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**ASSESSMENT OF LIQUEFACTION POTENTIAL
AND ITS TREATMENT FOR DESIGN AND CONSTRUCTION OF
CIVIL ENGINEERING STRUCTURES**

By

Ken-ichi TOKIDA

**GROUND VIBRATION DIVISION
EARTHQUAKE DISASTER PREVENTION DEPARTMENT
PUBLIC WORKS RESEARCH INSTITUTE
MINISTRY OF CONSTRUCTION**

1, Asahi, Tsukuba-shi, Ibaraki-ken, 305 JAPAN

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A B S T R A C T

This report has been prepared for engineers to understand soil liquefaction and disaster prevention for soil liquefaction and also to get a view of future earthquake disaster prevention.

Recent subjects and topics related to soil liquefaction as for civil engineering structures are described based on past earthquake experiences, research results and practical application. These are: mechanism of soil liquefaction, liquefaction-induced hazard in past earthquakes, procedures for estimating soil liquefaction potential, seismic design methods of civil engineering structures considering soil liquefaction, seismic inspection methods of existing structures for soil liquefaction and countermeasures against soil liquefaction.

Moreover, recent new topics on liquefaction problems (lateral ground flow induced by soil liquefaction and visual information system for zoning liquefaction potential) which have been studied in Public Works Research Institute (PWRI), Ministry of Construction, Japan are presented.

要 旨

本資料は、技術者が地盤の液状化及び液状化に対する防災について理解し、さらに今後の地震防災に対する示唆が得られるようにとりまとめたものである。

既往の地震経験、研究結果及び実務での適用に基づいて、土木構造物における地盤液状化に関係した最近の課題及び話題がまとめられている。その内容は、地盤の液状化のメカニズム、既往地震における液状化被害、液状化危険度の予測手法、液状化を考慮した土木構造物の耐震設計法、液状化に対する既存構造物の点検手法及び液状化対策である。

さらに、土木研究所で研究されてきている液状化問題に関する最近の新しい話題、つまり、液状化に起因する地盤の水平流動及び液状化危険度のゾーニングのためのビジュアル情報システムが報告されている。

ACKNOWLEDGEMENTS

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Dr. Yasushi Sasaki, Dr. Michio Okahara of Public Works Research Institute (PWRI), Mr. Toshiro Yamada of National Land Agency and Mr. Akiomi Shimazu of Japan Construction Information Center who worked at PWRI allowed the author to use their invaluable photographs taken during their visit after the 1990 Philippines Earthquake as experts of Japan Disaster Relief Team.

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CHAPTER 1 INTRODUCTION

IDNDR (International Decade of Natural Disaster Reduction) began in 1990 so that all countries of the world join in a concerted international action for reducing the adverse effects of such natural disasters as floods, droughts, hurricanes, earthquakes and so on. However severe earthquakes occurred in both Iran and Philippines in the first year of the Decade. An estimated 40,000 people perished in the Iran Earthquake with the Magnitude (M) 7.7 on the Richter scale in June 21, and an estimated 1,700 people perished in the Philippines Earthquake with M = 7.8 in July 16. The earthquakes will occur repeatedly all over the world in the future, and we should seriously prepare for future earthquakes to reduce the likelihood of earthquake disaster.

When an earthquake occurs, such natural phenomena as strong motion, soil liquefaction, fault, tsunami and so on occur first and induce damage to structures second. Soil liquefaction is a typical ground failure, inducing severe damage of structures during earthquakes. Therefore, it is very important to pay attention to soil liquefaction and make an effort to reduce liquefaction-induced damage.

This paper summarizes the recent subjects and topics related to soil liquefaction on civil engineering structures for the people who work on reduction of earthquake-induced damage to refer. Chapter 2 shows the mechanism of soil liquefaction to understand it fundamentally. Chapter 3 shows the liquefaction-induced hazard in past earthquakes to understand the effects of liquefaction. Chapter 4 shows the procedures for estimating soil liquefaction potential which are common in Japan and have been studied in the Public Works Research Institute (PWRI). Chapter 5 shows the seismic design methods of civil engineering structures considering soil liquefaction which are common and typical in Japan at present. Chapter 6 shows methods for inspecting existing structures considering soil liquefaction potential and its effects. Chapter 7 shows the countermeasures against soil liquefaction to reduce damage of structures which are common in Japan. Chapters 8 and 9 show the recent new topics on liquefaction problems: the lateral ground flow induced by soil liquefaction and the visual zoning information system which have been studied in the PWRI, respectively.

CHAPTER 2 LIQUEFACTION PHENOMENA AND ITS MECHANISM

2.1 LIQUEFACTION AT HORIZONTAL GROUND

2.1.1 Mechanism of Soil Liquefaction

The mechanism of soil liquefaction occurrence can be explained conceptually as shown in Fig. 2.1. The state of soil particles in the liquefiable layer can be classified into three stages according to the lapse of time in an earthquake as follows;

(1) Before Earthquake

The soil element is saturated with pore water and the frame of soil particles is loose. The bearing capacity of this soil element is related to the effective confining pressure equivalent with the effective overburden pressure (σ_v') and equivalent with the total overburden/confining pressure (σ_v) before an earthquake.

(2) During Earthquake

The soil element is forced to deform by shear stress during an earthquake. At this moment, the particles try to be dense (called as "negative dilatancy/shrinkage of element volume") because it is loose. However, because the duration time of seismic load is generally short, the pore water between soil particles is not drained and the total volume of soil element is not changed or constant. As a result of constant volume of a soil element, excess pore water pressure (Δu) occurs, and because the total confining pressure (σ_v) is constant, the effective overburden pressure (σ_v') during an earthquake is forced to decrease. This means that the bearing capacity of a soil element is decreased by the occurrence of excess pore water pressure, in other words, by soil liquefaction. In this stage, when the grain size of soil particles is small such as clay and silty clay, the soil particles find it difficult to be dense during an earthquake, in other words, not liquefied. Furthermore, when the soil particles are compacted densely before an earthquake (Fig. 2.1(C)), the soil particles try to be loose (called as "positive dilatancy/expansion of element volume") and the effective overburden pressure or bearing capacity of a soil element is increased.

(3) After Earthquake

The pore water can be drained after shaking and the excess pore water pressure in the soil element is decreased. According to the progress of water drainage, the volume of soil element is decreased and become stable finally. The effective overburden pressure or bearing capacity of a soil element is recovered again. The drainage of pore water and flow out of soil particles with it can be observed as a water spring and a sand boil during an earthquake, respectively.

2.1.2 Phenomena of Soil Liquefaction During Earthquakes

The typical phenomena of soil liquefaction during earthquakes can be shown such

as in Fig. 2.2 and Table 2.1. The visual phenomena on the ground surface induced by soil liquefaction are sand boil, water spring, crack, settlement, up-heave and lateral ground flow. In other words, the occurrence of soil liquefaction can be found from the above mentioned phenomena. The mechanism of liquefaction-induced effects can be classified into four groups: the decrease of lateral bearing capacity of ground, the decrease of vertical bearing capacity of ground, the decrease of shear strength of ground, the increase of up-lift force and the flow of soil particle/water. It should be noted that the damage patterns of structures by soil liquefaction are different according to the mechanism of liquefaction.

The typical liquefaction phenomena which can be observed visually during and after an earthquake are sand boil and crack on the ground surface. Photos. 2.1 [Ref. 2-4] and 2.2 [Ref. 2-5] show the sand boil and crack which occurred in the Miyagi-Ken-Oki Earthquake of 1978, in Japan and the Loma Prieta Earthquake of 1989, in U.S., respectively. The underground can be estimated to be liquefied during an earthquake by the liquefaction phenomena such as sand boil and crack. In other words, it is necessary to pay attention to sand boil and crack in the earthquake inspection just after an event. However, it should be noted that the liquefaction of interest from the engineering view point is the one which occurs at the site/area of human activity.

Photo. 2.3 [Ref. 2-6] indicates sand boil occurred at the Carmen Bridge and liquefaction-induced damage of the bridge pier in the Philippines Earthquake of 1990. Because the foundation of the bridge pier was not driven into the tight base-ground which were not liquefiable, the bearing capacity of the surrounding liquefiable ground of foundation was decreased by soil liquefaction, and the foundation was tilted and settled to cause the damage of superstructure as shown in Photo. 2.3.

Photo. 2.4 [Ref. 2-7] shows the flood occurred by rainfall after the 1989 Philippines Earthquake. Because liquefaction occurred severely and caused the ground settlement in Dagupan City, the rainfall after the Earthquake stayed in the street and hindered the activity and health of people. Some countermeasures were needed for drainage. It indicates that the effects of rainfall just after an earthquake should be noted as the secondary disaster.

2.2 LATERAL GROUND FLOW AT INCLINED GROUND

When a ground surface on the liquefiable layer is inclined, ground flow can occur. Hamada et. al. [Ref. 2-1] pointed out that the fairly big amount of lateral flow of ground could be seen broadly at the areas in and/or around Tokyo, Fukui, Niigata and Noshiro where the liquefaction of the ground apparently took place during the Kanto Earthquake (1923, M=7.9), the Fukui Earthquake (1948, M=7.3) the Niigata Earthquake (1964, M=7.5) and the Nihon-Kai-Cyubu Earthquake (1983, M=7.7), respectively. Furthermore, the lateral ground flow and induced damage of structures have been reported in the recent earthquakes such as the Philippines Earthquake of 1990 [Ref. 2-2] and the Valle de la Estrella Earthquake in Costa

Rica, in 1991 [Ref. 2-3] .

Fig. 2.3 shows the lateral ground flow occurred in the Niigata Earthquake of 1968, in Japan [Ref. 2-1] .

Photo. 2.5 [Ref. 2-8] shows the lateral ground flow and the induced crack and step difference occurred in the Nihon-Kai-Chubu Earthquake of 1983, in Japan. The ground surface can be seen to be slid in the direction of slope.

Photo. 2.6 [Ref. 2-9] also shows the lateral ground flow toward the river and the induced cracks observed in the Philippines Earthquake of 1990. Thus, most lateral ground flows in the past earthquakes had a tendency to occur at the site/area neighboring water area such as a river and a pond which are generally low comparing with the neighborhood, in the direction of water area. It is necessary to take special care at the site/area neighboring water area concerning the lateral ground flow induced by soil liquefaction during earthquakes.

Experimental studies with the use of a shaking table on lateral ground flow have been conducted in the Public Works Research Institute over the past a few years. **Photo. 2.7** shows the lateral flow and cracks on the surface of ground model reappearance in a shaking table test. The results of these tests are presented in Chapter 8.

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Photo. 2.1 Sand Boil and Crack (Miyagi-Ken-Oki Earthquake of 1978, Japan) [Ref. 2-4]



Photo. 2.2 Sand Boil and Crack (Loma Prieta Earthquake of 1989, USA) [Ref. 2-5]



Photo. 2.3 Sand Boil (Philippines Earthquake of 1990, Philippines) [Ref. 2-6]



Photo. 2.4 Settlement of Ground Induced by Soil Liquefaction and Flood (Philippines Earthquake of 1990, Philippines) [Ref. 2-7]



Photo. 2.5 Lateral Ground Flow and Induced Crack Ground Flow (Nihon-Kai-Cyubu Earthquake of 1983, Japan) [Ref. 2-8]

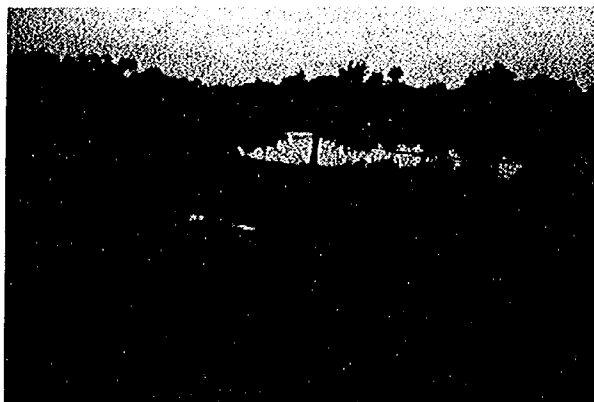


Photo. 2.6 Lateral Ground Flow and Induced Crack (Philippines Earthquake of 1990, Philippines) [Ref. 2-9]



Photo. 2.7 Lateral Ground Flow and Crack Observed in Shaking Table Tests at PWRI (see Chapter 8)

Table 2.1 Liquefaction and Induced-Damage

Liquefaction Phenomena of Ground	Effect of Liquefaction Against Structure	Example of Induced-Damage of Structure
Sand Boil	Decrease of Horizontal Bearing Capacity of Ground	Inclination/Deformation of Foundation of Bridge.
Water Spring		
Crack	Decrease of Vertical Bearing Capacity of Ground	Inclination/Settlement of Weighted Structure
Lateral Flow/Slide		
Settlement	Decrease of Dynamic Shear Strength of Ground	Crack/Settlement/Slide of Bank/Levee, Lateral Ground Flow of Inclined Ground
Up-heave		
etc.	Increase of Up-lift Force, Flow of Soil Particle/Water	Up-lift of Pipe/Manhole/Duct

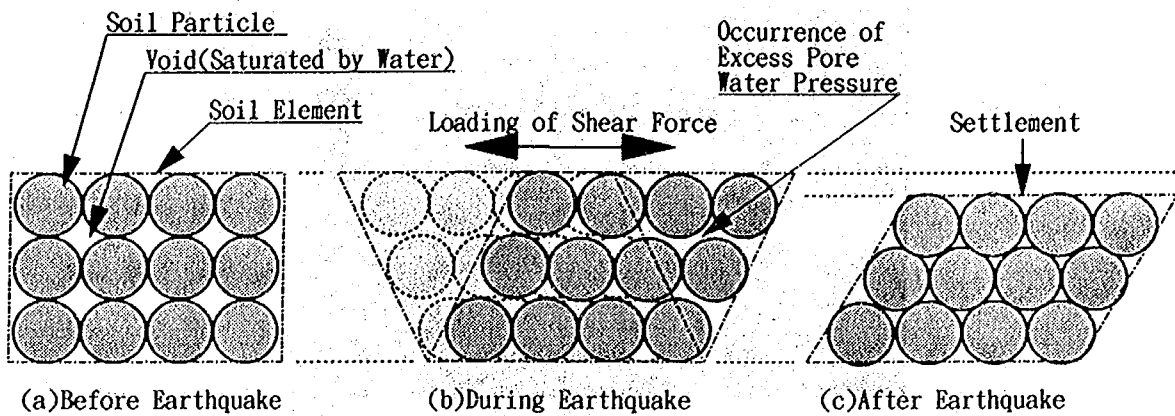


Fig. 2.1 Mechanism of Soil Liquefaction

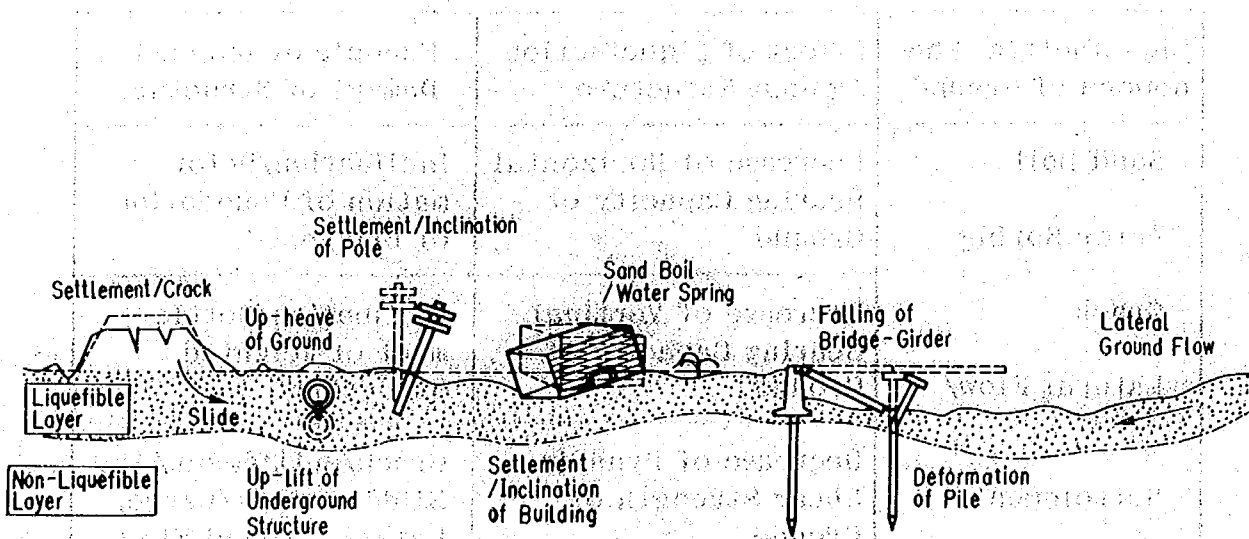


Fig. 2.2 Typical Phenomena of Soil Liquefaction

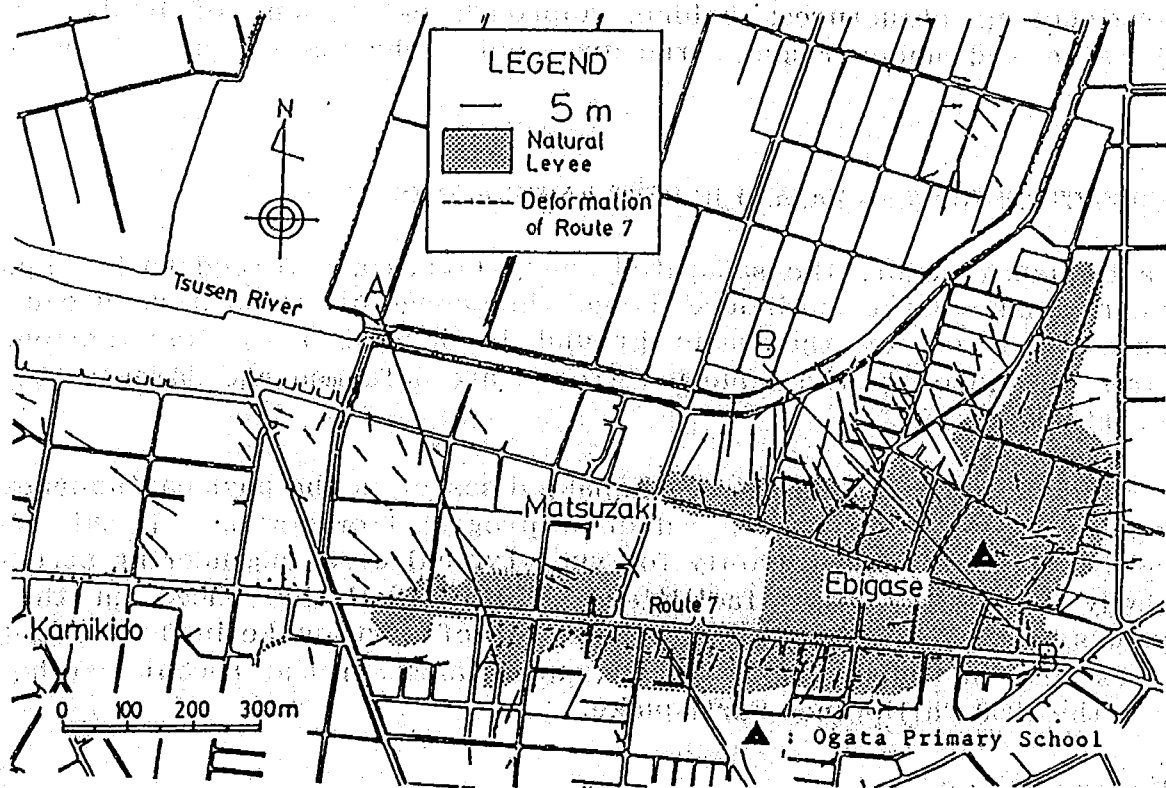


Fig. 2.3 Lateral Ground Flow Occurred in the 1964 Niigata Earthquake, Japan [Ref. 2-1]

CHAPTER 3 LIQUEFACTION-INDUCED BEHAVIOR/HAZARD

3.1 CLASSIFICATION OF LIQUEFACTION-INDUCED HAZARD

As shown in Table 2.1, the soil liquefaction/ground hazard causes several kinds of severe damage/hazard of structures according to the mechanism of ground hazard. Table 3.1 summarizes the relation between liquefaction-induced damage of typical civil engineering structures: bridge, approach embankment of bridge, levee, special levee and underground structure, and mechanism of ground hazard of interest.

3.2 LIQUEFACTION-INDUCED HAZARD IN PAST EARTHQUAKES

When earthquakes occur, the saturated sandy layers are forced to be liquefied during earthquakes first and induce second the damage of structures. Because the resistance potential of the basic ground is decreased by liquefaction, the structures on and/or in the liquefied ground are deformed and damaged severely during earthquakes.

It is very important to learn the earthquake disaster in the past earthquakes for planning the earthquake disaster reduction program from now on. In this report, the classification of damage mainly for such typical civil engineering structures as highway facilities, river facilities, lifeline facilities based on the past earthquakes are presented, and the examples of earthquake-induced damage of structures are introduced with photographs taken in the recent earthquakes including the 1990 Philippines Earthquake.

3.2.1 Highway Bridge

Highway bridges which are likely to be damaged by soil liquefaction during earthquakes can be anticipated in advance, and should be considered for checking the earthquake resistance of bridges in advance and examining the damaged parts just after earthquakes. The examples of damage and learned notation related to highway bridges damaged by liquefaction in recent earthquakes are shown as follows.

Photo. 3.1 [Ref. 3-1] shows the girder-falling of the Syowa-Ohashi Bridge in the 1964 Niigata Earthquake in Japan. The bridge consisted of 12 simple girders and the foundation of each pier/abutment was a pile-bent type as shown in Fig. 3.1. Around this bridge, severe liquefaction occurred and the surrounding ground of pile foundation lost its horizontal bearing capacity by soil liquefaction. Five simple girders between the pier-2 and the pier-7 were fallen down into the Shinano River. Although the bridge was opened for traffic in May of 1964, it was damaged by the earthquake in June of 1964, only one month after because the bridge was not designed considering the effect of soil liquefaction. After this earthquake, the research/study of soil liquefaction was started seriously in Japan.

Photo. 3.2 [Ref. 3-2] shows the damage of bridge approach banking at Gomyokou Bridge in the 1983 Nihon-Kai-Chubu Earthquake in Japan. Because the bridge approach banking was collapsed severely by cracks, settlement and lateral spreading, the bridge body was not damaged structurally but damaged functionally. However the repair of approach banking is generally easier than that of bridge body.

Photo. 3.3 [Ref. 3-3] shows Magsaysay Bridge of 6 simple reinforced concrete girders (total length: 102 m) damaged in the 1990 Philippines Earthquake. It is located in Dagupan City where liquefaction occurred most notably. Because the underlying sandy ground at this bridge was liquefied, the 5 pier foundations were sunk and tilted, and 4 simple girders fell down.

Photo. 3.4 [Ref. 3-4] shows the typical liquefaction phenomena observed visually at the ground surface around Carmen Bridge damaged in the 1990 Philippines Earthquake. Because the underlying ground was liquefied (see **Photo. 2.3**), 6 pier foundations of 13 simple truss girders (total length: 655 m) were tilted and 6 girders fell down.

Photo. 3.5 [Ref. 3-5] shows the damage of approach embankment of a bridge in the 1990 Philippines Earthquake, similar to **Photo. 3.2**. The base ground of the approach bank was liquefied and caused the sliding, settlement, and crack of the bank.

3.2.2 River Facility

Photo. 3.6 [Ref. 3-2] shows the sliding and settlement of a levee occurred in the 1983 Nihon-Kai-Chubu Earthquake in Japan. The ground below the bank was liquefied and induced the severe collapse of the levee.

Photo. 3.7 [Ref. 3-6] shows the settlement/step-difference at the top of levee locating at Panique along the Tarlac River damaged by soil liquefaction in the 1990 Philippines Earthquake. As shown in the Photograph, the bank slid and the step-difference of levee-top was about 1.5 m to 2.0 m.

Photo. 3.8 [Ref. 3-6] shows the collapse of special levees at Panique near Carvo Bridge along the Tarlac River in the 1989 Philippines Earthquake. The ground neighboring the special levees settled and seemed to be liquefied. Because the special levees were constructed with non-reinforced concrete, they seemed very weak for earthquakes structurally. The neighboring special levees which were retrofited with stones at the foot were not damaged and/or damaged slightly.

Photo. 3.9 [Ref. 3-6] shows the lateral movement of a special Levee in Dagupan City in the 1990 Philippines Earthquake. Because the ground around the special levee was liquefied and moved toward the river, the reinforced wall of the special levee was forced to move and/or tilt. The back fill was also settled and spread.

3.2.3 Underground Structure

The underground structures/facilities such as sewage pipe, water-supply pipe, gas pipe and duct are damaged severely by the ground hazard such as soil liquefaction. Because the underground structures are constructed in the ground, it is generally very difficult to repair and recover.

Photos. 3.10 and 3.11 show the up-lift of a sewage pipe and a manhole, respectively, by soil liquefaction in the 1964 Niigata Earthquake [Ref. 3-1]. Because the elevation of sewage pipe is very important for its function, the up-lift of pipes induces severe damage. Photo. 3.11 also shows the tilting of a building damaged by soil liquefaction.

Photo. 3.12 [Ref. 3-4] shows the floated gas tank at gas station in Dagupan City in the 1990 Philippines Earthquake. Such damage as up-lift of gas tanks occurred frequently in Dagupan City where liquefaction occurred severely. Because the unit weight of underground structures such as gas tank was smaller than that of its surrounding deposits, they were lifted up by the excess pore water pressure induced by soil liquefaction during Earthquake.

Photo. 3.13 [Ref. 3-4] shows the sticking out and bending of a water-supply pipe in Dagupan City in the 1990 Philippines Earthquake. The pipe was bent at a pipe joint by the compressive force induced by lateral deformation of surrounding ground liquefied during the Earthquake.

3.2.4 Others

Photo. 3.14 [Ref. 3-3] shows the sinking and tilting of a building in Dagupan City in the 1990 Philippines Earthquake. A number of buildings were damaged by liquefaction in Dagupan City, centering around the west-side district of Pantal River. The conditions on foundation of buildings, i.e., the structure and depth of foundations, seem inadequate to resist liquefaction. The liquefaction induced damage of buildings in this Earthquake is very similar to the one in the 1964 Niigata Earthquake when liquefaction occurred severely and caused a lot of damage (see Photo. 3.11).

3.3 EXPERIMENTAL STUDY ON DYNAMIC BEHAVIOR OF PILE FOUNDATION

Shaking table tests have been carried out at the Public Works Research Institute to investigate the behavior of both liquefied soils and pile foundations in liquefied soils experimentally [Ref. 3-7].

3.3.1 Test Conditions

A loose saturated sandy ground model was prepared in a square container: 6 meters long, 3 meters wide and 0.95 meters deep. The container was placed on a shaking table which was shaken in a sinusoidal wave with the frequency of 7 Hz as shown in Table 3.2. The magnitude of input acceleration ranged from 30 gals to 250 gals. Four pile foundation models with a different length/natural frequency were

set up in the ground model as shown in Fig. 3.2. The natural frequency of the Foundation Model-2 is the same as the frequency of input motion. The acceleration and pore water pressures in the ground model, acceleration and displacement of pile foundation models, and earth pressure on the pile models were measured at the points as shown in Fig. 3.2. The two longitudinal end walls were hinged at the bottom of walls to facilitate shear deformation of the ground model. The container has transparent glass in the two side walls to watch and measure the behavior of ground model during shaking.

3.3.2 Behavior of Liquefied Ground and Pile Foundation

Fig. 3.3 shows an example of the distribution of completely liquefied zones for several elapsed times (t :second) for Test No. 2 of Table 3.2. In this study, the extent of soil liquefaction at a certain depth was defined by the ratio of the excess pore water pressure (Δu) to the effective overburden pressure (σ_v') at a given depth. Because the zones with the ratio ($\Delta u/\sigma_v'$) of 1.0 can be judged to be liquefied completely, the completely liquefied zone with the thickness of H_L seems to spread from the ground surface to the bottom of the container according to the elapsed time. The ratio of the thickness H_L to the total thickness of a ground model (H_L/H_0) indicates the extent of completely liquefied zones.

Fig. 3.4 shows the relation between displacement at the top of the Foundation Model-2 and liquefaction extent of the ground model. From this figure, it can be seen that the displacement at the top of the piles tend to increase according to the increase of completely liquefied zones and input acceleration. The increase of the displacement of piles may be caused by the decrease of the horizontal bearing capacity of liquefied ground.

3.4 EXPERIMENTAL STUDY ON DYNAMIC BEHAVIOR OF UNDERGROUND PIPE

Shaking table tests on the behavior of a pipe in the liquefiable layer have been carried out at the Public Works Research Institute [Ref. 3-8] .

3.4.1 Test Conditions

Fig. 3.5 shows a liquefiable layer model and a pipe model laid in it. The layer model was prepared in a square container placed on a shaking table. The loose sand liquefiable layer was 0.3 meter deep and set on the stiff/nonliquefiable layer. The pipe model made of vinyl chloride was 1.0 meter long and the diameter and thickness were 60 millimeter and 1.8 millimeter, respectively. One of the pipe ends was free and the other one was hinged with a rigid beam hinged at the wall of a shaking table. Bending and axial strain gages were put on the upper/lower side of pipe model and two sides of it, respectively, and two tension meters connected with wires between pipe ends and the bottom of a shaking table were set to measure the time history of up-lift force to a pipe model. The acceleration and pore water pressure were also measured in the liquefiable layer.

The shaking table was driven by sinusoidal waves with a frequency of 5 Hz for 8 seconds, and the magnitude of input acceleration ranged from 80 to 207 gals as shown in Table 3.3.

3.4.2 Behavior of Pipes in the Liquefied Layer

The pipe model was forced to lift up according to the progress of soil liquefaction. Fig. 3.6 (a) shows the relations between up-lift force acting on pipe model and excess pore water pressure in the liquefied layer. From this figure, it can be indicated that in the case of Step-1, the layer was not liquefied so much that the up-lift force was very small, and on the other hand, in the case of Step-2, the up-lift force increased according to the increase of excess pore water pressure, in other words, to the progress of soil liquefaction. Fig. 3.6 (b) shows the relations between bending moment of a pipe calculated with measured strain gages which were same with Fig. 3.6 (a).

The maximum tension loads of left-end and right-end tension meters were 2.5 kgf and 2.0 kgf, respectively. On the other hand, because the weight of the pipe was 0.8 kgf, the volume of pipe model was $2,827 \text{ cm}^3$ and the unit weight of liquefied soils was 1.95 gf/cm^3 , the up-lift force acting on the pipe could be calculated to be 4.7 kgf ($= 2,827 \text{ cm}^3 \times 1.95 \text{ gf/cm}^3 - 0.8 \text{ kgf}$) when the soil layer was estimated to be liquefied perfectly. As mention above, the measured up-lift load 4.5 kgf ($= 2.5 + 2.0 \text{ kgf}$) was almost the same as the calculated one indicating that the layer liquefied perfectly and excess pore water pressure could be estimated by the structural condition of the pipe.

Fig. 3.7 shows the time history of bending moment distribution along the pipe. The Maximum bending moment occurred near the middle of a pipe with two free or hinged ends. According to the progress of liquefaction, the bending moments increased because the pipe was forced to heave between the two pipe ends.

From this study, it can be indicated that an underground pipe is forced to lift up by the excess pore water pressure induced by soil liquefaction. Furthermore, the upward displacement or the deformation of a pipe by up-lift force begins to increase rapidly when soil liquefaction progresses considerably. In other words, the effects of soil liquefaction against an underground pipe are small when the level of soil liquefaction or magnitude of excess pore water pressure is small.

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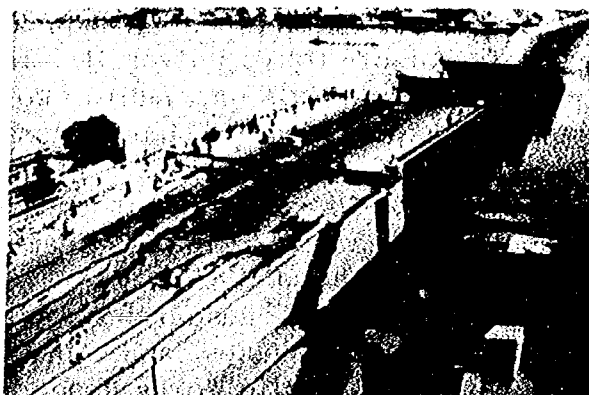


**Photo. 3.1 Girder Falling of Syowa-Ohashi Bridge
in the 1964 Niigata Earthquake [Ref. 3-1]**



**Photo. 3.2 Damage of Approach Embankment of Gomyokoh
Bridge in the 1983 Nihon-Kai-Chubu
Earthquake [Ref. 3-2]**

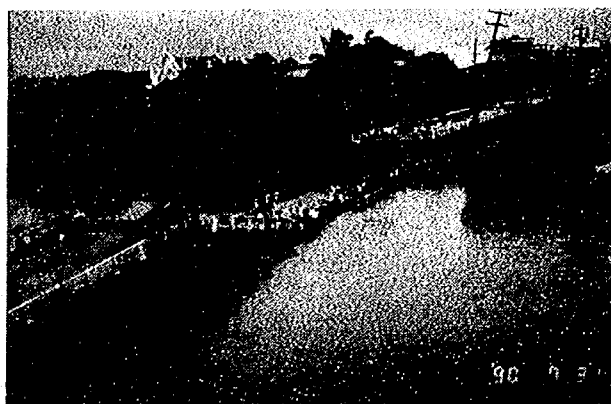


**Photo. 3.3 Girder Falling of Magsaysay Bridge in
the 1990 Philippines Earthquake [Ref. 3-3]**



Photo. 3.4 Girder Falling of Carmen Bridge in the 1990 Philippines Earthquake [Ref. 3-4]



Photo. 3.5 Damage of Approach Embankment in the 1990 Philippines Earthquake [Ref. 3-5]



Photo. 3.6 Sliding and Settlement of Levee in the 1983 Nihon-Kai-Chubu Earthquake [Ref. 3-2]



**Photo. 3.7 Slide and Settlement at the Top of Levee
in the 1990 Philippines Earthquake [Ref. 3-6]**

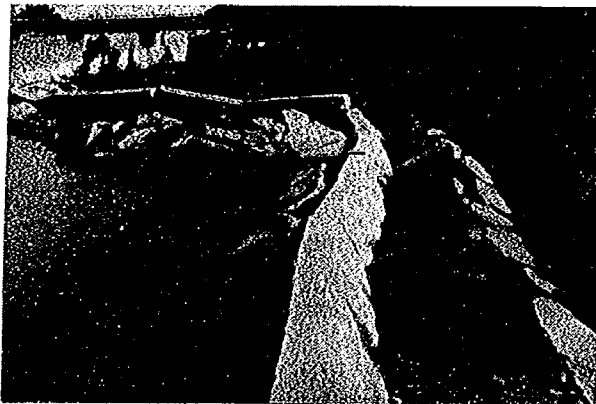


**Photo. 3.8 Collapse of Special Levee in the
1990 Philippines Earthquake [Ref. 3-6]**



**Photo. 3.9 Lateral Movement and Tilting of Special Levee
in the 1990 Philippines Earthquake [Ref. 3-6]**



Photo. 3.10 Up-lift of Sewage Pipe in the 1964 Niigata Earthquake [Ref. 3-1]



Photo. 3.11 Up-lift of Manhole and Tilting/Settlement of Building in the 1964 Niigata Earthquake [Ref. 3-1]



Photo. 3.12 Up-lift of Tanks at Gas Station in the 1990 Philippines Earthquake [Ref. 3-4]



**Photo. 3.13 Sticking Out of Water Supply Pipe in the
1990 Philippines Earthquake [Ref. 3-4]**

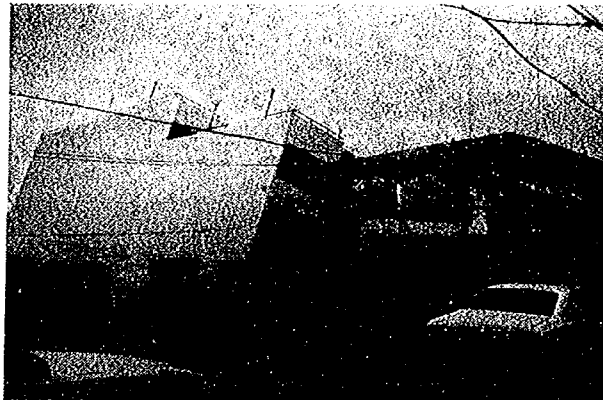


**Photo. 3.14 Tilting and Settlement of Building in the
1990 Philippines Earthquake [Ref. 3-3]**

Table 3.1 Liquefaction-Induced Hazard of Structures

Structures	Liquefaction-Induced Damage of Structures	Ground Hazard of Interest
Bridge	• Falling of girder • Tilting of abutment/pier • Settlement of abutment/pier/foundation • Collapse of foundation • Displacement of bearing support • Others	• Decrease of bearing capacity of ground • Lateral ground flow • Sliding of approach embankment • Settlement of ground
Approach Embankment of Bridge	• Sliding • Crack • Step difference of abutment • Settlement • Others	• Decrease of bearing capacity of ground • Sliding • Settlement
Levee (Earth Bank)	• Sliding • Settlement • Crack • Others	• Decrease of bearing capacity of ground • Lateral ground flow • Sliding • Settlement
Special Levee	• Sliding • Tilting • Overturning • Slipping/Opening joint • Others • Settlement • Crack	• Decrease of bearing capacity of ground • Lateral ground flow • Sliding • Settlement
Underground Structure (Pipe, Duct, Manhole, Tank etc.)	• Floating • Coming-off/Shear of joint • Crack • Bending/Buckling • Blockade of pipe by liquefied sand • Others	• Increase of up-lift by pore water pressure • Lateral ground flow • Lateral ground displacement

Table 3.2 Input Conditions of Shaking Table Tests on Pile Foundation [Ref. 3-7]

Step No.	Input Motion	Input Acceleration(gal)
1	Sinusoidal 7 HZ	30
2		80
3		80
4		50
5		80
6	Duration Time 30 second	150
7		150
8		250

Table 3.3 Input Conditions of Shaking Table Tests on Underground Pipe [Ref. 3-8]

Step No.	Input Motion	Input Acceleration(gal)
1	Sinusoidal 5 HZ	80
2		98
3		115
4		134
5	Duration Time 8 second	170
6		207

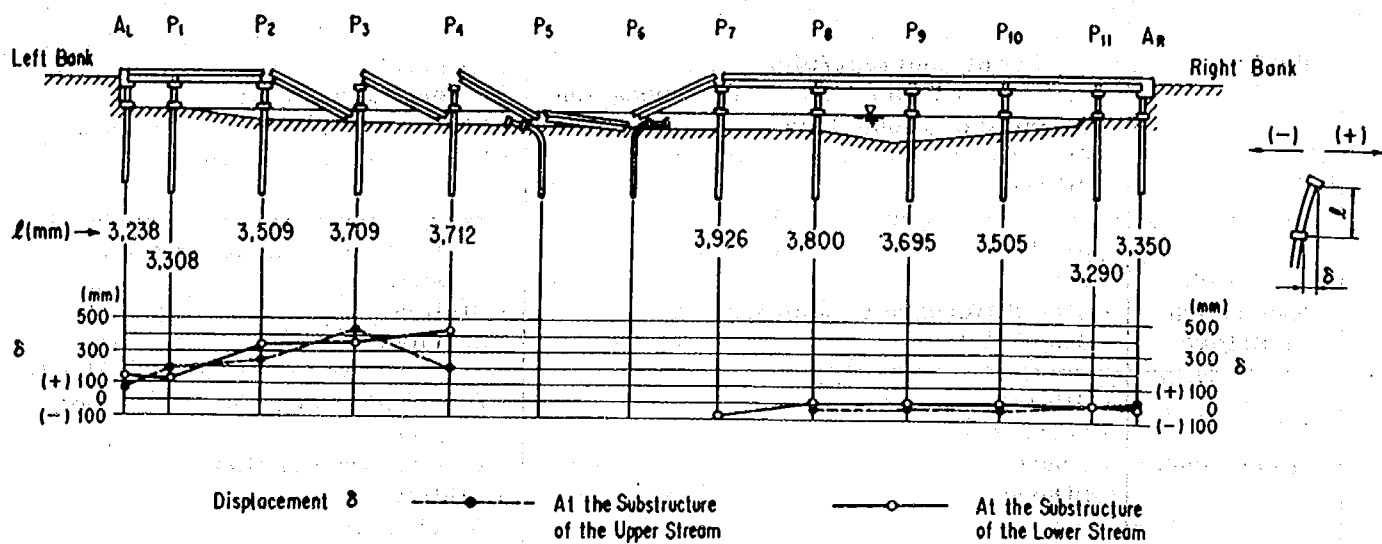


Fig. 3.1 Damage of Syowa-Ohashi Bridge in the 1964 Niigata Earthquake [Ref. 3-1]

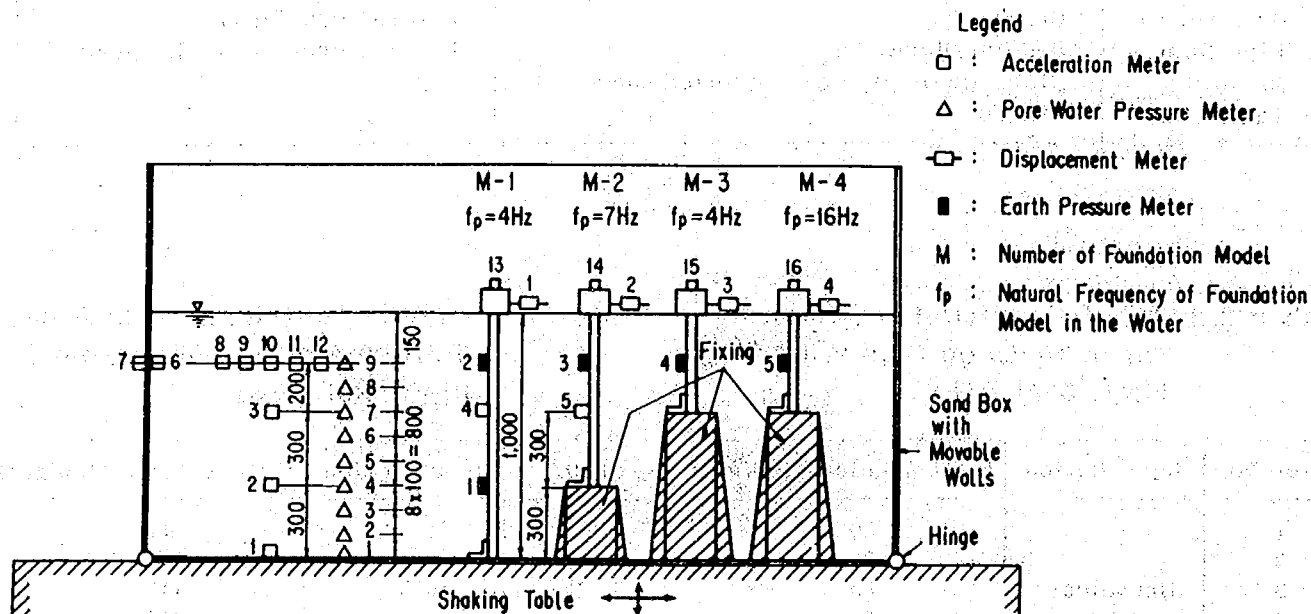


Fig. 3.2 Test Models and Measurement System [Ref. 3-7]

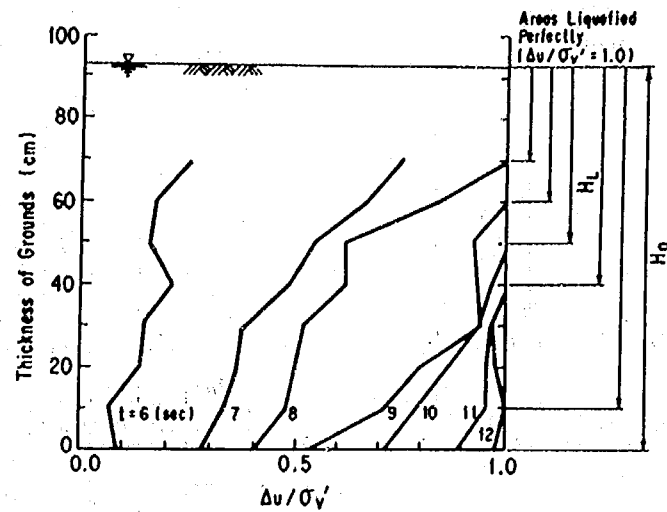


Fig. 3.3 Time History of Liquefied Zones [Ref. 3-7]

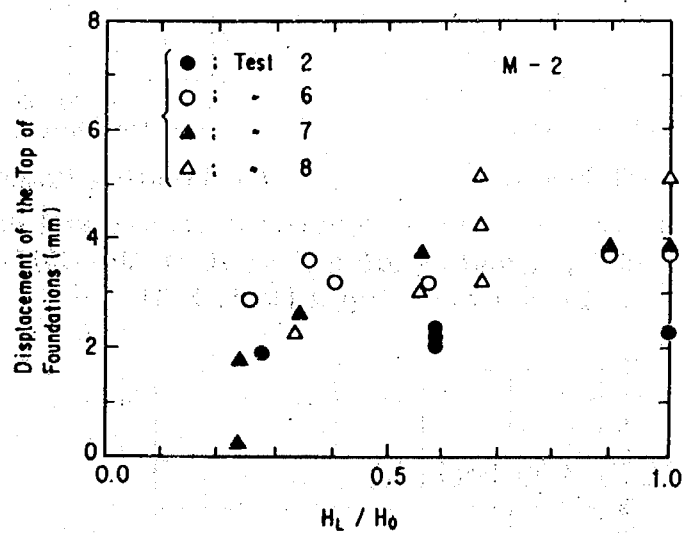


Fig. 3.4 Relation between Top Displacement of Foundation and Extent of Complete Liquefaction [Ref. 3-7]

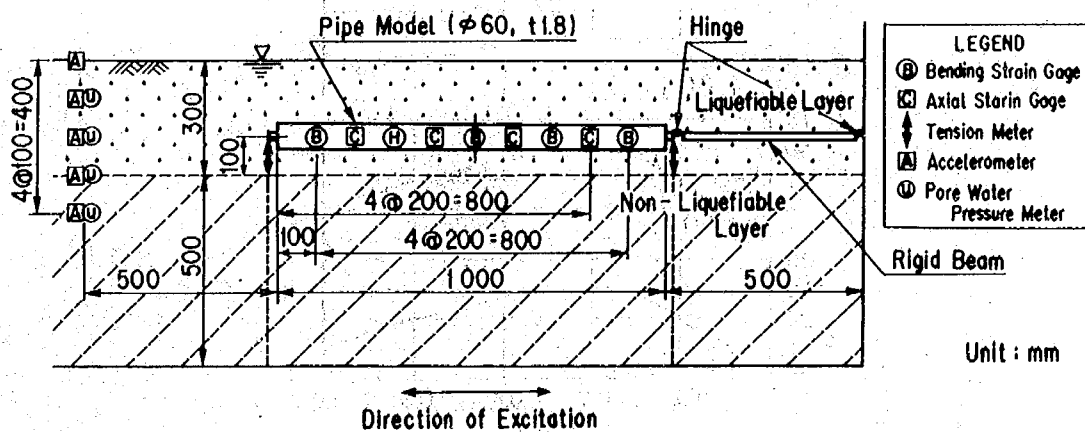


Fig. 3.5 Test Models and Measurement System [Ref. 3-8]

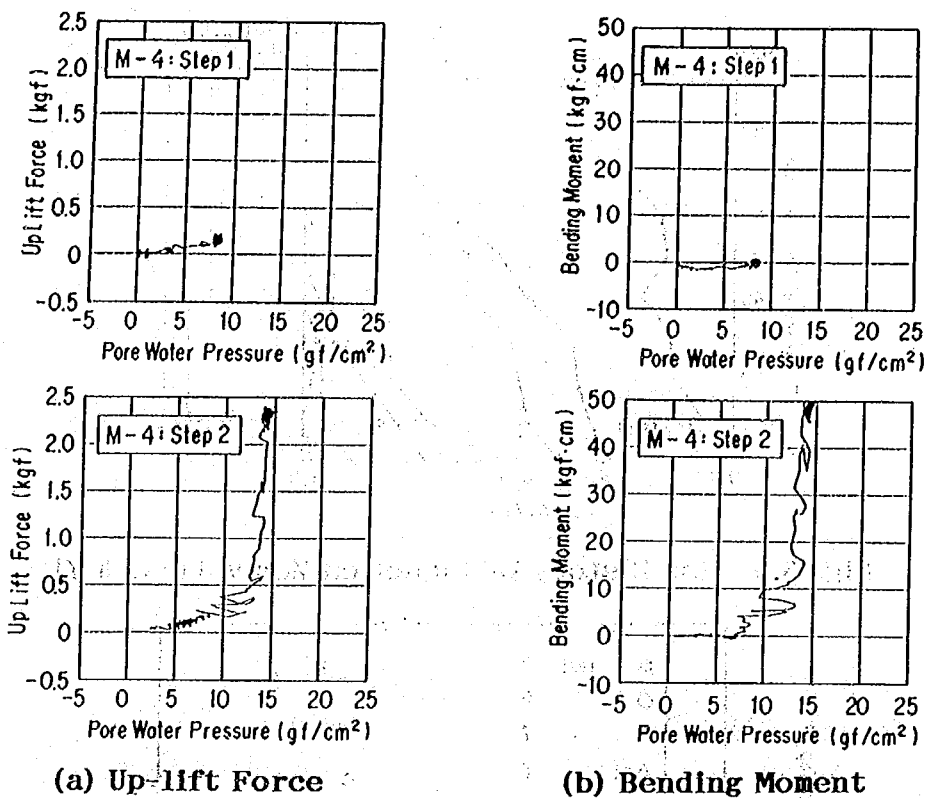


Fig. 3.6 Relation between Excess Pore Water Pressure and Up-lift Force at the End of Pipe, and Bending Moment at the Middle of Pipe in the Case of Step-1 and Step-2 [Ref. 3-8]

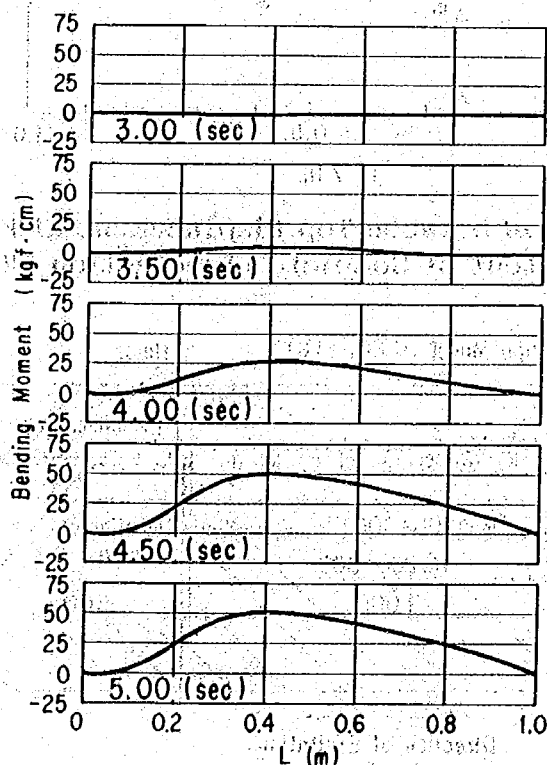


Fig. 3.7 Time History of Bending Moment Along Pipe [Ref. 3-8]

CHAPTER 4 PROCEDURES FOR ESTIMATING LIQUEFACTION POTENTIAL

4.1 CLASSIFICATION OF PROCEDURES

A number of studies and investigations on soil liquefaction have been conducted since the 1964 Niigata Earthquake in Japan. As a result of them, several simplified procedures to estimate liquefaction potential during earthquakes have been proposed and applied in the specifications and standards for the seismic design of structures.

A number of methods for estimating soil liquefaction potential have been studied, proposed and applied in Japan. For example, the methods can be classified into three groups as shown in Table 4.1, that is, empirical methods based on the liquefaction phenomena experienced in the past earthquakes, methods based on dynamic mechanism of soil liquefaction and methods applying and combining the above two methods. Details of each method will be introduced in the following sections.

4.2 EMPIRICAL METHODS BASED ON PHENOMENA IN PAST EARTHQUAKES

Fundamental characteristics for estimating liquefaction potential can be known qualitatively or quantitatively from the liquefaction phenomena and experiences during earthquakes. These empirical methods are generally applied to estimate the liquefaction potential simply and roughly.

