Thousand Yen (25,365)	Thousand Yen (80,626)
Aichi	26	806	432
Mie	73	1,578	4,230
Gifu	13	2,778	43
Nara	21	71	6,717
Wakayama	29	19,769	64,887
Okayama	14	346	4,174
Hiroshima	2	17	143
Shimane	1	small	small
Shikoku Island	(163)	(70,240)	(275,511)
Kagawa	7	478	1,033
Tokushima	19	3,687	8,243
Ehime	16	3,273	7,399
Kochi	121	62,802	258,826
Kyushu Island	(4)	(small)	(small)
Oita	1	small	small
Miyazaki	3	small	small
Total	346	95,605	356,137

(Note) Amount of loss was evaluated at the value at the time of the earthquake.

Kumano Bridge (Photo. 2.19)

The bridge, completed in March, 1935, was on the National Highway No. 41, located in Shingu, Wakayama Prefecture about 90 km north from the epicenter, and across the Kumano River. The JMA intensity at the site was 6, and the maximum acceleration was estimated to be 0.2g or more. The ground was of gravel layer with cobble stones. The abutments were of U-shape gravity type concrete structures, the dimensions of which are 15m in the transverse direction, 5m and 2.3m in the longitudinal direction at the base and at the crest, respectively, and 10m in height. The piers were of elliptical reinforced concrete columns with caisson foundations. The dimensions of the caisson were 11m by 4m in section and 13m (for main spans) or 8m (for side spans) in height. The superstructures, of the total length of 418.5 m,

consisting of three main spans (span length of 3 x 54 m) plus six side spans (6 x 41.4 m) and the width of 6m, were of simple steel Warren trusses.

Due to the earthquake the bridge sustained moderate damage to bearing supports and the appurtenant structures. Photo. 2.19 shows the damage on the 4th pier from the right-bank side, indicating the movement of the movable shoe (right on the photograph) and the failure at the fixed shoe (left). No significant structural damages were observed to the substructures (abutments and piers). The wing wall on the right-bank (Wakayama side) expanded about 20 cm and the approach banks settled about 30 cm at the maximum. Five trusses (the 4th span to 9th span from the right-bank) moved about 4 cm or more to the right-bank. As a result of the movements, the mortar and the concrete near the fixed bearing supports on the substructures were



Photo. 2.19 Damage to the Kumano Bridge (Movement of the Movable Shoe and Failure of the Fixed Shoe)

crushed. No lateral movements perpendicular to the bridge axis were observed.

Shimanto Bridge (Photo. 2.20)

The bridge was connecting Gudo and Nakamura, Hatata County, Kochi Prefecture about 250 km west from the epicenter, and across the Shimanto River. The abutments were of gravity-type reinforced concrete structures, and the piers were of reinforced concrete columns with caisson foundations. The superstructures, the total length of 438m and the width of 5.5m, consisted of eight main spans (steel Warren trusses of span length of $8 \times 46.05\text{m}$) and six side spans

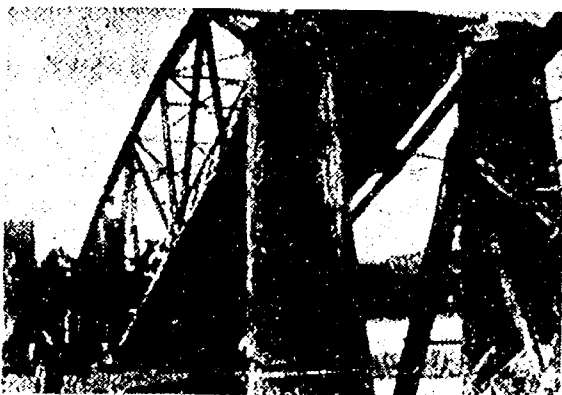


Photo. 2.20 Damage to the Shimanto Bridge (Fall of Six Trusses)

(reinforced concrete girders of span length of $6 \times 11.6\text{m}$).

Due to the earthquake the bridge sustained severe damage. The right-bank abutment had a serious crack at the parapet wall, and the wing wall failed at the downstream side. The piers supporting steel trusses sustained heavy damage due to the influences of the fall of the trusses, rather than the direct effects of the earthquake. Yet, the piers supporting the reinforced concrete girders had cracks due to the direct effects. The six steel trusses (2nd to 7th spans from the left) fell down into the river (Photo. 2.20).

Although no detailed descriptions are available on the causes of the fall, it is reported that the 4th span from the left fell down first, next the 3rd spans and finally the 2nd, fifth and sixth spans fell down almost simultaneously.

2.3 Fukui Earthquake of 1948²⁵⁾

A severe earthquake broke off inside the Fukui Plain, Fukui Prefecture, situated in the middle western part of Honshu Island on June 28, 1948. The magnitude was 7.3 on the Richter scale. Since the epicenter (see Fig. 2.14) was very close to several cities and towns in Fukui and Ishikawa Prefectures, a tremendous amount of damage was observed in the both prefectures. The Fukui Earthquake is one of the severest earthquakes which caused serious damage to engineering structures in Japan. The outline of the earthquake is as follows⁵¹⁾:

Name of	
Earthquake:	Fukui Earthquake
Date of	
Occurrence:	June 28, 1948
Epicenter:	Fukui Plain (36.1°N , 136.2°E)
Depth of	
Hypocenter:	20 km
Magnitude on the	
Richter Scale:	7.3

An extensive survey was made on the damage to engineering structures. The statistics on damage to highways and bridges due to the earthquake are summarized on Table 2.4. Comparing Table 2.4 with Table 2.3, it is apparent that the Fukui Earthquake caused very severe damage to the bridges in Fukui Prefecture.

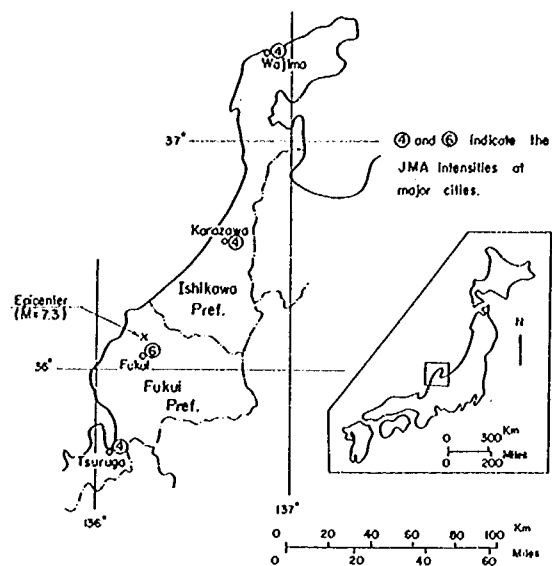


Fig. 2.14 The Fukui Earthquake of 1948

Table 2.4 Statistics on Damage to Highway Bridges due to the Fukui Earthquake of 1948

Prefectures	Bridge Damage		Highway Damage Except Bridges	
	Number of Bridges	Repairing Cost	Number of Sites	Repairing Cost
Fukui	180	Thousand Yen 189,869	475	Thousand Yen 205,945
Ishikawa	63	17,782	155	41,463
Total	243	207,651	630	247,408

(Note) Amount of loss was evaluated at the value at the time of the earthquake.

Nakazuno Bridge (Photos. 2.21 -- 2.26; Figs. 2.15, 2.16)

This bridge, completed in 1932, was on the Fukui-Kaga-Yoshizaki Route, one of the Fukui prefectural roads, located between Kawai and Nakafujishima, Yoshida County, Fukui Prefecture, and across the Kuzuryu River. The bridge was located about 8 km south from the epicenter, and the maximum acceleration at the site was estimated about 0.6 g from overturning of tomb stones nearby.

The abutments were of gravity-type reinforced concrete structures with pile foundations, and the thirteen piers were of reinforced concrete columns with pile foundations or caisson foundations. Fig. 2.16 shows the general view of a typical pier. The superstructures, the total length of 257 m and width of 5.5 m, were of 14-span steel plate simple girders (span length being 14 x 18.4 m).



Photo. 2.21 Damage to the Nakazuno Bridge (1) (Fall of the Girders)

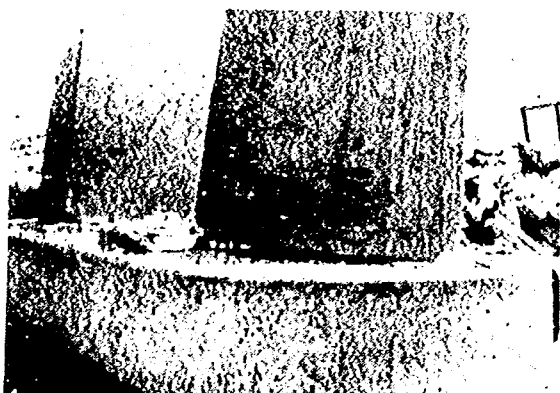


Photo. 2.22 Damage to the Nakazuno Bridge (2) (Crack at the Connection between Caisson Foundation and Column)



Photo. 2.23 Damage to the Nakazuno Bridge (3) (Overturning of a Pier)



Photo. 2.24 Damage to the Nakazuno Bridge (4) (Fall of the Girders)

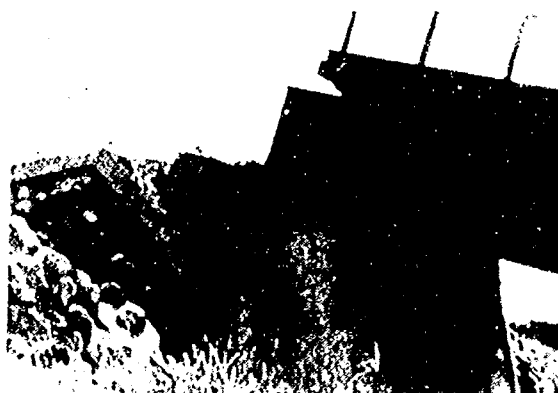


Photo. 2.25 Damage to the Nakazuno Bridge (5) (Failure near the Left Abutment)



Fig. 2. 15 Damage to the Nakazuno bridge

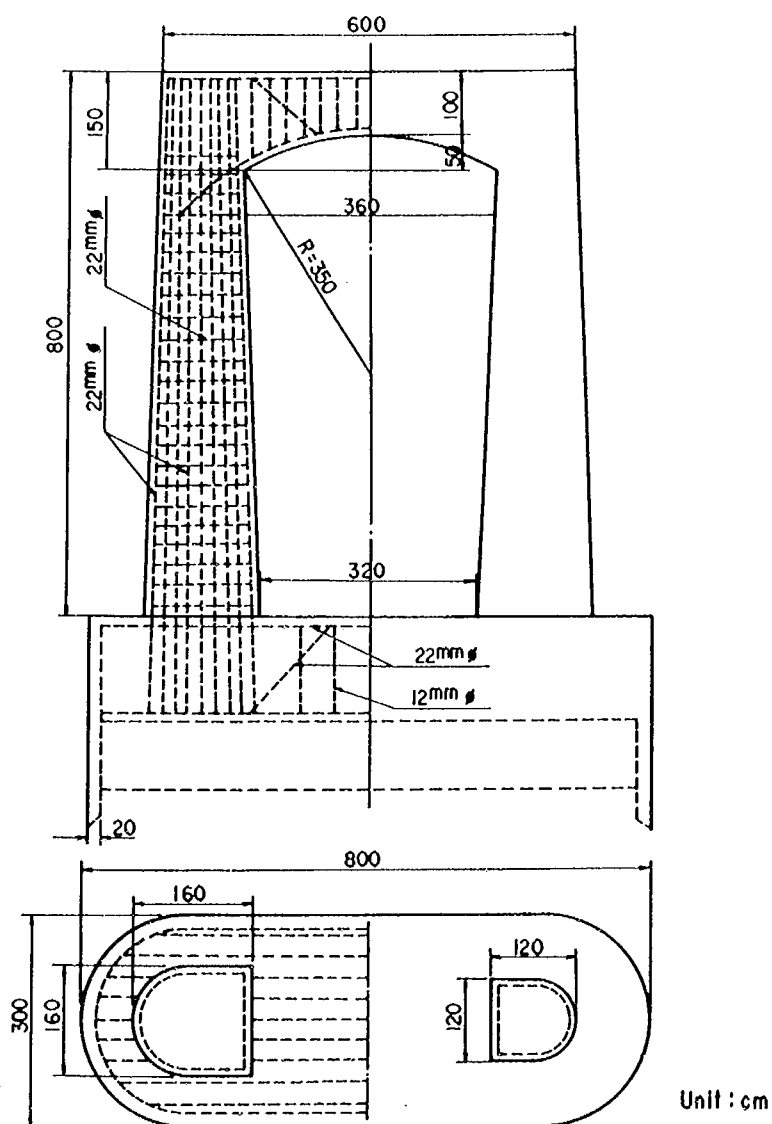


Fig. 2. 16 Detailed drawings of the pier of the Nakazuno bridge

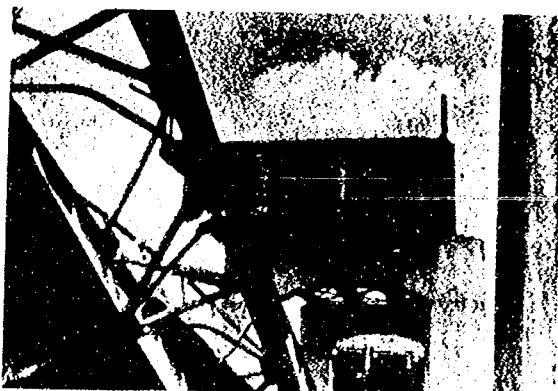


Photo. 2.26 Damage to the Nakazuno Bridge (6) (Fall of the Girders)

Due to the earthquake severe damage was observed both to the substructures and the superstructures. The left-bank abutment suffered cracks on parapet walls (Photo. 2.25), and the right-bank abutment inclined toward the center of the river. The 1st and 2nd piers from the left-bank did not suffer any significant damage. The 3rd to 7th, 9th and 10th piers tilted toward the left-bank, suffered heavy cracks at the connections between the columns and the caisson foundations (Photo. 2.22), and exposed reinforcements at the connections. Also heavy cracks were observed at the connections between the columns and the beams at the caps of the piers. The 8th and 10th piers overturned completely toward the left-bank due to the breakage at the connections between the columns and the caisson foundations (Photo. 2.23). The 12th and 13th piers tilted toward the right-bank, and suffered cracks at the connections between the columns and the beams.

Ten of the fourteen spans fell down into the river (Photo. 2.21, 2.24 and 2.26 and Fig. 2.15). Since they fell down on the soft sand layer, main girders and lateral members did not suffer any significant structural failure, and these fallen girders were still usable after easy repairing. It is understood that the fall of the superstructures was mainly caused by the failure of the substructures.

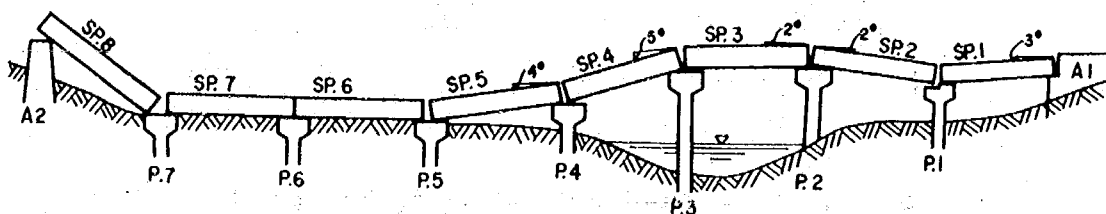


Fig. 2.17 Damage to the Nagaya bridge (1) (Settlements of all the Piers)

Nagaya Bridge (Photos. 2.27, 2.28; Figs. 2.17, 2.18)

The bridge was on the Katsuyama-Mikuniminato Route, one of the Fukui prefectural roads, located in Higashijugo, Fukui Prefecture, and across the Jugo Irrigation Canal. The bridge was very close to the epicenter (about 2 km north from the epicenter), and the maximum acceleration was estimated 0.5 to 0.6g near the site.

The abutments were of concrete structures, and the seven piers were of reinforced concrete rigid frame (two columns and a beam) structures.



Photo. 2.27 Damage to the Nagaya Bridge (1) (Settlements of the Piers)



Photo. 2.28 Damage to the Nagaya Bridge (2) (Failure of a Pier)

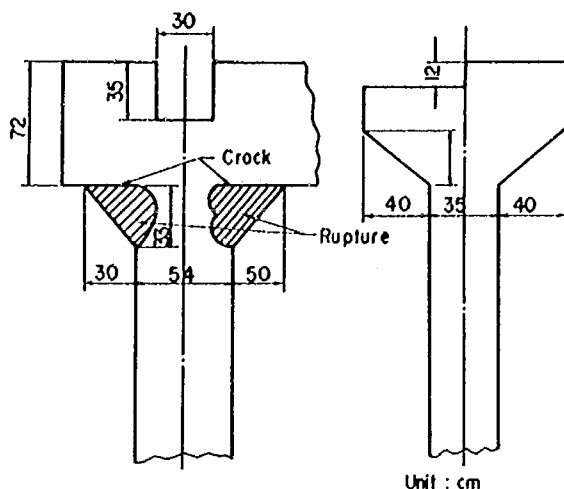


Fig. 2. 18 Damage to the Nagaya bridge
(Failure at a Pier)

The superstructures, the total length of 58.5m and the width of 4m, were 8-span I-shape steel girders (span length being $4 \times 6\text{m} + 3 \times 9.5\text{m} + 6\text{m}$). The bearing supports were of slide-type with two steel plates. Although the left-bank abutment did not suffer any significant damage, the right-bank abutment tilted toward the center of the canal. The 1st to 3rd piers from the left-bank settled considerably, and the 4th to 7th piers also settled and tilted considerably. Severe cracks were observed at the column-beam connections of the 4th to 7th piers (Photo. 2.28 and Fig. 2.18). Every span moved downwards drastically due to the settlements of the piers (Photo. 2.27 and Fig. 2.17)

the crest of the abutment. The anchor bolts moved about 40 cm to the left-bank and about 5 cm to the upstream. The right-bank abutment also tilted slightly. Every pier tilted toward the left-bank and settled. The maximum settlement of 25 cm was observed at the 2nd pier from the right-bank (Photo. 2.29). No significant structural damage was observed at the superstructure.

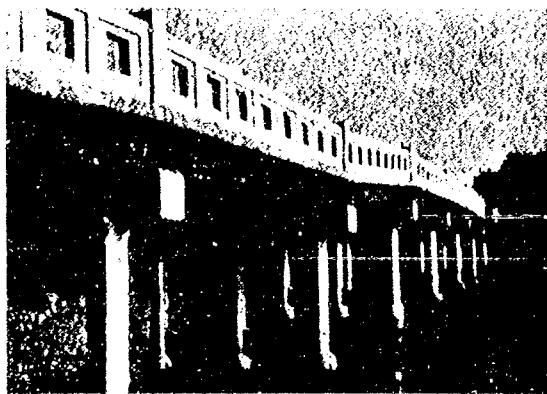


Photo. 2.29 Damage to the Shioya Bridge (Settlement of a Pier)

Shioya Bridge (Photo. 2.29)

This bridge was located near the mouth of the Daishoji River, Shioya, Ishikawa Prefecture, about 15 km north from the epicenter. The maximum acceleration was estimated about 0.5g at the site.

The abutments were of concrete structures, and the seven piers were of reinforced concrete rigid frame (3 columns and a beam) structures. The superstructures, the total length of 86 m and the width of 4.5 m, were of 8-span I-shape steel girders (span length being $8 \times 10.75\text{m}$).

Due to the earthquake moderate damage was observed to the substructures. The left-bank abutment tilted slightly and had a crack along the construction joint which was 2m lower than

Segoshi Bridge (Photo. 2.30)

This bridge was on one of the municipal roads at Segoshi, Ishikawa Prefecture about 15 km north from the epicenter, and across the Daishoji River. The maximum acceleration was estimated about 0.5 g.

The abutments were of concrete structures, and the seven piers were of reinforced concrete



Photo. 2.30 Damage to the Segoshi Bridge (Settlement of the Piers)

solid-slab-type structures. The superstructures, the total length of 87.2m and the width of 3.3m, were of 8-span I-shape steel girders. (span length being $8.5\text{m} + 6 \times 11.4\text{m} + 10.3\text{m}$).

By the earthquake moderate damage was caused at the substructures and the superstructures. The both abutments tilted toward the center of the river, which was caused by the sliding of the back fill. Several cracks were observed on the main wall and the wing walls. Every pier settled, as indicated in Photo. 2.30. The maximum settlement was 50 cm at the 4th pier. All the piers also tilted to the left-bank. Concrete slab for the superstructure moved generally to the upstream side. The main I-shape steel girders were deformed due to twisting and bending forces.

Itagaki Bridge (Photo. 2.31; Figs. 2.19, 2.20)

The bridge, completed in 1933, was on the Hashidate-Fukui Route, one of the Fukui prefectural roads, located in Fukui, Fukui Prefecture about 10 km south from the epicenter, and across the Ashiba River. The maximum acceleration was estimated about 0.6g from the overturning of tomb stones and the observation of damage in the area. The ground was of gravel layer at the bridge site.

The abutments were of gravity-type reinforced concrete structures, and the twelve piers were of reinforced concrete rigid frame (2 columns and a beam) structures with caisson foundations (Fig. 2.19). The superstructures, the

total length of 156m and the width of 4.5m, were of 13-span reinforced concrete T-shape girders (span length being $13 \times 12\text{m}$). The bearing supports were of tar paper. By the earthquake extensive damage was caused at the substructures and the superstructures. The both abutments had heavy cracks on the parapet walls and the wing masonry. The wing masonry at the left-bank abutment collapsed. The 3rd, 4th, 5th, 6th, 7th and 8th piers tilted about 10° , 4° , 12° , 3° , 1° and 1° to the left-bank, respectively. No permanent displacements were observed in the transverse direction to the bridge axis. The large tilting of the piers was caused by the failure at the connections between the columns and the caisson foundations and also by the tilting of the caisson foundations. Eight spans (out of thirteen spans) fell down into the river due to the failure of the piers (Photo. 2.31 and Fig. 2.20). Even the spans which did not fall, moved drastically, and sustained severe damage to the main beams and the slab.



Photo. 2.31 Damage to the Itagaki Bridge (Fall of Eight Spans)

Benten Bridge (Photo. 2.32, 2.33)

This bridge was on one of the municipal roads, located in Daishoji, Ishikawa Prefecture about 18 km north from the epicenter, and across a tributary of the Daishoji River. The maximum acceleration was estimated about 0.4 g near the site.

The abutments were of concrete structures, and the five piers were of reinforced concrete

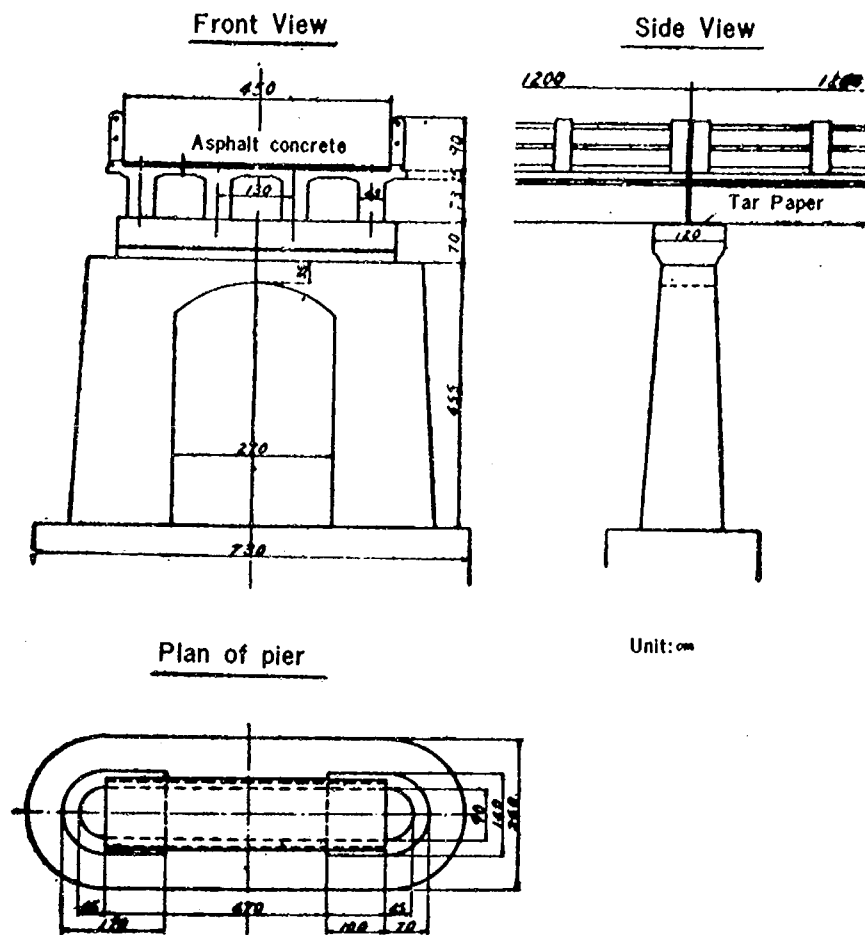
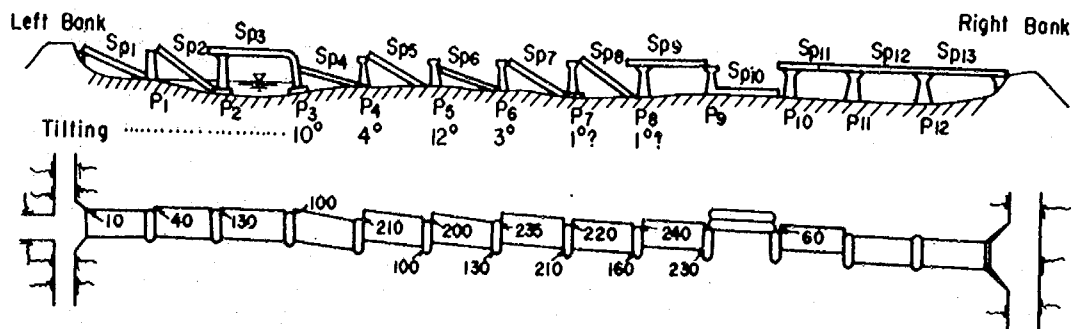


Fig. 2.19 General drawings of the Itagaki bridge



Figures indicate horizontal movements in cm

Fig. 2.20 Damage to the Itagaki bridge

rigid frame (4 columns and 3 beams) structures. The superstructures, the total length of 50.3m and the width of 3.3m, were of 6-span I-shape steel girders (span length being $2 \times 6 \text{ m} + 3 \times 10 \text{ m} + 8.3 \text{ m}$).

Due to the earthquake the bridge sustained severe damage. The right-bank abutment tilted toward the center of the river. The parapet walls at the both abutments collapsed and fell down. The 2nd pier from the left-bank deformed largely due to the fall of the 3rd span. The 3rd pier overturned completely into the river. The 4th and 5th piers tilted toward the left-bank by about 20° and 3° , respectively. The ends of the 3rd and 4th spans which were on the 3rd pier dropped down into the river (Photos. 2.32 and 2.33), that was caused by the overturning of the 3rd pier. The bearing support of the 6th span on the right abutment dislodged from the crest of the abutment.



Photo. 2.32 Damage to the Benten Bridge (1) (Fall of the 3rd and 4th Spans)



Photo. 2.33 Damage to the Benten Bridge (2) (Fall of the Girders and Tilting of the 2nd Pier)

Koroba Bridge (Photos. 2.34, 2.35; Fig. 2.21)

This bridge was also on one of municipal roads, located in Daishoji, Ishikawa Prefecture about 18 km north from the epicenter, and across a tributary of the Daishoji River. The maximum acceleration was estimated about 0.4 g near the bridge. The abutments were of concrete structures, and the two piers were of reinforced concrete rigid frame (3 columns and 2 beams at top and bottom) structures, as shown in Fig. 2.21. The superstructures, the total length of 36m and the width of 3.65m, were of 3-span reinforced concrete T-shape girders (span length being $3 \times 12 \text{ m}$).



Photo. 2.34 Damage to the Koroba Bridge (1) (Failure at the Piers)



Photo. 2.35 Damage to the Koroba Bridge (2) (Failure at the Column of a Pier)

Due to the earthquake the bridge also sustained severe damage. The wing walls of the both abutments suffered large cracks. The 1st pier from the left-bank had a large crack (width of 15 cm) at the lower beam. Heavy compressive failures were observed near the points 1m lower than the crest of the pier at the central and the downstream columns, and the reinforcements were exposed. The upstream column of the pier tilted to the upstream direction by 7° . The upstream column had a failure near the middle of

the height, and the reinforcements were exposed. Another crack was observed at the construction joint between the central column and upper beam.

The superstructure suffered heavy cracks near the mid-span of the main girders of the 1st and the 2nd spans. The damage to the girders was the severest among the damage caused by the direct vibrational effects of the earthquake on bridge superstructures.

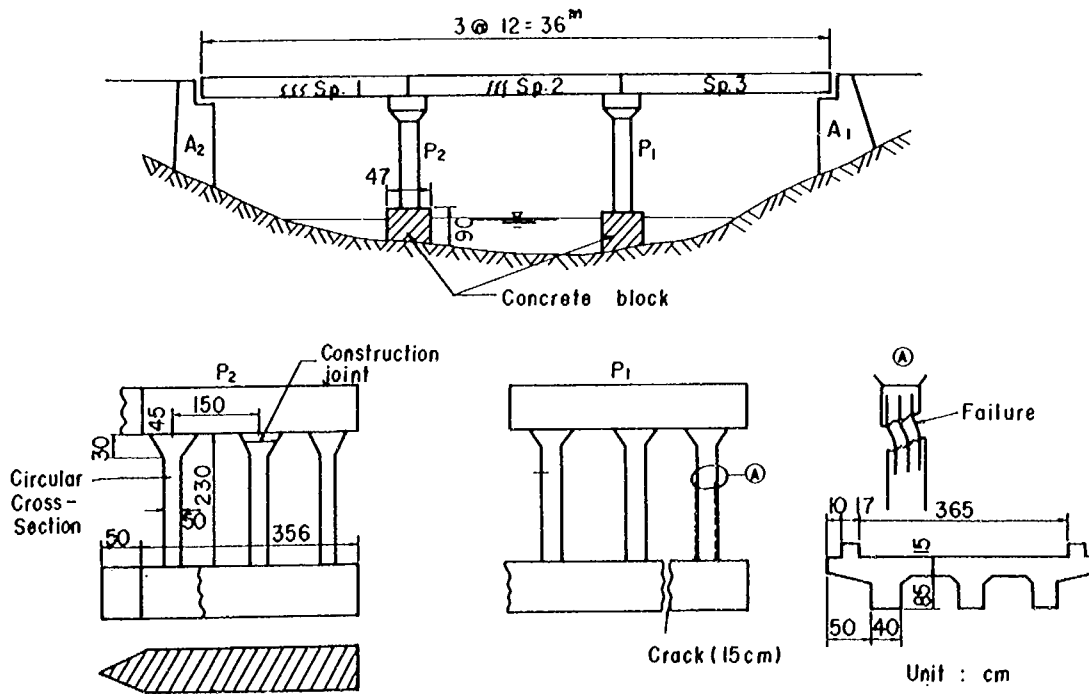


Fig 2. 21 Damage to the Koroba bridge

Summary of Bridge Damage

In view of the highway bridges damaged by the Fukui Earthquake of 1948, the damage characteristics of bridge structures are summarized as follows²⁵⁾:

The damage was mostly observed at substructures. The damage to superstructures was primarily caused by secondary effects of failure of other portions such as effects of severe shocks due to overturning or failure of the substructures. Degree of the damage to the superstructure was not severe. There was only one bridge (Koroba Bridge) which sustained superstructure damage undoubtedly caused by the direct effects of the earthquake excitation. The aspects of damage to individual portions of bridge structures are listed below.

(a) Damage to Abutments

Settlement, tilting (tilting generally toward the river), sliding, failure of parapet walls, settlement of approach roads, failure of wing walls, cracking (generally horizontal cracks on wing walls and the front walls of abutments).

(b) Damage to Piers

Settlement, tilting, overturning, cracking.

(c) Damage to Superstructures

(1) Failure at Bearing Supports

Deformation of anchor bolts, dislodging of anchor bolts from crests of abutments or piers, movement of girders due to the failure of roller supports.

(2) Damage to Main Girders

Cracking or compressive failure of the ends of main girders (generally in reinforced concrete girders) due to the collision with parapet walls on abutments.

(d) Other Significant Damage

Cracking or failure at construction joints in concrete bridges, damage due to lack of reinforcements in reinforced concrete bridges, damage due to inferior quality of concrete material.

2.4 Imaichi Earthquake of 1949²⁷⁾

Two earthquakes occurred at about 8:17 am and 8:25 am on December 26, 1949, the magnitude of which registered 6.4 and 6.7, respectively, on the Richter scale. The epicenters were in inland area near Imaichi, Tochigi Prefecture, about 110 km north from Tokyo (see Fig.

2.1). Due to the second earthquake various engineering structures such as buildings (mostly residential housings), water supply plants, railways, and highways within a limited area of about 16 km in diameter sustained severe damage, and landslides were observed at several locations. The outline of the earthquake is as follows⁵¹⁾:

Name of Earthquake:	Imaichi Earthquake
Date of Occurrence:	December 26, 1949
Epicenter:	Imaichi, Tochigi Prefecture (36.7°N, 139.7°E)
Depth of Hypocenter:	Shallow
Magnitude on the Richter Scale:	6.7 (for main shock)

It was estimated from the observation of the aftershocks that the vibration of the main shock near the epicenter might be very severe with large accelerations and short periods. The ground conditions near the epicenter were generally rocky layers or diluvial plateaus with loams.

As for the damage to bridge structures, no significant damage was observed. One railway bridge had failures at its bearing supports, and one highway bridge which overpassed a railway sustained very slight damage at the abutment.

2.5 Tokachi-oki Earthquake of 1952^{28), 46)}

A great earthquake occurred under the sea



Photo. 2.36 Damage to the Monbetsu Bridge (Crack at the 3rd Pier)

off Tokachi, about 50 km east from the Erimo Cape, Hokkaido on March 4, 1952, registering magnitude of 8.1 on the Richter scale. The outline of earthquake is as follows⁵¹⁾:

Name of	
Earthquake:	Tokachi-oki Earthquake of 1952
Date of	
Occurrence:	March 4, 1952
Epicenter:	Off Tokachi (42.2°N, 143.9°E)
Depth of	
Hypocenter:	45 km
Magnitude on the	
Richter Scale:	8.1

Due to the earthquake many engineering structures such as public works, transportation facilities, buildings, and housings in the southeast part of Hokkaido sustained heavy damage. The total loss in engineering structures was estimated about 15 billion yen, among it the loss of

highways (184 locations) was about 410 million yen, and that of highway bridges (128 bridges) was about 200 million yen. The amounts of the loss were counted at the value at that time.

Although some temporary wooden bridges collapsed entirely, no significant damage was observed for permanent concrete bridges and steel bridges. However, bearing supports, substructure caps near the supports, and hinge structures for suspended girders sustained slight damage for several bridges such as Monbetsu Bridge (Photo. 2.36), the Shizunai Bridge (Photo. 2.37), the Horoman Bridge (Photo. 2.38), and the Otanoshike Bridge.

2.6 Northern Miyagi Earthquake of 1962^{30), 46)}

A moderate earthquake occurred in an inland area near the northern part of Miyagi Prefecture in Honshu Island, about 50 km north from Sendai, Miyagi Prefecture on April 30, 1962 (Fig. 2.1). The outline of the earthquake is as follows⁵¹⁾:

Name of	
Earthquake:	Northern Miyagi Earthquake
Date of	
Occurrence:	April 30, 1962
Epicenter:	Northern Miyagi (38.7°N, 141.1°E)

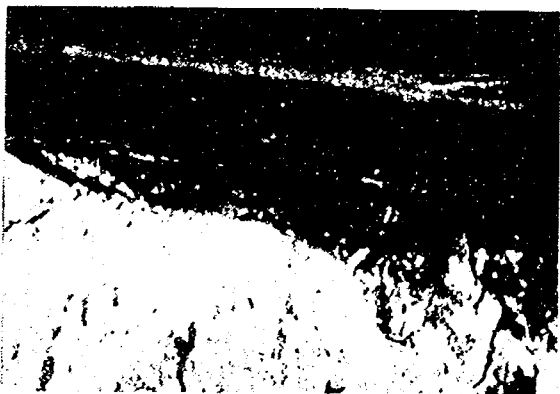


Photo. 2.37 Damage to the Shizunai Bridge (Failure at the Fixed Shoe of the 1st Pier)



Photo. 2.38 Damage to the Horoman Bridge (At the Hinged Support)



Photo. 2.39 Movements of the Girders at the Eai Bridge

Depth of

Hypocenter: About 10 km

Magnitude on the

Richter Scale: 6.5

Due to the earthquake many engineering structures such as public works, industrial facilities, water supply systems, agricultural facilities, housings, etc., sustained moderate damage in a very limited area of the northern part of Miyagi Prefecture. The total loss was estimated about 4 billion yen. The amounts of loss of highways and highway bridges were about 102 million yen and 43 million yen, respectively.

Although many bridges (187 bridges) sustained very slight damage, no significant damage was observed. Only two bridges (the Eai Bridge (Photo. 2.39) and the Konoh Bridge) sustained moderate damage at their bearing supports and the caps of the substructures. The slight collision of adjoining girders was observed at the Eai Bridge.

2.7 Niigata Earthquake of 1964 ³²⁾⁻⁴²⁾

The Niigata Earthquake, measuring magnitude of 7.5 on the Richter scale, occurred in the northwestern part of Honshu Island on June 16, 1964. The epicenter was under the sea near Awashima Island, about 55 km north from Niigata City (Figs. 2.1 and 2.22). The outline of the earthquake is as follows⁵¹⁾:

Name of

Earthquake: Niigata Earthquake

Date of

Occurrence: June 16, 1964

Epicenter:

Near Awashima Island,
Niigata (38.4°N, 139.2°E)

Depth of

Hypocenter: 20 - 30 km

Magnitude on the

Richter Scale: 7.5

Severe damage was caused in the alluvial plain near the mouths of the Shinano River and

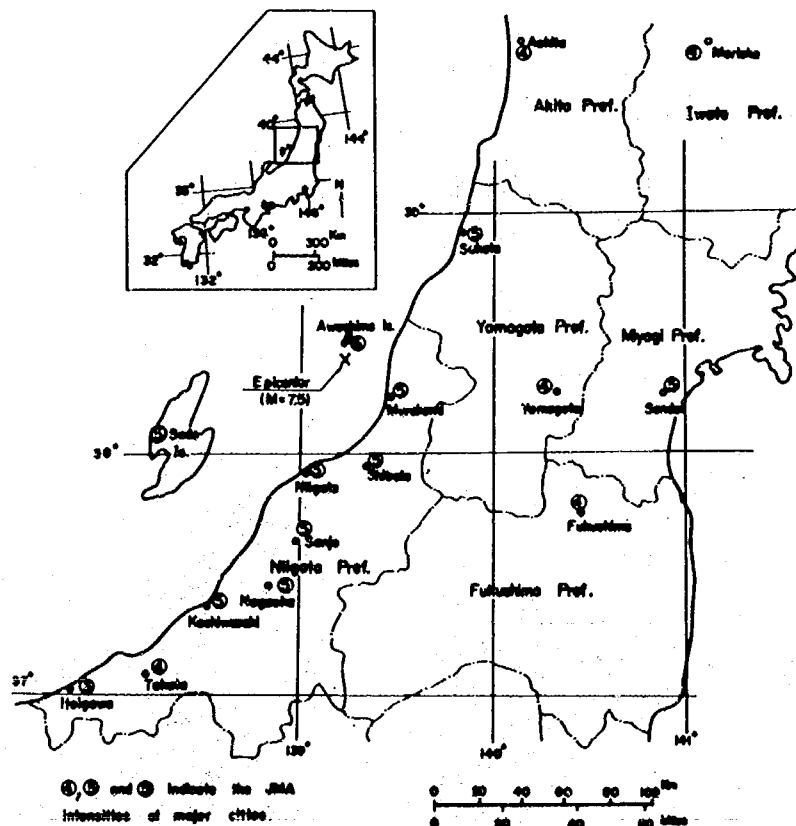


Fig. 2. 22 The Niigata Earthquake of 1964

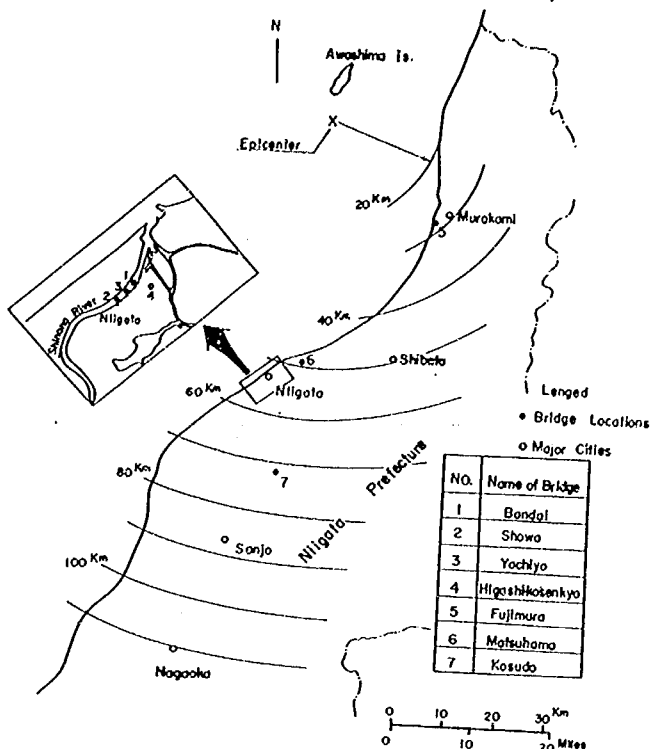


Fig. 2 23 Locations of the Seven Bridges which Sustained Severe Damage

the Agano River in Niigata City. Especially in the area near the mouth of the Shinano River where loose sand layers with high water table exist, reinforced concrete buildings, highway bridges and other structures sustained considerable damage due to the liquefaction phenomena of the ground soils.

Two sets of strong motion accelerographs, installed at the basement floor and the roof of a damaged apartment building located along the Shinano River, triggered the complete acceleration records during the earthquake. The maximum accelerations at the basement were about 0.15g (period of about 2 seconds) horizontally, and about 0.05g (period of about 0.3 seconds) vertically.

The followings are the descriptions on the damage to seven bridges which sustained relatively serious damage. Their locations are shown in Fig. 2.23.

Bandai Bridge (Photos. 2.40, 2.41; Figs. 2.24 – 2.28; Table 2.5)

This bridge completed in 1929, is on the

National Highway No. 7, across the Shinano River in Niigata City, Niigata Prefecture about 54 km south from the epicenter. The JMA intensity was 5, and the maximum accelerations measured at the building basement nearby were about 0.15 g horizontally and about 0.05 g vertically.

The ground conditions are of sandy soils (Figs. 2.25 and 2.26). The abutments are of gravity-type reinforced concrete structures with

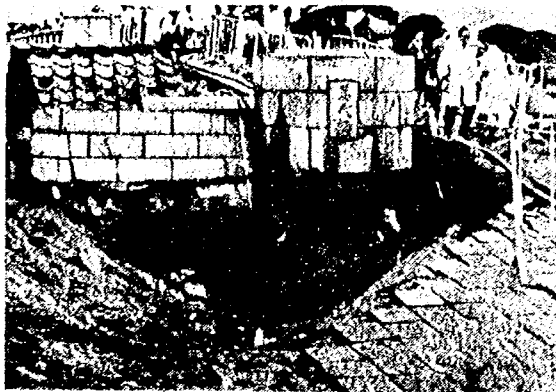
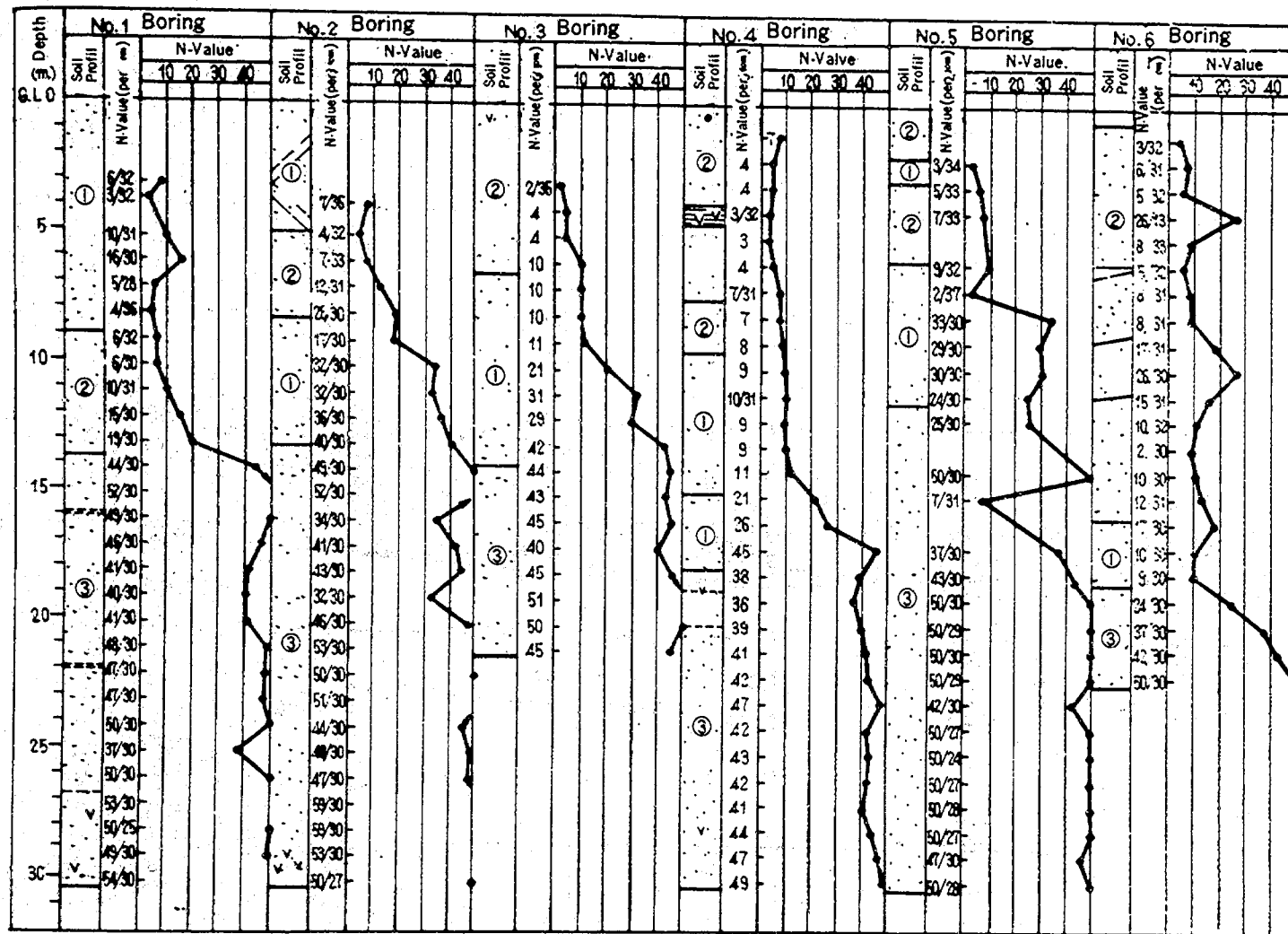


Photo. 2.40 Damage to the Abutment at the Bandai Bridge



Note)
 ① Medium sand
 ② Coarse sand
 ③ Fine sand

Fig. 2.25 Boring logs at the Bandai bridge

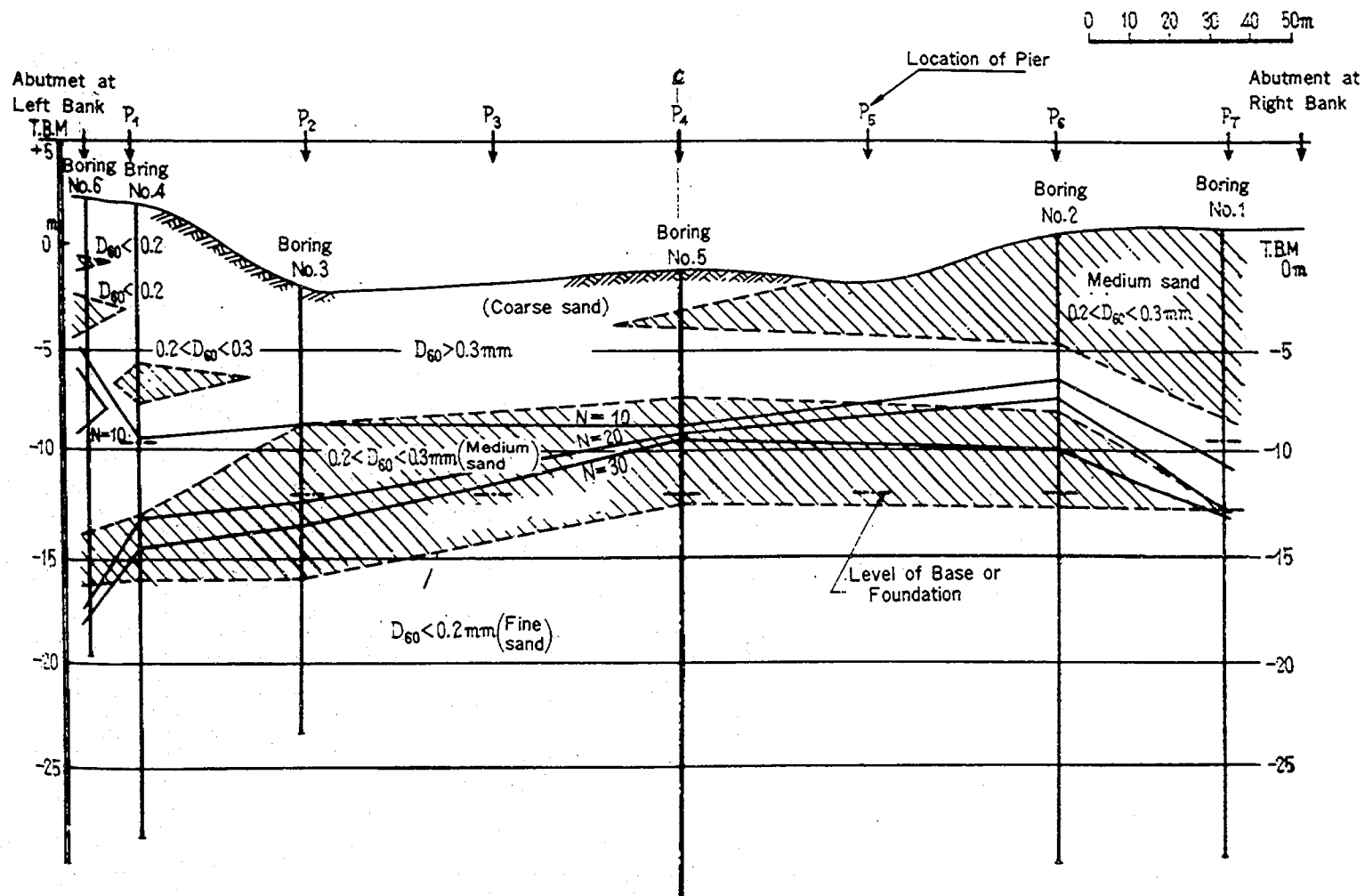


Fig. 2. 26 Soil Profile at the Bandai bridge

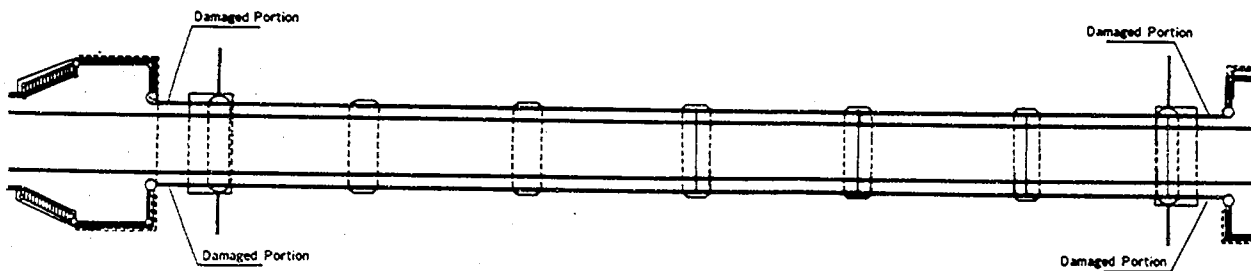
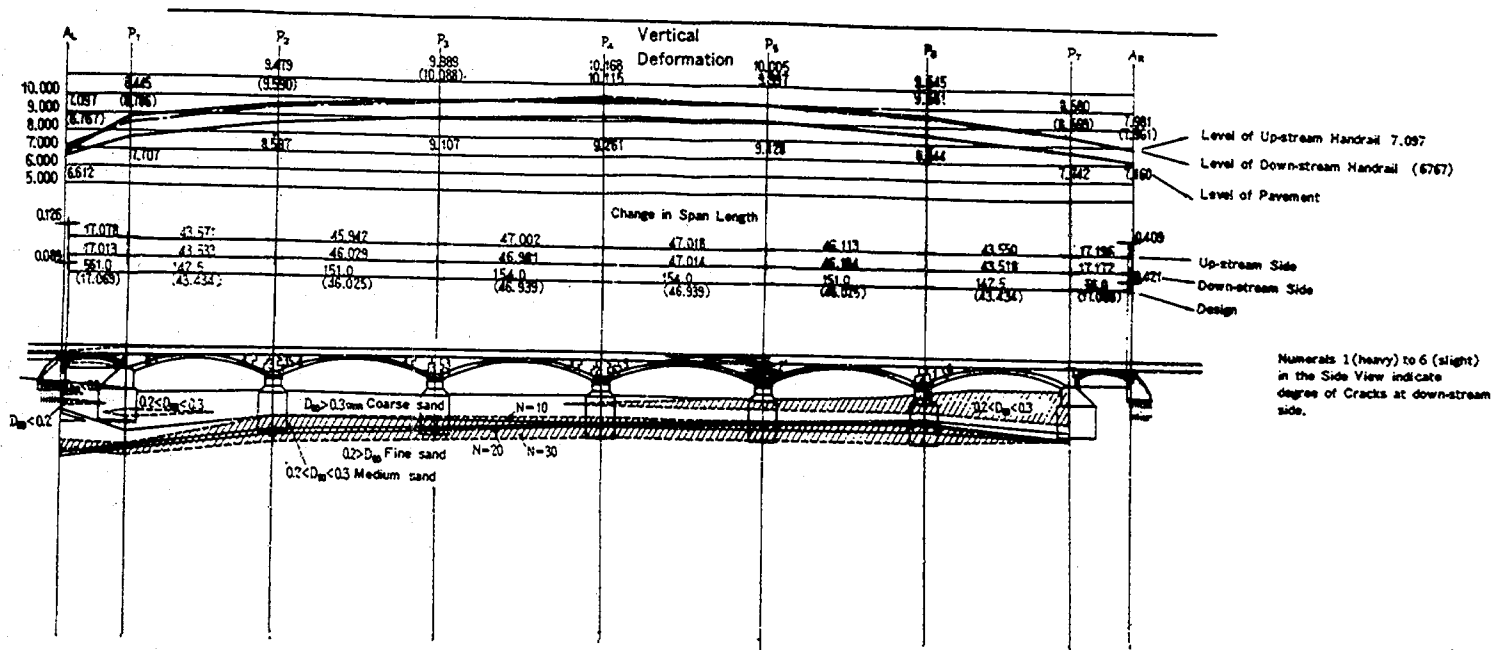


Fig. 2.27 Damage to the Bandai bridge

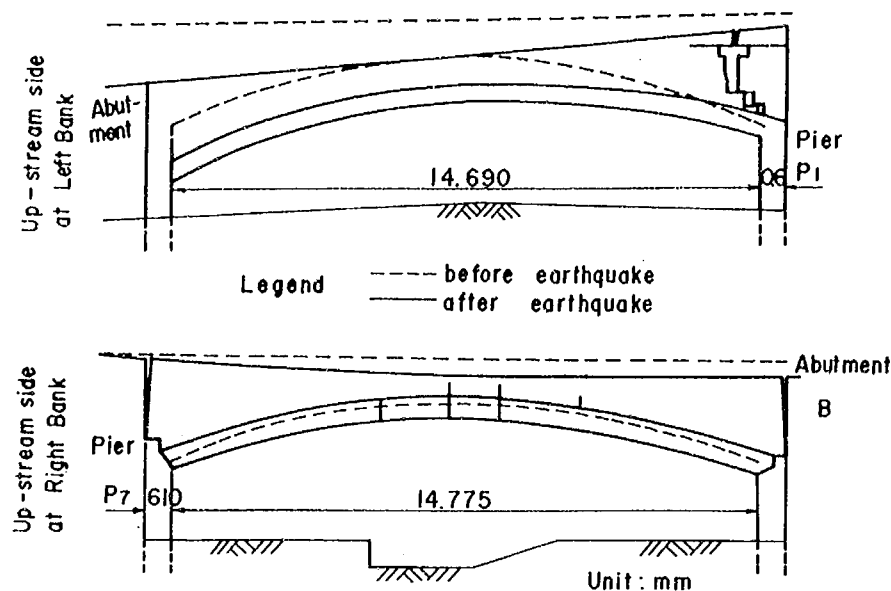


Fig. 2.28 Damage to the side spans of the Bandai bridge

wooden piles, and the piers are of solid-slab-type reinforced concrete structures with caisson foundations. The superstructures are of 8-span reinforced concrete arch structures, with two 2-hinge arches for side spans and six continuous fixed arches for main spans (Fig. 2.24). The total length is 309 m $(= (17.1 \text{ m} + 43.6 \text{ m} + 46.0 \text{ m} + 46.9 \text{ m}) \times 2)$, and the width is 21.8 m. Table 2.5 indicates the results of tests on the bearing powers of the ground at the bottom of the caissons.

Due to the earthquake the bridge sustained severe damage (Photos. 2.40 and 2.41; Figs. 2.27 and 2.28). The both abutments settled considerably (1.4 m at the left and 0.4 m at the right), and tilted slightly toward the center of the river. The back fills also settled largely. The parapet walls separated from the abutments and moved relatively to the abutments (1 m at the left and 2 m at the right).

Two piers also settled moderately (30 cm at P1 and 15 cm at P6). Drastic deformations and heavy cracks were observed at the two extremely

side spans due to the settlements of the substructures (Fig. 2.28). Slight lengthening of the right side span was measured. As for six continuous spans slight permanent deflection (less than 1 cm at midspan points) and several slight cracks in stone masonry near the piers were generally observed (Photo. 2.41).

At the arch rib of the 7th span appeared some slight cracks. The change in span length was measured to be slight (on the order of a few centimeters). After the earthquake the tilting of the abutments and the piers, the change in span length and the change in arch deflection were measured for a few months to predict successive damage.

Although the bridge was capable of carrying the traffic after a temporary repairing work which took only a couple of days, a long-period work was needed for complete repairing. The amount of loss of the bridge was estimated about 330 million yen, which was the largest among the damage to the highway bridges due to the Niigata Earthquake.

Table 2.5 Results of Bearing Powers Tests
at the Bandai Bridge Site

Caisso No.	Date of Tests	Elevation of Bearing Surface at Tests	Soils	Area of Bearing Surface at Tests	Ultimate Bearing Powers	Elevation of Base of Caisson Completed
1	Oct., 1927	-12.4m	Brown Coarse Sands	0.097 m ²	75.35 t/m ²	-12.4 m
2	Oct., 1972	-14.3	Gray Fine Sands	0.196 m ²	118.14	-15.2
3	Nov., 1927	-14.6	"	"	133.69	-15.2
4	Nov., 1927	-15.2	"	"	131.61	-15.2
5	Dec., 1927	-14.9	"	"	131.86	-15.2
6	Dec., 1927	-14.0	"	"	133.43	-15.2
7	Feb., 1928	-12.4	Brown Coarse Sands	"	111.97	-13.6

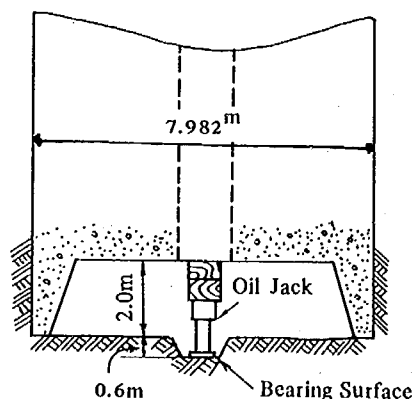
A sketch of the tests is shown on the figure.

Showa Bridge (Photos. 2.42 - 2.48; Figs. 2.29 - 2.34)

The bridge, completed in May, 1964 one month prior to the earthquake, was on the Niigata prefectural road No. 546, across the Shinano River at the point about 1.2 km above the Bandai Bridge. The bridge was located about 55 km south from the epicenter. The grounds are of sandy soils, comparatively loose near the left-bank and comparatively dense near the right-bank (Figs. 2.30 to 2.32).

The abutments are of pile bents (nine single-row piles of diameter of 609 mm and length of 22 m), and the piers are also of pile bents (nine single-row piles of diameter of 609 mm and length of 25 m) with collar braces and cap beams (Fig. 2.29). The design seismic coefficient for substructures was considered as 0.2 horizontally. The superstructures are of 12-span steel composite simple girders. The total length is 303.9 m (= 13.75 + 10 × 27.64 + 13.75), and the width is 24 m.

Due to the earthquake the bridge sustained very severe damage (Fig. 2.33). The left-bank abutment moved about 1m toward the center of the river, and the approach road on the left-bank settled considerably. On the other hand, the



right-bank abutment and the approach road sustained no significant damage. The first to fourth piers from the left-bank tilted toward the right-bank. The magnitudes of permanent deformation were 13 to 42 cm at their caps. The fifth and sixth piers collapsed completely into the river. However, the seventh to eleventh piers suffered only slight damage. Five girders (the third to seventh from the left-bank) out of twelve girders fell down into the river (Photo. 2.42). The sixth span fell down its both ends due to the failure of the fifth and sixth piers which had supported the span. Damage characteristics of the bridge are shown in Fig. 2.33.

The followings are revealed to be the main

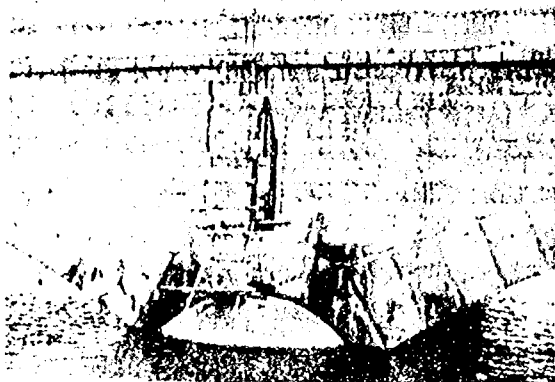


Photo. 2.41 Cracks at the Bandai Bridge

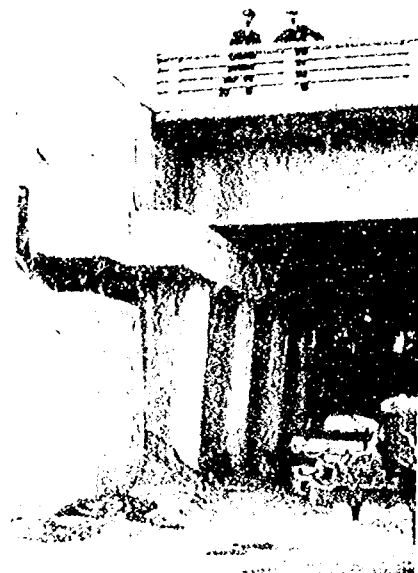


Photo. 2.44 Damage to the Left-Bank Abutment at the Showa Bridge

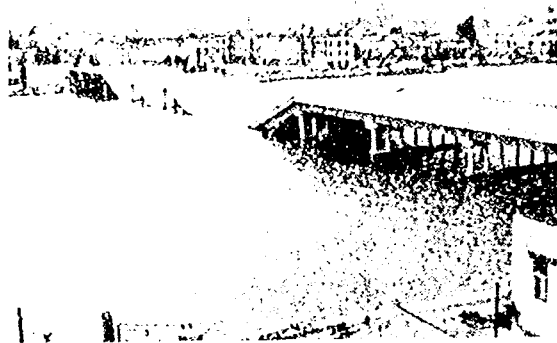


Photo. 2.42 Damage to the Showa Bridge (1) (General View of the Damage)



Photo. 2.45 The Ends of the Girders on the Seventh Pier at the Showa Bridge

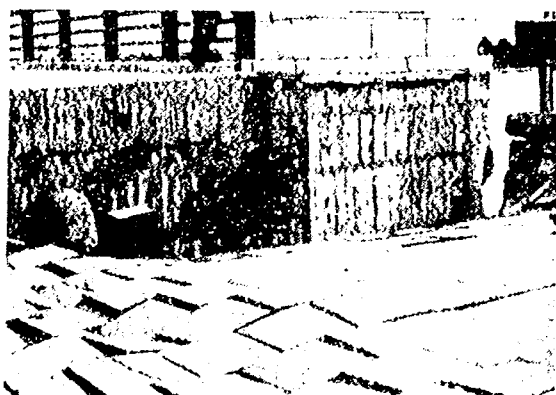


Photo. 2.43 Damage to the Showa Bridge (2) (Settlement of the Approach Road at the Left-Bank Abutment)

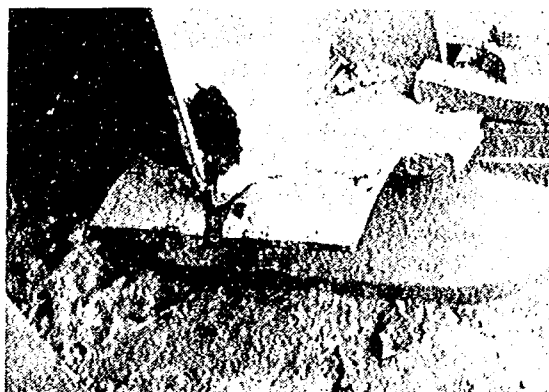
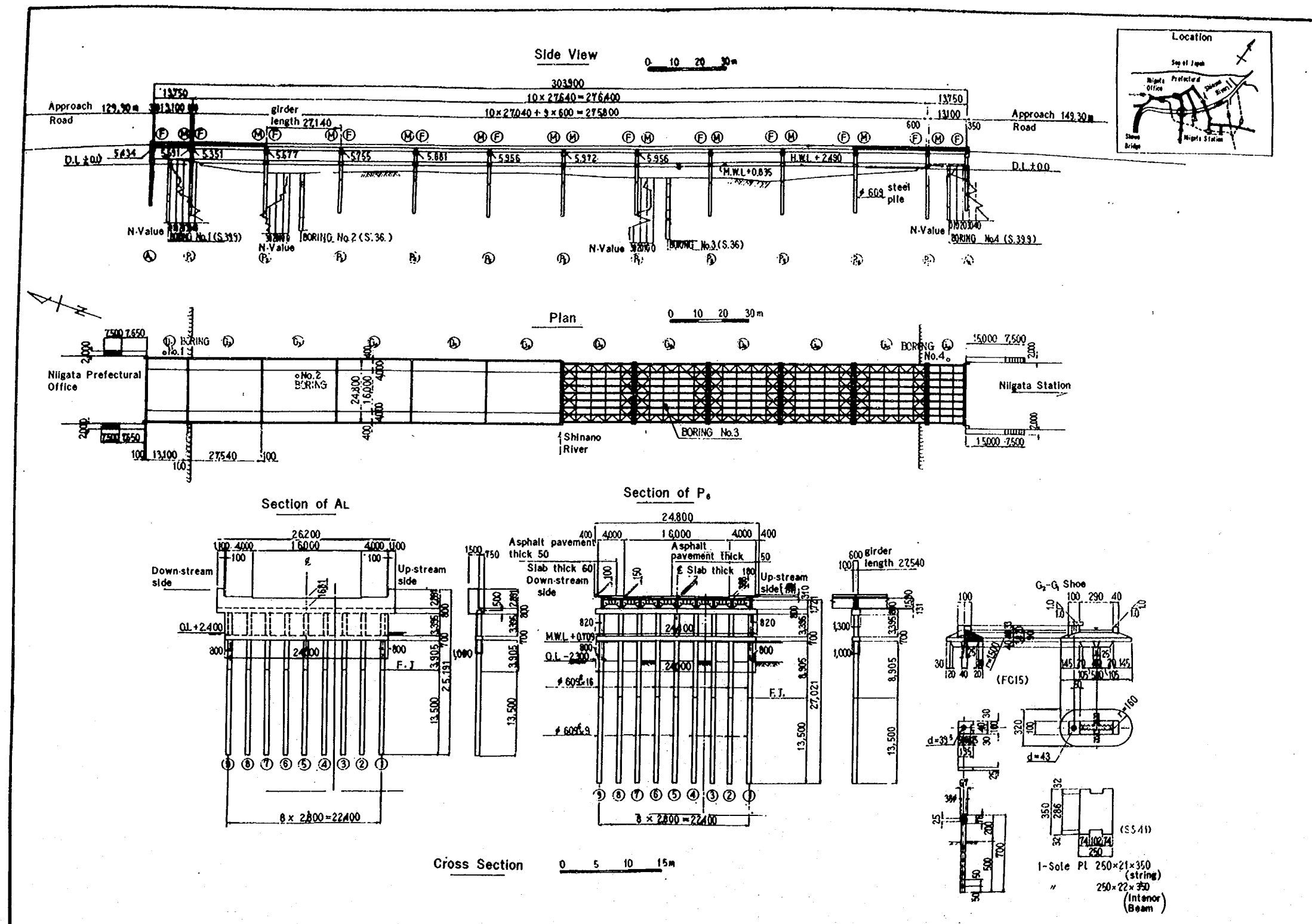


Photo. 2.46 The End of the Third Girder on the Second Pier at the Showa Bridge



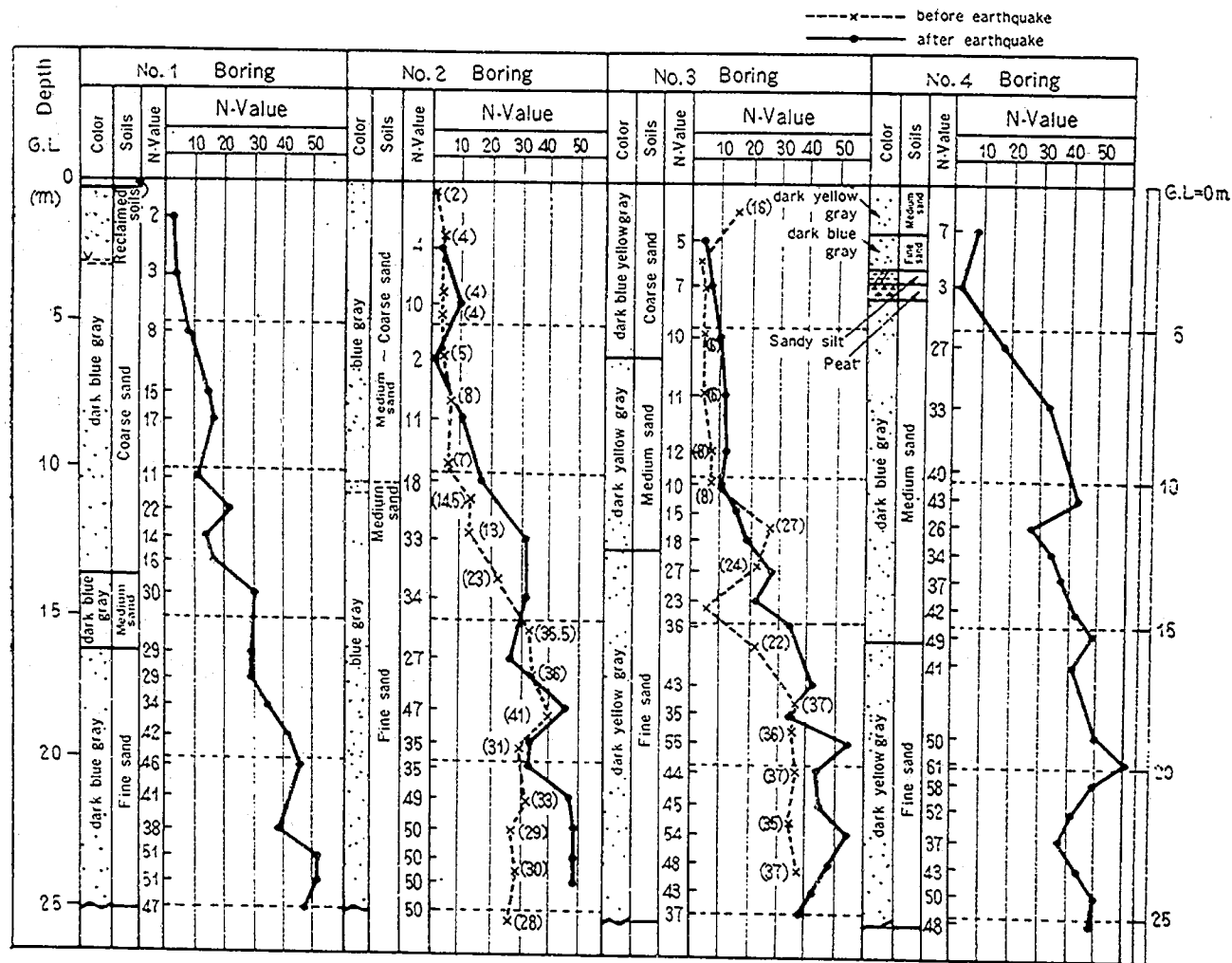


Fig. 2.30 Boring logs at the Showa bridge

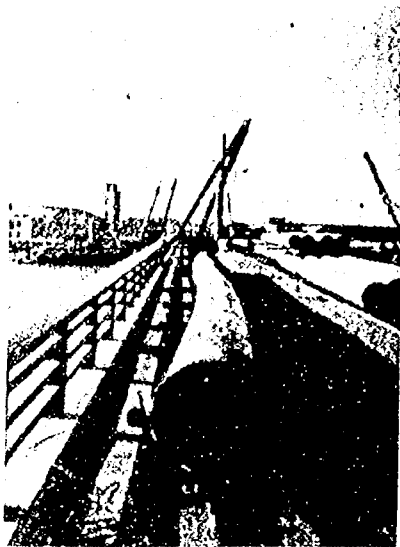


Photo. 2.47 A Pile Pulled out from the Fourth Pier and the Remaining Part of the Bridge



