EARTHQUAKE RESISTANT DESIGN FEATURES OF SUBMERGED TUNNELS IN JAPAN

CONTENTS

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I. INTRODUCTION

Construction of submerged tunnels has a long history of about 90 years, and yet, those tunnels have been undamaged by earthquakes until now. In recent decades, however, there has been a succesive and increasing demand for expansion of traffic networks in Japan, and this was followed by planning and construction of many submerged tunnels with a fairly great length. Since Japan is located in one of active seismic zones in the world, great many surveys and researches on earthquake resistance of the submerged tunnel have been carried out from various points of view. Those works clarified that the tunnel of this type has dynamic characteristics different from those of a structure on the ground surface.

In order to establish a rational method for earthquake-resistant design of a submerged tunnel, Japan Society of Civil Engineers organized a committee in 1971, and continued the survey and the examination related to the subject. In 1975, "Specification for Earthquake Resistant Design of Submerged Tunnels" was completed synthesizing the fundamental matters for a reasonable design.

The main part of this chapter is addressed to the description of this specification. Also mentioned here are the outlines of some representative submerged tunnels in this country, and of the latest researches on earthquake resistance of the submerged tunnels. With new experiences and knowledges obtained through these constructions and actively ongoing researches, it is expected that the improvement of the specification will be discussed in future.

2. SUBMERGED TUNNELS IN JAPAN

In Japan, the first submerged tunnel was completed in Ohsaka in 1944 for a subway system. By March, 1988, 18 submerged tunnels have been completed. The locations, dimensions and periods for construction of these tunnels are listed in Table 1 together with those under planning. Out of the latest tunnels completed, four representative tunnels with different types are selected to be described in more details in the following sections.

2.1 KEIYO-LINE HANEDA TUNNEL (TAMA RIVER)

The Keiyo-line Haneda Tunnel (Tama River) is a railway tunnel crossing the Tama River which is a borderline between Tokyo and Kawasaki City. This tunnel is 6393 m in lingth. Construction of this tunnel started in 1968 and ended in 1970. During the construction, the cut and cover method, shield driven method, and the caisson method were employed as the trench method was, regarding the conditions of construction sites. Near the Morigasaki canal, the trench method was employed, and this part leads to the submerged structure under the Tama River. The submerged structure is the central portion of the whole tunnel with the length of 6393 m. This part will be written as the "Tama

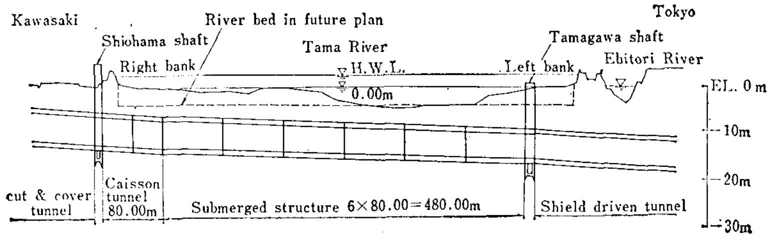


Fig. 1

River Tunnel" hereafter in this section.

The longitudinal section of the Tama River Tunnel is shown in Fig. 1. The tunnel leads, at its south end, to a part of 80 meter length constructed by the caisson method. Next to this part there exists a shaft in the right bank of the Tama River. The tunnel is connected, at its north end, to a shaft in the left Bank. A part of the tunnel beyond the shaft was constructed by the shield driven method. In plan the Tama River Tunnel has a curved alignment.

The typical cross section of the tunnel is illustrated in Fig. 2; and as it is seen in the figure the tunnel is used for a double track railway. The tunnel continues, at its north end, to two separate tunnel sections constructed by the shield driven method; each section carries single track of the railway. Therefore, the element of the north end has a gradually changing cross section from a binocular section to a section of two circules.

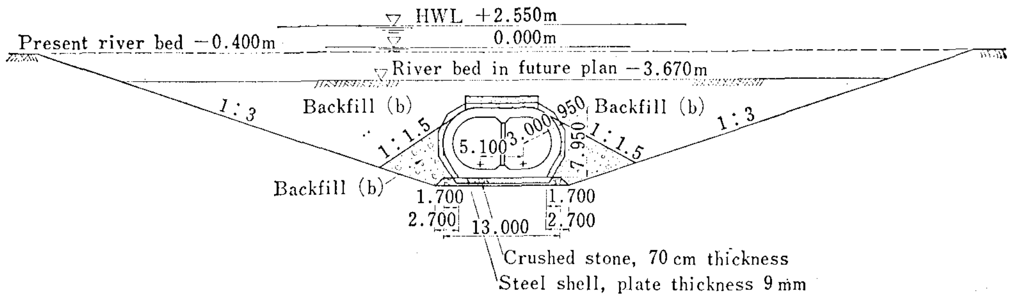


Fig. 2

Length of each element is 80 meters. Steel shells for the elements were fabricated in a dock, floated to the temporary quay, and there the reinforce concrete linings were executed. Weight of each element is about 6950 tons.

The joint between the elemnts has alsmost similar sectional rigidity to that of the intermediate section of the element.

2.2 KINUURA PORT TUNNEL

The Kinuura Port Tunnel is located about 270 kilometer west of Tokyo. It is a two lane vehicular tunnel with a walker lane linking both shores of Kinuura Bay. The longitudinal section and the cross section are shown in Figs. 3 and 4, respectively. The

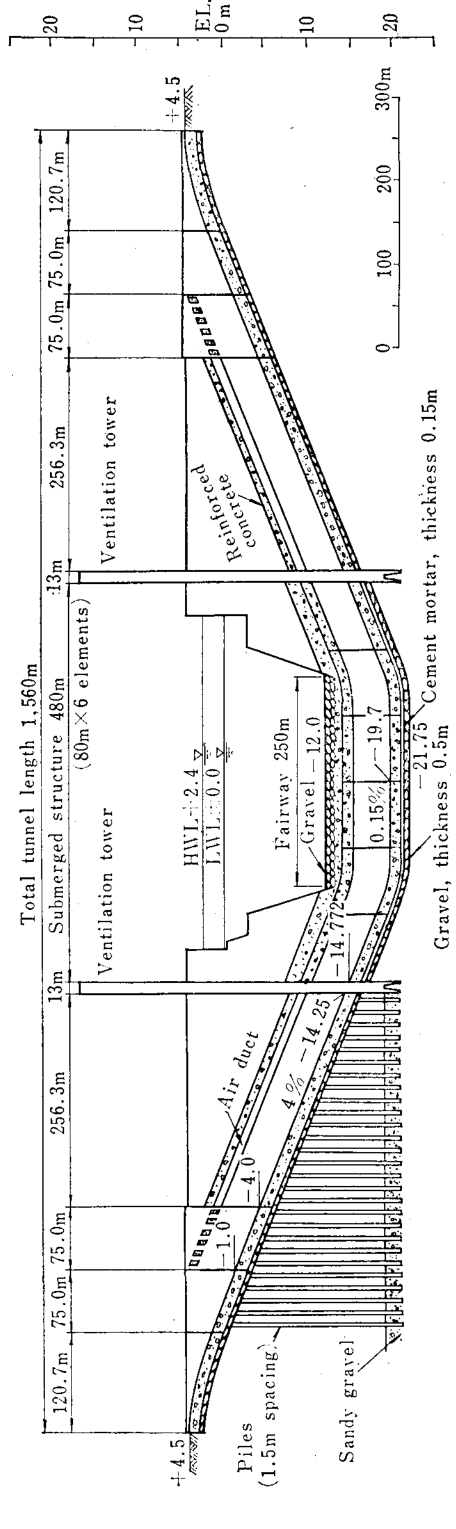

Total tunnel length 1,560m
Submerged structure 480m
(80m×6 elements)
120.7m
75.0m
75.0m
256.3m
13m
Ventilation tower
Air duct
Reinforced concrete
HWL +2.4
LWL ±0.0
Fairway 250m
Gravel −12.0
0.15%
−19.7
−21.75
−14.772
4%
−14.25
−4.0
−1.0
+4.5
Piles
(1.5m spacing)
Sandy gravel
Gravel, thickness 0.5m
Cement mortar, thickness 0.15m
EL. 0 m
10
20
0
100
200
300m

Fig. 3

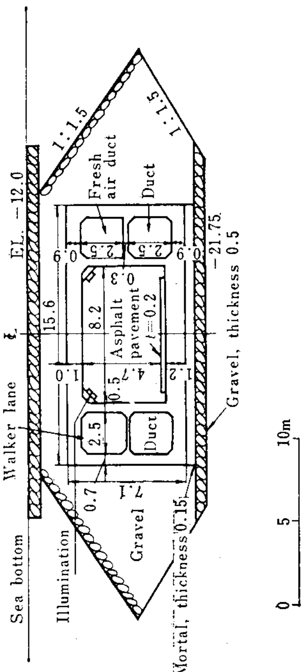

Sea bottom
Illumination
Walker lane
EL. −12.0
1:1.5
Fresh air duct
Duct
15.6
8.2
Asphalt pavement t=0.2
2.5
0.9
1.0
1.2
0.5
0.3
4.7
0.7
7.1
Gravel
Mortal, thickness 0.15
Gravel, thickness 0.5
−21.75
0
5
10m

Fig. 4

length of the tunnel constructed by the trench method is 480 meters and both ends are connected to the ventilation towers which were constructed by the pneumatic caisson method. The tunnel alignment in plan is straight.

The length of each element is 80 meters. The element consists of an exterior steel shell fabricated in a dock and a reinforced concrete lining executed in the vicinity of the site. The sectional rigidity is almost uniform through the tunnel elements and the joints between the elements. The weight of each elements is approximately 9000 tons.

The construction started in 1969 and the tunnel was completed in 1973.

2.3 TOKYO PORT TUNNEL

The Tokyo Port Tunnel is a part of a highway along Tokyo Bay. It is a six lane vehicular tunnel crossing the first fairway in Tokyo Port. The longitudinal section is shown in Fig. 5 and the typical cross section is in Fig. 6. The tunnel constructed by the trench method is 1035 meters in length, being connected at its both ends to the ventilation towers. The tunnel alignment in plan is straight.

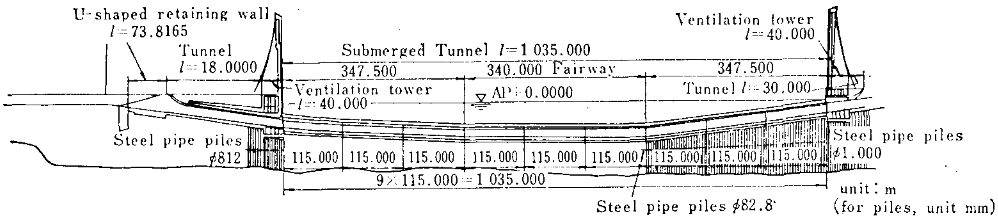


Fig. 5

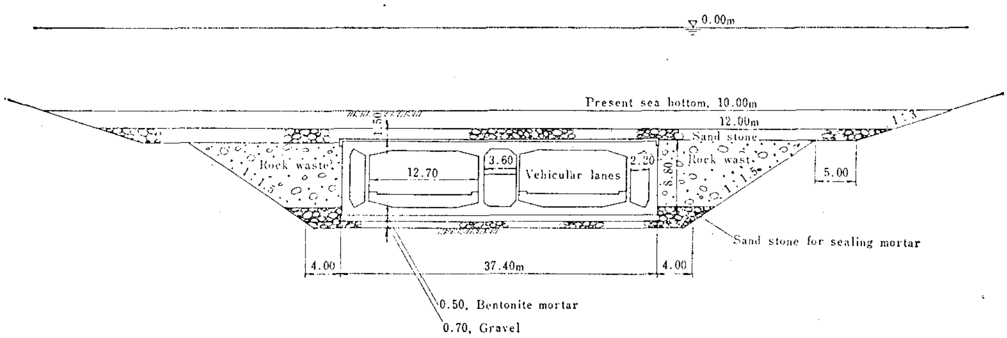


Fig. 6

Each element has 115 meter length and nine elements were used. All the elements were fabricated almost at a time in a temporary dock which was constructed at about 4 kilometers from the tunnel site. No steel shell was used for the element; for watertightening, however, exterior faces of side walls and bottom are covered with steel plates of 6 milimeter thickness. The weight of each element is about 38,000 tons.

The construction started in 1970 and completed in 1976.

