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2-28-32 Honkomagome, Bunkyo-ku, Tokyo, Japan

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PRESENT CONDITION OF SLOPE PROTECTION PROCEDURES AND STABILITY OF SLOPES DURING RAIN

by

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Highway Research Board, Washington, D.C.
January, 1972

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1. Introduction

Factors that may cause slope failure include rain water, melting snow, frost heave, and earthquakes, but the main cause for slope failure is the action of water.

Japan abounds with situations that may lead to the disasters of slope failure. Her terrain is steep and mountainous, and her natural conditions are such that are liable to invite heavy rains of typhoons and the wet season. In addition, the difficulty in the acquisition of land for highway construction which has become exceedingly serious in recent years has often resulted in the adoption of slopes of a steeper grade.

The recent disasters of slope failure which occurred successively were caused directly by seepage and surface water due to heavy rain, and the falling mud and rock crushed or swept away into the river those vehicles which were travelling on the highway, thereby posing a difficult problem in the control of traffic at the dangerous zone during heavy rain.

Especially the failure of cutting slope occurs frequently even while the highway is still under construction, and continual efforts have been made to protect slopes all the way through the stages of the highway planning, the highway construction, and the maintenance of highway after the completion of their construction and putting them into service to the public. In order to protect slopes from surface erosion and ensure safety against slope failure, the results of soil survey are being actively used for the selection of the optimum procedure of slope protection; but so far no selection

criterion and design method for the optimum procedure of slope protection have been established, which is applicable to all kinds of slopes, mainly because the effect of water on the shearing strength of soil has not yet been clarified, and the causes and process of slope failure are exceedingly complicated due to a wide variety of the composition of strata and soil conditions.

For the determination of optimum grades of embankment slopes or cut slopes and the selection of the optimum procedure of slope protection, a very rough guide exists which is based on the types of soil, condition of surface water and seepage water; but final decision is made according to the technical judgement of the engineer concerned which is based upon his experience and preference and aided by the information obtained from the actual construction which has been executed under similar conditions at the nearby site. Even if the subject is restricted to slope failure caused by water, the conditions involved are too complicated to allow a systematic description of the effectiveness of various procedures of slope protection. In this paper, therefore, the authors would like to deal with only ① the results of actual investigations on the procedures of slope protection which have been adopted on highways in Japan and ② the slope failure experiment which has been carried out by applying artificial rain to the full-scale model of a slope in order to clarify the causes of slope failure due to the flow of seepage water.

2. Actual Condition of Slope Protection

(1) Actual Investigation of Slope Protection

In order to obtain basic data for a criteria of selection of optimum procedures of slope protection, some parts of the national highways were selected and actual investigations were made on all the slopes lying in the parts. Side by side with the above, actual investigations of dangerous spots were made by selecting (1) those cutting slopes which caused comparatively frequent slope failures and (2) slopes which were considered exceedingly liable to cause alope failure.

The former investigations by complete enumeration were aimed at grasping the actual condition of application of various kinds of slope protection procedures to actual cutting slopes and clarifying the characteristic features of the kinds of slope protection by region or by kind of soil and by geological property. Whereas the latter investigation was aimed at finding out the actual condition of those slopes which were generally considered liable to cause slope failure and the kinds of slope protection which were applied to such dangerous slopes.

In both the above-mentioned investigations, detailed studies were made on the properties of soil and rock on the slope, presence of the fault of weathered rock, condition of fissures, conditions of surface water and seepage water on the slope, kinds of slope protection precedures, history of the past slope failures and their causes, and weather conditions under

which such failures occurred, and present condition of the slope protection and of the slope; and a judgement on the overall stability of each slope was given.

The locations of investigation were selected in the following way: Twelve routes of highways were selected which were considered representative of the respective regions of the entire country, and an investigation section of 50 km in length was set up in each of the routes, and all the slopes that were lying in the section were investigated. The number of locations investigated by complete enumeration reached 1,282.

The investigation of dangerous locations was made on those slopes which were considered specially to be in danger of failure, out of the designated sections for traffic control, such as those locations where the traffic was usually stopped at the time of heavy rains. The total number of the object locations in this survey amounted to 523.

(2) Results of Investigations

As a result of the above-mentioned investigations, various interesting facts have been clarified regarding the regional characteristics of the occurrence of slope failure, relationship between the type of soil and rock which forms the slope and the kind of slope protection procedure, stabilization of slopes by slope protection procedure, and relationship between the condition of water on the slope and the stability of the slope.

Slope failure tends to occur in regional concentration, and the areas

showing multiple-occurrence of failures are localized. As a general trend, it is observed that the ratio of occurrence of slope failure becomes gradually higher from the north to the south in Japan. As mentioned above, slope failure is caused by surface water or seepage water. Upon the investigation of a consecutive total rainfall up to the time of the occurrence of a slope failure, it has been observed that the consecutive rainfall increased from the north to the south in Japan. The annual rainfall in the south is considerably larger than that in the north, and the abpve-mentioned fact means that in the rainy district, slope failure will not easily occur, unless there is a large amount of rainfall. Several features which have been clarified as a result of the investigations are explained below:

i) Characteristic Features of Slope protection Procedures Used

One of the outstanding features of the procedures used, as far as slopes mainly made of soils are concerned, is the trend of protecting slopes by covering of vegetation, whenever possible, for the sake of economic reasons and the scenic effect of planting green grasses. The most frequently used method of covering with vegetation are the seed-jetting method and block-sodding method, which are applied to slopes made of various kinds of soils. For slopes made of stiff sandy soil (SW and SC), clayey soil, and clay (ML, CL, OL, MH, OH, and CH), the pick-hole planting method is often used with success.

The stone-pitching method and the concrete block- pitching

method are often used for the protection of slopes made of gravelly soil (GM and GC) and gravel (GW and GP), giving high stability to those slopes. Other methods which can be expected to give high stability to slopes are the retaining-wall method and concrete frame-work method. The retaining-wall method is designed, taking into account the earth pressure that corresponds to the soil conditions; therefore, disasters are rarely found in the case of this protection method. This method is effectively used for all kinds of soil. The concrete frame-work method is frequently applied to slopes made of sandy soil and the locations where spring water is found. The mortar-jetting method is rarely applied to slopes made of soils, but is applied to hard and dense slopes. This method, however, may pose problems in the cold district, because the slope employing this method of protection is easily broken by frost heaving.

As for slopes mainly made of rocks, disasters of slope failure have rarely occurred in the case of slopes protected by the concrete block-pitching method or the mortar-jetting method. This fact does not prove the high stability of slopes obtained by these protection methods; it may rather be more appropriate to consider that these methods are applied to slopes, which have rather been stable, for the only purpose of preventing their surfaces from being eroded.

ii) Shapes of Cutting Slopes

Cutting slopes mainly made of soils are dealt with here. In

general, slopes are often cut at a grade of about 1 : 1 regardless of the kinds of soil. For slopes made of gravelly soil or sandy soil, a grade within the range of 1 : 0. 8 to 1 : 2. 0 is adopted, depending on the heights of slopes and the condition of water on the slope.

When slopes having a grade of 1 : 0. 8 to 1 : 1. 0 are compared with those having grade of 1 : 1 to 1 : 1. 2, the ratio of the number of slopes which are judged to be stabilized with respect to the total number of slopes increases from 88 for the former slopes to 97% for the latter slopes.

The slope height is evenly distributed over the range of 5 m to 30 m for slopes made of various kinds of soil. Slope having a height of less than 10 m is generally judged to be stable, while slope having a height of more than 10 m poses problem. In case of slopes of more than 10 m high, however, the relationship between the height and the stability of slope is not clearly shown.

iii) Kinds of Procedures of Slope Protection and Stability of Slopes

From the results of the above-mentioned investigation by complete enumeration, Figs. 1, 2 and 3 are obtained, in which the ratios of the numbers of various slope protection procedures used with respect to the total number are indicated against the degree of stability of those slopes composed of soils, soils and rocks, and rocks only, respectively.

According to Fig. 1, slopes composed of soils are often protected by

the seed-jetting method, retaining-wall method, block-sodding method, or concrete block-pitching method; and it is very rare that the cutting surface is left untreated. Slopes to which these methods are applied have comparatively satifactory stability, showing that more than 85% of them are stable. The soil that forms slopes is mostly gravelly soil or sandy soil, both of which combined accounts for more than 70% of the total. In the case of slopes composed of soils and rocks accounts as shown in Fig. 2, slopes composed of gravelly soil or sandy soil also account for a greater part of the total. The procedures used frequently for the protection of this kind of slopes are the mortar-jetting method, retaining-wall method, seed-jetting method and concrete block-pitching method. There are only a comparatively small number of untreated slopes. Out of those procedures, the mortar-jetting method gives a comparatively higher stability, while the retaining-wall method and concrete block-pitching method give inferior stability. These results are contrary to the commonsense concerning the stability given by slope protection methods, but may imply the fact that the retaining-wall method and concrete block-pitching method are often used on slopes which are under unfavorable conditions especially in case of the retaining-wall method. Moreover, the slop which continues upwards from the retaining-wall has often experienced local slope failure. It is considered that these conditions frequently appear at the deteriorating slopes. In the case of slopes made of rocks as shown

in Fig. 3, slopes are often untreated, and those slopes of no treatment account for more than half of the total. The kinds of procedures which are used on this kind of slopes are mostly the retaining-wall method and mortar-jetting method throughout all the kinds of slopes formed with rocks, followed by, in order, the net method, seed-jetting method, and falling rock guard fence method. As the rock which forms slopes changes from unsound rock to hard rock, the stability increases; and this trend is clearly observed in the slopes of no treatment and those slopes which employ the net method and retaining-wall method.

iv) Effect of Seepage Water and Surface Water on Slopes to Stability of Slopes

The results of the investigation of dangerous locations are summarized in Fig. 4, which indicate the condition of water on slopes, and kinds of procedures of slope protection and stability of slopes. As can be observed from the figure, the stability deteriorates in slopes which are wet or have surface water or spring water, as compared with dry slopes, throughout all kinds of slope protection procedures. In the investigation by complete enumeration, the ratio of dry slopes to the total showed a considerably larger figure of 83% than that shown in Fig. 4. When the results of both the investigations are taken into account together, it is found that those slopes which are affected by water have lower stability than those which are not. The authors especially emphasize on the importance of drainage of water from the slope and

of taking into careful consideration the activities of water when the selection in procedures of slope protection is to be made.

v) Stability Given by Mortar-jetting Method

The most frequently used method for protection of slopes which are mainly formed by rocks is the mortar-jetting method. Since the evaluation of effectiveness of the mortar-jetting method varies widely with engineers, for the sake of information, Fig. 5 shows the relationship between the stability of slopes after the completion of their construction and their aging. The locations for investigation were picked up from those which had unfavorable conditions. As a result of this investigation, it was found that the slope surface to which the mortar-jetting method had been applied was weathered during the period of 1 to 3 years after the completion of the protection, and the number of those slopes, which still retained the original surface condition immediately after the completion, accounted for about 33% of the total. The number of slopes which showed discoloring and the fall of mortar and rock chips accounted for 20% of the total in 3 to 4 years after the completion. Especially the mortar-jetting on those slope surfaces which were wet or used to cause freezing and melting was short-lived, and should be considered only as a temporary protective work.

3. Slope Failure Experiment Using Artificial Rain

(1) Object and Method of Experiment

As explained before, most of the cases of slope failure are caused

directly by the effects of water, i.e., rain water and seepage water. The following is the description of the slope failure experiment which was conducted on the full-scale model of a slope for the purpose of tracing the detailed phenomena of slope failure, thereby clarifying the methods of design and the construction of highly stable slopes, and the procedure for the prevention of slope failure.

The slope failure which is caused by rain often occurs at the cutting slope, resulting in serious problems in the control and maintenance of highways. Since it is difficult to make an experiment on a natural slope in situ, the experiment was made on an embankment constructed for the purpose of the experiment. It is also often reported that embankment slopes along highways showed failure when heavy rain of a total rainfall of more than 250 mm (approx.) fell on them.

The slope failure experiment using rain was conducted at the large-scale laboratory of the Public Works Research Institute of the Ministry of Construction.

In the experiment, a full-scale experimental embankment was constructed in the laboratory, and artificial rain was applied to the embankment for a long time until the slope showed a failure.

The experimental embankment was 4 m high and had a grade of 1 : 1, 5. The material used for the embankment was Kanto-loam (a kind of volcanic-ash soil) which had been dried and then had its moisture content adjusted. In the construction of the embankment, the soil was

filled up by applying light rolling compaction to the fill at an interval of 30 cm in thickness in order to ensure the evenness of compaction of the fill.

Inside the embankment, pore pressure gauges, earth pressure gauges and aluminum pipes were installed, while on the surface of the embankment, surface strain gauges are fitted to measure the shifting, contraction and elongation of the surface. Alminum pipes were used for casing pipes to insert RI moisture meter and density meter.

The embankment was constructed in the laboratory of 32 m wide and 36 m long and equipped with artificial rainfall equipment capable of making rains of any given strength.

The experiment was conducted one month after the completion of the embankment. In the experiment, rain of a certain strength was applied to the slope of the embankment, and measurements and observations were continuously made on the condition of percolation of rain water and also on the cracks and deformation that occurred on the embankment.

The strength of rainfall was set to 15 mm/hr in order to ensure that nearly the entire rainfall would seep into the embankment, and no surface erosion and scouring would occur as a result of the surface flow. Penetration resistances at various parts of the embankment also were measured by the static cone penetrometer three times, i.e., before the commencement of the rain experiment, after its completion, and in between, in order to investigate the effect of the rise in the moisture

content exercised upon the shearing strength of the soil. When the rainfall reached 200 mm, the slope began to develop cracks; and when the rainfall reached about 300 mm, it was observed that the cracks expanded and the slip gradually progressed. At the completion of the experiment, the slope failure as shown in the photo 1 occurred.

(2) Results of Experiment and Discussion on Slope Stabilization

Changes in moisture content inside the embankment which have been observed by means of the experiment are expressed in the degree of saturation as shown in Fig. 6. The distribution of saturation degree inside the embankment at the commencement of the rain is almost uniform throughout the embankment, except a slightly larger value shown at the foot of the embankment. It is clearly discernible that as the consecutive total rainfall increases, the embankment increases its degree of saturation gradually from the dried state. Since the experimental embankment has been constructed upon an impermeable foundation, the saturation degree at the lowest part of the embankment rises. Except this part, however, the saturation degree does not rise very high, especially the moisture content in the center part of the embankment does not rise easily, which fact should be noted as one of the characteristic features of the embankment.

Fig. 7 shows an example of the penetration resistance measured by a static cone penetrometer. At the same time, the values of saturation degree of the soil also are plotted into the figure so that the relationship between the penetration resistance and the saturation degree can be clear-

ly found out. The variations of these two values are fairly similar. It has also been ascertained that the pore pressure which is generated inside the embankment while the rain is being applied, will disappear quickly after the completion of the experiment, except from that part at the bottom of the embankment which has been saturated.

This experiment reproduced the fact that as rain is applied to the embankment and the moisture content of the embankment rises, the stability of the slope gradually deteriorates until it develops a slope failure. The causes for this failure are: (1) the increase in the weight of the soil which acts as force causing the slip, (2) the rise in pore pressure, which acts as a factor to reduce the resistance against the slip and (3) decreasing in the shearing strength of soils due to an increase in the moisture content of soils. According to the results of the experiment, however, it is considered that the increase in the weight of soil and the rise in pore pressure have not any significant effects upon the slip. It is concluded, therefore, that the decrease in the shearing strength of the soil is the governing factor of the failure. Consequently, the phenomenon of failure can be explained to some extent, if the relationship between the moisture content of unsaturated soil and the shearing strength of the soil can be clarified.

The phenomenon of the shear failure of unsaturated soil, however, has not been fully investigated and contains many points which are still unknown. Consequently, as a sort of an expedient, it was decided to

obtain the relationship between the moisture content and the shearing strength; and in a laboratory experiment, a close investigation was made concerning the relationship among the cone penetration resistance, moisture content and saturation degree in respect of compacted samples. After making sure, by a consolidated undrained triaxial compression test, that the value of ϕ_{cu} remains more or less 10° even if the value of moisture content fluctuates slightly, the cohesion was calculated from the penetration resistance on the assumption that the frictional angle would be 10° and by using the formula for calculating the bearing capacity of piles. The relationship between the cohesion thus obtained and the degree of saturation is shown in Fig. 8.

The above-mentioned relationship is used for the examination of the stability of a slope, which is explained below: after the saturation degree distribution of the soil at the respective stages of the rainfall had been obtained in the experiment, the embankment was sliced into several zones according to the values of saturation degree for the purpose of calculation. Fig. 9 shows the constants of the soil which have been determined according to those zones after the slicing of the embankment. This figure also contains safety factors which have been obtained by the calculation of circular slips under the conditions as shown in Fig. 9. These safety factors are the minimum factors at the centers of those circular slips.

By looking at the figures included in Fig. 9 one by one from the top

to the bottom, it is possible to find how safety factors decrease as a result of a rise in the moisture content. Fig. 10 shows the relationship between the safety factor against slope slip and the total rainfall. This figure shows a good fit to the actual phenomenon in which the safety factor sharply drops until the rainfall reaches 200 mm, and the slope failure occurs at a rainfall of 250 to 300 mm.

Since it is difficult to explain the phenomenon of slope failure caused by the action of seepage water, by means of the results of the shearing test alone, an investigation has been made by full-scale model experiment, and an explanation has been given on the stability against slip by means of the variances of moisture content and shearing strength. It has been also ascertained that ① the paving of the top face of the embankment will decrease the penetration of seepage water and considerably improve the stability of a slope at the time of rain, and ② installing a drain near the toe of the slope and tamping the slope will be highly effective in enhancing the stability of the slope.

4. Concluding Remarks

This paper consists of two parts, the first being a summary with discussion of actual investigations which have been carried out on various procedures of slope protection applied to the national highways under the control of the Ministry of Construction, and the second part being an examination of the effects of rain upon the stability of a slope by applying artificial rain to the slope of a full-scale model.

The actual investigations were carried out on those slopes which were judged comparatively dangerous, and on all the slopes of certain investigation sections, and were able to give data concerning the general trend of slope protection procedure which had been adopted in all parts of the country, stability of slopes by slope protection method, and the effect of water upon the stability.

In the model experiment, it was considered that the slope failure due to rain is caused by the decrease of shearing strength with the rise in moisture content, and this concept was examined with special emphasis on the penetration resistance and the degree of saturation. By this experiment, it has become possible to connect the moisture content of soil with the stability of a slope against slip. The results of the studies dealt with in this paper may be utilized beneficially as data necessary for the determination of the selection criteria of various procedures of slope protection.

Procedure for slope Protection	Ratio of each method to the total		State of slope							
			Stable slope		Local slope failure		Falling rock		Slope failure	
	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80
① Retaining-wall method	78		69		8		0		1	
② Stone-pitching method	21		20		1		0		0	
③ Concrete-pitching method	8		7				0		1	
④ Mortar-jetting method	3		1		2		0		0	
⑤ Concrete frame-work method	6		5		1		0		0	
⑥ Concrete block-pitching method	33		29		4		0		0	
⑦ Strip-sodding method	0		0		0		0		0	
⑧ Block-sodding method	57		56		1		0		0	
⑨ Seed-jetting method	68		80		8		0		0	
⑩ Seed-board method	12		10		1		0		1	
⑪ Pick-hole planting method	24		22		2		0		0	
⑫ Net method	0		0		0		0		0	
⑬ Falling rock guard fence method	1		0		0		0		1	
⑭ Other methods	0		0		0		0		0	
⑮ No treatment	32		22		7		3		0	
Total	363		321		35		3		4	

Fig. 1 Relationship between state of slope and procedure for slope protection (slope composed of soils)

Procedure for slope Protection	Ratio of each method to the total		State of slope							
			Stable slope		Local slope failure		Falling rock		Slope failure	
	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80
(1) Retaining-wall method	49		33		5		8		3	
(2) Stone-pitching method	1		1		0		0		0	
(3) Concrete-pitching method	2		0		1		0		1	
(4) Mortar-jetting method	63		56		4		3		0	
(5) Concrete frame-work method	4		3		0		0		1	
(6) Concrete block-pitching method	11		7		0		0		4	
(7) Strip-sodding method	0		0		0		0		0	
(8) Block-sodding method	0		0		0		0		0	
(9) Seed-jetting method	19		16		0		3		0	
(10) Seed-board method	0		0		0		0		0	
(11) Pick-hole planting method	4		4		0		0		0	
(12) Net-method	0		0		0		0		0	
(13) Falling rock guard fence method	0		0		0		0		0	
(14) Other methods	0		0		0		0		0	
(15) No treatment	16		12		1		2		1	
Total	169		132		11		16		10	

Fig. 2 Relationship between state of slope and procedure for slope protection (slope composed of soils and rocks)

Procedure for slope Protection	Ratio of each method to the total		State of slope									
			Stable slope		Local slope failure		Falling rock		Slope failure	
	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80
① Retaning-wall method	73		58		7		5		3	
② Stone-pitching method	4		3		1		0		0	
③ Concrete-pitching method	4		3		1		0		0	
④ Mortal-jetting method	251		233		11		6		1	
⑤ Concrete frame-work method	8		4		3		0		1	
⑥ Concrete block-pitching method	16		15		0		0		1	
⑦ Strip-sodding method	1		1		0		0		0	
⑧ Block-sodding method	0		0		0		0		0	
⑨ Seed-jetting method	31		25		2		4		0	
⑩ Seed-board method	7		3		2		1		1	
⑪ Pick-hole planting method	5		4		0		0		1	
⑫ Net method	34		30		3		1		0	
⑬ Falling rock guard fence method	17		14		1		1		1	
⑭ Other methods	2		1		0		1		0	
⑮ No treatment	380		279		24		64		13	
Total	833		673		55		83		22	

Fig. 3 Relationship between state of slope and procedure for slope protection (slope composed of rocks)

Slope condition with respect to water	Ratio of each method to the total		State of slope									
			Stable slope		Local Slope failure		Falling rock		Slope failure	
	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80	Q'ty	Percent 20 40 60 80
Dry	1058		840		159		18		41	
Wet	108		48		43		12		5	
Surface water	85		63		0		4		18	
Spring water	27		17		5		1		4	
Total	1278		968		207		35		68	

Fig. 4 Relationship between state of slope and slope condition with respect to water

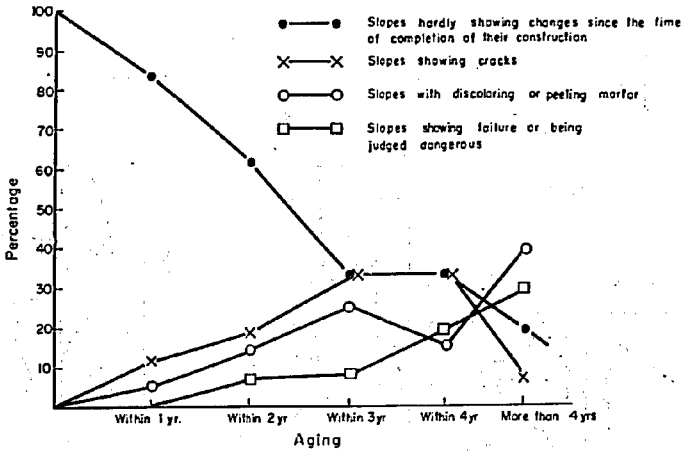


Fig. 5 Relationship between stability of mortar and aging

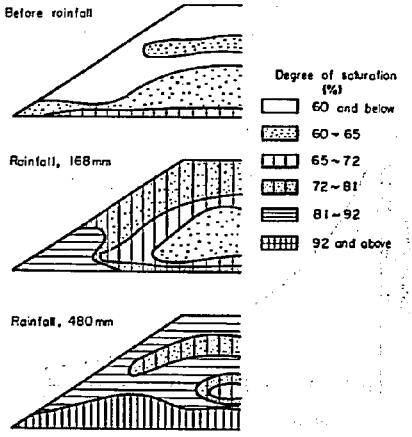


Fig. 6 Change of saturation degree in a fill

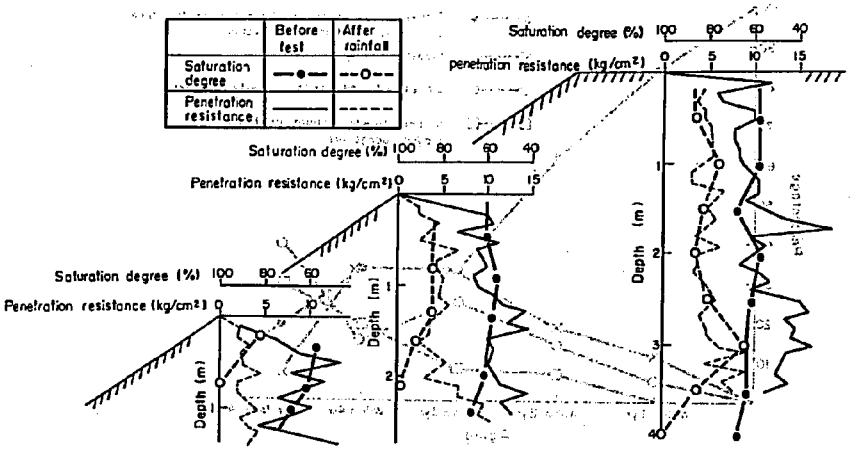


Fig. 7 Relationship between saturation degree and penetration resistance

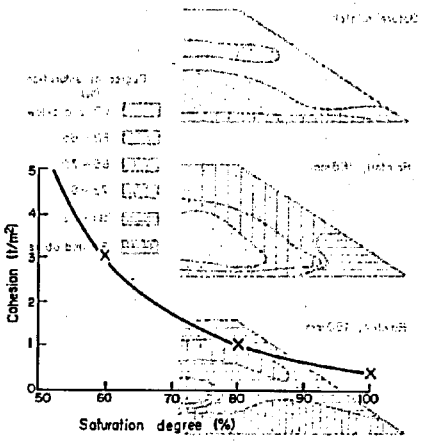
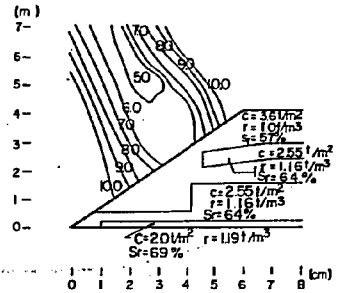
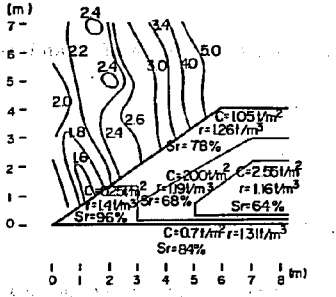


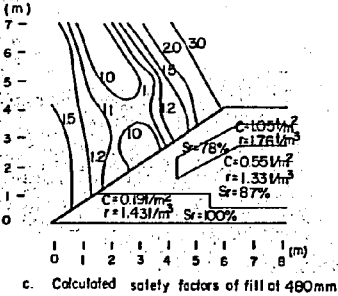
Fig. 8. Relationship between saturation degree and cohesion



a. Calculated safety factors of fill before rainfall



b Calculated safety factors of fill at 168mm rainfall



c. Calculated safety factors of fill at 480mm rainfall

Fig. 9 Calculated safety factors of fill

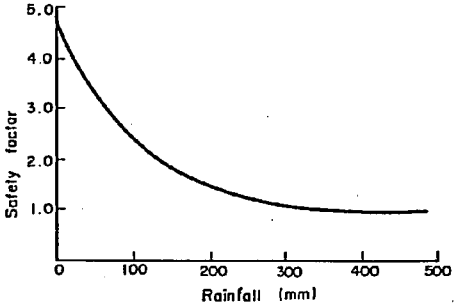


Fig. 10 Relationship between rainfall and safety factor

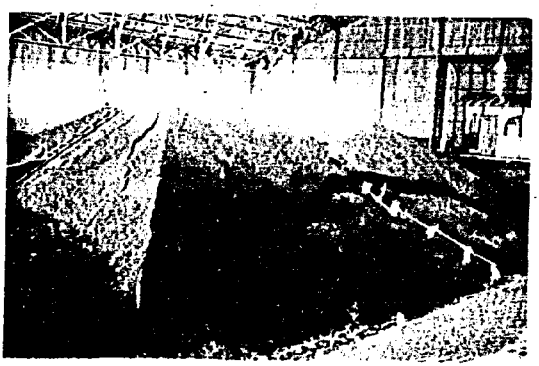


Photo 1 Failure of model slope

Actual Traffic Loadings on Highway Bridges and Stress Levels in Bridge Members

Charges de trafic actuel sur ponts-routes et niveaux de sollicitations dans les membrures des ponts

Gegenwärtige Verkehrslasten auf Strassenbrücken und Beanspruchungs-Niveau in Brückenelementen

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1. Introduction

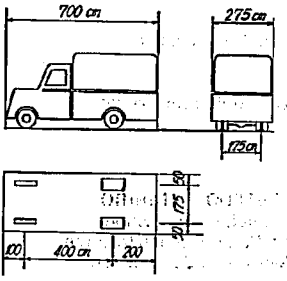
The design live load specified in the design specifications for highway bridges in Japan, as shown in Fig. 1 and Fig. 2, is fundamentally based on a 20 ton truck and traffic rows composed of a 20 ton truck and 15 ton trucks. The T-load in Fig. 1 is used for design of slab and floor system, while the L-load shown in Fig. 2 is used for design of main girder and main truss. These loads have been used since 1956, but the increase in both weight and number of trucks in these years is so remarkable that the loads shown in the current specifications have reached the state which may not be necessarily appropriate in contrast with the actual traffic situation. Besides, there exist a great number of old bridges which were designed for smaller load, than that of current specifications, and the safety of those bridges under the present heavy load is posing a problem.

In Japan, various surveys of actual traffic loadings and stress levels in highway bridge members are being carried out for the purpose of obtaining the data in re-examination of design live load, and also for control of safety of existing bridges.

2. Stress levels in bridge members[1)]

Stress levels caused in the main girder of 18 plate girder bridges (with span length of 20 to 60 m) and for each member of a truss bridge (with span length of 80 m) under the ordinary traffic loading were measured.

In measuring, the device enabling to automatically count up the frequency by each stress level separately was used. This device is composed of the strain detector in which differential transformer is used and recording device which classifies the detected strain separately by four class levels, and count up the respective frequency.



Total weight 20 ton
Weight of front wheel 2 ton
Weight of rear wheel 8 ton

Fig. 1 T - load

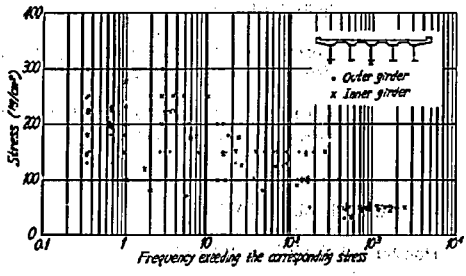


Fig. 3 Stress frequency (Plate girder bridge)

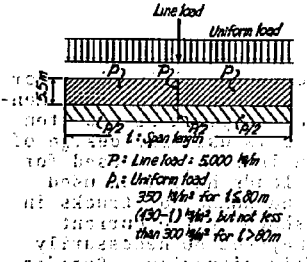


Fig. 2 L - load

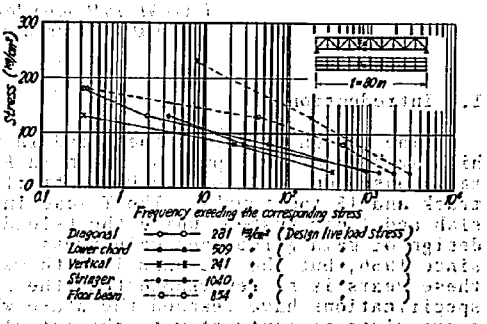


Fig. 4 Stress frequency (Truss bridge)

Type of Vehicles	Passenger Car	Bus	Small Size Truck (Gross Weight of 25 ton or less)	Large Size Truck (Gross Weight of 26 ton or more)	Total
Ratio (%)	429	181	15	125	100

Table 1. Percentage of various vehicles

The bridges for which the survey was made were selected from those which are located on the main national highways served for heavy traffic. The traffic amount per 24 hours at the survey point counts 10,000 to 40,000 vehicles, and the average traffic volume for the entire survey points was about 20,000 vechicles; the percentage of various types of vehicles being as shown in Table 1.

The measurement was curried out for 3 to 5 consecutive days. The result of measurements for plate girder bridges and a truss bridge are indicated in Fig. 3 and Fig. 4, respectively. In those

figures, the frequency at each stress level is shown per one day (24 hours). The relation between the range of the ratio of stress level measured in girder bridges to the calculated stress due to the current design live load (L-load) and the frequency caused per the traffic volume of 10,000 vehicles is shown in Fig. 5.

In the case of truss bridge shown in Fig. 4, the values of measured stress of floor system is greater than those of truss members. However, the ratio of measured stress level to design live load stress for floor system is smaller than that of truss members. This is supposed to be due to the fact the reinforced concrete slab supported by the stringers and floor beams may take charge a portion of the function of stringers or floor beams. Such a function of reinforced concrete slab was not considered in the design calculation.

According to the fact mentioned above, it is known that the live load stress caused in each member of bridge under ordinary traffic load is some 300 kg/cm^2 at most, which is fairly at lower level as compared with the calculated stress level due to design live load.

3. Actual status of wheel load[2)3)]

For the purpose of confirming the actual weight of wheel load of vehicle having great influence on the slab and floor system, measurement of weight of wheel load was carried out at 34 points on national highways in Japan.

The device used for the measurement is composed of weighing part and recording part. The weighing apparatus is the weighing meter consisting of a sheet of steel plate and four loadcells

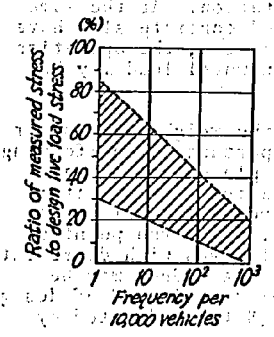


Fig. 5 Ratio of measured stress to design live load stress

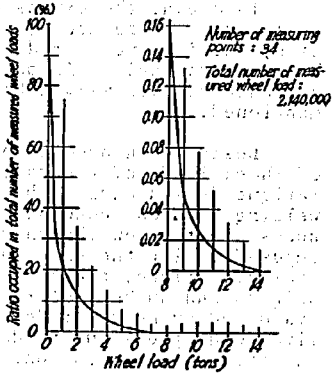


Fig. 6 Cumulative frequency distribution of wheel load

supporting the plate on its four corners. The weighing meter is installed in the reinforced concrete pit in such a way that the loading plate makes a part of the road surface covering the pit, so that the weight of wheel passing over the plate is detected without stopping the vehicles. The recording part has the function of classifying counting automatically each frequency of the detected weight by every 12 classes of level.

The survey points were selected from among the points located on main national highway, and besides, where the traffic is especially crowded. The traffic volume per 24 hours for the lane where the wheel load was measured indicated about 5,000 vehicles at point where the traffic volume was the least, and about 28,000 vehicles where the traffic volume was the most.

The measurement was carried out for 1 to 7 days for each point. It was made clear, as the result of the survey, that the frequency by different level of wheel load showed considerable difference according to the survey point, but on the same point, approximately the same frequency distribution was observed.

Fig. 6 represents the frequency measured by various weight of wheel load. As known from Fig. 6, although about 80% of the whole number of wheel load measured is one ton or less, and the number of heavier wheel load decreases rapidly, the fact that the number of wheel load exceeding design wheel load of 8 ton (which is approximately the same as the wheel load of HS20 of AASHO, see Fig. 1) amounts to 0.16% of whole number of wheel load, namely, that there exist about 32 vehicles of which wheel load exceeds design wheel load of 8 ton, per 10,000 vehicles, may not be ignored from the point of view of design of new bridges or maintenance and safety of the existing bridges. The reason of the existence of wheel loads heavier than design wheel load is due, needless to say, to the operation of large size trucks which are loaded more than the statutory loading limit.

In Japan, the expansion of truck size has been advanced in these years to reduce the cost of transportation. At the same time, the instances of damages of reinforced concrete slab have become remarkable. As one of the causes thereof, the operation of vehicles with weight heavier than design wheel load may be mentioned.

Besides, on account of the fact that the weight and number of wheel loads of trucks increased, the importance of performing fatigue design came to be claimed on steel plate deck where welding is used abundantly. In applying fatigue design, it is necessary to know the value and number of repetition of stress imposed during the expected life of bridge. From the point of view of fatigue, the effect of random wheel loads with different weights and numbers of repetition on steel plate deck may be evaluated by means of "equivalent number of repetition" of design live load stress due to design wheel load (8 ton), denoted by the following expression:

$$N_{eq} = K_1 \cdot K_2 \cdot N_t \qquad (1)$$

Where N_{eq} : Equivalent number of repetition of design live load stress

N_t : Number of total passing wheel loads during the expected life of the bridge

K_1, K_2: Coefficients

K_1 in Equation (1) is the coefficient used for converting the wheel load with certain weight into design wheel load of 8 ton, which may be expressed as Equation (2):

$$K_1 = \sum P(T) \left(\frac{T}{8}\right)^{\frac{1}{k}} \cdot \Delta T \qquad (2)$$

Where T : Weight of wheel load

P(T): Probability of occurrence of T

k : Coefficient determined from S-N diagram (the slope of S-N diagram in logarithmic sclae.)

The position where wheel loads pass in certain traffic lane is not stationary. Fig. 7, illustrated as an example, shows that the distribution of passing positions of wheel loads in a traffic lane may be regarded as a normal distribution with the standard deviation of approximately 35 cm. Fig. 7 was obtained as the result of observation at 20 different points on national highways with two or four lanes, all over Japan. K_2 in Equation (1) is the coefficient to be multiplied with the number of repetition determined on the assumption that the all wheel loads pass on a fixed position, and is given from the following equation:

$$K_2 = \sum \left(\frac{R(x)}{R_0}\right)^{\frac{1}{k}} \cdot P(x) \cdot \Delta(x) \qquad (3)$$

Where Ro : Design live load stress

R(x): Stress of member under the design wheel load passing on certain position, x

P(x): Probability of occurrence of R(x)

The value of K_2 for the longitudinal rib of steel plate deck which is commonly used in Japan, is approximately 0.2.

P(T) in Equation (2) is given by Fig. 6, and P(x) in Equation (3), by Fig. 7. Such data, actually observed concerning the weight and the passing position of wheel load are indispensable for the fatigue design.

4. Actual status of vehicle row load[2),3)]

L-load, shown in Fig. 2, depends upon the vehicle row load such as shown in Fig. 8.

In order to ascertain the magnitude of actual vehicle row

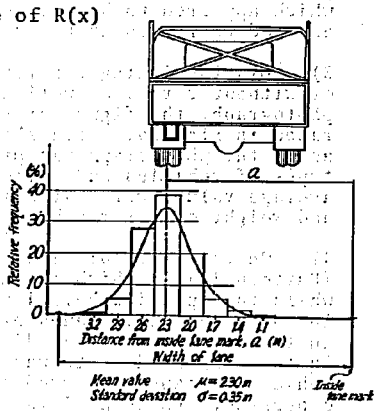


Fig. 7 Distribution of passing position of wheel

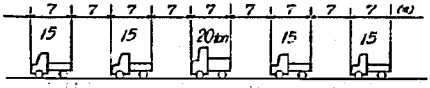


Fig. 8 Basic traffic pattern of L - load

Number of lanes	Number of points
2	25
4	40
6	8
	73

Table 2. Number of traffic pattern survey points

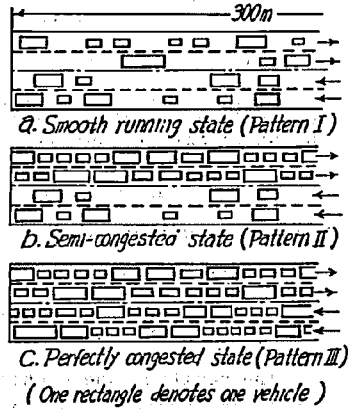


Fig. 9 Traffic patterns

load in comparison with the above design L-load, a survey on traffic pattern was carried out in the following procedure:

1) 73 survey points were selected (Table 2). These points are the places on the main national highways where the traffic is crowded and apt to be congested including a lot of large size vehicles.

2) At each survey point, traffic pattern extending over 300 m on the roadway was photographed intending to take as many heavy trucks as possible in the overlooked picture. Ten times of the photographing was carried out, and selected one case of pictures which appeared to be most congested in traffic, as they were assumed to represent the maximum of vehicle row load at the point.

3) Distinguishing the loading condition of each vehicle with or without cargo, as well as the type of each vehicle from the photograph, the figure of traffic pattern (Pattern I), as illustrated in Fig. 9a, which shows exactly same arrangement of vehicles as on the photograph was prepared. For the weight of different types of vehicles and different loading conditions, corresponding average values were established referring to the table of size and weight of motor vehicles in Japan.

4) On the basis of Pattern I, hypothetic pattern such as Fig. 9b (Pattern II) in which the half of traffic lanes is congested, while the opposite half lanes is smooth, and Fig. 9c (Pattern III) in which the whole lanes are perfectly congested, were prepared. In this case, the clear distance between two consective vehicles in the congested state was taken as 1 m, with reference to the vehicle distance observed on highways actually at the time of congested. Pattern II is the state which is usually observed in Japan, and Pattern III is the state which is very rarely observed.

5) In regard to Pattern I, II and III, the maximum bending moment Mo and maximum shear force So, which are imposed in a simply supported girder under each loading pattern, were calculated by means of electronic computer. In this case, Mo and So were obtained for nine cases of the span length of 20, 40, 60, 80, 100, 150, 200, 250 and 300 meters.

6) M/MD and So/SD were calculated, which are the ratio of Mo and So due to "actual vehicle load" obtained in 5), to beinding moment MD and shear force SD due to "design load" showed in Fig. 2.

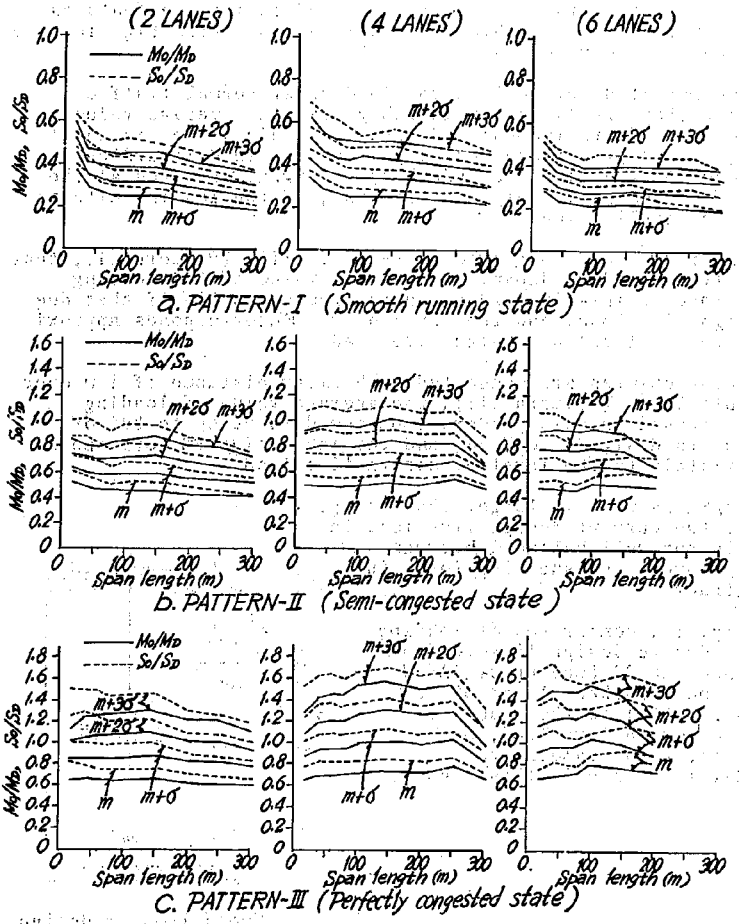


Fig. 10 Ratio of measured value to calculated value due to design load

The values of M_o/M_D and S_o/S_D thus obtained for all of observed points were gathered up for different number of lanes, and a histogram of M_o/M_D and S_o/S_D was drawn. The histogram showed that the form of distribution of M_o/M_D and S_o/S_D may be regarded as the normal distribution. Thereupon, the mean value "m" and the standard deviation "σ" of the values of M_o/M_D and S_o/S_D for different numbers of traffic lane and for different span lengths were calculated, then the relation between different expected magnitudes of the vehicle row load and their frequencies was presumed.

Fig. 10 was obtained in this way, indicating the features of magnitude of actual vehicles load in Japan. From Fig. 10, the following facts may be understood:

1) The magnitude of vehicle row load under normal traffic condition (Pattern I) is considerably low, the average value of the loading effects (moment and shear) in main girder is equal to 30%, at most, of those due to design live load regardless of the number of traffic lane and of span length, and even at the level of $(m+3\sigma)$, it is only 60%, at most, of the loading effects due to design live load.

2) Under the semi-congested state of traffic (Pattern II), that is usally occured in Japan, the average value of the loading effect imposed in main girder ranges from 40 to 60% of that due to design live load, and the level of $(m+3\sigma)$,corresponds approximately to the level of design live load.

3) When vehicles are congested with clear distance of 1 m over the whole lanes (Pattern III), the average value of loading effect imposed in main girder is presumed to be 60 to 80% of that due to design live load, and the level of $(m+\sigma)$ corresponds approximately to the level of design live load.

4) S_o/S_D is usually larger than M_o/M_D. Namely, the current design live load in Japan is not well-balanced between bending moment and shearing force in main girder.

Fig. 11, comparison of loading effects due to design live load of well-known HS 20 Loading of AASHO (1969) and "Brücken Klasse 60" of DIN 1072 (1967) to those due to Japanese design live load, may facilitate understanding the consideration made in this report.

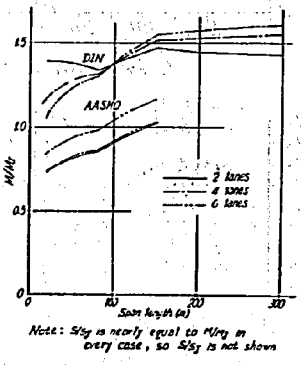


Fig. 11 Comparison of design loading effect in various countries

5. Conclusion

The following conclusion may be introduced concerning the load on highway bridges in Japan, on the basis of the results measured on the stress levels in bridge members, and on the wheel load and vehicle row load:

1) The live load stress imposed in main girder and main truss under usual traffic condition is 300 kg/cm^2 or so at most, which is considerably small as compared with the calculated design live load stress.

2) Vehicle row load under normal traffic condition is in considerably lower level as compared with the current design vehicle row load, but under the state in which the whole lanes are perfectly congested, there is a little chance that the vehicle row load may reach to the level of design load. Considering the fact that semi-congested state of traffic in which the half of traffic lanes are congested while the opposite half lanes are smooth is usually observed wherever in Japan, it seems to be suitable to determine the design load for main girder on the basis of such loading condition.

3) The relation between the weight and frequency of vehicle wheel load shows fairly big divergence according to area, but nearly constant trend on the same point. Generally speaking, the actual condition of wheel load is that it has no such allowance to design load as is seen in the vehicle row load, and the existence of such heavy wheel load exceeding design wheel load is amounting to a number which may not be ignored.

4) Since vehicle wheel load (single vehicle load) is in the severer condition than in the case of vehicle row load in comparison with the current design load, considerable unbalance of load carring capacity between main girder and floor system has been arisen. In order to design a bridge of which each member is well balanced in strength, it will be necessary, so long as the current design load is employed, to use larger safety factor for the floor system than that for main girder.

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Summary

As the result of our surveys of wheel load, row load of vehicles and of stress in bridge members, it was made clear that the magnitude of vehicle row load or corresponding stress in main girder has considerable allowance in comparison with the current design live load and corresponding stress, under any traffic loading condition except perfectly congested states, while the single vehicle load sometimes exceeds the design load and poses a problem in relation to the safety of slab and floor system.

On the Revised Wind Resistant Design Criteria for the Proposed Honshu-Shikoku Bridges

by

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To be presented at the Fourth Joint Meeting, UJNR
May 17-19, 1972
Washington, D.C.

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On the Revised Wind Resistant Design Criteria for the Proposed Honshu-Shikoku Bridges

by

T. Okubo

N. Narita

Summary

The gigantic project constructing long span bridges to connect the Honshu (the mainland of Japan) with the Shikoku has been expedited after promotion of the Honshu-Shikoku Bridge Authority in July, 1970. The wind resistant design criteria (1967)[1], which was formulated in 1967 with due regard to the type and scale of the bridges, and the social importance of the project, and has been applied to the contemplated design, was revised by using the recent outcome in the field.

This paper introduces the main features of the revised wind resistant design criteria for the proposed Honshu-Shikoku Bridges (1972)[2] and the related problems. The significant points which have been revised are as follows:

(1) To clarify the design procedure and the constitution of articles in the criteria.

(2) To reexamine the basic wind speed in due consideration of the recent wind data recorded in the Seto Inland Sea, and to set the basic wind speed for the respective zone or route.

(3) To determine the modification factors for converting a basic wind speed into the design wind speed, where analytical method of gust response is tentatively adopted.

(4) To revise the concrete values of drag coefficients, structural damping, wind inclination by using the recent data of observation.

It goes without saying that some of the problems are left unsolved. A more rational design criteria must be investigated by our serious

consideration.

Wind Resistant Design Criteria (1967) and its problems

The wind resistant design of an ordinary bridge in Japan is based upon the Design Specification of Highway Bridges[3] or the Design Standard of Steel Railway Bridges[4]. When the main span length of a suspension bridge exceeds several hundred meters, the dynamic behavior of the structure becomes important due to its flexibility. Moreover, the dimension of main elements can be determined by wind load. These matters, having nothing to do with short span bridges, are the reason why the great importance is attached to the wind resistant design of a long span suspension bridge in Japan located in the strongest wind area in the world.

In these circumstances the wind resistant design of the proposed Honshu-Shikoku Bridges has been based upon the Wind Resistant Design Criteria (1967), which was formulated by the ad hoc committee in the Japan Society of Civil Engineers. The Criteria is characterized in the following points;

(1) The design wind speed shall be assessed according to the topography of construction site, and the altitude and scale of the structure.

(2) Aerodynamic coefficients considered the structural composition are given for reference about the static design of the structure.

(3) It shall be the duty of the designer to perform a wind tunnel test in order to confirm the drag coefficient and to predict the critical wind speed for the self-excited oscillation of the structure.

(4) A design standard for the critical wind speed of elastic instability and self-excited oscillation is prescribed so that the sufficient safety shall be kept for the structure.

(5) It shall be the duty of the designer to take enough care of wind

stability not only after completion but also on each stage of construction.

Though the Wind Resistant Design Criteria (1967) includes the remarkable progress compared with the preceding design criteria, and it has been effective to designers to recognize the importance of wind stability problems, it is confined to give general regulation for the wind resistant design of long span suspension bridges. Main problems in the Criteria (1967) are as follows;

(1) Applicability of the criteria to other type bridges i.e. cable stayed bridges and cantilever bridges.

(2) Possibility of estimation of reliable basic wind speed at the construction site by the use of wind record.

(3) Reliability of the modification factors to basic wind speed which take into the effect of horizontal or vertical length of the structures.

(4) Relation between the wind inclination and the critical wind speed for self-excited oscillation.

(5) Rationality of safety factor for self-excited oscillation and elastic instability of the structure.

(6) Quantitative estimation of vortex-excited oscillation and its standardization in the criteria.

(7) Applicability of test results in wind tunnel testing to real structures.

The problems above mentioned seem to be difficult to be solved in a short period of time. In the following chapter the authors introduce the main items included in the revised design criteria (1972).

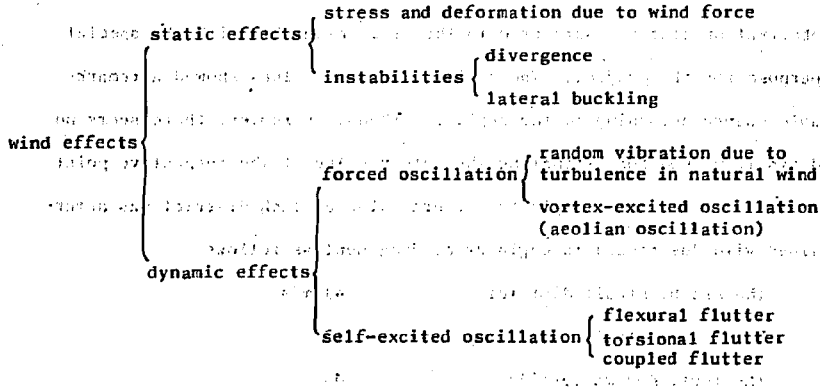
Wind Resistant Design Criteria (1972)

(Scope of application)

The Wind Resistant Design Criteria (1967) had no definite scope of application so that it gave the general regulation for wind resistant design. In the revised criteria it is confined to a truss-stiffened suspension bridge with the main span length of more than 500 meters. But it seems necessary to apply this criteria to a bridge being rich in flexibility.

(Procedure of wind resistant design)

As stated before it is often that the main members of a long span suspension bridge are determined by the wind resistant design. Accordingly the conscientious design is desired. Wind effects on a structure can be classified as follows;



Generally speaking, there is a fair chance for occurrence of some of wind effects in every structure, so that its wind resistibility is governed by the lowest critical wind speed against the phenomena. Self-excited oscillation, vortex-excited oscillation, and lateral buckling are the principal phenomena which shall be examined in the design work. But, it is not always possible to pursue theoretically the wind effects

on a long span suspension bridge on account of its unstreamlined shape, and the inspection of wind stability by wind tunnel tests becomes indispensable. Therefore, the wind resistant design shall be performed as shown in Fig. 2.1, that is, the proposed design by the static conditions shall be inspected through wind tunnel tests, and, when it shows insufficient aerodynamic characteristics, the design shall be modified to satisfy the design criteria.

(The basic wind speed)

The basic wind speed, defined by 10 minute mean wind speed at 10 meters above ground or sea level shall be determined by taking account of the wind record at the site, the life of structure, and the return period of wind speed. The return values were estimated by the use of recent wind record gathered at the governmental weather bureaus and the observation stations scattered in the Seto Inland Sea with the special purpose for the project. The estimated return values showed a remarkable change according to the applied methods. Moreover, there seems no definite method for estimating the return value at the respective point above the sea. Accordingly the return value at each district was determined with due regard to engineering judgement as follows;

The Akashi strait district	43 m/s
The Naruto strait district	50
The route Kojima-Sakaide	43
The route Onomichi-Imabari	40

(Design wind speed)

The basic wind speed (V_{10}) shall be converted to the design wind speed (V_D) by multiplying the modification factors;

$V_D = V_{10} \cdot S_1 \cdot S_2$ for suspended structures, cables, and hangers.

$V_D = V_{10} \cdot S_1 \cdot S_3$ for towers.

S_1, height factor, can be calculated by assuming that the vertical distribution of wind follows the power law. The standard value of power index is assumed to be 1/8.

The wind speed fluctuates with time as well as place. This fluctuation causes random forced vibration of the structure and may result in some magnification over the static response. This kind of problems - gust response problems - have recently been studied and they are tentatively adopted in some of building design codes. In the design criteria (1967) the fluctuation of wind was schematically simplified and the modification factor was derived by choosing the averaging time at which the horizontal scale of turbulence is equal to the horizontal length of the structure. So, the dynamic characteristic of the structure was not considered.

Gust response was analysed for both suspended systems and tower systems in order to determine concrete values of S_2 and S_3. The modification factors derived from the analysis ranged between 1.3 and 1.5.

In the revised design criteria (1972) effects of fluctuating wind are taken into account, but frequency response of the bridge is neglected as there has been no evidence of lateral vibration of suspension bridge structure due to gust loading.

(Design wind loads).

In the static design only the drag force shall be applied. For suspended structures following drag coefficients (C_D) can be used;

$C_D = 1.1 \cdot \sqrt{\frac{3}{2\phi}}$ for a single deck structure.

$C_D = 1.2 \cdot \sqrt{\frac{3}{2\phi}}$ for a double deck structure.

where ϕ is the solidity factor of the suspended structure. These expressions were empirically derived by wind tunnel tests.

(Lateral buckling)

In some occasion, the lateral buckling of the suspended structure is observed in wind tunnel test when the torsional rigidities are insufficient. The safety factor for this phenomenon shall be more than 1.7 times of design wind load as it is catastrophic to the structure.

(Structural damping)

Quantitative estimation of structural damping through theoretical analysis is impossible, and it depends upon the cumulation of field observations.

The structural damping of the whole system is decreased from 0.05 to 0.03 and the free-standing tower from 0.02 to 0.01 after due consideration of the observed data.

(Wind inclination)

The wind inclination is an important factor for aerodynamic stability of suspension bridges. The vertical inclination angle decreases with the mean wind speed. In the revised criteria (1972), three times of standard deviation of vertical inclination averaged over 30 seconds against wind speed is assumed to be the maximum of wind inclination. The averaging time of 30 seconds corresponds to some five times of the natural period of the lowest torsional oscillation in a suspension bridge with main span length of 1,500 meters.

(Critical wind speed of self-excited oscillation)

As the dynamic instability is included in self-excited oscillation, the thoughtful care is necessary. Especially in the case of coupled flutter, the oscillation of the structure grows so rapidly that the bridge may be broken in a comparatively short time. Horizontal length factor S_2 takes the effect of lateral bending into consideration and it

is not directly concerned with the self-excited oscillation. In the revised criteria (1972) the safety factor of the critical wind speed is determined as 1.3 against the basic wind speed multiplied with the height factor S_1.

Topics for further investigation

So that the more rational design criteria against wind effects is established many problems should be solved. Followings may be the principal topics;

(1) To establish the standard method for determining the return value of wind speed at a construction site.

(2) To establish the rational wind loads on a structure by taking account of wind fluctuation.

(3) To establish wind tunnel technique in which the characteristics of natural wind are simulated.

(4) To establish the rational safety factor for wind resistant design.

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A Résumé of the Aerodynamic Investigations

for the Kanmon Bridge

by

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Introduction

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Summary

The Kanmon Bridge is a truss stiffened suspension bridge, 1068 m between anchorages with a 712 m main span, 178 m side spans. The bridge is designed to support six lanes of highway traffic, and its capacity is 89,200 vehicles a day. A comprehensive investigation into the wind resistant design of the superstructure has been carried out because of the topographical condition at the construction site vulnerable to be attacked by such strong winds as typhoons and seasonal winds.

Described herein is the wind resistant design progress of structures including determination of design wind speed, the related wind tunnel tests on topographical configuration, the wind stability tests on suspended structures and towers, and a vibration test of the free-standing tower. Considerable numbers of anemometers and accelerometers are planned to be installed after completion of the bridge in the fall of 1973.

Introduction

The Kanmon Highway Tunnel, opened to the public in 1958, has been the trunk road connecting the Honshu to Kyushu Island. The traffic volume at the tunnel is estimated to exceed its capacity of 21,500 vehicles a day in 1972. In these circumstances the Ministry of Construction, Japan, decided to construct a new trunk line and adopted a suspension bridge for a new public service after comparison of the cost and technical feasibility between a suspension bridge and a submarine tunnel.

The wind problems play the most important role in the design and construction work of a long span suspension bridge in Japan which is attacked by typhoons and severe seasonal winds every year. The researches on this subject after the collapse of the Tacoma Narrows Bridge by gale in 1940 have shown the necessity of analysis of not only the bending and torsional rigidity of structure but also the aerodynamic characteristics of the stiffening structures. The authors point out that the major factors which should be taken into account in the wind resistant design of a long span bridge are (i) design wind speeds, (ii) aerodynamic characteristics of the structure, and (iii) rigidities of the structure.

The Structure Section of the Public Works Research Institute has been entrusted with the wind resistant design problems concerning the Kanmon Bridge on behalf of the Japan Highway Public Corporation since its preliminary design. The studies conducted by the Section include the followings:

(i) Wind tunnel tests on topographical models relating to the determination of design wind speed (1967)*

(ii) Determination of the design wind speed (1967)

(iii) Wind tunnel tests on the contemplated designs with truss-type stiffening girders (1967-1972)

(iv) Wind tunnel tests on the contemplated designs with box-type stiffening girders (1967-1968)

(v) Wind tunnel tests on the main tower at its erection stage (1968-1969)

(vi) Wind tunnel tests on the temporary structures to be used at construction stages (1969)

(vii) Field vibration tests on the free-standing tower (1971)**

* The test was carried out at the Meteorological Research Institute.
** The test was carried out in co-operation with the Earthquake Resistant Structure Section, P.W.R.I.

Estimation of basic wind speed

The wind records gathered from thirteen observation posts are listed in Table 1. No modification of records due to the difference of instruments was made by assuming that the effect of instrument on records of yearly maximum wind speed is negligibly small. The instantaneous wind speeds were converted to the 10 minute mean winds by multiplying the constant of 0.66, and the wind records prior to 1940, averaged over 20 minutes, were converted to 10 minute mean winds by multiplying the constant of 1.20. The gradient and level of regression line obtained from the modified records indicated that the longer the observation period is, the larger the predicted wind speed becomes. It means that the length of observation period should be unified. Therefore, two groups of wind records were chosen so that the one has the observation period of 5 years and the other 13 years. For each group were selected the five topographical factors having strong correlation to the estimated return values. The regression equations for R=5 and 20 can be written as

$$V_5 = -9.02+0.46X_1+0.063X_2+1.235X_3+8.128X_4+0.008X_5 \quad \cdots\cdots (1)$$
$$V_{20} = -16.46+1.048X_1+0.074X_2+1.855X_3+11.263X_4+0.011X_5$$

where R: return period, V_5, V_{20}: predicted wind speed for R=5 and 20 respectively, X_1: rate of the sea area within the area circled with a radius of 5 km (2.4), X_2: opening of the area circled with a radius of 30 km (185), X_3: the shortest distance to the reference point (1.0), X_4: the rate of convergence within the area surrounded by the circle with a radius of 15 km (1.5), and X_5: difference of the altitude between the highest point and the reference point within the area circled with a radius of 8 km (392). The values in the parentheses indicate the topographical values for R=100 estimated at the center of construction site. The return value for R=100 was obtained from the equations as 25.5 m/s assuming that the yearly maximum value satisfy the double exponential function. It was remodified by the records of more than 40 years at the Shimonoseki Observatory located about two kilometers apart from the construction site under the assumption that the return value varies at the same manner with that of the observatory. As the result of calculation the return value became 29.8±5 m/s including control

limit. When we choose three topographical factors the value becomes 31.7±4 m/s. Finally the basic wind speed was predicted as 35 m/s at the construction site.

Vertical profile of wind

The Kanmon Strait, hilling up to Mt. Hinoyama (268m) on the north and Mt. Kojoyama (175m) on the south, has the narrowest width of 700m, where the strong convergence of wind is expected. It means that the vertical profile of wind does not follow an usual power law. The vertical wind distribution was measured by wind tunnel tests on topographical models with scales of 1/3,000 and 1/10,000. It was also observed by using four anemometers fitted at an electric transmission tower near the site at heights 24.7, 34.6, 52.2 and 69.7 m above the ground. It was found from the experiments and observation that the vertical profile of wind at the strait does not follow a power law, and that the wind speed at the center of the channel is a little lower than that of the coast, but the wind speed at altitudes more than about 60 m is higher than the general flow of the tunnel. It is considered to be the effect of wind convergence by the strait. Vertical profile of the south-western wind was calculated by the use of test results because the wind from that direction was considered to give the most severe effect on the bridge. The modification factors to the stiffening girder and cable and/or hanger were calculated as 1.45 and 1.54 respectively. The power index of the vertical profile at the tower position was assumed to be 1/5 from the tests, and the magnification factor became 1.55. Modification factors due to horizontal and/or vertical length of structure are 1.09, 1.09 and 1.17 for cable, stiffening girder and tower in accordance with the design criteria. Finally the design wind speeds were determined as 53.5, 59.0 and 63.5 m/s.

Wind tunnel tests

In connection with any project of a major suspension bridge wind tunnel tests should be performed. These are indispensable because static and dynamic effects of wind, though they play important role in the

design stage compared with other loads, can not easily be predicted prior to the tests. Even a small change of the cross sectional shape may sometimes affects considerably the result. The wind tunnel tests carried out for the Kanmon Bridge are as follows:

(i) Wind tunnel tests on the truss-type girders,

(ii) Wind tunnel tests on the box-type girders, and

(iii) Wind tunnel tests on the main tower.

The tests were performed at 3.0 x 1.8 m wind tunnel at the Structure Section, Public Works Research Institute, Ministry of Construction. In the test (i) and (ii) the section model techniques have been applied, and such experiments, contrary to full model tests, we are relatively free in regard to model law and cost, are well suited for parallel tests during the preparation of design work. Experience has shown, moreover, that the section model technique will give enough information according to the comparison of tests results on full shape model carried out, for example, at the National Physical Laboratory, England and the University of Washington, U.S.A. In the test (iii) a rigid model elastically supported by gimbals at the tower base was used. It is because the damping capacity small enough for the real structure, say 0.02 in logarithmic decrement, may not be expected in the aeroelastic model.

Wind-tunnel tests on girders

In respect to girders the tests have been performed on 21 cross section designs as shown in Table 2. The major purpose of the tests was to find a model satisfying the provisions of the design criteria in which it is prescribed that the lowest wind speed for self-excited oscillation should not be less than 1.2 times of the design wind speed within the range of incidental angle of wind $-5<\alpha<5$ degrees. The test was started with the truss girder T-1, which had stiffening trusses of 6.0 m in height and distance of 29.0 m. The fundamental natural frequencies were calculated as 0.403 and 0.147 c/s for fundamental symmetric torsional and vertical bending mode respectively, and so reduced wind

speed V/NB (V: the lowest critical wind speed for the predicted catastrophic oscillation, N: natural frequency of torsion, and B: distance between stiffening trusses) must be not less than 5.53. As indicated in Fig. 1.1 wind stability of the model was not satisfactory. In order to improve the aerodynamic characteristics modifications were made, namely, (i) to increase the opening on the deck (T-2, T-4, and T-6), and (ii) to provide cross beams between slab and stringers (T-3 and T-5), but those were in vain. As a drastic reform of the cross section the truss height was increased to 9.0 m (T-7), so that the fundamental torsional frequency of the prototype was increased to 0.454 cycles per second. But, the aerodynamic characteristics seems not to have been refined so much as shown in Fig. 1.1. For this reason it was considered that wind disturbance due to the deck as well as the main chords of truss caused the instability. In models T-8 to T-17 the influence of the relative height of the deck to wind stability examined. These tests brought about a desired result. When the deck is put in the wake of the upper chord the wind characteristics at high angle of attack seems to be improved. The model T-10 was chosen as the best in relation to the design of the real structure (Fig. 1.2, and Fig. 1.3). In due course of time box-type girders were proposed as comparative design to truss girders. A box-type girder, complete break-away from conventional truss girder, has the low wind drag resulting from its shape and enables us to do the rational design. The original design of box-type girder had the width of 31.0 m and the maximum depth of 3.2 m (B-1). Due to its large torsional rigidity, the prototype had the fundamental torsional frequency of 0.662 cycles per second, and the frequency ratio was more than 3. The test results are summarized in Fig. 1.4 in which two types of oscillation are included, the one is a restricted oscillation, and the other a catastrophic oscillation. The restricted oscillator occurs within a certain range of wind. Though such type of oscillation is not fatal to the structure when no fatigue failure is expected, it should be avoided as far as possible. There may be two methods to eliminate the restricted oscillation, namely, the prevention of the separation of flow, and the provision of a damping capacity high enough to suppress the oscillation. In model B-4 no restricted oscillation was observed.

The catastrophic oscillation is fatal to the structure because the amplitude of oscillation will increase with wind velocity. The lowest predicted wind speed for catastrophic oscillation was far higher than the required value, and it was concluded that the proposed design B-4 was suitable from the viewpoint of wind stability. In the final design a truss-type girder similar to the model T-10 was adopted after comparison of construction cost and workability. (Fig. 2)

Section model tests on the modified suspended structure put a conclusion to the theme having been continued since 1967.