Photo. 2.48 Buckling of the Pile Pulled out

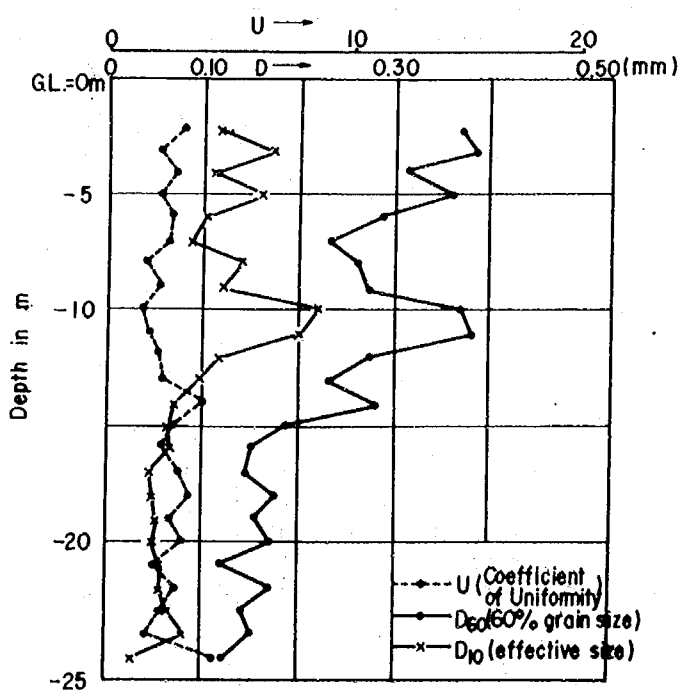


Fig. 2.31 Characteristics of soil grain size at the Showa bridge

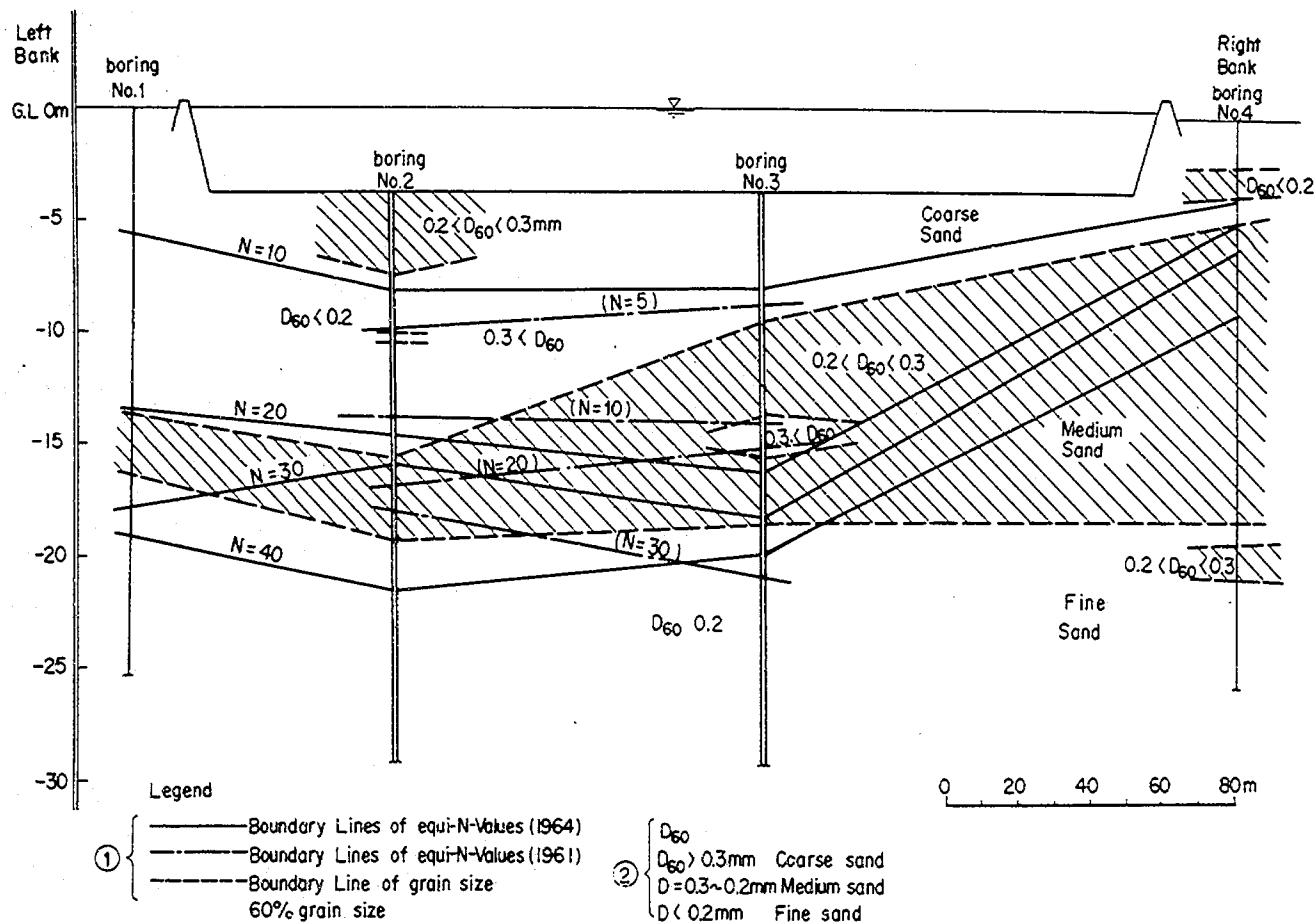


Fig. 2.32 Soil profile at the Showa bridge

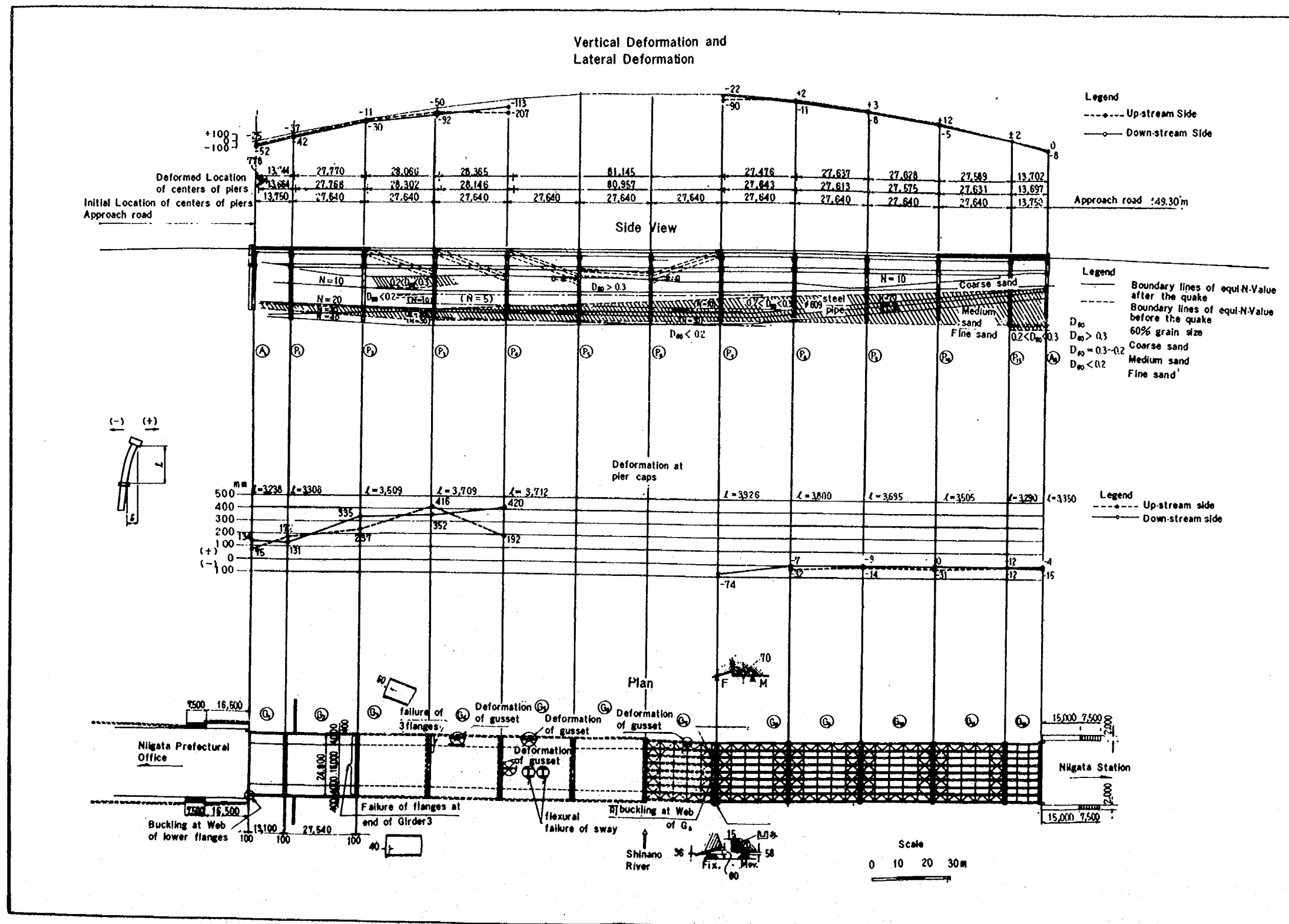


Fig. 2.33 Damage to the Showa bridge

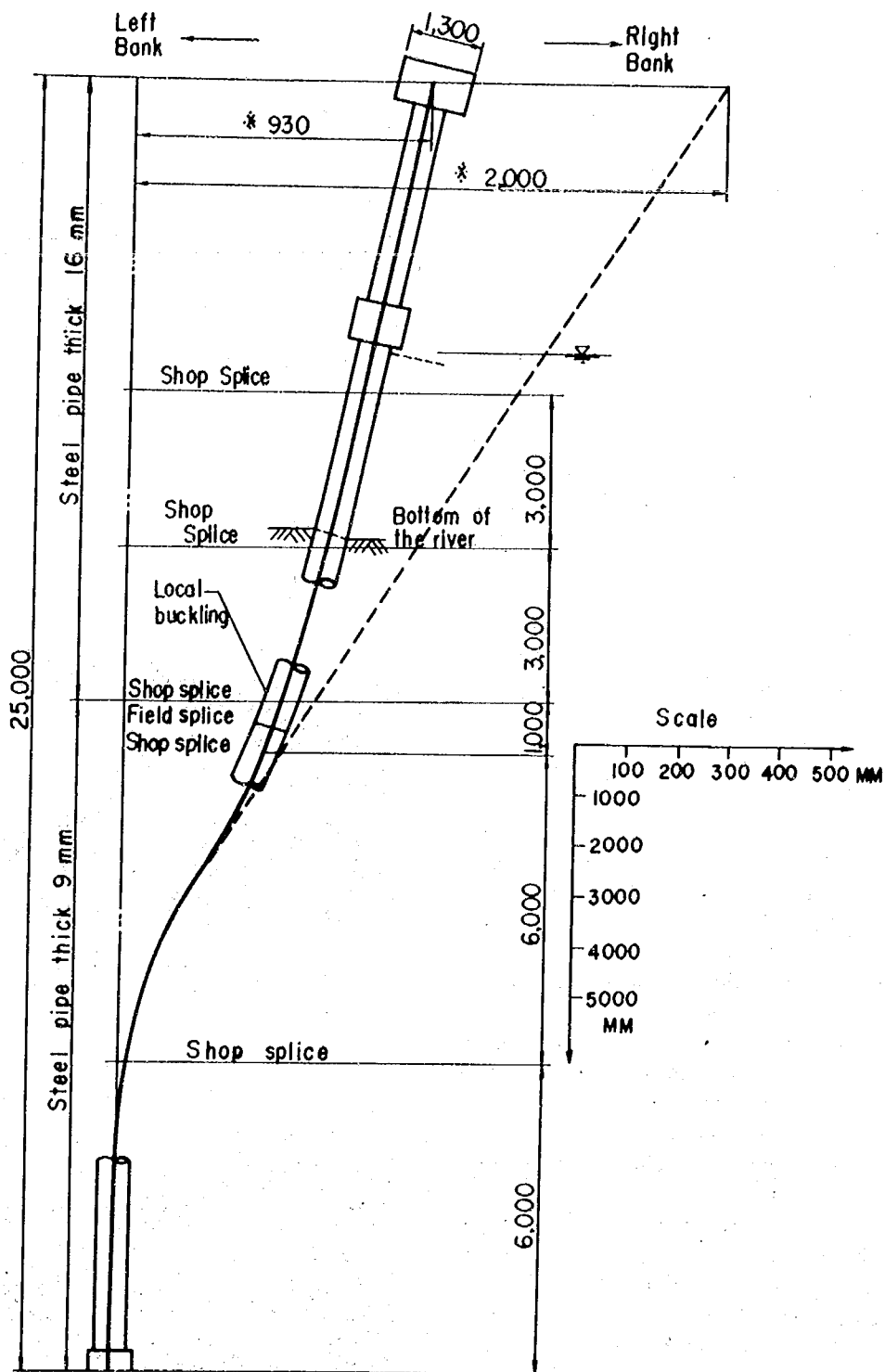


Fig. 2. 34 Permanent deformation of a pile pulled out at the Showa bridge

causes of the heavy damage:

- 1) The substructures made by single-row steel piles were too flexible.
- 2) Liquefaction phenomena occurred at the bridge site (except near the right-bank).
- 3) Both bearing supports on the sixth pier were movable ones. By the reason the girders on this particular pier were easy to dislodge from the pier.
- 4) The type of the superstructure was of simple girders, and the girders were not connected each other.
- 5) Near the left-bank abutment, a catastrophic sliding of the ground was observed.

On the damage to the bridge were conducted extensive surveys and studies including soils studies, measurements of dynamic properties of the bridge for the remaining portions less damaged, measurements of the deformation for the whole structure and of the deformation of piles under the ground by pulling out (Photos. 2.47 and 2.48; Fig. 2.34), and the dynamic analyses of the bridge subjected to the strong-motion record during the earthquake.

Yachiyo Bridge (Photos. 2.49 – 2.53; Figs. 2.35 – 2.39)

The bridge, completed in 1962, is on a Niigata municipal road, is located about 55 km south from the epicenter, and is also across the Shinano River at the point between the Bandai Bridge and Showa Bridge. The grounds are of sandy soils (Figs. 2.36 and 2.37).

The abutments are of gravity-type reinforced concrete structures with reinforced concrete pile foundations, and the piers are of solid-slab-type reinforced concrete structures with reinforced concrete pile foundations. The superstructures are of 4-side-span prestressed concrete simple girders and 10-mid-span steel composite simple girders. The total length of the bridge is 307.4 m ($=2 \times 7.48\text{m} + 10 \times 27.5\text{m} + 2 \times 8.72\text{m}$), and the width is 8m (Fig. 2.35).

Due to the earthquake the bridge also sustained serious damage (Photos. 2.49 to 2.53; Figs. 2.38 and 2.39). The both abutments moved to the center of the river, and tilted toward the opposite direction. The 1st, 2nd, 12th the 13th piers from the left-bank, which supported the prestressed concrete side-span girders, moved to the center of the river (the maximum was 1.1 m at the 2nd pier) by the movement of the



Photo. 2.49 Damage to the Yachiyo Bridge at the Left-Bank Abutment, the First and Second Piers

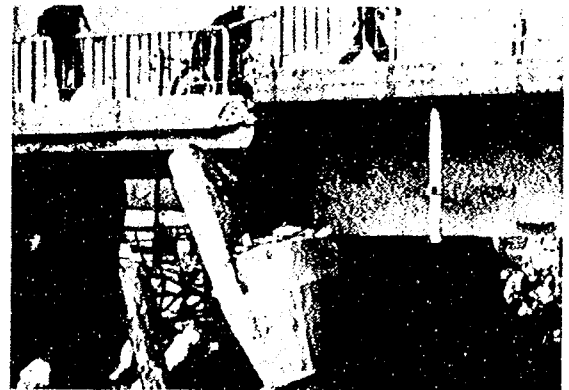


Photo. 2.50 Failure of the Second Pier of the Yachiyo Bridge (1)



Photo. 2.51 Failure of the Second Pier of the Yachiyo Bridge (2)

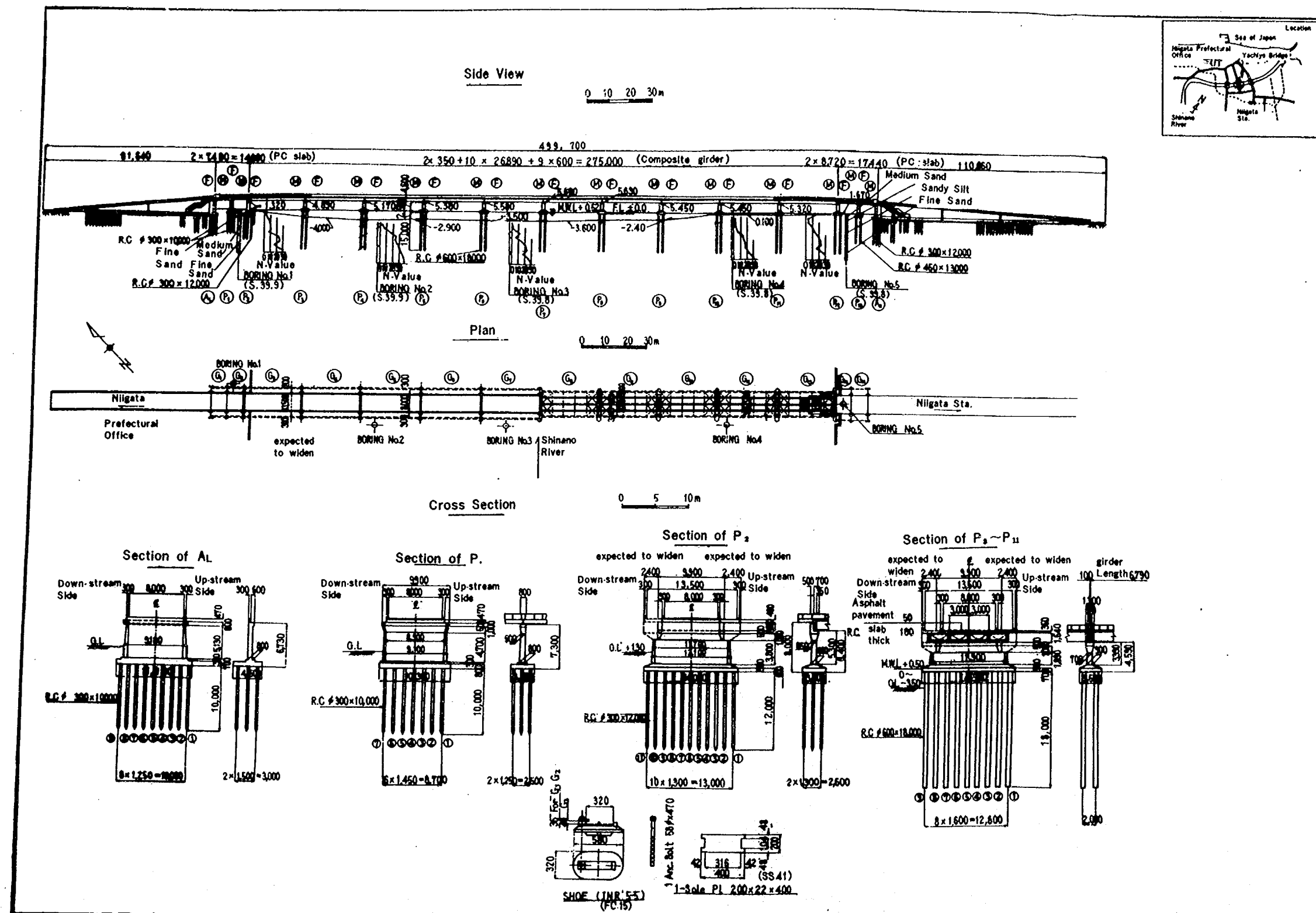
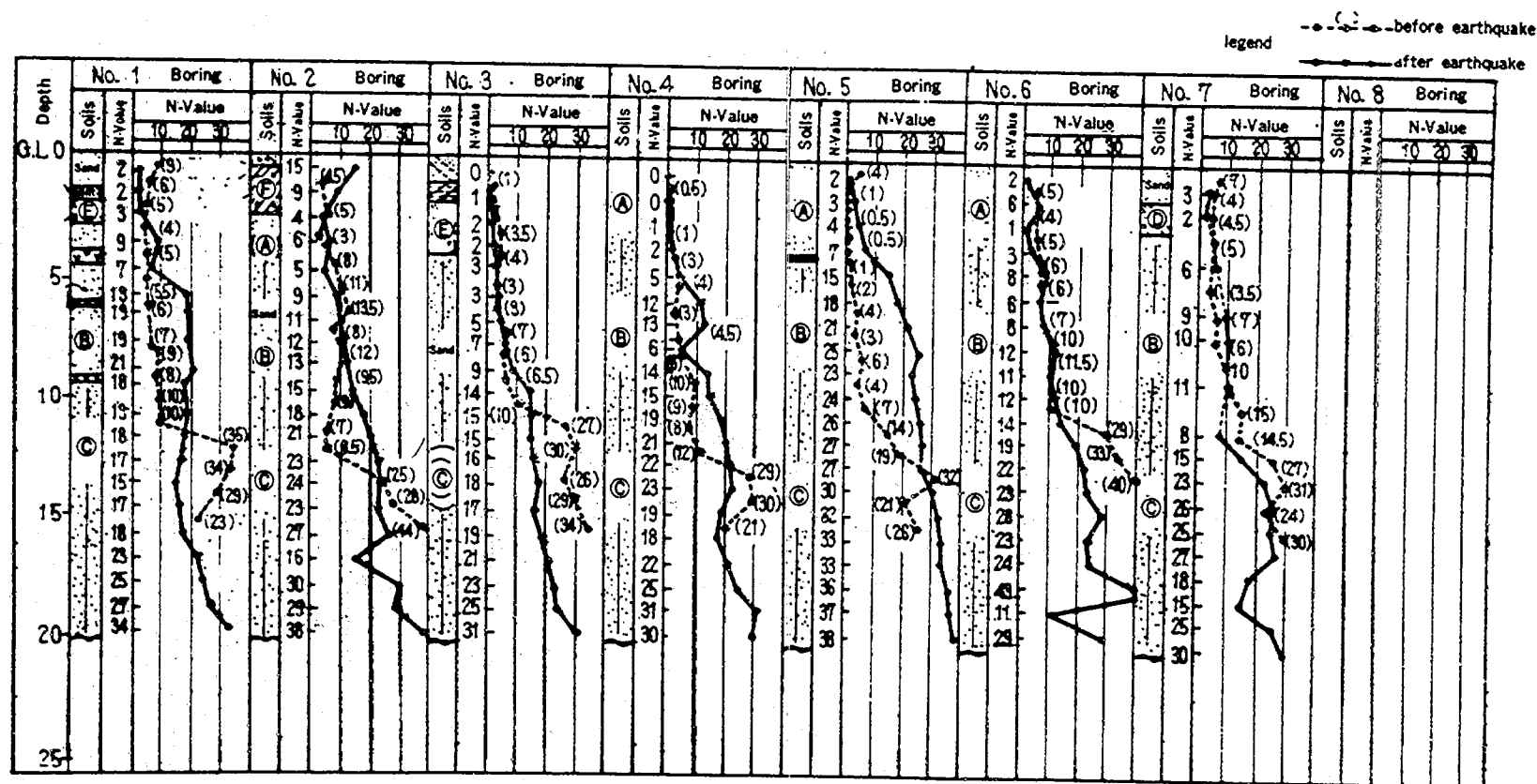


Fig. 2.35 General view of the Yachiyo bridge



Note)

Ⓐ Coarse sand	Ⓓ Silty sand
Ⓑ Medium sand	Ⓔ Sand with silt
Ⓒ Fine sand	Ⓕ Clay silt with gravel
	Ⓖ Sand with organic matters

Fig. 2. 36 Boring logs at the Yachiyo bridge

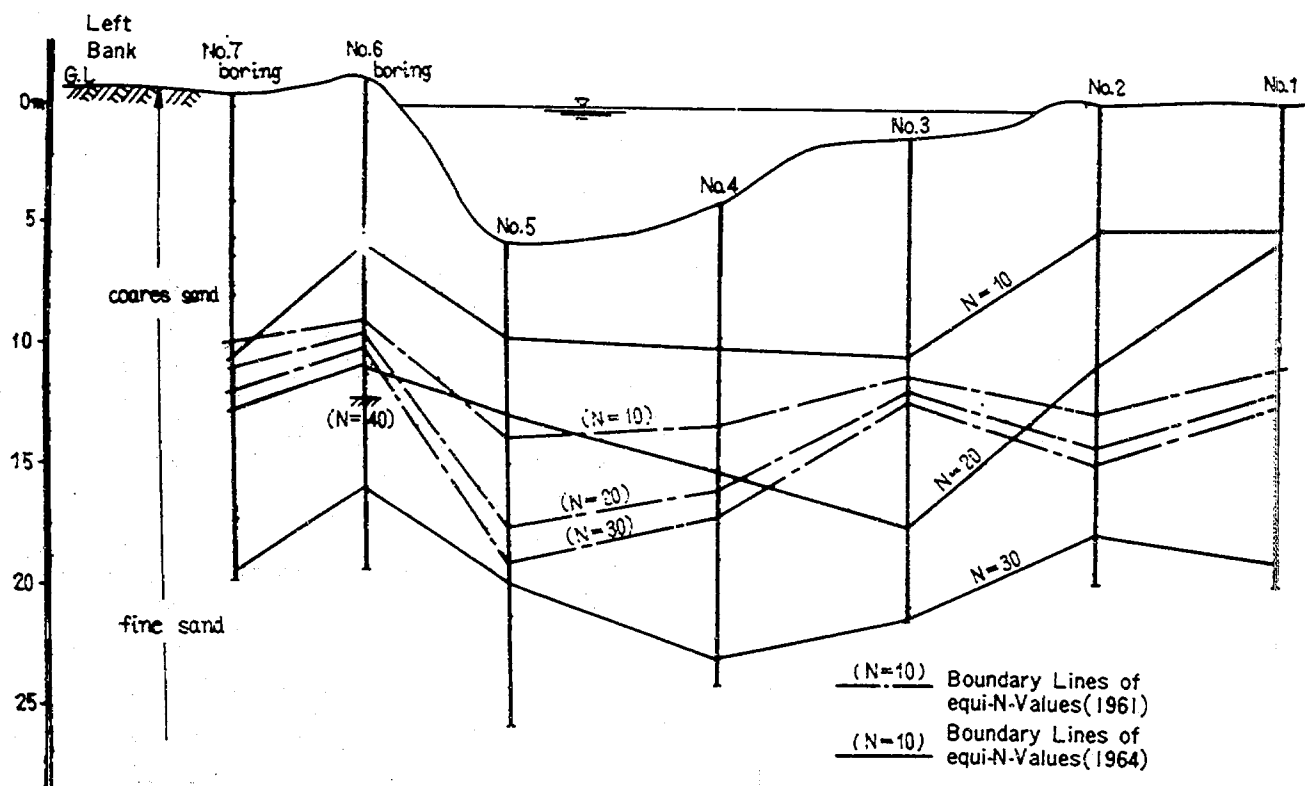
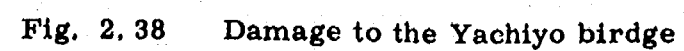


Fig. 2. 37 Soil profile at the Yachiyo bridge



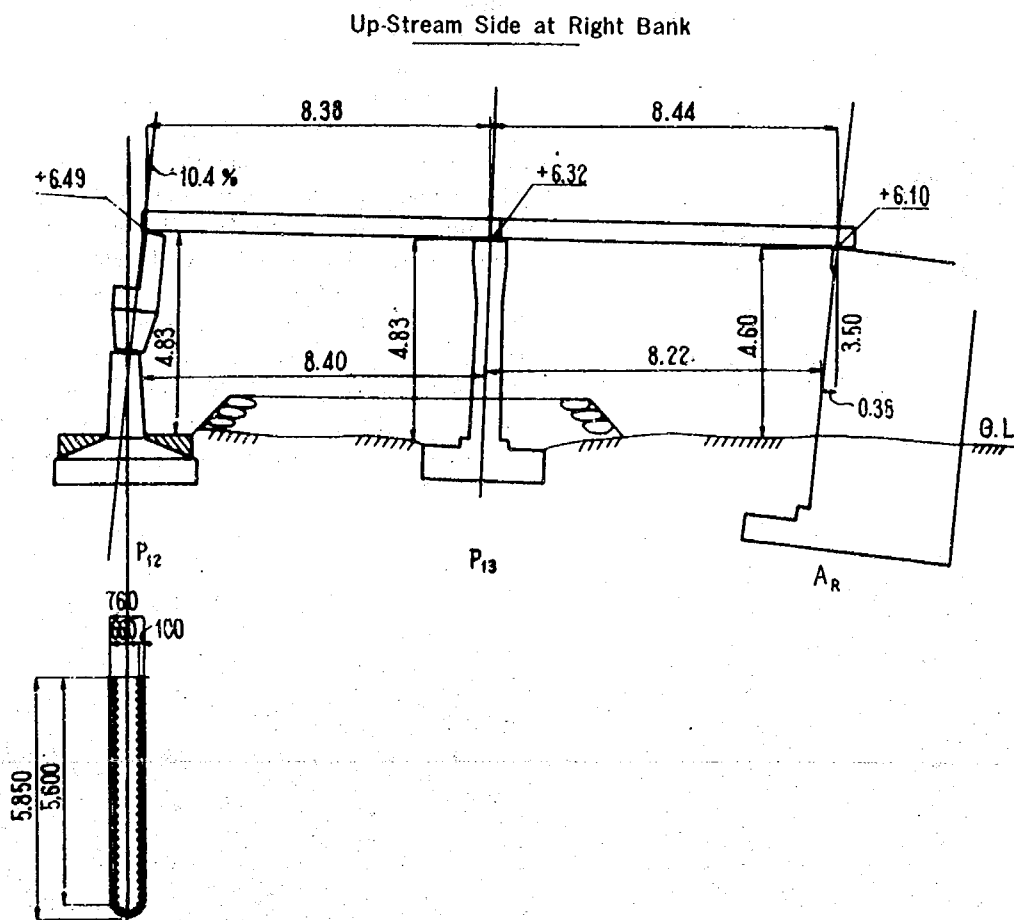
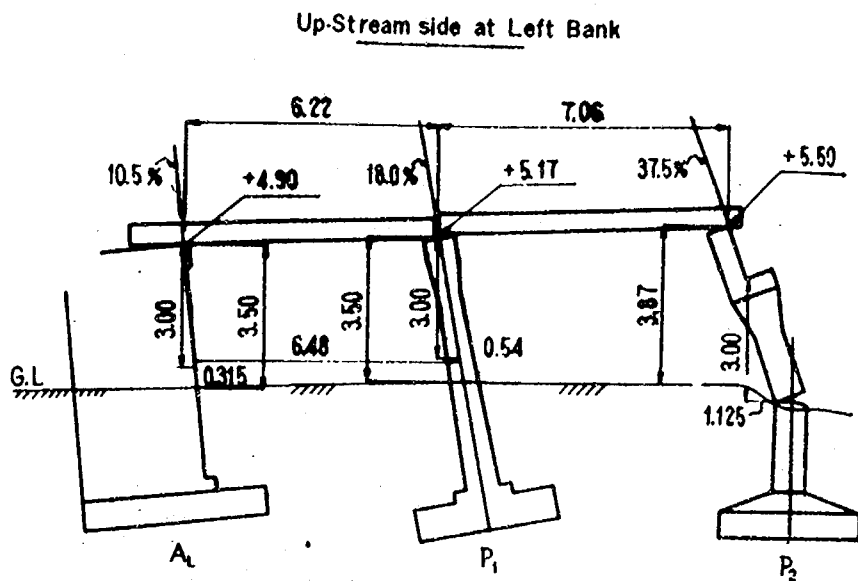


Fig. 2.39 Deformation of the side spans of the Yachiyo bridge

grounds, and tilted toward the opposite side due to the resistance of the girders. The 2nd and 12th piers cracked very seriously. The 3rd to 11th piers moved (the maximum was 40 cm at the 9th pier) and tilted moderately. The 4th and 9th piers suffered heavy cracks at the supports. It was understood that the sliding of the grounds (moving toward the center of the river) near the both abutments affected the damage to the bridge seriously.



Photo. 2.52 Damage to the End of the Tenth Girder on the Ninth Pier of the Yachiyo Bridge (1)

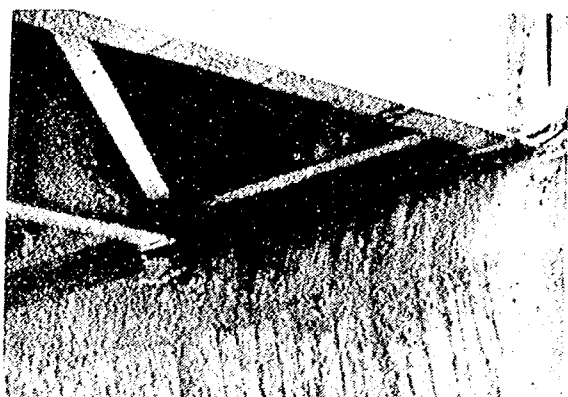


Photo. 2.53 Damage to the End of the Tenth Girder on the Ninth Pier of the Yachiyo Bridge (2)

The total length of the bridge superstructures shortened by 40 cm due to the sliding of the grounds (together with the substructures) toward the center of the river. The girders which were supported by the 2nd, 4th, 9th and 12th piers moved extensively and sustained serious damage. Since no girders fell down into the river, the bridge was usable for pedestrians to cross the river even immediately after the earthquake.

Higashi-kosenkyo Bridge (Photos. 2.54 -- 2.57; Figs. 2.40 -- 2.44)

The bridge, completed in 1963, is on a Niigata municipal road, overpassing the Shinetsu Line of the Japanese National Railways. The bridge is located in Niigata, Niigata Prefecture about 54 km south from the epicenter. The grounds are of sandy soils (Figs. 2.41 and 2.42), and the water table is about 2 to 3m below the ground surface.

The abutments are of solid-slab-type reinforced concrete structures with wooden pile foundations, and the two piers for the main span are of solid-slab-type reinforced concrete structures with reinforced concrete pile or wooden pile foundations. Thirteen piers are of rigid frame structures of three columns with wooden pile foundations. The seismic coefficient for the design of the substructures was considered to be 0.2 horizontally. The superstructures are of 15-side-span prestressed concrete simple girders and single-main-span steel composite simple girder. The total length is 229.5m ($= 8 \times 13.5\text{m} + 26.5\text{m} + 7 \times 13.5\text{m}$), and the width is 8m (Fig. 2.40).

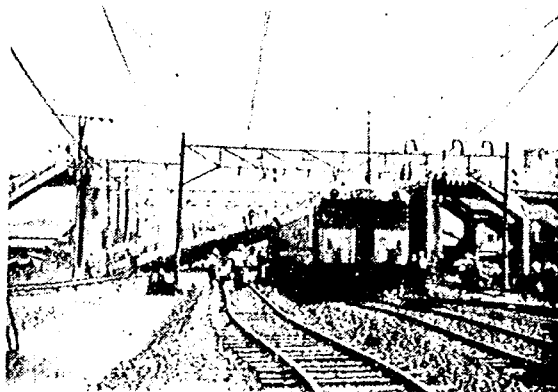


Photo. 2.54 Damage to the Higashi-Kosenkyo Bridge (General View of the Damage)

Due to the earthquake the bridge sustained severe damage at the main-span girder (photos. 2.54 to 2.57; Figs. 2.43 and 2.44). No significant damage was observed at the Niitsu-side abutment and the first to seventh piers from the Niitsu-side. The eighth and ninth piers tilted about 2% toward the Niitsu-side and 4% toward the Niigata-side, respectively. Furthermore, the ninth pier settled about 2.5 cm. The tenth to fifteenth piers and the Niigata-side abutment deformed horizontally about 5 cm, and settled considerably (about 40 cm at the maximum). After the earthquake all reinforced concrete piles at the ninth pier were pulled out for investigations of the amount of deformation during the earthquake. The maximum deformation was about 20 cm (see Photos. 2.56 and 2.57 and Fig. 2.44).

The eight spans on the Niitsu-side did not sustain any significant damage. The ninth span (main span) fell down on the railroad at its movable end on the Niigata side. This collapse was due to lengthening of the distance about 67 cm between both supports, which occurred by the large deformation of the eighth and ninth piers toward the opposite directions. The seven spans on the Niigata-side moved horizontally and vertically, and the asphalt pavement cracked. Furthermore, the failure of the bearing supports and the collision of the girders was observed.

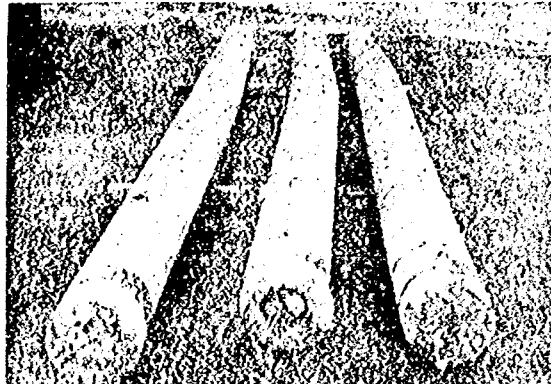


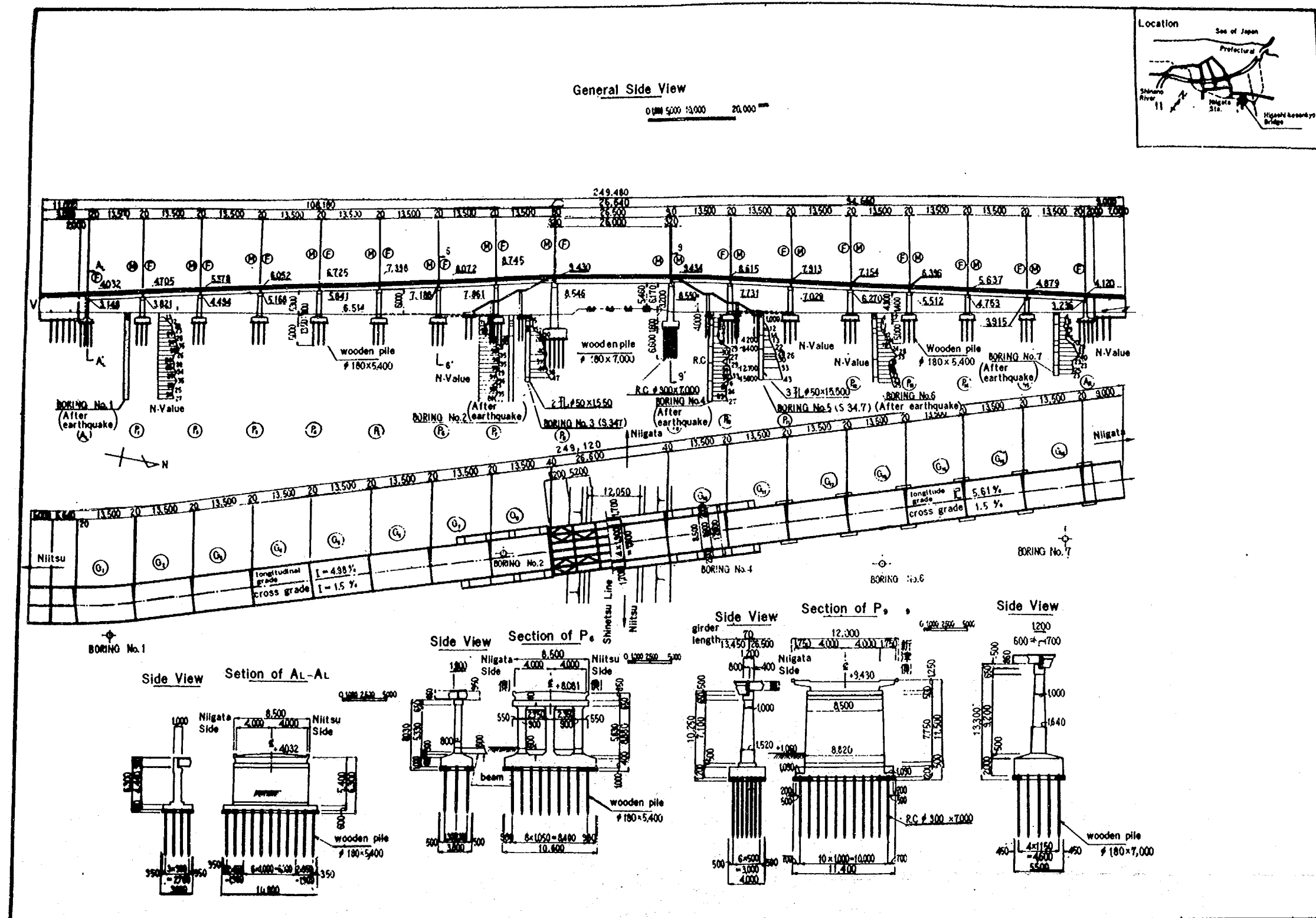
Photo. 2.56 Piles Pulled out from the Ninth Pier of the Higashi-Kosenkyo Bridge (1)



Photo. 2.55 The Pier Cap at the Ninth Pier of the Higashi-Kosenkyo Bridge



Photo. 2.57 Piles Pulled out from the Ninth Pier of the Higashi-Kosenkyo Bridge (2)



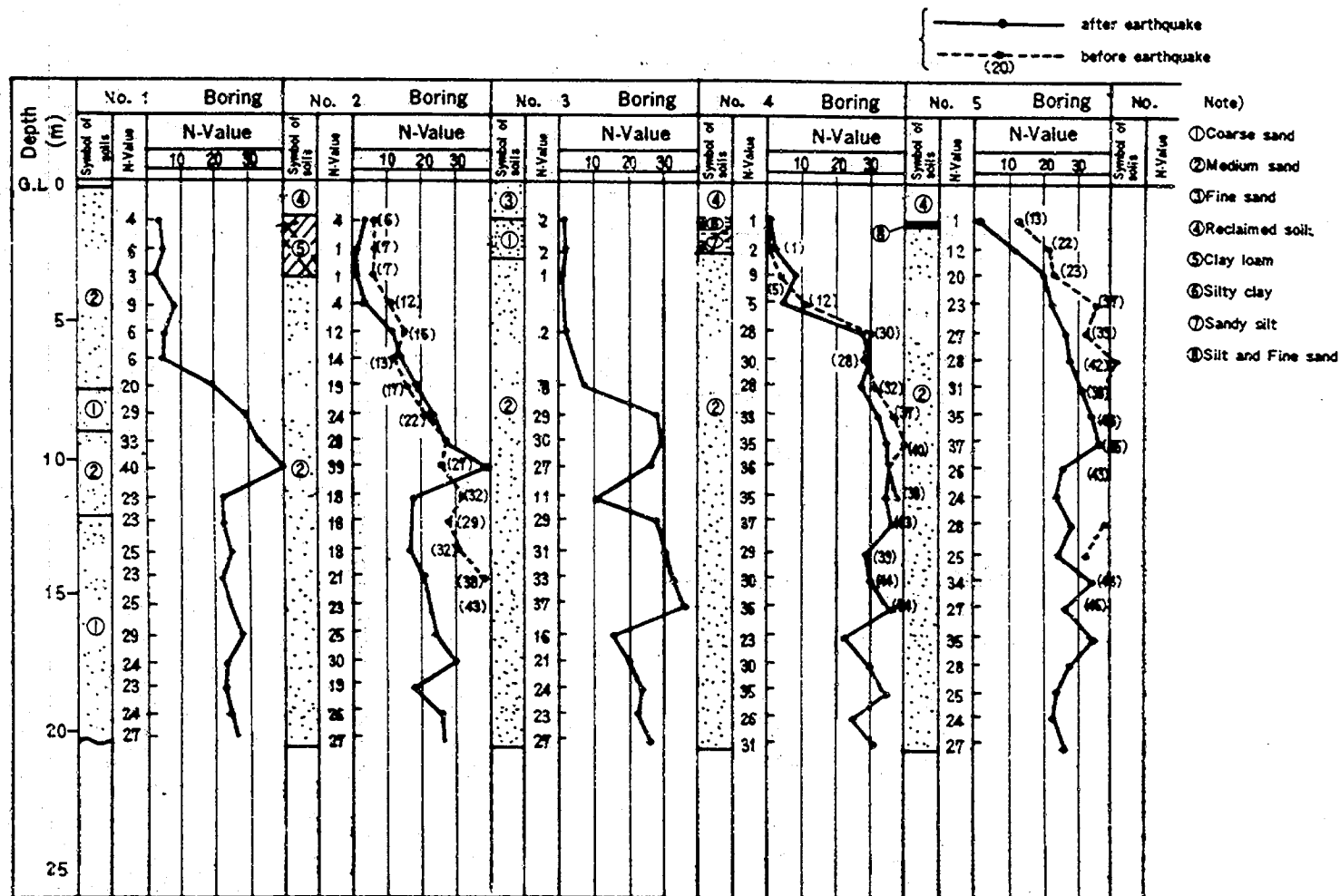


Fig. 2.41 Boring logs at the Higashi-Kosenkyo bridge

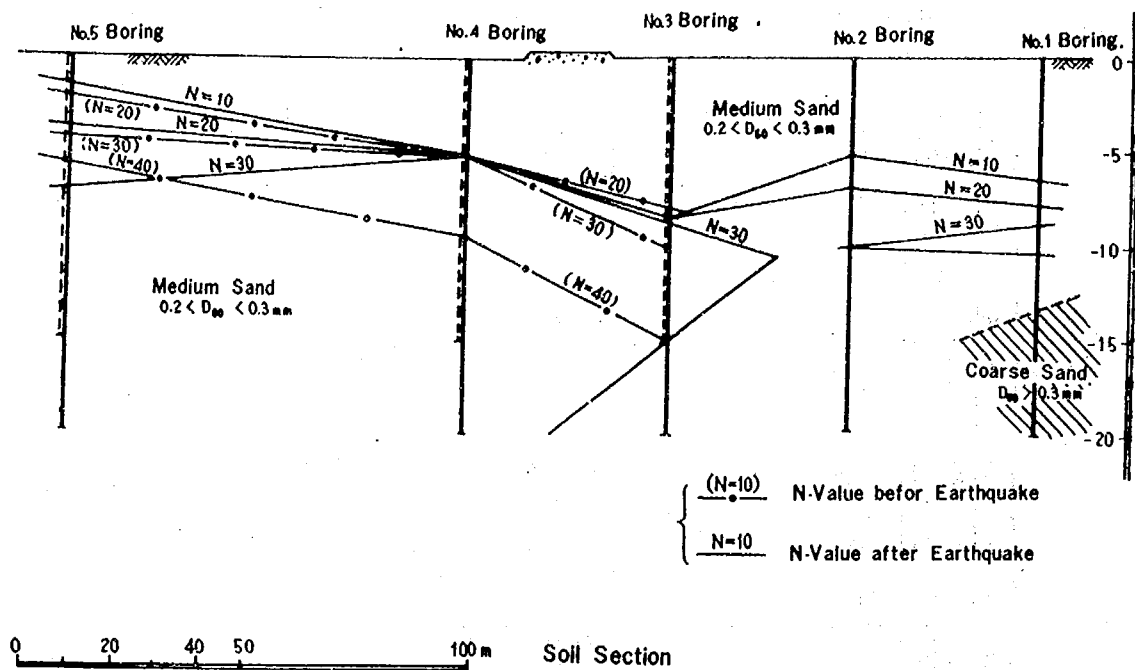


Fig. 2.42 Soil profile at the Higashi-Kosenkyo bridge

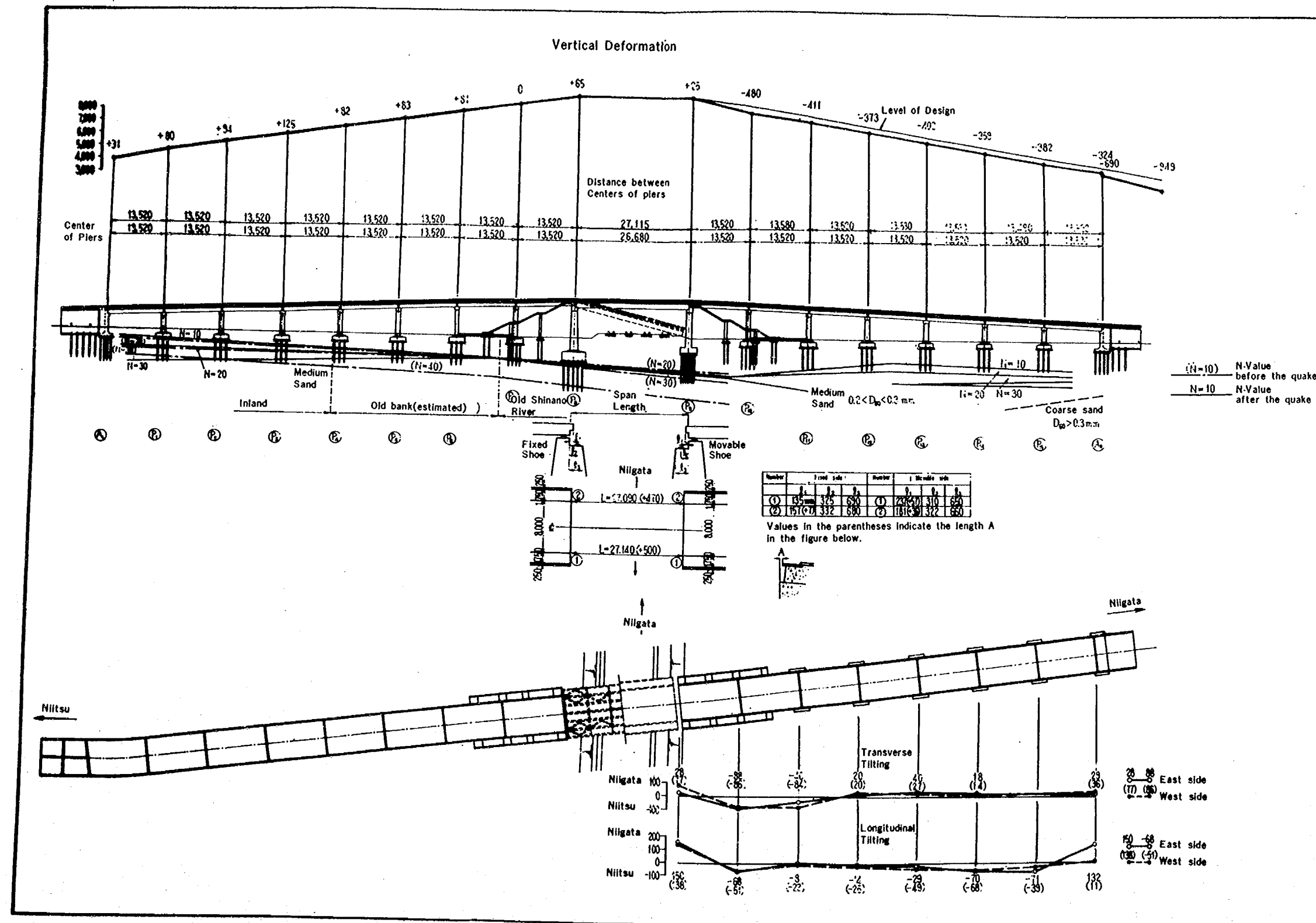


Fig. 2.43 Damage to the Higashi-Kosenkyo bridge

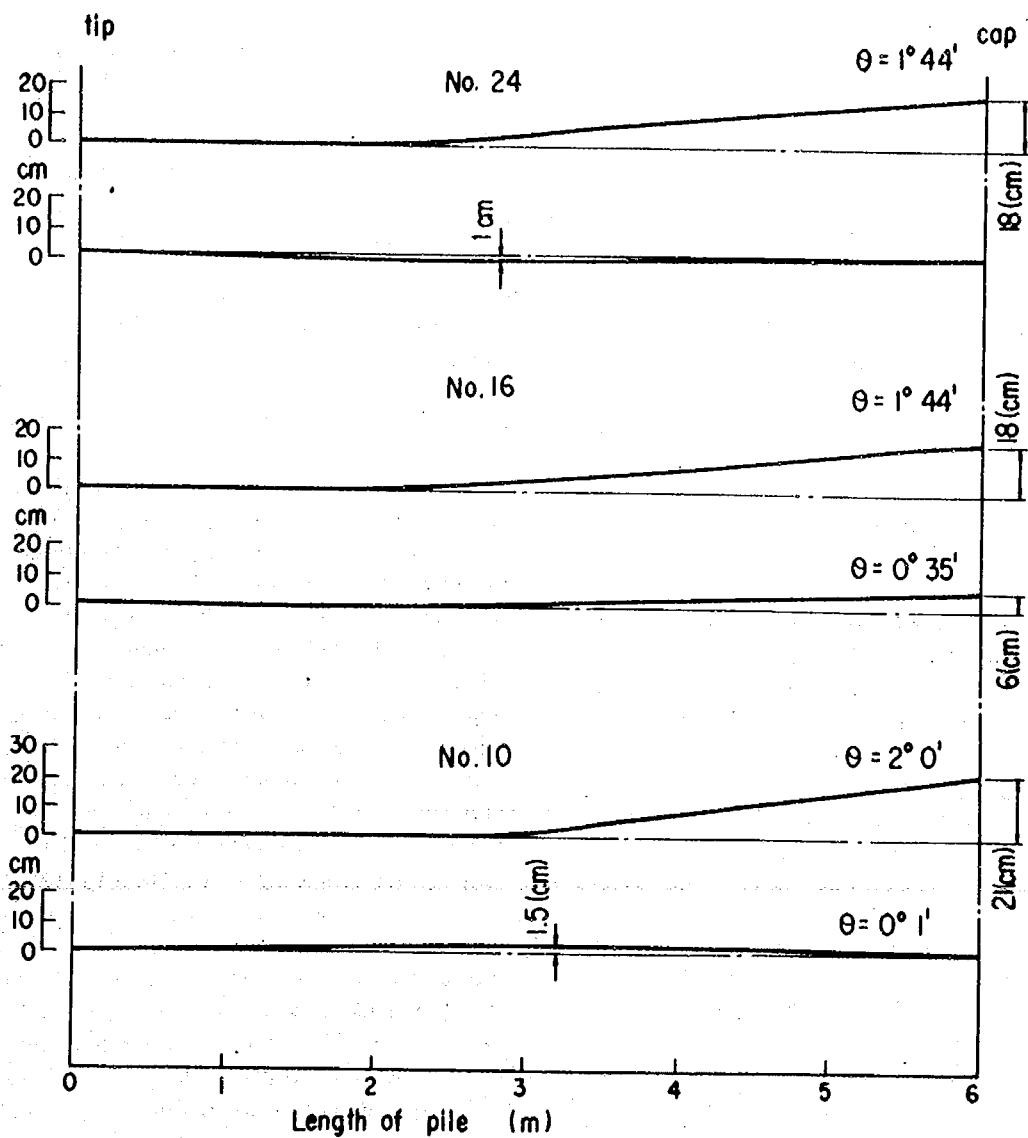


Fig. 2. 44 Deformations of some piles at the Higashi-Kosenkyo bridge

Fujimura Bridge

This bridge, completed in 1933, is on the Niigata prefectural road No. 53, located in Shiunji, Kita-Kambara County, Niigata Prefecture, about 28 km southeast from the epicenter, and is across the Ochibori River. The bridge is one of the bridges situated very close to the epicenter.

The abutments are of semi-gravity-type reinforced concrete structures with spread footings, and the three piers were of rigid frame reinforced concrete structures with caisson foundations. The superstructures, the total length of 33m and the width of 3.7m, are of 4-span T-shape reinforced concrete simple girders.

Due to the earthquake the bridge sustained serious damage. The abutments slid, settled, tilted, and cracked heavily. Main girders also cracked seriously. A complete repairing work was needed.

Matsuhama Bridge (Photos. 2.58, 2.59; Figs. 2.45 - 2.47)

This bridge was on the prefectural road No. 503, located in Matsuhama, Niigata, Niigata Prefecture, about 49 km south from the epicenter, and was across the Agano River. The ground conditions are of sandy soils, as shown in Fig. 2.46.

The left abutment is a gravity-type reinforced concrete structure with wooden pile foundation ($4 \times 7 = 28$ piles of the diameter of 20 cm and the length of 4.5 m). The right abutment is a gravity-type reinforced concrete structure with caisson foundation (the cross section of 3.4m by 10.6m and the depth of 7.5

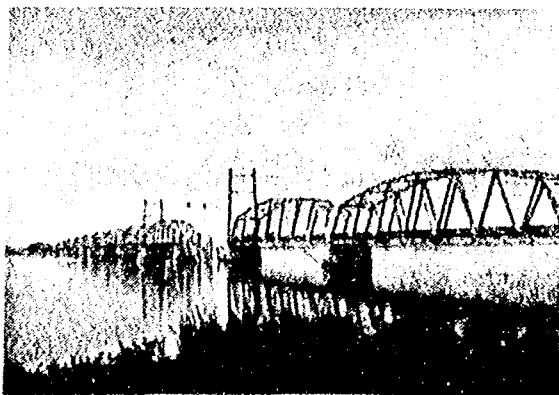


Photo. 2.58 Damage to the Matsuhama Bridge (Fall of a Truss)



Photo. 2.59 Damage to the Shoes on the Thirteenth Pier of the Matsuhama Bridge

m). The thirteen piers are of solid-slab-type reinforced concrete structures with caisson foundations (the cross section of 4.6m by 12.2m and the depth of 18m to 23m). The superstructures are of 14-span steel Warren simple trusses. The total length is 921m ($= 14 \times 65.8$ m), and the width is 6m. The supports of each span are a pin support and a roller support (Fig. 2.45).

At the time of the earthquake the bridge was still under construction. All the substructures had been completed. The first and second spans from the left-bank had been almost completed and the slab concrete for the two spans had been already placed. The tenth and eleventh spans were just at the stage of erection of the steel trusses, and no upper chord members were connected for the eleventh span. For the ten remaining spans the erection of trusses had been completed, but slab concrete was not placed yet.

Due to the earthquake the bridge sustained damage at both the substructures and the superstructures (Fig. 2.47). The right-bank abutment moved considerably toward the center of the river (about 1m), and several piers moved horizontally and vertically. The ninth pier moved 23cm toward the downstream direction. The movements of the other piers were less than several centimeters. The eleventh span fell down into the river at its roller end, due to the failure of the timbering (Photo. 2.58). The fall of the truss was mainly caused by the fact that upper chord members were not completed yet at the time of the earthquake. As for the other spans the moderate damage was observed, such as failure of anchor bolts (Photo. 2.59), failure of mortar near the supports, and slight damage to the expansion joints due to the movements of the girders.

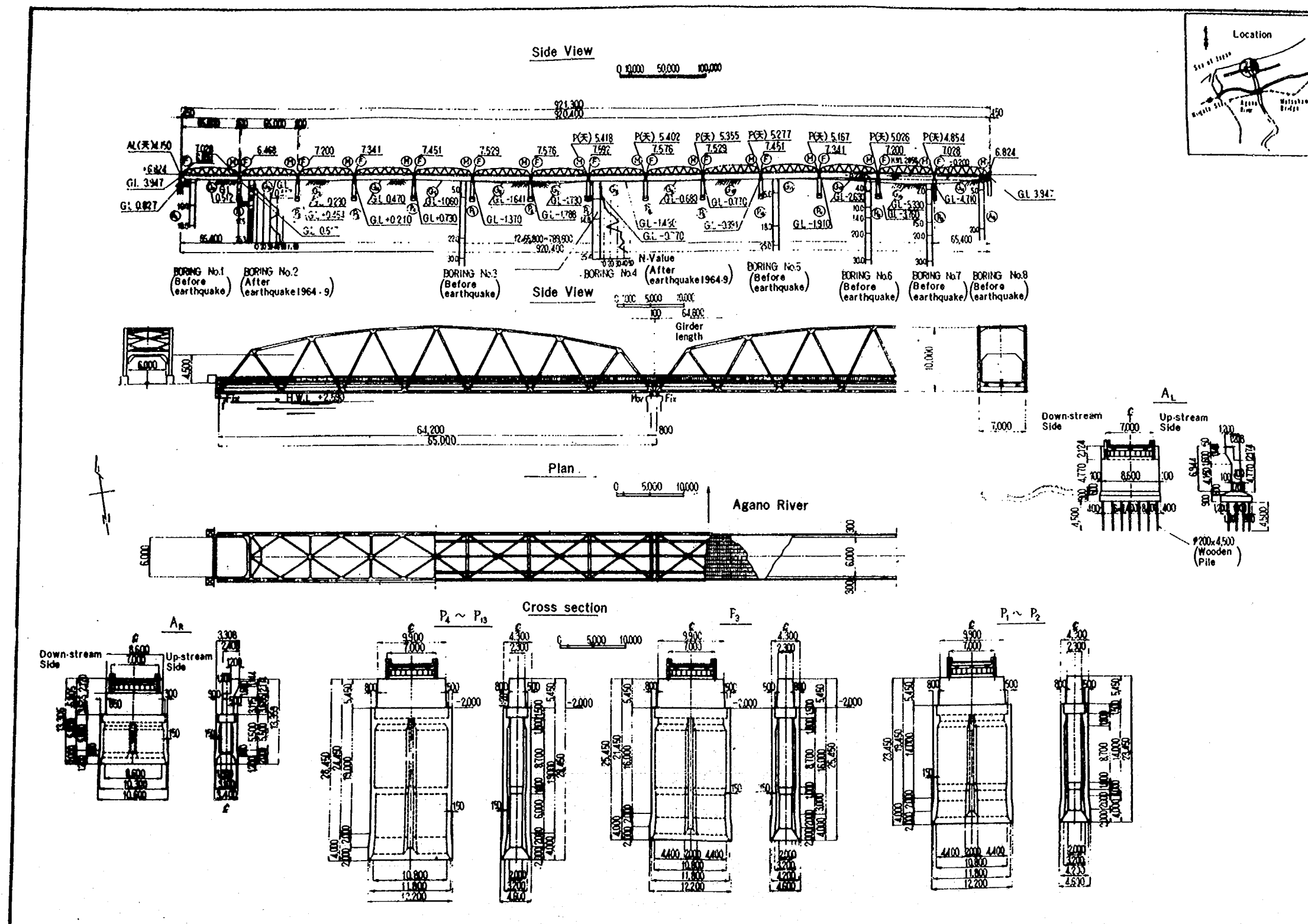


Fig. 2.45 General view of the Matsuhama bridge

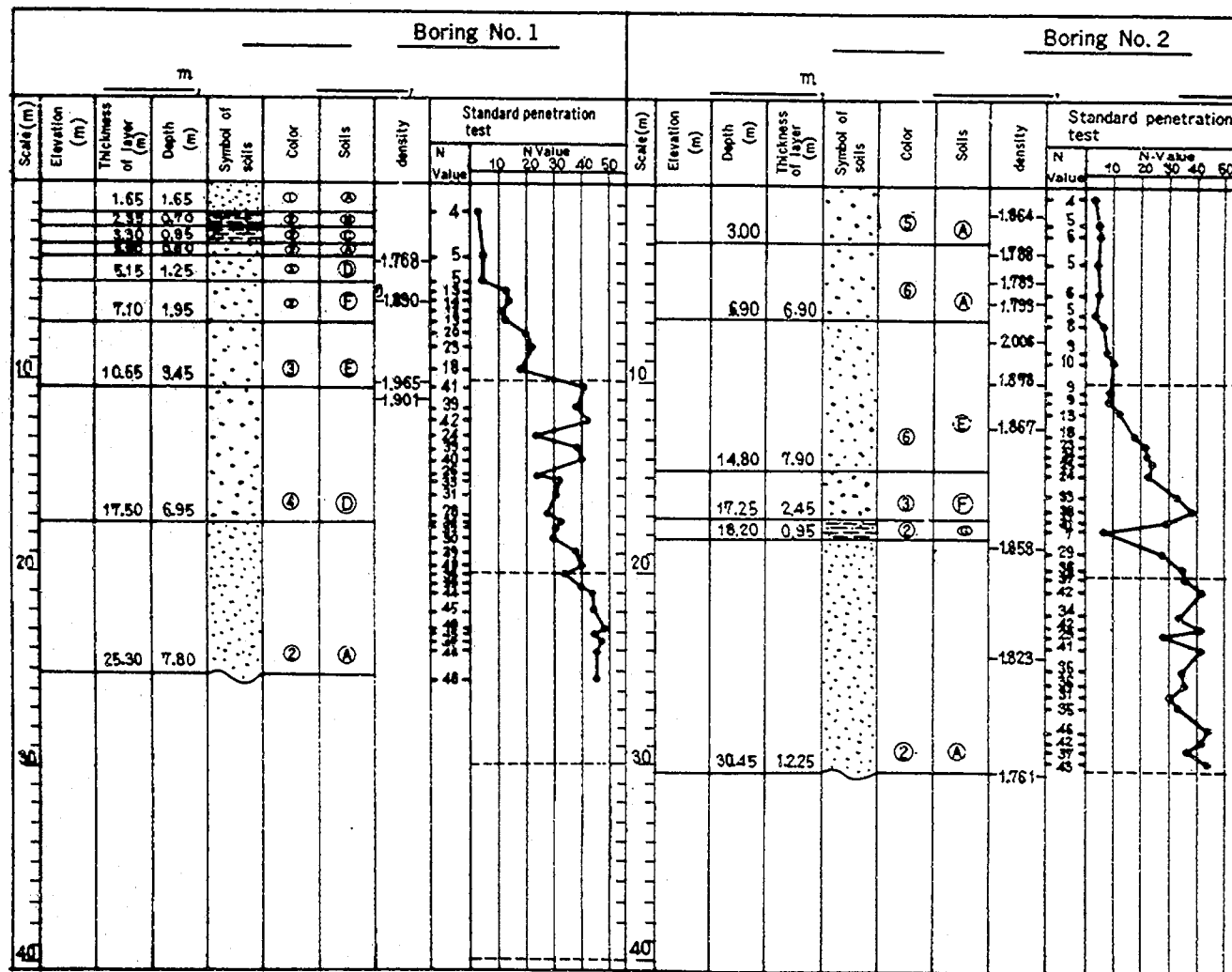
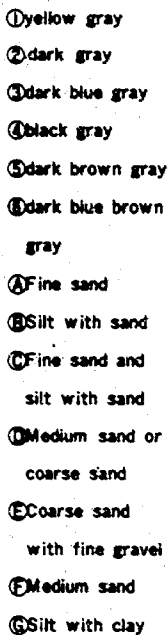


Fig. 2.46 Boring logs at the Matsushima bridge

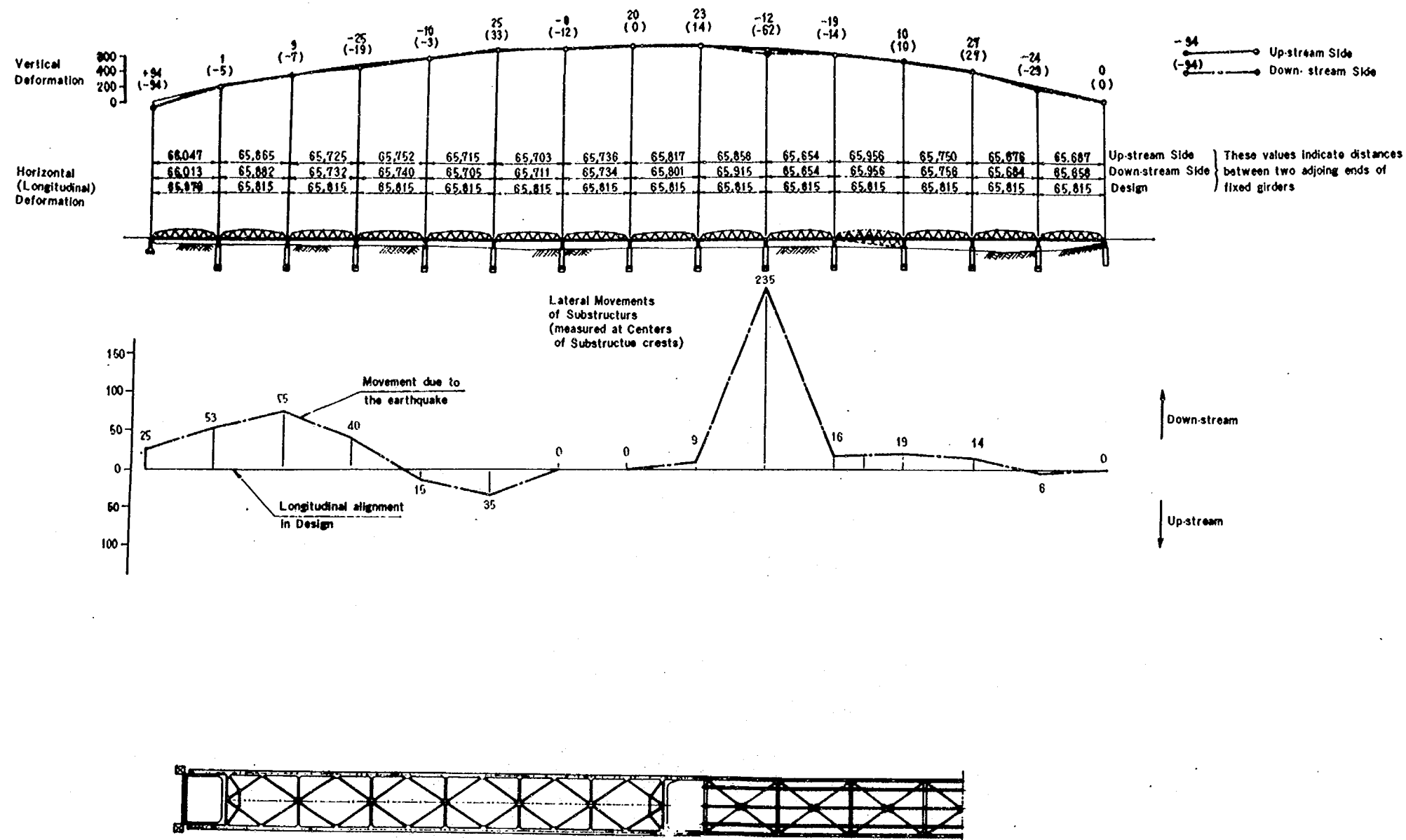


Fig. 2. 47 Damage to the Matsuhama bridge

Kosudo Bridge (Fig. 2.48 – 2.50)

The bridge, completed in December, 1963, is on the prefectural road No. 538, located in Kosudo, Nakakanbara County, Niigata Prefecture about 72 km south from the epicenter, and is across the Shinano River. The ground conditions are of clayey silts or silty clays (Fig. 2.49), which are finer than the soils in Niigata.

The abutments and piers are of solid-slab-type reinforced concrete structures with caisson foundations (the cross section of 4.6m by 12.2m and the depth of 13m). The superstructure is of 3-span steel simple Warren trusses, the total length is 189m (= 3 × 62.9m), and the width is 4m (Fig. 2.48).

Due to the earthquake the bridge sustained moderate damage (Fig. 2.50). All the substructures settled without horizontal movements. Amounts of settlements at the left-bank abutment, the first pier, the second pier and the right-bank abutment were 25 cm, 75 cm, 10 cm, and 7 cm, respectively. Slight cracks were observed at the right-bank abutment. Although the superstructures sustained slight damage at the railing, the expansion joints, and the supports due to the settlements of the substructures, no significant structural damage was observed at the superstructures themselves.

Summary

The Niigata Earthquake can be characterized by the fact that extensive field surveys were made on structural damage. Experimental and analytical investigations were also conducted to clarify the seismic damage to various structures and ground soils. In dynamic analyses of some highway bridges, the acceleration records obtained during the earthquake were employed as the inputs. The following is a summary of the results from the extensive surveys on the damage to highway bridges.

Table 2.6 indicates the number of bridges damaged in the influenced prefectures. The total loss of all the highway bridges was 1.47 billion yen. On the other hand, the loss of the highways except bridge structures was 2.34 billion yen. The both amounts are at the value at that time. Most of the loss was occurred in Niigata Prefecture (mostly in or near Niigata City).

A detailed survey was made for all bridges (including undamaged ones) located in or near

Niigata City. As shown in Table 2.7, there were 86 bridges within an area about 60 km from the center of the city. Fifty two bridges (60%) were damaged, and among them seven bridges were severely damaged. However, 34 bridges (40%) did not sustain any significant damage to be repaired. As for the seven bridges which were severely damaged, all of them sustained damage to substructures, and four bridges to superstructures. It was noted that most damages to superstructures were caused by failure or large deformation of the substructures.