4.2.1 Maximum Epicentral Distance of Liquefaction Occurrence

Because sandy layers become more liquefiable according to the increase of seismic ground motion and on the other hand, the intensity of seismic ground motion generally decreases far away from the epicenter, the occurrence of soil liquefaction can be estimated based on the epicentral distance at the site of interest. The relation between epicentral distance of sites where soil liquefaction phenomena were observed in the past earthquakes and the Earthquake Magnitude (M) is drawn as shown in Fig. 4.1 [Ref. 4-6]. Based on the relation such as Fig. 4.1, the maximum epicentral distance can be estimated roughly according to the Earthquake Magnitude of interest.

4.2.2 Ground Water Level and Depth of Layers

Because sandy layers become less liquefiable according to the increase of effective overburden/confining pressure, and the seismic ground-motion/load generally decreases with the ground depth of interest, the ground water level and/or the depth of layers can be used as one of the parameters to estimate the liquefaction potential.

Fig. 4.2 shows the relation between the liquefaction resistance factor F_L (see 4.3.2) and the depth of layers [Ref. 4-2]. F_L -Values were calculated at 23 points in Niigata City where boring data could be obtained and estimated to be liquefied or not in the 1964 Niigata Earthquake. It can be noted that the almost liquefied layers with the F_L -Values less than 1.0 are located in the depth of 0 to 10 meter, and on the other hand, somewhat liquefied layers are also located in the depth of 10 to 20 meter. The depth of layers seems one of factors to estimate liquefaction potential.

Fig. 4.3 shows the relation between underground water level (H_w) or thickness of non-liquefied layers at surface (H_o) and estimated thickness of saturated sandy layers (H_s) for both 64 liquefied points and 23 non-liquefied points in the past 6 earthquakes in Japan such as the 1891 Nohbi Earthquake ($M=8.0$), the 1944 Tohankai Earthquake ($M=8.0$), the 1948 Fuku Earthquake ($M=7.3$), the 1964 Niigata Earthquake ($M=7.5$) and the Miyagi-Ken-Oki Earthquake [Ref. 4-7]. Based on Fig. 4.3, it can be known that soil liquefaction is difficult to occur and/or does not reach to the ground surface when the ground water level or thickness of non-liquefied surface layer is larger than 2 to 3 meter or the thickness of saturated sandy layer is less than 2 to 3 meter.

4.2.3 Grain Size of Soils

Clayey/silty soils are generally difficult to liquefy because soil particles are dense and cohesive. On the other hand, gravelly soils are also difficult to liquefy because excess pore water pressure disappears rapidly during an earthquake. Fig. 4.4 shows an example of grain size distributions of sands boiled in the past earthquakes [Ref. 4-8]. The soil data on the 1990 Philippines Earthquake were also drawn. Fig. 4.5 shows an example of a range of grain size distribution to estimate liquefaction potential with use of grain size distribution [Ref. 4-9]. As mentioned above, grain size of soils is very important factor and effective to estimate liquefaction potential of soils/layers of interest.

4.2.4 Geographical Condition

Because the loose saturated sandy layers are generally liquefiable during earthquakes and the geographical ground condition indicates roughly soil condition, the geographical condition can be used as one of factors to estimate liquefaction potential. Fig. 4.6 shows the liquefied area and geographical conditions in Niigata City damaged by the 1964 Niigata Earthquake [Ref. 4-2]. In Fig. 4.6, it can be seen that almost all of the liquefied area is located at old riverbeds, reclaimed lands and inter sand dune lowlands.

The liquefaction potential level can be classified into three groups: A, B and C, based on the geographical conditions such as Table 4.2 [Ref. 4-1]. In this table, existing river bed, old river bed and so on should be judged as highly liquefiable sites. Fig. 4.7 summarizes the classification of 501 sites where liquefaction phenomena observed during past earthquakes in Japan according to

the standard of Table 4.2. Because about 75 percent of liquefied sites were classified into Group-A and about 95 percent of sites into Group-A and Group-B, the classification of liquefaction potential by Table 4.2 could be considered suitable and useful to estimate liquefaction potential roughly.

4.2.5 Maximum Blow Count by Standard Penetration Test

Because the soil liquefaction is related to the dynamic strength of soils, the blow count by standard penetration test (SPT) is commonly used as one of factors to estimate liquefaction potential. Fig. 4.8 shows examples of critical SPT N-values to cause soil liquefaction with respect to depth proposed in the past in Japan [Refs. 4-11, 4-14, 4-15, 4-16 and 4-17]. Although critical N-values are considerably different among the various proposals, it is possible to refer to them to estimate liquefaction potential roughly.

4.3 METHODS BASED ON DYNAMIC MECHANISM OF SOIL LIQUEFACTION

4.3.1 Dynamic Resistance and Dynamic Load Against Soils

Several methods have been proposed to estimate liquefaction potential considering dynamic resistance of soils and dynamic load induced by seismic ground motion in Japan. In this section, a simplified procedure which has been studied and proposed at the Public Works Research Institute is introduced.

4.3.1.1 Liquefaction Resistance Factor, F_L

Fig. 4.9 illustrates schematically dynamic behavior of a subsoil against seismic ground motion during earthquakes [Refs. 4-2 and 4-4]. Fig. 4.9(a) identifies the cyclic triaxial strength (R) of soil specimens by dynamic triaxial tests with sinusoidal stress waves, and Fig. 4.9(b) identifies the dynamic shear stress (τ) or the dynamic load (L) against a soil element when a bedrock motion with a peak acceleration of α_B or a surface motion with a peak acceleration of α_S is subjected to the sublayer.

The liquefaction resistance factor F_L is defined as :

$$F_L = \frac{R}{L} \quad (4-1)$$

where R is the in situ dynamic shear strength of soils and L is the dynamic load applied to the soils. Simplified methods to estimate the dynamic shear strength of soils and the dynamic load (L) are introduced as follows.

4.3.1.2 Resistance of Soil Element to Dynamic Load

Fig. 4.10 shows an example of dynamic triaxial test results for undisturbed soil specimens sampled from the site with the effective confining pressure (σ_o') equivalent to the estimated in situ effective overburden pressure (σ_v') [Ref. 4-2]. As shown in Fig. 4.10, three or four soil specimens are needed to estimate the cyclic triaxial strength, changing the axial load (σ_{dp}) or the input stress ratio ($\sigma_{dp}/2\sigma_o'$). The data are plotted according to the number of cycles of loading (N_o) and the double amplitude (DA) of axial strain of 1, 2, 6 and 10 % and also the points where the pore water pressure (Δu) becomes equal to the effective confining pressure (σ_o') are shown.

The cyclic shear strength of a soil specimen obtained directly from the dynamic triaxial test (R_1) is defined as

$$R_1 = \frac{\sigma_{dp}}{2\sigma_o'} \quad \text{for } N_o = 20 \text{ and DA} = 5\% \quad (4-2)$$

which is the stress ratio ($\sigma_{dp}/2\sigma_o'$) when the numbers of cyclic loading (N_o) is 20 for the double amplitude of axial strain (DA) of 5 % [Ref. 4-2].

In situ dynamic shear strength (R) can be obtained as follows, based on the cyclic triaxial shear strength (R_1) defined in equation (4-2).

$$R = C_1 \cdot C_2 \cdot C_3 \cdot C_4 \cdot C_5 \cdot R_1 \quad (4-3)$$

Where C_1 to C_5 are correction factors accounting for different conditions between the laboratory test state and the in situ one as follows:

C_1 : Correction factor for the difference of confining pressure, and $C_1 = (1 + 2K_o)/3$, where K_o is the coefficient of earth pressure at rest [Ref. 4-22] and is generally estimated to be 0.5 and then C_1 is 2/3.

C_2 : Correction factor for the loading pattern between the sinusoidal wave in a dynamic triaxial test and the in situ earthquake wave. Values of C_2 are 1/0.55 and 1/0.7 for a shock-type seismic and a cyclic-type seismic motion, respectively [Ref. 4-23], and C_2 can be estimated to be 1.62 on the average.

C_3 : Correction factor for soil disturbance by sampling and handling of specimens. It is assumed to be more than 1.0.

C_4 : Correction factor for soil densification by sampling and handling of specimens. It is assumed to be less than 1.0.

C_5 : Correction factor for multi-direction shaking, saturation of soils and so on. Seed estimated the factor, C_5 , to be 0.9 [Ref. 4-24].

If the product of C_3 and C_4 could be assumed to be unity, equation (4-3) becomes as follows:

$$R = (2/3) \times 1.6 \times 1.0 \times 0.9 \times R_1 = R_1 \quad (4-4)$$

Equation (4-4) indicates that the in situ dynamic shear strength (R) can be nearly equivalent to the cyclic triaxial shear strength (R_1) for undisturbed specimens sampled from the field.

Fig. 4.11 shows the relation between the mean grain size (D_{50}) and DR_1^* indicated in equation (4-5), obtained from dynamic triaxial tests for a number of undrained specimens sampled at in situ sites [Ref. 4-2].

$$DR_1^* = R_1 - 0.0042 \cdot D_r^* \quad (4-5)$$

where R_1 is the cyclic shear strength and D_r^* is the empirical equation proposed by Gibbs and Holtz [Ref. 4-18] as

$$D_r^* = 21\sqrt{N/(\sigma_v' + 0.7)} \quad (4-6)$$

where N is the blow counts by the standard penetration test (SPT) and σ_v' is the effective overburden pressure (in kgf/cm²) in situ. Based on Fig. 4.11, DR_1^* can be approximately expressed by

$$DR_1^* = \begin{cases} 0.225 \log_{10}(0.35/D_{50}) & (\text{for } 0.02 \leq D_{50} \leq 0.6 \text{ mm}) \\ -0.05 & (\text{for } 0.6 \leq D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (4-7)$$

From equations (4-5), (4-6) and (4-7), the cyclic shear strength (R_1) can be expressed by

$$R_1 = \begin{cases} 0.0882\sqrt{N/(\sigma_v' + 0.7)} + 0.225 \log_{10}(0.35/D_{50}) & (\text{for } 0.02 \leq D_{50} \leq 0.6 \text{ mm}) \\ 0.0882\sqrt{N/(\sigma_v' + 0.7)} - 0.05 & (\text{for } 0.6 \leq D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (4-8)$$

The effective overburden pressure (σ_v' in kg·f/cm²) at a certain depth of subsurface ground can be obtained by (see Fig. 4.9)

$$\sigma_v' = \frac{h_w}{10} \left[\int_0^{h_w} \gamma_t \cdot dz + \int_{h_w}^Z \gamma' \cdot dz \right] \quad (4-9)$$

where γ_t is the unit weight of soil above the water table (t·f/m³), γ' is the unit weight of soil below the water table (t·f/m³), h_w is the depth of the water table (m) and Z is the depth of a soil element (m).

With the use of equations (4-4) and (4-8), in situ dynamic shear strength of soil liquefaction (R) can be approximately estimated as functions of SPT N-values, effective overburden pressure (σ_v' in kg·f/cm²) and mean grain size (D_{50} in mm).

4.3.1.3 Dynamic Load Applied to Soil Elements

Dynamic load (L) forced to a soil element by seismic ground motions can be expressed by (see Fig.4.9)

$$L = \tau_{\max} / \sigma_v' \quad (4-10)$$

where $\tau_{\max}$ is the maximum single amplitude of shear stress in a soil element and σ_v' is the overburden pressure at the static state. After Seed and Idriss [Ref. 4-19], the dynamic load can be estimated by (see Fig. 4.9(b))

$$L = \frac{\alpha_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \quad (4-11)$$

where $\alpha_{\max}$ (in gals) is the maximum acceleration on the ground surface, g is the acceleration of gravity (= 980 gals), σ_v is the total overburden pressure, σ_v' is the effective overburden pressure and r_d is the reduction factor of the dynamic shear stress in soils accounting for the elastic ground deformation.

Figs. 4.12(a) and 4.12(b) show examples of the results of one-dimensional earthquake response analysis with the computer program "SHAKE" [Ref. 4-20] for a soft alluvial ground and a very soft alluvial ground, respectively [Ref. 4-21]. Based on the study on the reduction factor in the Public Works Research Institute, the following equation has been proposed.

$$r_d = 1 - 0.015 \cdot Z \quad (4-12)$$

where Z is the depth (in m) of a soil element or a layer of interest.

Figs. 4.13(a) and 4.13(b) show examples to calculate liquefaction resistance factor F_L with the use of equations (4-1), (4-8) and (4-11) for a nonliquefied site and a liquefied site in the 1964 Niigata Earthquake, respectively [Ref. 4-2]. As shown in Fig. 4.13, the simplified method with the use of the liquefaction resistance factor F_L seems adequate to estimate liquefaction potential.

4.4 METHODS MODIFYING EMPIRICAL AND DYNAMIC METHODS

The liquefaction resistance factor F_L mentioned above is very useful to estimate not only the liquefaction potential of a soil element or a layer of interest but also the distribution of liquefaction potential from the ground surface to the base ground. However, it is necessary to establish another index in place of the F_L -value to estimate the total liquefaction potential of a ground of interest for zoning of liquefaction potential widely. In this section, two liquefaction potential indices studied in the Public Works Research Institute will be introduced to estimate liquefaction potential of not a soil element or layer but a ground.

The liquefaction potential of a soil element or layer to cause damage of structures constructed on liquefiable ground is different according to the magnitude of F_L -value less than 1.0 and the depth of a liquefiable/liquefied layer. Two parameters were proposed to estimate the ground liquefaction potential: liquefaction potential index, P_L [Ref. 4-2] and site liquefaction potential, I_{PL} [Refs. 4-12 and 4-13] as (see Fig. 4.14)

$$P_L = \int_0^{20} (1 - F_L) \cdot W(z) dz \quad (4-13)$$

where P_L is the liquefaction potential index [$0 \leq P_L \leq 100$], $W(z)$ is the function of weighting based on the depth of layer which was estimated as follows in this paper

$$W(z) = 10 - 0.5 \cdot z \quad (4-14)$$

$$I_{PL} = \frac{H^*}{20} \quad (4-15)$$

$$H^* = H_1^* + H_2^* \quad (4-16)$$

$$H_1^* = 1.5H_{FL1} + 1.0H_{FL2} + 0.5H_{FL3} \quad (4-17)$$

$$H_2^* = 1.0H_{FL1} + 0.5H_{FL2} \quad (4-18)$$

where I_{PL} is the site liquefaction potential [$0 \leq I_{PL} \leq 1.25$], H^* is the modified thickness of liquefiable layers, H_1^* is the modified thickness of liquefiable layer at the depth of 0 to 10 m, H_2^* is the modified thickness of liquefiable layer at the depth of 10 to 20 m, H_{FL1} is the total thickness of layers with $F_L \leq 0.6$ (m), H_{FL2} is the one with $0.6 < F_L \leq 0.8$ (m) and H_{FL3} is the one with $0.8 < F_L \leq 1.0$ (m). The coefficients in the parameters H_1^* and H_2^* are decided for convenience sake, by referring the relation in Table 4.3 [Ref. 4-4] where bearing capacities of liquefied ground are either neglected or reduced by reduction factors D_E determined in accordance with F_L -values.

The P_L -value considers only the degree of soil liquefaction, and on the other hand, the I_{PL} -value considers both the degree of soil liquefaction and the effect of soil liquefaction against structures, for example, a pile foundation of highway bridge, relating to the bearing capacity of ground.

Fig. 4.15 shows the number of cases and accumulative percentage of P_L -values calculated at 64 liquefied sites and 23 nonliquefied sites in the past 6 earthquakes in Japan [Ref. 4-2]. From Fig. 4.15, the liquefaction potential was classified into 4 groups as shown in Table 4.4.

Fig. 4.16 shows the relation between P_L -values and I_{PL} -values at the same sites. The following relation can be indicated

P_L	0	5	15	100
I_{PL}	0	0.1~0.2	0.3~0.4	1.25

4.5 ESTIMATING METHODS OF LIQUEFACTION POTENTIAL IN DESIGN CODES OF CIVIL ENGINEERING STRUCTURES IN JAPAN

4.5.1 Classification of Design Codes

Table 4.5 summarizes the estimating methods of liquefaction potential in earthquake resistance design for typical civil engineering structures such as highway bridges, water supply facilities, sewage facilities, common utility duct, quaywalls/piers and buildings in Japan [Ref. 4-3] .

In this chapter, the specifications for highway bridges will be introduced as one of the design methods for soil liquefaction.

4.5.2 Specifications for Highway Bridges [Refs. 4-4 and 4-5]

1) Sandy Soil Layers Examined for Liquefaction

The sandy layers with the following conditions should be considered to be vulnerable to liquefy during earthquakes, and their liquefaction potential should be examined.

- ① Saturated alluvial sandy layers
- ② Layer's depth within 20 m from ground surface
- ③ Ground water level within 10 m from ground surface
- ④ D_{50} -values between 0.02 mm and 2.0 mm for grain size accumulation curve

2) Simplified Procedure for Liquefaction Potential

As for sandy soil layers judged to be vulnerable to liquefy, liquefaction potential should be examined based on liquefaction resistance factor F_L defined by the following equation.

$$F_L = R/L \quad (4-19)$$

where F_L : Liquefaction resistance factor

R : Resistance of soil elements to dynamic loads, and

$$R = R_1 + R_2 + R_3 \quad (4-20)$$

$$R_1 = 0.0882 \sqrt{N/(\sigma'_v + 0.7)} \quad (4-21)$$

$$R_2 = \begin{cases} 0.19 & (0.02\text{mm} \leq D_{50} \leq 0.05\text{mm}) \\ 0.225 \log_{10}(0.35/D_{50}) & (0.05\text{mm} < D_{50} \leq 0.6 \text{ mm}) \\ -0.05 & (0.6 \text{ mm} < D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (4-22)$$

$$R_3 = \begin{cases} 0.0 & (0 \% \leq F_o \leq 40 \%) \\ 0.004F_o - 0.16 & (40 \% < F_o \leq 100 \%) \end{cases} \quad (4-23)$$

L : Dynamic load to soil elements

$$L = r_d \cdot K_a \cdot (\sigma_v / \sigma_v') \quad (4-24)$$

$$r_d = 1.0 - 0.015 \cdot z \quad (4-25)$$

$$K_a = C_z \cdot C_G \cdot C_I \cdot K_{so} \quad (4-26)$$

R_1 : Resistance based on relative density (see Fig. 4.17)

R_2 : Resistance based on D_{50} -values (see Fig. 4.18)

R_3 : Resistance based on fines content (see Fig. 4.19)

N : Blow count by standard penetration test

D_{50} : Mean grain size (mm),

F_o : Fines content (%)

z : Depth from the actual ground surface (m)

K_a : Seismic coefficient for evaluating liquefaction

C_z : Seismic zone factor (see Fig. 4.21)

C_G : Ground condition factor (see Table 4.6)

C_I : Importance factor (see Table 4.7)

K_{so} : Standard seismic coefficient (= 0.15)

σ_v : Total overburden (kgf/cm²)

σ_v' : Effective overburden pressure at the static condition (kgf/cm²)

Soil layers with liquefaction resistance factor F_L smaller than 1.0 are judged to liquefy during earthquakes. When detailed data of the soils are not obtained, the values of unit weight below and/or above ground water level, mean grain size and fines content of soils can be estimated roughly based on soil classifications with use of Table 4.8.

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**Table 4.1 Typical Methods for Estimating
Soil Liquefaction Potential**

1. Empirical Methods Based on Phenomena in Past Earthquakes (Approach-1)	1) Maximum Epicentral Distance
	2) Ground Water Level/Depth of Layer
	3) Grain Size of Soils
	4) Geographical Condition
	5) Maximum Blow Count by Standard Penetration Test
2. Methods Based on Dynamic Mechanism of Liquefaction (Approach-2)	6) Dynamic Resistance and Dynamic Load Against Soils (Ex. Liquefaction Resistance Factor: F_L)
	7) Pore Water Pressure and Effective Overburden Pressure
3. Methods Applying/ Combining Approach -1 and Approach-2 (Approach-3)	8) Distribution of Liquefaction Resistance Factor (No.1) (Ex. Liquefaction Potential Index: P_L)
	9) Distribution of Liquefaction Resistance Factor (No.2) (Ex. Site Liquefaction Potential: I_{PL})

**Table 4.2 Geographical Condition and
Its Liquefaction Potential [Ref. 4-1]**

Topographical Group	Liquefaction Potential	Topographical Conditions
A	Highly Possible to be Liquefied	Existing Channel, Existing River Bed, Old Channel, Old River Bed, Lowland between Sand Hills, Reclamation of Old Water Surface (Reclaimed Area of Seashore, Lake-shore, Paddy Field, etc.)
B	Possible to be Liquefied	Wet Land, Natural Bank, Sand Hill, Sandbar, Sandy Beach, Other than the above (A) and below (B)
C	Not Easy to be Liquefied	Plateau, Hill, Fand Land

**Table 4.3 Reduction of Bearing Capacity of Ground According
to Liquefaction Potential [Ref. 4-4] and Coefficient
for Modified Thickness**

F_L	Depth, Z (m)	Reduction Factor, D_E	Coefficient for * Modified Thickness
$F_L \leq 0.6$	$0 < Z \leq 10$	0	1.5 **
	$10 < Z \leq 20$	1/3	1.0
$0.6 < F_L \leq 0.8$	$0 < Z \leq 10$	1/3	1.0
	$10 < Z \leq 20$	2/3	0.5
$0.8 < F_L \leq 1.0$	$0 < Z \leq 10$	2/3	0.5
	$10 < Z \leq 20$	1	0.0 ***
$1.0 < F_L$	$0 < Z \leq 20$	1	

Note: * is assumed to calculate the modified thickness of the site liquefaction potential, I_{PL} .

** is assumed for 0 of D_E .

*** is assumed for 1 of D_E .

Table 4.4 Liquefaction Potential Index, P_L -Value and Its Liquefaction Potential [Ref. 4-2]

P_L -Value	Liquefaction Risk and Investigatio/Countermeasures Needed
$P_L = 0$	Liquefaction risk is very low. Detailed investigation is not generally needed.
$0 < P_L \leq 5$	Liquefaction risk is low. Further detailed investigation is needed for specially important structures.
$5 < P_L \leq 15$	Liquefaction risk is high. Further detailed investigation is needed for structures. Countermeasures of liquefaction is generally needed.
$15 < P_L$	Liquefaction risk is very high. Detailed investigation and countermeasures are needed.

Table 4.5 Estimating Methods of Liquefaction Potential in Earthquake Resistance Design for Typical Structures in Japan [Ref. 4-3]

Sht 1 of 2

Specifications	Estimating Methods	Ground Conditions Applied
Specifications for Highway Bridges. Part V. Earthquake Resistance Design (Japan Road Asociation : 1990. 2)	Liquefaction potential should be examined based on liquefaction resistance factor F_L , and soil layers with F_L smaller than 1.0 are judged to liquefy during earthquakes. $F_L = R/L$ where, R: Resistance of soil elements against dynamic loads $R = R_1 + R_2 + R_3$ L: Dynamic load to soil elements induced by earthquake $L = r_d \cdot K_a \cdot \sigma_v' / \sigma_v'$ $r_d = 1.0 - 0.015 \cdot z$ $K_a = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot K_{a0}$ $K_{a0} = 0.15$	1) Saturated alluvial sandy layer 2) Ground water table within 10m from ground surface. 3) Layers within 20m from ground surface. 4) D_{50} -values on the grain size accumulation curve. $0.02 \leq D_{50} \leq 2.0 \text{ mm}$ 5) Fines content F_o on the grain $0 \leq F_o \leq 100 \%$ 6) Relative density D_r $D_r = 21\sqrt{N/(\sigma_v' + 0.7)} < 60 \%$
Guidelines of Earthquake Resistant Design for Water Supply Facilities (Japan Water Works Association : 1979.12)	The same as the above, i.e. with F_L . But R and K_a are a little different, as follows. $R = R_1 + R_2$ $K_a = \Delta_1 \cdot \Delta_2 \cdot K_{a0}$ $(\Delta_1 = \nu_1, \Delta_2 = \nu_2)$ The affect of fines content, F_o to R and importance factor (ν_3) for K_a are not included.	The same as the above without condition of 5) on Fines content F_o .

Specifications	Estimating Methods	Ground Conditions Applied
Guidelines of Earthquake Countermeasures for Sewage facilities (Japan Sewage Works Association : 1981. 9)	The same as the above. But K_0 is a little different, as follows. $K_0 = \Delta_1 \cdot \Delta_2 \cdot \Delta_3 \cdot K_{00}$ The importance factor $\Delta_3 (= \nu_3)$ is considered.	The same as the above.
Design Guidelines for Common Utility Ducts (Proposal) (Japan Road Association : 1986. 3)	The same as the above. ($K_0 = \nu_1 \cdot \nu_2 \cdot \nu_3 \cdot K_{00}$) Further the excess pore water pressure induced during earthquakes should be estimated with F_L for predicting uplift loads on utility ducts.	The same as the above added with the condition of N-value limit as follows. $N \leq 30$
Earthquake Resistance Design for Quaywalls and Piers (Japan Port and Harbor Association : 1989. 6)	Liquefaction potential is estimated based on grain-size distribution, equivalent N-value ($(N)_{0.60}$), equivalent acceleration α_{eq} , and fines content for each layer. Further, probability of liquefaction for the objective ground is judged considering thickness and depth of the ground, synthetically. $N+1.282(\sigma_{v0}'-0.66)$ $(N)_{0.60} = \frac{0.399(\sigma_{v0}'-0.66)+1}{\alpha_{eq} = 0.7 \times (\tau_{max}/\sigma_{v0}') \times g}$	1) Loose consolidated and saturated sandy layer. 2) Appropriate grain-size distribution. 3) Equivalent N-value, $(N)_{0.60}$ $2 \leq (N)_{0.60} \leq 40$ 4) Equivalent acceleration, α_{eq} $0 \leq \alpha_{eq} \leq 500$ gals 5) Fines content, F_c $0 \leq F_c (\%) (\leq 20)$ 6) Effective overburden pressure, $0 \leq \sigma_{v'} \leq 3$ kgf/cm ²
Design Guidelines for Foundation Structures of Buildings (Japan Building Association : 1988. 1)	Liquefaction potential should be examined based on liquefaction safety factor F_1 , and soil layers with F_1 smaller than 1.0 are judged to liquefy during earthquake. $F_1 = (\tau_1/\sigma_{s'})/(\tau_d/\sigma_{s'})$ where, Liquefaction Resistance Ratio $(\tau_1/\sigma_{s'}) \sim \gamma (\approx 5\%) \sim N_d$ Equivalent Shear Stress Ratio $(\tau_d/\sigma_{s'}) = r_n(\alpha_{max}/g)(\sigma_s/\sigma_{s'}) r_d$ $r_n = 0.1(M-1)$ $r_d = 1-0.015 \cdot z$ $\alpha_{max} = 200$ gals	1) Saturated sandy layer ($F_c \leq 20\%$) and/or poor silty layer ($20 < F_c \leq 50\%$). 2) Layers within 20m from ground surface. 3) Fines content, F_c $F_c \leq 50\%$ 4) Modified N-value, $N_d < 26$

[Note] R1 : Resistance related to relative density,

R2 : Resistance related to mean grain size,

R3 : Resistance related to fine content,

r_d : Reduction factor of dynamic load,

r_n : Modification factor for magnitude (M) of earthquake,

ν_1, Δ_1 : Modification factor for zone,

ν_2, Δ_2 : Modification factor for ground condition,

ν_3, Δ_3 : Modification factor for importance,

K_0 : Design seismic coefficient at ground surface,

K_{00} : Standard design seismic coefficient,

α_{max} : Maximum acceleration at ground surface

N : N-value, τ_{max} : Maximum shear stress, F_c : Fines content, D_{50} : Mean grain size,

σ_v, σ_s : Total overburden pressure

$\sigma_{v0}', \sigma_{s'}$: Effective overburden pressure at the static condition

Table 4.6 Ground Condition Factor, C_G [Refs. 4-4 and 4-5]

Ground Group	I	II	III
C_G	0.8	1.0	1.2

Table 4.7 Importance Factor, C_I [Refs. 4-4 and 4-5]

Group	C_I	Definition
1st class	1.0	Bridges on expressway (limited access highways), general national road and principal prefectural road. Important bridges on general prefectural road and municipal road.
2nd class	0.8	Other than the above

Table 4.8 Roughly Estimated Values of Unit Weight, Mean Grain Size and Fine Content of Soils [Ref. 4-4]

Soil Classification	Unit Weight below Ground Water Level γ_{sat} (tf/m ³)	Unit Weight above Ground Water Level γ_{dry} (tf/m ³)	Mean Grain Size D_{50} (mm)	Fines Content FC (%)
Topsoil	1.7	1.5	0.02	80
Silt	1.75	1.55	0.025	75
Sandy Silt	1.8	1.6	0.04	65
Silty Fine Sand	1.8	1.6	0.07	50
Very Fine Sand	1.85	1.65	0.1	40
Fine Sand	1.95	1.75	0.15	30
Middle Sand	2.0	1.8	0.35	10
Coarse Sand	2.0	1.8	0.6	0
Gravel	2.1	1.9	2.0	0

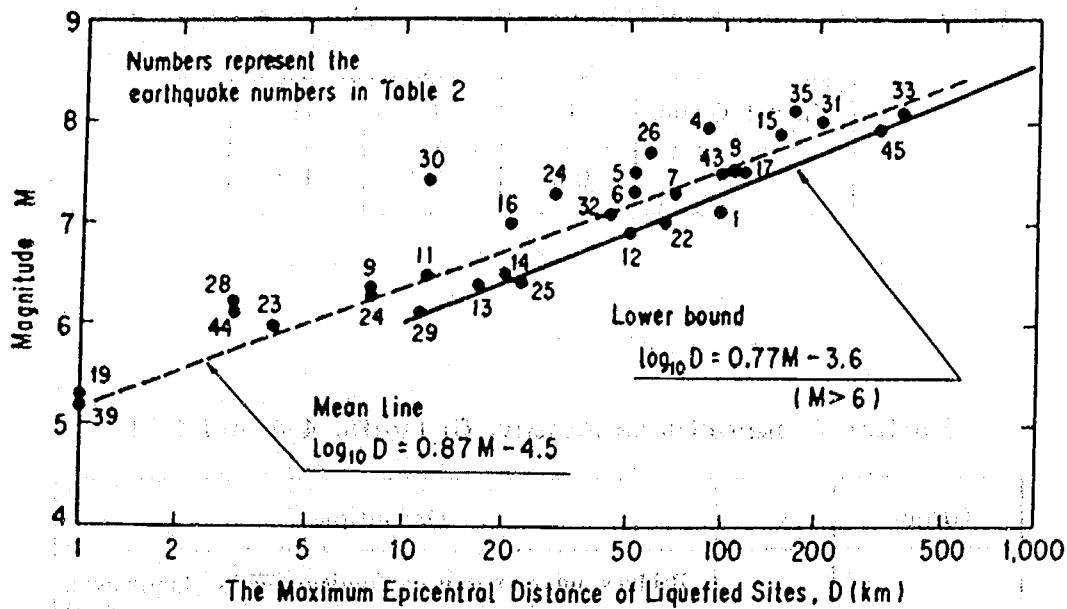


Fig. 4.1 Relation between Maximum Epicentral Distance of Liquefied Sites and Earthquake Magnitude [Refs. 4-6]

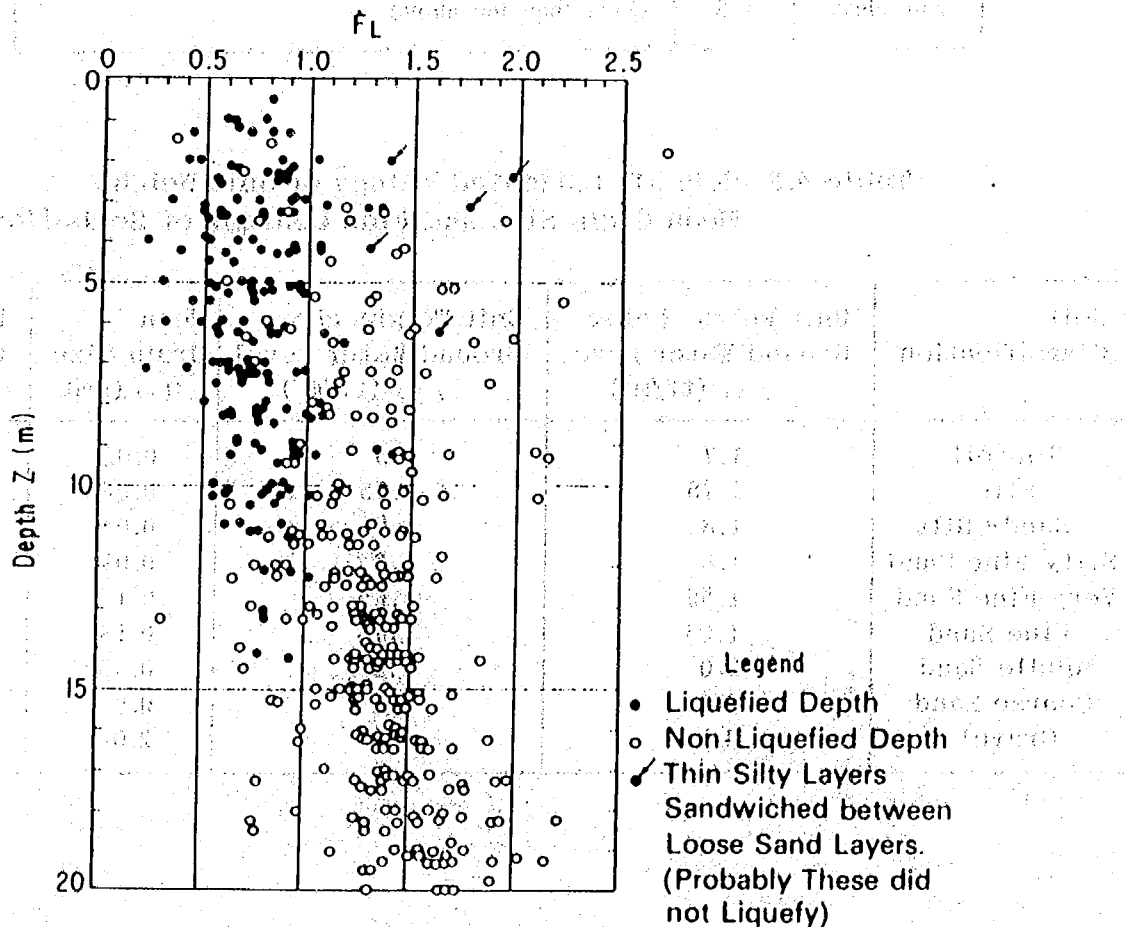


Fig. 4.2 F_L -Values Analysed at 23 Sites in Niigata City [Ref. 4-2]

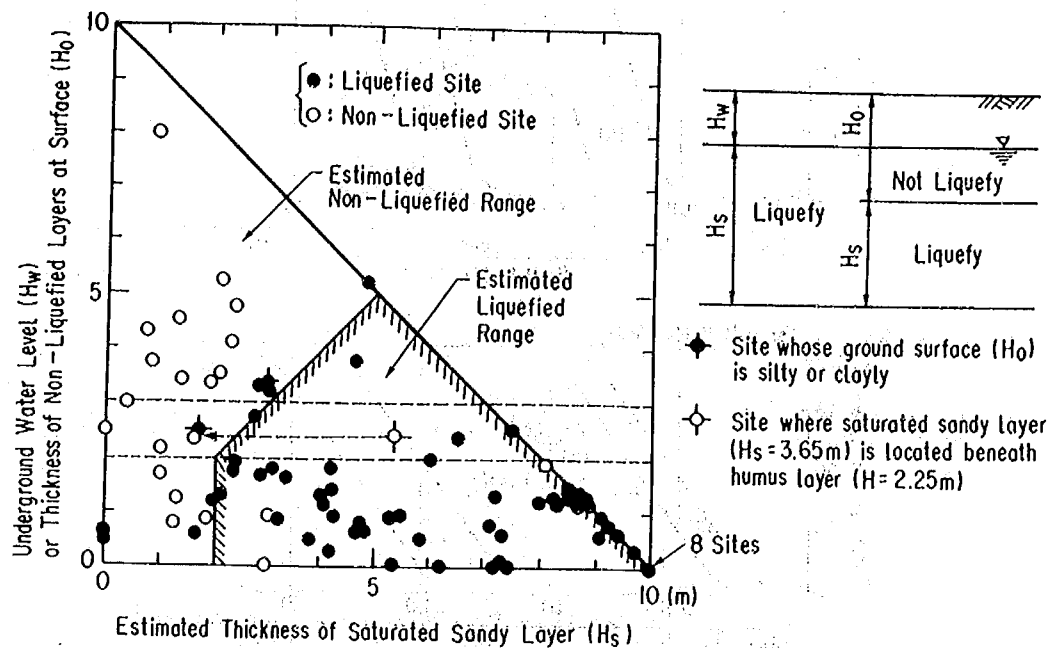


Fig. 4.3 Relation between Estimated Thickness of Saturated Sandy Layers (H_s) and Underground Water Level (H_w) or Thickness of Non-Liquefied Layers at Surface (H_o) [Ref. 4-7]

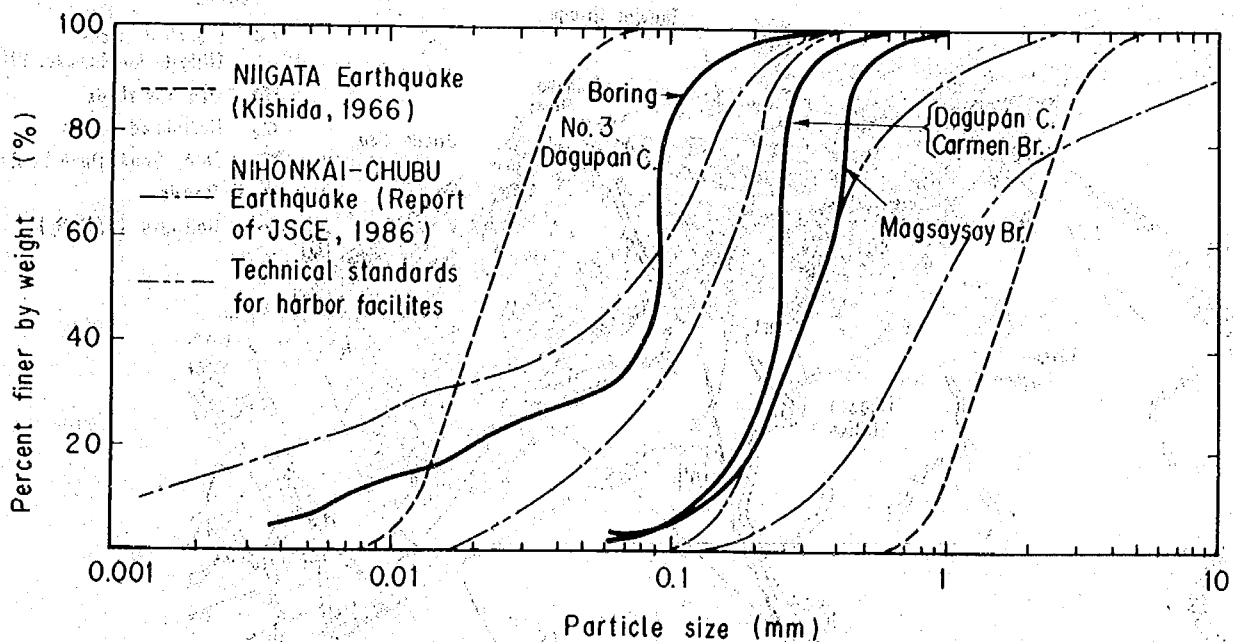


Fig. 4.4 Example of Grain Size Characteristics of Liquefied Soils in the Past Earthquakes [Ref. 4-8]

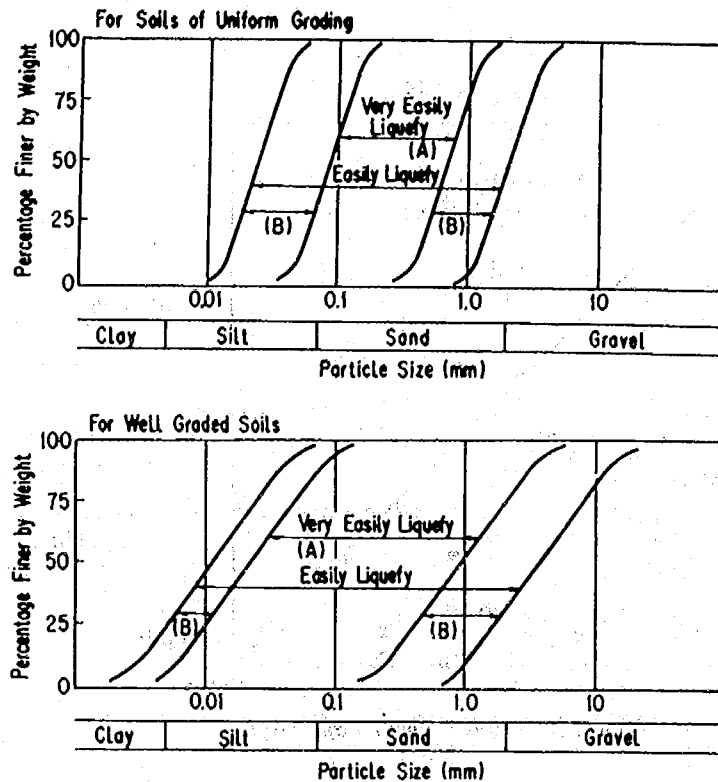


Fig. 4.5 Relation between Grain Size Characteristics of Soils and Liquefaction Potential [Ref. 4-9]

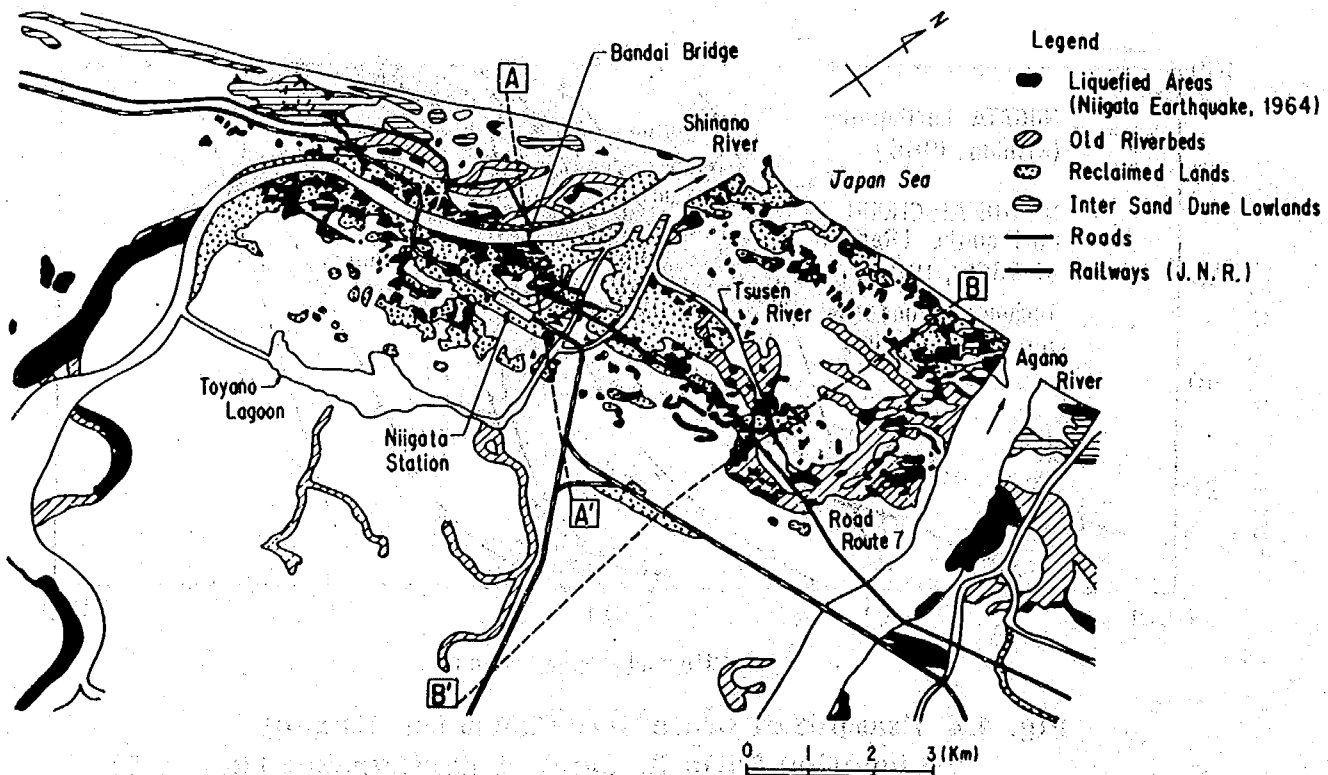


Fig. 4.6 Liquefied Area in Niigata City during the 1964 Niigata Earthquake [Ref. 4-2]

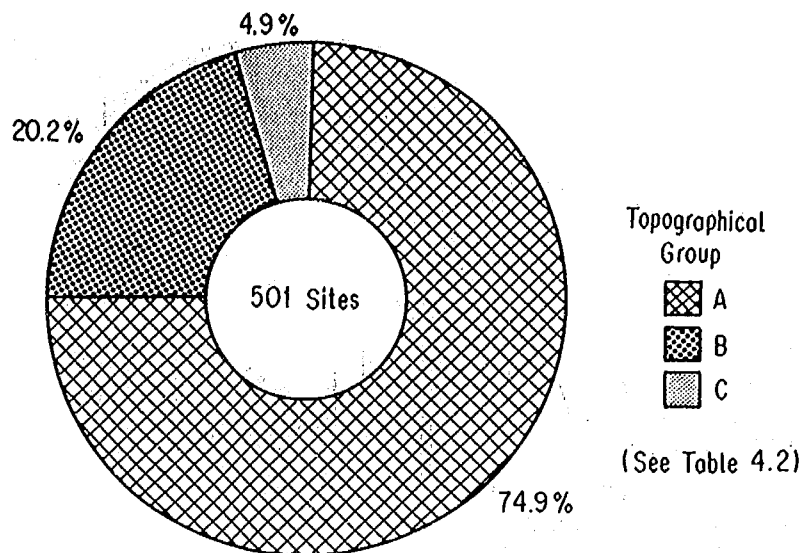


Fig. 4.7 Classification of Geographical Condition at 501 Sites Liquefied in the Past Earthquakes in Japan [Ref. 4-1]

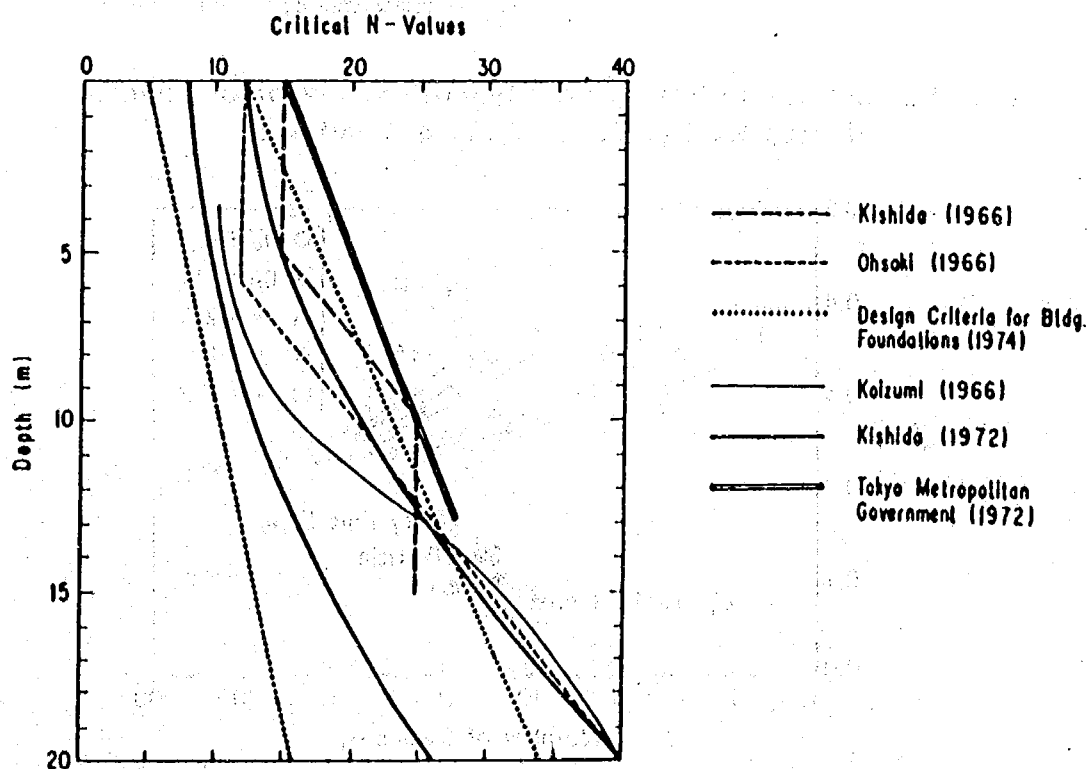


Fig. 4.8 Relation between Depth and Critical N-Values to Cause Liquefaction [Refs. 4-10 and 4-11]

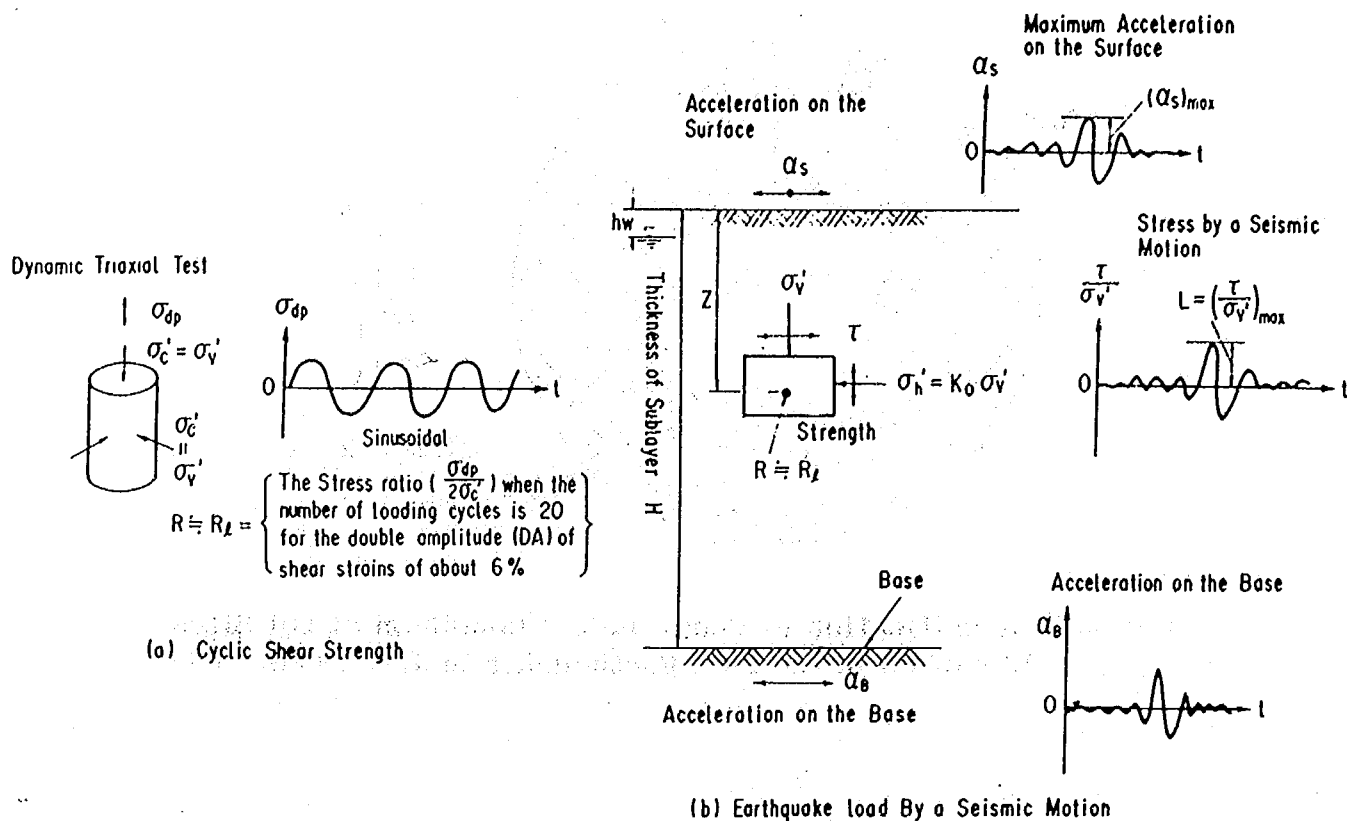


Fig. 4.9 Schematic Drawing on Dynamic Behavior of Layers during Earthquakes [Refs. 4-2 and 4-4]