Effects of center of rotation, solidity in inspection walk, and damping capacity on the wind instability were investigated by using the models T-18 ∿ T-23 (Fig. 3). The natural frequencies were calculated as 0.450 c/s for the fundamental symmetric torsional mode and 0.219 c/s for the fundamental symmetric vertical bending mode. The wind excited oscillations of the models were predominantly torsional, and there was no significant vertical motion at any time during the test. One of the test results is shown in Fig. 4, and the predicted critical wind speeds are plotted in Fig. 5 against the various angles of attack.

It may be concluded from the test that (i) the predicted critical wind speed at three degree of angle of attack is the lowest, and it is more than 60 m/sec, (ii) the critical wind speed decreases as the center of rotation increases, and (iii) solidity of the inspection walk and damping capacity give significant effect on the critical wind speed.

Win tunnel tests on the main tower

The towers of the Forth Road Bridge were subjected to wind excited oscillation during their erection and in their completed freestanding state before the catwalk cables were installed. These oscillations are reported to have been excited by wind directed approximately in line with the tower legs, and to have had a period of oscillation close to the calculated natural period in the fundamental bending mode. As the oscillation are due to vortex-shedding rather than galloping excitation, the resonant wind speed for a major suspension bridge is low. In

connection with the Kanmon Bridge Tower, 134.0 m high, wind tunnel tests were carried out in order to get data necessary to make a plan of erection work. These tests were made on 1/60 scaled rigid models supported elastically at its base by springs with low damping. The model conditions were simulated to three erection stages, namely, (i) completed freestanding state without catwalk cables, (ii) as the case (i), but with a creeper crane at the top of the tower, and (iii) during the rection stage and tower height is 103.4 m with the crane on the top. The natural frequencies in the fundamental bending mode were calculated as 0.305, 0.288 and 0.479 c/s respectively. Test result of case (i) is shown in Fig. 6. It was found that the resonant wind was ranged in a narrow band and the amplitude of oscillation decreased rapidly with the increase of yaw angle of wind to the legs' direction, and disappeared at the angle more than 20 degrees. The maximum amplitude at the top of tower was estimated some 50 cm and the top of the tower was connected with a sliding weight near the ground by ropes in order to protect workmen from hazardous oscillation. The maximum amplitude at the top of the Shimonoseki tower was reported to have been about 11 cm at wind speed of 11 to 15 m/s during at its freestanding state. The observed amplitude seems to be much influenced by the fluctuation of wind direction as well as the wind speed.

Wind tunnel tests on catwalk cable systems

Catwalk cables, used for erection work of main cable and hangers, were composed of ropes, wire-netting and timber. They were spanned between anchorages through the tops of towers, and were jointed to the storm cables by hanger ropes. The catwalk having inverse π cross section is flexible from its structure and is expected to oscillate by wind. In the first, vibration analysis was made on two types of catwalk system, and it was found that a trussed hanger system had higher natural frequencies of vertical bending mode than a parallel hanger system. In spite of the ambiguity in similitude law for a netting structure wind stability tests were daringly carried out on 1/10 and 1/7 scaled section models. It was observed that when the vertical sides were shielded to wind both bending and torsional motions occurred independently, and when

they were not shielded and had small damping capacity galloping type oscillation would occur. The predicted critical wind speed was 20 to 30 m/s according to the damping capacity.

Vibration test of the main tower

The dynamic characteristics of the Moji Tower (Kyushu Island) were measured at its freestanding state. The tower was forced by a twin mechanical exciter set on the upper strut. The force, 15 ton each at 10 cycle, was applied longitudinally in the first test, transversaly in the second, and torsionally in the third. A set of rocket boosters was also fired on the strut in order to supplement the deficiency of longitudinal excitation. The capacity of boosters is illustrated in Fig. 7. The response of the structure was recorded by 14 accelerometers attached to the tower shafts.

The observed natural frequencies and structural dampings are summarized in Table 3, and the observed modes of oscillation in Fig. 8. The observed frequencies are fairly well coincided with the calculated values, and the structural damping seems to increase with the degree of modes. The damping capacity of the lowest mode is smaller than the expected value.

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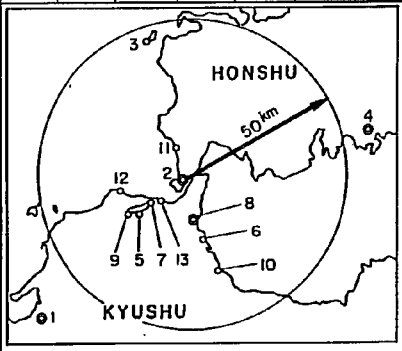
(4) Wind tunnel tests on the Kanmon Bridge

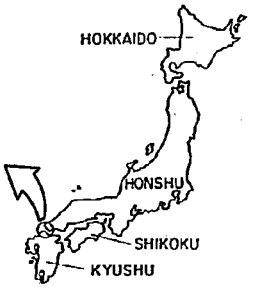
St. Sec., Tech. Rep. of P.W.R.I. No. 451, 452, 538, 596, 754 (in Japanese)

(5) An investigation of the aerodynamic stability of the tower proposed for the River Severn Suspension Bridge
Scruton C. and Walshe D.E., NPL Aero Report 1052, 1063

Table – 1 WIND RECORDS

NO.	1	2	3	4	5	6	7	8	9	10	11	12	13
LOCATION	FUKUOKA	SHIMONOSEKI	TSUNOSHIMA	HOUFU	YAWATA	KARITA	TOBATA	KOKURA (AIRPORT)	KUROSAKI	TSUIKI	YOSHIMI	WAKAMATSU	KOKURA
INSTRU-MENTS *	3C,4C	3C, 4C	3C	3C	4C,P2	P1	4C,P2	3C	4C	P1	P2	3C	3C
READING **	M	M	M	M	M	I	M	I	M	I	I	I	I
PERIOD (years) / YEAR	37	37	17	15	16	13	9	8	8	7	6	5	5
1929	17.5 m/s	25.8 m/s											
30	32.8	35.8											
31	17.2	25.2											
32	14.2	24.7											
33	18.0	27.0											
34	16.3	30.7											
35	16.6	35.8											
36	28.1	26.8											
37	15.8	26.9											
38	15.4	27.2											
39	16.1	26.5											
40	21.8	20.0											
41	23.3	24.2											
42	23.3	34.2											
43	15.3	22.6											
44	21.2	22.5											
45	28.7	24.2											
46	19.3	23.0											
47	16.7	26.0	20.3										
48	16.7	21.7	23.3										
49	22.0	30.7	27.9										
50	27.2	29.5	20.8	25.4	21.1								
51	32.5	24.1	35.6	20.2	19.6								
52	21.1	18.0	21.1	23.9	16.5								
53	19.4	18.0	22.9	15.5	18.0	18.0							
54	25.5	23.5	29.2	20.8	22.0	21.5							
55	21.4	30.3	24.1	33.4	19.6	24.1							
56	30.2	29.8	22.6	33.6	23.5	26.7							
57	20.1	23.5	20.7	20.5	21.7	18.0	22.0						
58	18.7	18.4	23.7	18.2	19.0	11.6	18.0	31	19.6				
59	23.5	23.8	23.5	25.1	24.7	33.0	24.9	41	21.3	27.8			
60	16.8	27.0	23.3	18.5	15.8	15.0	20.2	38	19.6	20.7	31.8		
61	22.8	19.7	23.3	19.5	16.5	36.0	25.2	32	21.0	26.2	33.4	32.7	24.0
62	14.2	18.7	22.0	18.8	14.7	22.8	19.6	27	19.6	21.1	32.9	26.8	28.3
63	18.2	20.0	25.0	18.7	13.9	23.0	18.4	31	19.6	23.6	35.5	22.3	22.0
64	18.2	20.3		18.7	16.5	22.1	21.1	35	21.3	29.2	35.0	20.0	30.0
65	18.3	26.0			18.0	25.0	29.8	32	24.4	29.5	34.5	26.7	25.1





Note

* INSTRUMENTS
- 4C : 4-Cup anemometer
- 3C : 3-Cup anemometer
- P1 : Propeller type anemometer (Aero vane)
- P2 : Propeller type anemometer (Koshin vane)

** READING
- I : Instantaneous value
- M : Mean value (Averaged over 10 min.)

TABLE. 2.1 MODELS

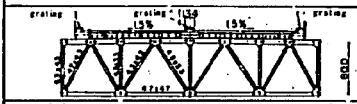
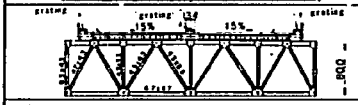
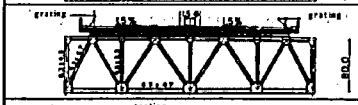
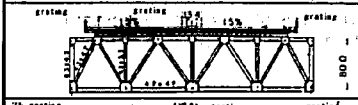
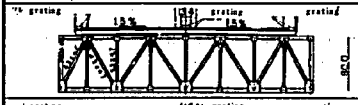
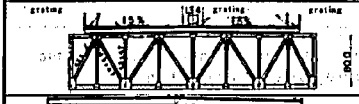
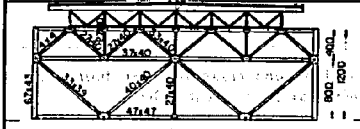
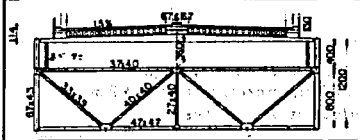
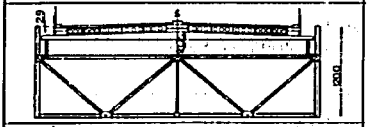
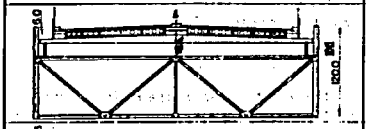
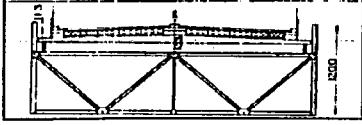
Sketch of models	desig-nation	Description
	T-1	Original design of truss girder truss height 6.0 m floor system : slab is fitted on truss stringers number of slots : 3
	T-2	T-1, but with 5 lanes of slots
	T-3	T-1, but with slab fitted on trussed stringers through floor girder (height of floor girder 30 cm)
	T-4	T-3, but with 5 lanes of slots
	T-5	T-3, but height of floor girder increased to 60 cm
	T-6	T-5, but with 5 lanes of slots
	T-7	Truss height : 9.0 m floor system : slab in fitted on trussed girders
	T-8	T-1, but the truss height is 9.0m and the road crown is higher than upper chord axis at about 130 cm
	T-9	T-8, but the road crown is higher than upper chord axis at about 25 cm
	T-10	T-8, but the road crown is about the same level as the upper chord axis
	T-11	T-8, but the road crown in lower than upper chord axis at about 40 cm

TABLE. 2.2 MODELS

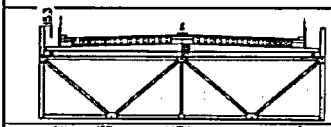
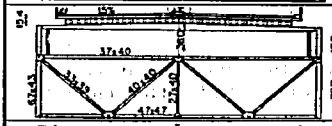
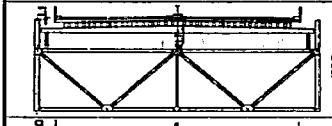
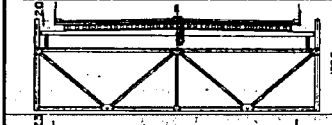
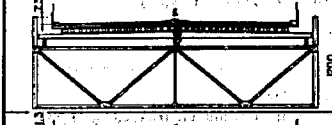
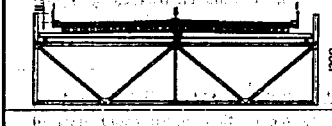
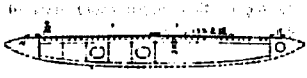
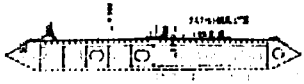
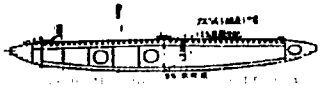
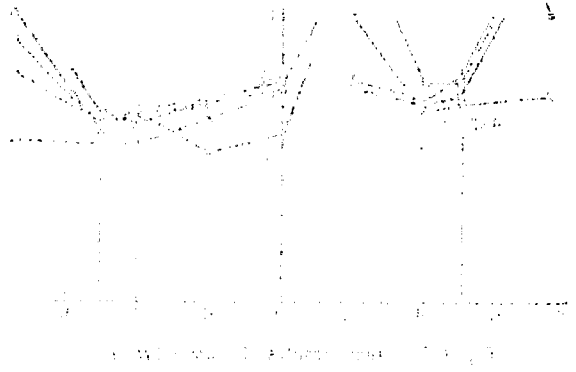
Sketch of models	desig-nation	Description
	T-12	T-8, but the road crown is lower than upper chord axis at about 70 cm
	T-13	T-3, but the truss height is 9.0m and the road crown is higher than upper chord axis at about 160 cm
	T-14	T-13, but the road crown is heigher than upper chord axis at about 55 cm
	T-15	T-13, but the road crown is heigher than upper chord axis at about 30 cm
	T-16	T-13, but the road crown is lower than the upper chord axis at about 9 cm
	T-17	T-13, but the road crown is lower than the upper chord axis at about 40cm
	B-1	Original design of box-type girder width : 31.0 m, maximum depth : 3.2 m lower surface is stream lined
	B-2	B-1, but the lower surface is polygonal and the maximum depth is 3.2 m
	B-3	Revised design width : 31.0 m, depth : 3.2 m
	B-4	B-1, but the width is increased to 32.5 m, maximum depth is 3.2 m

Table 3 Frequencies and Damping

Longitudinal Oscillation			
Mode	Freq. (Calculated)	Freq. (Observed)	Structural Damping
1	0.25 c/s	0.26 c/s	δ = 0.009
2	1.35	1.40	0.021
3	3.76	3.90	0.036
4	4.57		
5	4.81		
6	7.49	6.9	
Transverse Oscillation			
Mode	Freq. (Calculated)	Freq. (Observed)	Structural Damping
1	1.18 c/s	1.17 c/s	δ = 0.014
2	3.94	4.10	
3	6.80	6.20	
Torsional Oscillation			
Mode	Freq. (Calculated)	Freq. (Observed)	Structural Damping
1	0.74 c/s	0.75 c/s	δ = 0.011
2	2.16	2.29	0.076
3	4.75	4.61	0.023



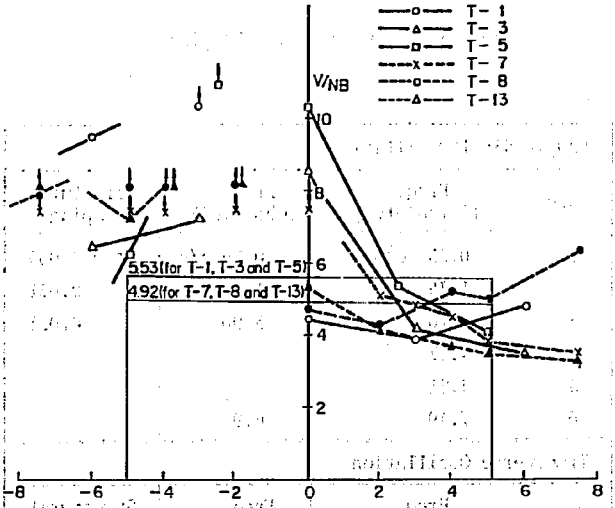


Fig. 1.1 Test results (Truss-type)

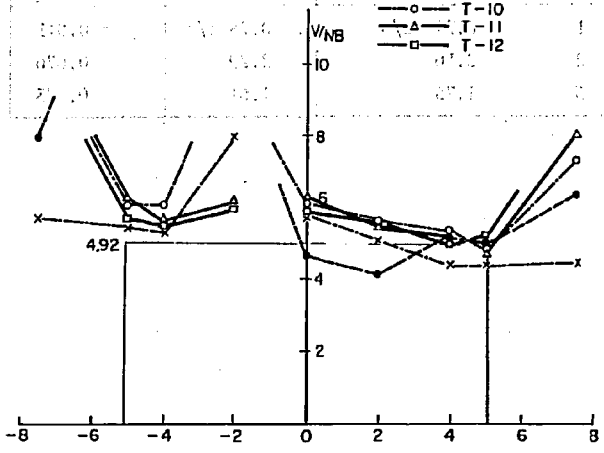


Fig. 1.2 Test results (Truss-type)

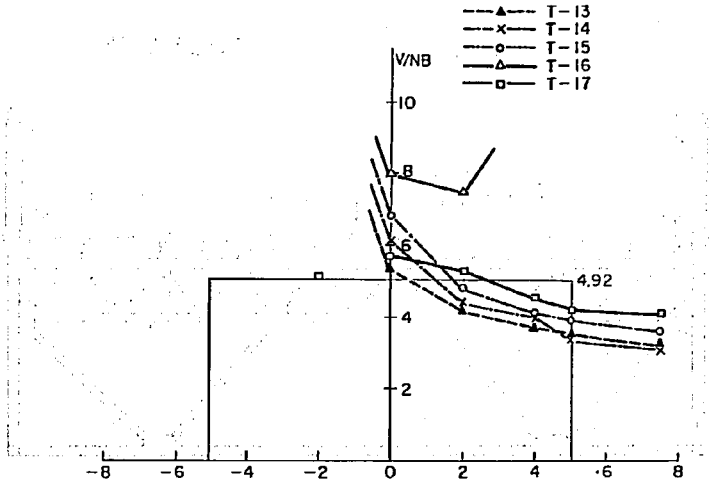


Fig. 1.3 Test results (Truss-type)

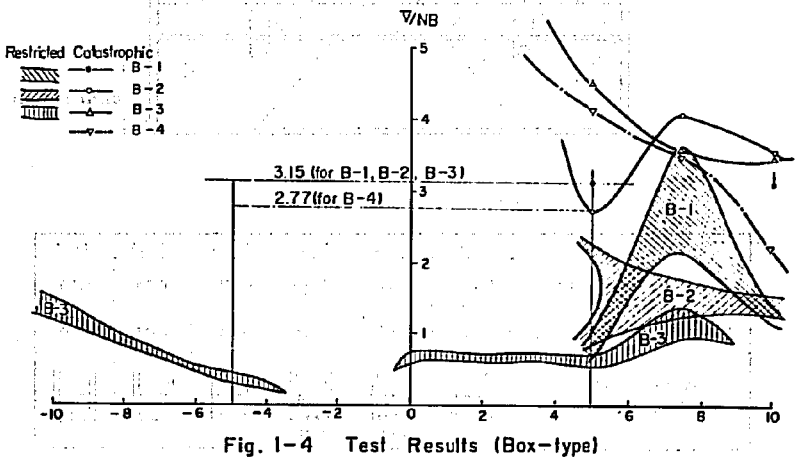


Fig. 1-4 Test Results (Box-type)

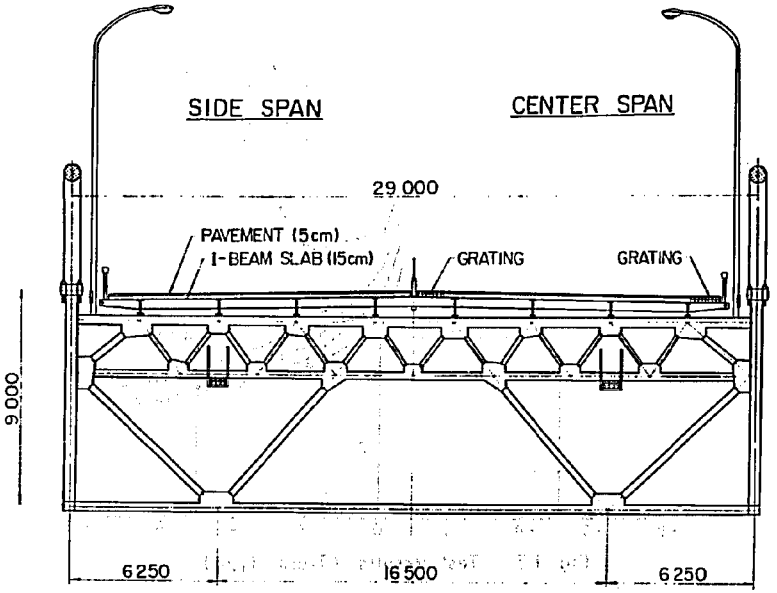
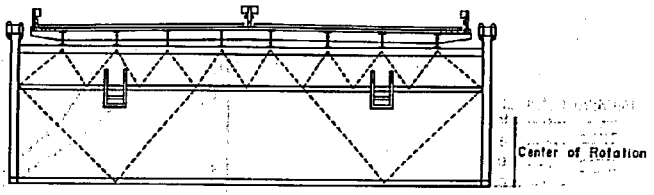


Fig. 2 Cross section (Final design)



	T-18	T-19	T-20	T-21	T-22	T-23
Center of Rotation (m)	5.20	5.20	5.20	6.75	6.75	5.98
Structural Damping	0.05	0.05	0.05	0.05	0.03	0.05
Configuration of Inspection Walk		plate	plate			

Fig. 3 Modified design

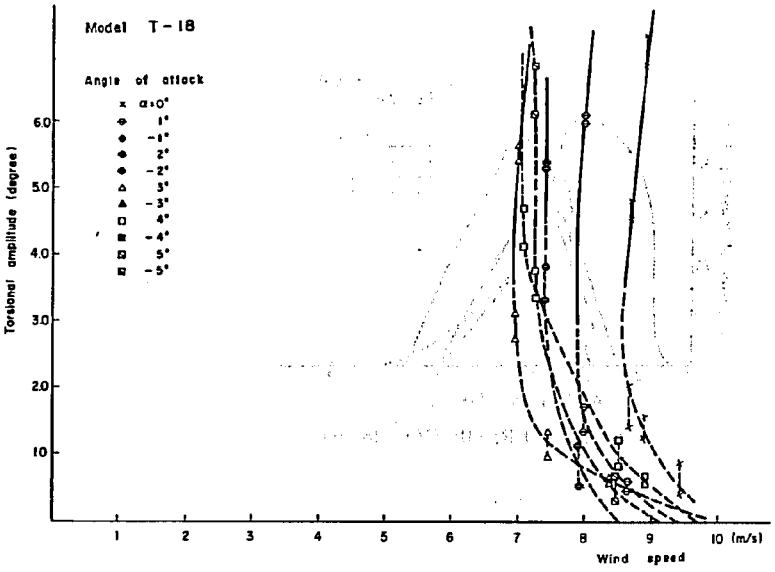


Fig.4 Wind speed vs amplitude

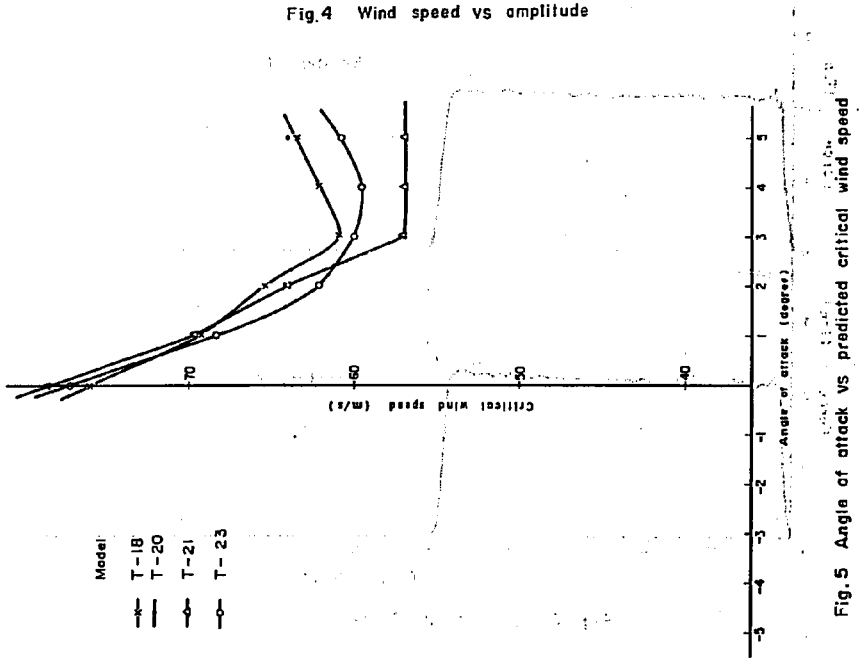


Fig.5 Angle of attack vs predicted critical wind speed

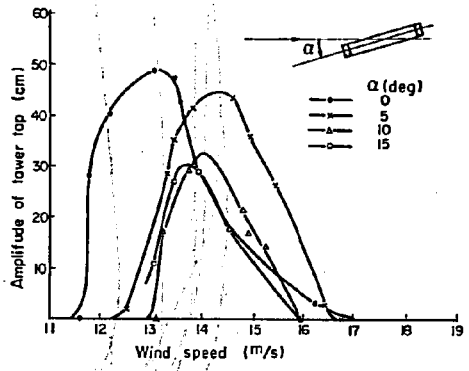


Fig. 6 Test Results (Main tower)

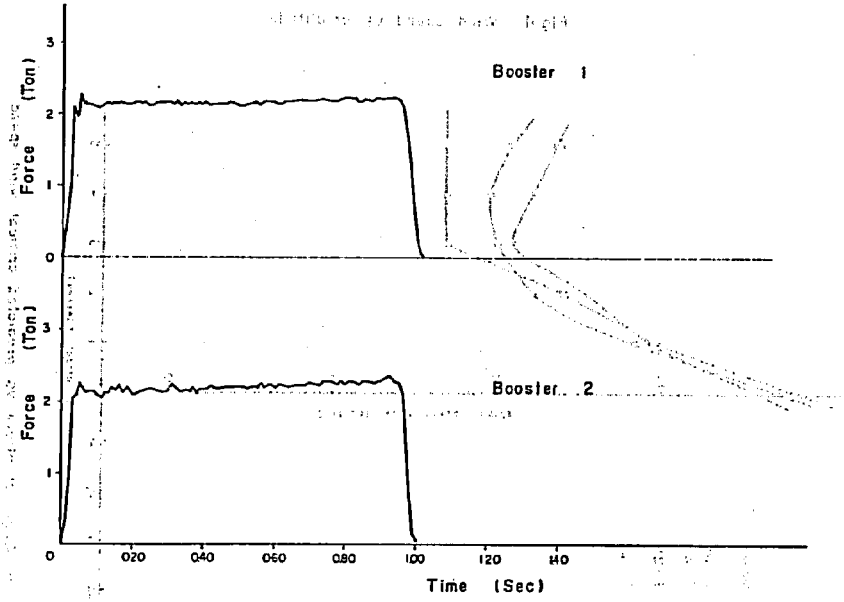
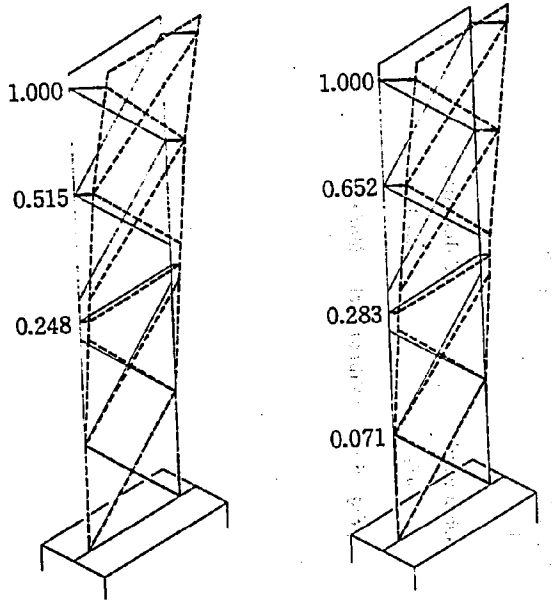


Fig. 7 Rocket of Force

$f = 0.25$ c/s
(Longitudinal)

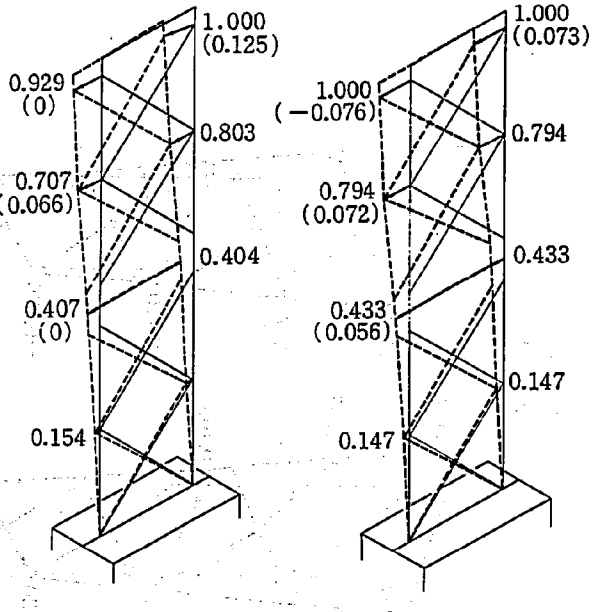


Observed
0.26 %

Calculated
0.25 %

Fig. 8-1 Vibrational Mode

$f = 1.18$ c/s
(Transverse)

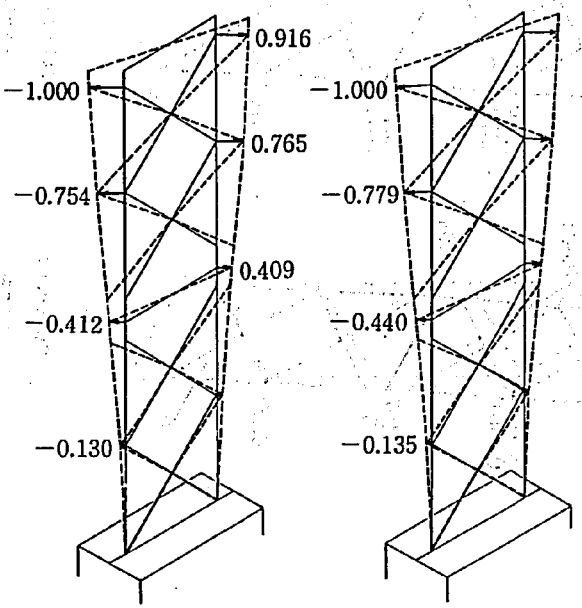


Observed
1.17 %

Calculated
1.18 %

Fig. 8-2 Vibrational Mode

$f = 0.74$ c/s
(Torsional)



Observed
0.75%

Calculated
0.74%

Fig. 8-3 Vibrational Mode

DYNAMIC BEHAVIOR OF STRUCTURES AND SUBSOIL
DURING EARTHQUAKES

Earthquake Response of Highway Bridges and Its Standardization

by

Eiichi Kuribayashi, Chief
Toshio Iwasaki, Senior Research Engineer
and Yutaka Iida, Research Engineer

Earthquake Engineering Research Section
Structure and Bridge Division
Public Works Research Institute
Ministry of Construction

Fourth Joint Meeting
U.S.-Japan Panel on Wind and Seismic Effects
UJNR, Washington, D.C., May 17 - 19, 1972

INTRODUCTION

"Specifications for Earthquake Resistant Design of Highway Bridges and Their Running Comentary" was published by the Japan Road Association in January 1972. The provisions are given in full as the Appendix of this paper.

Significant provisions on the design seismic coefficients for bridges with high-rise piers are provided among reformative ones incorporated.

In this paper the essential backgrounds related to the provisions are described.

1. Earthquake Response of Bridges

Although earthquake response analyses of highway bridges are found a lot and the dynamic characteristics have been gradually clarified, comparatively few examples of observed earthquake response of highway bridges are found in Japan.

The lack of the data on the observed earthquake response of highway bridges leads that earthquake response spectrum curves based on strong-motion earthquake records on various grounds are effectively applied to design.

Fortunately it is known that the response due to the fundamental natural vibration actually predominates during earthquakes. In almost all of cases of ordinary highway bridges except such particular ones as suspension bridges. Therefore the method using the earthquake response spectrum curves is strongly effective to estimate the response of bridges.

2. Design Standard Spectra

The design seismic coefficient k_{hm} in the modified seismic coefficient method considering structural response is obtained by multiplying the design seismic coefficient k_h in the seismic coefficient method

by the modification factor β depending on the fundamental period of the bridge and the ground condition of the structural site.

The modification factor β is provided by simplification of the four kinds of original average spectrum curves corresponding to four kinds of ground conditions.

In order to determine the four kinds of average spectrum curves the strong-motion earthquake records of 44 components observed recently in Japan are used.

The response acceleration ratio spectrum curves are obtained as follows.

When a vibrational system of single-degree-of-freedom, consisting of a mass m with spring coefficient k and damping coefficient c as shown in Fig. 1, is subjected to earthquake ground motion $\ddot{z}$ at its reference axis, the relative displacement y will be produced. The equation of motion for the system is

$$m\ddot{y}+c\dot{y}+ky=-m\ddot{z} \quad \cdots\cdots (1)$$

The solution of equation (1) is given by the following integral.

$$y=-\frac{1}{p\sqrt{1-h^2}}\int_0^t \ddot{z}(\tau)e^{-ph(t-\tau)}\sin p\sqrt{1-h^2}\,(t-\tau)d\tau \quad \cdots\cdots (2)$$

where

$p=\sqrt{k/m}$: natural circular frequency (natural period $T=2\pi/p$)

$h=c/2\sqrt{mk}$: damping ratio

The maximum response displacement $|y|_{max}$ computed for earthquake ground acceleration $\ddot{z}$ in equation (2) is a function of the natural period T with damping ratio h as a parameter. This function defines the response displacement spectrum (S_D) curves. Equation (2) may be written as

$$S_D=|y|_{max}=\frac{1}{p\sqrt{1-h^2}}\left|\int_0^t \ddot{z}(\tau)e^{-ph(t-\tau)}\sin p\sqrt{1-h^2}(t-\tau)d\tau\right|_{max} \quad \cdots\cdots (3)$$

The response velocity spectrum (S_V) curves and the response acceleration specrum (S_A) curves are defined respectively as

$$S_V = \frac{2\pi}{T} S_D \quad \cdots\cdots (4)$$

$$S_A = \left(\frac{2\pi}{T}\right)^2 S_D \quad \cdots\cdots (5)$$

The ratio of the value of response acceleration spectrum curves to the maximum value of earthquake ground acceleration $\ddot{z}_{max}$ means the magnification of the response acceleration of the vibrational system to the earthquake ground acceleration. This ratio defines the response acceleration ratio spectrum curves.

Envelopes of four kinds of average response acceleration ratio spectrum curves corresponding to four kinds of ground contions are shown in Fig. 2. The abscissa of Fig. 2 means natural periods.

The system shown in Fig. 1 can be supposed to represent the structural system of a bridge. The response of the system vary dependent on its natural period, damping ratio and the ground condition of the site as shown in Fig. 2.

The modification factor β shall apply to the design with bridges with fundamental natural periods longer than 0.5 sec. as shown in Fig. 4.2 in the specifications.

The maximum value of the modification factor β is 1.25 and the minimum value is 0.5.

The value in long period range decreases inversely with the natural period.

The value of the horizontal design seismic coefficient increases 25% at 0.5 sec. of the natural period.

The reasons for the modification are explained as follows.

i) The modification factor β's for the bridges with long natural periods in the ground of the group 4 correspond with the average response acceleration ratio spectrum curves for the group 1 and

the group 4 with 2 per cent damping ratio respectively. For the group 2 and the group 3 the modification factor β's are decided to trisect the difference between the ones for the group 1 and the group 4 as shown in Fig. 4.2 in the specifications.

ii) According to the fact on the basis of field vibration test results on actual highway bridges that damping ratios increase inversely with natural periods in the medium period range, the damping ratio h can be estimated at from 0.2 to 0.4 in the period range around 0.5 sec. From this point of view the acceleration ratio does not exceed 1.4 as shown in Fig. 2. The modification factor β in the medium period range is fixed at 1.25 on the average.

iii) In the period range shorter than 0.5 sec. the damping ratio can also be estimated at from 0.2 to 0.4. According to this estimation the response acceleration ratio becomes about 1.0 as shown in Fig. 2. In this case it can be said that the value of the horizontal design seismic coefficient in the modified seismic coefficient method considering structural response is nearly equal to the one in the seismic coefficient method.

Although the deduction processes are quite different from each other, the results are nearly equal in the short period range and there is not a contradiction between the two methods.

iv) The damping ratio here is a kind of measure that shows the degree of attenuation in the vibration of the system of single-degree-of-freedom considered to represent a bridge structure. When the value of the damping ratio h is greater than 1.00, the free vibration of the system is not induced. The smaller the value becomes below 1.00, the more easily the system becomes to vibrate.

According to field vibration test results on actual highway bridge structures, the relationship between the damping ratio h and the natural period T can be said as follows.

A. For the bridges with relatively short natural periods (shorter than 1 sec.), $h = 0.02/T$ (T shown in sec).

B. For the bridges with relatively long natural periods (longer than 1 sec), $h = 0.02$ or less than 0.02.

On the other hand the observed strong-motion earthquake response of ordinary bridges with short natural periods such as 0.3 sec shows that the response acceleration is equal to or below the ground acceleration.

This means that the equivalent damping ratio h is greater than 0.4 which exceeds the value calculated by the formula shown above in A, that is, $h = 0.02/0.3 = 0.067$.

It can be explained as follows.

The results of field vibration tests only show the elastic characteristics of bridges at very low amplitudes but the the response to strong-motion earthquakes is affected by the dynamic behavior of bridges not only in their elastic ranges but also in their inelastic ranges at large amplitudes.

The value of the damping ratio h for bridges with long natural periods is estimated as 0.02 with reference to the provisions of the Japan Society of Civil Engineers' Specifications for Earthquake Resistance Design of the Honshu-Shikoku Bridges (1967) although it corresponds to the maximum value obtained from field vibration test results.

The lowest value of the modification factor is limited to 0.50 from the engineering point of view that the value of the horizontal design seismic coefficient in the modified seismic coefficient method should not be less than the half in the seismic coefficient method.

Since the value of the modification factor β is decided to give the horizontal design seismic coefficient k_{hm} at the center of gravity of a bridge structure, it can be concluded, from the fact that its center of gravity generally locates at its upper part, that the uniform distribution of the design seismic coefficient maintains sufficient safety in spite of the larger amplitudes of vibration at its upper part.

3. Method for Obtaining Fundamental Periods

The fundamental natural period of a bridge is desirable to be obtained for the structural system as a whole, but the equations would become very complicated ones in this case. In order to simplify the problem the method to obtain the fundamental natural period of a bridge for the individual system consisting of each substructure and the part of superstructures supported by it is adopted. Although the values of the natural periods obtained by this method involve some errors, the effect of errors on the value of the horizontal design seismic coefficient is considered to be very small.

Bridges are classified in two groups by their substructures. One group consist of bridges supported by Spread foundations or pile foundations and the other consists of bridges supported by caisson foundations.

It is known that bridges supported by spread foundations and bridges supported by pile foundations show similar dynamic behavior in field vibration test.

Equations from (4. 3) to (4. 6) in the specifications give the natural periods of bending vibration and equation (4. 7) gives the natural periods of rocking vibration.

In the case of bridges supported by caisson foundations the coupled vibration may be excited between bending vibration and rocking vibration.

Since the equations to obtain the natural period of the coupled vibration become complicated ones, it is assumed that the bending vibration and the rocking vibration are excited individually.

In the case of bending vibration the actual individual system can be considered as the vibrational system shown in Fig. 3

The fundamental natural period of this system is given by the following equation.

$$T=\frac{2\pi}{\alpha^2}\sqrt{\frac{W_p}{EIg}}h^3 \quad \cdots\cdots(1)$$

where α is the minimum positive root of the following equation.

$$\frac{1+\cosh\alpha\cos\alpha}{\cosh\alpha\sin\alpha-\sinh\alpha\cos\alpha}=\mu\alpha-\frac{\chi}{\alpha^3} \quad \cdots\cdots(2)$$

where

$$\mu=\frac{W_u}{W_p} \quad \cdots\cdots(3)$$

$$\chi=\frac{Kh^3}{EI} \quad \cdots\cdots(4)$$

K : the spring constant which represents the effective rigidity of the superstructure for the substructure

Applying Rayleigh's method, equation (1) becomes

$$T=2\pi\sqrt{\frac{\frac{33}{140}W_p+W_u}{(3+\chi)EIg}h^3} \quad \cdots\cdots(5)$$

Equations (1) and (5) correspond the solid curve and the dotted curve in Fig. 4 respectively. Since the value α is limited below 1.875 in actual cases, the approximate value by equation (5) is accurate sufficiently for the practical purpose.

The value χ in actual bridges of the type 1 explained in the specifications' Table 4.6 shows about from 0.004 to 0.053 for reinforced concrete piers and about from 1.13 to 1.67 for steel piers in the transverse direction. Assuming $\chi = 0$ and $\chi = 1.5$ in equation (5), equations (4.3) and (4.4) in the specifications are derived respectively. The numeral coefficient of W_p is raised from $33/140 \fallingdotseq 0.24$ in equation (5) to 0.3 in equations (4.3) and (4.4) in consideration of the mass effect of footings.

The effective rigidity of the superstructure for the substructure is considered to be zero in a bridge of the type 2 in the transverse direction. Equation (4.6) is derived from equation (5) on the assumption $\chi = 0$.

The effective rigidity of the superstructure becomes so large in comparison with the bending rigidity of the pier in a bridge of the type 1 in the longitudinal direction that the value χ can be estimated as in-

finite. It corresponds to the assumption that the pier is considered as a fixed-hinged beam of $\alpha = 3.927$. Assuming $\alpha^2 \fallingdotseq 16$, equation (4.5) is derived from equation (1).

The total weight of the continuous beam shall be taken as W_u in case of an pier with its fixed support similar to the system of the type 1 in the longitudinal direction.

Equation (4.7) applied to the rocking vibration is derived based on the assumption that the center of rotation is located at the point $l/3$ from the bottom.

The natural period of the rocking vibration is given by the following equation.

$$T = 2\pi\sqrt{\frac{J}{K_\theta}} \quad \cdots\cdots (6)$$

where the rotational inertia due to the superstructure

$$\left(h + \frac{2}{3}l\right)^2 \frac{W_u}{g}$$

the rotational inertia due to the pier

$$\left\{\frac{h^2}{12} + \left(\frac{h}{2} + \frac{2}{3}l\right)^2\right\} \frac{W_p}{g}$$

the rotational spring constant due to the subsoil reaction at the side of the caisson foundation in the horizontal direction

$$K_H \frac{bl^3}{36}$$

the rotational spring constant due to the subsoil reaction at the bottom of the caisson foundation in the horizontal direction

$$K_S A \times \left(\frac{l}{3}\right)^2$$

the rotational spring constant due to the subsoil reaction at the bottom of the caisson foundation in the vertical direction

$$K_V I$$

Then $$J=\left(h+\frac{2}{3}l\right)^2\frac{W_u}{g}+\left\{\frac{h^2}{12}+\left(\frac{h}{2}+\frac{2}{3}l\right)^2\right\}\frac{W_p}{g}+\left\{\frac{l^2}{12}+\left(\frac{l}{2}-\frac{2}{3}l\right)^2\right\}\frac{W_c}{g} \quad \cdots\cdots (7)$$

and

$$K_\theta=K_H\frac{bl^3}{36}+K_SA\times\left(\frac{l}{3}\right)^2+K_VI \quad \cdots\cdots (8)$$

Equation (4.7) is derived from equations (6), (7) and (8).

The effective rigidity of the superstructure is neglected in the rocking vibration.

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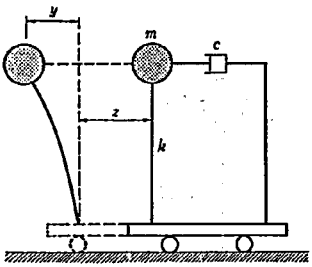


Fig. 1

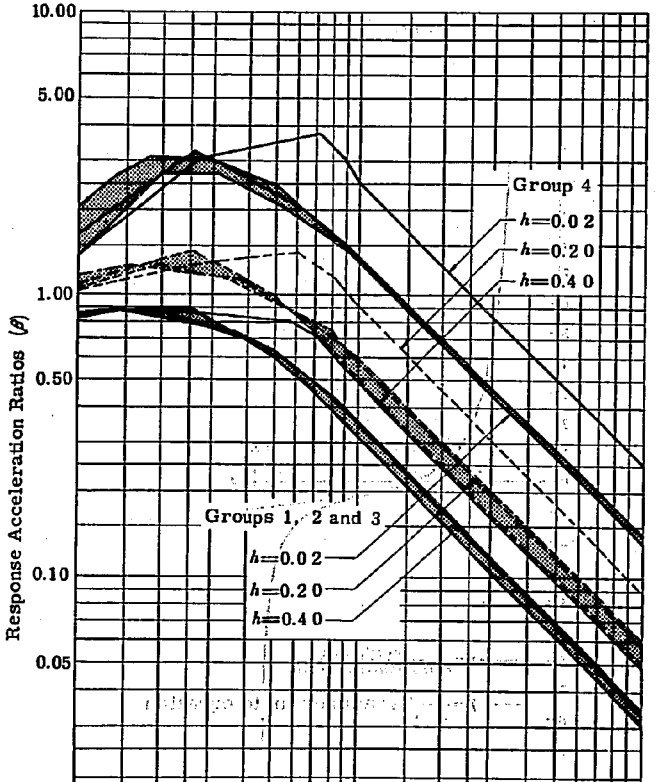


Fig. 2 Average Response Acceleration Ratio Spectrum Cruves

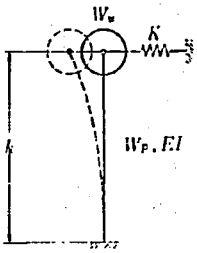

W_w
K
W_p, EI

Fig. 3

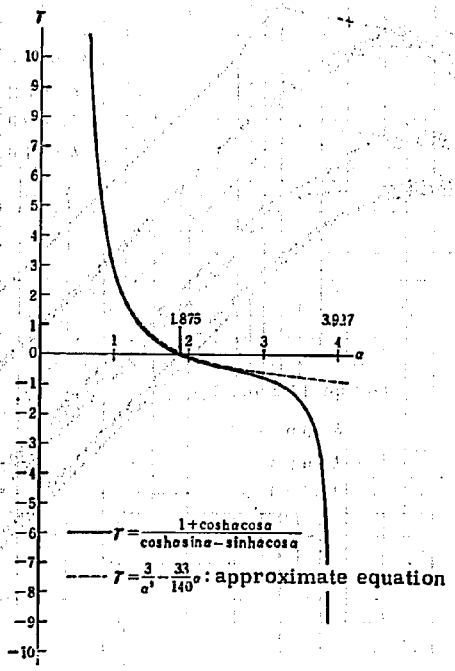

T
10
9
9
7
6
5
4
3
2
1
0
−1
−2
−3
−4
−5
−6
−7
−8
−9
−10
1.875
3.927
1
2
3
4
α
$T=\frac{1+\cosh\alpha\cos\alpha}{\cosh\alpha\sin\alpha-\sinh\alpha\cos\alpha}$
$T=\frac{3}{\alpha^3}-\frac{33}{140}\alpha$: approximate equation

Fig. 4

WIND DAMAGE DUE TO TYPHOON "YOLING" (1970) IN GREATER MANILA AREA

to be presented at
the Fourth Joint Meeting,
UJNR,
17-19, May, 1972
Washington D.C.

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1. Introduction

The Philippines are located in a region having the greatest frequency of tropical cyclones in the world. In 1970, as a whole twenty-one tropical cyclones affected the Philippines.

As well as in the U.S., tropical cyclones have womens name in the Philippines. Among the twenty-one tropical cyclones mentioned above, Pitang (Georgia in the U.S. name), Sening (Joan) and Yoling (Patsy) brought devastating damages to the Luzon Island.

Especially, it was said that Yoling was the most devastating typhoon over Manila, since the establishment of Philippine Weather Bureau about 106 years ago.

At the request of the Philippine Government, Unesco sent a servey mission to Manila for studying the nature and extent of damage and for formulating recommendations to prevent buildings and structures damage in the event of future typhoons through consultation with the Philippine authorities.

The mission consisted of three persons, that is, Prof. S. Mackey, Hong Kong University, Mr. C.E. Finney, Unesco Expert, Asian Regional Institute for School Building Research, Ceylon and the author.

In the wake of survey by the mission, in November 1971, Unesco organized a Regional Seminar on Wind Effects on Buildings and Structures in Manila. At this seminar, the members of survey mission mentioned above served as consultants for technical direction.

This report introduces an outline of the survey results in 1970 published by Unesco[2] and of the seminar in 1971.

2. Statistics of typhoon in the Philippines

After Dr. R. L. Kintanar[1], Director of Philippine Weather Bureau, as shown in Table 1, an annual mean number of tropical cyclones which affect the Philippines is 19.7. In the period of 1948-1970, the annual number of tropical cyclones varied from a minimum of 12 in 1951 to a maximum of 32 in 1964.

Table 1 Mean Monthly Frequency of Tropical Cyclones (1948-1970)

Month	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
Mean Frequency	0.44	0.35	0.30	0.39	0.78	1.48	3.26	3.57	2.96	2.30	2.57	1.35	19.7
%	2.2	1.8	1.5	2.0	3.9	7.5	16.5	18.1	15.0	11.7	13.0	6.8	100

In 1970, twenty-one tropical cyclones affected the Philippines as shown in Table 2. In the table, T.T., S. and T.D. mean typhoon, tropical storm and tropical depression, respectively and are defined by the maximum wind speed exceeding 64 knots, between 34 and 64 knots and less than 34 knots, respectively.

The paths of these tropical cyclones are shown in Figure 1. According to Kintanar, "During the months of April, May and June, the cyclones which hit the Philippines generally cross the Visayas. During the months of July, August and September, most of these cyclones cross northern Luzon or the Batanes Islands. Again, during the period from October to March, these cyclones generally cross the Visayas". In 1970, several typhoons struck southern Luzon successively during the months of October and November.

Table 2. Tropical Cyclones in 1970 affected to the Philippines

	Classification	Date of Occurrence	Name		Lowest Pressure (mili-bar)		Highest Wind Speed (m/s)		Maximum Radius of 1000 mb Isobar line (km)
			Philippine	U.S.	Over Ocean	Over Land	Over Ocean	Over Land	
1	T.	II 23-27	Atang	Nancy	949	960.5	66.7	61.1	200
2	T.D.	VI 13-14	Bening		996.7	1,000.2	11.1	8.3	400
3	T.S.	VI 29-VII 1	Claring	Pamela	980	1,001.2	27.8	-	110
4	T.	VI 29-VII 3	Deling	Olga	907	-	76.4	-	360
5	T.	VII 12-15	Emang	Rubby	989	996	26.4	18.1	550
6	T.D.	VII 30-VIII 1	Gading		1,002	1,002.4	12.5	18.1	90
7	T.S.	VIII 4-8	Heling	Violet	995	993.9	18.1	15.3	185
8	T.	VIII 9-12	Iliang	Wilda	950	1,002.3	51.4	8.3	550
9	T.	VIII 23-27	Loleng	Billie	995	-	69.4	-	500
10	T.D.	VIII 31-IX 2	Miding		-	-	-	-	-
11	T.S.	IX 3-6	Norming	Fran	-	-	-	-	-
12	T.S.	IX 4-7	Oyang	Ellen	-	-	-	-	-
13	T.	IX 8-13	Pitang	Georgia	904	977.5	66.7	61.1	260
14	T.D.	IX 21-25	Ruping		994	998	15.1	9.2	250
15	T.	X 10-16	Sening	Joan	901	870	76.4	76.4	600
16	T.	X 15-23	Titang	Kate	941	988.9	71.5	27.8	220
17	T.S.	X 24-27	Uding	Louise	996	1,005.4	26.4	11.1	240
18	T.S.	X 29-XI 4	Wening	Marge	989	1,000.2	27.8	23.6	60
19	T.	XI 15-20	Yoling	Patsy	912	954.8	61.1	55.6	200
20	T.D.	XI 22-24	Aning						
21		XII							

3. Records of typhoon Yoling

According to the Philippine Weather Bureau, typhoon Yoling had started its life as a tropical depression by 8:00 on November 14th in the vicinity of 155°E, about 1,100 km east of Guam. By 14:00 of the same day, it had become a tropical storm and by 7:00 of November 16th, it was upgraded to typhoon strength in the vicinity of 15°N and 141°E. Increasing its energy, typhoon Yoling landed on the east coast of Luzon at about 10 o'clock of November 19th and swept the Greater Manila Area at about 12:00 of the same day. By 01:00 of November 21st, it was downgraded to tropical storm on the South China Sea. Table 3 shows the vicissitudes of life of typhoon Yoling based on the reports of reconnaissance aircraft.

By the reconnaissance of aircraft, the maximum sustained wind velocity was 135 knots (69.5 m/s) near the center of typhoon. Because of the wind damage to Weather Bureau installations combined with the power failure of electricity by typhoon, there was very few reliable data of wind.

Table 4 shows wind records at Manila International Airport, about 8 km south of the center of Manila City. The instrumental height of anemometer was about 20 meters above the ground. The maximum gust wind speed was 108 knots or 55.6 m/s. The lowest barometric pressure at the Airport was 971.2 mili-bars.

The record of barometric pressure obtained at MERALCO Headquarters in the Quezon City where the typhoon eye swept is shown in Figure 2.

TABLE 3

RECORD OF TYPHOON YOLING

REPORT NO.	DATE (Nov. 1970)	TIME (GMT)	FORECAST POSITION		MAX/MIN WIND	RADII OF ACTION (IN N.M.) OF WINDS			STORM SPEED	REMARKS
			LAT.	LONGT.	kt.	100 kt.	50 kt.	30 kt.	kt.	
01	14	0500	12.8N	152.7E	35			50	17	Recon Fix @0500 hr.
02	14	1100	13.7N	151.0E	35			50	17	Recon difficulty in Location
03	14	1700	13.8N	149.5E	35			50	18	Recon Fix @1500 hr.
04	14	2300	15.4N	146.5E	40			50	18	Recon Fix @20.30 hr.
05	15	0500	15.3N	144.7E	40			75	17	Recon Fix @03.15 hr.
06	15	1100	14.9N	143.8E	45			75	14	Land Radar and Recon Fix @08.35 hr.
07	15	1700	14.7N	142.9E	50			100	11	" " " " Recon Fix @15.18 hr.
			STORM UPGRADED TO TYPHOON							
08	15	2300	15.0N	141.1E	65		50	150	15	Eye fixed by Recon @18.00 hr.
09	16	0500	14.9N	139.5E	75		50	150	15	Recon Fix @0300 hr.
10	16	1100	15.0N	138.1E	80		50	150	14	Land Radar and Recon Fix @1018 hr.
11	16	1700	14.8N	137.1E	85		50	150	13	Recon Fix @1450 hr.
12	16	2300	14.5N	135.2E	85		50	150	14	Recon Fix @2100 hr.
13	17	0500	14.7N	134.0E	90		75	150	14	Recon Fix @0300 hr.
14	17	1100	14.5N	132.0E	95		75	200N 150S	15	Recon Fix @0903 hr.
15	17	1700	14.5N	130.8E	100		75	200N 150S	14	Recon Fix @1400 hr.
16	17	2300	14.5N	128.7E	120		75	200N 150S	15	Recon Fix @2100 hr.
17	18	0500	14.5N	127.4E	125	100				Recon Fix @0300 hr.
18	18	1100	14.4N	126.2E	130	40	100	200	13	Recon Fix @0859 hr.
19	18	1700	14.5N	124.4E	135	40	100	200	15	Recon Fix @1517 hr.
20	18	2300	14.8N	122.8E	135	40	100	200		Radar & Recon Fix @2200 hr.
21	19	0500	14.9N	121.1E	103	20	75	350NWQ 250ELS	13	Radar and Recon Fix @2200 hr.
22	19	1100	14.6N	119.2E	80		100NWQ 75ELS	300NWQ 250ELS	13	Recon Fix @0846 hr.
23	19	1700	14.7N	117.4E	80		100N 75ELS	300N 200ELS	15	Recon Fix @1418 hr.
24	19	2300	14.7N	115.9E	85		100N 75ELS	300N 200ELS	15	EXTRAP.
25	20	0500	15.2N	115.5E	80		75	200	14	Recon Fix @0300 hr.
26	20	1100	15.7N	114.7E	65		25	150	10	Recon Fix @0936 hr.
			DOWNGRADE TO TROPICAL STORM							
27	20	1700	15.8N	113.3E	50			120	12	Recon Fix @1414 hr.
28	20	2300	15.9N	111.9E	45			75	14	EXTRAP.
29	21	0500	15.8N	110.5E	40			50	14	Recon Fix @0300 hr.
30	21	1100	16.3N	109.4E	45			100NWQ 50ELS	10	Increased wind due to land influence Recon Fix @0915 hr.
31	21	1700	16.6N	108.5E	40			100N 50ELS	10	
32	21	2300	16.8N	107.3E	40			125N 50ELS	10	EXTRAP. Further recon impossible due to restricted flying area

Notes for Editor on Table 3.3

N.M. = Nautical Miles
KT = Knots
N = Northern semicircle
NWQ = Northwest Quadrant
ELS = Elsewhere
Recon = Aircraft Reconnaissance
EXTRAP = Extrapolation

TABLE 4

WEATHER BUREAU FORECASTING BRANCH
Manila International Airport
Pasay City

WIND OBSERVATIONS AT MIA DURING THE PASSAGE OF TYPHOON

YOLING ON NOV. 18/19 1970

DATE & TIME (GMT)	WIND DIRECTION (°) & SPEED (kt)	GUSTS (kt)	TIME (GMT)	WIND DIRECTION (°) & SPEED (kt)	GUSTS (kt)
18/2100	VRBL - 02	None	9/0430	* 50	76
2200	VRBL - 02	None	0435	* 40	64
2300	310 12	None	0440	* 50	76
19/0000	310 16	26	0445	S 40	72
0035	290 32	40	0450	S 40	68
0100	300 30	40	0455	S 50	62
0110	300 20	38	0500	S 46	62
0120	290 34	50	0510	S 40	58
0130	300 36	48	0520	S 38	56
0140	280 38	52	0530	S 35	44
0150	300 36	56	0540	S 36	56
0200	300 36	68	0550	SSE 40	64
0210	270 55	76	0600	SE 40	50
0220	270 48	68	0610	SE 36	48
0230	270 48	76	0620	SE 36	56
0240	280 52	80	0630	SE 36	52
0250	280 60	84	0640	SE 30	40
0300	290 60	90	0650	SE 35	45
0310	290 50	100	0700	SE 35	45
0320	290 56	96	0710	SE 35	40
0330	290 58	108	0720	SE 34	40
0340	270 65	100	0750	SE 25	40
0350	260 70	100	0740	SE 24	40
0400	240 60	92	0750	SE 28	38
0405	220 65	100	0800	SE 25	36
0410	210 65	96	0830	SE 26	36
0415	* 60	90	0900	SE 20	26
0420	* 56	72	0930	SE 22	28
0425	* 56	80	1000	E 18	None

ALL OBSERVATIONS WERE MADE ON TOP OF MIA BUILDING, ABOUT 20 METERS FROM THE GROUND.

DIRECTIONS GIVEN IN COMPASS POINTS ARE ESTIMATES (NOT RECORDINGS)

Notes : * Power failure (direction not recorded). VRBL = Variable

4. Outline of damage due to Typhoon Yoling

(1) Statistics

Casualties and amount of damages are as shown in Table 5. Pitang had a similar size with that of Yoling and also devastated.

Table 5. Casualties and Damages

Typhoon	Date	Maximum Wind Velocity		Casualties			
		Over Ocean (m/s)	Over Land (m/s)	Dead	Missing	Injured	Amount of Damage ( )
Pitang	IX 11	66.7	61.1				
Sening	X 13.14	76.4	76.4	675	193	1593	500,000,000
Titang	X 18.19	71.5	27.8	631	284	76	305,000,000
Yolling	XI 19	61.1	55.6	230	381	1756	116,000,000

(2) Classification by intensity of damage

In the Greater Manila area, structures could be classified in the following descending order of frequency of damage.

i. Large neon-sign and billboard structures
ii. Roof of school buildings
iii. Roof of houses
iv. Glass breakage
v. Tall antena masts
vi. Survice station canopies
vii. Roof of industrial and commercial buildings

5. Damage of Domestic Houses

In the low-cost housing areas, traditional small houses of one or two-storeys, built of luan, survived quite well in either of widely spaced area or densely concentrated area.

On the contrary, in the high-class residential area such as Makati, comprising large and costly houses, concentrations of extensive roof damage were observed. In these area, very low-pitched (almost flat) roofs, cantilevered out to a considerable degree beyond the external walls to form large awnings, were usually used. These concentrations of damage indicated, not necessarily higher wind velocities in these areas, but rather poor attention to structural and roofing details to resist wind effects and false economy in construction.

6. Damage of High-rise Buildings

No structural damage to modern high-rise buildings of reinforced concrete or structural steel construction caused by direct wind action was observed. Some indirect structural damage resulted from collapse of neon-signs and billboard structures on the roofs.

Non-structural damage such as glass breakage, stripping of roof coverings, failure of sun breakers and other external additions and damage of interior ornaments resulted from glass breakage were widely observed.

In an attempt to assess wind velocities in the urban area and wind pressure patterns on actual buildings, a survey of glass breakage was carried out on a selected number of modern high-rise buildings in the Greater Manila area. Buildings surveyed are listed in Table 6.

The breaking loads of glass were estimated by an empirical formula derived by S. Miyoshi.

$$P = \frac{C}{A}\left(t + \frac{t^2}{4}\right)$$

where P: breaking load in kg/m^2 A: area of glass in m^2

t: thickness of glass in mm C: coefficient, 80 for sheet glass and 60 for plate glass.

Then, wind velocities were calculated from the breaking loads.

Because of wide variance of breaking loads of glass caused by incautious handling, good estimation of wind velocity was not obtained. However, it can be pointed out that extreme pressure rise caused by unusual configuration of building and high suction near the corner of building were main reasons of glass damage together with the use of inadequate thickness of glass for the panel dimensions concerned.

Table 6 Building surveyed

Name Building	Manila Hilton Hotel	Filipinas Hotel	Ermita Center Bldg.	Savoy Hotel	Intercontionental Hotel
No. of Storey	21	7	12	10	13
Height (m)	65	23	40		38
Location	Maria Y. Arosa St., Manila	Padre Faura cor. Roxas Blvd., Manila	1350 Roxas Blvd., Manila	Roxas Blvd., Pasay	Commercial District, Makati

7. Damage of School Buildings

Generally, school buildings were single storey structures with all structural and non-structural components prepared to size in the factory. The types commonly used have the names and characteristics listed in Table 7. In all cases, the walls were perforated with grilles or louvers.

Comparing the two Marcos types, it was noted that damage was much more prevalent and extensive to Type (1) than to Type (2). Most

of the structural damage to the Type (1) was attributable to the inadequacy of the lateral bracing to withstand the wind forces. Inadequate anchorage of the portal-frame column bases into the supporting piers or footings was another important contributory cause of the extensive damage of these structures.

In the case of the Army types (3) and (4), the nature and amount of damage was erratic in character and depended very much on the quality of workmanship displayed in the construction.

Table 7. School Buildings

Type	Basic Characteristics
(1) MARCOS POST-1969	Steel Z-section columns and rafters: Channel purlins.
(2) MARCOS PRE-1969	Doubled steel channel columns, rafters and purlins.
(3) ARMY	Wood trusses, stud walls and purlins.
(4) ARMY, VARIANTS	Wood trusses, square posts and purlins.

8. Damage to Roofs

Except in the case of modern high-rise buildings, the normal practice for roof construction in the Philippines is to use light-gauge, galvanized corrugated metal sheeting, fastened either to timber or steel purlins which are supported, in turn, on timber or steel rafters. As mentioned in the paragraph 5, the vast majority of houses are built with very low-pitched roofs, with the result that the full roof area is exposed to negative wind pressure or suction. In view of the type and quality of roof construction generally used, this leaves them particularly vulnerable to damage from wind attack.

The main causes of roof failure may be attributed to the following:

(1) Toe-nailing (Skew-nailing)

As shown in Figure 3, in most cases this type of nailing was used for fastening members together. Wind uplift pressure on the roof caused these nails to act in tension, resulting in easy withdrawal.

(2) Rusting and corrosion around seam bolts or nails

The use of round washers with bolts or nails for fastening the light-gauge roof sheetings to purlins often lead to local indentation of the sheet corrugations at the fastening points. These form pockets for collection of moisture under the washers and around the bolt or nail holes resulting in rusting and corrosion of the sheeting metal. Subjected to wind suction, the sheeting then readily tore away from its weakened fastenings.

(3) Inadequate anchorage of awnings and overhangs

Deep overhangs of roof to form large awnings are particularly vulnerable to wind attack and by high suction around the end of eave, the roof sheeting was peeled upwards or by the uplift force, roof was lifted from the wall due to straightening of bolt's hook which fastened the purlin to wall.

(4) Separation of flow

Damage of domed roof of Araneta Coliseum consited of aluminium sheets of 0.74 mm thick as shown in Figure 4 is considered as the results of the separation of air flow around the semi-sphere at high Reynolds Number exceeding 10^8.

9. Damage to Highway and Bridge

In general, there are two kinds of typhoon damage, that is, direct damage due to wind effects and indirect damage resulting from flooding.

landslides, storm surges, collapse of neighbouring structures and so on. In the case of bridges and highways, no direct damage was reported but damage by flooding and landslides was widespread and heavy.

Flood action resulted in bridge damage of two kinds; tilting and/or settlement of bridge piers by scouring, and washing away of bridge girders due to overtopping of bridge deck by flood water.

Highway damage largely consisted of surface erosion due to running water combined with damage arising from landslides.

10. Conclusions and Recommendations

The author would like to introduce the whole sentence of the "General Conclusions and Recommendations" of the Survey Missions Report[2].

(1) Storms in General

The vulnerability of the Philippines Archipelago to cyclonic storms and the amount of damage to life and property which can result from such attacks make it necessary for the Government of the Philippines (in collaboration with Governments of other affected countries in the Region) to undertake, as a matter of urgency, a more widespread and systematic study of such storms than has been done in the past. It is only from such well co-ordinated investigations that adequate information relating to wind effects and to rainfall distribution and intensity can be obtained. Reliable statistics of this nature form an essential prerequisite to the development of an adequate forecasting system and a suitable Building Code to minimize the hazards resulting from wind action and flood waters.

In the Great Manila area, where concentration of population, building and industry is so high relative to other parts of the archipelago, particular attention should be paid to this problem. Because, in the past, Manila has been relatively free from typhoon attack, there is no

reason to assume that weather conditions, similar to those which caused typhoon Yoling to follow its path and cause such widespread damage and disruption, will not recur at any time in the future.

(2) Meteorological Data

Based on the nature and extent of the damage surveyed, reports of the Weather Bureau, the Manila press, and discussions held with various organizations and individuals, the Mission makes the following comments and recommendations concerning meteorological data:

(i) All damaged Weather Bureau installations should be repaired or replaced and put into working order as quickly as possible.

(ii) Such installations should be adequately designed and protected to operate under peak gust velocities ranging up to 70 m/s and should be free from attack by flying debris.

(iii) In all major installations, adequate stand-by power supplies, independent of the central electric power distribution system, should be installed to ensure continuous operation when breakdown of the main power supply develop.

(iv) A more systematic maintenance and recalibration system of weather recording equipment at all major weather stations should be put into operation. In this connexion, re-calibration might best be organized on a regional basis in collaboration with other countries and organizations operating in the south east Asian region, using "roving" systems of calibration which could be flown from one country to another as required.

(v) An improved warning system involving direct radio communication to the mass-communication media should be evolved, with interruption of regular radio and television programmes at specified time intervals for broadcasts of storm warning

signals when necessary. (The Mission understands that communication to mass-communication media from the Philippine Weather Bureau largely depends upon land-line telephones).

(vi) Because of increasing development of high-rise building and the installation of tall radio and television masts in the Greater Manila area, it is necessary for designers of such structures to have reasonable information relating to maximum wind velocities and wind velocity profiles likely to be encountered. This information can be achieved by instrumenting selected tall masts at various intervals of height, at relatively low cost. The mission recommends that such instrumentation be carried out in locations such that continuous operation and maintenance can be handled by university Institutions under the control of the Weather Bureau.

(vii) Typhoon frequency maps covering the whole of the Philippines should be prepared for each 100 km square, based on the work of Chine, referred to earlier, and on the records available in the Philippine Weather Bureau. From these frequency maps, wind-zoning maps could then be prepared, based on return-value scales as suggested by Shellard, Sherlock, Thom and others. These zoning maps should be incorporated into the revised National Building Code relating to wind forces.