Table 2.8 indicates damage percentages for individual structural portions of highway bridges. In the table numbers of spans, abutments, and piers are counted, respectively, by classifying into several types of structures. From the extensive survey it is concluded that:

- a) Although the damage to bridge structures was observed in Akita, Fukushima, Niigata and Yamagata prefectures, severe damage was concentrated in the area near the mouths of the Shinano River and the Agano River in or near Niigata City, where ground conditions were very bad.
- b) The degree of the damage to bridges was roughly proportional to JMA seismic intensities. In various areas with the same intensity, ground conditions affected considerably degree of damage.
- c) There was no significant discrepancy among damage ratios for several types of bridge girders except wooden girders which sustained severe damage. As for substructures, abutments sustained much damage than piers. It is believed that the earth fills behind the abutments had significant effects on the stability of abutments. There was no significant discrepancy among damage ratios for substructures with several types of foundations.
- d) Since loose saturated sandy soils near the ground surface liquefied and the bearing powers of the grounds decreased considerably, the substructures sustained severe damage such as tilting, settlement or sliding.
- e) Although the damage due to liquefaction of sandy soils prevailed in Niigata City, only minor damage was observed at the bridges which had deeper foundations rested on dense sandy layers with N-value of 25 or more by the standard penetration tests.
- f) As for the damage to superstructures, failure

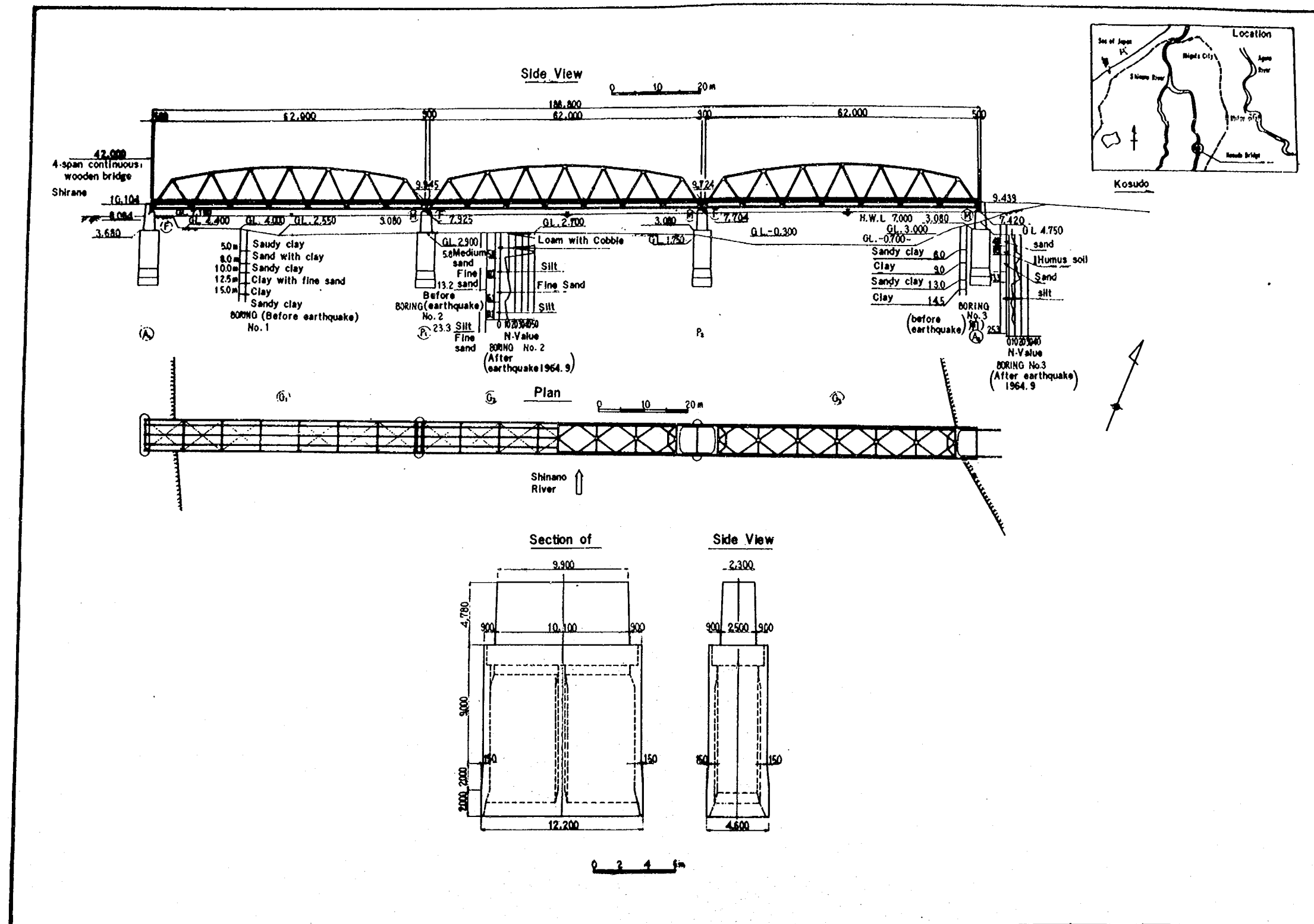


Fig. 2.48 General view of the Kosudo bridge

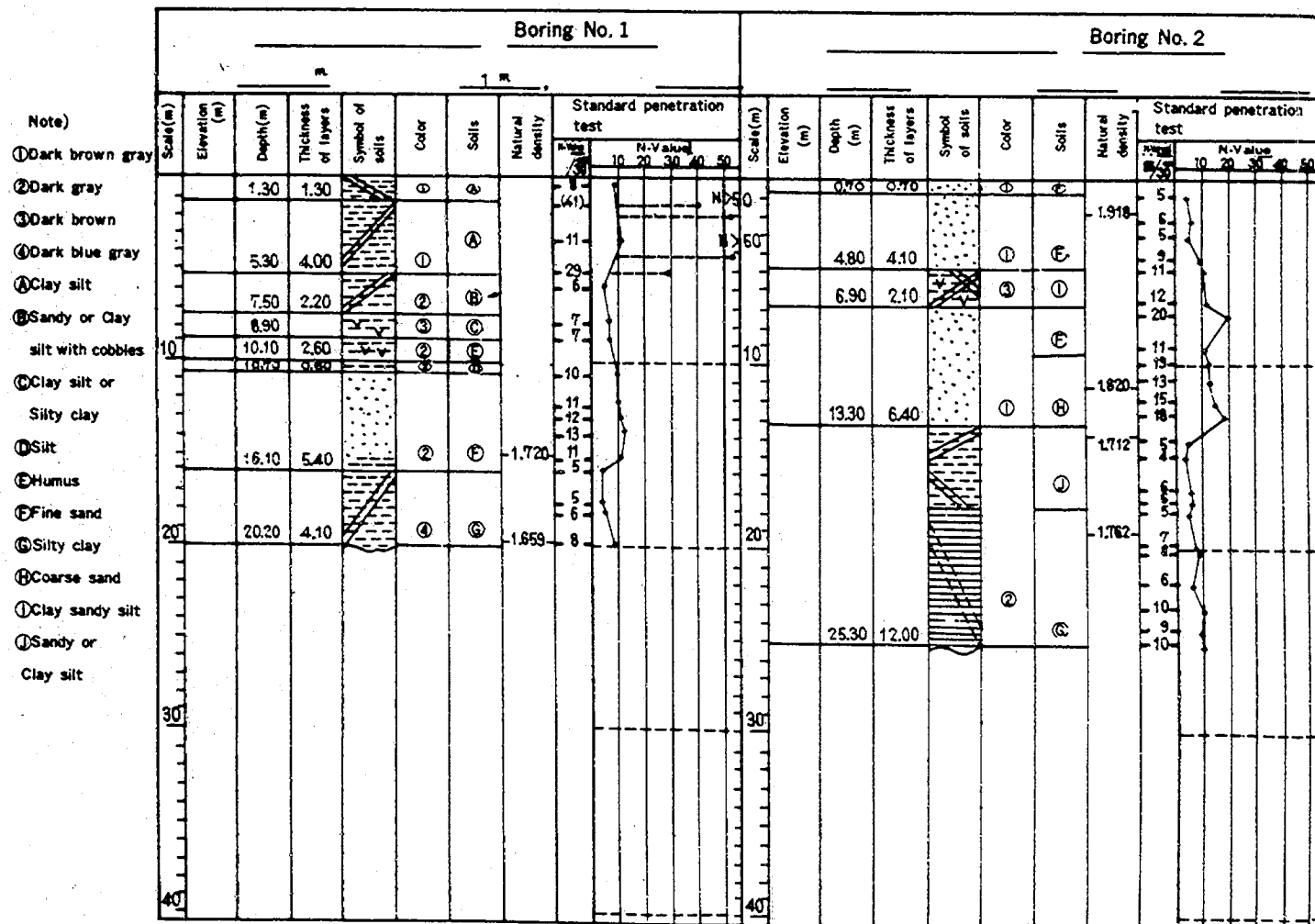


Fig. 2.49 Boring logs at the Kosudo bridge

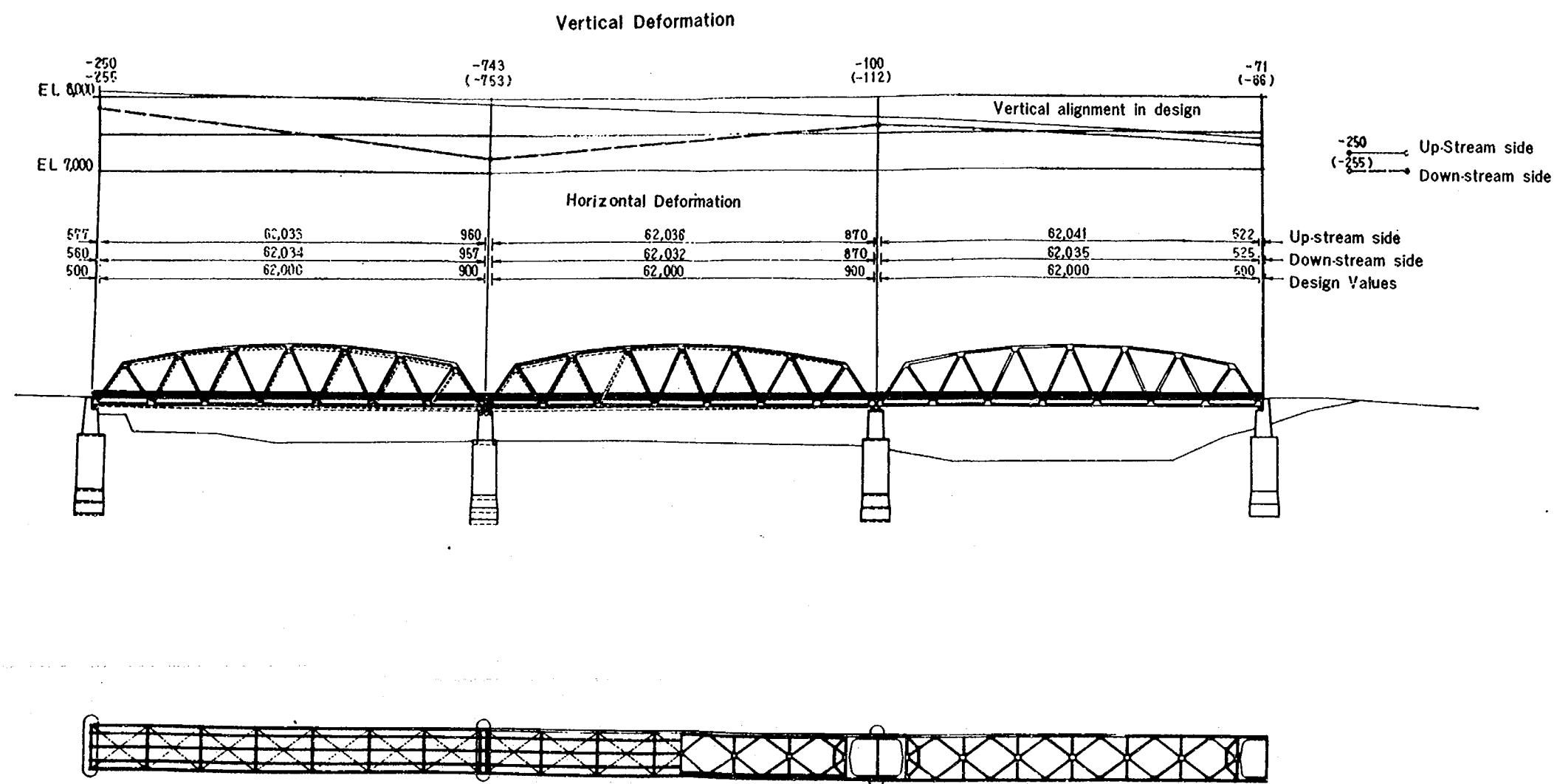


Fig. 2. 50 Damage to the Kosudo bridge

Table 2.6 Statistics on Damage to Highway Bridges (except Wooden Bridges) due to the Niigata Earthquake of 1964

Prefectures	Number of Damaged Bridges	Number of Severely Damaged Bridges	Number of Fallen Bridges	Approximate Epicentral Distance
Akita	7	0	0	140 - 160 km
Fukushima	5	0	0	120 - 150 km
Niigata	74	8	3	30 - 90 km
Yamagata	12	0	0	60 - 100 km
Total	98	8	3	

Table 2.7 Numbers of Damaged Bridges with various kinds of foundations due to the Niigata Earthquake of 1964 - For Highway Bridges within 60km from the center of Niigata City -

Varieties of Foundations	Total Number of Bridges Surveyed	Number of Bridges Damaged (Including Slight Damage)	Number of Bridges Damaged Severely			
			At Substructure	At Superstructure	In view of Traffic Capacity	Overall Evaluation
Bridges with Spread Footings	18.5	10	1.5	0.5	0.5	0.5
Bridges with Caisson Foundations	21	13	1	1	0.5	0.5
Bridges with Pile Foundations	46.5	29	4.5	2.5	4	2
Total	86	52	7	4	5	3

Note) When there are two kinds of foundations for one bridge, the number of the bridge was accounted such as half for one foundation and half for the other foundation.

Table 2.8 Damage Percentages of Individual Portions of Highway Bridges within 60km from the Center of Niigata City

Structural classification	Type of structures		Number of structures Surveyed	Damaged structures	
				Number of structures	Percentages
Superstructures	Steel Girders		168 spans	19 spans	11.3 %
	Reinforced Concrete Girders		222 spans	33 spans	14.9 %
	Prestressed Concrete Girders		132 spans	11 spans	8.3 %
	Wooden Girders		88 spans	8 spans	100 %
	Total		530 spans	71 spans	13.4 %
Substructures	Abutments	With Spread Footings	24	4	16.7 %
		With Pile Foundations	99	19	19.2 %
		With Caisson Foundations	29	7	24.0 %
		Sub-Total	152	30	19.7 %
	Piers	With Spread Footings	40	0	0 %
		With Pile Foundations	214	21	9.8 %
		With Caisson Foundations	180	15	8.3 %
		Sub-Total	444	36	8.1 %
	Total		596	66	11.1 %

of bearing supports, expansion joints and handrails were observed in addition to the fall of the girders.

- g) It seemed that the damage to superstructures was caused mostly by failure or large deformation of substructures.

The most important lessons from the damage to bridge structures due to the Niigata Earthquake seem to be the following two.

- 1) Liquefaction of saturated sandy soils affects considerably the stability of bridge structures.
- 2) A particular attention should be given to design details such as methods for preventing the fall of girders and structural details of connections.

2.8 Ebino Earthquake of 1968⁴³⁾

An earthquake attacked Ebino, Nishimoro-

kata County, Miyazaki Prefecture, southern part of Kyushu Island on February 21, 1968, accounting magnitude of 6.1 on the Richter scale (Fig. 2.1). The earthquake was accompanied with one before shock of magnitude of 5.6 which had broken off about two hours prior to the main shock and with many aftershocks including three major ones, one (magnitude was 5.5) on February 22 and two on March 25. The outline of the main shock is as follows⁵¹⁾:

Name of Earthquake	Ebino Earthquake
Date of Occurrence	February 21, 1968
Epicenter	Ebino, Miyazaki Prefecture (32.0°N, 130.8°E)
Depth of Hypocenter	Very shallow
Magnitude on the Richter Scale	6.1

Within a limited area (about 10 km in diameter) in Ebino, Miyazaki Prefecture and Yoshimatsu, Kagoshima Prefecture, where the

grounds are generally of deposit layers of volcanic sandy soils, several landslides occurred at steep slopes, many civil engineering structures sustained moderate damage, and hundreds of residential wooden houses sustained severe damage.

As for highway bridge structures, some ten bridges sustained moderate damage due to the earthquakes. The Kamezawa Bridge sustained the severest damage among these bridges.

Kamezawa Bridge (Photo. 2.60)

The bridge, completed in 1964, is on a Ebino municipal road, located in Ebino, Miyazaki Prefecture, and across the Sendai River.

The abutments are of gravity-type reinforced concrete structures with reinforced concrete caisson foundations, and four piers are of solid-slab-type reinforced concrete structures with wooden pile foundations. The superstructures are of 5-span prestressed concrete simple girders. The total length is 131 m, and the width is 3.5 m.

Due to the earthquake the left-bank abutment suffered a heavy lateral crack near the ground surface and tilted toward the backfill (Photo. 2.60). The fourth pier suffered slight breakage at the pier cap near the support. The end of the concrete girder of the first span failed severely on the left-bank abutment, and the support also failed (Photo. 2.60). The earth fills for approach road to the bridge settled about 20 cm at the both sides.



Photo. 2.60 Damage to the Kamezawa Bridge (Failure at the Abutment and at the PC Girder)

Kamimasaki Bridge (Photo. 2.61)

This bridge, completed in 1965, is on another Ebino municipal road, and across the Sendai River.

The abutments are of gravity-type reinforced concrete structures with pile foundations, and four piers are of solid-slab-type reinforced concrete structures with caisson foundations. The superstructures are of 5-span prestressed concrete simple girders. The total length is 140.5m, and the width is 3.6m.

Due to the earthquake the second girder from the left-bank moved about 20 cm toward the downstream and about 5 cm toward the right-bank. Due to the aftershocks of March 25, the movement toward the downstream reached about 26 cm (Photo. 2.61). The concrete pier caps near the supports failed slightly at the first and second piers. The approach roads also settled.

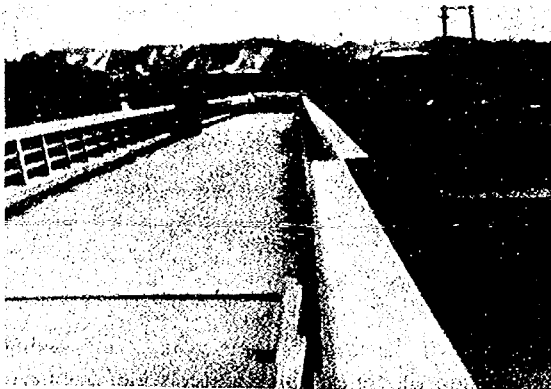


Photo. 2.61 Damage to the Kamimasaki Bridge (Movement of the PC Girder)

Ikejima Bridge (Photo. 2.62)

This bridge, completed in 1964, is on an Ebino municipal road, and across the Ikejima River. The ground conditions are of deposit layers of volcanic sandy soils. The abutments and two piers are of solid-slab-type reinforced concrete structures with spread footings and wooden pile foundations. The superstructures are of 3-span steel H-shape simple girders. The total span is 49.6 m, and the width is 6 m.

The first pier from the left-bank settled about 25 cm due to the earthquake (Photo. 2.62). To the other portions of the bridge, however, no significant damage was observed. On the river bed near the bridge the ground surface cracked, settled considerably, and some sandy materials from the deeper layers were boiled out through the cracks. It is believed from the facts that liquefaction took place at the site.

Several bridges other than the above bridges suffered the failures at the bearing supports, the movements of the girders, and the settlements (about 5 to 20 cm) of the banks for the approach roads. A reinforced concrete Gerber bridge (Kakuto Bridge) sustained very slight damage at the hinge.



Photo. 2.62 Damage to the Ikejima Bridge (Settlement of a Pier (25 cm) due to Liquefaction)

2.9 Tokachi-oki Earthquake of 1968⁴⁶⁾-50)

The Tokachi-oki Earthquake, measuring magnitude of 7.9 on the Richter scale, occurred off Tokachi on May 16, 1968. The epicenter was under the bottom of the sea about 140 km south from Erimo Cape of Hokkaido, and about 170 km east from the northeast coast of Honshu Island (Fig. 2.1). The outline of the earthquake is as follows⁵¹⁾:

Name of Earthquake	Tokachi-oki Earthquake of 1968
Date of Occurrence	May 16, 1968
Epicenter	Off Tokachi (40.7°N 143.6°E)
Depth of Hypocenter	About 20 km
Magnitude on the Richter Scale	7.9

Various engineering structures such as buildings, highways, river and coastal embankments, etc. sustained severe damage. However, no fatal damage was caused to any bridge structures, and only minor damage was observed at a large number of bridges in Hokkaido and Aomori prefectures (see Table 2.9). The amount of loss for bridge structures was surprisingly low, when compared with those for other structures. The

followings are brief descriptions on the damage features of the highway bridge structures, which were quite similar to the previous experiences.

As for the substructures, pile bent piers sustained moderate damage due to the lack of stiffness (Photo. 2.63), and a pier settled about 20 cm due to the lack of bearing powers of the foundation which had been scoured by the river stream (Photo. 2.64). Several abutments were pushed out toward the center of the river (Photo. 2.65), and some abutments had cracks at the concrete near the bearing supports (Photos. 2.66 and 2.67).

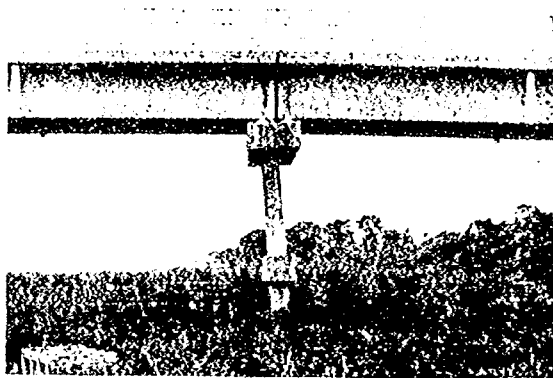


Photo. 2.63 Damage to the Kaimei Bridge (Tilting of the Pile-Bent Pier)

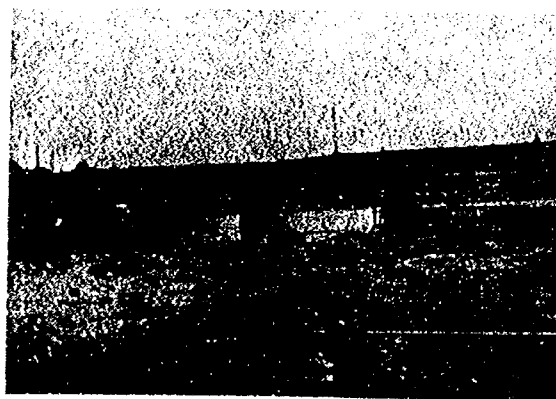


Photo. 2.64 Settlement of the Pier at the Shiriuchi Bridge

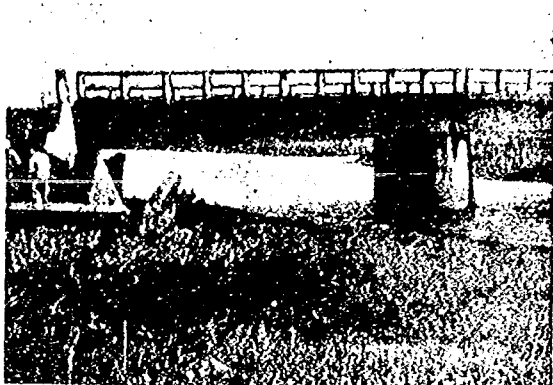


Photo. 2.65 Damage to the Abutment at the Noushi Bridge

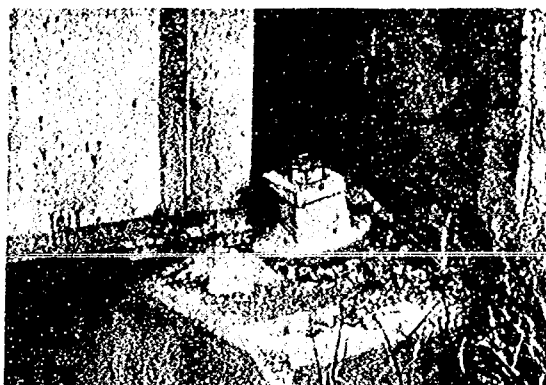


Photo. 2.68 Failure of the Support at the Kaimei Bridge



Photo. 2.66 Failure of the Abutment at the Deto Bridge

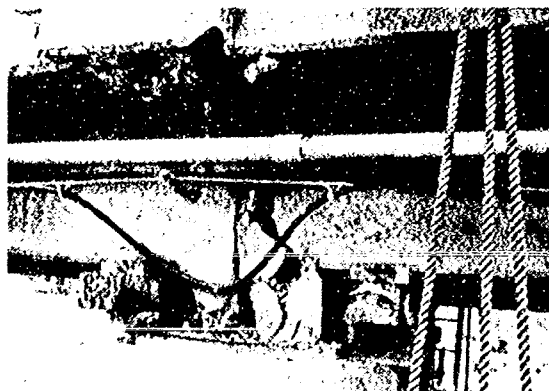


Photo. 2.69 Failure of the Concrete Girders at the Komoto Bridge

As for the superstructures, the girders moved slightly and the bearing supports sustained moderate damage including the failure of the concrete nearby (Photo. 2.68).

The ends of the girders at a reinforced concrete bridge collided and crushed (Photo. 2.69). Settlements of the banks for approach roads were often observed (by a few decimeters), which seemed to be caused by the compaction of the earth fills during the earthquake.

2.10 General Features of Seismic Damage to Bridge Structures

Seismic damage to bridge structures generally observed at abutments, piers, girders and supports which are consisting of a complete bridge structure (Fig. 2.51). The mechanisms of the bridge damage can be classified into three categories²⁹⁾.

1) Due to the Weakness of Supports: Any

Table 2.9 Statistics on Damage to Highways and Highway Bridges due to the Tokachi-oki Earthquake of 1968

Administrative Agency		Highways		Highway Bridges		Total		Remarks
		Number of Sites	Loss	Number of Bridges	Loss	Number	Loss	
Ministry of Construction		111	Thousand Yen 969,604	—	—	111	Thousand Yen 969,604	Mostly to National Highways in Aomori Prefecture
Prefectural Governments	Hokkaido	63	319,359	30	Thousand Yen 206,176	93	525,535	
	Aomori	818	1,167,060	64	187,770	882	1,354,830	
	Iwate	41	34,178	3	19,000	44	53,178	
	Miyagi	—	—	3	5,100	3	5,100	
	Akita	4	4,600	1	3,000	5	7,600	
Total		1,037	2,494,801	101	421,046	1,138	2,915,847	

Note) Amounts of loss were estimated at the value as of June, 1968.

portions of a bridge structure will be exerted to move in any directions during earthquakes. If the supports are not sufficient to undergo the differential movements between the superstructures and the substructures, they may fail. If the failure of a support occurs, the superstructures will move extensively relative to the cap of the substructures. Occasionally the superstructures may dislodge from the substructures and fall down into the river or on the underpass. Both the superstructures and the substructures will suffer considerable damage by the strong shock of the fall of the superstructures.

2) Due to the Weakness of Substructures: If a substructure is not sufficient to resist its own inertia force and seismic forces of the girders transmitted through the supports, it may deform extensively, and sometimes fail or even overturn completely. The superstructures supported by the substructure will sustain considerable damage caused by the lack of the resistance of the substructure.

3) Due to the Weakness of Surrounding Soils: If the soils surrounding a substructure are vulnerable to earthquake excitations, the substructure may settle vertically or move horizontally during earthquakes. If a drastic movement of substructure occurs, the superstructures supported by the substructure can not keep their initial positions and may sustain considerable damage, and sometimes even fall down.

As a result of the behavior described above the following failures are often observed at

individual portions of a bridge structure.

1. Substructures: Tilting, settlement, sliding, cracks, overturning.
2. Superstructures: Movement, crack or failure, fall of girders.
3. Supports: Failure of joints, cut-off or pull-out of anchor bolts.
4. Appurtenant Structures: Settlement of approach roads (especially in case of banking), settlement and sliding of wing walls, separation of wing walls from abutments, and failure of parapet walls.

In view of the experiences of seismic damage due to the recent earthquakes, it seems reasonable that bridge structures which were properly designed and constructed in accordance with the bridge specifications (Specifications for Design of Steel Highway Bridges, by Japan Road Association published in 1956 and revised in 1964) can resist major earthquakes without sustaining fatal damage, provided that they were designed with a special attention to (1) geological consideration to evade catastrophic ground disasters, (2) decrease in bearing capacities of the subsoils during earthquakes (such as liquefaction of sandy soils) and (3) the design details for preventing the fall of girders and for avoiding severe damage caused by the failure at joints.

On the basis of the viewpoint, the new specifications for seismic design of highway bridges were stipulated by the JRA in January, 1971, which will be mentioned in Chapter 4 and Appendices B and C.

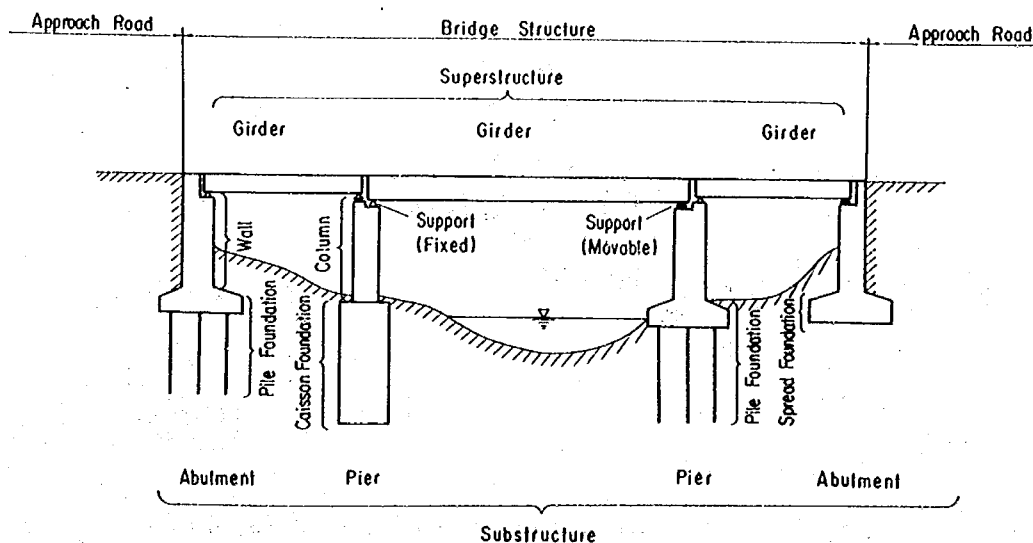


Fig. 2.51 Schematic Sketch of a Typical Highway Bridge

CHAPTER 3 RESEARCH ACTIVITIES ON SEISMIC EFFECTS ON BRIDGE STRUCTURES

This chapter deals with the research activities primarily in Japan which are related to earthquake-resistant design of bridge structures.

One of the most important aspects in the design of a structure for seismic forces is to evaluate the seismic activity (seismicity) in an area where the structure will be constructed

(3.1). In addition to the determination of the basic magnitude of seismic forces for design, it is also very important to obtain the characteristics of seismic forces in consideration of the dynamic properties of the structure and the ground (3.2).

For earthquake resistant design of a substructure the dynamic effects of the surrounding soil and water on the substructure should be properly taken into account (3.3 – 3.5). Moreover, for better understanding of the behavior of structures during earthquakes it is necessary to know the dynamic properties of the structures such as damping capacities, frequency character-

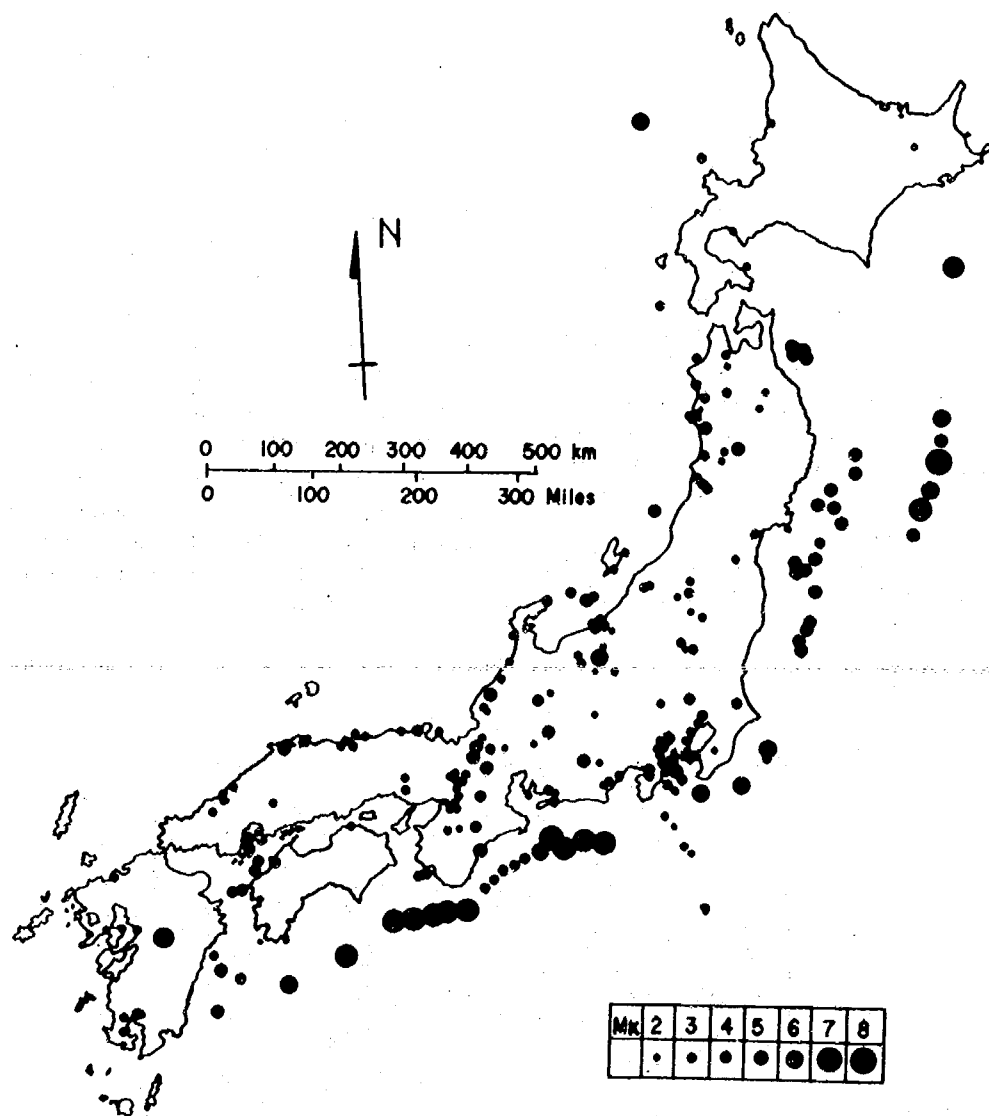


Fig. 3.1 Distribution of epicenter and magnitude of major historical earthquakes in Japan (after H. Kawasumi (80))

istics, and mode shapes (3.6), and also dynamic responses during the actual strong earthquakes (3.7). Furthermore, the dynamic analyses (3.8) and the model experimental studies (3.9) are essential to clarify dynamic behavior of bridges during earthquakes.

3.1. Seismicity in Japan

H. Kawasumi⁸⁰⁾ performed a study on seismic risk maps, which are obtained from the available history of the seismic activities in Japan. He listed up 342 major earthquakes

occurred in or near Japan for 1351 years between 599 and 1945 A.D., determined the epicenters and the magnitudes as shown in Fig. 3.1, and evaluated the maximum accelerations expected for certain periods of years throughout the country. The results will be well represented by Fig. 3.2. The three maps in the figure indicate the contours of expectation values of the maximum accelerations expected for (A) 75, (B) 100, and (C) 200 years, respectively.

Although the study was conducted primarily for establishing a seismic regulation for building structures, the recent seismic regulations for

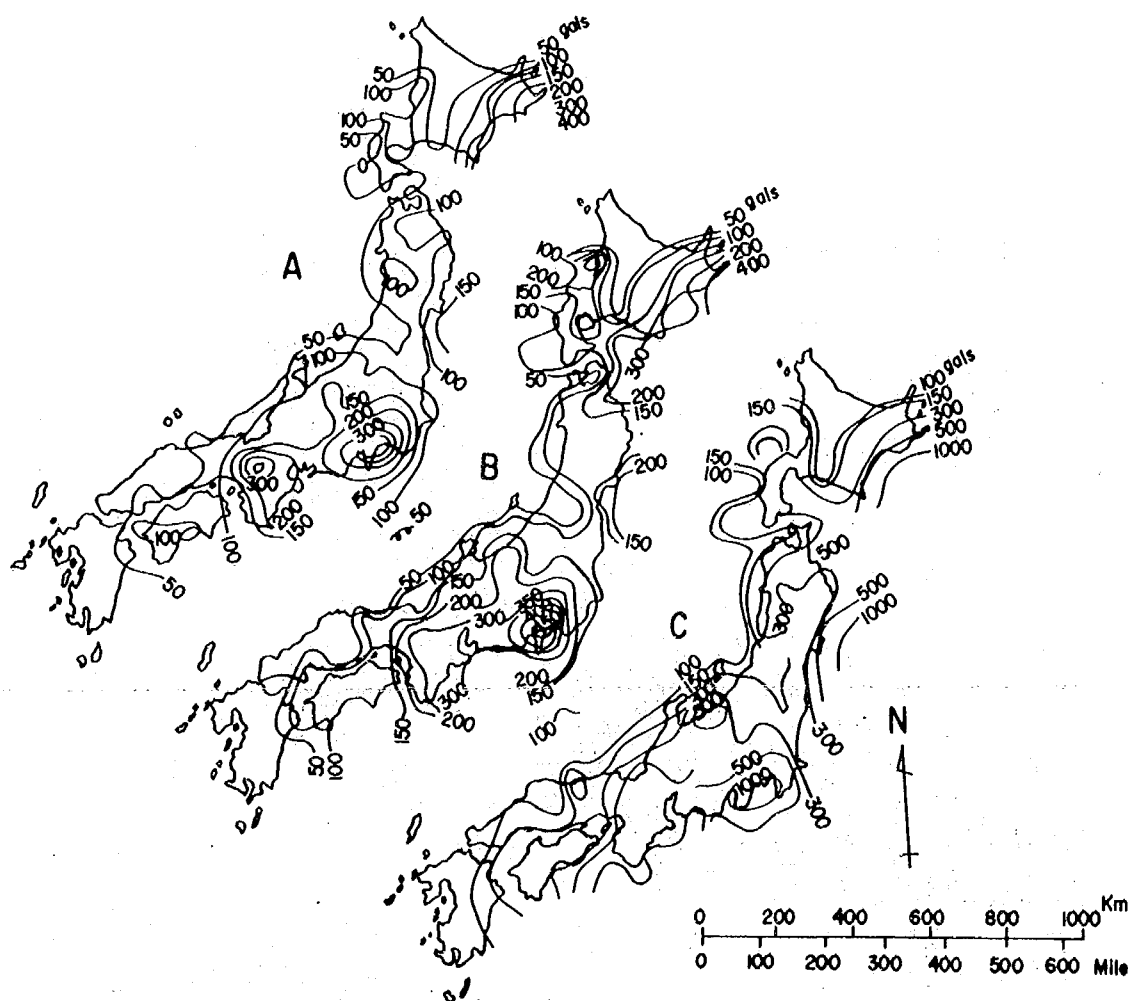


Fig. 3.2 Contours of maximum earthquake accelerations expected in Japan within periods of (A) 75, (B) 100, and (C) 200 years (after H. Kawasumi (80))

most of engineering structures are made up on the basis of the seismic maps.

S. Okamoto⁸¹⁾ attempted to make charts of seismic activity in Japan, in which maximum ground displacements during earthquakes occurred since 1605 are shown graphically in the chronological order at 236 points in the country. The maximum ground displacement at a certain point during an earthquake was determined as a function of the magnitude and the epicentral

distance.

H. Goto and H. Kameda^{82), 83)} conducted a statistical study on the maximum ground motion during strong earthquakes. Figs. 3.3 and 3.4 are the results of the analyses on the maximum ground accelerations and the maximum ground velocities expected for 75 years.

T. Okubo and T. Terashima⁸⁵⁾ proposed new zoning maps of earthquake risk on the basis of the Kawasumi's maps and the distribution of

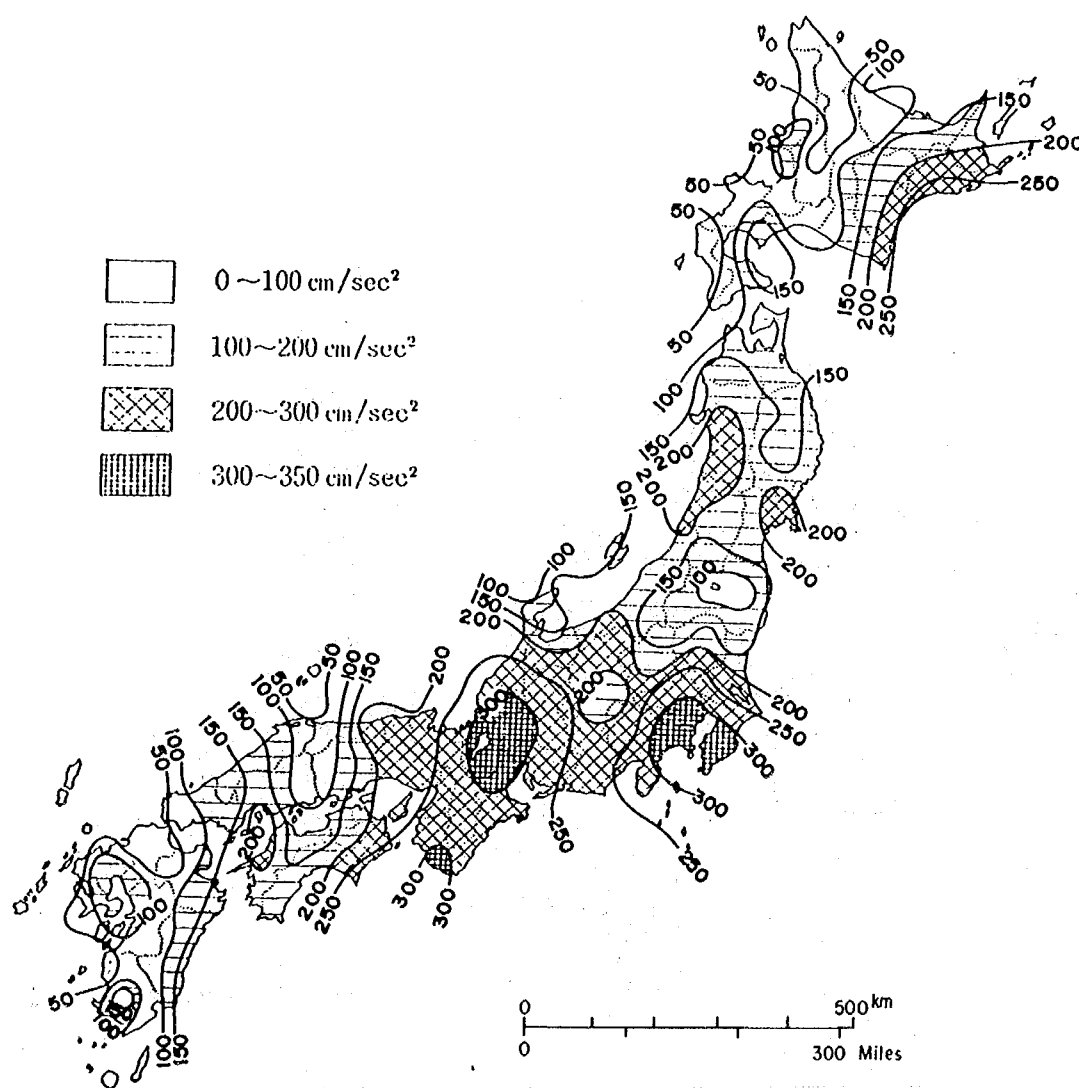


Fig. 3.3 Contours of maximum seismic accelerations expected within a period of 75 years (after H. Goto and H. Kameda (82))

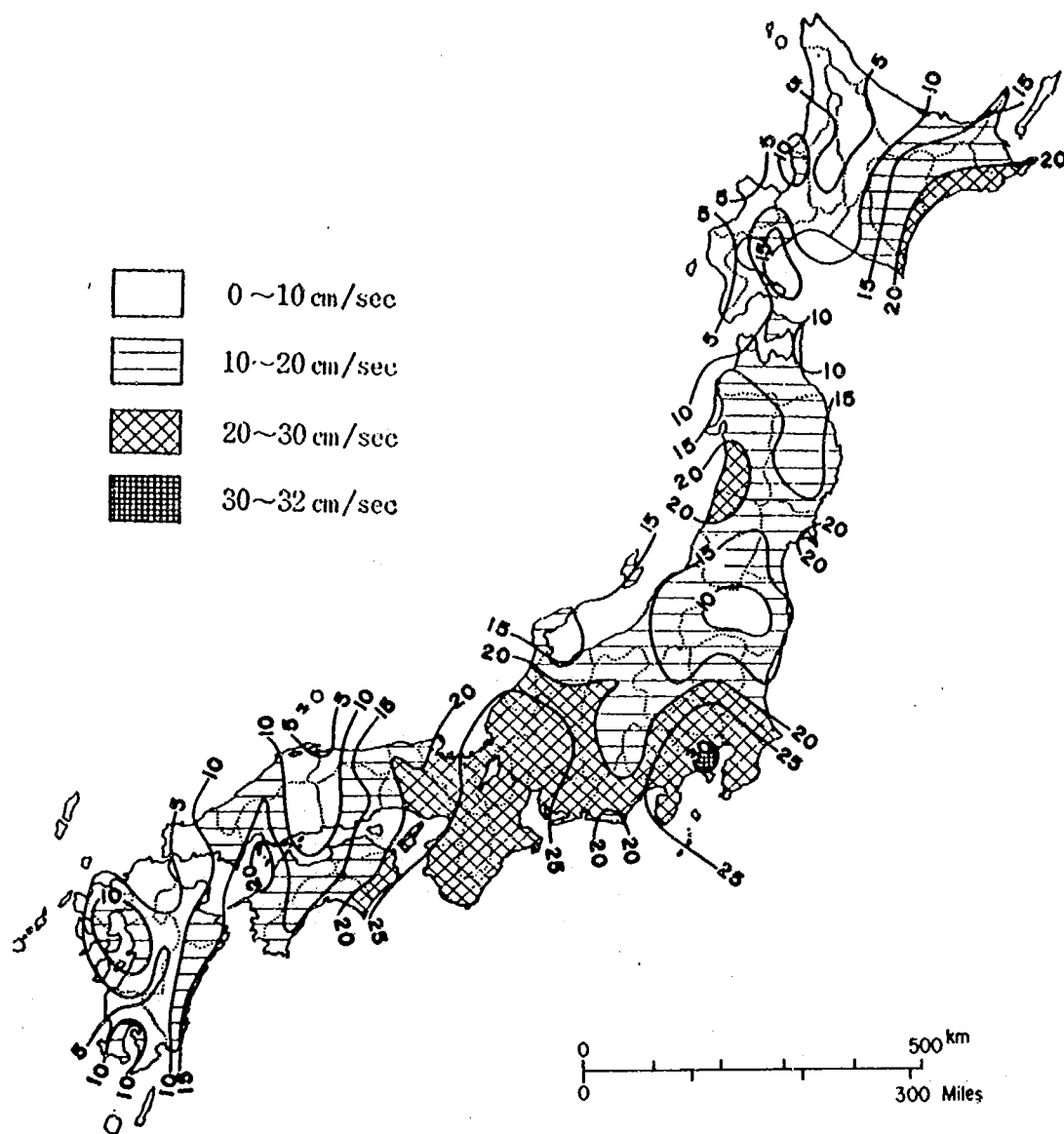


Fig. 3.4 Contours of maximum seismic velocities expected within a period of 75 years (after H. Goto and H. Kameda (82))

seismic intensities measured by the Japan Meteorological Agency (JMA) during major earthquakes occurred since 1923. Fig. 3.5 is a result of their zoning maps.

Seismic forces in the design of engineering structures can be determined in reference to these results on the seismicity in Japan, together

with consideration of other factors including social and economical aspects of the structures as well as engineering judgement (162).

3.2. Characteristics of Strong Ground Motions

Earthquake response spectral curves are of-

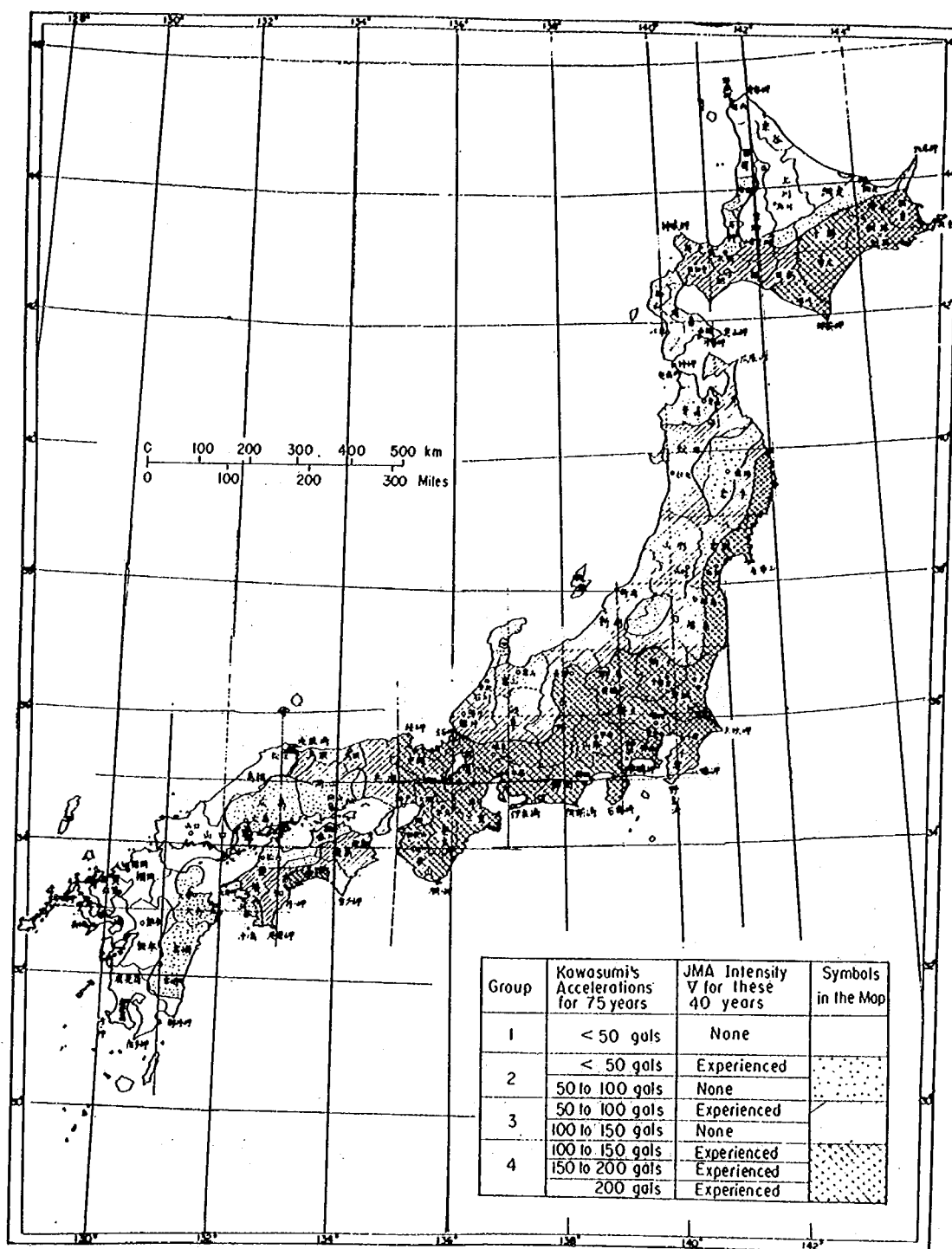


Fig. 3.5 Seismic risk map (after T. Okubo and T. Terashima (85))

en used as an index for representing dynamic effects of earthquake ground motions on structures. M. A. Biot⁸⁶) developed a method in which seismic forces for long-period structures can be evaluated reasonably using a response spectral curve. G. W. Housner, R. R. Martel, J. L. Alford⁸⁷) analyzed several strong ground motions recorded in the U.S.A. and obtained response spectra for a linear single-degree-of-freedom system with viscous damping. Furthermore, G. W. Housner⁸⁸) proposed average spec-

trum curves as shown in Fig. 3.6 (A). Fig. 3.6 (B) indicates spectrum curves of magnification factors (ratios of the response absolute acceleration to the maximum ground acceleration), obtained from Housner's average spectrum curves mentioned above, by assuming the maximum ground acceleration to be 4 ft/sec^2 .

T. Takata, T. Okubo and E. Kuribayashi⁸⁹) also proposed average spectrum curves for the same linear system by analyzing 20-component strong motions recorded in Japan, including a

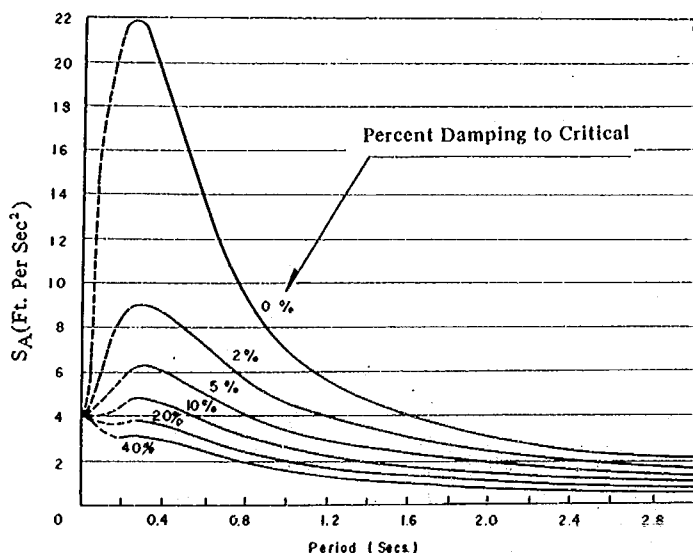


Fig. 3.6 (A) Average acceleration spectrum (after G. W. Housner (88))

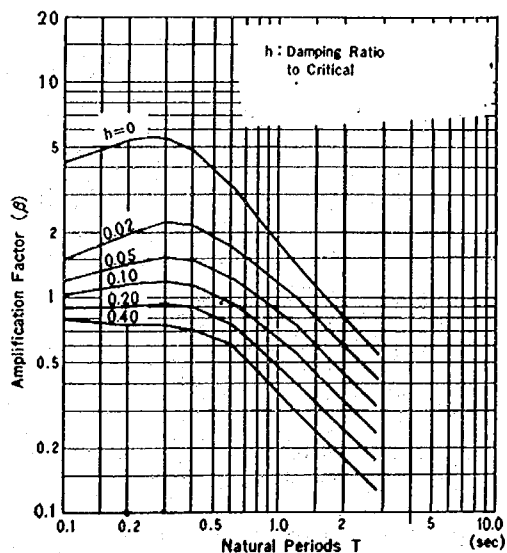


Fig. 3.6 (B) Amplification factor spectrum obtained from Fig. 3.6 (A)

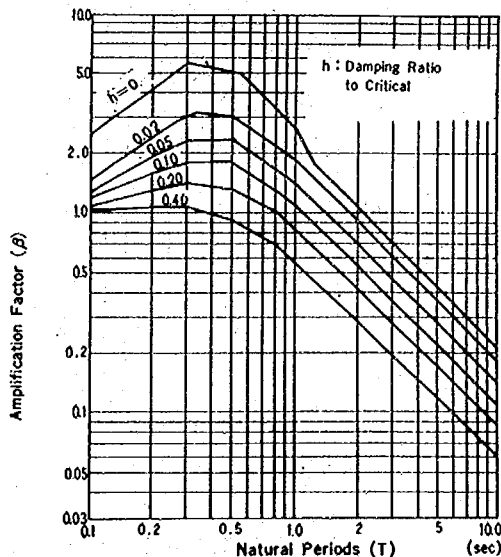
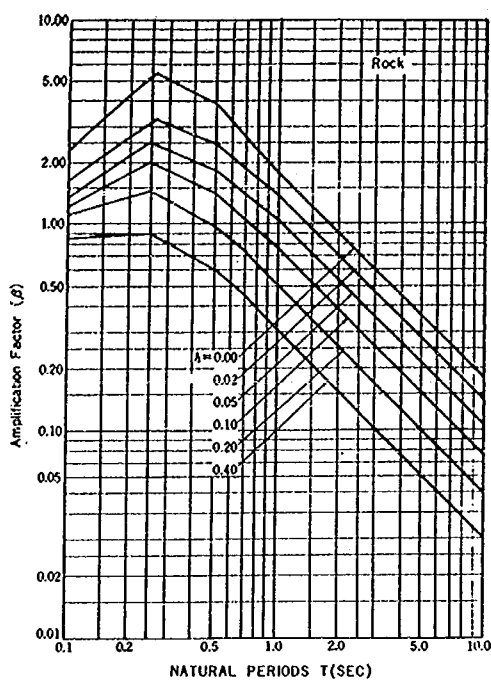
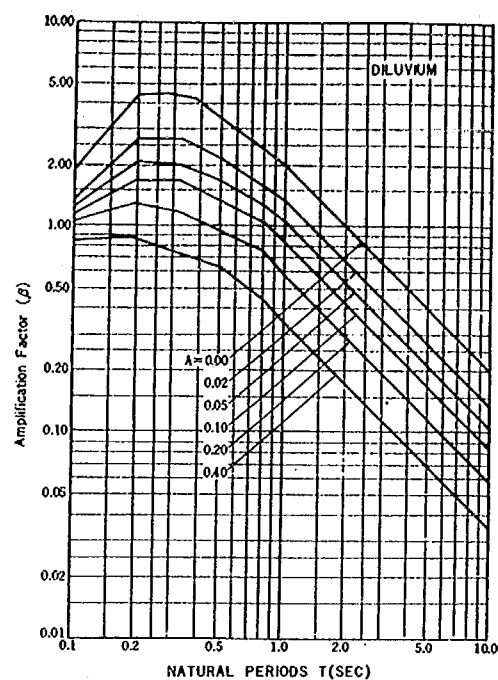


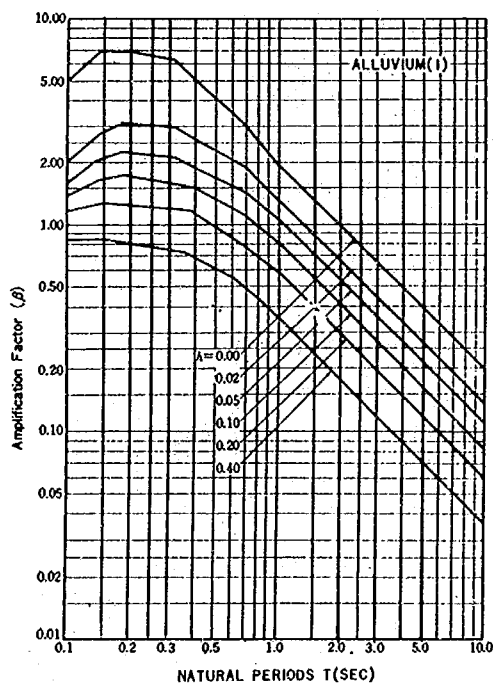
Fig. 3.7 Amplification factor spectrum (after T. Takata, T. Okubo, E. Kuribayashi (89))



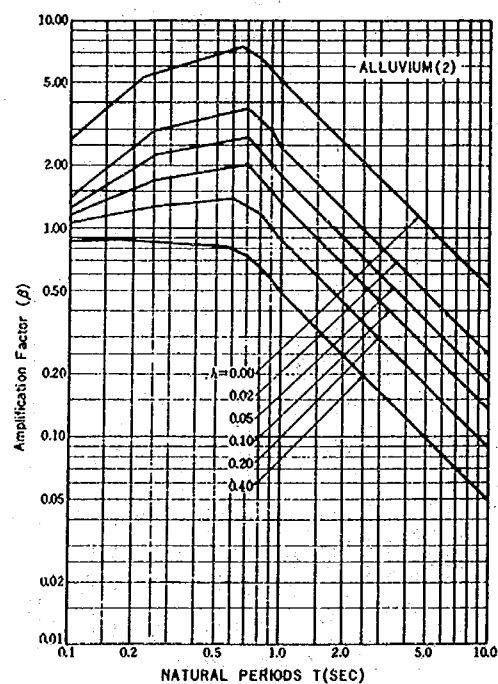
(A) AVERAGE SPECTRA FOR ROCK



(B) AVERAGE SPECTRA FOR DILUVIUM



(C) AVERAGE SPECTRA FOR STIFF ALLUVIUM



(D) AVERAGE SPECTRA FOR SOFT ALLUVIUM

Fig. 3.8 Amplification factor spectrum for four kinds of ground condition (after E. Kuribayashi, T. Iwasaki, K. Tuji (92))

record during the Niigata Earthquake of 1964. Fig. 3.7 represents a result of the average spectrum curves. It is noted that the values of β in Fig. 3.7 are generally greater than those in Fig. 3.6 (B) for systems with non-zero damping ratios. The spectral curves in Fig. 3.7 have been applied to dynamic analyses of several highway bridges (see Section 3.8), and also to the specifications for earthquake-resistant design of the proposed Honshu-Shikoku Suspension Bridges⁹⁾ (see Section 4.2).

T. Katayama⁹⁰⁾ computed 70-component Japanese strong motion records and clarified the effects of the magnitude of earthquakes and the magnitude of ground accelerations on the characteristics of linear spectral curves. S. Hayashi, H. Tsuchida, and E. Kurata⁹¹⁾ also computed 61-component strong motion records obtained at 21 stations during the Tokachi-oki Earthquake of 1968 and its aftershocks, and discussed the effects of ground conditions on the characteristics of linear spectral curves.

E. Kuribayashi, T. Iwasaki, and K. Taji⁹²⁾ studied on the factors which may affect the characteristics of linear spectral curves, discussed the effects of the magnitude of earthquakes, the magnitude of ground accelerations, the epicentral distance, and ground conditions, and proposed four different spectral curves as shown in Fig. 3.8, for various kinds of subsoil conditions from rocky ground to soft alluvial ground.

3.3 Consideration on Ground Disasters and Bearing Capacities of Subsoil

Any bridge superstructures are supported by the substructures with the foundations which are rested on or surrounded by the subsoil. As seen in the examples of seismic damage to bridge structures described in Chapter 2, ground disasters such as faulting, land sliding, liquefaction, etc. often cause severe damage to bridge structures. In designing a reasonably safe structure, accordingly, it is very important to know the possibility of occurrence of these catastrophic ground disasters, by utilizing ample geological knowledge as well as available engineering knowledge. It is preferable to avoid construction of important structures at sites where the catastrophic ground disasters can be anticipated with high possibility. A particular attention should be given to those structures which are inevitably constructed at such sites.

Since lack of bearing capacities of subsoils seems to be a cause of severe damage to bridge structures, bridge foundations should be designed under an appropriate estimation of the bearing capacities of subsoils by making use of ample knowledge on soil mechanics and foundation engineering. It should be noted that the bearing capacities for dynamic loads during earthquakes may be considerably different from those for static loads. An extensive decrease in the bearing capacities is often observed for loose saturated sandy soils due to the liquefaction during earthquakes. Once liquefaction occurs, the liquefied soils may exert large pressures on substructures during the latter periods of major earthquakes which have comparatively long duration. Moreover, in worse cases the ground may slide entirely together with the structures supported.

As for the subjects described here very briefly it is recommended to refer to ample publications on soil dynamics as well as those on soil mechanics and foundation engineering.

3.4 Earth Pressures on Abutments during Earthquakes

Coulomb's Formulas for Earth Pressures

For cohesionless soils the static earth pressures acting on abutments or retaining walls are approximately given by the well-known Coulomb's formulas²⁰⁷⁾. The active earth pressure P_a at depth x acting on the wall (shown in Fig. 3.9) is given by eq. (3.1).

$$P_a = \left(\gamma x + \frac{q \cos \theta}{\cos(\theta - \alpha)} \right) C_a \quad (3.1)$$

In which C_a is the coefficient of active earth pressure expressed by

$$C_a = \frac{\cos^2(\phi - \theta)}{\cos^2 \theta \cos(\theta + \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \alpha)}{\cos(\theta + \delta) \cos(\theta - \alpha)}} \right]^2} \quad (3.2)$$

and

- γ : unit weight of the soil,
- q : uniform vertical surcharge on the back fill,
- H : height of the wall,
- θ : angle between the vertical line and the wall surface,
- α : angle between the horizontal line and the surface of the back fill,
- ϕ : angle of internal friction of the soil, and
- δ : angle of friction between the wall and the

soil, and usually assumed as

$$|\delta| = \left(\frac{1}{2} \sim \frac{3}{4}\right) \phi \leq 15^\circ.$$

The passive earth pressure P_p at depth x acting on the wall (shown in Fig. 3.9) is given by eq. (3.3),

$$P_p = \left(\gamma x + \frac{q \cos \theta}{\cos(\theta - \alpha)}\right) C_p \quad (3.3)$$

in which C_p is the coefficient of passive earth pressure expressed by

$$C_p = \frac{\cos^2(\phi + \theta)}{\cos^2 \theta \cos(\theta - \delta) \left[1 - \frac{\sin(\phi + \delta) \sin(\phi + \alpha)}{\sqrt{\cos(\theta - \delta) \cos(\theta - \alpha)}} \right]} \quad (3.4)$$

Figs. 3.10 and 3.11 indicate the values of the coefficients C_a and C_p for various values of ϕ and δ for the case where $\theta = 0$ (vertical wall), $\alpha = 0$ (horizontal back fill), and $q = 0$ (without surcharge).

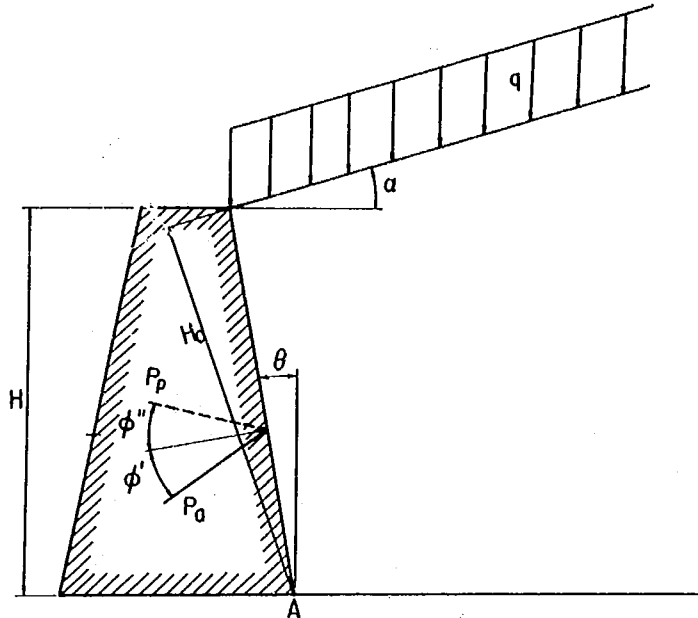


Fig. 3.9 Earth pressure acting on wall

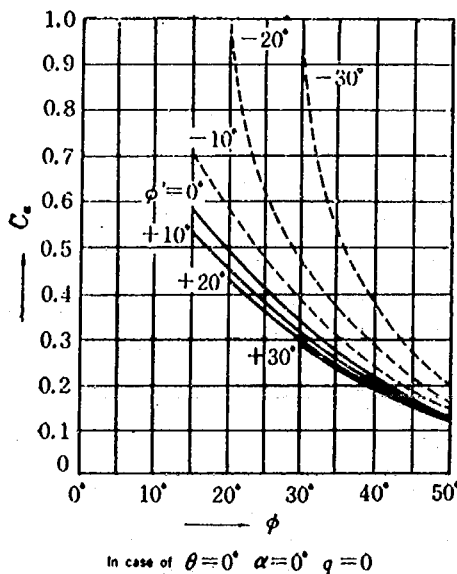


Fig. 3.10 Coefficient of active earth pressure

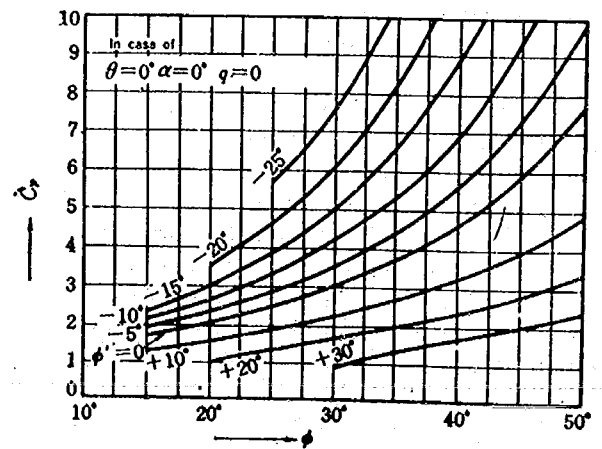


Fig. 3.11 Coefficient of passive earth pressure

Mononobe-Okabe Formulas for Earth Pressures during Earthquakes

N. Mononobe²⁰⁶⁾ and S. Okabe⁹³⁾ developed two quasi-static methods, one based on the Rankine's theory of earth pressures and the other on the Coulomb's theory, to approximately evaluate earth pressures acting on a wall during earthquakes. The basic idea of the methods is to modify the Rankine's theory or the Coulomb's theory by simply assuming that the wall statically incline together with the back fill toward the front of the wall for evaluating the active earth pressure during earthquakes, and toward the reverse direction for the passive earth pressure during earthquakes. The angle of inclination θ_o may be expressed by eq. (3.5), when horizontal acceleration and upward vertical acceleration are taken into consideration simultaneously (Fig. 3.12).

$$\theta_o = \tan^{-1} \frac{k_h}{1 - k_v} = \tan^{-1} k \quad (3.5)$$

where θ_o : angle of inclination of the wall and the back fill,

k_h : horizontal seismic coefficient

k_v : vertical seismic coefficient

$$k = \frac{k_h}{1 - k_v} : \text{resultant seismic coefficient}$$

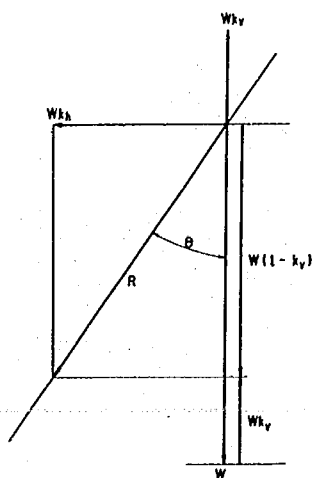


Fig. 3.12

Resultant seismic coefficient

Fig. 3.13 indicates the values of k and θ_o for various values of k_h and k_v .

After the wall and the back fill rotate by θ_o , the horizontal line and the vertical line will tilt by θ_o and the apparent acceleration of gravity may be assumed as $(1 - k_v) \sec \theta_o \cdot g$ instead of ordinary acceleration of gravity (g). From the assumptions the seismic earth pressures acting on the wall shown in Fig. 3.9 will be expressed by eqs. (3.6) and (3.7), when subjected to the horizontal acceleration $k_h \cdot g$ and upward vertical acceleration $k_v \cdot g$, simultaneously. The active earth pressure (p_{ea}) and the passive earth pressure (p_{ep}) during earthquakes at depth x will be

$$p_{ea} = (1 - k_v) \left[\gamma x + \frac{q \cos \theta}{\cos(\theta - \alpha)} \right] C_{ea} \quad (3.6)$$

$$p_{ep} = (1 - k_v) \left[\gamma x + \frac{q \cos \theta}{\cos(\theta - \alpha)} \right] C_{ep} \quad (3.7)$$

where C_{ea} and C_{ep} are the coefficients of active and passive earth pressures during earthquakes expressed by

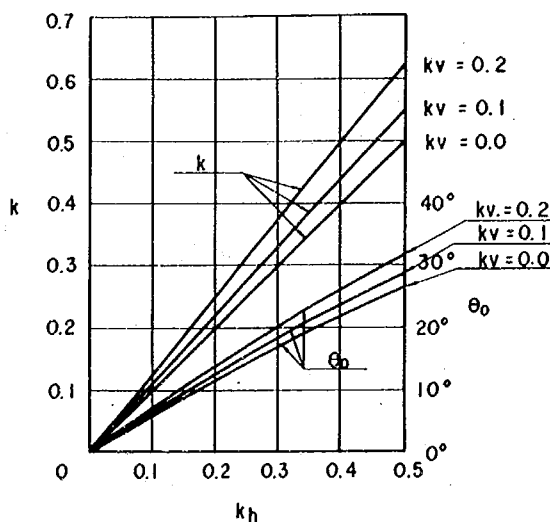


Fig. 3.13 k and θ_o values for various k_h and k_v

$$C_{ea} = \frac{\cos^2 (\phi - \theta_o - \theta)}{\cos \theta_o \cos^2 \theta \cos(\delta + \theta + \theta_o) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \theta_o + \alpha)}{\cos(\delta + \theta + \theta_o) \cdot \cos(\alpha - \theta)}} \right]^2} \quad (3.8)$$

$$C_{ep} = \frac{\cos^2 (\phi - \theta_o + \theta)}{\cos \theta_o \cos^2 \theta \cos(\delta + \theta - \theta_o) \left[1 - \sqrt{\frac{\sin(\phi - \delta) \cdot \sin(\phi - \theta_o + \alpha)}{\cos(\delta + \theta - \theta_o) \cdot \cos(\alpha - \theta)}} \right]^2} \quad (3.9)$$

It should be noted that the earth pressures given by eqs. (3.6) and (3.7) are the total pressures which are the sum of ordinary pressures and increases during earthquakes.

Although the inclination θ_o of the wall and the back fill is assumed intentionally for the calculation of the earth pressures during earthquakes, it is not meant that they actually tilt by θ_o . The direction of the earth pressures during earthquakes, therefore, is still same as that of the ordinary earth pressures expressed by eqs. (3.1) and (3.3).

Figs. 3.14 and 3.15 indicate the values of the coefficients C_{ea} and C_{ep} for various values of k and ϕ for the case where $\theta=0$, $\alpha=0$, $q=0$, and $\delta=0$ (zero friction between wall and soil) or $\delta=\pm 15^\circ$. From the figures the magnitudes of seismic earth pressures for the case of $k=0.2$ are about 1.5 times ordinary for active earth pressure, and about 90 percent ordinary for passive earth pressure.

Other Works on Earth Pressures during Earthquakes

N. Mononobe²⁰⁶⁾ also conducted an experiment on seismic earth pressures using a container set up on a harmonic shaking table and obtained results which agree quite satisfactorily with the formulas mentioned above. Fig. 3.16 indicates a result of the experiment.

