

PROGRESS REPORT OF RESEARCH WORKS IN BRIDGE EARTHQUAKE ENGINEERING
AT THE PUBLIC WORKS RESEARCH INSTITUTE, JAPAN

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SUMMARY

This paper firstly describes results of investigations on seismic damages to existing bridge structures due to the past major earthquakes, with emphasis on the damage features of highway bridges during the Miyagi-ken-oki Earthquake of June 12, 1978, registering 7.4 on the Richter magnitude. General features of seismic damages to bridges are then summarized. This also discusses the effects of soil liquefaction on bridge foundations, the earthquake measurements at highway bridges and neighboring grounds, and the new seismic design criteria proposed for highway bridges.

INTRODUCTION

A number of strong earthquakes have hit Japan and caused extensive damages to modern engineering structures including highway bridges. Bridges sustained considerable damages during the Kanto Earthquake of 1923, the Fukui Earthquake of 1948, the Niigata Earthquake of 1964, and also the Miyagi-ken-oki Earthquake of 1978. These experiences have encouraged bridge engineers to perform research works in the field of earthquake engineering associated with bridge design. This paper briefly summarizes recent research activities at the Public Works Research Institute, Japan which are related to seismic effects on highway bridges, and introduces new seismic design criteria for bridges.

For getting better understanding of seismic effects on bridges and for pursuing a reasonable design method against seismic forces, it seems very important to investigate seismic damage to existing bridge structures due to earthquakes previously experienced. Fig. 1 and Table 1 provide the brief information of eleven major earthquakes which caused comparatively severe damage to bridge structures since 1923. Details of seismic damage to bridge structures observed during nine earthquakes (except two recent ones in Table 1) are already summarized in the previous papers^{1),2)}. Firstly, therefore, damage characteristics caused by the recent Miyagi-ken-oki Earthquake of June 12, 1978 will be summarized and general features of seismic damage to bridges will be discussed.

MIYAGI-KEN-OKI EARTHQUAKE OF JUNE 12, 1978

Outline of the Earthquake³⁾ — On June 12, 1978 a strong earthquake took place under the sea bottom approximately 120 km 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 30 km. The earthquake caused very severe damages 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

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earthquake ($M = 6.7$) whose epicenter was approximately 60 km 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 Records³⁾ — During the Miyagi-ken-oki Earthquake of June 12, 1978 a number of strong-motion seismographs traced acceleration records at various stations in Hokkaido and northern Honshu. Fig. 2 shows a relationship between epicentral distance and maximum acceleration. These acceleration records were obtained at structures such as highway bridges and port structures and on ground surfaces near the structures. Fig. 3 shows typical strong-motion records (by SMAC-B2 type accelerographs) which were obtained at Kaihoku Bridge located 80 km west from the epicenter. Although very high accelerations (more than 500 gals) were measured on the pier top, Kaihoku Bridge did not sustain any structural damage. Only fixed bearing supports and oil dampers were slightly damaged to their anchor bolts.

Next, Fig. 4 shows a record at a pier cap of Date Bridge located near Fukushima City (epicentral distance of 160 km). The maximum accelerations on the pier cap were 480 gals in the longitudinal direction, and 320 gals in the transverse direction. This bridge suffered moderate damages to bearing supports and a truss member above the fixed bearing.

Features of Bridge Damage³⁾ — Fig. 5 shows locations of severely damaged highway bridges (black circles), places where liquefaction was observed (white circles), and geological conditions in Miyagi Prefecture. It is seen from Fig. 5 that most of major bridge damages and liquefaction took place in alluvial lands along large rivers such as Kitakami, Naruse, Yoshida, Natori, and Abukuma.

Sendai Bridge — Sendai Bridge, completed in 1965, is located in south part of Sendai City, and is crossing over Hirose River as a part of the National Highway No. 4. Superstructures are 9-span simply supported composite steel-plate girders, with span length of 9×33.840 m, total length of 310 m, and width of 19 m. Substructures are T-shape columns (6.1 m high) founded on rigid well foundations (9 to 18 m deep) embedded into rather stiff sands. Bearing supports are of type of line bearings.

Due to the earthquake (the epicentral distance to the bridge is $\Delta = 120$ km), all of the nine pier columns sustained damages. Piers 1 through 4 cracked horizontally at the column bases and surface concrete pieces separated heavily from the columns near the bases. Piers 5 through 8 had similar damages near the haunches which connect columns and beams. Pier 6 which has the lowest free height sustained the severest cracking at both sides (see Fig. 6). Concrete pieces separated at the haunch and reinforcing bars buckled. Near the haunch number of reinforcing bars as well as concrete sectional area change rapidly.