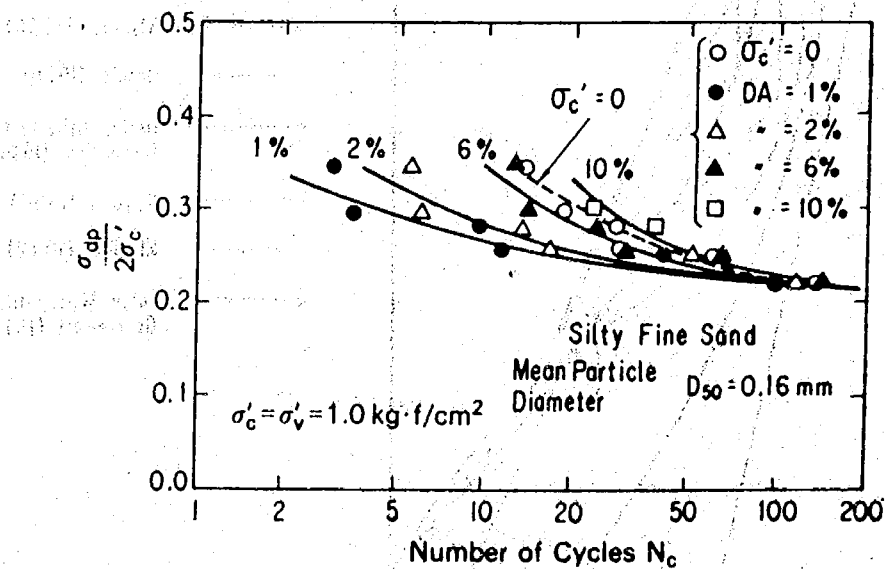


Fig. 4.10 Example of Dynamic Triaxial Test Results to Decide Dynamic Shear Strength of Un-drained Soil Specimen [Ref. 4-2]

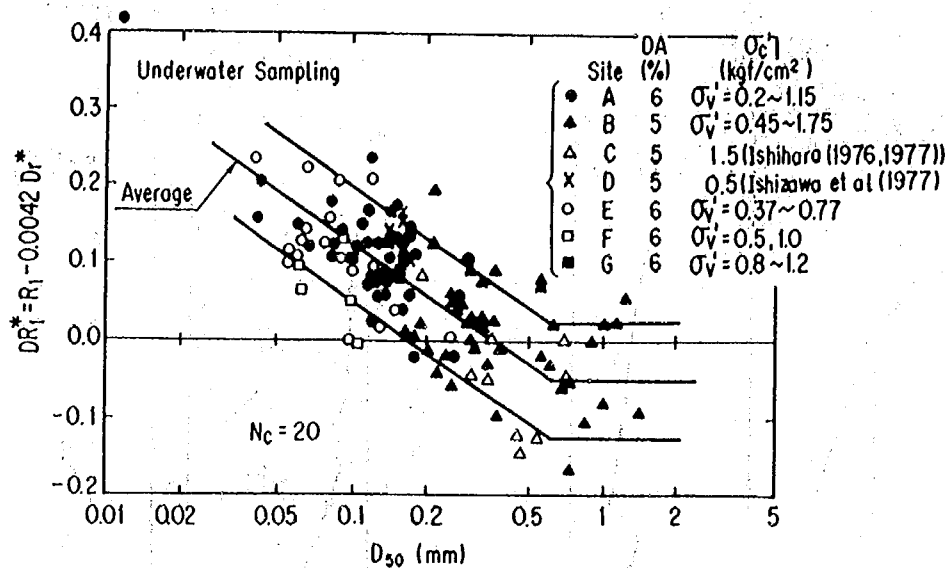
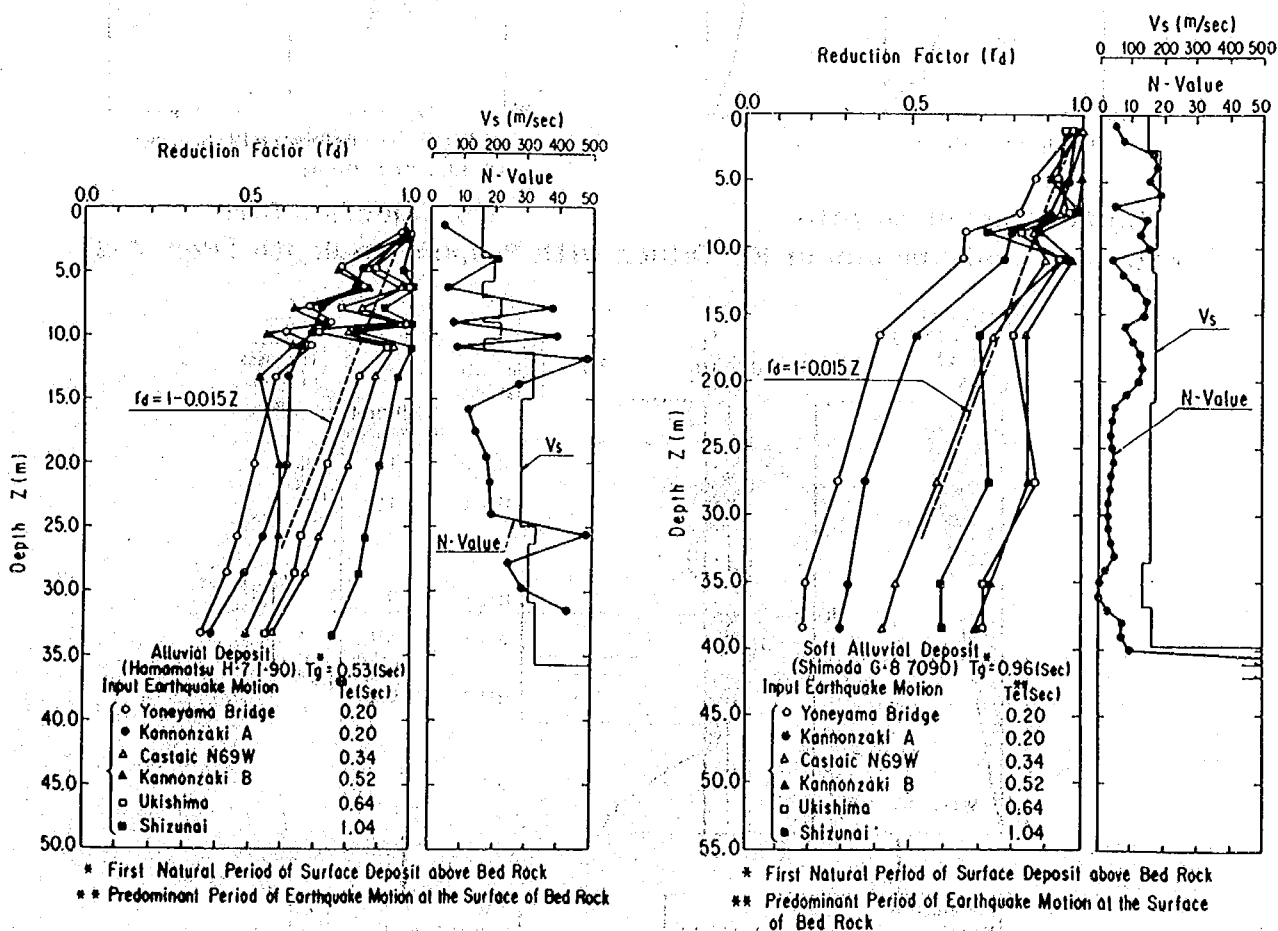


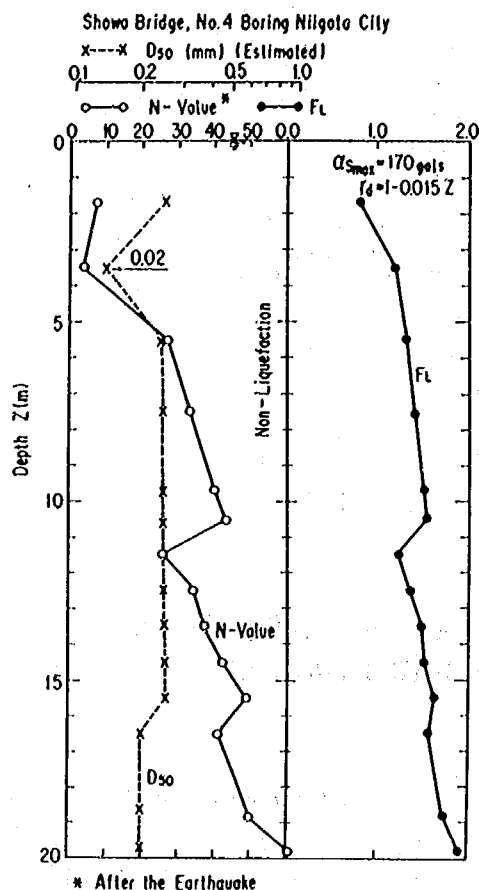
Fig. 4.11 Relation between Mean Grain Size of Soils (D_{50}) and Cyclic Shear Strength (R_1) [Ref. 4-2]



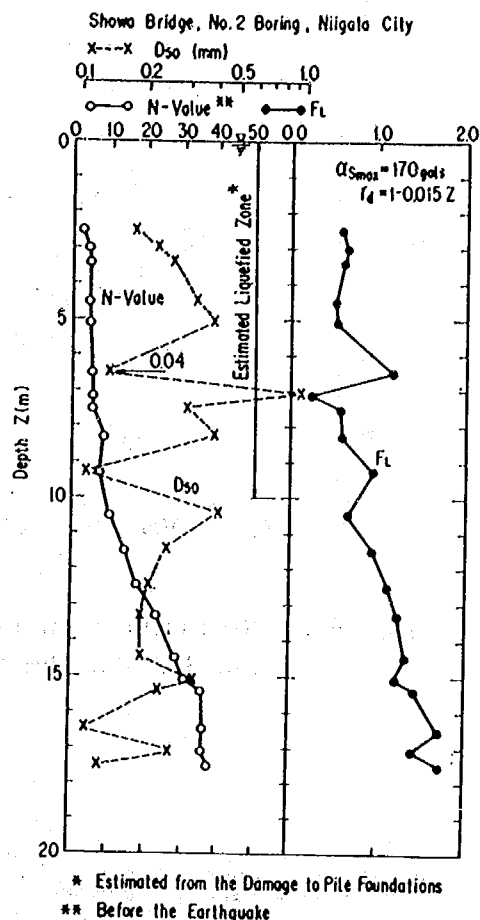
(a) Site on Alluvial Deposit

(b) Site on Soft Alluvial Deposit

Fig. 4.12 Reduction Factor, r_d [Ref. 4-2]



(a) Non-Liquefied Site



(b) Liquefied Site

Fig. 4.13 Distribution of F_L-Values with Respect to Depth [Ref. 4-2]

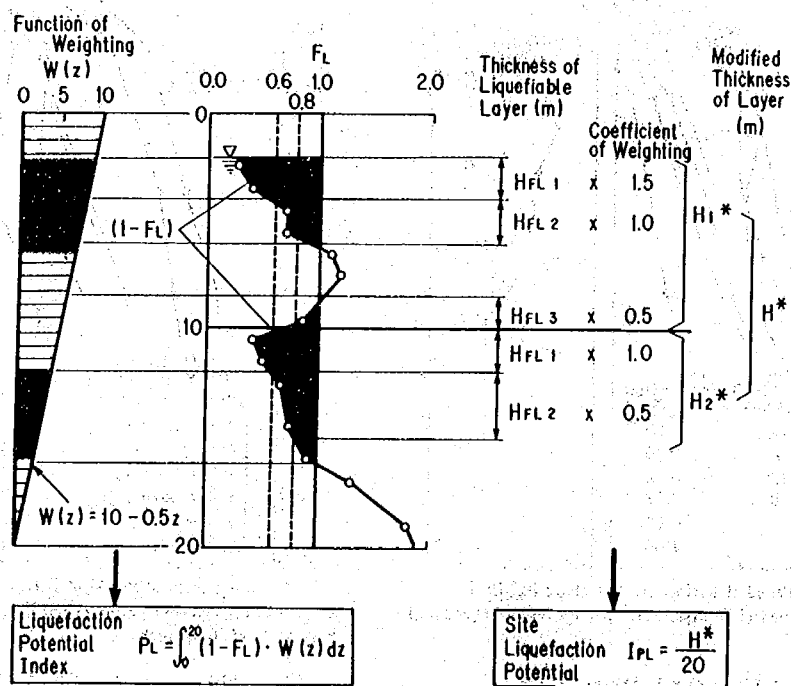


Fig. 4.14 Identification of P_L-Value and I_{PL}-Value [Refs. 4-12 and 4-13]

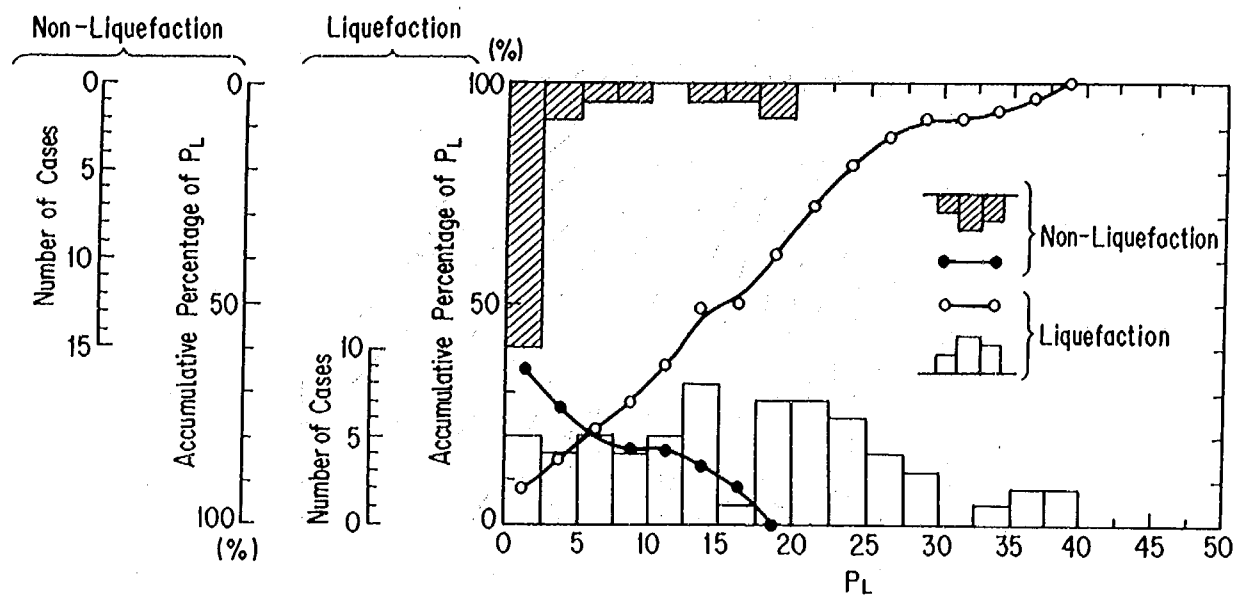


Fig. 4.15 P_L -Value Distribution at Liquefied Sites and Nonliquefied Sites in the Past Earthquakes [Ref. 4-2]

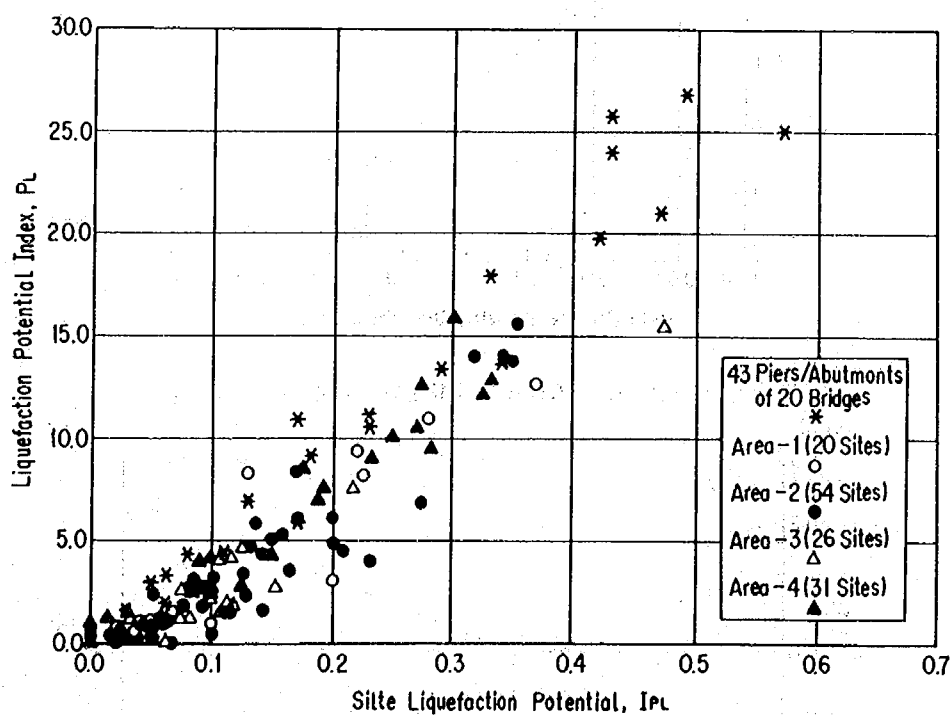


Fig. 4.16 Relation between P_L -Values and I_{PL} -Values [Ref. 4-13]

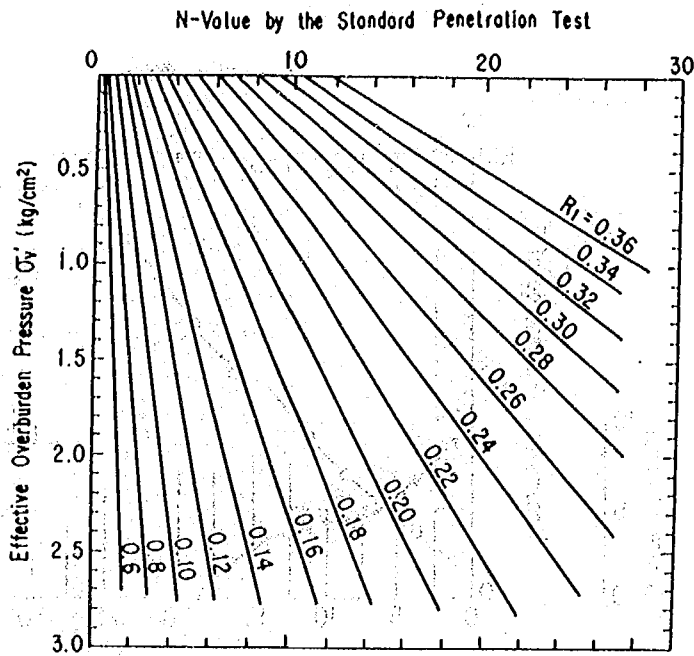


Fig. 4.17 R_1 -Value [Refs. 4-4 and 4-5]

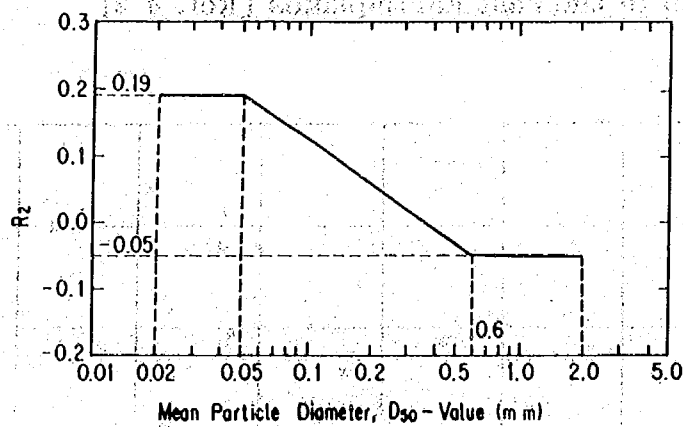


Fig. 4.18 R_2 -Value [Refs. 4-4 and 4-5]

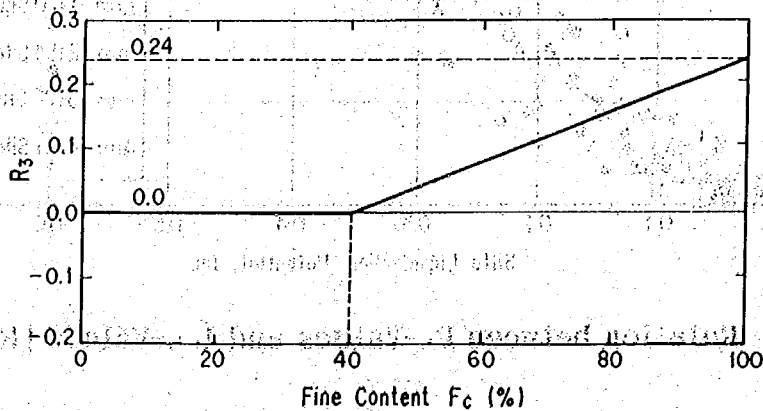


Fig. 4.19 R_3 -Value [Ref. 4-4]

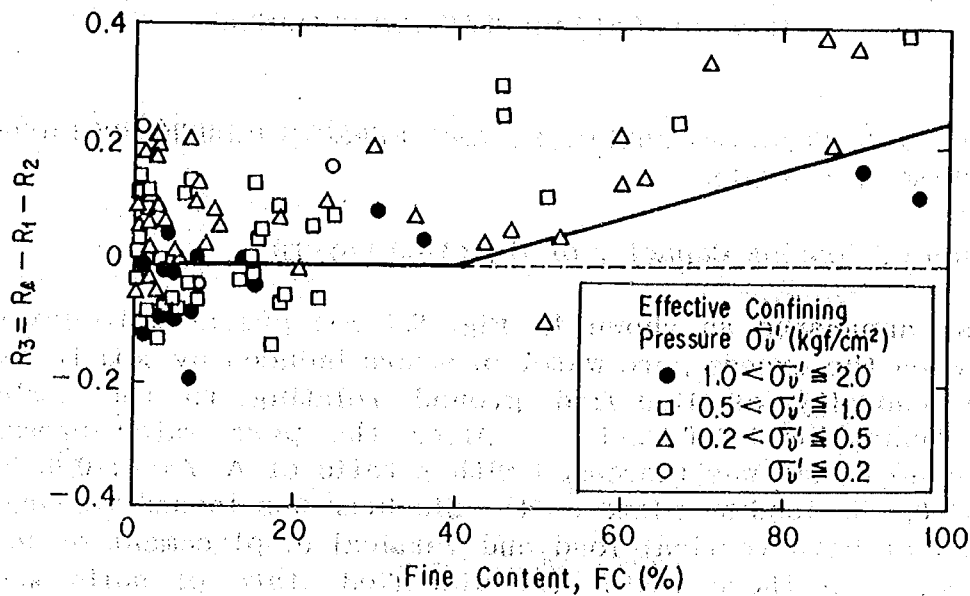


Fig. 4.20 Relation between Fines Content (FC) and Cyclic Shear Strength (R_a) [Ref. 4-4]

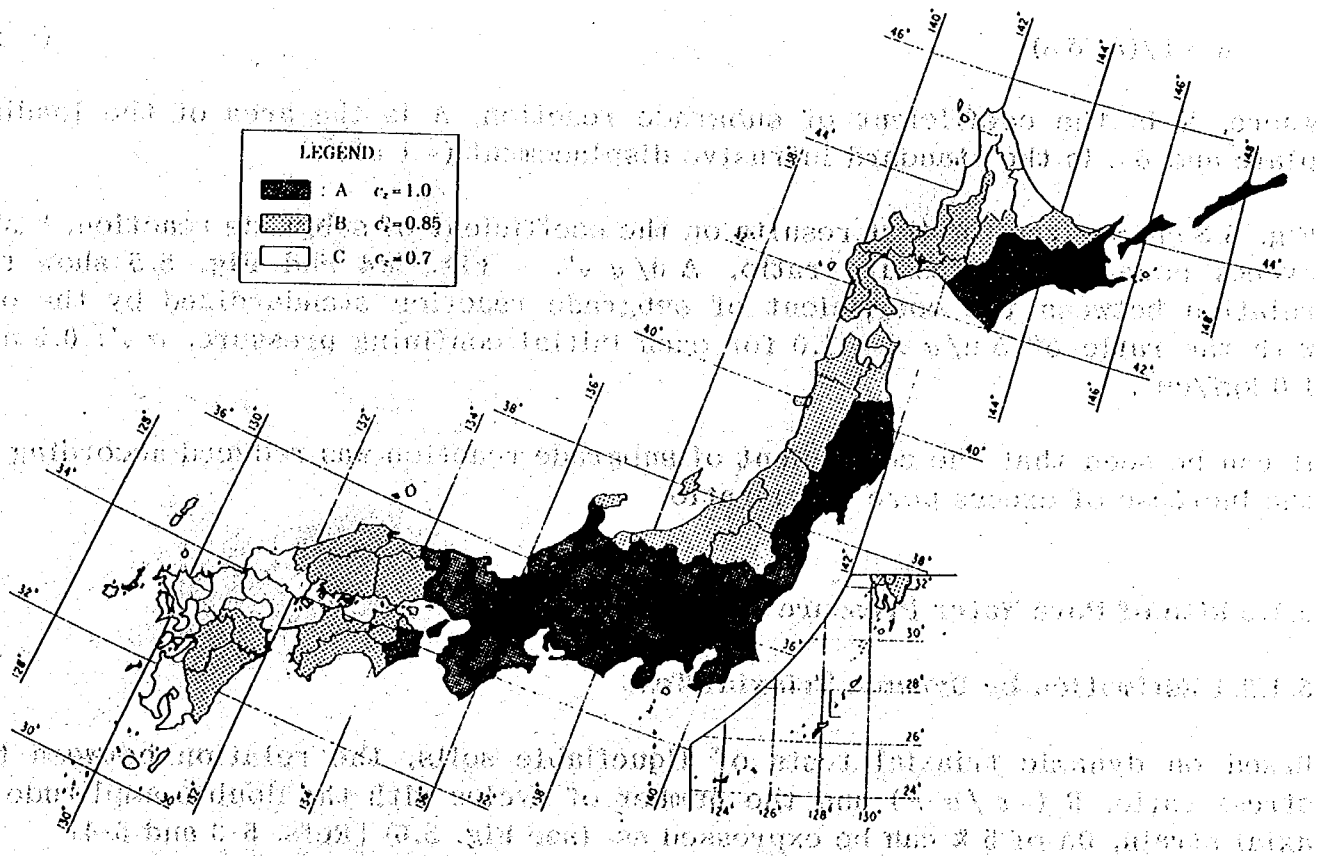


Fig. 4.21 Seismic Zoning Map and Modification Coefficient, C_z
[Refs. 4-4 and 4-5]

CHAPTER 5 SEISMIC DESIGN METHODS OF CIVIL ENGINEERING STRUCTURES CONSIDERING LIQUEFACTION POTENTIAL

5.1 SIMPLIFIED ESTIMATING METHODS OF DYNAMIC EFFECTS INDUCED BY LIQUEFACTION TO STABILITY OF STRUCTURES

5.1.1 Reduction of Bearing Capacity of liquefied Layers

A simple test apparatus as shown in Fig. 5.1 was prepared to investigate the relation between the excess pore water pressure induced by soil liquefaction and the bearing capacity of liquefied ground relating to the stability of a structure's foundation [Ref. 5-1]. After the pore water pressure in the saturated Toyoura sand was increased with a ratio of $\Delta u / \sigma_v'$: 0.0, 0.2, 0.4, 0.6, 0.8, 0.9 and 1.0, the loading plate on the diaphragm was loaded vertically and the time histories of both vertical load and vertical displacement of loading plate were measured. In these tests, the liquefied state of soils was simulated statistically by controlling the pore water pressure artificially. Table 5.1 and Fig. 5.2 show the test conditions and the results of loading tests, respectively.

Based on the relation such as Fig. 5.2, the bearing capacity of soils was identified as

$$K = P / (A \cdot \delta_o) \quad (5-1)$$

where, K is the coefficient of subgrade reaction, A is the area of the loading plate and δ_o is the standard intrusive displacement ($= 1 \text{ mm}$).

Fig. 5.3 summarizes the test results on the coefficient of subgrade reaction, K and excess pore water pressure ratio, $\Delta u / \sigma_v'$. Fig. 5.4 and Fig. 5.5 show the relation between the coefficient of subgrade reaction standardized by the one with the ratio of $\Delta u / \sigma_v' = 1.0$ for each initial confining pressure, σ_v' : 0.5 and 1.0 kgf/cm^2 .

It can be seen that the coefficient of subgrade reaction was reduced according to the increase of excess pore water ratio.

5.1.2 Rise of Pore Water Pressure

5.1.2.1 Estimation by Dynamic Triaxial Test

Based on dynamic triaxial tests of liquefiable soils, the relation between the stress ratio, $R (= \tau / \sigma_v')$ and the number of cycles with the double amplitude of axial strain, DA of 5 % can be expressed as (see Fig. 5.6) [Refs. 5-3 and 5-4]

$$(\tau / \sigma_v') = a \cdot N_1^b \quad (5-2)$$

where τ is the shear stress (kgf/cm²), σ_v' is the effective overburden/confining pressure, N_1 is the number of cycles and a and b are the constants decided according to the strain amplitude, DA .

The dynamic shear strength ratio of soils, R and the dynamic shear stress ratio, L can be expressed as

$$R = (\tau_R / \sigma_v') \quad (5-3)$$

$$L = (\tau_L / \sigma_v') \quad (5-4)$$

where τ_R is the dynamic shear strength (kgf/cm²) and τ_L is the dynamic shear stress (kgf/cm²). Furthermore, the following relation can be drawn based on the relation shown in Fig. 5.7.

$$R = a \cdot N_R^b \quad (5-5)$$

$$L = a \cdot N_L^b \quad (5-6)$$

where N_R and N_L are the number of cycles corresponding to the dynamic shear strength ratio, R and the dynamic shear stress ratio, L , respectively. From equations (4-1), (5-5) and (5-6), the relation can be expressed as

$$F_L = R/L = (a \cdot N_R^b) / (a \cdot N_L^b) = (N_R/N_L)^b \quad (5-7)$$

On the other hand, the relation such as Fig. 5.8 can be obtained in dynamic triaxial tests. Fig. 5.8 indicates that excess pore water pressure ratio, $\Delta U / \sigma_v'$ can be estimated with use of cycle number ratio, N/N_1 . When N_R and N_L are assumed to be a number of cycles to be liquefied perfectly and a number of cycles going to liquefy, the relation can be drawn as

$$N_R/N_L = N/N_1 \quad (5-8)$$

From equations (5-7) and (5-8),

$$N/N_1 = (F_L)^{1/b} \quad (5-9)$$

Finally, the excess pore water pressure can be estimated with use of equation (5-9) and Fig. 5.8 as

$$\Delta U / \sigma_v' \sim N/N_1 = (F_L)^{1/b} \quad (5-10)$$

For example, if $\Delta U / \sigma_v'$ is related to N/N_1 linearly as shown in Fig. 5.7, the following equation can be obtained.

$$\Delta U / \sigma_v' = N/N_1 = (F_L)^{1/b} \quad (5-11)$$

where the constant b is - 0.17 when DA is 5%.

5.1.2.2 Estimation by Shaking Table Test

Based on the shaking table tests shown in Chapter 3.3, several results on the relation between F_L -value and excess pore water pressure also have been obtained as follows [Refs. 5-5 and 5-6]. Fig. 5.9 shows the characteristics of ground models during shaking table tests.

Fig. 5.10 shows the relation between acceleration, α_0 measured in the ground during shaking and static cone penetration resistance, q_0 at the depth of acceleration measurement before shaking. The relation was classified into two groups: partially liquefied state of $\Delta U/\sigma'_{v'} < 1.0$ and perfectly liquefied state of $\Delta U/\sigma'_{v'} = 1.0$.

Fig. 5.11 shows the relation between acceleration measured in the ground during shaking and dynamic shear strength ratio, R_1 . In this case, the dynamic shear strength ratio, R_1 was estimated by the equation (4-8) with use of modified N -value estimated by

$$N = 4 \cdot q_0 \quad (5-12)$$

Fig. 5.12 shows the examples of time histories of excess pore water pressure, ground surface acceleration and estimated liquefaction resistance factor during shaking.

Fig. 5.13 shows the relation between liquefaction resistance factor, F_L and excess pore water pressure ratio, $\Delta U/\sigma'_{v'}$ ($= L_u$) drawn from such results as Fig. 5.12. From this figure, the relation can be estimated as

$$\left[\begin{array}{ll} F_L \leq 0.6 : & L_u = 1.0 \\ 0.6 < F_L \leq 0.8 : & 0.9 \leq L_u < 1.0 \\ 0.8 < F_L \leq 1.0 : & 0.5 \leq L_u < 0.9 \\ 1.0 < F_L : & L_u < 0.5 \end{array} \right] \quad (5-13)$$

Fig. 5.14 compares the relations between liquefaction resistance factor, F_L and excess pore water pressure ratio, $\Delta U/\sigma'_{v'}$ proposed in the past in Japan. In this figure, the relation by PWRI was drawn from Fig. 5.13.

Based on the results of Fig. 5.5, the relation between ratio of coefficient of subgrade reaction, K/K_0 and excess pore water pressure ratio, L_u can be classified into 4 groups as

$$\left[\begin{array}{ll} K/K_0 = 0 & : L_v = 1.0 \\ K/K_0 = 1/3 & : 0.9 \leq L_v < 1.0 \\ K/K_0 = 2/3 & : 0.5 \leq L_v < 0.9 \\ K/K_0 = 1 & : L_v < 0.5 \end{array} \right] \quad (5-14)$$

From equations (5-13) and (5-14), the relation between F_L -value and ratio of coefficient of subgrade reaction, K/K_0 can be drawn as (see Fig. 5.15)

$$\left[\begin{array}{ll} F_L \leq 0.6 & : K/K_0 = 0 \\ 0.6 < F_L \leq 0.8 & : K/K_0 = 1/3 \\ 0.8 < F_L \leq 1.0 & : K/K_0 = 2/3 \\ 1.0 < F_L & : K/K_0 = 1 \end{array} \right] \quad (5-15)$$

The results of equation (5-15) were applied in the Specifications for Highway Bridges in Japan (see Chapter 5.2.1).

5.1.3 Methods for Investigating Seismic Stability of Structures Considering Effects of Liquefaction

The methods for investigating the stability of structures considering the effects of soil liquefaction with use of liquefaction resistance factor, F_L -value can be summarized as shown in Fig. 5.16 [Ref. 5-1].

In the following Chapter 5.2, examples of the application of the procedures shown in Fig. 5.16 will be introduced for highway bridges and river dykes.

5.2 SEISMIC DESIGN METHODS OF TYPICAL STRUCTURES

5.2.1 Highway Bridge

5.2.1.1 Specifications in Japan

It is necessary to introduce the treatment of layers judged to liquefy by the above procedure for reasonable seismic design of structures. In the Specifications, the bearing capacities for layers judged to liquefy should be either neglected or reduced by multiplying the original bearing capacities with the ratio of coefficient of subgrade reaction considering F_L -value shown in equation (5-15). In the Specifications of Highway Bridges in Japan [Ref. 5-2], the ratio of coefficient of subgrade reaction is identified as the reduction factor, D_E and specified as shown in Table 5.2. In the Specifications, the effects of liquefied layers at the depth of 10 to 20 meters are assumed to be smaller than that at the depth of 0 to 10 meters.

As the relation between F_L -value and D_E should be provided as shown in Table 5.2, the reduction factor D_E should be determined in accordance with both range of F_L -values and depth of liquefied layers.

5.2.1.2 Example of Estimation of Liquefaction-Induced Effects

Fig. 5.17 shows the Showa-Ohashi Bridge damaged in the 1964 Niigata Earthquake [Ref. 5-8]. The pile foundations were reinforced after the earthquake as shown in Fig. 5.18 [Refs. 5-5 and 5-7]. This was one of retrofit methods of foundations against soil liquefaction. 9 piles of 1 row were constructed more and the length of the pier top was also spread.

Fig. 5.19 shows the results of dynamic response for piers before and after reinforcement shown in Fig. 5.18. It can be seen that the lateral displacement of pier top were decreased by the reinforcement of piers, and increased by the reduction of bearing capacity of liquefied soils. In this analysis, the reduction factor was decided based on the distribution of F_L -values in Fig. 5.18 and Table 5.2.

5.2.2 River Dyke

5.2.2.1 Design Methods in Japan

The circular arc analysis method is common to estimate the minimum safety factor for stability of a river dyke in Japan. In this method, the seismic coefficient and the excess pore water pressure are considered as the effects induced by earthquakes against stability of river dykes as shown in Table 5.3. The seismic coefficient and excess pore water pressure are applied separately to calculate minimum safety factor for seismic slide stability of a levee. In this paper, the former method will be called Kh-Method and the latter one Δu -Method.

The concept of a circular arc analysis method can be shown in Fig. 5.20 and the safety factor for sliding can be calculated by the following equations.

Static State:

$$F_{SS} = \frac{\sum [c \cdot l + (W - u_0 \cdot b) \cos \alpha \cdot \tan \phi]}{\sum (W \cdot \sin \alpha)} \quad (5-16)$$

Seismic Analysis (Kh-Method):

$$F_{SE}(K_h) = \frac{\sum [c \cdot l + \{ (W - u_0 \cdot b) \cdot \cos \alpha - K_h \cdot W \cdot \sin \alpha \} \cdot \tan \phi]}{\sum (W \cdot \sin \alpha + K_h \cdot W \cdot Y/R)} \quad (5-17)$$

Seismic Analysis (Δu -Method):

$$F_{SE}(\Delta u) = \frac{\sum [c \cdot l + \{W - (u_0 + \Delta u) \cdot b\} \cos \alpha \cdot \tan \phi]}{\sum (W \cdot \sin \alpha)} \quad (5-18)$$

where, F_{SS} is the safety factor for non-earthquakes, F_{SE} is the safety factor during earthquakes, c is the cohesion (tf/m²), ϕ is the internal friction angle of soil, u_0 is the initial pore water pressure (tf/m²), Δu is excess pore water pressure (tf/m²), α is the tilt angle of sliding surface, W is the total weight of sliced section (tf/m), K_h is the horizontal seismic coefficient, l is the length of base of sliced section (m), b is the width of sliced section (m), R is the radius of sliding circle (m) and Y is the vertical distance between the gravity center of sliced section and the center of sliding circle (m).

Fig. 5.21 illustrates the methods to estimate excess pore water pressure with the use of F_L -values calculated from boring data in situ.

5.2.2.2 Example of Design

Fig. 5.22 shows an example of analytical model of levee and subsurface ground model. Fig. 5.23 was the relation between F_L -value and excess pore water pressure used in the analysis. Table 5.4 shows the results of excess pore water pressure calculated in each cell according to Fig. 5.21.

Fig. 5.24 shows the results of circular slide surface and minimum safety factor of levee by circular arc methods. Table 5.5 shows the minimum safety factors calculated.

5.2.3 Common Utility Ducts

5.2.3.1 Design Guidelines in Japan

In this chapter, the Design Guidelines for Common Utility Ducts (Proposal) in Japan will be introduced [Ref. 5-9].

1) Basic Principle for Seismic Design

Because common utility ducts are extended structures embedded in the ground, they are generally affected by the deformation of the surrounding ground. The basic principles for seismic design of common utility ducts are introduced in the Guidelines as follows.

- ① The seismic design for common utility ducts shall be considered for soft ground and ground condition varied site.
- ② The seismic design shall be carried out with use of Response Displacement

Method in the direction of longitudinal axis.

③ The safety against the up-lift induced by liquefaction shall be checked.

2) Design Earthquake Ground Motion

The horizontal ground displacement amplitude on the ground surface should be calculated based on the natural period of surface ground and the zone. The horizontal and vertical ground displacement at the depth from ground surface are estimated by equation (5-19) and equation (5-20), respectively.

$$U_h(z) = (2/\pi^2) \cdot S_v \cdot T_s \cdot \cos(\pi z / 2H) \quad (5-19)$$

$$U_v(z) = (1/2) \cdot U_h(z) \quad (5-20)$$

where, $U_h(z)$ is the horizontal ground displacement amplitude (m), $U_v(z)$ is the vertical ground displacement (m), S_v is the design response velocity (m/sec) (refer to Fig. 5.25), T_s is the natural period of ground surface (sec), H is the thickness of surface ground (m) and z is the depth from ground surface (m).

The design response velocity S_v can be obtained with the use of Fig. 5.2 based on the natural period (T_s) and the objective zone. The zoning map is same with Fig. 4.21. The natural period (T_s) can be estimated by the equation (5-21).

$$T_s = 1.25 T_G \quad (5-21)$$

$$T_G = 4 \sum_{i=1} (H_i / V_{s,i}) \quad (5-22)$$

where, T_G is the characteristic value of surface ground (sec), H_i is the thickness of i -th sublayer (m), $V_{s,i}$ is the shear wave velocity of i -th layer (m/sec) and i is the sublayer's number counted from ground surface.

3) Check for Liquefaction and Lift-up of Ducts

Because common utility ducts are buried in the ground, the up-lift of ducts induced by soil liquefaction during earthquakes should be considered for seismic design. The flow-chart to investigate the possibility of soil liquefaction and check the stability of ducts by liquefaction is introduced in the Standards. At the beginning, the layers which are needed to check liquefaction are judged with the same method mentioned later in Chapter 4.5.2.

If the ducts are predicted to be located in the liquefiable layers, the check on the stability of ducts for lift-up by liquefaction should be carried out based on the equation (5-23) (refer to Fig. 5.26).

$$F_s = \frac{W_s + W_B + Q_s + Q_B}{U_s + U_D} \quad (5-23)$$

where, F_s is the safety factor for lift-up, W_s is the weight of soils above the duct (tf/m), W_B is the body weight of duct (tf/m), Q_s is the shear resistance force (tf/m), Q_B is the friction resistance force at sides of duct (tf/m), U_s is the uplift force by static water pressure to base of duct (tf/m) and U_D is the uplift force by excess pore water pressure to base of duct (tf/m).

The uplift force can be estimated by the equation (5-24) based on the excess pore water pressure.

$$U_D = \Delta u \cdot B = L_u \cdot \sigma_v' \cdot B \quad (5-24)$$

where, B is the width of duct (m), σ_v' is the effective overburden pressure at the depth of base of duct (t/m²), L_u is the excess pore water pressure ratio ($= \Delta u / \sigma_v'$) and Δu is the excess pore water pressure (t/m²).

The excess pore water pressure ratio (L_u) can be estimated with the use of liquefaction resistance factor (F_L) indicated by the equation (4-19) mentioned in the Seismic Design Specifications of Highway Bridges as shown in Fig. 5.27.

4) Treatment of Check for Up-lift of Duct

The safety factor (F_s) should be more than 1.1. When the safety factor is less than 1.1, detailed liquefaction resistance factor (F_L) should be calculated by detailed investigations, i.e., dynamic response analysis on layers and dynamic tests on soil elements.

When the following conditions are satisfied, countermeasures for liquefaction should be applied adequately.

- ① The probability for liquefaction occurrence of layers is high.
- ② The safety factor for up-lift of duct calculated by detailed investigations is less than 1.1
- ③ The occurrence of damages to duct is predicted.

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Table 5.1 Test Conditions on Bearing Capacity of Liquefied Soils [Ref. 5-1]

NO	Sample	σ'_{ν} (kgf/cm ²)	Void Ratio e	[K] $\Delta U / \sigma'_{\nu} = 0.0$ (kgf/cm ² /mm)
1	ST-1	1.0	0.787	19.0
2	ST-2		0.722	17.1
3	ST-3		0.721	18.5
4	ST-4		0.744	18.0
5	ST-5		0.810	16.6
6	ST-8	0.5	0.770	10.2
7	ST-10		0.719	7.8
8	ST-11		0.791	8.1
9	ST-12		0.691	10.9

Table 5.2 Reduction of Bearing Capacity of Ground According to Liquefaction Potential in the Specifications for Highway Bridge in Japan [Ref. 5-2]

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

Table 5.3 Circular Arc Analysis Methods for Estimating the Seismic Slide Stability of Levee which are Common in Japan

Purpose of Estimation	Analytical Conditions
1. Stability against Inertia Load by Seismic Ground Motion (K_h -Method)	1) Seismic effect is considered as an inertia load. 2) Seismic coefficient should be estimated for an inertia load. 3) Only horizontal load is considered generally. 4) Minimum safety factor (F_{s1}) for sliding of levee is calculated.
2. Stability Against Pore Water Pressure Induced by Soil Liquefaction (Δu -Method)	1) Seismic effect is considered as excess pore water pressure induced by soil liquefaction. 2) Excess pore water pressure should be estimated properly. 3) Only horizontal load is considered generally. 4) Minimum safety factor (F_{s2}) for sliding of levee is calculated.

Table 5.4 Results of Average of F_L -value and Induced Excess Pore Water Pressure Ratio

Liquefiable Layers Analysed		Cells Dividing Cross Section of Levee											
		B-1	B-2	B-3	B-4	B-5	B-6	B-7	B-8	B-9	B-10	B-11	B-12
As1	F_L	1.14	1.10	1.34	1.31	1.38	1.39	1.42	1.44	1.37	1.12	0.91	0.92
	$\Delta u / \sigma'_{v'}$	0.45	0.55	0.17	0.20	0.14	0.14	0.12	0.11	0.15	0.52	1.00	1.00
As2	F_L	1.28	1.26	1.45	1.43	1.50	1.52	1.55	1.58	1.50	1.28	1.15	1.16
	$\Delta u / \sigma'_{v'}$	0.23	0.26	0.11	0.12	0.09	0.08	0.07	0.06	0.09	0.23	0.42	0.40
As3	F_L	0.87	0.86	0.98	0.96	1.03	1.04	1.09	1.12	1.04	0.89	0.82	0.82
	$\Delta u / \sigma'_{v'}$	1.00	1.00	1.00	1.00	0.85	0.77	0.61	0.50	0.77	1.00	1.00	1.00

Table 5.5 Minimum Safety Factor of Bank Calculated by Circular Arc Method

Static State	Seismic Analysis	
	K_h -Method	Δu -Method
$K_h = 0$ $\Delta u = 0$	$K_h = 0.12$	$\alpha_{max} = 230$
3.40	1.83	1.10

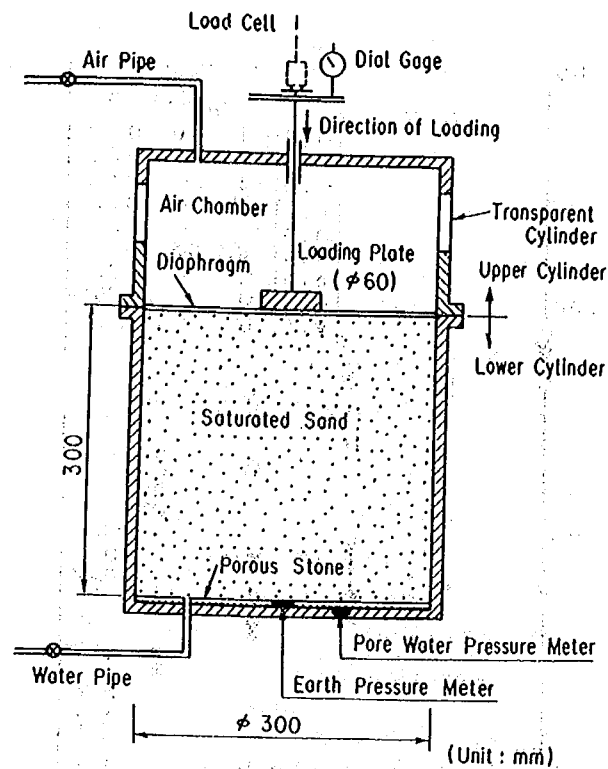


Fig. 5.1 Test Apparatus to Measure Bearing Capacity of Saturated Sand [Ref. 5-1]

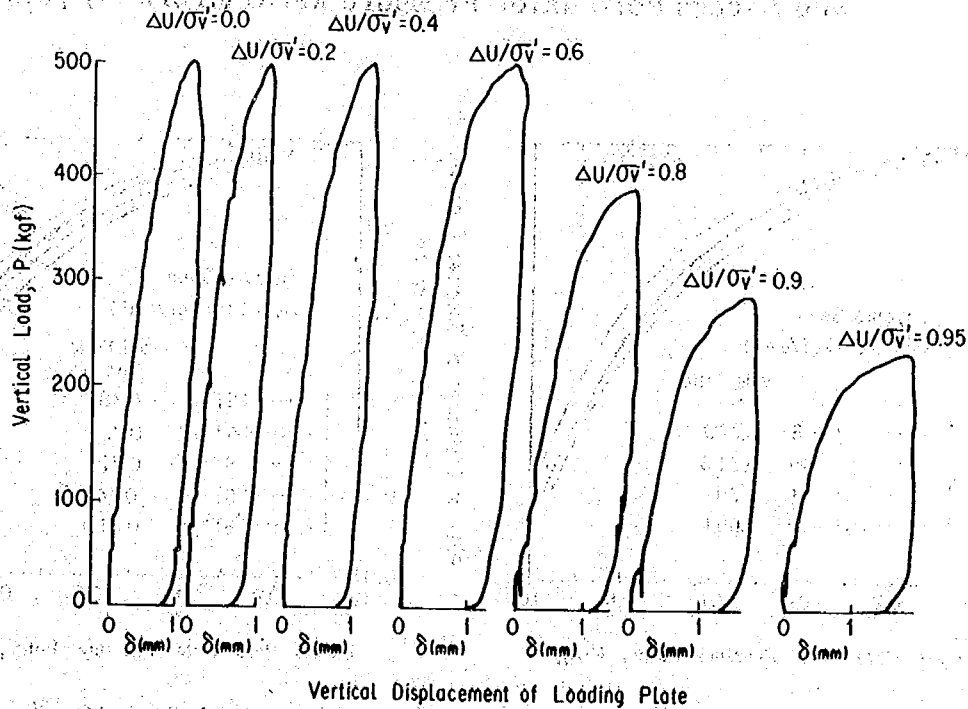


Fig. 5.2 Relation between Load and Displacement of Plate According to Pore Water Pressure Ratio [Ref. 5-1]

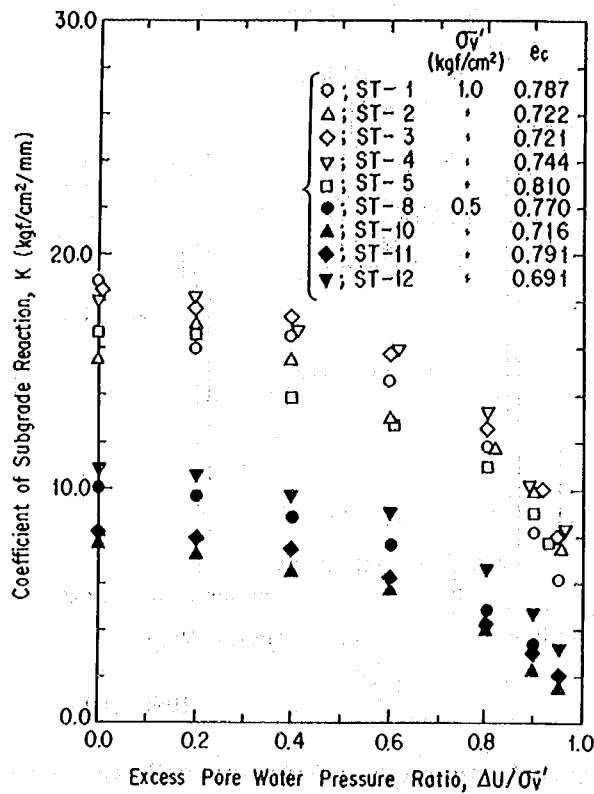


Fig. 5.3 Relation between Coefficient of Subgrade Reaction (K) and Excess Pore Water Pressure Ratio ($\Delta U/\sigma'v$) [Ref. 5-1]