(viii) Since flood damage is also a matter of serious concern, a comprehensive programme of rainfall recording should be commenced over the whole country and zoning maps of rainfall frequency and intensity prepared as for wind. To achieve the objectives recommended here and in Recommendation (vi) above, at minimum cost, it is suggested that wherever possible, selected schools should be used as Weather Stations and staff instructed accordingly, in order to ensure that records are continuous, reliable and regularly sent to the Weather Bureau.

(3) Building Regulations

It is understood that all buildings and structures erected in the Greater Manila area must be designed and constructed in accordance with the requirements of a Uniform Building Code. It is also understood that various government and professional organizations involved in the building industry in the Philippines are currently undertaking a review and revision of existing rules, with the objective of producing a more comprehensive set of Building Regulations for the Philippines in general. Because of the necessity to cater especially for the forces due to wind, earthquakes, flooding and volcanic action in varying degrees throughout the archipelago, there is a pressing need for this work. From the nature and extent of damage to buildings and other structures caused by the recent typhoons, it is apparent that either current building regulations in the Philippines are inadequate to cater for the wind forces encountered, or they are not being enforced adequately.

To eradicate deficiencies in existing building and structural engineering practice in respect of wind forces, the following recommendations are made:

(i) The Government of the Philippines should be urged to allocate adequate funds for the establishment of a well-staffed and efficient Building and Engineering Inspectorate, charged with the tasks of rigorously enforcing current Codes of Practice relating to building and engineering works and of revising and up-dating such Codes from time to time in order to meet the particular needs of the country and to keep pace with modern developments. Such an Inspectorate should be centrally controlled but could be organized on a provincial basis, with a provincial supervisor in charge of each regional office which would be responsible for issue of all building permits in the region and for enforcement of the Building Code locally. All inspectors should be well-qualified professionally. They

should have wide experience of commercial, industrial and domestic building construction and should be able and ready to advise on whether any particular structure conforms to the Code or not.

(ii) Every effort should be made to expedite work now under way on the revision of the current Code, with particular emphasis on the section dealing with wind loads on structures. Since the relevant meteorological data will take time to compile, it is suggested that, as a temporary expedient, use might be made of current regulations governing this important aspect of structural design, which are in force in neighbouring countries also subject to typhoon attack (for example Japan and Hong Kong).

(iii) If rationalization and economy of building development is to be achieved over the country as a whole, zoning maps, covering both wind intensity and rainfall intensity, should be prepared. In areas primarily rural in character, these may be done on a macroscale but in intensely-developed urban areas such as Greater Manila, additional micro-zoning will be required.

(4) Education and Training

In those areas which are subject to recurrence of natural disasters arising from typhoons, earthquakes and other violent natural phenomena, it is vitally important that the community should be made aware of practical measures which can be taken to minimize loss of life and damage to property arising from such causes. It is in the planning and organization of such educational and training programmes that Unesco could play a very important role. With this objective in mind the Mission makes the following recommendations:

(i) Unesco, in collaboration with the Government of the Philippines should sponsor a National Seminar on "Design of Buildings and Structures to resist Typhoon Winds" to be held in Manila at an

early date. Such a seminar should be essentially practical in its outlook and should cater for the building industry as a whole, embracing architects, structural engineers, building inspectors, meterorologists and building contractors. Topics for discussion might include: nature and characteristics of natural wind; its effects on buildings and structures; selection of suitable design criteria; exposure factors and local sheltering effects; likely damage patterns in relation to specific types of buildings and structures; structural design and detailing to avoid such damage; relative costs of improving design and detailing of buildings to resist wind damage, and design of glazing to resist wind forces.

(ii) Supplementary to the above seminar and arising from it, the Government of the Philippines, through its Department of Education, should be encouraged to undertake a programme of technical education and training in elementary building construction, with particular reference to the use of timber and corrugated sheeting in building work. The programme should include courses of short duration aimed particularly at carpenters, roofers, building supervisors, clerks of works and other operatives involved in such work. The individual courses should be simple in character and well-presented, with full use made of audio-visual aids and models to illustrate correct and incorrect procedures. They should be incorporated into all full-time curricula of related courses in technical education at Trade School, Technical College and University Levels. Additionally, in order to achieve maximum dissemination of knowledge in minimum time, they should be offered as special evening courses in adult education programmes and, in this latter connexion, the mass-media should be used to achieve the objectives. Much of the content of these courses should be aimed at correcting the elementary faults mentioned in the body of this report.

11. Regional Seminor on Wind Effects on Buildings and Structures

The regional seminor on wind effects on buildings and structures, sponsored by Unesco and organized by Unesco National Commission of the Philippines, was held on 22nd to 26th November, 1971, in Manila. 16 participants from 9 southeast asian countries, 5 consultants for technical direction, a representative from FAO Regional Office in Bangkok, 3 representatives from WMO Meteorological Training and Research Office in Manila, 4 representatives from ECAFE Typhoon Committee Secretariat and about 120 attendees from the Philippines were participated the seminar.

The programme of the seminar was as follows:

Day	Time	
22 Nov.	a. m.	Opening Ceremony
		Opening Remarks by Dr. G. Y. Zara
		Welcome Address by Dr. A. R. Roces
		Remarks by Dr. J. M. McDivitt
		Keynote Address by Sen. H. Z. Benitez
	p. m.	Organization Meeting
23 Nov.	a. m.	Session 1. "The Nature of Typhoons" by Dr. R. L. Kintanar
		Session 2a. "Unesco Report on Typhoons of October and November 1970 (Philippines)" by Prof. S. Mackey
	p. m.	Session 2b. "Recommended Design Loads for Wind in Philippines" by Mr. A. O. Hizon
24 Nov.	a. m.	Session 3a. "Wind effects on High-rise Buildings" by Dr. T. Okubo
	p. m.	Session 3b. "Design Methods for Wind Forces on Buildings and Structures" by Mr. A. R. Flores
25 Nov.	a. m.	Session 4a. "Materials and Construction Techniques for Wind Forces on Buildings and Structures" by Mr. C. E. Finney

	p. m.	Session 4b.	"Recommended Construction Supervision and Procedure to Minimize Damages due to Wind Effects" by Mr. A. Avecilla
26 Nov.	p. m.	Closing Meeting	Adoption of Final Report
		Closing Ceremony	Closing Address by Sen. E. Pelaez Distribution of Certificates

After each presentation in the sessions of seminar, the floor was opened to a open forum or free discussion and the seminar was closed with the general conclusions and recommendations[3] of the seminar outlined as follows:

(i) The final report is the report of the regional participants of the Seminar and thus deals with rather general concepts and recommendations which are of concern to all countries of the region.

(ii) Within the framework of the seminar, there was discussion on the need of additional research related to typhoons and was strong support for the establishment of research programmes in the region under both national and international auspices. The establishment of a research center on typhoons in the Philippines, which would have research divisions on wind effects on buildings and structures, was recommended.

(iii) The general conclusions and recommendations of the Unesco report entitled "The Typhoons of October and November 1970 (Philippine)", which concerned to the improvement of meterological data, building regulations and education and training, were unanimously endorsed by the participants of the seminar.

(iv) The improved warning systems against natural disasters, which should be based on an adequate and rapid feed-in of relevant meteorological information from a wide regional network linking observation stations in all concerned countries, were necessitated.

(v) It was recognized and agreed that the establishment and enforcement of suitable building codes were matters of critical importance.

(vi) It was agreed that there was a pressing need for a comprehensive programme of education and training designed specifically to provide updated knowledges for specialized people such as engineers, scientists, architects and building contractors and workmen and to assist the general public in combatting the ravages of natural disasters.

(vii) All programmes concerned with prevention of natural disasters should have a high degree of public involvement and participation.

(viii) All countries in the region should increase their support for scientific agencies and services concerned with the application of science and technology to lessen the impact of typhoons and other natural disasters.

(ix) Recognizing that typhoons and other natural disasters pay no attention to national boundaries, and that there is much to be gained by the sharing of information and experience through the design of regional programmes, it was requested that concerned regional and international agencies including, ECAFE, WMO and Unesco, take the results of this Seminar into account in designing their related programmes and as appropriate offer their technical and advisory services to countries where windstorms and other natural phenomena may lead further disasters which can be alleviated through collaborative action.

12. Conclusion

The author has introduced the survey results on typhoon damages in the Philippines made by the Unesco Survey Mission and the outcome of the regional seminar on wind effects on buildings and structures resulted from the activities of Survey Mission and the related gentlemen.

In both of general conclusions and recommendations of the Survey Mission and the Seminar, similar items dealing with the establishment of regional and international system on the exchange of information and experience, the establishment of research center, the improvement of education and training and refinement of structural codes and regulations are emphasized.

The next step which should be taken, the author considers, is the movement of the Governments of developed countries for cooperating to the regional and international programmes and for supporting the scientific and technological activities in the region and in each country.

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2) Mackey, S; Okubo, T and Finney, C.E.: Unesco Report on Typhoons of October and November 1970 (Philippines): Unesco Report 2387/RMO.RD/SCE. 197

3) Final Report, Unesco Regional Seminar on Wind Effects on Buildings and Structures, Manila 22-26 November, 1971: UNESCO/FSO/SEA, Nov. 1971, Manila.

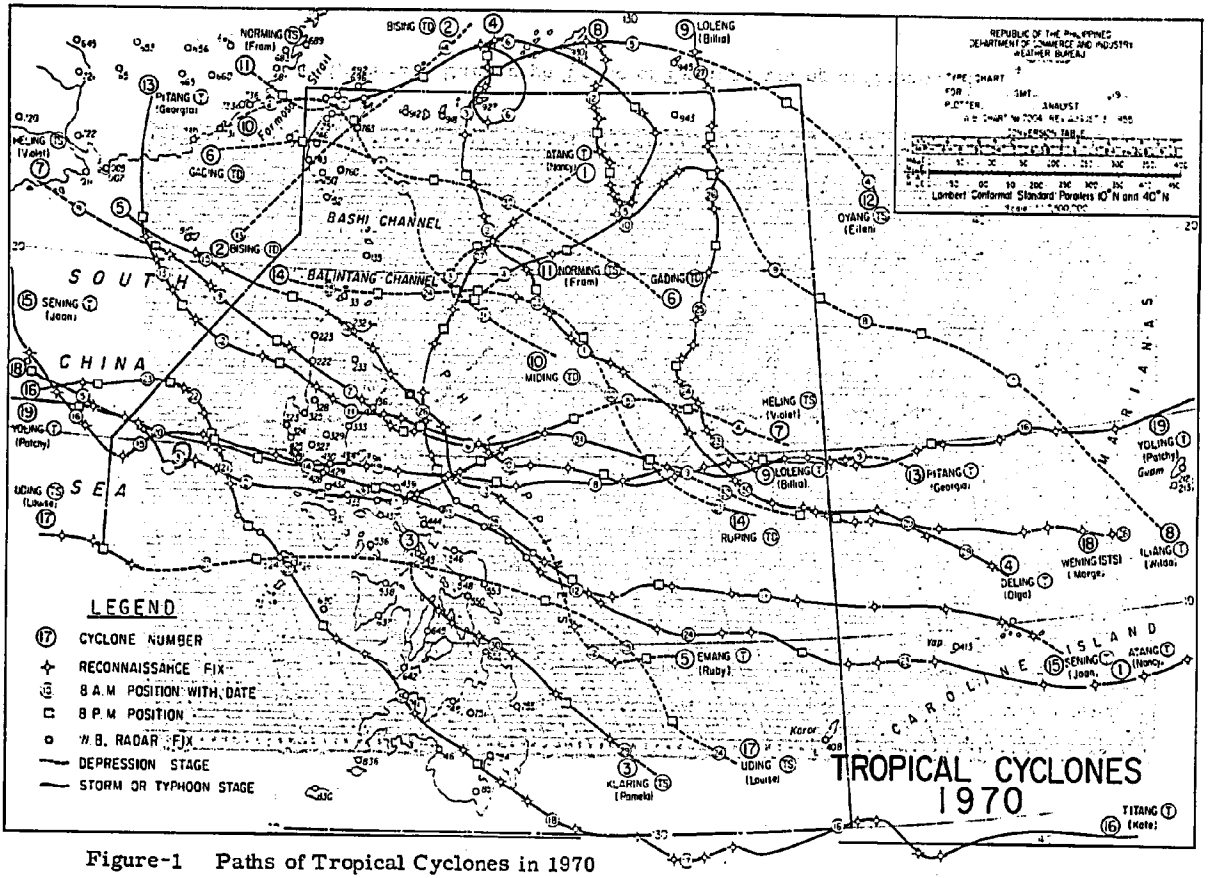


Figure-1 Paths of Tropical Cyclones in 1970

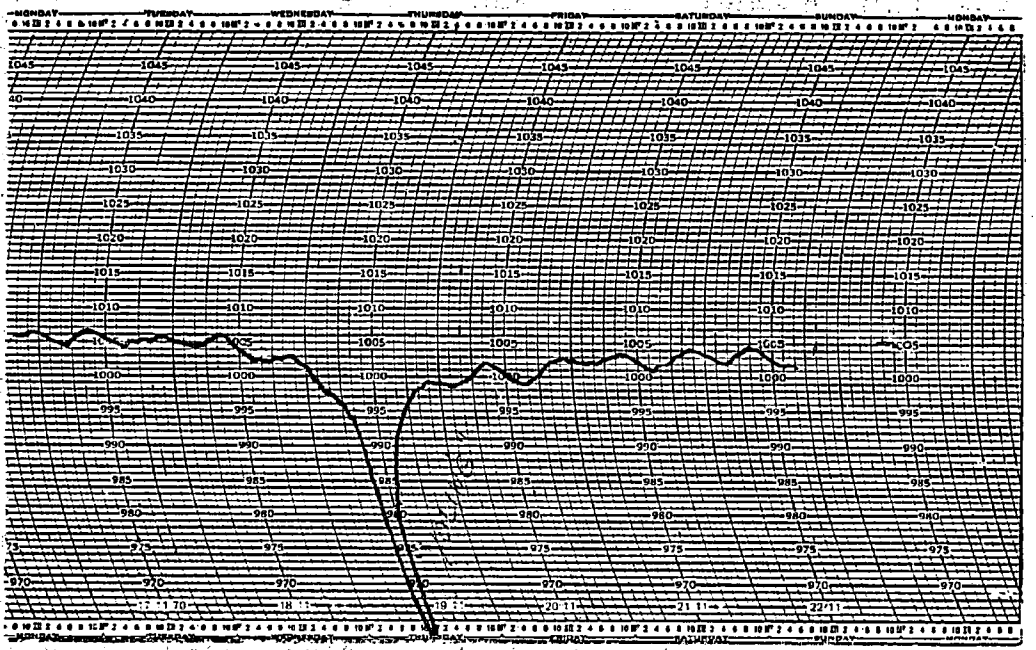


Figure-2 Barograph Trace of Typhoon Yoling at Quezon City

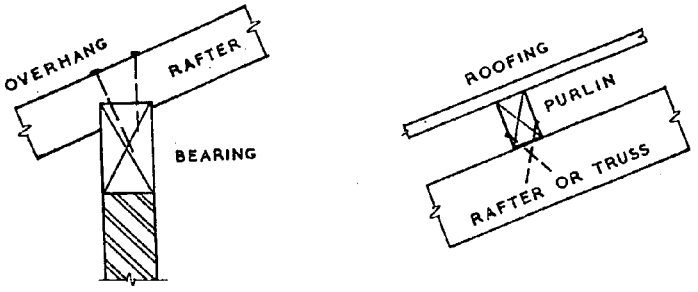


Figure-3 Toe-nailing

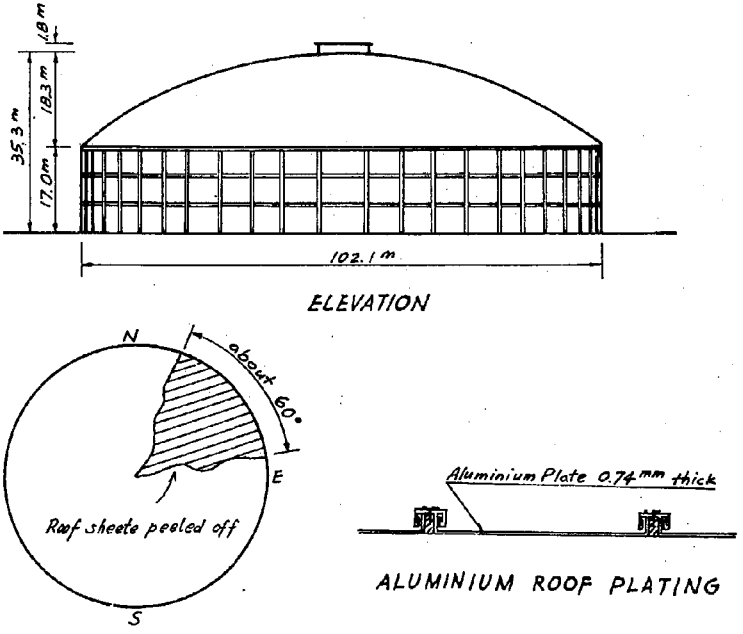


Figure-4 Wind Damage of Domed Roof
— Araneta Coliseum —

DYNAMIC BEHAVIOR OF STRUCTURES AND SUBSOIL DURING EARTHQUAKES

Earthquake Resistibility of Submerged Tunnels

by

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INTRODUCTION

Shirley Gut Siphon, which was constructed in 1894 as a part of the drainage system at Boston, U.S.A., is the first tunnel of prefabricated subaqueous tubes. A portion with length of 96 m among the total (457 m) of the Siphon, was built by prefabricated work. The portion was made of elements with length of 14.6 m and 20.0 m, and with circular cross section of diameter of 2.6 m. The number of submerged tunnels (or prefabricated subaqueous tunnels) completed afterward or under construction at the present registers fifty one in the world, for some eighty years.

The Ajigawa Highway Tunnel near Osaka, which is a two-lane tunnel started in 1935 and completed in 1944, is the first one in Japan. Among the total length of the tunnel of 80.2 m, an element with length of 49.2 m was built by prefabricated work.

Since that highways and railways have gone across rivers and bays mostly by bridges, with some exceptions such as the Kanmon Railway Tunnel (constructed between 1936 and 1944) and the Kanmon Highway Tunnel (constructed between 1938 and 1958) which are connecting Honshu and Kyushu islands and were built partially by shield work. Since the 1960's, however, tunnel construction by shield work and prefabricated work has adopted frequently.

The number of these types built since 1960 or under construction is thirteen, three by shield work and ten by prefabricated work.

It is expected that construction of tunnels of these types will become more frequent in Japan for the future, in view of considerable extension of demands for social investment as transportation facilities and sewage systems, especially in great cities.

A typical example of the demands is prefabricated undersea tunnels across the Tokyo Bay, proposed as a portion of the Tokyo Bay Loop Highways.

All over the land of Japan, similar to the State of California, U.S.A., belongs to one of the most active seismic zones in the world. Particularly, severe earthquakes have occurred frequently in the past around the Tokyo Bay.

Accordingly specific attention shall be given to the design and the construction of the submerged tunnels, for providing adequate structural resistance to seismic disturbances.

The Public Works Research Institute (PWRI), the Ministry of Construction has conducted comprehensive studies on earthquake resistibility of submerged tunnels since 1968. Also, since 1971 the Institute has commissioned to the Japan Society of Civil Engineers (JSCE) to make up uniform standards for the aseismic design of prefabricated tunnels embedded under the bottom of the sea consisting of soft soil layers.

In the present report interim results of these investigations are described briefly.

1. STATUS OF RESEARCH ACTIVITIES ON SEISMIC EFFECTS ON SUBMERGED TUNNELS IN JAPAN

Research activities on seismic effects on submerged tunnels and underground structures have been extensively carried out for these years.

First of all, investigations on earthquake-resistance of undergound pipes for electric cables have been conducted at electric companies and the Central Research Institute of Electric Power Industry.

Research works on prefabricated tunnels embedded under the water, have been practically carried out at the Ministry of Construction, the Ministry of Transport and the Japanese National Railways. The works are very comprehensive, covering geological aspects, soil mechanics, seismic activities, ground motions, aseismic design, structural details, dynamic analyses, experimental studies, etc.

Furthermore, model experiments on dynamic behaviors of tunnels and their dynamic analyses have been conducted at universities. The Institute of Industrial Science of the University of Tokyo is the leading institution on the subject.

In order to make up a reasonable earthquake regulation by summarizing the various investigations, PWRI has requested to JSCE to establish "Committee on Earthquake-Resistant Design of Submerged Tunnels." The committee (chairman is Professor Shunzo Okamoto) was organized in July, 1971 by participants from universities and governmental institutions concerned. The committee presented the first interim report[1] in March, 1972, as a result of the activities in the beginning year. The following chapter summarizes the report.

2. INVESTIGATIONS AT THE JAPAN SOCIETY OF CIVIL ENGINEERS

The chapter describes briefly the interim report[1] of "Committee on Earthquake-Resistant Design of Submerged Tunnels" organized at JSCE.

2.1 Studies on Seismic Ground Motions

The following four subjects relating to seismic ground motions, have been studied.

1) Seismic activities
2) Characteristics of seismic ground motions
3) Measurements of seismic ground motions
4) Seismic ground motions to be considered in the design

In 1) magnitudes and other characteristics of earthquakes to be considered in the design of tunnels are generally investigated, in consideration of seismicity of individual construction sites, in 2) characteristics of seismic ground motions are studies in connection with dynamic properties of grounds, in 3) measurements of

seismic motions are advised from the viewpoint of studying dynamic behaviors of grounds and underground structures and supervising the stability of these structures, and in 4) various conditions for determing seismic ground motions to be considered in the design of the tunnels are examined.

The following are principal results from the investigations in the beginning year.

a) Information on seismic activities in Japan,

b) Estimation of ground acceleration at a certain site, giving magnitude and epicenter of an earthquake,

c) Fundamental factors to be considered in determining seismic ground motions for the design.

2.2 Studies on Subsoil

In designing submerged tunnels against seismic forces, it is recommended to survey velocities of elastic waves propagating in the grounds and to obtain coefficients of subgrade reaction of the grounds, in addition to general soils surveys that are usually conducted in construction of ordinary engineering structures.

Although there are various methods to measure these quantities, specifie attention shall be paid to the magnitudes of strains in the grounds during soils surveys and also during earthquakes considered. There are several ways to estimate these quantities from the results of the standard penetration tests or other standard tests.

It seems probable that submerged tubes are sometimes constructed in sandy soil layers. Moreover, sandy soils are often employed as fill materials surrounding the tubes.

In view of experiences in Niigata and other places, loose sands are vulnerable to liquefaction during earthquakes. It seems obvious that the effects of liquefaction on tubes are very serious.

Accordingly particular consideration shall be given to avoiding the occurrence of the liquefaction phenomena, in view of the importance of these structures and their structural characters.

Several methods are available in assessing occurrence of liquefaction. It is noted, however, that some discrepancies arize between the various methods. Accordingly it seems that further investigations are needed on the subject.

In providing adequate resistance to seismic disturbances, it is also advised to clarify behaviours of cohesive soils during earthquakes, as well as sandy soils.

In addition, it seems that investigations on changes in soil properties during and after earthquakes, and on stability of embankments and slopes are essential in the proper aseismic design of tubes.

As for tubes which are constructed across or near faults, the effects of the faults during earthquakes should be studied. So for preliminary investigations have been performed through discussions with geophysical specialists on the subject, and through field surveys at faulting areas.

2. 3 Studies on Structural Design

"Earthquake Resistant Work of Water Supply Systems" issued by the Japan Water Supply Association, is only one criterion for the aseismic design of underground structures embedded in the ground. It stipulates the following matters in providing adequate structural resistance to seismic disturbances.

1) Physical properties of foundation works shall be uniform among those of adjoining structures,

2) Joints between adjoing tubes and connections between different structures shall be designed so that they are capable of resisting their dislocations,

3) Earth pressures shall be taken into account in the design. Morevoer,

it specifies concrete ways to be considered in construction of individual structures.

On the other hand, although it seems that numerous underground structures sustained seismic damage in the past, not many reliable documents are available, and comprehensive studies on the damage characteristics have not been conducted.

Earthquake-resistant design methods of date are roughly classified into two groups. One is the conventional static design method that is called the seismic coefficient method, the other is the dynamic design method in which dynamic behaviors of tubes and surrounding soils are taken into account. In Japan, the both methods are employed in the recent examples of the aseismic design of submerged tunnels.

In the dynamic design method structures are designed for critical conditions of the structures subjected to seismic wave motions propagating in the ground. Since submerged tunnels are usually embedded in the comparatively soft soil layers, it is essential to reasonably obtain dynamic properties of the soft soil layers. Although the problems to evaluate dynamic properties of soil layers are very hard to solve, it seems that the dynamic design method is an effective way to approximately grasp dynamic behaviors of these structures.

The static design method is powerful in examining the structural stability of tubes and in calculating the thickness of walls and reinforcements required in concrete structures. The method, however, seems not sufficient in examining the overall behaviors, as a structural system involving various structures and the surrounding soils. Therefore it seems that further investigations are needed, with respect to the method.

Precise dynamic responses of submerged tunnels can be obtained by following two approaches.

1) Analytical approach

2) Experimental approach

It is expected that the both methods will become powerful means in evaluating earthquake-resistance of submerged tunnels.

3. INVESTIGATIONS AT THE MINISTRY OF CONSTRUCTION

The Ministry of Construction has been conducting investigations on earthquake-resistance of submerged tunnels since 1968, for the purpose of establishing a resonable aseismic design criterion. The followings are research-oriented subjects at PWRI, which are parts of extensive studies at the Ministry.

(1) Seismicity

(2) Measurements of Seismic Responses of Grounds Using Down-hole Seismometers

(3) Analyses on Seismic Responses of Subsoil

(4) Dynamic Analyses of Underground Structures

(5) Model Experiments on Dynamic Behaviors of Submerged Tunnels

The subjects are individually discussed in the followings.

3.1 Seismicity [1), 2)]

Epicenters of damaging earthquakes since the dawn of history (A.D. 700) are indicated in Fig. 1. It is noted from the figure that any structures throughout the country (with limited areas of exception) may be strongly affected by seismic effects and that Tohoku, Kanto, Tokai, and Nanki regions have experienced especially numerous numbers of great earthquakes.

The followings are the results of the survey on seismicity around the Tokyo Bay.

Fig. 2 indicates epicenters of earthquakes that caused damage in the Tokyo Bay area between 818 and 1924. In the figure larger circles

denote epicenters of earthquakes with magnitude of 7.9 (on the Richter scale) or greater, and smaller circles are those with magnitude of 7.8 or smaller. Furthermore, black ones, shaded ones and white ones are roughly corresponding to earthquakes with maximum JMA seismic intensities around the Tokyo Bay (Yokohama, Tokyo or Chiba) of 7, 6 and 5, respectively.

Fig. 3 completely covers major earthquakes which occurred between 1961 and 1970 within a circle with its center at the midpoint of the Tokyo Bay (35.5°N and 139.85°E) and with a radius of 150 km. In the figure earthquakes are classified according to magnitudes and hypocentral depths.

Fig. 4 shows relationships between M and $\Sigma N(M)$, for three concentric circles with radii of 50 km, 100 km and 150 km. In the figure M denotes magnitude of earthquake on the Richter scale, and $\Sigma N(M)$ means accumulated numbers of earthquakes (expected to occur per 100 years) of magnitude M and greater. Assuming that log $\Sigma N(M)$ varies linearly with M, it is expected for 100 years that an earthquake with magnitude of about 7 will occur within the circle of radius of 50 km, that one with about 7.5 will occur within the circle of radius of 100 km, and also that one with about 8 will occur within the circle of radius of 150 km. It is noted, however, that the assumption of the linearity between M and $\Sigma N(M)$ seems to yield earthquakes with slightly greater magnitudes, as suggested in the curves in the figure. Conclusively,

Conclusively, for the aseismic design of structures it seems very significant to adequately evaluate seismicity to be considered at the structural sites.

3.2 Measurements of Seismic Responses of Grounds Using Downhole Seismometers [3]

Downhole seismometers are installed at three sites around the Tokyo Bay, as shown in Fig. 5. These were set up in 1970 and 1971 by the Ministry of Construction, to measure dynamic behaviours of the surface soil layers during actual earthquakes.

At the site of the Futtsu Cape, indicating a comparatively uniform sand layer, three-component downhole seismometers (acceleration transducers) are equipped at three levels; surface, -70 m and -110 m. At the Ukishima Park, consisting of reclaimed soils, silts and sands from the surface, four seismometers are installed at surface, -27 m, -67 m and -127 m. At Kannonzaki, indicating a rocky layer with a surface soil layer of 2 m deep, three seismometers are set up at surface, -80 m and -120 m. Furthermore, a SMAC-type strong-motion accelerograph is also equipped on the ground surface at each of these sites, in order to surely obtain ground acceleration records during strong earthquakes.

Fig. 6 shows a set of seismic records obtained at the Ukishima Park during a moderate earthquake (with a magnitude of 4.8 and its hypocenter at 40 km directly underneath the observation site) on September 30, 1970.

Fig. 7 indicates relations between maximum accelerations measured at various levels underground and depth, taking accelerations at the surface as units. It seems obvious that the results from the measurements will give valuable information in evaluating seismic effects on underground structures like prefabricated subaqueous tunnels.

3.3 Analyses on Seismic Responses of Subsoil [4]

To analytically evaluate seismic motions in soil layers, dynamic responses of surface layers were computed by means of the Kanai's multi-reflection method, which is to numerically analyze seismic responses of grounds considering the propagation of shear waves in multi-layered grounds.

Analyses were conducted for the ground at the Ukishima Park, where seismic records are available as mentioned in 3.2. Since shear velocities at the site are not clarified, twelve models with various shear moduli, shown in Fig. 8, were analyzed. Fig. 9 shows some of the results obtained. Fig. 9(a) indicates the results for Model 1 subjected

to an incident wave measured at the ground surface, and Fig. 9(b) is those for Model 1 subjected to an incident wave measured at the lowest (-127 m). Fig. 10 compares the results analyzed with measured ones. From these the degree of discrepancies between analyses and measurements may be known for the case. It seems essential to adequately estimate analytical models in these analyses.

In addition to the analyses mentioned above, dynamic analyses of surface grounds were performed by means of the finite element method.

3.4 Dynamic Analyses of a Submerged Tunnel [4]

Dynamic analyses were conducted for a submerged tunnel proposed across the Yokohama Bay, shown in Fig. 11. Dynamic behaviors of sections of grounds and of sections of tunnel together with the surrounding soils were analyzed at three various transverse sections. Fig. 12 indicates finite element models for the three sections.

The followings briefly describe the results for Section B (tunnel and surrounding soils). Assuming that the section is in plane strain and has a damping ratio of 10%, and that a shear wave velocity is 50 m/sec for the surrounding soil, the fundamental period of about 2.6 seconds is obtained. Fig. 13 indicates the maximum responses of displacements, and accelerations obtained for Section B when subjected to the average response spectra proposed in 1964 by PWRI [5], with the maximum incident acceleration of 200 gals.

3.5 Model Experiments on Dynamic Behaviors of a Submerged Tunnel [6]

1) Summary

Model experiments on dynamic behaviors of a prefabricated submerged tunnel were performed. The prototype is shown in Fig. 11. Three models consisting of soil layers and a tunnel were fabricated on a large-size shaking table. In the experiments similarity relations between the prototype and the models were considered. To statisy

the similitude gelatin was employed as ground soils, and rubber as the tunnel which will be actually made of reinforced concrete.

2) Elastic Properties of Gelatin

It is noted that gelatin can be advantageously used for soft soil layers in model experiments, due to the fact that its Young's modulus varies from about 0.04 to 2 kg/cm^2 according to the content of gelatin in water and the temperature. Prior to the dynamic experiments, elastic properties of gelatin were examined through static compression tests. Fig. 14 summarizes the result. From the figure it is noted that Young's modulus changes considerably with the temperature. Therefore it is essential in the experiments to control the temperature of the gelatin models.

3) Models in the Dynamic Experiments

Three models (shown in Fig. 15) were fabricated in consideration of model to prototype ratios of 1/500 in length and 1/5 in time. Model 1 is a model of two-layered ground with the upper layer with shear velocity of 100 m/sec and uniform depth of 25 m, and the lower layer with 300 m/sec and uniform depth of 95 m. Model 2 is of two-layered ground with varying depths. Model 3 includes a tunnel in the ground of Model 2. Table 1 summarizes the outline of the prototype and the models.

4) Outline of the Dynamic Experiments

Dynamic experiments were conducted using a large-size shaking table possessed by the National Research Center for Disaster Prevention, Science and Technology Agency. The following three steps of experiments were performed, respectively, for the three models in two horizontal directions.

a) Harmonic Excitation

b) Free Damped Vibration

c) Seismic Excitation

Deformation of the models was measured using an optical displacement meter unbonded to the models, and two 16 mm moving cameras.

5) Results of the Experiments

Fig. 16 indicates resonance curves when subjected to harmonic excitation, and Fig. 17 illustrates mode shapes during resonant states. From these it is found that the fundamental mode shapes are of shear deformation of the ground and that for Model 2 and Model 3 there are two predominant natural periods which are corresponding to the resonances of the two different sections. Fig. 18 illustrates mode shapes obtained from one of 16 mm moving cameras. It is obvious from Fig. 18(a) that the corners tend to vibrate with greater amplitudes than other portions. Since this seems different from actual behaviors for the prototype without any particular corners, it is advised that the effects of the corners are removed.

Fig. 19 indicates damping characteristics of the models, which are obtained from free damped oscillation. For Model 1 that has uniform soil layers, damping ratio seems constant regardless of amplitude, but for Model 2 and Model 3 that have varying soil layers, damping ratio varies with amplitude. The damping ratio was generally found to be 1% of critical for the most cases, the damping ratio, however, for longitudinal oscillation of the sections with the deeper soft layer, was about 2.5%.

Fig. 20 shows typical examples of responses of Model 2 and Model 3 when subjected to seismic excitation in the longitudinal directions. Time-acceleration records employed for the models are adjusted from seismic acceleration records obtained at El Centro during the Imperial Valley Earthquake of 1940 and at Hachinoe during the Tokachi-oki Earthquake of 1968. White noise oscillations that are roughly flat between 0.2 and 5 Hz for the prototype, was also considered. In adjusting the seismic records and the white noise, a model-to-prototype ratio of 1/5 was taken for time from the similarity requirements, and maximum

accelerations were decided about 50 gals and 100 gals, in consideration of accuracy of measuring instruments.

For the three models, displacement responses to the Tokachi-oki record are greater than those to the El Centro record. This seems due to the fact that response spectra of the Tokachi-oki record are more predominant for structures with longer periods, than those of the El Centro record.

4. CLOSING REMARKS

Regarding to earthquake-resistance of submerged tunnels, technical knowledge up to the present, a summary of the research work reported recently from JSCE, and the results of investigations at the Ministry of Construction are briefly described in the report. It is expected in the near future to complete resonable specifications for the earthquake-resistant design of prefabricated subaqueous tunnels, through continuing research activities and summarizing various results.

It is obvious that liquefaction of sands during earthquakes is a crucial factor affecting the aseismic stability of underground structures. Although the topic is not discussed in the present report, investigations on the effects on submerged tunnels should have been carried out at PWRI and the other institutions concerned.

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Table 1 Comparison of Prototype and Model

Prototype or Model		Prototype		Model (Required from Similitude)		Model (Actual)	
	Layer	First Layer	Second Layer	First Layer	Second Layer	First Layer	Second Layer
Ground	Material	Clayey silt	Siltstone	Gelatin	Gelatin	Gelatin	Gelatin
	Dimension (plan)	750 m × 1,350 m		1.50 m × 2.70 m		1.50 m × 2.70 m	
	Depth	30 - 55 m	95 - 70 m	0.06 - 0.11 m	0.19 - 0.14 m	0.06 - 0.11 m	0.19 - 0.14 m
	Shear Wave Velocity	100 m/sec	300 m/sec	1.0 m/sec	3.0 m/sec	1.1 m/sec	3.0 m/sec
	Shear Modulus	153 kg/cm^2	1,830 kg/cm^2	0.0102 kg/cm^2	0.0917 kg/cm^2	0.013 kg/cm^2	0.09 kg/cm^2
	Young's Modulus	459 kg/cm^2	5,320 kg/cm^2	0.0306 kg/cm^2	0.266 kg/cm^2	0.040 kg/cm^2	0.27 kg/cm^2
	Unit Weight	1.5 t/m^3	2.0 t/m^3	1.0 t/m^3	1.0 t/m^3	1.0 t/m^3	1.0 t/m^3
	Poisson's Ratio	0.5	0 0.45	0.5	0.45	0.495	0.495
	Fundamental Period	2.0 sec		0.4 sec		0.37 sec	
Tunnel	Material	Reinforced Concrete		Rubber		Rubber	
	Dimension (plan)	37.4 m × 1,350 m		0.075 m × 2.7 m		0.075 m × 2.7 m	
	Depth	8.45 m		0.017 m		0.017 m	
	Young's Modulus	270,000 kg/cm^2		15 kg/cm^2		12 kg/cm^2	
	Unit Weight	2.5 t/m^3 (Net 1.0 t/m^3)		1.25 t/m^3 (Net 1.0 t/m^3)		1.35 t/m^3 (Net 1.0 t/m^3)	

Similitude

Length $n = L_p / L_m = 50$

Density $m = \rho_p / \rho_m = 1.53$ (first layer), 2.04 (second layer)

Time $t = T_p / T_m = 5$

Velocity $v = V_p / V_m = n/t = 10$

Acceleration $a = A_p / A_m = n/t^2 = 20$

Young's Modulus $e = E_p / E_m = m n / t^2 = 1.5 \times 10^4$ (first layer), 2.0×10^4 (second layer)

Poisson's Ratio $v = V_p / V_m = 1$

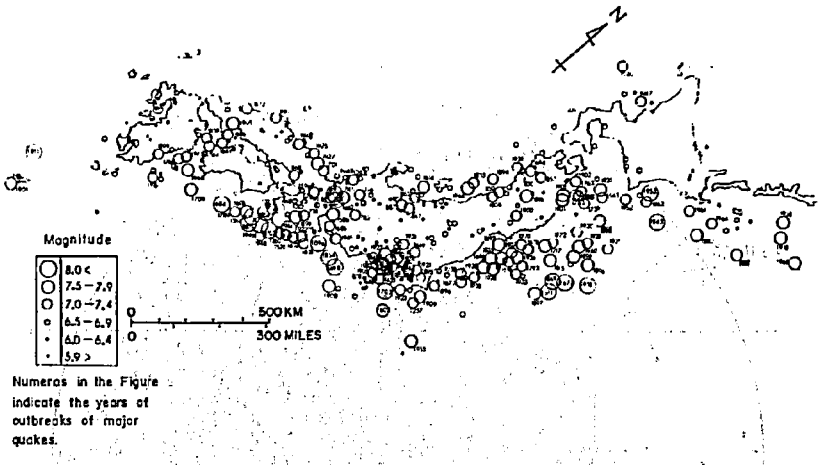


Fig. 1 Epicenters of Earthquakes Caused Damage in Japan between 701 and 1968.

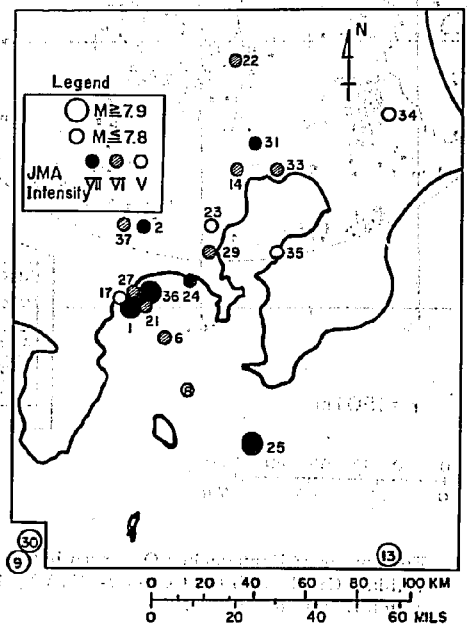


Fig. 2 Epicenters of Earthquakes Caused Damage in Tokyo Bay Area between 818 and 1924.

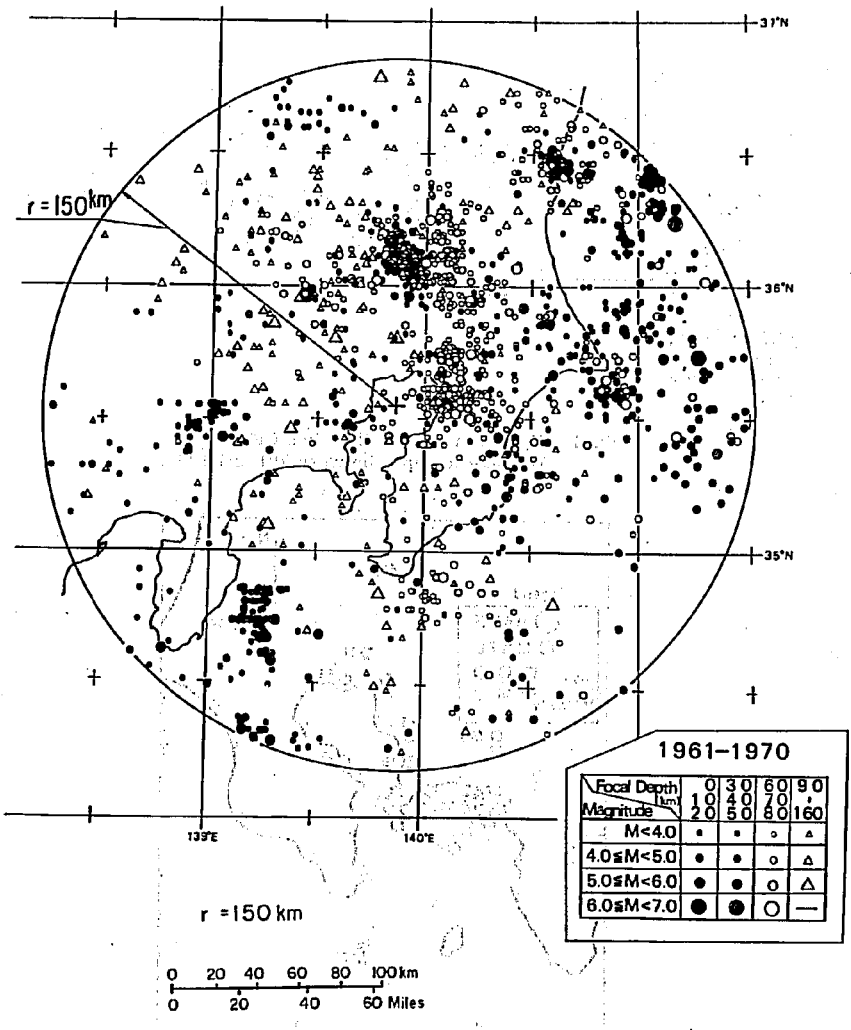


Fig. 3 Epicenters of Earthquakes Occurred between 1961 and 1970 within a Circle with the Center at 35.5°N, 139.85°E and with a Radius of 150 km.

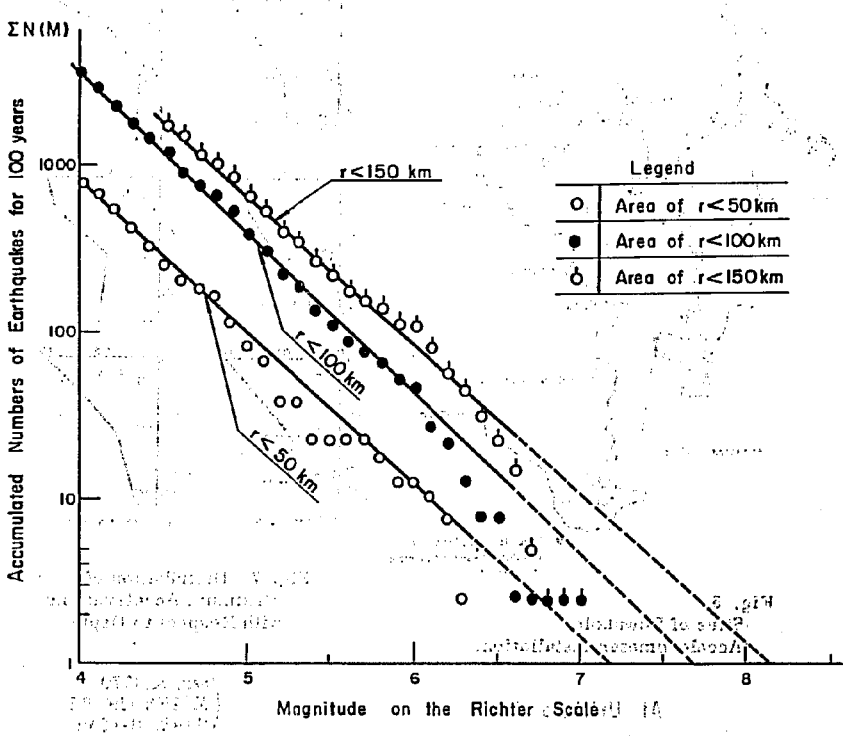


Fig. 4 Relation between Magnitudes and Numbers of Earthquakes Greater than M, Around Tokyo Bay Area.

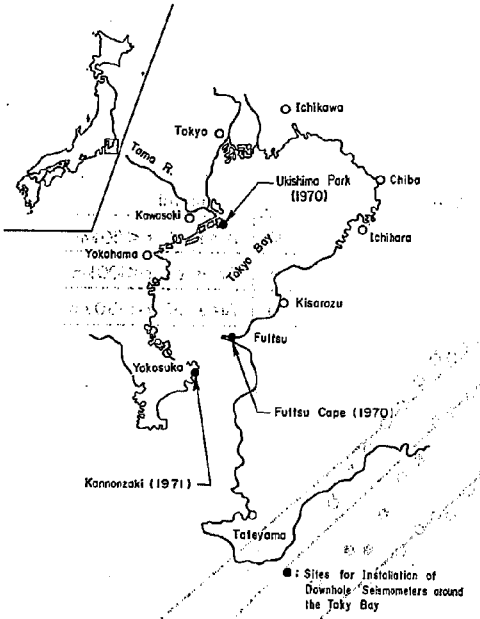


Fig. 5 Sites of Downhole Accelerometer Installation.

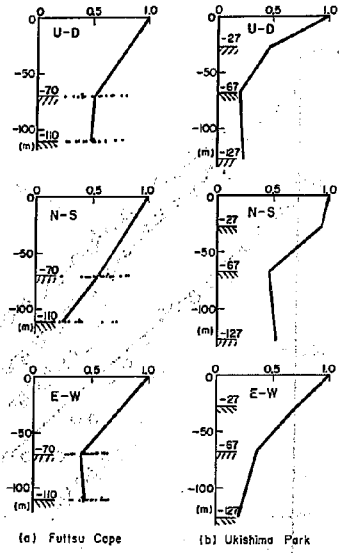


Fig. 7 Distribution of Maximum Accelerations with Respect to Depth

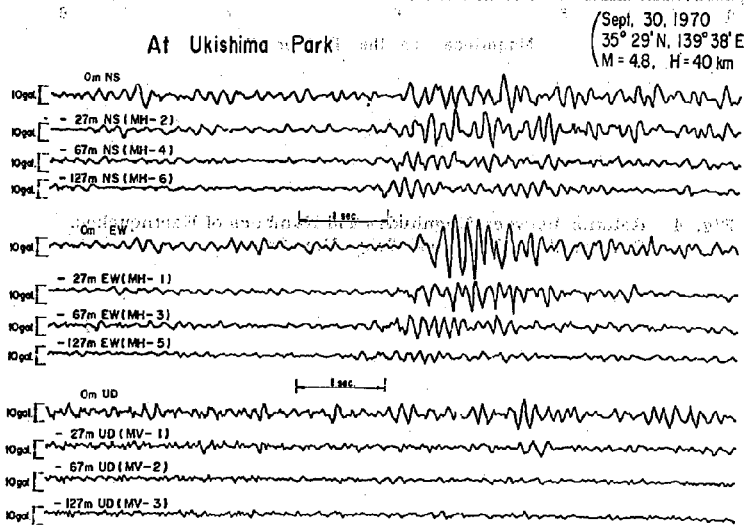


Fig. 6 Seismic Records from Downhole Accelerometers.

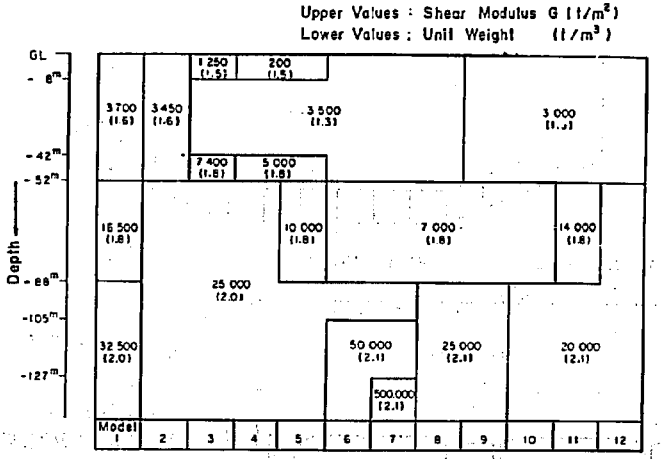
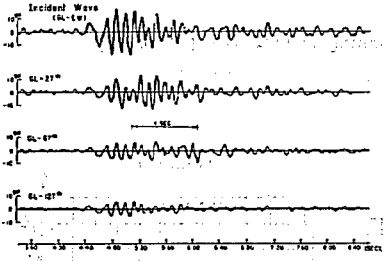
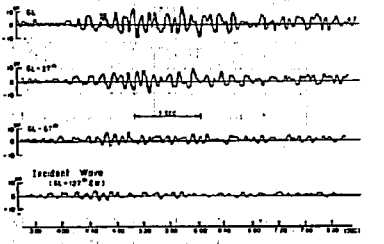


Fig. 8 Various Models of Surface Soil Layers at the Ukishima Park for Analyses by Multi-Reflection Theory.

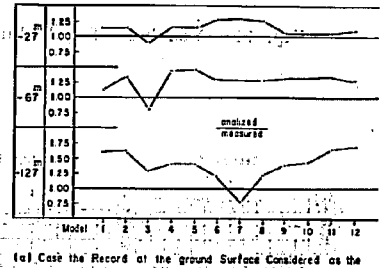


(a) Case the Record at the ground Surface Considered as the Incident Wave (for Model 1)

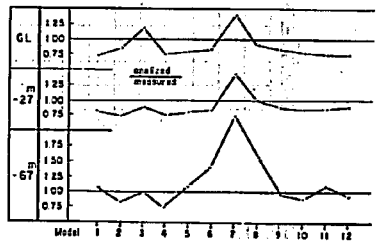


(b) Case the Record at -127m Wave Considered as the Incident Wave (for Model 1)

Fig. 9 Examples of Analyses by the Multi-Reflection Theory.



(a) Case the Record at the ground Surface Considered as the Incident Wave (for Model 1)



(b) Case the Record at -127m Wave Considered as the Incident Wave (for Model 1)

Fig. 10 Rations of Analyzed Accelerations to Measured Ones.

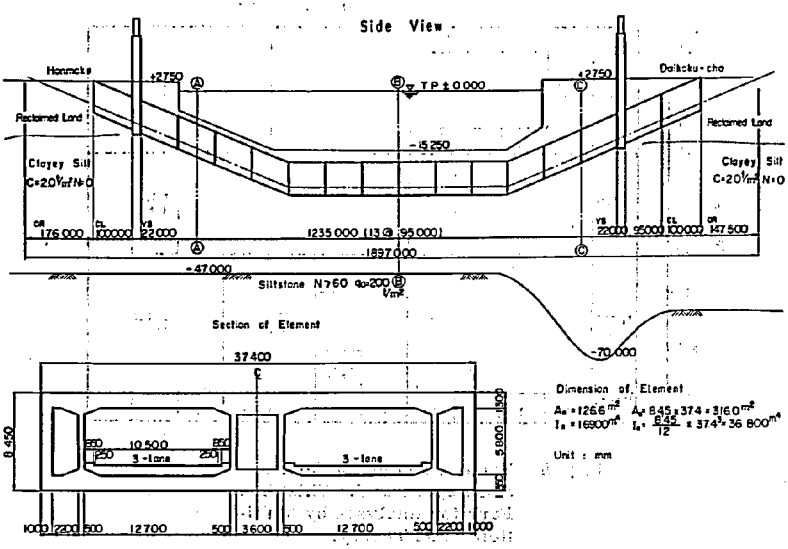


Fig. 11 Yokohama Bay Undercrossing Tunnel (Proposed)

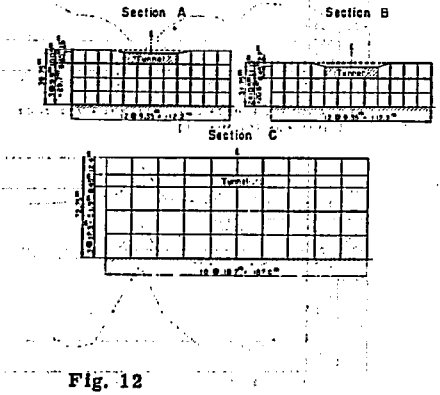


Fig. 12 FEM Models for Three Sections

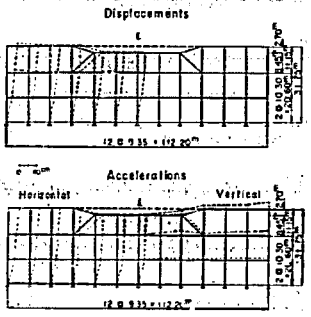


Fig. 13 Results of A Dynamic Analysis for Section B.

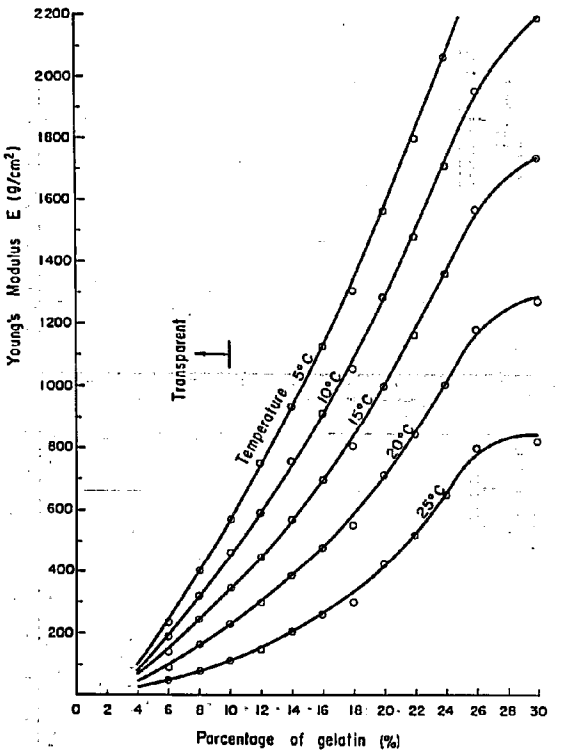


Fig. 14 Elastic Constants of Gelatin

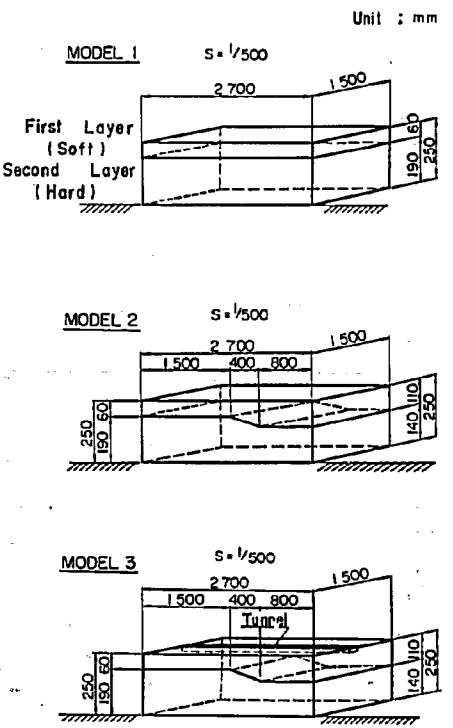


Fig. 15 Three Models Experimented.

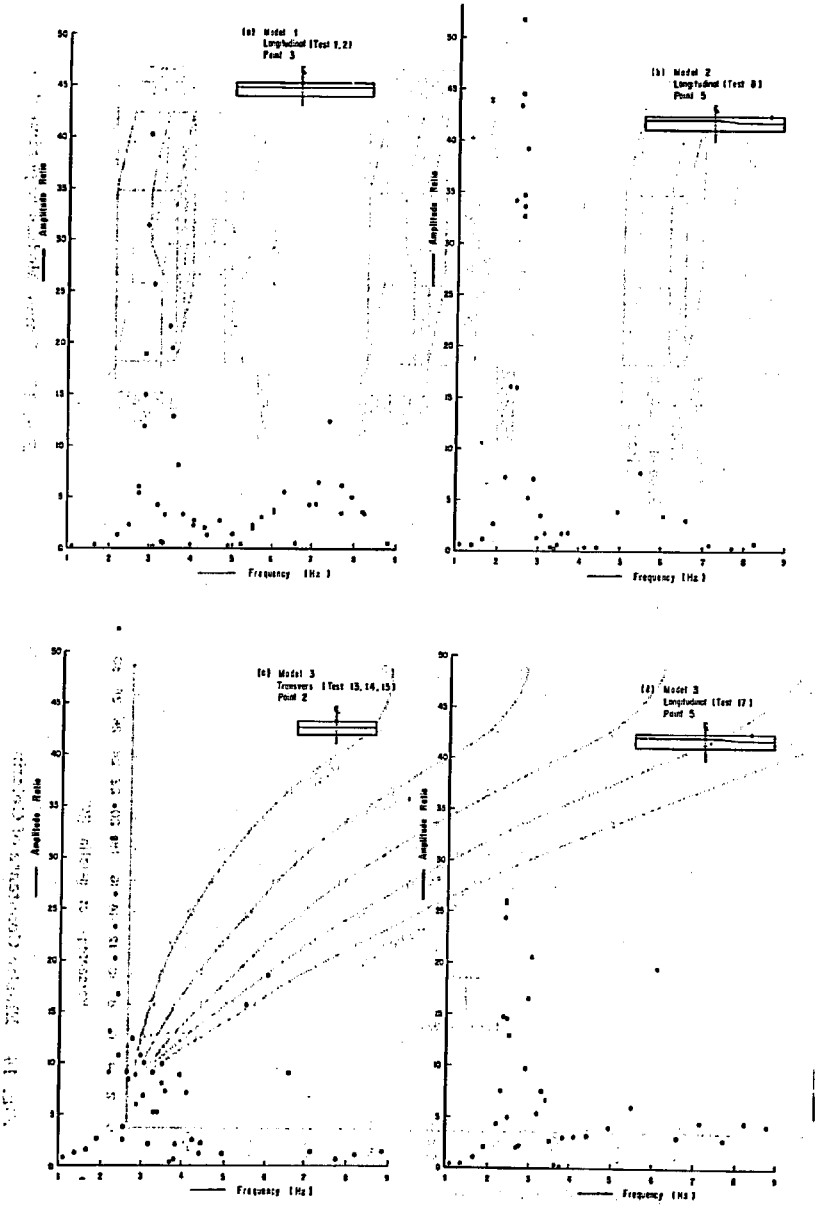


Fig. 16 Resonance Curves (Frequency vs. Amplitude Ratio)

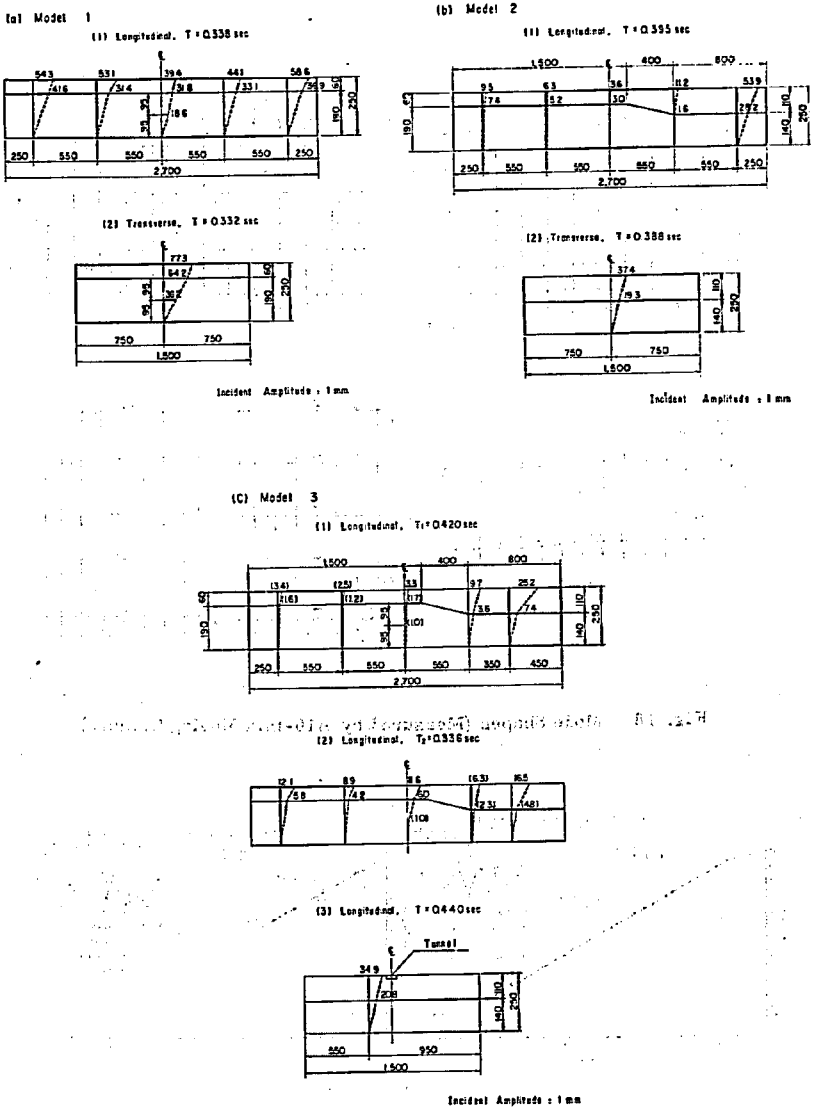


Fig. 17 Mode Shapes (Measured by An Optical Displacement Meter)

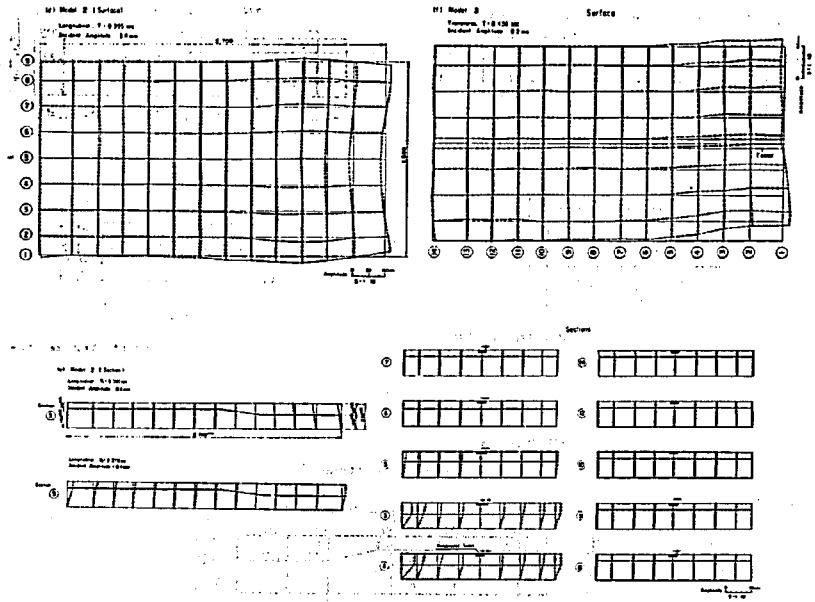


Fig. 18 Mode Shapes (Measured by A16-mm Moving Camera)

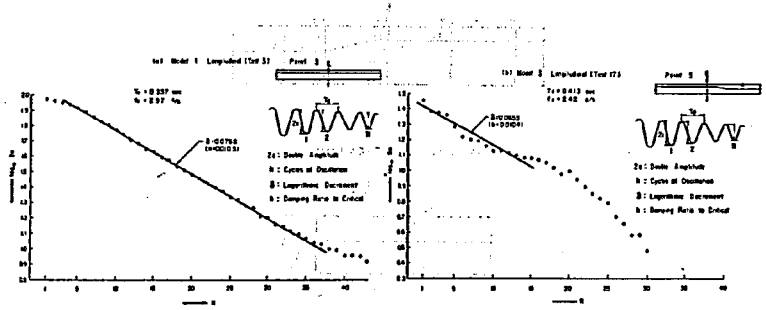
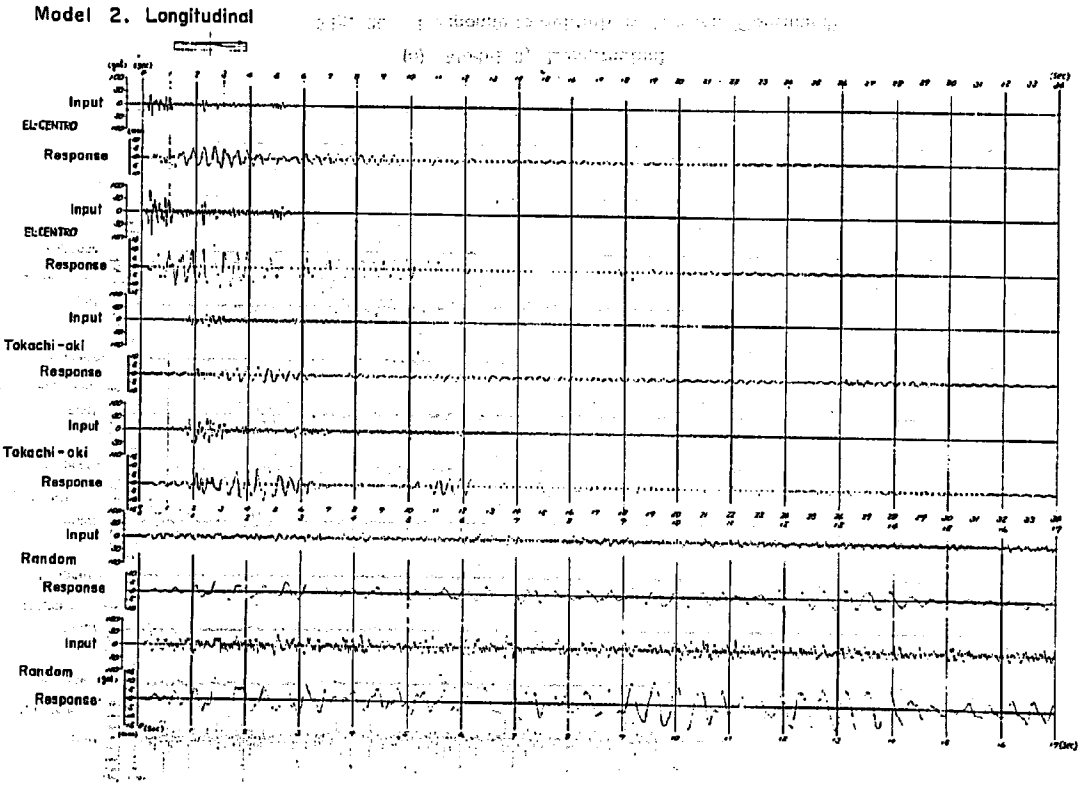
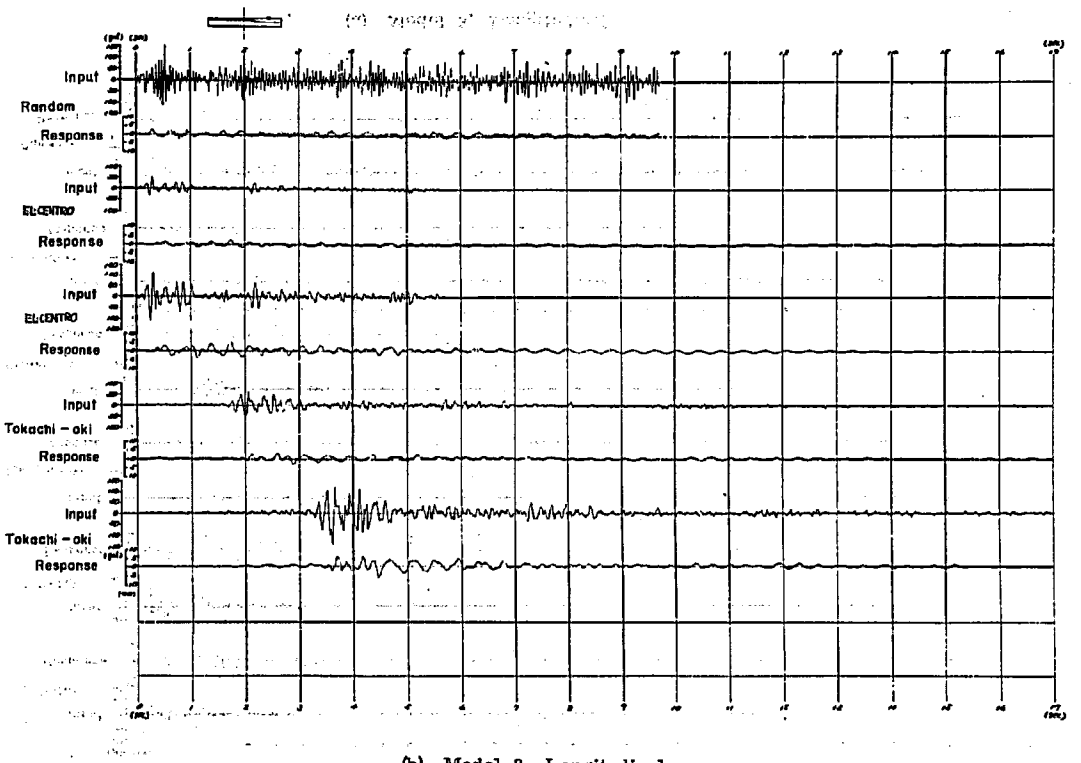


Fig. 19 Damped Oscillation



(a) Model 3, Longitudinal

Fig. 20 Response to Seismic Excitation



(b) Model 3, Longitudinal

Fig. 20 Response to Seismic Excitation (Continued)

DEFLECTION CRITERIA FOR ASPHALT PAVEMENTS

Sadao Nagumo*, Minoru Tsukinari*, and Seiichi Tanimoto*

ABSTRACT

The structural design of asphalt pavements in Japan is based on the CBR method, but recently attempts have been made to utilize surface deflection in the design of pavements for lightly trafficed roads and those on weak subgrades as well as in the determination of overlay thicknesses.