Fig. 7 shows temporary frame works supporting the girders near Pier 6. Since the bridge is very important, damaged piers were repaired without stopping traffic even for a short time. Fig. 8 illustrates an example of permanent repairing work at Pier 6. The thickness of added concrete was 50 to 70 cm, and vertical reinforcing bars were fixed by epoxy adhesive into the well foundation and lateral bars were fixed to the columns. Moreover, chemical resin was placed into small cracks. It took only one month to

completely repair the all damages to this bridge.

Kin-noh Bridge — Kin-noh Bridge, completed in 1956, is on National Highway No. 346, and crossing over Kitakami River. The superstructures of the bridge are single-span steel plate girder, 5-span simply supported steel trusses, and 9-span Gerber-type steel plate girders from the left to right. The total length and the width are 575.5 m and 6.0 m, respectively. Sub-structures are RC columns on caisson foundations for the truss span, and RC columns on footing foundations with RC piles for the Gerber plate girder span. Soils are of soft silts and sands, and a firm sand layer exists approximately 30 m below the ground surface. During the June earthquake one girder of this bridge fell down (see Fig. 9). This Kin-noh Bridge is only one bridge which completely fell down during the June Earthquake.

This bridge was damaged three times by three different earthquakes, namely Northern Miyagi-ken Earthquake of 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 Earthquake⁴) side blocks of bearing supports (oval line bearing) of Gerber girders failed, and concrete near the fixed bearing supports on the right-bank abutment cracked. After the 1962 Earthquake, a repairing work to add stiffening plates was undertaken at three piers (P8, P9, and P10).

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

Since there were only four months after the February Earthquake, repairing works of these bearings were still undertaken at the time of the June Earthquake. Accordingly, all the girders were possible to move freely without any restraints. A suspended girder between Piers 7 and 8 fell down on the river bed, as shown in Fig. 9. The superstructure moved toward the right-bank side by 55 cm on the top of Pier 8 (see Fig. 10). Since the upper support dislodged from the bearing and lower flange supported the dead weight of the girder, a local buckling took place at the web of the girder. All the Gerber spans between Pier 8 and the Right Abutment moved to the right. The girder moved 10 cm toward the right on the right-bank abutment, and the end of girder collided into the parapet of the abutment. The asphalt pavement of the backfill heaved due to the collision.

On the other hand, truss girders were also heavily damaged during the June Earthquake. Fig. 11 is a picture taken after the June Earthquake. Anchor bolts of the upstream fixed bearing at Pier 6 were severely pulled out by about 20 cm at most, presumably due to rocking and translation motions of the bearing, and some concrete underneath the lower bearing plate was taken out and the bearing sunk 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 after the February 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 Earthquake. Fig. 12 shows the state after the June Earthquake at the upstream movable bearing on Pier 5 whose rollers completely had rolled out. As for pier columns, only the right-bank side of Pier 8 sustained heavy cracks. It is estimated that these cracks would have taken place when the superstructure collided with the right abutment and the reaction toward the left bank acted to the pier.

Yuriage Bridge — Yuriage Bridge, completed recently in 1972, is crossing over Natori River near its mouth. The superstructures are of 3-span continuous PC box girders with a center hinge and of 7-span simply supported post-tension PC beams (T-shape) with 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 Earthquake the nine pier columns sustained many cracks (almost all around) mostly at the level of the ground. Pier 1 (first pier from the left) sustained numerous heavy cracks (see Fig. 13), and concrete pieces separated from the column. Stoppers of single-roller-type movable bearing on Pier 1 were damaged, guide pieces of the bearing failed, and the roller was almost rolling down from the shoe.

A simply supported PC beam on Pier 6 moved 6 cm toward downstream. The ends of one handrail inserted into the ends 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. A number of ground cracks and sand boils were observed on the river bed near the right bank. The subsoils of sands are loose near the surface and medium to dense underneath. A hard layer exists 70 m below the surface.

Date Bridge — Date Bridge, completed in 1963, is crossing over Abukuma River near Fukushima City, with a epicentral distance of approximately 160 km. Superstructures are 4-span continuous steel truss girders, with the total length of 288 m and the width of 7 m. The two abutments are on steel-pipe pile foundations, and the three piers are of tall RC columns on caisson foundations embedded into gravel and sand layers.

Due to the June Earthquake a lower chord member buckled just at the fixed bearing on Pier 2 (see Fig. 14). Several pins at the fixed bearing and one of the movable bearings were sheared off and came out of the shoes. The substructures did not sustain any damage. A strong-motion accelerograph is set up on the cap of Pier 2, and triggered a complete time history of the acceleration at the pier cap (see Fig. 4).

