

A CASE HISTORY OF BRIDGE PERFORMANCE DURING EARTHQUAKES IN JAPAN

(To be presented at the International Conference on
Case Histories in Geotechnical Engineering,
St. Louis, Missouri, USA, May 6-11, 1984)

February 1984

**Earthquake Engineering Division
Earthquake Disaster Prevention Department
Public Works Research Institute
Ministry of Construction**

PREFACE

This Technical Memorandum of the Public Works Research Institute presents the paper entitled "A Case History of Bridge Performance during Earthquakes in Japan," which will be presented at the International Conference on Case Histories in Geotechnical Engineering, to be held in St. Louis, Missouri, U.S.A., May 6–11, 1984.

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SYNOPSIS This paper describes the results of investigations on seismic damages to highway bridges due to the past major earthquakes occurred in Japan, with emphasis on geotechnical engineering aspects. More than three thousand highway bridges sustained seismic damages for these sixty years since 1923. Among them some thirty bridges completely fell down, including nine bridges failed due to the effects of fires. For some typical bridge damages, results of detailed analyses are also given in order to clarify the causes of the damages. From the review of comprehensive studies on actual bridge damages, this paper makes several conclusions regarding seismic damage features of bridge structures. Important factors to be considered in the seismic design of bridges, and research subjects on which further investigations are necessitated for improving the current seismic design procedures.

1. INTRODUCTION

In order to appropriately understand the effects of earthquakes on civil engineering structures and to establish rational seismic design codes, it is most important to grasp structural behavior during actual earthquakes experienced. In this paper, seismic damage features of highway bridges during past major earthquakes in Japan will be described, and relevant results of analyses for damaged bridges will be introduced to understand the causes of destructive damages.

Table I and Fig. 1 indicate an outline of highway bridge damages for thirteen major earthquakes which caused extensive damages. Table I lists dates of earthquake occurrence times, names of earthquakes, Richter magnitudes determined by the Japan Meteorological Agency (JMA), numbers of bridges damaged, numbers of bridges fallen, and damage costs of bridge at the time of each earthquake occurrence. Fig. 1 shows the epicenters of the thirteen earthquakes and their occurrence years. From Table I, it is understood that more than 3,000 highway bridges sustained seismic damages since 1923 in Japan, and that 29 bridges were fallen (including 9 bridges fallen by fires).

In the following sections damage features of highway bridges during major eight earthquakes (occurred in 1923 to 1983) will be mentioned, with emphasis on geotechnical engineering characteristics (T. Iwasaki et al. 1972).

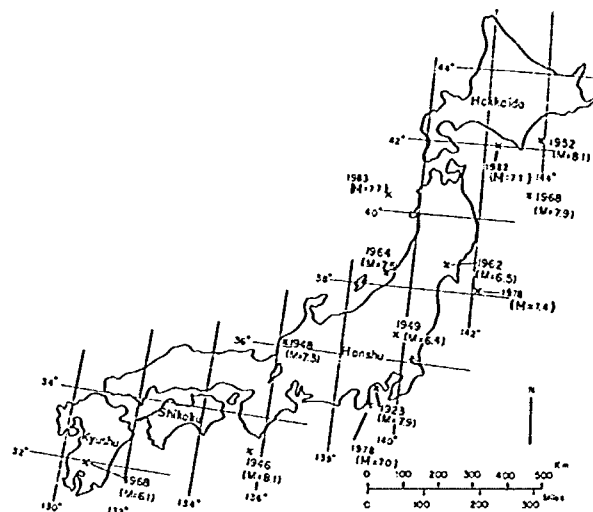


Fig. 1. Epicenters of Thirteen Earthquakes

Table I. Thirteen Earthquakes Causing Bridge Damages

No.	Date	Name	M [*]	No. of Damaged Bridges	No. of fallen Bridges
1	Sept. 1, 1923	Kanto	7.9	1785	17 ^{**}
2	Dec. 21, 1946	Nankai	8.1	346	1
3	Jun. 28, 1948	Fukui	7.3	243	7
4	Dec. 26, 1949	Imaichi	6.4	1	0
5	Mar. 4, 1952	Tokachi-oki	8.1	128	0
6	Apr. 30, 1962	Northern Miyagi	6.5	187	0
7	Jun. 16, 1964	Niigata	7.5	98	3
8	Feb. 21, 1968	Ebino	6.1	10	0
9	May 16, 1968	Tokachi-oki	7.9	101	0
10	Jan. 14, 1978	Izu-Oshima Kinkai	7.0	7	0
11	Jun. 12, 1978	Miyagi-ken-oki	7.4	108	1
12	Mar. 21, 1982	Urakawa-oki	7.1	11	0
13	May 26, 1983	Central Japan Sea	7.7	133	0
Total				3158	29

(*) Magnitudes are on the Richter scale

(**) Nine bridges were fallen by the effects of fires.

2. KANTO EARTHQUAKE OF 1923 (Magnitude = 7.9)

On September 1, 1923 a severe earthquake hit Sagami Bay off the southern coast of Kanto area which includes Tokyo, Yokohama, and other major cities in Japan. The earthquake recorded magnitude 7.9 on the Richter scale. The epicenter was 35.2°N, 139.3°E, and its hypocentral depth was estimated to be in the range of 0 – 20km. The epicenter location is shown in Figs. 1 and 2. Fig. 2 also shows the intensity distribution as determined by JMA. The definitions of the JMA Seismic Intensity Scale are described in Table II. In this table, corresponding magnitudes of accelerations at ground surface as suggested by Prof. H. Kawasumi (1951) are also shown.

Table II. 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, swirving 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. Kawasumi (1951)

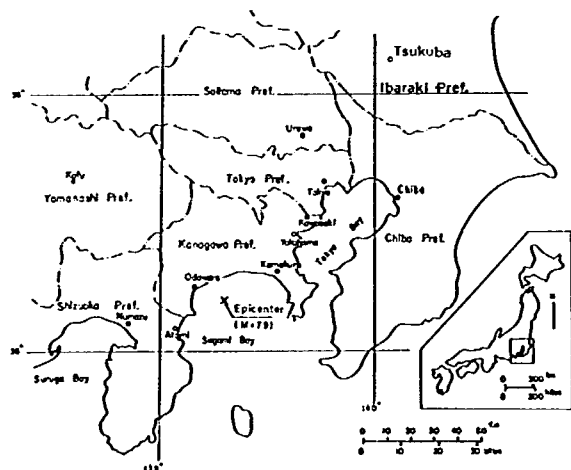


Fig. 2. The Kanto Earthquake of 1923

Substantial damages to bridge structures as well as other engineering structures were caused throughout the southern Kanto Area, especially in Tokyo, Yokohama, and the vicinity of the epicenter. Although a great amount of damage was caused by extensive fires in the large cities, vibrational effects of the earthquake also caused extensive damage to engineering structures.

The Kanto Earthquake was certainly one of the most significant earthquakes in Japan, and it greatly affected seismic design procedures of engineering structures. Before the earthquake, no distinct seismic regulations existed. Therefore, no structures had been designed with adequate lateral strength. After the earthquake, however, seismic design specifications were quickly introduced.

During the Kanto Earthquake, nearly two thousand highway bridges suffered light to heavy damages as shown in Table III. Damage caused by ground vibration was severest in Kanagawa Prefecture located near the epicenter. In the city of Yokohama, which is the capital city of Kanagawa Prefecture, the percentage of bridges damaged by vibrational effects was very high.

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

(A) Total Number of Bridges Damaged

Prefecture 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%	
Chiba	690	65	9.4%	Only wooden bridges suffered inside the affected area
Total	7,950	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		
		Vibration + Fires	Vibration Only	Fires Only
Wooden	35	26 (74.3%)	25 (71.4%)	8 (22.9%)
Steel	31	11 (35.5%)	16 (51.6%)	3 (9.7%)
Reinforced concrete	2	0 (0%)	2 (100%)	0 (0%)
Total	108	37 (33.3%)	43 (39.8%)	11 (10.2%)

Among a number of highway bridges damaged in Kanagawa Prefecture, the damages to Banyu Bridge and Aikoku Bridge will be introduced as typical ones in which interactions of soils and structures affected the stability of the bridge foundations.

Banyu Bridge

This bridge was located on National Highway No. 1 across Banyu River (presently Sagami River) about 15km northeast of the epicenter, between Chigasaki and Hiratsuka cities, Kanagawa Prefecture. The bridge substructures were under construction at the time of the earthquake.

Both abutments of gravity-type reinforced concrete with pile foundations had been completed at the time of the earthquake. The piers were reinforced concrete rigid frames with concrete well foundations. Among the total of 56 piers, only 6 had been completed in the left bank of the river. The well foundations of the remaining 42 piers had either just been completed or were under construction at the time of the earthquake. The superstructures had not been erected at the time of the earthquake.

During the Kanto Earthquake, the substructures sustained extensive damage as shown in Fig. 3 to 5. The right and left abutments tilted about 12° and 4° , respectively, towards the center of the river. Major failures occurred in the horizontal beams of the piers (Fig. 3). Insufficient curing of the concrete could have been an important factor causing this damage.

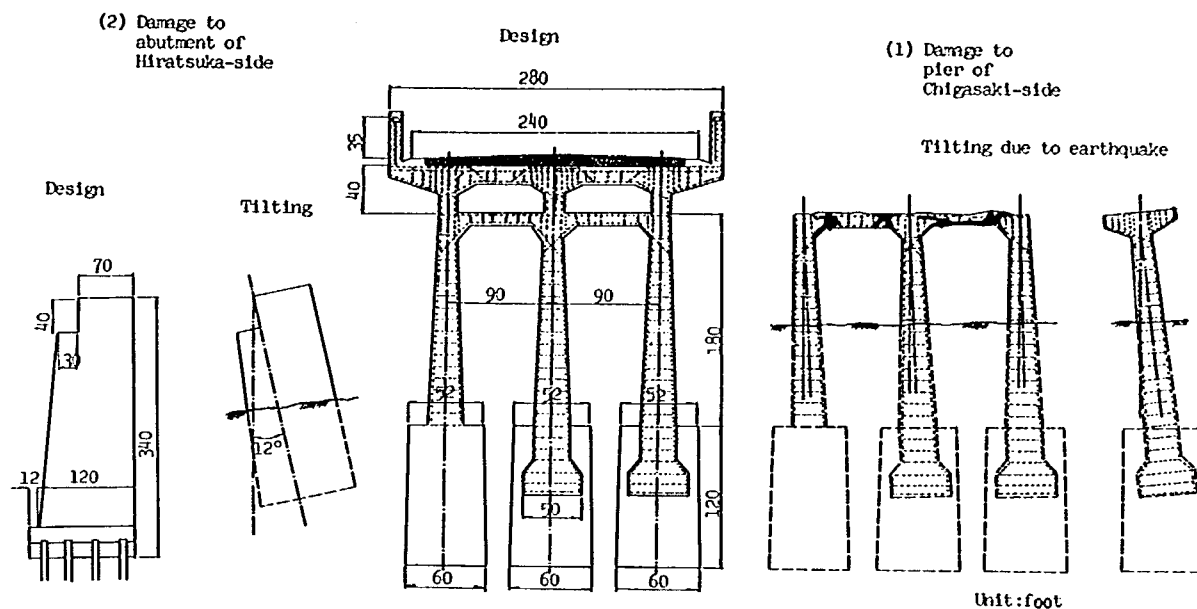


Fig. 3. Damage to the Banyu Bridge



Fig. 4. Damage to the Piers of the Banyu Bridge while under Construction



Fig. 5. Damage to the Caisson Foundations of the Banyu Bridge while Construction

Large displacements and floating of several well foundations were observed (Fig. 4). In view of the damage features of the foundations, it seems obvious that liquefaction of soils took place at this bridge site.

Bankoku Bridge

Bankoku Bridge, completed in 1903, was located about 40km northeast of the epicenter in Yokohama City, Kanagawa Prefecture. The two abutments were constructed of brick masonry with concrete block foundations resting on mudstone. A single span pony steel truss weighing 314 tons, with length of 36.6 m, and width of 12.2 m, was constructed on the abutments, as shown in Fig. 6.

The earthquake caused severe damage to the both abutments. The northern abutment slid by 1.2m horizontally toward the center of the river. This drastic sliding was accompanied by a relative movement between the upper and lower blocks of the foundation (see Fig. 7). Also observed was a huge vertical crack of the width of 2.7 cm in the abutment near the center line of the bridge axis. Excessive earth pressures forced on the two wing walls to cause their separation from the abutments. The upper portion of the southern abutment also slid toward the center of the river. The amount of sliding at this location totalled about 30 cm. Both wing walls at this abutment also separated.

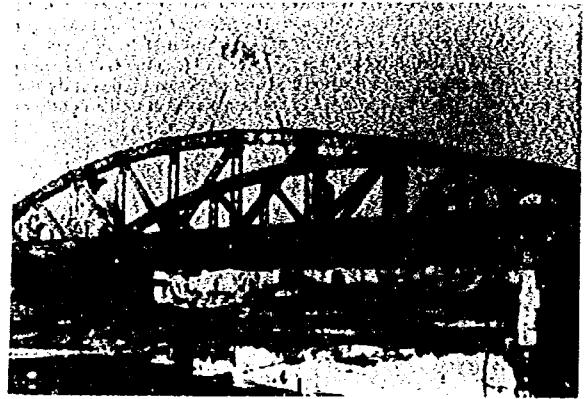


Fig. 6. Damage to the Bankoku Bridge

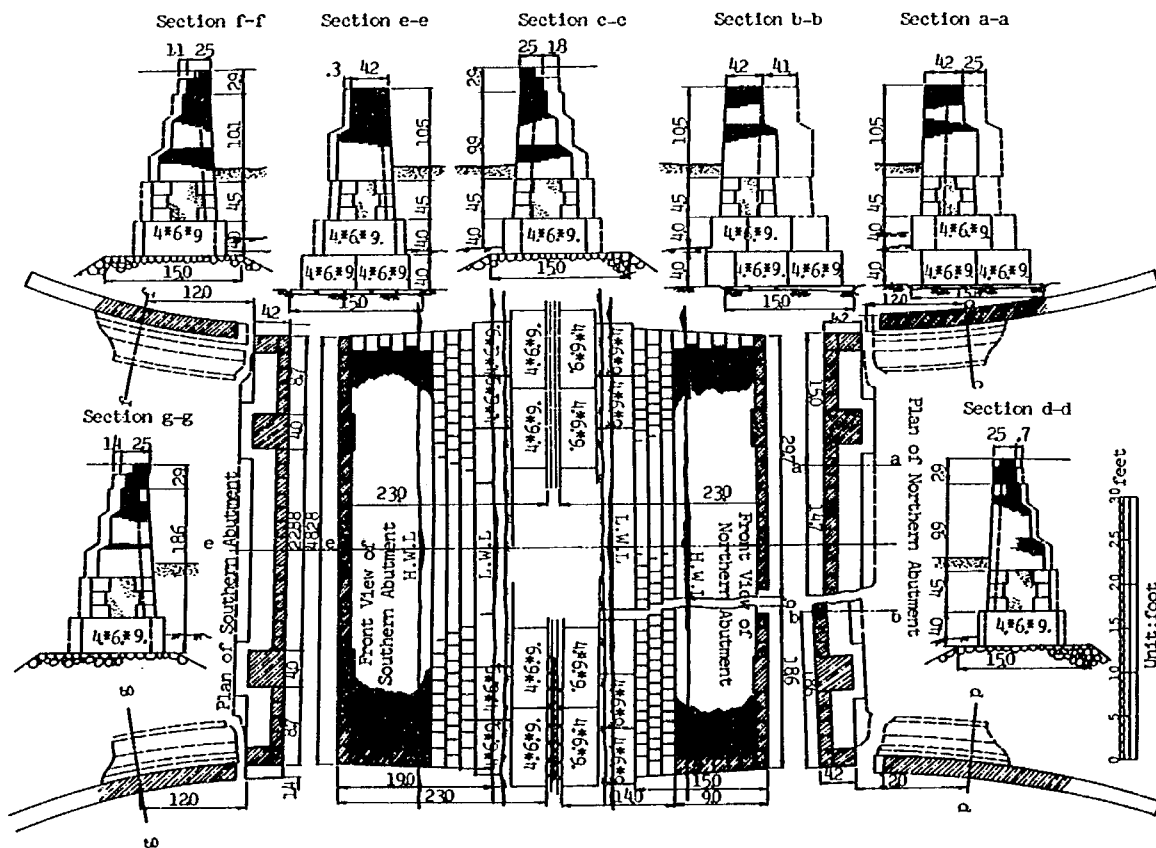


Fig. 7. Damage to the Northern Abutment of the Bankoku Bridge

The bridge truss girder suffered no significant damage. However, several lattice beams attached to the lower chords buckled. The movable roller support on the northern abutment moved considerably (40 cm on the east side and 30 cm on the west side) towards the south, together with the abutment. At the east side of this support the sole plate attached to the end of the truss was completely dislodged from the rollers (see Fig. 8). The fixed support on the southern abutment moved to the north (22 cm at the eastern side and 34 cm at the western side) along with the abutment, and all anchor bolts on the both sides were sheared off, as shown in Fig. 9.