Table 1 Submerged tunnels in Japan, completed and under construction

Name	Location	Use	Total length (m)**	Elements					Period for construction
				Number	Length (m)	Width (m)	Height (m)	Sectional shape	
Aji River Tunnel	Osaka	2 lane road	49.2	1	49.2	14.0	7.0	Rectangular	1935-1944
Haneda Tunnel	Tokyo	4 lane road	56	1	56	20.1	7.4	Rectangular	1963-1964
Haneda Tunnel	Tokyo	2 track monorail	56	1	56	10.95	7.4	Rectangular	1963-1964
Dojima River Tunnel	Osaka	2 track railway	70.5	2	36.0 34.5	11.00	7.78	Ractangular	1967-1969
Dotonbori River Tunnel	Osaka	2 track railway	24.5	1	24.9	9.65	6.96	Rectangular	1967-1969
Keiyo-line Haneda Tunnel (Tama River)	Tokyo	2 track railway	480	6	80	13.0	7.95	Binocular	1968-1970
Keiyo-line Haneda Tunnel (Keihinunga)	Tokyo	2 track railway	328	4	82	12.74	7.99	Binocular	1969-1971
Atsumi Power Plant Intake Channel	Aichi Prefecture	Cooling water intake channel for power plant	36.01	1	36.01	8.4	4.0	Rectangular	1970
Dokai Bay Tunnel	Kita-Kyushu	2 belt conveyers for iron ore and coke	1363.2	18	30 80.1	8.218	4.55	Rectangular	1970-1972
Kinuura Port Tunnel	Aichi Prefecture	2 lane road	480	6	80	15.6	7.1	Rectangular	1969-1973
Ohgishima Tunnel	Kawasaki	4 lane road	664.3	6	110	21.6	6.9	Rectangular	1971-1975
Tokyo Port Tunnel	Tokyo	6 lane road	1035	9	115	37.4	8.80	Rectangular	1970-1976
Sumida River Tunnel	Tokyo	2 track railway	201.5	3	67.5 67.0	10.30	7.80	Rectangular	1973-1976
Kawasaki Port Tunnel	Kawasaki	4 lane road	840	8	110 100	31.00	8.54	Rectangular	1972-1979
Tokyo Port 2nd Fairway Tunnel	Tokyo	4 lane road	744	6	124	28.40	8.80	Rectangular	1973-1980
Keiyo-line Daiba Tunnel	Tokyo	2 track railway	672	7	96	12.8	8.0	Binocular	1976-1980
Dokai Bay Tunnel	Kita-Kyushu	for gas pipes	434	9 1	45 26.5	3.2	3.2	Circular	1976-1977
Keihin Minami Canal Tunnel	Tokyo	1 belt conveyer for mud haulage	120	3	40	4.6	4.1	Rectangular	1980-1981
Tama River Tunnel	Tokyo-Kawasaki	6 lane road	1549.5	12	128.6	39.9	10.0	Rectangular	1986~
Kawasaki Fairway Tunnel	Kawasaki	6 lane road	1187.4	9	131.2	39.7	10.0	Rectangular	1986~
Osaka South Port Tunnel*	Ohsaka	4 lane road & 2 track railway	1025	10	102.5	34.8	8.8	Rectangular	1990~
Niigata Port Road Tunnel*	Niigata	4 lane road	850	8	107.5 110.5	28.6	8.75	Rectangular	1989~

* Tentative English names
** Total length of submerged structure part

2.4 KAWASAKI PORT TUNNEL

Kawasaki Port Tunnel is located in Tokyo Bay. The tunnel crosses the Keihin canal and links Chidori-cho with Ohgishima which is the manmade island. The longitudinal section and cross section are shown in Fig. 7 and Fig. 8, respectively. The length of the tunnel constructed by the trench method is 840 m and this part consists of eight elements. As the tunnel is constructed in soft clay layer, non-uniform settlement was considered in design process. Therefore, the land tunnel of Ohgishima is supported by steel pipe piles.

Two ventilation buildings are located on both shores and away from the tunnel. Each

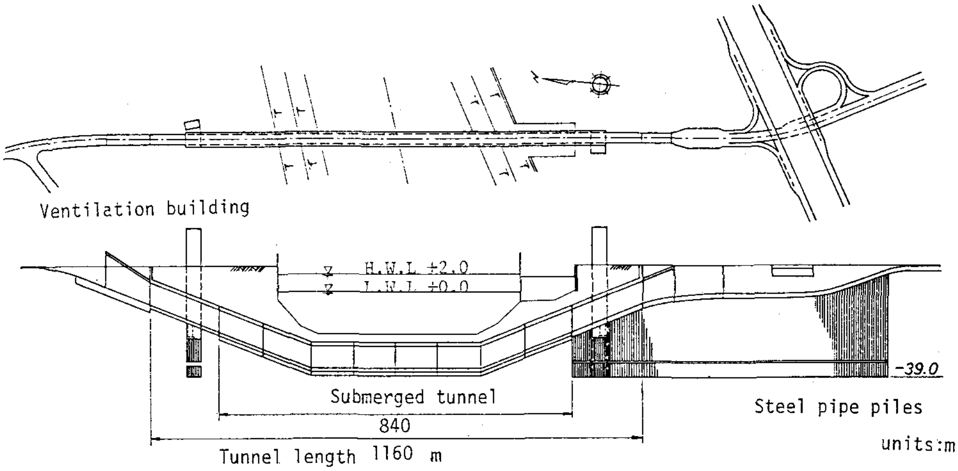


Fig. 7 Longitudinal Section

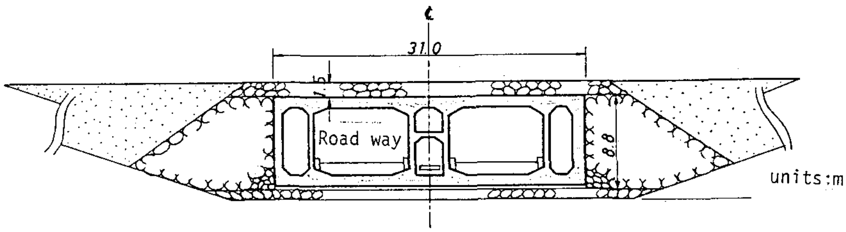


Fig. 8 Cross Section

element was fabricated by steel type method. The steel shells were built in the dock, and towed out onto the sea. Then, after joining reinforcing bars, placing of concrete was carried out at sea. Large reinforcing bars with the diameter of 51 mm were adopted for longitudinal bars to resist axial force during earthquakes.

The construction was completed in 1980 and various kinds of observations including measurement of earthquake response have been being carried out since the completion.

3. SPECIFICATIONS FOR EARTHQUAKE RESISTANT DESIGN OF SUBMERGED TUNNELS (JSCE-1975)*

This chapter describes the abridged specifications for earthquake resistant design of submerged tunnels (JSCE-1975). Though the specifications have not been revised since 1975, minor changes of expression were made within the limited pages. Complete translation is available in the last edition (1988).

* Although these specifications have not been revised since the last edition of this book, English translation here have been improved including corrections of minor mistakes.

3.1 GENERAL

3.1.1 Scope (omitted)

3.1.2 Related specifications (omitted)

3.1.3 Definitions of Terms (omitted)

3.2 SURVEYS

3.2.1 General (omitted)

3.2.2 Investigations on Earthquakes and Earthquake Ground Motions

3.2.2.1 Earthquakes (omitted)

3.2.2.2 Earthquake Ground Motions (omitted)

3.2.3 Investigation on Geology and Ground

3.2.3.1 General (omitted)

3.2.3.2 Preliminary Investigation (omitted)

3.2.3.3 Site Investigation (omitted)

3.2.3.4 Procedures of Tests (omitted)

3.2.3.5 Configuration and Depth of Bedrock (omitted)

3.2.4 Investigation on Engineering Properties of Grounds and Soils

3.2.4.1 Engineering Properties of Grounds and Soils (omitted)

3.2.4.2 Strength of Soils under Dynamic Loading (omitted)

3.2.4.3 Measurement of Seismic Wave Velocities (omitted)

3.2.4.4 Measurements of Dynamic Characteristics of Grounds (omitted)

3.2.4.5 Damping Factor of Surface Layer (omitted)

3.2.4.6 Strain-dependent Properties of Soils (omitted)

3.2.5 Ground Failure during Earthquakes (omitted)

3.2.6 Seismic Stability of Soils

3.2.6.1 Stability of Sandy Soils (omitted)

(1) Liquefaction potential estimation in design of port structures (omitted)

(2) Liquefaction potential estimation in design of highway bridges (former specifications, 1971) (omitted)

3.2.6.2 Stability of Cohesive Soils (omitted)

3.2.7 Investigation on Materials and Structures of Submerged Tunnels (omitted)

3.2.8 Investigation on Safety Countermeasure against Earthquakes (omitted)

3.3 EARTHQUAKE RESISTANT DESIGN

In earthquake resistant design of submerged tunnels, stability of the tunnel shall be checked by the seismic coefficient method and the seismic deformation method for the individual parts. The submerged tunnels shall be also checked by the dynamic analysis for the total structural system including effects of topography and geology at proposed tunnel sites, in accordance with the provisions in "3–4 Dynamic Analysis".

Further, it is necessary to ensure safety of the tunnels by installing safety equipments which are necessary for the tunnel in earthquakes, in accordance with the provisions in "3–5 Safety Countermeasure against Earthquakes" and by executing maintenance opera-

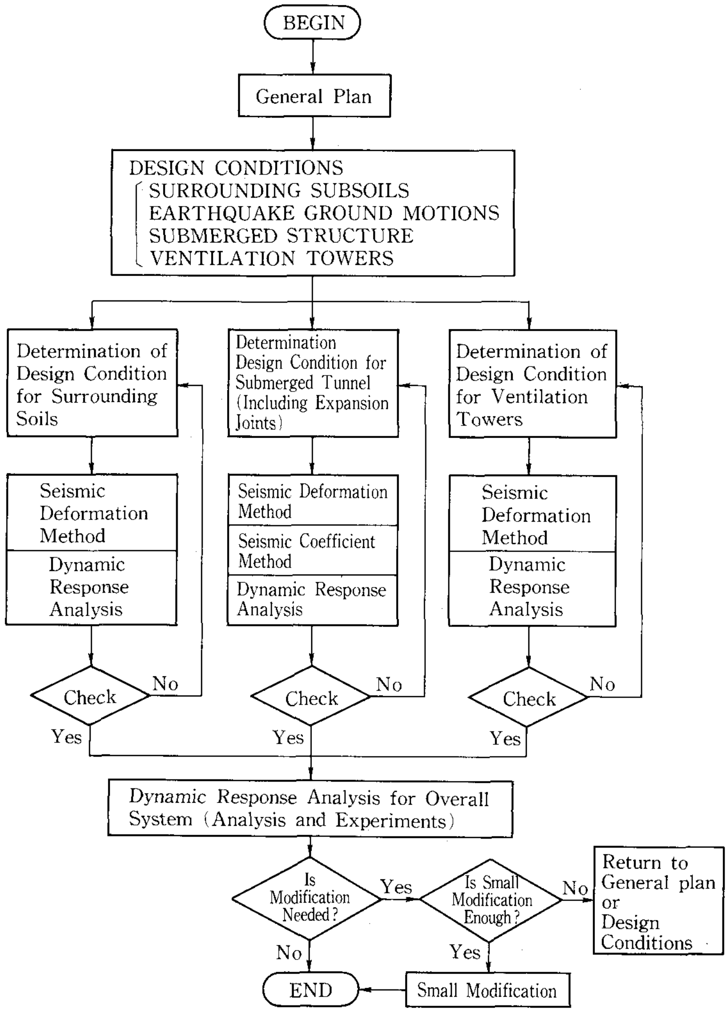


Fig. 9 Flowchart for Seismic Design

tion and inspection of the tunnel.

An example of the above-mentioned system of earthquake resistant design is shown in a form of a flowchart in Fig. 9. In this figure, the term "planning conditions" means traffic utilization conditions and the conditions of traffic lane composition and of the approximate route. The term "design conditions" means the conditions of design earthquakes, the ground and the tunnel which are related to the structural design, but does not include numerical values such as dimensions which are necessary for design calculations. The term "design data" means numerical values such as dimensions which are necessary for the design and are to be determined by the design conditions.

3.3.2 Basic Principles of Earthquake Resistant Design

This section indicates a basic principle of the earthquake resistant design of submerged tunnels. As mentioned in "3.3.1 General", the submerged tunnel is a structure subjected to various kinds of special condtions, and its design requires particular consideration for earthquake resistance as follows:

(1) In selecting routes and determining lengths of the tunnel and positions of its ventilation towers at design stage, it is recommendable to carry out thorough examinations of the topography and geology of the sites as well as the overall structural system and to avoid an extremely soft ground, an area that is liable to cause ground failure such as landslides, and a portion where discontinuities in topography and geology exist. It is also recommended to avoid discontinuities in the structure itself.

(2) In earthquake resistant design of the submerged structure, either the displacement of ground or the design seismic coefficient shall be taken into account, and in case of the ventilation tower and other appurtenant structures, the design seismic coefficient shall be considered. In addition to the designs of the abovementioned parts of the tunnel, it is also neccessary to design and examine the total structural system, by conducting the dynamic analysis.

(3) In designing portions which have considerable structural discontinuities such as joints between elements and joints between the tunnel and the ventilation towers (or approaches), examination shall be given in respect to influences and effects on earthquake resistance, particularly for their waterproof abilities before and after earthquakes.