Lessons from Bridge Damage due to the Miyagi-ken-oki Earthquake — In view of the damages to bridge structures during the Miyagi-ken-oki Earthquake of June 12, 1978, the following lessons can be derived.

- 1) Damages to superstructures concentrated on bearing supports and adjoining portions. On the other hand, most damages to substructures were cracks and separations of concrete at pier columns and abutments.

2) Damages to bearing supports were frequently observed. It is advisable to investigate design practices of bearing supports and to develop better bearings which are properly strong against seismic disturbances. It seems, however, that breakage of bearing supports have reduced failure of bridge girders and also failure of substructures.

3) Because of causing the extensive damages to the whole bridge structure, fall of bridge girders should be avoided.

4) A number of older bridges sustained severe damages. In these bridges, Gerber-type or simply supported type is used, the width of pier caps is narrower, and no consideration to prevent girder fall is introduced. It seems important to retrofit these older bridges by widening pier caps, or installing devices to prevent girder fall.⁵⁾

5) In view of the damages to pier columns at Sendai and Yuriage Bridges which were recently constructed according to the current specifications, it is recommended to consider ductility of pier columns when designing short reinforced concrete columns. In this respect further experimental and analytical researches are necessitated, and seismic specifications should include an appropriate regulation on ductility of columns.

GENERAL FEATURES OF SEISMIC DAMAGE TO BRIDGES

Damage to bridges generally observed at supports, abutments, piers, and girders consisting of a bridge structure (Fig. 15). The mechanisms of the bridge damage can be classified into three categories.¹⁾

1) Due to the Weakness of Supports — Any portions of a bridge structure will be exerted to move 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 substructures. Occasionally the superstructures may dislodge from the substructures and fall down. Both the superstructures and the substructures will suffer considerable damage by the fall of the superstructures.

2) Due to the Weakness of Substructures — If a substructure is not sufficient to resist its inertia force and seismic forces of girders transmitted through the supports, it may crack, deform, and sometimes overturn. 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 or move horizontally during earthquakes. An extensive decrease in the bearing capacities of the soils is often observed at loose saturated sandy soils due to liquefaction. If a drastic movement of substructure occurs due to the weakness of soils, 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 in the above the following failures are often observed at individual portions of a bridge structure.

Substructure: Tilt, settlement, slide, crack, overturn.
Superstructure: Movement, buckle, crack or failure, girder fall.
Support: Failure of bearings, cut-off or pull-out of anchor bolts.
Appurtenant Structures: Settlement of approach road (especially in case of banks), settlement and slide 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 essential to pay 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), (3) design details for preventing the fall of girders and for avoiding severe damage caused by the failure at supports, and (4) sufficient ductility for preventing cracking at pier columns especially at short rigid reinforced concrete columns. On the basis of the viewpoint, new specifications⁶⁾ for seismic design of highway bridges have been prepared.

EFFECTS OF LIQUEFACTION ON BRIDGE FOUNDATIONS

The Public Works Research Institute has been conducting laboratory shaking table tests on dynamic behavior of pile foundations constructed in loose saturated sand deposits^{8),9)}. Fig. 16 shows a typical test setup, and a description of the five test cases experimented. Figs. 17 and 18 show some test result when subjected to sinusoidal input motions. From the tests the following remarks are derived tentatively.

1) Liquefaction phenomena expand with respect to time, and with respect to space. Therefore, the effects of liquefaction of sands on foundations vary with time and space. It can be generally recognized that the acceleration response of a pile foundation is small prior to the initiation of liquefaction, becomes considerably large in the course of occurrence of liquefaction, and finally decreases after complete liquefaction takes place. During the complete liquefaction sands behave as heavy water.

2) In estimating behavior of a pile embedded into liquefiable sands, the relationship among the natural frequency of the pile foundation prior to liquefaction, the natural frequency of the pile in completely liquefied sands, and the predominant frequency of input motion seems the most important factor. Dynamic behavior of the pile in the course of liquefaction will be considerably affected by this relationship.

3) While the complete liquefaction succeeds, the soil-structure system will behave as a structure submerged into heavy water. Since its natural frequency becomes longer, it may resonate when subjected to seismic motions with longer periods during the latter time of an large earthquake.

STRONG-MOTION EARTHQUAKE MEASUREMENT AT BRIDGES

In Japan the observation of strong-motion earthquakes for engineering structures was initiated in 1953. As of March, 1979, the number of strong-motion accelerographs installed on engineering structures is over 1,200 totally. Among them, 199 SMAC-type accelerographs are equipped on 93 highway bridge structures (105 accelerographs) and on ground surfaces (94

accelerographs) near those bridges. In addition to the 93 highway bridges, 13 bridges equip electro-magnetic-type seismographs. Moreover, there are nine stations where dynamic behavior of subsurface soils during earthquakes are being measured using downhole seismometers (deeper ones are more than 100 m below the surface) in connection with large bridge projects.