The Ministry of Construction carried out performance surveys of pavements on about 750 sites of national highways and about 3,600 sites of lightly trafficed roads during the period 1963 - 1965, and 1964 - 1967 respectively.

Full-scale experimental pavements have been constructed successively since 1967. The results of observations of those experiments have indicated that the stabilization or treatment of base or subase materials with binders is quite effective in reducing the surface deflection and that the surface deflection is much influencing on pavement failures.

From the results of performance surveys on existing roads and experimental pavements, the authors obtained the following formula which expresses the relationship between the critical deflection for asphalt pavements and the accumulated traffic volume:

$$\log_{10}N = 0.179\,d^2 - 1.117d + C$$

where N: Accumulated traffic volume of heavy commercial vehicles, when the pavement becomes to require heavy repairs ... (vehicle/in one direction)
d: Critical deflection expressed by Benkelman Beam rebound deflection using a wheel load of 4.1 tons ... (mm)
C: Constant, 6.772 for general national highways and 6.385 for lightly trafficed roads, might be a little varied depending upon the layer thickness of asphalt mixtures and the types of bases and subbases.

When this relationship is applied to the surface deflection of the newly constructed pavements of national highways, it is predicted that no large-scaled repair works will be necessary for at least five years after construction for medium trafficed roads (traffic volume of heavy commercial vehicles: 1,000 vehicles/day, in one direction).

In the design of pavements of lightly trafficed roads which consists of a comparatively thin bituminous surfacing and a base on the top of the existing gravel roads, the authors obtained a design curve which makes it possible to determine the additional layers thickness to give a target deflection on the surfaces.

1. INTRODUCTION

The structural design of asphalt pavements in Japan have been based on the CBR method. Recently, however, serious pavement failures were often observed, particularly on suburban roads of major cities due to rapid increases in the traffic volume and the weight of vehicles. In order to cope with such a situation hot-mixed bitumen-bound bases came to be adopted tentatively, and after the AASHO Road Test results were published in 1962, this trend was further encouraged. In 1967, the Japan Road Association revised the manual and introduced the following formulae for the structural design of asphalt pavements: [1) 2)]

$$H = \frac{58.5\,P^{0.4}}{CBR^{0.6}} \quad \cdots\cdots\cdots\cdots\cdots (1)$$

$$T_A = \frac{12.5P^{0.64}}{CBR^{0.3}} \quad \cdots\cdots\cdots\cdots\cdots (2)$$

where H: Pavement thickness (cm)
P: Design wheel load (t)
CBR: CBR of subgrade soil after 4 days' soaking
T_A: Pavement thickness when all the materials are assumed to be equivalent to hot mixed asphalt concrete (cm)

The design wheel load P can be obtained from the following formula:

$$3,650 \times \sum_i N_i P_i^4 \times \alpha = 10^6 \times P^4 \quad \cdots\cdots\cdots\cdots (3)$$

where P_i: Wheel load of travelling vehicles (t)

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N_i: Daily traffic volume of vehicles of wheel load P_i

α : Rate of increase in the estimated traffic, five years after construction.

Fig. 1 shows the design wheel load calculated from the actually measured wheel load distribution on national highways.

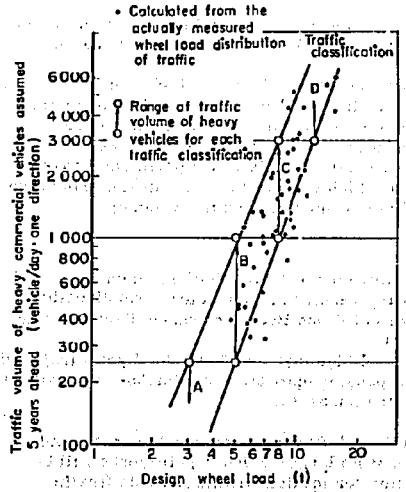


Fig. 1 Relationship between Traffic Classification and Design Wheel Load

The CBR of subgrade soil is usually obtained by the test made on the compacted sample of natural moisture content. In the case of wet cohesive soil (volcanic ash type), an unrealistically low CBR is often obtained and result in over-estimation of pavement thickness when the above-mentioned formulae are used. Also the subgrade of such a low CBR will fall beyond the scope of application of the original CBR design curve, and it has been tried to examine the pavement structure by utilizing the theoretical solution of a multi-layered system.

The formulae could be applied to the pavements of lightly trafficed local roads in the same way, but the special circumstances involved in Japanese local road pavements--i.e., they are beeing constructed on the existing gravel roads and particularly with smaller contracts and a great deal of sites -- are making it generally not practicable to measure subgrade CBRs, and it has become a usual practice to examine the pavement structure based on the surface deflection.

One of the simplest methods for evaluation of the pavement performance is the surface deflection measurement, and when trials are made to design pavement based on the deflection, it will become necessary to specify the criteria for the level of deflection that can be allowed to pavements.

2. PERFORMANCE SURVEYS OF ASPHALT PAVEMENTS

2.1 A Survey of Pavements of National Highways

The object of these surveys is to investigate the causes of pavement failure and to examine the effect of various factors on it. The collected data through the survey were also expected to supply a basic information for examining the previous method of the structural design. These surveys were carried on national highways by regional working offices, the Ministry of Construction, during the period 1963 - 1965, and the number of surveyed sections reached approximately 750. The investigation was made on the items concerning road environments such as pavement width, whether in cut or fill, traffic volume of heavy commercial vehicles as well as those concerning pavement structures and performance such as layer thickness, type of construction, Benkelman Beam deflection and cracking index. All these informations were collected by means of punched cards.

The performance evaluation was made simply on the basis of the amount of surfacing cracks in terms of cracking index, because the surfacing cracks are considered to stimulate the development of pavement failure excessively in Japan where it is rainy, and pavement engineers are liable to regard surface cracks as pavement failure itself. The cracking index is calculated as follows:

$$\text{Cracking Index} = \frac{\left(\begin{matrix}\text{Sum of}\\ \text{cracked areas}\end{matrix}\right) + \left(\begin{matrix}\text{Sum of}\\ \text{patching areas}\end{matrix}\right)}{\left(\begin{matrix}\text{Length of sur-}\\ \text{veyed section}\end{matrix}\right) \times \left(\begin{matrix}\text{Width of sur-}\\ \text{veyed section}\end{matrix}\right)} \times 100\ (\%) \quad \ldots\ldots\ldots\ldots\ (4)$$

Benkelman Beam deflection measurements are made to take rebound deflection under wheel-load 4.1 tons (tire size: 9.00 - 20 in., air pressure: 100 lb/sqi).

According to the results of consideration on the causes of pavement failures made by field engineers as shown in Table 1, it is obvious that the pavement failure should be markedly

influenced by the traffic volume and bearing capacity of bases and subbases. As a result of statistical analysis, it was found that the types of bases and subbases were significant in the correlation with a cracking index, and also that the layer thickness of asphalt mixtures had some relation to the index.

The development of cracks on the pavement of national highways becomes conspicuous as the accumulated traffic volume increases, and the degree of crackings varies with surface deflection as shown in Fig. 2. The term "heavy commercial vehicles" used here includes trucks of unladen weights exceeding five tons, buses having more than thirty seats, and special vehicles such as crane trucks. The general types of base materials in Fig. 2 are graded crushed rock and dry bound macadam, and the layer of asphalt mixtures includes those of asphalt concrete, hot-mixed bitumen-bound granular materials and asphalt penetration macadam. The tendency of an increase in cracking index is thought to be expressed by a downwards convexed curve in Fig. 2, and it has been assumed that cracking indexes in about 90% of the surveyed sections are lower than the level indicated by the curve.

When the layer thickness of asphalt mixtures varies, the tendency of development of cracks also varies even if the surface deflection remains the same, and for instance the layer thickness reduces to 5 - 10cm as shown in Fig. 3, the cracking index becomes higher than in Fig. 2.

The effect of cement stabilization or treatment of base materials on surface cracks could also be clarified, as shown in Fig. 4, that the pavements having cement bound bases show the comparatively lower cracking index than those having the other types of bases at the same surface deflection.

TABLE 1. Causes of Pavement Failure Pointed out by the Field Engineers.

Causes of pave-ment failure	Number of cases	%	Causes of pave-ment failure	Number of cases	%
Increase in traffic volume	63	19.1	Insufficient compaction of subgrade	20	6.1
Weak subbase	46	14.0	Poor drainage	19	5.8
Settlement of fill	27	8.2	Hardening of asphalt binder	18	5.5
Insufficient pave-ment thickness	27	8.2	Weak subgrade	14	4.2
Frost heave	25	7.6	Others	71	21.3

Note) Other causes are,
- Low binder content
- Cracking of cement stabilized layer
- Seasonal period of construction
- Poor asphalt mixtures
- Tire chain
- Aggregate of low grade.

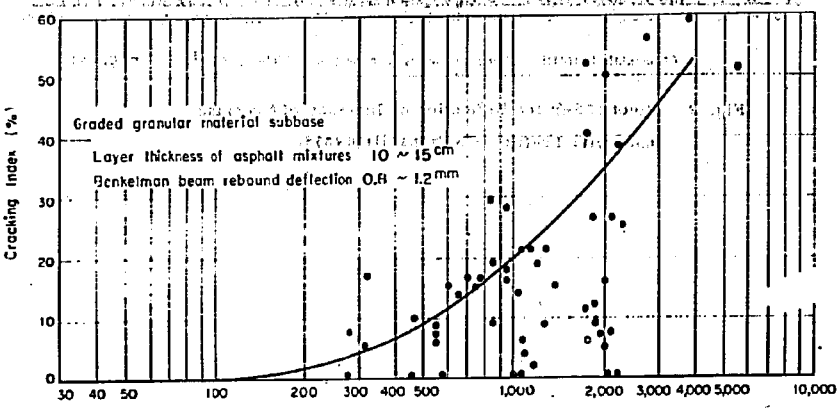


Fig. 2-1

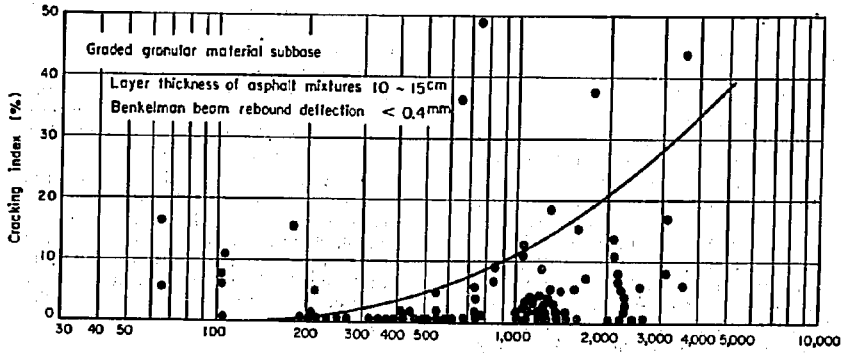


Fig. 2-2

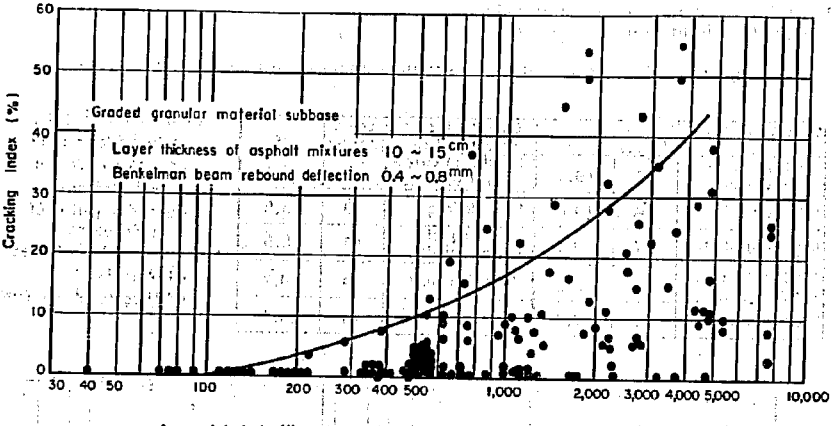


Fig. 2-3

Fig. 2 Effect of Surface Deflection on Increase of Cracking Index with Traffic, - National Highways.

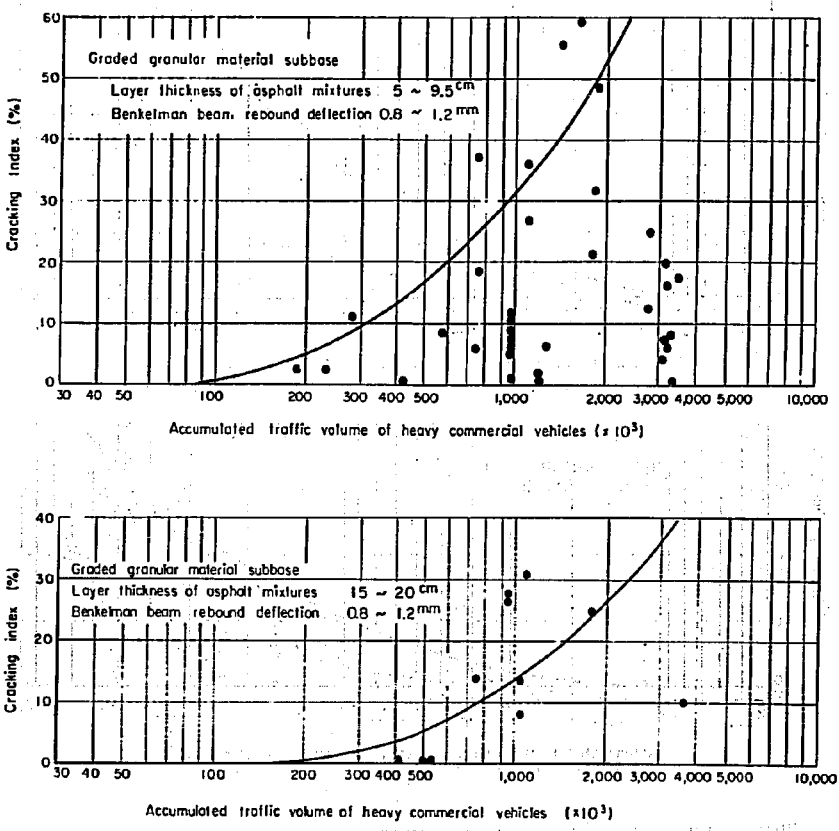


Fig. 3 Effect of Asphalt Mixtures Thickness on Increase of Cracking Index, - National Highways.

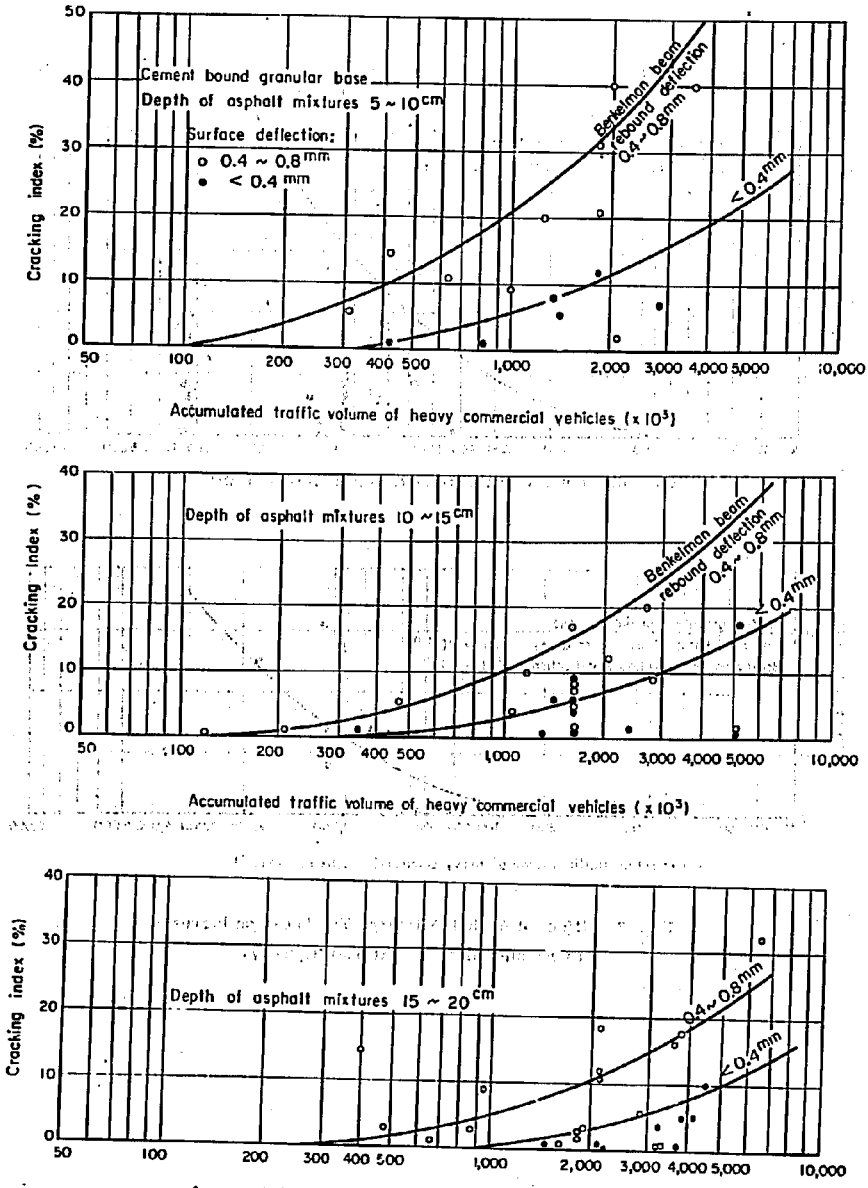


Fig. 4 Effect of Cement Bound Granular Bases. - National Highways.

2.2 A Survey of Local Road Pavements

The Japan Road Association issued the manual for lightly trafficed road pavements in 1964. According to the manual, the pavements should be constructed on existing gravel roads and consist of a graded granular materials base or a stabilized base of 6 to 12cm thick and a surfacing of 3 to 4cm. The Prefectural Divisions of Road Bureau have commenced a five-years performance survey on these types of pavement. The total number of sections already surveyed during the period 1964 - 1967, reached about 3,500, and the subject of investigation covers more than 20 items relating to road environment, pavement structures and performance, as in the case of national highways.

The results of survey on the pavements constructed during the period 1964 - 1965 indicated as shown in Fig. 5, that the factors influencing markedly on cracking indexes observed about a year after construction were the surfaces deflection, bearing capacity K_{30} on bases, types of surfacings, and traffic volume.

Fig. 6 clearly indicates the effect of the surface deflection on the degree of cracks of the pavements comprising an asphalt concrete surfacing and a graded granular material base, which are the most common type of local road pavements. As supposedly graded granular materials are not necessarily of good quality, pavements having a base of this sort of material show higher cracking indexes than those having a dry bound macadam base or a crusher-run rock base, as seen in Fig. 7.

When pavements are surfaced with an asphalt penetration macadam, they give a higher cracking index than those with hot asphalt mixtures, even if both the type of base and the level of surface deflection are the same. (Fig. 8) And if the surface deflection becomes larger, the difference between the two will become much remarkable.

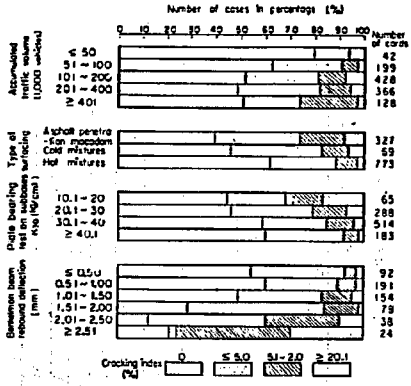


Fig. 5 Factors Affecting on Pavement Failure, - Lightly Trafficed Roads.

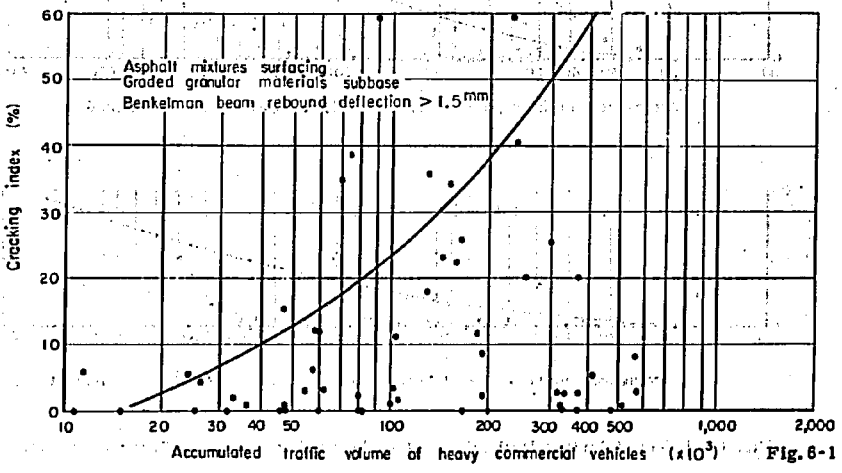


Fig. 6-1

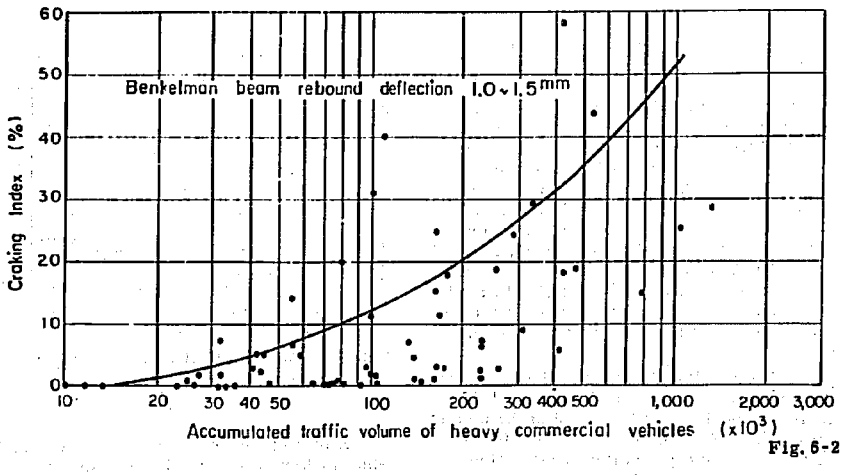


Fig. 6-2

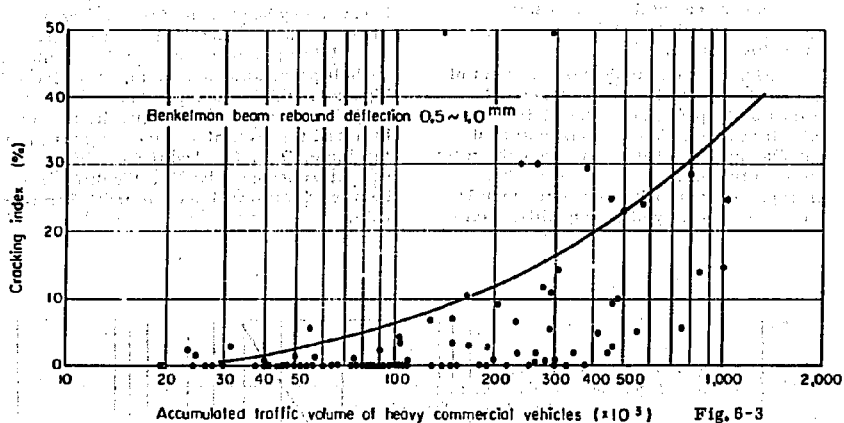


Fig. 6-3

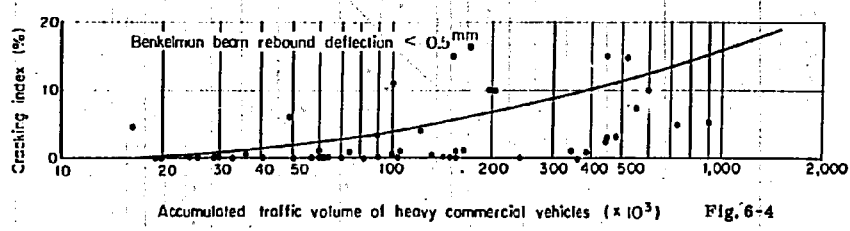
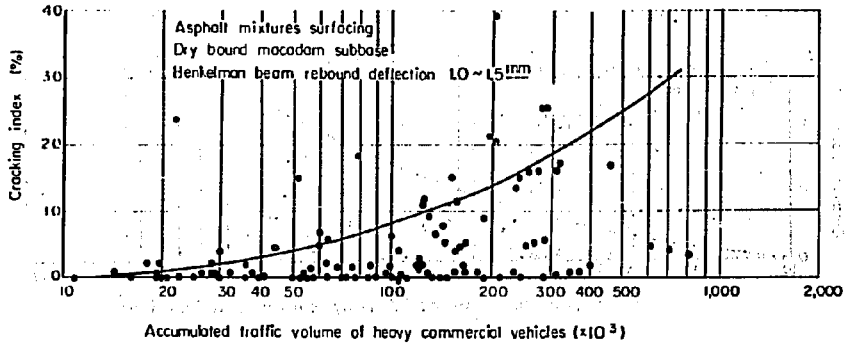


Fig. 6-4

Fig. 6 Effect of Surface Deflection on Cracking Index. - Lightly Trafficed Roads.



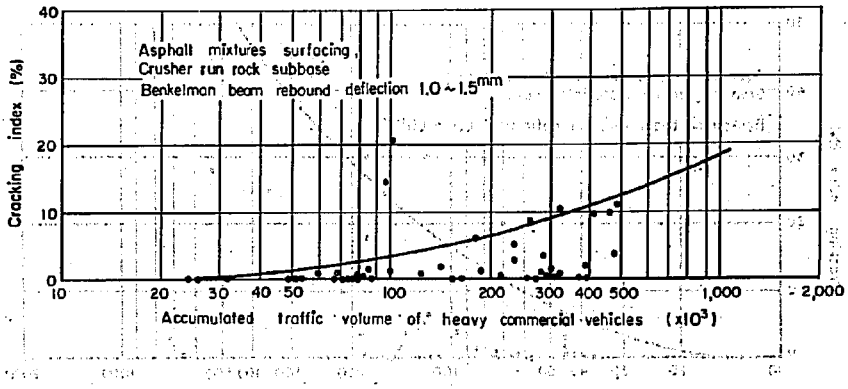
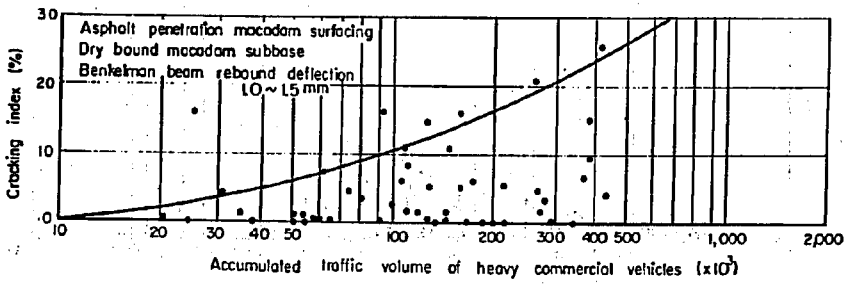


Fig. 7 Effect of Base Materials on Cracking Index, - Lightly Trafficed Roads.



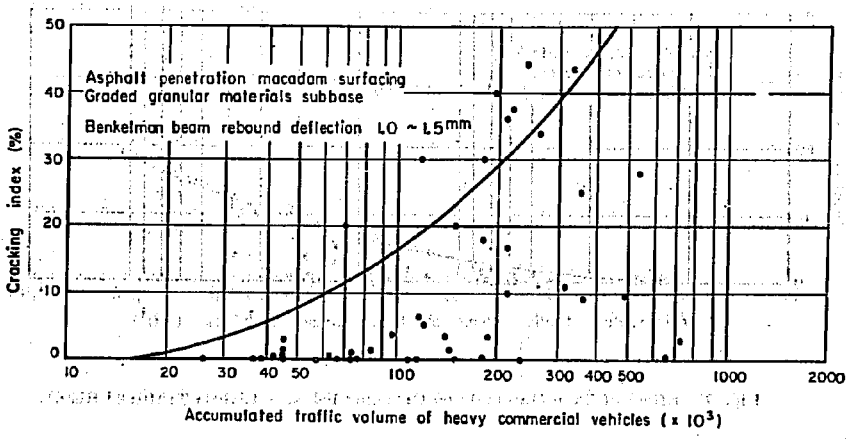


Fig. 8 Effect of Surfacing Types on Cracking Index, - Lightly Trafficed Roads.

3. OBSERVATION OF FULL-SCALE ROAD EXPERIMENTS

Full-scale experimental road sections of as long as about 1km have been successively constructed since 1967, and the number of main experimental sections has reached 13 so far.

Four sections relating to base and subbase materials are now under observation, and two of lightly trafficed road pavements have already been observed.

3, 1 Experimental Sections for Base Materials

The first full-scale experimental section was the MATSUDO experiment for studying the effect of cement bound base, which was constructed in 1965 and resurfaced after about three years. (Fig. 9)

Fig. 10 indicates apparently the effect of the cement bound base thickness on the rate of cracking developed during the period under traffic. The curve shown in the Figure was obtained from the survey on the national highways. When it is compared with the results of the observations made a little more precisely on experimental roads, it can be assured that the shape of the curve assumed previously and the corresponding level of deflection is fairly appropriate. The DAINICHI experimental section constructed in 1968 covered almost all the base materials and types of construction which were available at present in Japan (Fig. 11 and Table 2). The principal object of MEISHI experimental

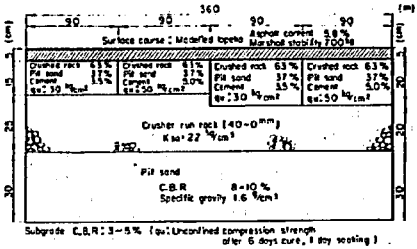


Fig. 9 Layout of MATSUDO Experimental Section.

section constructed in 1969 was to examine the effect of base materials and binder contents (Fig. 12 and Table 3). As the surface cracks have not yet observed on the sections, only the seasonal variation of deflection is shown in the figures.

It can be seen that the surface deflection is generally the smallest at sections consisting of a cement-bound or lime-bound granular base, while the largest at those of a non-bound granular materials. When bituminous materials are used as binders, the deflection becomes larger on the sections using cut back asphalt, tar and emulsified asphalt than penetration grade asphalt. As the sections showing a comparatively larger deflection represents the wider seasonal variation, it is presumable that the higher the deflection is, the more it is affected by both temperature and moisture.

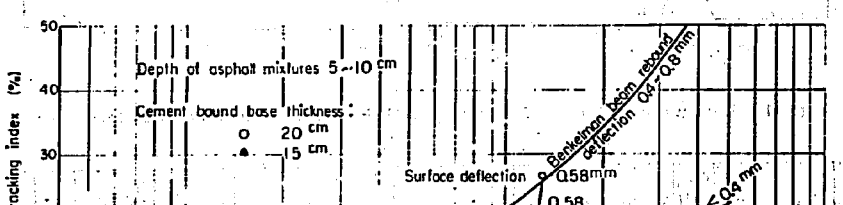


Fig. 10 Comparison of Performance Survey Data of MATSUDO Experiment and National Highways.

In the performance rating of pavements, the evaluation is often made based on the depth of rutting created on the surfaces. First, the transverse surface profile is recorded on the paper, then the distances from the reference line are measured. The square-roots of the unbiased variance of these readings are thought to indicate the surface unevenness. Fig. 13 shows the correlation of transversal surface unevenness of the DAINICHI experiment after trafficed by approximately 800 thousands of commercial vehicles and surface deflection. However, the transversal surface unevenness is not so remarkable at present as could be appreciated by the panels, if the present correlation between the transversal unevenness and the deflection will be pronounced in future, it is conceivable that the base and subbase materials which reduce the surface deflection would reduce the transversal deformation as well. The fact that the survace deflection would be related not only to the development of cracks but to a surface irregularity of pavements, makes the deflection measurements more advantagous in evaluating a pavement performance.

TABLE 2. Base Materials Used in DAINICHI Experiment.

Section	Materials in (%)			Binder content (%)		Marshall stability (kg) Compression strength (kg/cm²) C.B.R. (%)
	Crushed rock	Screenings	Pit sand			
A10	65	22	13	Asphalt pen. 60 ~ 100	4.0	663~818 (kg)
A15	65	22	13	Asphalt pen. 60 ~ 100	4.0	1015~1368 (kg)
A25	65	22	13	Asphalt pen. 60 ~ 100	4.0	917~1015 (kg)
T15	63	20	17	Road tar H. No.2	4.0	367~522 (kg)
T20	63	20	17	Road tar H. No.2	4.0	508~606 (kg)
S15	61	33	6	Cut back Asphalt	4.0	649~761 (kg)
E15	63	22	15	Emulsified asphalt mixing type MK-3	7.0	592~705 (kg)
E20	63	22	15	Emulsified asphalt mixing type MK-3	7.0	522~705 (kg)
C15	48	36	16	Portland cement	3.7	29~32 (kg/cm²)
C20	48	36	16	Portland cement	3.7	30~35 (kg/cm²)
C25	48	36	16	Portland cement	5.5	51~58 (kg/cm²)
EC15	63	22	15	Cement Bituminous modified	4.0 4.0	11~15 (kg/cm²)
L15	48	36	16	Hydrated lime No.1	100	10.1~10.7 (kg/cm²)
L20	48	36	16	Hydrated lime No.1	K10	10.6~12.1 (kg/cm²)
M20	63	22	15	——	——	58~72 (%)
M30	63	22	15	——	——	45~54 (%)
MS15	Blast furnace slag (40 ~ 0mm) 100			——	——	80~110 (%)
MS20	Blast furnace slag (40 ~ 0mm) 100			——	——	97~138 (%)
MS'15	Blast furnace slag (40 ~ 0mm) 70 Water granulated slag (40 ~ 0mm) 30			——	——	59~60 (%)
G30	Gravel (40 ~ 0mm) 100			——	——	40~57 (%)

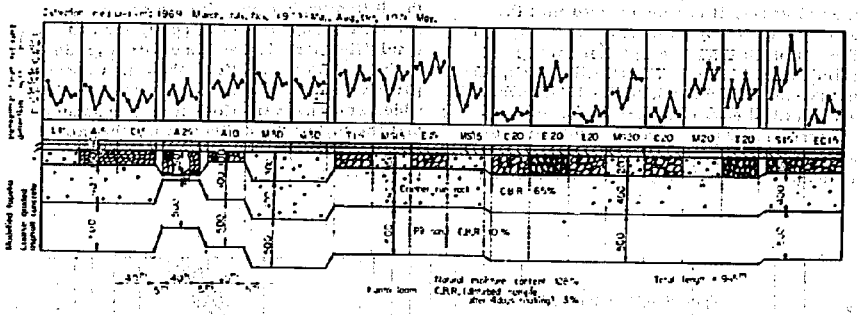


Fig. 11 Layout and Deflection Variation in DAINICHI Experimental Section (in mm).

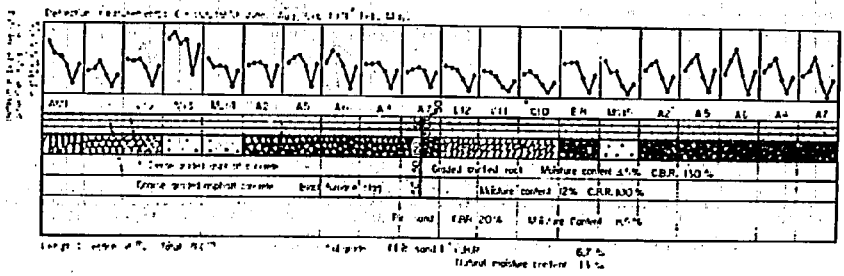


Fig. 12 Layout and Deflection Variation in MEISHI Experimental Section (in mm).

TABLE 3. Base Materials Used in MEISHI Experiment.

Section	Materials in (%)			Binder content (%)		Marshall stability (kg) Compression strength (kg/cm²) C.B.R (%)
	Crushed rock	Screenings	Pit sand			
AM1	Crushed rock 60~25mm Penetration macadam			Emulsified asphalt penetration type	ir4.2 kg/m²	—
A 2	40	50	10	Asphalt pen. 80~100	5.0	796 ~ 848 (kg)
A 3	40	50	10	Asphalt pen. 80~100	3.0	488 ~ 580 (kg)
A 4	90		10	Asphalt pen. 80~100	5.0	688 ~ 684 (kg)
A 5	90		10	Asphalt pen. 80~100	3.0	500 ~ 564 (kg)
A 6			100	Asphalt pen. 80~100	3.5	352 ~ 392 (kg)
A 7	80		River sand 20	Asphalt pen. 80~100	4.0	728 ~ 748 (kg)
E 8	90	10		Emulsified asphalt mixing type	8.0	615 ~ 655 (kg)
C 9	70	30		Portland cement	5.5	51 ~ 84 (kg/cm²)
C 10	70	30		Portland cement	3.0	25 ~ 34 (kg/cm²)
C 11	30	20	50	Portland cement	4.0	25 ~ 34 (kg/cm²)
L 12	90	-	10	Hydrated lime	2.5	9.2 ~ 11 19 ~ 63 (kg/cm²)
M13	50	20	30			85 (%)
Ms14	Blast furnace slag, A 100				-	147 (%)
Ms15	Blast furnace slag, B 100			—	—	120 (%)

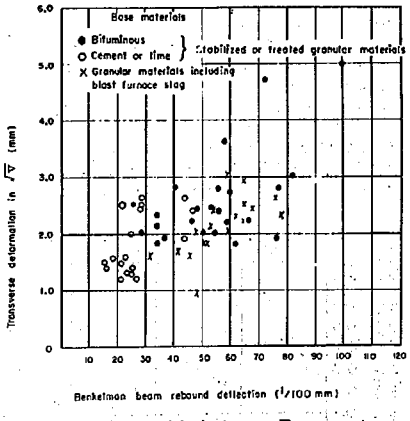


Fig. 13 Relationship between Pavement Deformation and Surface Deflection Obtained from DAINICHI Experiment.

3. 2 Experimental Sections for Subbase Materials

Both the YOKOTO and ITSUKAICHI experimental sections constructed in 1969 and 1970 respectively are aiming to study the effect of stabilization of local materials. The former (Fig. 14) is relating to subbases made of cement or lime stabilized wet cohesive soil called KANTO loam and pit sand, while the latter (Fig. 15) is to stabilized MASA (weathered granite) widely distributed in the CHUGOKU District.

As a result of observations, the effect of the stabilization of subbase materials on surface deflection is remarkable, and in particular the use of cement or lime with pit sand and MASA is

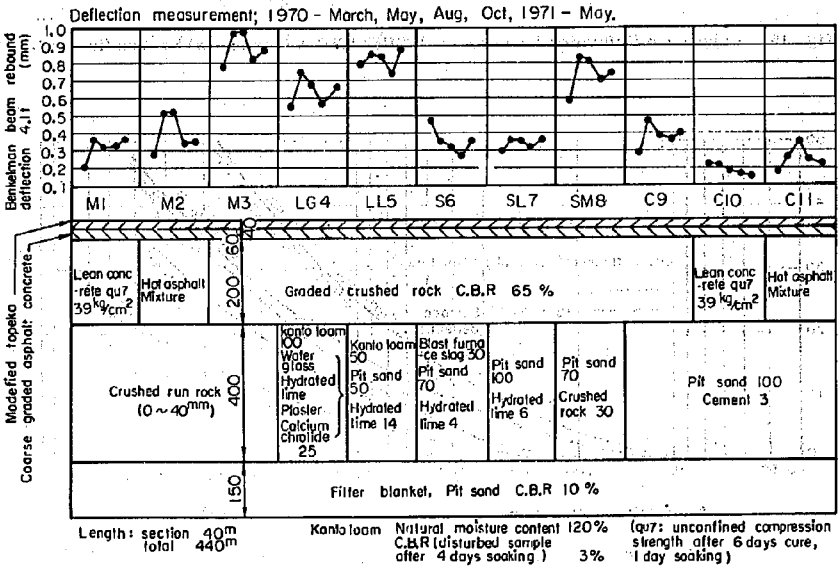


Fig. 14 Layout and Deflection Variation in YOKOTO Experimental Section (in mm).

the most effective; whereas the sections comprising subbases made of non-bound granular materials show as in the case of bases, larger deflection and larger seasonal variation.

At the sections showing larger surface deflection of the YOKOTO experiment, surface cracks already have partially observed. Fig. 16 shows the relationship between surface deflection and the cracking index at those sections having granular material bases, so as to be compared with the results of the nation-wide surveys. The cracking index is too high compared with the national highway pavements and is considered due to the non-uniform settlement of the fill of the wet cohesive soil.

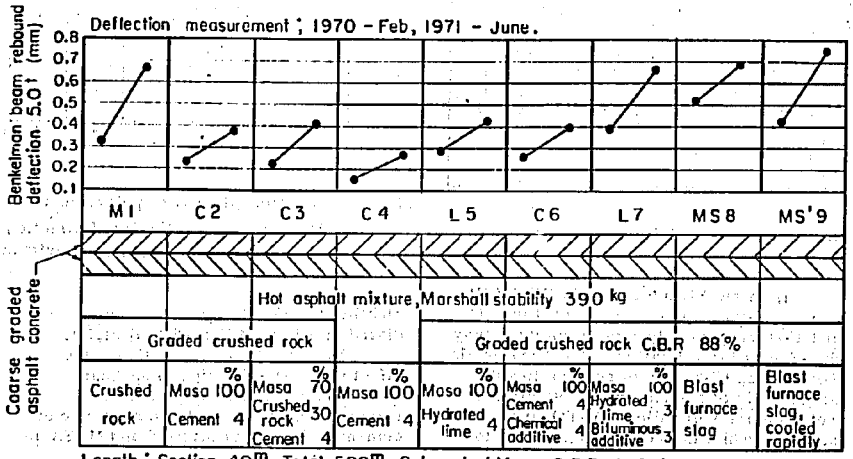


Fig. 15 Layout and Deflection Variation in ITSUKAICHI Experimental Section (in mm).

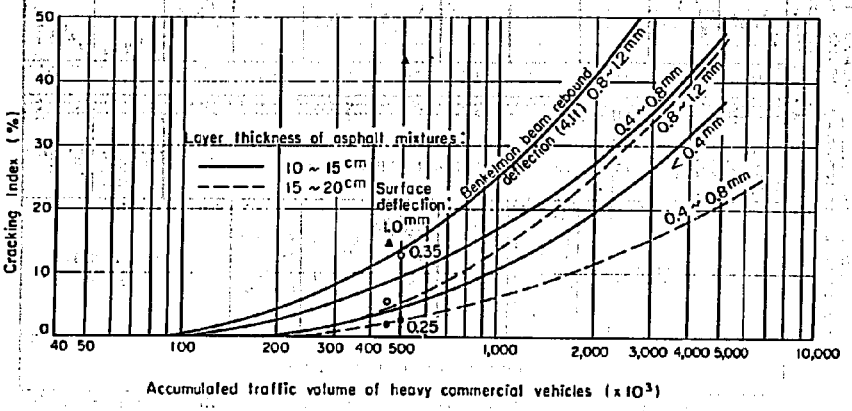


Fig. 16 Comparison of Performance Survey Results between Field Experiments and National Highways.

3. 3 Road Experiments on Lightly Trafficed Road Pavements

The MIYANOGI and HATAMACHI experimental sections constructed in 1964 and 1965 respectively, overlaid after 4 to 5 years' observation. (Figs. 17 and 18)

The development of surfacing cracks during the period is shown in Fig. 19.

The curve in the Figure represents the results of the nation-wide surveys, and does not seem to need any modification, when compared with that of the experiments. Throughout the two experiments, it can be said that the effect of materials and types of bases, subbases and surfacings on the pavement performance is far less distinct than that of the surface deflection.

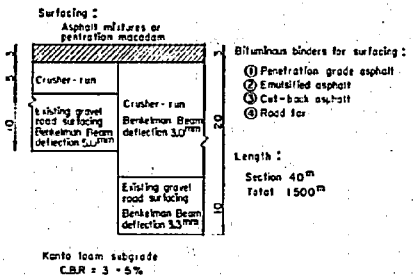


Fig. 17 Pavement Structures in MIYANOGI Experimental Section (in cm).

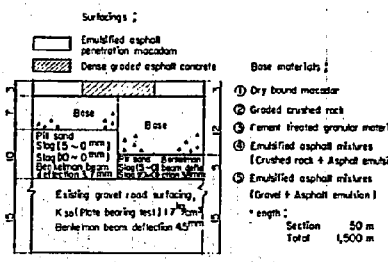


Fig. 18 Pavement Structure in HATAMACHI Experimental Section (in cm).

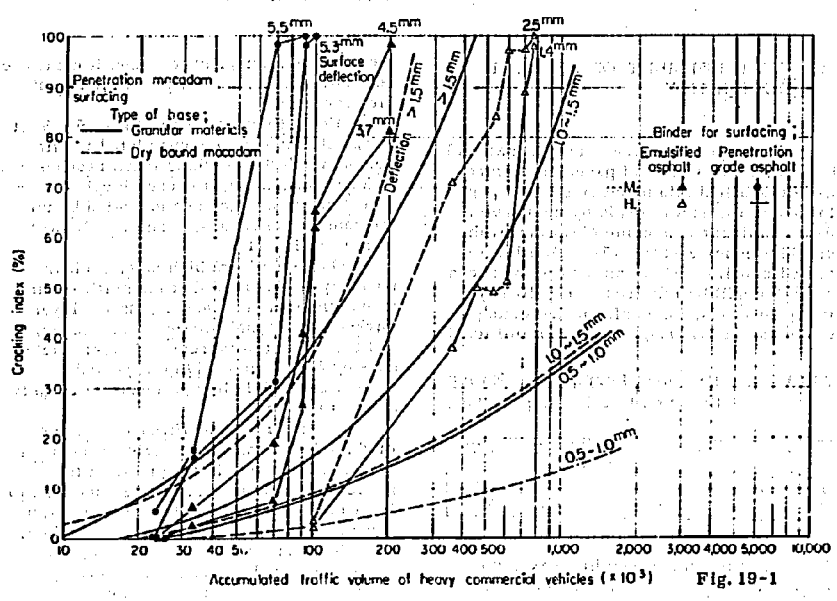


Fig. 19-1

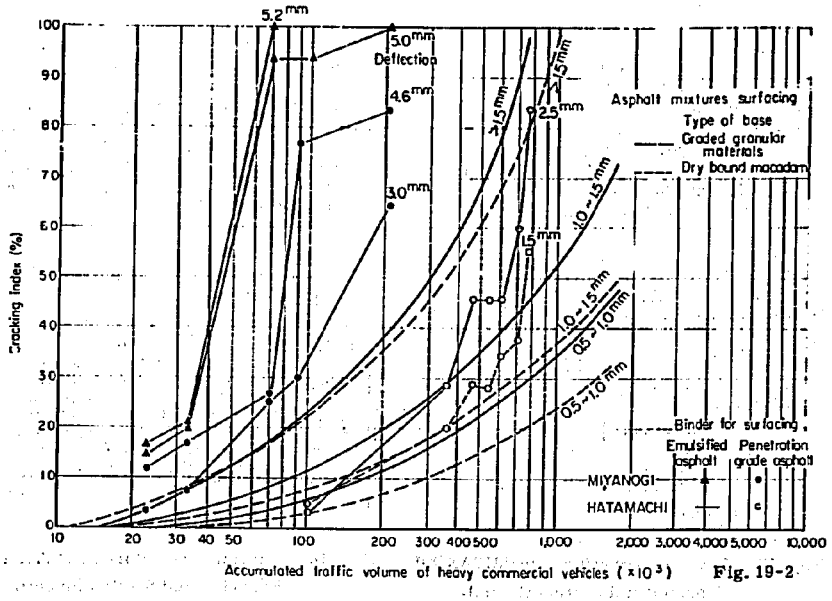


Fig. 19-2

Fig. 19 Comparison of Performance Survey Results between Field Experiments and Lightly Trafficed Roads.

4. ESTABLISHMENT OF CRITICAL DEFLECTION

In the performance survey of national highways, the degree of pavement failures were classified into three stages and performance rating was made by the panels. As a result it has become clear that if the cracking index exceeds 15%, quite large-scaled repairs such as an overlay or reconstruction will be required. For lightly trafficed roads, the performance rating was made in four stages, as seen in Fig. 20.

TABLE 4. Pavement Performance Rating Based on Level of Repair Work.

Rating	Required repair work	Cracking Index	Average cracking index	Average Deflection (mm, at center)
1.	Overlay or resurfacing	≥15%	31.1%	0.84mm
2.	Seal coat or patching	≥5%	14.1%	0.74mm
3.	Not required	≤5%	2.3%	0.53mm

As in the case of national highways, the lightly trafficed roads showing the cracking index exceeding 15% also require heavy repairs. Therefore, the cracking index of 15% could be taken as a border line for the pavement performance when it comes to necessitate a consideration on the life of the pavement.

From Figs. 2 - 4 and Figs. 6 - 8, it is possible to obtain accumulated traffic volume when the cracking index reaches 5%, 15% and further 40% for each level of surface deflection which are indicated by the medians of the ranges. The results are shown in Fig. 21.

On the national highways, a clear tendency is observed that the critical surface deflection increases, with the layer thickness of asphalt mixtures increases, and the amount of increase of layer thickness by 5 to 10cm could be expected to make a critical deflection increase roughly by 0.1 to 0.5mm.

On the lightly trafficed roads, the critical deflection which varies with types of base materials, is the smallest for graded granular materials and surmised to increase by as large as 0.3 to 0.6mm for crusher-run rock or dry bound macadam.

The critical deflection in Fig. 21 are represented by individual curves passing through the center of the ranges, and these are put together in Fig. 22.

The lightly trafficed road pavements

surfaced with asphalt macadam could be expected to perform better than those surfaced with asphalt mixtures, if the surface deflection falls below the level slightly under 1.0mm. Similarly, the national highway pavements having a cement bound base are comprehensible to show a excellent performance.

Including all the roads from the national highways having a layer of asphalt mixtures 10 to 15cm thick to local roads 3 to 4cm thick, the hatched range shown in the Figure can be considered the range of critical deflection for every typed asphalt pavements in Japan, however, in the case of roads in the suburbs of major cities, the layer of asphalt mixtures is generally thicker, and consequently the critical deflection is thought to be a little higher. The upper and lower limits of the ranges are expressed by the following formulas.

For national highways:-

$$\log_{10}N = 0.179\,d^2 - 1.117\,d + 6.772 \quad \cdots\cdots\cdots (5)$$

For lightly trafficed roads:-

$$\log_{10}N = 0.179\,d^2 - 1.117\,d + 6.385 \quad \cdots\cdots\cdots (6)$$

where N: Accumulated traffic volume of large commercial vehicles when the pavement comes to require heavy repairs (vehicle/lane)

d: Benkelman Beam rebound deflection measured by wheel-load 4.1 tons ... (mm)

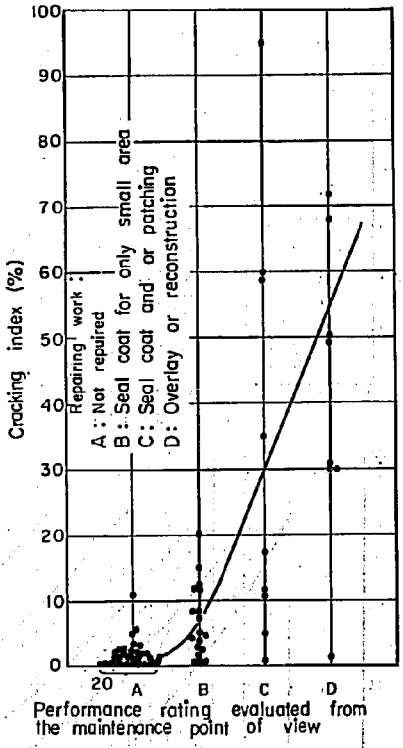


Fig. 20 Cracking Index VS Pavement Performance Rating.

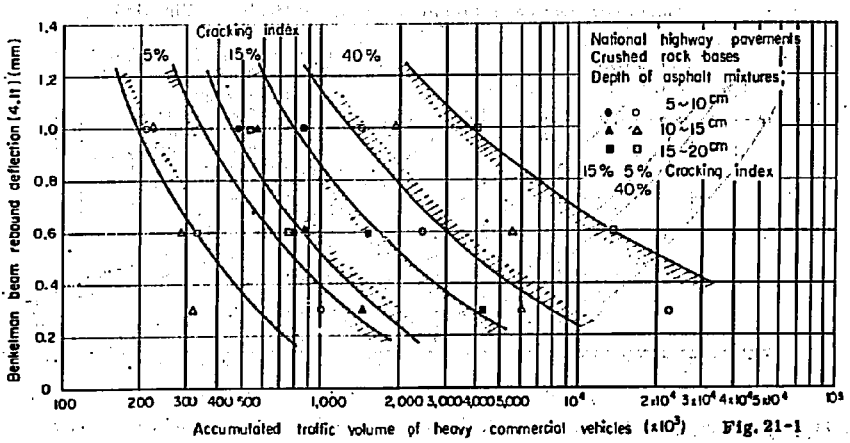


Fig. 21-1

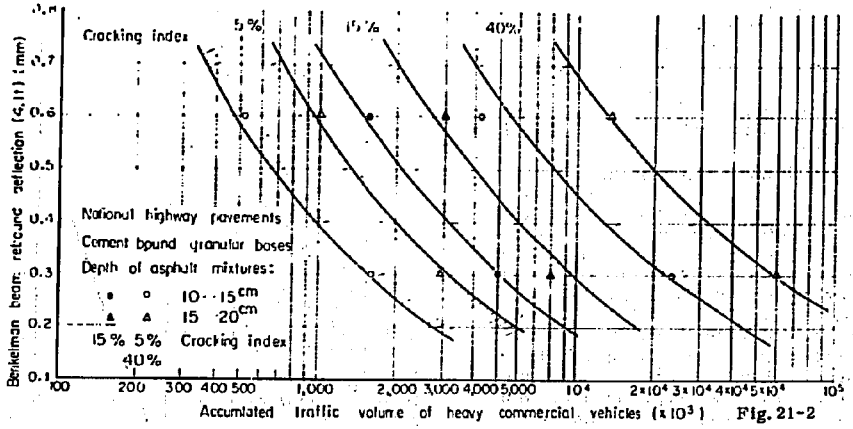


Fig. 21-2

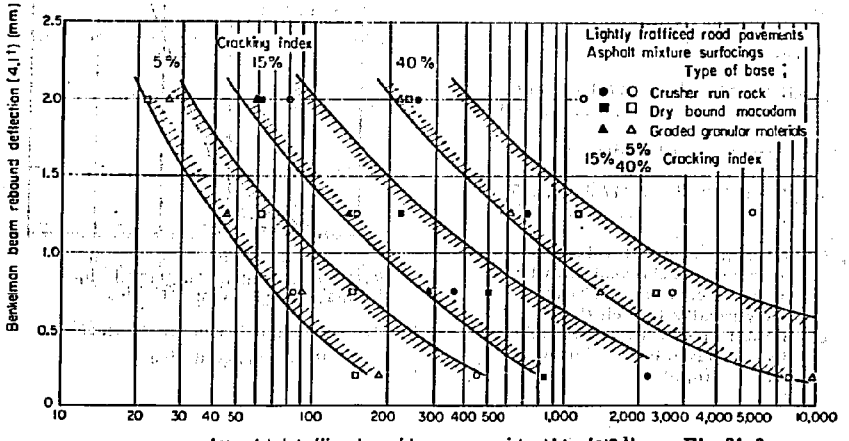


Fig. 21-3

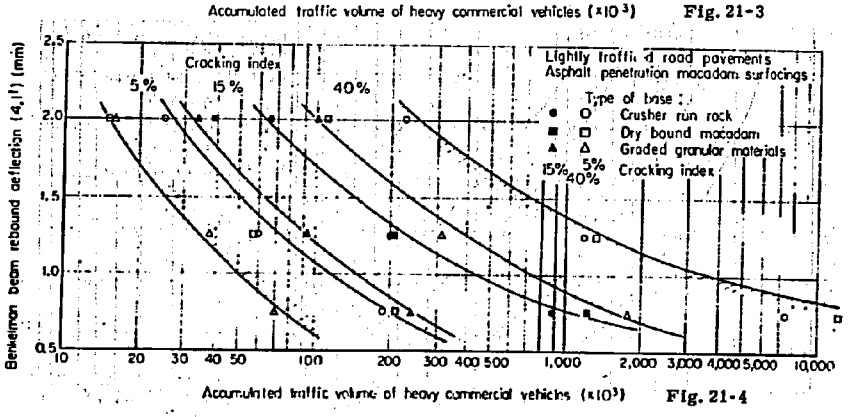


Fig. 21-4

Fig. 21 · Trends of Critical Deflection Decrease with Accumulated Traffic Volume Increase.

In the formula (5), the constant term can be made slightly larger if the layer thickness of asphalt mixtures is increased. In the formula (6), it should be made a little smaller if poorer base or subbase materials other than crushed rock are employed. When cement bound bases are employed, the formula should be given in another form.

For the calculation, Miura's method[3] was employed taking Poisson's ratio 0.35 for all materials. Measured deflections were taken when the surface temperature was around 20°C. If the wheel load was other than 5 tons, a proportional modification was made on the deflections. The result is shown in Table 5. However, even if the materials are the same, the E value

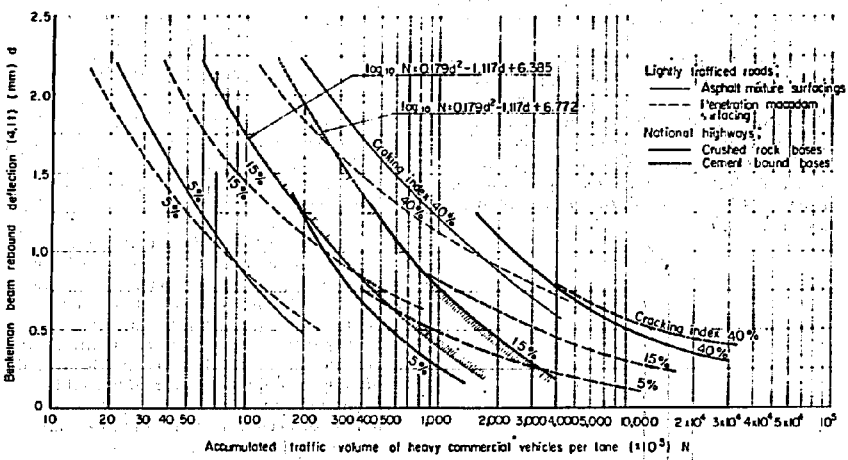


Fig. 22 Comparison of Deflection Criteria for Various Typed Asphalt Pavements.

5. APPLICATION OF SURFACE DEFLECTION TO STRUCTURAL DESIGNS

5.1 Trial Estimation of Pavement Materials E Moduli by Theoretical Calculation

A trial was made to estimate materials E value by applying a theoretical solution of multi-layered system to the structures of the full-scale experimental pavements already mentioned, adjusting E values so as to make the difference between the calculated surface deflections and measured ones obtained by using Benkelman Beam as small as possible. In the calculation, the contact area of double tires of the loading vehicle was assumed to be circular. Compared the rebound deflection of rigid circular plates of different radii with Benkelman Beam deflection, the contact area of tires was found to be equivalent to a circle of 20cm radius, ignoring the thickness of the pavement and the rigidity of the plate.

may vary depending on the positions in the layers, and considerably on the load bearing capacities of the under layers.

TABLE 5. Estimated E-Value from Comparison of Measured and Calculated Pavement Deflection.

Layers	Material	Estimated E (kg/cm²)
Surface or base course	Hot mixed asphalt concrete	40,000
Base	Hot asphalt mixture	20,000 ~ 30,000
	Tar mixture	10,000
	Cut back asphalt mixture	8,000
	Emulsified asphalt mixture	3,000
	Cement and asphalt emulsion treated mixture	15,000
	Cement bound crushed rock or lean concrete	20,000 ~ 40,000
	Lime bound crushed rock	200,000
	Graded crushed rock	4,000 ~ 5,000
	Crushed blast furnace slag	3,000 ~ 5,000
	Blast furnace slag, water cooled	30,000 ~ 40,000
Subbase	Crusher run rock	3,000 ~ 4,000
	Lime stabilized KANTO loam with chemical additives	2,500 ~ 3,500
	Lime stabilized KANTO loam	500 ~ 1,000
	Mixture of pit sand, slag and hydrated lime	50,000
	Mixture of pit sand and crusher run	1,000 ~ 2,000
	Cement treated pit sand	15,000
Subgrade	KANTO loam	300 ~ 500

5.2 Surface Deflection and Life of Newly Constructed Pavements on National Highways

In the Manual revised in 1967, a new traffic category D (estimated traffic volume exceeding 3,000 heavy commercial vehicle per day in one direction, five years after construction) was given, and consequently the pavements of heavily trafficed roads have been strengthend and those of general national highways improved slightly above their previous level.

The surface deflection of newly constructed pavements on national highways surveyed through the country in 1969 - 70 is shown in Fig. 23. It has been observed that Benkelman Beam deflection under 5 tons wheel-load rarely exceeds 1.2mm, and at the sites of above 60% lies within the range of 0.2 to 0.6mm (0.16 to 0.48mm under 4 tons).

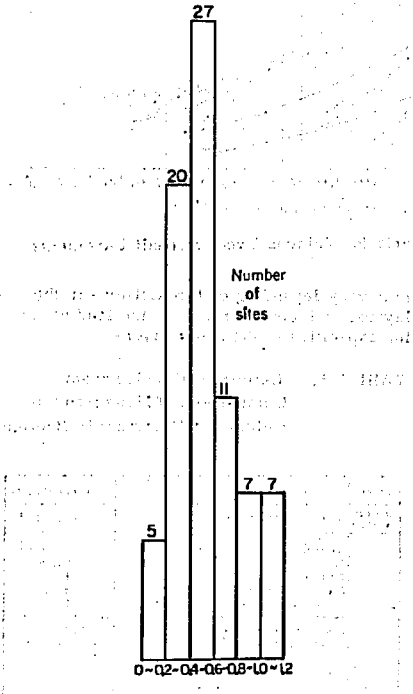


Fig. 23 Histogram of Deflection Data Obtained from Newly Constructed Pavement Surfaces of National Highways.

According to Fig. 22, the accumulated traffic volume of heavy commercial vehicles that corresponds to a deflection 0.5mm is about 1,800,000 and this figure decreases to 700,000 if the deflection increases to 1.0mm. The expected life of pavements creating such a level of deflection as shown above could have been estimated as seen in Table 6.

TABLE 6. Expected Life of the Newly Constructed Pavements Estimated from Surface Deflections.

Traffic Categories	Traffic Volume of Heavy Commercial Vehicle (veh/d)	Deflection (mm) 0.5	1.0
B ~ C	1,000	≥ 1,800 days	≥ 700 days
C ~ D	3,000	≥ 600 "	≥ 230 "

It can be surmised that more than half the number of existing national highways in Japan will perform well without any heavy repairs at least for five years under a medium traffic and for twenty months under a heavy traffic. In general, however, heavily trafficed roads often have a thicker layer of hot laid asphalt mixtures, and consequently the period of twenty months might be considered not practical.

5.3 Design of Pavements of Lightly Trafficed Roads

The survey was made on about thirty local roads to obtain the layer thickness required for establishing the critical deflection on road surfaces as shown in Fig. 22 and the formula (6). A section about 200m long was selected on gravel-surfaced road and a pavement conforming to the structure given in the Manual which originally obtained by the formula (1) using a design wheel load of 1.4 tons was constructed on the half length of the section. After opened to traffic the deflection variation was measured on both asphalt surfaced and gravel-surfaced section through the year. From Fig. 24 showing a correlation between the deflections on both sections, the design curve similar to that utilized in Canada[4] was obtained, as seen in Fig. 25. The additional layer mainly consists of a graded crushed rock base and a 3 to 4cm thick surfacing and curve A can be applied to roads under comparatively heavy traffic.

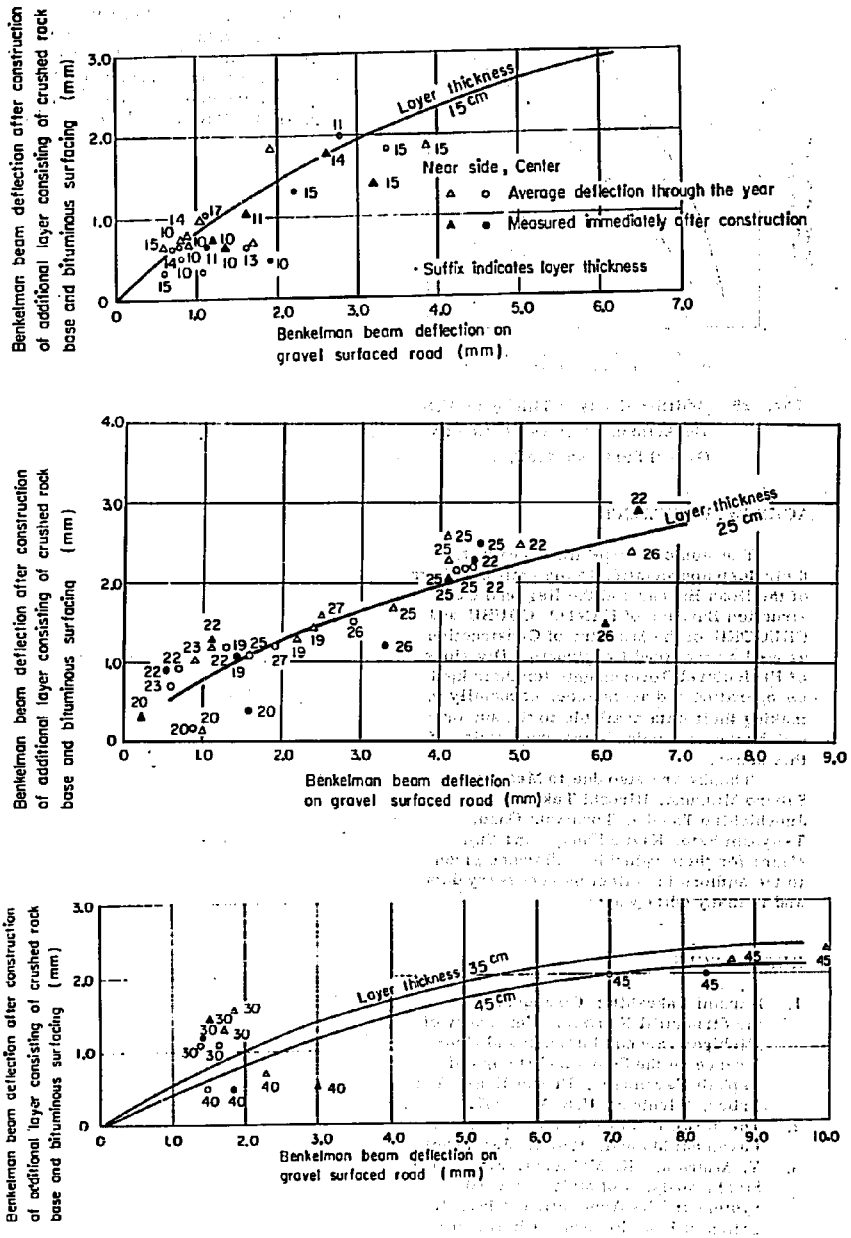


Fig. 24 Deflection Decrease on Road Surfaces after Pavement Construction.