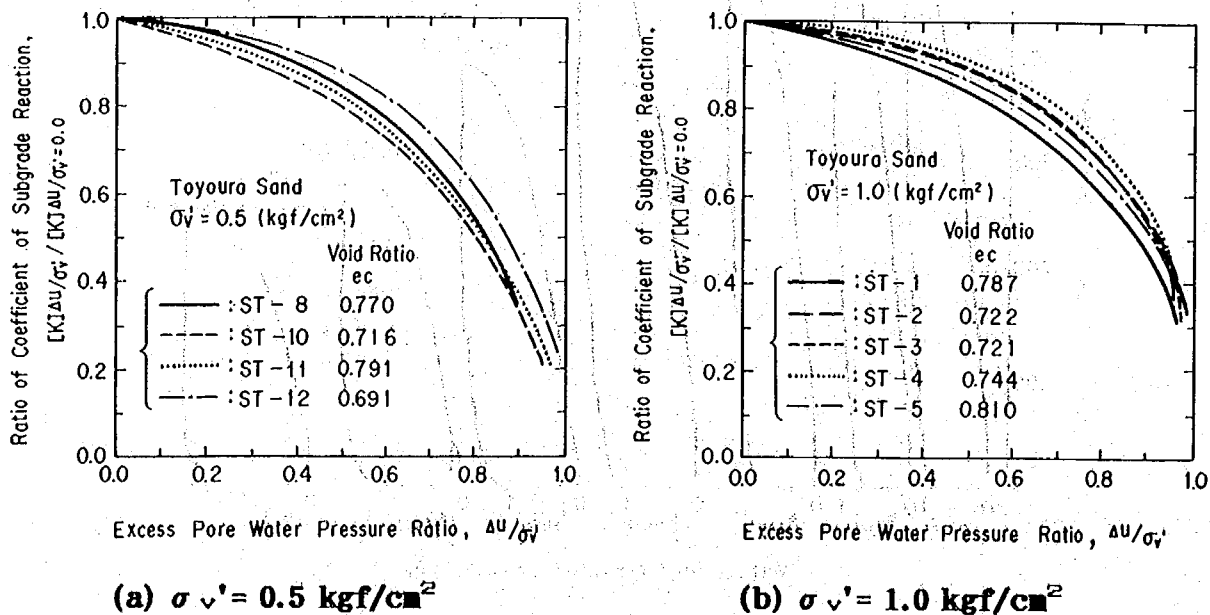


Fig. 5.4 Relation between K/K_0 and $\Delta U/\sigma'v$ [Ref. 5-1]

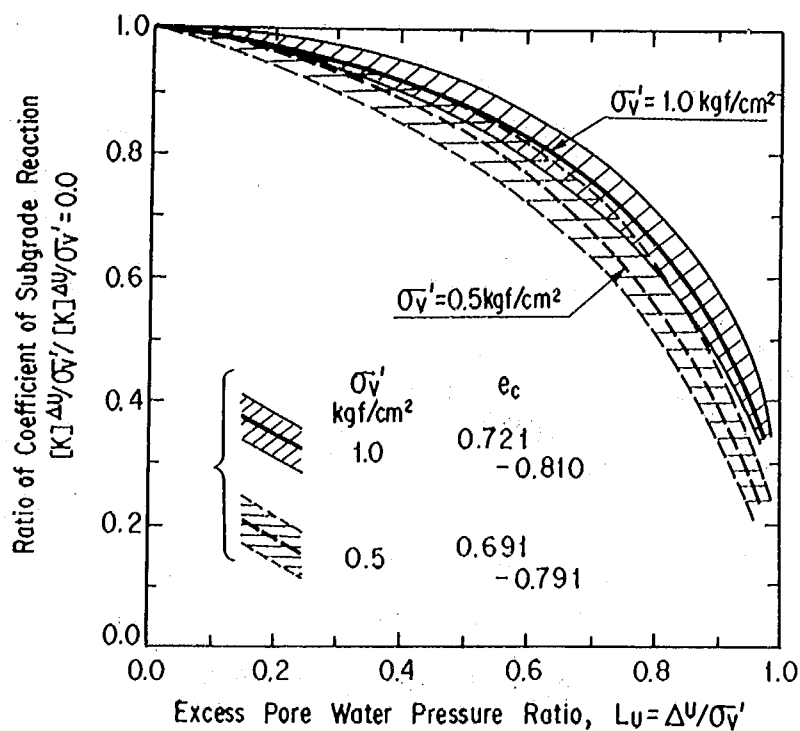


Fig. 5.5 Relation between K/K_0 and $L_u (= \Delta U / \sigma_v')$ [Ref. 5-1]

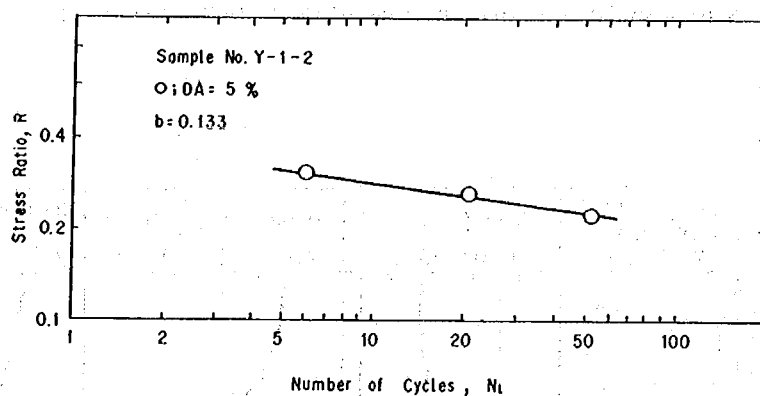


Fig. 5.6 Relation between Stress Ratio and Number of Cycles to Liquefy for Double Amplitude of Axial Strain of 5 % [Refs. 5-3 and 5-4]

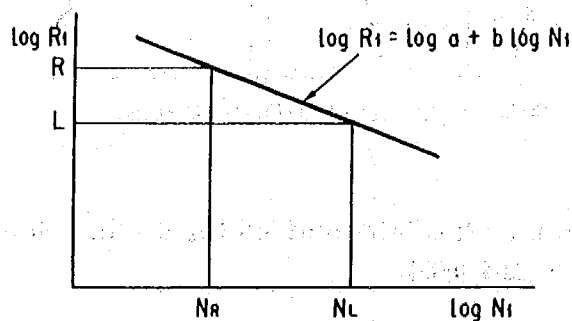


Fig. 5.7 Concept of Relation between R , L and N_R , N_L [Refs. 5-3 and 5-4]

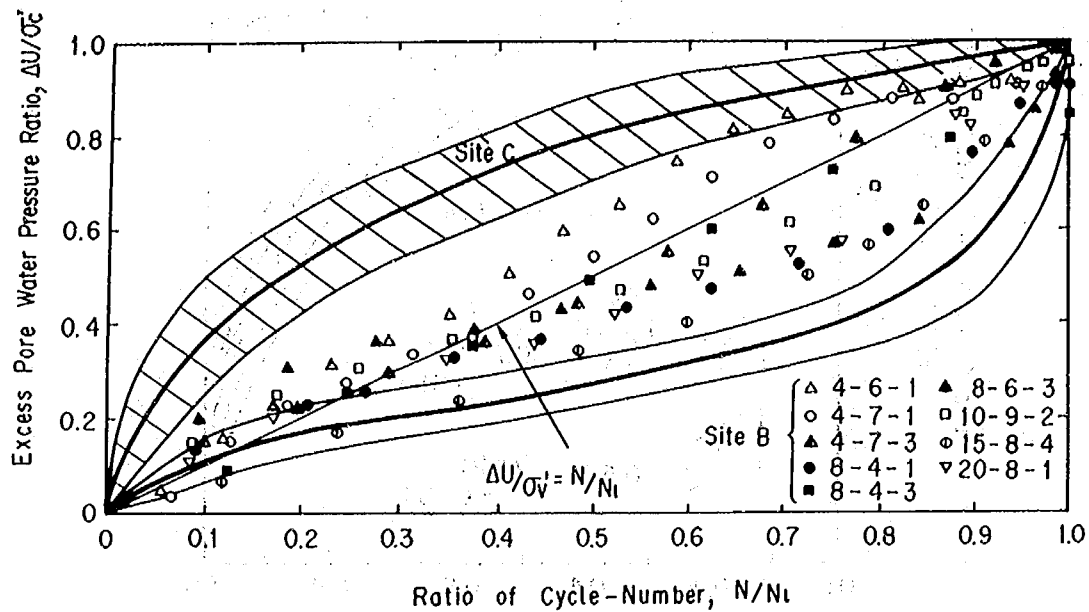


Fig. 5.8 Relation between Excess Pore Water Pressure Ratio and Cycle Number Ratio [Refs. 5-3 and 5-4]

Test No.	Stage	Average Settlement Ratio (%)	Void Ratio e		Relative Density D_r (%)	Dry Unit Weight γ_s (g/cm ³)		Wet Unit Weight γ_t (g/cm ³)	
		2 4 6	0.6 0.7 0.8 0.9			1.4 1.5		1.9 2.0	
0	Before Shaking								
1	After Shaking of 30gal								
2	After Shaking of 80gal								
3	After Shaking of 80gal								
4	After Shaking of 50gal								
5	After Shaking of 80gal								
6	After Shaking of 150gal								
7	After Shaking of 150gal								
8	After Shaking of 250gal								

* Average Settlement Ratio = (Surface Settlement) x 100 / (Total Thickness)

Fig. 5.9 Characteristics of Ground Model During Shaking Table Tests [Refs. 5-5 and 5-6]

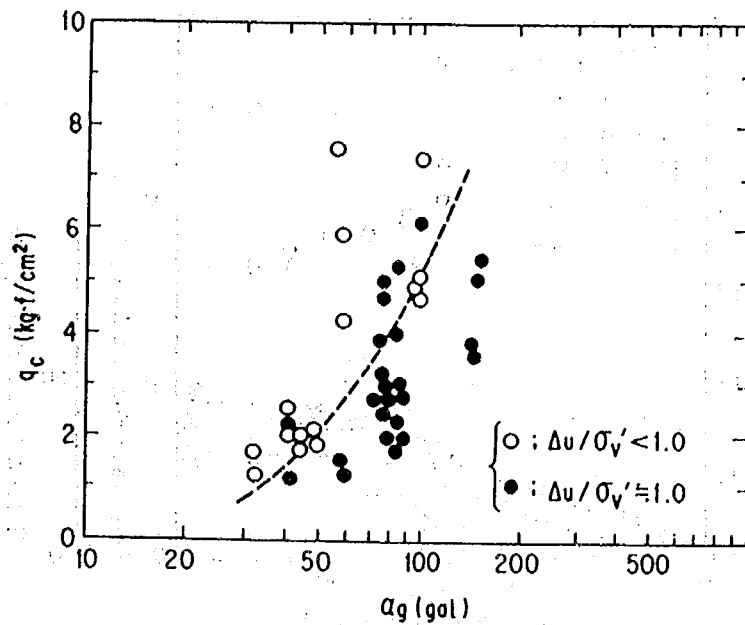


Fig. 5.10 Relation between Ground Surface Acceleration (a_g in gals) and Static Cone Penetration Resistance (q_c in kgf/cm^2) [Refs. 5-5 and 5-6]

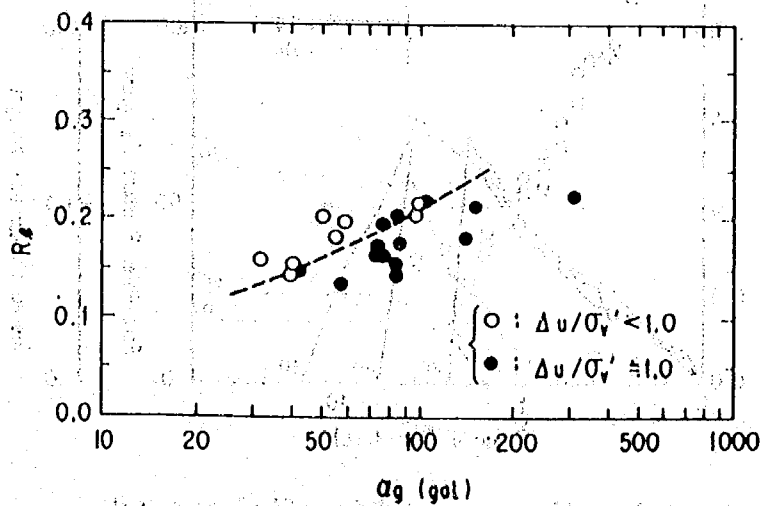
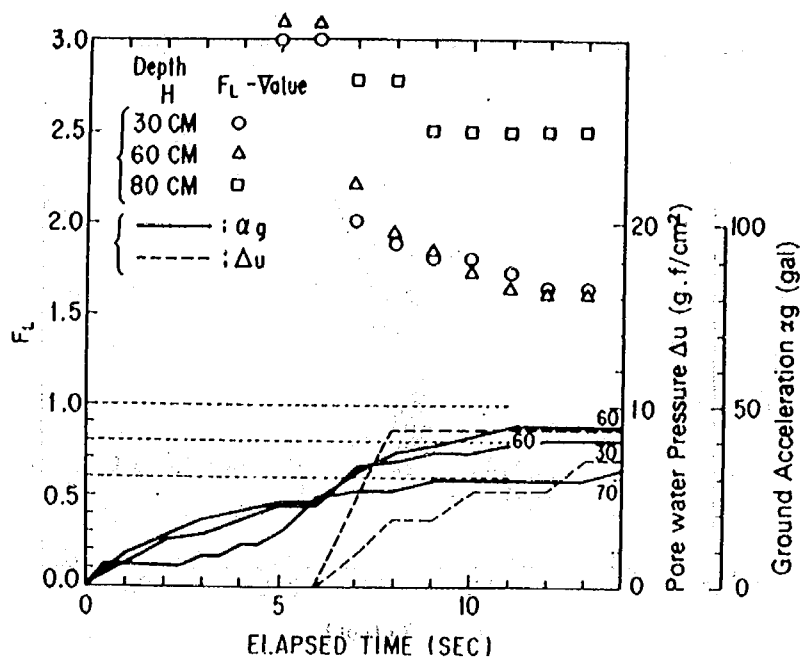
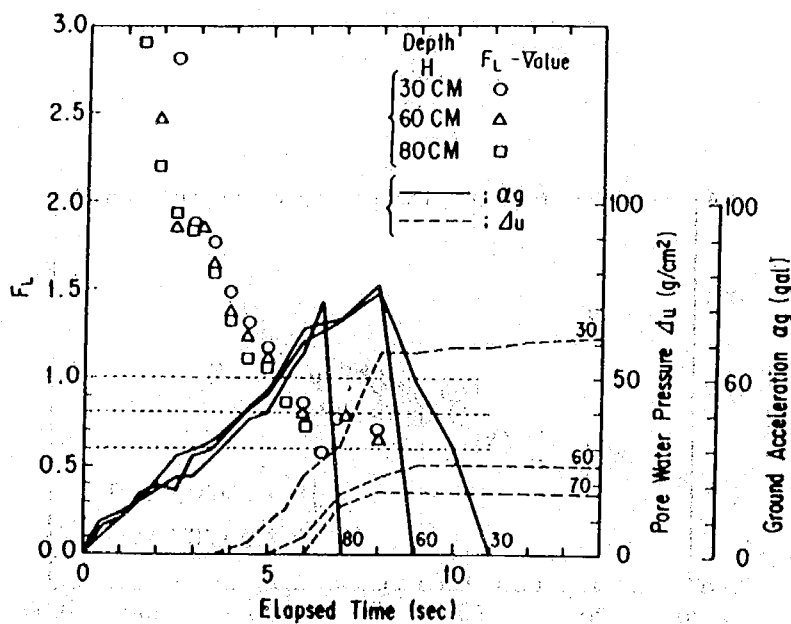


Fig. 5.11 Relation between Ground Surface Acceleration (a_g in gals) and Cyclic Shear Strength (R_s) [Refs. 5-5 and 5-6]



(a) In the Case of Nonliquefied ground



(b) In the Case of Liquefied ground

Fig. 5.12 Time Histories of Pore Water Pressure (ΔU), Ground Surface Acceleration (α_g) and Liquefaction Resistance Factor (F_L) During Shaking [Refs. 5-5 and 5-6]

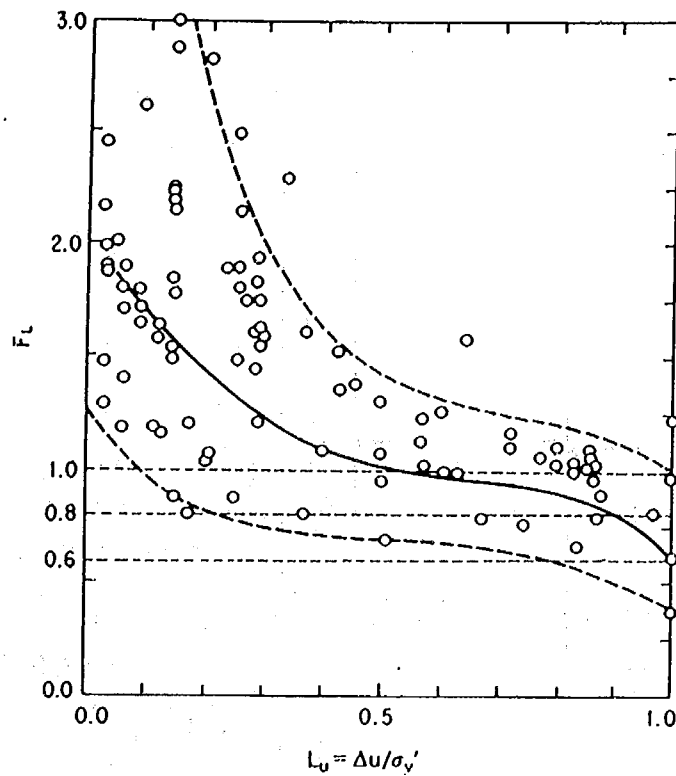


Fig. 5.13 Relation between Liquefaction Resistance Factor (F_L) and Excess Pore Water Pressure Ratio ($\Delta U/\sigma_v'$) Based on Shaking Table Tests [Refs. 5-5 and 5-6]

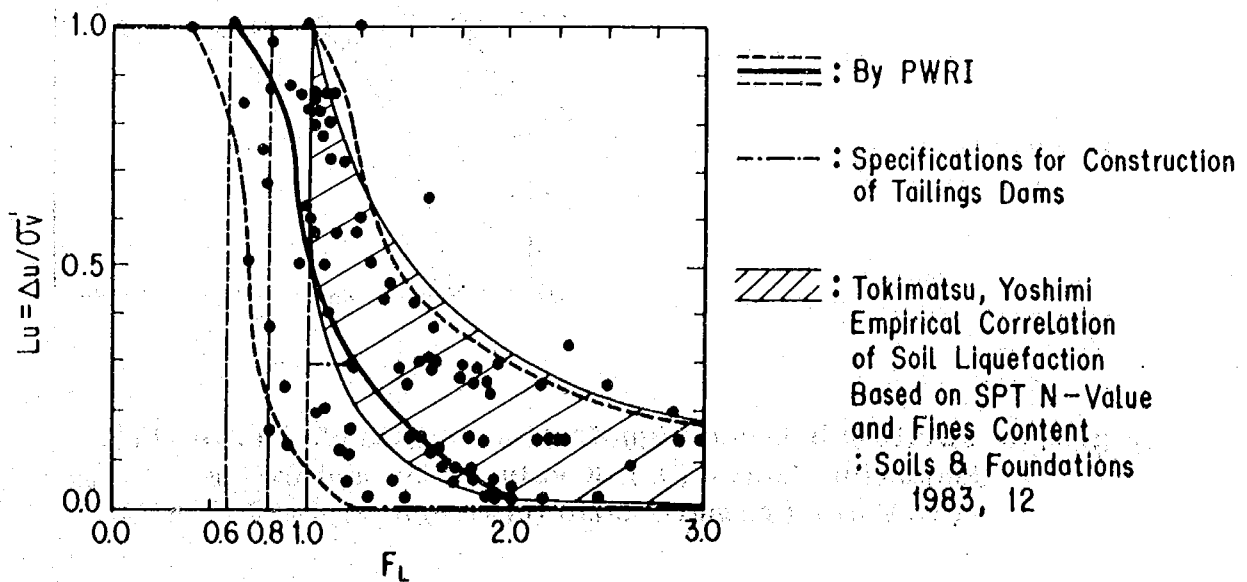


Fig. 5.14 Comparison of Relation between Liquefaction Resistance Factor (F_L) and Excess Pore Water Pressure Ratio ($\Delta U/\sigma_v'$) [Ref. 5-6]

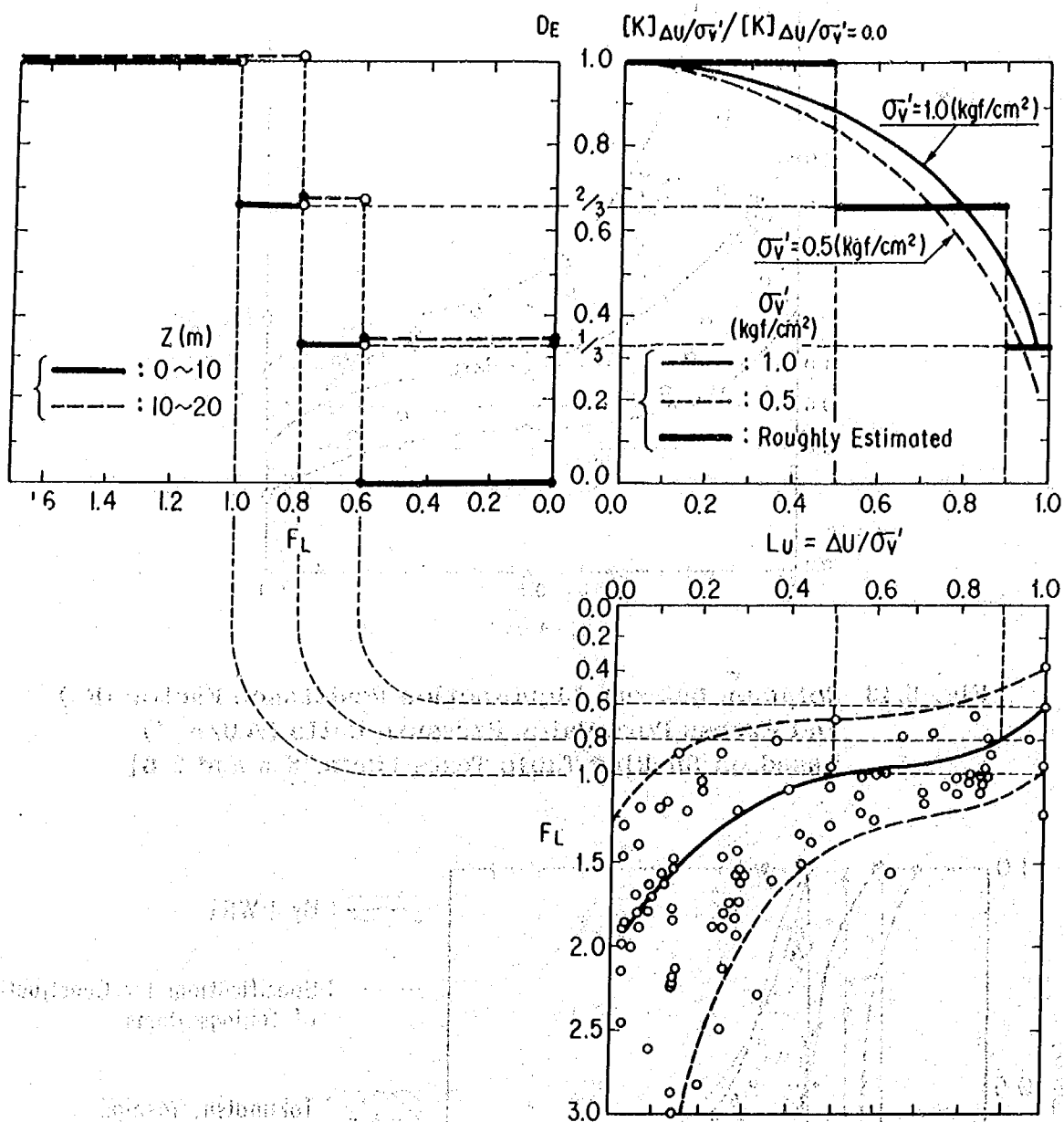


Fig. 5.15 Relation between Liquefaction Resistance Factor (F_L), Reduction Factor (D_E), K-value Ratio and Excess Pore Water Pressure Ratio ($L_u = \Delta U / \sigma'_v$)

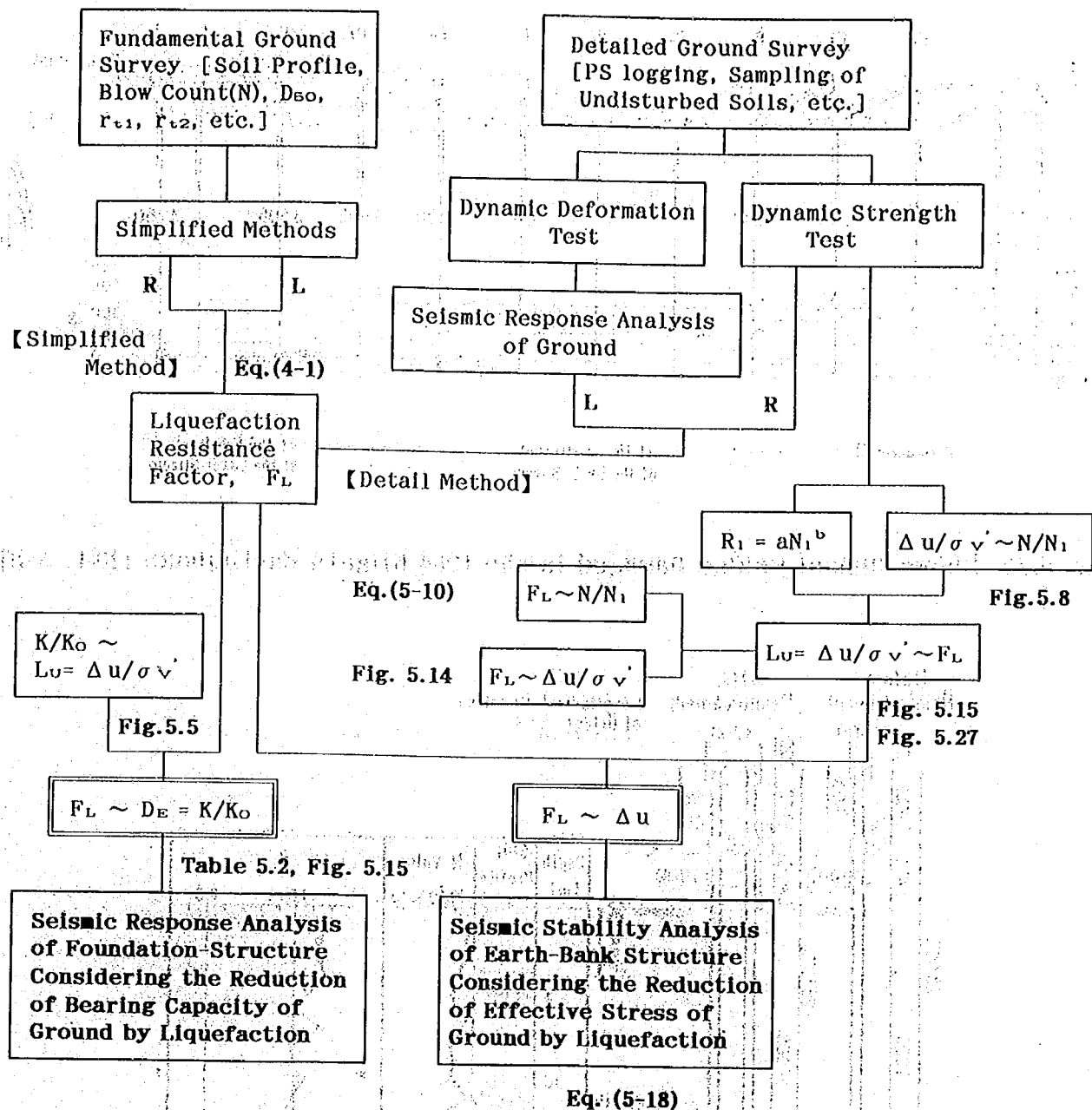


Fig. 5.16 Methods for Investigating the Seismic Stability of Structures Considering the Effects of Liquefaction with use of F_L -value [Ref. 5-1]

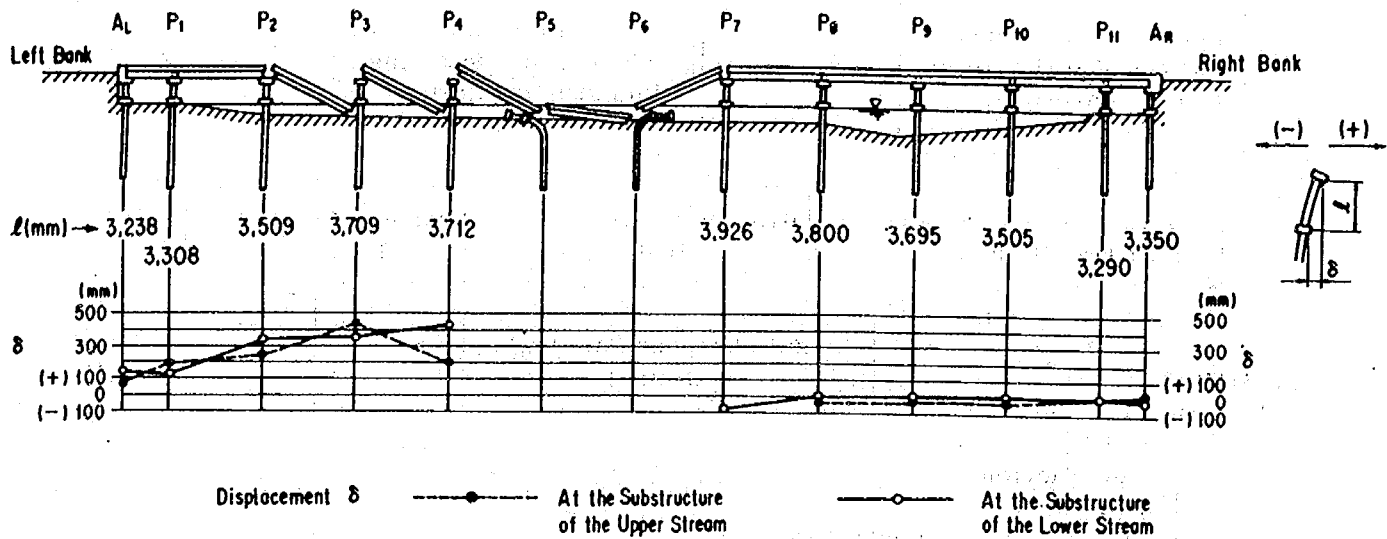


Fig. 5.17 Showa-Ohashi Bridge Damaged in the 1964 Niigata Earthquake [Ref. 5-8]

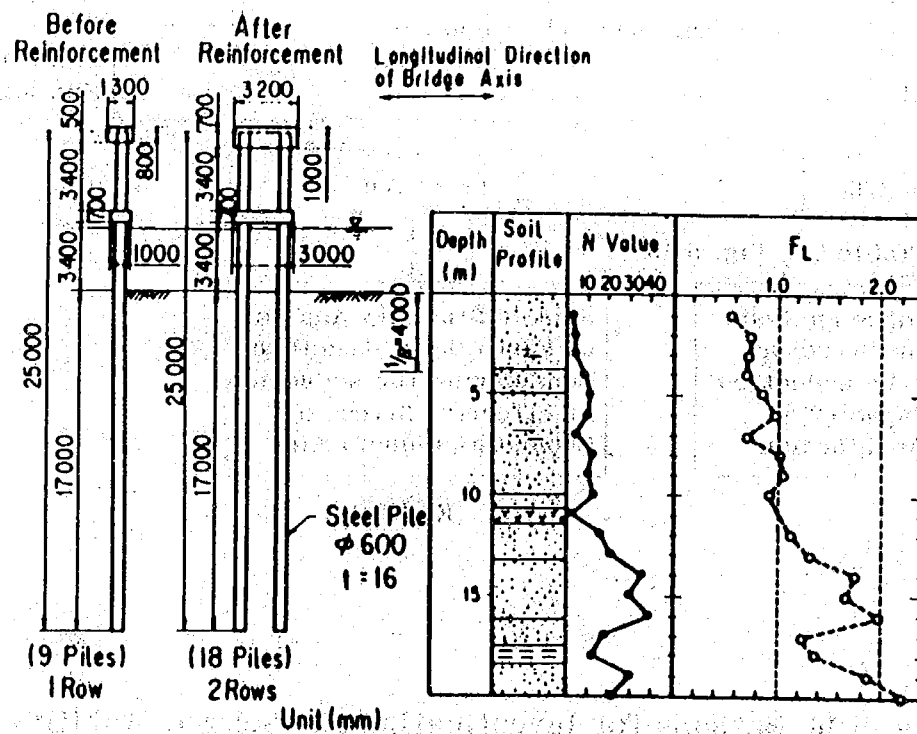


Fig. 5.18 Showa-Ohashi Bridge Pier 5 Reinforced After the Niigata Earthquake and Liquefaction Potential of Subsurface Ground with use of F_L -value [Refs. 5-6 and 5-7]

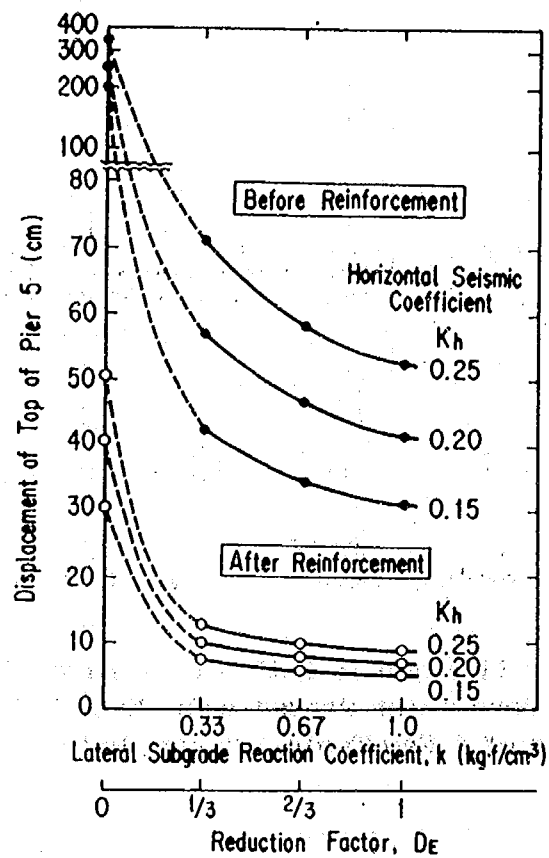


Fig. 5.19 Relation between Maximum Lateral Displacement of Top of Pier 5 and Reduction Factor, D_E or Coefficient of Lateral Subgrade Reaction of Ground, k (kgf/cm^2) According to Horizontal Seismic Coefficient, K_h [Refs. 5-6 and 5-7]

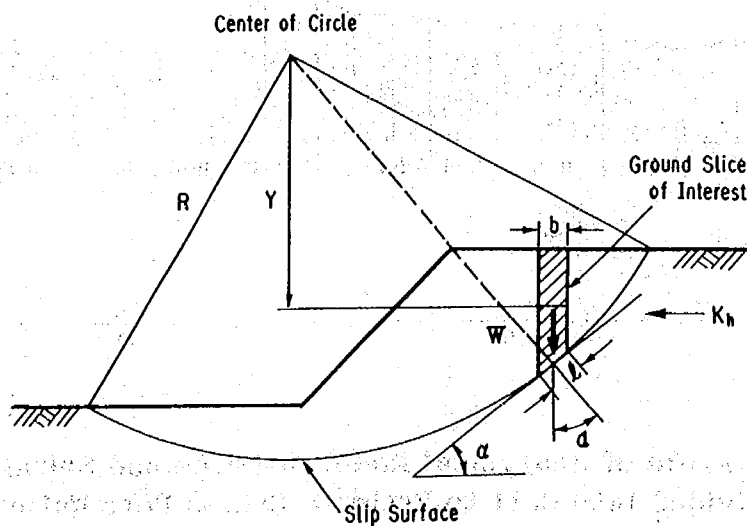


Fig. 5.20 Circular Arc Analysis Method

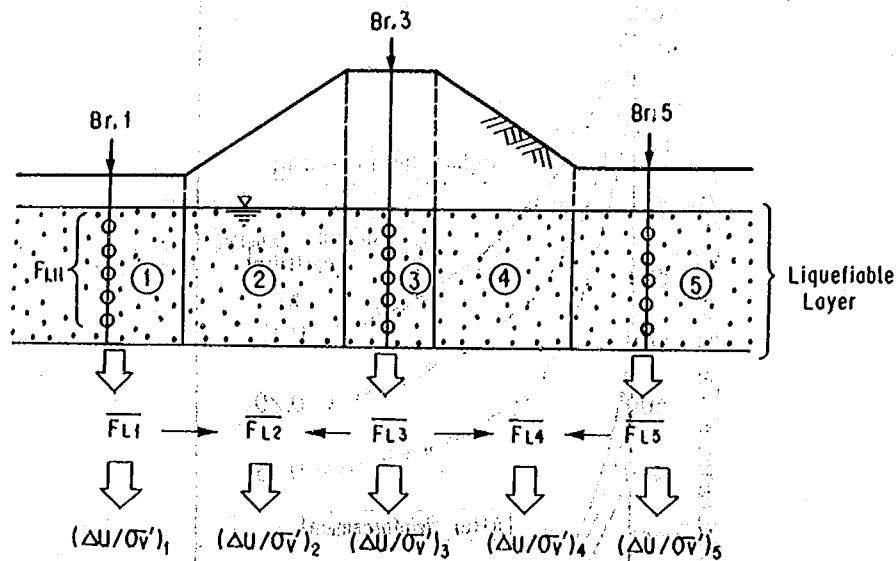


Fig. 5.21 Simplified Method to Estimate Average of F_L -value and Excess Pore Water Pressure for Each Cell of Layers with Use of A Few Typical Boring Data

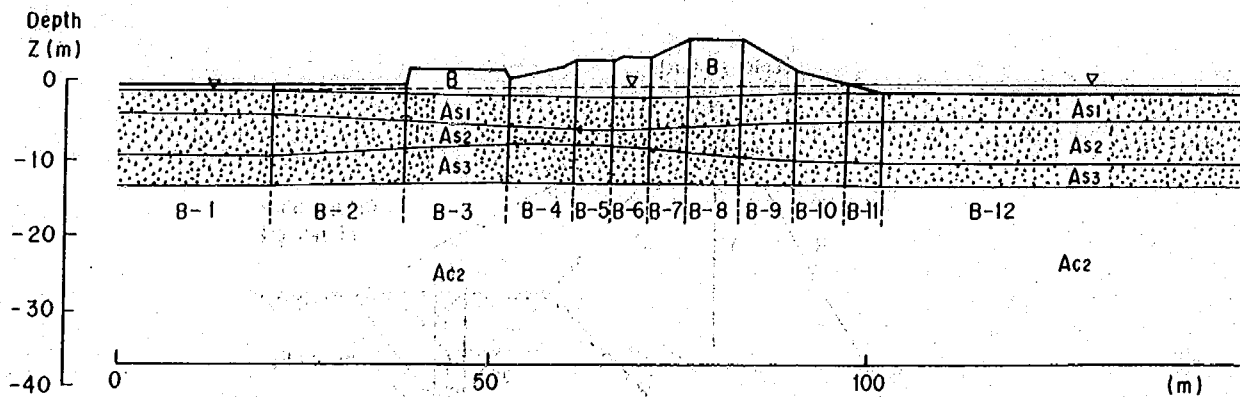
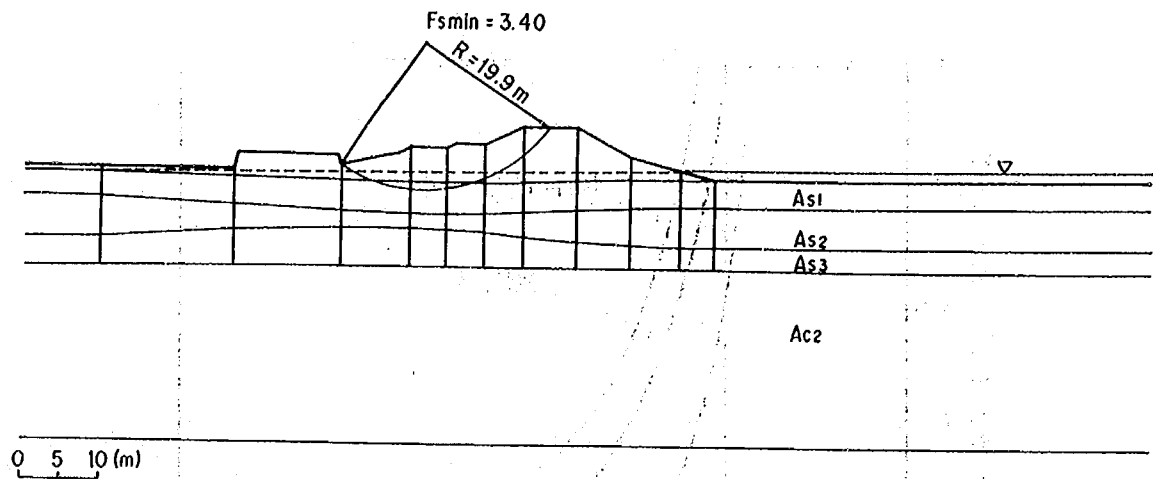
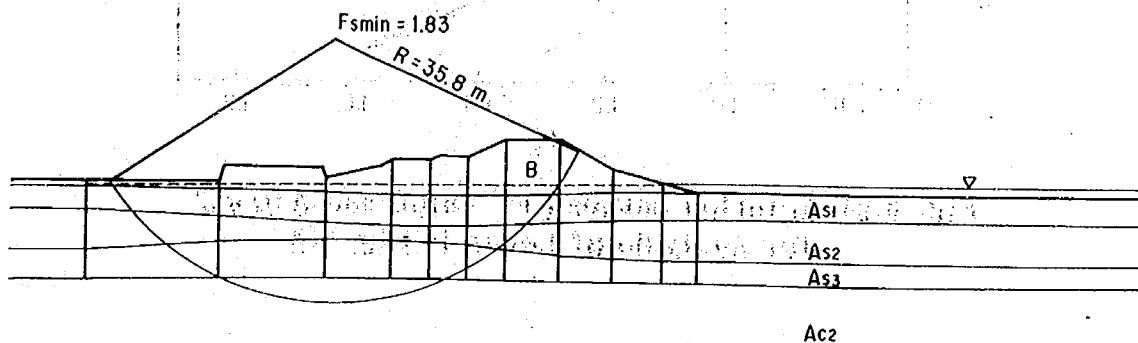


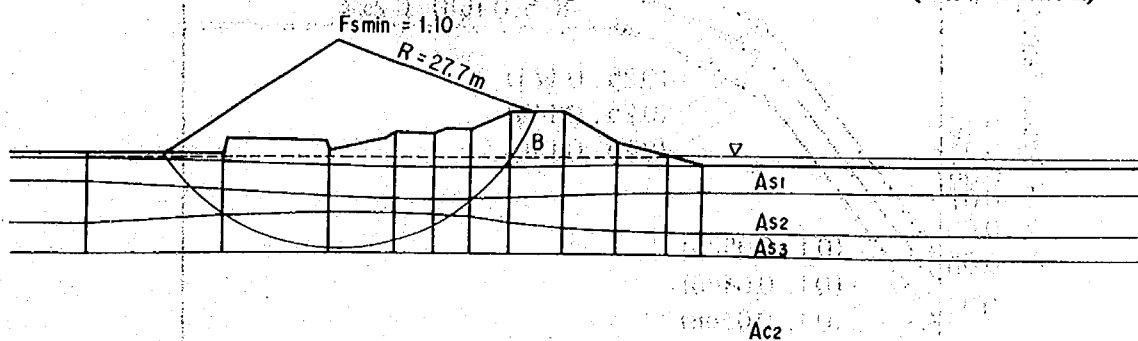
Fig. 5.22 Example of Analytical Model of Levee and Subsurface Ground Divided into Cell to Estimate Excess Pore Water Pressure



(a) Static State without Seismic Coefficient, K_h and Excess Pore Water Pressure Δu



(b) Seismic Analysis Considering Seismic Coefficient (K_h -Method)



(c) Seismic Analysis Considering Excess Pore Water Pressure (Δu -Method)

Fig. 5.24 Circular Slide Surface and Minimum Safety Factor of Levee Calculated by Circular Arc Method

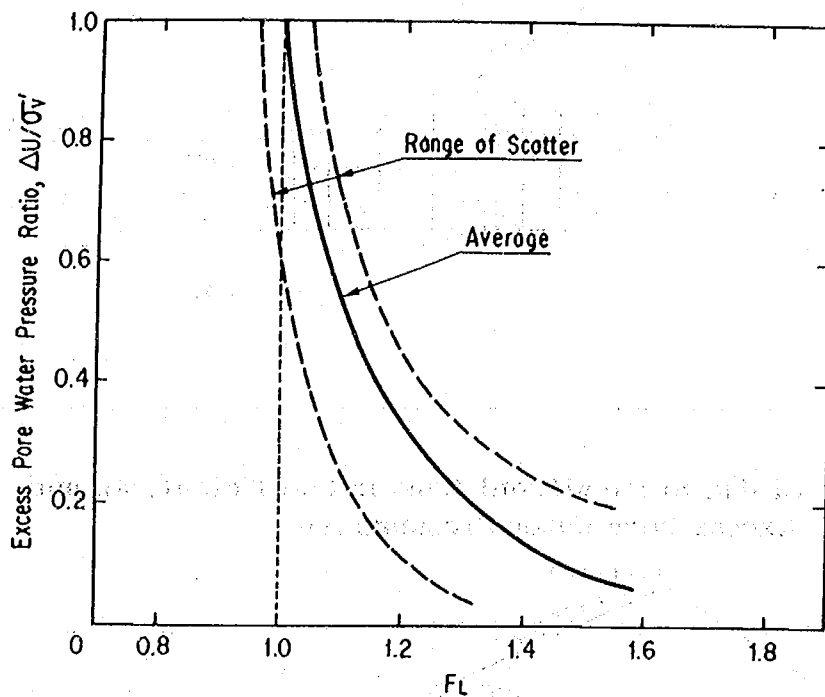


Fig. 5.23 Relation between F_L -value and $\Delta u / \sigma_v'$ for Analysis of Levee in Fig. 5.21

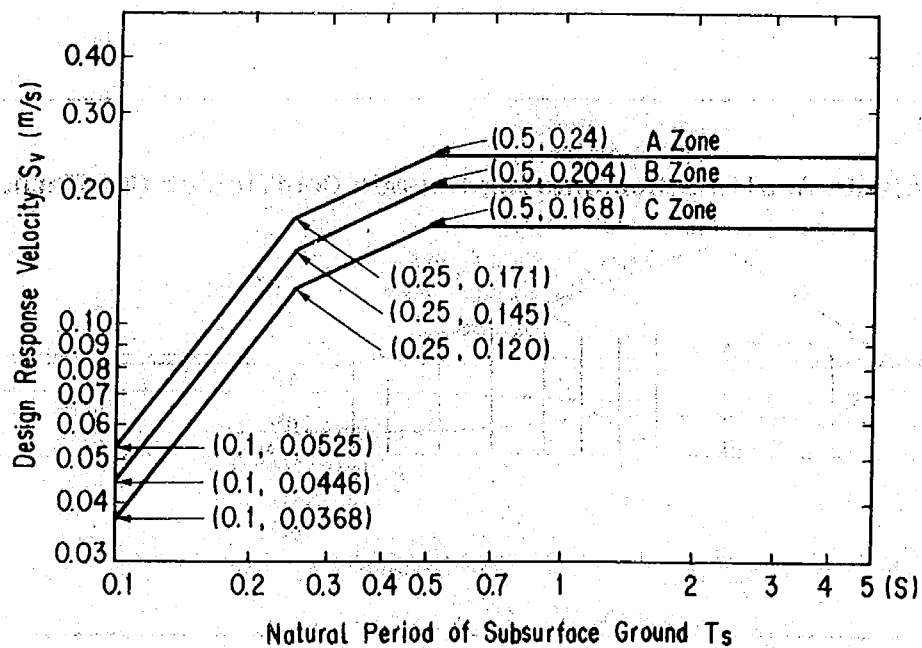


Fig. 5.25 Design Response Velocity Spectrum for Seismic Design of Common Utility Duct [Ref. 5-9]

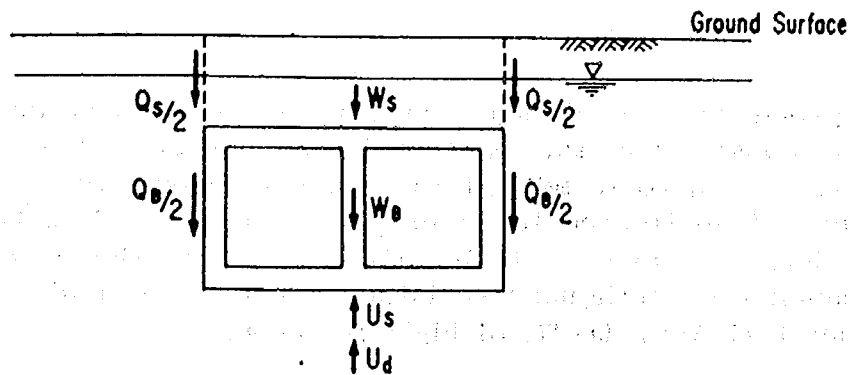


Fig. 5.26 Design Force and Resistance Related to Common Utility for Checking Up-lift [Ref. 5-9]

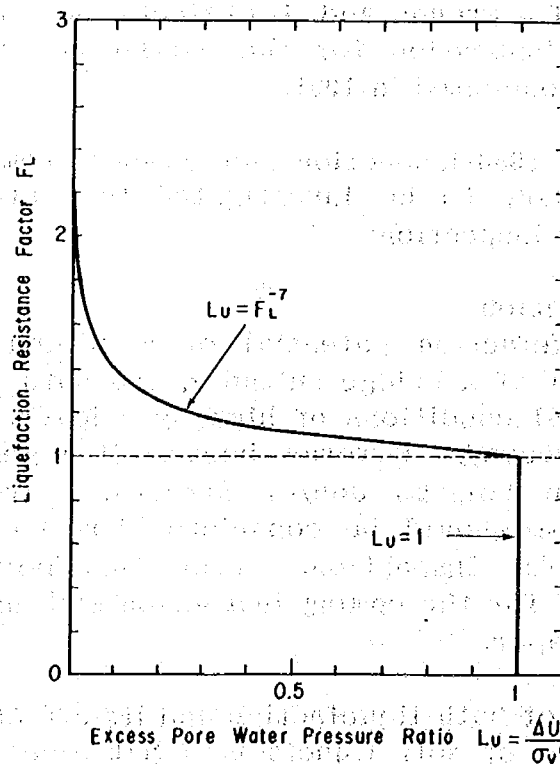


Fig.5.27 Relation between F_L -value and Excess Pore Water Pressure Ratio for Design of Common Utility Duct [Refs. 5-9 and 5-10]

CHAPTER 6 SEISMIC INSPECTION PROCEDURES OF EXISTING STRUCTURES CONSIDERING LIQUEFACTION POTENTIAL

6.1. FOREWORD

Two kinds of procedures to reduce the earthquake-induced damage of highway bridges can be considered; the earthquake design method for bridges to be constructed and the retrofit method for existing ones. The earthquake design methods have been investigated for a long time and applied to the actual design of highway bridges in Japan [Refs.9-1 and 9-2]. On the other hand, the inspections to check the earthquake resistance of existing bridges also have been carried out since 1971 for retrofit of highway bridges.

Table 6.1 shows the history of procedures for inspecting the earthquake resistance potential of highway bridges in Japan. As shown in this table, soil liquefaction was not inspected directly at the first stage of inspection in 1971 and 1976. However, liquefaction potential of a ground was noted concretely for the first time in the 1979-inspection. Furthermore, in the 1986-inspection, both liquefaction potential of a ground and its effects on bridge foundations were considered. The next inspection for the earthquake resistance potential of highway bridges will be conducted in 1991.

Based on results of the 1986-inspection and lessons from the past earthquakes, the following is necessary to be investigated for establishing a reasonable procedure for the coming inspection:

1) Lack of data for inspection

For inspecting the liquefaction potential of a ground and the liquefaction-induced damage potential of a bridge suitably, the data related to both ground conditions and structural conditions of highway bridges, especially, foundation conditions should be surveyed. However, because it is generally difficult to get the data, especially on borings and/or structures related to the bridges constructed long ago, it should be considered how to deal with the bridges without suitable data for inspection. From this point of view, simplified procedures are proposed for the coming inspection with use of data which can be obtained easily in this paper.

2) Reasonable estimation of both liquefaction and its effects on structures

Not only the occurrence of soil liquefaction but also the effects induced by liquefaction should be considered. From this point of view, two kinds of indexes; the site liquefaction potential and the liquefaction-induced damage potential are proposed to estimate the scale of liquefaction and damage of highway bridges more quantitatively than the 1986-inspection in this paper.

6.2. GROUND LIQUEFACTION POTENTIAL

6.2.1 Indexes of Ground Liquefaction Potential

As a first stage for inspecting the earthquake resistant potential of highway bridges, a ground liquefaction potential defined as the probability and/or scale of occurrence of soil liquefaction at the bridge site should be estimated roughly and/or in detail according to the data obtained. In this paper, indices for estimating the ground liquefaction potential are proposed as follows :

1) Ground Group and Topographical Group;

Because these indices can be obtained easily compared with boring data, they can be applied for inspecting the ground liquefaction potential roughly [see Section 6.2.3] .