(4) To ensure seismic safety of the submerged tunnel, practical provisions shall be made for maintenance and countermeasures against earthquakes—in addition to structural design—and consideration shall be given both to machinery and electrical equipments necessary for the maintenance and countermeasures and to control systems for properly operating such machinery and equipments.

3.3.3 Loads and Conditions in Earthquake Resistant Design

3.3.3.1 General

(1) In earthquake resistant design, the following loads in addition to the effects of earthquakes shall be taken into consideration. The kinds of loads to be employed shall be selected properly.

a) Dead loads

b) Earth pressures

c) Hydrostatic pressures
d) Buoyance or uplifts
e) Live loads
f) Effects of consolidations and settlements of subsoils
g) Effects of temperature change
h) Effect of shrinkage of concrete due to humidity
i) Other loads (tsunami, wave force, sunken ship and forsaken anchor)

(2) As the effects of earthquakes, consider the following:
a) Deformation of subsoils and structures during earthquakes
b) Inertia forces arising from the dead weight of structure
c) Earth pressures during earthquakes
d) Hydrodynamic pressures during earthquakes

(3) Combination of loads

For combining loads during earthquakes, consider the loads in Items a) to g) of (1) in general and consider also Items h) and i) depending upon the conditions at the construction site.

3.3.3.2 Ground Displacement during Earthquakes

(1) The displacement of the ground surface where the submerged structure is burried shall be determined in accordance with the provisions in "3.3.4.2.1 Ground Displacement in Earthquake Resistant Design." The ground displacement to be adopted in the design of the submerged structure shall be the ground displacement at the level of the longitudinal axis of the submerged structure.

(2) The horizontal and vertical planes which include the logitudinal axis of the submerged structure shall be considered.

There is no need of treating the submerged structure as being excited by the ground and causing resonance with it during earthquakes. The tunnel moves according to the motion of the ground. Therefore, if it is possible to determine the displacement mode of the natural ground during earthquakes, the tunnel can be treated as a beam which is connected to the ground through springs. The term "longitudinal axis" of the submerged structure means the central line in the height and the width of the element.

When the submerged tunnel is buried shallow near the ground surface, the bottom plane

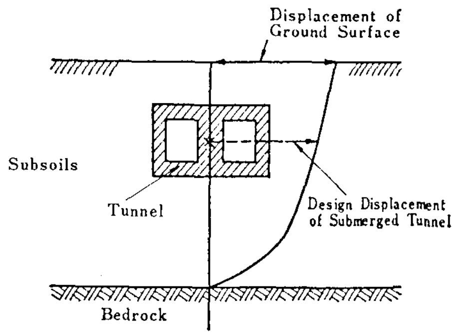


Fig. 10 Design Ground Displacement

of the tunnel is positioned at a depth of several to ten meters from the ground surface. Therefore, the displacement of the ground at each position of the tunnel differs from that on the ground surface. In this section, it is stipulated that displacement at the position of the longitudinal axis may be employed (refer to Fig 10).

In general, it is desirable that all the planes that include the point under consideration are considered, but in the design of the submerged tunnel, the horizontal and vertical planes are assumed as principal planes.

3.3.3.3 Inertia Forces

(1) Inertia forces due to earthquakes can be obtained by multiplying the dead weight of structure and soil mass by the design seismic coefficient which is stipulated in "3.3.4.2.3 Design Seismic Coefficient in Seismic Coefficient Method."

(2) The position on which inertia forces act shall be the position of the center of gravity of the dead weight, and the direction of the inertia forces shall, in principle, be two horizontal directions and one vertical direction.

Designs which are made by using inertia forces include the stability calculation of the sliding of ground, design of ventilation tower, design of approach and design of appurtenant facilities. In addition to the above, it is allowed in these Specifications that the cross section of the submerged structure can be designed by using inertia forces.

Since appurtenant facilities installed inside the tunnel and the ventilation tower are liable to have large inertia forces, it is necessary to increase the design seismic coefficient which is stipulated in "3.3.4.2.3 Design Seismic Coefficient in the Seismic Coefficient Method". In such a case, the extra factor shall be determined according to each situation with an upper limit of two times, and more or less two times shall be adopted. In case of the tunnel which is used for vehicle traffic, no inertia forces arising from the weight of live loads are required to consider.

The "two horizontal directions" mean, in ordinary cases, the axial direction and the direction perpendicular to the axis.

3.3.3.4 Earth Pressures during Earthquakes

(1) Horizontal earth pressures during earthquakes shall be determined by Mononobe and Okabe's earth pressure formula.

(2) Overburden pressures on the upper plane of the submerged tunnel shall be determined by multiplying the weight of overburden soils by $(1 \pm kv)$, where kv is the vertical design seismic coefficient.

3.3.3.5 Hydrodynamic Pressures during Earthquakes

(1) Hydrodynamic pressures during earthquakes which act to walls shall be determined in accordance with provisions in Revised Design Standard of Dam (Japan National Committee on Large Dams, 1971)

(2) Hydrodynamic pressures during earthquakes which act to columns shall be determined in accordance with provisions in Specifications for Earthquake Resistant Design of Honshu-Shikoku Bridges (Japan Society of Civil Engineers, 1967).

The hydrodynamic pressures during earthquakes shall be taken into consideration in the design when the ventilation towers are constructed in the water.

3.3.3.6 Soil Layers Where Shear Strength is Neglected in Earthquake Resistant Design

In earthquake resistant analyses of ventilation towers and other structures, the shear strength of soil layers stipulated in Items (1) and (2) below shall be neglected.

(1) Sandy Soil Layer

The sandy soil layer which has been judged to be liable to cause liquefaction by the test procedures shown in "3.2.6.1 Stability of Sandy Soils"

(2) Soft Cohesive Soil Layer and Silty Soil Layer

The soft cohesive soil layer which has been judged to be liable to cause sudden changes in properties during earthquakes by the test procedures shown in "3.2.6.2 Stability of Cohesive Soils."

In general, submerged tunnel is often constructed in the soft soil layer. Therefore, if the proposed tunnel site has a soil layer which is stipulated in this section, it is necessary to take proper countermeasures such as replacement or improvement of soils. In case of a ventilation tower, however, loads are transmitted to the bedrock directly or indirectly. Even if the site of the ventilation tower has a soil layer which is liable to become unstable during earthquakes, such a soil layer will little affect the stability of the structure. It is stipulated, therefore, that earthquake resistant analysis can be carried out by disregarding the shearing strength of the soil layers stipulated in this section.

One of the methods of determining the stability of cohesive or silty soil layer is to conduct a vibration test, a cyclic tri-axial compression test and a cyclic simple shear test on undisturbed soil samples as mentioned in "3.2.6.2 Stability of Cohesive Soils" and to check whether or not the samples become unstable state such as sudden deformation.

In the Specifications for Earthquake Resistant Design of Highway Bridges (Japan Road Association, January, 1971), it is stipulated that the shearing strength of a soil layer which has an unconfined compressive strength of less than 2.0 t–f/m^2, shall be disgregarded.

3.3.4 Design Earthquake Ground Motions

3.3.4.1 General

(1) In the earthquake resistant design of a submerged tunnel, the displacement of ground during earthquakes or the design seismic coefficient shall be taken into consideration.

(2) For the submerged structure, the ground deformation method and the seismic coefficient method may be used for the design.

(3) For the ventilation tower and other appurtenant structures, the seismic coefficient method may be used for the design.

The earthquake resistant design of ordinary structures has been made by using the seismic coefficient method and the earthquake resistance of such structures has a historical background. Therefore, it is stipulated that for those structures which can be seen in ordinary structures, out of the structural system of the submerged tunnel, earthquake resistant designs shall be made with use of the seismic coefficient method. Behavior of the submerged structure during earthquakes, however, is affected mainly by the three dimensional dynamic behaviours of its surrounding ground during earthquakes. For this reason, it is stipulated in the Specifications that either the ground displacement or the seismic coefficient shall be taken into consideration.

The earthquake resistant design for the submerged tunnel shall be made, basing on the ground displacement. For the estimation of the stability of sliding of the submerged tunnel in transverse direction, the seismic coefficient method is used for practical purpose, because any other rational method has not been established.

In earthquake resistant design of the ventilation tower and other appurtenant facilities, the seismic coefficient method shall apply as previous cases. The seismic coefficient method includes the modified seismic coefficient method considering structural response (applicable to tall ventilation towers).

3.3.4.2 Design Earthquake Ground Motion

3.3.4.2.1 Ground Displacement in Earthquake Resistant Design (abridged)

Ground displacement to be considered in the earthquake resistant design shall be based on the observed value and determined by taking the nature of the earthquake ground motion and ground conditions into consideration.

In order to obtain ground displacement during earthquakes, one of the following methods may be employed:

Method by Observation

This is the method to determine the ground displacement on the basis of the results of field observation.

Method Using Response Spectral Curve

a) The horizontal ground displacement amplitude on the ground surface can be obtained by the following formula:

$$U_h=\frac{2}{\pi^2}S_v \cdot T \cdot A_{oh} \cdot \nu_1 \qquad \ldots\ldots(3\text{–}1)$$

where U_h: Horizontal ground displacement amplitude on the ground surface (cm)

S_v: Response velocity value per unit acceleration at the bedrock (cm/s/gal).

(Refer to "Earthquake resistant design of bridges (3.5)")

Fig. 11 Response Velocity per Unit Acceleration (after Public Works Research Institute, Ministry of Construction) (omitted)

Fig. 12 Response Velocity per Unit Acceleration (after Port and Harbour Research Institute Ministry of Transport) (omitted)

This value can be obtained from Fig. 11 or Fig. 12 when the natural period T and damping coefficient h of the ground are specified.

T: Fundamental natural period of the subsurface ground (s)

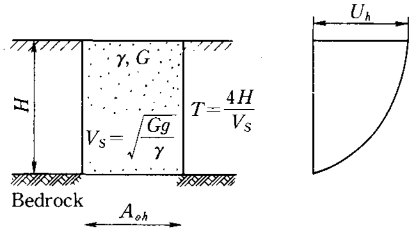
A_{oh}: Horizontal acceleration on the bedrock surface stipulated in "3.3.4.2.2 Bedrock Accelerations on Earthquake Resistant Design." (gal)

ν_1: Importance Factor (Refer to Table 2 below)

Table 2 Importance Factor

Kind	ν_1
Tunnel serving higher public interests	1.0
Others	0.8

Eq. (3–1) is a formula for calculating the deformation of ground by means of spectral curves, when bedrock is assumed to underlay homogeneous ground (having a thickness of H) as shown in Fig. 13 and seismic ground motion having a maximum acceleration of A_{oh} is applied to the bedrock.



(a) Surface Ground Model (b) Relative Displacement to Bedrock

Fig. 13 Model of Ground

Natural period T may be obtained either by a) method of obtaining the predominant period from micro-tremor observations or b) method of obtaining shear modulus G (or the S-wave velocity V_s) of the ground by means of insitu tests or laboratory tests and then obtaining the natural period, using thickness H according to formula $T=4H/V_s$. In any case, it is recommended to determine the natural period by taking the magnitude of strain that will occur in the ground during earthquakes into consideration.

b) Vertical ground displacement amplitude U_v on the ground surface shall be 1/2 to 1/4 of U_h.

(3) Method Employing Dynamic Analysis

In this method, the ground displacements shall be estimated according to "3-4 Dynamic Analysis."

(4) Method Employing Equivalent Sinusoidal wave Spectra

In this method, deformations of the submerged structures are calculated on the assumption that sinusoidal wave motions propagate through the ground along the submerged structure. Amplitudes of the sinusoidal wave motions under consideration are given for various periods (they are called the equivalent sinusoidal wave spectra); and sweeping the period, the deformation of the submerged structure is repeatedly calculated until the maximum deformation is obtained. The equivalent sinusoidal spectra may be obtained on the basis of strong motion earthquake records.

3.3.4.2.2 Bedrock Acceleration for Earthquake Resistant Design

(1) Horizontal acceleration to be considered at the level of the design bedrock surface shall be determined taking the magnitude of earthquakes at the proposed tunnel site and the degree of importance of the submerged tunnel into consideration.

(2) Vertical acceleration to be considered at the bedrock shall be 1/2 of the horizontal acceleration.

It is stipulated that the horizontal acceleration of the earthquake ground motion to be considered at the bedrock surface shall be determined by taking account of the importance of the submerged tunnel on the basis of the investigations mentioned in "3.2.2.1 Earthquakes" and "3.2.2.2 Earthquake Ground Motions", and existing data observed at underground. The "design bedrock surface" mentioned here shall, in principle, be determined by the method stipulated in "3.2.3.5 Configuration and Depth of Bedrock Surface".

The horizontal acceleration on the design bedrock surface can be obtained by the following formula:

$$A_{oh} = \nu_z \cdot A_o$$

where A_{oh}: Horizontal acceleration on the design bedrock surface.