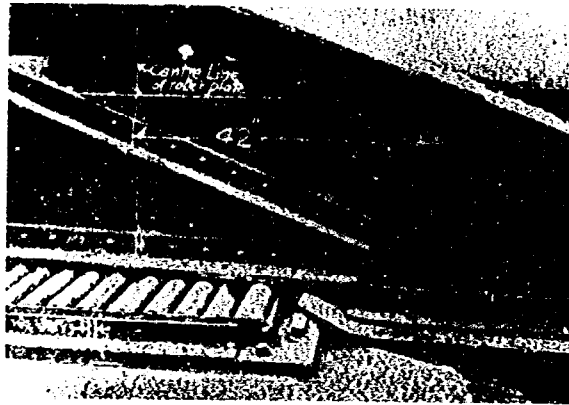


Fig. 8. Damage to the East Support at the Northern Abutment of the Bankoku Bridge

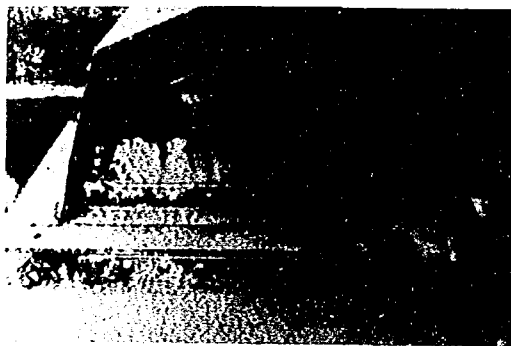


Fig. 9. Damage to the East Support at the Southern Abutment of the Bankoku Bridge

It is clear from the damage experienced by this bridge that severe damage can be caused due to sliding between blocks in the abutments.

3. FUKUI EARTHQUAKE OF 1948 (Magnitude = 7.3)

On June 28, 1948 a severe earthquake hit Fukui Plain, Fukui Prefecture, located in the midwestern part of Honshu Island. Since this earthquake, registering 7.3 on the Richter scale, had its epicenter at 36.1°N and 136.2°E which was very close to several cities and towns in Fukui and Ishikawa Prefectures, it caused a tremendous amount of damages in the two prefectures. The hypocentral depth was estimated about 20 km.

The Fukui Earthquake was one of the most damaging earthquake which have ever occurred in Japan. An extensive survey on the damages inflicted on civil engineering structures was carried out. Statistics on damages to highway bridges are summarized in Table IV.

Table IV. 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 203,945
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

Nakazuno Bridge, completed in 1932, crossed Kuzuryu River on Fukui-Kagayoshi-zaki Route, one of Fukui Prefectural roads located between Kawai and Nakafuji-shima Area, Yoshida County, Fukui Prefecture. It was located about 8 km south of the epicenter where peak ground acceleration was estimated about 0.6 g, basing on the evidences of overturned tombstones nearby.

The abutments were of gravity-type reinforced concrete construction with pile foundations. Thirteen piers were reinforced concrete columns resting either on pile or caisson foundations. A general view of a typical pier is shown in Fig. 10. The superstructures having a total length of 257 m and a width of 5.5 m, consisted of 14 simple span steel plate girders (14 x 18.4 m).

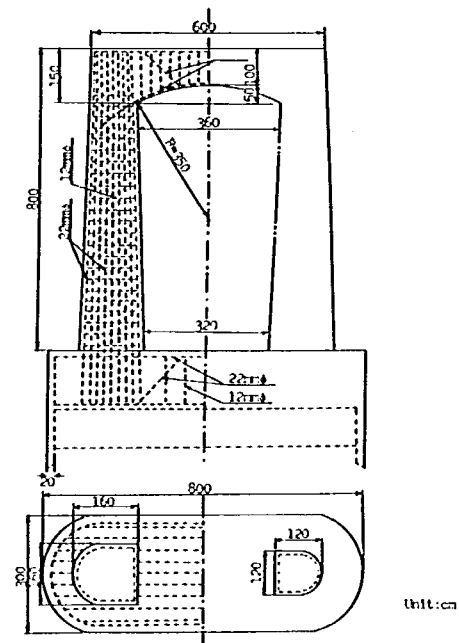


Fig. 10. Detailed Drawings of the Pier of the Nakazuno Bridge

During the Fukui Earthquake, both the superstructures and the substructures sustained heavy damage. The left abutment suffered cracking at its parapet walls (see Fig. 11), and the right abutment inclined toward the center of the river. The 1st and 2nd piers from the left bank did not suffer any damage. The 3rd to the 7th, 9th and 10th piers, however, tilted toward the left bank, suffering heavy cracking at the connections between columns and caisson foundations (see Fig. 12) exposing the reinforcing bars. Extensive cracking was also observed at the connections between columns and beams at the pier caps. The 12th and 13th piers tilted toward the right bank and suffered cracking at the connections between columns and beams.

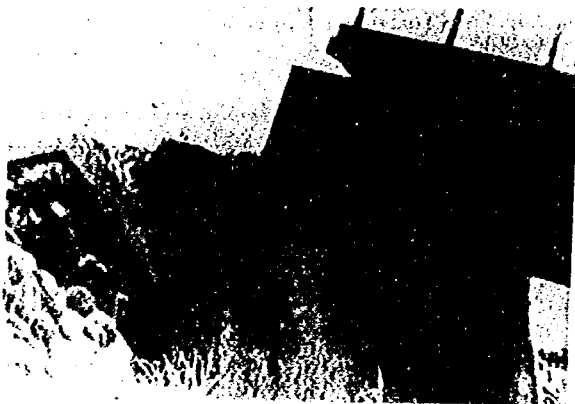


Fig. 11. Damage to the Left Abutment of the Nakazuno Bridge

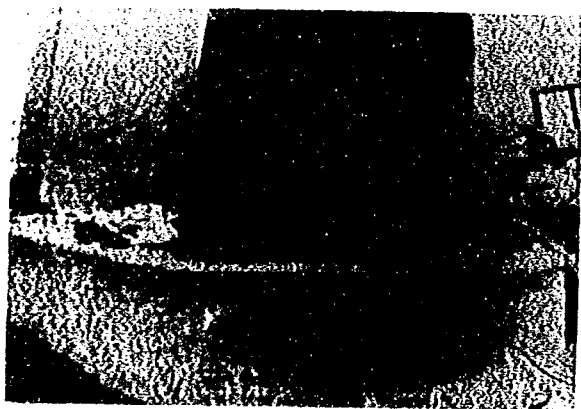


Fig. 12. Damage to the Nakazuno Bridge at the Column-to-Foundation Connection

Ten of the 14 spans fell down into the river (Figs. 13, 14, 15 and 16). Because they fell on the soft sand layer, the girders and their lateral members suffered no significant damage and were easily repaired for later use. Failure of the substructures was one of main causes of the behavior of the superstructures.

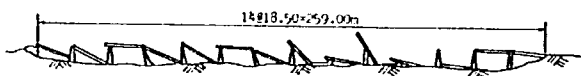


Fig. 13. Damage to the Nakazuno Bridge



Fig. 14. Damage to the Nakazuno Bridge



Fig. 15. Damage to the Girders of the Nakazuno Bridge

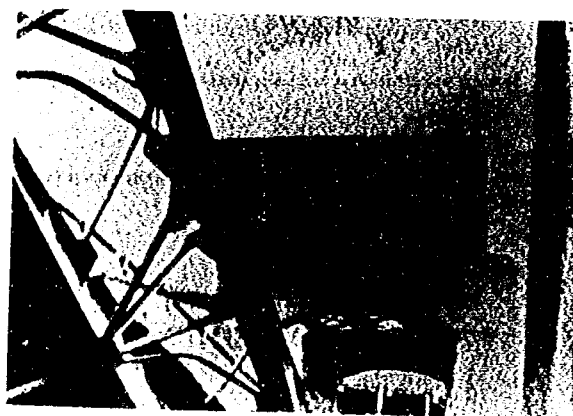


Fig. 16. Damage to the Girders of the Nakazuno Bridge

4. NIIGATA EARTHQUAKE OF 1964 (Magnitude = 7.5)

The Niigata Earthquake, registering 7.5 on the Richter scale, occurred in the north-western part of Honshu Island on June 16, 1964. Its epicenter (38.4°N , 139.2°E) was under the sea near Awashima Island about 55 km north of Niigata City (Figs. 1 and 17), and its hypocentral depth was estimated to be in the range of 20 – 30 km.

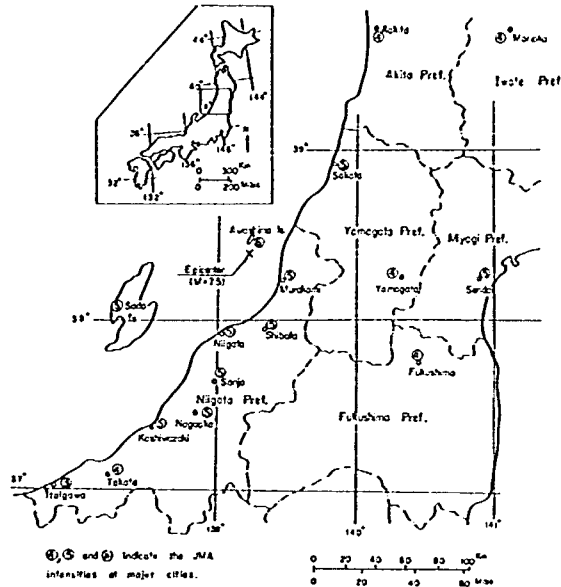


Fig. 17. The Niigata Earthquake of 1964

Severe damages were caused on the alluvial plain near the mouths of Shinano River and Agano River in Niigata City, especially in the area near the mouth of the Shinano River where loose sand layers with a high water table existed. In this area, many of reinforced concrete buildings, highway bridges and other structures sustained considerable damage due to liquefaction of sandy soils.

Strong Motion Records

Two strong motion accelerographs, installed in the basement floor and the roof of a damaged apartment building located along Shinano River, triggered time histories of acceleration during the earthquake shaking. The peak acceleration recorded at the basement level was about 0.15 g (predominant period of 2 seconds) horizontally and about 0.05 g (predominant period of 0.3 seconds) vertically.

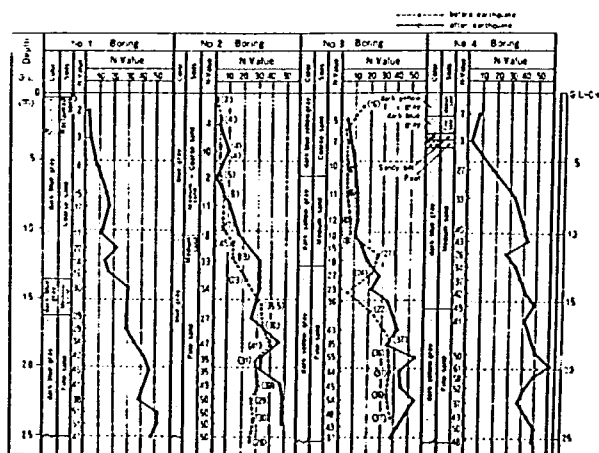


Fig. 18. Boring Logs at the Showa Bridge

Damage features of Showa bridge will be described below.

Showa Bridge

This bridge crossed Shinano River about 4 km located up to the mouth. The bridge was located some 55 km south of the epicenter. Construction was completed in May 1964 just one month prior to the earthquake.

The ground at this site consisted of sandy soils which were comparatively loose near the left bank and dense near the right bank, as shown in Figs. 18 to 20. Two abutments were pile bents (9 single-row piles of diameter 609 mm and of length 22 m) and so were piers (9 single-row piles of diameter 609 mm and of length 25 m). These bents had collar braces and cap beams, as shown in Fig. 21. The design seismic coefficient for the bridge structure was 0.2 horizontally. The superstructures having a total length of 303.9 m and a width of 24 m consisted of 12 composite steel simple span girders ($13.75 + 10 \times 27.64 + 13.75$ m).

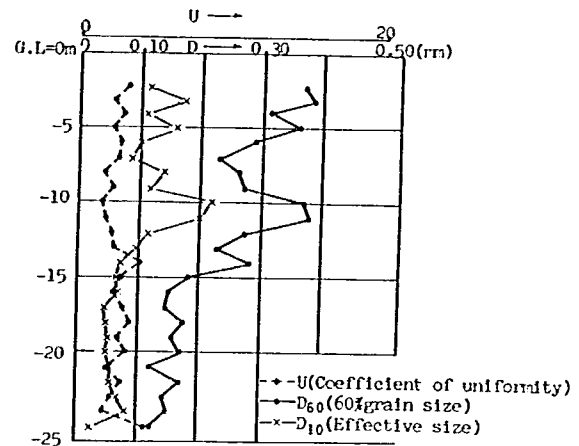


Fig. 19. Characteristics of Soil Grain Size at the Showa Bridge

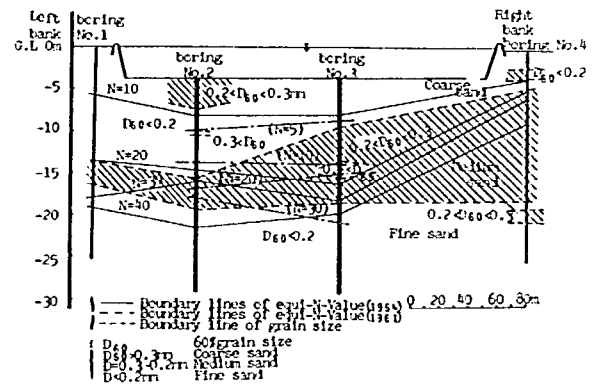


Fig. 20. Soil Profile at the Showa Bridge

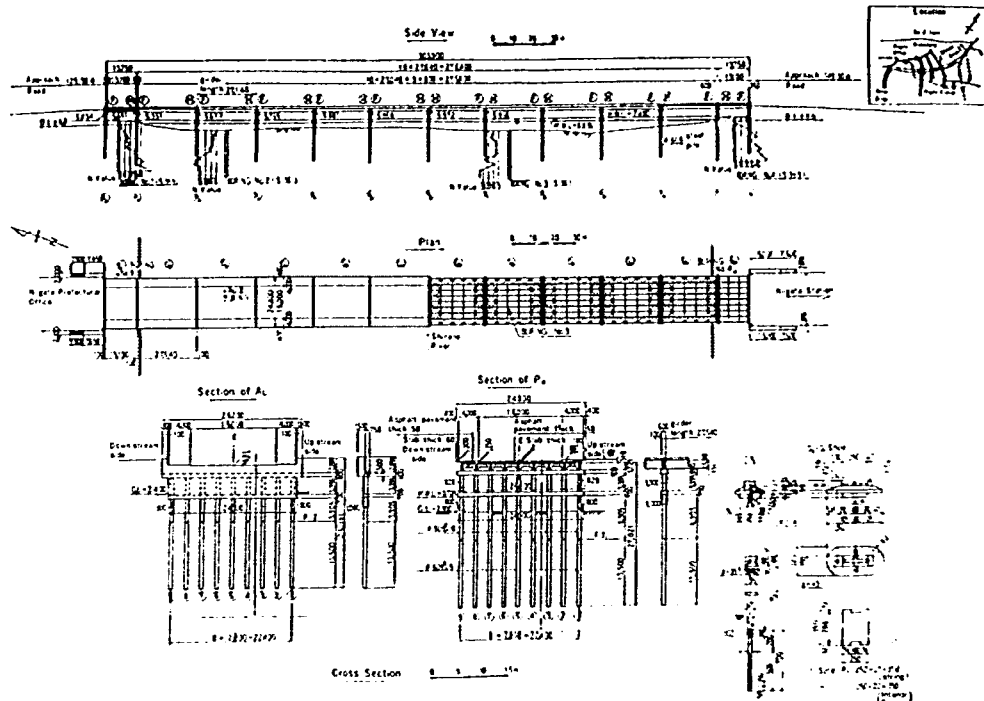


Fig. 21. General View of the Showa Bridge

Due to the Niigata Earthquake, the bridge sustained severe damage (Fig. 22). The left abutment moved about 1m toward the center of the river and its approach

road settled considerably. In contrast with this behavior, the right abutment and its approach road sustained no significant damages.