Strong-motion records from the above stations have been published periodically by the Public Works Research Institute⁷⁾, together with those from other public works such as highway tunnels, dam structures, etc.

SPECIFICATIONS FOR EARTHQUAKE-RESISTANT DESIGN OF HIGHWAY BRIDGES (1980)

Efforts for revising the current specifications for Earthquake-Resistant Design of Highway Bridges (1971)¹⁰⁾ have been undertaken, and new Specifications⁶⁾ have been completed in 1980. The features and changes in design procedures in the new Specifications are briefly described.

Seismic Zoning Map — The newly developed seismic zoning map illustrated in Fig. 19 is based on the Proposal for Earthquake Resistant Design Methods¹¹⁾, and slight modifications were introduced on the proposed original map from the viewpoint of administrative considerations.

Classification of Ground Conditions — In the current Specifications, classification of ground conditions are determined in accordance with geological conditions. However, since ground responses during earthquakes would generally be more largely affected by the predominant period of the ground, it is considered more reasonable to classify grounds into some groups in terms of the period of the ground. Consequently in the New Specifications, the ground conditions will be classified into four groups according to Table 5, in which characteristic value of ground T_g is stipulated to be principally calculated by the following equation:

$$T_g = \sum_i \frac{4H_i}{V_{si}} \dots\dots\dots (1)$$

where

- T_g : Characteristic value of ground (second)
- H_i : Thickness of i-th subsoil layer (m)
- V_{si} : Shear wave velocity of i-th subsoil layer at low strain (around 10-4%)

Liquefaction of Sandy Soil Layers — In the current Specifications, it is stipulated that saturated sandy soil layers which are within 10 meters of 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 and 0.5 mm, shall have a high potential for liquefaction during earthquakes. Bearing capacities of these layers shall be neglected in design. After the Niigata Earthquake of 1964, comprehensive studies have been conducted to assess vulnerability of saturated sandy soils. Based on these studies, the provisions for liquefaction are improved in the New Specifications as follows:

- (1) Sandy Soil Layers Needed to be Checked for Liquefaction - Saturated sandy layers which exist under water table and do not coincide with

any of the following conditions are vulnerable to liquefaction, and their liquefaction potential shall be estimated according to item (2).

- 1) Soil layers which exist at 20 meters below the surface or deeper.
- 2) Soil layers in the case where the water table exists deeper than 10 meters below actual ground surface.
- 3) Soil layers which have a D₅₀-value on the grain size accumulation curve either smaller than 0.02 mm or larger than 2 mm.
- 4) Soil layers formed in the diluvial era or older.

- (2) Estimation of Liquefaction - For those soil layers which are judged to be vulnerable for liquefaction, liquefaction potential shall be checked based on liquefaction resistance factor F_L defined by

$$F_L = \frac{R}{L} \dots\dots\dots (2)$$

where

- F_L : Liquefaction resistance factor,
 R : Resistance of soil elements to dynamic loads, and can be determined in accordance with Eq. (6),
 L : dynamic loads to soil elements induced by earthquake motion

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

$$r_d = 1.0 - 0.015z \dots\dots\dots (4)$$

- z : depth from the actual ground surface (m)
 k_s : seismic coefficient for evaluation of liquefaction, and shall be determined by the following equation:

$$k_s = v_1 \cdot v_2 \cdot v_3 \cdot k_{SO} \dots\dots\dots (5)$$

- v_1, v_2, v_3 : seismic zone factor, ground condition factor and importance factor.

$$k_{SO} = 0.15$$

σ_v : total overburden pressure (kgf/cm²)

σ'_v : effective overburden pressure (kgf/cm²).

Soil layers having the liquefaction resistance factor F_L smaller than 1.0 shall be judged to liquefy during earthquakes. For R -value the following equations were proposed based upon the results of laboratory dynamic triaxial tests on undisturbed soil specimens.⁸⁾

$$R = \begin{cases} 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.19 & (0.02 \leq D_{50} \leq 0.05\text{mm}) \\ 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} + 0.225 \log_{10} \left(\frac{0.35}{D_{50}} \right) & (0.05 < D_{50} \leq 0.6\text{mm}) \\ 0.0882 \sqrt{\frac{N}{\sigma'_v + 0.7}} - 0.05 & (0.6 < D_{50} \leq 2.0\text{mm}) \end{cases} \quad (6)$$

- (3) Treatment of Soil Layers which were Judged to Liquefy - For soil layers which were judged to liquefy by the above estimation, bearing

capacities and other soil constants shall be reduced in the seismic design, by multiplying the original bearing capacities by reduction factors D_E which are determined by Table 3.