ν_z: Seismic zone factor (refer to "Earthquake resistant design of bridges (3.5)")

A_o: Standard bedrock horizontal acceleration at bedrock surface (which shall be namely equal to 100∼150 gals).

The vertical acceleration is stipulated to be 1/2 of the horizontal acceleration by taking the examples of observations and those of other structures into consideration.

3.3.4.2.3 Design Seismic Coefficient in the Seismic Coefficient Method

(1) The design horizontal seismic coefficient shall be determined by taking account of the magnitude of earthquakes at the proposed site of the submerged tunnel, the kind of ground, and the importance of the submerged tunnel.

(2) The design vertical seismic coefficient shall, in principle, be 1/2 of the design horizontal seismic coefficient.

Design seismic coefficients are stipulated to be determined, in principle, on the basis of the results of "2.2 Investigation on Earthquakes and Earthquake Ground Motions" and

Fig. 14 Seismic Zoning Map (omitted)

"2.3 Investigation on Geology and Grounds" and by taking the topography at the site, ground conditions and the importance of the tunnel into consideration.

The design horizontal seismic coefficient in the seismic coefficient method can be obtained by the following method:

a) The design horizontal seismic coefficient is obtained by the following formula:

$$k_h = \nu_z \cdot \nu_I \cdot \nu_S \cdot k_{oh}$$

where k_h: Design horizontal seismic coefficient

ν_z: Seismic zone factor (refer to Fig. 14 of 3.3.4.2.2)

ν_I: Importance factor (refer to Table 2 of 3.3.4.2.1)

ν_S: Ground condition factor (refer to Table 3)

k_{oh}: Standard design seismic coefficient (=0.2)

Table 3 Ground Condition Factor, ν_S

Group	Definitions[1]	Value of ν_S
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 abobe bedrock (2) Alluvial layer[3] with depth less than 10 meters above bedrock	1.0
3	Alluvial layer[3] with depth less than 25 meters, which has soft layer[4] with depth less than 5 meters	1.1
2	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 tunnel 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, gravelly layer, or cobble layer.

3) Alluvial layer implies a new sedimentary layer made by a landslide.

4) Soft layer is defined in Section "3.3.3.6 Soil Layer where Shearing Strength is Neglected in Earthquake Resistant Design."

b) The design seismic coefficient to be used for earthquake resistant analysis of the submerged structure and the approach shall take the value mentioned in Item a) in the above.

c) The design seismic coefficient to be used for earthquake resistant analysis of the ventilation tower shall be obtained by multiplying the value of Item a) mentioned above by correction coefficient in which response of the ventilation tower is con-

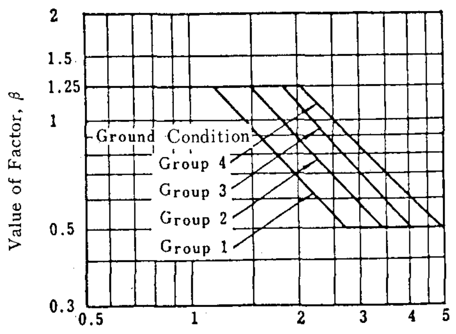


Fig. 15 Factor β

sidered. For correction coefficient β, use the value in Fig. 15. Provided that when the natural period of the ventilation tower is less than 0.5 s, use $\beta=1.0$.

(3) The design vertical seismic coefficient k_v is stipulated to be 1/2 of horizontal seismic coefficient k_h, by referring to the available strong motion earthquake records and other earthquake resistant design standards.

3.3.5 Earthquake Resistant Design of Submerged Structures

3.3.5.1 General

For the earthquake resistant design of the submerged structure, the seismic deformation method shall be used. For the design of cross section in the transverse direction and for the examination of the stability of sliding of the submerged tunnel, the seismic coefficient method or seismic deformation method shall be applied.

The submerged tunnel structure, being generally an long and narrow structure and buried under soft ground, is mainly affected by the displacement of the surrounding soil layer during earthquakes. Vibration characteristics of the ground near surface where the tunnel is constructed are affected by the topography, soils, properties, and stratigraphy of the nearby ground. Therefore, it is necessary to design submerged tunnel by taking the displacement or deformation of ground into consideration. Since sufficient knowledge has not yet been obtained concerning the above-mentioned vibration characteristics, it is stipulated that either vibration theory or wave propagation theory can be used.

The procedure stipulated in "3.3.5.2 Seismic Deformation Method" is to determine stresses and displacements of the submerged structure along the longitudinal axis assuming the submerged structure has been buried in a homogeneous soil layer.

In the earthquake resistant design of the submerged structure in its cross-sectional direction, it is necessary to examine the thickness of the cross-sectional wall so as to resist against loads during earthquakes and also to examine the stability of the submerged structure against sliding. It is stipulated that in these examinations, the conventional seismic coefficient method may be employed which is used for various civil engineering structures such as the caisson foundation, due to reasons mentioned in "3.3.4 Design Earthquake Ground Motion," and it is further stipulated that the results of the investigations shall be confirmed by dynamic analysis.

In the examination of the thickness of the cross-sectional wall, first the cross-sectional stress shall be calculated by taking account of the inertia forces, earth pressures and water pressures which are conceivable to be applied to the wall during earthquake, and then necessary measures shall be taken for reinforcing the cross section of the submerged structure or increasing the thickness of wall.

The stability against sliding shall be examined by taking account of both active and passive earth pressures that apply to the submerged structure and also shearing resistance forces which act from the adjoining cross sections of the submerged structure.

When foundation structures such as pile foundations are to be used for the submerged structure, it is necessary to examine the submerged structure and the foundtaions simultaneously by paying attention to connecting conditions between the two structures.

Phenomena of floating and dynamic settling shall be also considered. To cope with such phenomena, it will be proper first to take various measures such as the strict selection of soil fill material, improvement of soils and conscientious execution of these works and then to give necessary examination to the cross-sectional design of the submerged structure (refer to "3.2.6 Seismic Stability of Soils" and "3.3.8 Stability of Subsoils (Soil fill).").

3.3.5.2 Seismic Deformation Method

For the earthquake resistant analysis of the submerged structure, the ground deformation method (the analysis method is based on the displacement of ground during earthquakes) shall be employed.

Deformation taking place in a structure like a submerged tunnel which has larger rigidity but smaller density than the ground is governed by the displacement of soil layers surrounding the tunnel. By taking these properties of the submerged tunnel into consideration, mathematical model of the submerged tunnel to be used in design shall be taken as a beam on elastic foundation in analysing deformation and stress of the model.

More precise analyses such as earthquake response analyses are recommended for the design of submerged structure by taking account of discontinuities of topography, geology and structural joints. Maximum displacement of the ground to be employed in design shall be determined in accordance with provisions in "3.3.3.2 Ground Displacement during Earthquakes" and "3.3.4.2 Design Earthquake Ground Motion."

3.3.5.3 Seismic Coefficient Method

In the seismic coefficient method inertia forces arising from the weight of the structure itself and that of soil fill, shall be considered.

Directions and positions of inertia forces and combinations of inertia forces shall, in principle, be determined so that the stresses, straines and deformations of the structure become the critical.

In the seismic coefficient method, analysis shall be made by taking account of inertia forces arising from the weight of the structure itself and its contents, the weight of surrounding water and soil fill, and the weight of the ground. In making this analysis, it is necessary to grasp phenomena (for instance, an increase in surcharge and sliding due to unbalanced loading) and to examine individual cases. For simplicity the directions and positions of inertia forces to be applied and the combinations of inertia forces due to the horizontal seismic coefficient and those due to the vertical seismic coefficient shall, in principle, be determined so that structure will be placed in the worst state in respect to stress or strain.

3.3.6 Earthquake Resistant Design of Ventilation Towers

3.3.6.1 General

In making earthquake resistant design for ventilation towers consideration shall be given to their connections to the submerged structure so that the total structure will become earthquake resistant.

3.3.6.2 Earthquake Resistant Design of Ventilation Towers

For the earthquake resistant design of ventilation towers, the seismic coefficient method shall be employed, and the design seismic coefficients shall be determined in accordance with provisions in "3.3.4.2.3 Design Seismic Coefficient in the Seismic Coefficient Method."

For methods of application and analysis of seismic forces in the earthquake resistant analysis of the ventilation tower, provisions in related specifications shall be applied depending upon the structural types of ventilation towers, and conditions of the proposed sites of these structures.

3.3.7 Earthquake Resistant Design of Approaches

3.3.7.1 General

In making earthquake resistant design of approaches, consideration shall be given to the connection of approaches to the submerged structure and the ventilation towers so that the total structure will become earthquake resistant.

The structural types of approaches that are connected to the submerged structure have various types such as the tunnel construction by excavation and back-fill process, caisson process and shield process and the open construction by the semiunderground U-shaped construction and retaining-wall construction.

In the earthquake resistant design of these approaches, it is necessary to consider not only the earthquake resistance of the approaches but also the behavior of the submerged structure to which the approaches are connected.

Even if the construction process is different between the approaches and the submerged structure, it is advisable that approaches that are connected to the submerged tunnel should be provided with the same degree of earthquake resistant analysis as that of the submerged structure, if the approaches have similar structural conditions such as cross-sectional dimensions and rigidity to those of the submerged structure.

3.3.7.2 Earthquake Resistant Design of Approaches

The earthquake resistant analysis of the approach may be carried out by other related standards and specifications depending upon the respective structural types.

3.3.8 Stability of Subsoils

3.3.8.1 General (omitted)

3.3.8.2 Stability Analysis of Subsoils (omitted)

3.3.8.3 Stability of Soil Fills for Submerged Structures

(1) Liquefaction (omitted)

(2) Stability against Sliding (omitted)

3.3.9 Allowable Stresses

3.3.9.1 General

The allowable stresses of materiais and increase in allowable stresses during earthquakes shall be determined, considering provisions of related specifications depending upon the structural types and the degree of importance of the structure, and taking account of the assumptions in design calculations and the methods of construction and maintenance.

3.3.9.2 Allowable Stresses of Concrete (abridged)

(1) The allowable stresses of concrete to be used for the submerged tunnel shall conform to the following specifications:

"Standard Specifications of Concrete"* by the Japan Society of Civil Engineers.

(2) The allowable stresses of the concrete placed in the water shall be decided on the basis of the experiments simulating the field conditions as the quality of the concrete depends on the placing methods.

3.3.9.3 Allowable Stresses of Steel

(1) The allowable stresses (normal state) of steel to be used for the submerged tunnel shall conform to the following specifications:

a) Reinforcing Bars
"Standard Specifications of Concrete" by the Japan Society of Civil Engineers.

b) Steel Other than Reinforcing Bars
"Standard Specifications for Highway Bridges (Part of Steel Bridge)" by the Japan Road Association.

(2) When cracks are of the particularly harmful, allowable tensile stresses (under ordinary condition) of reinforcing bars shall be determined by taking the effects of cracks into consideration.

Due to the reasons shown in "3.3.9.2 Allowable Stresses of Concrete," this provision is stipulated. For steels which act as a single unit together with concrete, no reduction in allowable stresses against buckling is required.

The tensile strength of concrete is small and a concrete structure will develop cracks at the place where its tensile stress exceeds a certain limit. In case of reinforced concrete, the tensile strength of concrete is ignored in design analysis and, therefore, cracks on reinforced concrete poses no particular problem for resisting force. However, large cracks may cause the lack of waterproof of the reinforced concrete and result in rusting of bars. It is necessary, therefore, to impose some restrictions on cracks.

The magnitude of the width of a crack is complicatedly affected by the diameter of reinforcing bars and the depth of cover concrete. As the tensile stress of bars increases, the width of the crack is generally known to become larger. In order to restrict the width of the crack, therefore, it will be effective to restrict the allowable tensile stress of the reinforcing bars. Regarding the allowable width of the crack and the relation formula between the crack width and the stress of the reinforcing bar, many theories have bee proposed, and the allowable width of a crack varies considerably with conditions. Generally speaking, the crack width which is allowable from the viewpoint of waterproof is less than 0.1 mm, and that which is allowable from the viewpoint of rust prevention is considered to be about 0.2 mm for members that are always immersed in water and about 0.15 mm for members that are placed near the water surface and severely affected by water conditions.

The relationship between the stress of the reinforcing bar and the width of the crack, according to tests, is mostly within the ranges of 0.05 mm to 0.10 mm at a tensile stress of the reinforcing bar of 1,000 kg–f/cm^2, 0.15 to 0.20 mm at 2,000 kg–f/cm^2 and 0.2 to 0.3 mm at 3,000 kg–f/cm^2. Allowable stresses in the Standard Specifications of Concrete have

*) The "Standard Specifications of Concrete" were revised in 1986 adopting the limit state design method. The specifications cited here denote the former version.

been stipulated by taking the limits for the crack width allowable from the durability of general members in the ordinary exposing state into consideration. It is also stipulated that if the crack is particularly harmful, the allowable stress shall be reduced.