Similar experimental works on earth pressures were extensively performed by H. Matsuo and S. Ohara⁹⁴⁾, S. Niwa⁹⁵⁾, Y. Ishii, H. Arai, H. Tsuchida⁹⁶⁾ and other several investigators. Y. Ishii, H. Tsuchida and T. Furube⁹⁷⁾ conducted an experiment on dynamic earth pressures and dynamic pore water pressures in saturated sands considering the behavior of the saturated sands after the liquefaction takes place.

As for dynamic earth pressures of cohesive soils no definite information is available. In the design of most structures, however, it is assumed

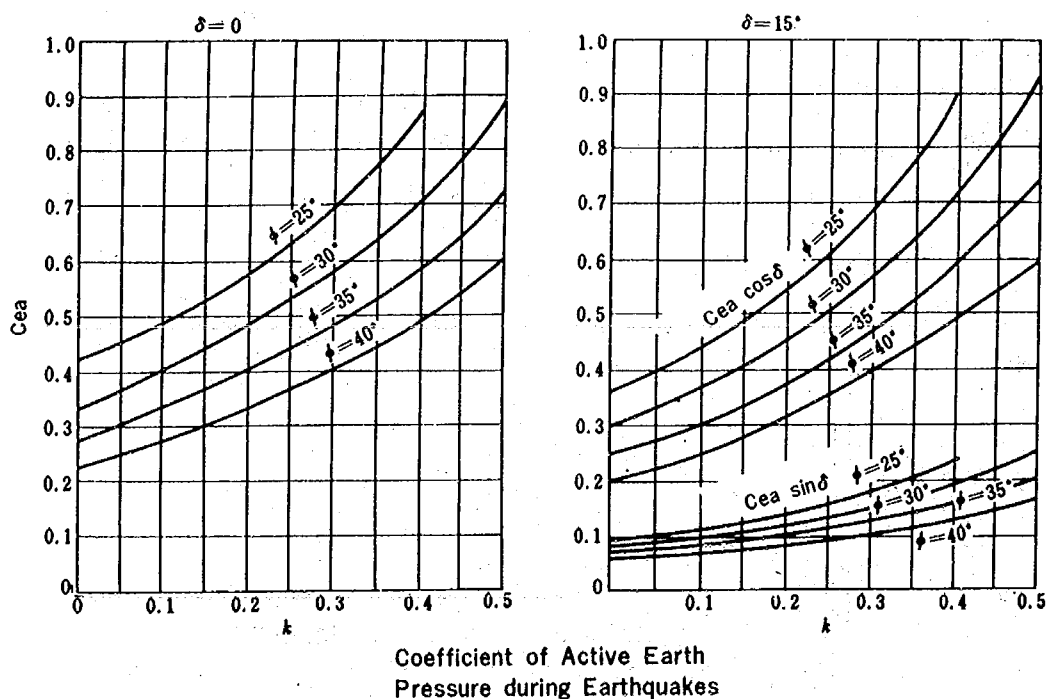


Fig. 3.14 Coefficient of active earth pressure during earthquakes

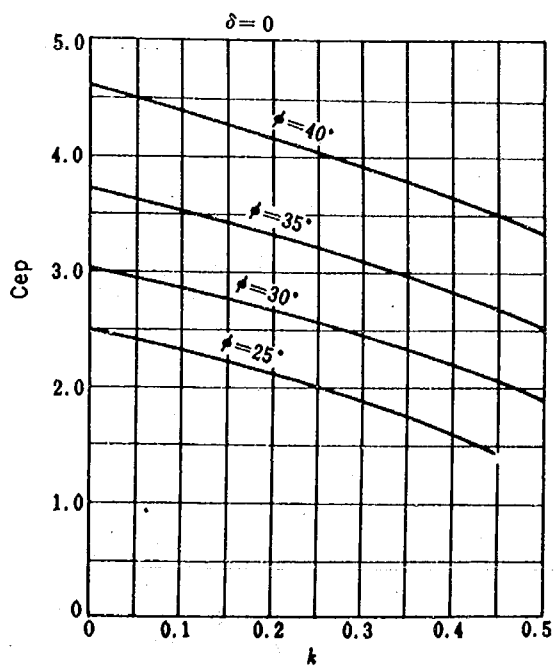
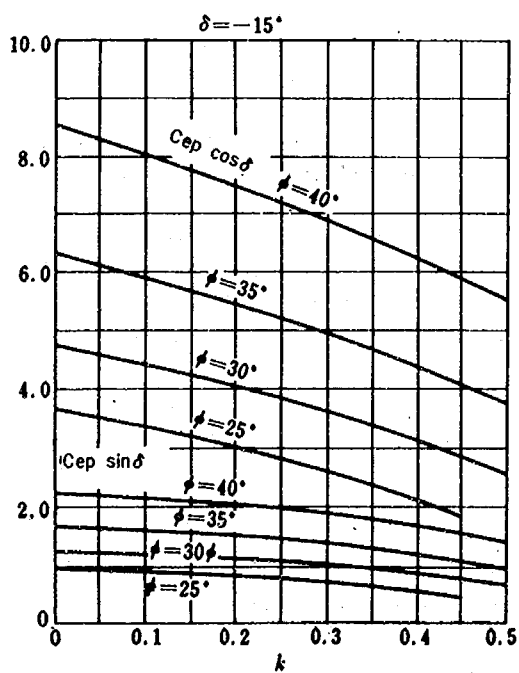
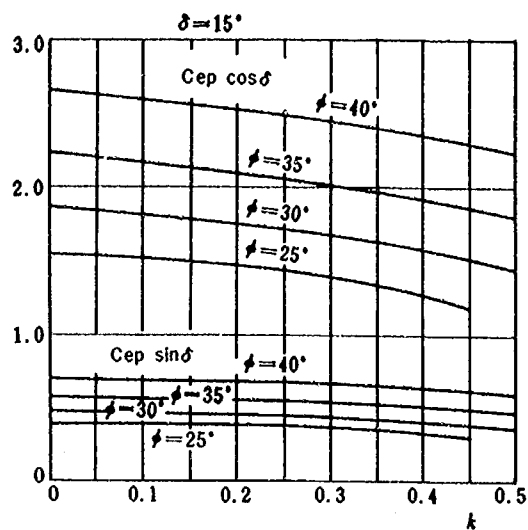


Fig. 3.15 Coefficient of passive earth pressure during earthquakes

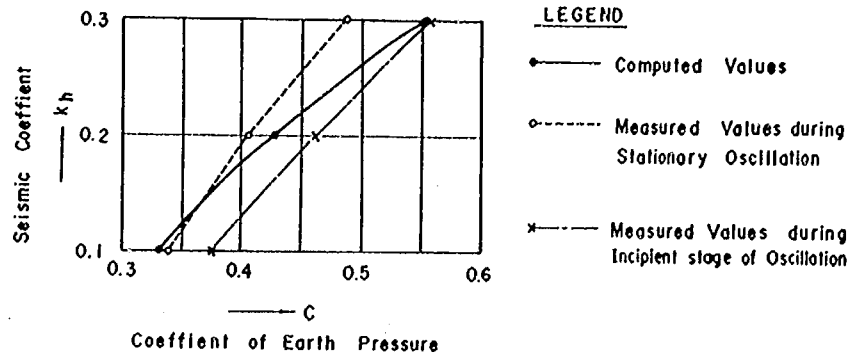


Fig. 3.16 Experimental results of earth pressure during harmonic excitation (after N. Mononobe (206))

that earth pressures of cohesive soils during earthquakes will be equal to ordinary pressures. Greater earth pressures, however, are sometimes considered in the aseismic design for the margin of safety for seismic forces.

3.5 Hydrodynamic Pressures on Substructures during Earthquakes

To evaluate hydrodynamic pressures acting on dam structures during earthquakes, H. M. Westergaard⁹⁸) developed the following simplified formula given by eq. (3.10) by assuming a dam as a two-dimensional vertical wall (Fig. 3.17).

$$P = \frac{7}{8} k_h \gamma_w \sqrt{hx} \quad (3.10)$$

where p : hydrodynamic pressure at depth x ,
 k_h : horizontal seismic coefficient,
 γ_w : unit weight of water,
 h : depth of reservoir.

The formula is widely used in the design of gravity dams. Afterward extensive research works have been conducted on hydrodynamic pressures on various types of dam structures.

There are also several research works on dynamic effects of reservoirs on columns such as bridge piers. First of all, J. L. Savage⁹⁹) attempted to evaluate the dynamic effects of

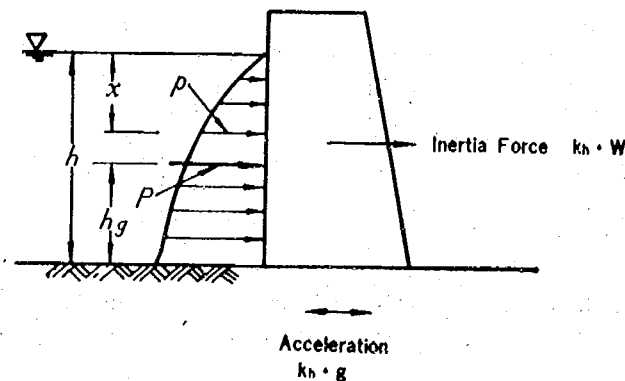


Fig. 3.17 Hydrodynamic pressure on vertical wall

reservoir on piers of the Pit River Bridge. R. W. Clough¹⁰⁰) conducted extensive model experiments on the dynamic effects of water on rigid and flexible columns. A. Sakurai¹⁰¹) performed three-dimensional analyses and experiments on hydrodynamic pressures acting on vertical circular columns. H. Goto and K. Toki¹⁰³) also conducted three-dimensional analyses and experiments and presented the following formulas given by eq. (3.11) for practically computing hydrodynamic pressures on vertical columns when subjected to seismic motions (Fig. 3.18).

$$P = \alpha k_h \gamma_w A \frac{b}{a} \left(1 - \frac{b}{4h}\right)^3 \sqrt{\frac{x}{h}} \quad \text{in case of } \frac{b}{h} \leq 2$$

or

$$P = \alpha k_h \gamma_w A \frac{b}{a} \left(0.7 - \frac{b}{10h}\right)^3 \sqrt{\frac{x}{h}} \quad \left. \begin{array}{l} \\ \text{in case of } 2 < \frac{b}{h} \leq 4 \end{array} \right\} \quad (3.11)$$

where p : hydrodynamic pressure acting on columns (per unit vertical length) at depth x ,

A : cross-sectional area,

α : a coefficient dependent on vibrational mode of column ($\alpha=1.0$ for case of translation motion).

The definitions of other notations can be seen in eq. (3.10). The formulas given by eq. (3.11) are usually used in the design of bridge piers (sometimes with slight modification).

S. Kotsubo¹⁰²) and T. Iwasaki¹⁰⁴) also conducted theoretical analyses and experimental works on hydrodynamic effects on bridge piers with particular cross-sections.

3.6 Dynamic Properties of Highway Bridges

Extensive studies have been carried out for evaluating dynamic properties of bridge structures such as natural periods, mode shapes, damping characteristics, etc. Numerous cases of dynamic tests were conducted on actual bridge structures consisting of foundations, piers and overall bridge structures.

Although in most of the tests the amplitudes of vibration are generally small when compared with those expected during strong earthquakes, the results can be reflected not only on further investigations of the bridges experimented, also on dynamic analyses of similar bridge structures

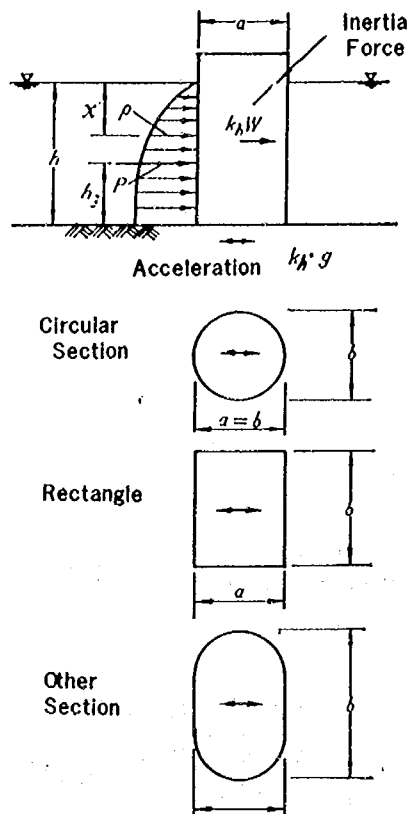


Fig. 3.18 Hydrodynamic pressure on column

in the future.

The followings are typical examples of dynamic tests for three bridges and the general features of dynamic properties of highway bridges.

Shinkatsushika Bridge¹¹⁶) (Figs. 3.19~3.24, Table 3.1~3.3)

This bridge, constructed by the Ministry of Construction in 1965, is on the National Highway No. 6, across the Edo River located between Tokyo and Chiba. Fig. 3.19 indicates the general view of the bridge, a typical cross-section of the superstructures which are of steel box girders, and subsoil conditions near the site of Pier 5 and results of the standard penetration test. The length of the bridge is 442m ($\div 8 \times 55$ m), and the width is 17.7m. Figs. 3.20 and 3.21 give brief information on Pier 4 and Pier 5 in which several transducers (accelerometers, earth pressure meters and pore-water pressure meters) were installed during the construction, to measure their

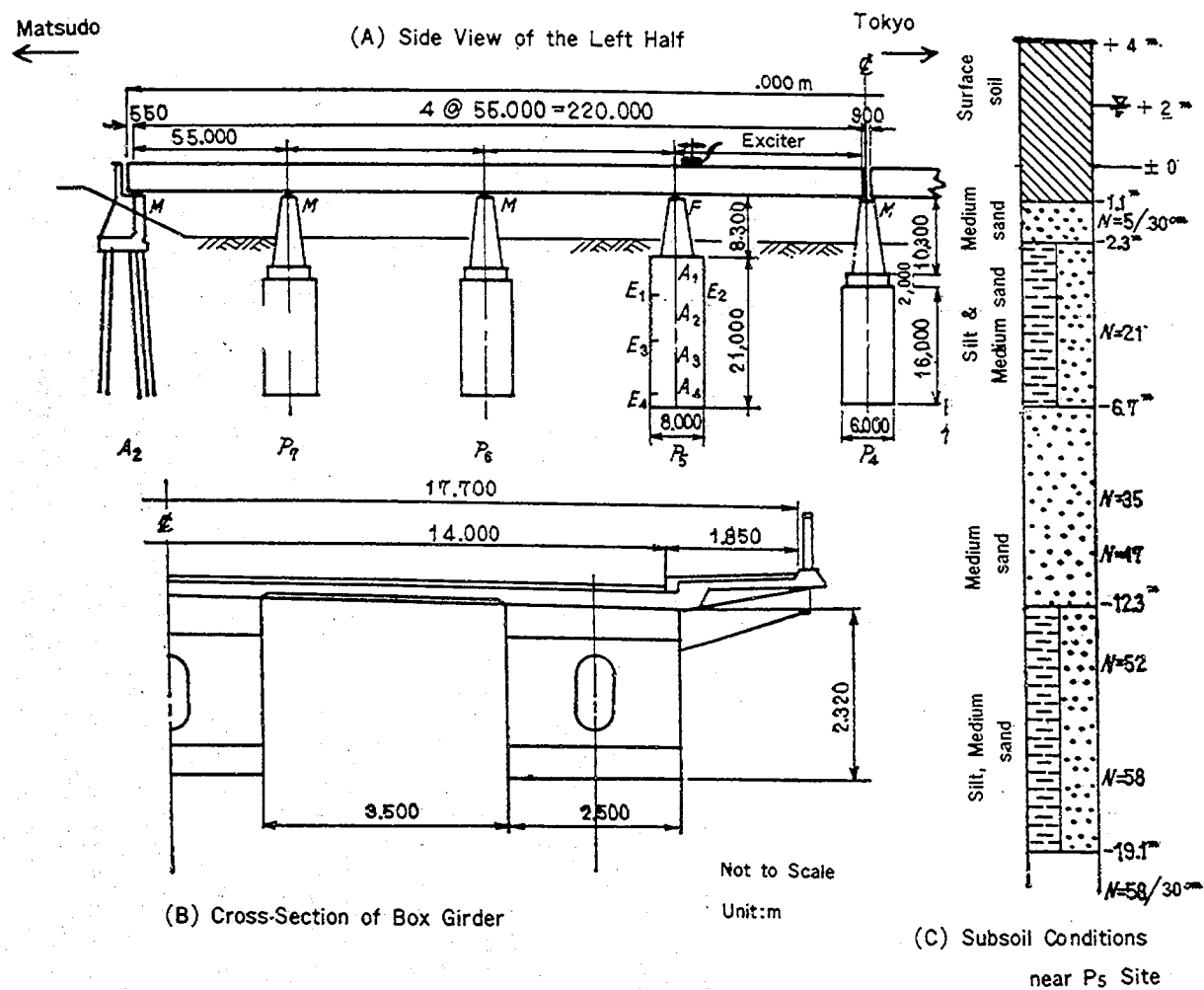


Fig. 3.19 General view of the Shinkatsushika bridge

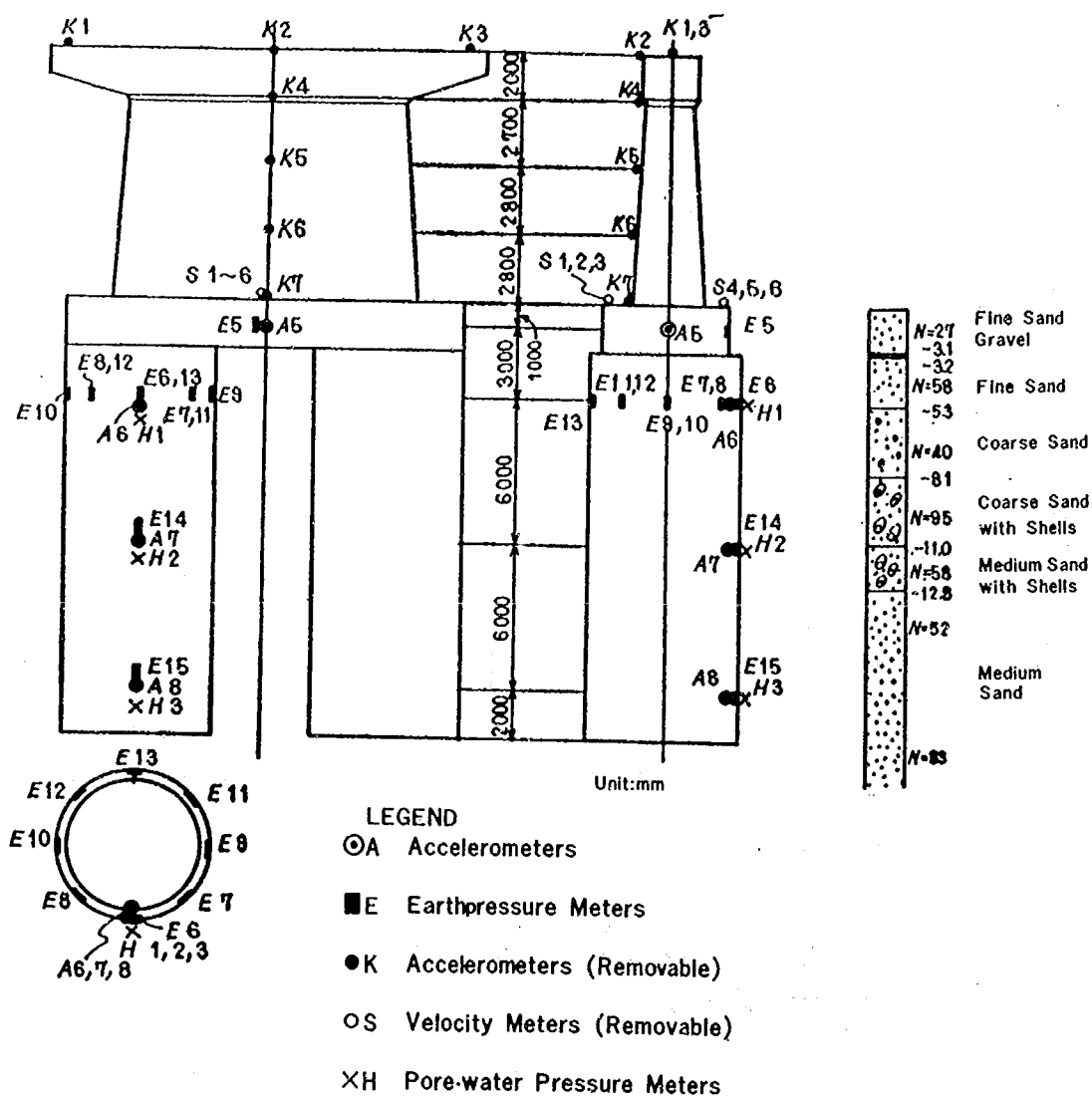


Fig. 3. 20 General view of pier 4 of the Shinkatsushika bridge

dynamic behavior. The subsoil conditions are generally of very dense sandy layers as shown in the figures. The both abutments are of reinforced concrete buttressed abutments with steel pipe piles, and the seven piers are of reinforced concrete structures with open caisson foundations. The design seismic coefficients of the

bridge were considered as 0.25 for horizontal and 0.1 for vertical, simultaneously.

Employing an centrifugal-type excitor which is capable of exerting dynamic horizontal forces (up to 40 tons) with a range of frequency of 1 to 13 Hertz, the Public Works Research Institute of the Ministry of Construction conducted compre-

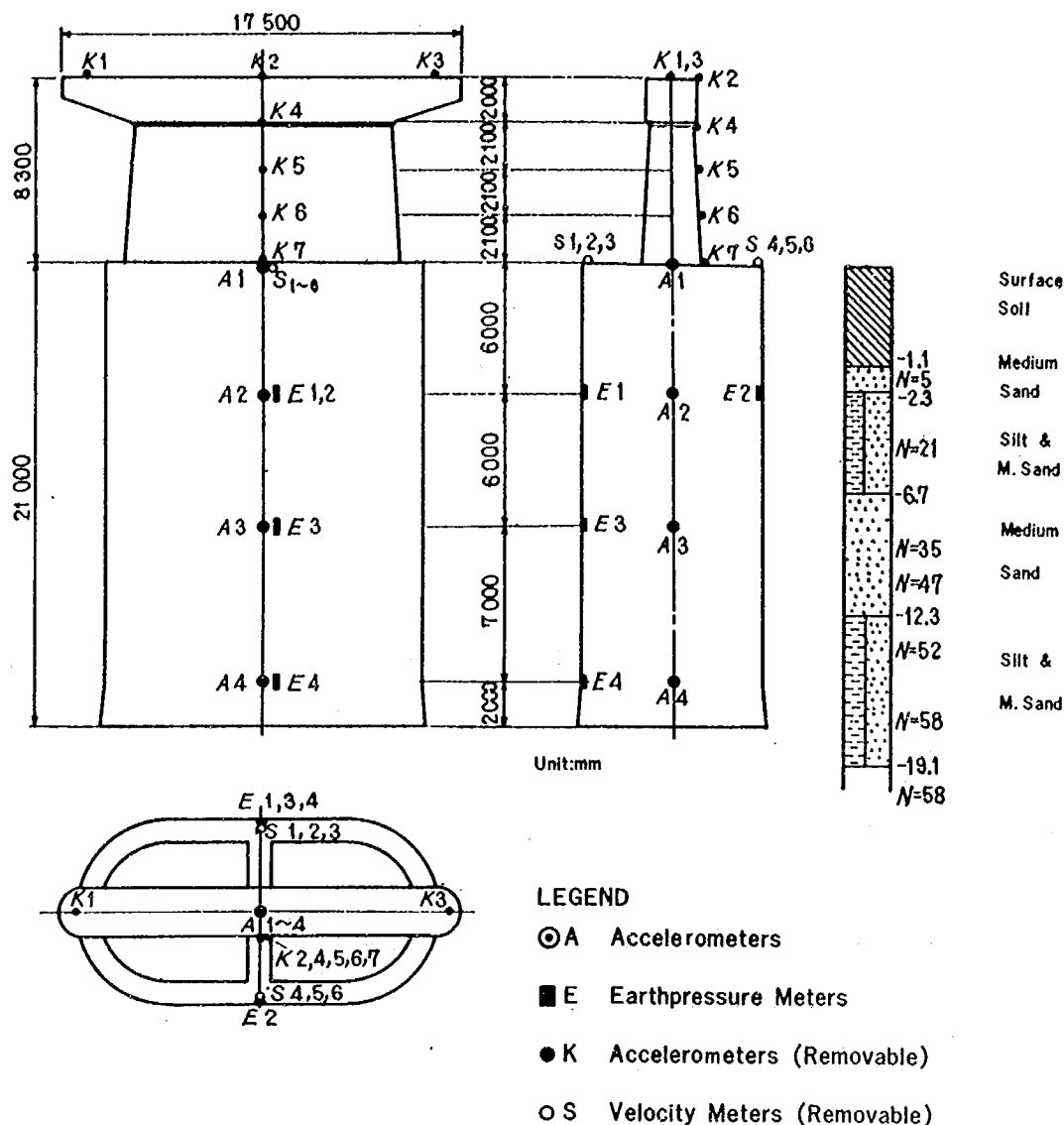


Fig. 3. 21 General view of pier 5 of the Shinkatsushika bridge

hensive dynamic tests on the two piers (Pier 4 and Pier 5) in 1963, and on the overall structure immediately after the completion of the bridge in 1965. The object of these tests was primarily to obtain the dynamic properties of the substructures and of the whole structure in the longitudinal direction.

The test results for Pier 4 are briefly

summarized in Table 3.1 and Fig. 3.22, including typical resonance curves at the pier cap, damping ratios and distribution of dynamic earth pressures for various magnitudes of exciting forces. The similar results are shown in Table 3.2 and Fig. 3.23 for Pier 5. Furthermore, the results for the whole structure are summarized in Table 3.3 and Fig. 3.24.

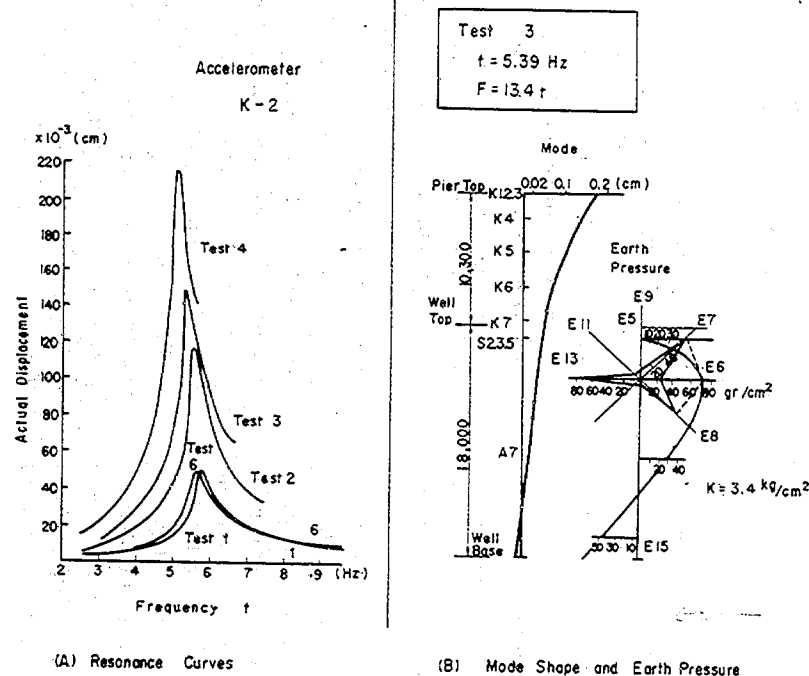


Fig. 3.22 Vibration test results for pier 4 of the Shinkatsushika bridge

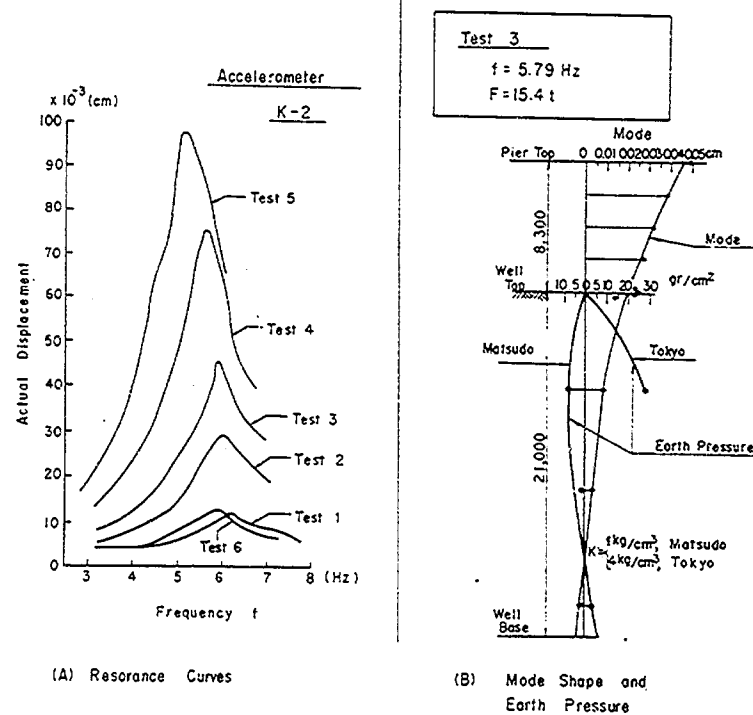
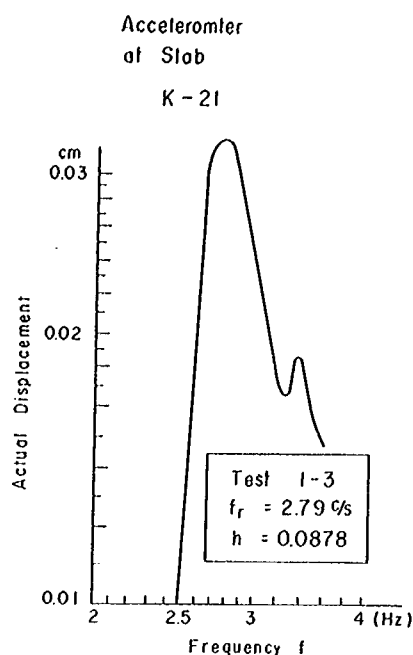
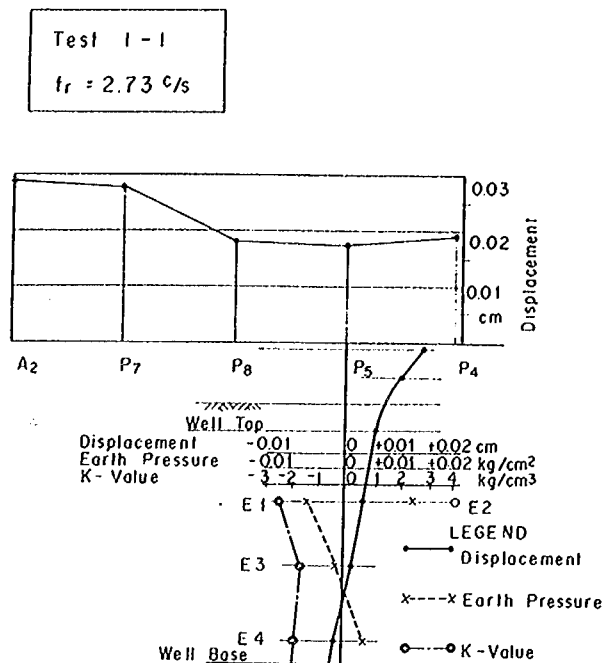


Fig. 3.23 Vibration test results for pier 5 of the Shinkatsushika bridge



(A) Resonance Curve



(B) Mode Shape, Earth Pressure, and K-Value

Fig. 3.24 Vibration test results for the whole bridge structure of the Shinkatsushika bridge

From these results the following may be concluded.

1) The fundamental resonant frequencies of the piers were between 4.9 and 5.8 Hertz for Pier 4 and between 5.0 and 6.0 Hertz for Pier 5, and obviously varied with the magnitudes of exciting forces. The maximum accelerations at the caps of the piers during the various tests were about 60 to 220 gals for Pier 4, and 20 to 110 gals for Pier 5. The damping ratios were 5 to 7 percent critical for Pier 4, and 8 to 12 percent critical for Pier 5, and did not show clear relation with amplitudes. The shapes of deformation during vibrations were primarily rocking motion with the centers of rotation at about one third from the bottom for the caisson foundations and simple bending for the columns. The dynamic earth pressures acting on the sides of the caisson foundations were approximately proportional to the dynamic amplitudes, and dynamic coefficient of subgrade reaction (ratio of earth pressure to amplitude) was approximately 4 kg/cm^3

2) As for the whole structure the fundamental

frequency was about 2.8 Hertz in the longitudinal direction, and the damping ratio was about 6.6 percent critical. The maximum acceleration at the superstructures was about 14 gals for acting forces of 9.1t. The Pier 5 having fixed supports vibrated together with the superstructure, although the other substructures (P4, P6, P7 and A2) having movable supports did not move at all. The shape of deformation of the Pier 5 was very similar to that of the pier itself without the superstructure.

Dynamic behavior of the Shinkatsushika Bridge during actual earthquakes have been measured by six transducers (three accelerometers and three earth pressure meters) installed for the dynamic tests in the caisson foundation of Pier 5, and two strong-motion accelerographs set up after its completion permanently on the crest of Pier 5 and on the ground nearby. Some results from these instruments will be discussed in 3.7.

Torii Bridge¹⁶⁾ (Figs. 3.25~3.29)

Table 3.1 Results of Vibration Test for Pier 4 of the Shinkatsushika Bridge

Test No.	Resonant Frequency f_r (Hz)	Damping Ratio * h	Maximum Amplitude during Resonance			Maximum Earth Pressure during Resonance		Coefficient of Subgrade Reaction K (kg/cm ³)	Acting Force during Resonance F (t)	Remarks
			Location	Acceleration α (gals)	Displacement a (cm)	Location	Earth Pressure p (kg/cm ²)			
1	5.78	0.0599	Crest of Pier	65.3	0.0496	E-6	0.039	—	4.1	
2	5.58	0.0504	Crest of Pier	148.0	0.120	E-6	0.060	7.0	9.0	
3	5.31	0.0689	Crest of Pier	164.0	0.147	E-6	0.080	3.4	13.0	
4	5.09	0.0485	Crest of Pier	225.0	0.220	E-6	0.095	7.3	21.0	
5	4.86	—	Crest of Pier	211.0	0.226	E-6	—	—	27.0	
6	5.62	0.0640	Crest of Pier	60.5	0.0486	E-6	0.025	2.0	3.7	
Average	5.55	0.0583	—	—	—	—	—	4.7	—	

* Damping ratios are obtained from the shape of resonance curves.

Table 3.2 Results of Vibration Test for Pier 5 of the Shinkatsushika Bridge

Test No.	Resonant Frequency f_r (Hz)	Damping Ratio* h	Maximum Amplitude during Resonance			Maximum Earth Pressure during Resonance		Coefficient of Subgrade Reaction K (kg/cm ³)	Acting Force during Resonance F (t)	Remarks
			Location	Acceleration α (gals)	Displacement a (cm)	Location	Earth Pressure p (kg/cm ²)			
1	6.12	0.118	K ₂	17.6	0.0119	E ₂	0.0083	3.0	4.5	
2	5.98	0.109	K ₂	41.0	0.0290	E ₂	0.0186	2.5	10.4	
3	5.91	0.080	K ₂	62.9	0.0456	E ₂	0.029	3.6	16.1	
4	5.63	0.100	K ₂	94.2	0.0752	E ₂	0.045	2.5	25.4	
5	5.26	0.099	K ₂	108.0	0.0990	E ₂	0.093	4.9	31.7	
6	6.00	0.103	K ₂	15.6	0.0110	—	—	—	4.3	
Average	5.8	0.111	—	—	—	—	—	—	—	

* Damping ratios are obtained from the shape of resonance curves.

Table 3.3 Results of Vibration Test for the overall structure of the Shinkatsushika Bridge

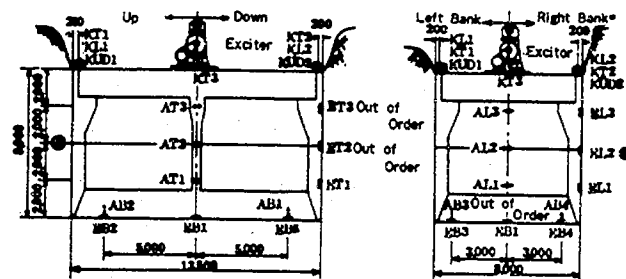
Test No.	Resonant Frequency f_r (Hz)	Damping Ratio * h	Maximum Amplitude during Resonance			Maximum Earth Pressure during Resonance		Coefficient of Subgrade Reaction K (kg/cm ³)	Acting Force during Resonance F (t)	Remarks
			Location	Acceleration α (gals)	Displacement a (cm)	Location	Earth Pressure p (kg/cm ²)			
1-1	2.79	0.0728	K-55	8.66	0.0295	E ₂	0.0125	4.1	6.24	
1-2	2.69	0.0787	K-21	9.52	0.0346	E ₂	0.0194	4.4	8.28	
1-3	2.77	0.0642	K-52	12.57	0.0401	—	—	—	8.78	
1-4	2.77	0.0584	K-52	11.35	0.0380	E ₂	0.0179	3.6	8.78	
2-1	2.78	0.0675	K-55	14.28	0.0458	E ₂	0.0179	4.5	9.84	
2-2	2.82	0.0455	K-22	9.76	0.0315	—	—	—	9.10	
Average	2.77	0.0660	—	—	—	—	—	4.2		

* Damping ratios are obtained from the shape of resonance curves.

Fig. 3.25 General view of the Torii bridge

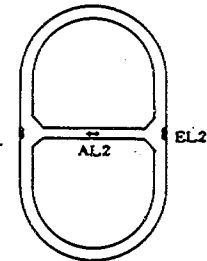
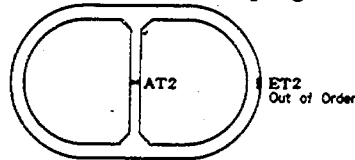
Transverse

Longitudinal

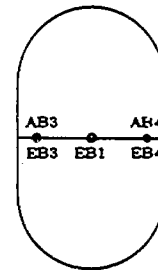
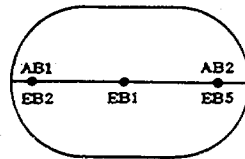


Unit:mm

Section
A-A



Base



S=1/200
0 1 2 3 4 5 m

See Note at the Left

Note

- 3-Component Accelerometer
- ↔ Accelerometer
- Earth Pressure Meter
- AT Transverse Accelerometer
- AL Longitudinal Accelerometer
- AB Accelerometer at the Base
- ET Transverse Earth Pressure Meter
- EL Longitudinal Earth Pressure Meter
- EB Earth Pressure Meter at the Base
- KT Transverse Accelerometer (Removable)
- KL Longitudinal Accelerometer (Removable)
- KUD Vertical Accelerometer (Removable)

Fig. 3.26 Open caisson foundation of pier 1 of the Torii bridge

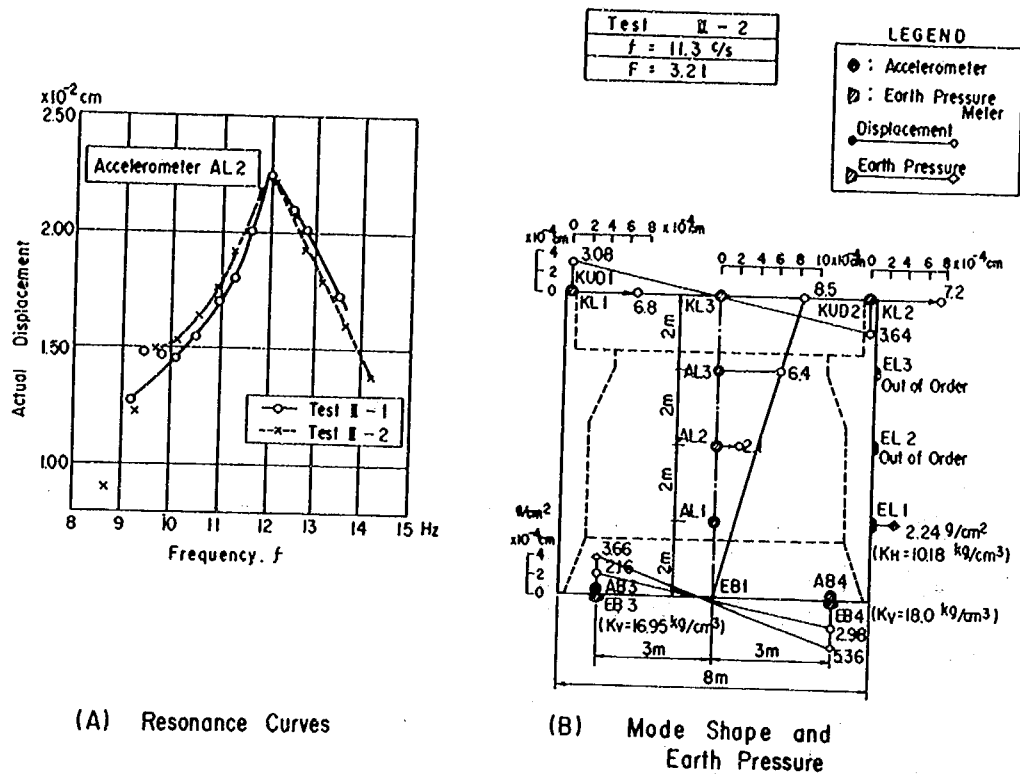


Fig. 3.27 Test results for the foundation of pier 1 of the Torii bridge (longitudinal direction)

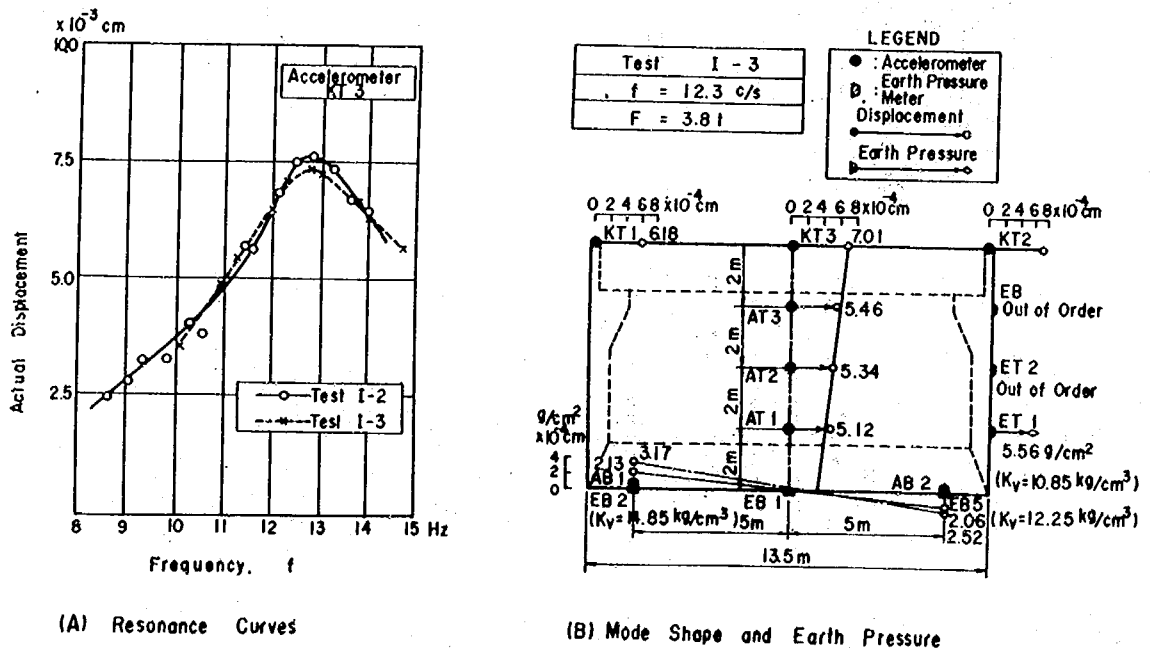


Fig. 3.28 Test results for the foundation of pier 1 of the Torii bridge (transverse direction)

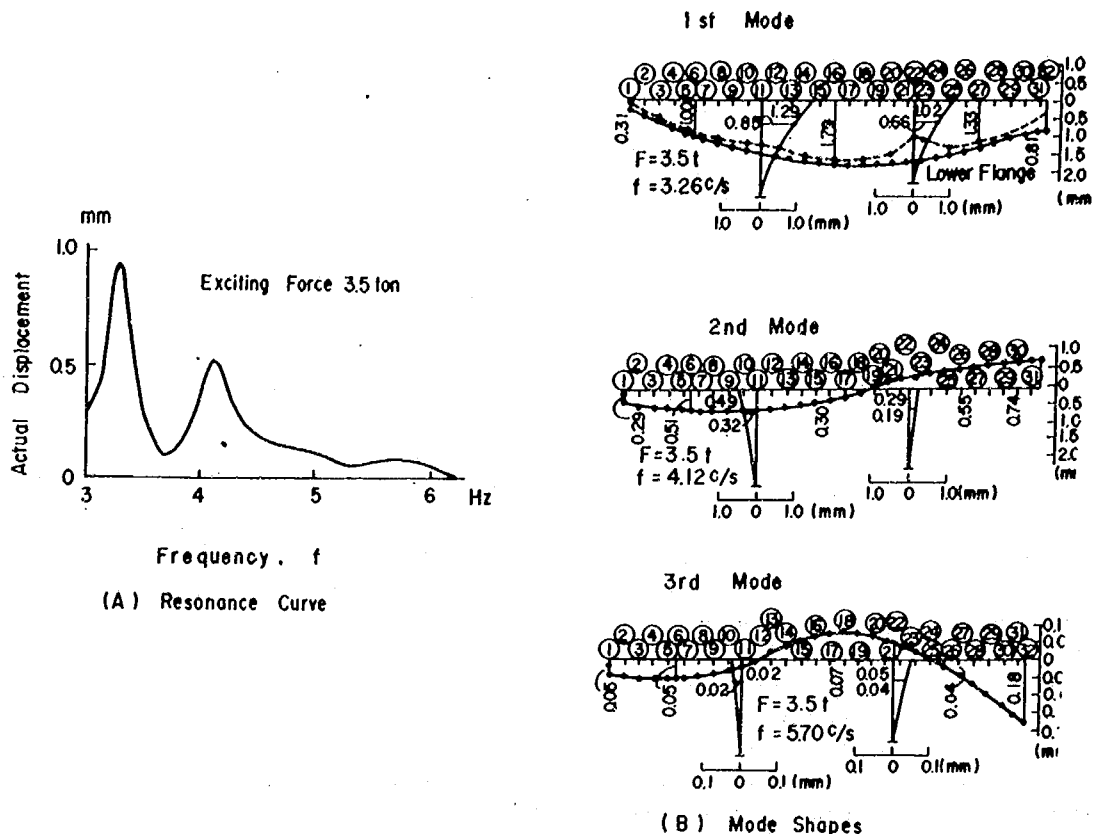


Fig. 3.29 Test results for the whole bridge structure of the Torii bridge (transverse direction)

This bridge, also constructed by the Ministry of Construction in 1968, is on the National Highway No. 49, and located at the bound between Fukushima and Niigata Prefectures. Fig. 3.25 indicates the general side view of the bridge. Fig. 3.26 shows the brief drawings of the caisson foundation of Pier 1, in which several transducers (ten accelerometers and eleven earth pressure meters) were equipped during its construction for dynamic testing.

The both abutments are of L-shape reinforced concrete buttressed structures with keys to resist horizontal forces, and two piers are of reinforced concrete solid columns (the cross-sections of 3.0m by 10.2m at the bottom and of 1.5m by 10.2m at the cap for Pier 1) with caisson foundations (the cross-section of 8m by 13.5m and the depth of 8m for Pier 1). The design seismic coefficients were considered as 0.25 horizontally and 0.1 vertically. The foundations of the four substructures rest on tuff, and the two caisson foundations are surrounded by tuffy clay with gravels. The superstructure, length of 115m and width of 7m, is of a 3-span continuous curved steel plate girder with a fix support at the left abutment, two pin supports at the piers and a movable support at the right abutment.

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For the bridge two steps of field dynamic tests were performed. In 1967 the caisson foundation of Pier 1 was actuated horizontally (both in longitudinal and transverse directions) by setting up a excitor, which is capable of exerting vibratory forces (up to 10t) with a range of 0.5 to 20 Hertz, on the crest of the foundation. Also in 1968 the dynamic properties of the whole structure in the transverse direction was obtained by equipping another excitor (maximum force of 20ton) on the slab between the two piers. Figs. 3.27 and 3.28 indicate the results of the first series of dynamic tests on the foundation of Pier 1 in the longitudinal and the transverse directions, respectively. Fig. 3.29 is the results of the second series on the overall structure in the transverse direction. These indicate typical results of resonant curves, resonant frequencies, damping ratios and mode shapes. From these the followings may be concluded.

- 1) The fundamental frequencies of the caisson foundation are 12.0 Hertz in the longitudinal and 13.0 Hertz in the transverse, and the damping ratios are about 15 percent critical in the both directions. The mode shape is the combination of rocking motion and translation motion, having the center of rotation at about the base of the foundation for the longitudinal, and at a deeper point (about 10m below the base) for the transverse. The vertical coefficient of subgrade reaction (ratio of earth pressure to displacement) at the base was measured about 15 kg/cm^3
- 2) For the complete structure vibrating in the transverse direction the three lowest resonant frequencies (1st to 3rd) are 3.26, 4.12 and 5.70 Hertz. The damping ratios for the three resonances are equally 2.8 percent of critical. The shapes of deformation were measured for the three resonant frequencies. After the vibration tests, an extensive analytical study was conducted to obtain better mathematical models in dynamic analyses both for the substructure and the superstructure.

Sokozawa Bridge ¹¹⁶⁾ (Figs. 3.30~3.34, Table 3.4)

The Sokozawa Bridge, completed by the Japan Highway Public (Corporation as a link of Chuo Expressway in 1968, is located at Sokozawa in Sagamiko, Kanagawa Prefecture about 50km west from Tokyo, where Chuo Expressway crosses a deep creek. A general view of the bridge

and the dimensions of Pier 3 are shown in Figs. 3.30 and 3.31, respectively. The both abutments are of gravity-type reinforced concrete structures, and four piers are I-shape steel framed reinforced concrete structures with spread footings (for Pier 1 and Pier 4 cast-in-place concrete pile foundations underneath the footings) rested on hard rock. Columns of two piers (Pier 2 and Pier 3) are about 50-meter high, and the other two are about 30-meter high.

The superstructures consist of two continuous steel truss girders: one is a two-span continuous girder and the other is three-span continuous girder. The superstructures are hinged to pier caps, and the longitudinal seismic forces exerted from the mass of the superstructures and some parts of the piers are designed to be resisted by the two rigid gravity-type abutments.

The bridge was designed in accordance with the design specifications proposed by the Subcommittee on Highrise Bridge Piers, the Expressway Research Foundation, which will be presented in 4.2. The basic seismic coefficient was considered as 0.2 horizontally and 0.1 vertically at the ground level. According to the specifications the design seismic coefficients for the piers in the transverse direction were increased by multiplying modification factors which have values from 1.0 to 1.66 varying with the height of piers.

For the bridge two series of field dynamic tests were conducted. The first series were tests for Pier 3 in 1967 before the erection of the superstructure, and the second series were tests for the overall structure in 1968 immediately after the completion of the bridge.

The first series of experiments for Pier 3 consisted of steady state forced vibration tests in the longitudinal direction by a 15-ton excitor and in the transverse direction by another 40-ton excitor, and step-function forced vibration tests by utilizing the propulsion of a rocket in the longitudinal direction. Since the fundamental period of vibration of the pier was estimated very long (about 1.4 sec.) in the longitudinal direction, a rocket engine which is capable of yielding thrust of 2t with duration of 1 second was fixed on the pier cap for obtaining free damped vibration records of the pier after the release of the thrust. The usage of rocket engines which are now often used for dynamic tests of long-period structures was initiated at the tests of the highrise pier of the Sokozawa Bridge.

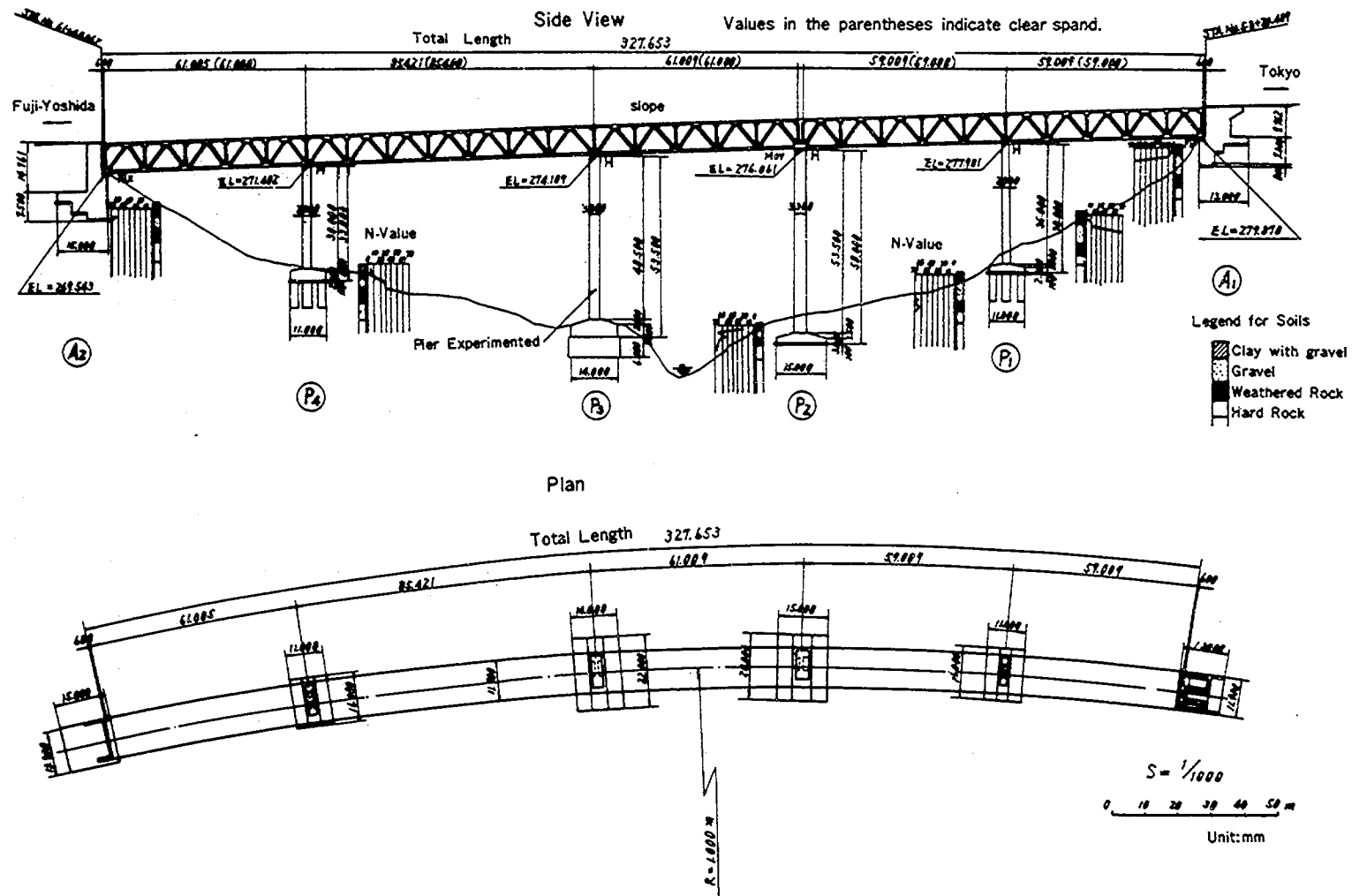


Fig. 3. 30 General view of the Sokozaawa bridge

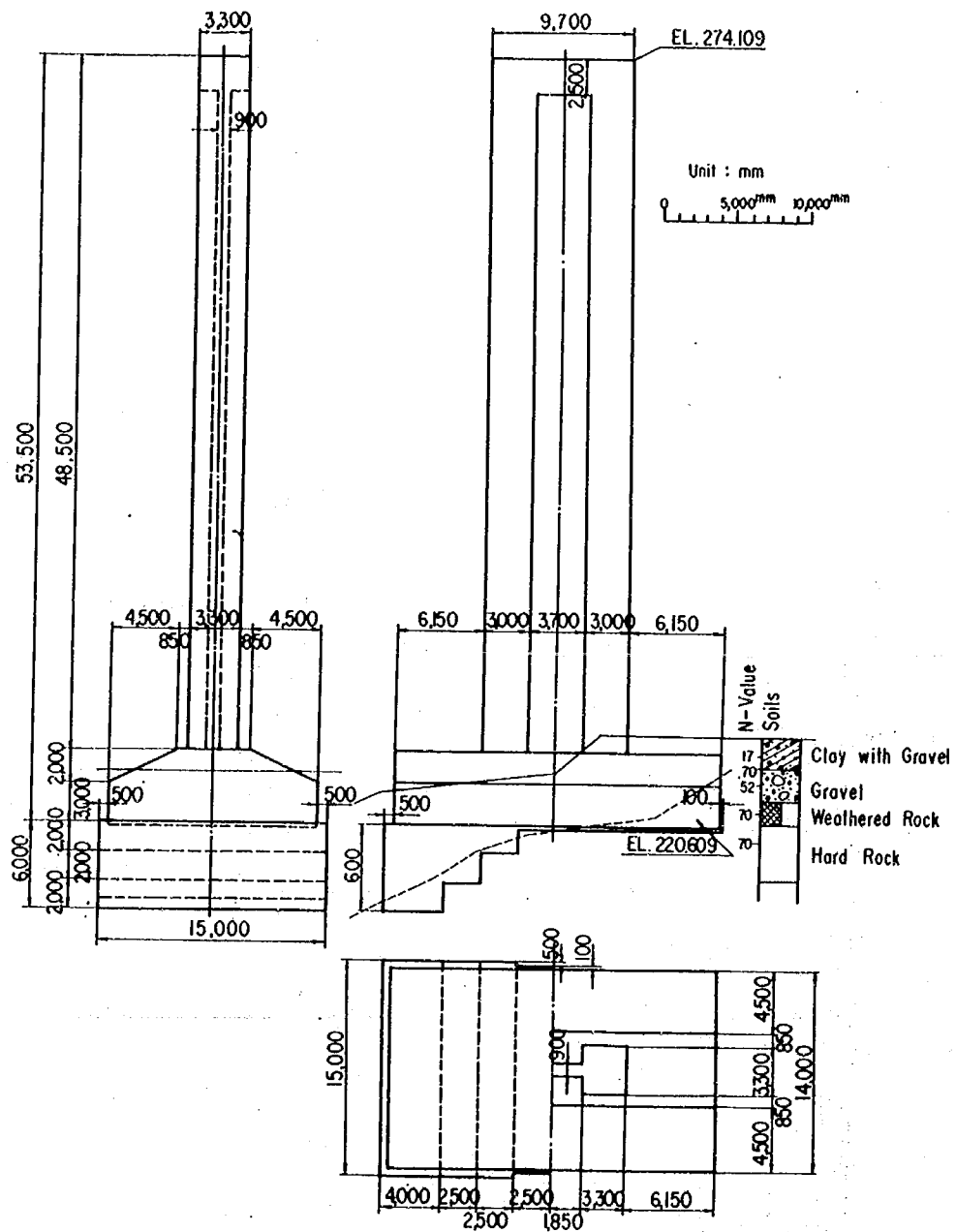


Fig. 3.31 Pier 3 of the Sokozawa Bridge

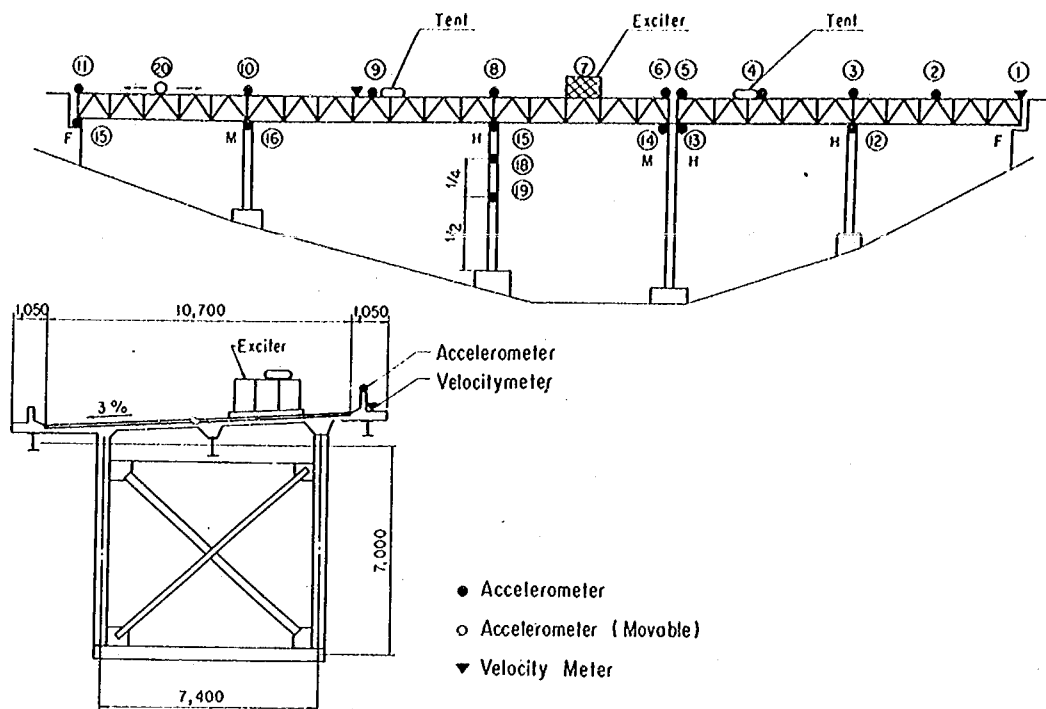


Fig. 3. 32 Locations of exciter and pick-ups

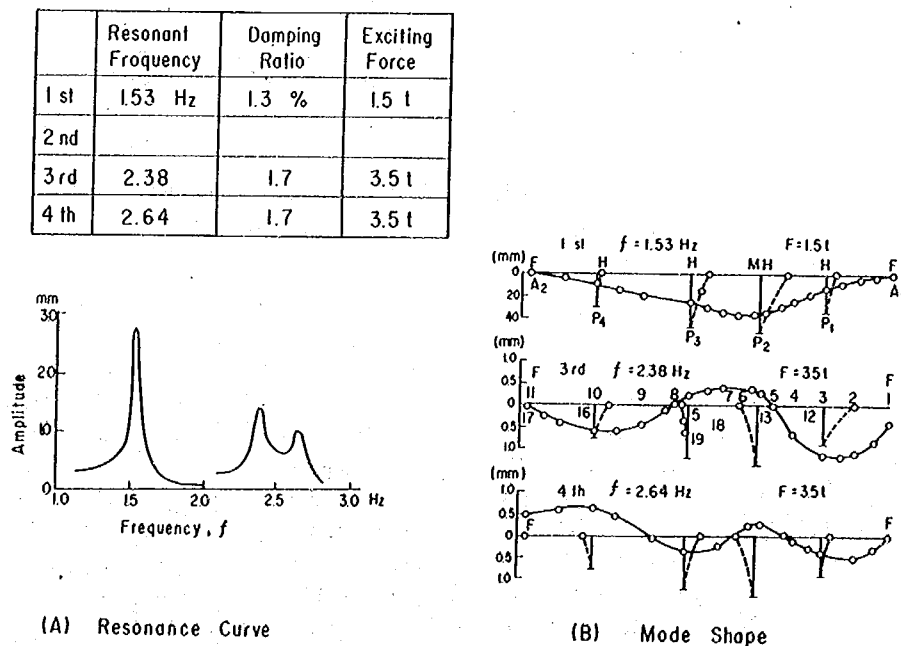


Fig. 3. 33 Test results for the whole bridge structure of the Sokoza bridge (transverse direction)

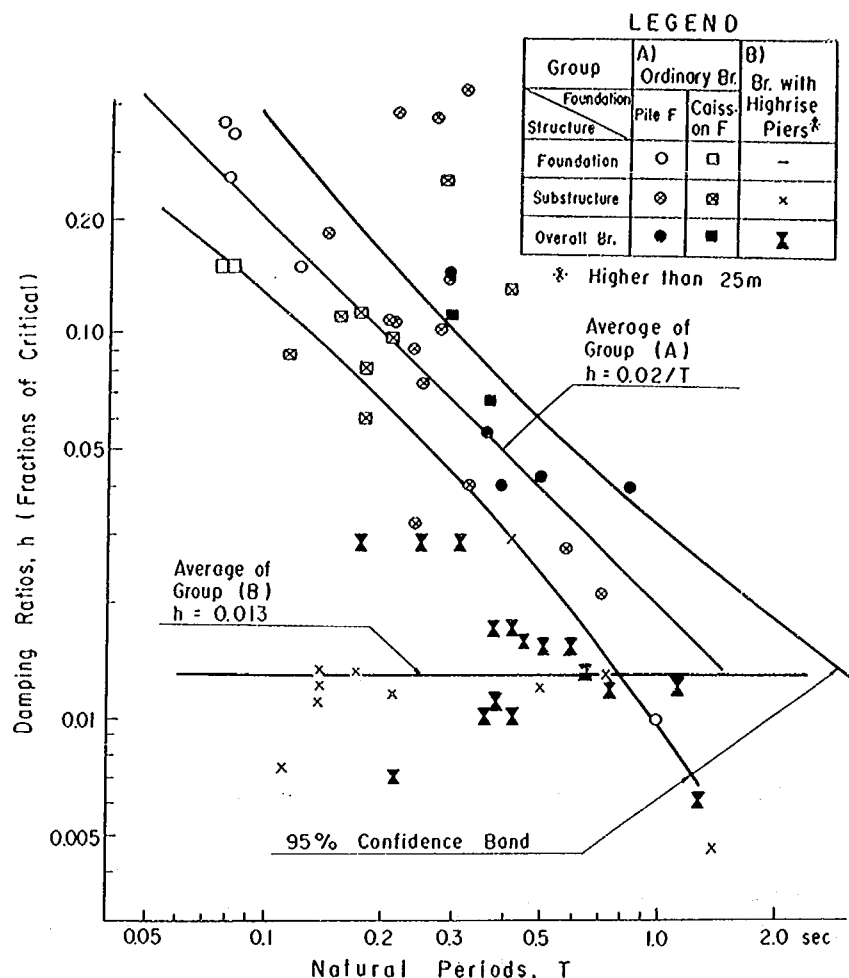


Fig. 3. 34 Relationship between natural period and damping ratio for bridges structures

The results of the tests for Pier 3 are tabulated in Table 3.4, together with the calculated values. The resonant frequencies are 0.77 and 4.62 Hertz in the longitudinal, and 2.38 Hertz in the transverse. The damping ratios are 0.6 to 1.0 percent of critical.

The second series were steady state forced vibration tests for the whole structure in the transverse direction by using an excitor set up on the slab of the midspan. Fig. 3.32 indicates the locations of the excitor and twenty transducers. Fig. 3.33 (A) is an example of resonance curves measured at the midpoint of the midspan. The three lowest resonant frequencies are 1.53, 2.38 and 2.63 Hertz, and the damping ratios for the

resonances are 1.3, 1.7 and 1.7 percent of critical, respectively. Fig. 3.33 (B) indicates measured mode shapes at the three resonant frequencies.

For the bridge an extensive study of dynamic analyses was conducted for evaluating dynamic properties and earthquake responses, which will be briefly described in 3.8.

Moreover, field dynamic tests of several highrise bridges on Chuo Expressway and Tomei Expressway of the Japan Highway Public Corporation such as Yokobuki Bridge, Tsuru-kawa Bridge, Sakawa-gawa Bridge, etc. which are similar to the Sokozawa Bridge, have been carried out.

Table 3.4 Test Results for Pier 3 at the Sokozawa Bridge

Outline of Experiment	Direction		Longitudinal	Transverse		Remarks
	Excitor		40t - Excitor	1st - Excitor	2t - Rocket	
	Date		Oct. 11, 1967	Oct. 12, 13, 1967	Oct. 14, 1967	
Results Experimented	Order of Modes		1st	1st	1st 2nd	
	Resonant Frequency		2.38 Hz	0.77 Hz	0.77 Hz 4.62 Hz	
	Period		0.42 sec	1.30 sec	1.30 sec 0.62 sec	
	Damping Ratio		1.0%	0.7%	0.6% 0.7%	From Free Damped Oscillation
			less than 2%	0.6%	- -	From Resonance curve
	Made Shape		Bending Deformation of Cantilever	Bending Deformation of Cantilever	Bending Deformation of Cantilever	
	Maximum Amplitude	Acceleration	190 gal	7.0 gal	3.45 gal 2.8 gal	Maximum Amplitude at the Crest of Pier
		Displacement	0.95cm	0.30cm	0.15cm 0.0033cm	
Results calculated	Natural Frequency		2.58 Hz	0.710 Hz		4.45 Hz
	Natural Period		0.388 sec	1.41 sec		0.225 sec

General Features of Dynamic Properties of Highway Bridges

In view of the results of extensive field dynamic tests carried out between 1958 and 1969 for twenty six highway bridges (three of them were briefly presented above), E. Kuriyayashi and T. Iwasaki¹¹⁶⁾ summarized dynamic properties of highway bridges for horizontal vibration, especially the relationship between natural periods and damping ratios.

Fig. 3.34 indicates the relationship between natural periods (periods for resonance phenomena) and corresponding damping ratios (fractions of critical), for all the meaningful data from the twenty six bridges. In the figure Group A denotes results of fundamental natural oscillations for ordinary bridges, and Group B denotes those of first to third natural oscillations for bridges having highrise piers higher than 25 meters above the crest of the foundations. The field dynamic tests were conducted for foundations without piers, substructures (piers), or overall bridge structures. In Fig. 3.34, the data are classified depending on the varieties of structures (foundations, substructures or overall structures) and the kinds of foundations (footings, piles or caissons).

Fig. 3.35 denotes the natural period-damping ratio relation for only Group B. In the figure numerals indicate orders of natural oscillations, and data from one structure were linked by a solid line. Fig. 3.36 indicates the relationship between height of piers and fundamental natural periods, for the horizontal motions of the overall bridge structures, together with the corresponding isolated piers. From the results the followings may be concluded.

- (1) Fundamental natural periods in the horizontal motions are about 0.07 to 1.0 second for foundations, 0.1 to 1.3 second for piers and 0.3 to 1.2 second for overall bridge structures. Fundamental periods of bridge structures get longer, as piers become higher.
- (2) Damping ratios of bridge structures in horizontal motions are approximately $0.02/T$ for ordinary bridge structures including foundations, piers and overall structures (where T denotes the fundamental natural periods in seconds of the structures), and 0.013 for highrise piers and bridges with highrise piers.
- (3) The above are the results of field dynamic tests within small amplitudes of vibration.

From the results for the Shinkatsushika Bridge it seems that dynamic properties of bridge structures vary with slight changes of amplitudes of vibration. Consequently, dynamic properties for large amplitudes may differ from the above.

It is expected, however, that the natural periods get longer and the damping ratios become larger with an increase of amplitudes.

3.7 Measurements of Strong-Motion Earthquakes at Bridges

Network of Measurements of Strong-Motion Earthquakes at Highway Bridges in Japan

In Japan the observation of strong-motion earthquakes for engineering structures was initiated after manufacturing of SMAC-type accelerographs was commenced in 1953. As of March, 1969, the number of strong-motion accelerographs installed on engineering structures is 510 totally, as shown in Table 3.5⁶⁵⁾. Among them 106 accelerographs are equipped on highway bridge structures and on the grounds near those bridges, and the number increased up to 119 as of March, 1970⁷²⁾. Fig. 3.37 indicates a network of strongmotion observation at 57 highway bridges (119 accelerometers, 61 on structures and 58 on grounds) as of March, 1970. Installation of the instruments is generally conducted by individual organizations such as the Ministry of Construction, the Hokkaido Development Agency, Prefectures and Highway Public Corporations, which are in charge of construction and maintenance of the structures.

In addition to the 119 SMAC-type accelerographs on 57 bridges, seven bridges (three of them have SMAC-type accelerographs at the same time) install electro-magnetic-type seismographs which are also to record dynamic behavior of the bridges. Two of them have earth pressure meters in the foundations as well as accelerometers.