2) Liquefaction Resistance Factor (F_L), Modified Thickness of Liquefiable Layer (H^*) and Site Liquefaction Potential (I_{PL});

Although it is necessary to get boring data and test results of soil properties to calculate these indexes, they can be applied for inspecting the ground liquefaction potential in detail [see Section 6.2.2] . Furthermore, the relation between the site liquefaction potential of the objective bridge site and the neighboring site of the bridge is investigated in Section 6.2.4.

6.2.2 Procedure for Detailed Inspection of Ground Liquefaction Potential

Liquefaction potential for sandy soil layers can be examined based on the liquefaction resistance factor F_L defined by the following equation applied in the "Specification for Highway Bridges" [Refs. 6-1 and 6-2] .

$$F_L = \frac{R}{L} \quad (6-1)$$

where F_L is the liquefaction resistance factor, R is the resistance of soil elements to dynamic loads and L is the dynamic load applied to the soil elements. R and L are estimated simply with use of the following equations:

$$R = R_1 + R_2 + R_3 \quad (6-2)$$

$$R_1 = 0.0882 \sqrt{N/(\sigma_v' + 0.7)} \quad (6-3)$$

$$R_2 = \begin{cases} 0.19 & (0.02 \text{ mm} \leq D_{50} \leq 0.05 \text{ mm}) \\ 0.225 \log_{10}(0.35/D_{50}) & (0.05 \text{ mm} < D_{50} \leq 0.6 \text{ mm}) \\ -0.05 & (0.6 \text{ mm} < D_{50} \leq 2.0 \text{ mm}) \end{cases} \quad (6-4)$$

$$R_3 = \begin{cases} 0.0 & (0 \% \leq F_c \leq 40 \%) \\ 0.004 \cdot F_c - 0.16 & (40 \% < F_c \leq 100 \%) \end{cases} \quad (6-5)$$

$$L = r_d \cdot k_s \cdot (\sigma_v / \sigma_v') \quad (6-6)$$

$$r_d = 1.0 - 0.015 \cdot z \quad (6-7)$$

$$k_s = C_2 \cdot C_G \cdot C_1 \cdot k_{s0} \quad (6-8)$$

where R_1 is the resistance based on relative density, R_2 is the resistance based on D_{50} -values, R_3 is the resistance based on fines content, N is the blow count by the standard penetration test, D_{50} is the mean grain size (mm), F_o is the fines content (%), z is the depth from the actual ground surface (m), k_s is the seismic coefficient for evaluating liquefaction, C_z is the seismic zone factor, C_o is the ground condition factor, C_i is the importance factor, k_o is the standard seismic coefficient ($= 0.15$), σ_v is the total overburden (kgf/cm^2) and σ_v' is the effective overburden pressure under static conditions (kgf/cm^2).

Soil layers with the liquefaction resistance factor F_L smaller than 1.0 are judged to liquefy during earthquakes. However the F_L -value is very suitable to estimate the liquefaction potential of a certain layer at the objective depth, another index is necessary to estimate the liquefaction potential of the whole subsurface ground in stead of F_L -value. In this paper, the following index is proposed as the simplified parameter to represent the ground liquefaction potential.

$$I_{PL} = \frac{H^*}{20} \quad (6-9)$$

$$H^* = H_1^* + H_2^* \quad (6-10)$$

$$H_1^* = 1.5H_{FL1} + 1.0H_{FL2} + 0.5H_{FL3} \quad (6-11)$$

$$H_2^* = 1.0H_{FL1} + 0.5H_{FL2} \quad (6-12)$$

where I_{PL} is the site liquefaction potential [≤ 1.25], H^* is the modified thickness of liquefiable layers, H_1^* is the modified thickness of liquefiable layer at the depth of 0 to 10 m, H_2^* is the modified thickness of liquefiable layer at the depth of 10 to 20 m, H_{FL1} is the total thickness of layers with $F_L \leq 0.6$ (m), H_{FL2} is the one with $0.6 < F_L \leq 0.8$ (m) and H_{FL3} is the one with $0.8 < F_L \leq 1.0$ (m).

The coefficients in the parameters H_1^* and H_2^* are decided for convenience sake, by referring the relation in Table 6.2 [Ref. 6-1] where bearing capacities of liquefied ground are either neglected or reduced by reduction factors D_E determined in accordance with F_L -values.

The damages of bridges induced in past earthquakes in Japan are investigated in view of the effects of liquefaction and so on. Table 6.3 shows typical examples of bridges which can be estimated to be damaged mainly by liquefaction. Site liquefaction potential (I_{PL}) is calculated with the use of boring data at the bridges as shown in Table 6.3. Furthermore, the damages of bridges related to liquefaction during earthquakes are classified into five levels as shown in Table 6.4 in this paper. For the bridges in Table 6.3, the relation between the damage level of bridges and the site liquefaction potential (I_{PL}) can be drawn as shown in Fig. 6.1. The standards on site liquefaction potential can be classified into three groups as follows, considering the damage level of bridges as shown in Table 6.3.

【 $0.0 \leq I_{PL} < 0.2$ 】 There was not any case of damage related to girder fall induced by liquefaction. The scale of liquefaction and its effects on bridges can be assumed to be small.(Estimation-1)

【 $0.2 \leq I_{PL} < 0.4$ 】 There were cases of damage related to liquefaction. The effects of liquefaction on bridges can be assumed to exist.(Estimation-2)

【 $0.4 \leq I_{PL} \leq 1.25$ 】 There were cases of girder fall induced by liquefaction. The scale of liquefaction and its effects on bridges can be assumed to be very severe.(Estimation-3)

From the above results, it can be indicated that the index of I_{PL} can be used as one index to estimate the ground liquefaction potential at a bridge site. The I_{PL} values of 0.2 and 0.4 can be applied as the boundary to judge the level of ground liquefaction potential, considering the damage level of bridges.

6.2.3 Procedure for Macro-Inspection of Ground Liquefaction Potential

Both ground group and topographical group are selected as the indices for macro-inspection of ground liquefaction potential. Their effectiveness is investigated in this section.

6.2.3.1 Ground Group and Site Liquefaction Potential

The ground condition can be classified roughly according to the stiffness and/or amplification characteristics of ground at the bridge site. In this paper, the ground conditions are assumed to be classified into three groups; Ground Group I, Ground Group II and Ground Group III as shown in Table 6.5 which shows the classification of ground groups defined in the "Specification for Highway Bridge" [Refs. 6-1 and 6-2].

Table 6.6 shows the results investigated on the relation between the ground group and the site liquefaction potential (I_{PL}) at 217 sites of bridges. As shown in Table 6.6, the sites included in Ground Group I can be assumed as the ones with a low liquefaction potential related to the damage of bridges, considering the Estimation-1 of Section 2.2. In other words, the sites classified as Ground Group II or III are in danger to have high liquefaction potential related to damage of bridges.

6.2.3.2 Topographical Group and Site Liquefaction Potential

It is well known that the topographical condition is related to the probability of liquefaction occurrence. In this paper, topographical conditions are classified roughly into three groups, i.e., A, B and C which are proposed as Table 6-7 for inspecting the earthquake resistant potential of highway bridges.

Table 6.8 shows the results investigated on the relation between the

topographical group and the site liquefaction potential (I_{PL}) at 332 sites of bridges. As shown in Table 6.8, the site classified as Topographical Group A can be assumed to have high liquefaction potential related to the damage of bridges. In other words, the sites classified as Ground Group B or C can be assumed to have low liquefaction potential related to damage of bridges.

6.2.4 Liquefaction Potential at Neighboring Sites

For dealing with the problem on the lack of boring data at the site of the objective bridge, the relation of liquefaction potential between the bridge site and its neighboring site is investigated with use of the index defined by the equation (6-13).

$$HT = H_1^{**} + H_2^{**} \quad (6-13)$$

$$H_1^{**} = 1.0H_{FL1} + (2/3)H_{FL2} + (1/3)H_{FL3} \quad (6-14)$$

$$H_2^{**} = (2/3)H_{FL1} + (1/3)H_{FL2} \quad (6-15)$$

where HT is the modified thickness of liquefiable layer (m) [$\leq 16(2/3)m$], H_1^{**} is the modified thickness of liquefiable layer at the depth of 0 to 10 m and H_2^{**} is the modified thickness of liquefiable layer at the depth of 10 to 20 m. H_{FL1} , H_{FL2} and H_{FL3} are shown in Equations (6-11) and (6-12). The coefficients in the parameters H_1^{**} and H_2^{**} are also decided in the same way with the parameters H_1^* and H_2^* shown in the equations (6-11) and (6-12).

For the above investigation, 187 boring data with liquefiable layers are selected within a radius of 750 m around 10 bridges. The modified thickness of liquefiable layer (HT) of selected sites distributes as shown in Fig. 6.2. Five items which seem related to liquefaction and their categories are selected to investigate the relations between the boring data of a bridge site and the ones at its neighboring sites as follows:

(Item-1) Difference of above sea level

Category: $\sim -3m \sim -1m \sim 1m \sim 3m \sim$

(Item-2) Distance

Category: $0 \sim 100m \sim 300m \sim 500m \sim 750m$

(Item-3) Difference of ground water level

Category: $\sim -3m \sim -1m \sim 1m \sim 3m$

(Item-4) Difference of ground group

Category: -1, 0, 1, 2

(Item-5) Difference of topographical group

Category: -1, 0, 1, 2

Fig. 6.3 shows the distribution of data according to the categories of each items. The regression coefficient and correlation between the difference of modified thickness (ΔHT) and the above items with each category are calculated with use of type I quantification analysis, and the results are summarized in Table 6-9. Based on the results of Table 6.9, however all the correlations are very low, the three correlations on the differences of ground water level, topographical group and ground group are relatively high compared to other items. Furthermore, as for the two items of difference of ground water level and topographical group, the difference of modified thickness (ΔHT) is inclined to increase according to the increase of them.

From the above investigation, the difference of topographical group, difference of ground group and difference of ground water level seem to relate to the liquefaction potential between two sites, relatively. The guidelines to adopt the boring data at the neighboring site for estimating site liquefaction potential (I_{PL}) of the objective bridge can be proposed as shown in Table 6.10 for convenience sake.

6.3. LIQUEFACTION-INDUCED DAMAGE POTENTIAL OF BRIDGE

6.3.1 Indices Related to Damage of Bridge by Liquefaction

For surveying the indices related to the liquefaction induced damage of bridge foundations, the following five indices are selected considering the results of Table 6.3:

- I_G : Supported condition on pier-top;
- I_F : Foundation type;
- I_{FS} : Bearing condition of foundation;
- I_{PL} : Site liquefaction potential; and
- I_{GC} : Ground group.

Table 6.11 shows the categories of each index mentioned above. With use of type I quantification analysis, five cases for analyzing the effects of these indices to the damage of bridges are conducted by combining them as shown in Table 6.12. In this analysis, the damage level (see Table 6.4) of bridge foundations is defined as a dependent variable and the independent variables are combined with the indices as show in Table 6.12.

Table 6.13 summarizes the results of category-scores and correlation-coefficients for each case. From these results, it can be seen that the three indices, i.e., I_F , I_{FS} and I_{HL} can explain the damage level of bridge foundations relatively well.

6.3.2 Simplified Method for Estimating Liquefaction-Induced Damage Potential of Highway Bridge

It should be noted that the damage patterns of bridge foundations induced by soil

liquefaction is different according to the foundation type as shown in Table 6.14. In other words, it is desirable to select the suitable indices to explain the cause of damage induced by liquefaction. Based on results in Section 3.1 and lessons from past earthquakes, three indices, i.e., P I, P II and P III are proposed in Equations (6-16-1), (6-16-2) and (6-16-3), respectively, for convenience sake to estimate the liquefaction-induced damage potential of bridge foundations according to the foundation type (see Table 6.15). For estimating these indexes quantitatively, the weighted points are assumed roughly, considering the scale of effects to damage of bridges.

Abutment:

$$P I = (I_G + I_J + I_F + I_{FS}) \times I_{PL} \times I_{HB} \quad (6-16-1)$$

Movable Pier:

$$P II = (I_G + I_F + I_{FS}) \times I_{PL} \times I_{GC} \quad (6-16-2)$$

Fixed Pier:

$$P III = (I_F + I_{FS}) \times I_{PL} \quad (6-16-3)$$

where,

I_G : Effect of device against fall of a girder

Continued girder, Fixed girder : 0.0

Both insurance of S_E and

Installation of stopper/connection : 0.5

Insurance of S_E or Installation of

stopper/connection : 1.0

Not insured : 1.5

(Note) S_E : Distance between the end of girder and the crest of a substructure.

I_J : Effect of girder-support condition on the top of pier/abutment

Movable bearing : 0.0

Fixed bearing : 1.0

I_F : Effect of foundation-type

Pile, Caisson : 1.0

Wooden pile, Pile bent : 2.0

I_{FS} : Effect of bearing condition at the bottom of foundation

Perfect support : 1.0

Imperfect support : 5.0

Shallower than liquefied layer : 6.0

I_{HB} : Effect by flow of abutment-backfill

(Represented with Height of backfill: H_B)

$H_B < 5.0$ m : 1.0

$H_B \geq 5.0$ m : 1.2

I_{PL} : Effect by liquefaction (Site liquefaction potential : I_{PL})

$0.2 \leq I_{PL} < 0.4$: 1.0

$0.4 \leq I_{PL}$: 1.5

(Note) The site with $I_{PL} < 0.2$ is omitted because of low liquefaction potential.

I_{GC} : Effect by deformation of surrounding ground (Represented with Ground group)

Ground group II : 1.0

Ground group III : 1.2

6.3.3 Liquefaction-Induced Damage Potential of Highway Bridge in the Past Earthquakes

The simplified procedure for estimating liquefaction-induced damage potential of highway bridge proposed in Section 3.2 are applied to the bridge foundations damaged in the past earthquakes as shown in Table 6.3. Figs. 6.4(1), 6.4(2) and 6.4(3) show the relations between liquefaction-induced damage potential and damage level for abutment, movable type of pier and fixed type of pier, respectively. Based on these results, the boundary values of indices related to the severe damage of bridges, i.e., with the damage level more than 4 can be estimated as follows:

Abutment:

$PI < 6.0$: Damage induced by liquefaction can be estimated to be small.

$PI \geq 6.0$: Danger of severe damage. Detailed investigation should be necessary. (Relation-4)

Movable Pier:

$PII < 6.0$: Damage induced by liquefaction can be estimated to be small.

$PII \geq 6.0$: Danger of severe damage. Detailed investigation should be necessary. (Relation-5)

Fixed Pier:

$PIII < 4.5$: Damage induced by liquefaction can be estimated to be small.

$PIII \geq 4.5$: Danger of severe damage. Detailed investigation should be necessary. (Relation-6)

6.4. SIMPLIFIED PROCEDURE FOR ESTIMATING EARTHQUAKE RESISTANT POTENTIAL OF HIGHWAY BRIDGE AGAINST SOIL LIQUEFACTION

From the above investigation and other lessons in the past earthquakes, the simplified procedure for estimating earthquake resistant potential of highway bridge against soil liquefaction can be proposed such as in Fig. 6.5. As shown in this flow-chart, the inspection is classified into four stages as follows:

1) First-stage of Fundamental Inspection

In this stage, before inspecting the site liquefaction potential and the liquefaction-induced damage potential, the bridges in the following conditions can be excluded.

(1) Bridges designed with use of the specifications after 1971 in which the effects of liquefaction are considered for earthquake resistant design of bridges.

(2) Bridges whose superstructure can be considered as the types not effected by soil liquefaction severely, for example, Deck Arch Bridge, Rigid-frame Bridge and Culvert Bridge.

2) Second-stage of Site Liquefaction Inspection

In this stage, the site liquefaction potential is inspected before inspecting the effects of liquefaction on bridges. The inspection flow is divided into two flows according to the existence of boring data at the bridge site. When the boring data is possible to get, the site liquefaction potential can be calculated with use of the index of I_{PL} proposed in Equation (6-9). On the other hand, when the data can not be obtained, the ground liquefaction potential is estimated roughly with the use of either ground group or topographical group of the bridge site based on the results in Section 2.3.

Furthermore, for the bridge site judged as both Ground Groups II and III and Topographical Groups A, the boring data at the neighboring site of the bridge site which can be selected by the standard proposed in Table 6.10 is possible to use to calculate the site liquefaction potential (I_{PL}). When the site liquefaction potential (I_{PL}) is less than 0.2, the effects of liquefaction are judged to be small by the Relation-1 shown in Section 2.1 and further inspection is not necessary. When the soil data for the index of I_{PL} can not be got at both bridge site and neighboring site, the inspection should progress in the next stage.

(3) Third-stage of Damage Potential Inspection

In this stage, the liquefaction-induced damage potential of P I, P II and P III should be calculated by the equations (6-16-1), (6-16-2) and (6-16-3) for the abutment, movable pier and fixed pier, respectively. When the values of P I, P II and P III are less than the boundary shown in Relations-4, -5 and -6 in Section 3.3, the damage induced by soil liquefaction is assumed to be small and further inspection is not necessary.

(4) Four-stage of Detail Stability Inspection:

In this stage, the detail inspection should be conducted to calculate the stability and resistance of foundations by the earthquake resistant design of highway bridge [Refs. 6-1 and 6-2]. For unstable bridges for soil liquefaction, countermeasures for reducing the effects of liquefaction should be selected and designed for planning the retrofit of bridges in the future.

6.5. FUTURE REMARKS

The results introduced in this paper will be referred in the next inspection of earthquake resistant potential for highway bridges conducted in 1991. The further investigation will be carried out after obtaining the results of the next inspection to improve the procedure more reasonably in the future.

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Table 6.1 History of Inspection for Earthquake Resistance Potential of Highway Bridge Considering Liquefaction Effects in Japan

Stage		Inspection Procedure for Liquefaction
1	1971	• Views by Site Observation. • Liquefaction is not considered directly.
2	1976	• Check List for Present Condition and Damage. • Damage Potential (A, B, C) for Each Check-Index. • Liquefaction is not considered directly.
3	1979	• First- and Second-stage Inspection. • Ground /Superstructure/Substructure. • Calculation of Bearing Capacity of Pile (2nd-stage). • Estimation of Cost for Countermeasure. • Liquefaction potential of ground is considered. 1st-stage : Existence of Saturated Sandy Layers (Yes/No). 2nd-stage : Thickness of Saturated Sandy Layers (~ 1.0 m ~ 10.0 m ~).
4	1986	• First-, Second- and Third-stage Inspection. • Ground /Superstructure/Substructure. • Calculation of Bearing Capacity of Pile (2nd-stage). • Estimation of Cost for Countermeasure. • Liquefaction potential of ground and its effect to foundation are considered. 1st-stage : Existence of Saturated Sandy Layers (Yes/No). 2nd-stage : Calculate F_L and Thickness (H_L) of Layer with F_L less than 0.6. - Compare H_L with Depth of Foundation.
5	1991	• First-, Second- and Third-stage Inspection. • Ground /Superstructure/Substructure. • Check of Foundation Stability for Liquefaction. • Selection of Countermeasure. • Liquefaction potential of ground and its effect to foundation are considered in detail more than 1986 Inspection. 2nd-stage: Survey Several Indexes (Topographical Co., Ground Co. and Existence of Saturated Sandy Layer). - Calculate F_L and r_{HL}. - Calculate P_I, P_{II} and P_{III}. 3rd-stage: Calculate Stability of Foundation.

Table 6.2 Relation Between F_L and D_E [Refs. 6-1, 6-2] and Coefficient for I_{PL}

F_L	Depth, Z (m)	Reduction Factor, D_E	Coefficient for * Modified Thickness
$F_L \leq 0.6$	$0 < Z \leq 10$	0	1.5 **
	$10 < Z \leq 20$	1/3	1.0
$0.6 < F_L \leq 0.8$	$0 < Z \leq 10$	1/3	1.0
	$10 < Z \leq 20$	2/3	0.5
$0.8 < F_L \leq 1.0$	$0 < Z \leq 10$	2/3	0.5
	$10 < Z \leq 20$	1	0.0 ***
$1.0 < F_L$	$0 < Z \leq 20$	1	—

Note: * is assumed to calculate the modified thickness of the site liquefaction potential, I_{PL} .

** is assumed for 0 of D_E .

*** is assumed for 1 of D_E .

**Table 6.3 Typical Bridges Considered to be Damaged
by Liquefaction**

Earth-quake (EQ)	Name of Highway Bridge	Typical Damage of Bridge Structures	Objective Substructure			Damage Level	Liquefaction Phenomena	Site Liq. Pot. I _{FL}
			No.	and Its Damage				
Niigata EQ 1964.6 M= 7.5	Syowa-Bridge	• 5 simple girders (P2~7) fell down. • 2 piers subsided and others inclined. • Pile deformed. etc.	1	P1	Settled(3cm), Pier-top Disp.(34cm)	4	Severe liquefaction occurred.	0.57
			2	P6	Subsided in the River.	5		0.47
			3	P7	Settled(9cm), Pier-top Disp.(7cm)	4		0.47
	Higashi-Kosen Bridge	• 1 simple girder (P8~9) fell down. • Pier subsided and inclined. • Piles deformed. • Retaining wall abutment subsided. etc.	4	AL	Slight damage (Unknown in detail)	2	Severe liquefaction occurred.	0.00
			5	P7	"	2		0.06
			6	P9	Settled(2.5cm), Pier-top Disp.(15cm)	5		0.43
			7	AR	Settled(32cm), Pier-top Disp.(13cm)	4		0.23
	Yachiyo Bridge	• Substructure broke. • 1 girder(P2~3) was in danger of fall. • Retaining wall abutment subsided. etc.	8	AL	Settled(40cm), Inclined with 10.5°	4	Movement of ground by liquefaction occurred.	0.34
			9	P2	Pier-body broke.	5		0.34
			10	P10	Up-lift(7cm), Pier-top Disp.(7cm)	3		0.13
	Taihei Bridge	• Pier inclined. • Slightly damaged.	11	P1	(Not damaged)	1	A few meters of surface liquefied.	0.42
			12	P8	Pier-top Disp.(6cm)	3		0.17
	Shin-Matsushima Bridge	• 1 girder (P10~11) fell down. • Under-construction.	13	P1	(Not damaged)	1	"	0.23
			14	P7	Settled(2cm), Pier-top Disp.(20cm)	4	10 m more liq.	0.78
	Kosudo Bridge	• Abutment and pier subsided.	15	P1	Settled(75cm)	3	(Unknown)	0.17
			16	AR	Settled(7cm)	3		0.00
Miyagi-Ken-Oki EQ 1978.6 M= 7.4	Abukuma Bridge	• Crack/peel-off of pier. Bearing broke.	17	P8	Pier cracked. Bearing broke.	3	Slightly sand boiled.	0.03
	Yuriage Bridge	• Crack of pier/girder /parapet. • Joint damaged.	18	P1	Peel-off of pier. Bearing broke.	3	Slight sand boil.	0.18
			19	P6	Movement of girder(6cm)	3		0.00
	Kaihoku Bridge	• Bearing broke. • Slightly damaged.	20	P2	Bearing broke.	3	Not liquefied.	0.00
			21	P4	Slightly damaged.	2		0.06
			22	AR	Slightly damaged.	2		0.06
	Eai Bridge	• Crack/peel-off of pier	23	P5	Pier cracked. Movement of girder(6cm)	3	(Unknown)	0.43
Kimazuka Bridge	• Base of bearing broke. • In danger of fall.	24	P3	Base of bearing broke.	3	Sand boil.	0.08	
Nihon-Kai-Chubu EQ 1983.5 M= 7.7	Oga Bridge	• Pier subsided and inclined. • Approach settled.	25	P4	Settled(13cm), Pier-top Disp.(23cm)	3	(Sand boil, Crack)	0.00
			26	P5	Settled(15cm), Pier-top Disp.(28cm)	3		0.00
	Ryuma Bridge	• Abutment cracked. • Bearing broke. • Slightly damaged.	27	AL	Abutment-body cracked.	4	Sand boil.	0.00
			28	P2	(Not damaged)	1	(Sand boil)	0.00
	Gomyokoh Bridge	• Backfill of abutment subsided severely. • Bridge-body was damaged slightly.	29	AL	Pile-bent cracked.	3	Sand boil, Crack.	0.03
			30	AR	Slightly damaged.	2		0.00
	Narusawa Bridge	• Backfill of abutment subsided. • Bridge-body was damaged slightly.	31	AL	Slightly damaged.	2	Sand boil, Crack.	0.00
32			P1	Not damaged.	1	0.00		

Note: () shows the unknown or estimated results without actual data.

Table 6.4 Classification of Damage Level of Highway Bridges Induced by Liquefaction

Damage Level	Liquefaction Related to Damage of Bridge	Standard to Estimate Damage Level of Bridge and Countermeasure
5	Liquefaction is main factor and causes severe damage of bridge.	• Girder fell down. • Substructure is broken. • Countermeasure for liquefaction is needed preferentially.
4	Damage of bridge is relatively severe, and liquefaction is considered as one of causes.	• Pier-top displacement by subsidence and/or inclination is more than 10 cm. • Width of crack is more than 1 mm. • Countermeasure for both liquefaction and other cause of damage is needed.
3	Liquefaction is not directly related to damage of bridge.	• Pier-top displacement by subsidence and/or inclination is less than 10 cm. • Width of crack is less than 1 mm. • General retrofit and/or repair for damage is needed.
2	Slightly damaged.	• Slight repair for damage is needed.
1	Not damaged.	

Note: Damage level for severe damage which was not induced by liquefaction is considered to be 3.

Table 6.5 Classification of Ground Group [Refs. 6-1, 6-2]

Group	Characteristic of Ground	TG (s) *
I	• Rock, Diluvial Ground (Firm)	$TG < 0.2$
II	• Alluvial Ground (Medium-stiff)	$0.2 \leq TG < 0.6$
III	• Soft and Thick Alluvial Ground (Soft)	$0.6 \leq TG$

* : Characteristic Value of Ground

Table 6.6 Relation between Ground Group and Site Liquefaction Potential (I_{PL})

Ground Group	Without Saturated Sandy Layer	$I_{PL} < 0.2$	$0.2 \leq I_{PL} < 0.4$	$0.4 \leq I_{PL}$	Total
I	32	7	0	0	39
II	14	75	13	2	104
III	0	34	27	13	74
Total	46	116	40	15	217

Estimate Suitably	Est. Dangerously
Est. Safely	Estimate Suitably

Table 6.7 Classification of Topographical Group

Group		Topographical Condition
A	High Liquefaction Potential	• Reclaimed/Filled-up land • Banked-up ground on the alluvial low ground • Banked-up ground on the former water/sea area • Slight heights on the former river bed • Land by drainage • Tidal flat • Coastal plain • Delta • Natural levee • Former river bed • (Former) Moat • Flood plain • Sand dune • Sand bar • Back low land • (Former) Marsh
B	Medium Liquefaction Potential	• Except for Group A and Group C [Banked-up ground except for A, Fan etc.]
C	Low Liquefaction Potential	• Terrace • Plateau • Heights • Hill • Slope • Mountain

Note: Existing river bed should be estimated, considering the topographical group of the surrounding site of the objective highway bridge.

Table 6.8 Relation between Topographical Group and Site Liquefaction Potential (I_{PL})

Topographic- al Group	Without Saturat- ed Sandy Layer	I _{PL} < 0.2	0.2 ≤ I _{PL} < 0.4	0.4 ≤ I _{PL}	Total
C	21	3	0	0	24
B	15	9	0	0	24
A	50	172	44	18	284
Total	86	184	44	18	332

Estimate Suitably	Est. Dangerously
Est. Safely	Estimate Suitably

Table 6.9 Correlation between Difference of Modified Thickness (ΔHT) and Difference of Indices at Both Objective and Neighboring Sites

Dependent Variable : Difference of Thickness ΔHT (Multiple Correlation Coefficient R = 0.721)					
Independent Variable			Regression Coefficient	Variation Ratio	Correlation
Item	Category				
Difference of above the Sea Level *	< -3m	1	0	0.725	0.031
	< -1m	2	- 0.569		
	≤ 1m	3	- 0.871		
	≤ 3m	4	- 0.492		
	> 3m	5	- 0.557		
Distance	≤ 100m	1	0	0.786	0.066
	≤ 300m	2	- 0.743		
	≤ 500m	3	- 0.221		
	≤ 750m	4	0.079		
Difference of Ground Water Level *	< -3m	1	0	9.283	- 0.392
	< -1m	2	- 2.323		
	≤ 1m	3	- 3.050		
	≤ 3m	4	- 6.575		
Difference of Ground Group *,**	-1	1	0	3.143	0.214
	0	2	- 1.363		
	1	3	- 0.381		
	2	4	0.694		
Dif. Topo- graphical Group ***	0	1	0	7.861	- 0.13
	1	2	- 0.996		
	2	3	- 1.910		

* : Difference = (Objective Site Condition)
- (Neighboring Site Condition)

** : (Example) Category 3 → Dif. 1 = (Ground Group II) - (I)

*** : Topographical Group A = 3, B = 2, C = 1. Dif. is absolute value.

Table 6.10 Standard for Selecting Neighboring Site to Apply Its Boring Data to Objective Site

Item		Limit for Selection
1	Difference of Topographical Group with Objective Site	Same Group or Close Group with 1-Grade High-potential
2	Difference of Ground Group with Objective Site	Same as the Above
3	Difference of Ground Water Level with Objective Site	Within ± 2 m

Note: 1) It is better to select the neighboring site at outside or inside of the river within 200 m or 500 m far from the objective site, respectively, and also within ± 10 m high from the objective site.
 2) The ground water level at the site should be estimated based on the boring data and surrounding ground condition.

Table 6.11 Indices and Its Categories Related to Damage Level of Bridge

Category	Index				
	Supported Co. on Pier-Top I_J	Foundation Type I_F	Bearing Co. of Foundation I_{FS}	Site Liquefaction Potential I_{PL}	Ground Group I_{GC}
1	Movable	Pile, Caisson	Perfect Support	$I_{PL} < 0.2$	II
2	Fixed	Wooden Pile, Pile Bent	Imperfect Support	$0.2 \leq I_{PL} < 0.4$	III
3	Movable and Fixed	—	Shallower than Liquefied Layer	$0.4 \leq I_{PL}$	—

Table 6.12 Indices to Investigate Effects to Damage Level of Bridge

Case	Index				
	I_J	I_F	I_{FS}	I_{PL}	I_{GC}
1	○	○	○	○	○
2	○	○	○	○	—
3	—	○	○	○	○
4	—	○	○	○	—
5	—	—	○	○	—

Note: ○ is selected to investigate its effect to Damage Level.

Table 6.13 Correlation between Indices and Damage Level of Bridge

(Number of Objective Substructure = 32)

Index, Category		I ₁			I _F		I _{FS}			I _{FL}			I _{OC}		Correlation Coefficient
Case		1	2	3	1	2	1	2	3	1	2	3	1	2	
1	Range	3.31			3.38		11.31			11.15			5.55		0.66
	Category Score	1.07	1.58	- 1.04	- 0.95	2.43	- 5.18	4.09	6.13	- 3.06	1.88	8.09	2.08	- 3.4	
2	Range	4.15			3.58		8.67			12.56					0.62
	Category Score	1.42	2.42	- 1.73	- 1.01	2.58	- 4.47	3.87	4.20	- 3.69	3.86	8.87			
3	Range				4.46		11.90			9.75			5.59		0.64
	Category Score				- 1.25	3.21	- 5.06	3.74	6.84	- 2.75	2.13	7.02	2.10	- 3.49	
4	Range				5.11		9.32			10.95					0.61
	Category Score				1.44	3.67	- 4.39	3.55	4.93	- 3.34	4.22	7.61			
5	Range						8.11			12.36					0.57
	Category Score						- 4.20	3.91	3.07	- 3.63	3.75	8.74			

Note: Category Score ☐ Plus(+) = Severe Damage
☐ Minus(-) = Minor Damage

Table 6.14 Classification of Damage of Foundation Induced by Liquefaction

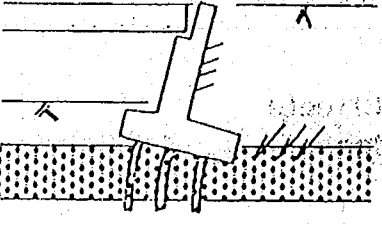
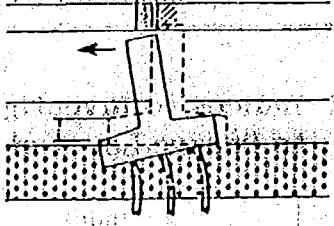
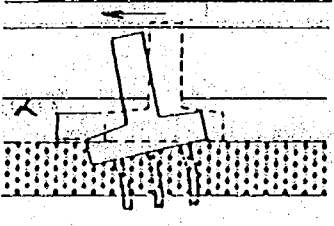
Abutment	Movable Pier	Fixed Pier
		
• Fall-down induced by large displacement of pier-top. • Foundation-collapse induced by loss of bearing of ground. • Settlement induced by loss of friction of ground.	• Fall-down induced by large displacement of pier-top. • Settlement induced by loss of friction of ground.	• Foundation-collapse induced by loss of bearing of ground. • Settlement induced by loss of friction of ground.

Table 6.15 Indices to Estimate Liquefaction-induced Damage Potential of Highway Bridge

Index		Substructure Type		
		Abutment	Movable Pier	Fixed Pier
Device Against Fall	I_{C*}	○	○	—
Supported Co. on Pier-Top	I_J	○	—	—
Foundation Type	I_F	○	○	○
Bearing Co. of Foundation	I_{FS}	○	○	○
Height of Backfill	I_{HB*}	○	—	—
Site Liquefaction Potential	I_{PL}	○	○	○
Ground Group	I_{GC}	—	○	—

Note: * Is added Index on Liquefaction-Induced Damage Potential.

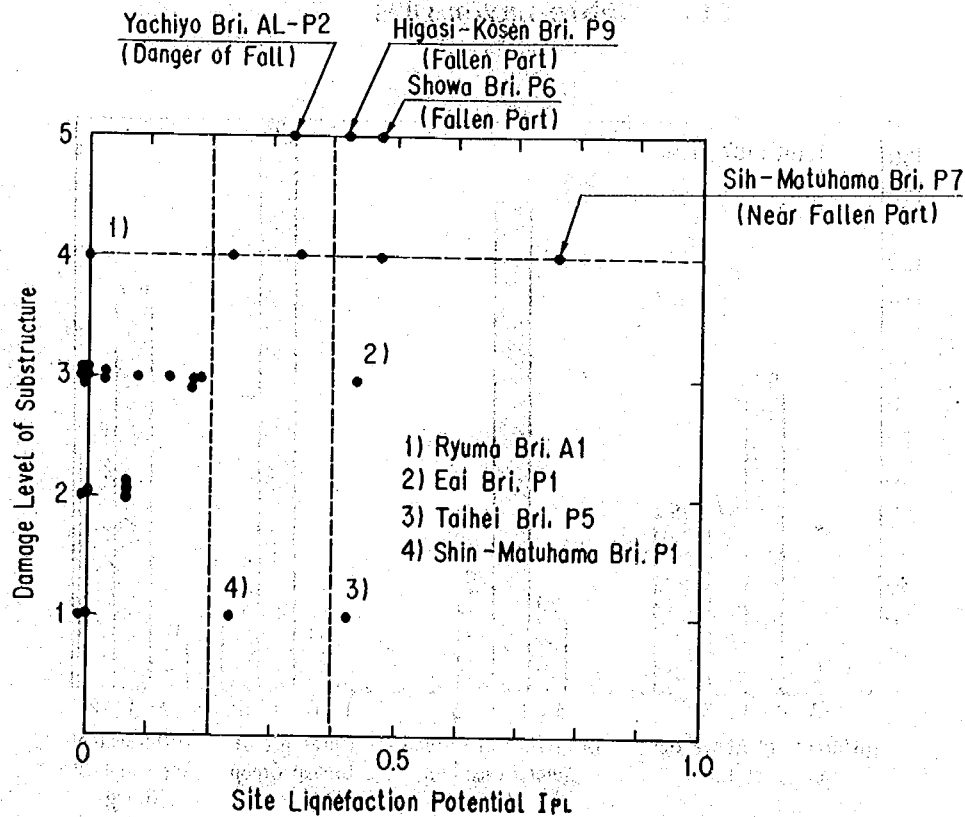


Fig. 6.1 Relation between Site Liquefaction Potential (I_{PL}) and Damage Level of Substructure

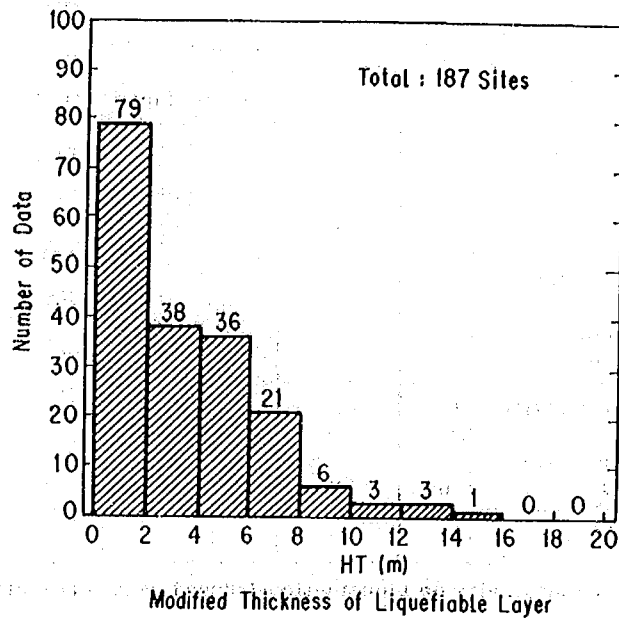


Fig. 6.2 Distribution of Modified Thickness of Liquefiable Layer (HT)

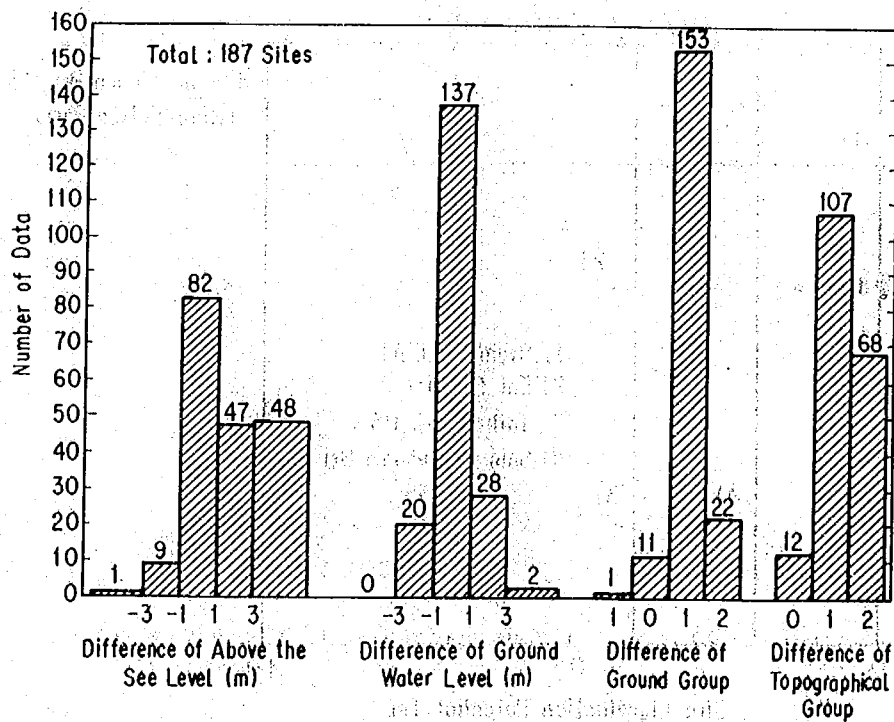
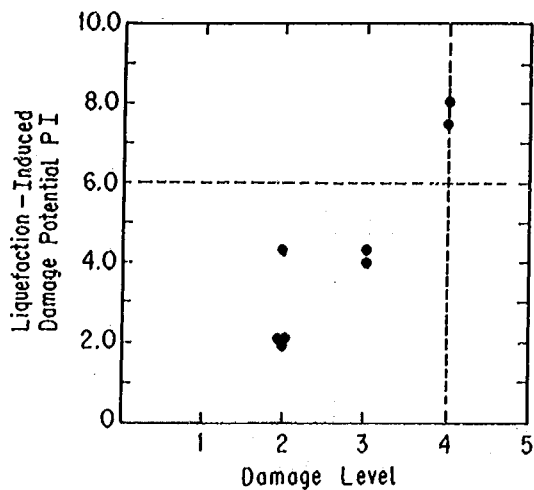
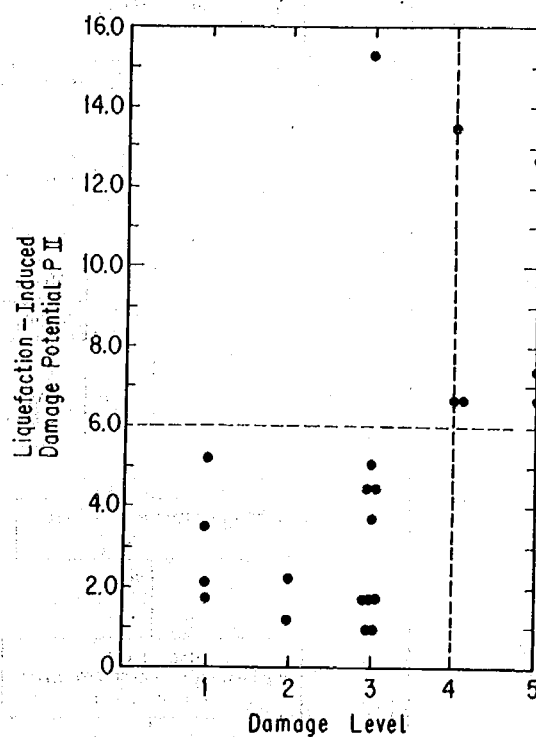


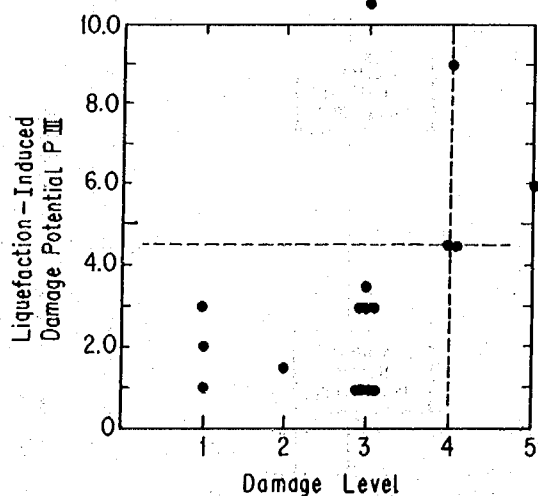
Fig. 6.3 Distribution of Categories in Each Item



(1) PI for Abutment



(2) PII for Movable Pier



(3) PIII for Fixed Pier

Fig. 6.4 Relation between Damage Level of Substructure and Liquefaction-Induced Damage Potential

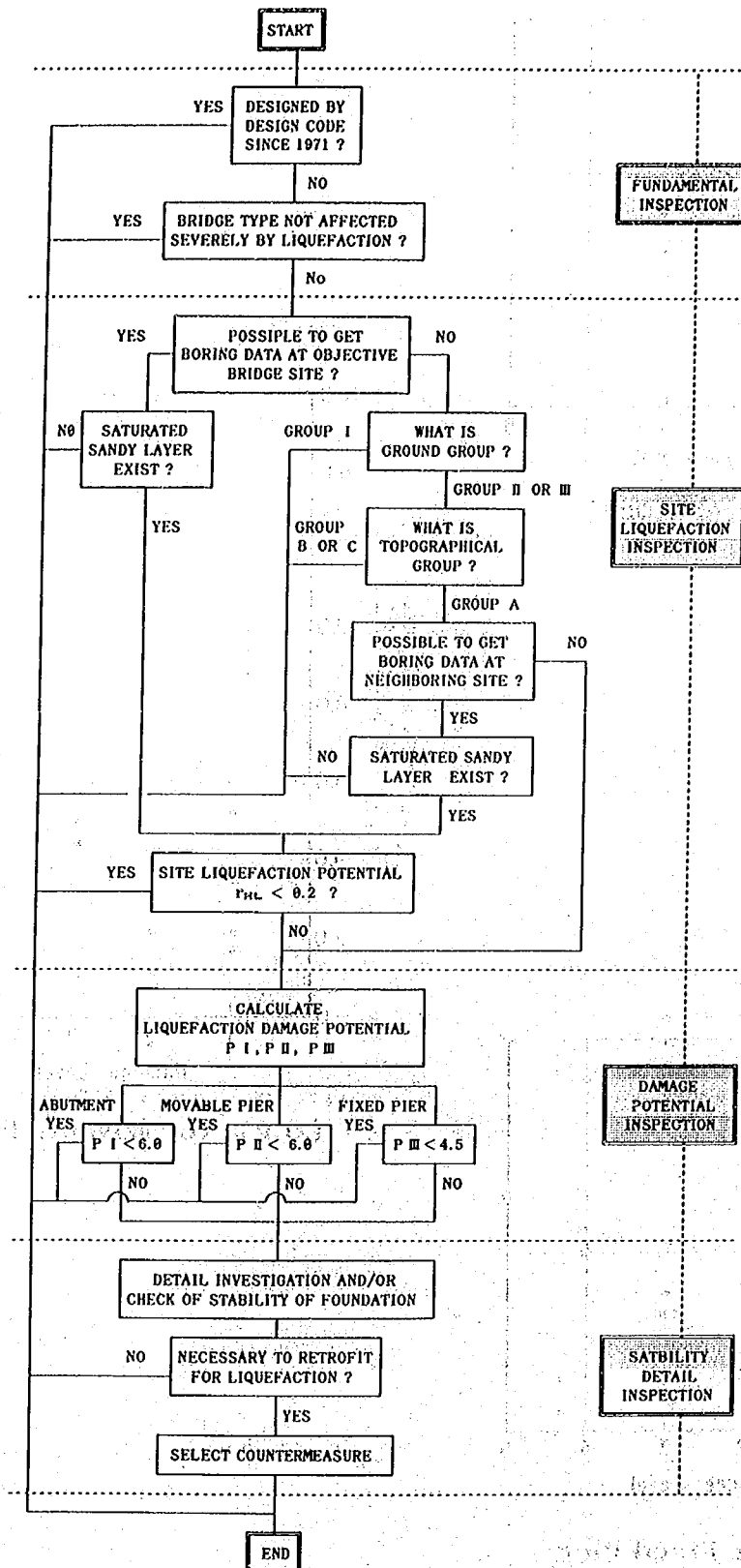


Fig. 6.5 Example of Flow Chart for Inspecting Earthquake Resistant Potential of Highway Bridge Foundation

CHAPTER 7 COUNTERMEASURES AGAINST SOIL LIQUEFACTION

7.1 PERMANENT COUNTERMEASURES

Permanent countermeasures against soil liquefaction can be classified into two groups as shown in Table 7.1: ground improvement and structure retrofit. The former controls and/or reduces the occurrence of soil liquefaction itself, and on the other hand, the latter supports the stability of structure in the liquefied layers.

7.1.1 Ground Improvement

Table 7.2 summarizes the typical ground improvement methods against soil liquefaction in Japan. These countermeasures are selected considering ground conditions, scale of countermeasures, construction works, costs and so on. Because these countermeasures generally need large scale works and huge costs, they are not common methods at present in Japan and are going to be applied more in the future.

7.1.2 Structure Retrofit

Table 7.3 summarizes the typical structure retrofit methods considering the effects of soil liquefaction. Because these countermeasures are relatively easy to apply to design of structures, they are common at present in Japan.

Photo. 7.1 shows the pile-piers of the Showa-Ohashi Bridge which were retrofited after the 1964 Niigata Earthquake. 9 piles of 1 row were changed to 18 piles of 2 rows for each pier to increase the rigidity of pile foundation against soil liquefaction. The seismic resistance of the retrofited piers are described in Chapter 5.2.1.2.

7.2 URGENT RETROFIT METHODS APPLIED JUST AFTER EARTHQUAKES

An urgent repair or retrofit of a structure just after an earthquake is necessary and effective to prevent the secondly induced damage by an after-shock and the spread of damage by a main shock. The simple and effective urgent retrofit methods should be examined and guided for one of methods to reduce earthquake disaster. Table 7.4 shows the urgent methods applied to retrofit damaged structures by soil liquefaction just after earthquakes.

It is very important and instructive to learn the experience in the past earthquakes. The practical urgent and temporary repair methods for civil engineering structures conducted in the past earthquakes will be shown as follows.

Photo. 7.2 [Ref. 7-5] shows the temporary bridge constructed neighboring the Magsaysay Bridge whose girders were fallen by soil liquefaction in the 1990 Philippines Earthquake. The temporary bridge was made by driving wood piles and putting steel truss girders and square timbers.

Photo. 7.3 [Ref. 7-2] also shows the temporary bridge constructed over a fallen simple girder of the Manicla Bridge whose abutments were tilted by soil liquefaction in the 1990 Philippines Earthquake. The temporary bridge was made by putting steel truss girders and square timbers.

Photo. 7.4 shows the Kinnoh Bridge whose one girder was fallen down in the 1978 Miyagi-Ken-Oki Earthquake in Japan. Because other parts besides fallen girder were not damaged severely, temporary girder of steel members were constructed after the earthquake only for walkers and light vehicles as shown in the right side of **Photo. 7.4**. However, another temporary bridge was needed for passing heavy vehicles, and constructed with use of steel pipe piles, H-shape steels and covering plates as shown in the left side of **Photo. 7.4**.

Photo. 7.5 shows the bailey bridge constructed temporarily in the 1984 Nagano-Ken-Seibu Earthquake in Japan. The former bridge was destroyed and flowed away by land slide of slope neighboring the bridge. The bailey bridge of truss-type and the neighboring roads made from asphalt were constructed after the earthquake.

Photo. 7.6 [Ref. 7-2] shows the temporary support of concrete girders of the Tabora Bridge in danger of falling-down because the pile-bent foundation was tilted in the 1990 Philippines Earthquake. The temporary supports were made with square timbers and wood members.

Photo. 7.7 [Ref. 7-4] shows the temporary retrofit of piers of elevated two-level viaduct at the section of Interstate 280 in San Francisco damaged in the 1989 Loma Prieta Earthquake. Because the pier was cracked and dangerous, the girder was supported by wood members tightened with steel wires.

Photo. 7.8 [Ref. 7-4] shows the temporary retrofit of piers of elevated two-level viaduct at the section of Interstate 101 in San Francisco damaged in the 1989 Loma Prieta Earthquake. The piers which were not collapsed but cracked were tightened with use of steel cables and guide wood plates.

Photo. 7.9 [Ref. 7-2] shows the temporary retrofit of a bridge approach embankment damaged in the 1990 Philippines Earthquake. Because the approach fill and back fill of an abutment were slid, the back fill was constructed temporary with wood members and stones.

Photo. 7.10 [Ref. 7-2] shows the filling of step difference between bridge surface and approach banking sunk in the 1990 Philippines Earthquake. The filling was filled with earth/gravel in this case, and asphalt was used in other case.

Photo. 7.11 [Ref. 7-2] shows the temporary repair of step difference between a bridge girder and an approach embankment applied in the 1990 Philippines Earthquake. Square timbers and wood members were used simply and temporary.

Photo. 7.12 [Ref. 7-2] shows the movable shoes slipped out and threatening to fall at the pier crest of the Sicsican Bridge damaged in the 1990 Philippines Earthquake. Although the girders of this bridge were not fallen down during the earthquake, the bridge was very dangerous as shown in this photograph and necessary to be inspected in detail just after the earthquake because the danger was not able to be found on the bridge deck.

Photo. 7.13 [Ref. 7-5] shows the prevention work for special levee to prevent water inflow from the river in Dagupan City after the 1990 Philippines Earthquake. Because the retaining wall was sunk and tilted by liquefaction, temporary cofferdam with sand bags was built up.

Photo. 7.14 also shows the prevention work of levee damaged in the 1983 Nihon-Kai-Chubu Earthquake in Japan. Because the penetration of ground leakage was feared, cut-off steel piles were driven into the ground to prevent the leakage and to lower the pore water pressure. The levee were also reinforced by banking with earth and piling with sand bags.

7.3 PRELIMINARY RETROFIT METHODS WORKING IN PAST EARTHQUAKES

Based on the experience in the past earthquakes, many lessons can be obtained as for both preliminary retrofit methods before earthquakes and permanent retrofit methods after earthquakes which are effective for reducing earthquake damage of existing structures. Several examples on the effect of preliminary retrofit countermeasures can be learned in the 1990 Philippines Earthquake.

Examples of preliminary retrofit methods which acted on effectively in the past earthquakes and can be referred against soil liquefaction are shown as follows.

Photo. 7.15 [Ref. 7-2] shows the retrofit for bridge girder and substructure in advance. One pier supporting concrete girders had been retrofited with attached piers and pier crests to prevent the girders not to fall. However the retrofited pier was not sunk during the Earthquake, the neighboring pier was sunk by liquefaction as shown in **Photo. 7.15**. The retrofit seems to act very effectively to reduce earthquake damage.