Modified Seismic Coefficient Method — In the current Specifications, the modified seismic coefficient method is provided to apply to bridges which have flexible piers and long fundamental periods (longer than 0.5 seconds), such as those with piers taller than 25 meters above the ground surface. Accounting for seismic responses, magnification factors (β) for the modified seismic coefficient method are stipulated. However, it has been pointed out that fundamental natural periods sometimes exceed 0.5 seconds even for those bridges with pier lower than 25 meters. Basing on experimental data on the relation between fundamental natural periods and pier heights, it is stipulated in the New Specifications that the modified seismic coefficient method shall apply to bridges which have flexible piers and long fundamental periods, such as those with piers higher than 15 meters. In addition to the above, the following modifications were introduced:

- 1) The magnification factors (β) are modified as shown in Fig. 20 so as to avoid a sudden change of β -value at a period of 0.5 seconds.
- 2) In the current Specifications, effects of subsoils are not precisely considered in estimating natural periods. Since the effects of subsoils would be predominant in calculating natural periods, especially for bridges with short piers, it is stipulated in the new Specifications that the effects of subsoils shall be taken into account for those bridges which are constructed into soft grounds. It is recommended to estimate the natural period for the individual system consisting of each substructure and the part of superstructures supported by it by

$$T = 2.01 \sqrt{\delta} \dots\dots\dots (7)$$

where

- T: Fundamental natural period in seconds of the system consisting of a substructure and the superstructures supported by it.
- δ : Maximum horizontal displacement (in meter) of the pier when subjected to the dead weight of the section of superstructure supported by the substructure plus 80 percent of the dead weight of the substructure above ground surface.

Design Seismic Coefficient for Considering Ductilities — In order to avoid brittle failure of reinforced concrete substructures during earthquakes, a provision that stipulates the seismic coefficient used for the design of reinforced concrete piers with adequate ductilities is newly introduced in the new Specifications. It is stipulated that the design seismic coefficient considering ductilities shall be determined by

$$k_{hd} = v_4 \cdot k_h \dots\dots\dots (8)$$

where

- k_{hd} : design seismic coefficient with consideration of ductilities
- v_4 : structural characteristics factor (greater than 1.3)
- k_h : horizontal design seismic coefficient for ordinary bridges.

CONCLUSIONS

From a review of recent studies related to seismic effects on highway bridges, the following 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 sometimes overturn; superstructures would move, cause cracks or failures, or fall down; and bearing supports may cause failures. Moreover, appurtenant structures and approach banks settle, or separate from the abutments.

2) For providing bridges with 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, (b) soil dynamics consideration such as liquefaction of surrounding soils, (c) details for protecting girder fall and for evading severe damage caused by the failures at structural joints, and (d) ductility of pier columns.

3) Further investigations are needed on the subjects shown below, for improving aseismic design of bridge structures.

- (a) Evaluation of Seismicity and Ground Motions
- (b) Structural Planning
- (c) Effects of Subsoils on Bridge Structures during Earthquakes
- (d) Aseismic Design Method for Bridge Substructures
- (e) Design Details of Superstructures and Bearing Supports
- (f) Measurement of Dynamic Properties and Seismic Response of Bridges
- (g) Dynamic Analysis
- (h) Laboratory Experiment on Dynamic Behavior of Bridges
- (i) Quantitative Evaluation of Ultimate Strength of Bridges

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Table 1 Eleven Major Earthquakes Causing Highway Bridge Damages

Date	Name	M*1	No. of Damaged Bridges	No. of Fallen Bridges	Amount of Loss of Bridges*3
Sept.1,1923	Kanto	7.9	1,785*2	17*4	Unknown
Dec.21,1946	Nankai	8.1	346	1	95,605 Thousand Yen
Jun.28,1948	Fukui	7.3	243	7	207,651 "
Dec.26,1949	Imaichi	6.4	1	0	minor
Mar. 4,1952	Tokachi-oki	8.1	128	0	200,000 "
Apr.30,1962	Northern Miyagi	6.5	187	0	43,000 "
Jun.16,1964	Niigata	7.5	98	3	1,470,000 "
Feb.21,1968	Ebino	6.1	10	0	50,000 "
May 16,1968	Tokachi-oki	7.9	101	0	421,046 "
Jan.14,1978	Izu Ohshima Kinkai	7.0	7	0	39,000 "
Jun.12,1978	Miyagi-ken-oki	7.4	108	1	4,000,000 "
Total			3,014	29*4	-

*1 Magnitudes are on the Richter scale, after either Annual Report of Science or Japan Meteorological Agency.