Moreover, in connection with the proposed extensive projects such as the Honshu-Shikoku Bridges and the Tokyo Bay Loop Highways, there are several stations where the dynamic behavior of subsoils during earthquakes are being measured using downhole seismometers (some are deeper than 100 m) as well as seismometers and/or SMAC accelerographs on the surface. The observation is being carried out to provide appropriate information for the earthquake-re-

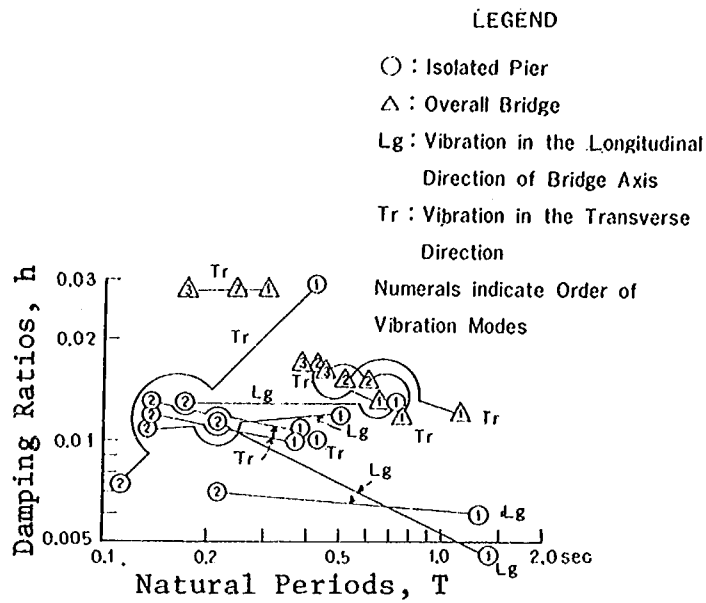


Fig. 3.35 Relationship between natural period and damping ratio for bridges with highrise piers

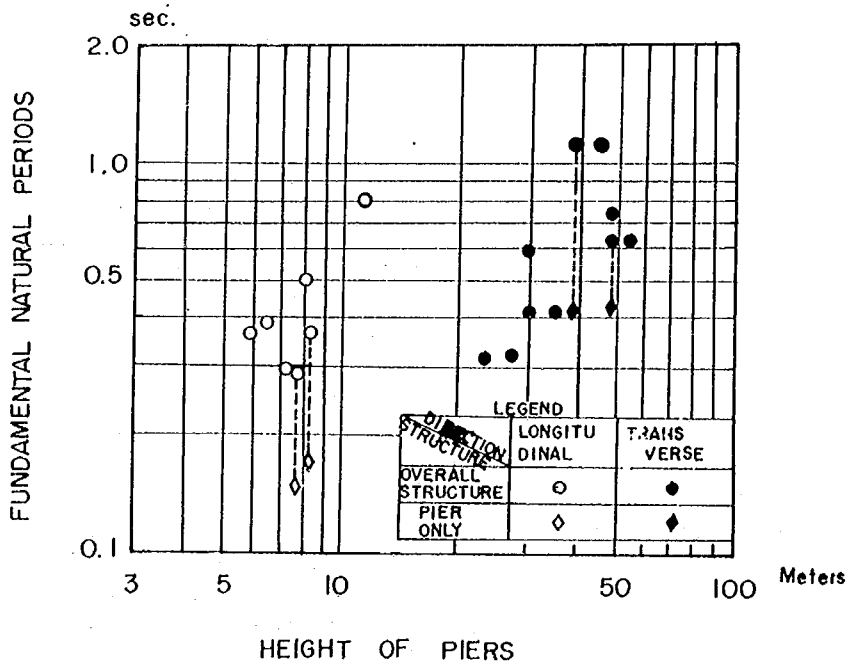


Fig. 3.36 Relationship between height of pier and fundamental natural period

Table 3.5 Numbers of Strong-Motion Accelerographs

(As of March 31, 1969)

Number of Accelerographs Installed at	At Structures	On Grounds	TOTAL
Building	192	10	202
Bridge	55	51	106
Railway	20	36	56
Harbor	3	43	46
Telephone Office	37	0	37
Power Plant	12	1	13
Atomic Power Plant	5	14	19
Dam	11	7	18
River	2	4	6
Road	0	7	7
TOTAL	337	173	510

sistant design of the huge bridges and other structures under consideration.

The records from the abovementioned instruments set up on bridge structures have been published annually by the Public Works Research Institute since 1966, together with those from other public work like highway tunnels, dam structures, river and coastal embankments, etc. (57), (58), (60), (64), (72). The followings are the general features of the results, and individual results of strong-motion earthquake measurements at several bridges (63), (67), (68).

General Features of Earthquake Response of Highway Bridges

To get better understanding of dynamic characteristics of highway bridges during actual earthquakes, 119 SMAC-type accelerographs at 57 bridges and electro-magnetic-type seismographs at 7 bridges were installed as of March, 1970, as mentioned in the preceding article. Most of these bridges have two (or three) sets of accelerographs, one on the ground surface near the bridge and one (or two) on the crests of representative substructures. The ground locations are chosen by considering topography and subsoil conditions at the bridge site and the

ground observation point, and the distance from the bridge.

Accelerographs at 28 bridges out of the above, have already triggered earthquake records which are meaningful in assessing dynamic behavior of bridges. Table 3.6 summarizes the outlines of these bridges. The subsoil conditions at these bridges are found to be very various, from soft clayey soils to hard rocks. Types of their foundations are spread foundations, caisson foundations (involving pneumatic caisson foundations and open caisson foundations), or pile foundations. Dimensions of these bridges are mostly ordinary ones in Japan; namely, the heights of their substructures above the ground level are less than 20 m, superstructures are of girders or trusses with spans less than 100 m and they are designed considering seismic coefficients of 0.2 to 0.3. Among them, however, two bridges have higher substructures (both 42 m) and one bridge is of trussed Langer with span length of 150 m. Fundamental natural periods of these bridges are estimated about 0.2 to 1.0 seconds in the horizontal direction.

Table 3.7 tabulates descriptions of earthquakes and maximum accelerations on grounds with those at crests of substructures of these bridges (except the Ochiai Bridge which will be

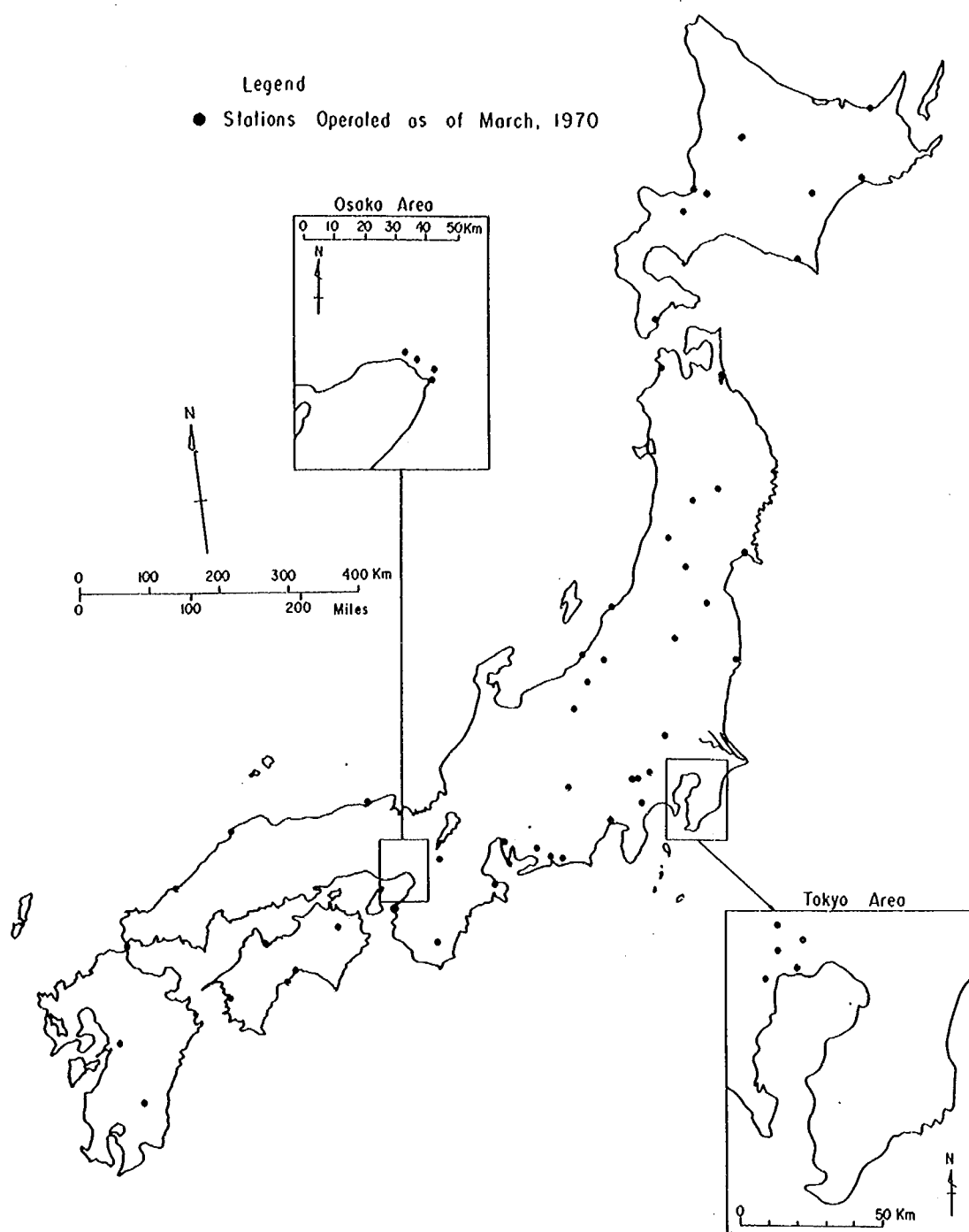


Fig. 3. 37 Network of strong-motion accelerographs at highway bridges

Table 3.6 Outline of Twenty Eight Bridges where Seismic Records are Available

Bridge No.	Name of Bridge	Structural Features					Numbers of Accelerographs			Dynamic Properties in the Longitudinal Direction		Number of Structural Location	Remarks
		Total Length	Width	Superstructure	Foundation	Soil	On Ground	At Bridge		Period (sec)	Damping Ratio		
								Pier*	Abutment*				
1	Ajigawa	1,368m	33 m	Box Girder	Caisson	Silt	1	1(8.1)m		0.5	0.1	1-1	Pier 1
								1 (12.6)		0.8	0.1	1-2	Pier 1
2	Amagasaki	280	28.3	Plate Girder	Caisson	Silt	1	1 (4.2)		0.6	0.1	2	Viaduct
3	Chiyoda	706	6	Warren Truss	Caisson	Sand	1	1 (9.5)				3-1	
				RC Girder	Spread				1 (4.0)			3-2	
4	Date	288	7	Warren Truss	Caisson	Gravel	1	1 (9.7)		0.5	0.1	4	
5	Fumimaki	144	7.6	RC Girder	Caisson	Sand	1	1 (5.8)				5	
6	Hirai	622	20	Gerber Box Girder	Caisson	Silt	1	1 (8.8)				6-1	Pier 5
								1 (8.8)				6-2	Pier 6
7	Horman	140	7.5	RC Girder	Caisson	Gravel	1	1 (4.8)				7	
8	Ichinohashi	—	8.8	Plate Girder	Cast-in-place RC Pile	Sitstone	1	1 (16.0)				8	Viaduct
9	Ishiseto	82	6	Howe Truss	Spread	Sand	1	1 (12.2)				9	
10	Ishizuchi	211.55	6	PC Girder	Caisson	Gravel	1	1 (5.5)				10	
11	Itajima	125.2	6	Plate Girder	Caisson	Silt	1	1 (5.0)		0.55	0.1	11	
12	Kinokawa	747.35	10.5	Box Girder	Caisson	Sand	1	1 (16.0)				12	
13	Myoken	237.70	8	Composite Girder	Steel Pile	Clay	1	1 (5.0)				13	
14	Nishiara	444.6	15	Gerber Plate Girder	Caisson	Sand	1	1 (6.7)				14-1	Pier 7
					Steel Pile			1 (7.4)				14-2	Pier 9
15	Ochiai	948	6	Gerber Plate Girder	Caisson	Gravel	2	1 (6.5)		0.35	0.1	15	
16	Otanoshike	220.8	11.8	Composite Girder	Steel Pile	Sand	1	1 (4.5)				16	
17	Otome	275	6	Plate Girder	Caisson	Gravel	1	1 (4.8)				17	
18	Sakaigawa	227.1	10.45	Warren Truss	Spread	Rock	1	1 (42.0)				18	
19	Shinkatsushika	442	17.7	Box Girder	Caisson	Medium Sand	1	1 (8.3)		0.36	0.07	19	
20	Shintonegawa	654	10.85	Plate Girder	Steel Pile	Sand	1	1 (7.5)				20	
21	Shitoku	101	7	Trussed Langer	Spread	Rock	1	1 (42.0)				21	
22	Soka	—	8.2	Composite Girder	Steel Pile	Clayey Silt	1	1 (6.5)				22	Span Congthof 150m
23	Taira	141.6	19.3	Plate Girder	Steel Pile	Silt	1		1 (6.0)			23	Viaduct
24	Takatsu	325.6	10.2	Composite Girder	Caisson	Fine Sand	1	1 (6.0)				24	
25	Toyohama	561	6.8	Composite Girder	Steel Pile	Gravel	1	1 (6.6)				25	
26	Ubakubo	481.79	25.5	Plate Girder	Cast-in-place RC Pile	Gravel	1	1 (18.0)				26	
27	Uonuma	205.6	7	Plate Girder	Caisson	Gravel	1	1 (13.4)		0.2	0.1	27	
28	Yoshida	270.2	22	Box Girder	Caisson	Sand	1	1 (7.8)		0.2	0.1	28-1	Pier 1
								1 (7.8)		0.3	0.1	28-2	Pier 2

* Values in the parentheses indicate the height of substructure above the ground surface.

Table 3.7 Comparison of Ground Accelerations (a_g) and Response Accelerations (a_R) at Bridges from Measurements during Earthquakes

Bridge No.	Name of Bridge	Date of Occurrence of Earthquake	Magni- tude	Depth of Hypo- center (km)	At Bridge Site		Maximum Accelerations (gals)						No. of Structural Location	Data No.
					I	Epicentral Distance (km)	Longitudinal		Transverse		Vertical			
							a _g	a _R	a _g	a _R	a _g	a _R		
1	Ajigawa	Mar. 27, 1963	6.9	0	4	121	21.9	67.0	34.0	25.0	14.0	6.0	1-1	1
2	Amagasaki	Mar. 27, 1963	6.9	0	4	121	28.0	28.0		25.0		—	1-2	2
3	Chiyoda	Mar. 12, 1967	4.5	10	3	35	32.5	31.3	25.0	41.3	5.3	7.5	3-1	4
								33.8		23.8		10.6	3-2	5
		July 5, 1967	4.1	170	3	71	33.8	58.8	28.8	27.5	5.3	13.8	3-1	6
								42.5		25.6		15.0	3-2	7
		Sept. 19, 1967	—	90	3	195	23.8	36.5	26.3	33.8	5.3	6.3	3-1	8
								20.0		27.5		6.3	3-2	9
		May 16, 1968	7.9	20	4	241	87.5	131.3	72.5	112.5	25.0	25.0	3-1	10
								91.3		75.0		31.3	3-2	11
	May 16, 1968	7.5	40	3	172	37.5	77.5	31.3	50.0	18.8	17.5	3-1	12	
							43.8		36.3		18.8	3-2	13	
4	Date	Jan. 17, 1967	6.3	30	3	151	22.1	28.8	19.1	43.8	—	8.8	4	14
		July 5, 1968	6.4	50	4	169	23	20	18	30	5	5	4	15
5	Fumimaki	Dec. 24, 1963	—	—	—	—	37.0	43.9	27.5	22.4	—	—	5	16
6	Hirai	Mar. 19, 1967	—	80	3	41	15.6	25.0	17.5	16.3	10.0	—	6-1	17
		Mar. 6, 1968	5.2	50	3	39	30	20	25	18	8	8	6-1	18
								58		13		8	6-2	19
		July 1, 1968	6.1	50	4	52	58.1	108	48.8	63.0	25.0	30	6-1	20
								128		60.0		30	6-2	21
		Oct. 8, 1968	—	70	3	34	30	48	35	35	5	15	6-1	22
						65	28	15		6-2		23		
7	Horoman	May 16, 1968	7.9	20	5	157	68.8	72.5	51.3	90.0	23.8	36.3	7	24
		May 16, 1968	7.5	40	5	74	56.3	68.8	43.8	87.5	18.8	25.0	7	25
8	Ichinohashi	Mar. 6, 1968	5.2	50	3	46	20	40	20	50	5	10	8	26
		July 1, 1968	6.1	50	4	46	38	140	48	173	18	25	8	27
		Oct. 8, 1968	—	70	3	39	55	120	55	125	13	28	8	28
9	Ishiseto	Feb. 21, 1968	6.1	0	4	39	22.5	50	20	25	10	5	9	29
		Mar. 25, 1968	5.7	0	3	39	22.5	50	22.5	35	10	5	9	30
		Apr. 1, 1968	7.5	30	4	131	25	70	30	35	15	10	9	31
10	Ishizuchi	Apr. 1, 1968	7.5	30	4	182	35	83	55	65	10	23	10	32
		Aug. 6, 1968	6.6	40	4	89	63	95	63	78	20	23	10	33
11	Itajima	Jan. 1, 1967	4.6	10	2	248	27.5	18.8	17.5	28.8	—	—	11	34
		Apr. 1, 1968	7.5	30	4	101	169.9	213	186.2	375	43	55	11	35
		Apr. 1, 1968	6.3	0	3	99	34	35	36	65	10	13	11	36
		Aug. 6, 1968	6.6	40	5	11	437.5	200	360.9	233	140	100	11	37
		Aug. 6, 1968	4.1	30	3	19	55	30	55	65	10	8	11	38
	Aug. 6, 1968	4.8	40	3	18	50	100	60	65	8	20	11	39	
12	Kinokawa	Mar. 30, 1968	5.0	0	4	6	80	115	70	55	85	30	12	40
13	Myoken	Apr. 1, 1968	7.5	30	3	337	35	45	48	48	20	8	13	41
		Aug. 6, 1968	6.6	40	3	226	18	25	18	25	5	20	13	42
14	Nishiarai	July 1, 1968	6.1	50	4	41	80	95	48	33	15	10	14-1	43
		Oct. 8, 1968	—	70	3	42	40	40	50	38	8	5	14-1	44
							33	50		10		14-2	45	
16	Otanoshike	May 16, 1968	7.9	0	4	257	31.3	45.0	41.3	43.8	12.5	12.5	16	46
17	Otome	Nov. 28, 1967	—	130	2	68	16.1	23.7	10.0	8.7	—	—	17	47
18	Sakaigawa	July 1, 1968	6.1	50	4	45	25	85	55	125	10	15	18	48
19	Shin- katsushika	Mar. 2, 1967	—	85	3	36	15.0	38.8	21.3	28.7	—	—	19	49
		Nov. 10, 1967	—	80	3	42	18.9	43.8	14.4	48.8	—	6.0	19	50
		July, 1, 1968	6.1	50	4	47	37.0	50	38.6	55	10	10	19	51
		Oct. 8, 1968	—	70	3	37	35	38	28	45	5	10	19	52
20	Shintonegawa	July 1, 1968	6.1	50	4	31	75	90	65	53	25	13	20	53
21	Shitoku	July 1, 1968	6.1			136	30	40	25	98	18	13	21	54
22	Soka	Mar. 7, 1968	5.1			42	18	60	25	35	5	5	22	55
		July 1, 1968	6.1			38	50	75	50	65	15	8	22	56
		Oct. 8, 1968	—			45	35	45	48	63	10	5	22	57
23	Taira	Feb. 26, 1968	5.4			64	50	23	40	35	20	10	23	58
		May 16, 1968	7.9			473	23	8	25	15	8	5	23	59
24	Takatsu	Apr. 1, 1968	7.5			271	20	118	25	75	5	20	24	60
		Aug. 6, 1968	6.6			159	25	95	33	83	8	20	24	61
25	Toyohama	May 9, 1969	5.6			58	30	60	23	85	8	10	25	62
26	Ubakubo	July 1, 1968	6.1	50	4	32	35	48	45	178	15	15	26	63
27	Uonuma	Jan 9, 1966	5.2			42	28.0	65.0	32.5	27.5	—	—	27	64
		Sept. 8, 1966	5.1			5	50.0	63.0	45.0	63.0	14.0	13.0	27	65
28	Yoshida	Aug. 19, 1961	7.0			135	15.8	38.0	19.0	—	—	—	28-1	66
								43.2		—		—	28-2	67

Note: Data for the Ochiai Bridge (No. 15) are shown in Table 3.8.

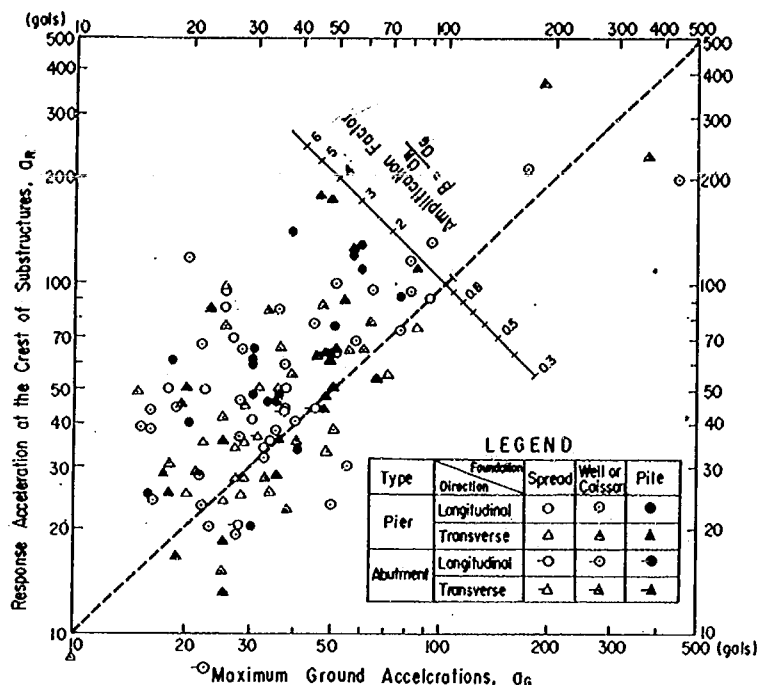


Fig. 3.38 Results of earthquake measurements at twenty seven bridges

discussed later). Fig. 3.38 compares maximum horizontal accelerations on grounds (a_g) with those at substructures (a_r). From these the followings may be indicated.

- 1) In most cases, response accelerations at crests of piers are more than those on ground surfaces.
- 2) Ratios of response accelerations at piers to ground accelerations tend to decrease, as values of ground accelerations increase.
- 3) Response accelerations at crests of abutments are usually less than ground accelerations.

Fig. 3.39 shows a result for the vertical direction. From this and Table 3.7 it is noted that vertical ground accelerations are smaller ($1/3$ to $1/2$) than horizontal ones. For vertical motions it is also indicated that response accelerations at crests of substructures are $1/2$ to 2 times ground accelerations.

Highrise Bridges ⁶³⁾

Among 28 bridges shown in Table 3.6, there are eight bridges which have substructures higher than 10m above ground surfaces. Thirteen strongmotion records are available for these eight

highrise bridges. Fig. 3.40 indicates a relationship between maximum ground accelerations (a_g) and maximum response accelerations at crests of substructures (a_r), and Fig. 3.41 shows a relationship between heights of substructures (H) and ratios of response accelerations to ground accelerations (β). From these the followings may be known.

1) Ratios of response accelerations to ground accelerations (β) are approximately 2 to 4, and are almost independent on heights of bridge substructures. In other words, there is no distinct relation between heights of bridge substructures and their responses. It is seen that magnitudes of ground accelerations have more significant effects on bridge responses, than heights of substructures.

2) For highrise bridges there are no records of experiencing severe ground motions with accelerations more than 100 gals. When subjected to moderate or weak ground motions with accelerations less than 100 gals, ratios of response accelerations (β) at highrise bridges are mostly 1 to 4. It is estimated that highrise bridges are likely to respond as linear structures for moderate ground motions.

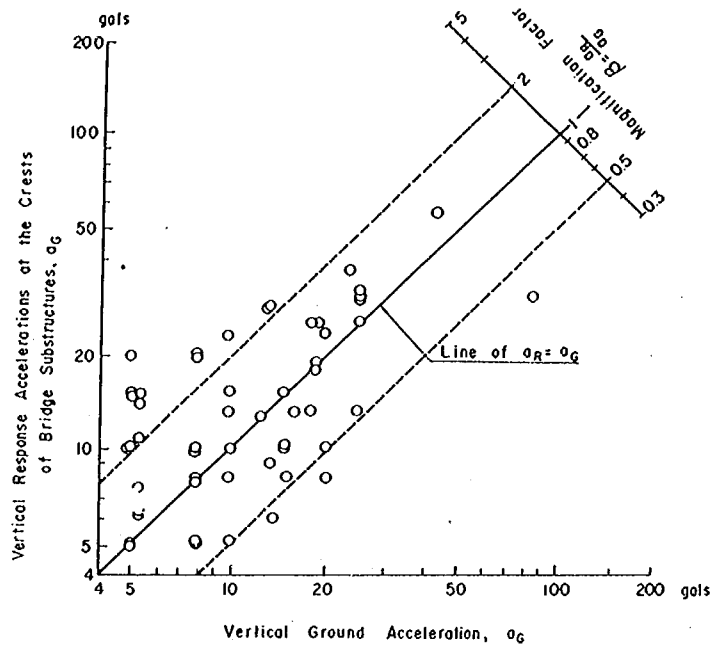


Fig. 3.39 Relation between ground acceleration and response acceleration at the crest of bridge structures in the vertical direction

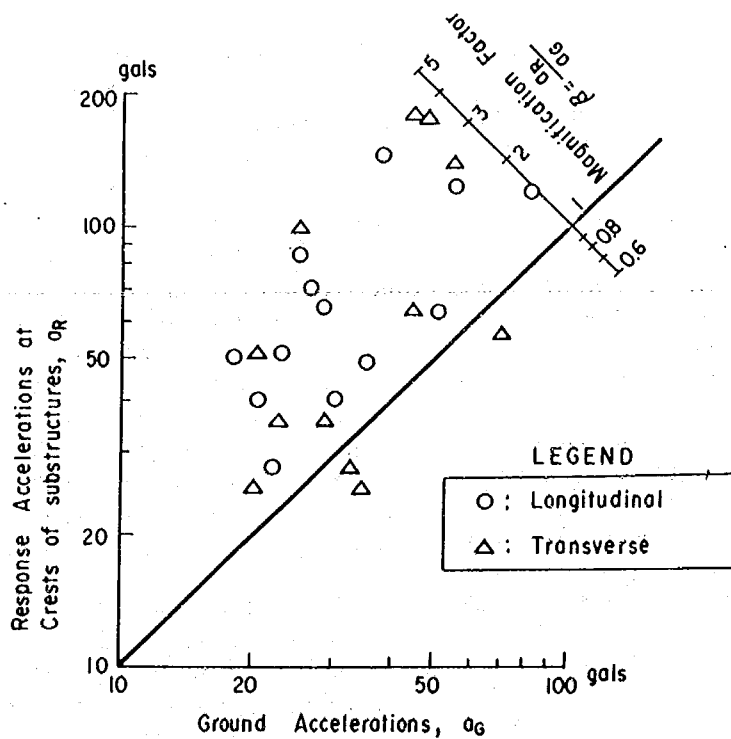


Fig. 3.40 Results of earthquake measurements at eight bridges with highrise piers (over 10 m above ground surface)

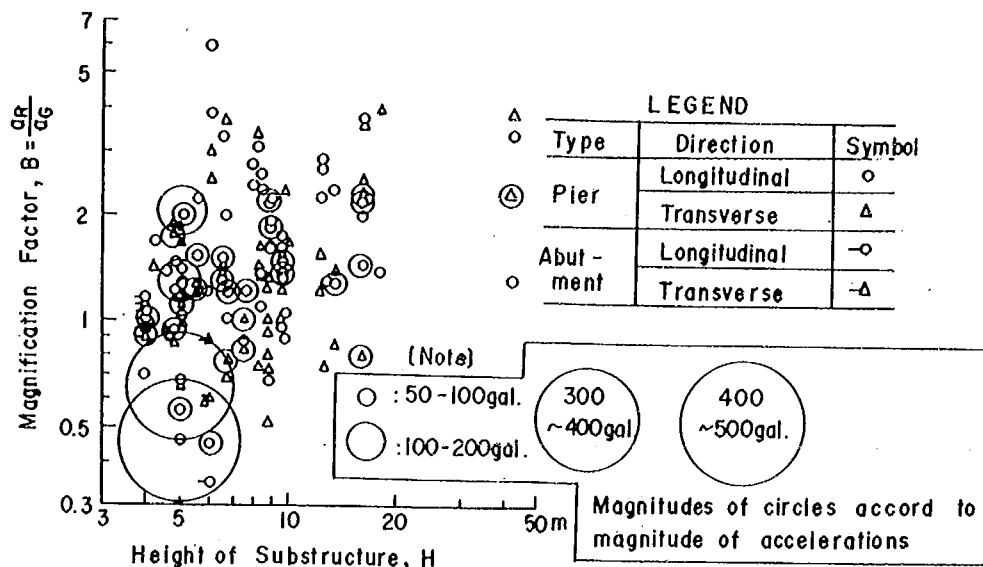


Fig. 3.41 Magnification factors measured vs. height of bridge structures

Ochiai Bridge 54), 55), 58), 59), 63), 68)

This bridge, completed in 1966, is on one of Nagano prefectural roads, and is located near the confluence of two rivers (Chikuma and Sai) in Wakaho, Nagano City. The general view of the bridge and the outline of a typical pier (Pier 11) are shown in Figs. 3.42 and 3.43, respectively. The subsoil near the bridge is an alluvial layer of consolidated coarse granular sediment, as shown in Fig. 3.43.

The right-bank and left-bank abutments are reinforced structures rested on caisson foundation and concrete pile foundation, respectively. Fifteen piers (P1 to P15) are reinforced concrete structures with caisson foundations, and the six remaining piers near the left-bank are similar structures with concrete pile foundations. The superstructures, total length of 948 m and with of 6m, are consisted of 12-span steel Gerber girders and 10-span steel simple girders, as shown in Fig. 3.42.

The bridge happens to be situated about 7 km north from Matsushiro Town, where a group of numerous earthquakes occurred since the middle of 1965. A peak of the group of earthquakes was brought about in April, 1966. Only for the month more than one hundred

thousand quakes broke off and among them some twelve thousand quakes were felt by human beings. The earthquakes can be characterised by the fact that they had very shallow hypocenters and very short periods of duration of shaking, as well as a surprising number of quakes. Fortunately magnitudes of these were not so great, yet several greater ones registered 5.0 to 5.2 on the Richter scale.

Although these earthquakes did not cause any significant damage to modern engineering structures including highway bridges, a number of strong-motion records were obtained by strong-motion seismographs set up on ground surfaces, in downholes, and at various locations of structures.

In the end of 1965 the Public Works Research Institute installed three accelerographs (Type SMAC-B2), one at the crest of Pier 11 of the Ochiai Bridge, one on the river bed nearby (see Fig. 3.43), and one on the ground surface in the vicinity of the central part of Nagano City. The principal purpose of the observation is to obtain quantitative information on seismic effects on the particular bridge and other engineering structures. By the end of 1969 the three accelerographs triggered 786 strongmotion records. Some typical examples are shown on Fig.

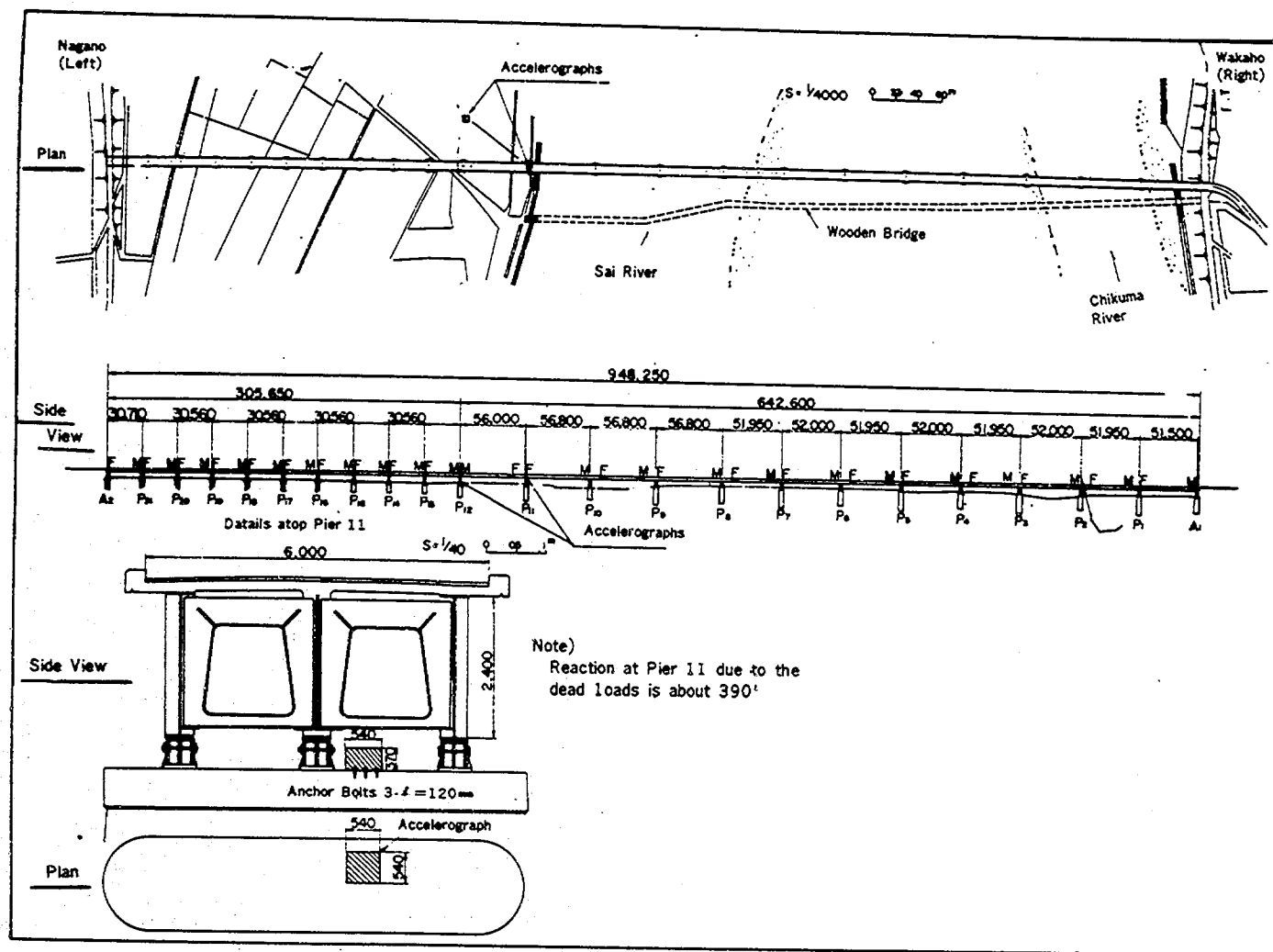


Fig. 3.42 General view of the Ochiai bridge



accelerations at the crest of the pier and on the ground in the direction of bridge axis, respectively. From the figure it seems probable that a_R is greater than a_G for smaller a_G , yet a_R gradually becomes less comparing with a_G , for larger a_G . In other words, the ratios of a_R/a_G tend to decrease in accordance with the increase of a_G . The following relation can be given empirically.

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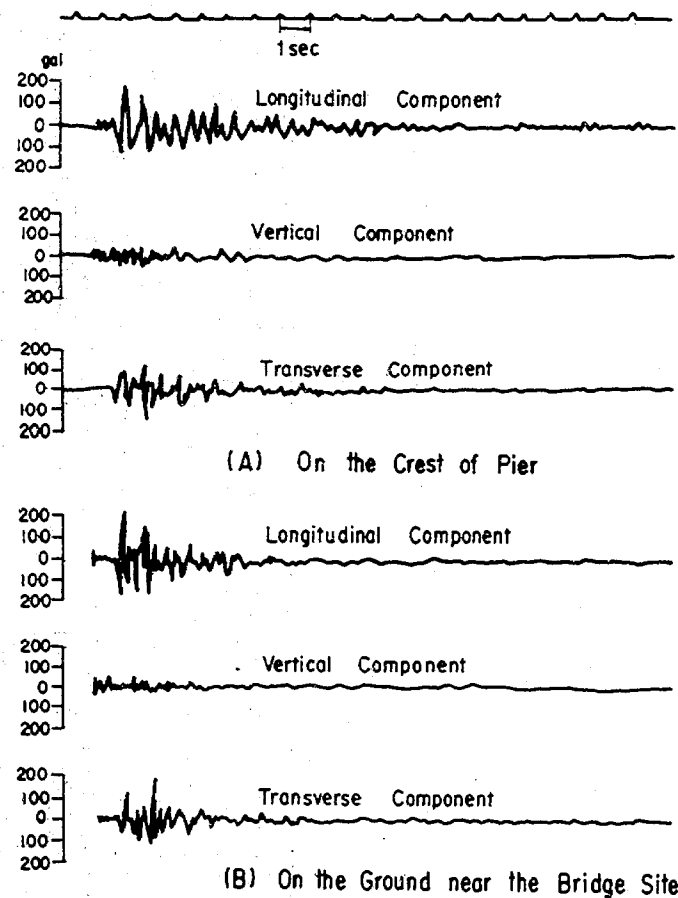


Fig. 3.44 Examples of acceleration records observed at the Ochiai bridge station

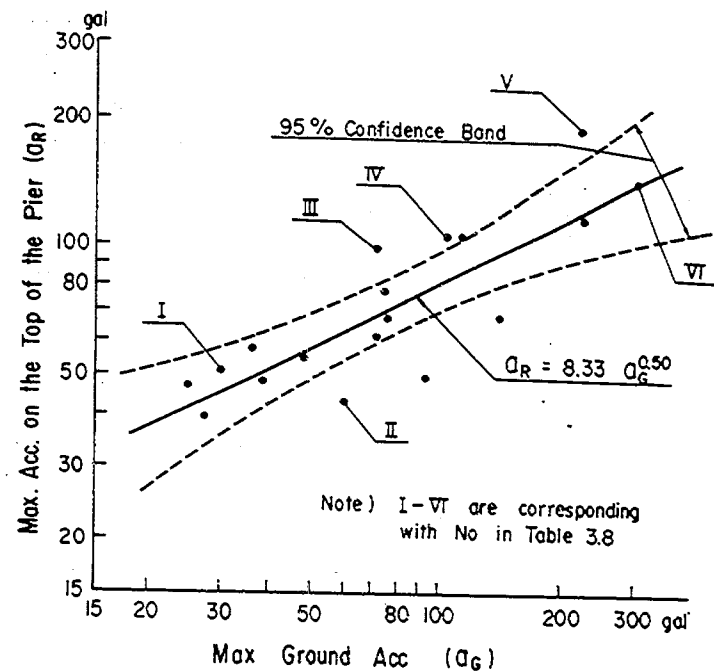


Fig. 3.45 Relation between ground acceleration and response acceleration observed at the Ochiai bridge in the longitudinal direction

Table 3.8 Observed Accelerations and Analyzed Responses at the Ochiai Bridge (Longitudinal Direction)

Earthquake		Observed Max. Acc.		Analyzed Response Acc.			Percentages of Analyzed Acc. to Observed Acc.		
No.	Date Time	Ground a_g (gal)	Top of Pier a_r (gal)	Linear a_r^L (gal)	Nonlinear Case 1 a_r^{N1} (gal)	Nonlinear Case 2 a_r^{N2} (gal)	$\frac{a_r^L}{a_r} \times 100$ (%)	$\frac{a_r^{N1}}{a_r} \times 100$ (%)	$\frac{a_r^{N2}}{a_r} \times 100$ (%)
I	Apr. 5, '66 17:52	30.0	51.3	43.7	43.7	43.7	85.2	85.2	85.2
II	Feb. 12, '66 04:05	60.0	43.8	55.1	55.1	54.0	125.8	125.8	123.3
III	May 6, '66 19:08	70.0	100.0	97.1	97.1	80.5	97.1	97.1	80.5
IV	May 28, '66 14:21	102.5	107.5	156.4	128.1	106.6	145.5	119.2	99.2
V	Apr. 5, '66 17:51	212.5	190.0	325.8	187.0	195.5	171.5	98.4	102.9
VI	Apr. 17, '66 10:21	302.5	145.0	303.4	202.7	211.8	209.2	139.8	146.1

Note) Linear: Linear System ; $T = 0.35^{\text{sec}}$, $h = 0.1$

Nonlinear Case 1 : Bi-linear System ; $T = 0.35^{\text{sec}}$, $h = 0.1$, $\eta = 0.6$, $X_y = 0.3^{\text{cm}}$

Nonlinear Case 2 : Bi-linear System ; $T = 0.35^{\text{sec}}$, $h = 0.1$, $\eta = 0.4$, $X_y = 0.15^{\text{cm}}$

The above equation suggests that the bridge apparently responds as nonlinear structure.

Table 3.8 tabulates the maximum accelerations measured on the ground and at the bridge, with respect to six representative earthquakes between February 12 and May 28, 1966. The table also indicates the results of dynamic analyses for three analytical models including nonlinear ones.

In addition to the above, extensive studies were performed by utilizing the strong-motion records obtained at the Ochiai Bridge and by conducting dynamic analyses⁶³⁾.

Itajima Bridge^{63), 68)}

This bridge, completed in 1965 in Uwajima City, Ehime Prefecture of Shikoku Island, is on one of prefectural roads. The general view of the bridge is illustrated in Fig. 3.46. The ground is of a comparatively soft alluvial layer of clayey loam or silt, as shown in the figure. The both abutments are reinforced concrete structures

with steel pile foundations (length of 18'm), and four piers are of reinforced concrete structures with open caisson foundations (depth of 16 m). The superstructures, total length of 125m consisting of 5-span (= 19.5 m + 3 × 28.4 m + 19.7 m) and width of 6m, are simple composite girders.

Immediately after its completion, a pair of accelerographs (Type SMAC-B2) were installed, one on the crest of Pier 3 and the other on the ground surface in a schoolyard located about 200 m apart. The bridge was attacked by several severe earthquakes in the past.

The Hyuganada Earthquake, which registered a magnitude of 7.5 on the Richter scale and was located under the sea 101 km south from the bridge site, broke off on April 1, 1968. The Earthquake accompanied a major aftershock. Fig. 3.47 indicates acceleration records obtained during the main shock at the two locations. The maximum horizontal accelerations were 186 gals on the ground and 375 gals at the crest of the bridge pier.

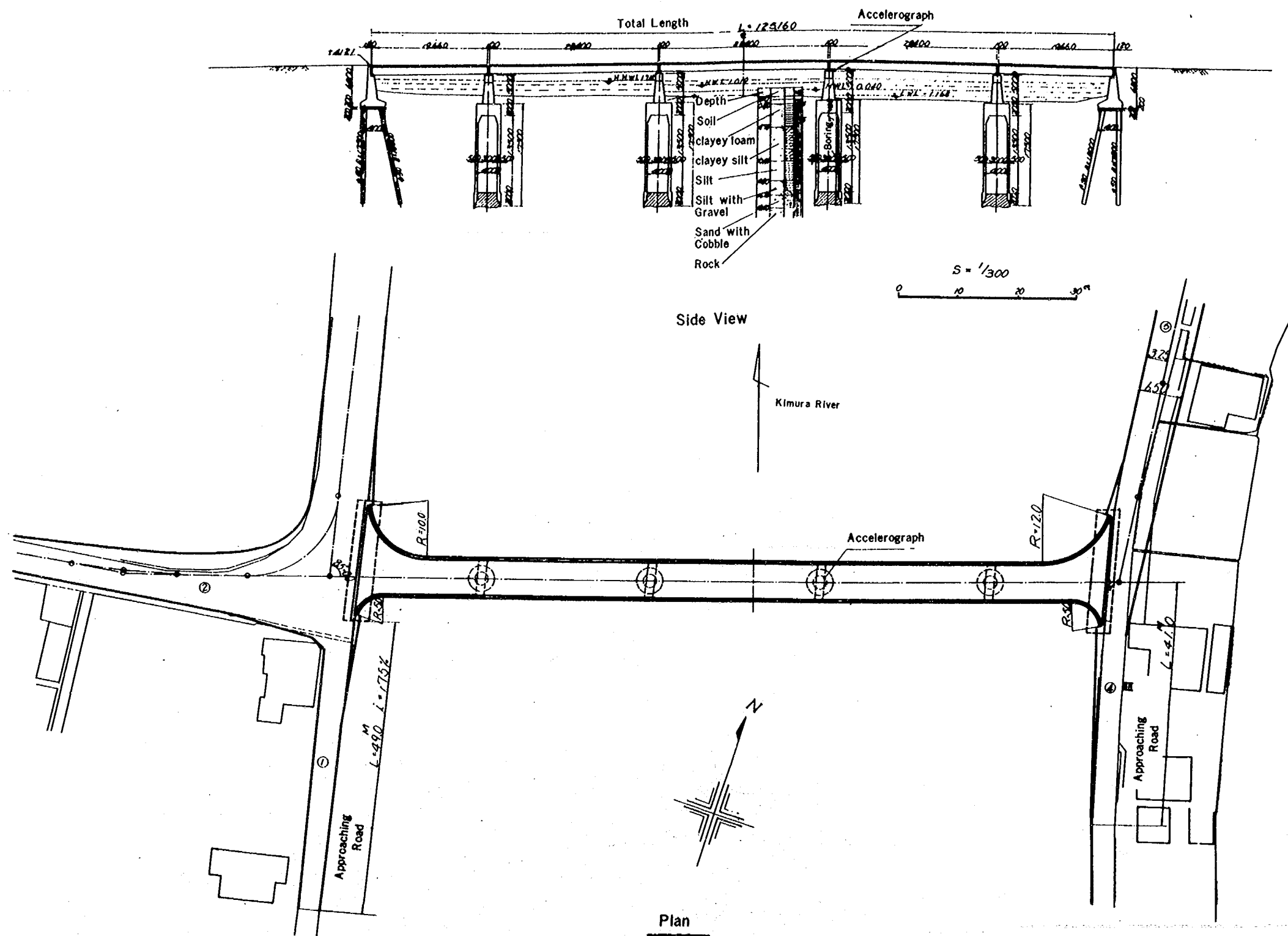
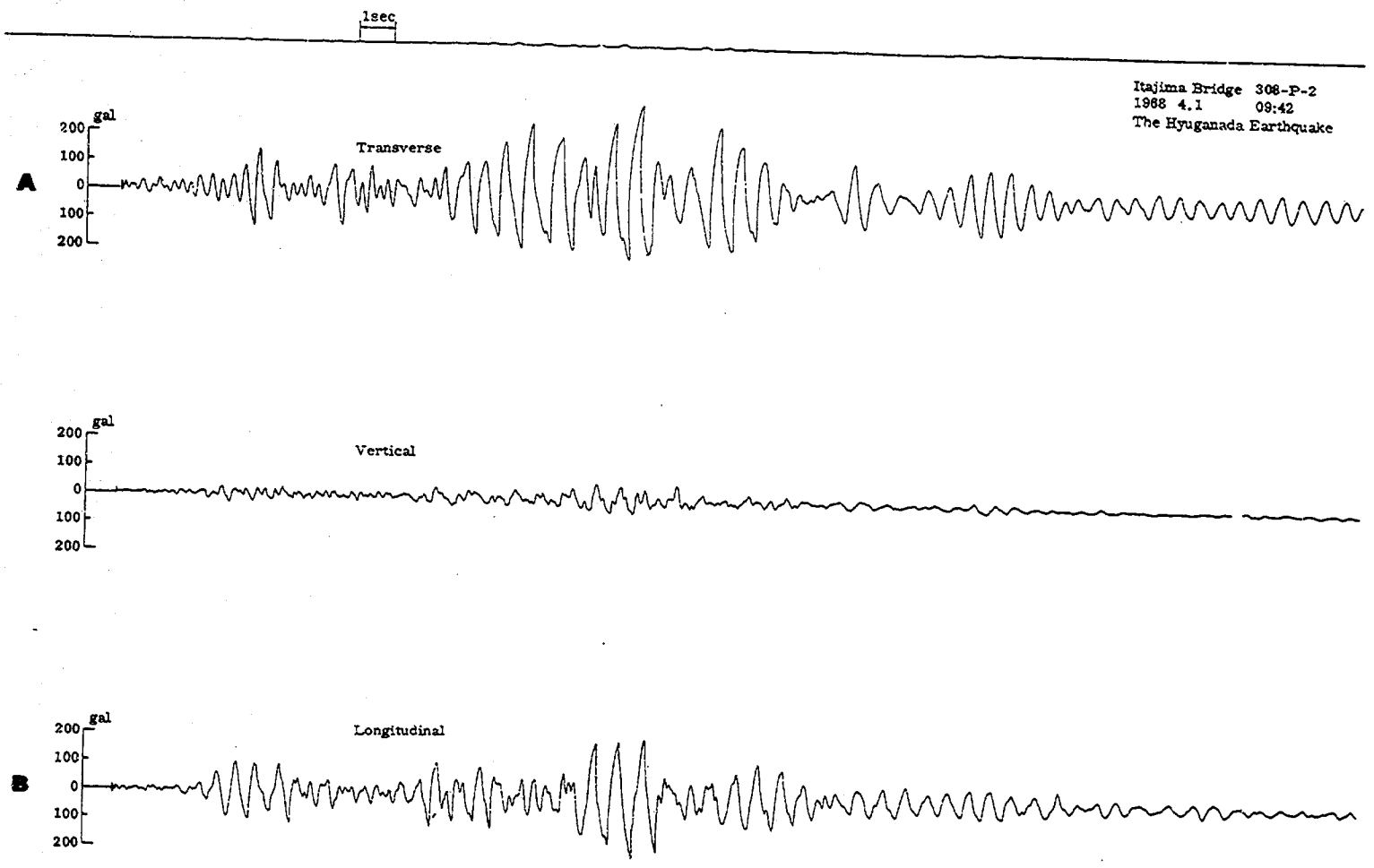
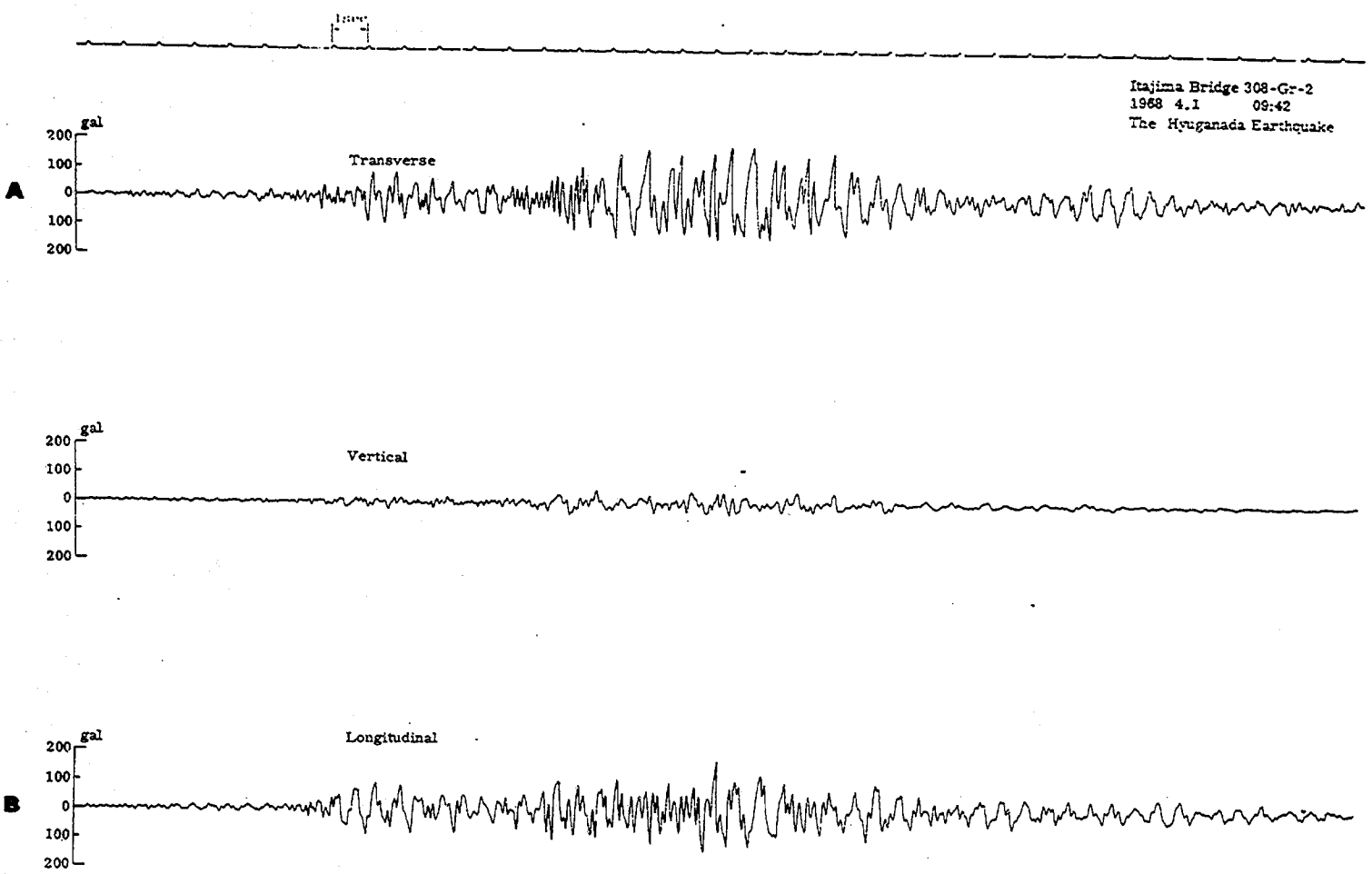


Fig. 3.46 General view of the Itajima bridge

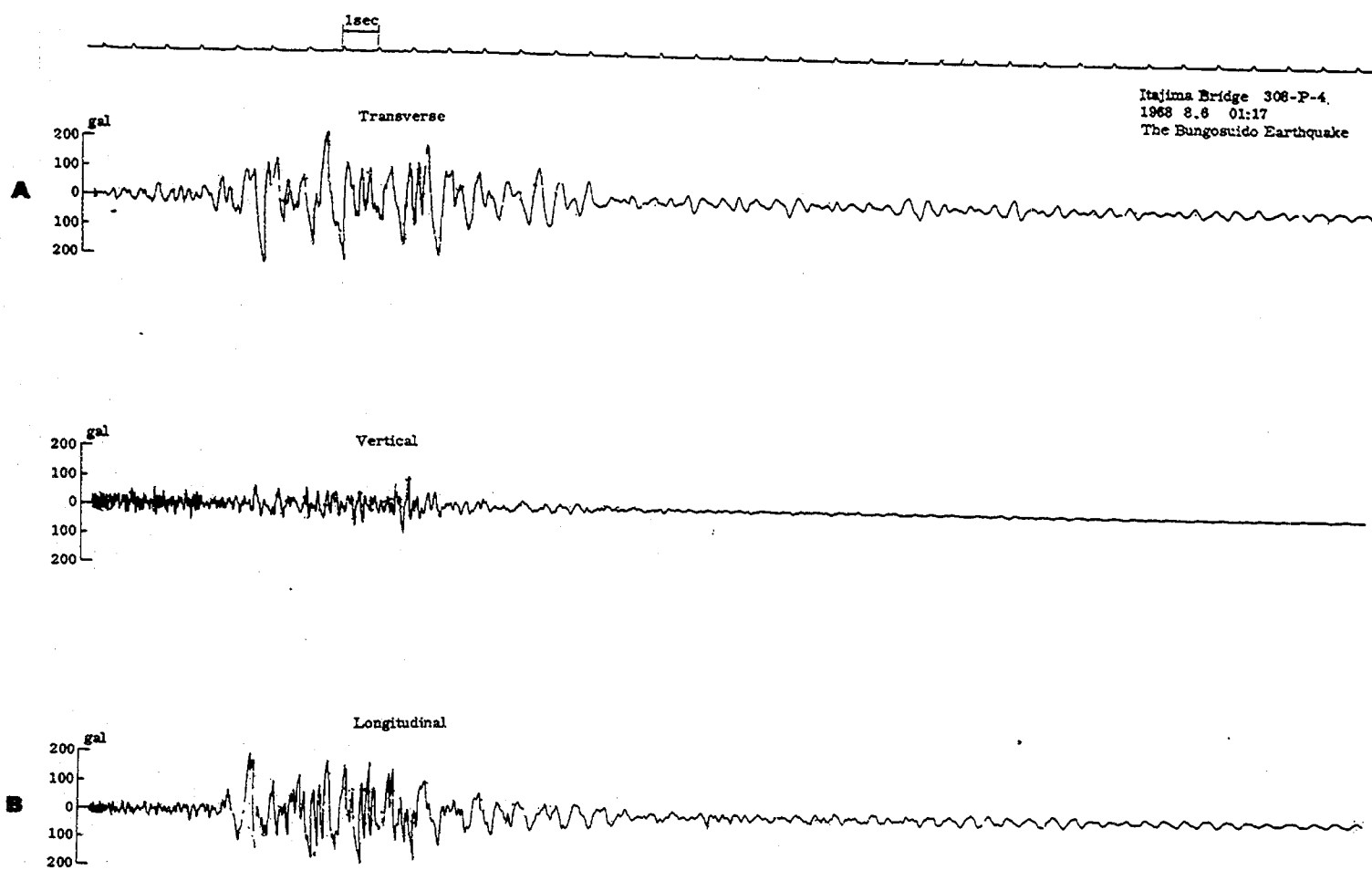


(A) at the Crest of Pier 3 (Record No. 308-P-2)

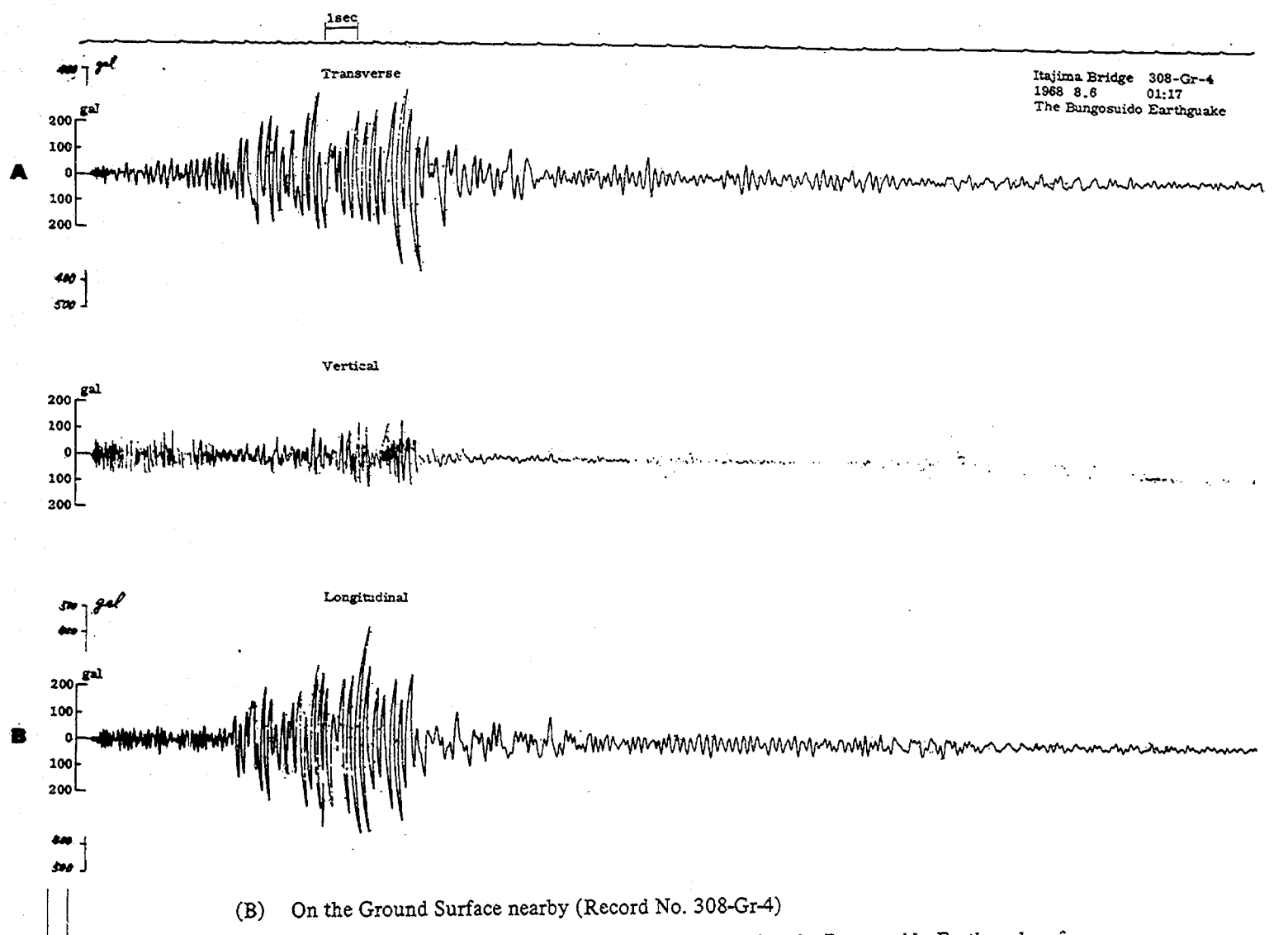


(B) On the Ground Surface nearby (Record No. 308-Gr-2)

Fig. 3.47 Acceleration records at the Itajima bridge during the Hyuganada Earthquake of April 1, 1968 [64]



(A) at the Crest of Pier 3 (Record No. 308-P-4)



(B) On the Ground Surface nearby (Record No. 308-Gr-4)

Fig. 3.48 Acceleration records at the Itajima bridge during the Bungosuido Earthquake of August 6, 1968 [64]

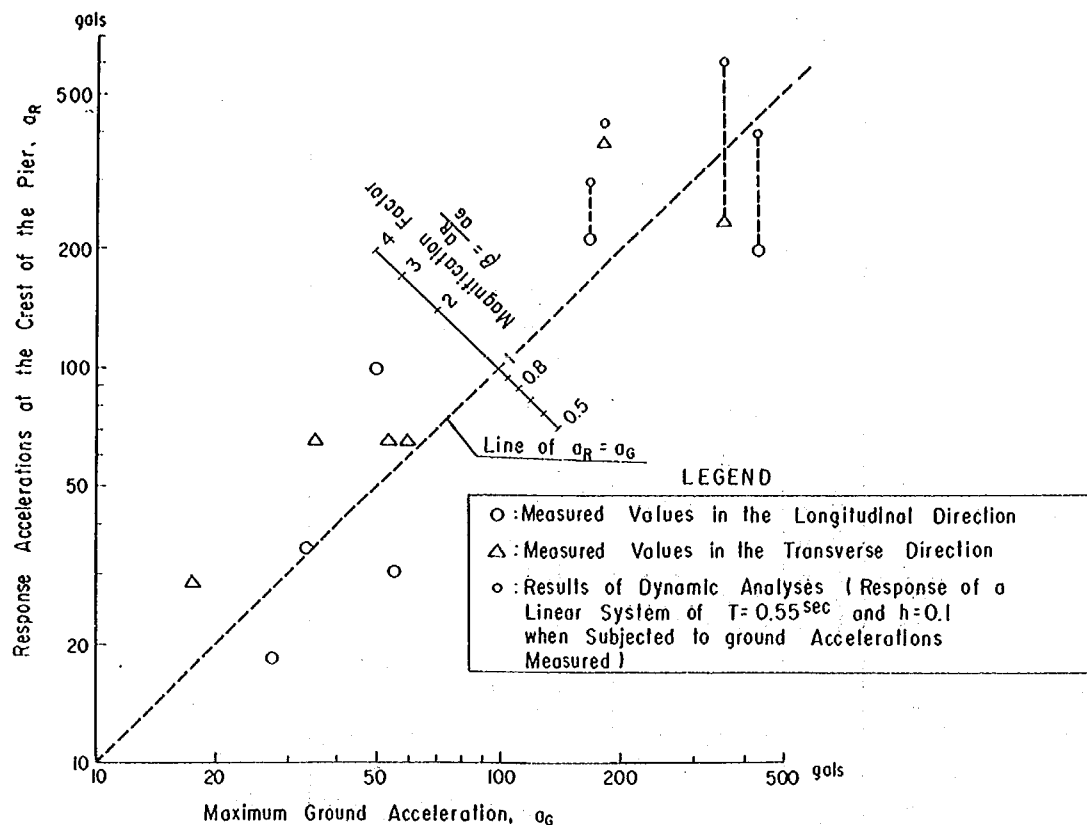


Fig. 3.49 Results of measurements and dynamic analyses at the Itajima bridge

The Bungosuido Earthquake, which registered a magnitude of 6.6 on the Richter scale and was located only 11 km from the bridge, occurred on August 6, 1968. The depth of the hypocenter was about 40 km. The earthquake accompanied six major aftershocks. Fig. 3.48 shows two records during the main shock. The maximum horizontal accelerations were 438 gals on the ground and 233 gals at the crest of the bridge pier.

Table 3.7 summarizes 3-component maximum accelerations recorded at the Itajima Bridge, as well as other bridges. In addition to the two aftershocks shown in Table 3.7, the Bungosuido Earthquake had a number of aftershocks including four major shocks (magnitudes were 4.9, 5.3, 4.2 and 4.4 on the Richter scale and maximum values of horizontal accelerations on the ground were 130, 280, 80 and 88 gals, respectively).

A relationship between maximum values of ground accelerations (a_g) and those of response accelerations (a_R) at the crest of the bridge pier,

is given in Fig. 3.49. In the figure some results on dynamic response analyses are also indicated for major four records. Response spectra were calculated for a linear single-degree-of-freedom system when subjected to strong-motions recorded on the ground. Calculated response accelerations in Fig. 3.49 were obtained assuming that the bridge will be idealized as a simple system having a natural period of 0.55 second and a damping ratio of 10% of critical.

In comparison of measured response accelerations and calculated ones, shown in Fig. 3.49, it may be known that results of linear analyses will be in good agreement with measured response accelerations at the bridge for moderate ground accelerations about 200 gals or less, but results of analyses will give greater responses than actual ones for severe ground accelerations greater than 300 gals. It should be also noted that the bridge did not sustain any damage even after it was attacked by several severe earthquakes mentioned above.

Table 3.9 Results of Earthquake Observation at the Shinkatsushika Bridge

Date of Earthquake	Magni- tude	At Bridge Site		Strong-Motion Accelerographs						Transducers Equipped in the Caisson Foundation of Pier 5						Data No.	Remarks
				Maximum Accelerations (gals)						Max. Accelera- tions (gals)			Max. Earth Pressures (kg/cm ²)				
		JMA Intensity	Epicentral Distance (km)	On ground			At Crest of Pier 5			A-1	A-2	A-3	E-1	E-3	E-4		
				L	T	V	L	T	V	L	L	L	L	L	L		
Mar. 2, 1967	—	3	36	15.0	21.3	—	38.8	28.7	—	—	—	—	—	—	—	50	
Nov. 10, 1967	—	3	42	18.9	14.4	—	43.8	48.8	6.0	17.4	27.9	8.2	0.199	0.046	0.034	51	
July 1, 1968 (Higashi- Matsuyama Eq.)	6.1	4	47	37.0	38.6	10	50	55	10	32.1	25.0	21.8	1.295	0.218	0.108	52	See Figs. 3.50~ 3.53
Oct. 8, 1968	—	3	37	35	28	5	38	45	10	23.6	17.9	12.7	—	0.112	—	53	

Notes) L: Longitudinal Direction

T: Transverse Direction

V: Vertical Direction

Data Numbers correspond to Table 3.7.

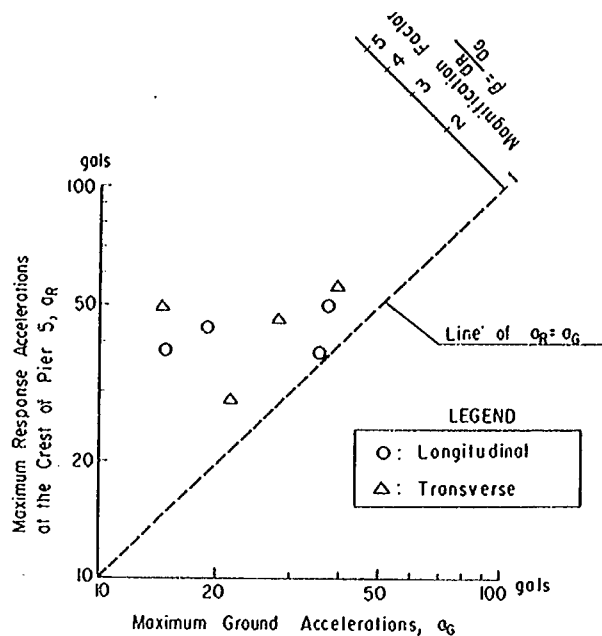


Fig. 3.50 Results of earthquake measurements at the Shinkatsushika bridge

Shinkatsushika Bridge 63), 68)

The outline of this bridge was described in 3.6. For the bridge two strong-motion accelerographs (Type SMAC-B2) are installed, one on the crest of Pier 5, and the other on the ground nearby. In addition to these two, six wire-strain gage type transducers (four accelerometers and four earthpressure meters) are equipped in the caisson foundation (depth of 21 m) of the Pier 5.

Several earthquake records were obtained for the past moderate or minor earthquakes. Table 3.9 summarizes the results of observation at the bridge. Fig. 3.50 indicates a relationship between maximum horizontal accelerations measured on the ground and at the crest of the bridge pier, from the two accelerographs. Figs. 3.51 and 3.52 show the records of two accelerographs obtained during the Higashi Matsuyama Earthquake of July 1, 1968. For the earthquake, Fig. 3.53 indicates the distribution of maximum values of accelerations and earthpressures in the longitudinal direction of the bridge axis. From the records, the followings may be noted.

(1) During earthquakes with maximum ground accelerations of 40 gals or less, accelerations at the crest of the pier were greater (1.1 to 3.3 times) than ground accelerations, yet accelerations at the caisson foundation underground were less.

(2) Although through the dynamic tests in the longitudinal direction of the bridge the fundamental period and dynamic coefficient of subgrade reaction (ratio of earthpressure to amplitude) were revealed to be 0.36 seconds and about 4 kg/cm³, respectively, during an actual earthquake with ground acceleration of about 40 gals it responded with a predominant period of 0.50 seconds, and the dynamic coefficient of subgrade reaction surrounding the caisson foundation was measured about 2 kg/cm³.

Magnitudes of amplitudes near the crest of the well foundation were estimated about 0.1 mm during the dynamic tests and about 2 mm during the earthquake. From the facts, it may be concluded that rigidity of ground soils tends to decrease for larger amplitudes of vibration.