Submerged tunnels include those which are protected by steel shells at the outside so that concrete will not directly touch water. In case of the waterway tunnel, in which waterproof is particularly required, it is necessary, to determine the allowable tensile stress of reinforcing bars by taking into consideration the abovementioned characteristics of the structure.

Further, steels which are buried in the concrete and act as a single body with concrete shall be taken for a sort of reinforcing bars and stipulations for reinforcing bars shall also apply to such steels. At present, an allowable stress of reinforcing bars of less than 1,800 $kg \cdot f/cm^2$ is used, although further examination is need for this value.

3.3.9.4 Increase in tne Allowable Stresses in Earthquake Resistant Design

(**1**) Allowable stresses in the earthquake resistant design can be obtained by increasing the allowable stresses stipulated in "3.3.9.2 Allowable Stresses of Concrete" and "3.3.9.3 Allowable Stresses of Steel", which are applicable to ordinary state. The increasing percentage shall be within the ranges shown in the following table, depending on the combinations of loads.

Table 4 Limit of Increase in Allowable Stresses

Combination of Loads	Increase Percentage
Ordinary Load + Earthquake Load	50%
Ordinary Load + Effects of Temperature Changes + Shinkage + Earthquake Load	65%

In this case, the allowable stresses of reinforcing bars which will be used as reference values, can be the value stipulated in "3.3.9.3 Allowable Stresses of Steel." When methods other than the principles of the Specifications which are stipulated in "3.3.9.2 Allowable Stresses of Concrete" and "3.3.9.3 Allowable Stresses of Steel" are used in the analysis of the allowabie sectional forces of members, allowable stresses or safety factors for ordinary design and the earthquakes resistant design shall be separately determined, in view of test results.

Seismic loads to be taken into consideration for design generate very rarely within the period of the life of the submerged tunnel, and the duration time of action of these loadings is also short. The values of allowable stresses have been increased by taking these facts into account. To obtain increase percentages, those shown in the Standard Specifications of Concrete have been used.

The design of the reinforced concrete structure has been made on the basis of the elasticity theory. Recently it is proposed, in view of the deformation characteristics of concrete, to apply non-elastic design methods such as ultimate strength design methods for critical situations such as earthquakes. It is also proposed that since concrete has some what tensile strength, a certain degree of tensile strength may be allowed to the concrete structure during earthquakes. However, since there are only few structures which have been designed based on the above-mentioned design methods, it is stipulated here that if such design methods are made, allowable stresses or safety factots may be determined by investigations and tests.

If the allowable stress of a reinforcing bar in ordinary loading conditions is assumed to be 1,400 kg–f/cm^2, then the allowable stress of the reinforcing bar in the earthquake resistant design can be 2,100 kg/cm^2 and the strain can be 10^{-3}. That is, when displacement of the ground or the submerged tunnel during earthquakes is known beforehand and there is a portion in which strain exceeds the above-mentioned limit, it is recommended to install mechanical joints in order to reduce the stresses.

3.3.10 Details of Earthquake Resistant Design

3.3.10.1 General

In detailed design of the submerged tunnel, attention shall be paid to earthquake resistance. Especially at the portion of sudden changes in the topography, geology and the structure of the suberged tunnel, servere displacement or deformation is liable to occur during earthquakes, hence the detailed design of the surroundings shall be conducted.

3.3.10.2 Structural Joints

The types of the structural joints and their spacing shall be determined by considering the design for normal state and construction stage and also by examining the earthquake resistance.

In selecting the positions, types and spacing of structural joints in the submerged tunnel, the following points are normally considered: the manufacturing process of tunnel elements at the time in construction, towing and sinking, and uneven settlement and temperature changes of the tunnel after its completion.

For the earthquake resistance of the submerged tunnel, however, construction and spacing of structural joints are important factors. In selecting the structural joints, sufficient examination is required from the point of view of earthquake resistance.

The type of structural joints are classified into two: the rigid joints in which joints have rigidity and strength equal to those of the cross section of the tunnel element so that joints can sufficiently withstand deformation and strain in earthquakes and the flexible joints in which joints have flexibility to absorb deformation during earthquakes. Naturally, joints with intermediate characteristics between the two types mentioned above are conceivable. Hereunder the respective characteristics of these joints and points of caution of the types are explained.

Flexible Joints—When flexible joints are used, displacements and deformations of joints and the effects on the entire length of the tunnel vary depending upon the positions of joints and their degree of flexibility. It is desirable to evaluate the effects of flexible joints during earthquakes by conducting the earthquake response analysis and experimental model tests.

In designing the joints, the effects of temperature changes and uneven settlement that may develop in the submerged structure should be taken into consideration so that joints maintain the capacity to absorb deformation which may occur during earthquakes.

Since flexible joints have structural characteristics which are different from those of general cross sections, careful examinations are required to prevent joints from becoming weak points of structure.

Rigid Joints—When rigid joints are used, it should be considered that the joints will not become weak points of the structure.

3.3.10.3 Pile Foundation

When a pile foundation is used for the submerged tunnel, examination shall be made on the effects of piles on the behavior of the submerged tunnel during earthquakes.

3.3.10.4 Ventilation Towers

In earthquake resistant design of ventilation towers, careful attention shall be given to the structures, and positions of connections between the ventilation tower and the submerged structure and approaches shall be arranged so that excessive cross-sectional forces do not develop in the structure.

3.4 DYNAMIC ANALYSIS

3.4.1 General

The dynamic analysis for the design of the submerged tunnel is divided into the one for the partial structural systems and the one is for the total structural system. The former is conducted for the each partial structural design, and consists of investigations on dynamic behavior of the individual partial structural systems such as the surrounding ground, the submerged structure and the ventilation tower. The latter is conducted for the total earthquake resistant design of the submerged tunnel, to grasp the dynamic behavior of the submerged tunnel as a total system consisting of the partial structural systems.

In the dynamic analysis it is desirable to perform both the earthquake response analysis and the dynamic model test. In the earthquake response analysis the earthquake response of the submerged tunnel agaiust various earthquake ground motions can be calculated in detail with an idealized analytical model. Meanwhile in the dynamic model test the three-dimensional dynamic behavior of the ground and the structures can be examined. As both analyses have their own merits, they are required to be used properly depending on the purposes. For example, the dynamic model test can be used for examining appropriateness of the analytical model used in the earthquake response analysis.

Details of the earthquake response analysis and the dynamic model test are described in "3.4.2 Earthquake Response Analysis," and "3.4.3 Dynamic Model Test." In the dynamic analysis the analytical model is established in accordinace with the dimensions of the structures and the dynamic characteristics of the ground described in "3.2 Surveys." Then, the earthquake response of the total structural system and the individual partial structural systems is evaluated, using the input earthquake ground motions described in "3.4.2.4 Input Earthquake Ground Motion." According to these analyses the effects of the sectional forces, the dynamic earthpressure, and the joints between elements and ventilation towers on the earthquake resistance of the submerged tunnel can be claritied.

3.4.2 Earthquake Response Analysis

3.4.2.1 General

The earthquake response analysis of the submerged tunnel should be conducted for the individual partial structural systems, such as the surrounding ground, the submerged structure and the ventilation towers, and also for the total structural system, considering the dynamic soil-structure interactions and the characteristics of earthquake ground motions, by procedures which can simulate the dynamic response of the submerged tunnel as correctly as possible.

3.4.2.2 Methods of Earthquake Response Analysis

The earthquake response analysis can be conducted by using one of the following two methods.

(A) Method using the average response spectra

(B) Method using the time-history records of earthquake ground motions

3.4.2.3 Analytical Model

Analytical model for the earthquake respouse analysis should be made for grounds, the submerged structure and the ventilation tower, respectively, and the total submerged tunnel. The effects of the existance of the water should be taken into account when necessitated. Dynamic properties of the grounds, such as natural periods, vibration modes, damping characteristics and so on, and those of the appurtenant structures, and the dynamic interactions between the grounds and the structures should be taken into account in establishing the analytical models.

Though the analytical model representing the dynamic characteristics of the ground and the structure can not be easily established, the following may be useful information for establishing the analytical model.

(1) In the ground, oscillation of the natural period is predominant. In case that the natural period of the ground is from 1 second to 3 seconds, it is thought that the oscillation of fundamental mode gives the largest amplitude during a great earthquake. Generally the earthquake response of the ground is not influenced by existance of the submerged tunnel except in the vicinity of the tunnel.

(2) It may be assumed that the damping of the ground during the earthquake is viscous damping. In this case it is thought that the damping factor ranges approximately from 10 to 20 percent. It may be allowed that the damping factor of stiff ground has a smaller value in the range and that of soft ground has a larger value.

(3) If the dynamic characteristics of the ground varies due to the sudden change in the ground condition, the influence is limited to the vicinity of the place where the change exists.

(4) Earthquake response of the submerged tunnel against motions of the surrounding ground is usually negligible in case of the ordinary ground condition and the submerged structure behaves like a beam in the ground subjected to static ground deformations.

(5) The influence of an external force working on a part of the submerged tunnel is limited to a small part of the structure, especially in case of the bending deformation.

Through the analyses conducted for the ground and for a combination of the ground and the tunnel with use of several methods including 3-dimensional finite element method, the following is usually accepted. First, 2-dimensional analysis of slices of the ground in the transverse direction to the tunnel is carried out to estimate vibrational characteristics of the slices. Then, the slices are represented by equivalent mass-spring systems and the masses are connected each other in the longitudinal direction by springs representing rigidity of the ground.

The springs of the above mentioned analytical model is usually linear; however, the dynamic behavior of the real ground-structure system is always non-linear. After consideration of this point, not only linear analysis but also non-linear analysis for the analytical model with non-linear springs may be required in some cases. In these cases, appropriate consideration of the similarity between the model and the prototype, and the degree of difficulty of the analysis should be taken into consideration for determining the analytical model.

3.4.2.4 Input Earthquake Ground Motion

The input earthquake ground motions of the tunnel site should be determined by the information on seismicity specified in "3.2 Surveys," considering the life of the tunnel, the distribution of earthquake damage, the seismic intensity based on the probability theory and the expectancy of the earthquake occurrence. At the same time it is necessary to consider the method for the determination of the input earthquake ground motions based on the magnitude of earthquake and the epicentral distance. The earthquake observation results show that the strains of the submerged tunnel are influenced by the characteristics of the earthquake ground motions rather than the maximum acceleration. The earthquake magnitude is one of the major factors which determin the characteristics of earthquake ground motions. Therefore, the earthquake magnitude is an important factor from the design point of view.

Though the earthquake ground motion of the tunnel site is desirable to be obtained from the earthquake observation, there is usually little possibility. In the case when the earthquake ground motion records at the site are not available, it is appropriate to use the earthquake ground motions obtained at the other site with similar ground condition. Then, when the amplitudes of the ground motions are multiplied by a constant value to fit the maximum amplitude which is assigned in the design conditions, it is necessary to consider difference of the frequency characteristics of the ground motions due to differences of the earthquake magnitudes and the maximum accelerations.

When selecting response spectra to be used in the analysis, it is necessary to take similar consideration as mentioned above.

The input earthquake ground motions should be considered in principle at the bedrock surface specified in "3.2.3.5 Configuration and Depth of Bedrock".

3.4.3 Dynamic Model Test (abridged)

Dynamic model tests for the submerged tunnel is desirable conducted for the individual partial structural systems and also for the total structural system, considering the dynamic soil-structure interactions and the characteristics of the earthquake ground motions by using the method which can simulate the dynamic response of the submerged tunnel as close as possible.

3.4.4 Safety Assessment

The assessment on safety can be made by comparing the results of dynamic analysis with "3.3.9 Allowable Stresses". In the dynamic analysis, very large responses for a very short duration are sometimes obtained. In this case, analyses of other similar structures may be referred to for the assessment of these responses. The assessment of the displacement and the strain of which allowable values are not specified in "3.3.9 Allowable Stresses" is made in the similar way to that mentioned above.

Furthermore, it is necessary to examine the possibility that the harmful deformation and relative displacement of the structures will occur, and the possibility of the drastic decrease in soil strength (such as liquefaction).

3.5 SAFETY COUNTERMEASURES AGAINST EARTHQUAKES

3.5.1 General

As the submerged tunnel is an underwater structure, a careful attention should be given to protection of water leakage. For this purpose, the magnitudes of earthquake motions, stress and strain of the submerged tunnel, and leakage of water in the tunnel shall be measured by equipments. When tsunami warning is received, and arrival time and magnitude of tsunami are estimated, necessary countermeasures should be taken to preserve safety of the traffic and the structure and to prevent flooding in the tunnel.

Special attention should be given to prevent damage to the safety equipments due to earthquake ground motions, leakage of water, and flooding.

A flowchart of the safety countermeasures against earthquakes is shown in Fig. 16.