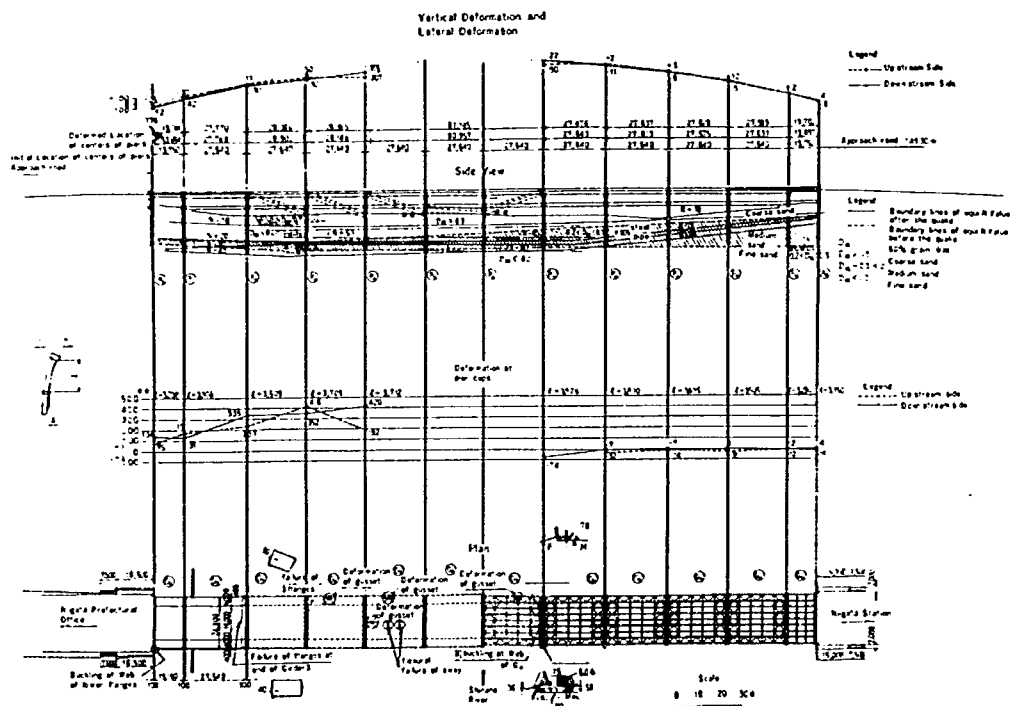


Fig. 22. Damage to the Showa Bridge

The first to fourth piers from the left bank tilted toward the right bank. The magnitudes of permanent deformations were 13 to 42 cm at their caps. The fifth and sixth piers collapsed completely into the river beds, while the seventh through the eleventh piers suffered only slight damages.

Five girders (third through seventh from left bank) out of twelve, fell down into the river water as shown in Figs. 23 and 24. Only the sixth span fell at both ends, which was due to failure of the fifth and sixth piers.

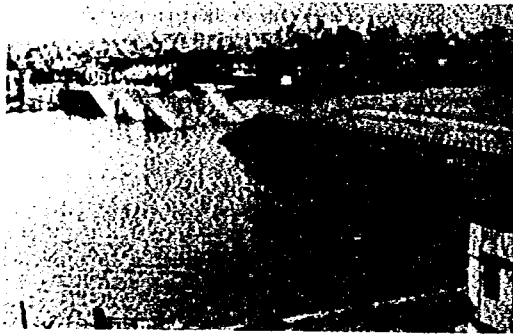


Fig. 23. Damage to the Showa Bridge

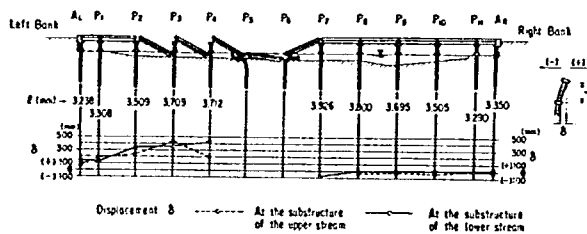


Fig. 24. The Circumstances of Showa Bridge Damaged during the Niigata Earthquake

Damage characteristics of this Showa Bridge are illustrated in Fig. 25. These reveal the following as main causes of the damages:

- 1) The substructures consisting of single-row steel pipe piles were too flexible.
- 2) Liquefaction of the sandy soils occurred (except near the right bank).
- 3) Both bearing supports of the sixth span were movable in the longitudinal direction of the bridge.
- 4) The superstructures consisted of simple girders which were not connected together.
- 5) Catastrophic sliding of the ground due to sand liquefaction took place near the left bank.

Following the earthquake, extensive studies were made on this bridge, including soils investigations, measurements of the dynamic properties of the less damaged bridge portions, measurements of deformation for the entire structure, pile investigations by pulling (Figs. 25 to 27), and dynamic analyses for the bridge systems subjected to strong earthquake motions.

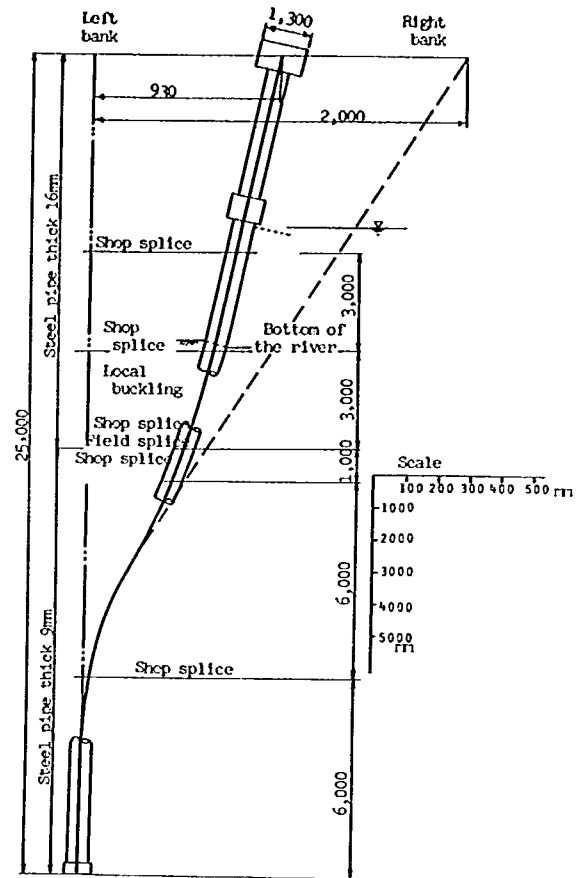


Fig. 25. Permanent Deformation of a Pile Pulled out at the Showa Bridge



Fig. 26. Damage to a Pulled-out Pile at the Fourth Pier of the Showa Bridge

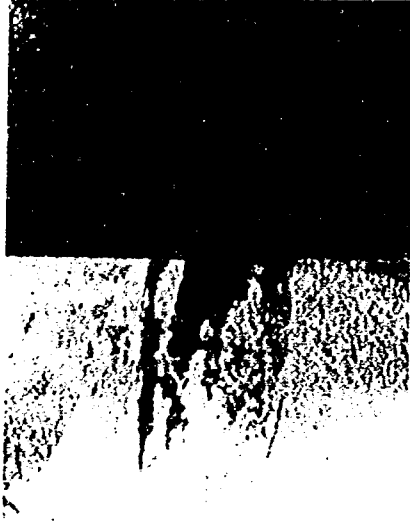


Fig. 27. Buckling Feature of the Pile Shown in Fig. 26

Fig. 28 shows the outline of pier 5 for both before and after the earthquake. As shown in the figure, the pier was reinforced by adding nine piles (totally 18 piles in two rows) after the earthquake. The figure also illustrates some results of insitu tests and assessment of soil liquefaction potential. N-values denote the number of blows obtained by the standard penetration test procedure. For the shallow depth N-values for mostly sandy soils are around 10 or less, which suggests the soils are very vulnerable to liquefaction. F_L -values, so called the liquefaction resistance factor, indicate an ability to resist liquefaction occurrence, and is defined by

$$F_L = \frac{R}{L} \quad (1)$$

where, R is the insitu resistance (or undrained cyclic strength) of a soil element when subjected to dynamic loads during an earthquake, and L is the dynamic load induced in the soil element by a seismic motion.

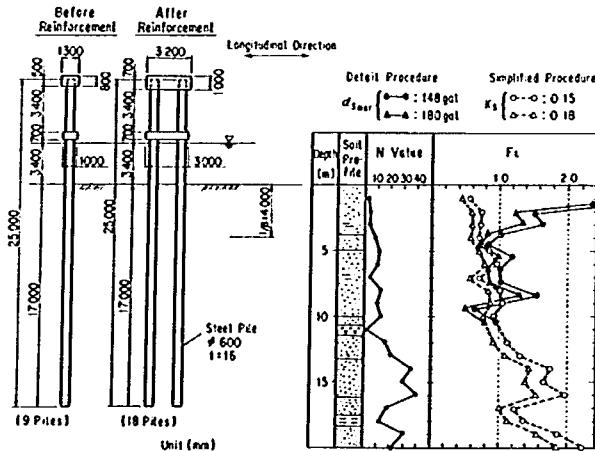


Fig. 28. The Substructure (P5) of the Showa Bridge and the Results of Assessment of Soil Liquefaction Using F_L -Values

R in Eq. (1) will be obtained either by dynamic triaxial tests for undisturbed specimens taken from the field (in the case of the detailed procedures), or the following empirical equations (in the case of the simplified procedure)

$$R = \begin{cases} 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.225 \log_{10} \left(\frac{0.35}{D_{50}} \right) & 0.04 \leq D_{50} \leq 0.6 \text{ mm} \\ 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} - 0.05 & 0.6 \leq D_{50} \leq 2.0 \text{ mm} \end{cases} \quad (2)$$

where, N : the number of blows of the standard penetration test
 σ'_v : effective overburden pressure (in kgf/cm^2)
 D_{50} : mean particle diameter (in mm)

On the other hand, L can be evaluated either by a dynamic response analysis of the soil ground (in the case of the detailed procedure), or the following empirical formula (in the case of the simplified procedure):

$$L = k_s \cdot \frac{\sigma_v}{\sigma'_v} \cdot r_d \quad (3)$$

where, k_s : seismic coefficient at ground surface, defined by the ratio of acceleration of ground surface to the acceleration of gravity,
 σ_v : total overburden pressure (in kgf/cm^2)
 r_d : reduction factor of dynamic shear stress to account for the ground deformation.

Normally $r_d = 1.0 - 0.015 z(z$: depth in m) is proposed. When the factor F_L at a certain depth is less than 1.0, the soil is judged to be likely to liquefy during a given earthquake. The right side of Fig. 28 illustrates the results of the calculation of F_L -values by two procedures, namely the detailed one and the simplified one. From this it is seen that liquefaction is likely to have taken place for the shallow layer up to the depth of about 10m.

Earthquake responses of the Pier 5 under the conditions for both before and after the earthquake are calculated using the conventional seismic coefficient method. The seismic coefficient (k_h) is taken as 0.15, 0.2, and 0.25. In the calculation, the coefficient of lateral subgrade reaction, k is taken 1.0 kgf/cm^3 as basic value without the effects of soil liquefaction. Reduced values of k (0.67 , 0.33 kgf/cm^3 and zero) are also taken to account for the decrease of subgrade reaction due to the effects of soil liquefaction.

Fig. 29 summarizes the relationship between the calculated maximum displacement at the crest of Pier 5 (in cm), and the coefficient of lateral subgrade reaction, and the reduction factor D_e . From this figure, it is found that the displacement of the pier increases according to the decrease to k -values. It is also seen that the displacement after the reinforcement work (after the earthquake) is considerably smaller comparing with the one for the original structure before the earthquake.

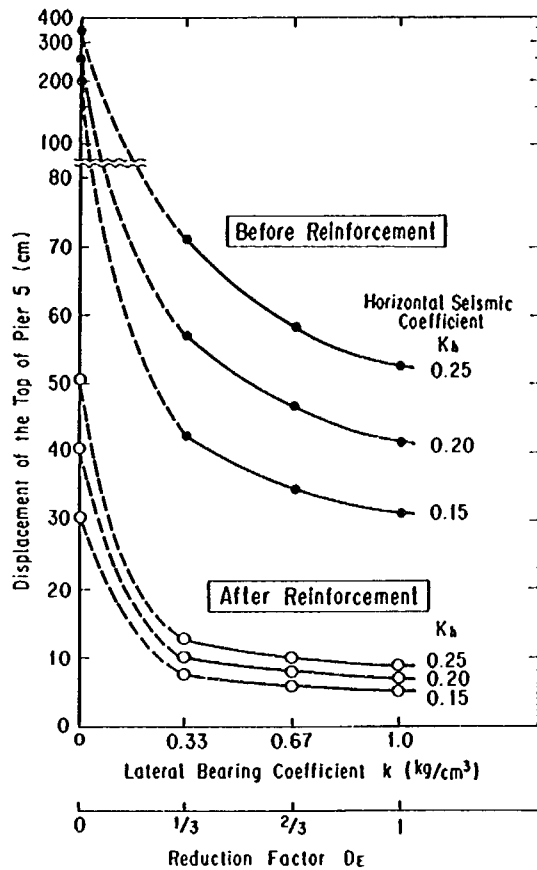


Fig. 29. Relationships between the Maximum Displacement of the Top of Pier 5 and the Reduction Factor D_e or the Lateral Bearing Coefficient of Grounds k

5. MIYAGI-KEN-OKI EARTHQUAKE OF 1978 (Magnitude = 7.4)

Outline

On June 12, 1978 a strong earthquake took place under the sea bottom approximately 120km east of Sendai City, Miyagi Prefecture. The Japan Meteorological Agency has reported a magnitude of 7.4 on the Richter scale and the focal depth of 30km. This earthquake caused very severe damage to various engineering structures including bridge structures. Due to the earthquake more than 100 highway bridges sustained structural damages. It should be also noted that another earthquake ($M = 6.7$) whose epicenter was approximately 60km north from the June earthquake had hit almost the same area on February, 20, 1978, and caused some minor damages to engineering structures.

Strong-Motion Records and Features of Bridge Damage

During the Miyagi-ken-oki Earthquake, a number of strong-motion seismographs triggered acceleration records at various stations in Hokkaido and northern part of Honshu. Fig. 30 shows a relationship between epicentral distance and peak acceleration on ground surfaces. Fig. 31 illustrates typical records (by SMAC-B2 type accelerographs) which were obtained at Kaihoku Bridge located about 80km west from the epicenter. Although very high accelerations (peak value of more than 500 gals in the longitudinal direction) were measured on the pier top, Kaihoku Bridge did not sustain any major structural damage. Only fixed bearing supports and oil dampers were slightly damaged to their anchor bolts.