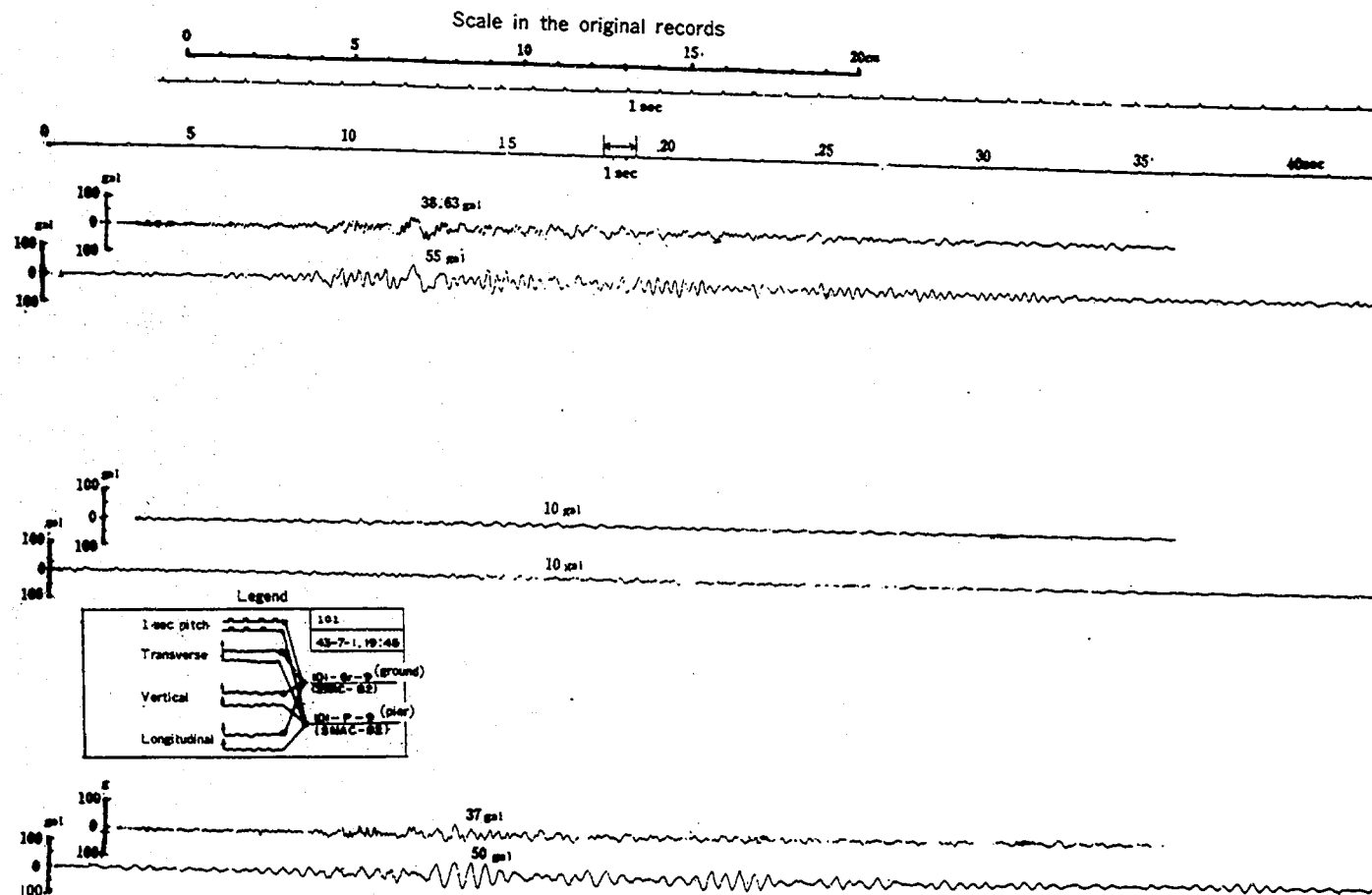


Fig. 3.51 Acceleration records at the Shinkatsushika bridge during the Higashi-Matsuyama Earthquake of July 1, 1968 (recorded by strong-motion accelerographs on the ground surface and at the crest of pier 5)

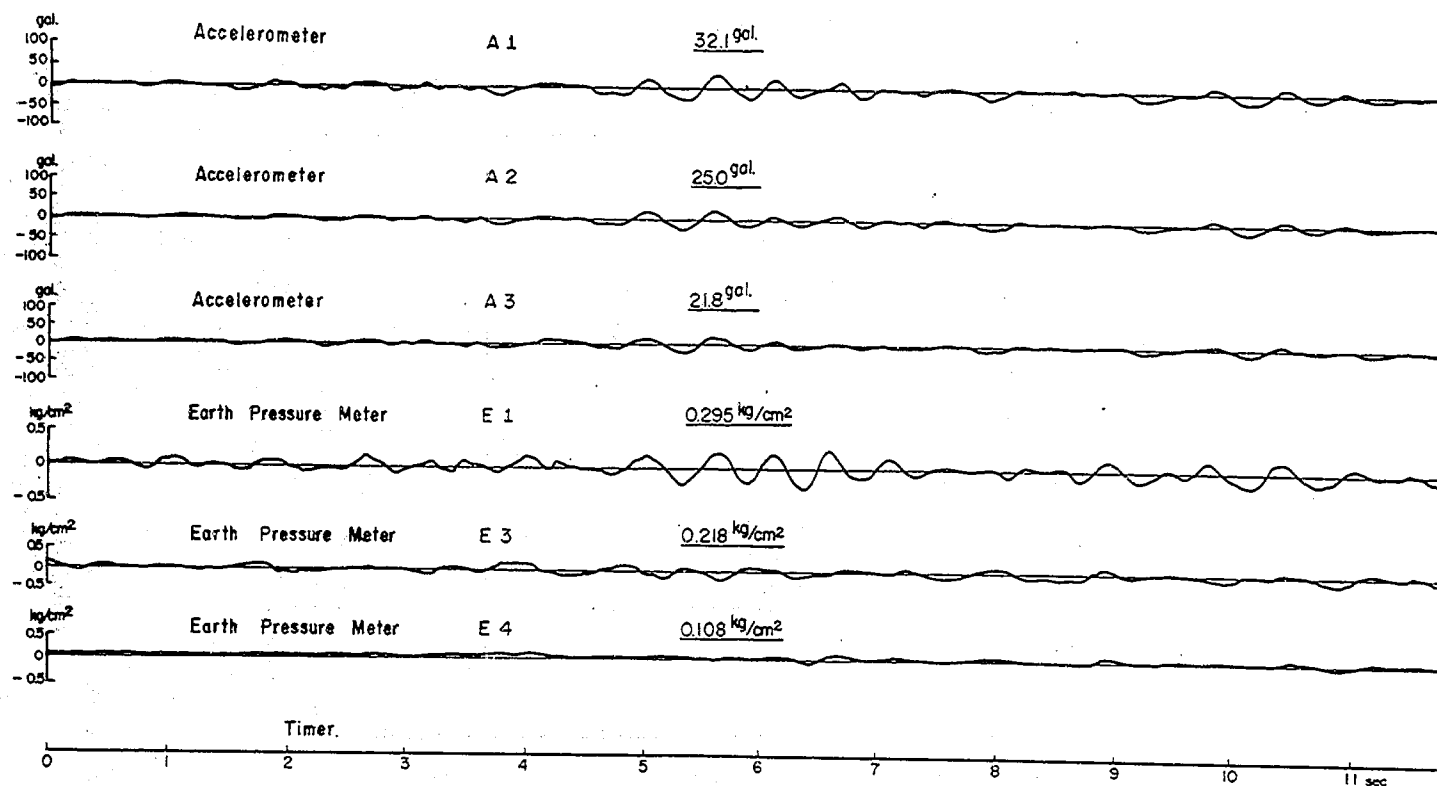


Fig. 3.52 Earthquake records at the Shinkatsushika bridge during the Higashi-Matsuyama Earthquake of July 1, 1968 (records of accelerations and earth pressures at the foundation of pier 5)

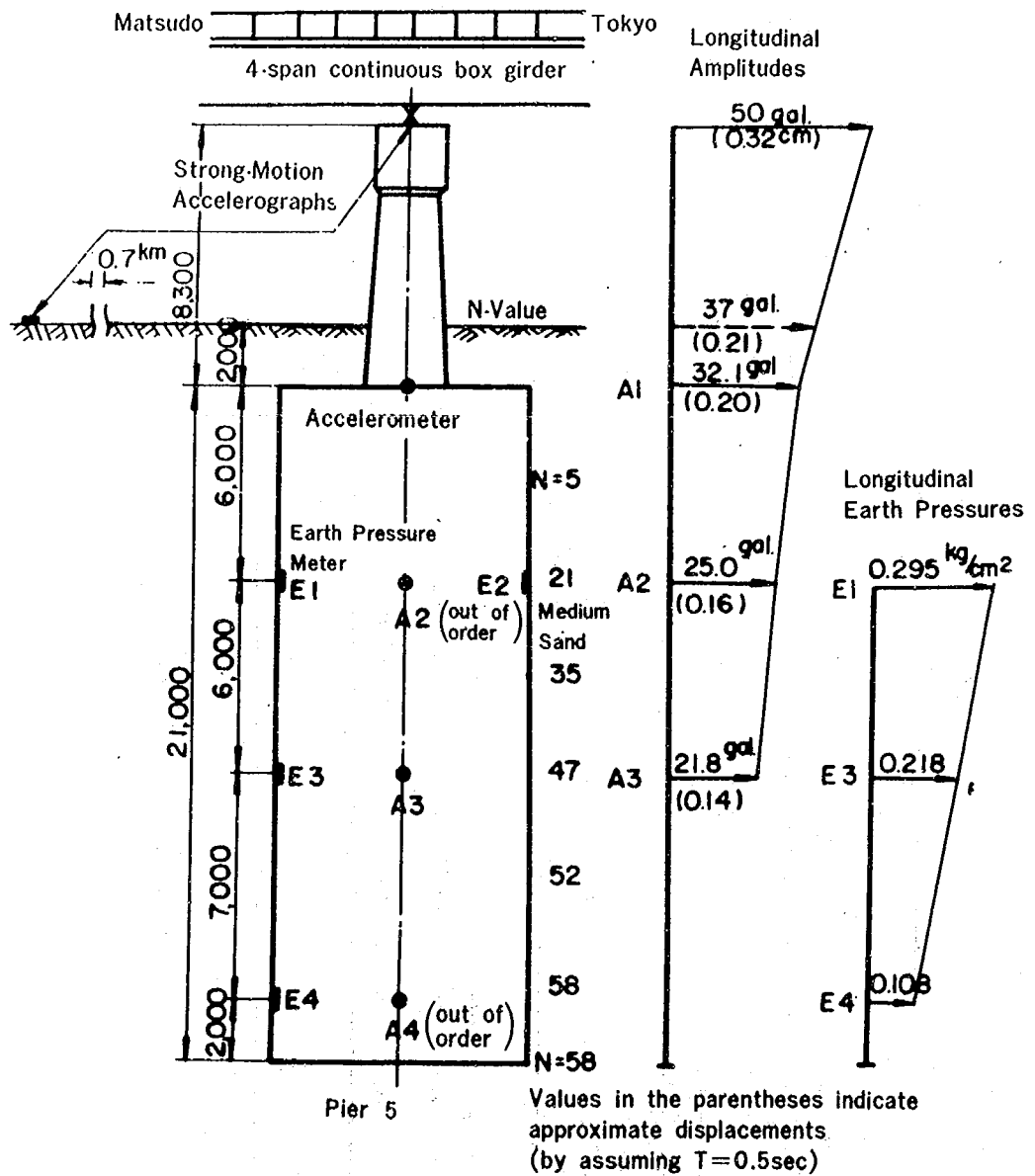


Fig. 3. 53 Results of earthquake measurements at the Shinkatsushika bridge during the Higashi-Matsuyama Earthquake of July 1, 1968

Some Other Bridges 63), 64)

Results of strong-motion measurements at Chiyoda Bridge, Hirai Bridge, and Soka Viaduct which experienced moderate ground motions several times, are shown on Figs. 3.54, 3.55 and 3.56, respectively. The maximum ground accelerations are about 50 to 100 gals for these bridges. For these bridges it may be noted that ratios of responses to ground accelerations are independent on magnitudes of ground accelerations for moderate ground accelerations less than 100 gals.

3.8 Dynamic Analysis on Earthquake Response of Bridges

Present Status of Dynamic Analysis of Bridges

In assessing earthquake-resistance of recent major highway bridges, dynamic analysis is frequently conducted, in addition to the earthquake-resistant design adopting the conventional seismic coefficient method or the modified seismic coefficient method considering structural dynamic response. It seems that this trend brought about due to the facts that some newly constructed highway bridges sustained severe

damage during the Niigata Earthquake of 1964 which emphasized the importance of dynamic effects of earthquake motions, and also that several highrise bridges were erected on Japan Highway Public Corporation expressways since around 1965. Moreover, this trend was expedited after the new specifications for earthquake-resistant design of bridges stipulate that response analysis shall be adopted for bridges which are required detailed investigations (see Chapter 4).

Although various analytical methods are available in analyzing dynamic behavior of structures, the following two are generally employed for highway bridges.

- (1) Response Spectrum Method: To calculate maximum values of structural responses on the basis of response spectra (average spectra or spectra from particular strong-motion records).
- (2) Numerical Integration Method: To calculate time history of structural responses by numerical integration using digitized data of specific seismic records.

In applying dynamic analysis to the design of bridges, it is indicated that important factors are (1) idealization of the actual structures considering the effects of substructures, foundations and surrounding soils, (2) application of seismic disturbances, and (3) assessment of re-

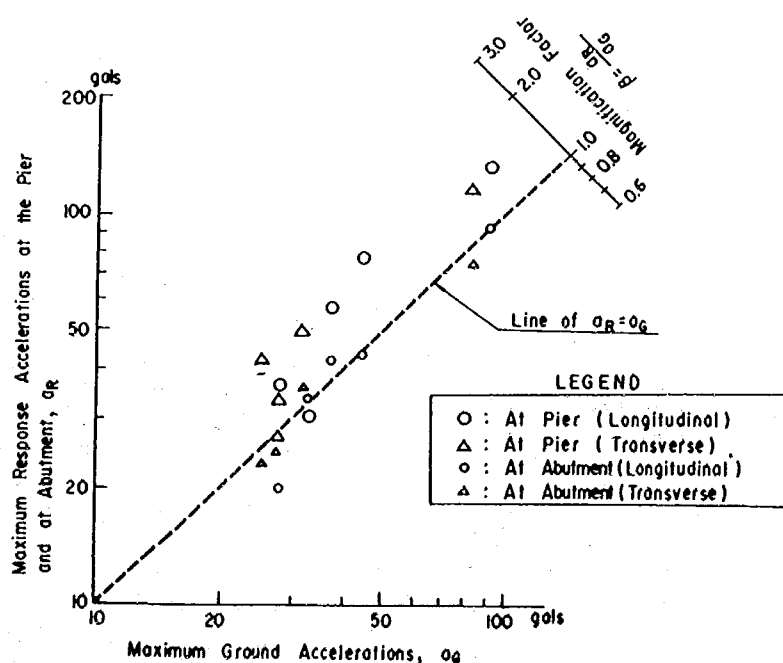


Fig. 3.54 Results of earthquake measurements at the Chiyoda bridge

sults of dynamic analysis in amending the initial design. Table 3.10 indicates some of highway bridges on which dynamic analysis have been conducted at various organizations¹²³⁾. In the table the outlines of dynamic analyses are summarized briefly. Analysis for some of these bridges are mentioned in the following.

Showa Bridge³⁵⁾

As indicated in 2.7, this bridge sustained severe damage during the Niigata Earthquake of June 16, 1964. To clarify the cause of the damage to the bridge, dynamic analysis was conducted among other extensive investigations. In preparing a computer program for the dynamic analysis, the prototype of a substructure was considered as shown in Fig. 3.57 in order to make wider use of the computer program for general bridges with substructures of pile foundations. Furthermore, the prototype was idealized as an analytical model shown in Fig. 3.58, for an analysis in the longitudinal direction of the bridge axis. In evaluating the analytical model the followings are assumed.

- (1) The bridge can be idealized by a four-degree-of-freedom system which represents a pier having a concentrated mass of the substructure rested on the crest of the pier and a spread footing with piles.
- (2) Piers and piles are considered to deform elastically.
- (3) Relative displacements of tips of piles to the surrounding soils are assumed zero.
- (4) A vertical spring constant of piles can be determined considering elastic deformation of piles fixed at the tips. A horizontal spring constant of piles is calculated assuming the piles as elastic beams rested on elastic foundation.
- (5) The superstructure is considered to be a mass concentrated on the crest of the pier.
- (6) The effects of mass of pier are considered to be concentrated at two points, the crest and the footing.
- (7) From the measurements for the bridge it is estimated that the coefficient of subgrade reaction for the surrounding soils is 1.0 kg/cm^3 , and the damping ratio is 2% of critical.

As for inputs to the bridge, the response spectra of the strongmotion record obtained at

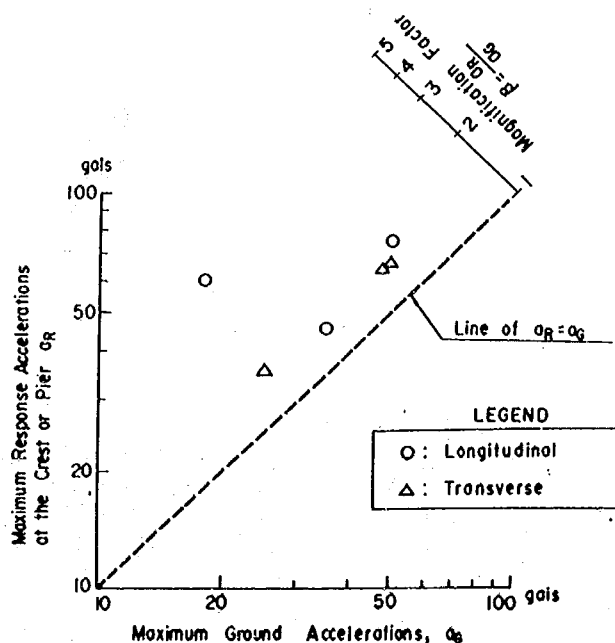


Fig. 3.55 Results of earthquake measurements at the Hina bridge

Table 3.10 Same Examples of Highway Bridges on which Dynamic Analyses were Conducted

Name of Bridges (Location)	Horizontal Design Seismic Coefficient	Analytical Model, Damping Ratio (h) (Direction Analyzed)	Method of Analysis	Organizations Administrated	Organizations Analyzed (Year)	Remarks
Amakusa No. 4 (Kumamoto, Kyushu)	0.15	34-mass spring system h=1, 2% for superstr. h=3, 5% for substr. (Transverse)	Direct Integration Method (Taft-1952, $a_G=150$ gals)	Japan Highway Public Corporation	JHPC and PWRI (Ministry of Construc- tion) (1967)	Field Experiment
Hirai (Tokyo, Honshu)	0.3	Elastic Continuum h=5% (Longitudinal and T)	Response Spectrum Method (PWRI-1964, $a_G=300$ gals)	Tokyo Metropolitan Government	Tokyo M.G. and PWRI (MOC) (1966)	◦ Field Experiment on Substructures ◦ Observation of Strong- Motion Earthquake
Honshu-Shikoku (Honshu-Shikoku)	0.2	94-mass spring system h=2, 5% (L, T ₁ and Vertical)	Response Spectrum Method (PWRI-1964, $a_G=200$ gals)	Kinki Regional Construction Bureau (MOC)	PWRI (MOC) (1966)	Model Experiment
Otani (Kitakyushu, Kyushu)	0.165	35-mass spring system h=2, 5% (L and T)	Direct Integration Method (El Centro-1940, $a_G=165$ gal)	JHPC	JHPC (1970)	Field Experiment
Sakai-gawa (Kanagawa, Honshu)	0.2	Elastic Continuum h=2.5, 10% (T)	Response Spectrum Method (PWRI-1964, $a_G=200$ gals)	JHPC	JHPC (1965)	Observation of Strong-Motion Earthquake
		41-mass spring system h=2.5, 10% (T)	Direct Integration System (El Centro-1940, $a_G=200$ gals)			
Showa (Niigata, Honshu)	0.2	Elastic Continuum h=5% (L)	Response Spectrum Method (PWR-1964, Niigata-1964, $a_G=200$ gals)	Niigata Prefectural Government	PWRI (MOC) (1964)	◦ Damage Experience ◦ Observation of Strong-Motion Earthquake
		One-mass bi-linear System, h=2%, (L)	Direct Integration Method (Niigata-1964, $a_G=157$ gals)			
Sokozawa (Kanagawa, Honshu)	0.2	Elastic Continuum h=2% (T)	Response Spectrum Method (PWRI-1964, $a_G=200$ gals)	JHPC	JHPC (1968)	Field Experiment
Yachiyo (Niigata, Honshu)	0.2	Elastic Continuum h=5% (L)	Response Spectrum Method (PWRI-1964, Niigata-1964, $a_G=200$ gals)	Niigata Municipal Government	PWRI (MOC) (1964)	◦ Seismic Damage Experience
Yoneyama (Niigata, Honshu)	0.2	Elastic Continuum h=2.5% (L and T)	Response Spectrum Method (PWRI-1964, $a_R=200$ gals)	Hokuriku Regional Construction Bureau (MOC)	Hokuriku Regional Construction Bureau (MOC) (1965)	Observation of Strong-Motion Earthquake

Notes: a_G : Maximum accelerations considered in dynamic analyses.
h : Damping ratios to critical damping, considered in analyses.

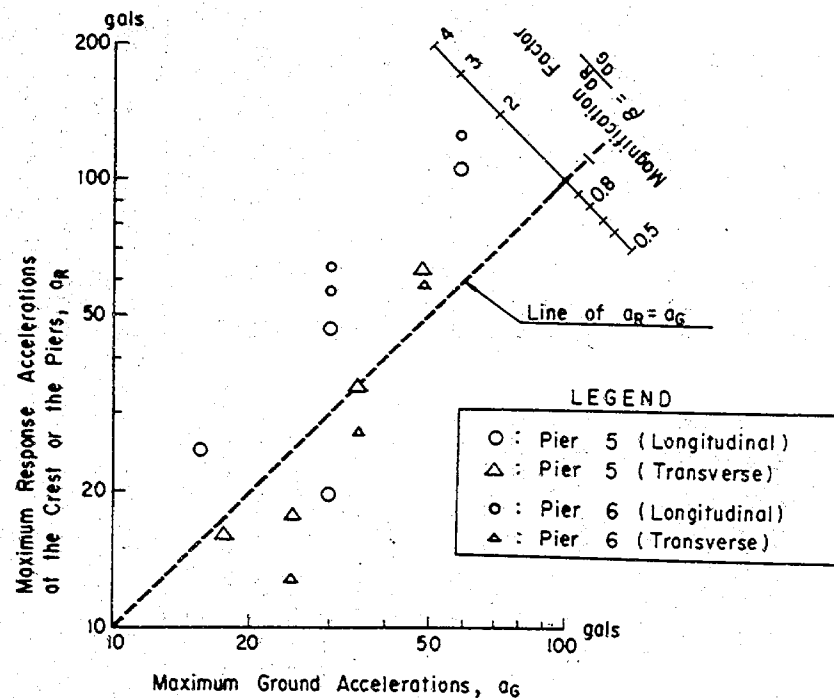


Fig. 3.56 Results of earthquake measurements at the Soka viaduct

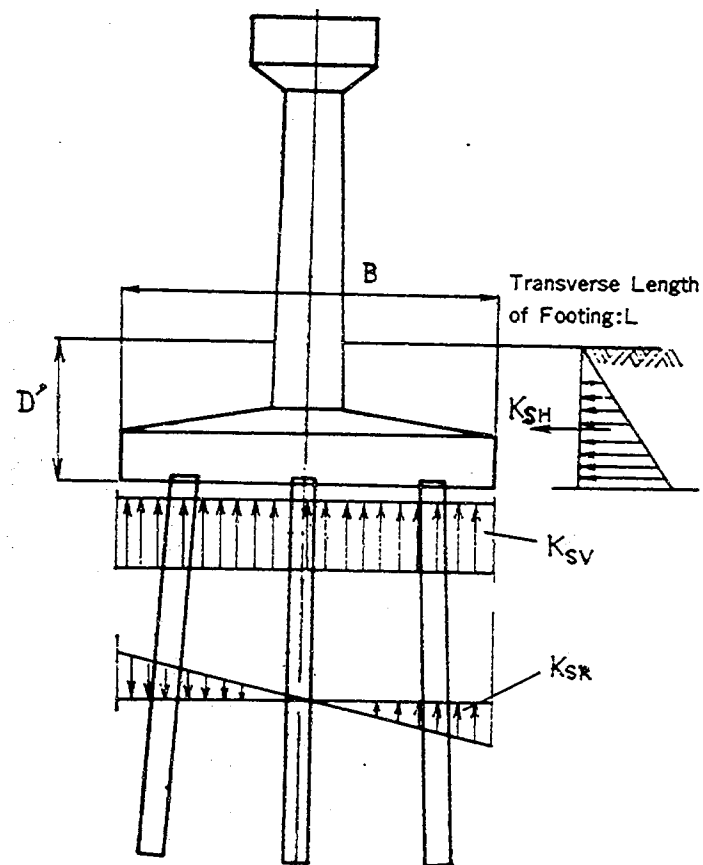


Fig. 3.57 General bridge substructure with pile foundation

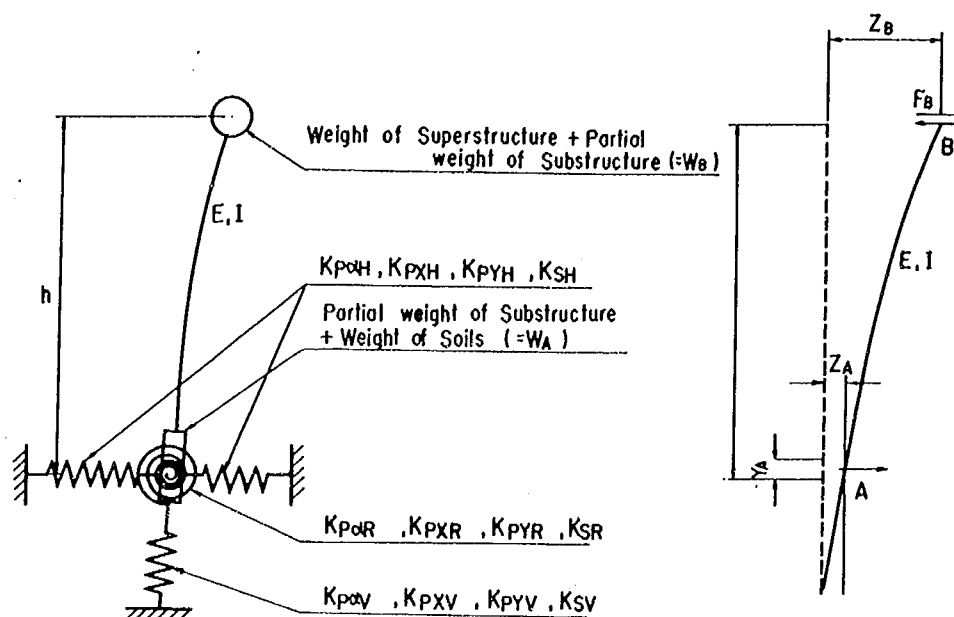


Fig. 3.58 Analytical model for bridge substructure with pile foundation

the basement of an apartment in Niigata was employed to analyze maximum response values. The maximum ground accelerations are about 0.15 g and 0.14 g for E-W and N-S components, respectively. From the analysis it was disclosed that the fundamental natural period would be 2.3 seconds and the maximum displacement would be 2.1 to 34 cm at the crest of the pier. It was also revealed that the maximum displacement would become 1.3 m or more provided that the surrounding soils around the piles lost the horizontal resistance for the depth of 10m below the surface due to the influences of liquefaction of the sands. Forces acting on the substructure were also calculated, and were found to be larger than allowable ones.

On the other hand, it seemed probable that the girders would fall down into the river provided that relative displacement increases 50 cm between two adjoining piers. This was due to the facts that the width of the crest of each pier, which supported a fixed shoe and a movable shoe, was 50 cm and that there were no devices to prevent the fall of girders.

In addition to the above, some dynamic analysis were performed by adopting a numerical integration method for a linear system and a

nonlinear system subjected to the strong-motion records. Through these studies the cause of the damage to the Showa Bridge was investigated.

Yoneyama Bridge 123)

This bridge, completed in 1956 by the Ministry of Construction as a link of the National Highway No. 8, is located in Yoneyama, Niigata Prefecture about 80 km southwest from Niigata City. The bridge is a slightly curved one with the height of piers of about 43 m, as shown in Fig. 3.59. The substructures are steel rigid frame with reinforced spread footings on rocky ground. The superstructures consist of 3-span continuous steel box girders (span length of 67 m, 93 m and 67 m) with steel slabs and of 2-span continuous steel plate girders (span length of 2 x 25 m) with concrete slabs.

Having highrise piers, the bridge has been investigated on the stability against earthquakes by conducting dynamic analysis, as well as the conventional earthquake-resistant design taking into account a horizontal seismic coefficient of 0.2. In the dynamic analysis seismic effects were evaluated by adopting average response spectra (Fig. 3.7) proposed by PWRI in 1964⁸⁹). Seismic

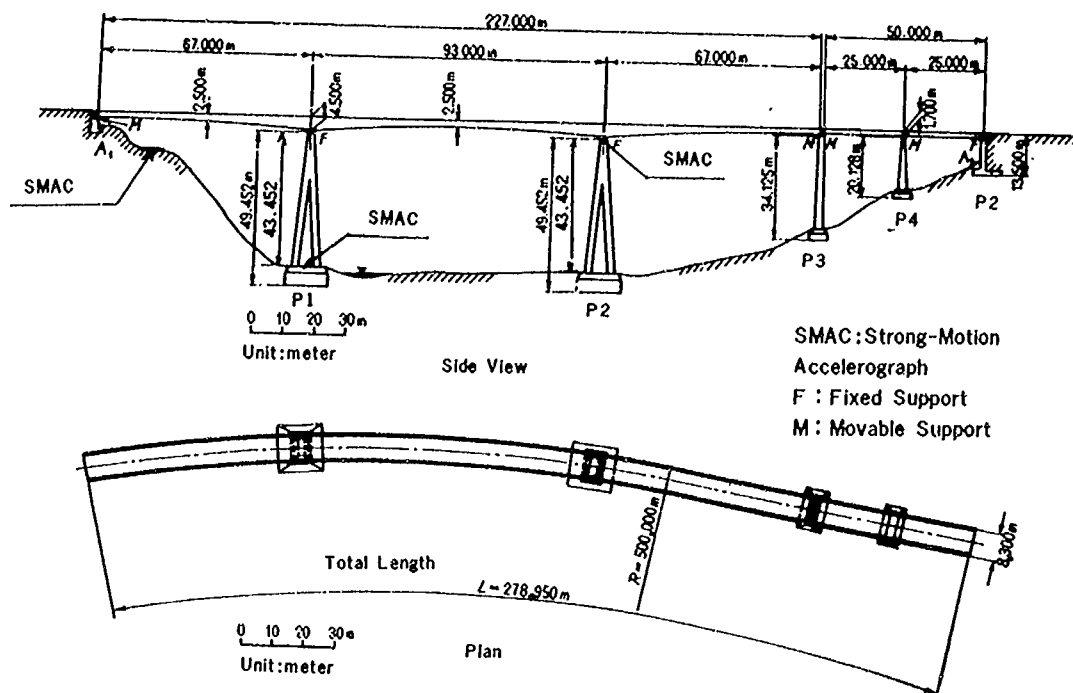


Fig. 3.59 General view of the Yoneyama bridge

motions are applied for two directions, the longitudinal and the transverse to the bridge axis assumed to be straight, and the maximum acceleration of ground motions was taken as 0.2 g. The followings summarize the assumptions and the results of the analysis for the transverse direction that yielded severer states for the stability of the bridge.

The bridge was idealized as an analytical model shown in Fig. 3.60. The model was set up on the basis of the following assumptions.

- (1) Any girders and piers are substituted with uniform members which have their properties shown in Table 3.11.
- (2) Girders between nodal points 0 and 3, and between 3 and 5 are continuous. At nodal point 3 shearing forces and bending moments can be transmitted through the Pier 3. At nodal points 1, 2 and 4 connections between girders and piers are made by fixed shoes.
- (3) Points 0, 5, 6, 7, 8 and 9 move simultaneously in phase during earthquakes. The bases of four piers are fixed perfectly on the spread footings at nodal points 6, 7, 8 and 9.

At nodal points 0 and 6, three cases of end conditions are considered: perfectly fixed, 90

percent fixed (when subjected to bending moments the end rotation will be 10 percent of perfectly free end), and 50 percent (50 percent of perfectly free end).

For the dynamic analyses two cases of damping ratios are considered, 2% and 5% of critical. In superposing each modal response value to obtain the maximum response, two combination methods are employed: one is the absolute sum of each modal maximum response (abbreviation in the figures is $\Sigma| \quad |$), the other is the square root of square sum of each modal maximum response ($\sqrt{\Sigma| \quad |^2}$).

Figs. 3.61 to 3.64 are the results of the analyses. Fig. 3.61 indicates the mode shapes of order of 1st to 5th. Figs. 3.62, 3.63 and 3.64 show maximum displacements, maximum bending moments and maximum shearing forces, respectively. Dotted lines in the figures indicate the initial design values obtained by adopting the conventional method where horizontal seismic coefficient of 0.2 applied to the superstructures. The design of the Yoneyama bridge was amended in the light of the results of the dynamic analyses.

Moreover, for the bridge three sets of SMAC

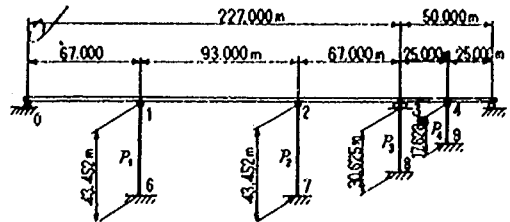


Fig. 3.60 Analytical model for the Yoneyama bridge

Table 3.11 Dimensions of Girders and Pier Columns of the Yoneyama Bridge

Girder	0 - 1	1 - 2	2 - 3	3 - 4	4 - 5
Span Length, L (m)	67.0	93.0	67.0	25.0	25.0
Moment of Inertia, I (m ⁴)	1.016	1.016	1.016	0.8	0.8
Unit Weight W (t/m)	5.7	5.6	5.7	7.39	7.39

Column	1 - 6	2 - 7	3 - 8	4 - 9	Remarks
Height, L (m)	43.452	43.452	30.625	17.628	Height above the crest of Foundation
Moment of Inertia, I (m ⁴)	1.010	1.010	0.203	0.0838	
Unit Weight, W (t/m)	4.43	4.43	1.137	0.7918	

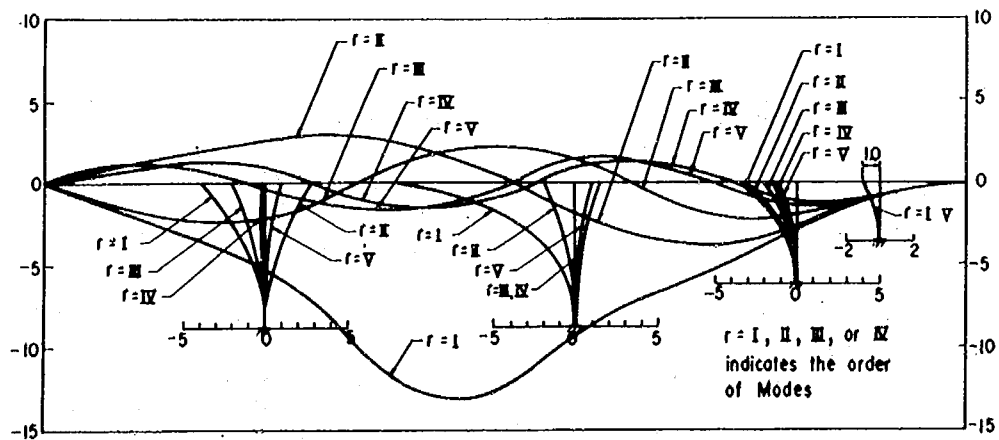


Fig. 3.61 Results of dynamic analysis - 1st to 40th mode shapes of the Yoneyama bridge in the transverse direction

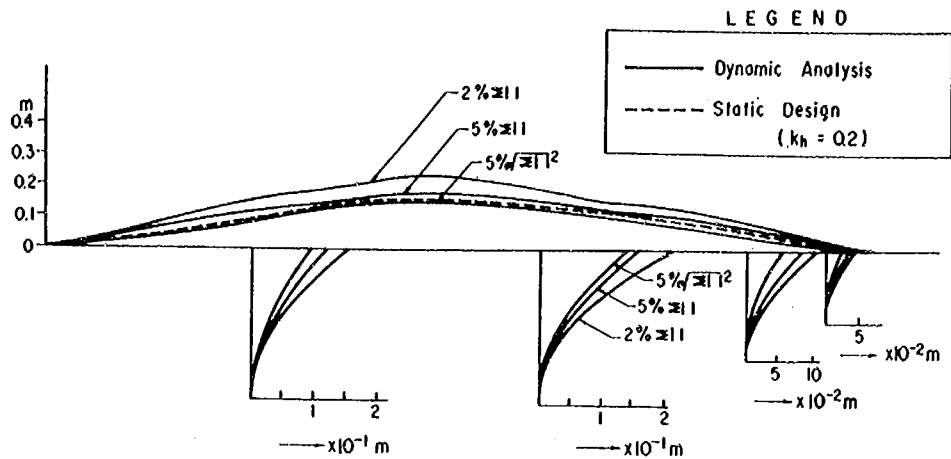


Fig. 3.62 Results of dynamic analysis - displacement

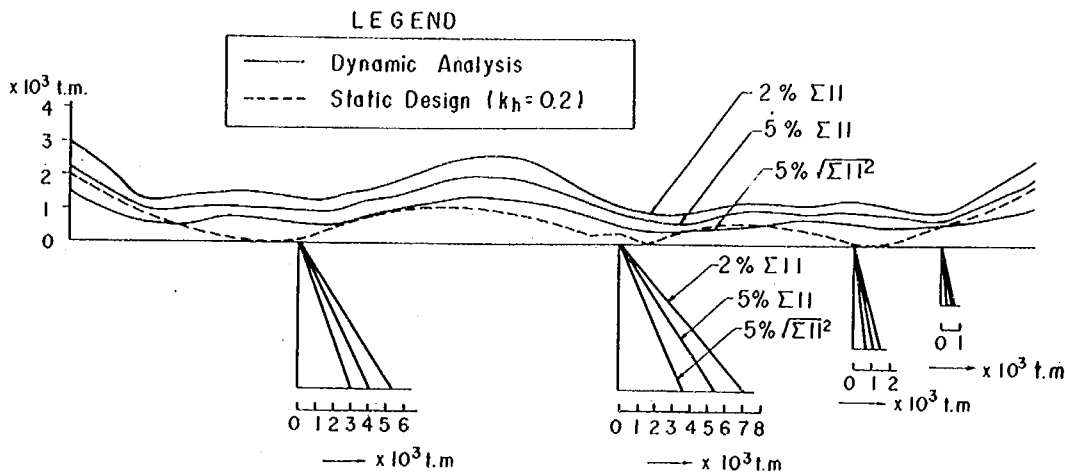


Fig. 3.63 Results of dynamic analysis - bending moment

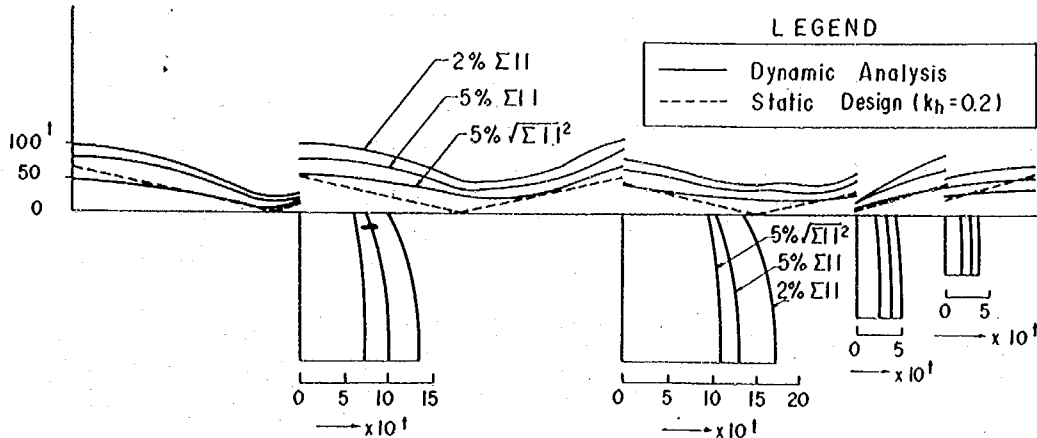


Fig. 3.64 Results of dynamic analysis - shearing force

accelerographs (see Fig. 3.59) were installed in 1966 after the completion, and its dynamic behavior is being measured during actual strong earthquakes.

Sokozawa Bridge^{(122), (124)}

As shown in 3.6, the dynamic properties of the Sokozawa Bridge were evaluated through the field dynamic tests. For the bridge dynamic analysis was also conducted. Fig. 3.65 shows the analytical model for the analysis in the direction perpendicular to the bridge axis. Six cases shown in Table 3.12 were considered with varying beam-column connection conditions and values of moments of inertia for concrete sections. Natural frequencies and mode shapes are indicat-

ed in Table 3.13 and Fig. 3.66, respectively, for 1st to 4th order. In these the results of the field tests are also shown. Case 6 among six cases of dynamic analysis is found comparatively in good agreement with the results of the field tests with small amplitudes.

For case where a ground motion with the maximum acceleration of 0.2g applies to the bridge model having a damping ratio of 2% of critical, maximum bending moments are 22,500 t-m, 49,000 t-m, 57,000 t-m and 28,000 t-m at the bases of four piers P1, P2, P3 and P4, respectively, and the absolute maximum displacement is about 20 cm. The above are the results for Case 3 (in Table 3.12) which is supposed to be most reasonable in analyzing dynamic re-

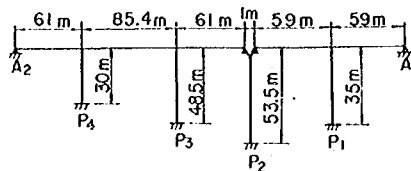


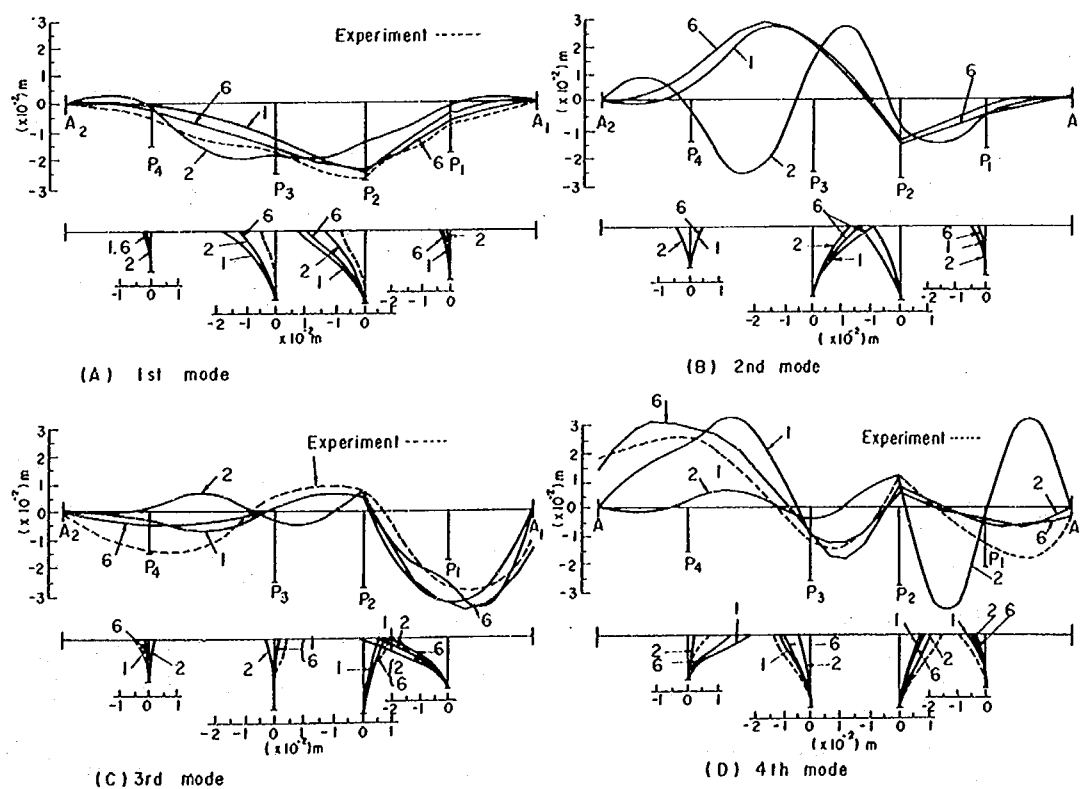
Fig. 3.65 Analytical model for the Sokozawa bridge
(T. Suruga, et al. (114))

Table 3.12 Six Cases Considered in the Analysis of the Sokozawa Bridge

Case No.	Bases of Columns	Connections at crest of piers		Evaluation of Moment of Inertia	
		Girder	Column		
1	Fixed	Hinged		Girder	Consider all cross-section of concrete
				Column	Consider all cross-section of concrete
2	Fixed	Hinged		Girder	Neglect tensile area of concrete
				Column	Consider all cross-section of concrete
3	Fixed	Hinged		Girder	Neglect tensile area of concrete
				Column	Neglect tensile area of concrete
4	Fixed	Fixed		Girder	Neglect tensile area of concrete
				Column	Consider all cross-section of concrete
5	Fixed	Fixed	Consider Twisting	Girder	Neglect tensile area of concrete
				Column	Consider all cross-section of concrete
6	Fixed	Hinged		Girder	Consider all cross-section of concrete and end bracing
				Column	Consider all cross-section of concrete

Table 3.13 Natural Frequencies Analyzed and Resonant Frequencies from the Field Experiment

Order of Modes		1st	2nd	3rd	4th
Experiment		1.53 Hz	—	2.38 Hz	2.64 Hz
Dynamic Analysis	Case 1	1.68	1.94	3.09	3.41
	Case 2	1.50	2.22	2.48	3.00
	Case 3	0.93	1.05	1.47	1.66
	Case 4	1.50	1.59	2.26	2.55
	Case 5	1.50	1.59	2.29	2.55
	Case 6	1.60	1.82	2.35	2.82



Note : numbers 1 to 6 indicate case numbers, of. Table 3.12

Fig. 3.66 Comparison of mode shapes by analysis and by experiment for the Sokozawa bridge (after T. Suruga, et al. (114))

sponses during strong-motion earthquakes.

Besides the Sokoza Bridge, several high-rise bridges on Chuo Expressway and Tomei Expressway of the Japan Highway Public Corporation such as Sakai-gawa Bridge, Yokobuki Bridge, Tsuru-kawa Bridge, Sakawa-gawa Bridge, etc. have been studied extensively by dynamic analysis.

Proposed Honshu-Shikoku Suspension Bridge 123), 140), 149)

Extensive studies are also conducted to analyze dynamic response of proposed Honshu-Shikoku suspension bridges connecting Honshu and Shikoku islands. The following describes an example of the dynamic analysis.

Table 3.14 summarizes the outline of a preliminary design of a suspension bridge with a center span of 1,500m. As shown in the table, four cases of ground conditions are considered in the analysis by varying the Young's modulus of the foundation rock. Fig. 3.67 indicates the coordinates for several sections and dimension symbols. Fig. 3.68 shows a mass spring model for the dynamic analysis. The model has 94 masses as a whole. In the figure symbols A, B, C, F, P and T represent anchor block, bent pier, cable, stiffening girder, main pier and tower, respective-

ly. L, M and R denote left span, midspan and right span, respectively. The definitions of the other symbols are as follows:

b: width (m)

E: Young's modulus (t/m^2)

G: shear modulus (t/m^2)

h: height (m)

I: moment of inertia (m^4)

J: moment of gyration ($t \cdot m \cdot sec^2$)

K_H : horizontal spring constant (t/m)

K_V : vertical spring constant (t/m)

l: span length (m)

u: horizontal displacement (m)

v: vertical displacement (m)

w: dead weight per unit length (t/m)

ϕ : rotational angle (radian)

Fundamental equations of motion for the suspension bridge when subjected to seismic disturbances are initially established for the analytical model shown in Fig. 3.67, then they are substituted with finite-difference equations for the discrete model shown in Fig. 3.68. A modal analysis is conducted to get natural frequencies and mode shapes. Maximum resultant response of displacements and forces are obtained by superimposing each modal response by utilizing average response spectra (Fig. 3.7) proposed by PWRI in 1964⁸⁹).

As a result of the analyses, Fig. 3.69 shows

Table 3.14 Preliminary Design of the Proposed Honshu-Shikoku Suspension Bridge Analyzed

(A) Superstructure

Definition	Symbol	Dimension	Value
Central Span Length	ℓ_M	m	1,500.0
Side Span Length	ℓ_L, ℓ_R	m	750.0
Cable Sag at Central Span	f_M	m	136.4
Cable Sag at Side Span	f_L, f_R	m	34.1
Height of Main Tower		m	210.0
Distance between shafts at Main Tower		m	36.0
Height of Stiffening Truss		m	12.0
Cross-sectional Area of Cable		m^2	2×0.689
Unit Weight of Cable		t/m	11.0
Unit Weights of Suspended Structure		t/m	20.0
Weight of Main Tower		t	24,300
Total Weight of Superstructure		t	117,400

Table 3.14

(B) Substructure

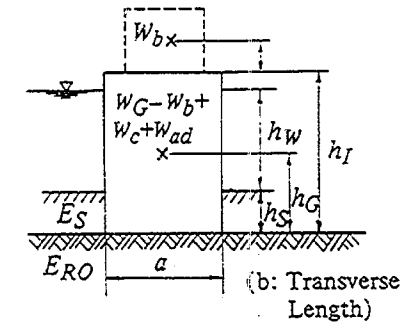
(a) Common Quantities

Sub-structure	h_a (m)	a (m)		b (m)	w_c (t/m ²)	h_b (m)	W_b (t)	W_c (t)	W_{ad}		W_G		V	
									L_g (t)	T_r (t)	L_g (t)	T_r (t)	Dry (t)	Sat (t)
Main Pier	75	L_g	50	80	2.0	21.5	0	6×10^5	281,000	165,000	884,000	765,000	657,000	417,600
		T_r	80	50					250,000	166,000	850,000	766,000		
Anchor Pier	65	L_g	100	70	2.0	21.5	1.7×10^5	9.1×10^5	171,000	267,000	1,251,000	1,347,000	1,082,000	732,000
		T_r	70	100					152,000	264,000	1,232,000	1,344,000		

(b) Quantities for four Cases

Case No.	h_s (m)	E_{RO} (t/m ²)	E_s (t/m ²)	M.P.				A.P.			
				h_w (m)	T		h_g (m)	h_w (m)	T		h_g (m)
					L_g (sec)	T_r (sec)			L_g (sec)	T_r (sec)	
1	10	10^{14}	10^{14}	50	0.09	0.08	36.2	40	0.03	0.04	38.82
2	10	600,000	16,000	50	0.51	0.41	36.2	40	0.46	0.54	38.82
3	10	80,000	10,000	50	1.28	1.32	36.2	40	1.21	1.43	38.82
4	10	40,000	5,000	50	2.02	2.32	36.2	40	1.64	2.03	38.82

Notes) W_{ad} Added Weight due to Water and Soils
 V Effective reaction at the base of Foundation
 L_g Longitudinal, T_r Transverse
 Dry No buoyancy, Sat Full buoyancy



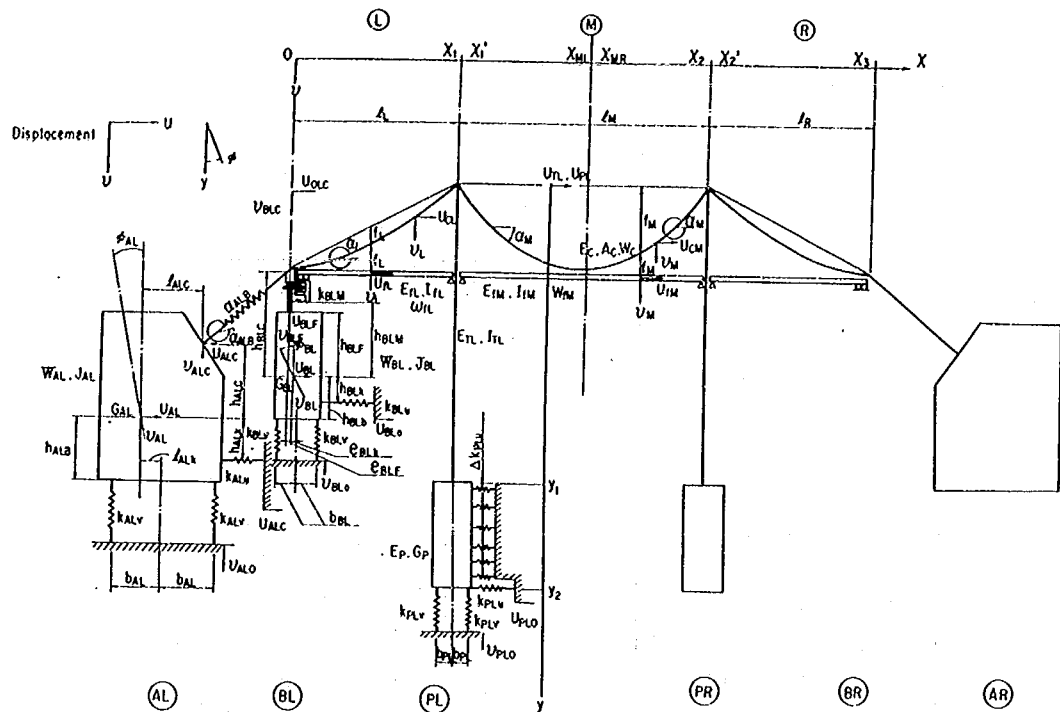


Fig. 3.67 Coordinates of a suspension bridge

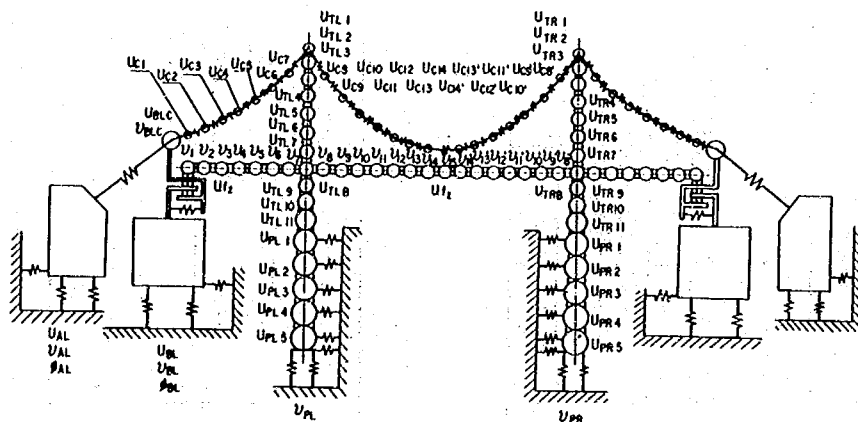


Fig. 3.68 An analytical model for dynamic analysis

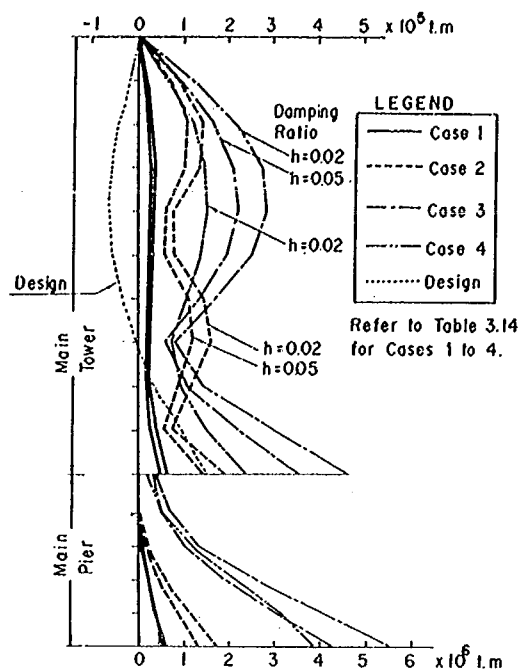


Fig. 3.69 Maximum bending moments for the main tower including pier in the longitudinal direction

maximum bending moments for the main tower and the main pier on which seismic motions are found to affect very significantly.

Otani Interchange Bridge¹¹⁵⁾

The bridge was constructed by the Japan Highway Public Corporation in 1970, as the rampway at the Otani Interchange of the Kita-Kyushu Bypass in northern Kyushu. The main portion of the bridge, between Pier 2 and Pier 8 (see Fig. 3.70), consists of a six-span continuous curved steel box girder. The portion has seven reinforced concrete hollow cylindrical piers (height of 20 to 30m and inner and outer diameters of 2.5m and 3.5m, respectively) supported by rigid spread foundations or open caisson foundations founded on the bed rock. The length of the above six spans is about 204m (= 30m + 4 × 36m + 30m). Five approaching spans at the both sides are of simple composite girders, and have the length of about 126m (each span is 20 to 29m). The effective width is 8.0 to 8.8 m.

S. Yamaguchi, et al.¹¹⁵⁾ conducted ex-

tensive studies on earthquake resistance of the bridge, in consideration of the fact that it has relatively high piers and a radius of the curvature is very small (40 m). Their studies included a field experiment on a isolated pier (Pier 7) before the erection of the superstructure, a field experiment on the overall structure completed, and dynamic analyses of the main six spans subjected to seismic excitation.

In the field test for the Pier 7, an excitor capable of generating sinusoidal forces up to 20 t was place on the crest of the pier and exerted it in a horizontal direction with harmonic forces within the range of 1.0 to 4.0 Hertz. From the test it is disclosed that the pier has the fundamental frequency of 1.58 Hertz and the damping ratio of 2.5% of critical, as shown in Fig. 3.71.

The field experiment for the overall structure was carried out to evaluate its dynamic properties, such as natural frequencies, mode shapes and damping ratios as a whole. To this aim the same excitor was set up on three points on the slab, above Pier 3, between Pier 4 and Pier 5, and above Pier 7, as shown in Fig. 3.70. It exerted the bridge in two horizontal directions (x

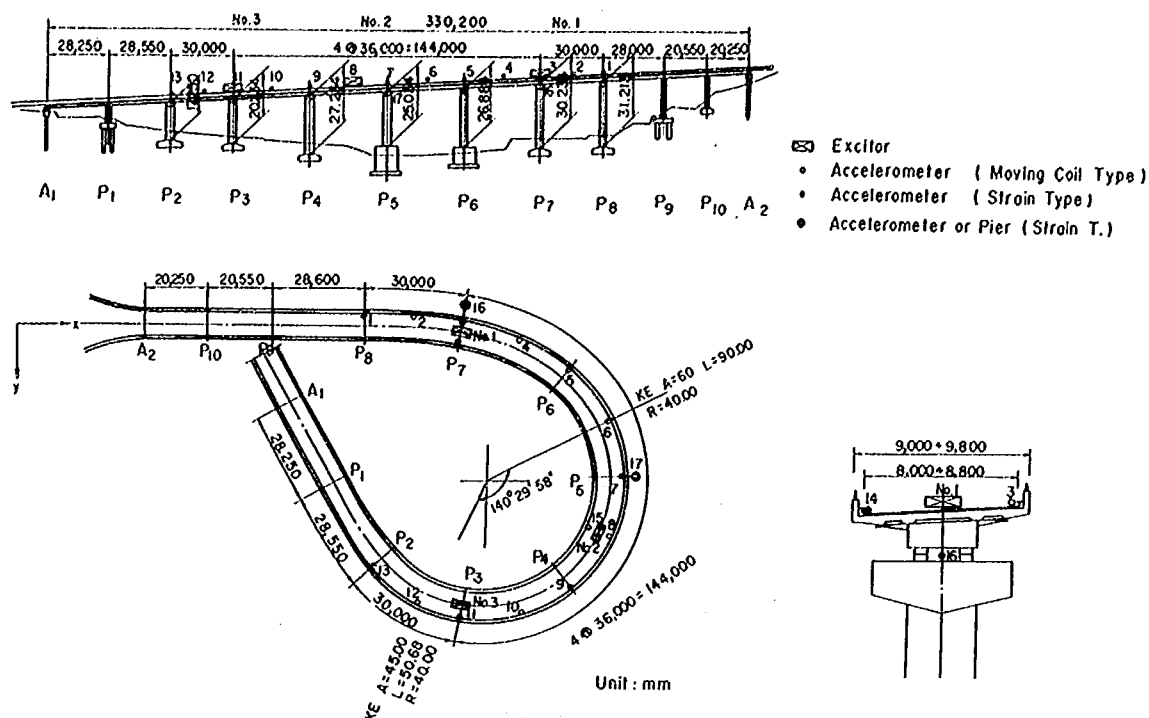


Fig. 3. 70 General view of the Otani interchange bridge (after S. Yamaguchi, et al. (115))

Table 3.15 Natural Frequencies from Test and Analysis for the Otani Interchange Bridge (After S. Yamaguchi, et al.¹¹⁵⁾)

(in Hertz)

Mode		1st	2nd	3rd	4th
Test		1.18	1.41	1.65	2.00
Computed	CASE1	(0.81) 0.96	(0.90) 1.06	(0.89) 1.26	1.64
	CASE2	(0.88) 1.04	(0.95) 1.34	1.96	2.27

Values in the parenthes ratios of frequencies computed to tested.

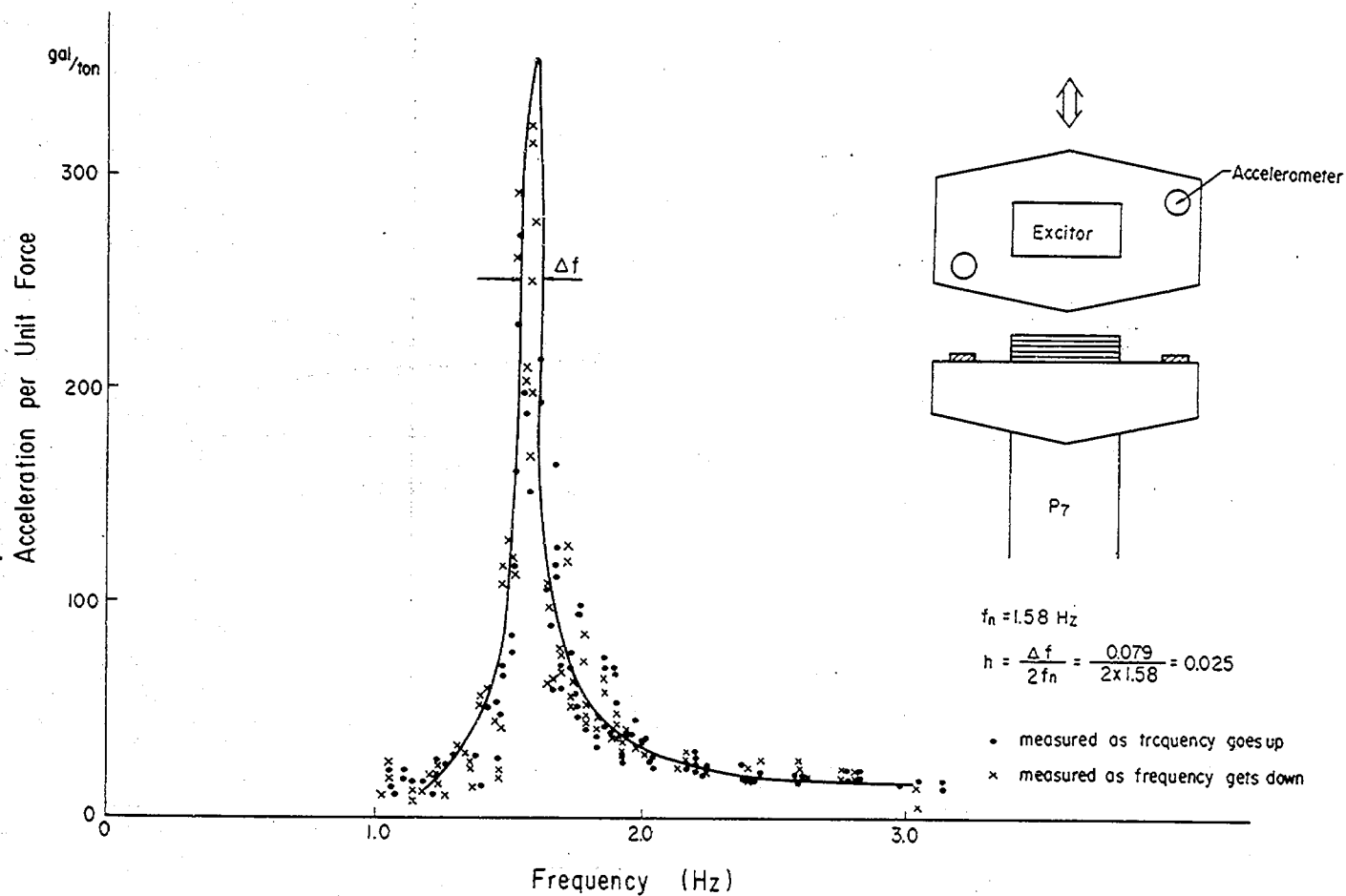


Fig. 3.71 Resonance curve for pier 7 of the Otani interchange bridge
(after S. Yamaguchi, et al. (115))

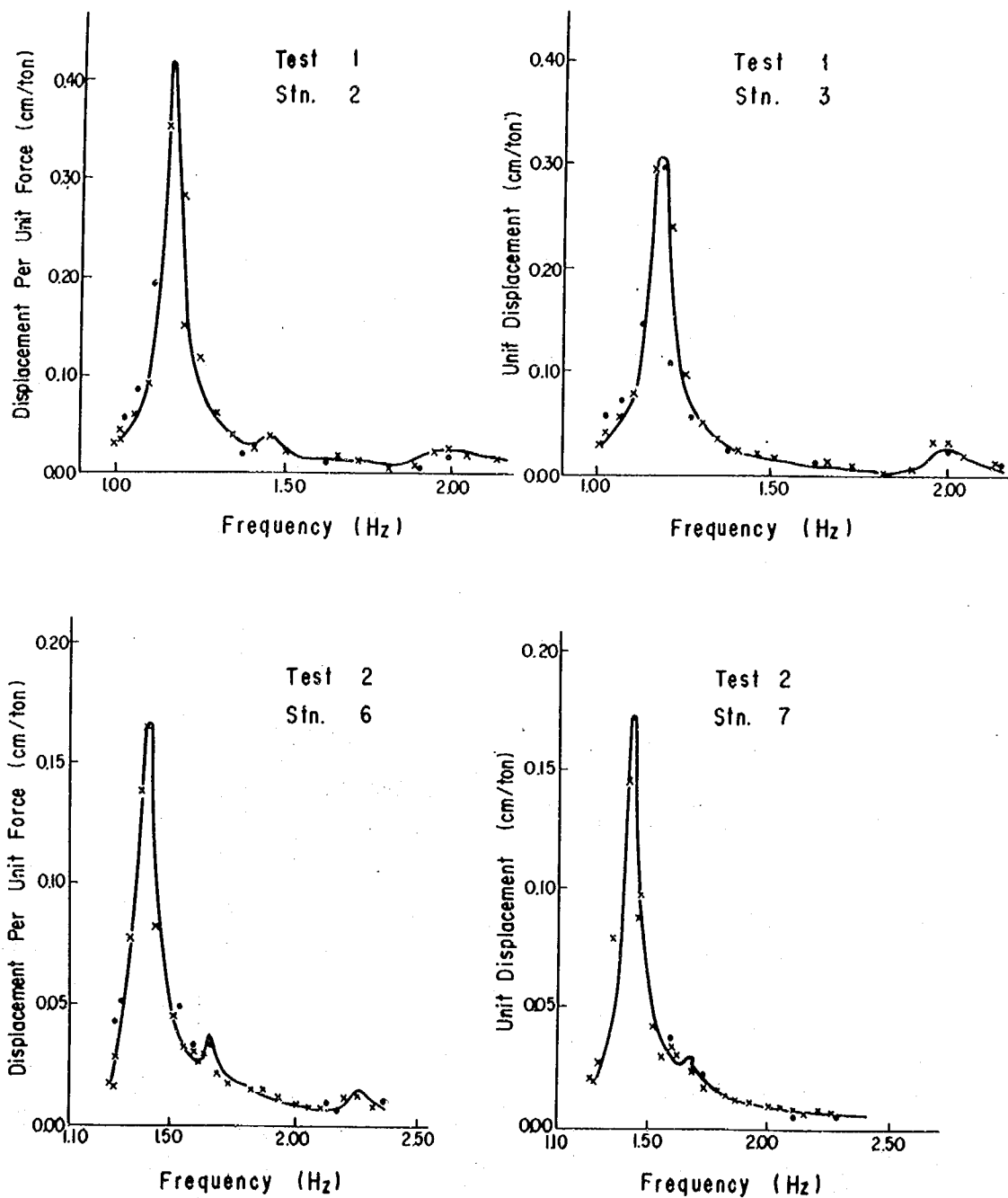


Fig. 3.72 Resonance curves for the overall bridge structure of the Otani interchange bridge (after S. Yamaguchi, et al, (115))

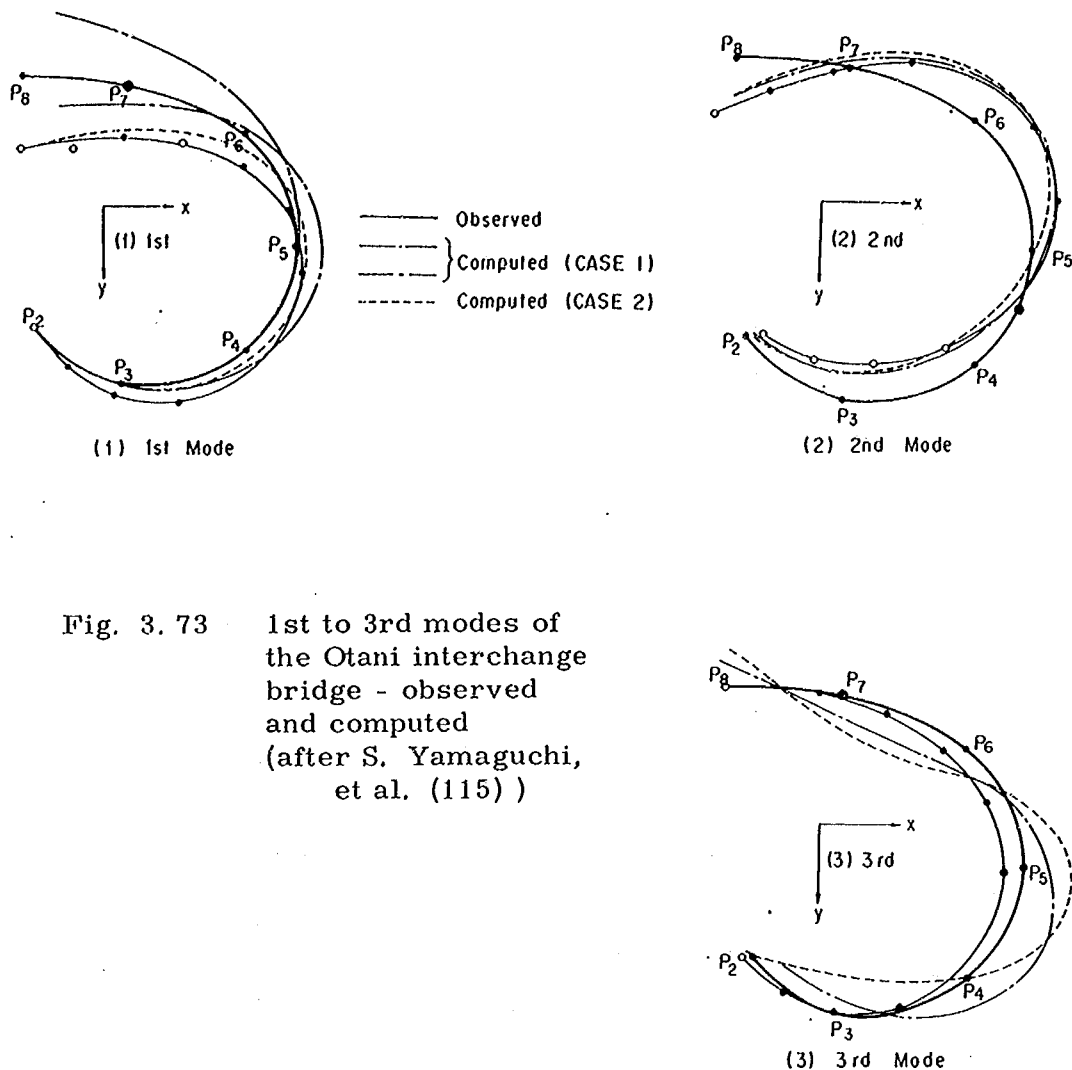


Fig. 3.73 1st to 3rd modes of the Otani interchange bridge - observed and computed (after S. Yamaguchi, et al. (115))

and y directions in the figure) with harmonic forces of 1.0 to 3.0 Hertz. Resonance curves, natural frequencies and mode shapes obtained are shown in Fig. 3.72, Table 3.15 and Fig. 3.73, respectively. The damping ratios were found to be about 2.2% of critical for the first mode and 1.8% of critical for the second mode.

The dynamic analysis was conducted to obtain seismic responses of the overall structure subjected to earthquake ground motions. Natural frequencies and mode shapes analyzed are indicated in Table 3.15 and Fig. 3.73, in comparison with the experimental results. Fig. 3.74 and Table 3.16 indicated results of the analysis when subjected to the N-S component of El Centro record of 1940, taking its maximum

acceleration as 165 gals. In Table 3.16 the results analyzed are compared with values in the design by means of the conventional seismic coefficient method taking a horizontal coefficient of 0.165. It is noted that the values from the two methods are not necessarily consistent.

3.9 Model Experiment on Dynamic Behavior of Bridges

In order to study on dynamic behavior of bridges, investigations have been conducted through laboratory model experiments employing shaking tables or exciters. The followings describe briefly some of experimental research works relating to dynamic behavior of bridges.

Table 3.16 Results of Dynamic Analysis for the Otani Interchange Bridge (After S. Yamaguchi, et al.¹¹⁵⁾)

(A) Maximum Displacement by Earthquake Wave in Direction x .

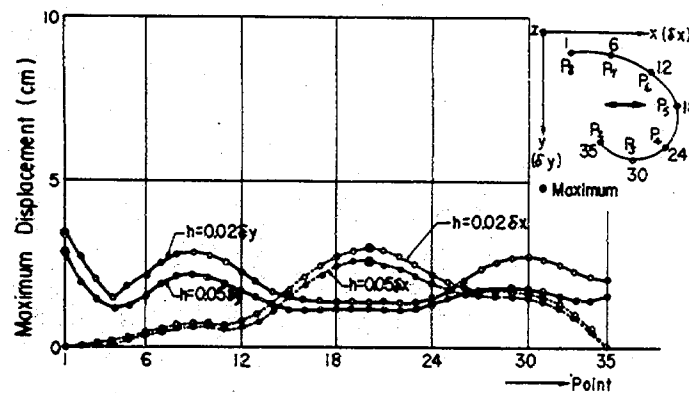
(B) Maximum Displacement by Earthquake Wave in Direction y .

unit cm

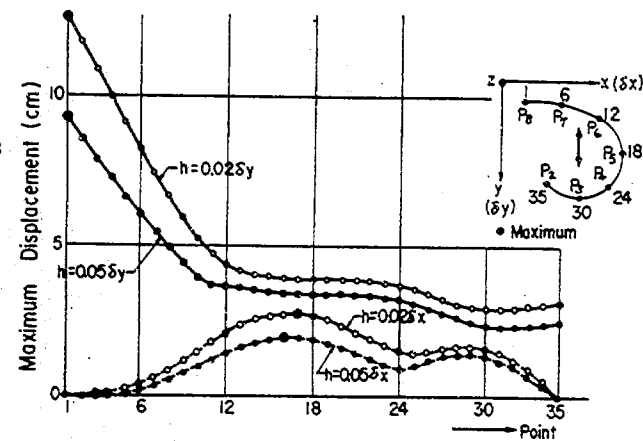
unit cm

Displacement Point	δ_x			δ_y		
	Method of Analysis			Method of Analysis		
	Seismic Coefficient	Modal Analysis		Seismic Coefficient	Modal Analysis	
	Method	h-0.02	h-0.05	Method	h-0.02	h-0.05
1	6.35	0	0	0.24	8.20	2.66
6	6.36	0.36	0.30	0.49	2.00	1.56
12	6.02	0.72	0.55	0.06	2.11	1.65
18	4.35	2.70	2.24	0.76	1.42	1.12
24	2.86	2.07	1.80	0.06	1.45	1.30
30	2.42	1.52	1.34	0.59	2.49	1.63
35	1.74	0	0	0.30	1.90	1.41

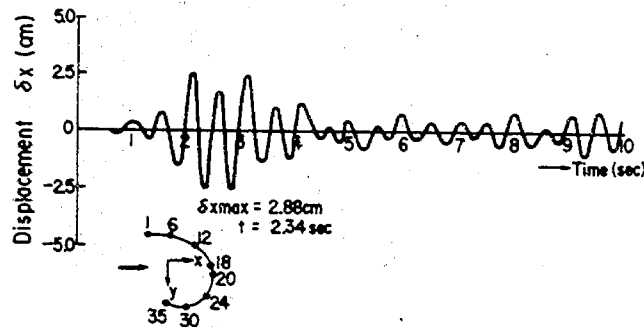
Displacement Point	δ_x			δ_y		
	Method of Analysis			Method of Analysis		
	Seismic Coefficient	Modal Analysis		Seismic Coefficient	Modal Analysis	
	Method	h-0.02	h-0.05	Method	h-0.02	h-0.05
1	0.39	0	0	6.76	12.24	8.30
6	0.36	0.37	0.25	6.56	7.75	5.69
12	0.28	2.27	1.53	5.22	4.71	3.45
18	0.48	2.63	1.80	5.00	4.45	3.46
24	0.06	1.46	0.90	4.69	3.72	3.16
30	0.42	1.44	1.29	2.22	2.71	2.24
35	0.13	0	0	1.22	2.93	2.34



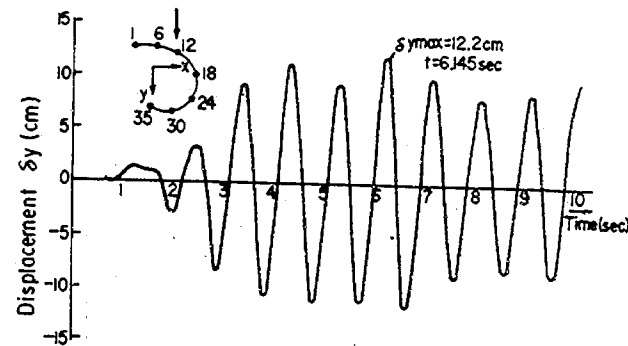
(A) Maximum Displacement by
Earthquake Wave in Direction X
(El-Centro N-S)



(B) Maximum Displacement by
Earthquake Wave in Direction Y
(El-Centro N-S)



(C) Displacement in Direction X, Point 20
Earthquake Wave ; El-Centro N-S
Damping ; $h = 0.02$



(D) Displacement in Direction Y, Point 1
Earthquake Wave ; El-Centro N-S
Damping ; $h = 0.02$

**Fig. 3.74 Results of dynamic analysis for the Otani interchange bridge
(after S. Yamaguchi, et al. (115))**

Model Test on Viaduct

E. Kuribayashi, K. Takada and K. Kimura¹²⁸⁾ performed model experiments on dynamic behavior of a viaduct made of four reinforced concrete columns and prestressed concrete girder, which was constructed on one of Tokyo expressways. The principal objectives of the model experiments were to know its dynamic characteristics and to observe degree of generation of cracks and failures on concrete columns during strong motions.