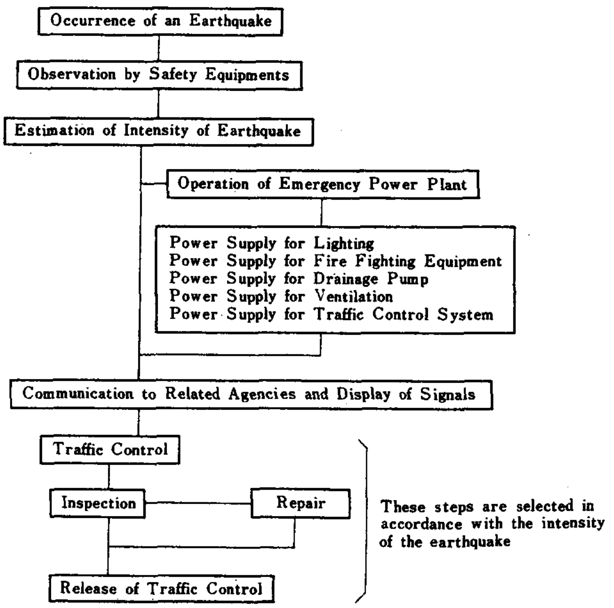


Fig. 16 A Flowchart of Safety Countermeasures against Earthquakes

3.5.2 Safety Equipments

For the purpose of the safety countermeasure in earthquakes, following equipments should be installed. Reliability of these equipment operation should be guranteed by the periodical inspections.

1) Sensors, recorders, and warning system of earthquake ground motions
2) Sensors, recorders, and warning system of tidal level and wave height
3) Traffic control system during and after earthquakes
4) Sensors, recorders, and warning system of settlement and water leakage
5) System for evacuation and guidance
6) Emergency power plant
7) System for observation of traffic
8) Drainage and system of protection from flooding
9) Other equipments (such as strain meters, stress meters, etc.)

3.5.3 Operation

When an earthquake occurs, countermeasures such as control of traffic in the tunnel and inspection of the structures should immediately be done in accordance with the earthquake intensity, the maximum acceleration, etc. at the site of the tunnel so that damage to the structure should be as small as possible and the safety of traffic should be maintained.

In the inspection after an earthquake, a special attention should be given to the joints of the elements, and crack and water leakage. If there is anything unusual, it should sufficiently be examined with respect to the safety of the traffic and the structures.

For the control of traffic, the safety equipments installed in the tunnel should be connected to operate warning equipments automatically. For the case that the automatic operation does not work properly, the warning equipments should be designed to operate manually, too.

The signals should be installed not only at the entrance of the tunnel but also in side the tunnel with an appropriate interval.

Information supply through a radio in a car with an antenna in the tunnel is effective.

A footpath to which people can access from the cars in the roadway may be helpful when the traffic in the roadway is seriously disturbed.

When a strong earthquake occurs, usual electric power supply may stop. In such a case, the emergency power plant should start immediately to supply electricity for the safety countermeasures such as fire fighting and drainage.

3.5.4 Inspection

The safety equipments and the tunnel structures should be steadily maintained in normal conditions by inspections for safety. The inspection should be periodically conducted. Aiso a special inspection should be conducted after earthquakes having the intensity stronger than the specified level.

3.5.5 Post-Earthquake Inspection

When an earthquake stronger than the specified one breaks out, the administrator of the submerged tunnel should immediately examine the influences on every portion of the tunnel. The results of the investigation and the countermeasures employed should be recorded.

4. EXAMPLE OF EARTHQUAKE RESISTANT DESIGN

4.1 TUNNEL STRUCTURE

The Tokyo-Bay-Shore Road is now under construction along the shore of the Tokyo Bay. The expressway section of this road between Kanazawa-ku in Yokohama-City and Kohya in Ichikawa-City is named 'the Expressway Bay-Shore-Route' and is now being constructed by the Metropolitan Expressway Public Corporation. (Fig. 17)

In this route submerged tunnels are used to cross the mouth of the Tama River and the adjacent Kawasaki Fairway. The procedure of the earthquake resistant design of these tunnels will be outlined in this section.

The submerged portion of Tama River Tunnel with the length of 1,550 m has the cross section, as shown in Fig. 18, for six lanes on both ways, where vehicular design speed is set to 80 km/h. This tunnel is made up of 12 elements, each about 130 m long. Its longitudinal slope is 4.0% at the tunnel entrance and 0.3% under the riverbed, while its horizontal alignment forms a curve.

Fig. 19 shows the geological profile of the site where the Tama River Tunnel is constructed. Here, the riverbed depth ranges from TP-7.0 to TP-12.0 meters. A thick and soft alluvial formation mainly consisting of cohesive silt forms the riverbed surface layer. The shear wave velocity in this layer is about 100 m/s. This layer overlies the diluvial formation with hard clay, sand, and gravel horizontally laminated. The shear wave velocity in this diluvial formation is from 200 to 250 m/s.

4.2 PROCEDURE OF EARTHQUAKE RESISTANT DESIGN

In the procedure of earthquake resistant design, general shape and dimension of each element and cross section are determined on the condition that only static loads are applied. Then the dynamic resistance is discussed taking the seismic activity at the site into account.

In case of the design of submerged tunnels, the cross-sectional dimensions and arrangement of reinforcing bars are usually determined considering static factors such as; water and soil pressures, clearance of freeboard during towing and the resistance against floating after completion.

On the other hand, forces in the longitudinal direction are small under static loading, unless unexpectedly great non-uniform settlement occurs. Thus, the longitudinal arrangement of reinforcing bars, the location of joints and their mechanical structures are determined by earthquake resistant design.

Fig. 20 shows the flowchart of earthquake resistant design and its procedure is described as follows.

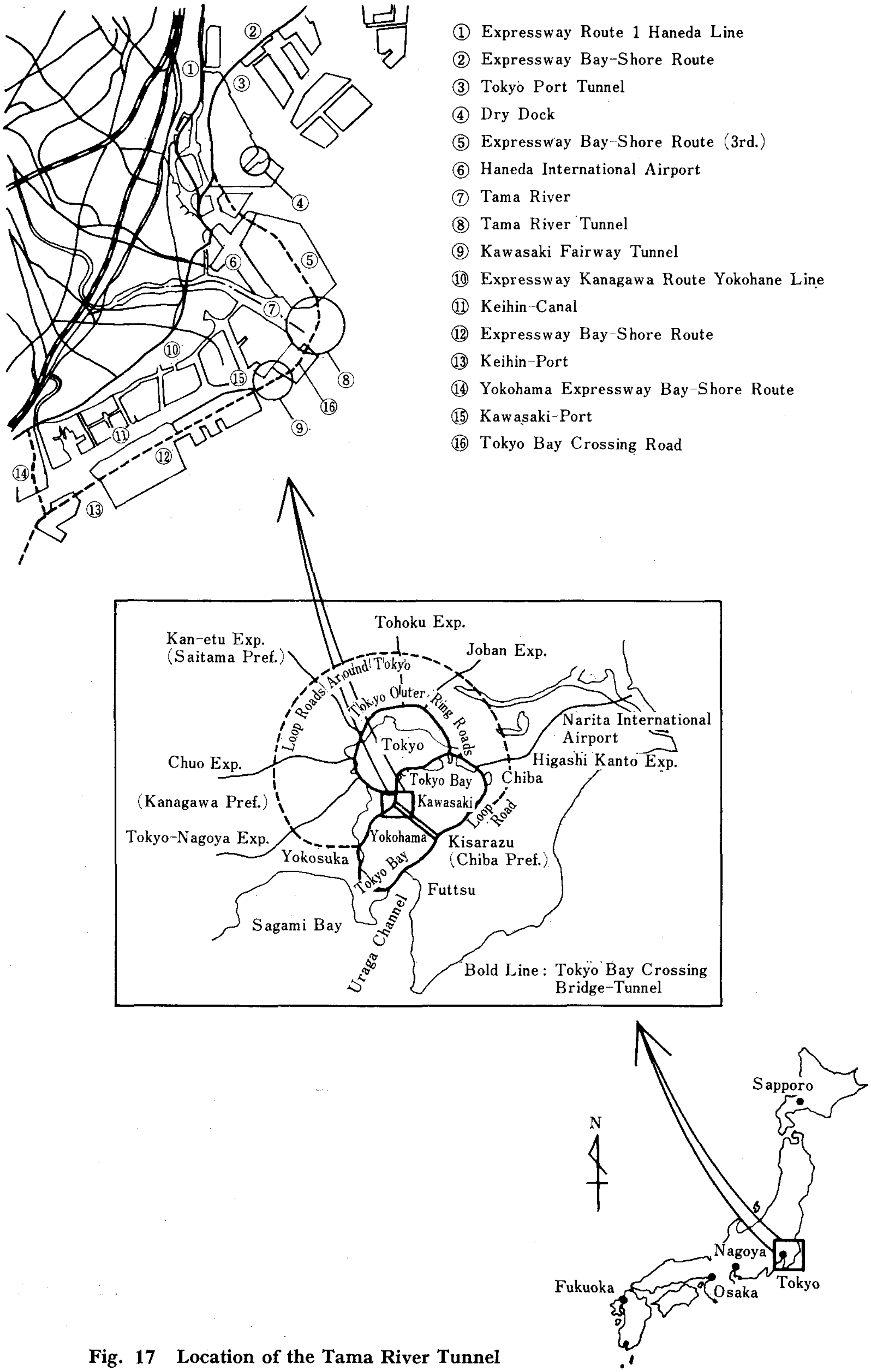


Fig. 17 Location of the Tama River Tunnel

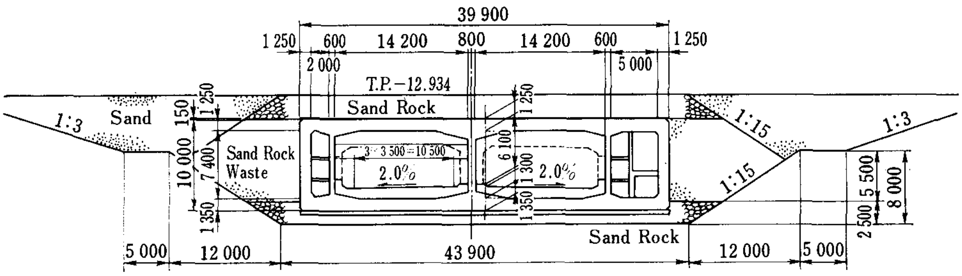


Fig. 18 Standard cross section

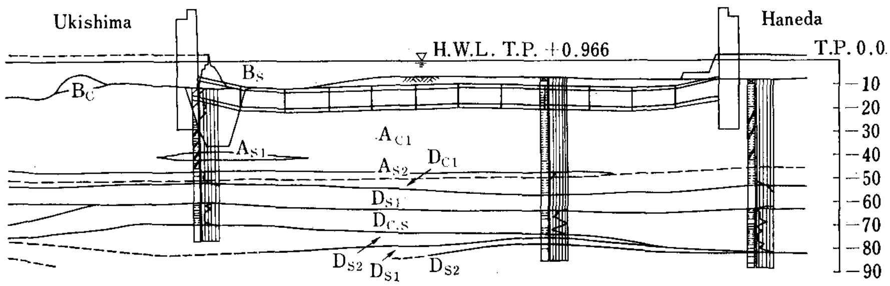


Fig. 19 Geological profile at the site

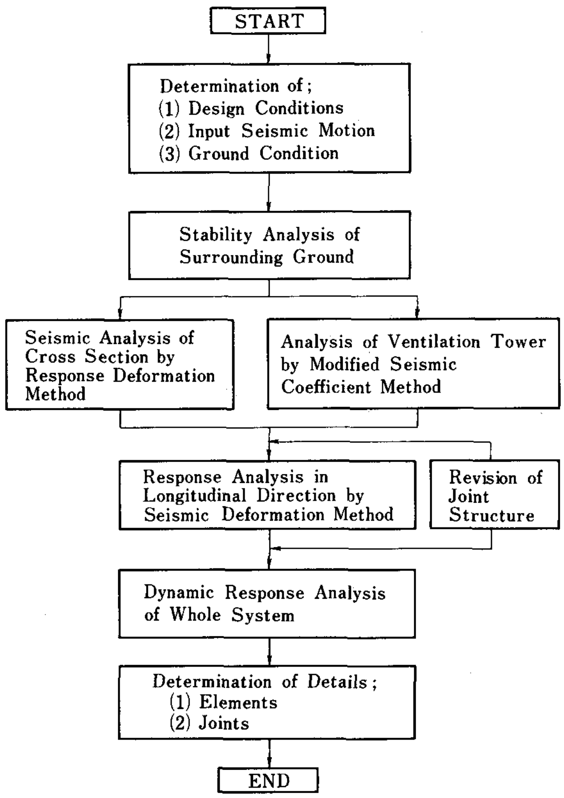


Fig. 20 Flowchart of earthquake resistant design of a submerged tunnel

(1) Input Seismic Motion

Characteristics of an input seismic motion and the position of layer where the input seismic motion is applied are determined taking the required service life of the tunnel, seismic activity and its characteristics at the site into consideration.