*2 The number includes bridges damaged by fires (roughly 400 bridges).

*3 Amounts of loss are estimated at the time of each earthquake occurrence.

*4 The numbers include 9 bridges fallen due to fires.

Table 2 Classification of Ground Conditions

Group	Characteristic Value T_g (seconds)
1	$T_g < 0.2$
2	$0.2 \leq T_g < 0.4$
3	$0.4 \leq T_g < 0.6$
4	$0.6 \leq T_g$

Table 3 F_L - D_E Relation

F_L	Depth, Z(m)	Reduction Factor, D_E
$F_L \leq 0.6$	$Z \leq 10$	0
	$10 < Z \leq 20$	1/3
$0.6 < F_L \leq 0.8$	$Z \leq 10$	1/3
	$10 < Z \leq 20$	2/3
$0.8 < F_L \leq 1.0$	$Z \leq 10$	2/3
	$10 < Z \leq 20$	1

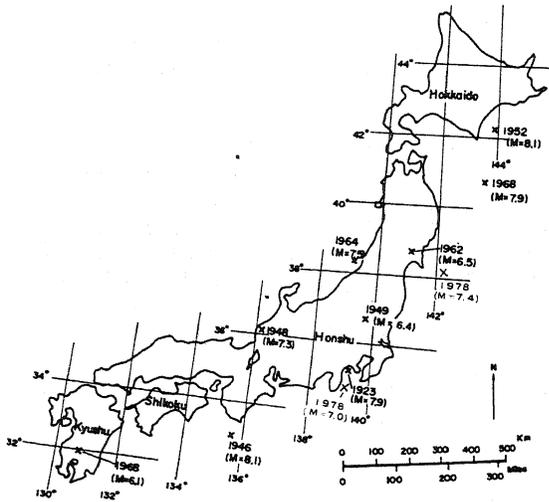


Fig. 1 Epicenters of Eleven Earthquakes which Caused Comparatively Severe Damage to Bridge Structures in Japan (See Table 1)