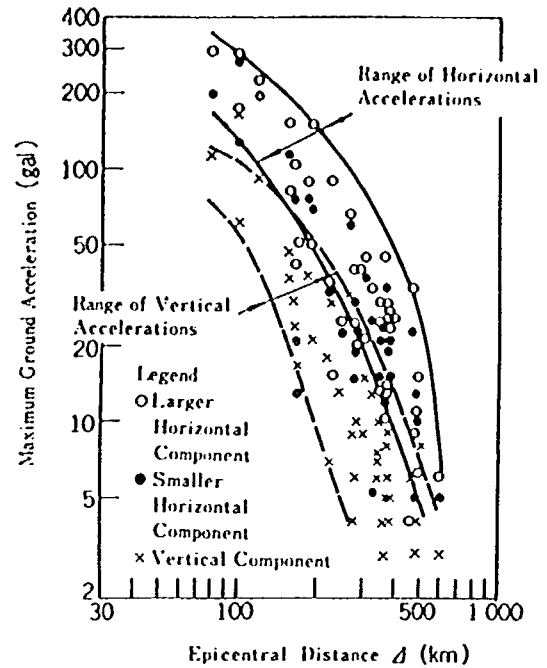


Fig. 30. Relation between Epicentral Distance and Maximum Ground Acceleration

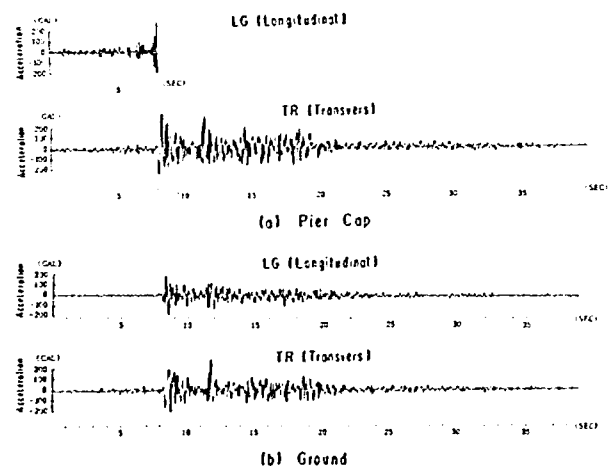


Fig. 31. Strong-Motion Records at Kaihoku Bridge Pier and Ground Nearby

Next, Fig. 32 shows a record at a pier cap of Date Bridge located near Fukushima City, Fukushima Prefecture (epicentral distance of 160km). The peak accelerations on the pier cap were 480 gals in the longitudinal direction and 320 gals in the transverse direction. This bridge suffered moderate damage to bearing supports and a truss member above a fixed bearing support.

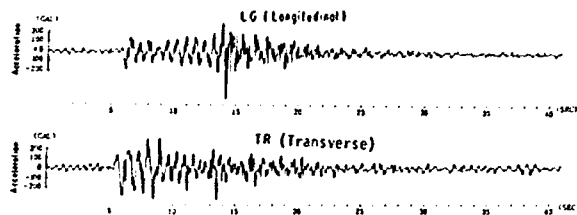


Fig. 32. Strong Motion Record at Cap of Pier 2, Date Bridge

Fig. 33 indicates locations of severely damaged highway bridges by black circles, places where soil liquefaction took place by white circles, and geological conditions in Miyagi Prefecture. It is seen from the figure that most of major bridge damage and liquefaction occurred in alluvial plains along large rivers such as Kitakami, Naruse, Yoshida, Natori, and Abukuma.

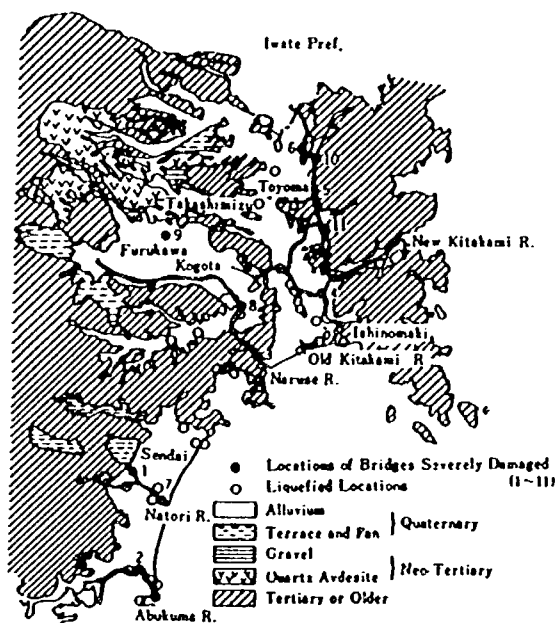


Fig. 33. Geological Features and Locations of Major Bridge Damages and Liquefaction Sites

Kin-noh Bridge

Kin-noh Bridge, completed in 1956, is located on National Highway No. 346, and crossing Kitakami River. The superstructures are single-span steel plate girder, 5-span simply supported steel trusses, and 9-span Gerber-type steel plate girders from the left bank to the right. The total length and the width are 575.5m and 6.0m, respectively. Substructures are reinforced concrete columns resting on caisson foundations for the truss spans, and reinforced columns resting on footing foundations with reinforced concrete piles for the Gerber plate girder spans. Soils are of soft silts and sands, and a firm sand layer exists approximately 30m below the ground surface. During the earthquake of June 12, 1978 one of suspended girders of this bridge fell down onto the river bed, as shown in Fig. 34. Kin-noh Bridge is only one bridge which completely fell down during the June Earthquake.

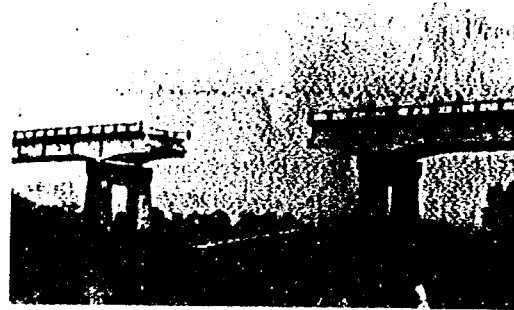


Fig. 34. Fall of a Suspended Girder, Kin-noh Bridge (During the Earthquake of June 12, 1978)

This bridge sustained damages three times by three different earthquakes, namely Northern Miyagi-Ken Earthquake of April 30, 1962 ($M = 6.5$, $\Delta = 15$ km), two Miyagi-ken-oki Earthquakes of February 20, 1978 ($M = 6.7$, $\Delta = 80$ km) and of June 12, 1978 ($M = 7.4$, $\Delta = 110$ km). Due to the 1962 Earthquake side blocks of bearing supports (oval line bearing) of the Gerber girder spans failed, and concrete near the fixed bearing supports on the right-bank abutment cracked. Following the 1962 Earthquake, a repairing work to add stiffening plates was undertaken at three piers (Piers-8, 9, and 10).

Anchor bolts of the bearing stiffening plates were sheared off during the February 1978 Earthquake. Side blocks of bearing supports, which did not sustain damages during the 1962 Earthquake were also failed during the February 1978 Earthquake. Most of bearing supports at the truss girders also failed, in addition to the failure of bearing supports at the Gerber girder spans. As for the truss spans, fixed bearing supports of pin-type were severely damaged at Pier-6. It seems that the bearing extensively rocked, rotated, and translated. Most of anchor bolts of fixed bearing supports on the other piers were also pulled out. Movable bearing supports of pin-roller-type were also failed.

Since the interval between the February 1978 Earthquake and June 1978 Earthquake was only four months, repairing works of the bearing supports caused by the February 1978 Earthquake were still undertaken at the time of June 1978 Earthquake. Accordingly, all the girders were easily movable without any restraints. A suspended girder between Piers-7 and 8 fell down on the river bed, as shown in Fig. 34. The superstructure moved toward the right-bank side by 55 cm on the top of Pier-8 (see Fig. 35). Since the upper shoe dislodged from the bearing support and the lower flange supported the dead weight of the girder, a local buckling took place at the girder web. All the Gerber spans between Pier-8 and the right-bank abutment moved toward the right. The girder moved about 10 cm toward the right on the right-bank abutment, and the end of girder collided with the parapet of the abutment. The asphalt pavement of the backfill heaved due to the collision.

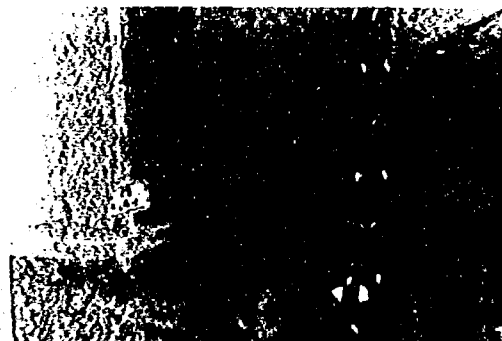


Fig. 35. Movement (55 cm) of a Plate Girder at the Downstream Support on Pier 8, Kin-noh Bridge (After June 12, 1978)

Furthermore, truss girders were also damaged due to the June 1978 Earthquake. Fig. 36 is a picture taken after the June 1978 Earthquake. Anchor bolts of the upstream fixed bearing at Pier-6 were severely pulled out as much as 20 cm, presumably due to rocking and translation motions of the bearing, and some concrete underneath the lower bearing plate crushed and the bearing sank by 2.5 cm. As for the downstream fixed bearing on Pier-6, a deformed bar which had been used as a temporary set bolt following the February 1978 Earthquake was sheared off again. The key of the upper shoe dislodged from the sole plate, and the sole plate deformed. As for pin-roller-type movable bearings, rollers had rolled out of the shoes during the February 1978 Earthquake. Fig. 37 shows a state following the June 1978 Earthquake at the upstream movable bearing on Pier-5 whose rollers completely rolled out. As for pier columns, the right-bank side of Pier-8 sustained heavy cracks. It is supposed that these cracks would have taken place when the superstructure collided with the right abutment and the reaction toward the left bank strongly acted to the pier.



Fig. 36. Pull-out of Anchor Bolts and Settlement of the Shoe at the Upstream Fixed Bearing on Pier 6, Kin-noh Bridge. (After June 12, 1978)



Fig. 37. Failure of Upstream Movable Bearing on Pier 5, Kin-noh Bridge (After June 12, 1978)

The superstructures for the Gerber-type spans were completely removed after the earthquake, and substructures and superstructures were newly constructed for the Gerber-type spans. Static lateral loading tests and forced vibration tests were conducted for piers which did not sustain any structural damages.

Yuriage Bridge

Yuriage Bridge, completed in 1972, is crossing over Natori River near its estuary. The superstructures are of 3-span continuous pre-stressed concrete box girders

with a center hinge and of 7-span simply-supported post-tension pre-stressed concrete T-shape girders. The bridge has the total length of 541.7 m and the width of 8 m. Two abutments are on steel pile foundations, two piers in the lower river bed are on caisson foundations, and 7 piers are on well foundations. Due to the June 1978 Earthquake, the nine pier columns sustained cracks mostly at the level of the ground surface. First pier from the left bank (Pier-1) received numerous heavy cracks as shown in Fig. 38 and concrete pieces separated from the column. Stoppers of simple-roller-type movable bearing on Pier-1 were damaged, guide pieces of the bearing failed, and the roller almost rolled down from the shoe.

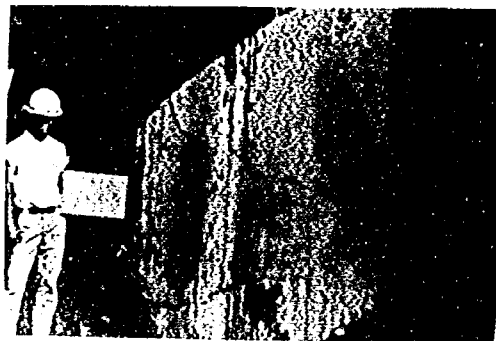


Fig. 38. Cracks at Pier 1, Yuriage Bridge

A simply-supported prestressed concrete beam on Pier-6 moved 6 cm toward the downstream. The ends of one handrail inserted into the end of another handrail above a pier. The length of insertion was 8 cm. During the earthquake the handrail ends completely came out of the adjacent handrail ends. It is understood that the two adjoining beams vibrated relatively at least 8 cm in the longitudinal direction to the bridge axis.

As shown in Figs. 39 and 40, sands spouted from ground surface cracks at numerous points near Pier-5 and between Pier-7 and 9. At these points the ground settles about 10 cm relatively to the piers. In situ soils investigations (see Fig. 40), laboratory tests for undisturbed specimens, and liquefaction assessment analyses were conducted after the earthquake. It was revealed that the subsoils of sands are loose near the surface and median to dense underneath. A hard layer exists 70 m below the surface. The results of liquefaction assessment are illustrated in Fig. 41.



Fig. 39. Sand Boils from Cracks on Ground near Pier 8, Yuriage Bridge (Natori River)

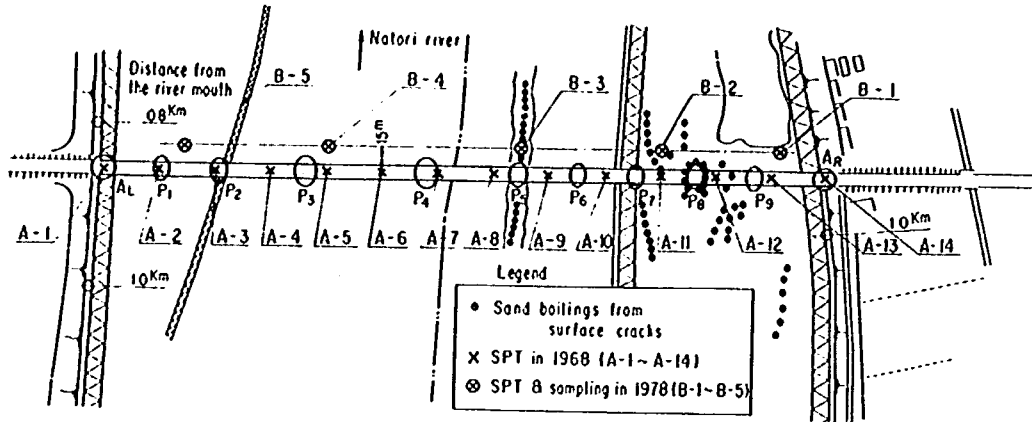


Fig. 40. Points of Sand Boils and Insitu Soil Tests at Yuriage Bridge

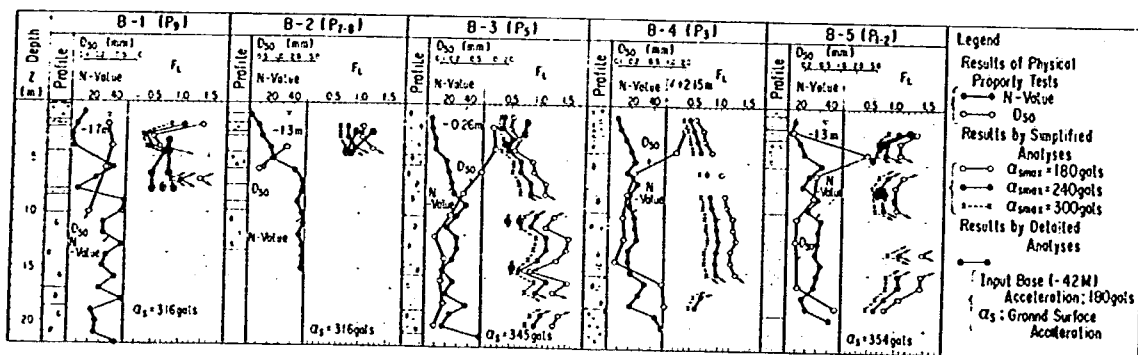


Fig. 41. F_L Values at Yuriage Bridge (B-1, B-2, B-3; Liquefied, B-4, Liquefaction Unconfirmed, B-5; Non-Liquefied)

In the figure F_L values, factor of liquefaction resistance as explained in the previous section, are estimated at five points, namely B-1, B-2, and B-3 (sand boils observed), B-4 (sand boils unconfirmed because of water stream), and B-5 (no sand boils). Two procedures, the detailed method and the simplified method, were employed in the liquefaction assessment analyses. In the detailed method, the acceleration record obtained at a dam site close to the bridge was selected. The peak acceleration value of 180 gals was taken at the level of 42m depth. On the other hand, in the simplified method, three values of the peak ground acceleration were assumed to be 180, 240, and 300 gals.

It is seen from Fig. 41 that F_L values less than 1.0 distribute nearly at the depth of 2 to 6m (at B-1 point), 2 to 5m (at B-2 point), 1 to 8m (at B-3 point), 1 to 7m (at B-4 point), and 3 to 9m (at B-5 point). F_L values are greater than 1.0 near the ground surface at B-5 point. It is supposed that at B-1, B-2, B-3, and B-4 points, liquefaction took place at the layers from the surface to the above-mentioned depths, and that at B-5 point liquefaction did not occur near the surface (up to about 3m), but liquefaction might have occurred at a deeper layer (between 3 to 9m). Sand layers with lower values of F_L well correspond to the layers of sand boiling.