(2) Stability of the Surrounding Ground

It is necessary to reconfirm the impossibility of liquifaction, although the surrounding soil consists mainly of cohesive clay with high resistance against liquifaction. Next, the stability against a sliding rupture of the surrounding soil will be discussed.

(3) Cross Section of Tunnel and Ventilation Tower

Earthquake resistance of a cross section of the tunnel designed for normal condition loads is examined. Considering that the tunnel box is deformed by the shear motion of the surrounding ground, response deformation method using finite element model is used to evaluate the induced forces.

The scheme of the ventilation tower connected to the tunnel is revised by consulting the analytical result of dynamic stability during an earthquake.

(4) Tunnel Responce in Longitudinal Direction and the Design of Joints

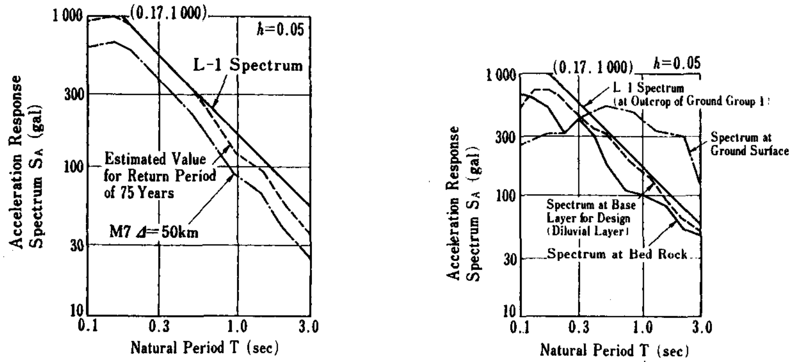
It is necessary to study the response characteristics of the tunnel including the ventilation towers in longitudinal direction, using the responce deformation method. The structural mechanism and the arrangement of the joints are determined considering the allowable displacement and strength of the joint. Then the analysis of earthquake response of the whole system is carried out.

4.3 INPUT SEISMIC MOTION

The input acceleration wave is calculated by the SHAKE. The layer which the input acceleration wave is applied is bed rock of which shear wave velocity is over 700 m/s.

The spectrum of the input acceleration wave is subject to L-1 spectrum shown in Fig. 21 (a). The L-1 spectrum is defined by a set of different factors: seismic motion with a return period of 75 years for the seismic activities around the tunnel; seismic motion of the earthquake whose magnitude and epicentral distance are 7.0 and 50 km, respectively. These factors correspond to seismic motions that are expected to occur once or twice during the designed service life.

An acceleration record with a similar spectrum shape as the L-1 is modified in the fre-



(a) L-1 Spectrum (at Outcrop of Ground Group 1)

(b) Acceleration Response Spectra Used for Earthquake Resistant Design (h=0.05)

Fig. 21

quency domain so that its spectrum shape coincides with the L-1. But L-1 spectrum shown in Fig. 21 (a) is defined as a spectrum at outcrop of the ground condition Group 1. So, the acceleration wave, at the surface of the base layer for the earthquake resistant design calculation (sandy diluvial layer), is obtained by converting the wave abovementioned to this base layer (see Fig. 21 (b)).

4.4 SEISMIC DEFORMATION METHOD

The seismic deformation method is used for determining the sectional force and strain induced in a tunnel. In this method, the deformation of the ground around a tunnel, to be considered at an earthquake, is applied to the tunnel.

Fig. 22 shows the fundamental vibration mode of the ground arround the tunnel.

In the study of transversal direction (discussed later), the vertical distribution of the displacement of the ground is taken into consideration. In the study of longitudinal direction, it is assumed that the amplitude of the displacement goes through a change with certain wavelength along the tunnel axis.

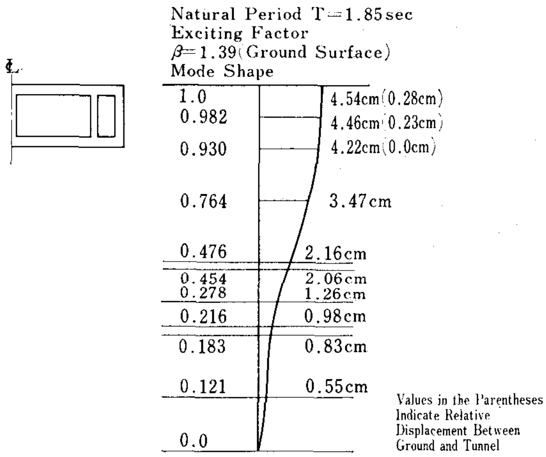


Fig. 22 Fundamental mode of surface layer

4.5 CROSS SECTION OF A TUNNEL

A FEM model is used for the design of the cross section of the tunnel. By using this model, a certain value of loads and displacements at the boundaries on both sides are

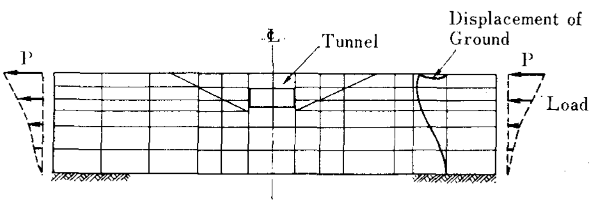


Fig. 23 Analytical model of tunnel cross section

determined so that the displacements of the ground shown in Fig. 22 are produced. Next, the FEM model where the tunnel box is replaced by the beam is made to calculate forces in the tunnel. And the sectional forces and displacements are calculated under the same boundary condition shown in Fig. 23. The sectional forces during earthquakes are about 10% of those at normal condition load.

4.6 DESIGN IN THE LONGITUDINAL DIRECTION OF THE TUNNEL AND DESIGN OF FLEXIBLE JOINTS

Though earthquake resistant design in the longitudinal direction of the tunnel is carried out by response deformation method and by dynamic response analysis, the results of dynamic response analysis are mentioned here.

A model shown in Fig. 24 is used for the dynamic response analysis of the whole system of the tunnel.

As a result of this analysis, the maximum response value and the distribution of the

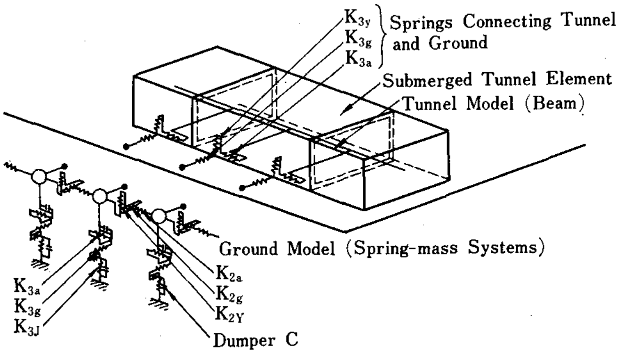


Fig. 24 Model Used for Earthquake Response Analysis

Table 5 Maximum Response Values of Dynamic Response Analysis

			Rigid joint	Flexible joint	Resisting force
Ground	Acceleration (gal)	Longitudinal direction	154	Same as left	At the grand surface
		Transeversal direction	158		
	Displacement (cm)	Longitudinal direction	9.2		
		Transeversal direction	9.9		
Tunnel	Axial force	(t)	45,300	9,660	24,200
	Bending moment	(tm)	341,000	75,600	438,000
	Shear force	(t)	5,730	1,630	7,230
Flexible joint	Axial force	(t)	—	4,880	22,200
	Bending moment	(tm)	—	27,000	198,000
	Shear force	(t)	—	506	2,860
	Elongation	(cm)	—	1.2	3.8
	Rotation	(rad)	—	3.7×10^{-4}	1.0×10^{-3}
	Dislocation	(cm)	—	0.3	—

Note: Acceleration is underestimated, because only the fundameotal mode of surface layer is taken into account.