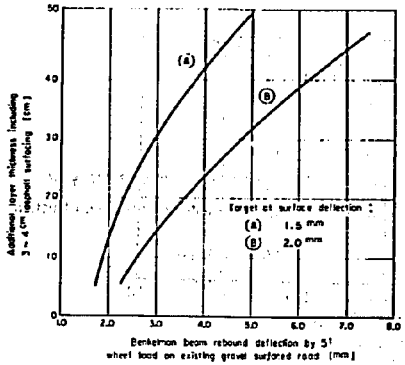


Fig. 25 Additional Layer Thickness V.S Benkelman Beam Deflection on Gravel Surfaced Road.

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INTERNATIONAL ASSOCIATION FOR HYDRAULIC RESEARCH
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SOME NOTES ON HYDRAULIC MODEL TESTS OF RIVER CHANNELS

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SYNOPSIS

Many kinds of large scale hydraulic model tests of main rivers in Japan have been performed at Kashima Hydraulics Laboratory. More than 30 projects have already been finished for recent ten years, and 14 are now being performed. The results of these experiments have been applied to the designs of actual river channels with favourable comments.

In this paper, the author will discribe the problems on some annoying similarity laws both for fixed and movable beds, new examination methods for verification of physical modeling, the methods for experiment plan and other inportant matters obtained from our experiences. For example, the special case in which the surface velocity distribution measured by aerial plotograph during a large flood was served to the verification of the model, determination method for time scales of movable bed models by measuring the rates of sediment transport both at actual river and fundamental two dimensional channel, fundamental study on the effects of distortion of the models, especially at curved reaches, and the effect of roughness of sidewall on the local erosion will be discussed.

Computational analysis is very useful for two dimensional problems, and physical model analysis is recognized to be valuable means, if skillfully arranged and appropriately conducted. From the view point of actual performance of model test, general experimental techniques developed at Kashima hydraulics laboratory will be presented.

INTRODUCTION

Hydraulic model tests of river channels have been recognized to be

effective methods in order to consider especially three dimensional phenomena, and many have been performed for a long time. The first regular model test in Japan was a coffer dam with roller-lift gate at Kitakami River near Sendai in 1930[1] which was conducted as small scale movable bed model test at Akabane branch, the Public Works Research Institute. Though the extents of models were restricted around the river structures at the initial stage of experiments, hydraulic phenomena around structures were proved to be necessary to be grasped in relation to the alluvial channels as a whole. In 1950s, technical development of verification of the model, and application of fundamental studies on alluvial channels were advanced. River models became larger-sized, most in fixed beds, and the number of performances increased in early 1960s. Recently, large scale hydraulic model tests in movable beds are remarkable, and techniques of physical modeling of the hydraulics of rivers develop a great deal. Table 1 shows the names of projects and their scales conducted at Kashima Hydraulics Laboratory, the Public Works Research Institute, in fiscal year 1971. Most of the experiments were carried out under the conditions of both steady and unsteady flows, and of movable beds using coal dust as bed material of the models except the cases where the sands of proper diameter are available.

Hear, the author will discribe some of current main topics and the results of some considerations about techniques of large scale physical modeling of alluvial channels, based on our experiments.

Change of subjects and ways of model tests

Model tests are directly related to river improvement works. Main subjects of physical model experiments carried out at initial era, were local scours and coefficients of discharge of weirs, coffer dams or groundsills, designs of groins, revetments or consolidation works, and design criteria for confluences, diversions and diversion channels. Lately, increase the experiments which contain such intricated problems as large scale dredging of low water channels, large scale temporary cofferings, gatherings of ballast at meandering channels, estuary improvements, water pollutions or rowing up of waves.

Model tests of former days contributed to designs of alluvial channels mainly in the from of supplements to the hydraulic calculations or field data. By and by, large scale fixed bed model tests became popular having need to understand the hydraulic phenomena caused by river structures, at whole river

Tabel 1 Projects of model test

at Kashima Hydraulics Laboratory in 1971.

Names of Rivers	dominant subjects	Design discharges	extents of models	kinds of beds	scales	
					horizontal	vertical
Yodo R.	dredging, coffer dam, confluence	12,000 m^3/s	7~40 Km	fixed bed	1/100	1/40
				movable	1/60	1/60
Tone R.	bends	8,000	35~46	movable	1/60	1/40
Tone R.	confluence, meandering	14,000	175~186.5	movable	1/70	1/35
Shinano R.	low water channel, meandering	11,000	13~23	movable	1/100	1/70
			13~26	movable	1/500	1/100, 1/200
Nagara R.	low water channel, temporary coffering	7,500	2~14	movable	1/60	1/40
O-yodo R.	estuary improvement	8,000	-1.6(sea)~3	movable	1/50	1/50
Mogami R.	estuary improvement	9,000	-2.0(sea) ~3.4	movable	1/60	1/60
Kitakami R.	retarding basin	11,000	24.8~38.2	fixed	1/100	1/20
Kyu-Yodo R.	dispersion	330	7~10	fixed	1/100	1/40
Ibi R.	rowing up of waves	———	-0.6(sea) ~8.2	fixed	1/65	1/32.5

extents affected. When sedimentation became to be considered the most important problem, attempts were tried to estimate river bed variations based on the results of fixed bed experiments, or informal movable bed model tests using sands as bed materials of the models. It is rather latest that the movable model tests were regularized, since coal dusts were applied to sediment materials of the models. Size of models became larger, and 150m to 300m length of models are commonly seen at our out-door experiment area.

According to the tendency that the purposes of experiments have become complex and pluralistic, expansion of model extent and accuracy of similarity as to water level, flow pattern and river bed variation, are required to the result of spurring the scale-up of models.

Therefore, on the other hand distorted models with small horizontal scales have also begun to be commonly used.

Moreover, there seem slight changes in expected experimental results from qualitative to quantitative (experiments hitherto contributed to comparisons of proposed schemes or estimations of quantity.), and hence there occur new

problems.

Subjects of discussions on experiment programs

The scales and contents of experiments are subject to their purposes. The ways to focus are different according to their goals, and unimportant factors are to be neglected because of the impossibility to reappear all of the hydraulic phenomena on the models. First decisions are weighty in experiment programs. When planning model tests, present standards and subjects as to scales and kinds of models may be as follows:

1) Extents of models: Length of 3~6 times of width of the channel is prefered to upstream of the object reach, considering the meandering flow lines, river bed variation, movement of alternate bars and supply method of sediment from upstream, and length of 2~5 times of width to downstream containing a gauge station of water level. This standard depends naturally on the purposes of experiments, conditions of channels and funds, and most of the present models are not so long.

2) Scales of models: Though larger the better in many cases, there are restrictions in experiment area, capacity of supply water discharge, funds and turn. Lower limit of scale may be judged by water depth (more than 3cm, neglecting the influence of viscosity and at turbulent state), similarities of critical tractive force and rate of sediment transport both for bed load and suspended load, measurement accuracy and measurement ability for velocity, water level, discharge and bed variation. Models will be distorted when these conditions can not be satisfied. There are many distorted models at movable bed experiments, originating some problems.

3) Fixed and movable bed models: Fixed bed models are prefered to simulate form losses due to the changes of hydraulic quantities, velocity distributions, and flow patterns, on assumption that bed variations are not so intense. On the other hand, movable bed model tests are direct to the phenomena of scour and deposit, but have weak points on roughness adjustment and expense.

4) Hydraulic conditions of experiments: Experiments of steady or unsteady flow conditions are carried out requiring equilibrium or time-depended quantities, respectively. Discharge and water level are determined according to pending phenomena such as design water level, scour, deposit and meandering. When scour is considered, minimum roughness coefficient must be adopted to estimate water level at down stream boundary. Uniform diameter of bed material is commonly used at experiments, and there may be some unsettled problems

concerning the phenomena as to graded natural sands, critical tractiveforces, rates of sediment transport, types of sediment transport (bed load, suspended load, wash load), roughness coefficients, sand waves, tightening ratio and porosity of the materials. There may be other problems of several kinds, and verification of model is recognized to be very significant.
The author will then discuss such special items in performing model tests as time scale, verification methods, and influences of distorted models.

Time scales of movable bed model tests

Similarity law is subject to the purpose of the experiment. If Froude's low is applied, the rate of time scale tr is

$$tr = xr\, hr^{-\frac{1}{2}}, \qquad \text{——————(1)}$$

here, xr and hr are the rates of horizontal and vertical scales, respectively. Besides this, in case of movable bed tests, another rate of time scale tsr introduced by the continuity condition of sediment transport,

$$tsr = (1-\lambda)r.\, hr\, xr\, qtr^{-1} \qquad \text{——————(2)}$$

must be considered. Here, λ means porosity of bed layor and qt is total rate of sediment transport. The values of tr are significant in the cases where unsteadiness and pondage effects are predominant, and tsr has important means in the cases where bed variations are emphasized. But calculated values of tsr are not a little different according to sediment formulas applied. Sediment transport characteristics at models are rather well understood because the hydraulic category of models is commonly within that of two dimensional fundamental studies. Sato, Kikkawa and Ashida's formula(S.K.A. formula), for example, in case of sands as bed materials of models and Brown's formula in case of coal dusts in which suspended sediment exists, are ascertained to be well applicable. Therefore, it is possible to increase greatly the accuracy of tsr if the field characteristics of sediment transport are grasped by measurements. Sediment functions are generally different between proto type and model, and tsr becomes the function of discharge.

Good result was obtained by determining time scale tsr, applying experimental equation in case of coal dusts and empirical relation obtained by field measurements of the spot, at the model test of down reach of Nagara River near Nagoya. At the model experiment of Mogami estuary, several time scales are computed as, tr =7.75, tsr =25.6 (S.K.A. formula), tsr =4.24 (Brown, mean diameter of proto type dp=0.6mm, and that of model dm=0.2mm,

coal dust) and tsr=244 (S.K.A. formula to proto type and Brown's formula to model). Experimental formula of coal dusts is close to that of Brown. When proto type discharge Q=6000m^3/s, calculated value of bed material load by Brown's formula is almost hundred times that by S.K.A. formula. Therefore, if there are not enough field data of sediment transport, experiment verification is necessary. Figures 1 and 2 show time variance of water levels and bed variations of proto type and model, respectively, when time scales tsr=25 and tsr=40. Movable bed modelings are in general recognized to be inevitable that simulation of water level can not be enough, and to pay mainly attention to bed variations. But recently, request to such water level as its maximum becomes strong. Influence factors to water level to time curve are considered to be pondage effects if tsr is greatly different from th, changes of roughness coefficient due to the development of sand waves on the bed (water level of model is generally higher than that of proto type at last stage of flood), changes of sediment transport due to the changes of roughness, dissimilarity of bed variation due to unsuitable sediment functions, and changes of time scale due to the change of discharge during the flood. These factors must be considered to value the experimental results. The difference of initial water level in Fgure 1 is caused by preliminary discharging (Q=2000t/s, 30minutes at tsr=25, and 15minutes at tsr=40). From the model test of Mogami esturary, the influences of time scale is large to the bed variations and not too large to water level curve.

By the way, if the experiments are carried out in the state of fully developed sand waves, it is theoretically possible to adjust water level by applying the formula of roughness coefficient of the state and determining suitable distortion ratio of the model. Or it may be a way to use a bed material of large diameter and small specific gravity in water in order to suppress the occurrence of bed waves.

Some notes on distorted models

It is a good policy to distort models if the influences of distortion and their amendment method are grasped. But the limit of distortion ratio exists due to the conditions of river channels and purposes of experiments. Table 2 shows the distortion ratios of main model tests performed at Kashima Hydraulics Laboratory before 1970. It may be possible to distort models greatly, but when scouring depth and location are the main subjects, models

can not be so distorted. Three dimensional equations of proto type can not be satisfied in distorted model, and it is not preferred to distort especially in the cases where the influences of vortex flows of horizontal axis are predominant. As experimental subjects, angle of repose of bed material, roughness coefficient, discharge coefficient(form loss) local flow pattern around structures, and local scour are mentioned. When model tests are carried out using distorted models, we try to amend the experimental results of scouring characteristics by doing proper experiments at another small channels. As an example of such fundamental studies, the effects of distortions on bed variations of curved channels are examined.[2] And especially entrance and exit vicinities of the curved channels are clarified to be greatly affected by distortions. Large scale vortexes of spiral motions in curved channels are divided into four regions: appearing, developing, developed and diminishing. Intensity of spiral motion is so strongly affected by depth of flow too, that especially in developing and diminishing regions, secondary current and tractive force are different according to the distortion ratio. At curved channels and compound curved channels of more than double bends, spiral motion of stream, scouring and depositing characteristics can not be simulated, if the depth of the flow is large. Distorted models had better be avoided as far as possible such cases of problems as local scour in bend, alternate bars and meandering, because depth is meaningful.

Gradient and roughness of revetment are also proved to affect the characteristics of local scour.[3] Verticaly upward component of secondary flow exists on the revetment of small gradient, but diminishes on steep revetment. This type of secondary flow pattern near the wall has some influences on scouring pattern and its position, but the effects of distortion on local scouring depth at the foot of revetment may not be serious, because scoured depth is in general larger in case of steep revetment of large distortion ratio than in case of small gradient. Roughness of side wall has also some effects on local secondary flow, velocity distribution and scouring pattern. At Tone river model test which was performed as movable bed using coal dust and at the model scale of λ_l=100 and h_r=50, scoured areas and their depths in bends where water depth is large, i.e. 10~25m near walls, and diameter of bed materiol is small i.e. 0.016~0.020mm, were strongly affected by the wall roughnesses of the revetments. Therefore, wall roughness of the model was adjusted to that of proto type and satisfactory agreement in scouring pattern

was obtained, without doing any treatment to the gradient of the revetment.

Table 2 Distortions of models (before 1970, at K.H.L.)

Rivers	Main subjects	Model beds	Scales		
			xr	hr	Distortion Ratio
Tone R. down stream	Unsteady flow, cofferdam	fixed	100	20	5
	Bends	movable	100	50	2
Tama R. down stream	Bridge piers	movable	60	60	1
Kiso R. Kawashima	diversion, confluence	movable	60	60	1
Kuma R. down stream	Bend	fixed	50	50	1
Nagara R. down stream	silting	movable	100	64 50	1.56 2
Fuji R. down stream	piers, revetments	movable	50	50	1
Kuji R.	estuary improvement	movable	40	40	1

Verification of modeling

Verification methods for reappearances of phenomena at models are determined according to the purposes of experiments, the phenomena to pay attention, and hydraulic quantities of limits. Verification of similarity at fixed model tests was hitherto to adjust water levels or roughness coefficients of flood flows. In 1969, surface velocity distribution was verified based on the measurement by aerial photograph of flood for the first time at the fixed model test of Yodo river. Figure 4 shows an example of surface velocity distributions both of proto type and the model.
It was interesting to be clarified by the experiment that the surface velocity distributions differed not a little, according to the ways of arraning gravels on the model bed which acted as roughness, even if at the same appropriate conditions for water levels. The flows at high water channels are seemed to be so much influenced, that some notices are necessary in arranging roughness gravels, especially when attentions are given to the hydraulic phenomena mear the boundary of low and high water channels. Aerial photograph gives another interesting knowledges such as flow pattern, longitudinal vortexes which are said[4] to occur at the intervals of double water depth, and qualitative concentration distribution of suspended sediment, besides surface velocity distribution. At the Tone river movable bed model test (35~46Km, 1971), aerial photographs of the model were taken from a helicopter, and

surface velocity distribution of flood flow, flow pattern, and other characteristics were compared to those of proto type which were also obtained by aerial photographs at the same stage of the flood. The results were almost satisfactory.(sea Photograph 1)

Commonly verification of similarity at movable bed models requires reapperances of bed variation patterns and scour depths. Questions here are the influences of sand waves which occur usually when comparatively small diameter bed materials were used. Ripples and dunes develop not uniform on the beds of the channels, depending upon the ununiformity of velocity distributions, and are considered to affect roughnesses, bed material load distributions, and turbulence structures of the flows. But, from the view point of the general situation, sand waves do not affect and change the flow patterns much, and can in general neglect their effects. Small sand waves occur on the alternate bars, and longitudinal voxtexes which were observed in proto type can be recognized in the flows of models even on the sand waves. To compare the rates of sediment transport may be another method of verification.

Conclusion

A great many hydraulic model tests of alluvial channels have been performed, and got fruitful results on river designs. Almost same phenomena as in models have also been observed in proto type, to increase confidence of model experiments, with the results of increase in number of requests to model tests. This may show the deepening of understanding for experiment effects that the model tests are useful to understand hydraulic phenomena, to increase security and to make safety factors uniform, roughly at the cost of about 0.1 per cent of working expenses.

But, more severe contents will be requested to model tests. Solidication of field data and application of fundamental studies are expected to make modeling techniques developed markedly. Especially, practical use of small scale and distorted models will be promoted in order to reduce cost and time. Needs of studies on development of new bed materials and experiment methods for rivers which have few field data are suggested. Progress of accuracy in measuring technique, automatic measurements, computer analysis of data and automatic drawings are necessary, and also indoor experiments are requested.

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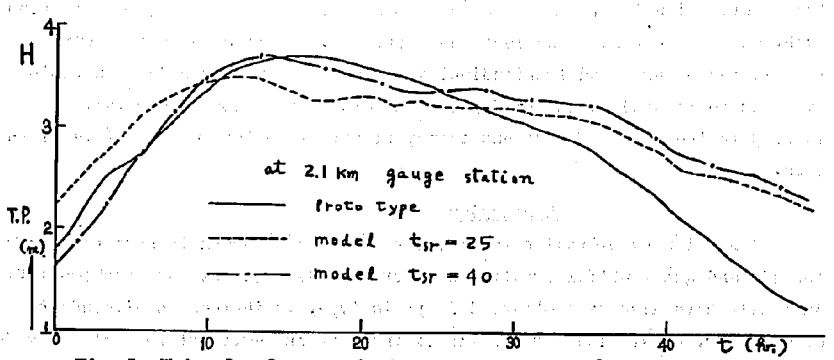


Fig. 1 Water-level curve during the flood of 1969 at Mogami estuary

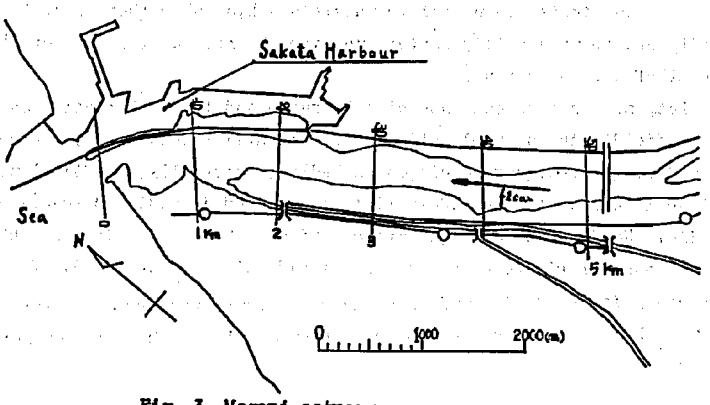


Fig. 3 Mogami estuary

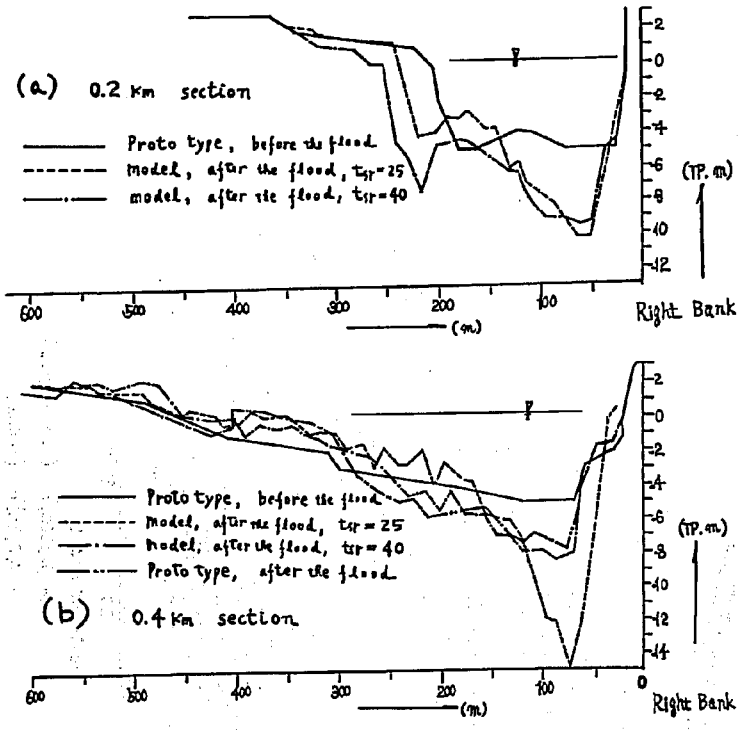
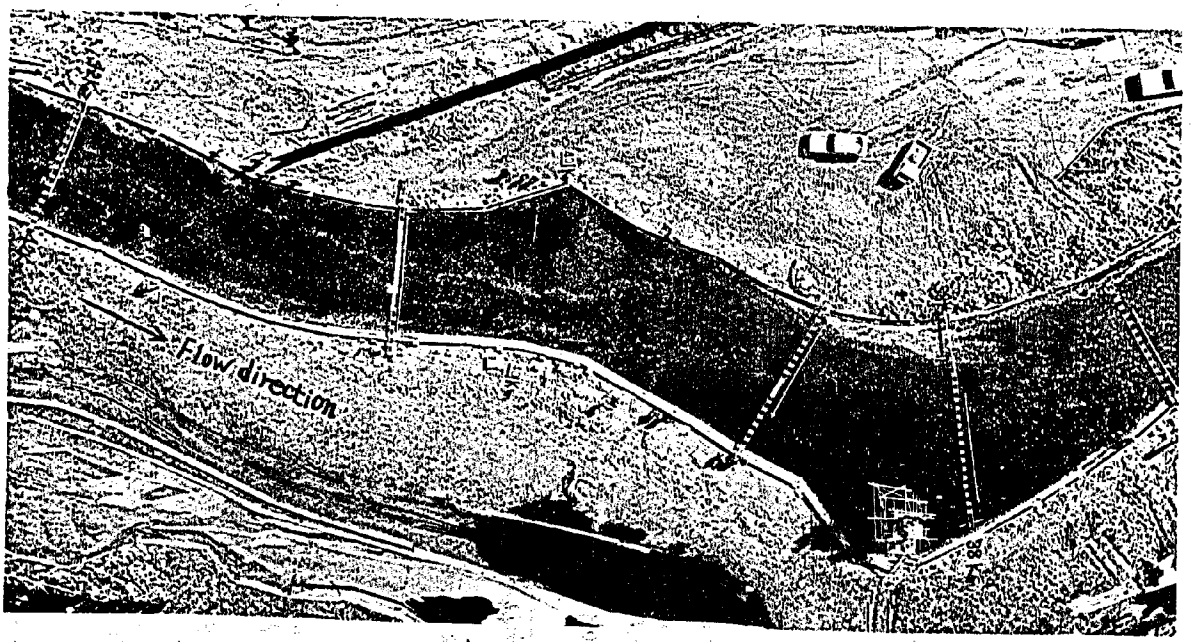


Fig. 2 Cross-sections at Mogami estuary
(flood of Aug. 1969 Q_{max}= 6067 m^3/s)



Fig. 4 Surface velocity distribution at Yodo river



scale of model

Q= 3,500m³/s (proto typo)

= 0.231m³/s (model)

Photo. 1 Aerial View of model of Tone river.

A Cable-stayed Bridge and Its Aerodynamic Design

by

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A Cable-stayed Bridge and Its Aerodynamic Design

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K. Yokoyama

Summary

Nowadays the number of construction work of cable-stayed bridges has rapidly increased in Japan. Though its economic span length is far smaller than a suspension bridge, a cable-stayed bridge expects much its bending and/or torsional rigidity on the cable system, so that it may be classified as a flexible structure and the due regard should be paid to its aerodynamic behavior in designing.

The paper outlines the present status of the aerodynamic design of a cable-stayed bridge. In the beginning it introduces the details about the progress of bridge span length with time and emphasizes the importance of aerodynamic design. Secondly the engineering problems occurring in the aerodynamic behavior and their interpretation are explained by the use of simple model. Thirdly some examples of countermeasure to prevent the aeolian oscillation (vortex excited oscillation) and the torsional flutter are introduced. In the end the paper refers to a comprehensive field test on model bridges which has been developed to deepen out understanding of the behavior of a bridge under wind action.

Development of a cable-stayed bridge

The principle of a cable-stayed bridge, in which the deck girder is stiffened by the tension members stretched diagonally from a main tower, is originated in the drawing by a Venetian engineer in the early seventeenth century, and a wooden bridge with short span length designed by the principle was constructed in the late eighteenth century in Germany.

Since then the cable-stayed bridge has been improved for the reasons that:

(a) The development of rapid and precise calculation technique of a structurally indeterminate system by not only the infinite deflection theory but the finite deflection theory by using electronic digital computers,

(b) The development of the steel slab structures has opened the way to design a bridge with wider deck compared with the conservative bridge composed of slab and beam system,

(c) The development of cable material not only with high tensile strength but with high fatigue strength has enabled an engineer to design such a bridge as requires a high rigidity by the cable system, and

(d) The cable-stayed bridge has its modern and functional beauty.

A number of cable-stayed bridge has been constructed in the world after the World War II. Fig. 1 shows how the span length of the world longest bridges has been developed with time according to the types of bridge. The suspension bridge, the engineering feature of which was completed by the late ninteenth century, has been constructed over estuaries or entrances of bay with the far long span and it seems that the span length of a suspension bridge has not been extended in recent years. The same tendencies may be seen in the cantilever truss bridge and the arch bridge, and the world longest bridge for each type was constructed by the thirties.

On the contrary, the cable-stayed bridge shows the rapid progress in its span length since the middle of 1950s, and the construction of cable-stayed bridge has been focused in these fifteen or twenty years.

In the meantime the construction of Katsuse Bridge in 1959 opened the history of the cable-stayed bridge in Japan, and Jinno Bridge, Maya Bridge, Onomichi Bridge, Toyosato Bridge, and Ishikari Bridge have been opened to the public successively. Besides Suehiro Bridge which will be referred in the following, there are several plans to construct the cable-stayed bridges with longer span of some 450 m in the belt road along the Tokyo Bay, Ise Bay, and Osaka Bay respectively. The Duisburg-Neuenkamp Bridge completed in 1970 in Germany has the center span length of 353 m and is proud of the grand appearance. However it seems

that she will give her seat in the near future when one comes to think of the progress in bridge engineering.

Aerodynamic problems in cable-stayed bridges

The deck girder of a cable-stayed bridge is usually composed of either box-girder with cantilevers on both sides or box-girder without cantilevers. The Lower Tamar River Bridge in Tasmania has a truss girder and such instances are few and far between. This is mainly because that there are more terms advantageous to box-girders than trusses in cable anchorage device and appearance. We may safely say that the characteristics of a cable-stayed bridge will be lost when we dare to use a truss even if it has a sufficient aerodynamic stability. The truss stiffened girder has proved to have sufficient aerodynamic stability. Therefore the discussion in the following shall be restricted within the cable-stayed bridge with box-girders.

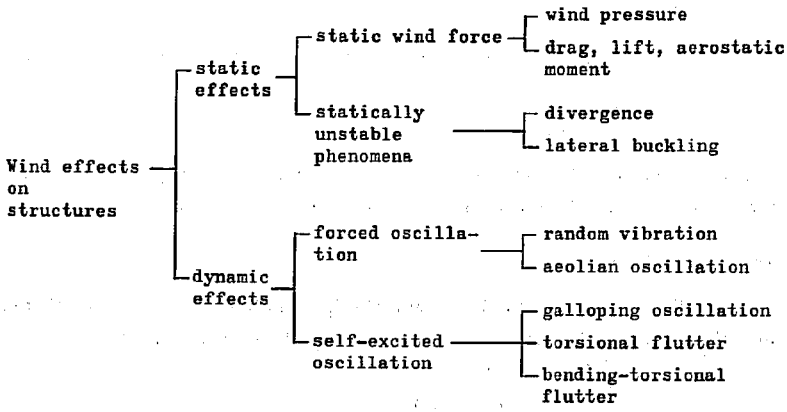
(Static wind loads)

The box-girder constituties the characteristic feature that it has a smaller drag coefficient and a large derivative of lift coefficient compared with a truss girder within the small angle of attack. This is seen much in the box-girder with cantilevers on both sides. The drag coefficient of a truss girder can be given roughly by an empirical formula, $1.1\times\sqrt{3/2\phi}$ (where ϕ is the rolidity ratio in side elevation). However it will be 1.0 - 1.5 for a box-girder without cantilevers and 0.7 - 1.0 for with cantilevers. Then drag force on a box-girder becomes 1/2 - 1/2.5 times of a truss girder, so that it is advantageous in a longer span bridge. Moreover the large derivative in lift coefficient means that the galloping oscillation which is fatal to the bridge may not occur. However from the theoretical viewpoint the large derivative seems to be disadvantageous to forced vibration due to turbulent wind. This kind of dynamics remains as an important problem which has to be studied by the field observation.

(Dynamic effects of wind)

There are proposals of classification system as to the dynamic effects of wind on a structure. The representatives are given by the

Special Committee on Wind Effects in the U.S. and by A.G. Davenport. T. Okubo, one of the co-authors, has proposed the following system of classification;



Now the following three phenomena seem to be most important, namely, torsional flutter, aeolian oscillation, and galloping oscillation, and the stability conditions for a sinusoidal motion under wind action are derived in the following paragraph by the use of a simple model.

(a) Heaving motion

Following Fig. 2, we may write the heaving motion of a structure and aerodynamic force acting on it as follows:

$$\begin{cases} h = h_0 e^{i\omega t} \\ L = (L_R + i L_I) e^{i\omega t} \end{cases}$$

where L_R and L_I denote the real and imaginary part of unsteady aerodynamic force respectively. When we integrate the work done by the wind during one cycle of oscillation, it will be $\pi h_0 L_I$, and the amplitude will increase with time when $\pi h_0 L_I > 0$ or $L_I > 0$. When we introduce the quasi-steady theory, $L_R = 0$ and $L_I = -\frac{1}{2}\rho V A \omega h_0 \left(\frac{\partial C_L}{\partial \alpha} + C_D\right)$ then the condition $L_I > 0$ may be substituted by $\frac{\partial C_L}{\partial \alpha} + C_D < 0$, where $\frac{\partial C_L}{\partial \alpha}$

and C_D denote the lift derivative and drag coefficient. This condition was given by Den Hartog for the analysis of galloping oscillation of an ice-cladded transmission line.

(b) Pitching motion

The similar manipulation can be applied for the pitching motion:

$$\begin{cases} \alpha = \alpha_0 e^{i\omega t} \\ M = (M_R + iM_I) e^{i\omega t} \end{cases}$$

The work done by wind during one cycle of oscillation may be given as $\pi \alpha_0 M_I$ and the condition of negative damping is $M_I > 0$

(c) Coupled motion

In the case of coupled motion, the heaving and pitching oscillation, and their aerodynamic forces may be expressed as follows:

$$\begin{cases} h = h_0 e^{i\omega t} \\ \alpha = \alpha_0 e^{i(\omega t - \phi)} \\ L = (L_R + iL_I) e^{i\omega t} \\ M = (M_R + iM_I) e^{i\omega t} \end{cases}$$

where ϕ denotes the phase difference between the heaving oscillation and the pitching one. After a similar calculation the work may be obtained as follows:

$$\pi h_0 L_I + \pi \alpha_0 (M_R \sin\phi + M_I \cos\phi)$$

By introducing the quasi-steady aerodynamic force, the condition will be

$$\frac{\partial C_L}{\partial \alpha}\left(-\frac{\omega h_0}{\alpha_0 V} - \sin\phi\right) < 0$$

under the assumption of $M = 0$

Though the manipulation mentioned above is useful to the qualitative understanding, the aerodynamic behavior of a real structure is much complicated due to the non-linearity in the aerodynamic force at a large amplitude of oscillation.

The aeolian oscillation is originated in synchronization of vortex frequency in the wake with a natural frequency of the girder in either bending or torsional mode. The frequency of vortex shedding is governed by Strouhal number S_t in the following formula

$$S_t = N_v \cdot D/V$$

where N_v, D and V denote the vortex frequency, the representative depth of girder, and wind velocity respectively. For instance $S_t = 0.125$ for a rectangular cylinder and $S_t = 0.19$ for a circular cylinder at the Reynolds number of some 5×10^5. So that the vortex frequency increases with wind speed and it coincides with the natural frequency of a girder when the structure starts to vibrate. This is the main cause of aeolian oscillation. Thus the vortex force still acts on the structure even when we restrict the motion of the structure. That is the reason we assign the aeolian oscillation to forced oscillation. However there is another thought that classifies it into self-excited oscillation as an interaction between the amplitude and the wind occurs at a large amplitude. Such phenomena as the divergence and the forced random oscillation seem to give slight effect.

Aerodynamic behavior of a cable-stayed bridge

It is recognized that the wind tunnel testing is the most reliable method to investigate the aerodynamic behavior of a structure. There are two kinds of testing method, namely, one is the full model test in which the whole structure of the prototype is reduced by the use of the similitude law, and the other is the section model test in which the typical section of the prototype is reduced. Usually the section model test is applied for a cable-stayed bridge.

Now Fig. 3 gives a qualitative example of wind tunnel test results on a proposed cable-stayed bridge. The ordinate is the amplitude, while the abscissa is wind velocity. When we increase the wind velocity the section model mounted by coil springs starts to vibrate around the critical velocity V_s at which the natural frequency of heaving or torsional mode coincides with vortex frequency defined by Strouhal number, and when the amplitude is fairly large, the frequency of vortex in the wake is kept constant within the narrow band of wind velocity around V_s by the effect of motion of the model (Lock-in phenomena). By the careful observation we may find a weak oscillation at the half of the critical wind velocity, and it seems to be a sub-harmonic oscillation.

In the high wind velocity range we observe another type of oscillation with far violence, and the change of amplitude may be determined according to the initial excitation as shown by arrows in Fig. 3. When the damping ratio of the system is small the amplitudes at Vs and Vc become large while it becomes difficult to distinguish Vs from Vc .

These circumstances may be explained by the help of unsteady aerodynamic force. The qualitative characteristics of imaginary part (L_I) of aerodynamic force acting on a model under heaving motion is given in Fig. 4 where the ordinate is L_I while the abscissa is nondimensionalized wind velocity (V/fD) It is easily expected that the flutter defined by $L_I > 0$ will occur at the velocity range where $L_I > 0$

It is necessary to prevent the occurrence of aeolian oscillation and flutter. The critical wind velocity of flutter must be higher than the expected wind velocity that may attack a bridge during its life as the flutter oscillation is fatal to the bridge. Normally the type of flutter expected in case of a cable-stayed bridge is torsional one. It is fairly easy to get the critical wind velocity of flutter free from danger by selecting a proper aerodynamic configuration of the girder because of its high torsional rigidity. On the contrary it is not so easy to suppress the aeolian oscillation. It is mainly because the cross section of a bridge girder is geometrically bluff and the separation of flow is inevitable. The intensity of aeolian oscillation is much influenced by the structural damping ratio and the mass as well as the geometrical configuration. The situation may be explained as follows by using the unsteady wind force.

Considering the heaving motion, we may write the equation of motion as

$$M\ddot{h} + K_1 h(1 + i g_h) = \frac{1}{2}\rho V^2 d l (L_2 + i L_3) e^{i\omega t}$$

Introducing the relation $h = h_0 e^{i\omega t}$ into the above equation and after a little manipulation, we obtain

$$\begin{cases} \omega^2 = \omega_h^2 - \dfrac{\rho V^2 d L_2}{2 M h_0} \\ L_1 - 4\pi \left(2\dfrac{M}{\rho d^2 l}\right)\delta_s \left(\dfrac{fd}{V}\right)^2 \left(\dfrac{h_0}{d}\right) = 0 \end{cases}$$

where $\omega^2_s = K_s/M$, $f = \omega_s/2\pi$ and $\delta = \widetilde{xy}_s$. Therefore we can obtain the relationship between h_o/d and V/fd against the parameter of $\frac{2M}{\rho d^2 l}\delta_s$ when we combine the above equation with unsteady aerodynamic coefficient as shown in Fig. 5.

How to suppress the oscillation of a cable-stayed bridge by wind

The authors have hitherto explained that there is a fair chance for occurrence of the aeolian oscillation in heaving or torsional mode at comparatively low wind velocity and of the flutter due to the negative damping effect of wind at high wind velocity, and that they are mainly caused by the separation of flow at the sharp edge of the girder.

The aeolian oscillation occurs at low wind velocity, say 10 - 15 m/sec, and it may be natural to consider that there are the pedestrians and vehicles on the deck at that time. So that the occurrence of the aeolian oscillation, even if it does not cause the structural damage, may decrease the essential serviciability of bridge. The flutter at high wind velocity must be prevented in order not to give the structural damage.

(a) Prevention of aeolian oscillation

The followings seem to be important to increase the aerodynamic characteristics:

(i) Aerodynamic configuration of a structure
(ii) Rigidity of a structure
(iii) Structural damping

By the way the draft design rules for highway box-girder of the U.K. provides the critical wind velocity which varies according to the nature of wind at the construction sites, namely, for instance 40 m/s for esturial bridges and 25 m/s for low level bridges inland, and if the critical wind velocity does not exceed the limiting velocity, consideration shall be given to vibration, fatigue of material, and expansion joints. This design rule seems to prevent the aeolian oscillation by increasing the structural rigidity. However it is difficult from the economic viewpoint to introduce the thought directly to the cable-stayed

bridge in which much rigidity is expected in the cable system.

The installation of mechanical damper to the structure to increase the damping capacity introduces problems in reliance and maintenance. There is a similar idea to install a dynamic damper to the structure, but there has not been any instance of installation to a real bridge.

It is after all the best way to increase the aerodynamic characteristics by modifying the cross sectional configuration of the girder. To accomplish the purpose the following methods may be proposed:

(i) To suppress the separation of flow as much as possible,

(ii) To break down the shedding vortices so as to diminish the vortex force.

As to the method (i) it is preferable to sharpen the side edges of the bridge which parts are the most important for the separation of flow. It is also preferable to install fairings around the leading edge. Therefore, such structural element as solid handrail or high curb is undesirable to the wind stability. As to (ii) it is preferable to suppress shedding of large scale vortex by changing the longitudinal cross sectional shape around the windward edge. Some of examples are to change the solidity ratio of handrail and to attach triangular or trapezoidal blocks around the edge.

Some of the examples are given in Fig. 5. It is impossible to say what method is the best because the effects vary according to the basic configuration of the girder. For example, in case of Suehiro Bridge, the fairing was found to be the most effective after the comprehensive wind tunnel test on section models in which the effects of the discrete closure of hand rail, the installation of sharp plate and fairing in both sides of the girder, and decrease of curb height were investigated. The aeolian oscillation was completely suppressed within the wind inclination angle between -6° and 6° by installing the fairing in both edge of the girder, and at the same time the critical wind velocity was increased from 75 m/s in the original design to some 100 m/s.

The good consideration is necessary for such accompanying effects of installation of the device mentioned above as the reduction of

serviceability by wind conversion over the bridge deck and the defilement of appearance. The wind conversion at the center of outer lane due to installation of fairing was 5 percent.

(b) Prevention of flutter

The torsional rigidity is the most important factor as the mode of oscillation in the flutter is torsional, and it is preferable to select the box girder for basic configuration to ensure the sufficient torsional rigidity. The effective height-width ratio of a box-girder with cantilever on both sides is so small that it may not happen galloping oscillation.

Other problems

The aerodynamic problems and the countermeasures to them in the cable stayed bridge have so far been discussed. The deep consideration should also be paid to the stability of the towers and bridge girders during their construction stage.

The final mention is to the wind tunnel test. Usually a wind tunnel test is carried out in the smooth flow so that the turbulence effects are neglected. The effects of air viscosity is not simulated either. The separation and reattachment of air flows may be essential in the stability of a streamlined structure similar to a aircraft wing. Therefore it is necessary to establish the similitude law which is applicable to this type of structures.

The bridge is originally designed to have sufficient wind resistibility up to the design wind velocity, and it is very difficult to reproduce and record accurately the dynamic behavior like aeolian oscillation in a real structure. By this reason there are few observed data which enable the comparison between the wind tunnel test results and those in the real structures.

In order to overcome the situation the Honshu-Shikoku Bridge Authority started a field test by using a sectional model in response to the proposal by the Special Committee on Aerodynamic Effects set up in the Japan Society of Civil Engineers with the request of the Authority.

The test started in the last January, and is expected to give us the broard information about the aerodynamic behavior of bridge structures under wind action.

Conclusion

The present status of aerodynamic design of cable-stayed bridge has been explained and from the discussion the following may be concluded.

(a) The due consideration shall be paid on the aerodynamic design of a cable-stayed bridge.

(b) The wind load on a box-girder is smaller than that on a truss girder, but much attention shall be given to the box-girder because it may cause the aeolian oscillation or the flutter.

(c) The characteristics of unsteady aerodynamic force gives good information when we study the aerodynamic behavior of a cable-stayed bridge.

(d) The supplementary device for suppressing the aeolian oscillation shall be chosen not to harm the original serviceability.

(e) It is firstly necessary to collect data of field observation in order to enhance the reliability of wind tunnel testing, and a field test for the purpose has recently been started in Japan.

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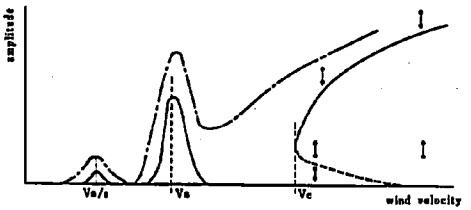


Fig-3 Wind velocity vs amplitude

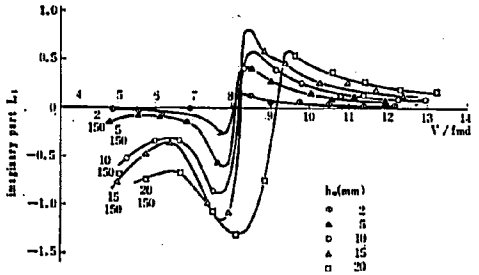


Fig-4 L_1 vs v/fd

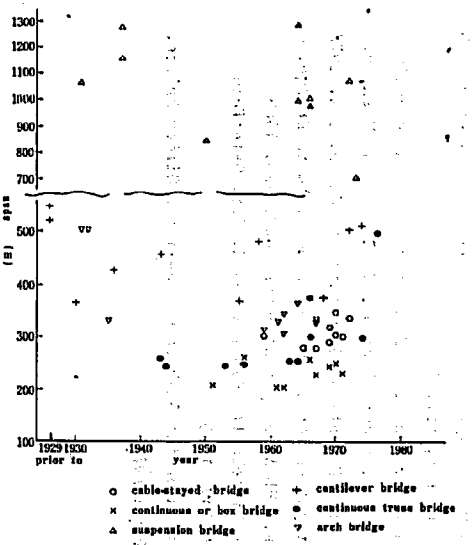


Fig-1 Bridge span vs construction year

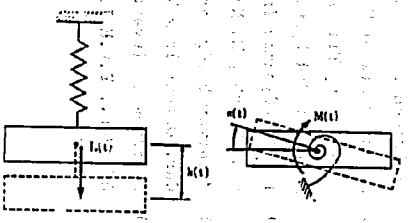


Fig-2 Model system

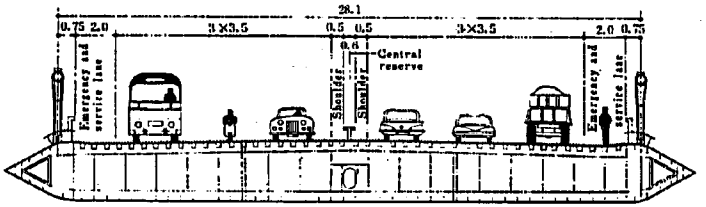


Fig-5.1 Little belt Bridge

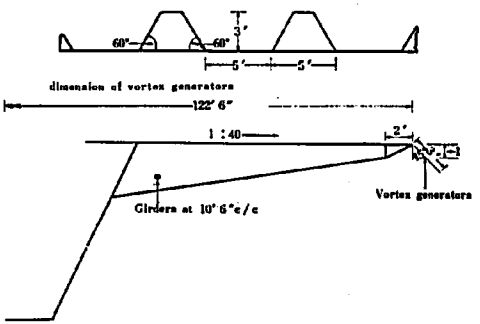


Fig-5.3 Vortex generator (Lower Yarra Bridge)

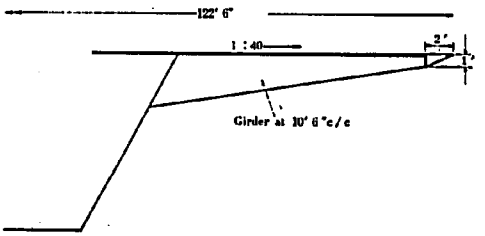


Fig-5.2 Lower Yarra Bridge

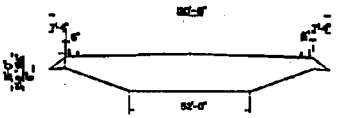


Fig-5.4 Proposed Burrard Inlet Bridge

EARTHQUAKE ENGINEERING DESIGN OF EARTH FILLS

by

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INTRODUCTION

Earth fills are divided into a number of types according to their applications such as fill dam, levee, road embankment, railroad embankment, etc., each being constructed in different size.

These various types of earth fill may have similar configurations but are intended differently. The fill dam or levee is assigned a function to stop water, while the road or railroad embankment has a role to carry the load of traffic traveling thereon. From the difference in the role or configurational dimensions of earth fill, it follows that should the earth fills be undergone a failure in the event of an earthquake, there is also a difference in the extent of damage or easiness of repair work. This leads to a variety of methods or principles of design of earth fills with respect to the earthquake engineering.

This report describes earthquake engineering designs in practice of earth fills and discusses concepts and problems concerned with the earthquake engineering design along with the results of failure tests of earth fills carried out by use of a large vibrating table as they relate to the slip calculations.

It must be noted that at the first and second joint panel meetings, Dr. Fukuoka reported the earthquake engineering design of soil structures in general so that the present report is limited to the earth fills only and particularly to those matters which have not been reported.

1. Fill Dam

As a standard for study of the stability of a dam at the time of an earthquake, a "Revised Dam Design Standard" [1] of the Japan Large Dam Council is presently available. The standard was established in 1971 through revision made recently of the Dam Design standard established in 1957 and used until the revision.

(1) Safety Factor against Slip

The revised standard provides that the dam and foundation should have a required degree of safety against slip failure. It also provides that the safety should be tested, as a rule, with respect to the circular slip surfaces under the slice method, and gives the safety factor as being expressed by the equation

$$n = \frac{\Sigma (c\, l + (N - U - N_e) \tan \phi)}{\Sigma (T + T_e)} \qquad (1)$$

where n: Safety factor,

N and T: Vertical and tangential components of the load working upon the slip surface of the respective slices,

U: Pore pressure acting upon the slip surface of the respective slices,

N_e and T_e: Vertical and tangential components of the seismic load working upon the slip surface of the respective slices,

ϕ and c: Internal friction angle and cohesion of the slip surface material of the respective slices,

l: Length of the slip surface of the respective slices.

The value of safety factor is given by 1.1 plus a specified value for allowance. The value of allowance should be determined properly in consideration of the property, quality control and method of application of the material, but a value of 0.1 is given as a recommended value.

(2) Seismic Force

The seismic force is assumed to act horizontally upon the dam in a value of the weight multiplied by a seismic coefficient given in the Table 1.

However, when the reservoir is empty or of a surcharge level, the seismic coefficient is taken to be 50 percent of the value shown in the table, and in the event of an unusual flood, no seismic coefficient is taken into consideration.

The values of seismic coefficient given in the table have a certain range, and thus any value to be used in design will have to be chosen appropriately within such range with consideration given to the district, soil condition and importance of the dam concerned.

What is different from the table under the previous standard is the shift of Aomori and Iwate from the right column to the left. This change was made in consideration of the Tokachi Oki and other earthquakes occurred in this district during the past several years.

Table 2 shows design seismic coefficients used for rock fill dams in our country against probable seismic coefficients obtained from the data of past earthquakes. As seen, relatively small values are employed for the design seismic coefficient. The value ① represents the probable seismic coefficient obtained from the data since 1605, and the value ② represents the probable seismic coefficient obtained from the map of Dr. Kawasumi.

(3) Conditions for Calculating Safety against Slip

The safety against slip is to be calculated under the following five conditions:

(a) When the reservoir is at a high water level while the seepage flow is in a steady state,

(b) Immediately after the completion with the pore water pressure developed during the construction work still remaining,

(c) When the reservoir is at an intermediate level while the seepage flow is in a steady state,

(d) For dams where rapid and extensive changes of the level are carried out usually, when the head declines sharply from the high water level to a low water level while the pore water

pressure is still remaining,

(e) At a level of surcharging is carried out.

In the cases of (d) and (e), the seismic coefficient value should be reduced by 50 percent as described in the foregoing.

2. Levee

Design of the cross section of a levee is made generally in accordance with the Ordinance for Structures of River Control Facilities, Etc. (draft)[2].

Factors to be considered in making a design are the river level, discharge, flow, topography, soil condition, flood continuation time, etc., and the seismic force is not considered normally.

It is provided that the levee should have a gentle slope gradient less than 1 to 2 with berms provided at every 2 to 5 meters of height.

Each river has a standard section provided so that it is not in practice to test the stability of a levee at respective points by calculations but a levee according to the standard cross section is designed and constructed. Since the soil engineering considerations are not given sufficiently, there are variations in the stability of levees. Particularly, as the past record indicates, where the soil condition is not good, earthquake damage is extensive.

Conventionally, earthquake engineering considerations were scarcely given to earth structures such as levee, because if damaged they could be relatively readily restored and also because of a very small possibility of a flood and an earthquake occurring simultaneously. However, with recent remarkable development of land along rivers and high level of utilization of low land around estuaries, the earthquake resistance of levees particularly within city boundaries has been called for urgently.

For improving the earthquake proofness of levee, various processes have been tested, and some of them have been executed actually.

They include enlarging the free board or crown of levee, providing a counterweight in front of levee in order to increase the resistance to slip and compacting the foundation with sand compaction piles.

Besides, a compound levee and its quake resistance will be described briefly.

The compound levee is a name given against the earth levee and includes a retaining wall constructed with concrete or other materials as an integral part of the levee.

According to the Structure Ordinance, the compound levee is an exception, the earth levee being a normal procedure. It is stated in the Ordinance that a major reason for making the earth levee as a rule is to prevent damage owing to an earthquake. For example, in the instance of Niigata Earthquake, the compound levee along the Agano River had the joints of retaining wall displaced greatly and permitted water to flow into the land.

In the case of an earth levee, if it had a settlement or slip caused, it will never break or collapse completely. Moreover, it can be readily repaired in the event of damage. Such advantages of the earth levee were strongly recognized. Thus, the Structure Ordinance makes it compulsory to build earth up to the estimated high water level for a compound levee and allows a retaining wall for the portion of free board only. (Fig. 1)

3. Railroad Embankment

The railroad embankment is constructed with a slope of a gradient at 1 to 1.8 as a standard [3]. It had no earthquake engineering aspect taken into account.

At the time of the Tokachi Oki Earthquake, a number of embankment collapses occurred along the Tohoku Line and other local lines, and it took a considerable period for the failures to be repaired. With this damage as an opportunity, the Japan National Railway organized

a committee on earthquake engineering of the railroad embankment and started a concentrative study on the earthquake resistance of embankment on soft ground.

The study covered many aspects including classification of the types of failure of embankment and vibratory analysis. But, what must be noted particularly was the model experiments using a large vibrating table. Although the results of the study are not yet summarized in the form of design criteria, they are highly valuable. Here, among other achievements, the stability analysis and earth fill earthquake engineering countermeasures [4] will be introduced briefly.

(1) Method of Calculation for Slip

The following two analytical processes were presented for the slip failure of earth fill.

(a) The lower end of slip passes approximately the toe of slope

A slip comprising a slip surface along the bottom of the embankment and that passing through the embankment was assumed.

In the calculation, the Coulomb's earth pressure is first obtained with the angle of slip surface within the embankment as β as shown in Fig. 2. The total of this earth pressure and the weight W_1 of the lump of earth in the wedge portion multiplied by the seismic coefficient $W_1 k$, being taken as a seismic earth pressure, the value of β is changed variously to give a maximum value P of seismic earth pressure. When U represents the total of the pore pressure acts along the bottom surface, the safety factor of the lump of earth CBOD against slip is expressed by the equation

$$F_s = \frac{(W_2 - U)\tan\phi' + c'\overline{OD}}{P + W_2 k} \qquad c' + W_1 k$$

where $$P = \tan(\beta - \phi')W_1 - \frac{\cos\phi'}{\cos(\beta - \phi')} \qquad c' + W_1 k \qquad (2)$$

$c = c' \overline{OB}$,

$U = c' \overline{OD}$,

u : Pore pressure.

(b) The slip surface crosses the foundation soil

This is a case where the earth fill and foundation soil are subjected together to a slip failure.

Assuming that the slip occurs along an arc K''KK' about the center O, the safety factor against sliding is expressed as follows from the equilibrium of moments. (Fig. 3)

$$F_s = \frac{(N \tan\phi' + \widehat{K'GK'} \cdot c')r}{Tr} = \frac{N \tan\phi' + \widehat{K'GK'} \cdot c'}{T} \quad (3)$$

where R : Resultant of four forces W_1, W_1k, W_2 and U,

N, T: Forces of two components of the resultant R,

W_2 : Submerged weight of KCK',

U : Total of dynamic pore water pressure along the slip surface.

The foregoing two analytical methods are used to describe the results of experiments by means of the large vibrating table, and each of them explains the phenomena well.

The features of these analytical methods are that the dynamic pore water pressure are introduced into the calculation and, in the case of the method (b), that the force developed by horizontal to the earth mass in the portion of foundations is neglected.

The dynamic pore pressure may be produced by vibration if the soil is saturated. While its value depends on the soil property and vibratory characteristic, the results of experiments indicate that it is produced at a value about 0.5 of the overburden pressure. It is stated that the seismic force upon the foundation soil is neglected, because the acceleration in the foundation soil at the time of an earthquake is small when compared with the acceleration in the earth fill.

The study also points out that for the horizontal acceleration to

be used in the calculation, the maximum acceleration at the time of an earthquake should not be employed and that it is necessary to use a reduced or modified seismic coefficient through introduction of the concept of average acceleration (or effective acceleration) and the cycle of vibration.

It seems that further study will be required for the way of applying the values of horizontal seismic coefficient and dynamic pore pressure.

(2) Earthquake Proof Measures for the Embankment

For protecting or reinforcing the railroad embankment against an earthquake, various methods are conceivable, and it is one of such methods to design so that the embankment may not have the pore water pressure increased. This will be achieved by constructing the embankment with coarse materials such as debris or providing a drain layer of coarse materials between the ground surface and the embankment.

There are also measures designed to restrain the embankment to prevent it from moving should it tend to fail owing to the rise of pore water pressure or shortage in the strength of the foundation soil. Such measures include installation of sheet piles in two rows at the locations of both toes of slope of the embankment or reinforcing the embankment by reinforcing materials installed at the bottom of, or inside, the embankment.

A process of reinforcing the soil for preventing liquefaction or lateral flow of the soil is also considered to be an effective measures. As belonging to such process, the sand compaction pile method may be listed.

Some of such works were chosen, and they were subjected to vibration tests by means of large models for comparison of the methods. Through comparison of the results of vibration tests with the earthquake damages recorded in the past, the protective works shown in Table 3 were proposed for the respective soil conditions. The proposition is highly evaluated because the effects of the protective works are com-

parable with one another.

4. Road Embankment

Similar in the scale to the railroad embankment is the road embankment, but it has no consideration made of protection against damage from an earthquake. According to the "Road Earth Work Manual"[5] and "Design Manual"[6], the gradient of the slope is determined by the type of soils in the case of ordinary embankments (less than 10 - 15 m). For exceptionally high embankments, it is provided to calculate the slip, but this test is intended for examination of the strength of soils and seepage water within the embankment.

As the past record indicates, the embankment built on the soft ground is vulnerable to damage by an earthquake. But, it is relatively easily repaired should it be damaged, and the secondary damage brought about by the embankment failure is not so great generally. Thus, with respect to the earthquake proofness, no consideration is given normally.

However, where an oil pipe line is installed under the road embankment as is carried out recently, it is necessary to investigate the stability of embankment at the time of an earthquake, and a consideration is being made of making it a requirement to investigate and test the earthquake proofness.

5. Artificial Island for Transverse Road across Tokyo Bay

Although a special type of earth fill, an artificial island and its stabilization now being contemplated as one of the investigations for the Tokyo Bay transversal road construction project[7] will be described briefly.

The artificial island program is still flowing, but it is designed to construct an artificial island through earth filling in water over the soft soil at the central part of Tokyo Bay.

Where the earth fill is to be made, the water has a depth of about 30 meters, while only sand is available as a material for earth fill. Thus, the program involves many problems with respect to earthquake engineering. With no experience for such a large scale earth fill, there are a number of unknown factors which have to be clarified such as liquefaction or deformation of the earth fill.

It is intended to prevent the liquefaction of earth fill by thorough compaction of sand and covering the surface of earth fill with rocks, and the stability of earth fill as well as ground soil is being examined also.

For testing the stability at the time of an earthquake, the slice method is employed, but the slices are positioned obliquely as shown in Fig. 4.

The safety factor is obtained by the equation

$$F_s = \frac{\Sigma c_i l_i + \Sigma \left(\sqrt{1+k^2}\, W_{is} \cos(\alpha_i + \theta) + W_{ir} \cos\alpha_i - u_i l_i \right) \tan\phi_i}{\Sigma \left(\sqrt{1+k^2}\, W_{is} \sin(\alpha_i + \theta) + W_{ir} \sin\alpha_i \right)}$$

$$= \frac{\Sigma c_i l_i + \Sigma \left((W_{is} + W_{ir}) \cos\alpha_i - k W_{is} \sin\alpha_i - u_i l_i \right) \tan\phi_i}{\Sigma \left((W_{is} + W_{ir}) \sin\alpha_i + k W_{is} \cos\alpha_i \right)} \quad (4)$$

where k : Design seismic coefficient in the horizontal direction,

θ : Angle of the seismic force with the vertical line, $\theta = \tan^{-1} k$,

W_{is} : Weight of the earth fill in the divided slice i (total weight including water),

W_{ir} : Weight of the ground in the divided slice i (total weight including water).

In the study of the artificial island, it is assumed that only the earth fill portion is subject to the horizontal acceleration, with no consideration given to the acceleration acting upon the ground soil. It is also assumed that the earth strength and pore water pressure are the same with those where no seismic force is present.

No horizontal acceleration is applied to the soil, because as de-

scribed previously, the vibration of the fill portion is far greater than that of the ground soil and from the consideration that in the case of a large earth fill as the present artificial island, the whole earth will not have a uniform acceleration applied so that a reduced seismic coefficient may be used.

6. Earth Fill Failure Tests by Means of Large Vibrating Table

Now the model tests carried out to reproduce the phenomenon of failure of an earth fill built on a loose sandy soil by an earthquake will be reported briefly.

The model is an earth fill constructed in a height of 1.5 meters over a soil of a thickness of 1.5 meters as shown in Fig. 5. The soil and fill are of the sand lightly compacted.

The failure tests were carried out under two conditions, that is, the ground water level corresponds to the surface of the soil so that the whole soil is saturated with water and the ground water is up to a height one half of the thickness of the soil.

The instruments installed to the model are as shown in Fig. 5. An increasing acceleration was applied to each of these two models until failure of the earth fill took place, and the process was observed. With the model having the soil saturated with water up to one half of the thickness, failure of the fill occurred at a vibrating acceleration of 400 gal, whereas the model having the soil saturated wholly with water collapsed at 200 gal. The acceleration referred to here is that of the vibrating table.

Excessive pore pressures produced during the tests were measured. As an example, the values measured at a vibrating acceleration of 200 gal are shown in Fig. 6.

The slip calculation was made with the circular slip method applied and the values of measurement of the pore pressures introduced.

A part of the results is represented in Fig. 7. In the figure, the

results of calculations are given with two values taken for the cohesion of the earth fill.

As seen from the figure, if the dynamic excessive pore pressures are taken into consideration, the failure of the earth fill in the tests can be well explained.

Conclusive Remarks

In this report, how the stability against earthquake is considered in the respective types of earth fills has been discussed. As described, the earthquake proofness is scarcely taken into account in most of them. If any consideration is given, it is limited to such an extent as to apply a simple method of taking the seismic coefficient and circular slip into consideration. The circular slip is surely a practical method and will be used in the future, but careful examination is particularly required as to what value of seismic coefficient is to be chosen.

Upon review of the methods of stability analysis now carried out for the various types of earth fills, this much can be said that a reduced modified seismic coefficient rather than the estimated maximum seismic coefficient may be used in the calculation and that if the dynamic excessive pore pressure produced in the soil is introduced in the calculation of slip, the phenomenon of failure can be explained with good result.

Comparative experiments of the effects of earthquake proof measures for earth fill will be useful as valuable data. But, for this, accumulation of more data is needed.