6. URAKAWA-OKI EARTHQUAKE OF 1982 (Magnitude = 7.1)

6.1 OUTLINE

On the morning of March 21, 1982, a severe earthquake with the Richter magnitude of 7.1 hit off Urakawa Town, southern part of Hokkaido Island in Japan. From the report (1982) of the Japan Meteorological Agency (JMA), the characteristics of the earthquake are outlined as follows:

- 1) Event Time : 11:32 am, March 21, 1982
- 2) Magnitude : 7.1 on the Richter scale
- 3) Epicenter : 20 km off Urakawa Town (142°36' E, 42°04' N)
- 4) Focal Depth : 40 km
- 5) Seismic Intensity (JMA Scale):
 - 6-Urakawa
 - 4-Tomakomai, Sapporo, Otaru, Iwamizawa, Hiroo, Kuchan, Obihiro
 - 3-Kushiro, Asahikawa, Muroran, Hakodate, Aomori, Morioka

Fig. 42 illustrates the epicenter and JMA seismic intensities at various locations. Fig. 43 is a detailed map showing the epicenter of the main shock, epicenters of numerous aftershocks, determined from a densely instrumented network of Science Department, Hokkaido University (1982). It is seen from Fig. 43 that the epicenter (the center of the largest Octagon) is located off Mitsuishi Town about 15 km west from Urakawa.

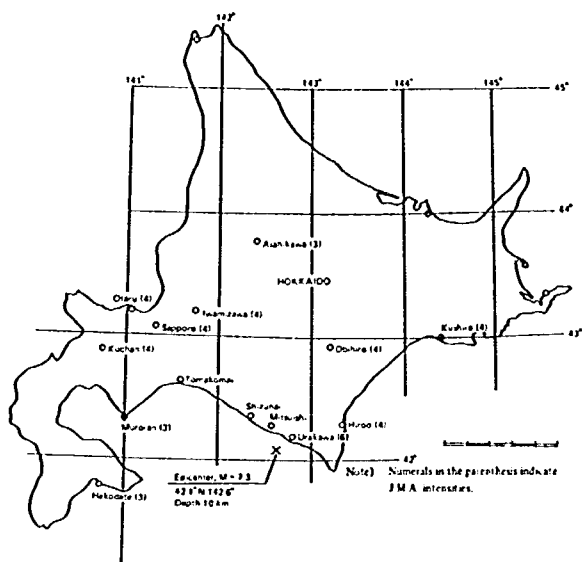


Fig. 42. Epicenter and JMA Intensities, Urakawa-oki Earthquake of March, 21, 1982.

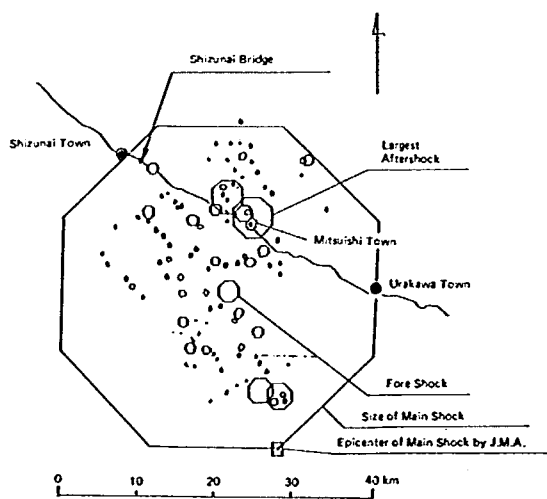


Fig. 43. Epicenters of Fore Shock, Main Shock and Numerous Aftershocks (After Science Department, Hokkaido Univ.)

Strong-motion accelerographs triggered time histories of accelerations at several locations. The locations and peak ground surface accelerations are shown in Figs. 44 and 45. The highest acceleration (about 300 gals) was recorded at Hiroo about 60 km east from the epicenter. At the surface of a firm ground near Horoman Bridge located about 40 km east from the epicenter, the peak acceleration was about 100 gals. An average value of peak accelerations at Sapporo City of the epicentral distance of 150 km was about 70 gals.

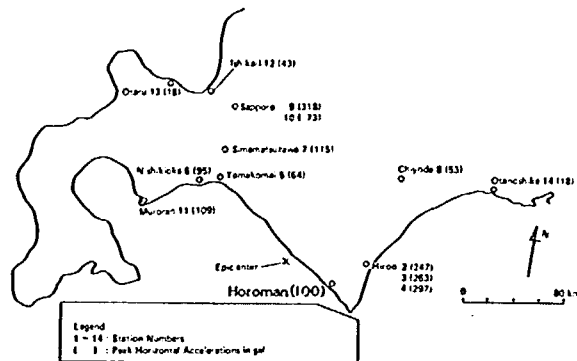


Fig. 44. Strong Motion Stations Recorded and Peak Horizontal Accelerations on Ground

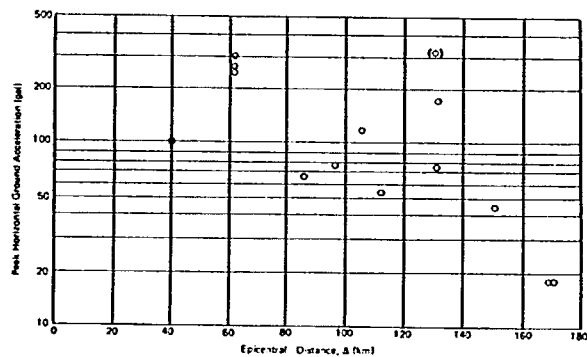


Fig. 45. Epicentral Distance Versus Peak Horizontal Ground Acceleration (Δ is measured from the center of octagon of Fig. 43.)

6.2 DAMAGE FEATURES OF SHIZUNAI BRIDGE

Pier columns of Shizunai Bridge located near the estuary of Shizunai River, were seriously damaged. It is seen from Fig. 43 that Shizunai Bridge is sited at the north-

ern edge of the focal area. General view of Shizunai Bridge is illustrated in Figs. 46 and 47. The bridge, completed in 1972, consists of 3 of 3-span continuous steel plate girders (9 spans totally), 2 abutments, and 8 piers.

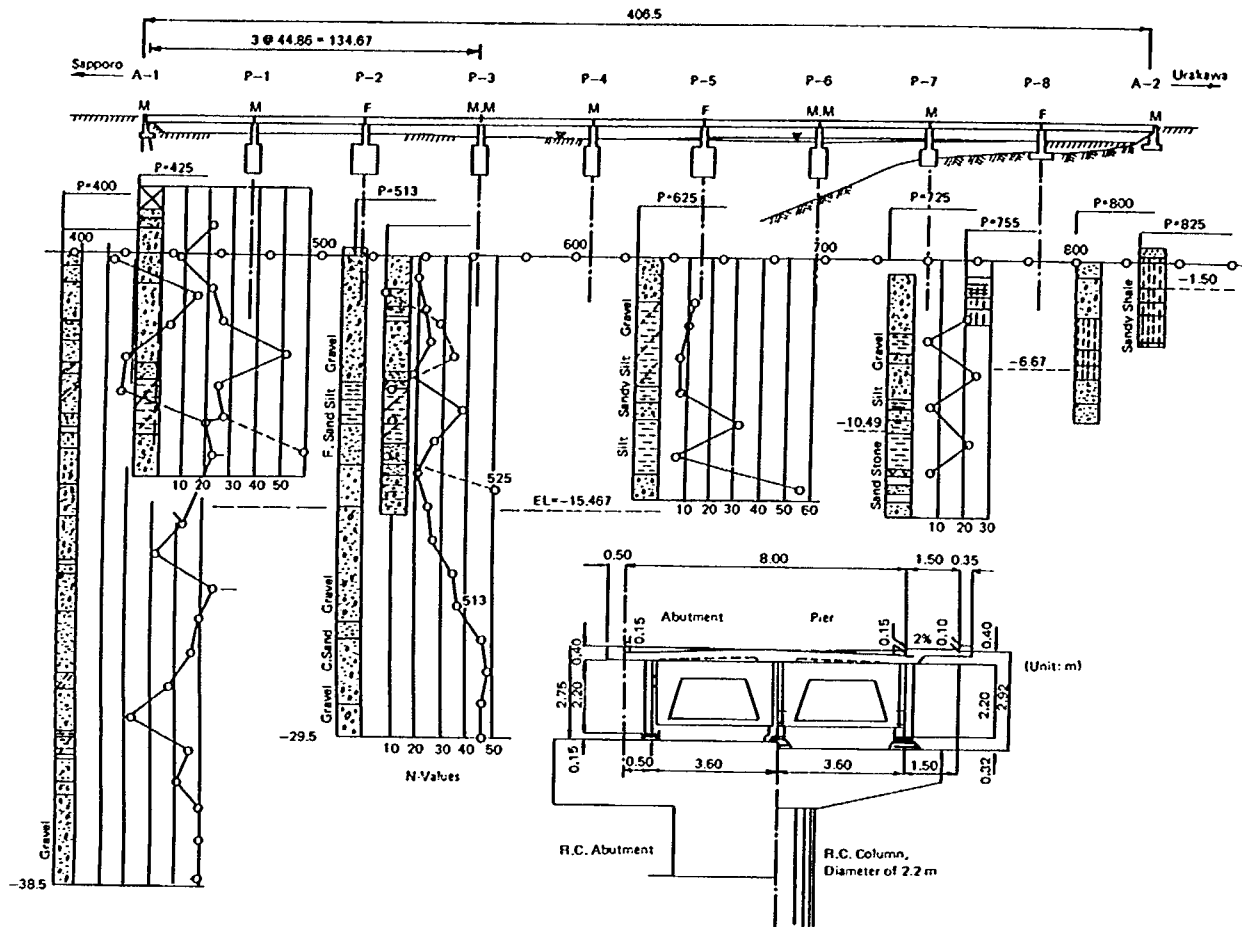


Fig. 46. General View, Cross Section, and Soil Profiles of Shizunai Bridge

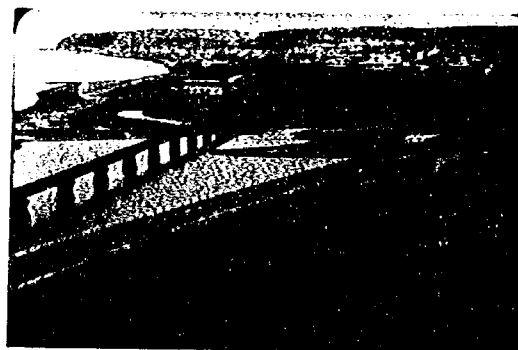


Fig. 47. View of Shizunai Bridge

Soil conditions at the bridge site are shown in Fig. 46. Soft soils are deeper at the right-bank side (or Abutment-1 side), and the mid-span part. Soft soils are very shallow at the left-bank side (or Abutment-2 side). A bedrock layer (mudstone of Tertiary Era) is seen on the surface near the site of Abutment-2. Pier-8 and Abutment-2 have spread foundations resting directly on the mudstone layer.

Four piers (Piers-1, 3, 4, and 6) have well foundations of diameter of 6 m, and embedment depth of 12 m, two piers (Piers 2 and 5 - fixed piers) have well foundations of diameter of 10 m and embedment depth of 12 m, one pier (Pier-7) has a well foundation of diameter of 6.5 m and embedment depth of 7 m, and the final pier (Pier-8) has a square spread foundation of 10 m x 10 m. Abutment-1 on the right-bank has a pile foundation.

All of the eight piers have reinforced concrete circular columns of a diameter of 2.2 m and height of 8.1 m. As for Piers-2 and -5 which have fixed bearing supports in the longitudinal direction to the bridge axis, diameters become greater at the lower sections.

Due to the Urakawaoki-Earthquake, two pier columns (Piers-3 and 6) sustained very heavy cracks. Especially Pier-3 almost failed as seen from Figs. 48 and 49. Fig. 50 shows a damage feature of Pier-6. The lower sections of the column under the water sustained diagonal cracks similar to the case of Pier-3. Figs. 51 and 52 indicate damages to Piers-2 and 4, respectively. Separations of concrete pieces and cracks are seen at higher sections of the columns. Diagonal cracks are rather minor for Piers-2 and 4.



Fig. 50. Damage to Pier-6 of Shizunai Bridge, Seen from Pier-7

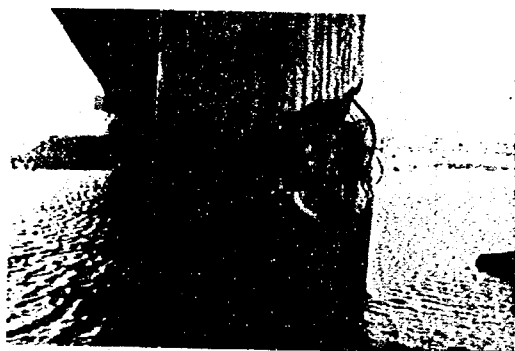


Fig. 48. Damage to Pier-3 of Shizunai Bridge, Seen from Pier-2 Site



Fig. 51. Damage to Pier-2 of Shizunai Bridge, Seen from Upstream of Pier-1 Side



Fig. 49. Closest View of Failed Section of Pier-3 of Shizunai Bridge, Seen from Downstream (Note cut-off of bars)



Fig. 52. Damage to Pier-4 of Shizunai Bridge

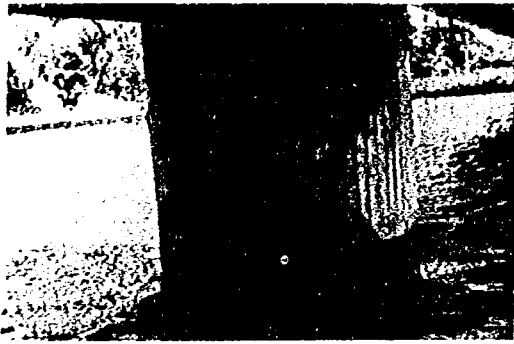


Fig. 53. Damage to Pier-5 of Shizunai Bridge, Diagonal Cracking on Downstream Side

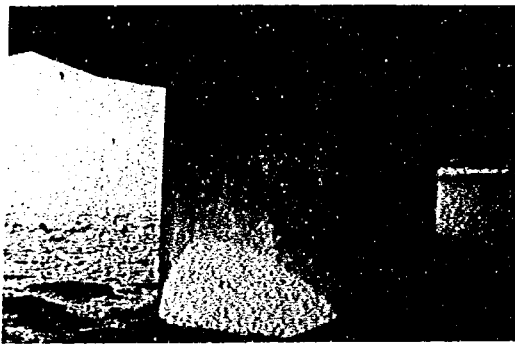


Fig. 54. Damage to Pier-7 of Shizunai Bridge, Light Horizontal Cracking

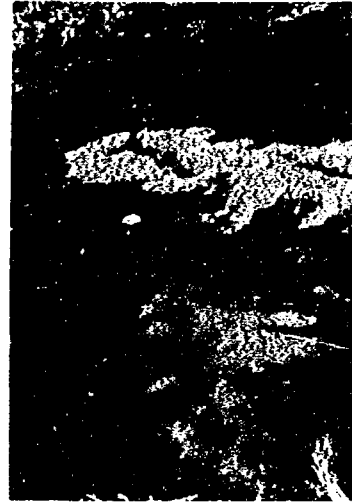


Fig. 55. Sand Boils on Ground near Pier-1, Shizunai Bridge

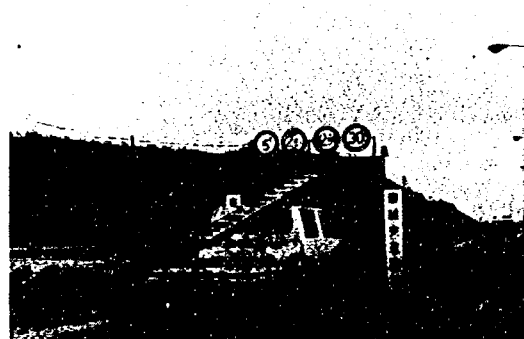


Fig. 56. Several Restrictions Such As a Speed Limit

Figs. 53 and 54 show minor damages to Piers-5 and 7, respectively. Horizontal and diagonal cracks are seen for Pier-5, and a horizontal crack is seen Pier-7. Fig. 55 shows an evidence of sand boilings observed near Pier-1, which received cracks at the base of the column under the ground surface.

Although seven piers (Piers-1 to 7) sustained minor to heavy cracks, Pier-8 did not receive any damage.