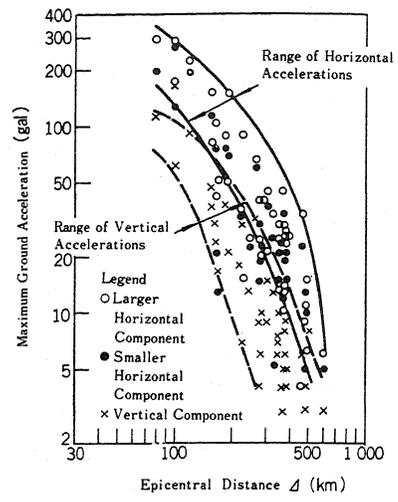


Fig. 2 Relation between Epicentral Distance and Maximum Ground Acceleration

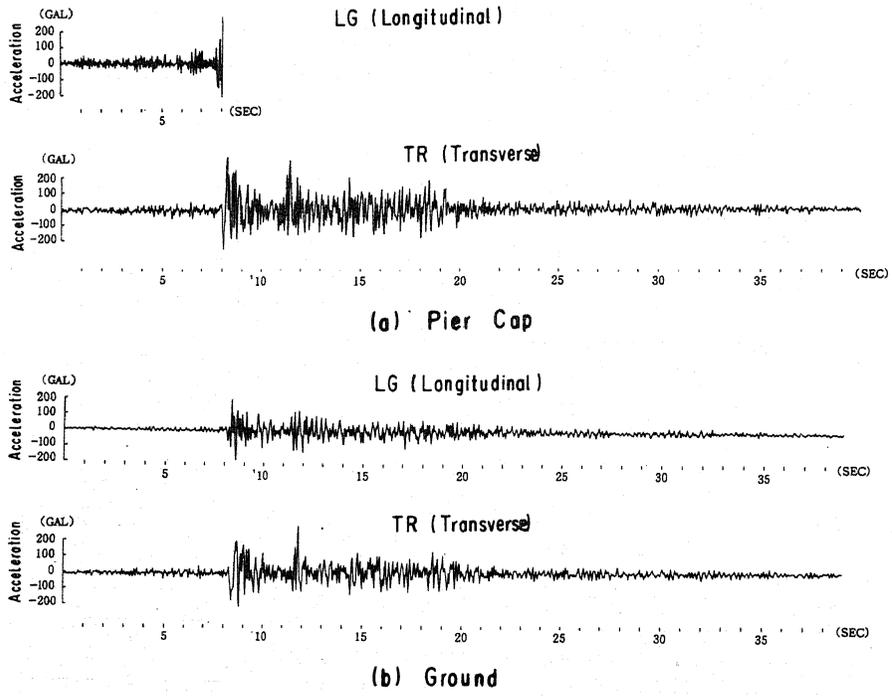


Fig. 3 Strong-Motion Records at Kaihoku Bridge Pier and Ground Nearby

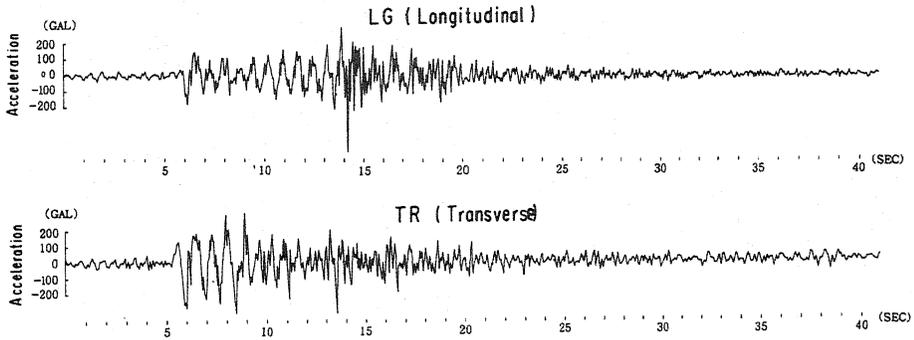


Fig. 4 Strong-Motion Record at Cap of Pier 2, Date Bridge



Fig. 6 Failure of Pier 6, Sendai Bridge

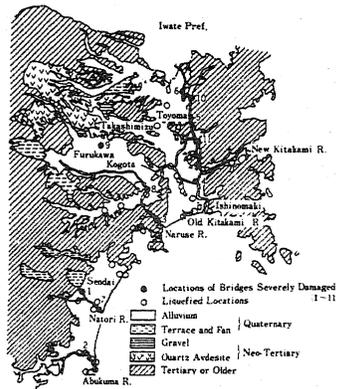


Fig. 5 Geological Features and Locations of Major Bridge Damages and Liquefaction Sites

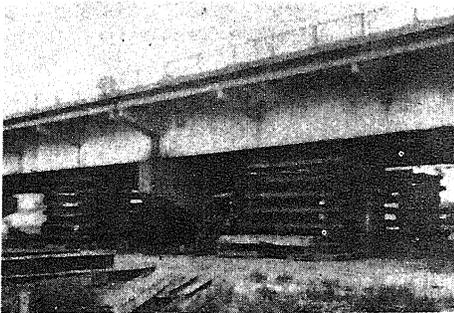


Fig. 7 Temporary Frame Supporting Girders near Pier 6, Sendai Bridge

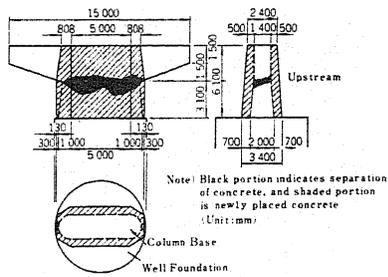


Fig. 8 Damage and Repair Work of Pier 6, Sendai Bridge

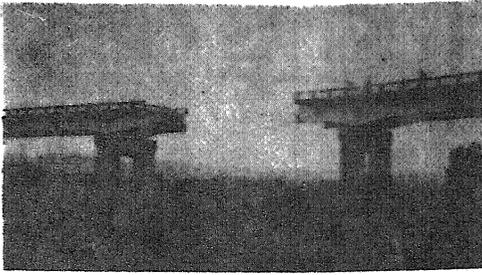


Fig. 9 Fall of a Suspended Girder, Kin-noh Bridge (During the Earthquake of June 12, 1978)

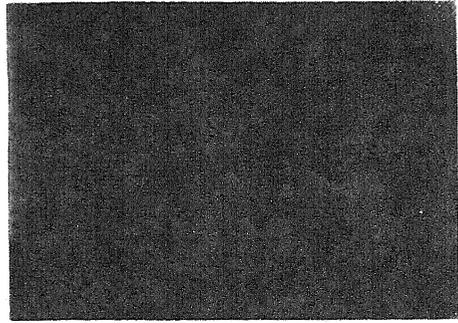


Fig. 10 Movement (55 cm) of a Plate Girder at the Downstream Support on Pier 8, Kin-noh Bridge (After June 12, 1978)