Photo. 7.16 [Ref. 7-2] shows the bridge pier of the Carmen Bridge which were reconstructed to prevent the effects of washing by a water current. Because the renewed pier was maybe constructed in the solid bearing stratum, it was not affected by the liquefied layers and this part of the bridge was not damaged during the Earthquake, compared with other part of it shown in **Photo. 3.4**.

Photo. 7.17 [Ref. 7-4] shows the elevated two-level viaduct retrofitted in advance with steel plates at the section of Interstate 408 in San Francisco which worked effectively in the 1989 Loma Prieta Earthquake.

Photo. 7.18 [Ref. 7-4] shows the girders of the viaduct retrofitted in advance by connecting two girders with wire a cable at the section of Interstate 880 in Oakland which seemed to work effectively in the 1989 Loma Prieta Earthquake.

Photo. 7.19 [Ref. 7-3] shows the foot of a special levee retrofitted in advance with stones which worked effectively against soil liquefaction in the 1990 Philippines Earthquake.

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- 7-3)Yamada T., Head of Earthquake Disaster Countermeasures Division, National Land Agency, Japan, who was a member of Expert Team of Japan Disaster Relief Team for the 1990 Philippines Earthquake.
- 7-4)Tamura K., Chief Researcher of Ground Vibration Division, Public Works Research Institute, Japan.
- 7-5)Towhata I., Assistant Professor of Tokyo University, Japan.

Table 7.1 View Points of Countermeasures Against Soil Liquefaction

- Ground Improvement : Prevent and/or decrease the occurrence of soil liquefaction by improving liquefiable ground.
- Structure Retrofit : Retrofit the structure not be damaged by soil liquefaction.

Table 7.2 Typical Classification of Countermeasures against Soil Liquefaction in Japan

Cause of Liquefaction	Principles of Countermeasures	Countermeasures
Loose Sandy Soil/Layer	Densification of Soil/Layer	• Sand Compaction Pile • Rod Compaction Pile • Vibro-Flotation • Vibro-Tamper • Others
Homogeneous Grain Size	Particle Improvement	• Replacement of Soil • Mixture Treatment
Deformation/Movement of Soil Particles	Particle/Soil/Layer Hardening	• Mixture treatment • Deep Mixing • Grouting/Hardening • Quicklime Pile
Saturated Soil/Layer	Lowering Degree of Saturation /Increase of Effective Stress	• Lowering Groundwater Level • Cutoff Wall
Shear Deformation of Soil/Layer	Restraint of Shear Deformation	• Underground Reinforced Concrete Wall • Deep Mixing Wall
Occurrence/Increase of Pore Water Pressure	Restraint/Dissipation of Pore Pore Water Pressure	• Gravel/Stone Drain • Artificial Material Drain
Flow Deformation of Liquefied Soil/Water	Shut-off/Restraint of Flow of Liquefied Soil/Water	• Rods-Tied Steel Sheet Pile • Underground Reinforced Wall • Steel/Reinforced-Concrete Pile

Table 7.3 Structure Retrofit Against Soil Liquefaction

Liquefaction-Induced Damage and Its Causes	Principle of Countermeasures	Countermeasures
Loss/Decrease of Lateral Bearing Capacity of Ground, Causing Deformation/Collapse of Piles	Increase of Rigidity of Piles in the Liquefied Ground.	• Increase of Number of Pile • Increase of Diameter of Pile
	Stop/Restraint of Deformation of Pile	• Anchoring
	Stop of Displacement of Girder	• Connecting of Girders • Seat Length(SE) • Stopper on Top of Pile
Loss/Decrease of Shear Strength of Ground, Causing Slide/Collapse of Embankment.	Increase of Shear Strength/Effective Overburden Pressure of Ground.	• Counterweight Fill • Mitigation of Slope of Bank
	Stop/Restraint of Deformation of Embankment	• Rod-Tied Steel Sheet Pile • Underground Reinforced Wall
Flow of Liquefied Soil/Water Under Structures, Causing Uplift of Under-structures.	Shut-off/Restraint of Liquefied Soil/Water.	• Underground Wall Foundation
	Stop/Restraint of Uplift	• Counterweighting • Anchoring (Pile, Cable, Stopper, etc.)

Table 7.4 Urgent Retrofit Methods Just After Earthquakes [Ref. 7-1, Compile]

Structures	Liquefaction-Induced Damage	Urgent Retrofit Methods
Bridge (Girder, Abutment, Pier, Foundation)	• Falling of girder • Tilting of abutment/plier • Settlement of abutment/plier /foundation • Collapse of foundation • Displacement of bearing support • Others	1) Temporary girders of steel members should be set instead of fallen girder. 2) Temporary bridge should be constructed neighboring collapsed bridge. 3) Berry bridge prepared in advance should be carried into and constructed. 4) Temporary girder holder/support should be constructed to prevent girder falling. 5) End of girders should be connected by welding with steel plate or cable. 6) Pier crest should be attached with stopper of steel bracket. 7) Gap occurred on bridge surface should be covered with plate. 8) Others
Approach Embankment of Bridge	• Sliding • Settlement • Crack • Step difference of abutment • Others	1) Step difference should be filled with earth, gravel and asphalt. 2) Banking with earth and gravel should be done to fill crack and recover embankment. 3) Sheet pile or wooden pile should be driven to prevent further collapse. 4) Others
Levee (Earth Bank)	• Sliding • Settlement • Crack • Others	1) Banking with earth should be done to fill cracks and recover levee-body. 2) Sand/gravel bags should be set to prevent flood and reinforce levee-body. 3) Sheet pile or wooden pile should be driven to prevent further collapse. 4) Others
Special Levee	• Sliding • Tilting • Overturning • Slipping/Opening joint • Others • Settlement • Crack	1) Sand/gravel bags should be set to prevent flood. 2) Earth should be banked to prevent flood. 3) Sheet pile or wooden pile should be driven to reinforce levee-body. 4) Others
Underground Structure (Pipe,Duct, Manhole, Tank, etc.)	• Floating • Coming-off/Shear of joint • Crack • Bending/Buckling • Blockade of pipe by liquefied sand • Others	1) Damaged parts of pipes should be dug and reinforced or changed. 2) Temporary lines should be constructed instead of damaged lines. 3) Others

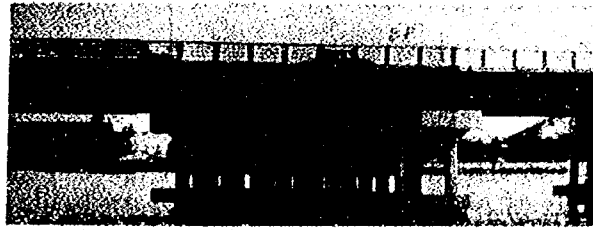


Photo. 7.1 Pile-Pier Retrofited Permanently at the Showa-Ohashi Bridge Damaged by Liquefaction in the 1964 Niigata Earthquake

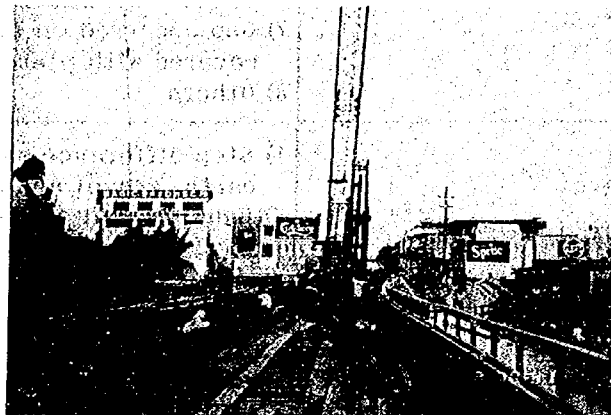


Photo. 7.2 Temporary Bridge Constructed Neighboring the Magsaysay Bridge Damaged by Liquefaction in the 1990 Philippines Earthquake [Ref. 7-5]



Photo. 7.3 Temporary Bridge Constructed over the Manicla Bridge Damaged by Liquefaction in the 1990 Philippines Earthquake [Ref. 7-2]



Photo. 7.4 Temporary Girder (Right-side) and Temporary Bridge (Left-side) Constructed at the Kinnoh Bridge Damaged in the 1978 Miyagi-Ken-Oki Earthquake, Japan

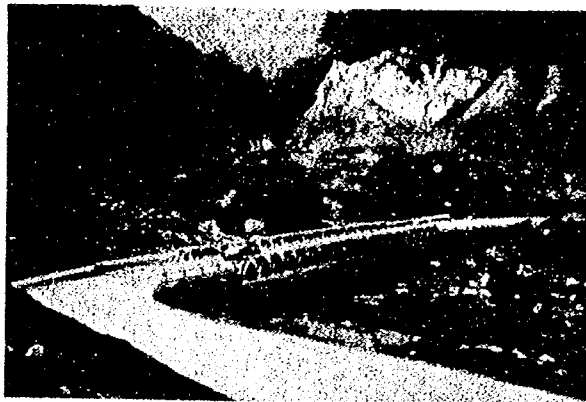


Photo. 7.5 Bailey Bridge Set Just After the 1984 Nagano-Ken-Seibu Earthquake, Japan

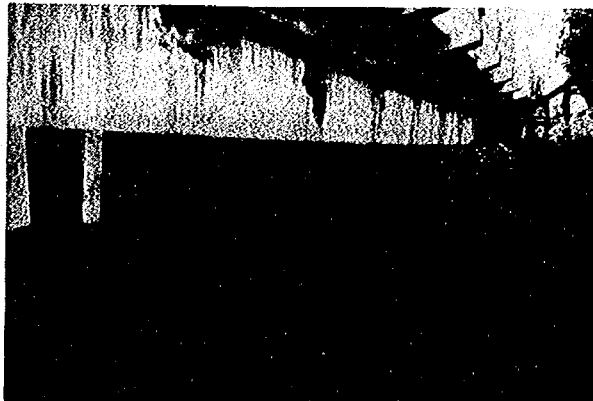


Photo 7.6 Temporary Wooden Support of Girder at the Tabora Bridge Damaged in the 1990 Philippines Earthquake [Ref. 7-2]

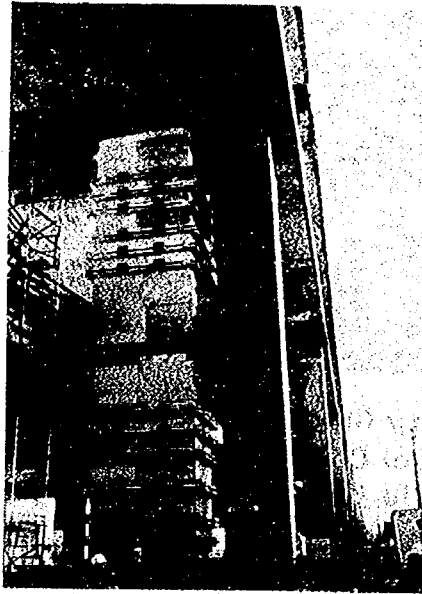


Photo 7.7 Temporary Retrofit of Piers of Elevated Two-Level Viaduct at the Section of Interstate 280 in San Francisco Damaged in the 1989 Loma Prieta Earthquake [Ref. 7-4]

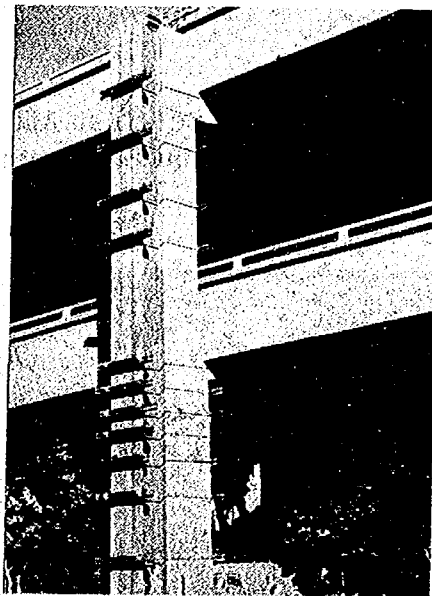


Photo. 7.8 Temporary Retrofit Support of Elevated Two-Level Viaduct at the Section of Interstate 101 in San Francisco Damaged in the 1989 Loma Prieta Earthquake [Ref. 7-4]



Photo. 7.9 Temporary Retrofit of Approach Embankment of Bridge Damaged in the 1990 Philippines Earthquake [Ref. 7-2]

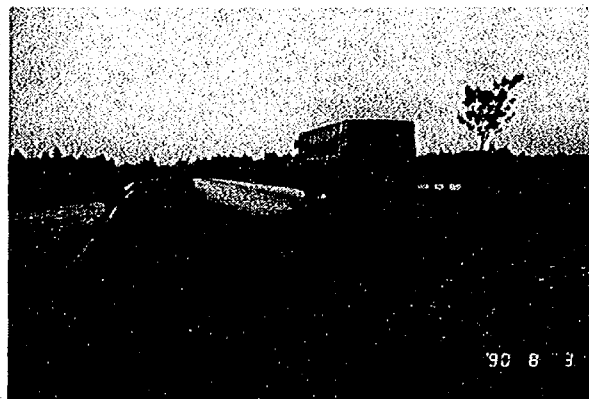


Photo. 7.10 Temporary Repair of Step Difference of Girders at the Carmen Bridge Damaged in the 1990 Philippines Earthquake [Ref. 7-2]



Photo. 7.11 Temporary Repair of Step Difference between Bridge Girder and Approach Embankment Damaged in the 1990 Philippines Earthquake [Ref. 7-2]



**Photo. 7.12 Movable Shoes Slipped Out and Threatening to Fall
at the Sicslean Bridge Damaged in the 1990
Philippines Earthquake [Ref. 7-2]**



**Photo. 7.13 Prevention Work of Levee with Sand Bags at Dugpan City
Just After the 1990 Philippines Earthquake [Ref. 7-5]**



**Photo. 7.14 Prevention Work of Levee with Sand Bags and Steel Piles
Just After the 1983 Nihon-Kai-Chubu Earthquake, Japan**

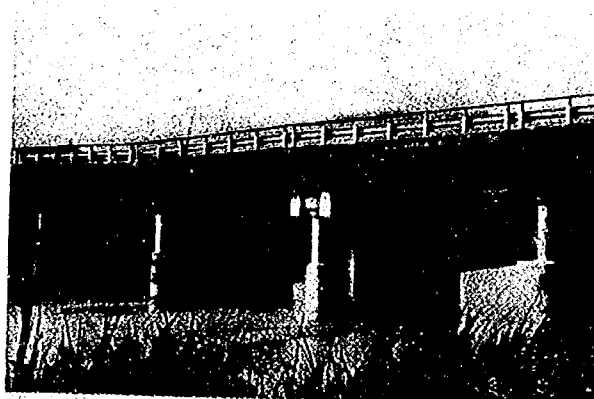


Photo. 7.15 Advance Pier-Retrofit Worked Against Liquefaction Effectively at the Aloragat Bridge in the 1990 Philippines Earthquake [Ref. 7-2]



Photo. 7.16 Bridge Pier Renewed at the Carmen Bridge to Prevent the Effects of Washing by Water Which Worked Effectively Against Liquefaction in the 1990 Philippines Earthquake [Ref. 7-2]

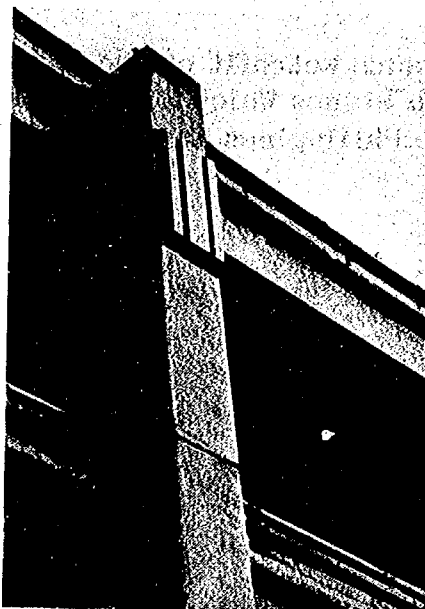


Photo. 7.17 Advance Pier-Retrofit with Steel Plates at the Section of Interstate 408 in San Francisco Which Worked Effectively in the 1989 Loma Prieta Earthquake [Ref. 7-4]



Photo. 7.18 Advance Retrofit by Connection of Girders with Wire Cable at the Section of Interstate 880 in Oakland [Ref. 7-4]

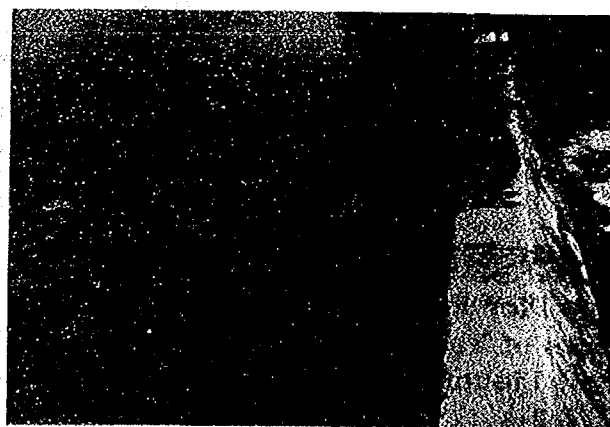


Photo. 7.19 Advance Retrofit of the Foot of Special Levee with Stones Which Worked Effectively in the 1990 Philippines Earthquake [Ref. 7-3]

CHAPTER 8 EXPERIMENTAL STUDY ON LATERAL GROUND FLOW INDUCED BY SOIL LIQUEFACTION

8.1 INTRODUCTION

The liquefaction has caused severe damage of structures during past earthquakes in Japan. Hamada et. al. (Refs. 8-1 and 8-2) pointed out that the fairly big amount of lateral flow of ground could be seen broadly at the areas in and/or around Tokyo, Fukui, Niigata and Noshiro where the liquefaction of the ground apparently took place during the Kanto Earthquake (1923, $M=7.9$), the Fukui Earthquake (1948, $M=7.3$) the Niigata Earthquake (1964, $M=7.5$) and the Nihon-Kai-Chubu Earthquake (1983, $M=7.7$), respectively.

Therefore it becomes necessary to consider two types of effect induced by soil liquefaction for the earthquake resistant design of structures; the loss of bearing capacity of liquefied ground and the lateral flow of the inclined ground induced by liquefaction. The loss of bearing capacity of liquefied layers has been considered in the "Specifications for Highway Bridges" (Ref. 8-3) in Japan.

However, the mechanism of ground flow induced by soil liquefaction, the predicting method of ground flow and the applying method of its effect to the earthquake resistant design of structures are not yet clarified at present.

Under these circumstances, as the first stage of investigations, the authors conduct a series of shaking table tests to study the mechanism of ground flow induced by soil liquefaction and to establish the procedure for estimating ground flow. This paper describes the essential factors relating to the lateral ground flow due to soil liquefaction explained from the experiments.

8.2 METHOD OF EXPERIMENTS

Shaking table tests on eight model grounds (Model-1~8) were carried out. The conditions of ground models are shown in Fig. 8.1 and Table 8.1. The two types of containers placed on a shaking table are prepared to set up model grounds: a square container for Model-1~7 is 6.0 m long, 0.8 m wide and 2.0 m high, and another semicircular one for Model-8 is 4.0 m long with a diameter and 0.4 m high. Four transparent glass windows (each is 0.8 m wide and 1.8 m high) are installed at the one side of the square container. Models-1~7 are prepared as a two-dimensional ground model to investigate the influence of the ground conditions on the lateral ground flow, and on the other hand, Model-8 is prepared as a three-dimensional ground model to know the influence of the direction of excitation on the lateral ground flow. The model grounds fundamentally consist of three layers: a lowermost nonliquefiable layer, a middle liquefiable layer and an uppermost unsaturated layer. However, Models-7 and 8 do not have the uppermost unsaturated layer and the lowermost nonliquefiable layer, respectively. The materials used for model grounds are shown in Table 8.2, i.e.,

mountain sand taken from Mt. Sengen-yama for three layers and shot of lead or gravel for the uppermost layer.

As for preparation of the model grounds, the lowermost sand layer is firstly compacted enough in the container not to liquefy during excitation, a certain amount of water is poured, and then the middle liquefiable sand layer is prepared by means of the underwater drop method. Upon completion of the lowermost and middle layers, the uppermost unsaturated layer is prepared forming the surface ground with a slope. In the Models-5, 6 and 8, shot of lead or gravel is used as the material of the unsaturated layer to give the larger overburden pressure and lesser cohesion than Mt. Sengen-yama Sand. Based on Table 8.2 as for the unit weight of unsaturated surface layer, the unit weight of gravel is not so different from that of Mt. Sengen-yama Sand, but the unit weight of shot of lead is about four times than that of Sand. The thickness of liquefiable layer (H_1), the slopes of surface (θ_s) and lower boundary of liquefiable layer (θ_b) are varied to have different conditions in order to examine the influence of these factors on the ground flow. The water level is set equal to the upper boundary of liquefiable layer, i.e., horizontal regardless of the slope of ground surface in this series of tests.

The typical conditions of each model ground are indicated as follows:

- Model-1: $\theta_s = 5\%$, $\theta_b = 0\%$ and $H_1 = 700$ mm. H_1 is larger than Model-4 ($H_1 = 350$ mm).
- Model-2: $\theta_s = 7.5\%$, $\theta_b = 0\%$ and $H_1 = 350$ mm. θ_s is larger than Model-4 ($\theta_s = 5.0\%$).
- Model-3: $\theta_s = 5\%$, $\theta_b = 5\%$ and $H_1 = 350 \sim 50$ mm. The lower boundary of liquefiable layer inclines comparing with Model-4 ($\theta_b = 0\%$).
- Model-4: $\theta_s = 5\%$, $\theta_b = 0\%$ and $H_1 = 350$ mm.
- Model-5: $\theta_s = 2.5\%$, $\theta_b = 0\%$ and $H_1 = 350$ mm. The slope of shot of lead (converted into about $\theta_s = 10\%$ as sand) is larger than Model-4 ($\theta_s = 5\%$) and Model-2 ($\theta_s = 7.5\%$).
- Model-6: $\theta_s = 0, 5\%$, $\theta_b = 0\%$ and $H_1 = 350$ mm. The ground surface is inclined and horizontal.
- Model-7: $\theta_s = 0\%$, $\theta_b = 5\%$ and $H_1 = 650 \sim 350$ mm. Only lower boundary of liquefiable layer is inclined.
- Model-8: $\theta_s = 15\%$, $\theta_b = 0\%$ and $H_1 = 250$ mm. The surface ground shapes a semi-cone with gravel.

In each test, constant sinusoidal acceleration with a frequency of 2 Hz is employed for 20 seconds (40 cycles), and several levels of maximum acceleration (60~190 gals) are applied stepwise to the shaking table as input motion. The measured maximum acceleration of the shaking table and the results of typical lateral displacement on the ground surface at Section-B are summarized in Table 8.3 for all tests.

As shown in Fig. 8.1, accelerometers, pore water pressure meters, strain gage meters and displacement meters were installed in or on the ground models to measure the time history of acceleration, pore water pressure and displacement of layers during excitation. These meters were installed at four sections, i.e.,

Sections-A,B,C and D for Models-1~7, and at two sections, i.e., Section-OA (direction of excitation) and Section-OE (right-angle direction of excitation) for Model-8. Vertical marked lines with a white-colored sand are also installed behind the transparent glass windows at intervals of 20 cm for Models-1~7 to trace or photograph the deformation of the layers in the direction of depth. Bench mark points are also set on the ground surface to measure manually or photograph the change of lateral displacement of ground surface.

8.3 RESULTS OF EXPERIMENTS

8.3.1 Distribution of Ground Flow

Photos. 8.1(a) and 8.1(b) show the deformation of model ground by the vertical marked lines through the four transparent glass windows at the Step-2 in the case of Model-4, comparing before excitation with after. It can be seen that the model ground is deformed in the direction of slope (left side of Photos.) due to excitation.

Figs. 8.2(a) and 8.2(b) show the successive change of vertical marked lines during excitation at the Step-2 in the cases of Models-4 and 6, respectively. These figures are drawn from photographs taken through the four transparent glass windows which are 4.0 m long on the side of the container. Based on these figures, it can be noted that the lateral displacement of the model ground increase gradually in the direction of slope according to the lapse of exciting time, and the unsaturated surface layer seems to deform uniformly in the direction of depth, i.e., the deformations of surface layers are not induced by the deformation of itself but by the deformation of the liquefied layer. Because the lowermost nonliquefied layer is not deformed, the displacement at the lower boundary of the middle liquefied layer is almost zero, and the lateral displacement increases upwards in the liquefied layer. The shape of deformation of liquefied layers seems to be different according to the position of the section and the lapse of exciting time. In the case of Model-6, the horizontal ground neighboring with the inclined surface ground is deformed laterally by the flow of slope, but the deformation decreases far away from the boundary of the slope.

Fig. 8.3 shows the typical distribution of lateral displacement and strain at the surface in the case of Models-4 and 6. The lateral displacement is estimated by measuring the positions of bench marked points set on the ground surface after each step of excitation, and indicate the magnitude of lateral movement in the direction of slope. The lateral strains are calculated based on the relative displacement and the distance of two bench marked points, and the tensile and compressive strains indicate the spread and shrink of the surface ground, respectively. In the case of Model-4, the middle sand layer doesn't liquefy and the ground flow doesn't occur at the Step-1, but the ground displacement increases according to the increase of input acceleration. Because the displacement is largest in the middle of slope, the tensile strains occur at the upper-side of slope, and on the other hand, the compressive strains occur at the

lower-side of one. In the case of Model-6, the displacement is largest at the middle of slope, the tensile strains occur at the slope and the compressive strains occur at the horizontal ground.

8.3.2 Relation between Acceleration, Excess Pore Water Pressure and Ground Flow During Excitation

Fig. 8.4 shows the typical relation of time histories between input motion, ground acceleration, excess pore water pressure in the liquefied layer and lateral displacement on the surface for Section-A at the Step-2 in the case of Model-6. From this result, it can be seen that the acceleration in the liquefiable layer seems to increase according to the increase of excess pore water pressure and becomes maximum just before the excess pore water pressure approaches the effective overburden pressure, and decreases to zero gradually after the sand layer liquefies almost perfectly. On the other hand, the lateral displacement of the surface seems to begin to increase just before the excess pore water pressure increase to maximum, and continue to increase according to the lapse of exciting time.

Fig. 8.5 shows the relation of time history between the lateral displacement at four points on the surface and the excess pore water pressure in liquefied layer at Section A, B, C and D. From these figures, the displacement on the ground surface also can be seen to begin to increase suddenly just after the excess pore water pressure ratio ($\Delta u/\sigma'_v$) approaches to the range of 0.8 to 1.0, and also continue to increase gradually during excitation after the complete liquefaction.

8.3.3 Behavior of Ground after Excitation

Fig. 8.4 also shows the behavior after stopping excitation. After excitation, the excess pore water pressure seems to keep the high pressure constantly by seepage pressure for about 2 minutes, and it takes more than 10 minutes for high excess pore water pressure to decrease to almost zero. On the other hand, the displacement on the ground surface increases a little after excitation, but it seems very small comparing with that during excitation. From the site observation on the liquefaction and lateral ground flow in past earthquakes, sand boils, water springs and ground movements are reported to continue for a while after the earthquake. However the water spring can be observed at the ground surface for a while after excitation in this experiments, the progress of lateral deformation of ground can't be observed. The cause of the difference on ground flow induced by liquefaction between the actual phenomena at sites during earthquakes and the experiments can not be cleared at present.

8.3.4 Relation between Input Motion and Direction of Ground Flow

Fig. 8.6(a) shows the direction and quantity of displacement of the bench mark points which are set on the radial lines (Section-0A ~ Section-0I) at the ground surface at the Step-2 in the case of Model-8, comparing the direction of ground flow with that of excitation. From this figure, the surface of the semi-cone

shaped slope and the neighboring horizontal ground seem to move almost radially, that is, in the direction of slope. Fig. 8.6(b) shows the distribution of displacement in the direction of slope for each Section, i.e., the vertical and lateral axes indicate the distance from the center of semi-circle in Fig. 8.6(a) and the position of Sections in the direction of circumference, respectively, and the shadowed area means the quantity of displacement in the direction of slope. As shown in Fig. 8.6(b), the displacement of surface ground in the direction of slope seems almost uniformly, except for the Sections neighboring the container which may be affected by the friction of the side wall.

Figs. 8.7(a) and 8.7(b) show the relation of time histories between excess pore water pressure (Δu) in the liquefied layer and the lateral displacement (D) at the middle of surface of semi-cone, comparing the meters set at the Section-OA in the direction of excitation with the one set at the Section-OE in the direction of right-angle of excitation, respectively. Based on Figs. 8.7(a) and 8.7(b), the dynamic behavior of the pore water pressure seems similar in each layer, and the displacement at the surface seems to begin to increase at almost the same time just before the perfect liquefaction (see Fig. 8.4). Because the displacement meter in Fig. 8.7(a) is set in the direction of excitation and the one in Fig. 8.7(b) is set in the direction of right-angle of excitation, the displacement at the Section-OA increases vibrationally in the direction of excitation and the one at the Section-OE increases gradually in the direction of right-angle of excitation. This means that the surface ground at the Section-OE is vibrated in the direction of excitation, but the ground flow occurs in the direction of slope. It can be noted that the excitation is not the cause for ground flow but that for soil liquefaction.

Judging from these characteristics concerning the ground flow, it seems that whichever the direction of excitation is, the ground flow due to soil liquefaction is induced towards the direction of slope. Because the direction of slope means that of the initial shear stress enforced by sloped overburden load, the ground seems to flow in the direction of the initial shear stress in the liquefiable layer before excitation.

8.3.5 Effects by Thickness of Liquefiable Layer, Surface Slope and Lower Boundary Slope

Fig. 8.8 compares the distribution of the lateral displacement and the lateral strains (see Fig. 8.3) for Models-1, 3, 4 and 7, because their test conditions are similar and suitable for investigating the influence by the thickness of liquefiable layer, slopes of surface and/or lower boundary of liquefiable layer on ground flow at the surface. Comparing the results for Model-1 ($\theta_s = 5\%$, $\theta_b = 0\%$, $H_1 = 700$ mm) with that for Model-4 ($\theta_s = 5\%$, $\theta_b = 0\%$, $H_1 = 350$ mm), the lateral displacement and strains for Model-1 are larger than that for Model-4, and so it can be seen that the thickness of the liquefiable layer relates to the ground flow, and the displacement of ground seems to increase according to the increase of the thickness of liquefiable layer. Comparing the results for Model-3 ($\theta_s = 5\%$, $\theta_b = 5\%$, $H_1 = 350$ mm) with that for Model-4, the lateral displacement and strains

for Model-3 are larger than that for Model-4, and so it can be noted that the ground with both surface slope and lower boundary slope seems to be induced more than that with only surface slope. Comparing the results for Model-7 ($\theta_s = 0\%$, $\theta_b = 5\%$, $H_1 = 650 \sim 350$ mm) with that for Model-1 and Model-4, the lateral displacement and strains for Model-7 are very small, and so it can be indicated that the effect of lower boundary slope by itself seems very small for the ground flow. However, comparing the results for Model-7 with that for Model-3 and Model-4, the lower boundary slope seems to become effective for the ground flow when the surface inclines.

As mentioned above, the influence of the slope of the lower boundary of liquefiable layer to the ground flow seems very small. However, when both ground surface and lower boundary of liquefiable layer incline as the case of Model-3, the slope of the lower boundary of liquefiable layer is likely to increase the ground flow.

8.3.6 Effects by Slope of Ground Surface

Table 8.3 shows the typical lateral displacement measured on the ground surface at Section-B for all Models and Steps of excitation. In this Table, the ground conditions for Model-2 ($\theta_s = 7.5\%$, $\theta_b = 0\%$, $H_1 = 350$ mm), Model-4 ($\theta_s = 5\%$, $\theta_b = 0\%$, $H_1 = 350$ mm) and Model-5 ($\theta_s = 2.5\%$, $\theta_b = 0\%$, $H_1 = 350$ mm) are similar except for the slope of ground surface. However, the shot of lead is used as the materials for Model-5, and so the slope of surface ground is converted to be about 10% as sand.

Comparing the results for Models-2 and 4, it can be seen that the displacement of the ground surface seems to increase according to the increase of the surface slope, but the results for Model-5 doesn't indicate the similar tendency with Models-2 and 4. As for the cause of the difference, it can be estimated that the geotextile sheet which is set only for Model-5 between the shot of lead and the liquefiable sand layer not to mix shot of lead with sand layer, and the cables connecting the meters laid in the ground may affect the ground flow.

8.3.7 Estimation of Ground Flow

From the above mentioned test results, it is deduced that the slope of surface (θ_s :%) relates to the ground flow, and the ground flow seems to increase according to the increase of the surface slope (see Table 3). And the thickness of liquefied layer (H :cm) where the excess pore water pressure ratio ($\Delta u/\sigma'_v$) is almost equal to 1.0 relates to the ground flow (see Table 8.3, Fig. 8.8 and Fig. 8.5). Further, the duration time of excitation (T :sec) while the ratio ($\Delta u/\sigma'_v$) is almost 1.0 also relates to the ground flow (see Fig. 8.4).

Therefore, these three parameters, i.e., θ_s , H and T can be considered as the main factors relating to the quantity of ground flow (D :cm) measured at the surface. Fig. 9 shows the relation between the parameter $D/(T \cdot H)$ [$\times 10^{-4}$ /sec] and the slope of surface (θ_s :%) at the Sections A, B, C and D at the Steps-1 and 2 for all Models excepting Model-8. The slope of surface is calculated as the average slope

around the measured point on the surface at each Section. Based on this figure, it can be indicated that the parameter $D/(T \cdot H)$ has a tendency to increase according to the increase of the slope θ_s .

8.4 CONCLUSION AND FUTURE RESEARCH

Based on the experimental study on the lateral ground flow due to soil liquefaction using a shaking table, the following can be concluded:

- (1) The pore water pressure in the liquefiable layer increases firstly during excitation, preceding the lateral ground deformation, and the lateral displacement of the inclined surface begins to increase remarkably when the excess pore water pressure ratio ($\Delta u/\sigma'_{v0}$) in the underlying layer exceeds about the range of 0.8 to 1.0, and continues to increase according to the lapse of exciting time.
- (2) The lateral deformation occurs only in the liquefied layers, and the unsaturated surface layer is displaced by the deformation of underlying liquefied layers.
- (3) The ground seems to be deformed by shear force during excitation and the deformation mode of ground in the direction of depth differs according to the position and the lapse of exciting time.
- (4) Just after stopping the excitation, the excess pore water pressure in the liquefied layer keeps the high pressure by seepage for a while and begins to decrease gradually. On the other hand, the lateral ground flow almost stops.
- (5) The lateral ground flow is affected severely by the slope of ground surface, and the slope of the lower boundary of the liquefiable layer doesn't affect so much by itself, but affects when the ground surface also inclines.
- (6) The direction of excitation has no significant influence to the direction of the lateral flow of ground, and the lateral ground deformation is related to the direction of the slope of surface, i.e., the direction of initial shear stress in the liquefiable layer before excitation. The excitation doesn't relate with the direct cause for ground flow but that for soil liquefaction.
- (7) The lateral ground flow at the ground surface (D) in this series of experiments can be expressed by the slope of surface (θ_s), the thickness of the almost perfectly liquefied layer (H) and the duration time of excitation after almost complete liquefaction (T). The relation between the parameter $D/(T \cdot H)$ and θ_s can be indicated as Fig. 8.9.

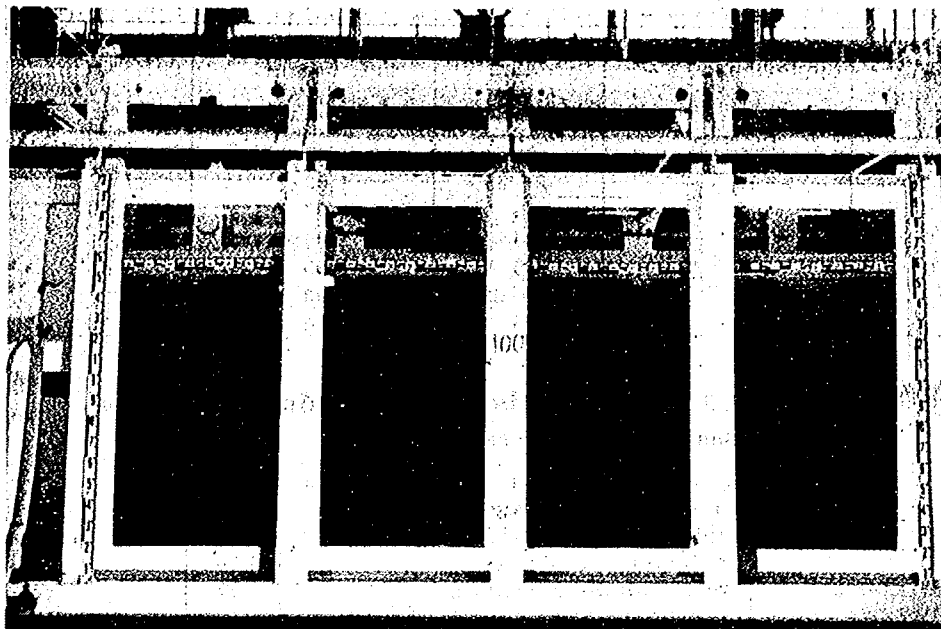
As mentioned above, the fundamental characteristics on ground flow due to soil liquefaction are investigated experimentally and the principal factors which cause the ground flow are clarified qualitatively. However, it should be noted that the results are derived from the limited conditions for shaking table tests and further investigation should be executed from now on. The subjects in the future can be considered as follows:

- (1) Establishment of the test conditions considering the actual site conditions during earthquakes, for example, the scale of a container, the relative density

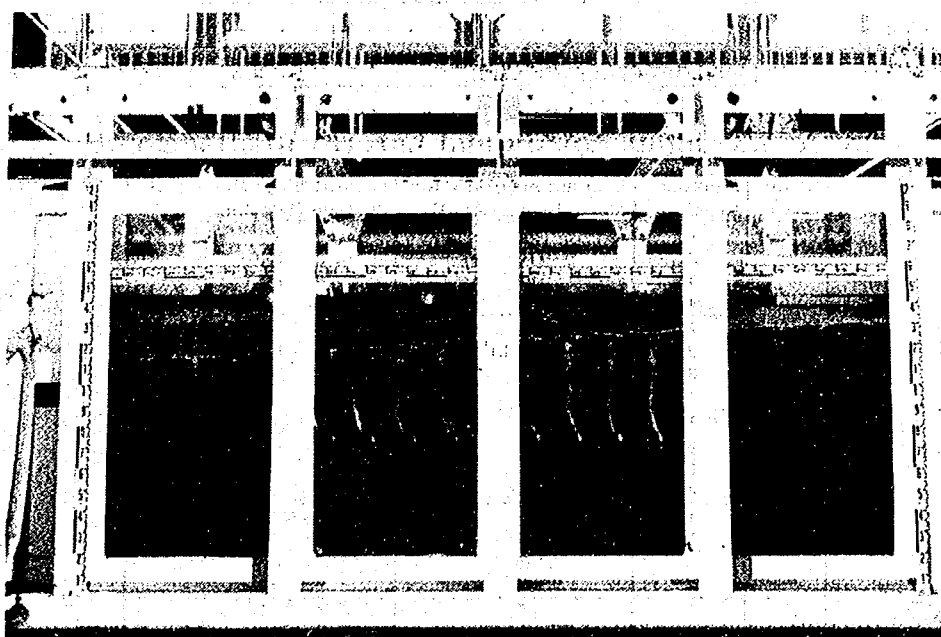
- of liquefiable layer, the ground water level.
- (2) Investigation of the cause of the difference between the experimental characteristics and the actual phenomena observed during earthquakes, for example, the progress of the ground flow after excitation.
 - (3) Investigation of the influence of ground flow on the structures, for example, pile foundations of bridges.
 - (4) Application of the effect of ground flow to the specifications for earthquake resistance design.

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(a) Before Excitation



(b) After Excitation

Photo. 8.1 Ground Flow due to Liquefaction

Table 8.1 Characteristics of Ground Models

Model No.	Slope of Surface $\theta_s(\%)$	Slope of Lower Boundary of Liquefiable Layer $\theta_b(\%)$	Thickness of Liquefiable Layer $H_L(\text{cm})$	Liquefiable Layer	Unsaturated Surface Layer	
				Unit Weight $\gamma_{s1}(\text{tf/m}^3)$	Unit Weight $\gamma_{s2}(\text{tf/m}^3)$	Materials
1	5	0	70	1.93	1.49	sand
2	7.5	0	35	1.88	1.37	sand
3	5	0	5~35	1.93	1.57	sand
4	5	0	35	1.90	1.49	sand
5	2.5	0	35	2.06	5.71	shot of lead
6	0~5	0	35	2.00	1.48	gravel
7	0	5	35~65	1.93		
8	15	0	25	1.84	1.36	gravel

Table 8.2 Characteristics of Ground Materials

1) Sand (Mt. Sengen-yama Sand)

Specific Gravity of Soil Particle G_s	2.655
Maximum Void Ratio e_{max}	0.976
Minimum Void Ratio e_{min}	0.596
Maximum Grain Size (mm)	4.76
Mean Grain Size D_{50} (mm)	0.27
Coefficient of Uniformity U_c	1.37

2) Shot of Lead

Specific Gravity of Soil Particle G_s	11.34
Mean Grain Size D_{50} (mm)	2.00

3) Gravel

Maximum Grain Size (mm)	9.52
-------------------------	------

Table 8.3 Input Motion and Lateral Displacement

Model NO.	Shaking Step NO.	Maximum Acceleration at Table $a_{max}(\text{gal})$	Lateral Displacement at Surface (Section B)(mm)
1	1	80	61
	2	110	169
	3	160	118
	4	220	56
2	1	65	28
	2	110	144
	3	155	165
	4	105	62
3	1	65	68
	2	110	156
	3	90	87
	4	85	35
4	1	65	8
	2	95	72
	3	140	142
	4	190	44
5	1	80	1
	2	101	4
	3	152	210
	4	154	69
6	1	60	1
	2	104	101
	3	153	106
	4	154	54
7	1	64	-2
	2	103	17
	3	154	11
	4	151	6
8	1	51	14
	2	94	11
	3	95	5

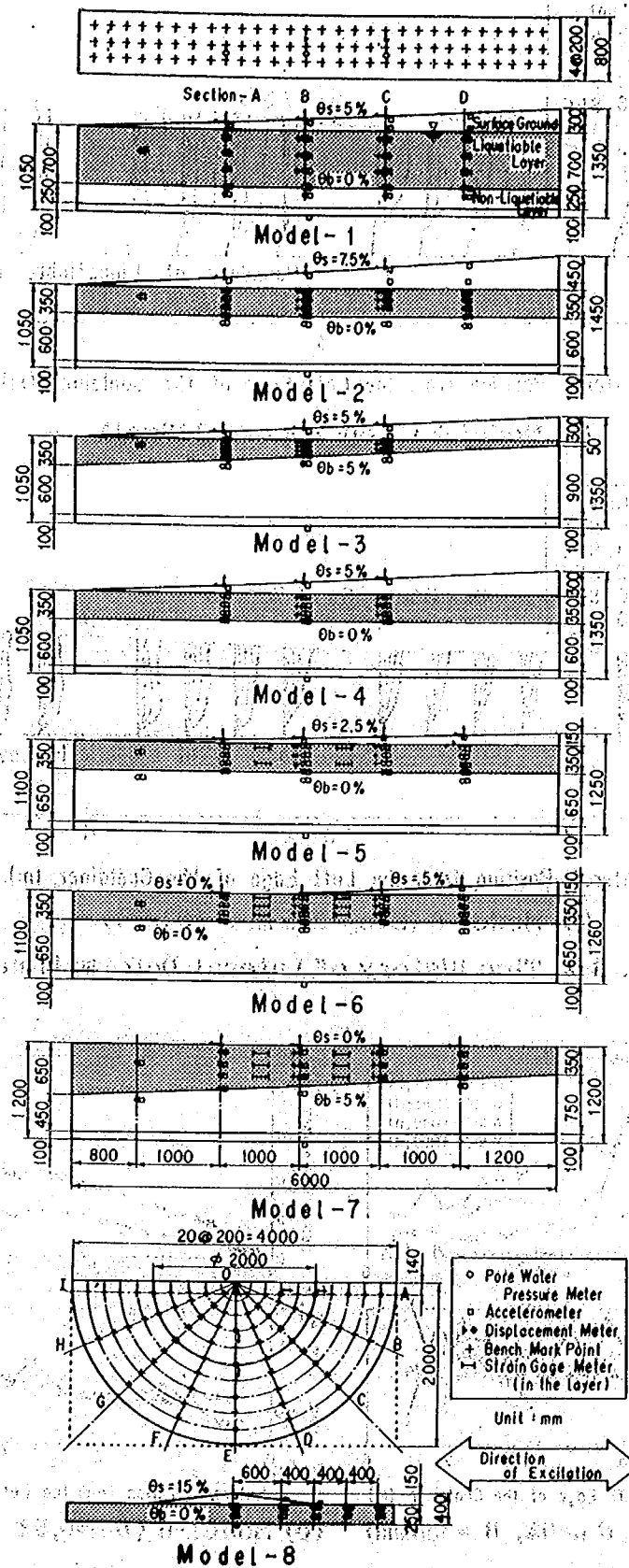
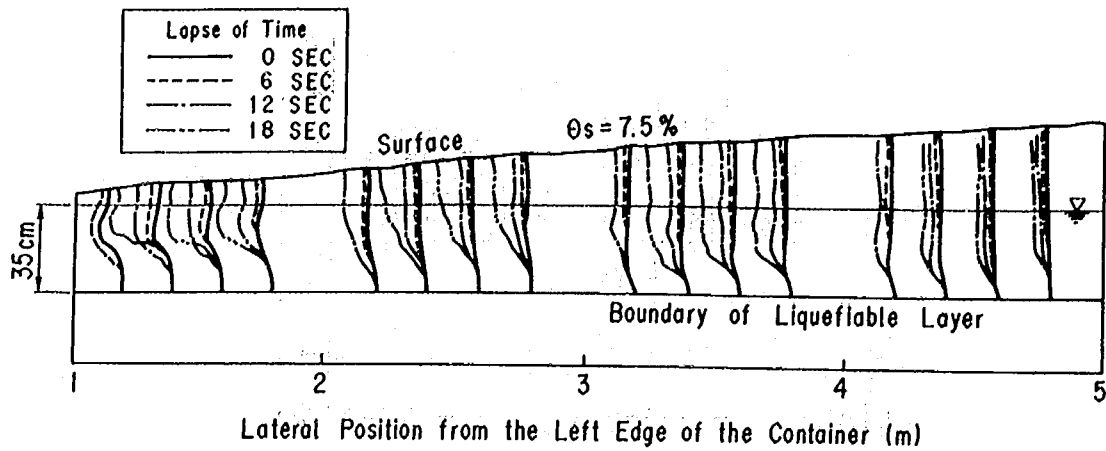
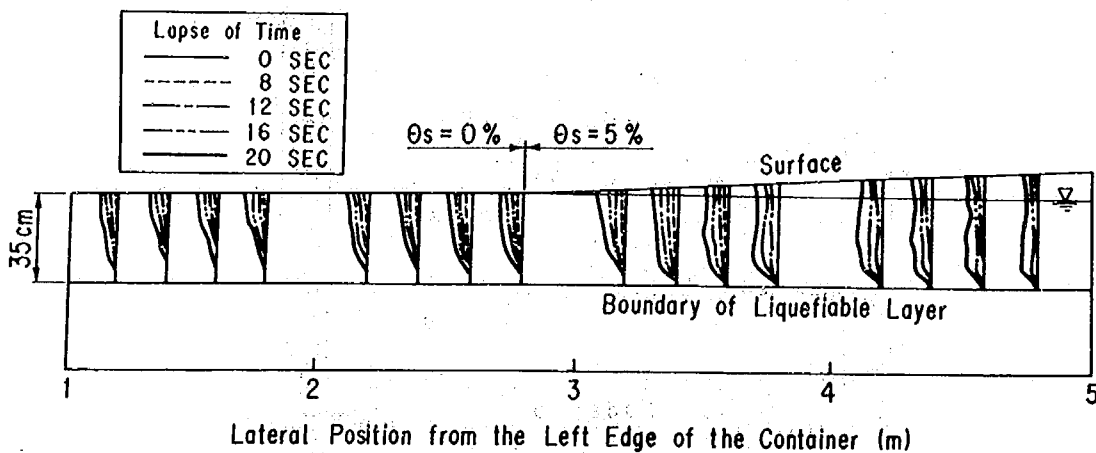


Fig. 8.1 Ground Models and Measurements

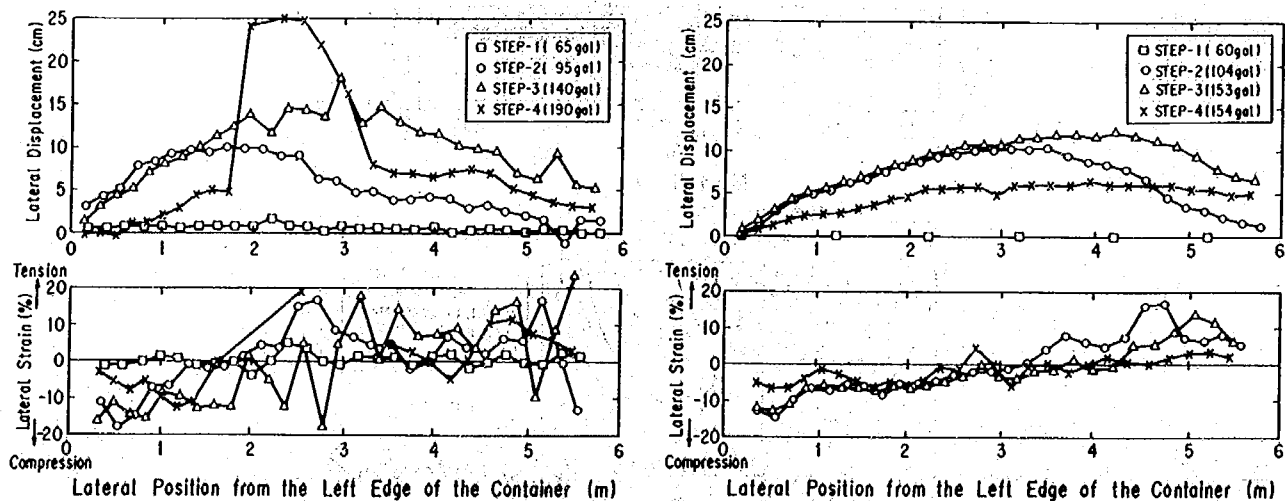


(a) Model-2 (Step-2: $\alpha_{max}=110gal$)



(b) Model-6 (Step-2: $\alpha_{max}=104gal$)

Fig. 8.2 Time History of Lateral Deformation



(a) Model-4 ($\theta_s=5\%$, $\theta_b=0\%$, $H_1=350mm$)

(b) Model-6 ($\theta_s=0.5\%$, $\theta_b=0\%$, $H_1=350mm$)

Fig. 8.3 Lateral Deformation of Ground Surface Measured by Bench Mark Points

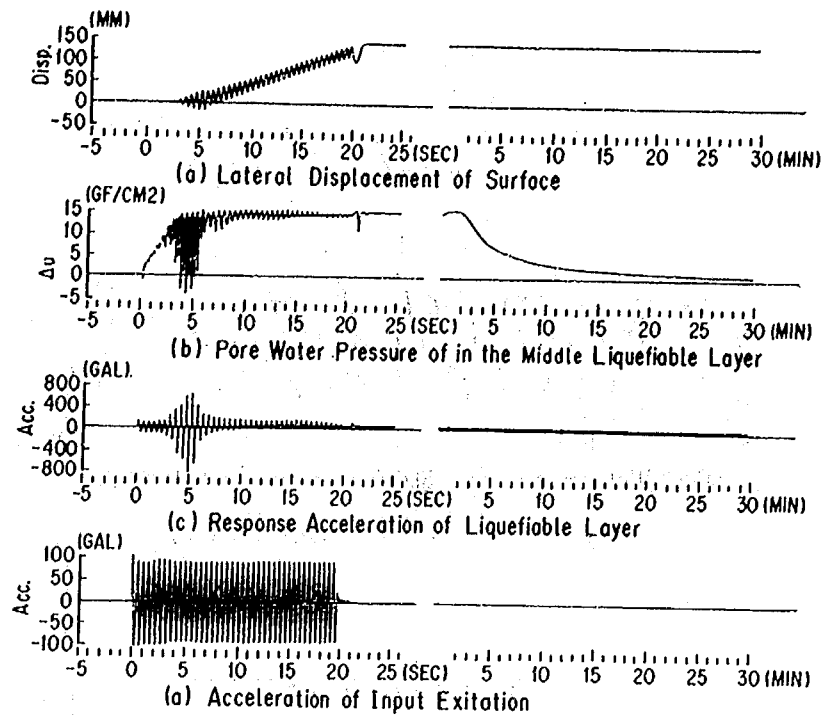


Fig. 8.4 Time History of Behavior of Ground (Model-6, Step-2, Section-A)