Quick repairing works for Piers-2, 3, and 6 completed by the middle of April, and one-lane of the bridge reopened on April 15 for light traffic (lighter than 5 tons) with several restrictions such as a speed limit (see Fig. 56). It took six months to completely repair all the damages, and the bridge eventually reopened on October 1, 1982 for all vehicles without any restrictions.

6.3 RESPONSE ANALYSIS FOR OVERALL SYSTEM OF SHIZUNAI BRIDGE

Objectives

In order to clarify earthquake response of each part of structural members of Shizunai Bridge, a dynamic response analysis was conducted for its overall system consisting of superstructures, substructures, and surrounding soils. In the analysis structural members were assumed to behave as elastic materials, although the surrounding

soils were assumed to have nonlinear properties. A principal aim of the analysis was to compare the differences of seismic responses for the eight piers, and to grasp the causes of the differences of damage extents for the eight piers.

Input Motions

Two seismic bedrock motions were employed as inputs for the dynamic analysis. First one was an acceleration record (peak value being 100 gals in one of horizontal components) obtained on the ground surface at the site of Horoman Bridge (epicentral distance being 36km north east from the epicenter) during the Urakawa-oki Earthquake of March 21, 1982. The second one was the acceleration record (peak value being 37 gals in one of horizontal components) obtained under the ground (40m below the surface) at the site of Shizunai Bridge (epicentral distance being 17km) during the Southern Hokkaido Coast Earthquake of January 23, 1981 (Magnitude of 7.1 and focal depth of 130km). Fig. 57 (A) shows locations of the epicenters of the above two earthquakes together with the two recording sites. The time history of the acceleration wave form and the response spectral curves of the second record (Shizunai Bridge Record) are shown in Fig. 57 (B) and (C). Unfortunately accelerograms at the site of Shizunai Bridge were not obtained, on March 21, 1982, because the accelerographs had been out of order and were just under repairing during the Earthquake of March 21, 1982.

In the dynamic analysis described below, acceleration amplitudes were adjusted in the manner that the peak surface motion becomes about 300 gals. The peak bedrock acceleration was taken as 180 gals in each of the two records. A time interval was taken as 0.01 s in the analysis. Only the results of analysis for the case of Shirunai Bridge Record will be described in the following.

Response Analysis of Grounds

A non-linear analytical computer program, named "NONSOIL" (1980) developed at the Public Works Research Institute, using Ramberg-Osgood models, was employed in the analysis of soil layers at the site of Shirunai Bridge.

The bedrock for the analysis was taken as the upper boundary of the Tertiary mudstone with the shear wave velocity of 520 m/s. From the results of soil surveys (measurements of P-wave and S-wave velocities), it is disclosed that a flat bedrock line exists about 46 m below the surface between the right bank (Abutment-1) site and Pier-6 site. On the other hand, from Pier-6 to Abutment-2 the bedrock line goes up to the surface. At the left bank (Abutment-2) site an outcrop of the bedrock appears on the surface.

In view of the above features of the bedrock line, a seismic analysis for soil layers was conducted for grounds between Abutment-1 and Pier-7. Fig. 58 illustrates a classification of soil layers considered in the analysis. In this figure, an analytical model for structural members is also shown. Table V tabulates shear wave velocities from insitu measurements for these soil layers. In the analysis, shear moduli and hysteretic damping constants were determined as functions of strain levels (non-linear properties), which were obtained from the results of dynamic torsional shear tests for soil specimens sampled from the field.

As a result of the analysis, Table VI shows predominant periods of vibration at several soil layers. Predominant periods for deeper soils at points between Abutment-1 and Pier-4 are about 0.7 to 1.0 s, those for shallow soils at points of Piers-5 and 6 are about 0.5 s, and that of a very shallow soil at Pier-7 is about 0.2 s. It is estimated that predominant periods at points of Pier-8 and Abutment-2 might be 0.2 s or shorter.

Fig. 59 indicates a distribution of response accelerations of grounds, when a motion of Shirunai Bridge record was input. The peak accelerations on the ground surface become 300 to 400 gals. The displacements on the ground surface (relative displacements to the bedrock) are 4.8 cm at Abutment-1 site, 3.1 cm at Pier-1 site, 3.3 cm

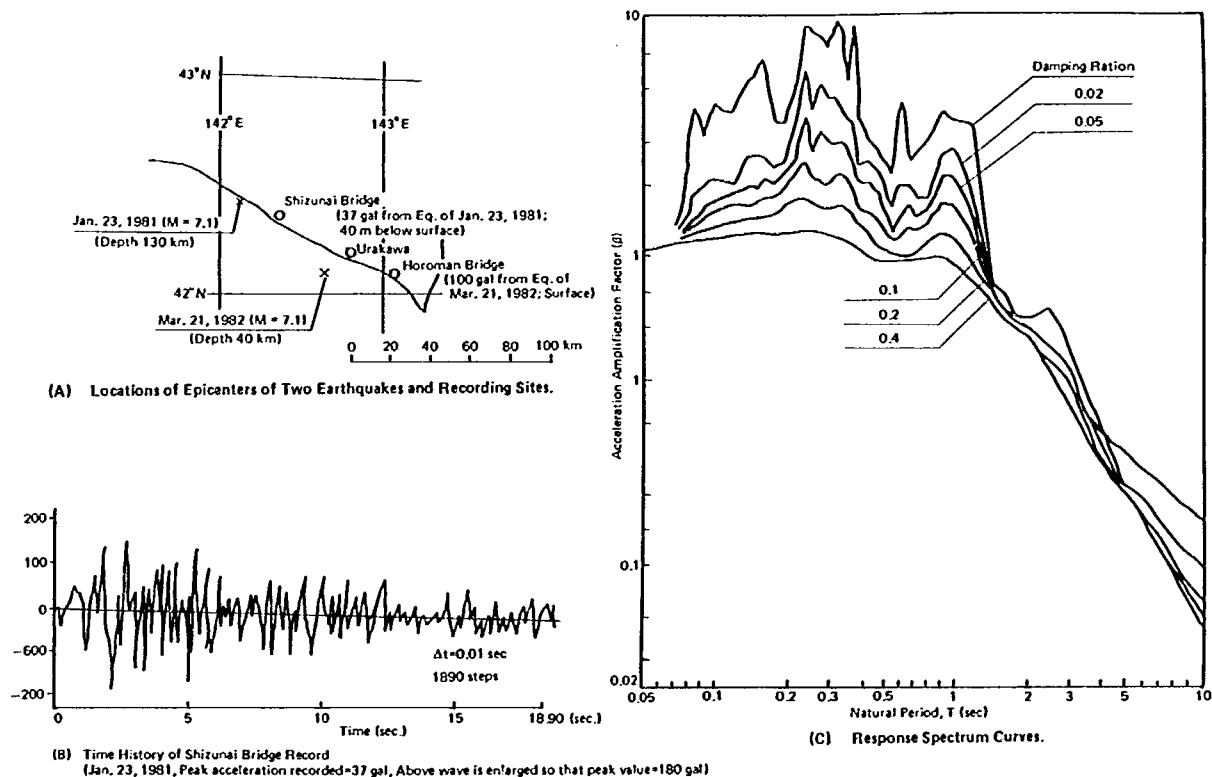


Fig. 57. Shirunai Bridge Record and Response Spectrum Curves

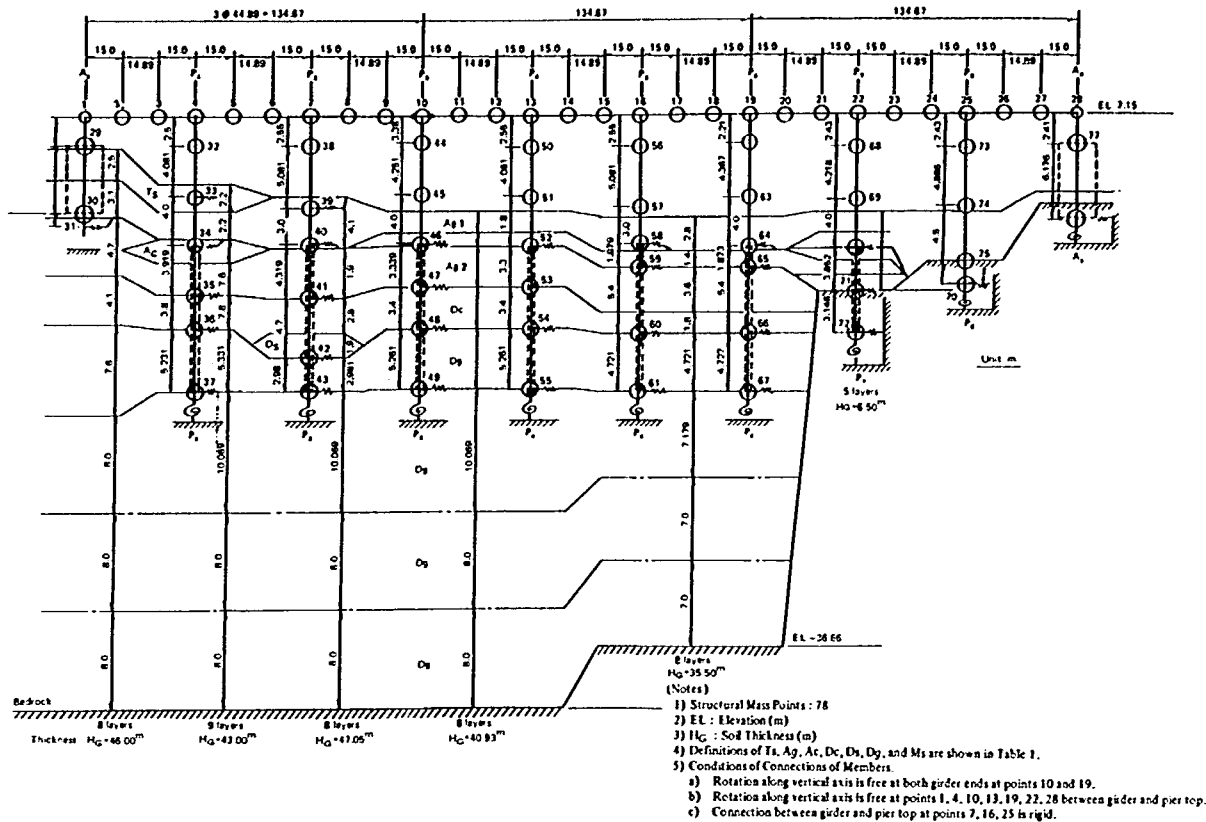


Fig. 58. Analytical Model of Overall System of Shizunai Bridge

Table V. Shear Wave Velocities of Soil Layers

Symbol of Soil Layers	Soil Classification	Shear Wave Velocities Measured (m/sec)
T_s	Top Soil	150
A_{g-1}	Alluvial Gravel (Upper)	170
A_c	Alluvial Clay	220
A_{g-2}	Alluvial Gravel (Lower)	250
D_c	Diluvial Clay	200
D_s	Diluvial Sand	200
D_g	Diluvial Gravel	400
M_s	Tertiary Mudstone	520

Note) See Fig. 7 for depth of each soil layer

Table VI. Predominant Periods at Various Sites

Sites	A1	P1	P2	P3, P4	P5, P6	P7
Predominant Period (ω)	1.00	0.75	0.75	0.68	0.47	0.23

at Pier-2 site, 2.6cm at Pier-3 to 6 sites, and 0.5cm at Pier-7 site. Furthermore, shear strains developed in ground are rather large in the diluvial clayey soils (D_c -layer) at sites between Abutment-1 and Pier-6, and the peak strain level becomes $(3.4 \text{ to } 4.9) \times 10^{-3}$. In the analysis, shear moduli of soils are taken smaller for high shear strain, according to the results of laboratory soil tests conducted for insitu soil specimens. Shear moduli for the shear level of $(3.4 \text{ to } 4.9) \times 10^{-3}$ become only about 10 percent of the original value of shear moduli G_0 ($= \rho V_s^2$ for shear strain of 10^{-6}). It is estimated from the field soil survey that the effects of soil liquefaction (observed near Pier-1) might have minor effects on the dynamic behavior of the ground and the bridge structures, because the thickness of a liquefied layer was disclosed very shallow.

Response Analysis of Bridge System

A response analysis for an overall system of Shizunai Bridge is also conducted in the transverse direction of the bridge axis, with use of a linear computer program named "DAST-M" (1980) developed at the Public Works Research Institute, in which multiple-point motions given from the results of the analysis for the soil layers (described in the previous section) can be considered as input motions to the bridge system. A model for the overall system is shown in Fig. 58. As shown in Fig. 58, although the effects of ground only below the top of the well foundations are considered for most piers, the effects of grounds above the top of the well foundations are taken into account for Piers-1 and 2, because the embedment of soils are deeper at the two pier sites.

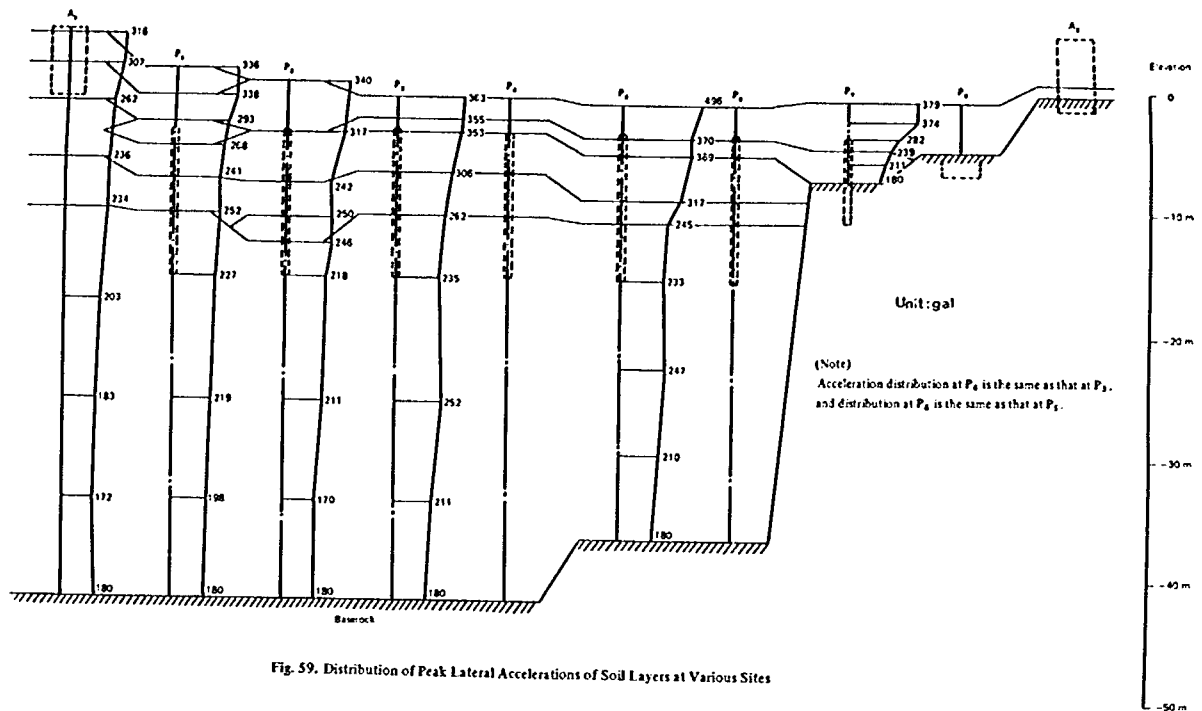


Fig. 59. Distribution of Peak Lateral Accelerations of Soil Layers at Various Sites

In modeling bearing supports which connect girders and pier crowns, rotations with respect to the longitudinal axis of the bridge and lateral displacements in the transverse direction are assumed to be identical at the two ends of the connections. In other words, the conditions of the bearing supports are assumed to be completely fixed for the rotations with respect to the bridge axis and the transverse displacements. On the other hand, rotations with respect to the vertical axis are assumed fixed at the fixed bearing supports (Piers-2, 5 and 8), and free at the movable bearing supports (Piers-1, 3, 4, 6 and 7, Abutments-1 and 2).

In the analysis, masses of girders, pier columns, and well foundations, torsional inertia of girders along the bridge axis, and rotational inertia of pier top beams along the bridge axis are considered. An equivalent viscous damping ratio of 20 percent to critical is assumed, accounting for the various damping effects.