Reference

1) Japan Large Dam Council: Revised Dam Design Standard, 1971

2) River Bureau, Ministry of Construction: Ordinance for Structures of River Control Facilities, Etc. (draft), 1972
3) Japan National Railway, Railway Construction Bureau: Manual of Design and Construction Works of Earth Structure, 1968
4) Japan Railway Establishments Association: Research Report on Earthquake Proof Design of Earth Fill, 1972
5) Japan Road Association: Road Earth Work Manual, 1967
6) Japan Road Public Corporation: Design Manual, Vol. 1, Earth Works, Pavement, Drainage and Landscape, 1970
7) Japan Road Association: Report on Design Standard of Transverse Road across Tokyo Bay, 1973

Table 1 Seismic Coefficients for Districts

Type of dam	Tohoku (Aomori, Iwate, Akita, Yamagata and Fukushima), Kanto, Chubu, Kinki and Minami Shikoku	Hokkaido, Tohoku (Miyagi), Hokuriku, Chugoku, Kita Shikoku and Kyushu
Concrete dam and rock fill dam	0.12 - 0.20	0.10 - 0.15
Earth dam	0.15 - 0.25	0.12 - 0.20

Table 2 Design Seismic Coefficient and Probable Seismic Acceleration

Name of dam	Max. acceleration recorded since 1605 (gal)	Design seismic coefficient	Probable seismic acceleration ①		Probable seismic acceleration ②		Remarks
			100 years (gal)	200 years (gal)	100 years (gal)	200 years (gal)	
Mihoro	300	0.12	165	275	170	500	completed in 1960, 131m in height
Makio	210	0.15	120	170	180	500	completed in 1961, 105m in height
Oshirakawa	300	0.12	175	245	170	500	completed in 1963, 95m in height
Yanase	200	0.12	140	175	130	300	completed in 1965, 115m in height
Honzawa	390	0.15	245	330	500	1000	completed in 1965, 73m in height
Kuzuryu	430	0.12	185	255	170	500	completed in 1968, 128m in height
Misakubo	190	0.12	75	110	380	800	completed in 1968, 105m in height
Kisenyama	230	0.12	180	230	320	500	completed in 1969, 95m in height
Shirakawa	140	0.15	90	125	50	150	under construction, 69m in height
Daisetsu	90	0.12	40	65	50	100	under construction, 92m in height
Hirose	180	0.12	150	180	400	1000	under construction, 75m in height
Takase	210	0.15	140	175	150	500	under construction, 176m in height
Nanakura	210	0.15	140	185	150	500	under construction, 125m in height
Shimokotori	260	0.12	140	190	170	500	under construction, 119m in height

(by Fill Dam Section, Public Works Research Institute)

Table 3 Earthquake Proof Measures for Embankment

	Embankment \ Ground	Diluvial soil or fair alluvial soil N>10 in sandy ground N> 5 in clayey ground	Soft ground 10>N>4 in sandy ground 5>N>2 in clayey ground	Extremely soft ground N<4 in sandy ground N<2 in clayey ground
A. Embankment to be made	Debris	- / -	b, c, d / -	a, b, c / -
	Clayey soil	- / 3, (1, 2, 5)	b, c, d / 3, (1, 2, 5)	a, b, c, (e) / 3, (1, 2, 5)
	Sandy soil	- / 1, 4, 5, (2)	b, c, d / 1, 4, 5, (2)	a, b, c, (e) / 1, 4, 5, (2)
B. Existing embankment	Debris	- / -	f, (a) / -	f, (a) / -
	Clayey soil	- / (1, 5, 6)	e, f, (a) / (1, 5, 6)	e, f, (a) / (1, 5, 6)
	Sandy soil	- / 1, 5, 6	e, f, (a) / 1, 5, 6	e, f, (a) / 1, 5, 6

(1) Measures to embankment

- Measures are not necessary
1 Drainage pipe method
2 Net method
3 Sand mat method
4 Interception of ground water
5 Installation of reinforcing material
6 Toe drainage blanket method

Measures to ground

- Measures are not necessary
a Counterweight
b Sand compaction pile method
c Caustic lime-pile method
d Partial displacement
e Installation of sheet pile
f Stabilization of ground at toe of slope
(Sand compaction pile for sandy soil)
(Caustic lime pile for clayey soil)

(2) Measures shown in parentheses are necessary for the embankment composed of volcanic soils

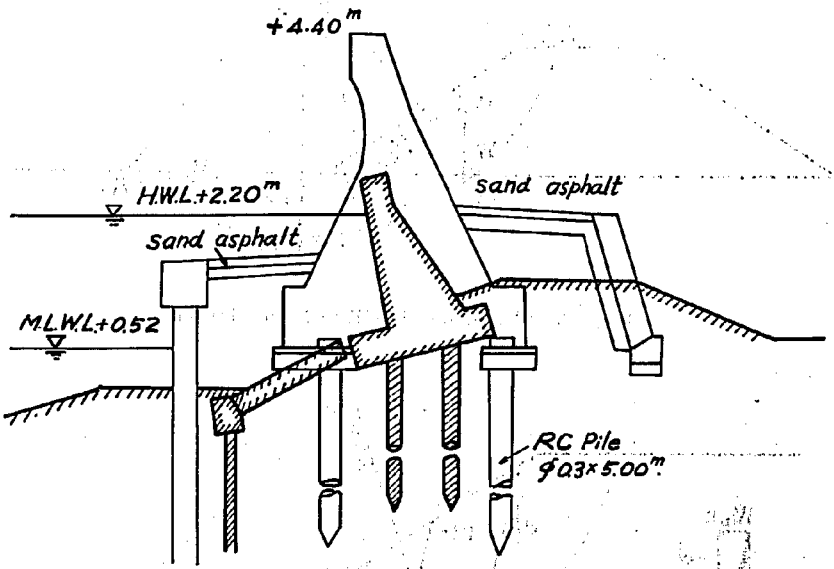


Fig. 1 Cross Section of Damaged and reconstructed Compound Levees at the Agano River

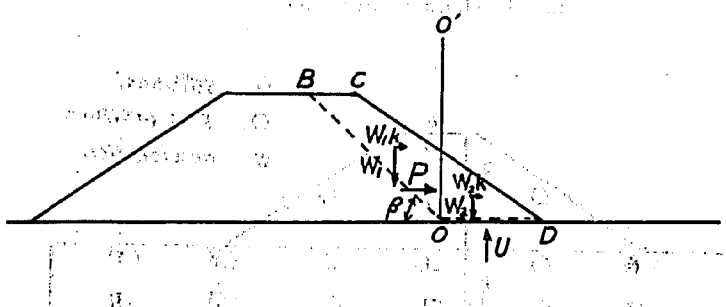


Fig. 2 Slip of Earth Fill

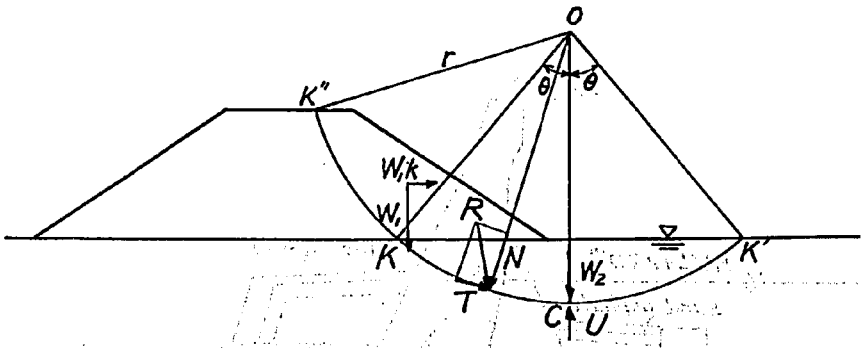


Fig. 3 Slip of Earth Fill and Foundation

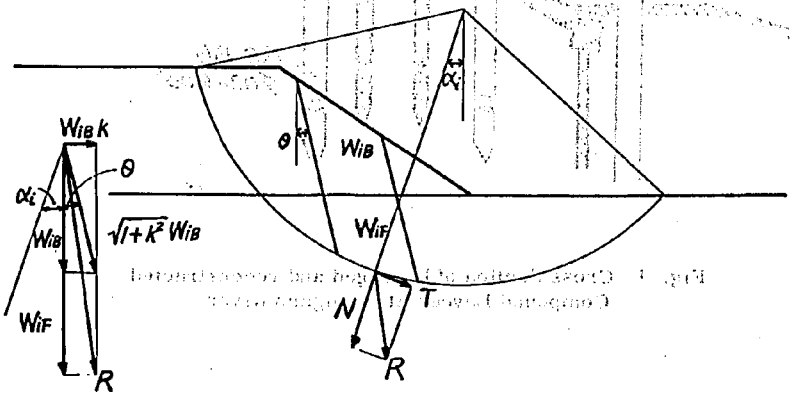


Fig. 4 Oblique Slice Method

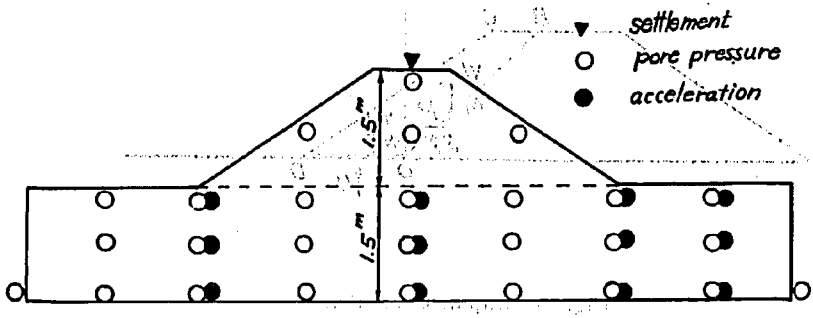


Fig. 5 Installation of Observation Instruments

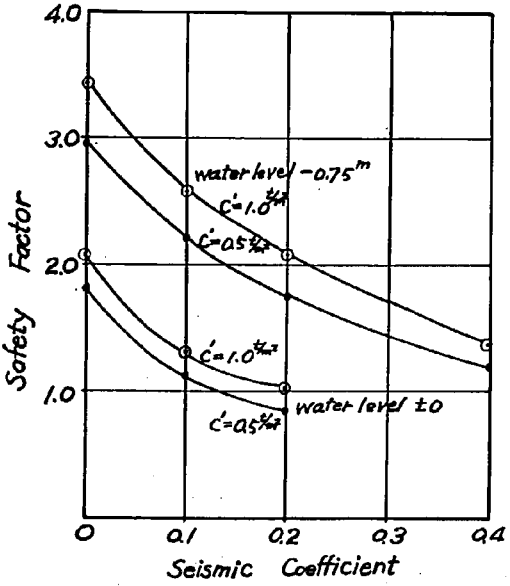


Fig. 6 Example of Slip calculation

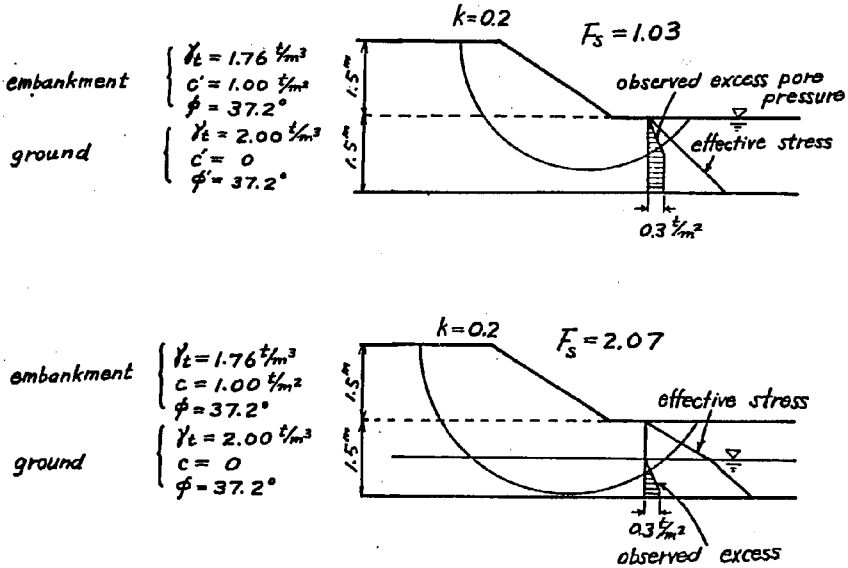


Fig. 7 Results of Calculation

Earthquake Resistant Design Codes

EATHQUAKE RESISTANT DESIGN CRITERIA FOR UNDERGROUND STRUCTURES

by

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1. Introduction

There are two types of underground structures: isolated linelike subsurface structures such as tunnels and pipelines, and foundation structures which support various engineering structures. In this paper proposed earthquake design criteria for pipelines and tunnels embedded into soft soil deposits are summarized briefly.

2. Earthquake Resistant Design Criteria for Petroleum Pipelines

Utility Act For Petroleum Pipelines has become effective since December, 1972 in Japan. It is expected that the Act will rationalize energy supply systems and provide a safer way to transport petroleum in Japan.

Since the land of Japan belongs to one of the most active seismic zones in the world and the population concentrates in coastal areas developed, in establishing technical criteria of petroleum pipelines a special attention was given to preserve safety against earthquake disasters. As for technical criteria for petroleum pipelines, Ministry of International Trade and Industry, Ministry of Transport, Ministry of Construction and Ministry of Autonomy jointly have enforced the ordinance which provides fundamental matters including provisions on structural design method of pipelines and their equipments, procedures for examinations, and apparatuses for preservation of safety of overall pipeline systems. A notification (draft) on detailed regulations is also provided jointly by the four Ministries.

Main parts of the technical criteria of petroleum pipelines are related to earthquake design. The following summarizes principal points of the earthquake design criteria specified in the ordinance and the notification (draft).

(A) Loads in Earthquake Design

The following loads shall be taken into account in the earthquake design.

1. Internal Petroleum Pressures
2. Earth Pressures
3. Vehicle Loads
4. Weight of Petroleum
5. Weight of Pipes
6. Seismic Effects
7. Effects of Differential Settlement of Ground
8. Hydraulic Pressures
9. Bouyancy
10. Other Loads

Appropriate loads shall be selected from the above list. The loads from 1 to 6 on the above shall be generally considered in earthquake design, and the loads from 7 to 10 shall be considered on the basis of the location, the type and the arrangement of the structure.

The following seismic forces shall be taken into account to determine seismic effects in design of the pipeline.

1. Inertia forces due to the dead weight of the structure and petroleum

2. Earth pressures during earthquakes

3. Hydrodynamic pressures during earthquakes

4. Ground displacement during earthquakes

(B) Design Conditions on Bearing Capacities of Soils

Bearing capacities of specified saturated sandy soils which are very vulnerable to liquefaction shall be neglected in design. Bearing capacities of specified cohesive soils and silty soils which are very soft shall be also neglected in design. The weight of soil layers whose bearing capacities are neglected shall have surcharge effects on the lower ground.

(C) Design Seismic Coefficient

The horizontal seismic coefficient at bedrock shall be determined by

$$k_{oh} = \nu_1 \nu_2 k_o \quad (1)$$

where

k_{oh}: Horizontal seismic coefficient at bedrock

k_o: The standard horizontal seismic coefficient at bedrock (= 0.15)

ν_1: Seismic zone factor

ν_2: Factor on utilization of land

The minimum value of k_{oh} shall be considered as 0.08.

The value of ν_1 shall be obtained by Table 1 and Fig. 1, and that of ν_2 shall be by Table 2.

The vertical seismic coefficient at bedrock, k_{ov} shall be one half of k_h.

The horizontal design seismic coefficient shall be determined by

$$k_h = \nu_3 \cdot k_{oh} \quad (2)$$

where

k_h : Horizontal design seismic coefficient

ν_s : Ground condition factor

The minimum value of k_h shall be 0.10. The value of ν_s shall be obtained by Table 3.

The vertical design seismic coefficient, k_v shall be one half of k_h.

(D) Ground Displacement in Earthquake Design

The horizontal displacement of ground surface which is taken into account in the earthquake design shall be given by

$$U_h = 0.202 S_v T k_{oh}$$

where

U_h : Amplitude of displacements at ground surface (cm)

S_v : Standard value of response velocity spectrum (cm/sec)

T : Natural period of surface layer (sec)

k_{oh}: Eq (1)

The vertical displacement of ground surface, U_h shall be one half of U_h. The value of S_v shall be obtained by Fig. 2. The value of U_h is given by Fig. 3 when taking k_o as 0.1. Similar values of U_h proposed by other two institutes are shown for comparison in Fig. 3.

(E) Stress Calculation (Refer to Fig. 4)

Axial stress in straight pipes buried into the soil layer shall be calculated by

$$\sigma_x = \sqrt{\sigma_L^2 + \sigma_B^2} \qquad (4)$$

where

σ_x : Resultant axial stress in the pipe, kg/cm^2

σ_L : Axial stress due to longitudinal deformation of the pipe, kg/cm^2

σ_B : Axial stress due to flexural deformation of the pipe, kg/cm^2

U_h : Eq. (3), cm

L : Eq. (8), cm

D : Outside diameter of the pipe, cm

E : Young's modulus of the pipe, kg/cm^2

α_1 : Coefficient on longitudinal displacement of the pipe. α_1 is expressed by

$$\alpha_1 = \frac{1}{1 + (\frac{2\pi}{\lambda_1 \ell})^2}$$

$$\lambda_1^2 = \frac{K_1}{EA} \quad , 1/cm^2$$

$$\ell = \sqrt{2}\, L \quad , cm$$

K_1 : Longitudinal Displacement Coefficient of subgrade reaction of the layer, kg/cm^2

A : Cross-sectional area of the pipe, cm^2

α_2 : Coefficient on transverse displacement of the pipe

α_2 is expressed by

$$= \frac{1}{1 + (\frac{2\pi}{\lambda_2 L})^4}$$

$$\lambda_2^4 = \frac{K_2}{EA} \quad ,1/cm^4$$

K_2 : Transverse Displacement Coefficient of subgrade reaction of the layer, kg/cm^2

I : Moment inertia of the pipe, cm^4

(F) Determination of Constants of Soil Layers

Shear wave velocities of soil layers shall be generally determined by the field measurements or laboratory experiments.

In measuring shear wave velocities in the surface layer, strains developed in the layer shall be approximately equal to strains expected during strong earthquakes. When utilizing the results of measurements for a small amplitude like an artificial seismic survey, shear velocities in design may be considered as one half of the measured values. When no measurements can be done, shear velocities may be determined in terms of N-value obtained by a standard penetration test.

The following empirical formulas may be employed.

$$\left.\begin{aligned} &\text{For bedrock} \quad : V_{os} = 4 \times 10^3 \sqrt{N.} \ \text{cm/sec} \\ &\text{For surface layer}: V_{is} = 2 \times 10^3 \sqrt{N.} \ \text{cm/sec} \end{aligned}\right\} \quad (5)$$

Next, elastic constants of soil layers shall be obtained by

$$G_s = \frac{\gamma_t}{g} V_{os}^2 \text{ or } G_s = \frac{\gamma_t}{g} V_{is}^2$$
$$E_s = 3 G_s \quad (6)$$

where

G_s : Shear modulus of the soil, kg/cm^2

E_s : Young's modulus of the soil, kg/cm^2

γ_t : Total unit weight of the soil, kg/cm^3

g : Acceleration of gravity ($= 9.8 \times 10^2 \ cm/sec^2$)

Coefficients of subgrade reaction of the soil layer shall be obtained by

$$K_1 = E_S$$

$$K_2 = E_S$$

where

K_1 : Longitudinal displacement coefficient of the subgrade reaction, kg/cm^2

K_2 : Transverse displacement coefficient of the subgrade reaction, kg/cm^2

E_S : Young's modulus of the soil, kg/cm^2

Natural period of the surface layer shall be given by

$$T = C \frac{H}{\bar{V}_{is}} \quad (7)$$

where

T : Natural period of the surface layer, sec.

H : Depth of the surface layer, cm

$\bar{V}_{is}$: For cohesive soil, $\bar{V}_{is}$ denotes average shear velocity of the surface layer whose shear velocities given by V_{is} in Eq. (5) are approximately uniform among the surface layer and for sandy soil $\bar{V}_{is}$ is the shear velocity at the base of the surface layer.

C = 4.0 for cohesive soil

C = 5.23 for sandy soil

Design wave length of the seismic wave propagated in the surface layer shall be given by

$$L = \frac{2 L L_2}{L_1 + L_2} \quad (8)$$

where

L : wave length in design

L_1 : $T\bar{V}_{is}$

L_2 : $T\bar{V}_{os}$

$\bar{V}_{1s}$: Shear velocity in bedrock

Bedrock shall be defined as a soil layer which is considerably stiffer than the surface layer. When it is hard to clearly define a bedrock, bedrock shall be defined as a layer which has a shear velocity of 300 m/sec or more or a layer which has a standard penetration value of 50 or more.

(G) Allowable Stresses and Safety Factor

Material and minimum thickness of pipes employed in petroleum pipelines shall be satisfactory in the light of the notification (draft) specified. Hoop and axial stresses developed in pipes during earthquakes shall be within the respective allowable stresses. Hoop stress due to internal petroleum pressures shall be within 40 percent of the minimum value of the standard yielding points. Also, the resultant stress due to hoop stress, axial stress and shearing stress perpendicular to the pipe axis shall be within 90 percent of the minimum value of the standard yielding points.

Four kinds of allowable stresses are specified: The allowable tensile stress, the allowable compressive stress, the allowable shearing stress, and the allowable bearing stress.

Both of the allowable tensile stresses and the allowable compressive stress shall be one half of the product of the minimum value of the standard yielding points and the efficiency of connections. The allowable shearing stress shall be 0.6 times the allowable tensile stress,

and the allowable bearing stress shall be 1.4 times the tensile stress.

Increases in allowable stresses for combination of various loads are tabulated in Table 4.

(H) Measures for Preventing Seismic Disasters

In addition to the earthquake design specified in the preceding items (A) to (G), measures for preventing seismic disasters shall be considered to cover destructive earthquakes stronger than those considered in the earthquake design. To this aim numerous equipments including a supervising devices of operation, safety controlling devices, pressure gages, leakage meters, cut-off values, removal systems of leaked petroleum, seismometers, dispatching systems, alarming systems, etc. shall be installed in the pipeline system.

Seismometers shall be two kinds: Seismic alarming meters which inform the outbreak of an earthquake which has an acceleration of 80 gals or more, and strong motion accelerometers which are capable of recording time history of acceleration ranging 10 to 1000 gals.

3. Draft of Earthquake-Resistant Specifications for Submerged Tunnels (JSCE)

Since 1971 the Japan Society of Civil Engineers (JSCE) has been making up uniform standards for the aseismic design of prefabricated tunnels embedded under the bottom of the sea consisting of soft soil layers, with a joint commission from the Ministry of Construction and the Ministry of Transport.

It is expected that the JSCE standards will be completed in March, 1974. A interim draft of earthquake-resistant specifications for prefabricated submerged tunnels which were reported in March, 1973 are briefly described hereafter. The draft includes five chapters including general, surveys, aseismic design, dynamic analysis and measures for preventing seismic disasters. It tentatively specifies ways of various surveys needed in aseismic design, design calculation method, methods of dynamic experiments and dynamic analyses, and countermeasures to prevent the safety against unexpected strong earthquakes.

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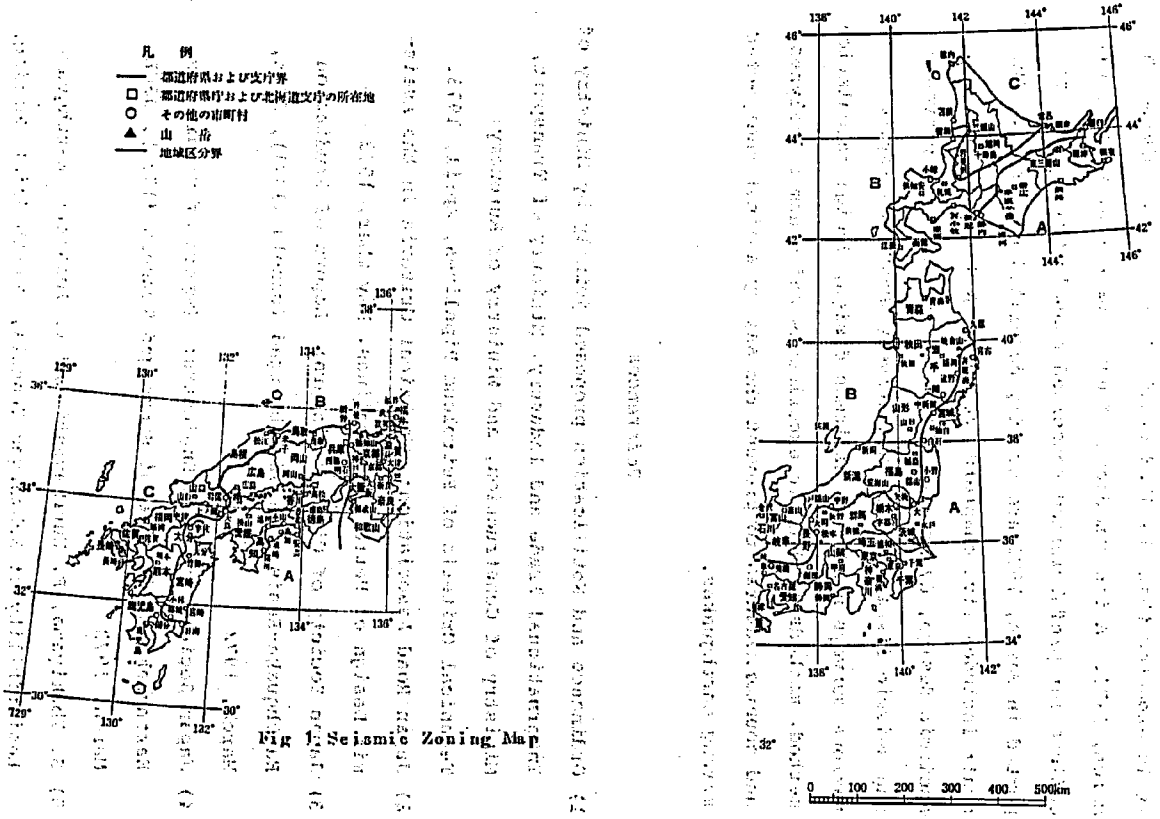


Fig 1 Seismic Zoning Map

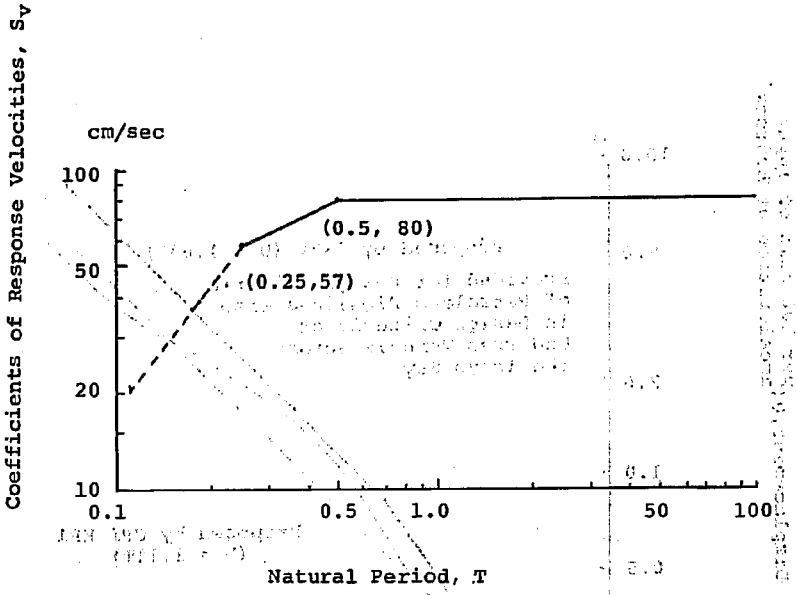


Fig. 2 Coefficient of Response Velocities

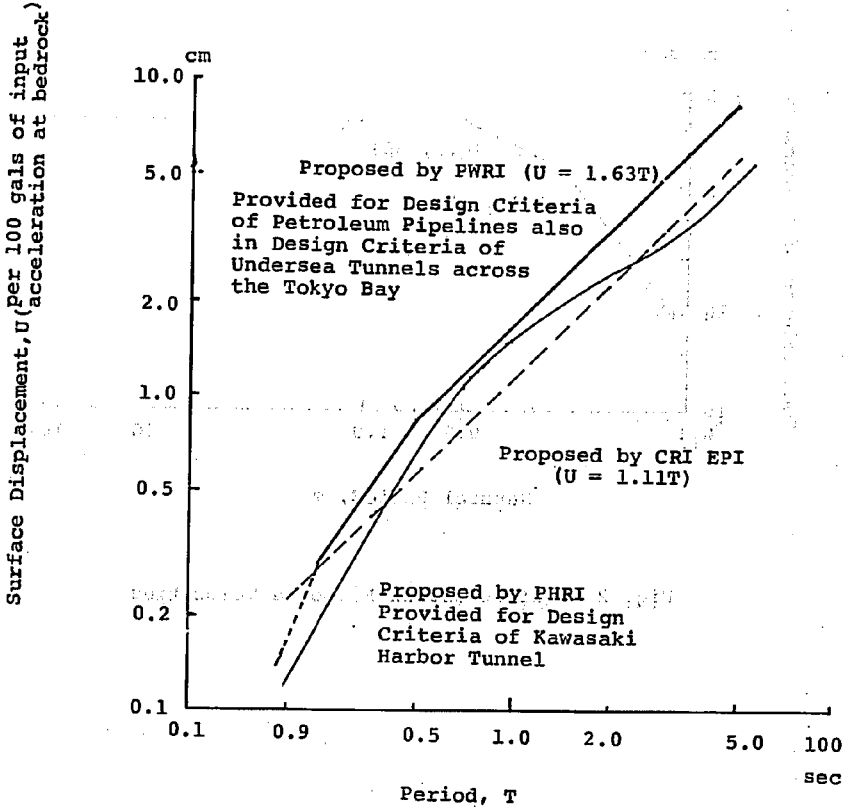


Fig. 3 Relationship between Periods and Surface Displacements

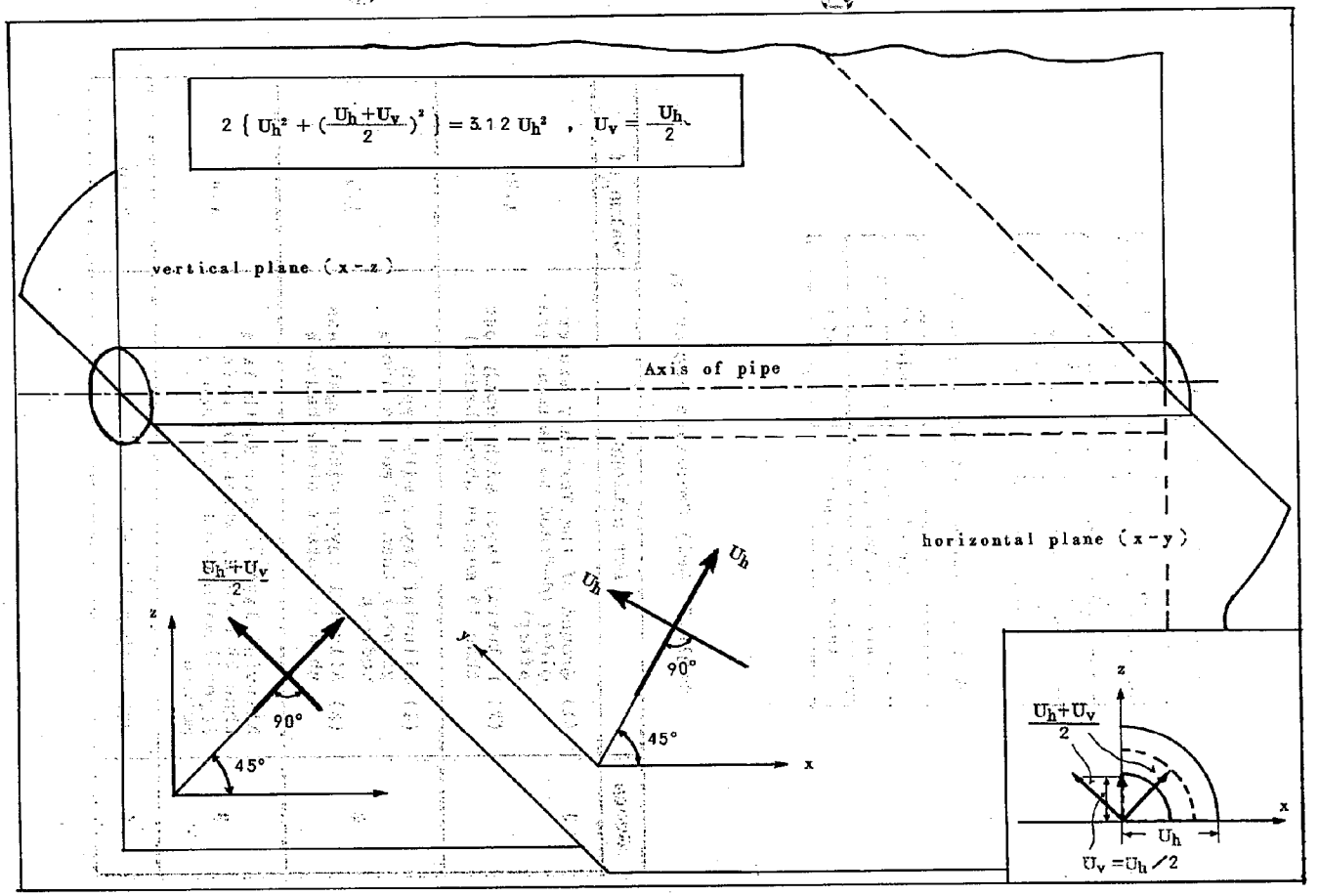


Fig.4 Wave Vectors in Horizontal and Vertical Planes

Table 1 Seismic Zone Factor ν_1

Zone	Value of ν_1
A	1.00
B	0.85
C	0.70

Table 2 Factor on Usage of Land, ν_2

Land	Value of ν_2
Forest or Widerness	0.80
Other than above	1.00

Table 3 Soil Condition Factor ν_3

Group	Definitions	Value of ν_3
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer with depth less than 10 meters above bedrock	1.20
2	(1) Diluvial layer with depth greater than 10 meters above bedrock (2) Alluvial layer with depth less than 10 meters above bedrock	1.33
3	Alluvial layer[3] with depth less than 25 meters, which has soft layer[4] with depth less than 5 meters	1.47
4	Other than the above	1.60

Table 4 Increases in Allowable Stresses

Combination of Loads	Increases in Allowable Stresses (%)
Principal Loads + Wind Loads	25%
Principal Loads + Snow Loads	25%
Principal Loads + Effects of Temperature Change	25%
Principal Loads + Seismic Loads	70%
Principal Loads + Effects of Own Construction Works	80%
Principal Loads + Effects of Construction Works of Other Structures	50%

Research Project on New Criteria for Earthquake Resistant Design for Over-All Structures

Mitsuru Nagao, Director
Public Works Research Institute
Ministry of Construction

Fifth Joint Meeting
U.S.-Japan Panel on Wind and Seismic Effects
UJNR, Tokyo, May 14-16, 1973

1. Five-Years Plan on Development for Construction Technology

It is an important problem to cause the national land to be redeveloped in accordance with various desires of people, so it is largely expected to develop the fundamental technology of construction. In the present situation, however, the ratio of investment on development for construction technology to total investment on construction works has been smaller with compared to that of other industires. Moreover, it has become difficult that the system of the development for construction technology responds to the rapid change of the social situation of Japan because of obstructions: (1) slowdown of the promotion of the development in response to needs of social and administration basis, (2) thin bodys of the system to evaluate and effectively utilize those of results.

Accordingly Five-Years Plan on Development for Construction Technology has been carried out since 1972. The investments for this plan amount to about twenty billion yens (eighty million dollars). This plan was decided to make up for the ratios of annual research expenditure in Ministry of Construction to total investments on construction works or to total investments on promotion of technology developments from the present level to that of ten years ago. In order to promote the development plan, following three methods have been selected.

(1) Establishment of Project for Systematic Development of Technology,

(2) Promotion for Development of Technology in private organizations related and efficient utilization of these results

(3) Promotion of special subjects of the Development

2. Service System of Project for Systematic Development of Technology

For the purpose of inquirying the fundamental direction of the technology development, the way that the selection of important research theme and the system of the technology development in cooperation with the private organization, Board of Consultants on Development for Construction Technology was established as an advisory organization of Ministry of Constructions in September, 1971. This board consists of twenty five of academic and sophisticated members and of five members from Ministry of Construction, and is able to organize the Sub-Committee for the purpose of the research and investigation. As one of the Sub-Committees, Sub-Committee on Earthquake Resistant Resistant Technology (Chairman: Dr. Kiyoshi Muto) has been organized to survey and discuss the fundamental direction of development for earthquake resistant technology as one of the disaster prevention policies.

On the basis of the resolution of this Sub-Committee, Construction Minister makes decision of the fundamental direction concerning the selection of research fields and subjects, management of the system and the financial basis.

Executive Committee on Development for Construction Technology has been capable to promote the research and investigation of three Auxiliary Institutes of Ministry of Construction: Geographical Survey Institute, Public Works Research Institute and Building Institute.

Based on the decision, Bureaus of Ministry of Construction, that is, Minister's Secretariat, Planning Bureau, City Bureau, River Bureau, Road Bureau and Housing Bureau and three Auxiliary Institutions serve the developments of technology under the system shown in figure 1.

3. Project for Development of Earthquake Resistant Technology

As one of the major research objectives in the Project for Systematic Development of Technology, Development of New Criteria of Earthquake Resistant Design has been included. In this project, following six large themes are selected.

(1) Research for Seismic Ground Motions

For establishing the rational earthquake resistant design, it is important to obtain the seismic intensity classified by regions and kinds of ground conditions. Accordingly, seismic activity levels classified by regions, vibration characteristics of bed-rocks, dynamic behaviors of surface layers and finally standard seismic intensities for design are classified.

(2) Research for Dynamic Properties of Soils

Dynamic strength, deformation and characteristics of soil and foundation grounds are researched.

(3) Research for Dynamic Properties and Dynamic Strength of Structural members

Dynamic strength and deformation of structural members and materials are researched.

(4) Research for Vibrational Characteristics and Earthquake Response Analysis of Structures

Dynamic behavior of structures which take account of the inter-action between grounds and structures and earthquake response analysis considering the ground conditions are researched.

(5) Establishment of New Criteria of Earthquake Resitant Design for Structures

New criteria of practical and rational earthquake resistant design based on the concept of dynamic design are developed.

(6) Research for Countermeasures against Earthquake Disasters

This subject consists of two items: (1) development of methodology of evaluation and reinforcement for earthquake resistibilities of existing buildings and structures to preserve much resistible function of large cities in which collapse of principal facilities located in the populated areas has to prevent during earthquakes, (2) research on methodology of planning earthquake disaster prevention.

Under these six research themes, sixty subjects of nineteen sub-themes are prepared. The Expenditure for the research project is estimated as a thousand and eight hundred million yen (seven million dollars) as shown in Table 1. The research have been carried out for five subjects in 1972 and twelve added in 1973. The project will provide new knowledges and criteria near future.

Table 1 Expenditure for Development of New Criteria of Earthquake Resistant Design (Yen)

Theme \ Fiscal Year	1972	1973	1974	1975	1976	Total
Research for Seismic Ground Motions	14,780,000	12,859,000	79,569,000	70,000,000	64,000,000	241,208,000
Research for Dynamic Properties of Soils	3,701,000	5,334,000	43,690,000	62,500,000	33,400,000	148,625,000
Research for Dynamic Properties and Dynamic Strength of Structural Members	0	18,948,000	120,000,000	101,000,000	51,000,000	290,948,000
Research for Vibrational Characteristics and Earthquake Response Analysis of Structure	0	27,561,000	117,000,000	148,000,000	150,000,000	442,561,000
Establishment of New Criteria of Earthquake Resistant Design for Structures	0	3,607,000	249,747,000	165,618,000	124,020,000	542,392,000
Research for Countermeasures against Earthquake Disaster	21,756,000	4,333,000	54,739,000	39,000,000	25,000,000	144,828,000
Total	40,237,000	72,042,000	664,745,000	586,118,000	447,420,000	1,810,562,000

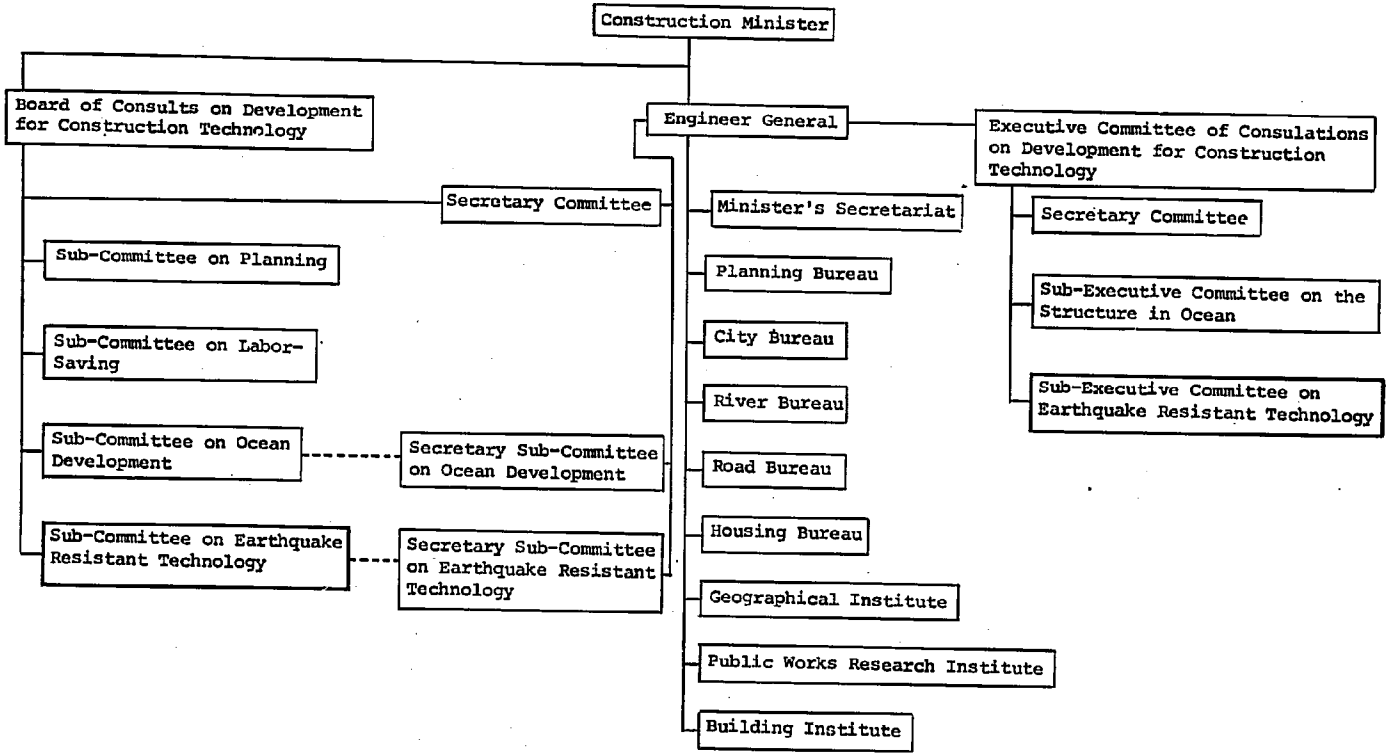


Figure 1. System of Research Project on Developments for Construction Technology

Overseas Technical Cooperation

Preliminary Survey on Seismology and Earthquake Engineering in the Union of Burma

by

Eiichi Kuribayashi

Chief, Earthquake Engineering Research Section,

Public Works Research Institute

Ministry of Construction

Fifth Joint Meeting

U.S.-Japan Panel on Wind and Seismic Effects

UJNR, Tokyo, May 14-16, 1973

1. INTRODUCTION

Objective of the Mission

An earthquake shook the area of Rangoon, the Capital of the Union of Burma in September 9, 1970, causing damage to the golden rod at the top part of the most respected pagoda of the Shwe Dagon and to several buildings including the State House of His Excellency Mr. Ne Win, the Chairman of the Revolutional Council of the Union of Burma.

Impressed with this earthquake and weighed with several other earthquakes of the slighter intensity which were also felt in the same area successively, the Government of the Union of Burma extended a request to the Government of Japan for dispatch of a mission composed of well qualified experts on seismology and earthquake engineering so that the scientific and technical level of Burma in the field of seismology and earthquake engineering could be improved quite urgently.

According to the request, the Burmese Government inquired dispatch of experts from Japan who can serve for longer period, say, one-two years, by which such materialized fruits as drafting of earthquake resistant design code and regulations for building and civil engineering structures, presenting guide line for building constructions and seismic zoning map of Burma, etc.

Careful and friendly examination on the request raised by the Government of Burma was made by the Japanese

authorities concerned which made it clear that, currently, they have too little information on the present status on seismology and earthquake engineering in Burma to send a full scale team of experts in the above fields. It came out consequently to send firstly a preliminary survey mission of short term for carrying out thorough investigation on the present status of Burma in these fields as a part of the long term cooperation of Japan which could be extended to Burma most effectively.

Upon the hearty acceptance of the Government of Burma on the above view of Japanese authorities it was agreed upon to send firstly a preliminary survey mission composed by four experts in the short period of two weeks.

The main objectives of the mission were (1) to investigate the present status of the seismological observation network in Burma and to find the way for improving the station, network and seismology, (2) to investigate the present status on the design and construction works on houses, buildings and civil engineering structures then to give pertinent advices for establishing code and regulations for earthquake resistant design, (3) to find the way of raising urgently the level of the field engineering in these field, (4) to give suggestions for the training of young researchers and engineers, (5) to enforce the curricula in the universities and institutions in the field of seismology and earthquake engineering.

Members of the Mission

The composition of the Japanese Mission to Burma on Seismology and Earthquake Engineering under Colombo Plan for Technical Cooperation in South and Souty-East Asia was as follows:

Chief	Syun'ichiro OMOTE,	International Institute of Seismology and Earthquake Engineering, Building Research Institute	General
Member	Eiichi KURIBAYASHI,	Public Works Research Institute, Ministry of Construction	Civil Engineering structures
Member	Masaji ICHIKAWA,	Meteorological Research Institute, Japan Meteorological Agency	Seismology
Member	Tsuneo OKADA,	Institute of Industrial Science, University of Tokyo	Building construction engineering

1. Period of the Survey in Burma

Eighteen days: from January 13 to 30, 1971

2. Summary of Survey of the Mission

Problems on seismology and earthquake engineering in Burma were first investigated in 1960, when a UNESCO Seismological Survey Mission of 5 members headed by Prof. V. V.

Beloussov, of USSR visited this country. The composition of the Mission was:

Prof. V. V. Beloussov, Head, USSR, President of International Union of Geodesy and Geophysics

Dr. K. Takeyama, Japan, Director, Building Research Institute, Ministry of Construction

Dr. G. A. Eiby, Newzealand, Research Staff, Seismological Observatory, Wellington

Dr E. V. Karous, USSR, Institute of Physics of the Earth, Moscow

Rer. Daniel Linehan S. J., USA, Weston Seismological Ovservatory, Weston, Mass.

Though it was only four days in which the UNESCO Mission stayed in Burma, the Mission carried out thorough investigations on the situation of seismology and earthquake engineering in Burma, compiling the result of their survey in a form of a report entitled "UNESCO Seismological Survey Missions, Part 1, Report of the Mission to South-East Asia; Monograph No. 15, January, 1962, International Union of Geodesy and Geophysics, UNESCO." In the report, based upon the analysis of their inspections, the Mission gave quite pertinent recommendations for improving the situations on seismology and earthquake engineering in Burma and suggesting the future plan to be carried out by the local people under the international cooperations.

At that moment, the building of the seismological observatory of Rangoon was under construction and no

seismological observation was in operation in Burma. At this present time, however, when our Mission visited Burma, it was observed a conspicuous improvement in the situation on seismology and earthquake engineering. For example, in the field of seismology there were existing two seismological stations in Rangoon and Mandalay equipped with seismographs in each of them, though the maintenance of these seismographs were problematic when the earthquake of Sept. 9, 1970 occurred. In the field of earthquake engineering, the Government of the Union of Burma had sent two trainees abroad, one to Bergamo Earthquake Engineering Center in Italy, in 1962, and the other to the International Institute of Seismology and Earthquake Engineering, Tokyo, in Japan in 1963, by which the Government expressed her definite desire for improving the situation in the field of earthquake engineering in Burma.

If it should be allowed to use the most straight forward expression, however, we cannot help recognizing that the speed of progress in this decade, in the field of seismology and earthquake engineering in Burma was excessively slow as compared with the wonderfully rapid progress carried out in many other countries in the same period.

The mission, however, found it the most delightful and took it the best fruits of the present visit to Burma that the definite desire was expressed by the people in the posts of high authority as well as responsible personnel in the Government and by the practical engineers and administration officers engaging in the field works.

In the case of the present Mission to Burma, though it was just only a short visit of about two weeks, making full use of the time, we visited as many organizations and stations as possible to meet almost all key persons who were carrying on important activity in the respective work, by which we were able to be provided quite a complete information to realize the current status of Burma in the field of seismology and earthquake engineering.

Above all it was extremely valuable for us that we could exchange mutual views and opinions on all the important items with the top level persons in the Government in an atmoaphere of most openhearted and friendly. Through such discussions and talks we could make them clear, in the field of seismology and earthquake engineering.

The following Section gives the recommendations on such items as (1) Establishing code and regulation for earthquake resistant design, (2) sending trainees on seismology and earthquake engineering abroad, (3) inviting foreign experts by which the increase of the number of trained experts and engineers in Burma could be done, (4) bringing up of the higher level field engineers on the specified construction project, (5) establishing new seismological observation stations in order to provide basic data for minimizing earthquake desasters, (6) enforcement of the curricula in the universities on seismology and earthquake engineering.

All the member of the Mission are prepared to continue their cooperation to Burma as much as they can but at the same time it is our sincere desire that upon receiving the

report of the Mission to Burma in the field of seismology and earthquake engineering, the Government of Japan will commence, from a higher view point, for extending most effective technical cooperations to Burma in this field through the practical channel of the Overseas Technical Cooperation Agency of Japan in the close contact with the diplomatic offices of Japan in Burma.

3. Recommendations

Restricting to the problems on seismology and earthquake engineering as mentioned above, after conducting as much investigation on the spots as possible and exchanging views and opinions with the local authorities and personnel as most thoroughly and friendly as possible and finally, after finishing necessary arrangement with the Government of the Union of Burma and the diplomatic offices of Japan in Rangoon, the mission presented the following recommendations to the Government of Japan and to the Government of the Union of Burma through the Overseas Technical Cooperation Agency of Japan.

Recommendations:

1) Advice for the establishment of a draft on earthquake resistant design regulation for structures and on design code of practice for specific structures with different materials

Taking into considerations on the general technical level of the construction engineering in Burma, presently,

it seems too difficult in Burma to build up a complete and well balanced earthquake resistant design regulation and design code of practice within a very short time. The establishment of such a code should be started with the accumulation of the specifications of respective construction project in Burma, then in the process of such an accumulation, good efforts will be born out for improving such specifications into more harmonized form. In such a way, after several years, it will become practicable for drafting somewhat a more simple design manual for different structures and for making out a design code of practice with respect to respective construction. Finally earthquake resistant design regulations should be included in the design code of practice.

2) Overseas training of experts in the field of seismology and earthquake engineering

Needless to say one of the most effective and rapid way for building up the higher level experts is to send ingenious persons abroad in a definite period of time to be trained at the most pertinent institutions or schools etc. in the advanced countries. Very careful examination will be most important for the selection of training institutions abroad as well as the selection of the individuals to be sent to foreign countries. There are several famous training institutions providing facilities for the trainees from other countries such as Earthquake Engineering Research Center, UC, Berkeley, USA,

Earthquake Engineering Center, Bergano, Italy, and International Institute of Seismology and Earthquake Engineering (IISEE), Tokyo. It is greatly advantageous to send trainees to such training facilities following the schedule prepared under the long term planning of the Government.

The IISEE, Tokyo, is providing a course of full one year under the well organized curriculum both for seismology and earthquake engineering fields, in a very high international reputation. The participants of the IISEE are supported by the fellowship including tuitions, living and flight expences which are sponsored by either the Government of Japan or UNESCO. It is recommended that the Government of Burma will send at least two trainees to the IISEE every year, one to the seismology course and the other to earthquake engineering course.

3) Inviting foreign experts to Burma for the training of engineers

In order to improve the general technical level of engineers in Burma it will provide an effective way of inviting suitable experts in that field from abroad asking them to give lectures to the young engineers in the Government organizations or to guide the way of applying new method of design and construction for some more qualified persons or to give practical training.

4) Advice for the construction practices on the specified construction project

In the case of a big construction project, to which high level Japanese experts should be invited, it could be so arranged as to work a group of well qualified young engineers in the Burmese Government or other semi-official or private organizations, under the Japanese experts throughout the whole period of construction project, by which local young engineers will be able to absorb completely all technics, knowledges, and at the same time all the experiences in that field. It is only through this way of contact and guidance that young engineers could be trained and brought up into a real technical expert in specified construction project.

It will be some merit to attend a class to receive lectures on a training course but the most effective and shortest way of bringing up a practically useful expert will be attained only through the direct absorption of the experiences of foreign experts by the young local engineers when the foreign expert is enthusiastically willing and actively devotioning to educate young Burmese engineers.

5) Urgent setting up of an effective seismological observation network and promotion of the development of seismology in Burma

It is strongly recommended to establish six new seismological stations along with the definite schedule upon year marked programme so that together with the two existing stations, 8-station network can be expected

to run into operation within a very near future.

It should be emphasized that successful completion of the new 8-station network system could only be attained with the setting up of the new attractive posts in the organization of the Meteorological Department of the Government of Burma.

It is also recommended here that in case when a destructive earthquake should take place somewhere in Burma, the Government should organize an emergency investigation team and should send it to the site in order to investigationed record all the data including causalties, damage statistics, destruction of houses and buildings, etc. Upon such a site investigation, seismic intensity map will be drawn up and detailed descriptions on the destructions caused in each individual building or house will be retained in records. Such a report of the site investigations will be greately valuable for the future development of seismology and earthquake engineering in Burma.

It is also recommended at least two sets of Strong Motion Seismographs should be settled in Burma, one in Rangoon and the other in Mandalay.

6) Establishing the new curriculum on seismology and earthquake engineering universities

The most foundamental way for improving the situation of one specific field in one country should be found in the education of students at the universities.

Presently there is no curriculum giving lectures on earthquake engineering or seismology at any university in Burma.

It is quite important, seeing from the long range programme, to create chairs on earthquake engineering in the department of construction engineering and on seismology in the department of geology or physics or elsewhere. However, as a first step it is advised that the lectures on earthquake engineering and/or seismology should be newly added in the curriculum of engineering and/or science as soon as possible. For this purpose it might be advisable for the very beginning years experts from foreign countries would be invited for giving co-operations and after several years main part of the lecture should be given by local professor trained in some way or other.

In some case such lectures in the University will be opened also for the young engineers in the Governmental and other organizations having good interest in that field for some limited period. Such lectures will be greatly valuable not only for broadening and deeping the knowledge level of engineers but also for calling upon the special interest in the higher level construction engineers and administration officers.

In order to bring all these items as mentioned above into realization, it is needless to say but should be strongly

requested that the Government of the Union of Burma should wrestle with these important problems with enthusiasm, ensuring the financial necessities for all facilities and equipment, electing new posts for the effective and powerful running of these new projects, encouraging and guiding the official personnel for devoting themselves to their respective sublime job.

It was the highest pleasure for the Mission to learn that the Mission could find through the present visit that the responsible persons in the Government were well aware of this situation and completely were determined to make their best effort along this line. However it should also be necssary to mention that these targets, herewith itemized are too large to be attained by the only effort of the Government of Burma herself but also be called upon the cooperation from outside. The Mission feels happily to express here that the Government of Japan is deeply solicitous so much in the rapid progress of the seismology and earthquake engineering in Burma that the Government will be pleased to extend its maximum cooperation in supplying equipment and materials and in sending experts to Burma upon the requests raised from the Government of the Union of Burma, through the channel of the Overseas Technical Cooperation Agency of Japan in so much as practicable.

A M E N D M E N T S

- A Draft of Earthquake Resistant Design Regulations -

A mission of five members headed by V. Tha Nyunt dispatched from the Government of the Union of Burma visited Japan in September 1972. One of the purposes of the mission was of framing a draft of Earthquake Resistant Design regulations proposed to the Government of the Union of Burma.

In cooperation with the mission and Oversea Technical Cooperation Agency, ex-members of the Japanese mission worked for drafting the Code shown as follows:

Earthquake Resistant Design Regulations (Draft)

1. Design Seismic Coefficient

1.1 The horizontal design seismic coefficient shall be determined by the following formula:

$$k_h = \nu_1 . \nu_2 . \nu_3 . k_0$$

where

k_h : Horizontal design seismic coefficient,

k_o : The standard horizontal design seismic coefficient (= 0.1),

ν_1 : Seismic zone factor,

ν_2 : Ground condition factor, and

ν_3 : Importance factor.

1.2 The vertical design seismic coefficient, k_v may generally be considered as zero.

2. Factors for Modifying the Standard Horizontal Design Seis-
Coefficient

2.1 Seismic Zone Factor

Seismic zone factor shall be determined in accordance with Table 1, in which the zone classification shall be determined from Fig. 1.

2.2 Ground Condition Factor

Ground condition factor shall be determined in accordance with Table 2.

2.3 Importance Factor

Importance factor shall be determined in accordance with Table 3.

3. Design Loads in Earthquake Resistant Design

The following loads shall be taken into account in earthquake resistant design. The appropriate loads shall be selected from this list on the basis of the location and the type of the structure.

(1) Dead Loads

(2) Earth Pressures

(2) Hydraulic Pressures

(4) Seismic Effects

(6) Other Loads

4. Allowable Stress and Safety Factor

4.1 Working stresses in each part of structural members subject to the loads specified in Article 3 shall not exceed the allowable stress specified below:

Allowable stress: Yielding stresses of materials or two third of breaking stresses of materials.

4.2 Axial compressive forces of structural members subject to the loads specified in Article 3 shall not exceed two third of bukling loads of the members.

4.3 Intensities of reactions in foundation ground subject to the loads specified in Article 3 shall not exceed two third of ultimate bearing capacities.

Table 1 Seismic Zone Factor ν_1

Zone	Value of ν_1
A	1.0
B	0.7

Table 2 Ground Condition Factor ν_3

Group	Definitions[1]	Value of ν_3
1	(1) Ground of the Tertiary era or older (defined as bedrock hereafter) (2) Diluvial layer[2] with depth less than 10 meters above bedrock	0.9
2	(1) Diluvial layer[2] with depth greater than 10 meters above bedrock (2) Alluvial layer[3] with depth less than 10 meters above bedrock	1.0
3	Alluvial layer[3] with depth less than 25 meters, which has soft layer[4] with d depth less than 5 meters	1.1
4	Other than the above	1.2

(Notes) 1) Since these definitions are not very comprehensive, the classification of ground conditions shall be made with adequate consideration of the bridge site.
Depth of layer indicated here shall be measured from the actual ground surface.

2) Diluvial layer implies a dense alluvial layer such as a dense sandy layer, gravel layer, or cobble layer.

3) Alluvial layer implies a new sedimentary layer made by a landslide.

4) Soft layer is defined in Section 3.7 "Soil Layer Whose Bearing Capacities are Neglected in Earthquake Resistant Design."

Table 3 Importance Factor ν_3

		Value of ν_3
Public buildings		1.5
Other buildings	Reinforced concrete	1.0
	Brick	0.5
	Wood	0
Dams		1.5
Bridges		1.0
Harbour structures		1.0

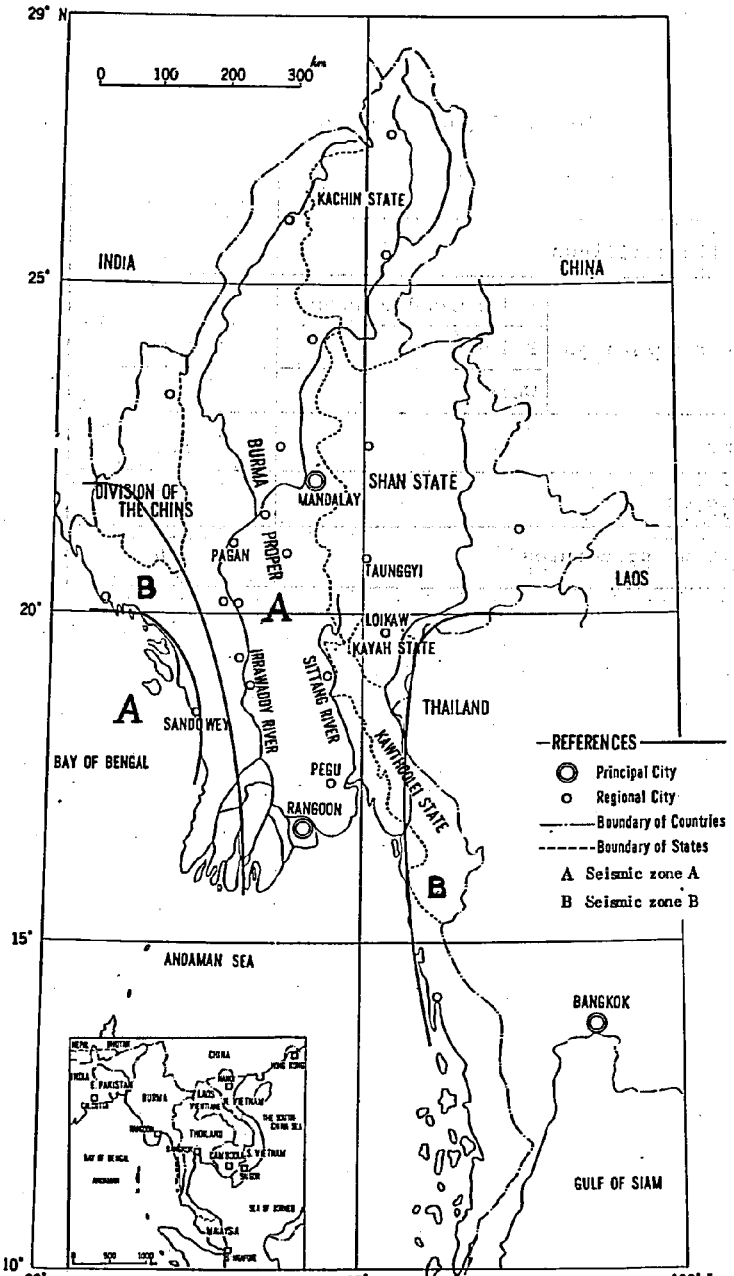


Fig 1. Seismic Zoning Map

A P P E N D I X

- Earthquakes in Burma -

Fig. A Seismicity of the Earth, 1961-1967, ESSA, CGS Epicenters, Depth 0-100 km.

Fig. B Seismicity of the Earth, 1961-1967, ESSA, CGS Epicenters, Depth 100-700 km.

Fig. C Epicentre distribution in Burma (After U Sein Shwe U)

Fig. D Four zones in Burma

Fig. E Distribution of Seismic Intensities and Isoseismals for the 1970 Rangoon Earthquake

Fig. F Commulative Frequency (N) of Total Duration of an Earthquake (F-B) Recorded by Willmore Seismograph

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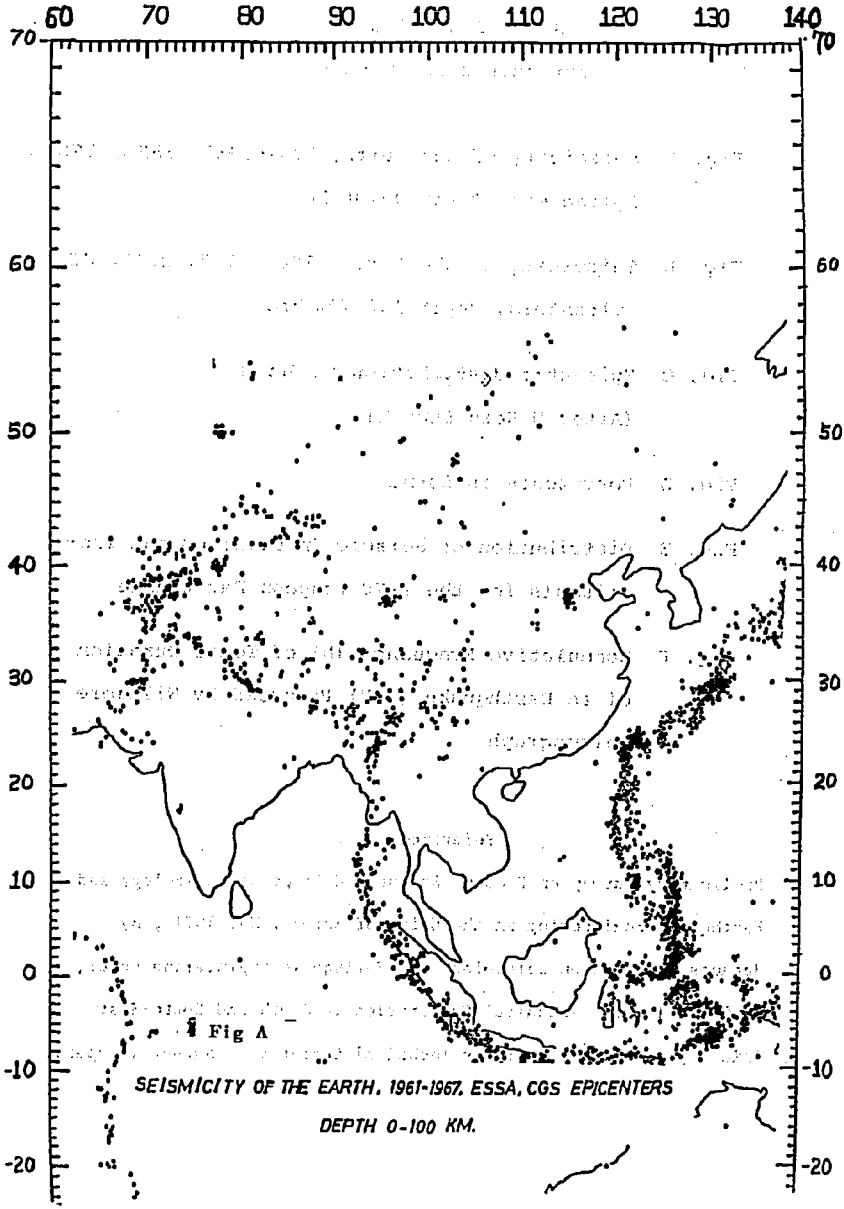


Fig A

SEISMICITY OF THE EARTH, 1961-1967, ESSA, CGS EPICENTERS

DEPTH 0-100 KM.

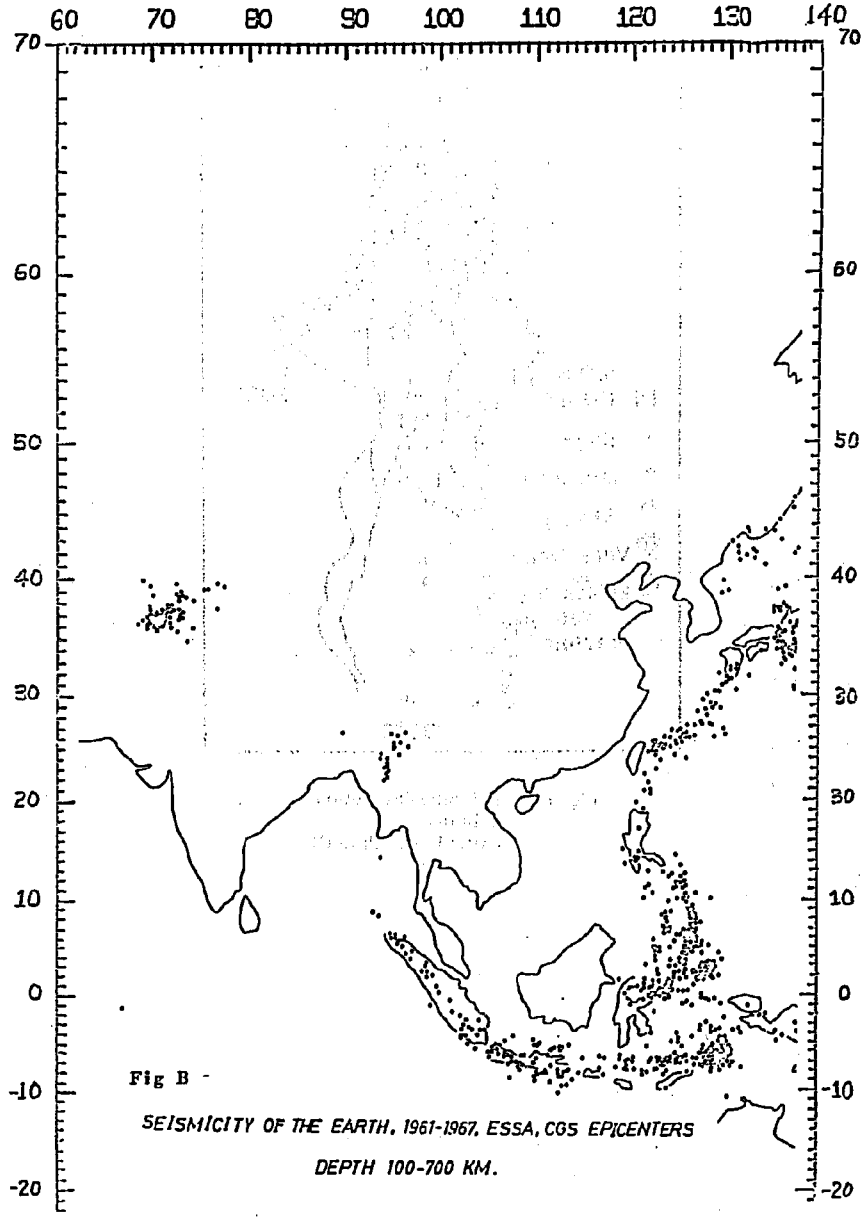


Fig B -

SEISMICITY OF THE EARTH, 1961-1967, ESSA, CGS EPICENTERS

DEPTH 100-700 KM.

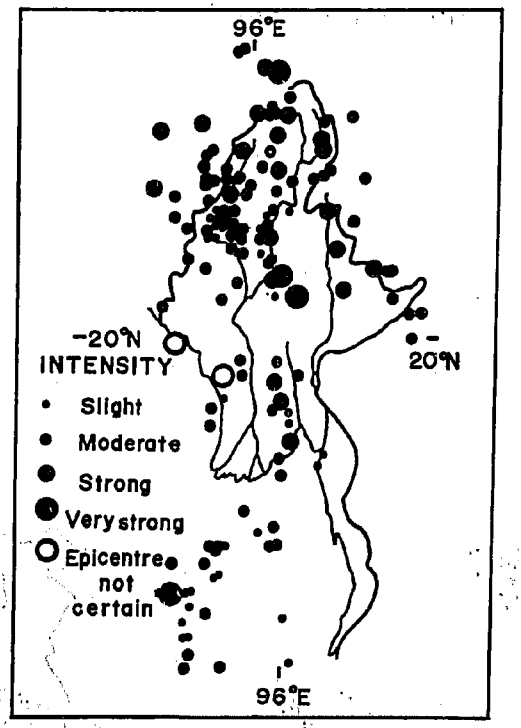


Fig. C Epicentre distribution in Burma (After U Sein Shwe U)

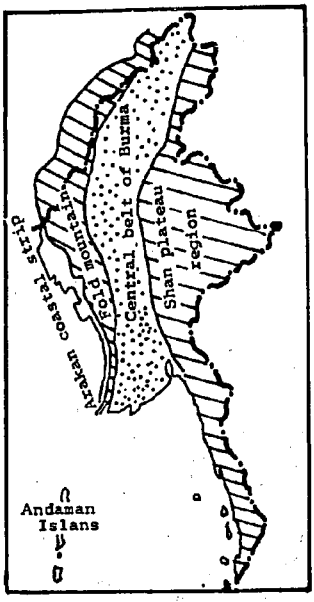


Fig. D Four zones in Burma

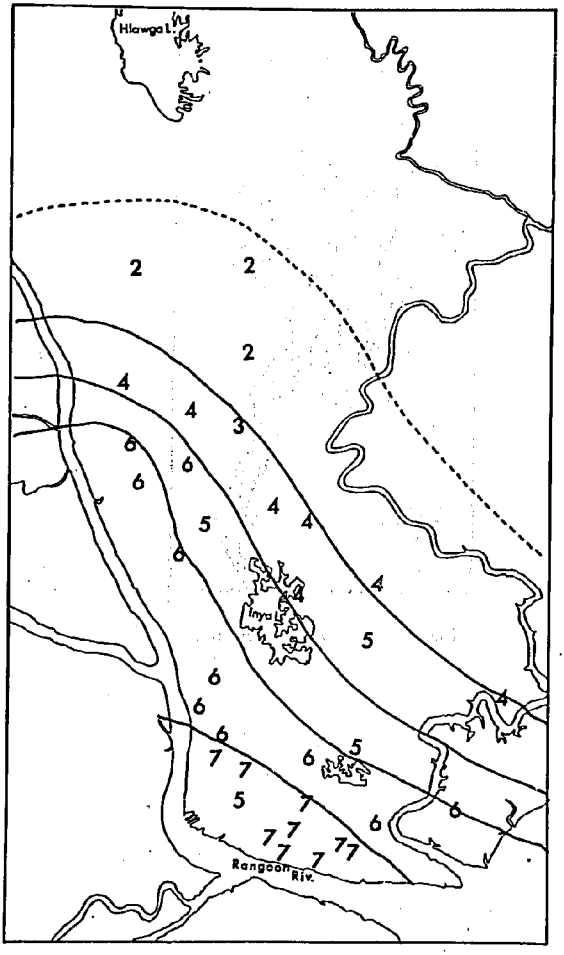


Fig. E Distribution of Seismic Intensities and Isoseismals for the 1970 Rangoon Earthquake

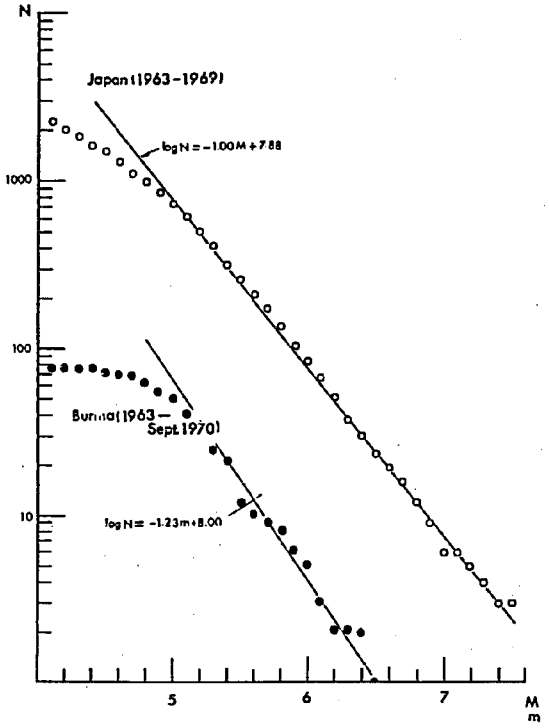


Fig. F Commulative Frequency (N) of Total Duration of an Earthquake (F–B) Recorded by Willmore Seismograph.