The model experiments were carried out by considering similarity relations between the prototype and the model. Two different models were made: Model I is an 1/3.75 model made by materials similar to the prototype and Model II is an 1/20 model made by plastics (rigolac).

As for the Model I, a centrifugal type excitor (maximum exciting force of about 10 t) was equipped inside the model, as shown in Fig. 3.75. Generation of cracks and failures on concrete columns was observed during relatively strong motions, and variation of dynamic characteristics was investigated in connection with the degree of growth of cracks. From the experiment it was revealed that many cracks were observed on the

four reinforced concrete columns when the model was subjected to acceleration of about 0.2 g, yet no cracks appeared on the prestressed concrete girder.

As for the Model II, a small model made by regolac was placed on a shaking table, as shown in Fig. 3.76, and the dynamic response was measured when it was subjected to a sinusoidal excitation with relatively small amplitude. In the experiment the regolac model seemed to behave within the elastic range.

Seismic Effects on Pile Foundation

N. Ogata and S. Kotsubo¹²⁹⁾ studied theoretically and experimentally on horizontal resistance of a pile foundation, in consideration of deformation of soil layer surrounding the piles. Fig. 3.77 illustrates the setup of the experiment. Bakelite tubes were embedded as piles, into dry sand layer in the box placed on the shaking table. Dynamic strains of the piles and accelerations at several points were measured during excitation of the table.

From the experiments and theoretical investigations, it is concluded that deformation of surface soil layer during earthquakes has significant effects on the horizontal resistance of pile foundations.

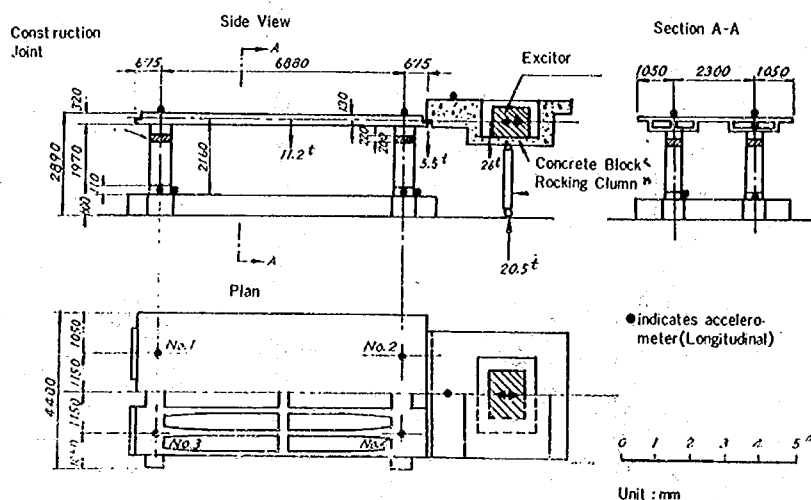
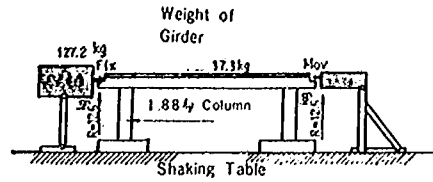
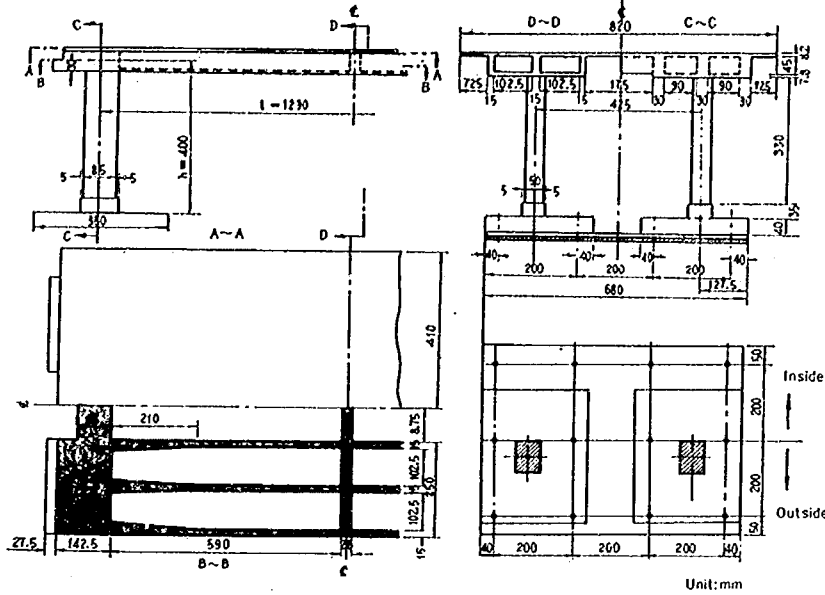


Fig. 3.75 Model I for dynamic tests of viaduct (after E. Kuribayashi, et al. (128))



(a) Sketch of overall Setup



(b) Detailed Drawing of the Model

Fig. 3.76 Model II for dynamic tests of a viaduct
(after E. Kuribayashi, et al. (128))

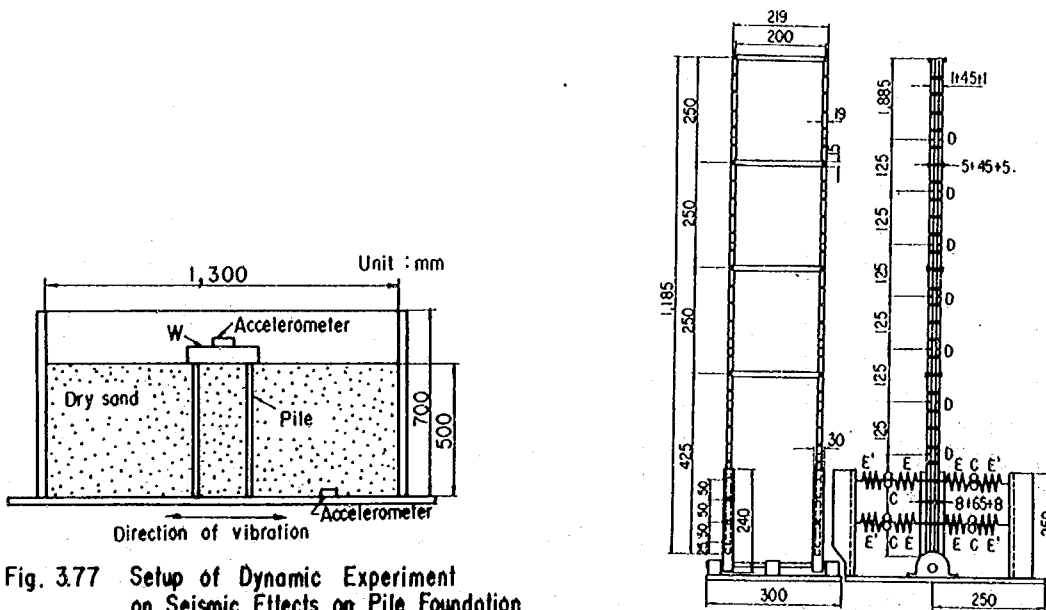


Fig. 3.77 Setup of Dynamic Experiment
on Seismic Effects on Pile Foundation
(After N. Ogata, S. Katsubo⁽²⁹⁾)

Fig. 3.78 A Model of Tower-pier System
of Suspension Bridges
(After I. Konishi et al.⁽³⁰⁾)

Earthquake Responses of Tower-Pier System of Suspension Bridge

I. Konishi, Y. Yamada, et al. have conducted extensive studies on seismic response characteristics of suspension bridges^{130), 134), 137),}

138). Among them an experiment for the tower-pier system was carried out to evaluate the effects of inelasticity of the foundation and the effects of the structural damping on the responses¹³⁰⁾

A model of the tower-pier system was experimented on an earthquake simulator, as shown in Fig. 3.78. From the experiment the following are emphasized in analyzing dynamic responses of suspension bridges to earthquakes. (1) In seismic design of a suspension bridge the design of the tower-pier system is most important, and the rotational deformation of the pier has significant effects on the stability.

(2) Inelastic properties of the foundations have considerable effects to reduce dynamic responses

of the structure, therefore, it is important to correctly estimate the inelastic behavior of the foundations.

Vibration Test of a Structure Supported by Pile Foundation

K. Kubo¹³¹⁾ conducted a model experiment on dynamic behavior of a liquid gas tank supported by a pile foundation, employing a large-size shaking table (see Fig. 3.79). The shaking table has a box of 10m long, 4m deep and 2m wide, and is capable of containing soil materials weighing 170 t. The shaking table is exerted by an actuator having an electro-hydraulic servo system and moves in one horizontal direction. The table has the properties of periods of vibration between 0.1 to 1.0 second, maximum amplitude of 10 cm and maximum acceleration of 400 gals.

From the experiment it is disclosed that the piles are exerted considerably by surrounding soil during vibration.

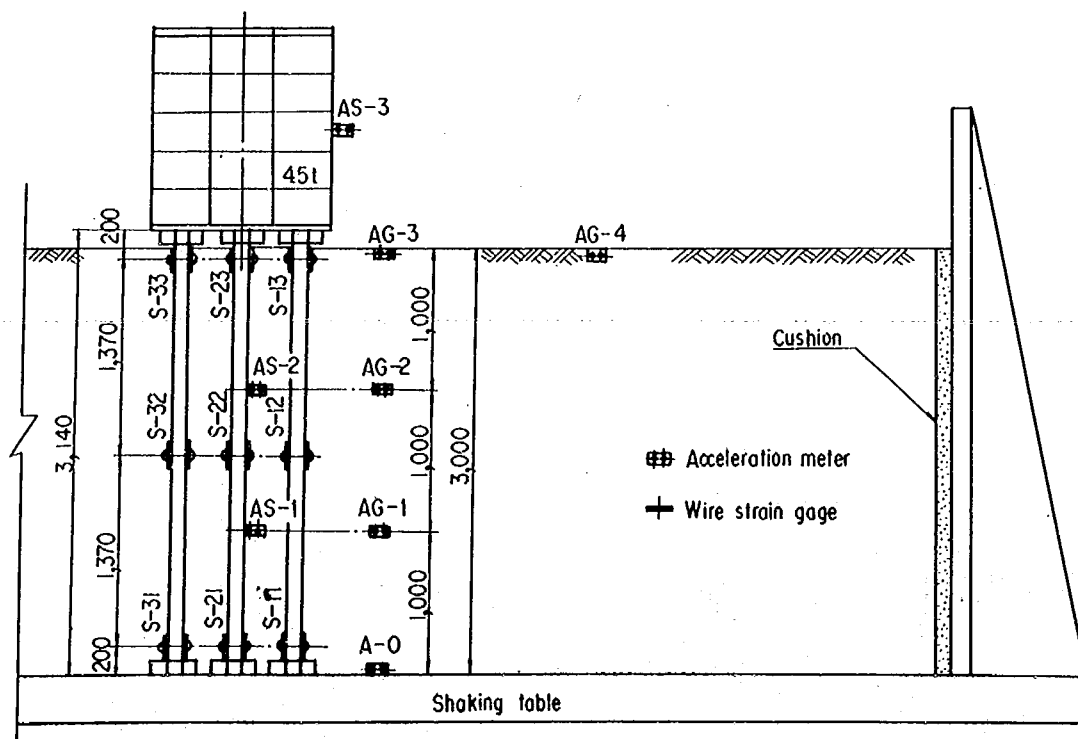


Fig.3.79 Sketch of Dynamic Test of a Model Structure Supported by Piles (After K. Kubo¹³¹⁾)

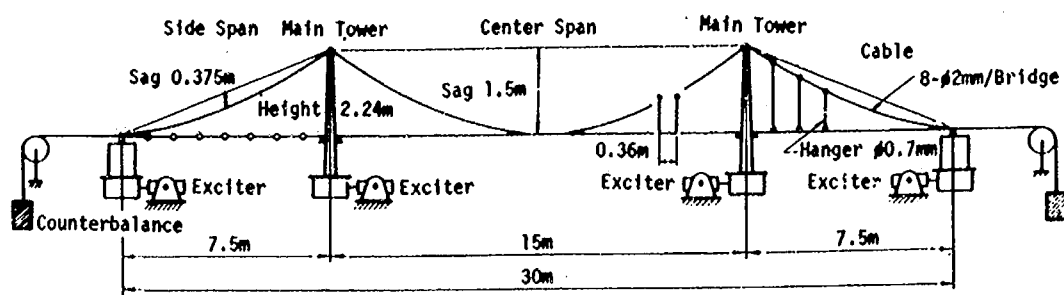


Fig. 3.80 General view of a model for a suspension bridge subjected to earthquake excitation (E. Kuribayashi, et al. (132))

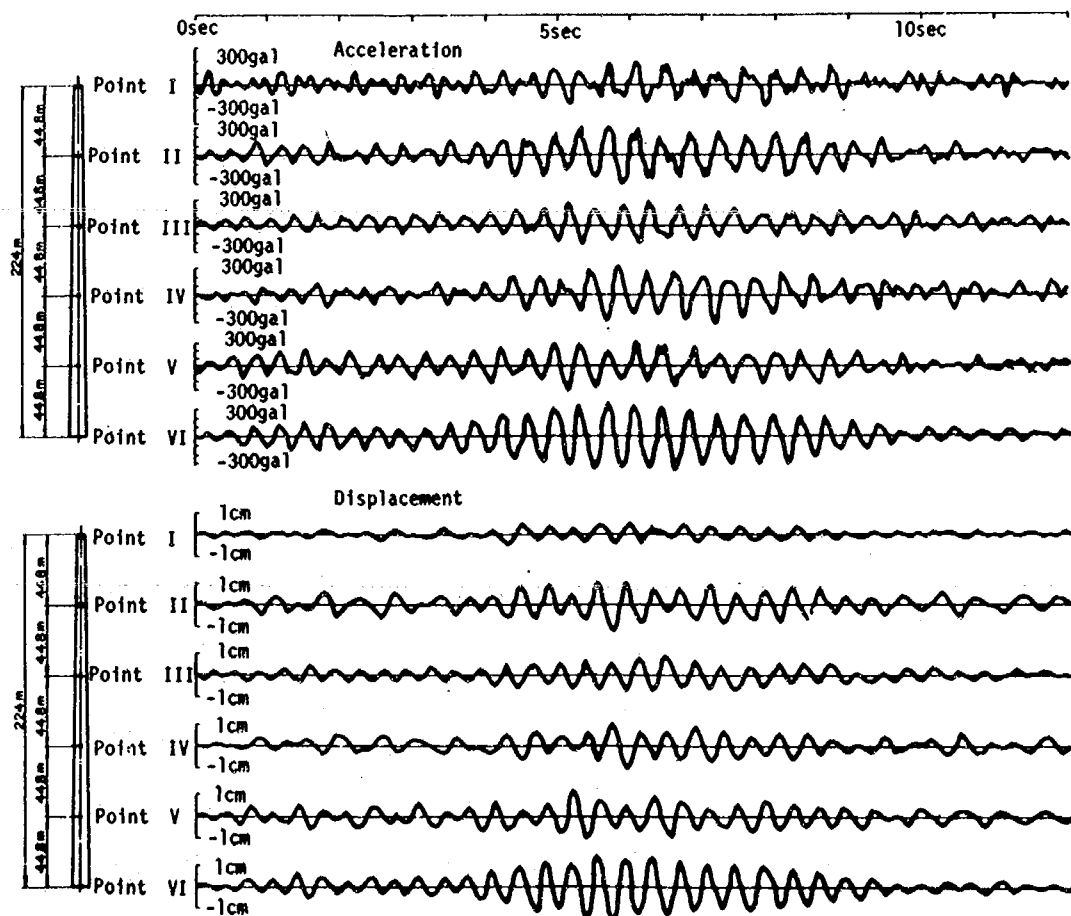


Fig. 3.81 Records of model response to the Kushiro Earthquake ground motion, E-W component (after E. Kuribayashi, et al. (132))

Model Test on Seismic Responses of Three-Span
Suspension Bridge

E. Kuribayashi, Y. Oyamada and Y. Iida 132)
studied seismic response of an overall model of a
three-span suspension bridge proposed to con-
nect Honshu and Shikoku islands. In the experi-
mental study an one hundredth model was

fabricated on a synchronized electro-magnetic
earthquake simulator system with four discrete
shaking tables, as shown in Fig. 3.80.

A result of the experiment, where the model
was excited at one table which supported one of
towers, is indicated in Fig. 3.81. The results of
the experiment were compared with the design
values and also with the results of dynamic
analysis.

CHAPTER 4 EARTHQUAKE-RESISTANT DESIGN CRITERIA FOR BRIDGES

This chapter is dealing with earthquake-resistant design criteria for bridges. Although the chapter covers primarily current Japanese bridge criteria, earthquake regulations for structures in other countries are also mentioned briefly.

Section 4.1 describes a short history of earthquake-resistant design criteria for highway bridges in Japan.

Section 4.2 outlines "Specifications for Earthquake-Resistant Design of Highway Bridges" proposed by the Japan Road Association in January, 1971, to make out current specifications for general Japanese highway bridges. The section also briefs four tentative specifications for highway bridges administrated by the Japan Highway Public Corporation (JHPC), the Tokyo Expressway Public Corporation (TEPC), the Hanshin Expressway Public Corporation (HEPC), and the Honshu-Shikoku Bridge Authority (HSBA). Furthermore, the section mentions about aseismic design of railway bridges administrated by the Japanese National Railways (JNR).

Section 4.3 summarizes earthquake regulations in various countries, and compares factors affecting design seismic coefficients and magnitudes of seismic coefficients to be taken into account in the aseismic design among various countries.

4.1 History of Earthquake-Resistant Design Criteria for Japanese Highway Bridges

The Ministry of Home Affairs stipulated in 1926 "Specifications for Design of Road," which are parts of "Road Laws." In the specifications seismic forces were first taken into account in the design of highway bridges, in consideration that several highway bridges sustained substantial damage during the Kanto Earthquake of 1923. The specifications provided that highway bridges shall be designed in accordance with the seismic coefficient method, in which horizontal seismic coefficients were taken from 0.1 to 0.4. The values of the coefficients were dependent on areas and ground conditions.

For bridges to be constructed in Tokyo and Yokohama, seismic coefficients of 0.3 or more were recommended.

This seems due to the substantial damage to bridge structures in the areas during the Kanto Earthquake.

The Ministry of Home Affairs issued in 1939 "Specifications for Design of Steel Highways," which took place of the former ones in the design of highway bridges. The specifications stipulated that seismic forces should be taken into account in accordance with the seismic coefficients method, considering a horizontal coefficient of 0.2 and a vertical coefficient of 0.1, simultaneously.

The specifications were revised in 1964 by the Japan Road Association, with a commission from the Ministry of Construction. The revised specifications specified that horizontal coefficient of 0.1 to 0.35 depending on areas and ground conditions and vertical coefficient of 0.1 should be considered in the aseismic design, simultaneously.

In view of recent technological advances in bridge engineering, earthquake engineering and other scientific fields, the Japan Road Association, also with a commission from the Ministry of Construction, drew up in January, 1971, comprehensive specifications exclusively for earthquake-resistant design of highway bridges. In the current specifications two methods are provided for the aseismic design. One is the conventional seismic coefficient method for rigid structures, where the horizontal coefficient ranges between 0.1 and 0.24 depending on areas, ground conditions, and importance. The other is the modified seismic coefficient considering structural response for comparatively flexible structures, where the horizontal seismic coefficient vary from 0.05 to 0.3 depending on the fundamental natural periods in addition to the above three factors.

On the other hand, characteristic criteria have been proposed, between 1966 and 1967, tentatively for the aseismic design of highway bridges relating to specific projects administrated by JHPC, TEPC, HEPC, and HSBA. Moreover, the Japanese National Railways stipulated in 1968 its own criteria for aseismic design of railway bridges.

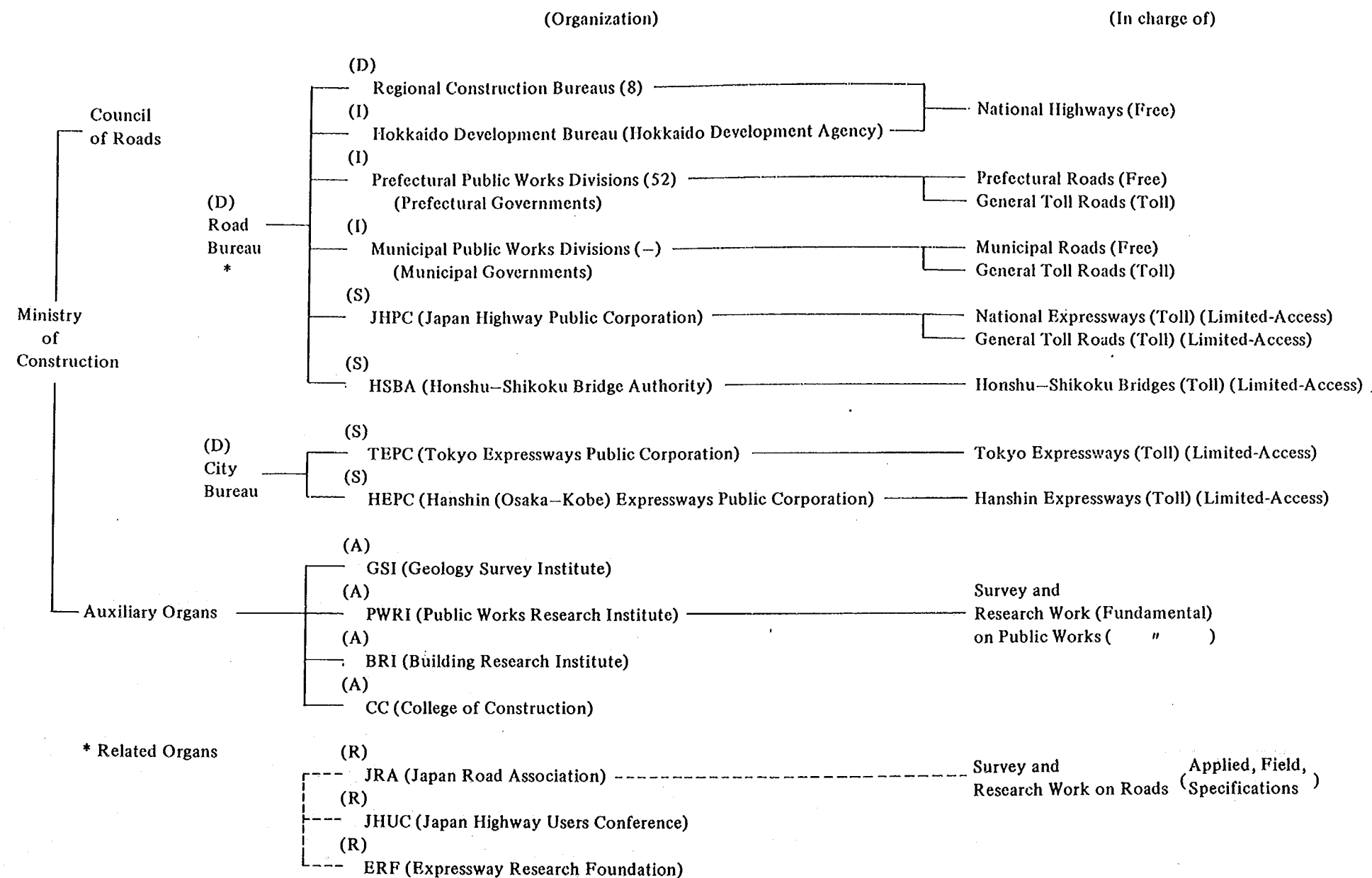
Table 4.1 tabulates briefly the above mentioned history of design loads (primarily seismic loads) for highway bridges in Japan. Table 4.2

Table 4.1 History of Design Loads for Highway Bridges in Japan (Primarily Seismic Loads)

Year	Name of Regulations	Class	Design Live Loads			Impact Loads	Seismic Loads k: Horizontal Seismic Coefficient	Other Seismic Regulations (k: seismic coef.)	Major Earthquakes
			Truck Roller Streetcar Loads	Uniform Loads	Line Loads				
1) 1886	Order No. 13 Ministry of Home Affairs (MHA)			U-454 kg/m	—	not considered	not considered		(M>7) after Rika Nenpyo (1971)
2) 1906	Order MHA		R-13.6 t S-12.7	U-400 600 kg/m (carriage way) U-270 400 kg/m (footway)	—	not considered	not considered		
3) 1919	Road Laws MHA		R-13.6 t T-11.2 t S-30 t	Same as 2)	—	not considered	not considered		
4) 1926	Specifications for Design of Roads, Road Laws, MHA	1st	T-12 t	U-600kg/m		considered	Seismic Coefficient Method $k=0.15\sim0.4$ depending on location and ground condition $k\geq 0.3$ advised in Tokyo, Yokohama	1924. Building Law ($k\geq 0.1$) 1950. Kawasumi's Seismic Map.	1923. Kanto (M=7.9)
		2nd	T-8 t	U-500 kg/m	—				
		3rd	T-6 t	U-500 kg/m					
5) 1939	Specifications for Design of Steel Highway Bridges, MHA	1st 2nd	T-13t T-9 t	U-500 kg/m U-400 kg/m		considered	Seismic Coefficient Method $k_h=0.2, k_v=0.1$	1950. New Building Law ($k=0.2, H\leq 16m$ increase 0.01 per an increase of 4m above 16m) 1964. High-Rise Building. ($H\geq 45m$) $k=(0.18\sim0.36)/T$ (T: period) ($0.05\leq k\leq 0.2$) 1965. Report of JSCE on Civil ERD of Engineering Structures.	1946. Nankai (M=8.1) 1948. Fukui (M=7.3) 1952. Tokachioki (M=8.2) 1964. Niigata (M=7.5)
6) 1956 (and 1964)	Revision of Specifications for Design of Steel Highway Bridges, JRA	1st 2nd	T-20 t T-14 t	— —	L-20 (5t/m) L-14 (3.5 t/m)	considered	Seismic Coefficient Method $k=0.1\sim0.35$ depending on location and ground conditions		
7) 1964 to 1971	Specifications for Design of Substructures of Highway Bridges, JRA	same as 6)					same k as 6) Detailed calculation methods		
8) 1966 (and 1970)	Design Standards, JHPC	same as 6)					Design Code for High-Rise Bridges, (Increase in k with the height of the piers)	1968. Report of JSCE on Earthquake-Resistant Design of Substructures of Railway Bridges.	1968. Tokachi-oki (M=7.9)
9) 1967	Specifications for Earthquake Resistant Design of Honshu-Shikoku Bridges, JSCE	Special Live Loads and Impact Loads for Suspension Bridges are Considered					Modified Seismic Coefficient Method (Basic Coef. $k=0.2$)		
10) 1967	Specifications for Earthquake-Resistant Design of Tokyo Expressways, TEPC	same as 6)					Seismic Coef. Method $k=0.2\sim0.3$		
11) 1968	Aseismic Design Standards, HEPC	same as 6)					Seismic Coef. Method $k=0.2\sim0.28$		
12) 1971	Specifications for Earthquake Resistant Design of Highway Bridges, JRA	same as 6)					Seismic Coef. Method $k=0.1\sim0.24$ (Rigid) Modified SCM. $k=0.05\sim0.3$ (Flexible)		

Note) Regulations in 8) to 11) are relating to special projects.

Table 4. 2 Highway Administration in Japan (Technical Field)



Abbreviation

- A : Auxiliary Organs of the Ministry of Construction
D : Directly Controlled by the Ministry of Construction (or Organs of the Ministry)
I : Indirectly Controlled by the Ministry of Construction
R : Related Organs
S : Supervised by the Ministry of Construction

- (Notes) 1. The above information may not be complete.
2. JSCE (Japan Society of Civil Engineers) is doing survey and research work in the academic field.

gives brief information on the technical fields of the highway administrations and research in Japan, for the convenience of the reader who is not familiar with them. Since the information shown in Table 4.2 is not official, it may not be correct.

4.2 Earthquake-Resistant Design Criteria for Japanese Bridges

Specifications for Earthquake-Resistant Design of Highway Bridges (JRA-1971)²²⁾

Since 1956, the earthquake-resistant design of highway bridges had been conducted in accordance with the provisions of Section 13 (Section 2.9 in the English version) of "Specifications for Design of Steel Highway Bridges,"⁴⁾ which were issued in 1956 by the Japan Road Association originally and revised in 1964 (the English version were issued in 1968). The Section provided the followings.

- (1) The horizontal design seismic coefficient shall be determined by Table 4.3.
- (2) The vertical design seismic coefficient shall be assumed as 0.10.
- (3) Increases in allowable stresses for seismic forces alone or for dead loads plus seismic loads shall be taken as
 - 70% for steel structures,
 - 50% for concrete structures and reinforced concrete structures.

Since the provisions in the Section 13 were not comprehensive, it was necessary to draw up new detailed specifications exclusively for

aseismic design of highway bridges. Current "Specifications for Earthquake-Resistant Design of Highway Bridges"²²⁾ were issued in January 1971, by the Japan Road Association, which apply to the design of highway bridges with spans not longer than 200 meters, to be constructed on expressways, national highways, prefectural highways and principal municipal highways.

The specifications basically stipulate to employ seismic coefficient methods and provide two methods in determining design seismic coefficients. One is the conventional seismic coefficient method that applies to the design of relatively rigid structures. The other is the modified seismic coefficient method considering structural response that applies to the design of relatively flexible structures. The followings are the principal points of the specifications.

(1) The horizontal design seismic coefficient for a rigid structure is determined systematically, depending on the geographical location of the bridge site, the ground conditions at each sub-structure site, and the importance of the bridge. The horizontal design coefficient for a flexible structure is determined depending on the fundamental natural period of each structural system.

(a) In the seismic coefficient method that is employed for relatively rigid structures, the horizontal design seismic coefficient (k_h) shall be determined by

$$k_h = \nu_1 \nu_2 \nu_3 k_0 \quad (4.1)$$

where

k_h : horizontal design seismic coef-

Table 4.3 Horizontal Design Seismic Coefficient (Out of Date)

Ground Conditions* Regions*	Weak	Ordinary	Firm
Where severe earthquakes have been frequently experienced	0.35 ~ 0.30	0.30 ~ 0.20	0.20 ~ 0.15
Where severe earthquakes have been occurred	0.30 ~ 0.20	0.20 ~ 0.15	0.15 ~ 0.10
Other regions	0.20	0.15	0.10

* no further discription on regions and ground conditions.

k_o : coefficient,
 standard horizontal design seismic
 coefficient (= 0.2),
 ν_1 : seismic zone factor,
 ν_2 : ground condition factor,
 ν_3 : importance factor.

Table 4.4 Seismic Zone/Factor ν_1
for/General Highway Bridges

Zone	Value of ν_1
A	1.00
B	0.85
C	0.70

The values of ν_1 , ν_2 , and ν_3 are shown in Tables 4.4, 4.5 and 4.6, respectively. The definitions of classification are specified in the provisions. The minimum value of k_h shall be considered as 0.10.

(b) In the modified seismic coefficient method considering structural response that is employed for relatively flexible structures such as a bridge with highrise piers higher than 25 m or a bridge with a fundamental period longer than 0.5 seconds, the horizontal design seismic coefficient (k_{hm}) shall be determined by

$$k_{hm} = \beta k_h \dots \dots \dots (4.2)$$

Table 4.5 Ground Condition Factor ν_2 for General Highway Bridges

Group	Definitions ¹⁾	Value of ν_2
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer ²⁾ with depth less than 10 meters above bedrock	0.9
2	(1) Diluvial layer ²⁾ with depth greater than 10 meters above bedrock (2) Alluvial layer ³⁾ with depth less than 10 meters above bedrock	1.0
3	Alluvial layer ³⁾ with depth less than 25 meters, which has soft layer ⁴⁾ with depth less than 5 meters	1.1
4	Other than the above	1.2

- (Notes) 1) Since these definitions are not very comprehensive, the classification of ground conditions shall be made with adequate consideration of the bridge site.
 Depth of layer indicated here shall be measured from the actual ground surface.
- 2) Diluvial layer implies a dense alluvial layer such as a dense sandy layer, gravel layer, or cobble layer.
- 3) Alluvial layer implies a new sedimentary layer made by a landslide.
- 4) Soft layer is defined in Section 3.7 "Soil Layer Whose Bearing Capacities are Neglected in Earthquake Resistant Design."

Table 4.6 Importance Factor ν_3 for General Highway Bridges

Group	Definitions	Value of ν_3
1	Bridges on expressways (limited-access highways), general national highways and principal prefectural highways. Important Bridges on general prefectural highways and municipal highways.	1.0
2	Other than the above	0.8

Note: The value of ν_3 may be increased up to 1.25 for special cases in Group 1.

where

k_{hm} : horizontal design seismic coefficient in the modified seismic coefficient method considering structural response,

k_h : coefficient given by eq. (4.1),
 β : a factor dependent on the fundamental period of the bridge, and obtained by Fig. 4.1.

For structures whose fundamental periods are shorter than 0.5 seconds, β may be considered as 1.0.

The minimum value of k_{hm} shall be 0.05.

(2) The vertical design seismic coefficient may generally be considered as zero, except for special portions such as bearing supports.

(3) The horizontal design seismic coefficient for structural parts, soils and water below the ground surface may be considered as zero.

(4) Hydrodynamic pressure during earthquakes are specified in the specifications. Earth pressures during earthquakes, however, are specified in the related specifications.

(5) A specific attention is paid to very soft soil layers and soil layers vulnerable to liquefaction during earthquakes. The bearing capacities of these layers are neglected in the design, in order to assure high earthquake-resistance for structures that are built in these layers.

(6) A special attention is also paid to the design of structural details, in consideration of the damage previously experienced to bridge structures. To this aim provisions are specified for bearing supports and devices for preventing bridge girders from falling.

(7) Increases in allowable stresses of materials

may be considered in the earthquake-resistant design, magnitudes of increases for various materials are specified in the several related specifications. The increasing rates are as follows:

concrete in reinforced concrete structures:	50%
reinforcements in reinforced concrete structures:	50%
structural steel for superstructures:	70%
structural steel for substructures:	50%
concrete in prestressed concrete structures against compressive forces:	65%
foundation soils:	50%

Detailed descriptions on the specifications are provided in Appendices B and C of this report.

Specifications for Design of Highways of JHPC (JHPC-1970) ¹⁸⁾

The Japan Highway Public Corporation provides comprehensive "Specifications for Design of Highways" issued in January, 1970 by the Expressway Research Foundation, which consist of three volumes. Section 6.2 of Part 5 in Volume 2 specifies provisions on seismic effects on bridge structures. The following summarizes the provisions of the Section.

(1) General

The earthquake-resistant design of a highway bridge shall provide sufficient stability against seismic disturbances for the structure as a unit and also for all parts thereof, including superstructures, substructures, and surrounding soils, in consideration of topographical and geological conditions at the site.

(2) Design Seismic Coefficient

(a) The horizontal design seismic coefficient shall be determined in view of survey on seismic effects on bridge structures previously experienced, geological features, characteristics of microtremors at the site, and the relevant specifications for earthquake-resistant design of engineering structures.

(b) The vertical design seismic coefficient shall generally be assumed as 0.10.

(c) Design seismic coefficient for structural portions underground and for portions lower than 15 m above the ground surface, shall be the basic coefficient mentioned in (a) and (b) of this item (2).

(3) Increase in Horizontal Design Seismic Coefficient for Highrise structures

(a) The horizontal design seismic coefficient for structural portions higher than 15 m above the ground surface, shall be determined by increasing the basic coefficient by 5% per an increase of 5 m, in height. The vertical design seismic coefficient does not need to increase.

(b) For those bridges which have continuous steel superstructure with reinforced concrete slab, each span length of which is about 50 to 90 m and have reinforced concrete or steel frame reinforced concrete substructures with solid section, a special increase in horizontal design seismic coefficient shall be needed in the transverse direction of the bridge axis. Horizontal design seismic coefficient for the abovementioned bridges vary with the height of the structure as follows:

basic coefficient, for $0 < H < 15$ m

5% increase per 5 m, for $15 \leq H \leq 25$ m,

14% increase per 5 m, for $25 \leq H \leq 40$ m,

same as the value for $H = 40$ m, for $H > 40$ m,

where H denotes the height of structural

portions above the ground surface.

Horizontal design seismic coefficient in the longitudinal direction shall be provided in accordance with (a) of this item (3).

It should be noted that the provisions in the Section are invalid, after the Specifications for Earthquake-Resistant Design of Highway Bridges²²⁾ took the place of them.

Aseismic Design Standards of Tokyo Expressways (TEPC-1967)¹⁰⁾

The Tokyo Expressway Public Corporation has its own tentative standards for aseismic design of bridges, which were issued in July, 1967. The main points of the standards are as follows:

(1) The design seismic coefficients are determined in accordance with Table 4.7. Ground conditions in the table shall be decided by Table 4.8. The following notes shall be considered in the aseismic design.

(a) To structures that are constructed in very soft soil layers and to structures whose fundamental periods may coincide approximately with the predominant periods of the ground, a particular attention shall be paid by conducting dynamic analysis.

(b) Parentheses in Table 4.8 are for those structures that have rigid foundations with ratio of depth to diameter (or shorter length at the base) of 3 or less.

(c) In cases where the depth of alluvial layer (or loam layer) is between 0 and 3 m and a firm layer exists below the thin layer, the thin layer shall be removed in the construction. In these cases the ground condition will be classified into 1.

(2) Combination of applied loads and allowable stresses in the earthquake-resistant design are tabulated in Table 4.9. In the table principal loads designate dead loads, earth pressures, hydraulic pressures, buoyancy, and uplifts.

Table 4.7 Design Seismic Coefficient for Tokyo Expressways

Classification of Ground Conditions		1	2	3	4
Horizontal Seismic Coefficient k_h	Above Ground	0.20	0.24	0.27	0.30
	Under-ground	0.15	0.18	0.20	0.23
Vertical Seismic Coefficient, k_v		0.1	0.1	0.1	0.1

Table 4.8 Classification of Ground Conditions for Tokyo Expressways*

Properties of Alluvial Layer and Loam Layer** Depth of Alluvial Layer and Loam Layer	Gravel	Sand, Clay, and Loam ($N \geq 5$)	Soft Soil	
			$N = 2 \sim 5$	$N < 2$
0 ~ 3 m	See Note (C)			
3 ~ 10 m	3 (2)	3 (2)	4 (3)	4 (3)
10 ~ 25 m	3 (2)	4 (3)	4 (3)	4 (4)
25 m or more	4 (3)	4 (3)	4 (4)	4 (4)

* See Notes (a) ~ (c) on the report.

** N indicates number of blow per 30 cm by the standard penetration test

Table 4.9 Combination of Loads and Allowable Stresses in the Earthquake-Resistant Design of Tokyo Expressways

Type of Structures		Combination of Loads	Increases in Allowable Stresses (%)	Allowable Stress of Concrete
Foundations or Retaining Walls	Reinforced Concrete Structures	Principal Loads + Seismic Loads	50	$\frac{\sigma_{2.8}}{3}$ or 100 kg/cm ²
	Reinforced Concrete Structure	Principal Loads + Temperature + Humidity + Seismic Loads	65	
	Steel Structures	Principal Loads + Seismic Loads	50	
Superstructures or Piers	Reinforced Concrete Structures	Dead Loads + Seismic Loads	50	$\frac{\sigma_{2.8}}{3}$
	Reinforced Concrete Structures	Dead Loads + Temperature + Humidity + Seismic Loads	65	
	Steel Structures	Dead Loads + Seismic Loads	70	

Table 4.10 Values of Factor m for Hanshin Expressways

Depth of Embedment of Foundation	N-Values of Sandy Soils	$N \geq 30$	$30 \geq N \geq 10$	$N < 10$
	N-Values of Cohesive Soils	$N \geq 8$	$8 \geq N \geq 4$	$N < 4$
less than 10 m		1.0	1.0	1.1
10 m ~ 23 m		1.0	1.1	1.25
25 m or more		1.1	1.25	1.4

Furthermore, foundations imply spread foundations, pile foundations, and caisson foundations, and piers denote parts of substructures above the crests of the foundations.

Seismic Design Standards of HEPC (HEPC-1968)¹⁶⁾

The Hanshin (Osaka-Kobe) Expressway Public Corporation also has its own tentative standards for earthquake-resistant design of engineering structures related to the expressways. The following summarizes the standards issued in December, 1968.

(1) Design Seismic Coefficient

(a) Civil engineering structures shall generally be designed by means of the seismic coefficient method provided below.

(b) The horizontal design seismic coefficient (k_h) shall be determined by

$$k_h = mk_0 \quad (4.3)$$

where

k_0 : basic seismic coefficient (= 0.2)

m : A modification factor dependent on the depth of embedment of the foundation and the average N value (Number of blow by the standard penetration test) at the structural site, and given by Table 4.10.

Seismic Coefficients for structural portions higher than 10m above the ground surface shall be increased by 1% per an increase of 1m in height.

(c) The vertical seismic coefficient shall be one half of the horizontal one.

(d) Seismic coefficients for retaining walls shall be 0.2 in the horizontal, and 0.1 in the vertical, simultaneously.

(2) Earth pressures during earthquakes shall be calculated by the Mononobe-Okabe method.

(3) Provisions on stability of structures and allowable stresses are specified.

Specifications for Earthquake-Resistant Design of Honshu-Shikoku Bridges (JSCE-1967)⁹⁾

The Japan Society of Civil Engineers issued "Specifications for Earthquake-Resistant Design of Honshu-Shikoku Bridges" in May, 1967, with

a commission jointly from the Ministry of Construction and the Japanese National Railways. The specifications are primarily for long-span suspension bridges with huge foundations under the deep sea, and are remarkable ones which stipulate first a modified seismic coefficient method in which dynamic structural response is considered. The principal points of the specifications are as follows:

Table 4.11 Value of Factor S for Honshu-Shikoku Suspension Bridges

Soil Condition	Factor S
Rock (Tertiary or older)	1.0
Diluvium	1.1
Alluvium	1.2

(1) The basic design shall be made by means of the conventional seismic coefficient method or modified seismic coefficient method considering structural response. The results of the basic design are desirable to be checked through dynamic analysis.

(a) Seismic Coefficient Method.

The horizontal seismic coefficient (k_h) and the vertical seismic coefficient (k_v) shall be determined by

$$k_h = S \cdot C_0 \quad (4.4)$$

$$k_v = \frac{1}{2} k_h \quad (4.5)$$

where

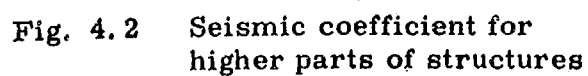
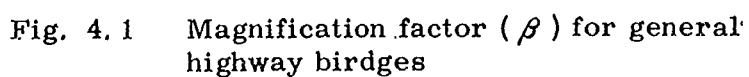
S : A modification factor depending on the ground conditions and given by Table 4.11.

C : Basic coefficient (= 0.2) (The value of C was amended to 0.18 according to the report of JSCE (1971)¹⁵⁾). Increases in k_h with the height of structures shall be given by Fig. 4.2.

(b) Modified Seismic Coefficient Method Considering Structural Response

The design seismic coefficient (k) shall be determined by

$$k = m_1 \cdot m_2 \cdot C \quad (4.6)$$



where

m_1 : A dynamic magnification factor for a single-degree-of-freedom system subjected to seismic excitation. The factor is depending on the structural characteristics such as fundamental periods, damping capacities, nonlinearity, etc., and is given by Fig. 4.3 for each structural part.

m_2 : A factor needed when assuming that actual structures are considered as simple systems to which simplified distribution of design seismic loads.

C: $C = C_0 = 0.2$ horizontally and 0.1 vertically (amended to 0.18 horizontally and 0.09 vertically, according to the report of JSCE (1971)¹⁵⁰) for substructures. The values of C shall be the coefficient k_T at the top of the corresponding main piers for main towers, and the coefficient k_T at the top of the abutment for suspended structures and cables, as shown in Fig. 4.4.

It is noted that the modified seismic coefficient method considering structural response is ordinarily employed for design practices, rather than the one mentioned in (a).

(c) Dynamic Analysis

To evaluate the earthquake response of a suspension bridge under consideration, dynamic analysis shall be carried out with regard to a whole structural system including both the superstructure and the substructure. Either one of response spectrum method and numerical integration method may be employed in the dynamic analysis. Dynamic properties of structures shall be determined with sufficient consideration.

According to the report of JSCE (1971)¹⁵⁰, the damping ratios (h) may be assumed as follows when no information is available.

$h = 0.1$ for rigid substructures,

$h = 0.05$ for flexible substructures made of concrete,

$h = 0.02$ for steel structures,

where h denotes the damping ratio expressed as fraction of critical damping.

(2) The effects of earth pressures and hydraulic pressures on substructures during earthquakes

shall be taken into account in the design and also in the dynamic analysis. Formulas to calculate hydrodynamic pressures during earthquakes are provided (see eq. (3.11)).

(3) Consideration on Structural Safety

(a) Combination of Applied Loads

In the design of superstructures, dead loads, live loads during earthquakes, and temperature change shall be combined with the seismic effects.

(b) Stability of Structures

In the design taking into account seismic loads, the allowable bearing capacity of the ground may be 1.5 times the allowable capacity for ordinary loads. The increased capacity, however, shall be less than three quarters of the yielding capacity of the ground, and the displacements of structures shall be smaller than the allowable limits. The point of the resultant force acting at the base of foundation shall exist within two thirds of the center of the base. The factor of safety against sliding shall be greater than 1.5, in the aseismic design of substructures.

(c) Strength of Members

Stresses in steel structures such as towers, cables, stiffening girders and hangers shall be within the elastic ranges. Stresses in local members may exceed the elastic ranges, provided that the structure does not suffer severely as a whole. Stresses in concrete structures shall be in accordance with the provisions in "Standard Specifications for Concrete"⁸) published by JSCE.

(d) Displacements and deformations of superstructures and substructures shall be less than the allowable limits that will be determined by the stability of the structure as a whole.

Aseismic Design Criteria for Railway Bridges (JSCE-1968)¹⁵

The Japan Society of Civil Engineers made up "Report on the Aseismic Design of the Substructures of Railway Bridges" in 1968, with a commission from the Japanese National Railways. The report proposes earthquake-resistant criteria for railway bridges. The principal points are as follows:

(1) Horizontal Seismic Coefficient

The horizontal design seismic coefficient

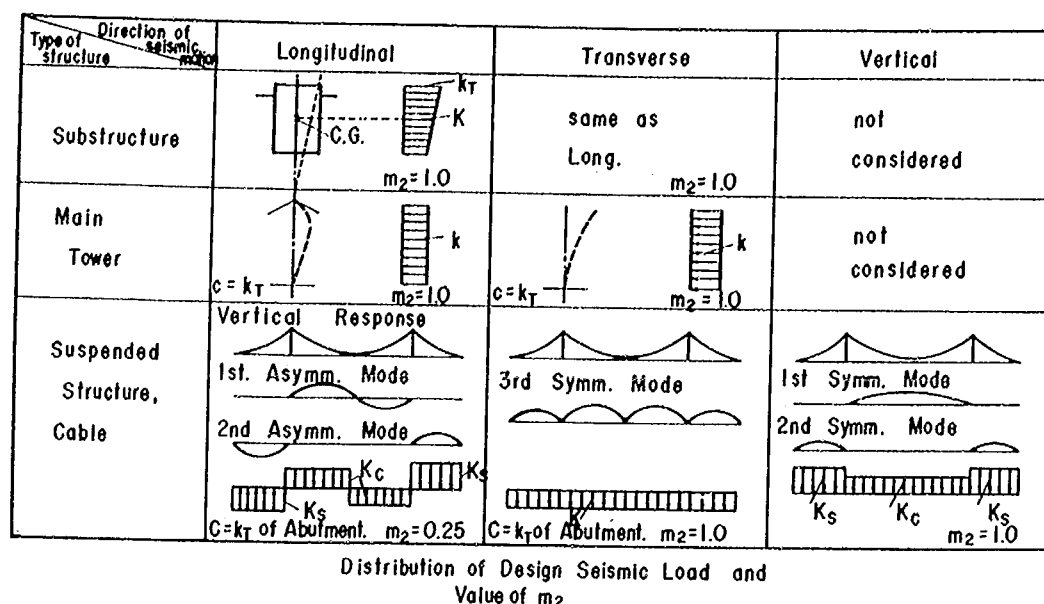


Fig. 4.4 Distribution of design seismic loads and value of m_2

(k_h) shall be determined by

$$k_h = m_s \cdot m_I \cdot k_o \quad (4.7)$$

$$k_h = m_o \cdot m_I \cdot k_o \quad (4.7)$$

where

k_o : basic seismic coefficient,
 $k_o = 0.2$ for zone A and 0.15 for zone B,

m_s : soil condition factor,
 $m_s = 1.2$ for soft layers, 1.0 for medium layers, and 0.8 for hard layers,

m_I : importance factor,
 $m_I = 1.1$ for more important structures, 1.0 for ordinary structures and 0.9 for less important structures.

Table 4.12 indicates the values of k_h for various conditions.

Definitions of soil conditions are as follows:

- Soft Layers: soft layers that have depth of 2 to 5m with N-value (Number of blow by the standard penetration test) of zero, depth of 5 to 10m with N-value less than 2, or depth of 10m or more with N-value less than 4,
- Medium Layer: Diluvial or alluvial layers except soft layers defined above,

Hard Layer: Rock layers formed in the Tertiary or older.

(2) The vertical design seismic coefficient shall be one half of the horizontal one, when required to be considered.

(3) The design seismic coefficient for portions of structures higher than 10m above the ground surface, shall be determined by increasing the values in Tables 4.12 by 1% per an increase of 1m in height, just similar to the preceeding specifications (see Fig. 4.2).

(4) The design seismic coefficient for underground portions of structures and underground structures shall be determined by considering m_s shown in Fig. 4.5 for various ground conditions.

(5) Earth pressures during earthquakes shall be obtained by the Mononobe-Okabe method.

(6) Hydrodynamic pressures during earthquakes, which shall be added to static pressures, are given by the same formulas as the preceeding specifications (see eq. (3.11)).

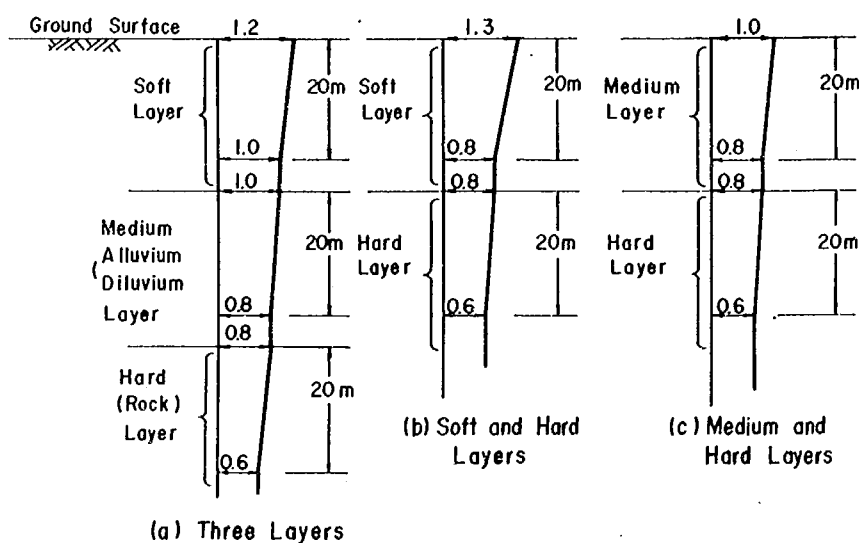
4.3 Earthquake Regulations in Various Countries

Table 4.13 summarizes earthquake regulations in twenty one countries, by referring to an available world list of earthquake regulations²¹⁾,

Table 4. 12 Design Horizontal Seismic Coefficient ($k_h = m_s \cdot m_i \cdot k_o$) for JNR Bridges

Zone			Λ ($k_o = 0.2$)			B ($k_o = 0.15$)		
Soil Condition			Soft	Medium	Hard	Soft	Medium	Hard
			$m_s = 1.2$	$m_s = 1.0$	$m_s = 0.8$	$m_s = 1.2$	$m_s = 1.0$	$m_s = 0.8$
Degree of Importance or Structure	Important	$m_i = 1.1$	0.25	0.2	0.2	0.2	0.15	0.15
	Ordinary	$m_i = 1.0$	0.25	0.2	0.15	0.2	0.15	0.1
	Less important	$m_i = 0.9$	0.2	0.2	0.15	0.15	0.15	0.1

k_o : Basic seismic coefficient

Fig. 4.5 Value of m_2 for underground layers

and also to the specifications for highway bridges in the U.S.A. and those in Japan. Some of the regulations shown in the world list are obliged to omit from Table 4.13, due to the language difficulties. In the table approximate ranges of horizontal design seismic coefficients in the highest seismic zone for each country, factors affecting the design seismic coefficients, and increases in allowable stresses of materials in the aseismic design, are presented. It should be noted, however, that most of these regulations are primarily for the design of buildings, and only four out of the twenty one countries present earthquake regulations for highway bridges.

As for magnitudes of horizontal seismic

coefficients, five countries take zero to 0.1 ten countries zero to 0.2, four countries zero to 0.3, and two countries zero to 0.4. As for factors affecting design seismic coefficients, most regulations point out seismicity (or seismic zones) and ground conditions (or types of foundations), and more than half countries indicate natural periods (or height, or number of stories), importance (or usage), and structural members (or structural properties). Furthermore, some regulations take into account the effects of mode shapes in determining design seismic coefficients.

Appendix B in this report is briefing individual regulations in these twenty one countries, in the prescribed form.

Table 4.13 Design Seismic Coefficients in Various Countries

Country	Approximate Range of Horizontal Design Seismic Coefficients In Various Countries In the Highest Seismic Zone for Each Country	Factors Affecting Design Seismic Coefficients ¹⁾						Increase in Allowable Stresses for Earthquake Resistant Design ²⁾	Structures to which the Regulations Apply Primarily
		Zone	Ground Conditions or Type of Foundations	Period, Height or No. of Stories	Importance or Usage	Structural Member or Structural Properties	Mode Shape		
	0 0.1 0.2 0.3 0.4								
Argentina		○	○	○	○	○	X		Buildings
Bulgaria		○	○	○	○	○	○	50%	"
Canada		○	○	○	○	○	X		"
Chile		X	Δ	○	○	○	X		"
Cuba { Static Dynamic		X	○	X	○	○	X	W, SS, B : 50% ME, C, MA, G : 33%	"
EL Salvador		○	X	X	○	○	X	W, SS, B : 50% ME, C, MA : 33%	"
Greece		○	○	X	X	X	X	C, S : 20%	"
India		○	○	X	X	○	X	G : 30 ~ 50%	Buildings and Bridges
Iran { Low High-Rise		○	○	X	○	X	X	33%	Buildings
Israel		○	○	○	X	○	X	33%	"
Italy		○	X	X	X	X	X	SS : 1400 kg/cm ² HS : 2000 kg/cm ²	"
Japan { Rigid Flexible		○	○	X	○	○	X	SS : 70% B, C : 50%	Highway Bridges (Japan Road Ascc.)
Mexico		○	○	X	○	○	X	W, SS, B : 50% ME, C, MA : 33%	Buildings
New Zealand		○	X	○	○	X	X		"
Peru		○	X	○	○	○	X	SS, C : 33%	"
Philippines		○	○	○	X	○	X		"
Portugal		○	○	X	X	○	X	SS : 2400 kg/cm ² C : 100%	"
Rumania		○	○	○	X	○	○		"
Turkey		○	○	○	X	○	X	50%	Buildings and Bridges
U.S.A. { AASHO California		X	○	X	X	X	X	33%	Highway Bridges
Venezuela		○	○	X	○	○	X	33%	Buildings

Notes: 1) Factors with circles are considered, and factors with triangles are considered partially or indirectly.

2) B : Bars for reinforcement, C : Concrete, G : Ground, HS : High-tension steel, MA : Masonry

ME : Metals other than steel, SS : Structural Steel, W : Wood

3) Mostly by "A World List of Seismic Regulations (1970)", 4) See Appendix B for further details.

CHAPTER 5 SUMMARY AND CONCLUSIONS

Seismic damage to bridge structures are extensively described in Chapter 2, by making full use of photographs and drawings available. Nine major earthquakes occurred in Japan since the Kanto Earthquake occurred in 1923, are surveyed for the purpose of clarifying features of seismic damage to bridges. The followings are their names and numbers of bridges on which detailed descriptions of seismic damage.

- (1) Kanto Earthquake of 1923
(Magnitude = 7.9) 10 bridges
- (2) Nankai Earthquake of 1946
(Magnitude = 8.1) 2 bridges
- (3) Fukui Earthquake of 1948
(Magnitude = 7.3) 7 bridges
- (4) Imaichi Earthquake of 1949
(Magnitude = 6.7) General Damage
- (5) Tokachi-oki Earthquake of 1952
(Magnitude = 8.1) General Damage
- (6) Northern Miyagi Earthquake of 1962
(Magnitude = 6.5) General Damage
- (7) Niigata Earthquake of 1964
(Magnitude = 7.5) 7 bridges
- (8) Ebino Earthquake of 1968
(Magnitude = 6.1) 3 bridges
- (9) Tokachi-oki Earthquake of 1968
(Magnitude = 7.9) General Damage

From the extensive study, general features of seismic damage to bridge structures are obtained.

Research activities related to seismic effects on bridges are summarized in Chapter 3. Various research works are intentionally classified into the following nine subjects, which seem essential to evaluate seismic effects on bridges.

- (1) Seismicity
- (2) Characteristics of strong ground motions
- (3) Ground disasters and bearing capacities of subsoils
- (4) Earth pressures
- (5) Hydraulic pressures
- (6) Dynamic properties of bridges
- (7) Measurement of earthquake responses of bridges
- (8) Dynamic analysis of bridges
- (9) Laboratory experiment on dynamic behavior of bridges

Current specifications for earthquakes-resist-

ant design of bridges are summarized in Chapter 4. A short history of changes in aseismic design criteria for highway bridges in Japan, is described briefly. The present specifications for earthquake-resistant design for general highway bridges, and four specifications for highway bridges on specific projects administrated by four Japanese Highway Public Corporations (JHPC, TEPC, HEPC and HSBA) are outlined. The aseismic design criteria of railway bridges in the Japanese National Railways are also briefed. Furthermore, earthquake regulations in twenty one countries are summarized, and factors affecting design seismic coefficients and magnitudes of seismic coefficients are compared among those countries.

From the comprehensive study through a complete review of the literature related to seismic effects on bridges, the followings may be concluded.

- (1) Seismic damage to bridge structures are generally caused by the lack of resistance at bearing supports, substructures, or surrounding soils. As the results of the weakness at these portions, substructures would tilt, settle, slide, cause cracks or failures, or overturn; superstructures would move, cause cracks or failures, or fall down; and bearing supports may cause failures. Moreover, it is often observed during earthquakes that appurtenant structures such as wing walls and approach banks settle, or separate from the main structures.
- (2) It seems that bridge structures properly designed and constructed in accordance with the current Japanese specifications, will withstand major earthquakes, without sustaining fatal damage. For providing adequate resistance to seismic disturbances, the magnitudes of horizontal design seismic coefficients are most significant. In addition, it seems important to give special attentions to (a) topographical and geological consideration to avoid ground disasters, (b) soil dynamic consideration such as liquefaction of surrounding soils, and (c) design details for protecting bridge girders from falling and for evading severe damage caused by the failures at structural joints.
- (3) Further investigations are needed on the subjects shown below, in improving aseismic design methodology of bridge structures.

(a) Evaluation of Seismicity and Ground Motions

For determining appropriate seismic forces to be considered in the design, it is important to statistically study frequencies of occurrences of strong earthquakes and to investigate characteristics of strong ground motions and their magnitudes in connection with various conditions concerned (such as ground conditions). It seems also important to clarify the differences of ground motions between one location and another nearby, from the viewpoint of the propagation of seismic waves during strong earthquakes.

(b) Structural Planning

For assuring appropriate structural resistance to seismic disturbances, it seems very significant to properly select structural types in consideration of seismicity, topography, geological conditions, soils conditions, etc. at the site. It is recommended to investigate for setting up a standard for selecting types of bridges including superstructures, substructures, and foundations, in connection with seismic effects.

(c) Effects of Subsoils on Bridge Structures during Earthquakes

It is necessary to evaluate dynamic earth pressures on substructures, bearing capacities of subsoils, effects of ground failures (such as faults, sliding, liquefaction, etc.) on bridge structures during earthquakes.

(d) Aseismic Design Method for Bridge Substructures

It is essential to establish a definite design method for substructures consisting of piers and abutments, together with various foundations (such as spread foundations, caisson foundations, pile foundations, etc.). The magnitude of seismic coefficient should be reasonable in the aseismic design. From the experiences of seismic damage and designing, the lowest value of horizontal seismic coefficient seems to be about 0.1 or greater, in providing ordinary bridge structures with indispensable resistance to seismic disturbances. It is also noted that appro-

priate information on the properties of ground soils is necessary in the aseismic design.

(e) Design Details of Superstructures and Supports

A specific attention shall be paid to design details of bearing supports and hinges, and also to devices to prevent the fall of girders from the crests of substructures. It seems reasonable that for superstructures design details are more significant than simply applying seismic forces on them.

(f) Measurement of Dynamic Properties and Seismic Response of Actual Bridges

It seems important to conduct field experiment for actual bridges in order to evaluate their dynamic properties such as natural periods, mode shapes, damping characters, etc. It is also significant to measure dynamic behavior during actual earthquakes by installing strong-motion seismographs on bridges and on grounds nearby.

(g) Dynamic Analysis

It seems reasonable that an analytical approach is a better way to estimate dynamic behavior of structures subjected to seismic excitation. In dynamic analysis it is very important to adequately evaluate analytical models which represent probable behavior of bridges during strong-motion earthquakes. For this aim it is essential to compare the results of the analysis with the measurement of response of actual structures to forced and seismic excitation. It is advisable to reproduce structural failures or collapses in analysis, with respect to the structures suffered by the past earthquakes.

(h) Laboratory Experiment on Dynamic Behavior of Bridges

Dynamic behavior of materials, simplified structural systems, and composite structures are preferable to be examined through laboratory experiment, employing exciters, actuators, or shaking tables. To make model

experiment more effective, it is necessary to establish similitude theory for structures which may behave beyond the elastic range, or may collapse when subjected to strong motions.

(i) Quantitative. Evaluation of Ultimate Strength of Bridges

To obtain quantitative information on ultimate strength of bridge structures, it is

effective to survey seismic damage to the similar structures experienced, and to clarify the differences of characteristics between damaged structures and undamaged ones. It is also important to establish standard ways to quantitatively clarify structural ultimate strength. For an example, dynamic experiment or dynamic analysis may be effective means to pursue structural behavior until suffering failures or collapses.

APPENDIX A

BIBLIOGRAPHY ON SEISMIC EFFECTS ON BRIDGES (PRIMARILY JAPANESE LITERATURE)

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APPENDIX B

BRIEFING OF SEISMIC DESIGN CODES IN VARIOUS COUNTRIES

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Country	Specifications	Seismic Coefficient, k (k in $V = kW$)	Increase in Allowable Stresses	Remarks																														
Argentina	CINEH Centro Para Estudio De Normas Estructurales Del Holmigon	$k = \gamma_e \cdot \gamma_d \cdot S \cdot C_o$ <table><thead><tr><th>Zone</th><th>Basic Coefficient C_o</th></tr></thead><tbody><tr><td>3</td><td>0.1</td></tr><tr><td>2</td><td>0.07</td></tr><tr><td>1</td><td>0.04</td></tr><tr><td>0</td><td>0</td></tr></tbody></table> S (Magnification Factor) σ_{al} : Allowable Bearing Power of the Ground <table><thead><tr><th>Group</th><th>Usage</th><th>γ_d</th></tr></thead><tbody><tr><td>A)</td><td>Specially Important</td><td>1.3</td></tr><tr><td>B)</td><td>Ordinary</td><td>1.0</td></tr><tr><td>C)</td><td>Less important</td><td>0.8</td></tr></tbody></table> <table><thead><tr><th>Type of Structure</th><th>γ_e</th></tr></thead><tbody><tr><td>Type I) strong</td><td>1.0</td></tr><tr><td>II) medium</td><td>1.3</td></tr><tr><td>III) weak</td><td>1.8</td></tr></tbody></table>	Zone	Basic Coefficient C_o	3	0.1	2	0.07	1	0.04	0	0	Group	Usage	γ_d	A)	Specially Important	1.3	B)	Ordinary	1.0	C)	Less important	0.8	Type of Structure	γ_e	Type I) strong	1.0	II) medium	1.3	III) weak	1.8		Regulation on Instrmentation
Zone	Basic Coefficient C_o																																	
3	0.1																																	
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Country	Specifications	Seismic Coefficient, k			Increase in Allowable Stresses	Remarks	
Bulgaria	State Committee for Building and Architecture, Ministry of Construction, Bulgaria. Code for Buildings in Earthquake Region (1964).	$k = \psi\beta\eta k_c$ Basic Seismic Coef. k_c			50%		
		Soil Condition	Seismic Intensity				
			VII	VIII			IX
		1. Rock	—	—			0.05
		2. Semi-rock, coarse gravel, rigid clay	—	0.033			0.067
		3. medium	0.025	0.050			0.1
		4. soft	0.033	0.067			0.133
		η : Coef. of Mode shape (1.35 for first mode) β : dynamic coef. of one-degree-of-freedom system $\beta = 0.9/T$. $0.6 \leq \beta \leq 3$ ψ : coef. accounting damping capacity of structures $\psi = 1.0$ — ordinary structures, $\psi = 1.5$ — flexible structures with small damping.					
		Usage	Region of Seismic Intensity in MM Scale				
			VII	VIII			IX
		A. Especially Important	VIII	IX			X*
		B. Important	VII	VIII			IV
		C. Ordinary	VII	VII			VIII
		D. Less Important	not considered				
		* Seismic Force for X = 1.3 × (IX)					

Country	Specifications	Seismic Coefficient, k	Increase in Allowable Stresses	Remarks
CANADA	National Building Code of Canada, 1970.	$k = (1/4) R \cdot K \cdot C \cdot I \cdot F$ R = Seismic Regionalization Factor depending on Climate Data $K = 0.67 \sim 3.00$ (on structure) $C = 0.10$ for 1 and 2-storey building $C = 0.05 / \sqrt[3]{T} (C \leq 0.1)$ $I = 1.3$ (post disaster service, school) or $= 1.0$ (other) $F = 1.5$ (soils of low dynamic shear modulus) or $= 1.0$ (other soils)		

Country	Specifications	Seismic Coefficient, k (k in $V = kW$)	Increase in Allowable Stresses	Remarks																								
Chile	Institute Nacional De Incestigaciones Technologicas Normalizacion (INDITECNOR)	$k = K_1 K_2 C$ <table><thead><tr><th>Usage</th><th>K_1</th></tr></thead><tbody><tr><td>a) Specially Important</td><td>1.2</td></tr><tr><td>b) Ordinary</td><td>1.0</td></tr><tr><td>c) Less Important</td><td>0.8</td></tr></tbody></table> <table><thead><tr><th>Type of Structure</th><th>K_2</th></tr></thead><tbody><tr><td>d) weak</td><td>1.2</td></tr><tr><td>e) moderate</td><td>1.0</td></tr><tr><td>f) strong</td><td>0.8</td></tr></tbody></table> To Natural Period T : Fundamental Natural Period <table><thead><tr><th>Ground Condition</th><th>To (sec.)</th></tr></thead><tbody><tr><td>Rock, Dense gravel</td><td>0.2</td></tr><tr><td>Dense Soil</td><td>0.3</td></tr><tr><td>Soft</td><td>0.9</td></tr></tbody></table>	Usage	K_1	a) Specially Important	1.2	b) Ordinary	1.0	c) Less Important	0.8	Type of Structure	K_2	d) weak	1.2	e) moderate	1.0	f) strong	0.8	Ground Condition	To (sec.)	Rock, Dense gravel	0.2	Dense Soil	0.3	Soft	0.9		Regulations on Instrumentation, Dynamic Analysis
Usage	K_1																											
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Dense Soil	0.3																											
Soft	0.9																											

Country	Specifications	Seismic Coefficient, k				Increase in Allowable Stresses	Remarks
Cuba	NOCAE No. 1 STANDARDS For STRUCTURAL DESIGN.	1) Static Method (For Mercalli 8 zone)				a) Wood and structural or reinforcing steel: 50% b) Other metals, Concrete and masonry: 33% c) Soil: 33%	Regulation on Limit for Horizontal Displacement
		Group by Usage	Type of Structure	High Compressibility zone	Low Compressibility zone		
		A (Specially Important)	1 Strong	0.08	0.05		
			2 Medium	0.1	0.1		
			3 Weak	0.2	0.13		
		B (Ordinary)	1 Strong	0.06	0.04		
			2 Medium	0.08	0.08		
			3 Weak	0.15	0.10		
		C (Less Important)	1 Strong	0	0		
			2 Medium	0	0		
			3 Weak	0	0		
		2) Dynamic Method					
		$k = K_c \beta \cdot \eta$ $K_c = 0.05$ (For Mercalli 8 zone) β = dynamic factor For shear deformation structures: $0.6 < \beta = \frac{0.9}{T} \leq 3$ For flexure deformation structures: $1 < \beta = \frac{1.5}{T} \leq 5$ (of small damping) η : coef. varing with mode shape Dynamic Method $\geq (60\%) \times$ Static One					

Country	Specifications	Seismic Coefficient, k				Increase in Allowable Stresses	Remarks	
El Salvador	Regulations for Seismic Design, Republic of EL Salvador, C. A. 21, January, 1966.	$k = C \text{ or } CD$				A) Timber, Structural or Reinforcing Steel: 50% B) Other metals, concrete, masonry: 33% C) Soils : Study	Regulations on Dynamic Analysis, Instrumentation	
		<u>Values of C</u>						
		Group A)	Group B)		Group C)			
		1.3 × (Group B)	Type of Structure	Seismic Zone	Seismic Zone			0
			1	0.12	0.06			
			2	0.24	0.12			
			3	0.30	0.15			
		For grouping and type of structure see Mexican Regulation (p. B-17). Reduction Factor $D = \sqrt{C/X_e} \quad (0.6 \leq D \leq 1.0)$ X_e = displacement of center of gravity of body due to seismic coef. C (cm)						

Country	Specifications	Seismic Coefficient, k				Increase in Allowable Stresses	Remarks	
Greece	Royal Decree About Anti-Seismic Regulation for Building Construction, June, 1959.	Seismicity of areas	Soil Condition				Concrete Steel) : 20%	
			(a)	(b)	(c)	(d)		
		I Slight	0.04	0.06	0.08	*		
		II Moderate	0.06	0.08	0.12			
		III Strong	0.08	0.12	0.16			
		Soil Condition:						
		(a) good	} Definitions are defined.					
(b) moderate								
(c) poor								
(d) worst								
* Construction of permanent building must be avoided except if there is possible to confront eventual risk of land slide or settling.								

Country	Specifications	Seismic Coefficient, k				Increase in Allowable Stresses	Remarks
India	Indian Standard Criteria for Earthquake Resistant Design of Structures (1966).	Zone No.	Horizontal Seismic Coefficient, k			Permissible Increase	Vertical Coefficient = 1/2 (Horiz.) Regulations on Hydrodynamic Pressure, Earth pressure on Retaining Walls. Regulation on Details for the Design of Bridges.
			Soil Type I	Soil Type II	Soil Type III		
		VI	0.08	0.1	0.12	Soil I 50%	
		V	0.06	0.08	0.10	Soil II 30%	
		IV	0.05	0.06	0.08	Soil III 30%	
		III	0.04	0.05	0.06		
		II	0.02	0.03	0.04		
		I	0	0.01	0.02		
		0	0	0	0		
		Soil Type I : $\sigma_a \geq 45t/m^2$ Soil Type II : $\sigma_a \geq 20t/m^2$ Soil Type III : $\sigma_a \geq 10t/m^2$ ($N \geq 10$) (σ_a : allowable bearing power of the soil) Loose sands with $N < 10$ (N : blows by standard penetration test) may liquefy. In important project, need special investigations for these soils. k : consider only above scour depth, and for portions below the scour depth, $k = 0$. Seismic force shall be calculated on the basis of depth of scour caused by mean annual flood.					