From a characteristic value analysis (or an eigenvalue analysis) for the overall system shown in Fig. 58, the fundamental mode shape (or the first mode) is shown in Fig. 60. The fundamental natural period is about 0.6s, and thus the entire parts of the system move to the same direction. Bending deformations of pier columns prevail for the fundamental mode. The participation factor for the fundamental mode is most essential comparing with the other modes.

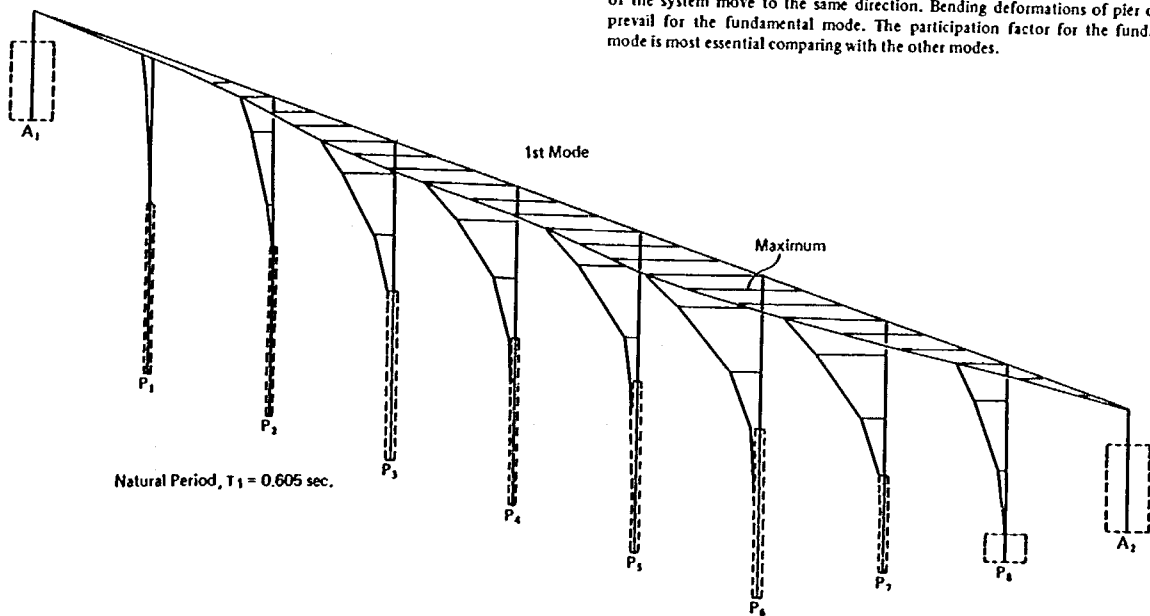


Fig. 60. Modal Shape of the 1st Mode for Overall System of Shizunal Bridge

Figs. 61 and 62 illustrate the distribution of peak response accelerations and peak response displacements, respectively, when Shizunal Bridge is subjected to the input motion of Shizunal Bridge record shown in Fig. 57 (the peak acceleration at the bed-rock is adjusted to be 180 gals), in the transverse direction to the bridge axis. Maxi-

mum response accelerations can be observed at pier tops and girders between Piers-1 and 6, and the peak acceleration value is very large (480 to 660 gals). Maximum response displacements at the sites between Abutment-1 and Pier-6 are also large (4.2 to 6.6 cm).

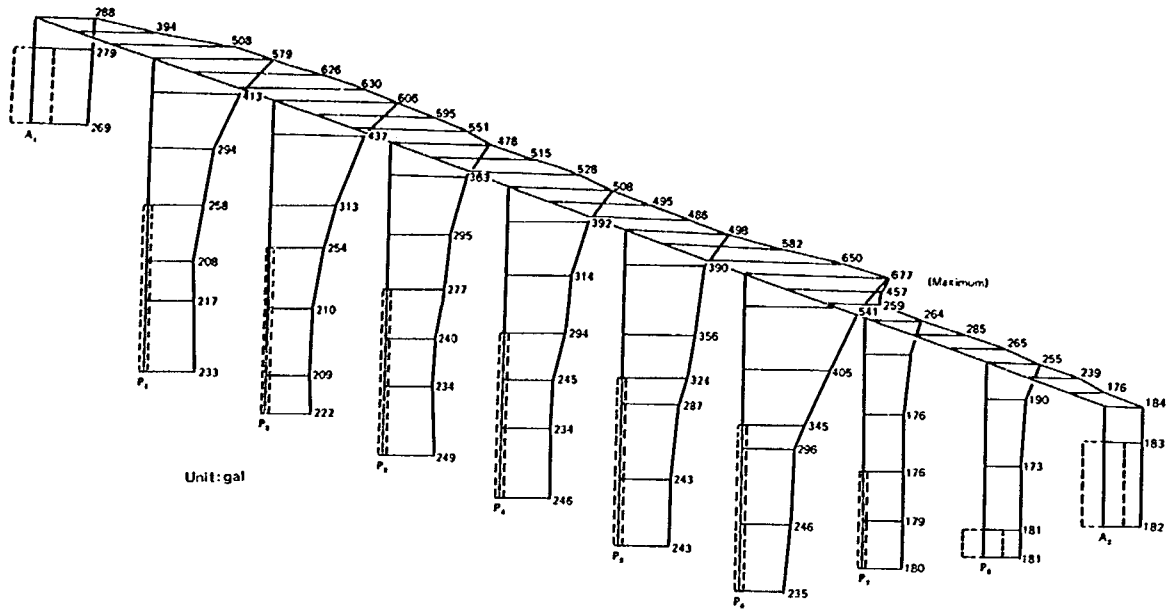


Fig. 61. Distribution of Peak Response Accelerations at Various Structural Points

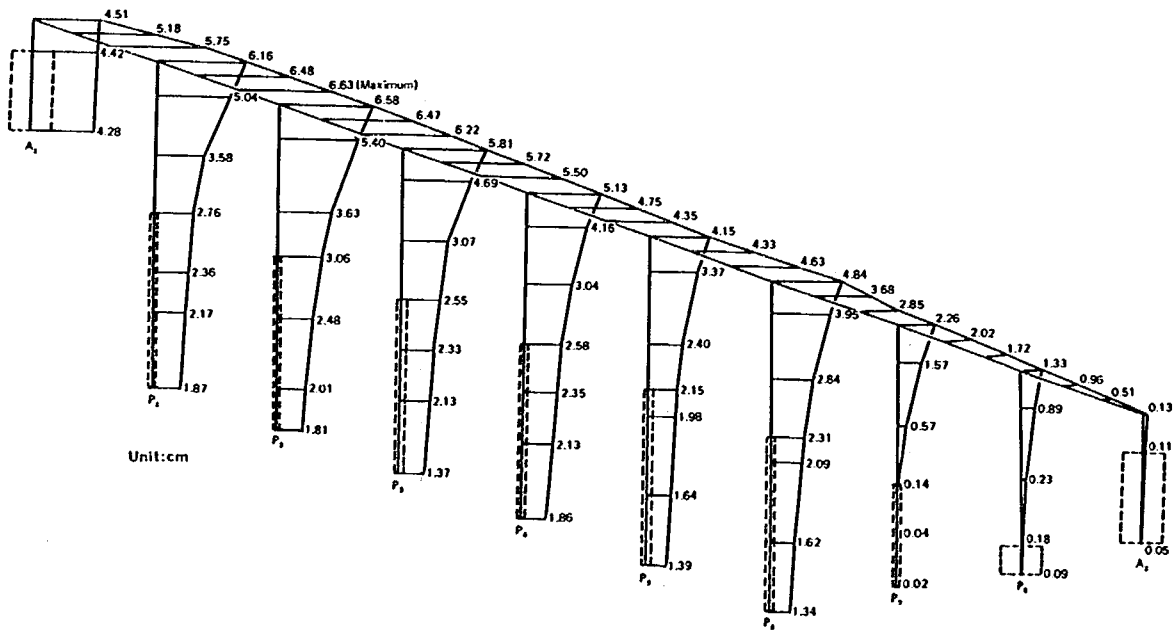


Fig. 62. Distribution of Peak Response Displacements at Various Structural Points

Next, Figs. 63 to 66 show the distributions of peak bending moments for all pier columns. In the figures, design bending moments (M_D), bending moments to cause initial cracks (M_C), bending moments to cause yielding (M_Y), and ultimate bending moments (M_U) are illustrated, as well as the peak response bending moments (M_{PI}) given for each pier from the dynamic analysis performed. It is seen from these figures that response bending moments (M_{PI}) are quite large ($M_{PI} > M_U$) at Piers-3 and 6 which sustained very heavy cracks during the Urakawa-oki Earthquake of

1982. Response bending moments at Piers-1, 2, 4, 5, 7 are nextly large ($M_{PI} > M_D$, and M_{PI} are nearly equal to M_U at the section of cut-off of reinforcing bars). Response bending moments at Pier-8, which did not receive any cracks, are small ($M_{PI} < M_D$). Peak response shearing forces are higher than shearing strengths at Piers-1, 2, 3, 5 and 6, although response shearing forces are less than shearing strengths at Piers-4, 7 and 8.

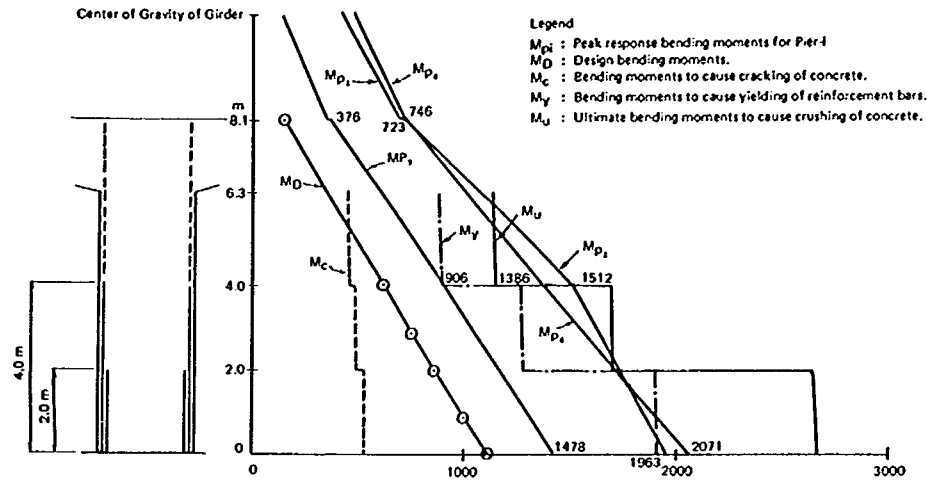


Fig. 63. Distributions of Peak Bending Moments for Pier-1, Pier-4, and Pier-7

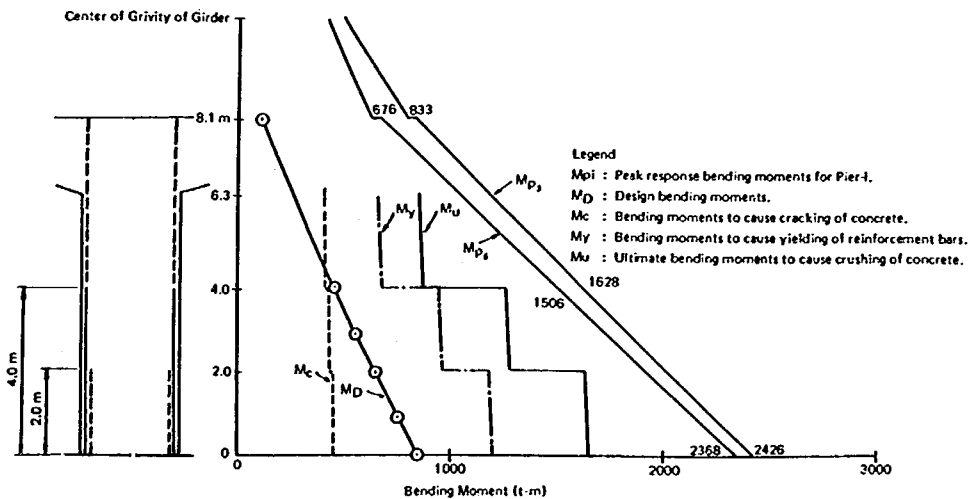


Fig. 64. Distributions of Peak Bending Moments for Pier-3 and Pier-6

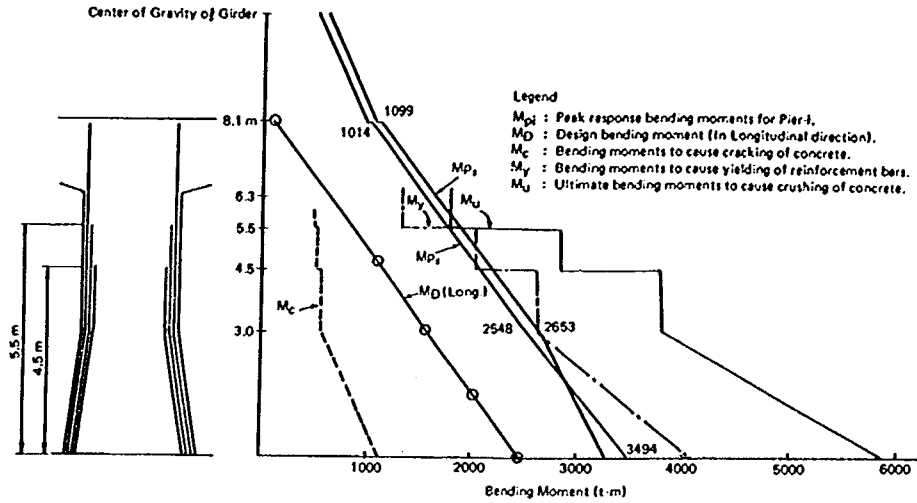


Fig. 65. Distributions of Peak Bending Moments for Pier-2 and Pier-5

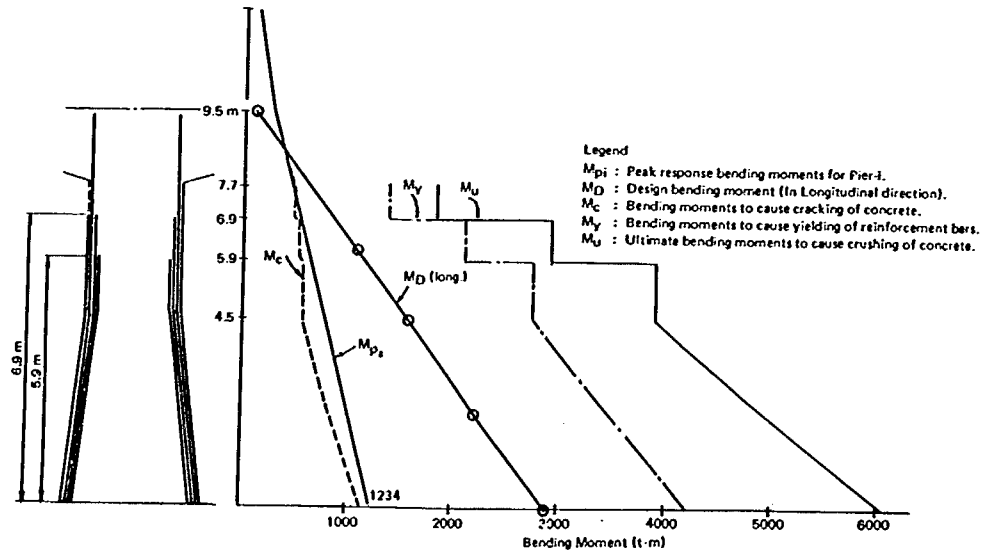


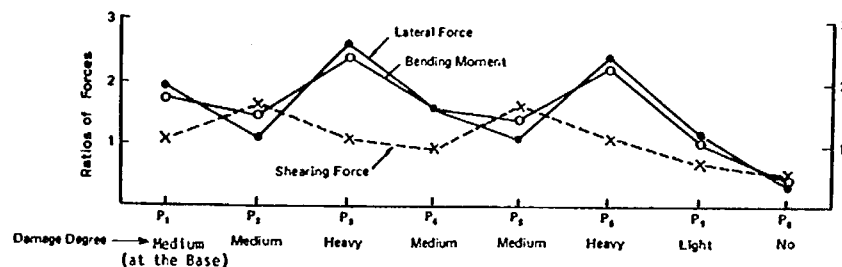
Fig. 66. Distributions of Peak Bending Moments for Pier-8

Table VII shows (A) comparison of design lateral forces and response lateral forces analyzed, (B) comparison of yielding bending moment and response bending moments analyzed, and (C) comparison of shearing strength at most dangerous sections and response shearing forces analyzed. A figure below the table shows the ratios of

response lateral forces to design lateral forces, the ratios of response bending moments to yielding bending moments, and the ratios of response shearing forces to shearing strengths for all the nine pier columns, indicating degree of damages during the Urakawa-oki Earthquake of 1982.