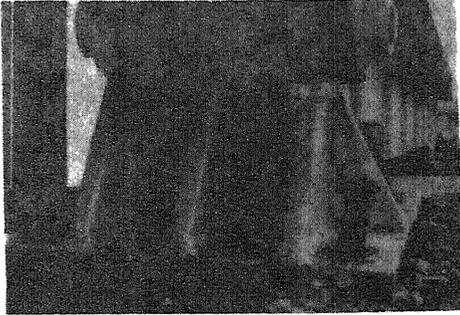


Fig. 11 Pull-out of Anchor Bolts and Settlement of the Shoe at the Upstream Fixed Bearing on Pier 6, Kin-noh Bridge. (After June 12, 1978)

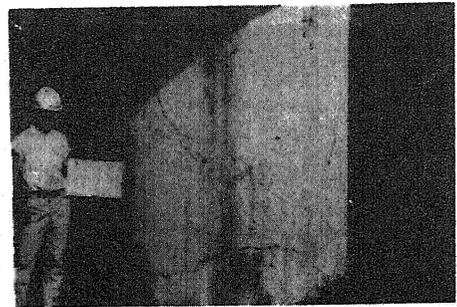


Fig. 13 Cracks at Pier 1, Yuriage Bridge



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



Fig. 14 Buckling of Lower Chord Member Above the Fixed Pin-Bearing on Pier 2, Date Bridge

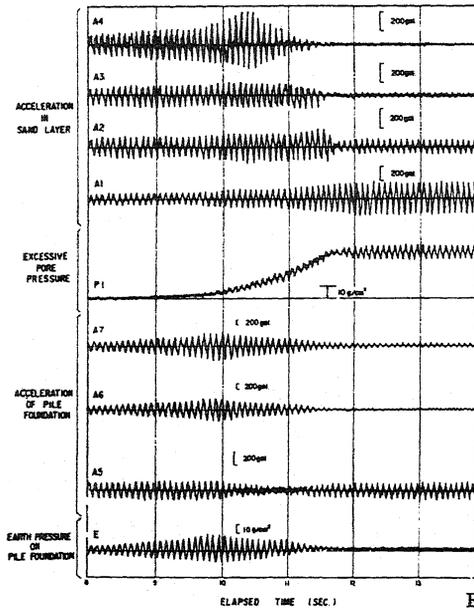


Fig. 17 Typical Test Records (Test 1)

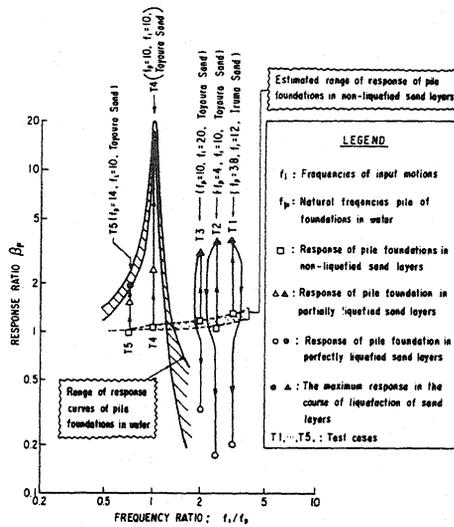


Fig. 18 Response of Pile Foundation Models Embedded in Liquefying Sand Layer (Tests 1 to 5)

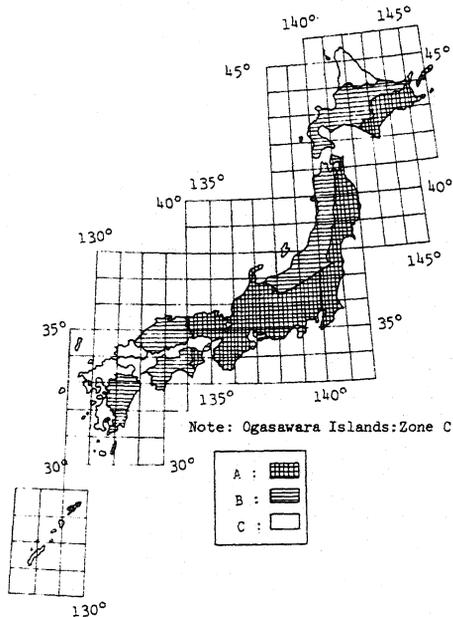


Fig. 19 Seismic Zoning Map (New Specifications)

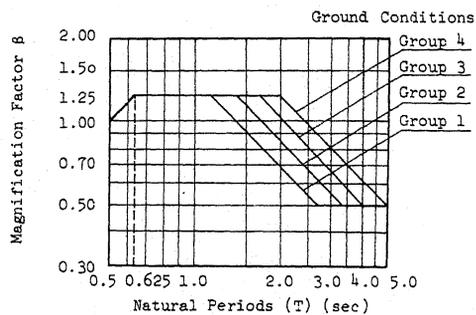


Fig. 20 Magnification Factor for the Modified Seismic Coefficient Method (New Specifications)