PRELIMINARY REPORT ON DAMAGE OF MANAGUA EARTHQUAKE, Dec. 23, 1972

Submitted to the Fifth Joint Meeting

of Japan and U.S. Panels

on Wind and Earthquake Resistant Structures

May 14, 1973

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1. Introduction

At 00:29 on December 23rd, 1972, Managua City, the capital of Nicaragua, was subjected to a strong earthquake. The central area of the City suffered destructive damages and numbers of deaths and injuries amounted to about 10,000 and 20,000 persons, respectively.

For surveying the aspects of damage of urban area caused by shallow earthquake just beneath the city and for finding the way of engineering cooperation with Nicaraguan people, the Japanese Government sent a survey team on damages of Managua Earthquake 1972, consisted of five members including a seismologist and a fire chief other than the authors from March 15 to April 1st 1973.

The survey team stayed in San Francisco on March 15 and 16, and carried out preliminary survey at Earthquake Research Center of U.S Geological Survey in Menlopark, Seismological Field Survey NOAA, Department of Civil Engineering of the University of California in Berkeley and other organizations, then surveyed to actual earthquake damage in Managua from March 17 to March 29.

In Managua, the survey team had contacts with both of concerned governmental and private sectors and collected valuable data. However, because numbers of the team and period of stay were limited, the survey was carried out on a part of damage.

One of the important harvest of the survey was that we obtained records of strong motion accelerograph in Managua. Using the strong motion record, the authors are now analyzing structural damages, so they present here a preliminary report introducing the outline of earthquake damages.

2. Brief Description of Managua Earthquake in 1972 and its Damages

2.1 Description of Earthquake

Origin time (GMT) 06:29:42, December 23rd, 1972

" (Local time) 00:29:42, December 23rd, 1972

Epicenter 12.4° N 86.1° W

Depth of hypocenter about 5 km

Magnitude 6 1/4 in Richter scale

The location of epicenter mentioned above is announced on the basis of instrumental estimation, but the damage pattern in Managua and locations of epicenters of aftershocks observed by Peter L. Ward et al of USGS suggest that the epicenter was actually located in the Lake of Managua near the Managua City, around 12°10' N and 86°15' W.

2.2 Acceleration of Earthquake Ground Motion and Isoseismal

At the time of earthquake, three units of strong motion accelerograph were installed in Managua but only one located at ESSO Oil Refinery recorded the earthquake ground motion. According to the record, maximum accelerations were 0.39 g, 0.34 g and 0.33 g in the east-west, north-south and up-down direction, respectively.[1)]

Taking into account the fact that the epicentral distances of center of Managua and ESSO Oil Refinery are about 3 km and 8 km (the focus distances are about 6 km and 9.5 km, respectively) based on the location of epicenter suggested in paragraph 2.1., together with the value of transmission function at both sites; the maximum acceleration at the city center is estimated for beyond the recorded maximum at ESSO.

The authors are now analysing the degree of damage by air-photos taken before and 10 hours after the earthquake for getting the isoseismal map and they consider that the isoseismal map shown by T. Matsumoto et al.[2] gives a general tendency. (Fig.-1)

2.3 Outline of Damage

The outline of damage was as follows:

Dead	8,000 ~ 10,000
Injured	about 20,000
Refugees	220,000 ~ 250,000
Affected area	27 km^2 with totally destroyed area of 13 km^2 and damaged area of 14 km^2
Damaged House	53,000 units
Shops and Factories	95% of small shops and factories and 11 large factories lost or seriously damaged.
Commercial buildings and warehouses	400,000 m^2 destroyed
Public and private offices	340,000 m^2 destroyed
Hospital	4 hospitals with 1,650 beds lost or seriously damaged.
School	740 classrooms lost or seriously damaged
Total loss	More than $1 billion.

2.4 Some Features of Earthquake

The Managua Earthquake 1972 has following features.

(i) There were foreshocks of at least 4 times since 2 hours before the main shock.

(ii) There were symptoms of earthquake.

(iii) Faults were found on the ground surface.

(iv) Damages were severe compared with the magnitude of earthquake because the hypocenter was shallow and was located in the vicinity of urban area.

At least, three foreshocks of intensities V to VII in Modified Mercalli scale with sounds took place at about 22:30 of December 22 and one shock of intensity V at 23:10 of the same day. It was said that many people frightened by these shocks went out of doors and stayed outside, resulted in the decrease of casualty by the main shock.

Several tens minutes after the main shock, two comparatively strong aftershocks of magnitudes 5.0 and 5.2 successively took place and destroyed buildings damaged by the main shock but still standing.

As symptomatic phenomena, it was reported that small animals showed unordinary behavior since 10 days before the earthquake, and that two days before the earthquake steam arose from a part of Lake Tiscapa. Also, several people reported that they saw light in the direction of epicenter at the time of earthquake.

According to our survey, seven faults caused by the earthquake were found in the urban area of Managua and the location of them are shown in Figure-1. The faults were left lateral faults with some amount of vertical displacement and the maximum relative displacement along fault line was 38 cm. In the vicinity of these faults, buildings suffered more severe damages than other area and taquezal buildings suffered destructive damage.

3. Faults and Ground Failure

3.1 Faults

Seven faults shown in Figure-1 mainly caused the ground failures by the earthquake. These faults are left-lateral and strike approximately N20°E to N40°E, more eastly in the northern part. The movement of Fault C was largest and maximum horizontal and vertical relative displacement were 38 cm and 6 cm, respectively. Fault C has already existed before the earthquake and is noted in the geological map of 1/250,000 published by Catastro E Inventario de Recursos Naturales.

Remarkable ground failures are caused by Fault C (thermal power plant, Lake Tiscapa), Fault D (customs office building, south of Lake Tiscapa), Fault E (American College), Fault B (Banco Central de Nicaragua) and Fault A (Somoza Stadium, Western river). Among these five faults, the location of Fault A approximately coincides with that of a fault reported by D.E. Sultan[4] at the occasion of the earthquake in 1931.

Damages of Thermal Power Plant, Customs Office Building and U.S. Embassy Building can be considered as examples of building damage caused by fault movement. Beside these, it can be observed by air photos that taquezal houses are more severely collapsed along faults. Other than above, road cracks, slope failures, breaking of water supply pipes, collapse of retaining walls were also severe along the faults.

3.2 Liquefaction of Sandy Ground and Settlement of Ground

No evidence of liquefaction of sandy ground was found and the followings may by the reason:

a) The sandy ground mainly consists of sand with gravel and sand with hard silt and the grain size is well distributed beyond the liquefaction limits.

b) There was a long drought before the earthquake resulted in low ground water surface.

c) In most part of Managua City, N-value at the ground surface exceeds 10 and that at the depth of 4 to 5 meters exceeds 50 and there were cemented strata containing lava.

Settlements of ground are found in filled-back soil and reclamated land but had caused no structural damage except oil tanks.

3.3 Land Slide and Slope Failure

Land slide and slope failure were found slopes inside of crater and steep slope near mountains. Slope failures along road surrounding Lake Tiscapa, at the entrance of Army Hospital and at the site of U.S. Embassy took place at steep natural slope of 60° to 75° surface dip and caused damages of retaining wall along road, cracking of pavement and collapse of Hospital gate and U.S. Embassy building.

Hillside failures are also found on the slope inside of crater of Lake Asososca. The slope is steeper than 70° and damage of surrounding road, intake facilities and their approach road and of buildings at cliff top was caused by hillside failure.

A big scale slope failure and sliding of filled soil are found on Pan-American Highway, passing through the hillside of Lake Nejapa.

At this site, one side of highway consists of untreated cut slope of 6 meters high and 45° steep and the other side consists of filled bank overlying on natural slope of 50 meters high and 50° steep and covered by vegetation. Failures of cut slope and rip of filled soil took place in the distance of 1 km.

Beside above, failures of natural slopes along river had taken place at many places.

In spite of wide occurrence of slope failures described above, these slope failures did not related to the disastrous damages including loss of lives. The reason can be attributed to the facts that 1. The ground soil mainly consisted of cemented silt or gravel formations and was stable, 2. ground water level was low by the long drought preceeding the earthquake, 3. the slope failures took place only during and immediately after the earthquake and had no progressive or successive failure after the quake and 4. the wide surplus spaces were provided at both sides of highways.

4. Outline of Building Damage

4.1 Damage of Residential Houses

It is said that about 54,000 houses suffered severe damages by the earthquake and about 70 percent of these houses were taquezal construction. In the taquezal houses, the roof system is supported on vertical wooden posts placed at the openings of wall and all corners and between the vertical posts wooden studs are located at the spacing of about 60 centimeters and wooden laths are nailed on both sides of the

posts and the studding. The shelves thus formed by the studs and the laths are filled with stones or balls of mud and both sides of the wall are coated by lime plaster.

According to the survey report on the damage of 1931 Managua Earthquake, the taquezal buildings occupied about 85 percent of buildings in the city and the newer taquezal buildings suffered minor damages such as the shaking off of roof tiles and wall plaster in comparison with that most of the above buildings suffered severe damages including entire collapse. As the reasons that the taquezal buildings suffered severe damages by 1972 earthquake, it is considered that the wooden posts enclosed in the thick wall were eated by termites or rotted by high humidity and temperature.

When our survey team arrived at the city, damaged taquezal buildings were already demolished and the team could not survey and inspect the aspects of damage. But, the team found that taquezal buildings were consisted of stress members of light weight and many of stress members stood after the earthquake without remarkable horizontal displacement.

4.2 Damages of Reinforced Concrete Buildings

Before the Managua Earthquake of 1972, there was building regulations concerning the city planning but no earthquake resistant building code and regulations in Nicaragua and the safety of building against earthquake completely depended on the judgment and choice of designer. Therefore, various types of reinforced concrete buildings existed in the City of Managua and the behavior of these buildings were tested by the earthquake.

The characteristic aspects are shown in the damage of reinforced concrete buildings depending on their structural types and sizes as follows:

(1) Moment resisting frame structure with shear wall

The lower part of Lang Building, Telecommunication Building and National Palace are examples of buildings of moment resisting frame structure with sufficient shear wall. All of these buildings are 2 to 3 stories building and the fundamental natural periods of them are estimated about 0.2 seconds. Figure-2 shows the acceleration response spectra calculated from the strong motion accelerograms recorded at ESSO Oil Refinery Plant, which has a sharp peak at about 0.2 seconds natural period and the amplification factor in the assumed damping ratio of 5% is about 2. Accordingly, it is considered that, during the earthquake, this type of buildings were subjected to the comparatively strong acceleration. In fact, all the shear walls suffered extensive shear cracking. However, because the rigidity of building was decreased by cracking of shear walls, moment resisting frames suffered minor damages and also, by the confinement effects of deformation by shear walls, damages of secondary members, interior coating and equipments were slight, so after repairing shear walls, these buildings can again be used.

(2) Moment resisting frame structure

The typical examples of high rise building of moment resisting frame structure are the Telecol Building (floor area of about 360 m^2,

7 stories), National Insurance Company Building (floor area of about 400 m^2, 5 stories) and the main building of INSS Building (floor area of about 460 m^2, 8 stories). In these buildings, damages of structural members were comparatively slight but damages of partition walls made of concrete blocks and hollow ceramic blocks, interior construction such as basewall and equipments were destructive and the buildings lost their functions.

(3) Moment resisting frame structure with core wall

The typical examples of this type of building are the main building of Supreme Court Building (floor area of 645 m^2, 5 stories), the Banco de America Building (17 stories) and the Banco Central de Nicaragua (15 stories). In these buildings, structural members suffered comparatively slight damage but secondary members such as partition wall and basewall, interior construction and equipments suffered severe damages. In the case of Supreme Court Building, the low rise building surrounding the main building suffered more severe damages. In the case of Banco de America Building, the beams connecting each independent core walls suffered local damage, because these beams had rectangular holes provided for piping ducts. In the case of Banco Central de Nicaragua Building, the core walls located eccentrically and connecting portions between core walls and moment resisting frame structure suffered severe damages together with the severe damage of secondary members, interior construction, equipments and penthouse on the roof.

(4) Buildings with long dimension

Many of school and hospital buildings were of 10 to 20 meters wide, 50 to 70 meters long and 3 to 4 stories high. Most of these buildings suffered severe damage of structural members and collapsed. As a structural type, these buildings belong to the moment resisting frame structure but they have ample of partition walls made of concrete block or brick. Because the flexural deformation of these buildings was restricted by the above mentioned secondary members, the rigidity of columns became various and stresses caused by the earthquake apted to locally concentrate. On the other hand, the distance between columns in these buildings was so large that when one column lost the bearing capacity to vertical force applied to it by the shear failure, then failures of other columns successively took place until the total collapse of building. Also, in many of the collapsed buildings, the floor system was consisted of concrete or ceramic block masonry work spanned between the stringers of precast or poured-in-place concrete, thus the floor system was easy to totally collapse by the local failure of members supporting the floor. From the viewpoint of earthquake engineering it can be concluded that the floor system of this type has a poor resistivity against earthquake.

Summarizing the damage of reinforced concrete buildings, it is concluded that buildings severely damaged by the 1972 Managua Earthquake had following problems in designing of them:

a. Mixed use of different structural types in the same building.

b. Abrupt change of rigidity of building in the vertical direction.

c. Adoption of insufficient floor system.

5. Damage of Bridges and Other Structures

5.1 Bridges

At the time of the 1972 Managua Earthquake, 17 road bridges and 3 railway bridges were located in the region of seismic intensity VII and VIII in Modified Mercalli scale and two pedestrian overcrossing bridges in the region of intensity IV in M.M. scale. These road bridges are small bridges of 2.6 m to 7.6 m in span length and 12 m to 18.2 m in width and suffered no damage except slight damages of masonry works at the abutments.

Two railway bridges out of three were a very primitive structure putting sleepers on the two beams composed of five used rails, three of them in the normal position and two interlayed among former three in the up side down position.and suffered damages at the abutments. The remaining one bridge was consisted of I beam and suffered no damage.

Two pedestrian bridges were located in front of the National Theater and were six spans continuous concrete slab girder bridge of 6 m in width and 3.4 m + 4@12.5 m + 2.6 m in span length. The direction of bridge axis is north to south and the bridge was subjected to the severe east-west ground motion which was strongest among the ground motion in three direction. The damage of the bridges were the separation of bridge girder from the abutments, crashing of concrete posts

between the parapet walls of girder and the abutments and horizontal crackings at the east and west ends of bridge piers. On a contact face of bridge girder and the abutment, the trace of scratching, 100 mm in westward, 55 mm in eastward and 85 mm in downward direction was found, which showed the violent relative displacement of girder and the abutment.

Because of the uncertainty of the ground conditions at the site and the construction of bridge piers, the earthquake response of the bridges is not yet analyzed, however, from the pseudo-static analysis applying the maximum acceleration observed at the ESSO Oil Refinery as a static load it is clearly recognized that the bridge piers were subjected to tensile stresses exceeding far beyond the tensile strength of concrete.

It should be noticed that the damage of bridges was slight compared with the building damages surrounding the bridges.

5.2 Road and Highway

The damages of road and highway were cracking of pavement by the passing faults, failure of curb stones, retaining walls and paving stones on sidewalks, breaking and interruption of road by slope failures and slides and the blocking of road by the collapsed buildings and electric poles but it can be said that the damage of road and highway was slight compared with the damage of other structures and facilities.

The most prominent highway damage took place near the Lake Nejapa on Pan American Highway 2 southwest of Managua. A part of Highway of

about one kilometer long was blocked by slope slides in cuts and embankments and it took about one day to reopen the Highway to the traffic.

5.3 ESSO Oil Refinery

The ESSO Oil Refinery is located west of Lake Asososca about eight kilometers apart from the estimated location of epicenter. The Refinery was completed in 1962 and the design of facilities was carried out based on API Standard including the earthquake loading. Crude oil is transported through the pipeline of 56 km long, 15 cm in diameter and 70 kg/cm^2 of design pressure (at the time of earthquake operating pressure was 38 kg/cm^2) from the Puerto Somoza located at the Pacific Coast of Nicaragua. Crude oil is stored in the material tanks and after processed at the refinery, oil products are stored in the product tank and transported to the consumers by tank lorries and railway trains.

The crude oil pipeline is embeded 0.9 m under the ground surface and gets on the ground at the pig treatment yard then again is embeded to the material tanks. The survey team found that the pipeline was warped at the upstream portion of the pig treatment yard. However, the warping of pipeline did not effect the operation and the transportation of oil was continued during and after the earthquake without interruption. Whole area the pipeline was embeded was inspected by the employee of the Refinery and no abnormality was found.

The most prominent damage in the Refinery was the settlement of foundation of two cylindrical product tanks about 4.5 cm and the associated cracking of asphalt pavement about 10 cm in width around

the tanks. These two tanks were full at the time of earthquake and others were not and the reason that the foundation settlement took place only at the two tanks was attributed to this fact. The pipes connected to these two tanks had flexible joints sufficiently bearable to the amount of settlement occurred and suffered no damage.

In the Refinery, a spherical propane gas tank supported by six steel posts was equipped. The diameter and the storage capacity of the tank were 14.6 m and 160 m^3, respectively. The six posts rest on the monolithic concrete foundation and the tank suffered no damage.

The ground of the Refinery consists of cemented sand with gravel or silt with sand, the ground water surface is 13 meters below the ground surface and N-value (standard penetration test) near the surface exceeds ten. The reason that the damage at the Refinery was very slight in spite of the severe ground motion of 0.39 g as shown on the strong motion accelerograms recorded at the Refinery can be attributed to the facts that the ground of the Refinery was in such a good condition mentioned above and the structures and facilities were designed against the earthquake load as same as those in the third zone of the State of California.

5.4 Thermoelectric Power Plant

The total power generated in Nicaragua is 192 MW (mega watts) consisted of 100 MW from two hydroelectric power plants located in the northern mountainous area, 77 MW from a thermal power plant in Managua including 2 MW by two diesel generators and 15 MW from a gas turbine power plant in the City of Chinandega.

The thermal power plant in Managua is located on the shore of Lake Managua northeast of the city. A fault caused by the earthquake passed through the yard of power plant and the damages in the plant were consisted of failure of boiler foundation, dislocation of transformer, tilting of a transmission tower, breaking of cooling tower and steel stacks, cracking of warehouse walls and falling of overhead crane girder in the machine shop. As a result of these damages, three (whole) generators were out of work after the earthquake. Besides above mentioned damages, settlement of filled soil around the fuel storage tanks, tilting of sheet piled retaining walls and crushing of concrete at the intersection of retaining walls took place.

The power failure at the thermal plant caused the shortage of electric power in Nicaragua and resulted in the power failure in other regions. Among three generators, the first one was repaired and reopened the operation after 18 days. The second one after 32 days and the last one more than 100 days.

5.5 Sewage System

The sewage trunk lines and the terminal treatment yard are now under the investigation and not yet established so suffered no damage by the earthquake. At present, the damage survey on the sewage pipes, total length of which is 500 km and on the man-holes of 6,000 units are partially carried out.

Among the surveyed sewage pipes of 153 km long, dislocation of joints and cracking of pipes were found on pipes of 31 km and among the surveyed man-holes of 1912 units, dislocation of joints was found on man-holes of 387 units. No buoying of man-holes caused by the liquefaction of sand layer was found.

5.6 Water Supply System

The source of the municipal water supply for 420,000 citizens is the Lake Asososca and the purification plant is located on the top of crater cliff. The water supply system suffered the damages such as the breaking of supply pipes in the city, toppling of a pumping unit among five into the crater lake, slope failure surrounding intake pipe line, pulling out of anchoring bolts at the foundation of storage tanks and cracking of nuts of anchoring bolts and break of water supply head tanks in the city.

The damage of supply pipelines mainly consisted of dislocation at the joint of main and supply pipe and dominated on the pipes in the east west direction. Where the pipes of asbestos were used, shearing off of pipes took place. These damages mainly took place in the region where the ground failure associated with the faults.

A reinforced concrete head tank of 40 meters of diameter and 6 meters high located in the city suffered the unequal settlement of foundation, the maximum settlement was 30 centimeters and cracking of wall covering two-thirds of total height.

5.7 River

Two small rivers are located in the east and west part of the city and the average depth is about 3 meters and width of river bed is 3 to 5 meters. In the dry season no water flows the rivers and no river dyke exists. The design discharge of west river is 90 m^3/sec and that of east river 80 m^3/sec and the gradient of river bed is 0.5%. The both sides of west river were consisted of natural slope of 1:0.6 and along the river, slope failures took place at many sites.

For preparing to the rainy season, protection work of river bed and slopes using concrete slates was carried out when the survey team visited the city.

6. Conclusion

The Managua Earthquake of 1972 showed an aspect of earthquake damage of modern city and offered us various problems which could be considered in the future. The problems are as follows:

a. Considering the geotectonical movements and volcanic activities near Managua, it is expected that there is high possibility of occurrence of earthquake of similar magnitude and violence near Managua in the future. Especially, A and C Faults in Figure-1 are existing faults and these faults will be actuated by the future earthquakes resulting in the increase of damage. Followingly, the site of reconstruction of capital city should carefully be selected.

b. In the past, there was no earthquake observatory in Nicaragua and the investigation of mechanism of earthquake and of regional seismicity has been hindered by the lack of observed data. With the Managua Earthquake as a turning point, it is desired to establish a network of seismological and volcanological observation in Nicaragua and to periodically observe the geotectonical movements of ground.

c. At present, there is no earthquake resistant building code and regulations except the regulation based on the city

planning in Nicaragua. It is necessary to establish a system of earthquake resistant building code, regulations and standards of practice satisfying the local condition in the country.

d. By the Managua Earthquake of 1972, the key point buildings, such as fire stations, schools and hospitals for refuge and emergency activities were damaged and nindered these activities, especially the operation of fire engines and ambulance cars. It is necessary to keep the sufficient earthquake resistivity of these key point buildings and to arrange their locations efficiently dispersed in the city.

e. In the tropical and subtropical region, thick earthen walls and roofs are frequently used in residential houses for insulating heat resulting in the increase of building damages and loss of lives. It is necessary to develop thermal insulation materials of low cost and light weight for residential houses.

f. Blocking of city streets by collapsed buildings and electric posts hindered the fire fighting, refuging and emergency activities. It is desired to establish the city planning preventing the blocking of street even in the residential area, together with to enforce the earthquake resistant design of buildings and to embed the electric supply lines.

g. Untreated steep natural slopes are liable to cause sliding by the earthquake resulting in loss of lives, blocking of road and damage of structures. It is necessary to improve the steepness of natural slopes and to construct failure preventing works.

h. It should be deemed that the municipal water supply systems loose their function by earthquake damages and it is necessary to keep water for fire fighting by means of adoption of independent water supply system or construction of underground water tanks.

i. When the earthquake damages are violent, not only the residents but members of emergency activities in the damaged area frequently loose their activities. It is desirable to establish rescue organizations in each region which can independently carry out their activity when they are despatched to the damaged area and to train them preliminary.

7. Acknowledgement

The authors wish to express their hearty thanks to the personnel in the concerned organization in Nicaragua and members of the earthquake damage survey team despatched from U.S. and Mexico who gave us valuable informations and recommendations and exchanged opinions.

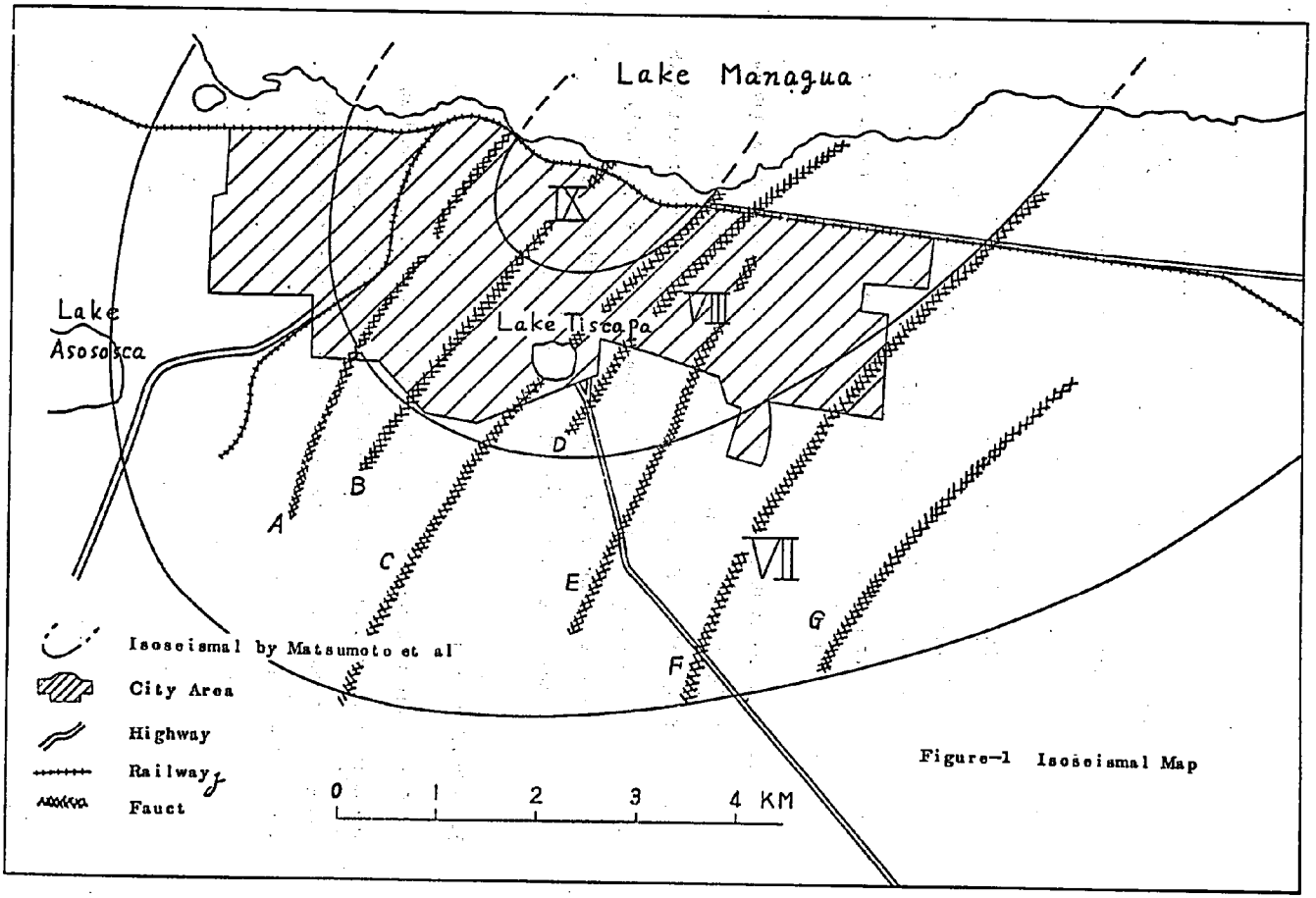
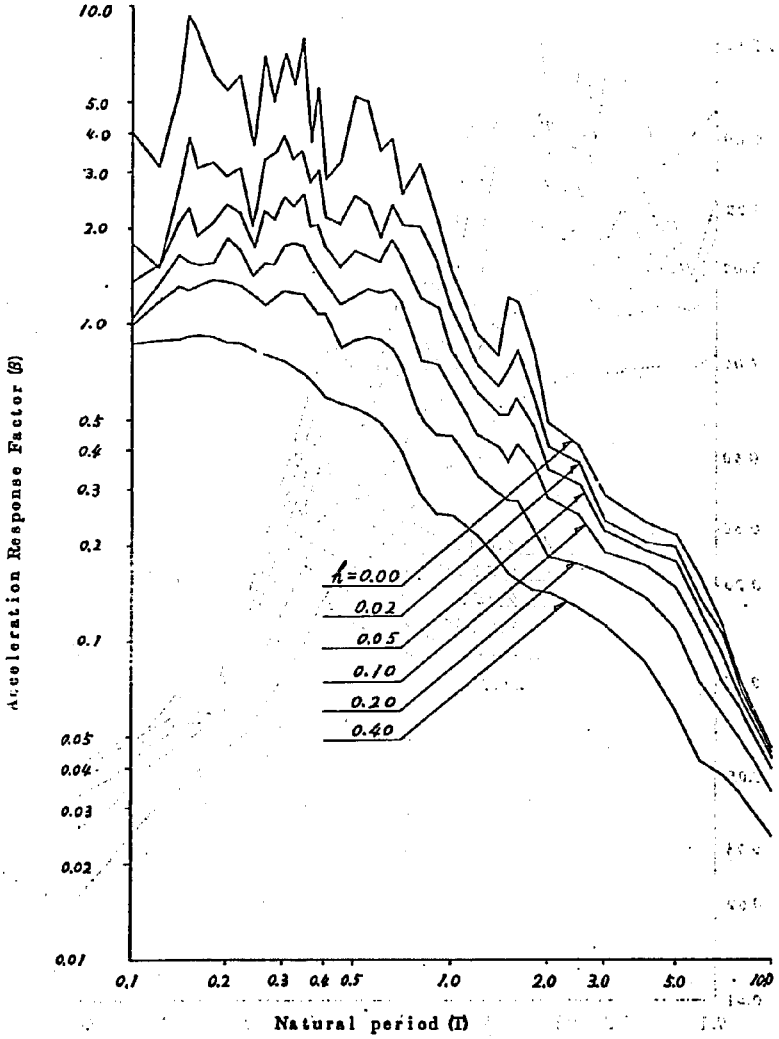
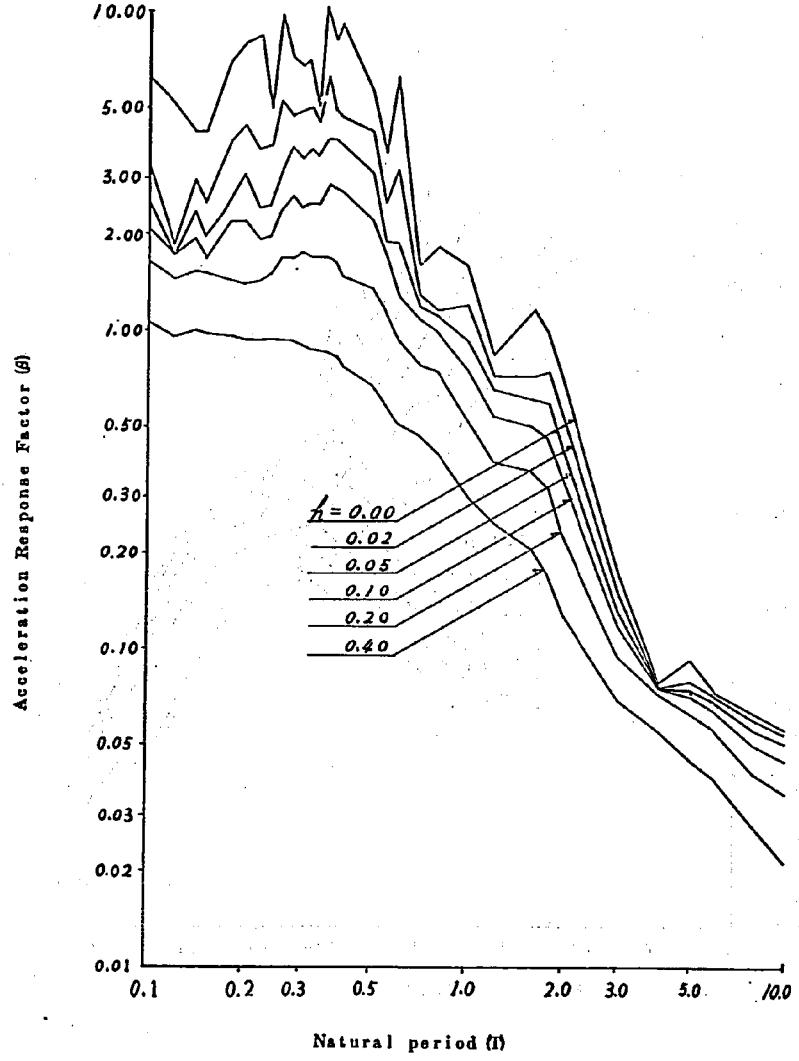


Figure-1 Isoseismal Map



Figure– 2(a) E–W Component



Figure−2(b) N−S Component

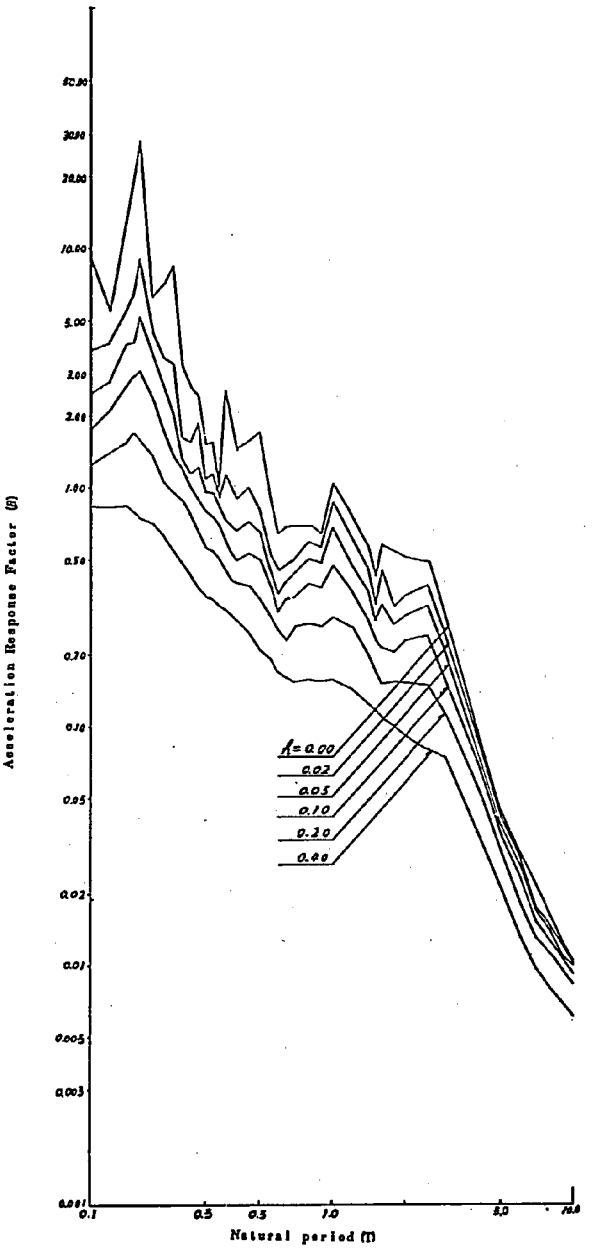


Figure-2(3) U-D Component

EFFECTS OF SEISMIC AND SUBSOIL CONDITIONS ON EARTHQUAKE RESPONSE SPECTRA

By

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ABSTRACT

Characteristics of earthquake response spectra of 44 strong-motion records observed recently in Japan are studied. Response spectra are classified into several groups in four ways, according to magnitudes, maximum ground accelerations, epicentral distances and subsoil condition, and consequently the spectra are averaged respectively in each group.

From this investigation the following conclusions are drawn.

1) The average response acceleration factors (the averages of the ratios of the maximum response accelerations to the maximum ground accelerations) for the records observed during earthquakes with large magnitudes show comparatively high level in the range of relatively long natural periods.

2) The factors, however, for the records with accelerations of 200 gals or more show comparatively low level.

3) The factors for the records with great epicentral distances show comparatively high level.

4) The factors for the records obtained on stiff subsoil layers are lower than the ones on soft subsoil layers.

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1. Introduction

Earthquake response spectra have been served to evaluate seismic effects on structures with relatively long natural periods.

The response spectrum technique was first proposed in 1942 by M.A. Biot(1) and it was the clue to development of reasonable earthquake resistant design for flexible structures. Later, in 1953, G.W. Housner et al.(2) calculated earthquake response spectra with consideration for the effect of damping. In 1959 G.W.Housner(3) proposed average spectra.

In Japan, T. Takata et al.(4) calculated earthquake response spectra of strong-motion records observed in that country and proposed average spectra from 20 horizontal acceleration records in 1964. The average spectra have been used in earthquake response analyses of public works such as highway bridges(5) and were adopted to the Japan Society of Civil Engineer's Specifications for Earthquake Resistant Design of the Honshu-Shikoku Bridges (1967)(6).

Recently T. Katayama(7) and S. Hayashi et al.(8) discussed effects of seismic and subsoil conditions on earthquake response spectra.

In this paper characteristics of earthquake response spectra of 44 strong-motion records observed recently in Japan are studied.

2. Earthquake Response Spectra

When a vibrational system of single-degree-of-freedom, consisting of a mass m with spring coefficient k and damping coefficient c is subjected to earthquake ground motion z at its reference axis, the relative displacement y will be produced. The equation of motion for the system is

$$m\ddot{y} + c\dot{y} + ky = -m\ddot{z} \qquad (1)$$

The solution of equation (1) is given by the following integral.

$$y = -\frac{1}{p\sqrt{1-h^2}} \int_0^t \ddot{z}(\tau) e^{-ph(t-\tau)} \sin p\sqrt{1-h^2}\,(t-\tau)\, d\tau \qquad (2)$$

where

$p = \sqrt{k/m}$: natural circular frequency (natural period $T = 2\pi/p$)

$h = c/2\sqrt{mk}$: damping ratio

The maximum response displacement $|y|_{max}$ computed for earthquake ground acceleration $\ddot{z}$ in equation (2) is a function of natural period T with damping ratio h as a parameter. This function defines the response displacement spectrum(S_D).

Equation (2) may be written as

$$S_D = |y|_{max} = \frac{1}{p\sqrt{1-h^2}} \left| \int_0^t \ddot{x}(\tau) e^{-ph(t-\tau)} \sin p\sqrt{1-h^2}\,(t-\tau) d\tau \right|_{max} \quad \cdots\cdots (3)$$

The response velocity spectrum (S_V) and the response acceleration spectrum (S_A) are defined respectively as

$$S_V = \frac{2\pi}{T} S_D \quad \cdots\cdots (4)$$

$$S_A = \left(\frac{2\pi}{T}\right)^2 S_D \quad \cdots\cdots (5)$$

The ratio of the response acceleration spectrum to the maximum ground acceleration $|\ddot{x}|_{max}$ means the magnification of the response acceleration of the vibrational system to the earthquake ground acceleration. This ratio defines the response acceleration factor β.

$$\beta = \frac{S_A}{|\ddot{x}|_{max}} = \frac{kS_D}{m|\ddot{x}|_{max}} \quad \cdots\cdots (6)$$

In this paper response spectra are represented by the β-spectra. The β-spectra of different earthquakes can be directly compared and averaged since the spectra are nondimentional values and they are normalized to the maximum ground accelerations.

3. Strong-Motion Earthquake Records(9)–(16)

For the study 44 digitized accelerograph records are selected. All of the records are observed recently during comparatively large earthquakes in Japan. The accelerographs are the SMAC and the DC strong-motion accelerographs.

The records are selected so that the magnitudes and the maximum accelerations may be over about five and 20 gals respectively and groups may be practically equal in the number of records in case of classification.

The records on subsoil layers where liquefaction or faults were observed nearby are excluded.

4. Classification of Records

The kinds of classification are shown in Table I. The 44 records are classified into several groups in four ways, according to magnitudes, maximum ground accelerations, epicentral distances and subsoil condition in order to examine the effects of these conditions on earthquake response spectra.

Furthermore in order to examine whether the 44 records are enough to take the average in each group, the 44 records are classified randomly into three groups.

The classification of subsoil condition is as follows.

GR 1: Bedrock or Diluvial or Alluvial layers with depth less than about three meters (10 feet) above bedrock.

GR 2: Diluvial layers.

GR 3: Alluvial layers which are stiffer than GR 4.

GR 4: Alluvial layers which have standard penetration test N-value of 10 or less extending from the surface to the depth of 10 meters (30 feet) or more.

5. Totally Average Spectra

The totally average spectra for the 44 records are shown in Fig. 1. The average spectra (1964) proposed by T. Takata et al.(4) are shown in Fig. 2. The average spectra in Fig. 1 and Fig.2 are very similar to each other. However, the former spectra are lower than the latters. This decrease probably results from following three reasons. First, the number of records of the average spectra (1970) are more than double of the average spectra (1964). Secondly, the 44 records include all kinds of records more equally. Thirdly, the 44 records include more records with large acceleration.

6. Classified Average Spectra.

Response spectra are classified into several groups in five ways and consequently they are averaged respectively in each group.

Four average spectra with damping ratio of 5% corresponding to four groups of records classified by magnitutes are shown in Fig. 3. It may be remarked that the average β -spectra for records observed during earthquakes with large magnitudes tend to show comparatively high level in the range of relatively long natural periods. For example, the average β -spectra with damping ratio of 5% for the groups with magnitude of 4.8 to 5.5 and of 7.5 to 7.9 are about 0.46 and 1.9, respectively, at 1 sec. of the natural period.

Three average spectra with damping ratio of 5% corresponding to three groups of records classified by maximum ground accelerations are shown in Fig. 4. It is difficult to find distinct difference between these spectra. However, it may be remarked that the β -spectrum for the group of records with accelerations of 200 gals or more shows comparatively low level in the period range around 1 sec. For example, the average β -spectra with damping ratio of 5% for the groups with maximum ground acceleration of 20 to 50 gals and of 200 to 438 gals are about 1.4 and 0.74, respectively, at 1 sec. of the natural period.

Three average spectra with damping ratio of 5% corresponding to three groups of records classified by epicentral distances are shown in Fig.5. It may be remarked that the average β -spectra for the group of records with great epicentral distances tend to show comparatively high level in the range of relatively long natural periods. For example, the average β -spectra with damping ratio of 5% for the groups with epicentral distance of 5 to 20 kilometers (3 to 13 miles) and of 120 to 322 kilometers (75 to 201 miles) are about 0.83 and 2.0, respectively, at 1 sec. of the natural period.

Five average spectra with damping ratio of 5% corresponding to five groups of records classified by subsoil condition are shown in Fig. 6. The fifth group is the union between the third and the fourth shown in Table II. It may be remarked that the average β -spectra for the records obtained on stiff subsoil layers tend to be lower than the ones on soft subsoil layers in the relatively long natural periods. For example, the average β -spectra with damping ratio of 5% on rocky grounds and on soft alluvial grounds are about 1.0 and 1.8, respectively, at 1 sec. of natural period. The average spectra for rock, diluvium and stiff alluvium do not show much difference from one another in the period range around 1 sec. However, the average spectrum for soft alluvium show high level and much differ from the others in the same period range.

Three average spectra with damping ratio of 5% corresponding to three groups of randomly classified records are shown in Fig.7. Since the three average spectra are practically agree with one another, it can be said that the amount of records are enough to examine effects of seismic and subsoil conditions on earthquake response spectra.

An example of the deviation of spectra is shown in Fig.8. It may be remarked that earthquake response spectra deviate considerably from one another.

7. Conclusion

From above investigation the following conclusion may be drawn.

1) The average β -spectra for the records observed during earthquakes with large magnitutes show comparatively high level in the range of relatively long natural periods.

2) The spectra for the records with accelerations of 200 gals or more show comparatively low level.

3) The spectra for the records with great epicentral distances show comparatively high level.

4) The spectra for the records obtained on stiff subsoil layers are lower than the ones on soft subsoil layers.

The spectra were adopted with some simplification to "Specifications for Earthquake Resistant Design of Highway Bridges", issued in January 1972, by the Japan Road Association.(17)

Acknowledgement

This paper largely depends upon the members who service the instruments at stations so effectively in spite of their busy daily works. Their efforts should be greatly appreciated. The authors also wish to express their appreciation to the organization in concern for their considerable supports to the observation of strong-motion earthquakes.

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Table I Data of 44 Records

No.	Record		Date of Earthquake	Magnitude	Depth (km/mile)	Epicenteral Distance (km/mile)	Maximum Acceleration (gal)	Length of Records Used (sec)	Station	Subsoil condition[1]
1	TOKYO	N-S	1956. 2.14	6.0	50/31	34/21	70.25	9.0	Earthquake Research Institute, Univ. of Tokyo SMAC-A	2
2	TOKYO	E-W					56.00			
3	TOKYO	N-S					66.63	10.0	Earthquake Research Institute, Univ. of Tokyo DC	
4	TOKYO	E-W					50.13			
5	KUSHIRO	N-S	1962. 4.23	7.0	60/38	96/60	243.88	39.0	Kushiro	2
6	KUSHIRO	E-W					372.75			
7	SHIOYA	N-S	1963. 3.27	6.9	0/0	142/89	26.88	6.0	Shioya	1
8	SHIOYA	E-W					24.25			
9	IWAYA	N-S				153/96	22.38	9.0	Iwaya	1
10	AKASHI	N-S				146/91	79.88	11.0	Akashi	2
11	AKASHI	E-W					36.13			
12	901-GR-4	N-S	1963. 8. 4	5.1	40/25	163/102	76.75	8.0	Public Works Research Institute, Chiba Branch	2
13	901-GR-4	E-W					71.00			
14	904-GR-1	TR	1965. 4.20	6.1	20/13	46/29	41.50	20.0	Shintenryugawa Bridge	3
15	904-GR-1	LG					100.40			
16	SHIZUOKA	N-S				9/6	108.30	10.0	Shizuoka	3
17	SHIZUOKA	E-W					67.00			
18	905-B-18	TR	1966. 4. 5	5.4	0/0	8/5	206.13	20.0	Ochiai Bridge	3
19	905-B-18	LG					186.88			
20	905-C-7	E-W				11/7	139.38	25.0	Nagano	4
21	905-C-7	N-S					230.38			
22	605-GR-2	N-S	1966. 5.28	5.3	2/1	12/8	52.75	10.0	Sunobana Dam	
23	605-GR-2	E-W					46.88			
24	605-GR-26	N-S	1966. 8.20	4.9	0/0	11/7	130.38	10.0		
25	605-GR-26	E-W					129.13			
26	202-GR-11	TR	1966. 9. 8	4.8	10/6	5/3	45.00	10.0	Uonuma Bridge	1
27	901-GR-17	E-W	1966.10. 8	4.9	40/25	55/34	89.40	15.0	Public Works Research Institute, Chiba Branch	2
28	901-GR-17	N-S					45.50			
29	801-GR-1	N40°E	1966.11.12	6.0	10/6	11/7	117.60	20.6	Ariake-Sea Embankment	4
30	801-GR-1	N50°W					86.50	20.4		
31	308-GR-2	LG	1968. 4. 1	7.5	30/19	104/65	169.88	40.0	Itajima Bridge	4
32	308-GR-2	TR					186.25			
33	1303-GR-1	N-S	1968. 5.16	7.9	0/0	322/201	181.25	50.0	Shin-Ishikari Bridge	4
34	1303-GR-1	E-W					174.88			
35	1305-GR-1	LG				156/98	77.50	30.0	Horoman Bridge	3
36	1305-GR-1	TR					52.25			

Table I Data of 44 Records (continued)

No.	Record		Date of Earthquake	Magnitude	Depth (km/mile)	Epicenteral Distance (km/mile)	Maximum Acceleration (gal)	Length of Records Used (sec)	Station	Subsoil condition[1]
37	1303-GR-2	N-S	1968. 5.16	7.5	40/25	223/139	96.88	50.0	Shin-Ishikari Bridge	4
38	1303-GR-2	E-W					170.88			
39	1305-GR-3	LG				74/46	57.00	30.0	Horoman Bridge	3
40	1305-GR-3	TR					46.25			
41	310-GR-15	LG	1968. 7. 1	6.1	50/31	89/56	58.13	25.0	Hirai Bridge	4
42	310-GR-15	TR					48.75			
43	308-GR-4	LG	1968. 8. 6	6.6	40/25	19/12	437.50	15.0	Itajima Bridge	4
44	308-GR-4	TR					360.88			

1) Classification of Subsoil Condition

1: Bedrock or Diluvial or Alluvial layers with depth less than about three meters (10 feet) above bedrock.

2: Diluvial layers.

3: Alluvial layers which are stiffer than the following 4.

4: Alluvial layers which have standard penetration test N-value of 10 or less extending from the surface to the depth of 10 meters (30 feet) or more.

Table II Classification of 44 Records

Classification		Definition	Number of Records	Average Values			Figure Number
				z_{max} (gal)	M	Δ (km/mile)	
T O T A L			44	116.8	6.3	84/53	1
Magnitude (M)	MAG1	4.8~5.5	13	111.5	5.1	40/25	3
	MAG2	5.5~6.5	12	72.6	6.1	37/23	
	MAG3	6.5~7.5	9	178.3	6.9	107/67	
	MAG4	7.5~7.9	10	121.3	7.7	176/110	
Acceleration (z_{max})	ACC1	20 ~ 50 gals	10	38.4	6.2	86/54	4
	ACC2	50 ~ 75	10	60.1	6.2	64/40	
	ACC3	75~120	9	92.6	6.3	91/57	
	ACC4	120~200	9	163.2	6.5	124/78	
	ACC5	200~438	6	308.6	6.3	42/26	
Epicentral Distance (Δ)	DIS1	5 ~ 20km (3~13 miles)	15	156.6	5.6	11/7	5
	DIS2	20~120 (13~75)	16	106.4	6.4	67/42	
	DIS3	120~322 (75~201)	13	83.9	7.0	189/118	
Subsoil Condition	GR 1	Rock	8	59.7	5.7	61/38	6
	GR 2	Diluvium	12	104.9	6.0	88/55	
	GR 3	Alluvium 1 (Stiff)	10	94.3	6.6	59/37	
	GR 4	Alluvium 2 (Soft)	14	175.7	6.7	89/56	
Random	RAN1		14	103.0	6.3	93/58	7
	RAN2		15	108.5	6.2	87/54	
	RAN3		15	137.8	6.4	73/46	

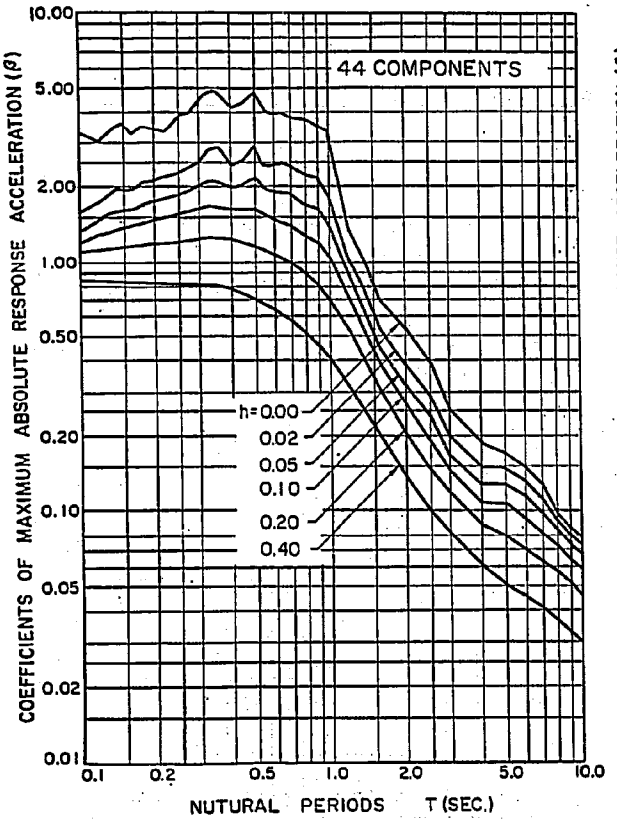


Fig. 1 Average Spectra (1970)

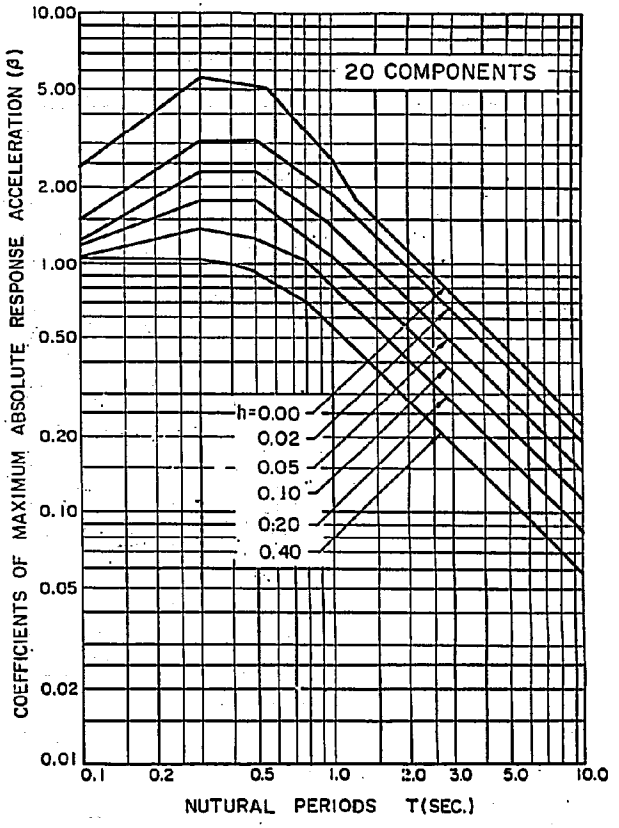


Fig. 2 Average Spectra (1964)

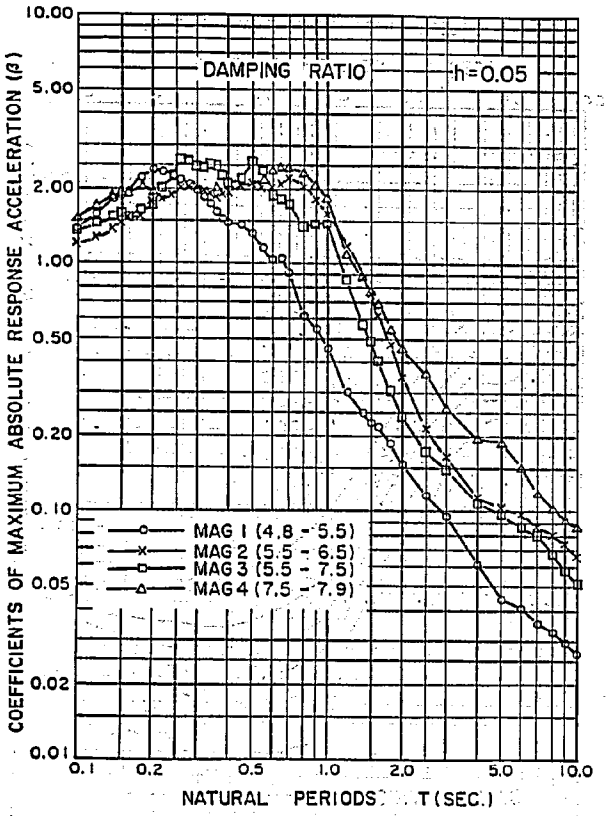


Fig. 3 Influence of Magnitude

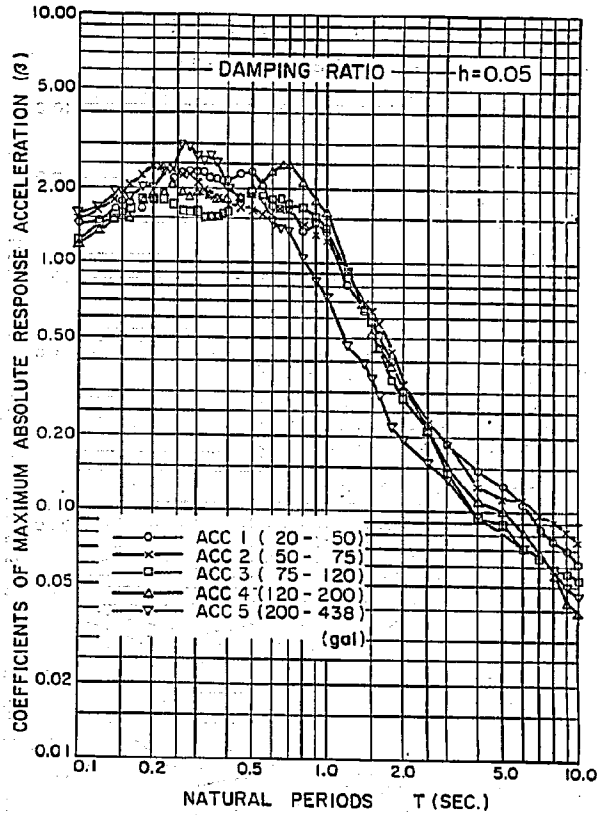


Fig. 4 Influence of Acceleration

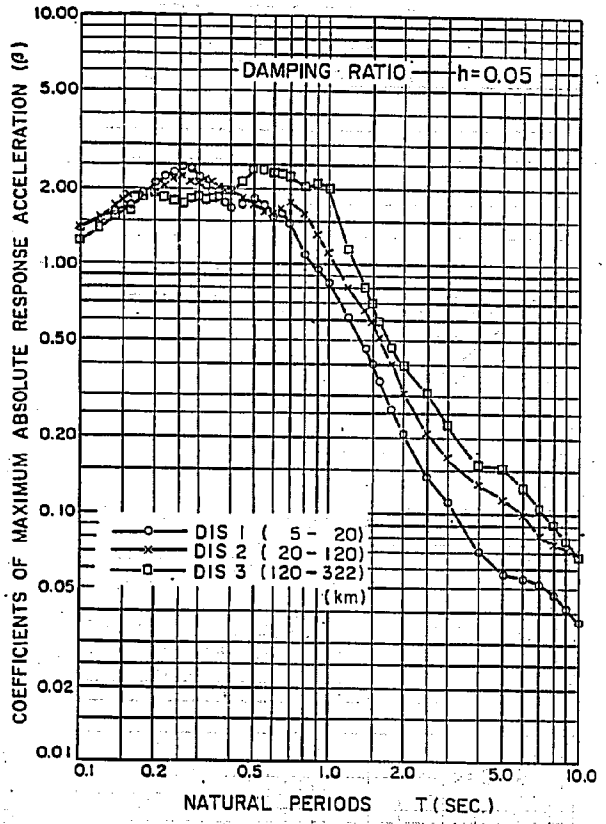


Fig. 5 Influence of Epicentral Distance

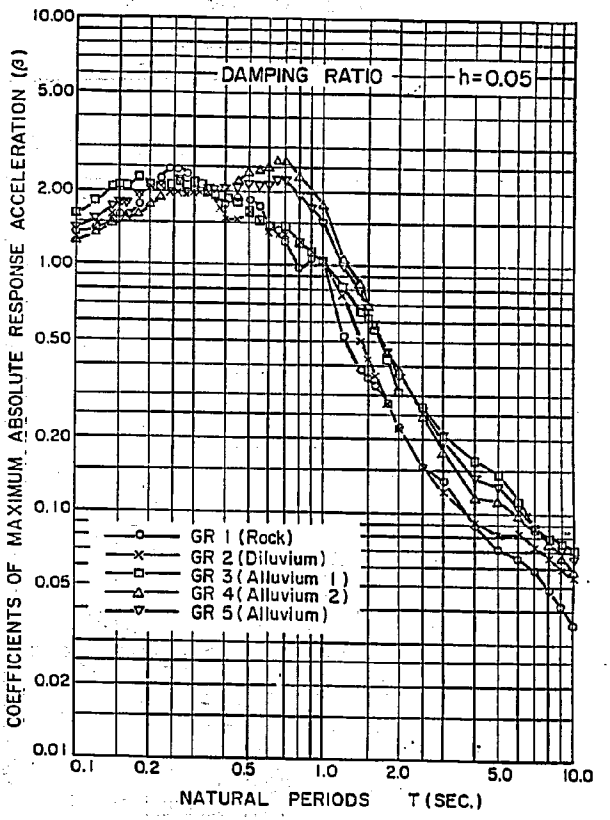


Fig. 6 Influence of Subsoil Condition

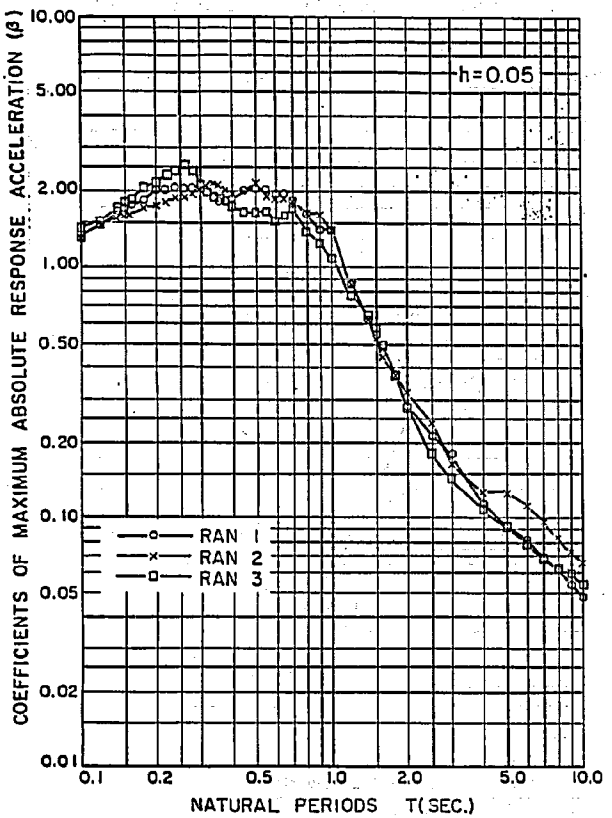


Fig. 7 Random Selection

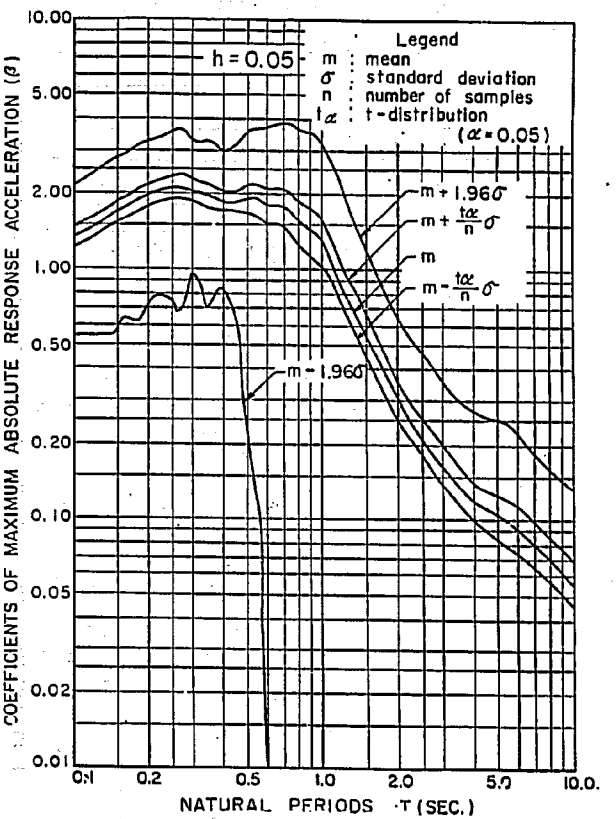


Fig. 8 Deviation of Average Spectra

DYNAMIC PROPERTIES OF HIGHWAY BRIDGES

(5th WCEE)

by

Eiichi Kuribayashi and Toshio Iwasaki

December , 1972

Public Works Research Institute
Ministry of Construction

DYNAMIC PROPERTIES OF HIGHWAY BRIDGES

by
Eiichi Kuribayashi[I] and Toshio Iwasaki[I]

SYNOPSIS

This paper presents the results of dynamic tests on some thirty highway bridges constructed in Japan during this decade, and summarizes their dynamic properties such as mode shapes, resonant frequencies, damping characteristics. The tests were conducted on foundations, pier columns, or complete bridge structures at various stages of construction, by applying lateral forces due to harmonic excitors or other equipments. From the study a relation between natural periods and damping ratios is evaluated for bridge structures. Also relations between heights of substructures above crests of foundations and natural periods, and between orders of natural modes and damping ratios are investigated.

INTRODUCTION

In the study of structural behavior to seismic excitation it is essential to evaluate structural dynamic properties, such as mode shapes, natural periods, and damping characteristics. To this aim numerous dynamic tests were performed on actual bridge structures.

In the following the results of the dynamic tests are described.

RESULTS OF DYNAMIC TESTS ON HIGHWAY BRIDGES

Between 1958 and 1969, some thirty highway bridges were tested dynamically. Most bridges experimented are common ones in Japan and designed in taking into account lateral seismic coefficients of 0.15 to 0.3 (average 0.2). The tests were conducted usually with excitation small in comparison with those expected during major earthquakes, by applying lateral forces caused by centrifugal-type excitors. For some bridges several experiments were carried out at various stages of construction (foundations, pier columns or complete bridges).

Fig. 1 indicates a relationship between natural periods and corresponding damping ratios (fraction of critical) for all meaningful data. In the figure Group A denotes results of fundamental modes for ordinary bridges, while Group B denotes those of 1st to 3rd modes for bridges with pier columns higher than 25 m above the crests of foundations. Data in Fig. 1 are classified on the basis of structural type (foundation, substructure, overall structure), and also data in Group A are classified into two on the basis of foundation type (pile, caisson).

The natural period-damping ratio relation for only Group B is shown in Fig. 2, in which numerals indicate orders of natural modes

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CLASSIFICATION OF RECORDS AND AVERAGE SPECTRA

The 44 records are classified into several groups in four ways, according to magnitudes, maximum ground accelerations, epicentral distances and subsoil conditions in order to examine the effects of these conditions on earthquake response spectra. Definition of subsoil conditions is shown in Table 1.

In this paper response spectra (represented by β -spectra) are defined to be the ratio of the response accelerations to the maximum ground acceleration, which means the magnification of the response accelerations of the vibrational systems to the earthquake ground acceleration.

Response spectra are classified into several groups in four ways and consequently they are averaged respectively in each group.

Four average spectra with damping ratio of 5% corresponding to four groups of records classified by magnitudes are shown in Fig. 1. It may be remarked that the average β -spectra for records observed during earthquakes with large magnitudes tend to show comparatively high level in the range of relatively long natural periods. For example, the average β -spectra with damping ratio of 5% for the groups with magnitude of 4.8 to 5.5 and of 7.5 to 7.9 are about 0.46 and 1.9, respectively, at 1 sec. of the natural period.

Five average spectra with damping ratio of 5% corresponding to five groups of records classified by maximum ground accelerations are shown in Fig. 2. It is difficult to find distinct difference between these spectra. However, it may be remarked that the β -spectra for the group of records with accelerations of 200 gals or more show comparatively low level in the period range around 1 sec.

Three average spectra with damping ratio of 5% corresponding to three groups of records classified by epicentral distances are shown in Fig. 3. It may be remarked that the average β-spectra for the group of records with great epicentral distances tend to show comparatively high level in the range of relatively long natural periods.