Table VII. Result of Dynamic Analysis for Overall System of Shizunai Bridge

Forces	Piers	P ₁	P ₂	P ₃	P ₄	P ₅	P ₆	P ₇	P ₈	Remarks
(A) Lateral Force at Pier Top	(1) Design (t)	102	(277)	74	102	(277)	74	102	(277)	Values in (): Longitud. Direction
	(2) Analysis (t)	195	300	192	159	301	176	114	91	
	(2)/(1)	1.91	1.08	2.59	1.56	1.09	2.38	1.12	0.33	
(B) Bending Moment at Critical Section of Pier Column	Yielding Moment (t·m)	890	1,325	677	890	1,325	677	890	1,371	=1.57 A
	(2) Analysis (t·m)	1,512	1,891	1,628	1,386	1,796	1,506	906	539	
	(2)/(1)	1.70	1.43	2.40	1.55	1.36	2.22	1.02	0.39	
(C) Shearing Force at Critical Section of Pier Column	Shearing Strength (t)	205	205	205	205	205	205	205	205	
	(2) Analysis (t)	221	327	220	183	326	217	132	103	
	(2)/(1)	1.08	1.60	1.07	0.89	1.59	1.06	0.64	0.50	



From the figure below Table VII the following can be pointed out. Ratios of response lateral forces to design lateral forces for Piers-3 and 6 which sustain heavy cracks, are very large (2.6 and 2.4). The ratios for Piers-1, 2, 4, 5, and 7 which received moderate to minor cracks, are between 1.1 and 1.5. The ratio for Pier-8 with no damage is very small (0.3). Ratios of response bending moments to yielding bending moments are also large for Piers-3 and 6, and small for Pier-8. The tendency is identical to the case of lateral forces. Furthermore, ratios of response shearing forces to shearing strengths are greater than 1.0 for Piers-1, 2, 3, 5 and 6, and less than 1.0 for Piers-4, 6, and 8.

It is obvious that some piers behaved extensively beyond the elastic limit. In the above, an elastic response analysis for the overall system of Shizunai Bridge are described. Another dynamic analysis has been conducted for Pier-3, which sustained the heaviest damage among the eight piers, by considering non-linear characteristics of reinforced concrete structures and surrounding soils. This analysis is done in order to accurately grasp dynamic behavior, including behavior after the outbreak of cracks. As well as the two different dynamic analyses, laboratory experiments are conducted for scaled models of Pier-3 of Shizunai Bridge. The details are described elsewhere (T. Iwasaki et al. 1983 and N. Narita et al. 1983).

6.4 SUMMARY

From the dynamic analysis of Shizunai Bridge damaged by the Urakawa-oki Earthquake of March 21, 1982 ($M = 7.1$), the following can be summarized.

- (1) It is revealed that Shizunai Bridge sustained seismic lateral forces much higher than the design forces. Especially for piers located in the right-bank side where deeper soft soils exist at the surface above the bedrock, earthquake responses were extensive. It should be noted that one of causes of the damage to these piers was an coincidence of periods of vibration between the predominant period of soils and the fundamental period of the bridge structure between Piers-1 and 6. In order to improve seismic design specifications for highway bridges, it seems essential to consider the effects of dynamic interactions between bridge structures and surrounding soils.
- (2) Ratios of response forces to design forces at pier tops and ratios of response bending moments to yielding moments are extremely high for Piers-3 and 6 which received heavy cracks during the Urakawa-oki Earthquake. The results of the analysis well coincide with the damage features of the eight piers.

- (3) Basing on the current Specifications of Seismic Design of Highway Bridges (1980), design lateral forces acting on piers which support ends of continuous girders (such as Piers-3 and 6 of Shizunai Bridge), become smaller than those acting on other piers, because the current Specifications require that design lateral forces are the product of the seismic coefficient and the dead loads from girders (or reactions) acting on piers, and reactions are smaller on the ends of continuous girders.

From the dynamic analysis for the overall system, however, response lateral forces generated in Piers-3 and 6, which are supporting ends of continuous girders, are not necessarily small. Therefore, the ratios of response lateral forces to design lateral forces become greater for Piers-3 and 6. This fact might be one of causes of heavy damage to selective pier columns of Shizunai Bridge.

- (4) Ratios of response shearing forces to shearing strengths are smaller than those of bending moments. From this fact it is supposed that cracks of pier columns of Shizunai Bridge might be caused firstly by bending moments at sections of cut-off of reinforcing bars, and secondly by shearing forces at the same sections where sectional area became smaller due to preceding bending failures. It seems that shearing forces eventually caused diagonal cracks and serious shear failures of the pier columns of Pier-3 and 6. In order to avoid these failures of reinforced concrete columns elongation of bars (at cut-off of reinforcing bars) should be carefully considered.

7. CENTRAL JAPAN SEA EARTHQUAKE OF 1983 (Magnitude = 7.7)

A large-size earthquake hit the coastal areas of Central Japan Sea, Akita and Aomori Prefectures, on May 26, 1983, registering a Richter magnitude of 7.7. The focal depth was rather shallow (about 5 km). Fig. 67 illustrates the location of the epicenter and the area of the aftershocks and the locations of highway bridges damages. Fig. 68 shows peak acceleration values observed by strong motion accelerographs. It is noted from this that peak accelerations are between 200 and 300 gals in the coastal areas of Akita and Aomori Prefectures. Extent of damages to highway bridges was rather moderate, although damages to soil structures (such as river dikes and road embankments) are very extensive due to the effects of liquefaction supporting sandy deposits.

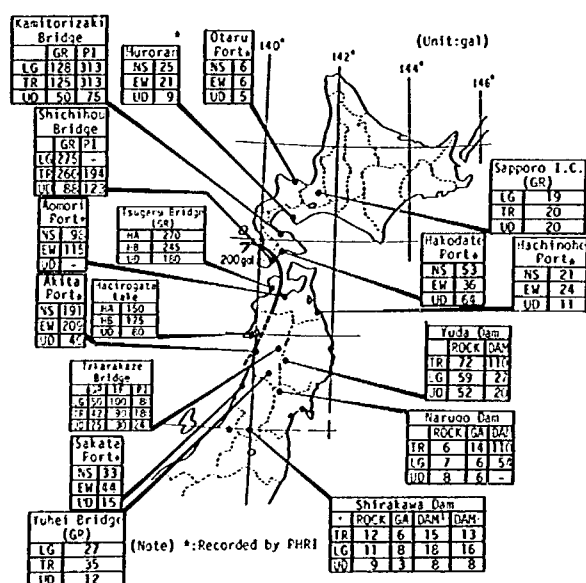
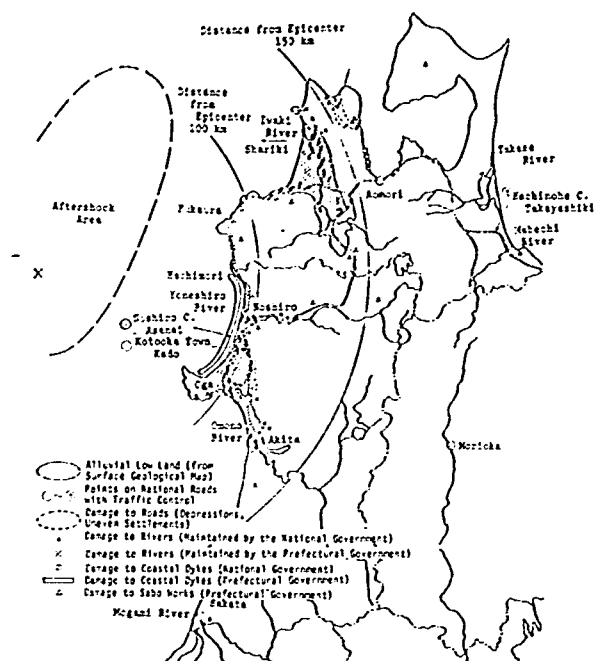


Fig. 69 shows a damage feature of a plain concrete abutment of Ryuma Bridge located in Akita Prefecture. A horizontal crack with width of 2 cm is observed at the construction joint. It was also seen that the abutment moved toward the river side due to the pressure of the backfill. A collision between the abutment parapet and the girder presumably caused the heavy lateral crack of the abutment.

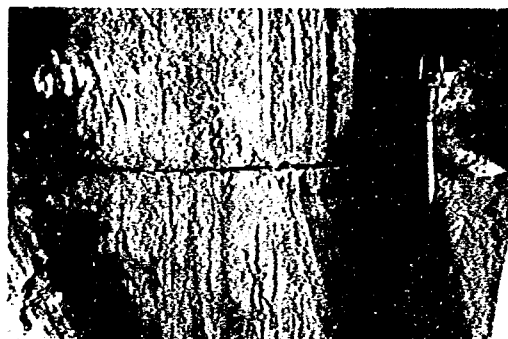
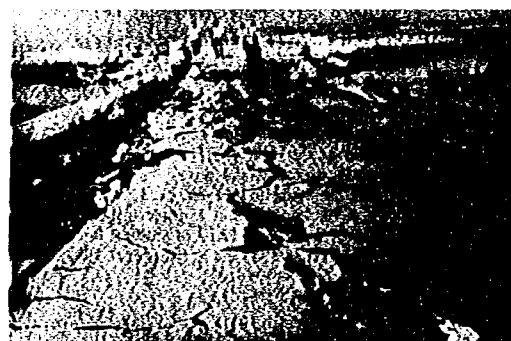


Fig. 70 shows extensive settlements of an approaching bank to an abutment of Gomyoko Bridge in Akita Prefecture. Those settlements were caused by liquefaction of supporting sandy layers. Fig. 71 indicates a view of Gomyoko Bridge which did not sustain any structural damage, in spite of serious settlement of approaching roads. Fig. 72 shows a view of Jusanko Bridge in the northern part of Aomori Prefecture, where settlement (about 50 cm) was observed at neighboring ground surface due to liquefaction of a sandy layers. Although serious settlements and cracks occurred at ground surfaces, the bridge did not receive any serious damage except a settlement of one pier of about 10 cm. Approaching banks to the both abutments settled seriously (1 m at most).



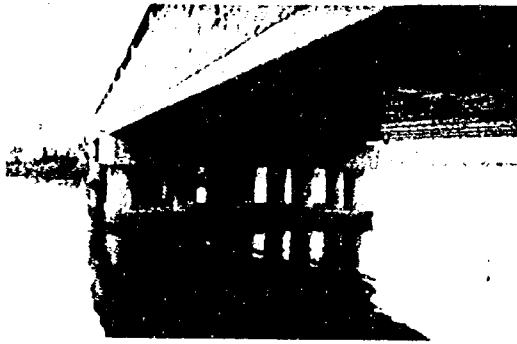


Fig. 71. View of Gomyoko Bridge



Fig. 72. View of Jusanko Bridge

Fig. 73 shows a heavy crack of a reinforced concrete pile foundation of Haguro Bridge, located next to Jusanko Bridge.



Fig. 73. Heavy Crack of Pile Foundation of Haguro Bridge

Fig. 74 shows cracks and separations of concrete pieces near a bearing support of Shin-Yamada Bridge in the northern part of Aomori Prefecture. It seems that the width of the pier crown was too narrow to transmit the lateral force due to the seismic excitation.

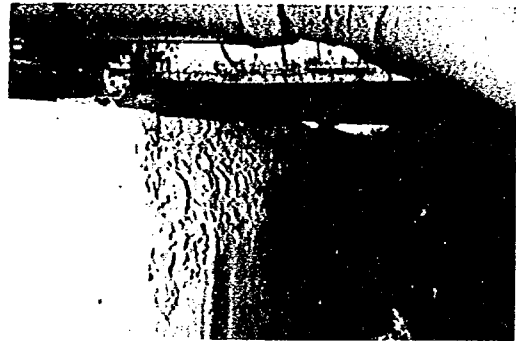


Fig. 74. Cracks and Separations of Concrete Pieces near a Bearing Support of Shin-Yamada Bridge

8. CONCLUSIONS

From a comprehensive review of the past experiences of seismic damages to highway bridges in Japan, the following may be concluded.

1) Seismic damage to bridge structures are generally caused by the lack of resistance at bearing supports, substructures and surrounding soils. As results of the weakness of these portions, substructures would tilt, settle, slide, cause cracks, and even overturn; superstructures would move, cause cracks, and sometimes fall down into river beds; and bearing supports would cause failures. Moreover, it is often observed that appurtenant structures such as wing walls and approaching banks settle, and separate from abutments.

2) For providing highway bridges with adequate strength against earthquake disturbances, the magnitude of design seismic forces is the most important factor in the design. In addition, it seems important to give a special attention to (a) topographical and geological consideration to avoid ground disaster, (b) geotechnical engineering consideration such as liquefaction of surrounding soils, (c) design details to protect bridge girders from falling, and (d) holding proper ductility of reinforced concrete pier columns.

3) In improving seismic design of highway bridges, further investigations would be needed on the following subjects:

(a) Evaluation of Seismic Activities and Ground Motions

For determining appropriate seismic forces to be considered in the design, it is essential to study probabilities of occurrence of strong earthquakes either by a statistical manner or a deterministic (earthquake prediction) manner, and to investigate characteristics of strong ground motions in connection with various factors involved (such as seismic conditions, subsoil conditions, etc.). It seems also important to clarify phase differences of ground motions between one site and another nearby, from the viewpoint of the propagation of seismic waves during strong earthquakes.

(b) Structural Planning

For assuring appropriate strength of bridges to seismic disturbances, it seems very significant to properly select structural types in consideration of seismicity, topography, geology, geotechnical conditions at the bridge site.

(c) Effects of Soils on Bridge Structures

It is necessitated to evaluate the effects of soil-structure interactions, earth pressures on abutments, bearing capacities of subsoils, and the effects of ground failures (such as liquefaction, faulting, and sliding) on bridge structures.

(d) Seismic Design Method of Substructures

It is essential to establish a definite design calculation method for substructures consisting of piers and abutments, together with various types of foundations (such as spread foundations, caisson foundations, pile foundations, sheet pile foundations, multi-column foundations, continuous-wall foundations, etc.). It should be noted that ample information on engineering properties of ground soils is necessary for properly conducting seismic design.

(e) Details of Superstructures and Bearing Supports

A specific attention should be paid to design details of bearing supports and hinges, and also to measures for preventing the fall of girders from the crests of substructures.

(f) Measurement of Dynamic Properties and Seismic Response of Highway Bridges
It seems important to perform field experiments for actual highway bridges in order to evaluate the dynamic properties (such as natural periods, mode shapes, damping characteristics, etc.). It is much more significant to measure dynamic behavior during actual strong earthquakes by installing strong-motion instruments on bridges and on grounds nearby.

(g) Dynamic Analysis of Highway Bridges

It seems reasonable that an analytical approach is a better way in estimating dynamic behavior of structures subjected to seismic excitation. In dynamic analysis, it is very important to adequately set up 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 excitations. It is advisable to reproduce structural failures or collapses in analysis, with respect to bridges severely suffered by the past earthquakes.

(h) Laboratory Experiments on Dynamic Behavior of Bridges

Dynamic behavior of materials, simplified structural systems, and composite structures are preferable to be examined through laboratory experiments, employing exciters, actuators, or shaking tables.

(i) Quantitative Evaluation of Ultimate Strength of Bridges through Seismic Damage Investigation

To obtain quantitative information on ultimate strength of bridge structures, it is effective to survey seismic damage to the similar structures, 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, a dynamic experiment or a dynamic analysis may be effective means to pursue structural behavior until suffering failures and collapses.

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A CASE HISTORY OF BRIDGE
PERFORMANCE DURING
EARTHQUAKES IN JAPAN

PWRI TECHNICAL MEMORANDUM
No. 2046
Feb. 1984
Toshio Iwasaki

Synopsis:

The report describes the results of investigations on seismic damages to highway bridges due to the past major earthquakes occurred in Japan, with emphasis on geotechnical engineering aspects.

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