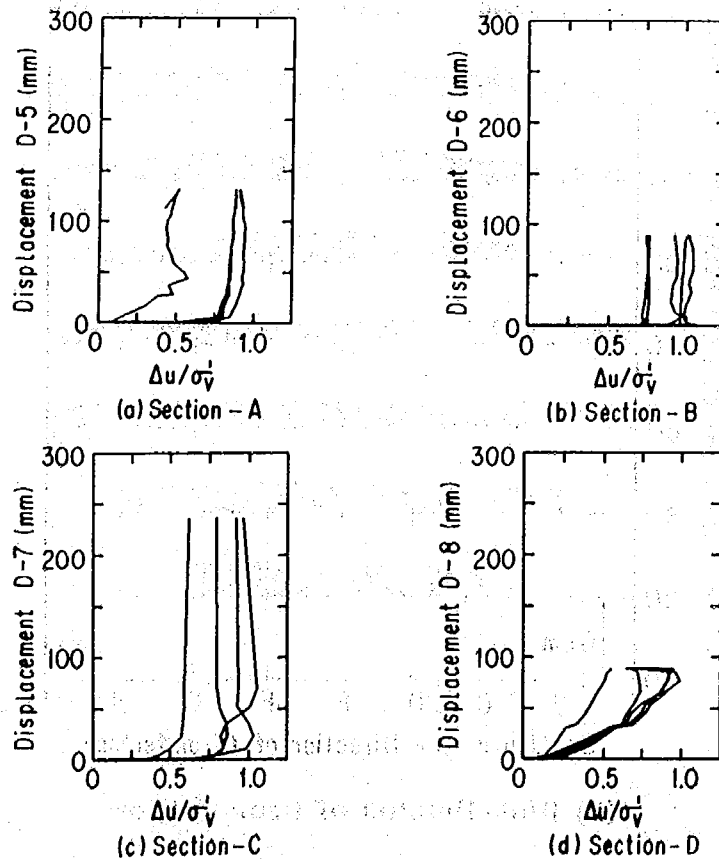
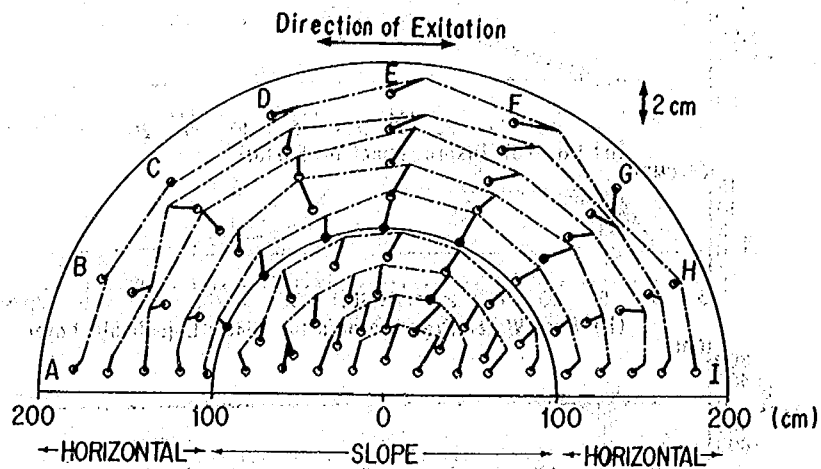
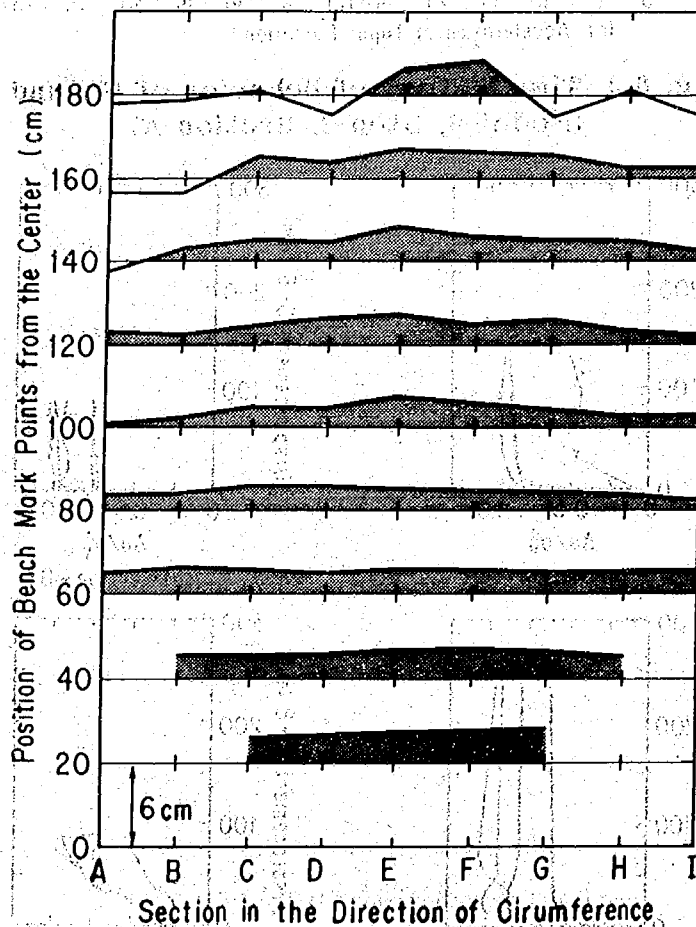


Fig. 8.5 Pore Water Pressure Ratio and Displacement of Surface (Model-6, Step-2)

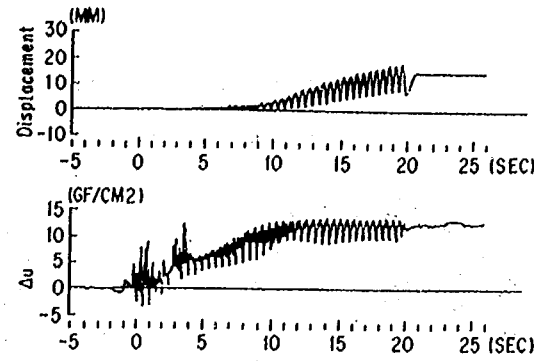


(a) Direction of Ground Flow

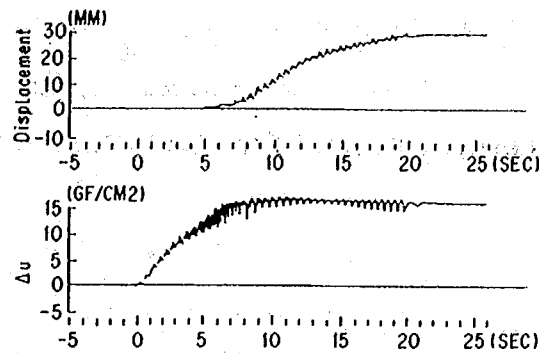


(b) Distribution of Ground Flow

Fig. 8.6 Directions of Ground Flow and Input Motion (Model-8, Step-2)



(a) Direction of Excitation



(b) Direction of Right-angle of Excitation

Fig. 8.7 Time History of Pore Water Pressure and Displacement (Model-8, Step-1)

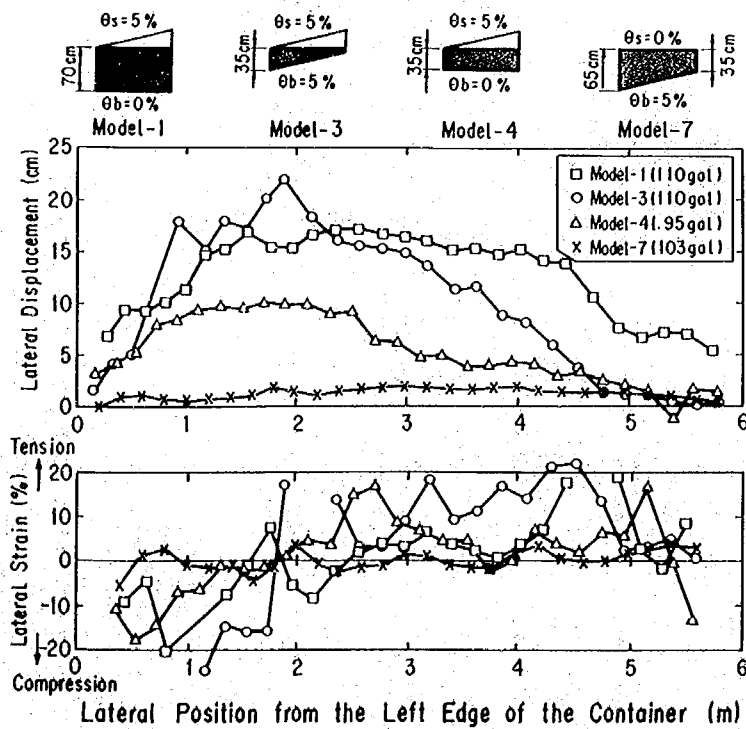


Fig. 8.8 Influence by Thickness and Slopes

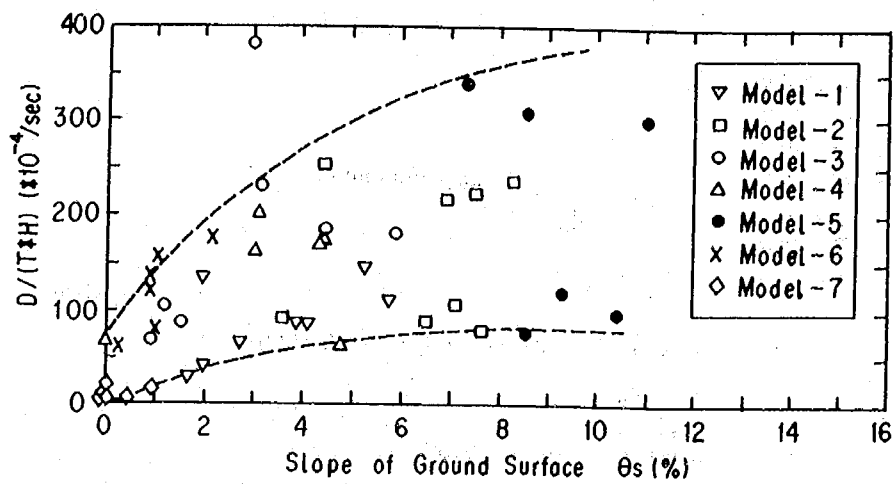


Fig. 8.9 Lateral Ground Flow and Slope of Surface

CHAPTER 9 ZONING OF LIQUEFACTION POTENTIAL AND ITS VISUALIZATION

9.1 INTRODUCTION

Because damaging earthquakes affect structures over a wide area when they hit urban regions, it generally takes a considerable amount of time and labor to understand the damaged state of facilities immediately after the occurrence of large earthquakes. Correct information on damage should be gathered quickly to take action as soon as possible for restoration just after an earthquake. In addition, the rapid collection of useful information from various information transferred in a confused state is necessary. The type and cause of the damage need to be estimated and to be given to the people who work on damage restoration. For this purpose, a visual display of the distribution of the seismic intensity in an area, even though it might be an estimated one, and the result of assessed ground hazard is thought to be a very effective aid to rapid restoration work just after an earthquake.

Although the underground structures for lifeline systems such as water pipes, gas pipes, and so forth may be severely damaged during earthquakes, it is difficult to directly find the damage because the structures are buried in the ground. It is necessary and effective to estimate the damage of underground structures indirectly with use of ground hazard zoning because the structures are easily affected by the dynamic behavior of ground. Because the causes of damage to underground structures are seismic ground motion, soil liquefaction and so on, the zonation of ground motion and ground hazard is effective for evaluating the hazard of lifeline systems extensively.

In this chapter, the seismic ground hazard information system, which is being developed to estimate the ground hazard and damage of underground structures quickly and correctly with use of a computer system, is introduced. The system is being developed as a pilot-system to investigate the usefulness of visualization methods in sieving correct information effectively out of various information which is reported just after the event. This is done by using a digitized map which has various information such as ground and structure condition, ground hazard, structural damage and location of structures.

9.2 SEISMIC INFORMATION RELATED TO GROUND HAZARD

9.2.1 Flow of Ground Hazard Information Just After Earthquakes

The purposes of establishing the information system of seismic ground motion and hazard are twofold (see Table. 9.1):

1. To make possible the urgent and rapid inspection and restoration of earthquake hazards just after an earthquake. Then, action can be taken

based on the actual or rationally estimated distribution of seismic ground motion, and actual or rationally estimated ground hazard and actual damage of structures.

2. To be utilized for strengthening the structures against future earthquakes based on the rationally estimated distribution of seismic ground motion, ground hazard and damage of structures.

Although there are several kinds of indices of earthquake ground motion and hazard types, the ones of interest in this study are those related to the earthquake-induced damage of structures, especially underground structures. They are as follows:

- 1) Seismic intensity of ground motion.
- 2) Soil liquefaction.
- 3) Lateral ground spread induced by soil liquefaction.

The flow of seismic information of ground motion, ground hazard and structure damage during the earthquakes can be summarized as shown in Fig. 9.1. The topics related to the seismic information in this study can be classified as follows:

- 1) Fundamental data of earthquake and induced-ground motion and hazard.
- 2) Estimated data of seismic ground motion and hazard.
- 3) Quantitative estimation methods of seismic ground motion and hazard.
- 4) Data base needed for seismic zoning of ground hazard.
- 5) Actual data of earthquake-induced damage of (underground) structures.
- 6) Quantitative estimation methods of earthquake-induced damage of (underground) structures.
- 7) Data base needed for estimating damage of structures by ground hazard.

In this chapter, the research results on the above subjects of 1) to 4) will mainly be shown.

9.2.2 Fundamental Data of Earthquake and Induced-Ground Motion and Hazard

The information related to seismic ground motion and hazard which can be recorded and collected just after an earthquake can be considered as follows.

- 1) Epicentral location of actual or estimated earthquake.
- 2) Magnitude of actual or estimated earthquake.
- 3) Seismic ground motion recorded by seismographs.
- 4) Ground hazard found at the site.

Because the location and Richter-Scale Magnitude of an earthquake are reported in Japan by the Japan Meteorological Agency (JMA) just after an earthquake, these fundamental data of an earthquake can be obtained easily through mass media. When the earthquake observation system is improved as an on-line system, the seismic ground motion records can be obtained without delay just after an earthquake and used for actual seismic zoning of ground hazard. Furthermore,

the ground hazard which is estimated by indirect information can be corrected with use of the actual hazard which is observed at the site and reported after the earthquake.

9.2.3 Estimated Data of Seismic Ground Motion and Hazard

Although the fundamental data of an earthquake and the recorded ground motion are very useful, they cannot necessarily yield enough information on the ground motion at every site of interest. Thus it is necessary to supplement the fundamental information with analyzed data for urgent and rapid inspection and restoration of earthquake-induced damage of structures. The information related to ground hazard which is needed and useful to supplement the fundamental information for restoration just after an earthquake can be classified as follows:

- 1) Seismic ground motion estimated at a site of interest in an area and its distribution.
- 2) Soil liquefaction potential at a site of interest in an area and its distribution.
- 3) Potential magnitude of lateral ground spread by soil liquefaction and its distribution.

9.2.4 Quantitative Estimation Methods of Seismic Ground Motion and Hazard

Reasonable methods for analyzing the original data from of an earthquake and the ground condition have been investigated and established. The following methods are necessary for estimating the secondary information quantitatively.

- 1) Method for estimating the attenuation of seismic ground motion.
- 2) Method for estimating the soil liquefaction potential.
- 3) Method for estimating the lateral ground flow induced by soil liquefaction.

9.2.5 Data Base Needed for Seismic Ground Hazard Zoning

It is necessary to establish a data base to indicate and analyze the original information before an earthquake occurs. The following data base should be established for seismic ground hazard zoning:

- 1) Mapping data which draw the fundamental information such as the coastline, boundaries of cities, locations of highways and rivers and so on.
- 2) Ground group information to estimate the seismic ground motion and soil liquefaction potential.
- 3) Soil profile, geology and topographical ground condition to estimate the soil liquefaction potential.
- 4) Contours of elevation to estimate the lateral ground spread by soil liquefaction.

9.3 METHODS TO ESTIMATE QUANTITATIVE GROUND HAZARD

9.3.1 Attenuation of Seismic Ground Motion

The maximum acceleration of the ground surface can be estimated with the use of equation (9-1) according to the ground group at the site of interest [Ref. 9-1] .

$$\alpha_{\max} = \begin{bmatrix} 87.4 \times 10^{0.216M} \\ 232.5 \times 10^{0.313M} \\ 403.8 \times 10^{0.266M} \end{bmatrix} \times (\Delta + 30)^{-1.218} \quad \begin{matrix} \text{(Ground Group I)} \\ \text{(Ground Group II)} \\ \text{(Ground Group III)} \end{matrix} \quad (9-1)$$

where $\alpha_{\max}$ is maximum acceleration of the ground surface (gal), M is the Richter-Scale Magnitude of an earthquake and Δ is the epicentral distance (km). The Richter-Scale Magnitude is reported by JMA just after an earthquake and the epicentral distance can be calculated from the location of an earthquake. The Ground Groups I, II and III as shown in Table 9.2 are rock or diluvial ground, alluvial ground, and soft alluvial ground or reclaimed ground, respectively [Ref. 9-2] .

The Seismic Intensity on the JMA scale is also estimated based on the maximum acceleration of the ground surface calculated with the use of equation (9-1) as follows.

$\alpha_{\max}$ (gal)	~ 0.8 ~ 2.5 ~ 8.0 ~ 25 ~ 80 ~ 250 ~ 400 ~							
Seismic Intensity	0	1	2	3	4	5	6	7

9.3.2 Soil Liquefaction Potential

The sandy layers at a site of interest with the following conditions can be considered to be vulnerable to liquefaction during earthquakes, and their liquefaction potential should be examined [Ref. 9-3] :

- 1) The layer is a saturated alluvial sandy layer.
- 2) The layer is within 20 m from the ground surface.
- 3) The ground water level is within 10 m from the ground surface.
- 4) The D_{50} -value is between 0.02 mm and 2.0 mm for grain size accumulation curve.

As for sandy soil layers judged to be vulnerable to liquefaction, liquefaction potential can be examined based on the liquefaction resistance factor F_L defined by the following equation (9-2) [Refs. 9-2 and 9-3] . Soil layers with a liquefaction resistance factor F_L less than 1.0 are judged to liquefy during the earthquake considered.

$$F_L = \frac{R}{L} \quad (9-2)$$

where F_L is the liquefaction resistance factor, R is the resistance of soil

elements to dynamic loads and L is the dynamic load applied to the soil elements. R and L are estimated simply with use of the following equations:

$$R = R_1 + R_2 + R_3 \quad (9-3)$$

$$R_1 = 0.0882 \sqrt{N/(\sigma_v' + 0.7)} \quad (9-4)$$

$$R_2 = \begin{cases} 0.19 & (0.02\text{mm} \leq D_{50} \leq 0.05\text{mm}) \\ 0.225 \log_{10}(0.35/D_{50}) & (0.05\text{mm} < D_{50} \leq 0.6\text{mm}) \\ -0.05 & (0.6\text{mm} < D_{50} \leq 2.0\text{mm}) \end{cases} \quad (9-5)$$

$$R_3 = \begin{cases} 0.0 & (0\% \leq F_o \leq 40\%) \\ 0.004F_o - 0.16 & (40\% < F_o \leq 100\%) \end{cases} \quad (9-6)$$

$$L = r_d \cdot k_s \cdot (\sigma_v / \sigma_v') \quad (9-7)$$

$$r_d = 1.0 - 0.015z \quad (9-8)$$

$$k_s = C_z \cdot C_a \cdot C_i \cdot k_{s0} \quad (9-9)$$

where R_1 is the resistance based on relative density, R_2 is the resistance based on D_{50} -values, R_3 is the resistance based on fine contents, N is the blow count by standard penetration test, D_{50} is the mean grain size (mm), F_o is the fines content (%), z is the depth from the actual ground surface (m), k_s is the seismic coefficient for evaluating liquefaction, C_z is the seismic zone factor, C_a is the ground condition factor, C_i is the importance factor, k_{s0} is the standard seismic coefficient ($= 0.15$), σ_v is the total overburden (kgf/cm^2) and σ_v' is the effective overburden pressure under static conditions (kgf/cm^2).

In the system in this study, the seismic coefficient (k_s) is assumed to be estimated by the equation (9-10) instead of the equation (9-9).

$$k_s = \alpha_{max}/g \quad (9-10)$$

where α_{max} is the maximum acceleration of the ground surface estimated by the equation (9-1) and g is the gravity acceleration ($= 980\text{cm/sec}^2$).

In this study, the liquefaction potential at the site of interest is defined with two indices: the total thickness of liquefiable layers with the F_L -Value less than 1.0 (H_{AS}) and the topographical ground condition. Based on experience in past earthquakes, the topographical ground conditions can be used as one of the indices to roughly estimate the liquefaction potential. The topographical ground conditions are classified into three groups as shown in Table 9.3 [Ref. 9-4]. The liquefaction potential of a site of interest can be classified into one of three groups as shown in Table 9.4.

Group-A : Site/Zone with high liquefaction potential.

Group-B : Site/Zone with low liquefaction potential.

Group-C : Site/Zone with negligible liquefaction potential.

9.3.3 Method to Estimate Lateral Spread of Ground by Soil Liquefaction

The lateral ground spread (flow/movement) induced by soil liquefaction has been investigated by the Public Works Research Institute [Refs. 9-5 and 9-6] and is being included as one ground hazard in the seismic information system. Because the simplified method to estimate lateral ground spread quantitatively is now under investigation, the equation (9-11), which has been proposed empirically in past earthquakes, has been tentatively plugged into the pilot-system in this study [Ref. 9-7] .

$$D = 0.75 \sqrt{H^3} \sqrt{\theta} \quad (9-11)$$

where D is the maximum lateral displacement of the ground surface (m), H is the thickness of liquefied layers (m) and θ is the maximum slope of the ground surface or of the lower boundary of the liquefied layer (%).

9.4 VISUALIZATION OF GROUND HAZARD

9.4.1 Purpose of Visualization

Information about the seismic ground motion, ground hazard and the damage of structures should be summarized simply for the urgent and rapid inspection and restoration of works after an earthquake disaster. Because graphically displayed information with the use of a computer system is very understandable, the visualization of actual and analysed information with a computer system is very helpful for people who work on earthquake disaster remediation. In this section, the visual information system, which has been developed as a pilot-system to investigate the methods to display information visually over the past few years at the Public Works Research Institute, is introduced using one case study. The above-mentioned methods to estimate the seismic ground motion and ground hazard are applied in the pilot-system.

9.4.2 Case Study Model Area for Visual Information System

The case study model area for the visual information pilot-system is the Southern Area of Kanto-District around Tokyo-Bay in Japan. The map of the model area can be displayed on the computer-monitor with three kinds of scale according to their purpose as follows (each map is overlaid with grid lines which form rectangular cells):

Map A : This map indicates the wide sized area (of 158 km $\times$ 130 km) with 14 $\times$ 14 = 196 cells each of which is same size as the entire Map B.

Map B : This map indicates the middle sized area (of 11.3 km $\times$ 9.25 km) with 20 $\times$ 20 = 400 cells each of which is same size as the entire Map C.

Map C : This map indicates the narrow sized area of 566 m $\times$ 466 m scale.

It is possible to select a cell of interest in Map A and B, and to display it as Map B and C respectively.

9.4.3 Example of Display in the Visual Information System

Typical examples of the display in the visual information pilot-system are shown in Photos. 9.1 to 9.9.

The hardware for the visual information pilot-system is composed of a memory apparatus, a display apparatus, a keyboard with a mouse, a printer and a mainframe computer with a map displayed as shown in Photo. 9.1. The system can be worked with a key or a mouse by selecting the menus shown on the display or inputting the necessary data.

The earthquake conditions, such as the epicentral location (longitude and latitude) and Magnitude (M) (which are reported by JMA just after an earthquake or estimated as a future earthquake), are necessary to be input as shown in Photo. 9.2. Furthermore, the seismic intensity reported at typical cities where JMA offices are located can also be input.

The maximum acceleration and the converted seismic intensity on JMA scale can be estimated with the use of the earthquake conditions above-mentioned by equation (9-1) and can be displayed on Maps A, B and C. Photo. 9.3 shows the display for Map A. Based on this visual information on the distribution of seismic ground motion, it is possible to estimate the location and size of heavily damaged areas for rapid and appropriate action just after an earthquake. Furthermore, it is possible to easily understand which areas have a high potential for ground hazard problems in future earthquakes included in the earthquake disaster prevention plan.

The data base of ground groups and topographical ground conditions above-mentioned in the equation (9-1) and Table 9.3, respectively, should be established in advance in order to be able to estimate the seismic ground motion by the equation (9-1) and the total liquefaction potential defined in Table 9.3. Photo. 9.4 shows the zoning of ground groups for each cell in Map B. The ground group has been determined with the use of typical boring data for each cell based on Seismic Design Specifications [Ref. 9-3]. Photo. 9.5 also shows the zoning of topographical ground conditions for each cell in Map B. The topographical ground condition has been determined by the average condition of the topography at each cell.

The liquefaction potential defined in Table 9.4 can be displayed with three kinds of colored grids added to the objective cell. As shown in Photo. 9.6, the cell of Group-A with high liquefaction potential is displayed with a red grid and the one of Group-B with low liquefaction potential is with a yellow grid. Furthermore, the Group-C with negligible liquefaction potential is with a blue grid. The liquefaction potential of colorless grids cannot be estimated for lack of soil profile data. Based on these visual information, it is possible to estimate the area with high liquefaction potential easily and widely for both rapid and appropriate action just after an earthquake and for future plans against earthquake disaster.

The lateral ground spread estimated by the equation (9-11) can be displayed with red-colored arrows for each cell as shown in Photo. 9.7. The magnitude of ground spread can be shown with the length of the arrow and the direction of it is shown with the direction of the arrow. The thickness of the liquefied layer (H) at each cell in the equation (9-11) is determined from the results of calculations of liquefaction resistance factor (F_L) defined by the equation (9-2) with typical boring data. On the other hand, the maximum slope of the ground surface (θ) at each cell is defined as the maximum slope between the cell of interest and the eight cells around the cell of interest which is calculated from the average height at the center of each cell.

The detailed location of the structures of interest can be displayed in Map C as shown in Photo. 9.8. The epicentral distance, ground group and total liquefaction potential of Map C can also be displayed. When the data base of structural conditions for each structure is introduced in advance, the conditions of structures can be extracted from the data base and the damage conditions of structures reported after earthquakes can also be input in Map C. The damaged structures can be visualized to a larger scale in Map B as shown in Photo. 9.9. This scale is very useful for comprehending the distribution of damaged structures.

9.5 FUTURE VIEW

1. Because lifeline systems are located over a large region under the ground, seismic zoning is very effective for both seismic inspection and restoration action just after earthquakes and for disaster prevention programs for future earthquakes.
2. Although seismic zoning of ground motions and ground hazards has been described in this paper, further investigation is needed to establish a simple but rigorous method to quantitatively estimate the magnitude of liquefaction induced lateral spread of the ground in the future.
3. Because various damage of underground structures such as lifeline facilities is caused by various ground hazards, it is necessary to investigate the relation between these ground hazards and the damage level of lifeline facilities in order to create useful seismic zoning.
4. The visual information system with the use of a computer system is very useful for people who work on earthquake disaster prevention so that they can understand the seismic information and data rapidly and adequately.
5. For establishing the useful zoning of seismic ground hazard and induced-damage of structures, fundamental data on the ground and structures is needed. Estimates of the ground hazard and damage of structures should be established in advance.

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Table 9.1 Application of Seismic Ground Motion and Hazard during Earthquakes

1) Urgent and Rapid Inspection and Restoration Just After Earthquakes.

- Based on —
- (1) Actual Distribution of Seismic Ground Motion.
 - (2) Estimated Distribution of Seismic Ground Motion and Hazard.
 - (3) Estimated Damage of Structures, Especially Underground Structures Induced by Ground Motion and Hazard.

2) Planning of Earthquake Disaster Prevention in Future Earthquakes.

- Based on —
- (1) Estimated Distribution of Seismic Ground Motion and Hazard.
 - (2) Estimated Damage of Structures, Especially Underground Structures Induced by Ground Motion and Hazard.

Table 9.2 Ground Groups [Ref. 9-2]

Ground Group	Characteristic of Ground	T_G^* (sec)
I	• Rock, Diluvial Ground (Firm)	$T_G < 0.2$
II	• Alluvial Ground (Medium-stiff)	$0.2 \leq T_G < 0.6$
III	• Soft and Thick Alluvial Ground (Soft)	$0.6 \leq T_G$

* :Characteristic Value of Ground [See Equation (5-22)]

Table 9.3 Relation between Topographical Ground Condition and Liquefaction Potential
[see Table 6-7]

Liquefaction Potential	Geographical Conditions
(A) Highly Possible to be Liquefied	Existing Channel, Existing River Bed, Old Channel, Old River Bed, Lowland between Sand Hills, Reclamation of Old Water Surface (Reclaimed Area of Seashore, Lake-shore, Paddy Field, etc.)
(B) Possible to be Liquefied	Inter Wet Land, Natural Bank, Sand Hill, Sandbar, Sandy Beach, Other than the above (A) and below (B)
(C) Not Easily to be Liquefied	Plateau, Hill, Fand Land

Table 9.4 Simplified Method for Estimating Liquefaction Potential

Ground Group	Thickness of Liquefiable Layer (m)		
	$0 \leq H_{As} \leq 2$	$2 < H_{As} \leq 5$	$5 < H_{As} \leq 20$
I	B	A	A
II	C	B	A
III	C	C	B

(Note) A : Site/Zone with High Liquefaction Potential

B : Site/Zone with Liquefaction Potential

C : Site/Zone with Negligible Liquefaction Potential

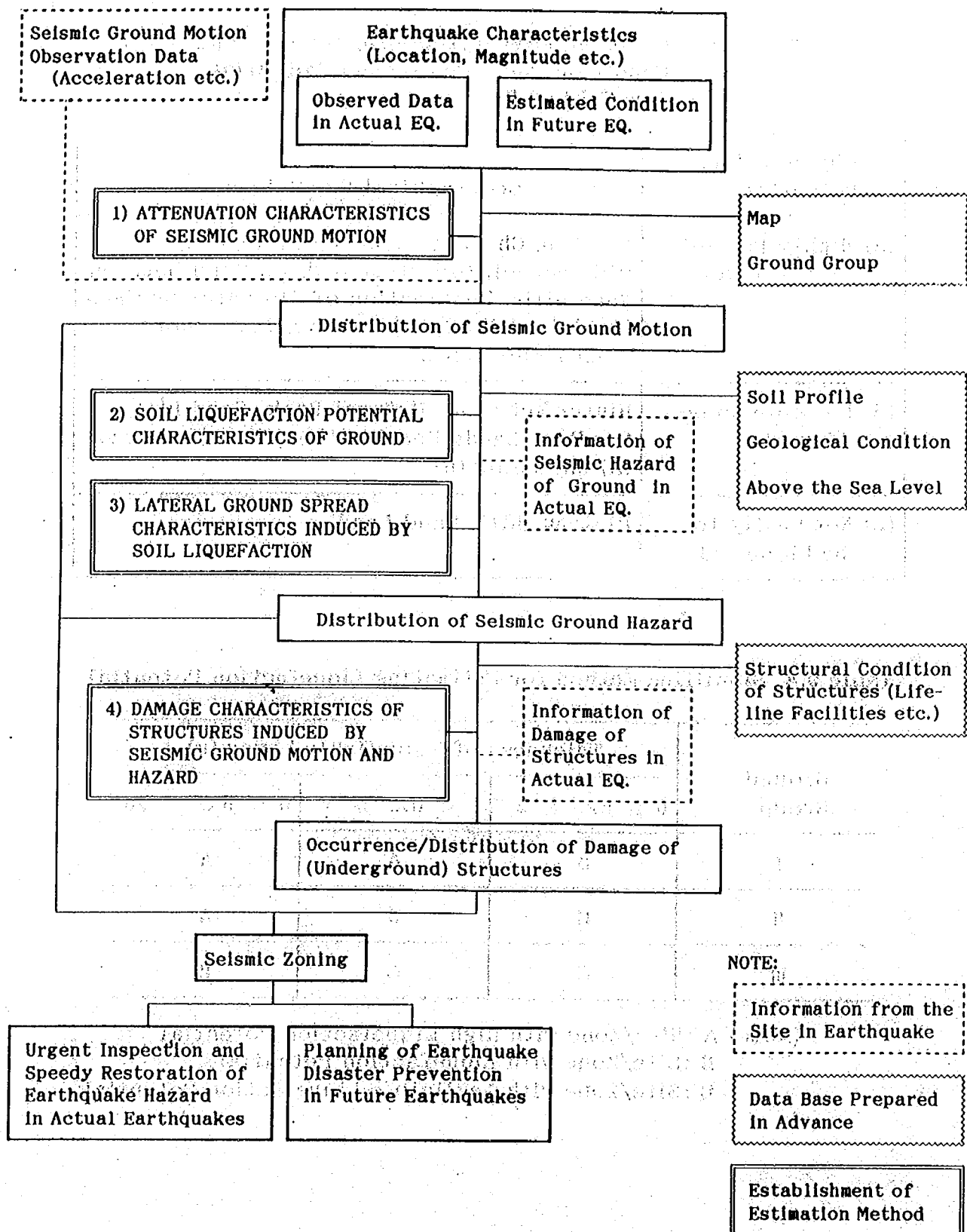


Fig. 9.1 Flow of Information Related to Seismic Ground Motion/Hazard and Induced-damage of Structures

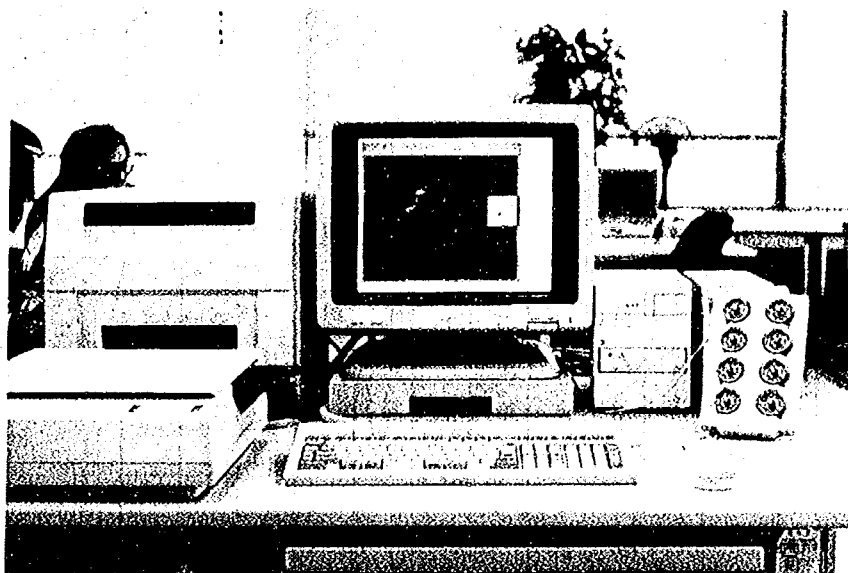


Photo. 9.1 External Appearance of Visual Information Pilot-system

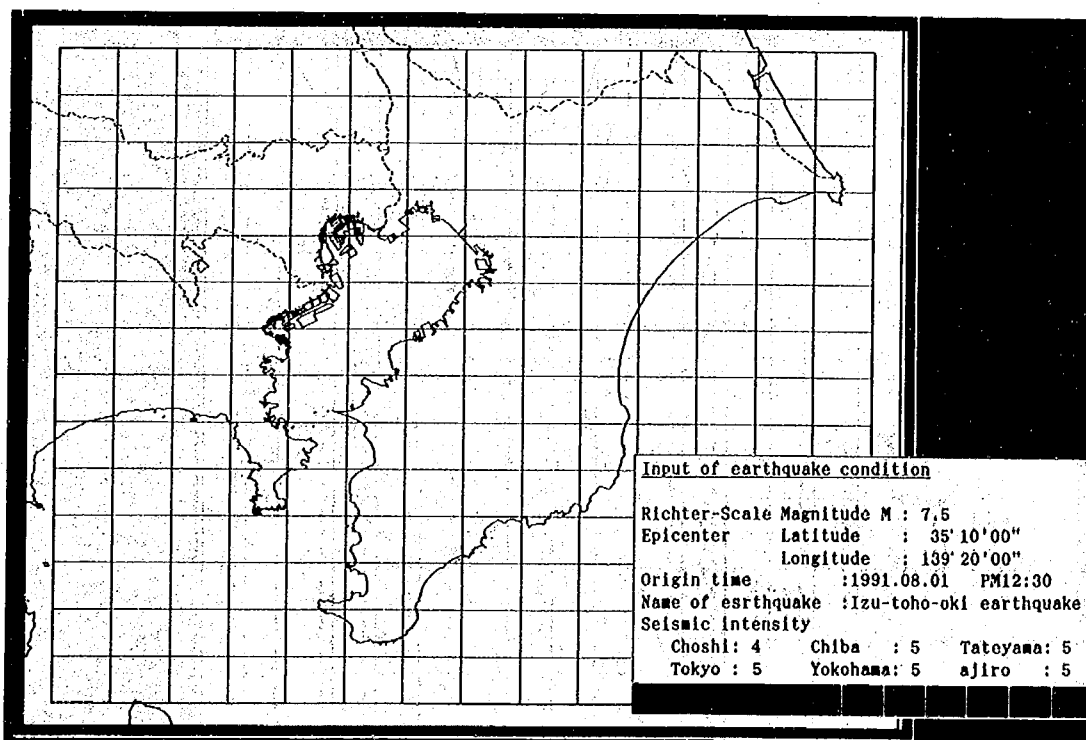


Photo. 9.2 Input of Earthquake Location (Longitude and Latitude), Richter-Scale Magnitude and Seismic Intensity Reported by JMA

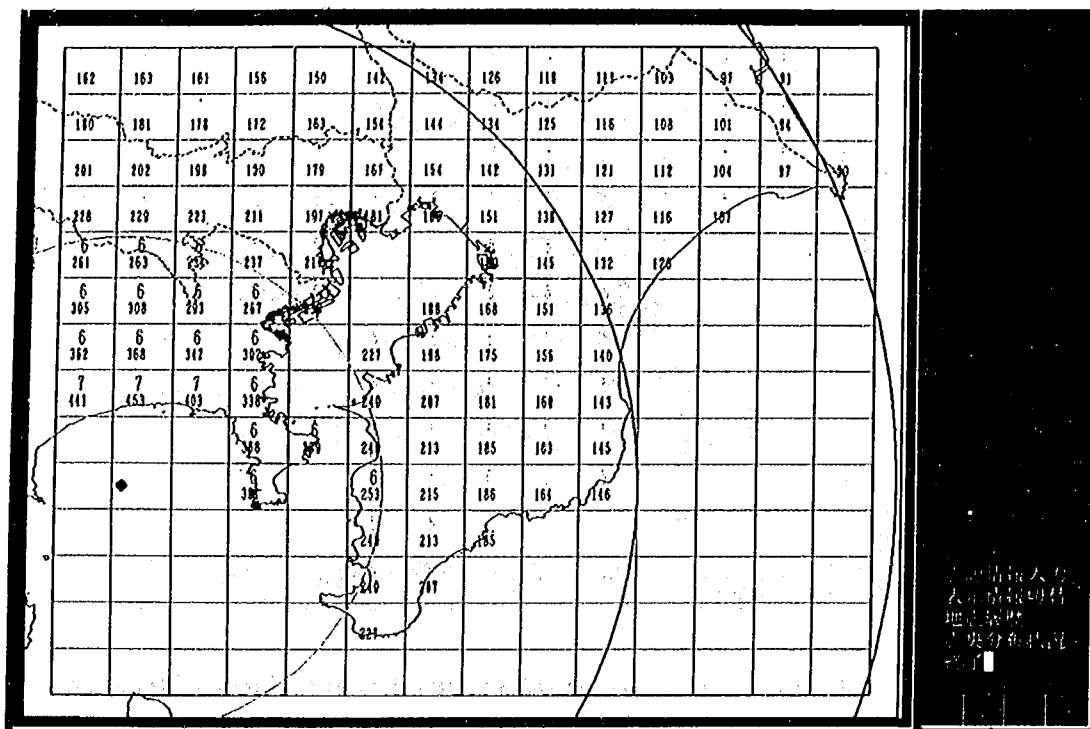


Photo. 9.3 Display of Maximum Ground Surface Acceleration and Seismic Intensity Estimated in Map A (Southern-Area of Kanto-District : 158km×130km, 196 cells)

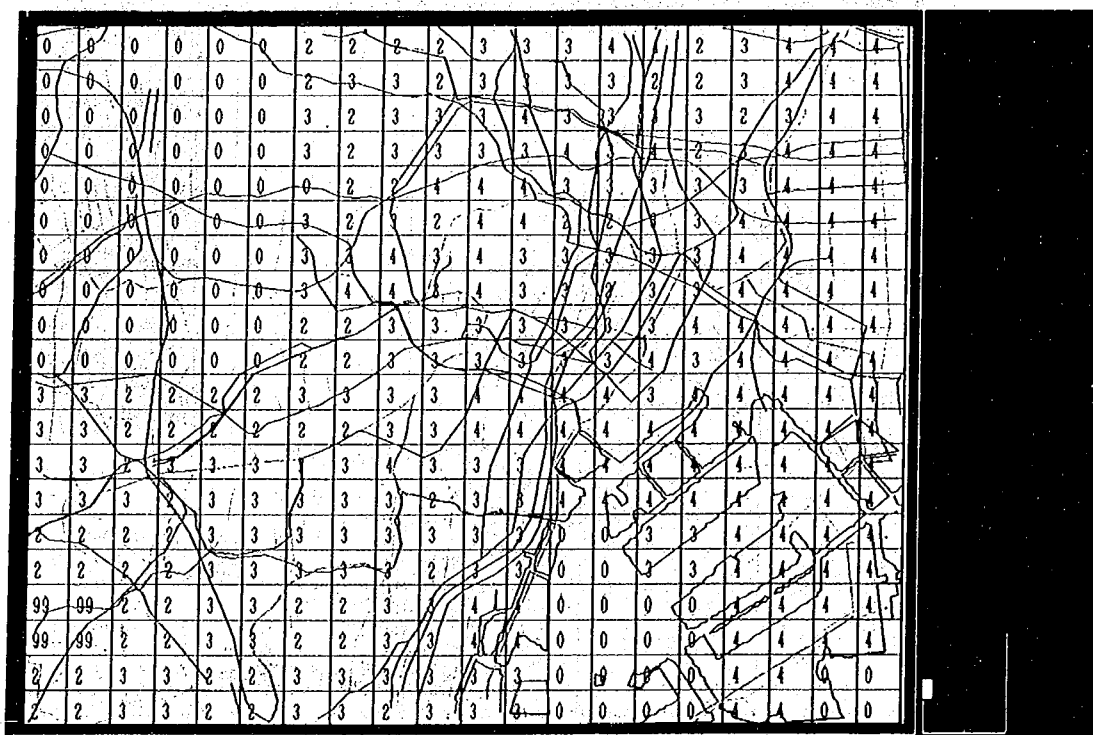


Photo. 9.4 Display of Map B of the Ground Groups Estimated from Actual Ground Data to Estimate Maximum Ground surface acceleration

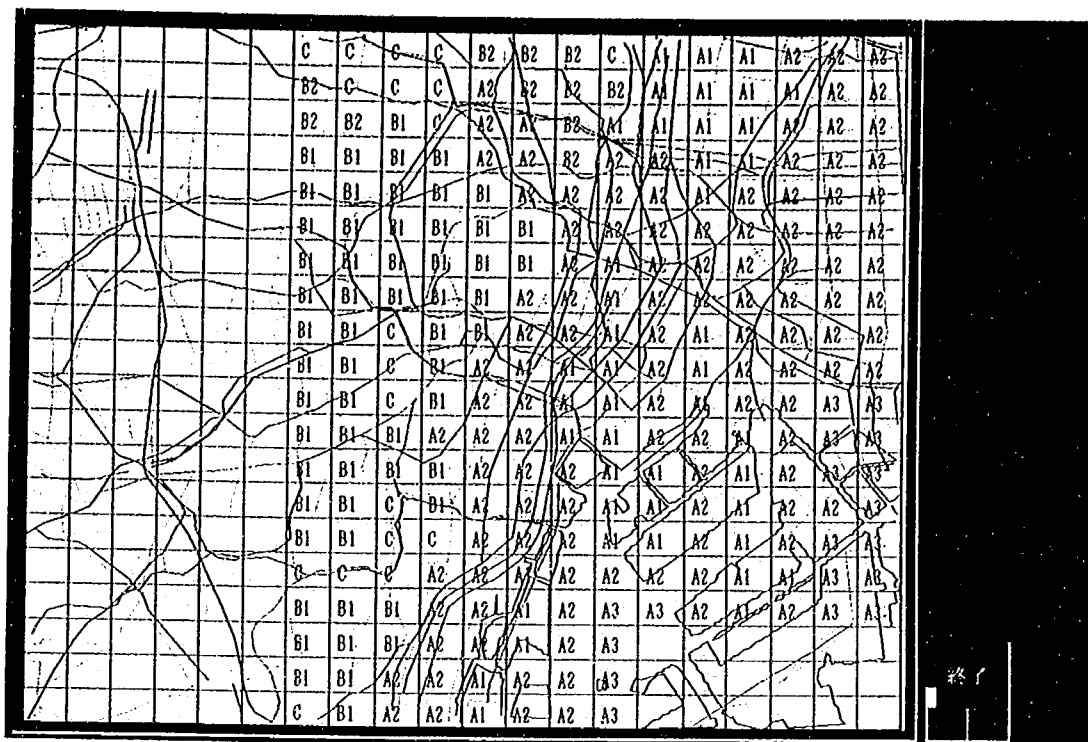


Photo. 9.5 Display of Map B of the Topographical Ground Conditions to Estimate Liquefaction Potential

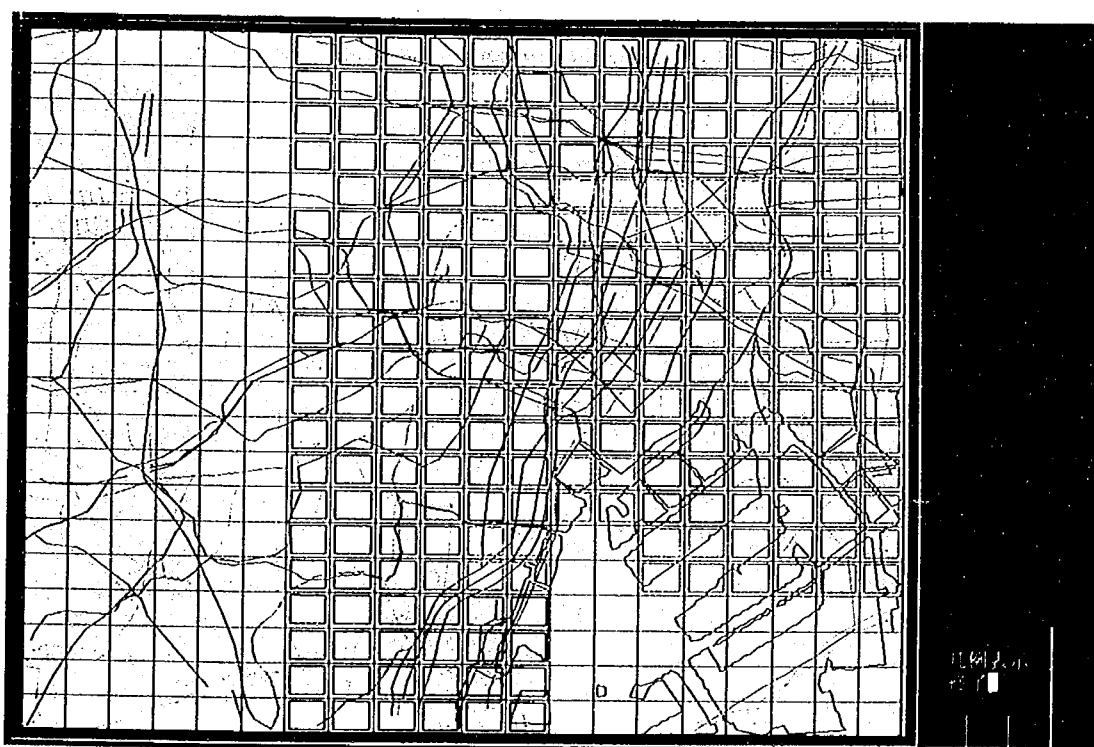


Photo. 9.6 Display of Map B of the Liquefaction Potential Estimated for Each Cell (Group-A : Red Grid Added, Group-B: Yellow Grid Added, Group-C : Blue Grid added)

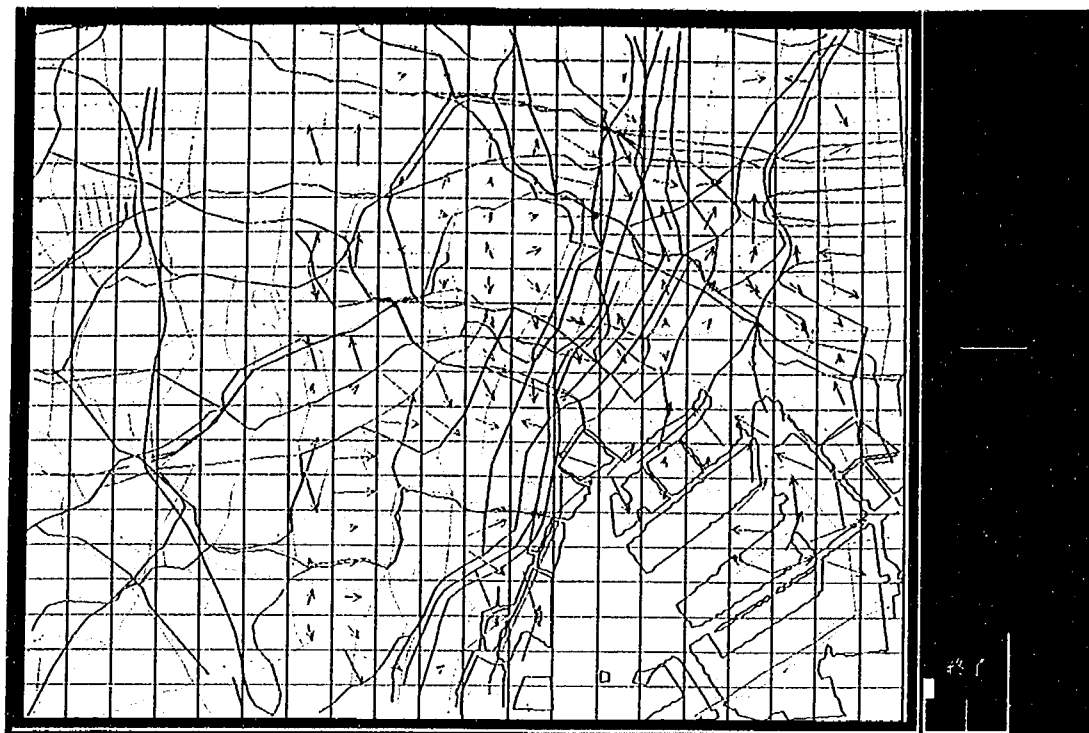


Photo. 9.7 Display of Map B of the Magnitude and Direction of Maximum Lateral Spread of Ground for Each Cell

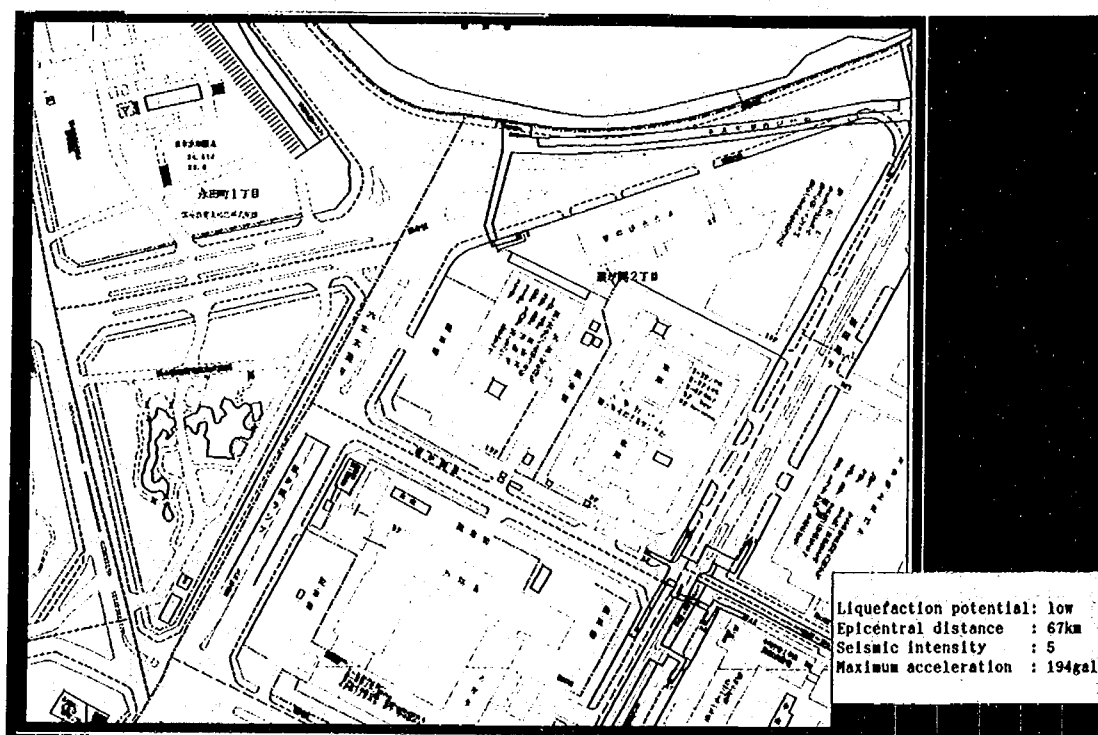


Photo. 9.8 Display of Map C of the Location of Structures and Liquefaction Potential

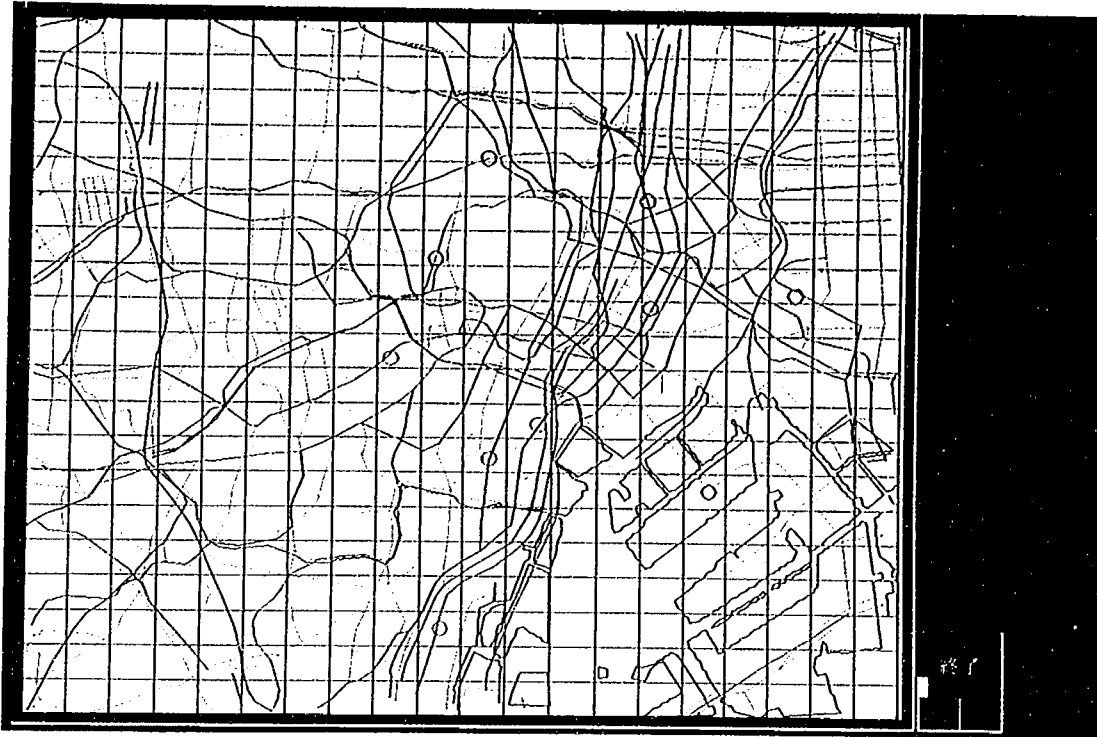


Photo. 9.9 Display of Map B of the Location of Damaged Structures Reported from Sites

REFERENCES

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1. LESSONS IN THE 1990 PHILIPPINES EARTHQUAKE	147
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THE UNIVERSITY OF CHICAGO

DEPARTMENT OF CHEMISTRY

REPORT OF THE CHAIRMAN OF THE COMMITTEE ON THE

PROGRESS OF THE DEPARTMENT OF CHEMISTRY

FOR THE YEAR 1960-1961

PRESENTED TO THE BOARD OF TRUSTEES OF THE UNIVERSITY OF CHICAGO

1. LESSONS IN THE 1990 PHILIPPINES EARTHQUAKE

We can learn several important lessons on the civil engineering structures from the 1990 Philippines Earthquake as follows:

(1) The slope failures occurred at a number of sites in the mountain area and the trunk roads for economic activity and social life were collapsed and damaged severely [Photos. R.1.1 and R.1.2] . The rapid restoration and opening of the trunk roads were needed to transport the relief goods and construction machines for rescue person's lives. The necessity of disaster reduction for highways in the mountain area has been recognized again.