Five average spectra with damping ratio of 5% corresponding to five groups of records classified by subsoil condition are shown in Fig. 4. The fifth group is the union between the third and the fourth shown in Table 1. It may be remarked that the average β-spectra for the records obtained on stiff subsoil layers tend to be lower than the ones on soft subsoil layers in the relatively long natural periods. For example, the average β -spectra with damping ratio of 5% on rocky grounds and on soft alluvial grounds are about 1.0 and 1.8, respectively, at 1 sec. of natural period. The average spectra for rock, diluvium and stiff alluvium do not show much difference from one another in the period range around 1 sec. However, the average spectrum for soft alluvium show high level and much differ from the others in the same period range.

(4) Since rigid foundations embedded into deep soils have short natural periods and large damping capacities, deep foundations seem effective in providing bridge structures with appropriate structural resistance to seismic disturbances.

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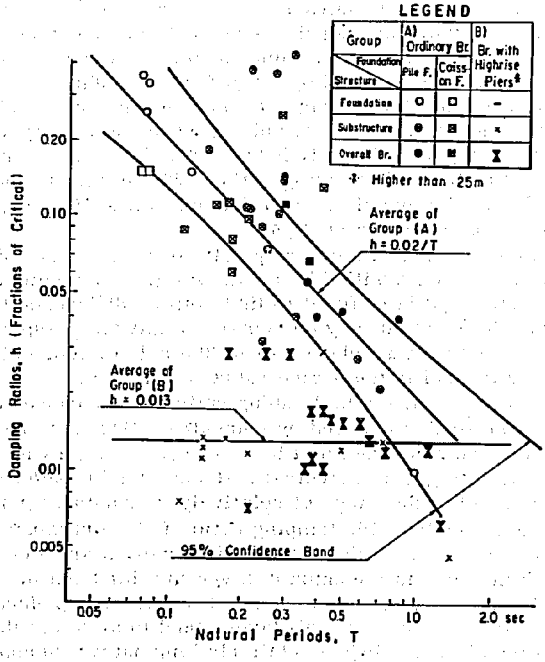


Fig. 1 Relationship Between Natural Periods and Damping Ratios for Bridge Structures

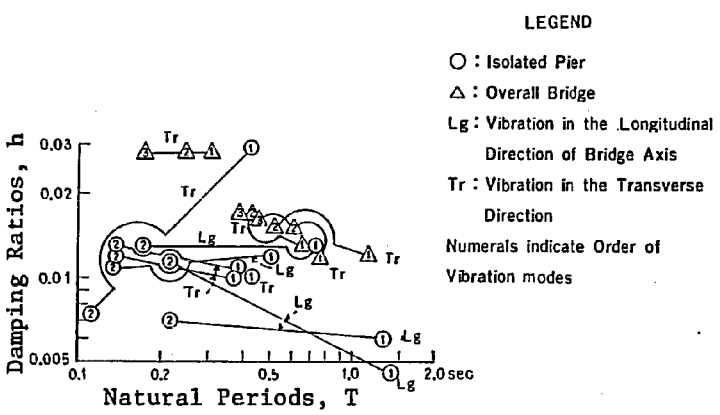


Fig. 2 Relationship between natural period and damping ratio for bridges with highrise piers

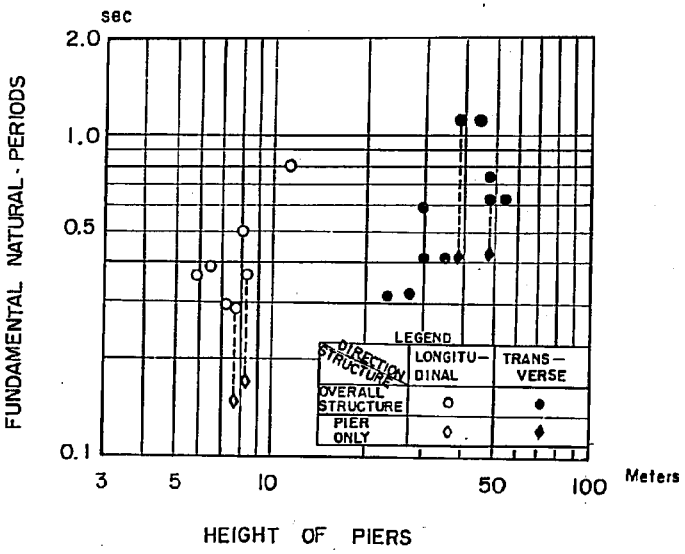


Fig. 3 Relationship between height of pier and fundamental natural period

EFFECTS OF SOIL DEPOSITS ON SEISMIC BEHAVIOR OF PREFABRICATED HIGHWAY TUNNELS

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Eiichi Kuribayashi[I] and Toshio Iwasaki[I]

SYNOPSIS

This paper deals with a comprehensive study on earthquake-resistance of prefabricated highway tunnels embedded into soft soil deposits undersea. The principal purpose of the study is to establish design methodology in providing adequate structural resistance to seismic disturbances. For evaluating dynamic behavior of soft soil deposits during earthquakes, measurements of seismic motions are carried out employing downhole accelerometers at three sites around Tokyo Bay, and dynamic analyses are conducted by utilizing seismic records obtained and applying the multi-reflection theory. Furthermore, model experiments and dynamic analyses on dynamic response of a highway tunnel embedded into soft soil deposits are conducted.

INTRODUCTION

Shirley Gut Siphon, which was constructed in 1894 as a part of the drainage system in Boston, U.S.A., is the first tunnel of prefabricated subaqueous tubes. A portion with length of 96 m among the total 457 m of the Siphon, was built by prefabricated work. The portion was made of elements with length of 14.6 m and 20.0 m, and with circular cross section of diameter of 2.6 m. The number of submerged tunnels (or prefabricated subaqueous tunnels) completed afterward or under construction at the present registers fifty one in the world for some eighty years.

It is expected that construction of tunnels of this type will become more frequent in Japan for the future, in view of considerable extension of demands for social investment as transportation facilities and sewage systems. A typical example of the demands is prefabricated undersea tunnels across the Tokyo Bay, proposed as a portion of the Tokyo Bay Loop Highways.

The land of Japan belongs to one of the most active seismic zones in the world. Particularly, major earthquakes have occurred frequently in the past around the Tokyo Bay. Accordingly, specific attention shall be given to the design and the construction of the submerged tunnels, for providing adequate structural resistance to seismic disturbances.

In order to make up a reasonable earthquake regulation by summarizing the various investigations, PWRI has requested to JSCE to establish "Committee on Earthquake-Resistant Design of Submerged Tunnels." The committee (chairman is Professor Shunzo Okamoto) was organized in July, 1971 by participants from universities and govern-

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mental institutions concerned. The committee presented the first interim report[1] in March, 1972, as a result of the activities in the beginning year.

Also, comprehensive studies on earthquake resistance of submerged tunnels have been conducted at the Public Works Research Institute (PWRI), Ministry of Construction. In the present report interim results of the investigations at PWRI are briefly described[2]-[4]

MEASUREMENTS OF SEISMIC RESPONSE OF SOIL DEPOSITS USING DOWNHOLE SEISMOMETERS

Downhole seismometers are installed at three sites around the Tokyo Bay, as shown in Fig. 1. These were set up in 1970 and 1971 by the Ministry of Construction, to measure dynamic behavior of the surface soil layers during actual earthquakes.[3]

At the site of the Futtsu Cape, indicating a comparatively uniform sand layer, three-component downhole seismometers (acceleration transducers) are equipped at three levels; surface, -70 m and -110 m. At the Ukishima Park, consisting of reclaimed soils, silts and sands from the surface, four seismometers are installed at surface, -27 m, -67 m and -127 m. At Kannonzaki, indicating a rocky layer with a surface soil layer of 2 m deep, three seismometers are set up at surface, -80 m and -120 m. Furthermore, a SMAC-type strong-motion accelerograph is also equipped on the ground surface at each of these sites, in order to surely obtain ground acceleration records during strong earthquakes.

Fig. 2 shows a set of seismic records obtained at the Ukishima Park during a moderate earthquake (Richter magnitude of 4.8 and hypocenter at 40 km directly underneath the observation site) on September 30, 1970. Fig. 3 indicates relations between maximum accelerations measured at various levels underground and depth, taking accelerations at the surface as units. It seems obvious that the results from the measurements will give valuable information in evaluating seismic effects on underground structures like prefabricated subaqueous tunnels.

ANALYSIS ON SEISMIC RESPONSES OF SUBSOILS

To analytically evaluate seismic motions in soil layers, dynamic responses of surface layers were computed by means of the Kanai's multi-reflection method, which is to numerically analyze seismic response of grounds considering the propagation of shear waves in multi-layered deposits.

Analysis was conducted for the ground at the Ukishima Park, where meaningful seismic records are available as mentioned above. Since shear velocities at the site are not clarified, twelve models with various shear moduli, shown in Fig. 4, were analyzed. Fig. 5 shows some of the results obtained. Fig. 5 (a) indicates the results for Model 1 subjected to an incident wave measured at the ground surface, and (b) is those for Model 1 subjected to an incident wave measured at the lowest

(-127 m). Fig. 6 compares the results analyzed with measured ones. From these the degree of discrepancies between analyses and measurements may be known for the case. It seems essential to adequately estimate analytical models in these analyses.

DYNAMIC ANALYSIS OF A SUBMERGED TUNNEL

Dynamic analysis was conducted for a submerged tunnel proposed across the Yokohama Bay, shown in Fig. 7. Dynamic behavior of sections of grounds and of sections of tunnel together with the surrounding soils was analyzed at three various transverse sections. Fig. 8 indicates finite element models for the three sections, A, B and C.

The following briefly describes the results for Section B (tunnel and surrounding soils). Assuming that the section is in plane strain and has a damping ratio of 10%, and that a shear wave velocity is 50 m/sec for the surrounding soil, the fundamental period of about 2.6 seconds is obtained. Fig. 9 indicates the maximum responses of displacements, and accelerations obtained for Section B subjected to the average response spectra proposed in 1964 by PWRI, when considered the maximum incident acceleration of 200 gals.

MODEL EXPERIMENTS ON DYNAMIC BEHAVIOR OF A SUBMERGED TUNNEL

Model experiments on dynamic behavior of a prefabricated submerged tunnel were performed.[3] The prototype is shown in Fig. 7. Three models consisting of soil layers and a tunnel were fabricated on a larger-size shaking table. In the experiments similarity relations between the prototype and the models were considered. To statisfy the similitude gelatin was employed as ground soils, and rubber as the tunnel which will be actually made of reinforced concrete.

It is noted that gelatin can be advantageously used for soft soil layers in model experiments, due to the fact that its Young's modulus varies from about 0.04 to 2 kg/cm^2 according to the content of gelatin in water and the temperature. Prior to the dynamic experiments, elastic properties of gelatin were examined through static compression tests. It is noted that Young's modulus changes considerably with the temperature. Therefore it is essential in the experiments to control the temperature of the gelatin models.

Three models (shown in Fig. 10) were fabricated in consideration of model to prototype ratios of 1/500 in length and 1/5 in time. Model 1 is a model of two-layered ground with the upper layer with shear velocity of 100 m/sec and uniform depth of 25 m, and the lower layer with 300 m/sec and uniform depth of 95 m. Model 2 is of two-layered ground with varying depths. Model 3 includes a tunnel in the ground of Model 2. Table 1 summarizes the outline of the prototype and the models.

Dynamic experiments were conducted using a large-size shaking

table possessed by the National Research Center for Disaster Prevention,Science and Technology Agency. The following three steps of experiments were performed, respectively, for the three models in two horizontal directions.

a) Harmonic Excitation
b) Free Damped Vibration
c) Seismic Excitation

Deformation of the models was measured using an optical displacement meter unbonded to the models, and two 16-mm moving cameras.

Fig. 11 illustrates mode shapes during resonant states. From these it is found that the fundamental mode shapes are of shear deformation of the ground and that for Model 2 and Model 3 there are two predominant natural periods which are corresponding to the resonances of the two different sections. It is found that the corners tend to vibrate with greater amplitudes than other portions. Since this seems different from actual behavior for the prototype without any particular corners, it is advised that the effects of the corners are removed.

For Model 1 that has uniform soil layers, damping ratio seems almost constant regardless of amplitude, but for Model 2 and Model 3 that have varying soil layers, damping ratio varies slightly with amplitude. The damping ratio was generally found to be 1% of critical for the most cases, the damping ratio, however, for longitudinal oscillation of the sections with the deeper soft layer, was about 2.5%.

Fig. 12 shows typical examples of responses of Model 3 when subjected to seismic excitation in the longitudinal directions. Time-acceleration records employed for the model are adjusted from seismic acceleration records obtained at El Centro during the Imperial Valley Earthquake of 1940 and at Hachinoe during the Tokachi-oki Earthquake of 1968. White noise oscillations that are roughly flat between 5 and 25 for the models, was also employed. In adjusting the seismic records and the white noise, a model-to-prototype ratio of 1/5 was taken for time from the similarity requirements, and maximum table accelerations were decided about 50 gals and 100 gals, in consideration of accuracy of measuring instruments and model strength.

For the three models, displacement responses to the Tokachi-oki record are greater than those to the El Centro record. This seems due to the fact that response spectra of the Tokachi-oki record are more predominant for structures with longer periods, than those of the El Centro record.

CONCLUSIONS

From the extensive studies through the field measurements, the laboratory dynamic experiments and the analyses described above briefly, the following conclusions can be drawn.

(1) Accelerations at deeper locations of soft soil deposits are small in comparison with the surface accelerations. Shapes of vertical

distribution of maximum accelerations are different among various earthquakes.

(2) The multi-reflection theory may be employed satisfactorily in estimating seismic response of multi-layered subsoils, provided that soils properties, primarily elastic moduli, are appropriately evaluated.

(3) For estimating three-dimensional behavior of long structures embedded into soft subsoils during earthquakes, model experiments would be an effective way. In model experiments gelatin can be advantageously used as soft soils.

(4) Dynamic analyses by means of the finite element method would be also effective in obtaining seismic response of underground structures.

(5) It is found that deformation of soft soil deposits has significant effects on the dynamic behavior of the embedded structures, and that soil deposits in resonance to seismic motions might exert the structures into critical state. Furthermore, the structures would be affected extensively in the vicinity of a site where depth of soft soil deposits varies steeply. This seems due to the fact that the deformation of deeper soil deposits is larger than that of the other section.

(6) It is expected in the near future to complete resonable specifications for the earthquake-resistant design of prefabricated subaqueous tunnels, through continuing research activities and summarizing various results. It is obvious, however, that liquefaction and failure of soils during earthquakes is a crucial factor affecting the aseismic stability of underground structures. Although the topic is not discussed in the present report, investigations on the effects on submerged tunnels have been carried out at PWRI and the other institutions concerned.

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3) E. Kuribayashi, T. Iwasaki, K. Tuji: An Experimental Study on Dynamic Characteristics of Prefabricated Tunnels, Technical Memorandum of PWRI, No. 747, March, 1972.
4) S. Ibukiyama, E. Kuribayashi, T. Iwasaki: Earthquake Resistibility of Submerged Tunnels, Fourth Joint Meeting of U.S. - Japan Panel on Wind and Seismic Effects, UJNR, Washington, D.C., May 17 - 19, 1972.

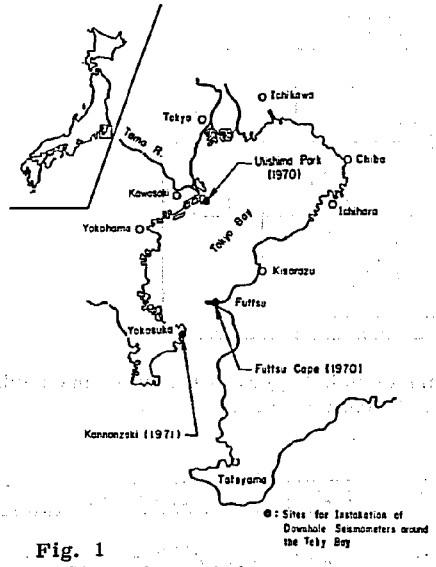


Fig. 1
Sites of Downhole Accelerometer Installation.

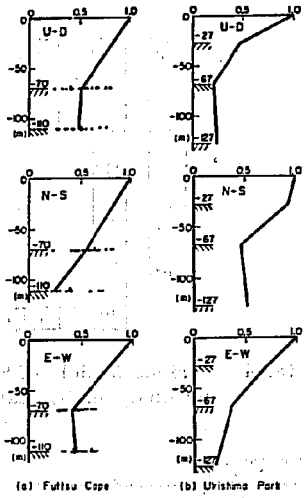


Fig. 3 Distribution of Maximum Accelerations with Respect to Depth

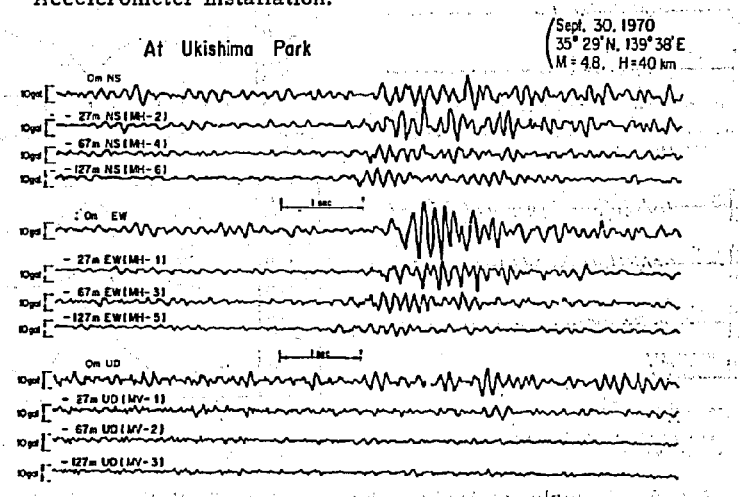


Fig. 2 Seismic Records from Downhole Accelerometers.

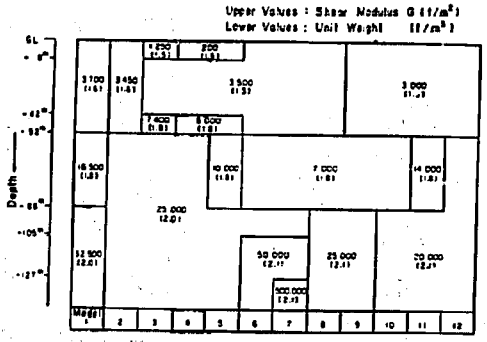
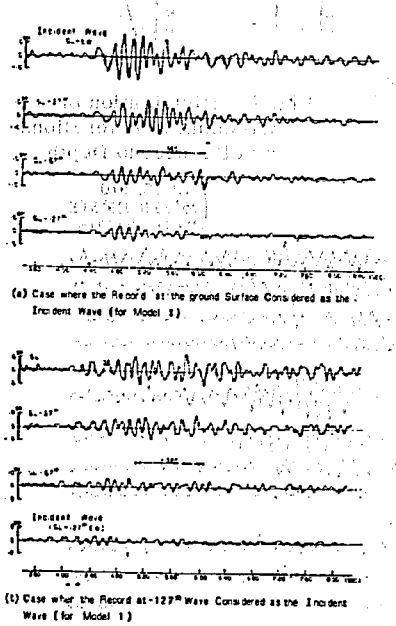


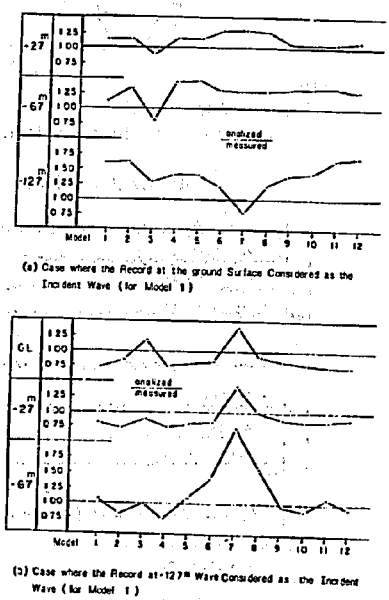
Fig. 4 Various Models of Surface Soil Layers at the Ukishima Park for Analyses by Multi-Reflection Theory.



(a) Case where the Record at the ground Surface Considered as the Incident Wave (for Model 1)

(c) Case wher the Record at -127ᵐ Wave Considered as the Incident Wave (for Model 1)

Fig. 5 Examples of Analyses by the Multi-Reflection Theory.



(a) Case where the Record at the ground Surface Considered as the Incident Wave (for Model 1)

(b) Case where the Record at -127ᵐ Wave Considered as the Incident Wave (for Model 1)

Fig. 6 Ratios of Analysed Accelerations to Measured Ones.

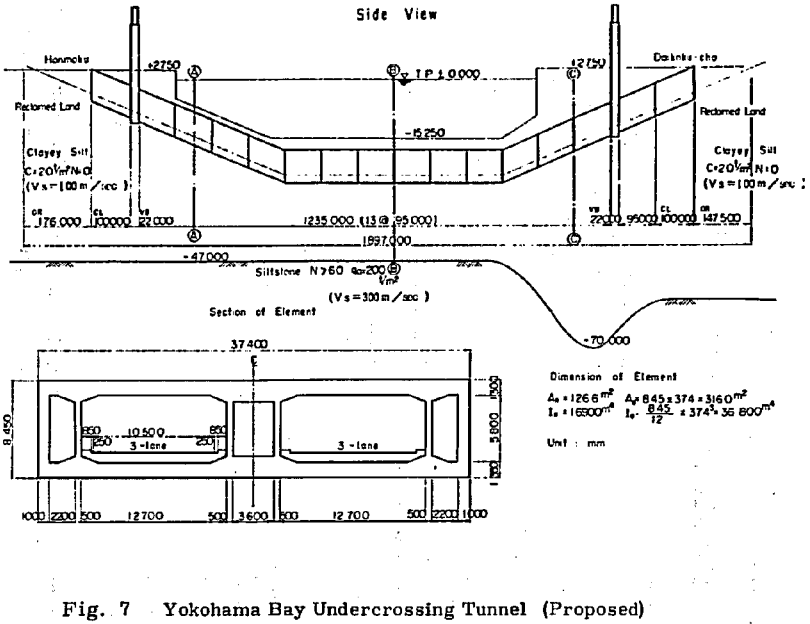


Fig. 7 Yokohama Bay Undercrossing Tunnel (Proposed)

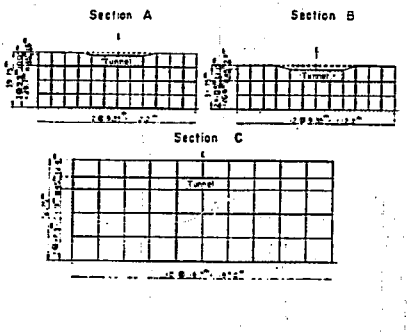


Fig. 8
FEM Models for Three Sections

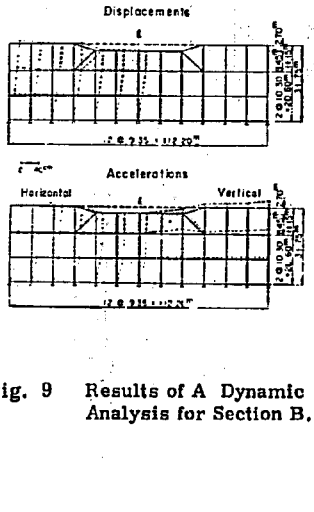


Fig. 9 Results of A Dynamic Analysis for Section B.

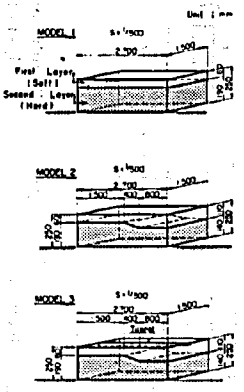
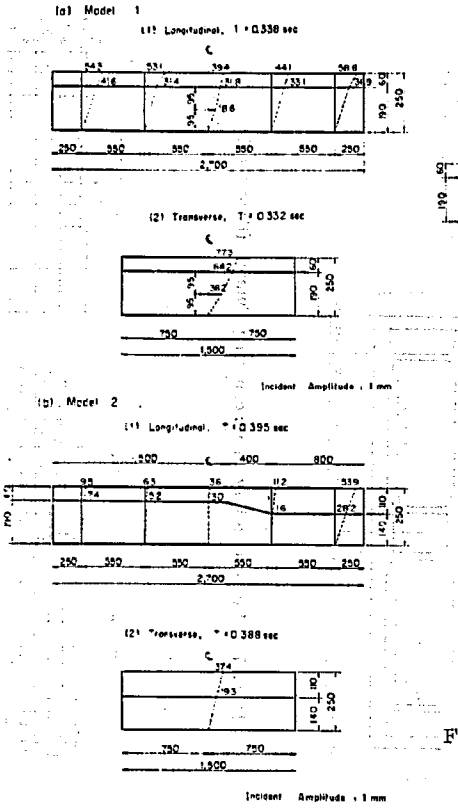


Fig. 10
Three Models Experimented.



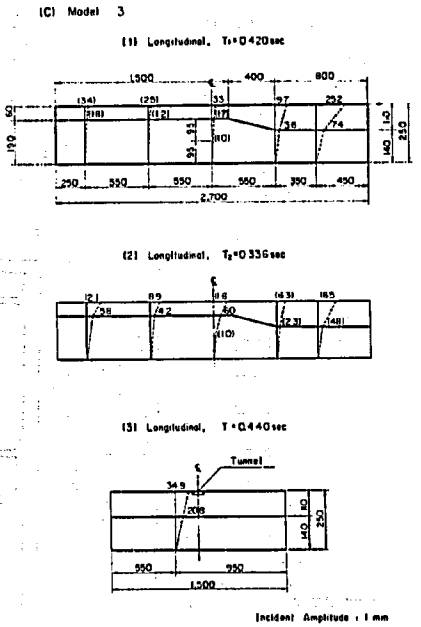


Fig. 11 Mode Shapes (Measured by An Optical Displacement Meter)

Model 3, Longitudinal



Fig. 12 Response to Seismic Excitation

Table 1 Comparison of Prototype and Model

Prototype or Model		Prototype		Model (Required from Similitude)		Model (Actual)	
	Layer	First Layer	Second Layer	First Layer	Second Layer	First Layer	Second Layer
Ground	Material	Clayey silt	Siltstone	Gelatin	Gelatin	Gelatin	Gelatin
	Dimension (plan)	750 m × 1, 350 m		1.50 m × 2.70 m		1.50 m × 2.70 m	
	Depth	30 - 55 m	95 - 70 m	0.06 - 0.11 m	0.19 - 0.14 m	0.06 - 0.11 m	0.19 - 0.14 m
	Shear Wave Velocity	100 m/sec	300 m/sec	1.0 m/sec	3.0 m/sec	1.1 m/sec	3.0 m/sec
	Shear Modulus	153 kg/cm²	1, 830 kg/cm²	0.0102 kg/cm²	0.0917 kg/cm²	0.013 kg/cm²	0.09 kg/cm²
	Young's Modulus	459 kg/cm²	5, 320 kg/cm²	0.0305 kg/cm²	0.268 kg/cm²	0.040 kg/cm²	0.27 kg/cm²
	Unit Weight	1.5 t/m³	2.0 t/m³	1.0 t/m³	1.0 t/m³	1.0 t/m³	1.0 t/m³
	Poisson's Ratio	0.5	0 0.45	0.5	0.45	0.495	0.495
	Fundamental Period	2.0 sec		0.4 sec		0.37 sec	
Tunnel	Material	Reinforced Concrete		Rubber		Rubber	
	Dimension (plan)	37.4 m × 1, 350 m		0.075 m × 2.7 m		0.075 m × 2.7 m	
	Depth	8.45 m		0.017 m		0.017 m	
	Young's Modulus	270, 000 kg/cm²		15 kg/cm²		12 kg/cm²	
	Unit Weight	2.5 t/m³ (Net 1.0 t/m³)		1.25 t/m³ (Net 1.0 t/m³)		1.35 t/m³ (Net 1.0 t/m³)	

Similitude

Length $n = L_r / L_m = 50$

Density $m = \rho_r / \rho_m = 1.53$ (first layer)
$= 2.04$ (second layer)

Time $t = T_r / T_m = 5$

Velocity $v = V_r / V_m = n/t = 10$

Acceleration $a = A_r / A_m = n/t^2 = 20$

Young's Modulus $e = E_r / E_m = mn^2/t^2 = 1.5 \times 10^4$ (first layer)
$= 2.0 \times 10^4$ (second layer)

Poisson's Ratio $\nu = N_r / N_m = 1$

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DESIGNS AND HYDRAULIC MODEL INVESTIGATIONS OF HYDRAULIC JUMP TYPE DISSIPATORS (*)

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1. INTRODUCTION

Multi-purpose dams in Japan are often constructed at locations which are comparatively near to populated areas. Therefore, complete dissipation is required of such dams. To achieve this objective, a hydraulic jump type dissipator consisting of a vertical sill is usually employed. In the hydraulic jump type dissipator, the length of the dissipator and the height of the end sill become important quantities. The length of the dissipator poses no problem, if it is set to a value 4.0 to 4.5 times as long as the conjugate depth of hydraulic jump h_2. Consequently, this paper deals with the design procedure and the method of experiments regarding the sill height of the dissipator.

2. FORMULAE FOR DESIGNING SILL HEIGHT

In calculating the sill height, the following two broadly differentiated concepts have been widely adopted :

1) The method of making the length of the stilling basin considerably shorter than that of hydraulic jump, and of considering the water pressure applied to the sill. In this method, analysis is made by means of momentum.

(*) *Projets et recherches sur les dissipateurs du modèle hydraulique du type saut de ski.*

2) The method of making the length of the stilling basin more than about 80 percent of the length of the hydraulic jump, and of considering the sill to be a device for ensuring the conjugate depth of hydraulic jump. The analysis is made by means of energy.

In the following pages, previously adopted formulae for calculating the height of sill will be reviewed, laying special emphasis on the assumptions taken in order to derive the respective formulae and on their characteristics features.

1) Doeringsfeld and Barker's Formula [1] :

$$\frac{\left(\sqrt{1+8\,F^2}-1-2\,\frac{d}{h_1}\right)^3}{\left(\sqrt{1+8\,F^2}-1+2\,\frac{d}{h_1}\right)}=\frac{3\,(\sqrt{1+8\,F^2}-1)}{64\,F^2} \qquad (1)$$

The formula shown above has been derived from the results of the experiment on the broad crested weir carried out by Doeringsfeld *et al.* and is based on the following assumption; namely, in Figure 1 below :

i) The momentum is continuous in respect of Cross-sections I, II, and III. (The condition of the fluid naturally exists. Therefore, in the formulas below, the mention of this fact will be omitted.)

$$\frac{h_2}{h_1}=\frac{\sqrt{1+8\,F^2}-1}{2}\,;$$

$$\frac{q^2}{g\,h_2}+\frac{h_2^2}{2}=\frac{q^2}{g\,h_3}+\frac{h_3^2}{2}+\frac{(2\,h_2-d)\,d}{2}.$$

ii) The relationship among h_2, d and h_3 which has been obtained by Doeringsfeld *et al.* from experiment is.

$$h_2-d=2\,h_3.$$

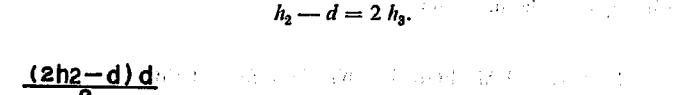


Fig. 1

As can be clearly seen from the above relationship, the formula is applicable when the sill is a broad crested weir and takes charge of water pressure corresponding to h_2.

2) Forster and Skrinde's Formula [2] :

$$\frac{d}{h_1} = \frac{\sqrt{1+8F^2}-1}{2} - \sqrt{3F^{4/3} - \frac{1}{2}(\sqrt{1+8F^2}+1)}. \quad (2)$$

This formula has been derived from the research on the sill by Forster *et al.* In Figure 2 below, this formula is based on the following assumption :

i) The momentum is continuous in respect of Cross-sections I, II and III.

$$\frac{h_2}{h_1} = \frac{\sqrt{1+8F^2}-1}{2};$$

$$\frac{\beta q^2}{g h_1} + \frac{h_1^2}{2} = \frac{\beta q^2}{g h_3} + \frac{h_3^2}{2} + \frac{(2h_2 - d)d}{2}.$$

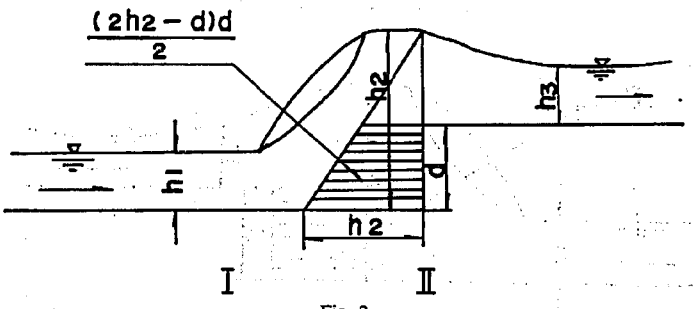


Fig. 2

ii) The water depth h_3 at Cross-section III becomes the critical depth :

$$\frac{v_3^2}{g h_3} = 1.$$

As can be clearly seen from the above relationship, the formula is applicable when the critical depth occurs above the abrupt rise which takes charge of hydrostatic pressure.

3) PWRI (The Public Works Research Institute) No. I. Formula (Mura) [3] :

$$\frac{d}{h_1} = \frac{\left(\frac{h_2}{h_1}\right)\left(\frac{C_D F^2}{K^2} + 2K\right) + \sqrt{\left(\frac{h_2}{h_1}\right)^2\left(\frac{C_D F^2}{K^2} + 2K\right)^2 + 4\left(\frac{h_2}{h_1}\right)\left\{\left(\frac{h_2}{h_1}\right)^3 - (2F^2+1)\left(\frac{h_2}{h_1}\right) + 2F^2\right\}}}{2\left(\frac{h_2}{h_1}\right)} \quad (3)$$

This formula has, in its derivation process, a characteristic feature that the water pressure applied to the sill is handled in the general form of P_x, and has been derived in the following way :

i) The momentum is continuous in respect of Cross-section I, II and III.

$$\frac{h_2}{h_1} = \frac{\sqrt{1+8F^2}-1}{2};$$

$$\frac{\beta q^2}{g h_1} + \frac{h_1^2}{2} = \frac{\beta q^2}{g h_3} + \frac{h_3^2}{2} + \frac{P_x}{w}$$

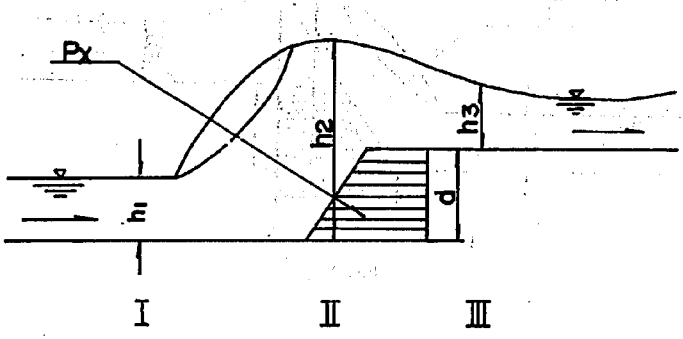


Fig. 3

From this assumption and the continuation equation, the following formula can be obtained :

$$\varphi = \left(\frac{h_2}{h_1}\right)^3 - \left(2F^2 + 1 - \frac{2P_x}{\omega h_1^2}\right)\left(\frac{h_2}{h_1}\right) + 2F^2 = 0.$$

Now, if we assume that :

$$\varphi_1 = \left(\frac{h_2}{h_1}\right)^3 - (2F^2 + 1)\left(\frac{h_2}{h_1}\right) + 2F^2;$$

$$\varphi_2 = -\frac{2P_x}{\omega h_1^2}\left(\frac{h_2}{h_1}\right);$$

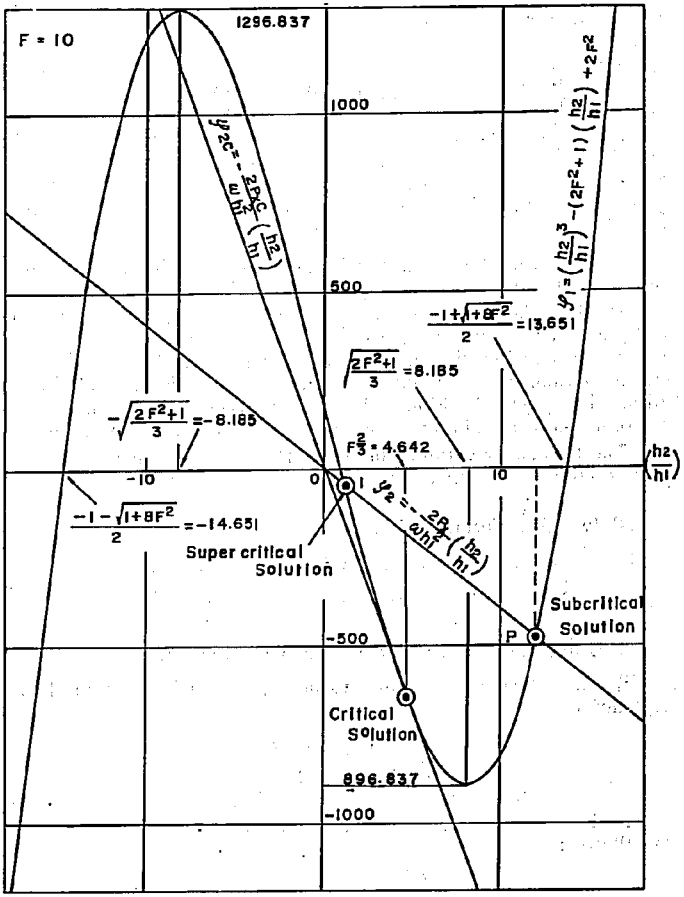


Fig. 4

then $\varphi = \varphi_1 - \varphi_2 = 0$, and the solution of $\varphi = 0$ is obtained at a point of intersection of φ_1 and φ_2. When $F = 10$, φ_1 and φ_2 will be as shown in Figure 4.

The procedure of calculating the necessary sill height using Figure 4 will be as follows :

1) From the requirements for the stilling basin, the value of h_1, h_2 and F are given; 2) when F is determined, a graph of φ_1 as shown in Figure 4 is obtained; 3) a point P on φ_1 which corresponds to the given h_2/h_1 is determined; 4) a straight line connecting P and the origin becomes φ_2; 5) P_x is obtained, from this φ_2, and 6) to obtain a sill to which a water pressure of P_x is applied become necessary. As can be clearly seen in Figure 4, P_x may become positive or negative with $h_2/h_1 = (\sqrt{1 + 8\,F^2} - 1)/2$ as boundary. $P_x > 0$ is considered to correspond to obstacles such as a sill, whereas $P_x < 0$ is considered to correspond to abrupt drop, etc. The most important problem in application of the formula is how to correlate P_x with the sill height.

ii) A coefficient of drag C_D [$= D/(p\,v\,d/2)$] is introduced and the velocity of flow near the sill v is represented by $v = v\,1/K$ (K : *a* constant).

The formula can be derived in the orthodox way, but the estimation of C_D and K will pose difficulties. As can be seen in Figure 4, φ has generally three roots, but the supercritical flow solution and the negative solution are meaningless, and only the maximum solution is physically feasible. Also, when φ gives multiple roots, namely, when φ_1 is contacted by φ_2, the roots show the solution of the critical depth and are also meaningless.

The formulas mentioned in Items 1), 2) and 3) above belong to Classification I mentioned earlier.

4) Iwasaki's Formula [4] :

$$\frac{d}{h_1} = \frac{(1 + 2\,F^2)\sqrt{1 + 8\,F^2} - 5\,F^2 - 1}{1 + 4\,F^2 - \sqrt{1 + 8\,F^2}} - \frac{3}{2}\,F^{2/3}. \qquad (4)$$

This formula is based on the following assumption :

i) The momentum is continuous in respect of Cross-sections I to II, as follows :

$$\frac{h_2}{h_1} = \frac{\sqrt{1 + 8\,F^2} - 1}{2}.$$

ii) The energy is continuous in respect of Cross-sections II to III, as follows :

$$\frac{v_2^2}{2\,q} + h_2 = \frac{v_3^2}{2\,q} + h_3 + d.$$

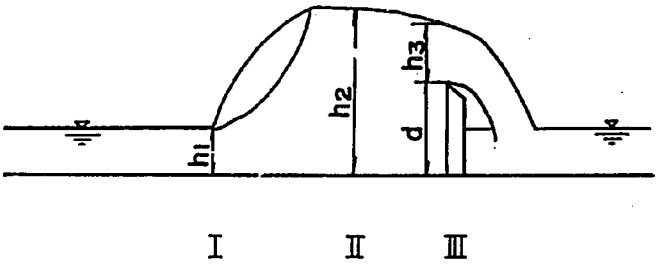


Fig. 5

iii) At Cross-section III, a critical depth is formed, as follows :

$$h_3 = F^{2/3} h_1.$$

The above-mentioned formula applies to a sharp crested weir at which no energy loss occurs in respect of Cross-sections II to III.

5) PWRI No. II. Formula (Kawamura, Fujimoto *et al.*) [5] :

$$\frac{d}{h_1} = \frac{(1 + 2F^2)\sqrt{1 + 8F^2} - 5F^2 - 1}{1 + 4F^2 - \sqrt{1 + 8F^2}} - \left(\frac{F^2}{2K^2}\right)^{1/3}$$

The above-mentioned formula employs the discharge coefficient of weir K obtained by the studies made by Homma and Nabeoka on the

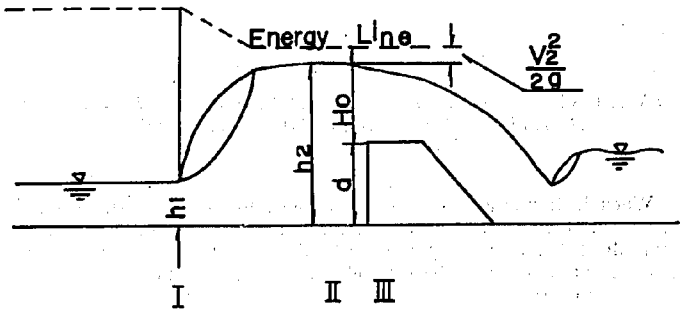


Fig. 6

discharge coefficient of a broad crested weir, and is based on the following assumption :

i) The continuation of momentum exists in respect of Cross-section I to II, as follows :

$$\frac{h_2}{h_1} = \frac{\sqrt{1 + 8\,F^2} - 1}{2}.$$

ii) Discharge Formula of Broad Crested Weir (6), (7) :

$$q = K\sqrt{2g}\,H_0^{3/2};$$

where :

$$H_0 = \frac{v_2^2}{2g} + h_2 - d.$$

The above-mentioned formula is a generalized form of Iwasaki's formula mentioned in Item 4) above, and the second term of the right side of the formula may slightly change depending upon what equation is employed as a discharge formula of the sill.

The formulas mentioned in Item 4) and 5) belong to Classification II mentioned earlier. To design this type of sills, the formula mentioned in 5) is most desirable, because in this formula, calculation can be made by including the factor ofs ill shapes, but the drawback of this formula lies in that by employing the Homma's and Nabeoka's experiments only, the scope where the discharge coefficient of weir can be obtained will be limited. In actual practice, therefore, the formula in Item 4) is used for simpler calculations.

3. EVALUATION OF HYDRAULIC JUMP TYPE DISSIPATORS IN HYDRAULIC MODEL INVESTIGATIONS

When hydraulic experiments of hydraulic jump type dissipators are carried out, the writer mainly uses the following three factors in evaluating draft designs, namely, the longitudinal allignment of water depth, water pressure applied to the sill and stream regimen (still photo). First, let us consider the ' evaluation ranks ' shown in Table 1.

TABLE 1

Rank	Evaluation
A	Excellent (too good quality)
B	Good (proper quality)
C	Fair (sometimes permissible)
D	No good (having no function as hydraulic jump type dissipator)

3.1. THE RELATIONSHIP BETWEEN THE LONGITUDINAL ALIGNMENT OF WATER DEPTH AND THE RANK IS SHOWN IN TABLE 2 AND FIGURE 7.

TABLE 2

Rank	Longitudinal allignment of water depth
A	Horizontal water surface at the end of hydraulic jump can be seen from the upper stream of point P*
B	Horizontal water surface at the end of hydraulic jump can be seen from a point between point P and the upstream surface of the sill
C	Maximum depth is seen at the upstream surface of the sill, and no horizontal water surface exists
D	Maximum depth occurs at the lower stream of the upstream surface of the sill

(*) Point P means a point lying upstream from the upstream surface of the sill at a distance equal to the height of the sill.

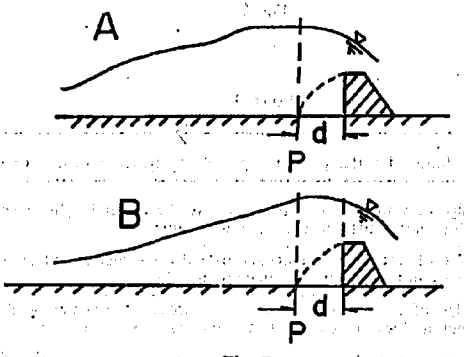


Fig. 7

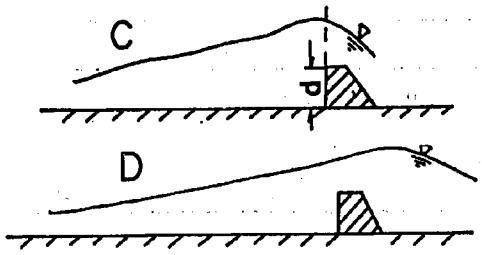


Fig. 7

3.2. THE RANKING OF THE VARIOUS KINDS OF WATER PRESSURE WHICH IS APPLIED TO THE UPSTREAM SURFACE OF THE SILL IS SHOWN IN TABLE 3 AND FIGURE 8.

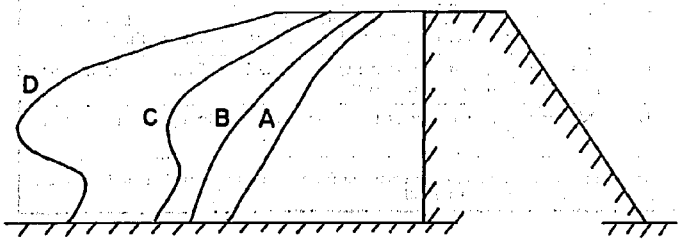


Fig. 8

TABLE 3

Rank	State of water pressure applied to upstream surface of sill
A	A bulge of water pressure occurs above the center height of sill, but it does not constitute the maximum value
B	A bulge of water pressure occurs around the center height of sill, but it does not constitute the maximum value
C	A bulge of the maximal value occurs around the center height of sill, but it does not constitute the maximum water pressure
D	The maximum water pressure occurs around the center of the sill height.

3.3. THE RANKING OF THE VARIOUS KINDS OF STREAM REGIMEN OF THE STILLING BASIN IS SHOWN IN TABLE 4 AND FIGURE 9.

TABLE 4

Rank	Stream regimen of stilling basin (scale : Approx. 1/50)
A	Turbulence of hydraulic jump does not reach the sill; the transparent portion (which does not contain air bubbles) of the flow exists from point P* to the upper stream and the depth of water that overflows the sill does not show time variation
B	Turbulence of hydraulic jump reaches the sill, but the flow between point P and the upstream surface of sill is more or less transparent, and the depth of water that overfows the sill shows only slight time variation
C	A part of turbulent flow overflows the sill; the region of the rectangular equilateral triangle connecting point P and the upstream crown of the sill is transparent; and the stream which overflows the sill shows time variation
D	The flows has excessive turbulence, overflowing the sill; the transparent region at the upstream surface of the sill barely exists; and the stream which overflows the sill scatters and falls on the river bed downstream

(*) The definition of the point P is the same as that mentioned in Item 3.1 earlier.

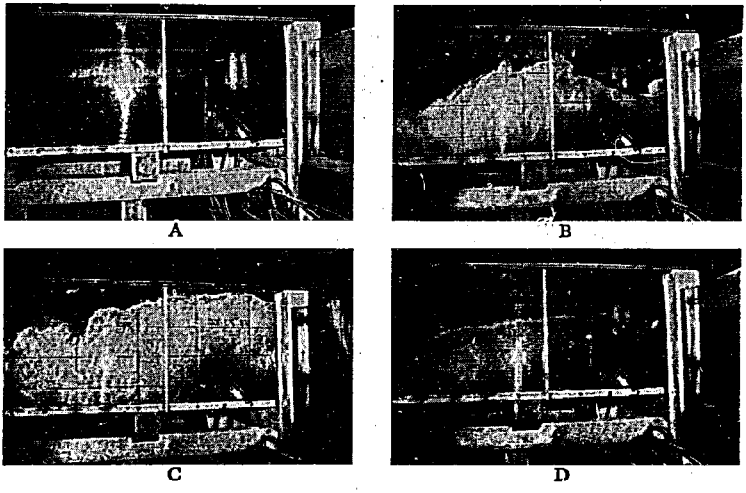


Fig. 9

4. SILL-HEIGHT CALCULATION FORMULAS AND RESULTS OF HYDRAULIC MODEL INVESTIGATIONS

At PWRI, more than 100 cases of hydraulic model experiments of hydraulic jump type dissipators have been carried out since 1953. Figure 10 is obtained by summarizing the d/h_1 and F in order to study the relationship between the ranking in each case of these experiments and the sill-height calculation formula. As can be seen in Figure 10, the value of K = 0.4 in the PWRI No. II Formula was adopted in these experiments in order to achive the ranking A or B. It is quite clear that the formula mentioned in Item 5) of Chapter 2 if the optimum formula. The specifications of the dissipator which constitutes the basis of the abovementioned data are shown in Table 5. To use the formula proposed by Forster and Skrinde will bring out the sill height which is considered to be on the dangerous side according to the writer's opinion.

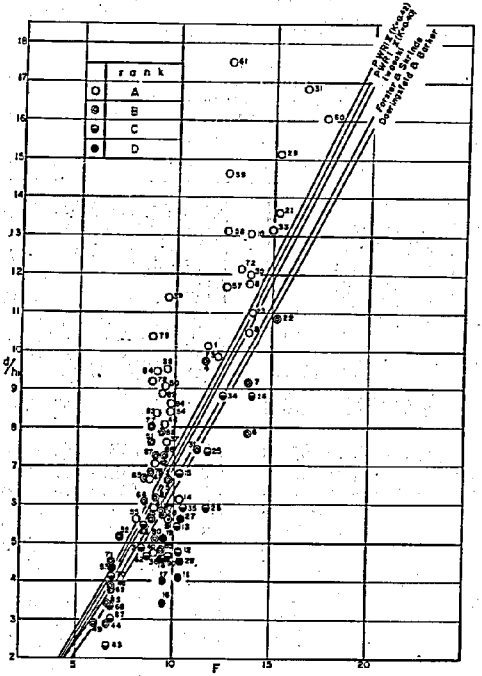


Fig. 10

Table 5-1

Dam no.	Height (m)	Design Flood (m³/s)	Crest B(m) / ±H(m) / ±N	Conduit B (m) / ±H (m) / ±N	Arrangement: Crest / Conduit	Breadth of Selfing Beam B (m)	Case no.	Discharge Q (m³/s)	Discharge for Unit Breadth q (m²/s/m)	Head H (m)	Inflow Velocity V1 (m/s)	Inflow Depth h1 (m)	Inflow Froude Number	Height of Endsill d (m)	d/h1	Rank: Profile	Rank: Pressure	Rank: Sediment	Rank: Total	Case no.
1	57.0	431	12.0 ±0.3 ±1	—		12.0	1	200	16.67	32.8	28.12	0.593	12.67	6.0	10.118	A	C	A	B	1
							2	100	23.00		28.66	0.872	9.80		6.881	B	C	B	C	2
							3	431	35.92	51.6	29.07	1.236	8.35		4.854	C	C	C	C	3
2	81.0	338	6.2 ±7.75 ±2	—		13.0	4	200	15.38	43.0	27.18	0.566	11.54	5.5	9.717	B	I	B	D	4
							5	300	23.08		27.69	0.833	9.69		6.603	D	I	D	D	5
3	106.0	4700	31.4 ±10.0 ±6	—		96.0	6	2505	26.04	99.0	37.75	0.766	13.17	6.0	7.65	B	I	B	B	6
							7							7.0	9.14	B	I	B	B	7
							8							8.0	10.44	A	I	A	A	8
							9							9.0	11.73	A	I	A	A	9
							10							10.0	13.08	A	I	A	A	10
							11	3000	57.87	99.0	39.13	1.479	10.28	6.0	4.06	D	I	C	C	11
							12							7.0	4.73	D	I	C	C	12
							13							8.0	5.41	D	I	C	C	13
							14							9.0	6.09	D	I	C	C	14
							15							10.0	6.75	D	I	C	C	15
							16	6000	69.44	99.0	39.44	1.761	9.49	6.0	3.41	D	I	D	D	16
							17							7.0	3.98	D	I	D	D	17
							18							8.0	4.54	D	I	D	D	18
							19							9.0	5.11	B	I	D	D	19
							20							10.0	5.68	B	I	D	D	20
4	64.0	460	8.0 ±5.6 ±2	4.4 ±4.4 ±1		22.6	21	240	10.71	50.8	29.09	0.368	15.31	5.0	13.59	A	A	A	B	21
							22							4.0	10.87	B	A	—	A	22
							23	300	13.39	50.8	28.51	0.454	13.89	5.0	11.01	A	C	—	B	23
							24							4.0	8.81	C	C	—	C	24
							25	450	20.54	50.8	30.22	0.680	11.71	5.0	7.35	C	C	—	C	25
							26							4.0	5.88	C	C	—	C	26
							27	610	27.23	50.8	30.63	0.889	10.38	5.0	5.62	C	D	—	D	27
							28							4.0	4.50	C	D	—	D	28
5	87.0	620	5.0 ±6.8 ±2	4.0 ±3.8 ±1		21.0	29	350	16.67	72.2	33.68	0.495	13.29	7.5	15.15	A	A	B	A	29
							30	450	21.43	81.8	34.23	0.626	13.82		11.36	A	A	B	A	30
							31	730	34.71	81.8	35.19	1.015	11.16		7.58	A	B	C	B	31
6	55.9	515	8.0 ±7.2 ±1	4.4 ±4.4 ±2		32.0	32	230	7.19	53.6	26.90	0.267	16.62	4.0	16.81	A	B	—	A	32
							33	320	9.38		27.41	0.342	14.87		12.16	A	B	—	B	33
							34	480	14.38		28.13	0.511	12.57		8.81	B	C	—	C	34
							35	700	21.88		28.75	0.768	10.83		5.91	C	C	—	C	35
							36	920	28.75		29.08	0.989	9.34		4.33	C	D	—	D	36

Table 5-2

Dam no.	Height (m)	Design Flood (m³/s)	Crest B(m) / ±H(m) / ±N	Conduit B(m) / ±H(m) / ±N	Arrangement: Crest / Conduit	Breadth of Stilling Basin B (m)	Case no. Fig 10	Discharge Q (m³/s)	Discharge for Unit Breadth q (m²/s/m)	Head H (m)	Inflow Velocity V1 (m/s)	Inflow Depth h1 (m)	Inflow Froude Number	Height of Endsill d (m)	d/h1	Rank: Profile	Rank: Pressure	Rank: Sediment	Rank: Total	Case no.
7	30.0	1700	10.4 ±7.8 ±3	4.0 ±4.0 ±2		43.6	37	500	11.47	39.0	21.76	0.527	9.58	4.0	7.59	A	—	—	A	37
							38							3.0	5.69	A	—	—	A	38
							39							5.0	9.49	A	—	—	A	39
							40	960	22.02	42.4	23.84	0.865	8.98	4.0	4.71	A	—	—	A	40
							41							5.0	5.89	A	—	—	A	41
							42							6.0	7.07	A	—	—	A	42
							43	2040	46.79	44.4	27.17	1.722	6.81	4.0	2.32	C	—	—	C	43
							44							5.0	2.90	B	—	—	B	44
							45							6.0	3.48	B	—	—	B	45
8	50.0	1350	9.0 ±12.5 ±3	4.5 ±4.5 ±2		48.0	46	700	14.58	35.0	23.45	0.622	9.50	5.0	8.04	A	B	—	A	46
							47	1210	27.71		23.63	0.750	8.72		6.67	A	B	—	B	47
							48	1700	31.33		24.09	1.297	8.76		5.08	B	B	—	B	48
							49	2000	41.67	41.0	24.29	1.716	8.92		4.29	B	B	—	B	49
9	77.0	3530	8.4 ±13.0 ±2	6.2 ±6.3 ±3		50.0	50	1060	40.00	69.0	32.91	1.215	9.54	11.0	9.05	A	—	—	A	50
							51	1460	40.00		33.15	1.448	6.80		7.61	A	C	—	B	51
							52	2360	71.20	67.5	33.29	2.139	7.27		5.14	B	C	—	B	52
							53	3345	66.80	70.5	34.18	2.548	6.85		4.33	C	D	—	D	53
10	83.0	2330	7.5 ±7.0 ±4	6.7 ±5.0 ±3		59.0	54	520	19.74	77.0	33.31	1.193	9.74	10.0	8.38	—	A	—	A	54
							55	1310	50.26		33.86	1.780	8.11		5.62	—	B	—	B	55
							56	1820								—	B	—	B	56
11	83.0	3800	8.0 ±11.1 ±2 12.0 ±12.1 ±2	4.4 ±4.4 ±3		49.0	57	1120	22.45	72.0	32.79	0.685	12.66	8.0	11.68	A	—	—	A	57
							58							9.0	13.14	A	—	—	A	58
							59							10.0	14.60	A	—	—	A	59
							60							11.0	16.06	A	—	—	A	60
							61							12.0	17.52	A	—	—	A	61
							62	2770	51.83		34.20	1.693	9.59	8.0	4.64	B	I	—	B	62
							63							9.0	5.22	B	I	—	B	63
							64							10.0	6.05	B	I	—	B	64
							65							11.0	6.96	B	I	—	B	65
							66							12.0	7.45	B	I	—	B	66
							67	3560	93.06		34.78	2.676	6.78	8.0	2.99	C	I	—	C	67
							68							9.0	3.36	C	I	—	C	68
							69							10.0	3.74	B	I	—	B	69
							70							11.0	4.11	B	I	—	B	70
							71							12.0	4.49	B	I	—	B	71

Table 5-3

Dam no.	Height (m)	Design Flood (m³/s)	Crest B (m)	Conduit B (m)	Arrangement	Breadth of Stilling Basin B (m)	Case no.	Discharge Q (m³/s)	Discharge for Unit Breadth q (m²/s)	Head H (m)	Inflow Velocity v₁ (m/s)	Inflow Depth h₁ (m)	Inflow Froude Number	Height of End sill d (m)	d/h₁	Rank Profile	Rank Pressure	Rank Regimen	Rank Total	Case no.
12	100.0	4400	31.0	3.6		60.0	72	1600	26.67	69.0	55.10	0.742	13.34	9.0	12.13	A	-	B	A	72
			× 2	4.8			73	2000	33.33		56.40	0.916	12.16		9.83	A	A	C	A	73
			× 5	6.4			74	4400	73.33		57.71	1.945	8.64		4.63	C	C	C	C	74
13	100.0	6400	18.0	3.0		75.0	75	2900	38.67	60.0	50.91	1.251	8.83	7.0	5.60	A	B	C	B	75
			× 10.0	25.6			76							8.5	6.80	A	B	B	B	76
			× 4	5.5			77							10.0	7.99	A	B	B	B	77
							78							11.5	9.16	A	B	A	A	78
							79							13.0	10.39	A	B	A	A	79
							80	3440	45.87	70.0	53.30	1.377	9.06	7.0	5.08	B	B	C	B	80
							81							8.5	6.17	A	B	B	B	81
							82							10.0	7.26	A	B	B	A	82
							83							11.5	8.35	A	B	B	A	83
							84							13.0	9.44	A	B	B	A	84
							85	3900	52.00	80.0	55.50	1.463	9.37	7.0	4.78	B	B	C	B	85
							86							8.5	5.80	B	B	B	B	86
							87							10.0	6.83	A	B	B	A	87
							88							11.5	7.85	A	B	A	A	88
							89							13.0	8.87	A	B	A	A	89
							90	4260	56.80	90.0	57.51	1.515	9.74	7.0	4.62	D	B	C	C	90
							91							8.5	5.61	B	B	B	B	91
							92							10.0	6.61	B	B	B	B	92
							93							11.5	7.60	A	B	B	B	93
							94							13.0	8.59	A	A	B	A	94

5. A NEW EXPERIMENT PROCEDURE OF HYDRAULIC JUMP DISSIPATOR

The experiment procedure mentioned above is a procedure of experiments carried out according to the classic depth measuring method (point gauge). Now a new expriment procedure which has been used as a trial in the writer's laboratory will be reviewed below. This procedure will be materialized by the block diagram illustrated in Figure 11.

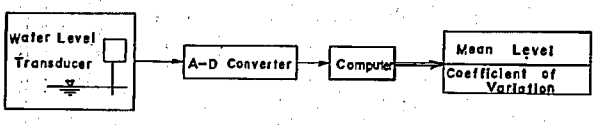


Fig. 11

In the classic method, only an inaccurate average value of water depth was obtained, but if the above-mentioned new experiment procedure is adopted, a considerably accurate time average value and fluctuation of water depth can be measured. Accordingly, the stream regimen shown in Figure 12 can be expressed by the diagram shown in Figure 13. Here, it is empirically known that if a hydraulic jump dissipator of a scale of about 1/50 can perform perfect dissipation, the coefficient of variation (C.V.) in the flow directry above the end sill is less than 5 percent.

This new experiment procedure can be widely used for the quantitative expression of stream regimen obtained in the experiment using a hydraulic model which has a plane expansion (for instance, a model having a completed plane shape such as the environs of a water gate, diversion point and confluence point).

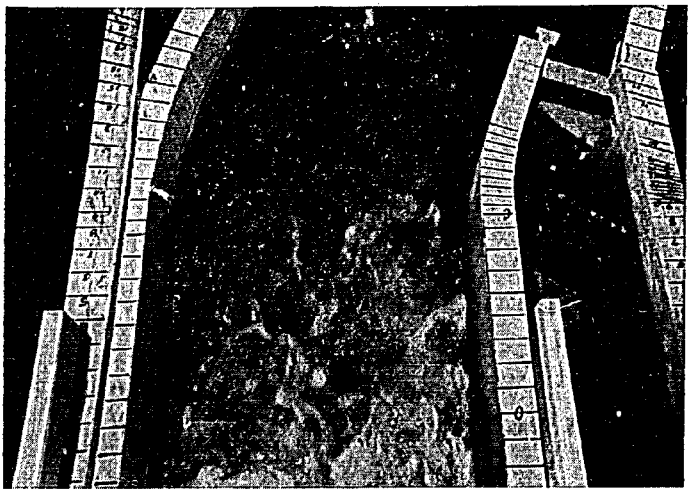


Fig. 12

Then an experiment using as apparatus as shown in Figure 14 was carried out to check how C.V. could vary with the difference in the scale. Namely, in Figure 14, all the lengths are determined by F and h_1; therefore, if F is assumed to be constant, h_1 will be the representative value showing the magnitude of the apparetus. When C.V. (*) is measured by employing a model, keeping that F is constant and changing h_1, we obtain Figure 15.

(*) The representative value of C. V. of the entire hydraulic jump is obtaine das follows : The jump length is divided into 10 equal parts and the C. V.'s of the ten parts have been measured, and then the average value of C. V.'s of five measureung points at the lower stream from the center point has been teken as the representative C. V. of the experiment case.

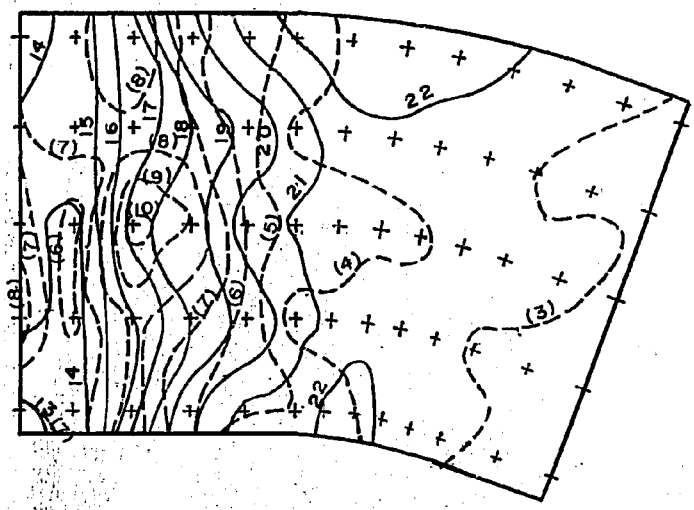

14
15
16
17
18
19
20
21
22
(8)
(7)
(6)
(5)
(4)
(3)
(2)
(1)
Mean Depth (cm) , Profondeur Moyenne (cm)
C. V. (%) , C. V. (%) .

Fig. 13

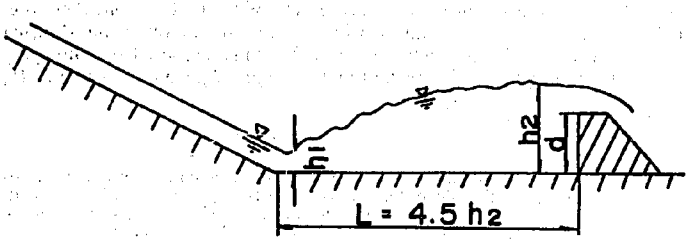

h1
h2
d
L = 4.5 h2

Fig. 14

From Figure 15, it is observed that as a rough figure of $h_1 > 2$ cm is obtained, C.V. will decrease. This may be attributable to viscosity and surface tension. In the model experiment, therefore, it is desirable that the septh of supercritical inflow be made about 2 cm or more.

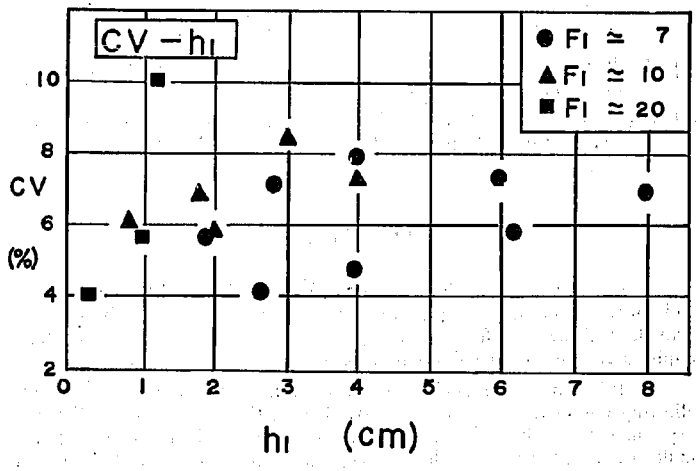


Fig. 15

CONCLUSION

By employing the above mentioned experiment procedure, it has become possible to establish objective critaria for hydraulic jump dissipators whose evaluation in the past was mingled with a great deal of subjective judgement. In future the measuring method mentioned in Chapter 5 will be further developed.

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SUMMARY

A number of methods are known for calculating the height of an end sill in hydraulic jump type dissipator work. However, such methods provide a sill height which is actually in shortage for complete dissipation required in most of the dams in Japan.

In this paper, a formula for calculation of the correct sill height allowing complete dissipation is derived from the data obtained through experiments with hydraulic models. In the process of producing the formula, a concept of ranking the flow is proposed for representing the extent of dissipation in the experiments of hydraulic jump type dissipator models. In the last, experiments using electric methods for measurement of the water level are introduced.

RÉSUMÉ

Il existe un grand nombre de méthodes pour calculer la hauteur du seuil aval pour un dissipateur d'énergie hydraulique du type à saut de ski. Toutefois, ces méthodes déterminent une hauteur de seuil qui ne convient pas pour une dissipation complète telle qu'elle est requise dans la plupart des barrages au Japon.

Dans ce rapport on détermine une formule pour calculer la hauteur de seuil correcte qui permet une dissipation complète, à partir de données obtenues dans des expériences sur modèles réduits. Pour établir cette formule on propose, pour représenter le degré de dissipation dans les expériences sur des modèles réduits hydrauliques de dissipateurs à sauts de ski un système de classement des débits. Enfin, on présente des expériences utilisant des méthodes électriques pour mesurer les niveaux d'eau.