Country	Specifications	Seismic Coefficient, k	Increase in Allowable Stresses	Remarks
Iran	Iranian Antiseismic Code for <u>Building</u> Construction	$k = K \cdot C$ $K = 1$ (Seismicity Coefficient) $C = 0.1$ (For Soil with normal bearing capacity less than 10 t/m^2) $C = 0.08$ (For Soil with normal bearing capacity more than 10 t/m^2) For public buildings : 25% increase. For buildings having 5 story or more structures high than 18m, C shall be determined by $C = \frac{0.025}{T} \geq 0.04$ T : fundamental period (sec.) $(C = \frac{0.05}{T} : \text{for chimney, towers, reserviors.})$	1/3	

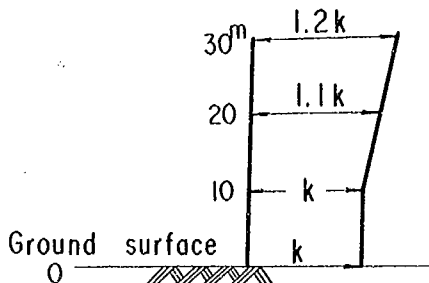
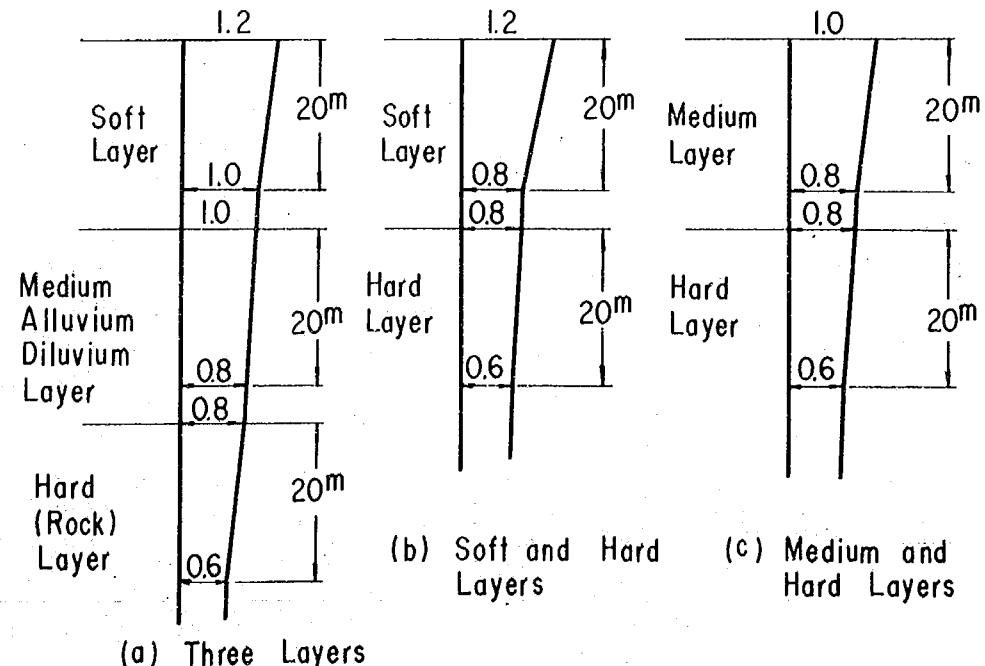
Country	Specifications	Seismic Coefficient, k		Increase in Allowable Stresses	Remarks	
Israel	Proposed Israel Standard	Seismicity Building and soil	Vaoue of k		1/3	Regulation on Seismic Coefficient for Structural Parts.
	REGION					
	A	B				
	Whole Buildings					
	1.1 Multi-Story Buildings founded on good soil ($\sigma_a \geq 15t/m^2$)	$\frac{0.1}{m + 4.5}$	$\frac{0.2}{m + 4.5}$			
	1.2 Multi-Story Buildings founded on bad soil ($\sigma_a < 15t/m^2$)	$\frac{0.2}{m + 4.5}$	$\frac{0.4}{m + 4.5}$			
	1.3 Various Structures such as reservoirs, connected building, etc., founded on good soil ($\sigma_a \geq 15t/m^2$)	0.02	0.04			
	1.4 Various Structures shown in 1.3, founded bad soil ($\sigma_a < 15t/m^2$)	0.04	0.08			
	1.5 Slender Structures (water towers, chimneys)	0.04	0.08			
	m : number of stories above one under consideration and assuming that the floor panels or other structural stiffness elements transmit the forces in their plane.					
	Region A : up to Intensity VII					
	Region B : up to Intensity IX					

Country	Specifications	Design Seismic Coefficient, k		Increase in Allowable Stresses	Remarks
Italy	Italian Standard Specifications for Buildings in Seismic Zones	k		mild steel	not simultaneous Horizontal and Vertical
		Zone a	Zone b	$f_s = 1,400 \text{ kg/cm}^2$	
		Basic $k = 0.1g$	$= 0.05g$	medium steel	
		$F_H = 0.1 (D + 1/3.L)$	$= 0.05 (D + 1/3.L)$	$t_s = 2,000 \text{ kg/cm}^2$	
		$F_V = 1.4 (D + 1/3.L)$	$= 1.25 (D + 1/3.L)$		
		$F_V > D + L$ D : Dead Load L : Live Load F_H : Horizontal Force F_V : Vertical Force			

Country	JAPAN	CODE	Japan Road Association, "Specifications for Earthquake-Resistant Design of Highway Bridges (1971)," and Related Specifications.																											
DESIGN SEISMIC COEFFICIENTS	(1) Conventional Seismic Coefficient Method (for relatively rigid structures, $T < 0.5$ sec) $k_h = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot k_o \geq 0.1$ $k_o = 0.2$ Table 1 ν_1 (Seismicity) <table><tr><th>Zone</th><th>ν_1</th></tr><tr><td>A) Active</td><td>1.0</td></tr><tr><td>B) Moderate</td><td>0.85</td></tr><tr><td>C) Mild</td><td>0.7</td></tr></table> Table 2 ν_2 (Soil) <table><tr><th>Soil Conditions</th><th>ν_2</th></tr><tr><td>1. Rock</td><td>0.9</td></tr><tr><td>2. Diluvium</td><td>1.0</td></tr><tr><td>3. Alluvium</td><td>1.1</td></tr><tr><td>4. Soft</td><td>1.2</td></tr></table> Table 3 ν_3 (Importance) <table><tr><th>Importance</th><th>ν_3 *</th><th>Remarks</th></tr><tr><td>Important</td><td>1.0</td><td>General</td></tr><tr><td>Less important</td><td>0.8</td><td></td></tr></table> * When exceptionally important, ν_3 can be up to 1.25.			Zone	ν_1	A) Active	1.0	B) Moderate	0.85	C) Mild	0.7	Soil Conditions	ν_2	1. Rock	0.9	2. Diluvium	1.0	3. Alluvium	1.1	4. Soft	1.2	Importance	ν_3 *	Remarks	Important	1.0	General	Less important	0.8	
	Zone	ν_1																												
A) Active	1.0																													
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Importance	ν_3 *	Remarks																												
Important	1.0	General																												
Less important	0.8																													
(2) Modified Seismic Coefficient Method Considering Structural Response (for relatively flexible structures such as ones with high-rise piers higher than 25m or ones with fundamental natural periods longer than 0.5 sec.) $k_{hm} = \beta k_h \geq 0.05$ (k_h: CSCM in (1)) β is shown in Fig. 1 ($0.5 \leq \beta \leq 1.25$), $\beta = 1.0$ for $T < 0.5$ sec. Magnification Factor (β) Soil 4 Soil 3 Soil 2 Soil 1 Fig. 1 The Value of Factor β Notes for both (1) and (2) : 1) Vertical Coefficient $k_v = 0$ except for special parts like supports. 2) $k_h = 0$ for structural parts under the ground (except underground structures).																														
Increases in Allowable Stresses	Concrete and Reinforcements in RC Structures : 50%, Prestressed Concrete : 65% for Compression Zone Steel : 70% for Superstructures and 50% for Substructures, Foundation Soils : 50%																													
Remarks	Provisions on the followings are specified. 1) Earth and water pressures during earthquakes, 2) Soils which have no resistance during earthquakes (due to liquefaction of sands) 3) Special attention to prevent fall of superstructures from substructures.																													

COUNTRY	JAPAN	CODE	Specifications for Earthquake Resistant Design in Tokyo Expressways Public Corporation. (July, 1967)								
DESIGN SEISMIC COEFFICIENTS	Table 1. Seismic Coefficients (See Note (a) below)					Table 2. Classification of Soil Condition (See Note (b) below)					
	Classification of Soil Conditions		1	2	3	4	Property of Alluvial Layer or Loam Layer	Gravel	Sand, Clay and Loam (N ≥ 5)	Soft Soil	
	Horizontal Coefficients, k_h	Above ground	0.2	0.24	0.27	0.3				N=2-5	N < 2
		Underground	0.15	0.18	0.20	0.23					
	Vertical Coefficients, k_v		0.1	0.1	0.1	0.1					
	Notes:										
	(a) For those structures which will be constructed in very soft soils, and whose natural periods may coincide with predominant periods of the ground, special attention should be drawn by dynamic analysis method.										
	(b) Values in the parentheses in Table 2 are for those structures which have rigid foundations, such as caisson or well foundations with depth to diameter (or shorter length of bases) ratios of 3 or less. N indicates number of blow per 30 cm by the standard penetration test.										
	(c) In cases where depth of alluvium layer or loam layer is between 0 and 3 m, and a firm layer is under this thin layer, the surface alluvium or loam layer should be removed. In the cases Soil Condition will be classified into 1.										
	Combination of Loads and Allowable Stresses	Type of Structures		Combination of Loads		Increases in Allowable Stresses (%)		Allowable Stresses of Concrete		Notes:	
Foundations or Retaining Walls		Reinforced Concrete Structures	Principal Loads + Seismic Loads		50		$\frac{\sigma_{28}}{3}$ or 100 kg/cm ²		(1) Principal Loads = Dead Loads+ Earth Pressures+Water Pressures+ Buoyancy+Uplift,		
		Reinforced Concrete Structures	Principal Loads + Temperature + Humidity + Seismic Loads		65				(2) Foundations imply spread footings, pile foundations, and caisson foundations,		
		Steel Structures	Principal Loads + Seismic Loads		50				(3) Substructures mean the part of substructures above the top of foundations.		
Superstructures or Substructures		Reinforced Concrete Structures	Dead Loads + Seismic Loads		50		$\frac{\sigma_{28}}{3}$				
		Reinforced Concrete Structures	Dead Loads + Temperature + Humidity + Seismic Loads		65						
		Steel Structures	Dead Loads + Seismic Loads		70						

COUNTRY	JAPAN	CODE	JSCE : "Specifications for Earthquake Resistant Design of the Honshu-Shikoku Suspension Bridges (1967)" July, 1967 JSCE : "Revision of Specifications (1967)" June, 1971.															
DESIGN SEISMIC COEFFICIENTS	(1) Seismic Coefficient Method (out of use) Horizontal : $k_h = S \cdot C_o$; Vertical : $k_v = (1/2) k_h$ ($C_o = 0.18$) Table 1 Value of S <table><tr><td>Soil Condition</td><td>S</td></tr><tr><td>Rock (Tertiary or older)</td><td>1.0</td></tr><tr><td>Diluvium</td><td>1.1</td></tr><tr><td>Alluvium</td><td>1.25</td></tr></table> Increase Seismic Coefficient for higher part (Fig. 1) Fig. 1 Seismic Coefficient for Higher Part of Structures			Soil Condition	S	Rock (Tertiary or older)	1.0	Diluvium	1.1	Alluvium	1.25							
	Soil Condition	S																
Rock (Tertiary or older)	1.0																	
Diluvium	1.1																	
Alluvium	1.25																	
(2) Modified Seismic Coefficient Method Considering Structural Response (employed) Design Seismic Coefficient, $k = m_1 \cdot m_2 \cdot C_o$ ($C_o = 0.18$ in Horizontal, 0.09 in Vertical). m_1 = Magnification factor for simple one-degree-of-freedom system (depends on type of structure, periods, damping, non-linearity, etc.) (Fig. 2) m_2 = A factor used when actual structures are considered to be simple systems to which simplified distributed seismic loads are applied (Fig. 3) Fig. 2 Magnification Factor m_1 <table><tr><th>Direction of Seismic Motion Type of Structure</th><th>Longitudinal</th><th>Transverse</th><th>Vertical</th></tr><tr><td>Substructure</td><td></td><td>Same as Longitudinal $m_2 = 1.0$</td><td>not considered</td></tr><tr><td>Main Tower</td><td></td><td></td><td>not considered</td></tr><tr><td>Suspended Structure, Cable</td><td> Vertical Response 1st Asymm. Mode 2nd Asymm. Mode </td><td></td><td></td></tr></table> Fig. 3 Distribution of Design Seismic Loads and Value of m_2			Direction of Seismic Motion Type of Structure	Longitudinal	Transverse	Vertical	Substructure		Same as Longitudinal $m_2 = 1.0$	not considered	Main Tower			not considered	Suspended Structure, Cable	Vertical Response 1st Asymm. Mode 2nd Asymm. Mode		
Direction of Seismic Motion Type of Structure	Longitudinal	Transverse	Vertical															
Substructure		Same as Longitudinal $m_2 = 1.0$	not considered															
Main Tower			not considered															
Suspended Structure, Cable	Vertical Response 1st Asymm. Mode 2nd Asymm. Mode																	
REMARKS	Provisions on the following subjects are specified: Dynamic Analysis, Earth Pressures, Hydrodynamic Pressures, Safety Factor, Other Consideration.																	

COUNTRY	JAPAN	CODE	JSCE (Commissioned from the Japanese National Railways): "Report on the Aseismic Design of Sub-Structures of Railway Bridges (1968)"						
DESIGN SEISMIC COEFFICIENTS	Table 1. Design Horizontal Seismic Coefficients. $k_h = m_s \cdot m_I \cdot k_o$							Definition of Soil Condition	
	Zone			A ($k_o = 0.2$)			B ($k_o = 0.15$)		
	Soil Condition			Soft	Medium	Hard	Soft	Medium	Hard
				$m_s = 1.2$	$m_s = 1.0$	$m_s = 0.8$	$m_s = 1.2$	$m_s = 1.0$	$m_s = 0.8$
	Degree of Importance of Structure	Important	$m_I = 1.1$	0.25	0.2	0.2	0.2	0.15	0.15
		Ordinary	$m_I = 1.0$	0.25	0.2	0.15	0.2	0.15	0.1
		Less Important	$m_I = 0.9$	0.2	0.2	0.15	0.15	0.15	0.1
	k_o : Basic Seismic Coefficient								
	Notes:								
	(1) Seismic Coefficients for Higher Portion of Structures (Fig. 1): For the part of structures higher than 10 m above ground surface, the design seismic coefficient shall be obtained by increasing by 1% per an increase of 1 m in height.								
(2) Seismic Coefficients below Ground Surface (Fig. 2): Seismic coefficients for underground portion of structures, underground structures, caisson foundations, etc. are obtained by multiplying m_s (shown in Fig. 2 for various ground conditions and depth) by $m_I k_o$ given in Table 1.									
(3) Vertical Seismic Coefficients, k_v shall be one half of the horizontal, when needed.									
(4) These specification shall apply to bridges and retaining walls.									
									
Fig. 1 Seismic Coefficient for Higher Portion									
									
Fig. 2 Values of m_s for Underground									

Country	Specifications	Seismic Coefficient, k (k in $V = kW$)				Increase in Allowable Stresses	Remarks	
Mexico	Provisions for Earthquake Resistant Design in the Federal District, Mexico. February, 1966 (For Building)	Group A	Group B			Group C	Timer, structural or reinforcing steel: 50% Other metals, Concrete, masonry: 33% Soils: according to the chapter on foundations.	Regulations on Repairing, Instrumentation, Dynamic Analysis, Maximum relative displacements.
			Type of Structure	Zone 1	Zone 2			
		1.3 x (Group B.)	1	0.06	0.04	0		
			2	0.08	0.08	0		
			3	0.15	0.1	0		
		Group A : specially important Group B : important Group C : less important Type 1 : strong structure Type 2 : moderate structure Type 3 : weak structure Zone 1 : zone of high compressibility Zone 2 : zone of low compressibility						

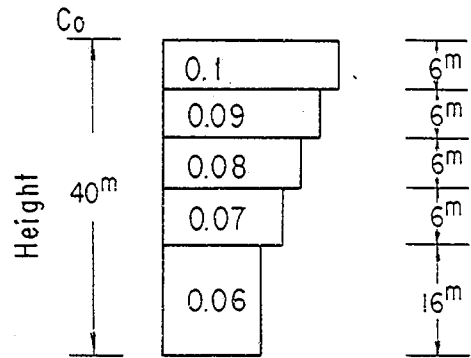
Country	Specifications	Seismic Coefficient, k	Increase in Allowable Stresses	Remarks
New Zealand	New Zealand Standard Mode Building By law (NZSS1900), Chapter 8 Basic Design Loads, Earthquake Provisions. (1965)	(a) Public Buildings (b) Others		

Country	Specifications	Seismic Coefficient, k					Increase in Allowable Stresses	Remarks
Peru	Preuvian Standards for Antiseismic Design. September 30, 1968.	$k = UKC$ U : Factor depending on the use of building and on the region.					Elastic Method:	Vertical seismic forces: It will be necessary to verify the effects of the vertical seismic force in the following cases: A) In cantilever element and heavily loaded tie rods, etc. B) Elements of great lengths, especially cantilever, bracings, and bridges. Regulations on dynamic analysis for non-conventional structures, instrumentation.
		Region Usage	1	2	3	Remarks	33% for (steel concrete	
		A) Rural	Special recommendations				Ultimate-Resistant Method: load factor 1.33	
		B) Ordinary	1	0.8	0.6			
		C) Public	1.2	1.0	0.7			
D) Special	Special Consideration Bigger than C)							
		$K = 1.33$ $= 1.0$ $= 0.80$ $= 0.67$ } modifying factor used to take into account structural response to aseismic action, according to its energy absorption capacity $C = 0.05/\sqrt[3]{T}$ Soil Classification I : Hard Soil—rocks, rocky soils, dense soils. II : Soft Soil—soil with low bearing capacity, high water content, and high water table. III: Undesirable Soil—Very soft soil. Notes: 1) For Soils II and III, foundations should be designed in such a way that effects of differential settlement and disorderly vibration will be minimarized. 2) On Soils III, type D should not be built, and Seismic forces should be increased for type A to C. 3) In urban planning, soils III should be used as green areas, recreation areas, and avenues.						

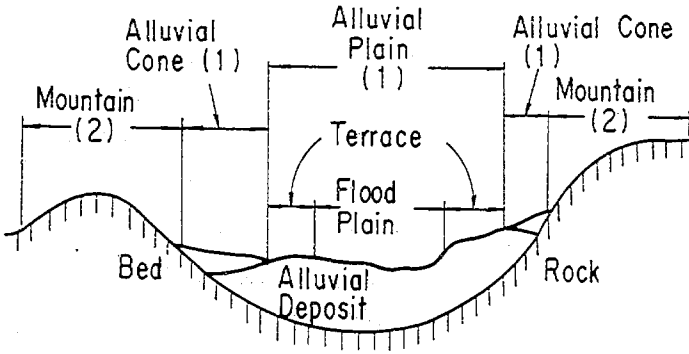
Country	Specifications	Seismic Coefficient, k				Increase in Allowable Stresses	Remarks
Philippines	Draft of the National Building Code of Philippines	$k = ZCK$					Regulation on Instrumentation
		Values of Z					
		Found. Zone	A	B	C		
		1	0.8	1.0	1.2		
		2	0.6	0.8	1.0		
		3	0.4	0.6	0.8		
$C = \frac{2.05}{\sqrt[3]{T}} \leq 0.1$ C = 0.1—one and two-story buildings K : Horizontal force factor (0.67 ~ 3) Found : Foundation Conditions							

Country	Seismic Coefficient, k						Allowable Stresses	Remarks
Portugal			Zone A		Zone B		Zone C	<u>Limit Design</u> mild steel : 2400 kg/cm^2 other steel : $0.8 \times$ (ultimate stress) Concrete : $2/3$ (rupture stress) = 120 kg/cm^2 (minimum rupture stress : 180 kg/cm^2) <u>Elastic Design</u> mild steel : 2400 kg/cm^2 (tensile, bending, or non-buckling compressive). For other type of loads: $1.7 \times$ (permissible stress for ordinary loads) Concrete : $2 \times$ (permissible stress for ordinary loads).
			(1)	(2)	(1)	(2)		
	Construction as a Whole	Construction with a reserve of strength due to non-structural bracing members	0.1	0.15	0.05	0.075	0	
		Construction without a reserve of strength due to non-structural bracing members	0.15	0.2	0.075	0.1	0	
Specifications	Building Members	Walls and Similar Members	0.20		0.10		0	
Portuguese Codes on Earthquake Resistant Structures		Balconies, chimneys and other members projecting from the outer walls and from the roofing	0.30		0.15		0	
	(1) : Usual cases of foundation ground. (2) : Foundation ground with particularly unfavorable seismic characteristics, like the case with mud, soft clay, silt and recent embankments, for layers at least about ten meters thick, even when the foundation structure goes through these layers and rests on harder ones.							

Country	Specifications	Seismic Coefficient, k	Increase in Allowable Stresses	Remarks																
Rumania	Earthquake Regulations	$k = K_s \beta \epsilon \psi$ $k \geq 0.02$ <table><tr><th>Design seismic degree</th><th>K_s</th></tr><tr><td>7</td><td>0.025</td></tr><tr><td>8</td><td>0.05</td></tr><tr><td>9</td><td>0.1</td></tr></table> β : Dynamic factor (σ_a: allowable bearing power) <table><tr><th>Soil</th><th>β</th></tr><tr><td>a) $\sigma_a \geq 20t/m^2$</td><td>$0.6 \leq \beta = \frac{0.9}{T} \leq 3$</td></tr><tr><td>b) $\sigma_a < 20t/m^2$</td><td>$0.75 \leq \beta = \frac{1.125}{T} \leq 3$</td></tr><tr><td>c) soft</td><td>$0.9 \leq \beta = \frac{1.35}{T} \leq 3$</td></tr></table> β for a) ϵ :: mode participation factor $\psi = 1.0$ = usual building = 1.2 = frame structure building of RC, building with hinged roofs or RC strut, = 1.5 = flexible structure such as chimneys, water tanks, towers, etc.	Design seismic degree	K_s	7	0.025	8	0.05	9	0.1	Soil	β	a) $\sigma_a \geq 20t/m^2$	$0.6 \leq \beta = \frac{0.9}{T} \leq 3$	b) $\sigma_a < 20t/m^2$	$0.75 \leq \beta = \frac{1.125}{T} \leq 3$	c) soft	$0.9 \leq \beta = \frac{1.35}{T} \leq 3$		
Design seismic degree	K_s																			
7	0.025																			
8	0.05																			
9	0.1																			
Soil	β																			
a) $\sigma_a \geq 20t/m^2$	$0.6 \leq \beta = \frac{0.9}{T} \leq 3$																			
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c) soft	$0.9 \leq \beta = \frac{1.35}{T} \leq 3$																			

Country	Specifications	Seismic Coefficient, k	Increase in Allowable Stresses	Remarks																																
Turkey	Regulation for Buildings in Disaster Areas, Highways	$k = C_0 \cdot n_1 \cdot n_2$  Above 40^m C_0 will be increase by 0.01 step per 3.0 meters <table><tr><th colspan="2"></th><th colspan="2">n_1</th></tr><tr><th colspan="2">Soil</th><th>Steel</th><th>RC</th></tr><tr><td>I</td><td>Hard rock</td><td>0.6</td><td>0.8</td></tr><tr><td>II</td><td>Compact soils</td><td>0.8</td><td>0.9</td></tr><tr><td>III</td><td>Soft</td><td>1.0</td><td>1.0</td></tr></table> <table><tr><th colspan="2">Earthquake Region</th><th>n_2</th></tr><tr><td>1st</td><td>Degree</td><td>1.0</td></tr><tr><td>2nd</td><td>Degree</td><td>0.6</td></tr><tr><td>3rd</td><td>Degree</td><td>0.6</td></tr></table>			n_1		Soil		Steel	RC	I	Hard rock	0.6	0.8	II	Compact soils	0.8	0.9	III	Soft	1.0	1.0	Earthquake Region		n_2	1st	Degree	1.0	2nd	Degree	0.6	3rd	Degree	0.6	50%	not considered for embankments of highways. Regulation on earth-pressure.
		n_1																																		
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3rd	Degree	0.6																																		

Country		Horizontal Seismic Coefficient in $V = kW$ (V = Horizontal Force, W = Total Weight)		Increase in Allowable Stresses	Remarks
U.S.A.		Structures on spread footing on soil of allowable bearing pressure $\geq 37t/m^2$	$k = 0.02$	1/3 (Dead Load + primary forces except live load)	
Specifications	(A)	Structures on spread footing on soil of allowable bearing pressure $\leq 37t/m^2$	$k = 0.04$		
		Structures on piles	$k = 0.06$		
(A) Standard Specifications for Highway Bridges, AASHO (1965). (Simple quasistatic formula)		$k = KC$ K : Horizontal Force Factor $\begin{cases} = 1.33 \text{ for systems where the lateral forces are restricted by diaphragm} \\ = 1.00 \text{ for columns or piers which have a height-length ratio greater than 2.5} \\ = 0.67 \text{ where lateral loads are resisted by rigid frame action} \end{cases}$			Based on "Recommended Force Requirements," SEAOC (1959)
(B) State of California Division of Highways (Modified seismic coefficient method)	(B)	$C = 0.05/T^{1/3}$ T : fundamental natural period (sec.) $= 0.32 \left(\frac{D}{P}\right)^{1/2}$ D : Dead load of vibrating part (lb) P : force to produce one inch of lateral deflection (lb/in.) $k \geq 0.02$			

Country	Specifications	Seismic Coefficient, k				Increase in Allowable Stresses	Remarks	
Venezuela	Venezuela Building Regulations	k for zone 3				33%		
		Type of Structure	Alluvial Deposit (1)		Bedrock (2)			
			Usage of Building		Usage of Building			
			Group 1	Group 2	Group 1			Group 2
		I	0.075	0.06	0.06			0.045
		II	0.09	0.07	0.11			0.085
		III	0.15	0.12	0.13			0.10
		Group 1 : Specially Important Group 2 : Important Group 3 : Less Important ($k = 0$) k for zone 2 : $1/2 \times (\text{zone 3})$ k for zone 3 : $1/4 \times (\text{zone 3})$ k for zone 0 : 0 I : bending frame II : shear, axial force III : one column, one line of resisting						
		<u>Soil classification</u>						
								

APPENDIX C

**SPECIFICATIONS FOR EARTHQUAKE-RESISTANT DESIGN
OF HIGHWAY BRIDGES (JANUARY 1971)**

Tentative English Version

JAPAN ROAD ASSOCIATION

FOREWORD

The earthquake-resistant design of highway bridges is currently being conducted in accordance with the provisions of Section 13 (Section 2.9 in the English version) of the Specifications for Design of Steel Highway Bridges. Since the provisions concerning earthquake-resistant design are not very comprehensive as stated in the preface of the Specifications, the design is in many respects dependent on the designer's interpretation. Accordingly the results of the various designs were not always consistent. To alleviate this problem it was necessary to revise the Section 13 of the Specifications and to draw up new detailed specifications exclusively for earthquake resistant design of highway bridges.

In response to the need for revisions, the Japan Road Association established the Earthquake Resistant Design Subcommittee under the Highway Bridge Committee in 1966. Between 1966 and 1968 a preliminary draft was prepared. The task was taken over by the Earthquake-Resistant Design Branch-Committee of the Specifications Subcommittee in 1968, when the Highway Bridge Committee was reorganized.

The Subcommittee and the Branch-Committee have seriously attempted to draw up the revision through surveys of the available research work, discussions with specialists from universities and other organizations, and trial calculations and bridge designs at several stages in accordance with the interim drafts, and finally completed the revision of the Specifications in 1971.

The new Specifications are completed basically in accordance with the provisions of Section 13 of the Specifications for Design of Steel Highway Bridges, and also by extensively referring to the results of recent investigations on earthquake engineering. Therefore the design methodology of Section 13 of the current Specifications was generally traced in establishing the new Specifications.

In determining the design seismic coefficient the importance factor is newly considered in addition to the traditional factors such as zone factor and ground condition factor. Moreover, for bridges which have longer natural periods the factor on the natural periods of the structures are considered.

Although the design horizontal seismic coefficient used to be 0.1 to 0.35 in the Section 13 of the current Specifications, the value is reduced to 0.1 to 0.3 and over 0.3 only for rare occasions in the new Specifications. This is based on the results of recent investigations that it seems more essential to pay attention to soil conditions rather than to the magnitude of seismic coefficients in design of bridges against seismic forces.

Since design seismic forces in the new Specifications were determined by referring to the seismic risk map expected for 75 years presented by Dr. Kawasumi and by reflecting the results of a recent study on the meteorological data since the Kanto Earthquake in 1923, seismic forces considered in the new Specifications would cover the severest earthquakes experienced in individual regions in Japan.

The seismic forces would also correspond to those due to the greatest earthquakes in the seismological point of view.

In the new Specifications the following aspects differ from the previous method.

- (1) The design seismic coefficients of a bridge are determined systematically dependent on geographical location of the bridge site, the ground conditions at each substructure site, and the importance of the bridge.
- (2) The vertical design seismic coefficients are generally neglected.
- (3) The design seismic coefficients for structural parts, soils and water below the ground surface are neglected.
- (4) Special attention is paid to very soft soil layers and soil layers vulnerable to liquefaction during earthquakes. The bearing capacities of these layers are neglected in the design in order to assure high earthquake-resistance for those structures which are constructed in these layers.
- (5) The design seismic coefficients for bridges with highrise piers are determined in accordance with the modified seismic coefficient method considering structural response, instead of the conventional way in which the design seismic coefficients increase with the height of the piers.

(6) Special attention is also paid to the design of structural details in view of earthquake damage previously experienced to bridge structures. To this end provisions are specified for bearing supports and devices for preventing superstructures from falling.

Although the Specifications were developed through extensive studies and discussions of research reports on earthquake engineering from throughout the world as well as Japan, some unexpected problems may arise when they are applied to actual structures. Because of this and the fact that technological improvements take place very rapidly, another revision will be required in the future. Everyone is urged to indicate any insufficient points or to make any suggestions for the present Specifications, so that they may be improved in subsequent revisions.

The present publication is limited to the provisions which are considered to be sufficient for the design of bridges. Comments which describe the basis and the application methods of the provisions will be issued immediately after their completion. They will be indispensable when it is necessary to revise the present Specifications again or when any parts of the provisions are found to be uncertain in their application.

Dr. Hiroshi Kawasumi, Dr. Tsutomu Terashima, and Mr. Tsutomu Tomita participated in the discussions on special matters as special members of the Committee. Their cooperation is greatly appreciated.

January, 1971

Earthquake Resistant Design Branch-Committee
Specifications Subcommittee
Highway Bridge Committee
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Members with * are also secretaries.

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* References are provided exclusively in the English version for the convenience of the reader.

CHAPTER 1 GENERAL

1.1 Scope

The provisions in the Specifications apply to earthquake resistant design of highway bridges with spans not longer than 200 meters, to be built on expressways, national highways, prefectural highways and principal municipal highways.

In regard to matters which are not specified herein, the following Specifications shall be conformed to, in accordance with the type of the structure considered.

Specifications for Design of Steel Highway Bridges,	Japan Road Association
Specifications for Design of Welded Highway Bridges,	Japan Road Association
Specifications for Design and Construction of Composite Beams for Steel Highway Bridges,	Japan Road Association
Specifications for Design of Reinforced Concrete Highway Bridges,	Japan Road Association
Specifications for Design and Construction of Friction Type Joints in Steel Highway Bridges Using High Strength Bolts,	Japan Road Association
Specifications for Design of Substructures of Highway Bridges,	Japan Road Association
Specifications for Prestressed Concrete Highway Bridges,	Japan Road Association

1.2 Definitions of Terms

The following definitions apply only to the provisions of these Specifications.

- (1) Earthquake: A phenomenon with propagation of vibration due to a sudden naturally occurring movement at a certain portion inside the earth.
- (2) Earthquake Ground Motion: Vibration of ground during earthquakes.
- (3) Design Seismic Coefficient: A coefficient indicating the magnitude of acceleration to be considered for earthquake-resistant design of structures, and expressed as a fraction of the acceleration of gravity.
- (4) Seismic Coefficient Method: A method for earthquake-resistant design in which seismic forces are assumed to act on structures as static forces.
- (5) Modified Seismic Coefficient Method Considering Structural Response: A method for earthquake resistant design which is developed through modification of the seismic coefficient method by considering characteristics of earthquake ground motions, dynamic properties of structures, etc. for those structures with long fundamental periods of vibration.
- (6) Modified Design Seismic Coefficient Considering Structural Response: A design seismic coefficient used in the modified seismic coefficient method considering structural response in which the design seismic coefficient is dependent on the fundamental period of the structure.
- (7) Standard Horizontal Design Seismic Coefficient: The Horizontal design seismic coefficient which applies to a bridge which is located in a zone where severe earthquakes have frequently occurred or where severe earthquakes have a high potential of occurrence, and which is constructed in soil layers with ground conditions corresponding to group 2 in Table 4.5.
- (8) Seismic Zone Factor: A factor to decrease the horizontal design seismic coefficient in accordance with the geographical location of the structural site. The factor has the value of 1.0 for those zones where severe earthquakes have frequently occurred or where severe earthquakes have a high potential of occurrence.
- (9) Ground Condition Factor: A factor to modify the horizontal design seismic coefficient depending upon the ground conditions of the structural site. The factor is intended to unify the margin of

safety against earthquake disturbances among structures constructed in various ground conditions, in consideration of damage previously experienced.

- (10) Importance Factor: A factor to modify the horizontal design seismic coefficient depending on the importance of the structure.
- (11) Inertia Force: Product of the weight of the structural body and the design seismic coefficient.
- (12) Seismic Force: Any forces such as inertia forces, earth pressures, etc. to which structures are subjected during earthquakes.
- (13) Coefficient of Subgrade Reaction: A coefficient given by the following formula, which is also shown in Section 3.1 of the Volume of Spread Foundations, the Specifications for Design of Substructures of Highway Bridges.

$$K = \frac{p}{\delta}$$

where

- K: coefficient of subgrade reaction in kg/cm³
- p: loading pressures in kg/cm²
- δ: displacement in cm

- (14) Earthquake Response Analysis: An analysis of the dynamic behavior of structures during earthquakes.

CHAPTER 2 BASIC PRINCIPLES FOR EARTHQUAKE-RESISTANT DESIGN

- (1) The earthquake-resistant design for a highway bridge shall provide sufficient stability against seismic disturbances for the structure as a unit and also for all parts thereof, including superstructures, substructures, and surrounding soils, considering topographical and geological conditions at the site.
- (2) The seismic coefficient method basically shall apply to the earthquake-resistant design of relatively rigid structures.
- (3) The modified seismic coefficient method considering structural response shall apply to the earthquake resistant design of relatively flexible structures which are of long fundamental periods of vibration, such as bridges with highrise piers.

The earthquake response analysis shall also be adopted for those structures, for which detailed investigations are required.

- (4) Particular attention shall be paid to preventing the fall of superstructures from substructures due to the movements during earthquakes.

When the design allowing for partial failures of the whole system or local failures of certain structural members is required on the basis of economical considerations, special attention to preventing the fall of superstructures shall be paid.

CHAPTER 3 LOADS AND CONDITIONS IN EARTHQUAKE-RESISTANT DESIGN

3.1 General

- (1) The following loads shall be taken into account in earthquake-resistant design. The appropriate loads shall be selected from this list on the basis of the location and the type of the structure.

- 1. Dead Loads
- 2. Earth Pressures
- 3. Hydraulic Pressures
- 4. Buoyancy or Uplifts
- 5. Effects of Temperature Change

6. Effects of Shrinkage due to Humidity in Concrete Structures (Including Creep Effects)
7. Seismic Effects
8. Effects of Friction at Bearing Supports
9. Effects of Consolidation and Settlement of Ground
10. Effects of Movements of Supports
11. Other Loads

(2) Combination of Loads

Design conditions shall be determined considering the following four cases. The combination of various loads listed above shall be decided in accordance with the provisions in individual Specifications according to the type of the bridge considered.

- Case 1 Maximum stresses will be expected in the members
 Case 2 Maximum reactions will be expected in the foundation soils
 Case 3 Maximum displacements will be expected at the supports
 Case 4 The structure will become critical conditions due to overturning, sliding, etc.

3.2 Seismic Effects

The following seismic forces shall be taken into account to determine seismic effects in design of a bridge structure.

1. Inertia forces due to the dead weight of the structure: The magnitude of the inertia forces shall be the product of the weight of the structure and the design seismic coefficient. The design seismic coefficient shall be obtained in accordance with the provisions in Chapter 4 "Design Seismic Coefficient."
2. Inertia forces due to the superimposed weight: The magnitude of the inertia forces shall be the product of the superimposed weight and the design seismic coefficient. The design seismic coefficient also shall be obtained in accordance with the provisions in Chapter 4 "Design Seismic Coefficient."
3. Earth pressures during earthquakes.
4. Hydrodynamic pressures during earthquakes.

3.3 Inertia Forces

- (1) Both for the seismic coefficient method and the modified seismic coefficient method, bridge substructures shall be subjected to the inertia forces of superstructures during earthquakes as follows (refer to Fig. 3.1):

- (a) Horizontal seismic force acting on A_L (left abutment) shall be

$$H_{AL} = R_{AL} \cdot f_{AL} \text{-----} (3.1)$$

$$\text{where } H_{AL} \leq \frac{1}{2} k_h W_A \text{-----} (3.2)$$

In other words,

$$H_{AL} = \text{smallest of } \begin{cases} R_{AL} \cdot f_{AL} \\ \text{or} \\ \frac{1}{2} k_h W_A \end{cases}$$

- (b) Horizontal seismic force acting on P_1 (Pier 1) shall be

$$H_{AR} + H_{BL} = \text{largest of } \begin{cases} k_h W_A \text{ (} H_{BL} = 0 \text{ in this case)} \text{-----} (3.3) \\ \text{or} \\ \frac{1}{2} k_h W_A + R_{BL} \cdot f_{BL} \text{-----} (3.4) \end{cases}$$

In equation (3.4), $R_{BL} \cdot f_{BL}$ shall satisfy

$$R_{BL} \cdot f_{BL} \leq \frac{1}{2} k_h W_B \text{-----} (3.5)$$

(c) Horizontal seismic force acting on A_R (right abutment) shall be

$$H_{BR} = k_h W_B \text{-----} (3.6)$$

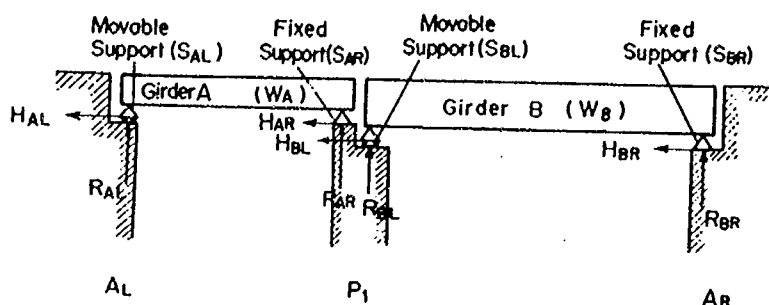


Fig. 3.1 Inertia Forces of Superstructures

where

- f_{AL} : Static friction coefficient at movable support A_L ,
- f_{BL} : Static friction coefficient at movable support B_L ,
- H_{AL} : Horizontal seismic force (inertia force or friction force) acting on A_L due to Girder A,
- H_{AR} : Horizontal seismic force (inertia force) acting on P_1 due to Girder A,
- H_{BL} : Horizontal seismic force (inertia force or friction force) acting on P_1 due to Girder B,
- H_{BR} : Horizontal seismic force (inertia force) acting on A_R due to Girder B,
- k_h : Horizontal design seismic coefficient (in accordance with the provisions in Chapter 4. "Design Seismic Coefficient": k_h in equation (4.1) for the seismic coefficient method, or k_{hm} in equation (4.2) for the modified seismic coefficient method, shall be employed),
- R_{AL} : Reaction at A_L due to W_A ,
- R_{AR} : Reaction at P_1 due to W_A ,
- R_{BL} : Reaction at P_1 due to W_B ,
- R_{BR} : Reaction at A_R due to W_B ,
- W_A : Dead weight of Girder A, and
- W_B : Dead weight of Girder B.

- (2) Inertia forces shall be assumed to act at the center of gravity of the structure. In designing the substructure, inertia forces exerted from superstructures may be assumed to act at the level of the base of the supports in the longitudinal direction to the bridge axis, and at the level of the center of gravity of the superstructures in the transverse direction.

In the transverse direction the level of the center of gravity of the superstructures may generally be taken as the lower level of the floor slab.

- (3) Inertia forces shall be assumed to come from two horizontal directions: longitudinal and transverse to the bridge axis, or parallel and perpendicular to the principal axis of any structural elements of the bridge.

3.4 Earth Pressures during Earthquakes

Earth pressures during earthquakes shall be determined in accordance with provisions in the Specifications for Design of Substructures of Highway Bridges (See Reference 1 at the end of the English

Version). The horizontal seismic coefficient in the calculation of earth pressures shall be in accordance with the provisions in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method."

3.5 Hydrodynamic Pressures during Earthquakes

Hydrodynamic pressures during earthquakes shall be determined by the following formulas. The pressures shall be assumed to act in the same direction as that of the inertia forces given by the provisions in Section 3.3 "Inertia Forces."

(1) Hydrodynamic Pressures on Walls

Hydrodynamic pressures acting on one side of a wall-type structure shall be determined as follows (refer to Fig. 3.2):

$$P = \frac{7}{12} k_h W_o b h^2 \text{-----} (3.7)$$

$$h_g = \frac{1}{2} h \text{-----} (3.8)$$

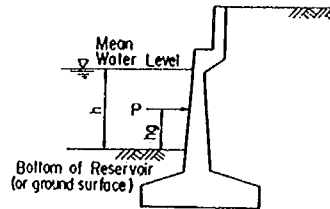


Fig. 3.2 Hydrodynamic Pressures on Wall Structures

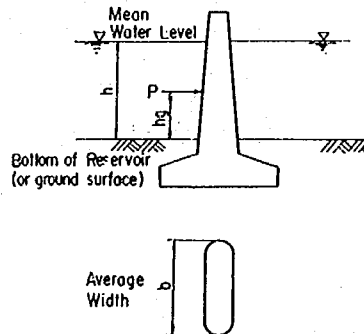


Fig. 3.3 Hydrodynamic Pressures on Column Structures

where

- b : Width of the wall in meters in the perpendicular direction to that of the pressure,
- h : Depth of water in meters,
- h_g : Height of the total hydrodynamic pressure in meters above the bottom of the water,

k_h : Horizontal design seismic coefficient given by the provisions in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method,"

P : Total hydrodynamic pressure in t, and

W_o : Unit weight of water in t/m³.

(2) Hydrodynamic Pressures on Columns

Hydrodynamic pressures acting on a column-type structure surrounding by water shall be determined as follows (refer to Fig. 3.3):

$$P = \frac{3}{4} k_h W_o b^2 h \left(1 - \frac{b}{4h}\right) \quad \text{for } \frac{b}{h} \leq 2 \quad \text{----- (3.9)}$$

$$P = \frac{3}{8} k_h W_o b^2 h \quad \text{for } \frac{b}{h} > 2 \quad \text{----- (3.10)}$$

$$h_g = \frac{1}{2} h \quad \text{----- (3.11)}$$

3.6 Ground Surface Assumed in Earthquake Resistant Design

In earthquake resistant design the bearing capacities of soil layers which are specified in Section 3.7 "Soil Layers Whose Bearing Capacities are Neglected in Earthquake Resistant Design" shall be neglected. Ground surface in design shall be assumed to be the level of the lower boundary of the neglected layer, if the layer extends continuously below the actual ground surface.

For this case the neglected layer shall be assumed to have properties of zero cohesion and zero angle of internal friction.

3.7 Soil Layers Whose Bearing Capacities are Neglected in Earthquake Resistant Design

(1) Sandy Soil Layers Vulnerable to Liquefaction

Saturated sandy soil layers which are within 10 meters below the actual ground surface, have a standard penetration test N-value less than 10, have a coefficient of uniformity less than 6, and also have a D_{20} -value on the grain size accumulation curve between 0.04 mm and 0.5 mm, shall have a high potential for liquefaction during earthquakes. Bearing capacities of these layers shall be neglected in design.

Saturated sandy soil layers which have a D_{20} -value between 0.004 and 0.04 mm or between 0.5 mm and 1.2 mm may liquefy during earthquakes, and shall be given particular attention. Estimation whether or not these layers will liquefy shall be made in accordance with the available information on liquefaction problems.

When a special investigation is performed, the provisions in this item (1) in Section 3.7 may not be required to apply.

(2) Cohesive Soil Layers and Silty Soil Layers

Bearing capacities of cohesive soil layers and silty soil layers, which are within 3 meters of the actual ground surface, and are very soft such as those with the compression strength, determined by unconfined compression tests or field tests, less than 0.2 kg/cm², shall be neglected in design.

(3) Weight of Soil Layers Whose Bearing Capacities are Neglected

The weight of soil layers whose bearing capacities are neglected shall have surcharge effects on the lower ground.

3.8 Buoyancy or Uplifts

Buoyancy or uplifts shall be determined in accordance with the provisions in the Specifications for Design of Substructures of Highway Bridges (See Reference 2 at the end of the English Version).

CHAPTER 4 DESIGN SEISMIC COEFFICIENT

4.1 General

The design seismic coefficient shall generally be determined in accordance with the provisions in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method." For those bridges, however, which have flexible piers and long fundamental periods, such as those with piers higher than 25 meters above the ground surface, the design seismic coefficient shall be determined in accordance with the provisions in Section 4.4 "Design Seismic Coefficient in the Modified Seismic Coefficient Method Considering Structural Response."

4.2 Design Seismic Coefficient in the Seismic Coefficient Method

- (1) The horizontal design seismic coefficient shall be determined by the following formula:

$$k_h = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot k_o \text{-----(4.1)}$$

where

- k_h : Horizontal design seismic coefficient,
 k_o : The standard horizontal design seismic coefficient (= 0.2),
 ν_1 : Seismic zone factor,
 ν_2 : Ground condition factor, and
 ν_3 : Importance factor.

The value of k_h shall be rounded to two decimals. The minimum value of k_h shall be considered as 0.10. The values of factors ν_1 , ν_2 and ν_3 shall be obtained by the provisions in Section 4.3 "Factors for Modifying the Standard Horizontal Design Coefficient."

- (2) The vertical design seismic coefficient, k_v , may generally be considered as 0. Vertical seismic forces for design of bearing supports, however, shall be determined in accordance with the provisions in Section 5.3 "Vertical Seismic Forces for Design of Connections between Superstructures and Substructures."
- (3) The horizontal design seismic coefficient may be considered as 0 for structural parts, soils and water below the assumed ground surface in design. The assumed ground surface in design shall be determined in accordance with the provisions in Section 3.6 "Ground Surface Assumed in Earthquake Resistant Design."

Item (3), however, shall not apply to underground structures such as culverts.

4.3 Factors for Modifying the Standard Horizontal Design Seismic Coefficient

(1) Seismic Zone Factor

Seismic zone factor shall be determined in accordance with Table 4.1, in which the zone classification shall be determined from Fig. 4.1 or Table 4.2.

Table 4.1 Seismic Zone Factor ν_1

Zone	Value of ν_1
A	1.00
B	0.85
C	0.70

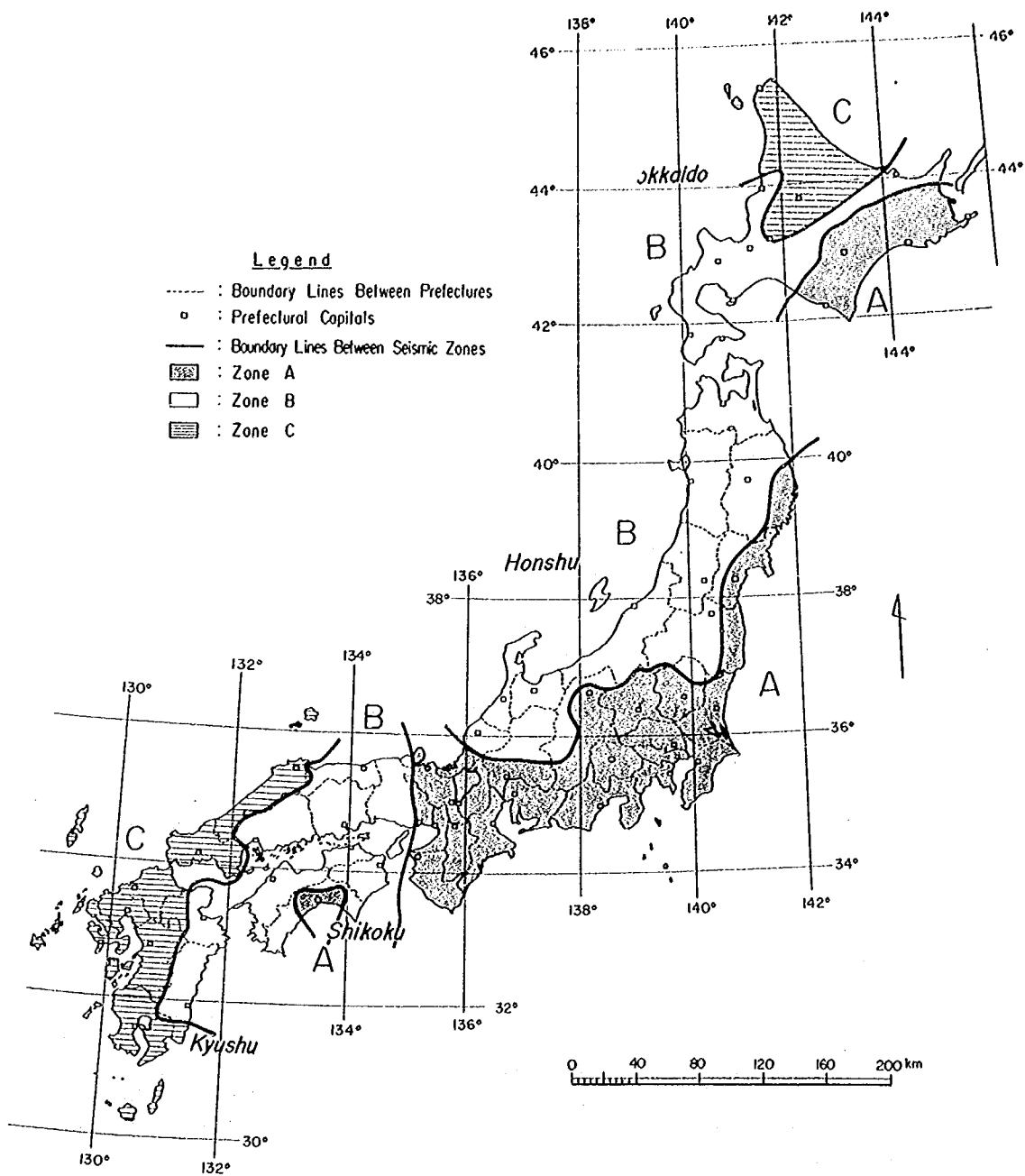


Fig. 4.1 Seismic Zoning Map

(Note)

Table 4.2, which indicates in detail the seismic zone through the use of place names, is omitted in the English version.

(2) Ground Condition Factor

Ground condition factor shall be determined in accordance with Table 4.3.

Table 4.3 Ground Condition Factor ν_2

Group	Definitions ¹⁾	Value of ν_2
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer ²⁾ with depth less than 10 meters above bedrock	0.9
2	(1) Diluvial layer ²⁾ with depth greater than 10 meters above bedrock (2) Alluvial layer ³⁾ with depth less than 10 meters above bedrock	1.0
3	Alluvial layer ³⁾ with depth less than 25 meters, which has soft layer ⁴⁾ with depth less than 5 meters	1.1
4	Other than the above	1.2

(Notes)

- 1) Since these definitions are not very comprehensive, the classification of ground conditions shall be made with adequate consideration of the bridge site.
Depth of layer indicated here shall be measured from the actual ground surface.
- 2) Diluvial layer implies a dense alluvial layer such as a dense sandy layer, gravel layer, or cobble layer.
- 3) Alluvial layer implies a new sedimentary layer made by a landslide.
- 4) Soft layer is defined in Section 3.7 "Soil Layer Whose Bearing Capacities are Neglected in Earthquake Resistant Design."

(3) Importance Factor

Importance factor shall be determined in accordance with Table 4.4.

Table 4.4 Importance Factor ν_3

Group	Definitions	Value of ν_3
1	Bridges on expressways (limited-access highways), general national highways and principal prefectural highways. Important Bridges on general prefectural highways and municipal highways.	1.0
2	Other than the above	0.8

Note: The value of ν_3 may be increased up to 1.25 for special cases in Group 1.

4.4 Design Seismic Coefficient in the Modified Seismic Coefficient Method Considering Structural Response

The design seismic coefficient specified in this Section shall apply to the design of superstructures and substructures of those bridges which have flexible piers and relatively long fundamental periods, such as ones in which the height of the substructures is 25 meters or more above the ground surface in design.

The above-mentioned ground surface in design shall be specified in Section 3.6 "Ground Surface Assumed in Earthquake Resistant Design."

The Design Seismic coefficients in this Section shall be determined by modifying the horizontal design seismic coefficient in Section 4.2 "Design Seismic Coefficient in the Seismic Coefficient Method," on the basis of characteristics of strong earthquake ground motions recorded, and dynamic properties of bridge structures, such as natural periods, mode shapes and damping capacities of bridge substructures.

After obtaining the design seismic coefficient, the method for applying seismic loads for the design shall be the same as specified by the provisions in Chapter 3 "Loads and Conditions in Earthquake Resistant Design."

The provisions in this Section shall not apply to the design of superstructures of those bridges in which superstructures are flexible and have longer periods, such as suspension bridges.

4.4.1 Design Seismic Coefficient

(1) The horizontal design seismic coefficient shall be determined by the following formula:

$$k_{hm} = \beta k_h \text{-----} (4.2)$$

where

k_{hm} : Horizontal design seismic coefficient in the modified seismic coefficient method considering structural response,

k_h : Horizontal design seismic coefficient given by eq. (4.1); and

β : A factor dependent on the fundamental period of the bridge, and obtained by Table 4.5 or Fig. 4.2.

For structures whose fundamental periods are shorter than 0.5 sec., β may be considered as 1.0

The value of k_{hm} shall be rounded to two decimals. The minimum value of k_{hm} shall be considered as 0.05.

(2) The vertical design seismic coefficient shall be provided in accordance with the provisions in Item (2) of Section 4.2.

(3) The horizontal design seismic coefficient for the portions below the assumed ground surface in design, shall be provided in accordance with the provisions in Item (3) of Section 4.2.

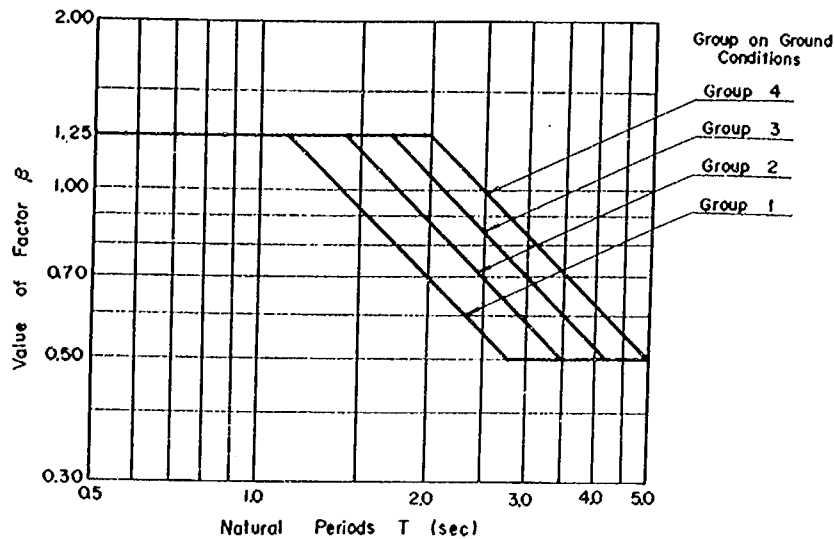


Fig. 4.2 Factor β (For reference of Table 4.5)

Table 4.5 Value of β

Group on Ground Conditions	Value of β for Fundamental Period T (sec.)		
1	$\beta = 1.25$ for $0.5 \leq T \leq 1.1$	$\beta = 1.40/T$ for $1.1 \leq T \leq 2.8$	$\beta = 0.50$ for $T \geq 2.8$
2	$\beta = 1.25$ for $0.5 \leq T \leq 1.4$	$\beta = 1.75/T$ for $1.4 \leq T \leq 3.5$	$\beta = 0.50$ for $T \geq 3.5$
3	$\beta = 1.25$ for $0.5 \leq T \leq 1.7$	$\beta = 2.10/T$ for $1.7 \leq T \leq 4.2$	$\beta = 0.50$ for $T \geq 4.2$
4	$\beta = 1.25$ for $0.5 \leq T \leq 2.0$	$\beta = 2.50/T$ for $2.0 \leq T \leq 5.0$	$\beta = 0.50$ for $T \geq 5.0$

(Notes)

- 1) Refer to Fig. 4.2.
- 2) Refer to Table 4.3 regarding groups on ground conditions.

4.4.2 Method for Obtaining Fundamental Periods

Fundamental natural periods of a bridge shall be determined for the individual system consisting of each substructure and the part of superstructures supported by it rather than for the structural system as a whole.

(1) Bridges Supported by Spread Foundations or Pile Foundations

For those bridges which are supported by spread foundations or pile foundations, the fundamental periods may be obtained from Table 4.6.

Any formulas in Table 4.6 shall apply to bridges in which the level of the base of the footing is lower than that of the assumed ground surface in design and the deformation of the substructure is mainly caused by the elastic flexural deformation of the pier which is the upper part of the substructures above the top of the footing. Therefore, they shall not apply to bridges in which the level of the base of the footing is higher than that of the assumed ground surface in design.

Table 4.6 Fundamental Periods of Bridges Supported by Spread Foundations or Pile Foundations

Type of Structural System	Direction	Formulas for Fundamental Periods	
		Material of Pier	
		Reinforced Concrete	Steel
1 A bridge where most superstructures are continuous, have fixed supports (or movable supports specified in Article 5.2.1) on most substructures, and also have rigid abutments, to one of which the extreme end of the superstructures is connected with a fixed support (See Fig. 4.3)	Transverse	$T = 2\pi \sqrt{\frac{0.3W_p + W_u}{3EI_g}} h^3$ (4.3)	$T = 2\pi \sqrt{\frac{0.3W_p + W_u}{4.5EI_g}} h^3$ (4.4)
	Longitudinal	$T = \frac{\pi}{8} \sqrt{\frac{W_p}{EI_g}} h^3$ (4.5)	
2 Other than the above: For example, a bridge with simple supports	Longitudinal or Transverse	$T = 2\pi \sqrt{\frac{0.3W_p + W_u}{3EI_g}} h^3$ (4.6)	

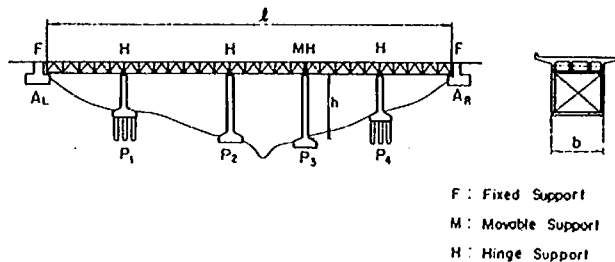


Fig. 4.3 An Example of Type 1 of Structural System in Table 4.6

where

- T : Fundamental period in second of the system consisting of a substructure and the section of the superstructures which it supports,
- W_p : The weight of the pier in t,
- W_u : The weight of the section of superstructures in t supported by the substructure being considered,
- E : Young's modulus of the pier in t/m²,
- I : Moment of inertia of the pier in m⁴ in the direction considered.
For piers with varying section with the height, I may be an average value,
- h : The height of the pier in m, and
- g : Acceleration of gravity (= 9.8 m/sec²).

(Note)

Eq. (4.4) shall apply to those bridges which have a ratio of the length between supports on both abutments to the width between outside girders, less than approximately 50 (refer to l/b in Fig. 4.3).

(2) Bridges Supported by Caisson Foundations

For those bridges which are supported by caisson foundations, the fundamental periods may be obtained from Table 4.7.

Table 4.7 Fundamental Periods of Bridges Supported by Caisson Foundations

Type of Structural System		Direction	Formulas for Fundamental Periods
1	Type 1 in Table 4.6	Transverse	one of eqs. (4.3) (or (4.4), and (4.7), which gives the largest value of β
		Longitudinal	eq. (4.5)
2	Type 2 in Table 4.6	Transverse	one of eqs. (4.6) and (4.7), which gives the largest value of β
		Longitudinal	

(Note)

$$T = 2\pi \sqrt{\frac{(h + \frac{2}{3}l)^2 W_u + (\frac{1}{3}h^2 + \frac{2}{3}hl + \frac{4}{9}l^2) W_p + \frac{l^2}{9} W_c}{g \left\{ K_H \frac{bl^3}{36} + K_s \frac{Al^2}{9} + K_v l B \right\}}} \quad (4.7)$$

where

T : Fundamental period in second of the system consisting of a substructure and the section of the superstructures which it supports,

W_p : The weight of the pier in t,

W_u : The weight of the part of superstructures in t supported by the substructure being considered,

W_c : The weight of the caisson foundation in t,

h : The height of pier in m,

A : Cross-sectional area in m^2 at the base of the caisson foundation,

I_B : Moment of inertia in m^4 at the base of the caisson foundation in the direction considered,

K_H : Horizontal coefficient of subgrade reaction in t/m^3 at the level of the base of the caisson foundation,

K_v : Vertical coefficient of subgrade reaction in t/m^3 at the base of the caisson foundation,

K_s : Horizontal coefficient of subgrade reaction in t/m^3 for shear deformation at the base of the caisson foundation, and

g : Acceleration of gravity ($= 9.8 \text{ m/sec}^2$).

CHAPTER 5 GENERAL PROVISIONS FOR DESIGN OF STRUCTURAL DETAILS

5.1 General

Every bridge structure or every portion thereof shall be designed and constructed to resist seismic forces as provided in Chapter 1 through Chapter 4 and to meet the provisions for design of structural details specified in this Chapter.

Moreover, attention shall be paid to the following respects.

- (1) For those abutments which are constructed in soft ground layers, the failure of the ground layer during earthquakes shall be checked.
- (2) For those bridges in which any adjacent substructures have different ground conditions, different type of structural systems, or different structural dimensions, special attention shall be paid to the design of structural details, considering that those two substructures may respond differently during earthquakes.
- (3) For those portions such as joints between superstructures and substructures, connections between piers and foundations, or connections between footings and piles in pile foundations, where the seismic forces may not be transmitted smoothly and seismic failure have been observed often in the past, particular attention shall be paid to the design of structural details, considering accuracy of evaluation of ground conditions, existence of construction joints, accuracy of construction, etc.

5.2 Devices for Preventing Superstructure from Falling

Movable supports shall have stoppers (special devices for resisting large movements of superstructures during earthquakes) to prevent the superstructures from falling from the substructures, caused by the dislocation of the upper shoes of the supports from the lower shoes during strong earthquakes (refer to 5.2.1).

For the girder ends one of the following methods shall be employed as well as the above-mentioned consideration.

- (1) A method extending the length between the end of the support and the edge of the substructure (or widening the width of the crest of the substructure in the longitudinal direction to the bridge axis) in order to prevent the superstructures from falling from the substructure (refer to 5.2.2 or 5.2.3).
- (2) A method connecting adjacent girders on the substructure to prevent the superstructures from falling from the substructure even if they become dislodged from the substructure (refer to 5.2.4).

5.2.1 Stoppers at Movable Supports

The allowable movable length in the design of stoppers at movable supports shall be assumed as the sum of the movement due to temperature change, the movement due to the deflection of the girder when subjected to live loads, a margin for covering construction errors, and 20 mm.

The above-mentioned provision need not apply to those stoppers which are not installed near the supports.

The horizontal design seismic coefficients for designing stoppers shall be determined by increasing the horizontal design seismic coefficient given by eq. (4.1) or (4.2) by 50% or more.

5.2.2 Method of Extending the Length between the End of the Support and the Edge of the Substructure

For those substructures which support the ends of girder, the length S (in cm) between the end of the support and the edge of the substructure, shall be equal to or more than the value given by the following formulas:

$$S = 20 + 0.5 \, l \quad \text{for } l \leq 100 \, \text{m}$$

$$S = 30 + 0.4 \, l \quad \text{for } l > 100 \, \text{m}$$

where

S : Length between the end of the support and the edge of the substructure in cm, and

l : Span length in meters.

For particularly important bridges constructed in soft ground layers (Group 4 in Table 4.3), the value of S shall be equal to 35 cm or more.

5.2.3 Suspended Joints

For suspended joints the length between the ends of girders shall be equal to 60 cm or more, as shown in Fig. 5.1. The length for those bridges constructed in soft ground layer (Group 4 in Table 4.3) shall be equal to 70 cm or more.

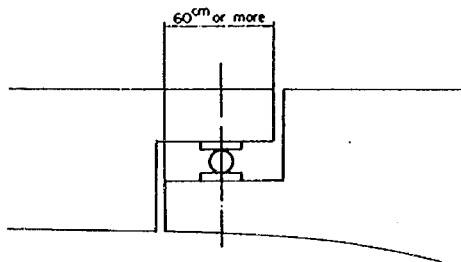


Fig. 5.1 Length between the Ends of both Girders

5.2.4 Method of Connecting Adjacent Girders

Devices for connecting adjacent girders on substructures shall have the movable length specified in Section 5.2.1, for cases in which at least one of two supports is a movable one.

The devices shall be designed to rotate freely to allow the rotation of the girder subjected to live loads, for cases in which both two supports are fixed on one pier.

5.3 Vertical Seismic Forces for Design of Connections between Superstructures and Substructures

The vertical design seismic coefficient for the design of connections between superstructures and substructures shall be assumed as 0.10. When the vertical design seismic coefficient applies upward, only seismic forces shall be considered, neglecting the effects of the dead loads.

The same value of the vertical design seismic coefficient shall be employed for the design of any connections similar to the above.

5.4 Methods for Transmitting Seismic Forces at Connections between Superstructures and Substructures

The method for transmitting seismic forces at connection between superstructures and substructures shall be as follows:

- (1) For cast-in-place reinforced concrete bridges, the means of transmission of seismic forces shall be due to the bearing pressure between the swelling at the base of the lower shoe and the concrete at the crest of the substructure. The concrete portion near the base of the lower shoe shall resist seismic forces as one body together with the pier of the substructure. In the above-mentioned cases, the means of transmission of seismic forces between the upper shoe and the girder shall be due to the anchors fixed on the upper shoe.

For a margin of safety, anchor bolts between the lower shoe and the substructure shall be designed to resist seismic forces alone, in consideration of cases where no resistance between the swelling at the base of the lower shoe and the concrete at the crest of the substructure can be expected.

The above method is recommended not only for cast-in-place concrete bridges, but, if possible, also for prefabricated concrete bridges or steel bridges.

- (2) In cases where no bearing resistance of concrete can be expected, anchor bolts shall be employed to transmit the seismic forces. In these cases one of the following two methods shall be considered.
- (a) A method in which a steel plate with anchor bolts is fixed firmly on the crest of a substructure while concrete is being placed, and then the lower shoe of the support is welded to the steel plate after the erection of the girders.
 - (b) A method in which a hole is prepared when concrete is placed, a support is set up near the hole, and then anchor bolts are fixed by placing cement mortar into the hole, or a method in which anchor bolts are set up while concrete is being placed, and then the lower shoe is fixed on the anchor bolts.
- (3) Anchor bolts used shall be 25 mm or more in diameter, and the depth of the anchor bolts fixed in the concrete shall be 10 times the diameter or more.

5.5 Devices Expected for Decreasing Seismic Forces

When any devices which are expected to decrease seismic force are employed for bridge structures, sufficient investigations shall be conducted on their effectiveness, and special attention shall be paid to preventing the superstructures from falling.

CHAPTER 6 MISCELLANEOUS PROVISIONS

When sufficient reasons exist, these Specifications need not apply to the design of bridges.

REFERENCE

References are provided exclusively in the English version.

[Reference 1]

Specifications for Design of Substructures of Highway Bridges

Volume for General Survey and Design

Part 3 Design, Chapter 2 Loads.

Section 2.5 Earth Pressures

Earth pressures acting on a wall shall be the distributed loads given by the following formulas:

(1) Normal Earth Pressures

- (a) Earth pressures acting on a movable wall during normal time shall be determined by the Coulomb's theory as follows:

i) For Sandy Soils

$$P_A = \gamma \cdot K_A x + K_A q$$

$$P_P = \gamma \cdot K_P x + K_P q$$

ii) For Cohesive Soils

$$P_A = \gamma \cdot K_A x - 2C\sqrt{K_A} + K_A q$$

$$P_P = \gamma \cdot K_P x + 2C\sqrt{K_P} + K_P q$$

- (b) Earth pressures on a fixed wall during normal time shall be determined by

$$P_A = \gamma \cdot K_s \cdot x + K_s q$$

(2) Earth pressures during earthquakes shall be determined by the Mononobe-Okabe method.

$$P_A = (1 - k_v) \gamma \cdot x \cdot K_{EA}$$

$$P_P = (1 - k_v) \gamma \cdot x \cdot K_{EP}$$

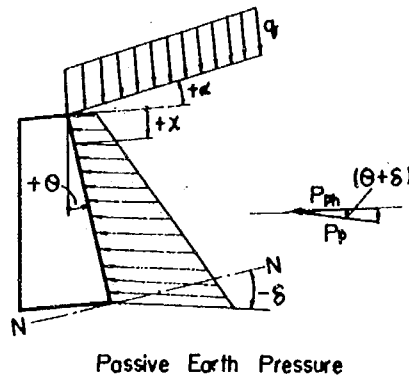
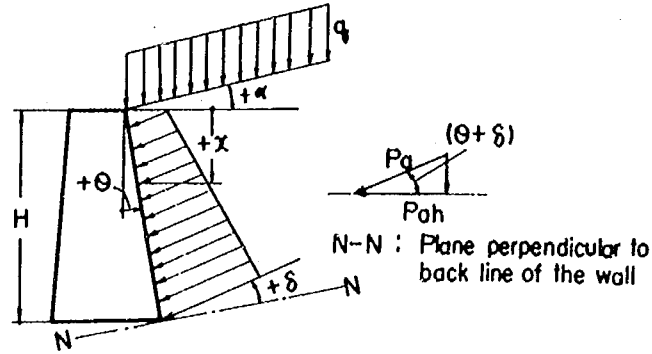


Fig. 1 Earth Pressure

where

- C : Cohesion of the soil in t/m^2 ,
- K_A : Active earth pressure coefficient for Coulomb's theory,
- K_P : Passive earth pressure coefficient for Coulomb's theory,
- K_{EA} : Active earth pressure coefficient during earthquakes,
- K_{EP} : Passive earth pressure coefficient during earthquakes,
- K_s : Earth pressure coefficient at rest,
- k_v : Vertical seismic coefficient,
- P_A : Active earth pressure in t/m^2 at depth of x meters,
- P_P : Passive earth pressure in t/m^2 at depth of x meters,
- q : Surcharge in t/m^2 on ground surface,
- x : Arbitrary depth in meters,
- α : Angle between the ground surface line and the horizontal line,
- γ : Unit weight of the soil in t/m^3 ,
- δ : Angle of friction between the wall and the soil,
- ϕ : Angle of internal friction of the soil, and
- θ : Angle between the back line of the wall and the vertical line.

(Comments)

K_A , K_P , K_{EA} and K_{EP} are expressed as follows:

$$K_A = \frac{\cos^2 (\phi - \theta)}{\cos^2 \theta \cdot \cos (\theta + \delta) \left[1 + \sqrt{\frac{\sin (\phi + \delta) \cdot \sin (\phi - \alpha)}{\cos (\theta + \delta) \cdot \cos (\theta - \alpha)}} \right]^2}$$

$$K_P = \frac{\cos^2 (\phi + \theta)}{\cos^2 \theta \cdot \cos (\theta + \delta) \left[1 - \sqrt{\frac{\sin (\phi - \delta) \cdot \sin (\phi + \alpha)}{\cos (\theta + \delta) \cdot \cos (\theta - \alpha)}} \right]^2}$$

$$K_{EA} = \frac{\cos^2 (\phi - \theta_0 - \theta)}{\cos \theta_0 \cdot \cos^2 \theta \cdot \cos (\theta + \theta_0) \left[1 + \sqrt{\frac{\sin \phi \cdot \sin (\phi - \alpha - \theta_0)}{\cos (\theta - \theta_0) \cdot \cos (\theta - \alpha)}} \right]^2}$$

$$K_{EP} = \frac{\cos^2 (\phi - \theta_0 + \theta)}{\cos \theta_0 \cdot \cos^2 \theta \cdot \cos (\theta - \theta_0) \left[1 - \sqrt{\frac{\sin \phi \cdot \sin (\phi + \alpha - \theta_0)}{\cos (\theta - \theta_0) \cdot \cos (\theta - \alpha)}} \right]^2}$$

where $\sin (\phi \pm \alpha - \theta_0) = 0$ when $\phi \pm \alpha - \theta_0 < 0$
and δ is assumed to be zero during earthquakes.

where

$$\theta_0 = \tan^{-1} \frac{k_h}{1 - k_v}$$

k_h : Horizontal seismic coefficient

k_v : Vertical seismic coefficient

[Reference 2]

Specifications for Design of Substructures of Highway Bridges

Volume for General Survey and Design

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Section 2.7 Buoyancy or Uplifts

When it is apparent that buoyancy forces or uplifts act on structures, they shall be taken into account in the design.

(Comments)

When it is unknown whether they act or not, both cases shall be taken into account in the design.