(2) Soil liquefaction occurred at many sites underlain by poor alluvium deposits and induced severe damage and collapse of buildings and civil engineering structures such as bridges and river-dykes [Photos. R.1.3 and R.1.4] . The countermeasure for reducing the disaster by soil liquefaction has been recognized.

(3) The flow out of woods and mud fallen with slope failures occurred by rainfall after the earthquake, and the flood also occurred at the cave-in area liquefied during the earthquake by the rainfall [Photos. R.1.5, R.1.6 and R.1.7] . Therefore, the secondary disaster such as flow out of woods and mud and flood by rainfall after an earthquake should be noted.

(4) The characteristics on the damage of structures are summarized as 1) Large active fault on the ground surface [Photo. R.1.8] , 2) Damage of buildings, bridges and river-dykes induced by soil liquefaction, 3) Cave-in by liquefaction and flood, 4) Slope failures and roads damaged in the mountain area, 5) Damage of high and middle-high buildings in the highlands.

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**Photo. R.1.1 Many Slope Failures Occurred at the mountainous Area
Along the Santa Fe River [Ref. R.1-2]**



**Photo. R.1.2 Large Slope Failure Occurred at the Dalton Pass
of the Pan Philippines Highway [Ref. R.1-2]**



**Photo. R.1.3 Soil Liquefaction Occurred at
the Carmen Bridge [Ref. R.1-3]**

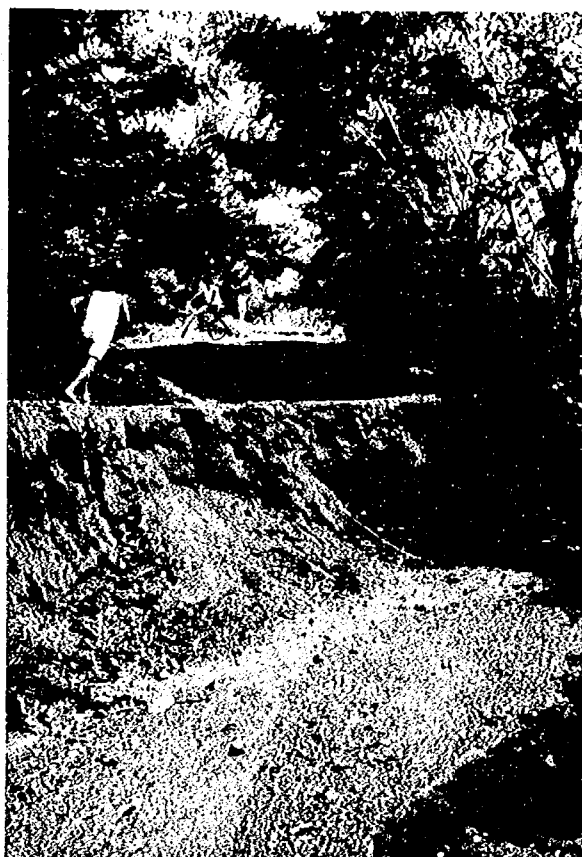


**Photo. R.1.4 Levee Damaged by Soil Liquefaction along
the Aguno River [Ref. R.1-4]**



**Photo. R.1.5 Flow Out of Woods and Mud at
the 2nd Puncan Bridge [Ref. R.1-2]**



**Photo. R.1.6 Temporary Dam by Slid Earth along
the Buedo River [Ref. R.1-2]**



Photo. R.1.7 Settlement of Ground Induced by Soil Liquefaction and Flood [Ref. R.1-4]

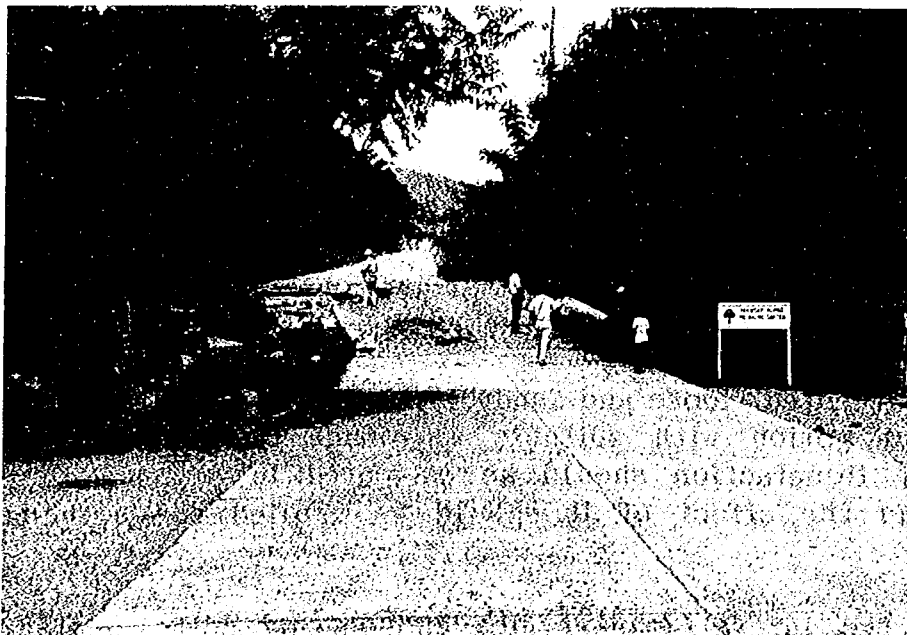


Photo. R.1.8 Fault (Maximum Lateral Displacement: About 5 m, Maximum Vertical Displacement: About 1.3 m) Appeared Across the Pan Philippines Highway at Digidig [Ref. R.1-2]

2. VIEW POINTS IN RECENT EARTHQUAKES

The characteristics of the phenomena and the damages by earthquakes are different in each earthquake and depend on the location of earthquake, the damaged site condition, the conditions of structures, and so on. The typical earthquakes occurred recently in the world and Japan are represented in Table R.2.1. The characteristics of recent earthquakes are as follows.

(1) Soft Ground and Damage

In the 1989 Loma Prieta Earthquake, the elevated I-880 Cypress viaduct was collapsed at the northern end and also damaged but not collapsed at the southern end. The northern end of the viaduct is underlain by soft to medium stiff recent estuarine deposits of silty marine clay known locally as Bay Mud, and on the other hand, the southern end of it is underlain by stiff alluvium. Fig. R.2.1 shows the relation between the geological condition and the collapse or damage of the Cypress viaduct, and it can be seen that the soft ground such as Bay Mud seems to affect the behavior of structures.

According to progress of urbanization near the coast, the reclaimed land has been spread rapidly. The dynamic behavior of the reclaimed land and soft alluvium is generally amplified during earthquakes, and the structures constructed at these ground are possible to be damaged such as the 1989 Loma Prieta Earthquake. Therefore, it is necessary pay attention to the seismicity of poor ground during earthquakes from now on.

(2) Soil Liquefaction/Induced Lateral Ground Flow and Damage

The soil liquefaction occurred and caused considerable damage at a number of sites such as the San Francisco Bay area in the 1989 Loma Prieta Earthquake and the Dugpan City in the 1990 Philippines Earthquake. Fig. R.2.2 shows the distribution map of liquefied sites at Marina District in San Francisco where many buildings and lifeline facilities were damaged.

Because the saturated, loose and sandy layers will be certainly liquefied by the strong ground motion with suitable magnitude, it must be noted that the possibility of liquefaction should be estimated in advance to construct the structures and the effect of it should be applied to earthquake design of structures.

Furthermore, the lateral ground flow induced by soil liquefaction occurred largely and induced severe damage of structures in the 1990 Philippines Earthquake. Because the lateral ground flow is likely to be affected by geographical conditions such as a sloped surface ground on liquefiable soils, it is necessary to pay attention to the sites near by a river.

(3) Slope Failure and Damage

When the earthquake occurs in and around the mountain area, many slope failures are induced, and the structures such as roads are damaged severely. The slope failures and damage of structures occurred in many earthquakes such as the 1990 Philippines Earthquake, the 1990 Iran Earthquake, the 1989 Loma Prieta Earthquake, the 1984 Nagano-ken-Seibu Earthquake and the 1978 Izu-Oshima-Kinkai Earthquake.

Because the roads in the mountain area are very important lifeline, the damage of them isolate the towns and villages from other area and the reconstruction of them is generally very difficult and takes much time. Therefore, it is noted that the countermeasures of structures such as roads for earthquake disaster and reconstruction plan must be examined beforehand.

(4) Lifeline Damage

The lifeline facilities such as water-supply, sewage, gas, electric power and telephone are very important fundamentals for social life at present. Because the lifeline facilities are generally constructed under ground, their damage induced by earthquakes is related to the dynamic behavior and damage of surrounding ground.

In the 1989 Loma Prieta Earthquake, the water-supply system and sewerage system were damaged severely in Marina District. Fig. R.2.3 shows the location of major water pipe and sewer break in the District, and it can be seen that the network system were damaged at the soft ground condition sites.

As mentioned above, because the damage of lifeline facilities by earthquakes influence social life directly, it will be needed to consider the countermeasure for the lifeline facilities still more.

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- R.2-2) Hough, S. E., et. al., "Did Mud Cause Freeway Collapse?", EOS Transactions, American Geophysical Union, November 21, 1989.
- R.2-3) Geological Survey, "Effects of the Loma Prieta Earthquake on the Marina District San Francisco, California", Open-File Report 90-253, April 20, 1990.

Table R.2.1 Recent Earthquakes in Japan and the World

Earthquake	M	Country	Date	Epicenter	Damaged Area	Phenomena	Typical Damage
Philippines EQ	7.8	Philippines	1990. 7.16	Inland	Mountain, Urban	Strong Motion, Liquefaction, Fault	Building, Bridge, Slope(Road)
Iran EQ	7.7	Iran	1990. 6.21	Inland	Mountain, Urban	Strong Motion, Fault	Building, Bridge Slope(Road)
Loma Prieta EQ	7.1	U.S.A.	1989.10.17	Inland	Mountain, Urban	Strong Motion, Liquefaction, Fault	Bridge, Slope (Road), Building (Fire)
Spitak EQ	7.0	U.S.S.R.	1988.12. 7	Inland	Mountain, Urban	Strong Motion, Fault	Building, Lifeline
Mexico EQ	8.1	Mexico	1985. 9.19	Ocean	Urban	Strong Motion	Building, Lifeline
Nagano-ken-Seibu EQ	6.8	Japan	1984. 9.14	Inland	Mountain	Strong Motion	Slope (Road)
Nihon-kai-Chubu EQ	7.7	"	1983. 5.26	Ocean	Urban, Coast	Liquefaction, Tsunami	Levee, Road Banking, Port and Harbor, Building
Miyagi-ken-Oki EQ	7.4	"	1978. 6.12	Ocean	Urban	Strong Motion, Liquefaction	Bridge, Levee, Banking
Izu-ohshima-Kinkai EQ	7.0	"	1978. 1.14	Ocean	Mountain	Strong Motion	Slope (Road)
Niigata EQ	7.5	"	1964. 6.16	Ocean	Urban, Coast	Liquefaction, Tsunami	Building, Bridge, Oil Tank(Fire)

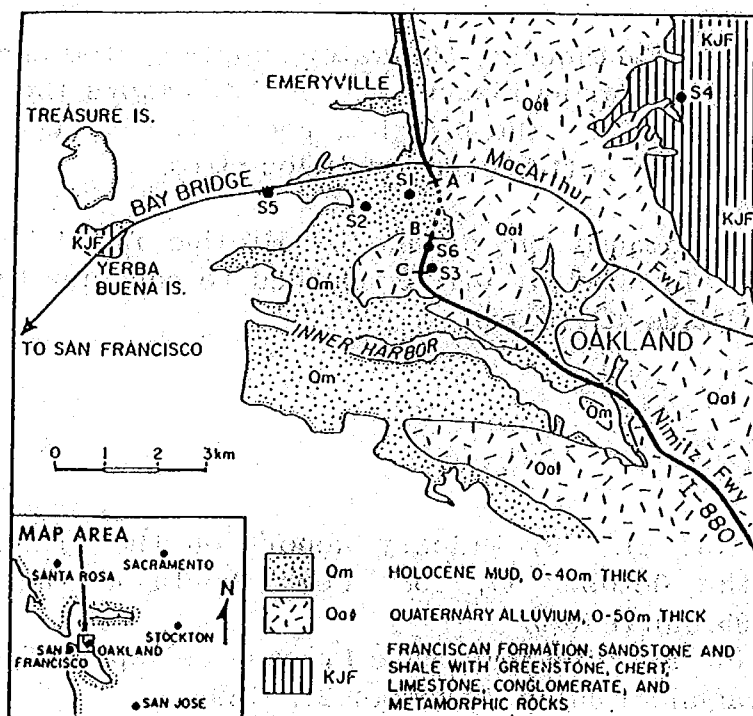
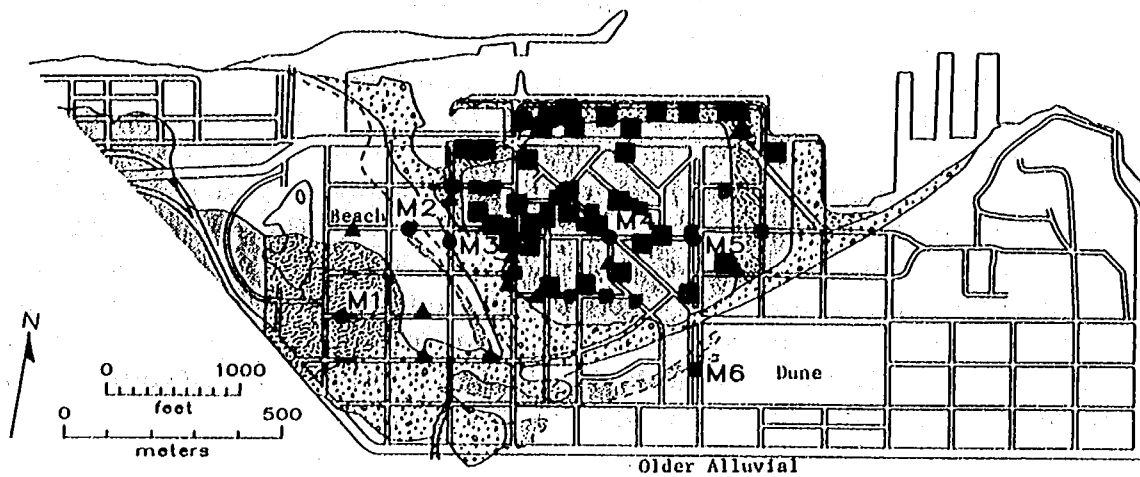


Fig. R.2.1 Relation Between Geological Conditions and Damage of Cypress Viaduct in the 1989 Loma Prieta Earthquake [Refs. R.2-1 and R.2-2]



ARTIFICIAL FILLS

- | | |
|--------------------------------|---|
| ■ 1906-1917 (principally 1912) | ▲ Brown sand boils, definite |
| ■ 1895-1906 | ▲ Brown sand boils, definite from pipe breakage |
| ■ 1869-1895 | ■ Gray sand boils, definite |
| ■ 1851-1869 | ■ Gray sand boils, definite from pipe breakage |
| ● M3 Location of USGS borings | |

Fig. R.2.2 Locations of Geologic Units and Sand Boils at Marina District in the 1989 Loma Prieta Earthquake [Draw up based on Ref. R.2-3]

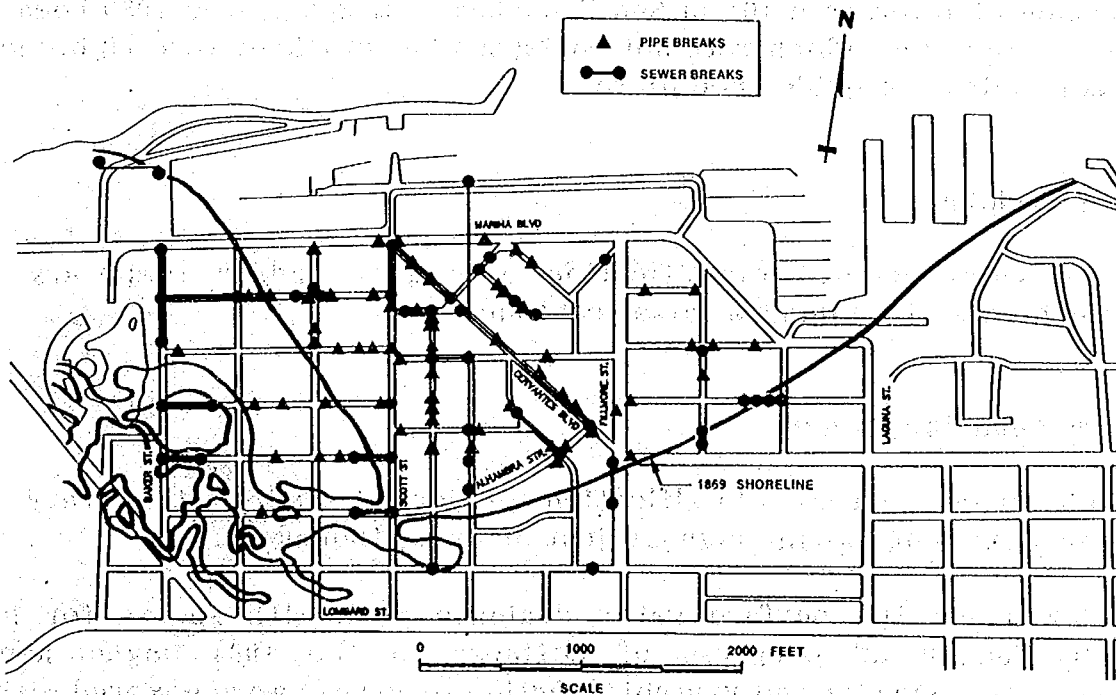


Fig. R.2.3 Locations of Major Water Pipe and Sewer Breaks at Marina District in the 1989 Loma Prieta Earthquake [Ref. R.2-1]

3. URGENT AND TEMPORARY REPAIR METHODS FOR DAMAGE

The structures damaged by the earthquake are required to be repaired, reinforced and renewed properly for rapid restoration of social and economic activity to earthquake disaster. Hereupon, the urgent and/or temporary repair is necessary and effective to prevent the secondly induced damage by after-shock and the spread of damage by main shock as emergency measures. The urgent and temporary repair methods should be examined and guided for one of methods to reduce post-earthquake disaster and retrofit before earthquake temporarily. Based on the investigation of reconstruction methods for earthquake disaster in the past earthquakes, "Manual for Repair Methods of Civil Engineering Structures Damaged by Earthquakes" [Ref. R.3-1] are proposed in Japan.

1) Highway Bridge

Table R.3.1 shows general classification of urgent and/or temporary repair methods for highway bridge just after earthquakes.

Photo. R.3.1 shows the temporary bridge under construction, neighboring Magsaysay Bridge whose girders were fallen in the 1990 Philippines Earthquake. The temporary bridge was made by driving wood piles and putting steel truss girder and wood plate.

Photo. R.3.2 shows the temporary retrofit of piers of elevated two-level viaduct at the section of Interstate 101 in San Francisco damaged in the 1989 Loma Prieta Earthquake. The piers which were not collapsed but cracked were tightened with use of steel cables and guide wood plates.

(2) Highway Banking

Table R.3.2 shows general classification of urgent and/or temporary repair methods for highway banking just after earthquakes.

(3) Slope Neighboring Highway

Table R.3.3 shows general classification of urgent and/or temporary repair methods for slope neighboring highway just after earthquakes.

Photo. R.3.3 shows the surface water drainage work with sheets for highway banking in danger of progress of collapse in the 1984 Nagano-Ken-Seibu Earthquake. The opening and unstable portion of cracks were covered with vinyl sheets weighted with sand bags to prevent inflow of water.

(4) Levee

Table R.3.4 shows general classification of urgent and/or temporary repair methods for levee just after earthquakes.

Photo. R.3.4 shows the prevention work of leakage of ground laying under the levee damaged during the 1983 Nihon-Kai-Chubu Earthquake. Because the penetration of ground leakage was feared, cut-off steel piles were driven into the ground to prevent the leakage and to lower the pore water pressure. The levees were also reinforced by banking with earth and piling with sand bags.

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Table R.3.1 Temporary Repair Methods for Highway Bridge [Ref. R.3-1, Compile]

Sht 1 of 2

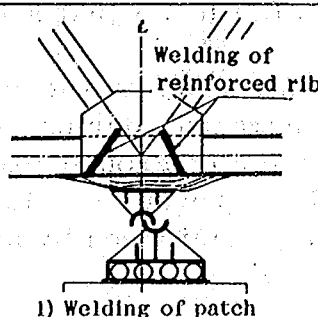
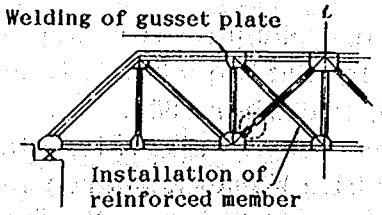
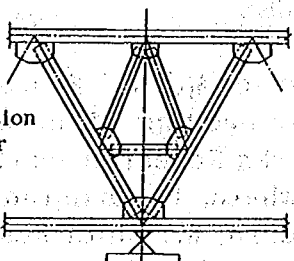
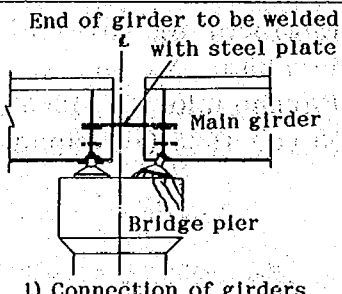
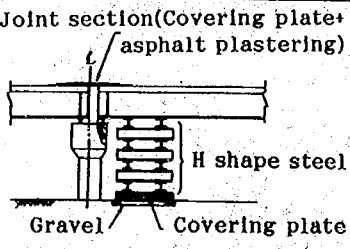
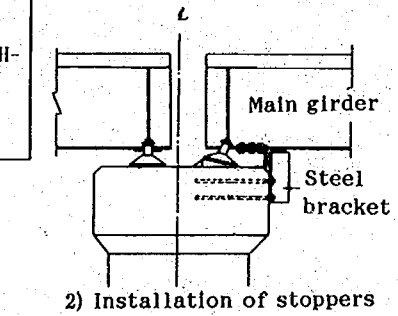
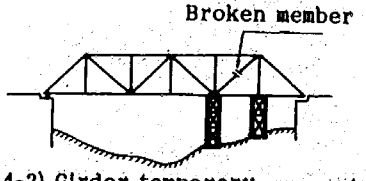
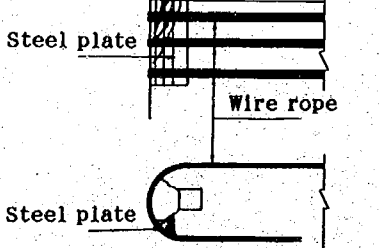
Main purpose	Kind of method	Description of work	Example of patten
Reinforcement of member	1) Welding of rib	1) Member buckled or cracked should be reinforced by welding ribs.	 1) Welding of patch
	2) Installation of reinforced member	2) Member buckled should be reinforced by installing members.	
	3) Installation of alternative member	3) Member cutted or losing its section should be reinforced by installing members and gusset plate.	
	 Welding of gusset plate Installation of reinforced member		 Installation of member
	3) Installation of alternative member		2) Installation of reinforced member
Falling-off-prevention	1) Connection of girders	1) End of girders should be connected by welding with steel plate or cable.	 End of girder to be welded with steel plate Main girder Bridge pier 1) Connection of girders
	2) Installation of stoppers	2) Pier crest should be attached with stopper of steel bracket.	
	3) Binding of pier crest	3) Pier crest cracked should be reinforced by binding with steel plate and wire ropes.	
	4) Girder support	4) Girder in danger of falling-off should be supported by H-shape steel, steel pipes and woods.	
	 Joint section(Covering plate+asphalt plastering) H shape steel Gravel Covering plate 4-1) Temporary girder holder		 Main girder Steel bracket 2) Installation of stoppers
	 Broken member 4-2) Girder temporary support		 Steel plate Wire rope Steel plate 3) Temporary binding of pier crest concrete by cables

Table R.3.1 (Continue)

Sht 2 of 2

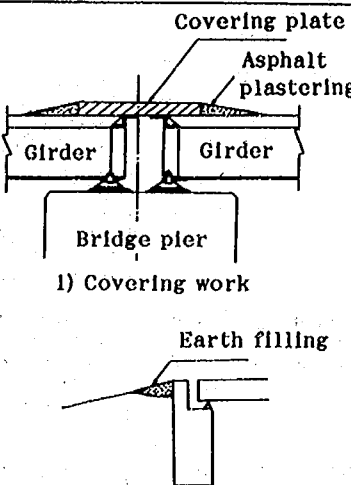
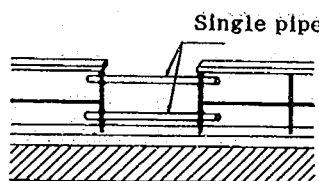
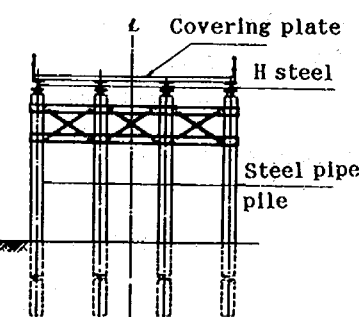
Main purpose	Kind of method	Description of work	Example of patten
Correction of bridge surface	1) Covering of plate 2) Filling of step	1) Gap occurred on bridge surface should be covered with plate. 2) Step difference should be filled with earth, gravel asphalt.	 1) Covering work 2) Repair of step difference
Repair of handrail	1) Covering plate and pipe	1) Handrail damaged should be repaired with steel bar, pipe and plate.	 Repair of handrail covering plate
Substitute of failed girder	1) Temporary girder 2) Temporary bridge	1) Girder fallen down should be changed with temporary girder of steel members. 2) Temporary bridge should be constructed neighboring collapsed bridge. 3) Berry bridge should be carried into and constructed.	 2) Temporary bridge at Kinou bridge

Table R.3.2 Temporary Repair Methods for Highway Banking [Ref. R.3-1]

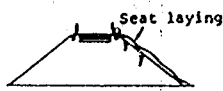
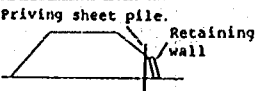
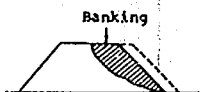
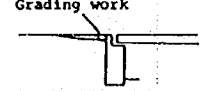
Main purpose	Type of work	Description of work	Typical drawing of sample
Prevention of collapse by rainwater infiltration	sand filling Asphalt seal Sandbag piling Earth curb Vinyl sheet laying Temporary drainage	To prevent not only the inundation of rainwater to the crack and collapse of surface and slope of banking face but also the collapse of banking.	
Prevention of embankment collapse by traffic	Banking Sandbag laying Sheet pile driving Wooden piling	Increase stability of embankment to prevent the collapse of banking by traffic.	
Securing of traffic (Securing of width and establishment of detour)	Banking and pavement Sheet pile driving Pier and temporary bridge (Bailey bridge) Construction work of detour	Securing of width to ensure passage. Works for detour will be done when damage is heavy.	
Securing of safety for passage (Treatment of faulting and step difference)	Grading works Sediment filling Asphalt patching	Treatment for damage of faulting for jointing of bridge and etc. shall be done to secure safety for traffic.	

Table R.3.3 Temporary Repair Methods for Slope Neighboring Highway
[Ref. R.3-1, Compile]

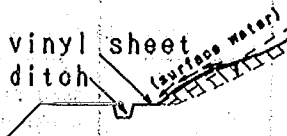
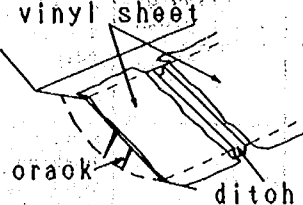
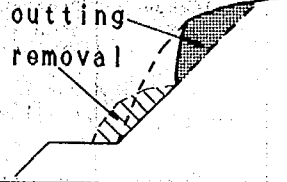
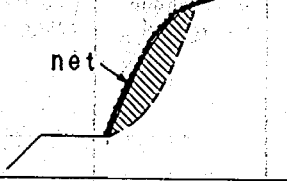
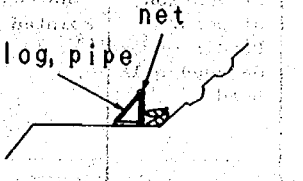
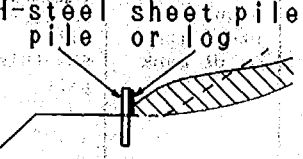
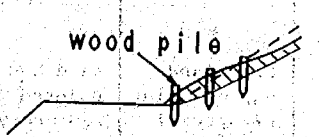
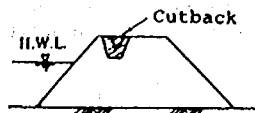
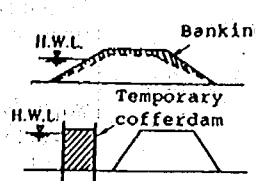
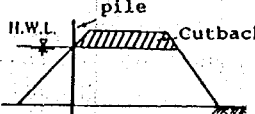
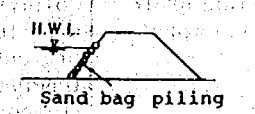
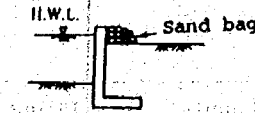
Type of construction	Detail of construction	Description of work	Typical drawing of sample
Removal of colluvial deposit	Removal of colluvial deposit	Colluvial deposit and rockfall piled on the road shall be removed and traffic function of road shall be provided.	 removal
Surface water drainage work	Surface water drainage work (Temporary drainage)	Temporary drainage ditch shall be provided with vinyl sheet and U-type ditch and surface water shall be drained.	 vinyl sheet ditch (surface water)
	Works of covering with sheet	The opening and unstable portion of cracks shall be covered with vinyl sheet so as to prevent inflow of water.	 vinyl sheet crack ditch
Cutting work	Cutting work of instability portion	Extremely unstable portion remained on a part of slope and cut slope suffered from damage shall be removed by cutting.	 cutting removal
Netting work	Netting work (rockfall prevention work)	The collapsed portion and or unstable portion shall be covered with synthetic resin net or wire net to prevent movement of small scale unstable portion.	 net
Slide protection work	Colluvial deposit prevention fencing work (Temporary protective fencing work)	Simplified fencing structures shall be made combining wire netting with log and scaffolding pipe and providing the road and catches small scale slide soil etc.	 net log, pipe
	H-steel sheet pile (Soldier beam and breast-board construction)	H-steel pile shall be driven to the road and pine log, pine sheet pile and steel sheet pile shall be laid across and catches small scale slide soil etc.	 H-steel sheet pile pile or log
Piling works	Wood piling work	Wood pile shall be driven to fix the shallow soil layer which was moved to the hill.	 wood pile
Counter-weight fill work	Piling soil sack (Soil sack piling work)	Sand back which has similar function as counterweight fill shall be piled instead of banking.	 sand bag

Table R.3.4 Temporary Repair Methods for Levee [Ref. R.3-1]

Main purpose	Kind of method	Explanation of method	Example of pattern
Prevention of infiltration of rainfall	Sand filling, banking Cutback	To prevent rain water from infiltration into bank through damaged spots and prevent the bank from weakening.	
Securing of levee crown height	Sand filling, banking Cutback, banking Sand bag piling Temporary cofferdam	To reinforce damaged spots with earth and sand to stabilize the bank against high water, etc. In large scale damage, emergency cofferdam should be studied.	
Prevention of leakage of bank	Sand filling, banking Sand bag piling Cutback Protection with good soil (outer slope) Cut-off sheet pile (Outer slope, H.W.L. position)	Leakage is feared because of cracks, etc. on bank, and the bank should be reinforced with earth and sand, and covered with infiltration material to prevent the bank from water infiltration and from weakening.	
Prevention of leakage of ground	Cut-off sheet pile (bottom of outer slope)	When penetration of ground leakage is feared, leakage of inner ground is prevented and pore water pressure is lowered to stabilize the bank.	
Prevention of scour Repair of damaged bulkhead	Sand bag piling Pointed concrete	To prevent bank from scouring and collapsing by rain water and river flow and weakening.	
Securing stabilization of bank	Sand bag piling Counterweight fill Sheathing sheet pile	To stabilize bank by counterweight to damaged bank. In case area is not wide enough, earth supporting work should be considered.	
Crack and gap of joint of special levee	Sand bag filling Pointed concrete Temporary cofferdam	To repair urgently damaged spot of damaged cut-off function of special levee, or building up temporary cofferdam.	

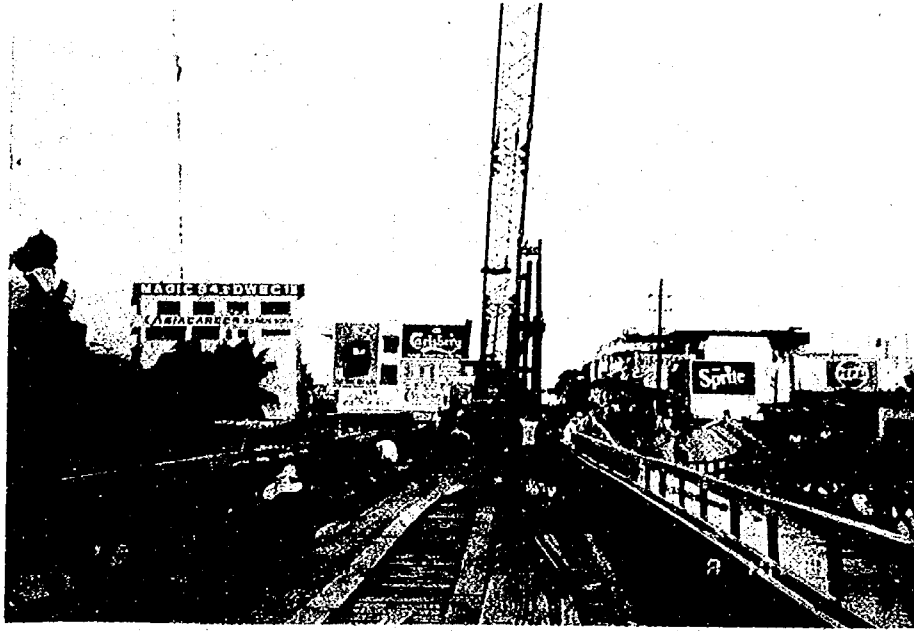


Photo. R.3.1 Temporary Bridge Constructed Neighboring Main Bridge in the Philippines Earthquake [Ref. R.3-2]

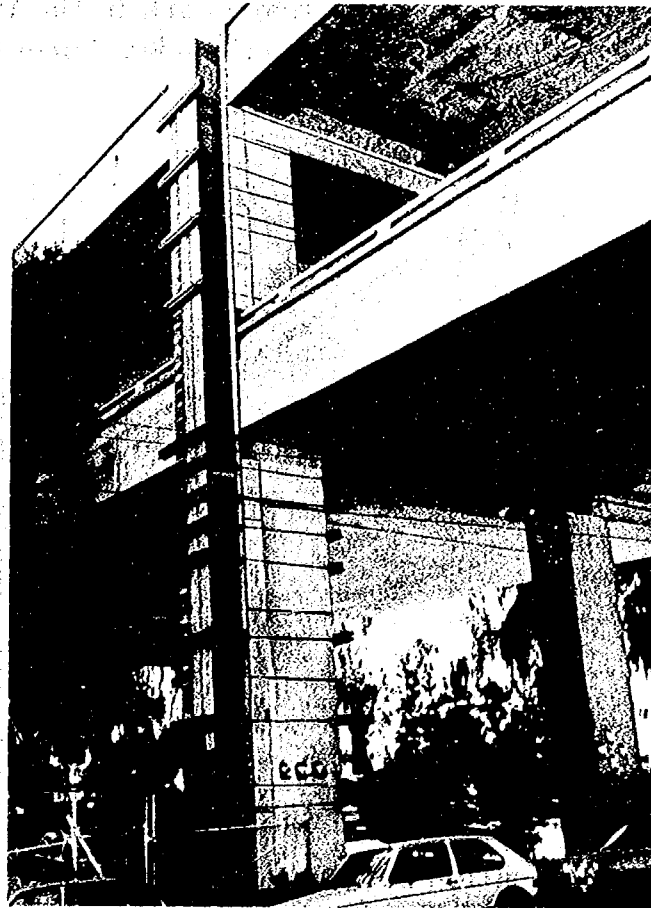


Photo. R.3.2 Temporary Retrofit of Bridge Pier in the 1989 Loma Prieta Earthquake [Ref. R.3-3]



**Photo. R.3.3 Surface Water Drainage Work in the 1984
Nagano-Ken-Seibu Earthquake, Japan [Ref. R.3-4]**



**Photo. R.3.4 Prevention Work for Levee with Sand Bags
and Steel Sheet Piles [Ref. R.3-5]**

4. RETROFIT METHODS

Based on the experience in the past earthquakes, many lessons can be obtained as for both preliminary retrofit methods before earthquakes and permanent retrofit methods after earthquakes which are effective for reducing earthquake damage of existing structures. Several examples on the effect of preliminary retrofit countermeasures can be learned in the 1990 Philippines Earthquake.

The retrofit methods to reduce the earthquake induced damage of highway bridge, highway banking, slope neighboring highway and levee can be shown in Tables R.4.1, R.4.2, R.4.3 and R.4.4, respectively, based on the lessons in the past earthquakes in Japan.

REFERENCES

- R.4-1)Public Works Research Institute, "Manual for Repair Methods of Civil Engineering Structures Damaged by Earthquakes", Technical Note of Public Works Research Institute, Vol. 45, December 1986.
- R.4-2)Japan Road Association, "Manual for Earthquake Disaster Prevention of Highway Facilities", February, 1988 (in Japanese).

Table R.4.1 Retrofit Methods for Highway Bridges
[Ref. R.4-1 and R.4-2, Compile]

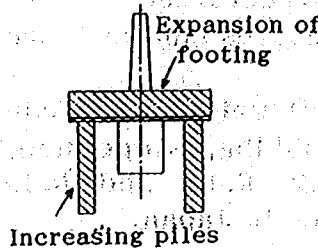
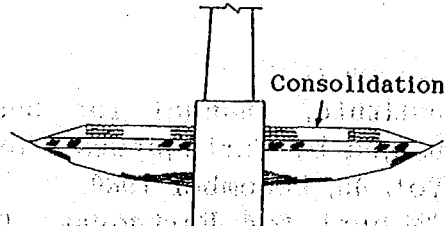
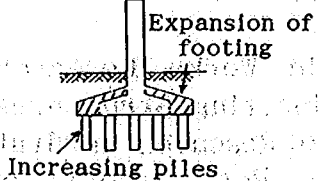
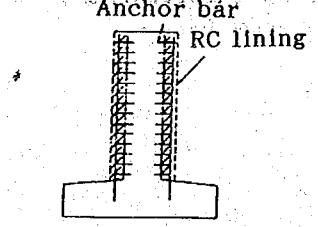
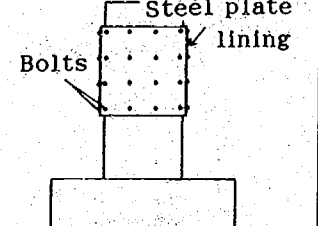
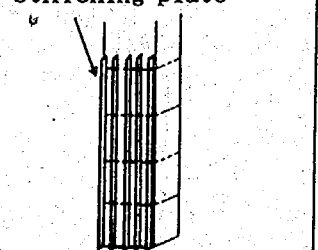
Structures	Kind of Method	Typical pattern
Foundation	- Increasing piles and footings. - Setting up underground continuous wall and continuous beam. - Steel sheet pile coffering work. - Ground improving work. - Earth anchor work. - Consolidation work of foundation. - Replacement.	 Expansion of footing Increasing piles
	 Consolidation  Expansion of footing Increasing piles	
RC pier Abutment	- Injection of resin, puttying, resin mortar repair, concrete repair - RC lining - Steel plate lining - Increasing wing walls - Bonding band steel plate - Reinforcing anchor - Increasing aseismic walls - Replacement	 Anchor bar RC lining  Steel plate lining Bolts
Steel pier	- Method of stuff inside - Method of splice the stiffening plate from outside - Replacement	 Stiffening plate

Table R.4.1 (Continue)

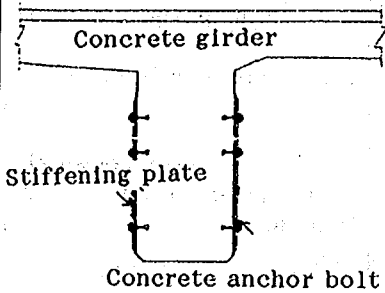
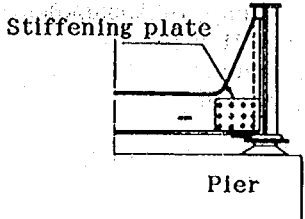
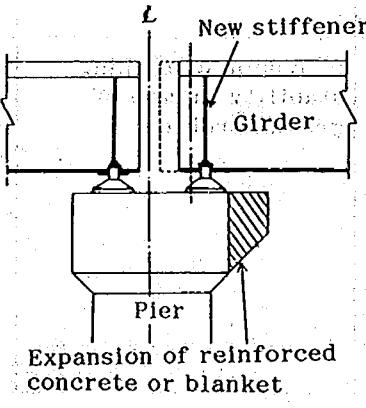
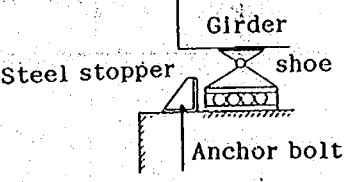
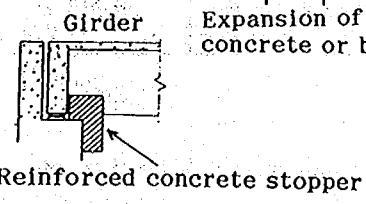
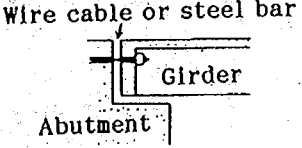
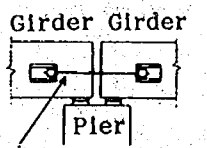
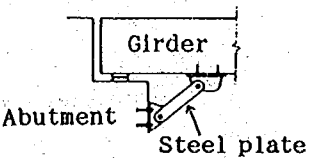
Superstructure	Concrete bridge	
	Steel bridge	
Supporting section	- Expansion of pier crest with reinforced concrete - Expansion of pier crest with blanket - Steel or reinforced concrete stopper set on the pier crest - Connection of girder with girder, pier and abutment by wire cable, steel bar and steel plate	
    		

Table R.4.2 Retrofit Methods for Highway Banking
[Ref. R.4-1, Compile]

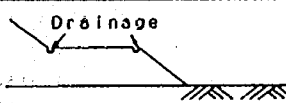
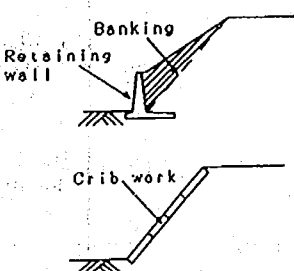
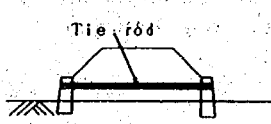
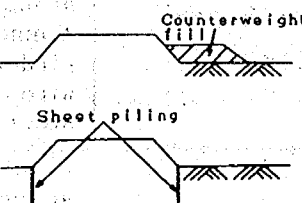
Main purpose	Kind of method	Typical pattern
Treatment of rain water	- Drainage ditch - Earth curb	
Securing of road function	- Banking and pavement - Approach cushion board - Asphalt sealing - Asphalt grouting to cavity	
Securing of embankment stability in case embankment comes into question	- Banking work - Retaining wall - Crib work	
Provision of banking stability in case of ground problem	- Counterweight fill - Ground improvement - Ground water drainage - Sheet piling - Tie rod	
	 	

Table R.4.3 Retrofit Methods for Slope Neighboring Highway
[Ref. R.4-1, Compile]

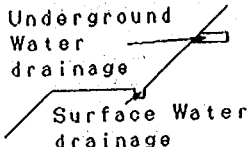
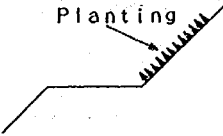
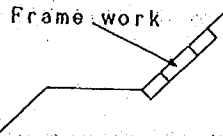
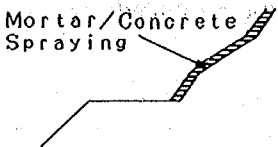
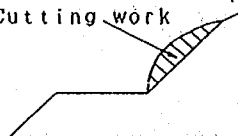
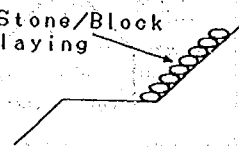
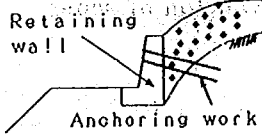
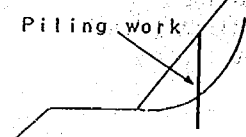
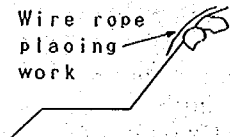
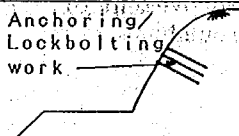
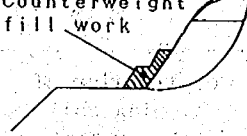
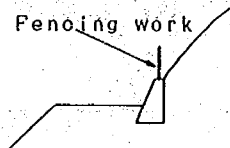
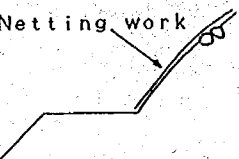
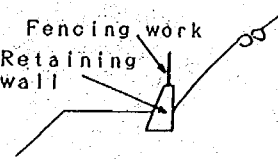
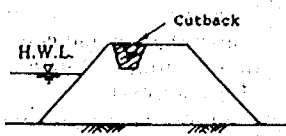
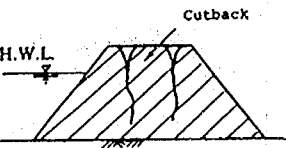
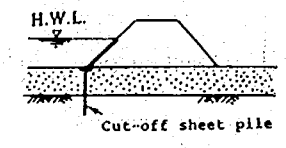
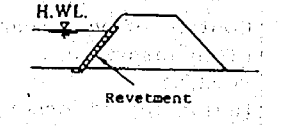
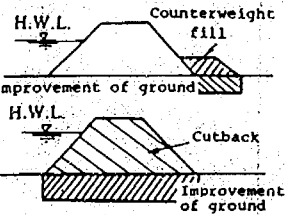
Main purpose	Kind of method	Typical pattern
Control of occurrence of erosion, collapse and rockfall (try not to be effected by rainfall etc.)	· Drainage work · Turfing work · Spraying work · Pitching work · Crib work · Partial cutting	
	 	 
Control of occurrence of collapse and rockfall (balancing of force)		
	· Retaining wall work · Anchoring work · Piling work · Counterweight fill · Rockfall prevention work	  
Control and restraint	 	
	· Fencing work · Wire cylinder	
Protection for collapse and rockfall (not to cause damage even if collapse and rockfall occurred)	· Netting work · Fencing work · Work for retaining wall	
	 	

Table R.4.4 Retrofit Methods for Levee [Ref. R.4-1]

Main purpose	Kind method	Typical pattern
Securing strength of levee	Cutback and Banking	
Prevention of leakage of bank	Cutback and Banking	
Prevention of ground leakage	Cut-off sheet pile (toe of outer slope)	
Prevention of scour	Revetment	
Securing Stabilization of levee	Counterweight fill Improvement of ground	
Securing stabilization of special levee	Construction of retaining wall Reinforcing of retaining wall Repairing of retaining wall	