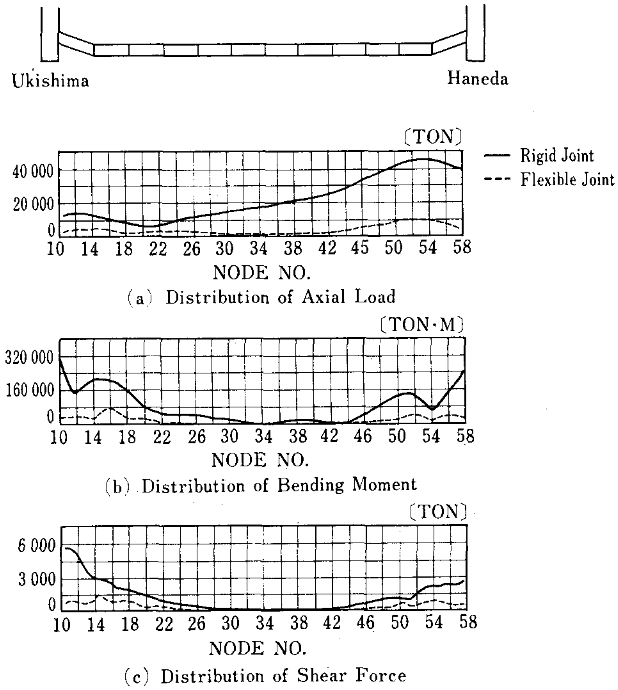


Fig. 25 Model for dynamic response analysis in longitudinal direction

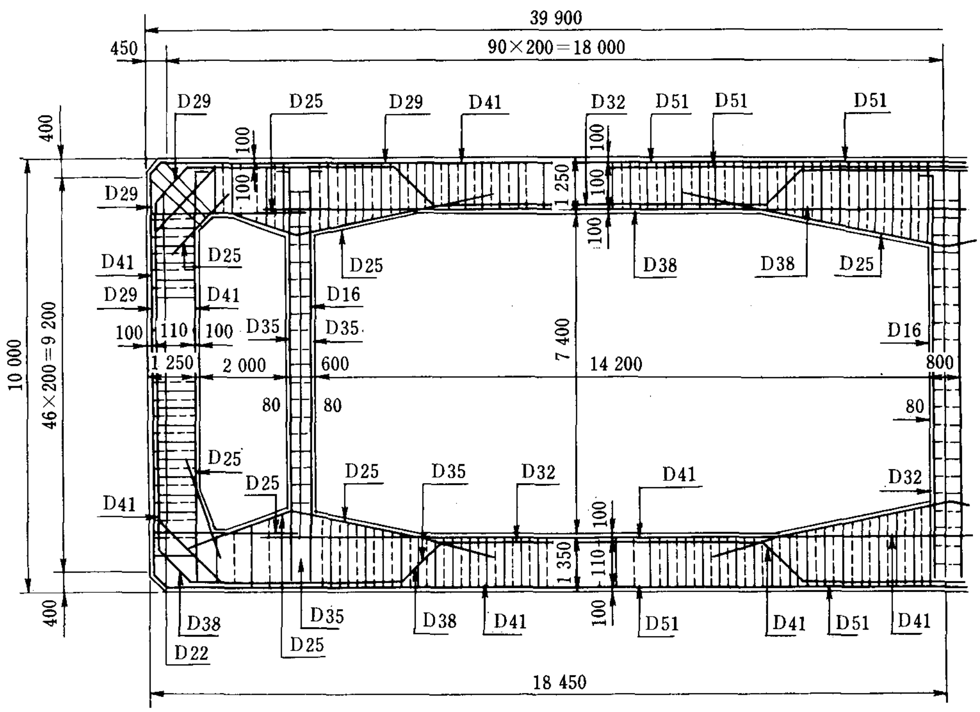


Fig. 26 Standard arrangements of reinforcing bars in tunnel element

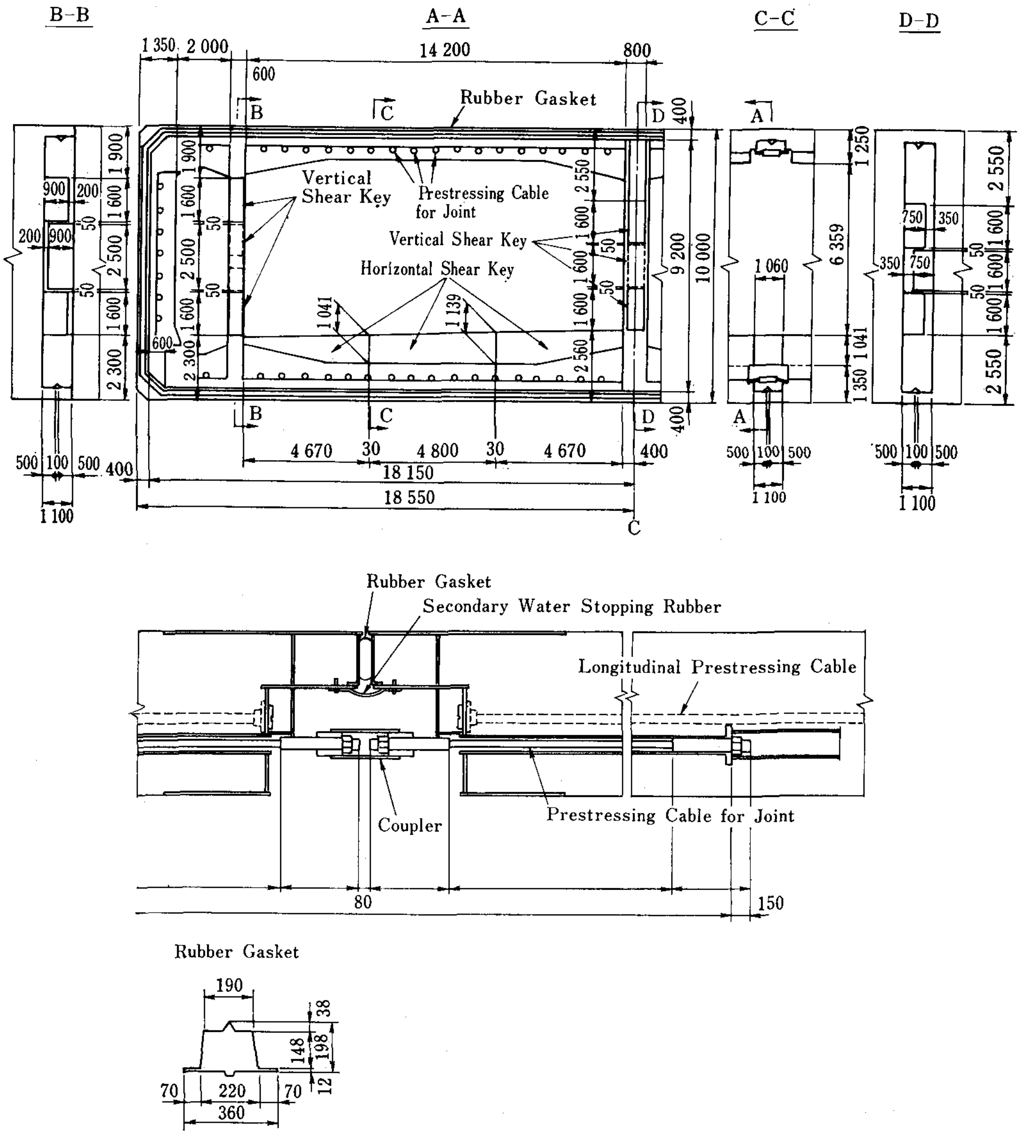


Fig. 27 Structure of flexible joint

largest sectional force are shown in Table 5 and Fig. 25, respectively. Table 5 and Fig. 25 show values in two cases where: (1) all joints are rigid; (2) flexible joints of rubber gaskets and prestressing cables are installed. These table and figure indicate that use of flexible joints declines the sectional forces remarkably.

Fig. 26 and Fig. 27 show the section of the tunnel and the structure of flexible joints determined as a result of this analysis. In the case of this tunnel, prestressing cables are placed along the tunnel axis and a prestressed load of about 10 kg/cm^2 is applied in order to enhance the durability and watertightness by suppressing excessive shrinkage and temperature cracking of concrete.

Fig. 28 is a flowchart for determining arrangements and dimensions of rubber gasket, prestressing cable and shear key that constitute the members of a flexible joint. The design and study based on this flowchart will bring a required level of rigidity, resistance and watertightness. Even when encountering ground displacement and earthquake force that are unforeseeable at the stage of the design, the deformation of flexible joints will reduce damages inflicted on the tunnel.

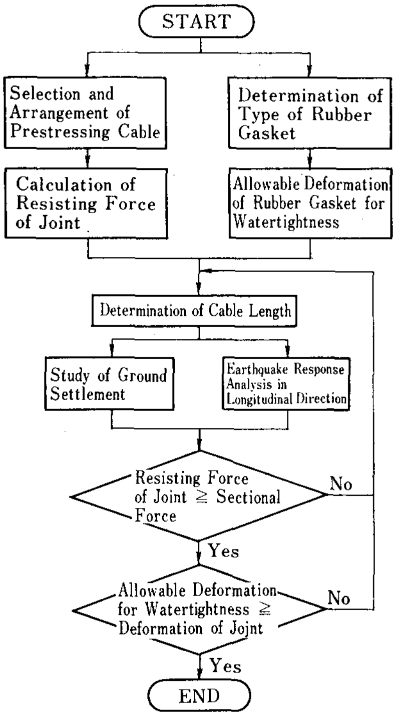


Fig. 28 Design flowchart of flexible joint

5. RESEARCH ACTIVITIES

Earthquake resistance of the submerged tunnel is examined by earthquake observations, dynamic model tests and earthquake response analysis.

5.1 OBSERVATION OF EARTHQUAKE GROUND MOTIONS

Deformation of the submerged tunnel during earthquakes is determined mostly by the variation of the earthquake ground motions along the tunnel. This is because that the tunnel has relatively small apparent unit weight and its rigidity is not so large in comparison with the surounding ground. As the tunnel is not heavy, response of the tunnel itself is not remarkable. As the tunnel is not so rigid, it deforms almost as the ground deforms. Therefore, uniform ground motions along the tunnel does not produce too much effect on the tunnel. However, variations of the ground motions along the tunnel, in other words, relative motions between different parts of the ground have large influence on the tunnel. From this point of view, knowledge of the earthquake ground motions is very important and useful for earthquake resistant design of the submerged tunnel.

The variations of ground motions are caused by two reasons. The first reason is propagation of seismic waves in horizontal direction. The second reason is difference of ground contitions. If ground conditions of two separate sites differ each other, response of the grounds will differ even to the same input motions from the baserock, and the difference causes relative motions between the two sites.

Fundamental data to increase the knowledge of the earthquake ground motions from view point described previousely can be provided by earthquake observation recording ground motions at several points simulteneously. The earthquake observation of this type is called as the seismic array observation and the instrumentation for the observation as the seismic instrument array or simply the array.

The seismic array observations have been performed by several researchers and organizations in Japan and provided useful information for the earthquake resistant design of the submerged tunnel. Many seismic instrument arrays are now under operation. The seismic array observations in Japan will be introduced hereafter.

(1) Vertical One Dimensional Array

A simple type of array to study the earthquake response of the ground is a vertical one dimensional seismometer array which consists of several borehole seismometers located at different levels in the ground, In this observation, mainly, the seismic waves propagating vertically in the ground are taken into consideration. The observations of this type have long history and have been carried out at many sites by different researchers. Followings are examples of the observations.

Okamoto and Kato installed two borehole seismometers at levels of 0.5 and 37 m below the ground surface in Yasuura, Chiba Prefecture in 1960.[1] The ground up to about 35 m below the ground surface at the site consisted of layers of fine sand and silt of which N-values ranged between 0 to 10. At 33.8 m a medium grain sand layer of N-value exceeding 40 appeared. Six earthquake events were recorded in 1960 and 1961. Fig. 29 shows one of the records obtained on October 3, 1960. The scale for amplitude in the figure is showing amplitude on the recording paper; however, the overall sensitivities of both seismometers were identical and it is possible to compare directly the amplitudes of both time histories. The peak accelerations were 9 gals at 0.5 m and 2.4 gals at 37 m. Followings are main findings from the analysis on the six records:

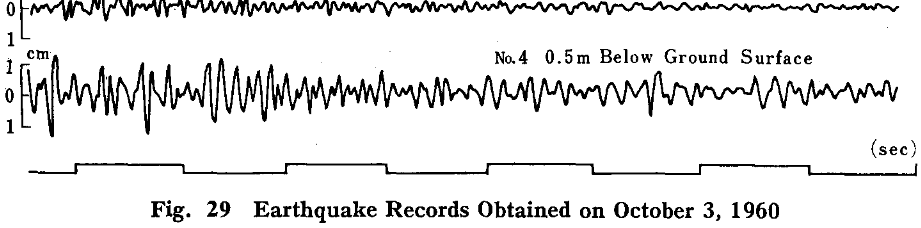


Fig. 29 Earthquake Records Obtained on October 3, 1960

a. The ground motions at 0.5 m had time lag of 0.13 second in relative to those at 37 m. Therefore, in average, velocity of wave propagation of the ground was found to be 280 m/s.
b. Ratios of peak accelerations between 0.5 and 37 m were in the range of 2 to 4.
c. In microtremor at the site, component of about 2.5 Hz was predominant. This frequency is considered to be the natural frequency of the first mode oscillation of the ground. The component of this frequency was observed in the earthquake records of large peak accelerations, but it was not clear in the records of small peak accelerations.

Sakurai, et al. installed vertical seismometer arrays at two sites in Matsushiro where many earthquakes of moderate magnitudes took place as the Matsushiro Earthquake swarm. The swarm was located in the central part of Honshu Island.

The Port and Harbour Research Institute (PHRI) has been operating many vertical one dimensional seismometer arrays and at present PHRI is operating the array in Funabashi, Tokyo, and Yokosuka.

The Public Works Research Institute (PWRI) also has been operating vertical one dimensional seismometer arrays in Futsu, Kawasaki, and Yokosuka.

(2) Horizontal One Dimensional Array

In some observations, seismometets were deployed straightly along the ground surface to study the horizontal propagation of the seismic waves and the relative motions of ground.

Sakurai, et al. made observation of this type during the period of the Matsushiro Earthquake swarm in 1966 through 1970 and existence of the surface wave propagating on the soft alluvial layer was confirmed.

Tamura, et al. made observation of this type at a site in Tokyo, where the surface layer

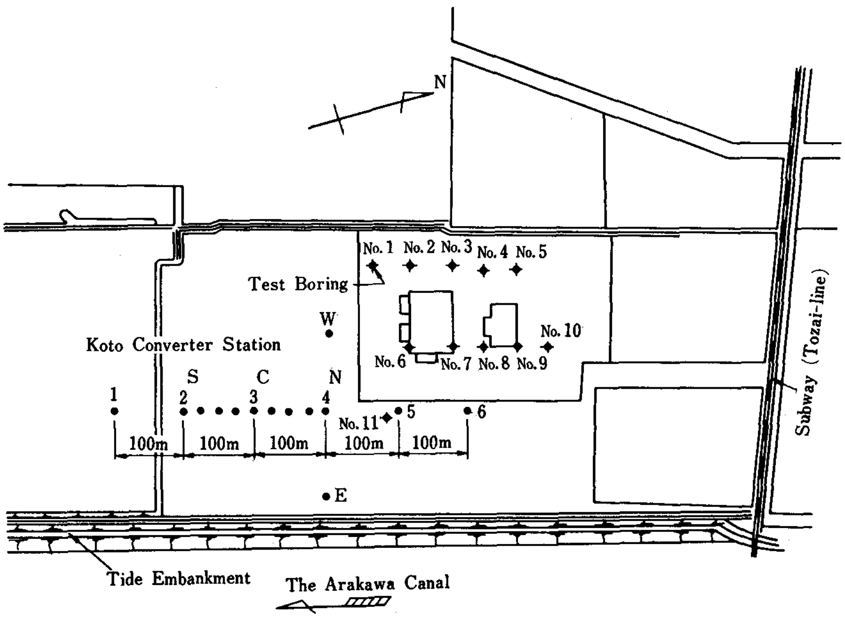


Fig. 30 Locations of Seismometers of Array by Tamura, et al. in Tokyo

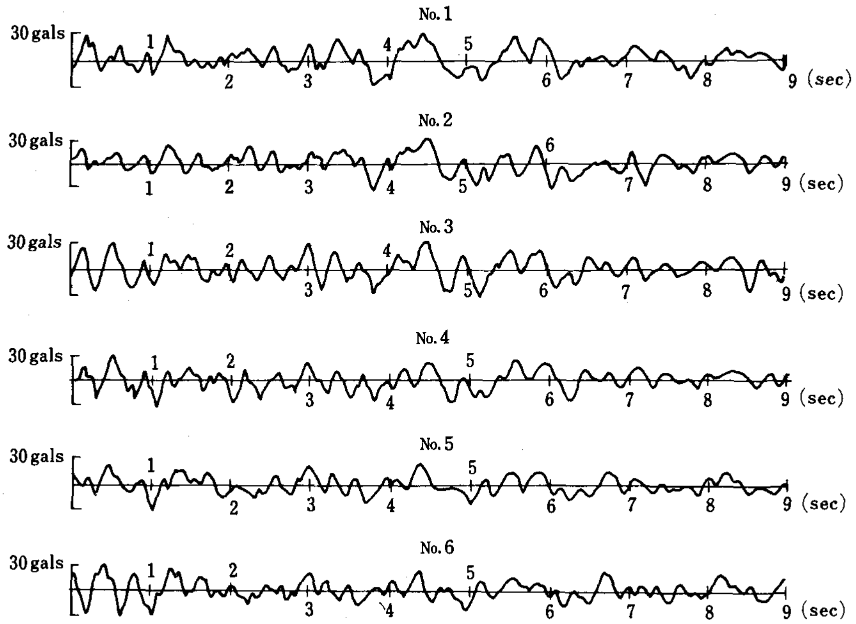


Fig. 31 Acceleration Records of Earthquake of July 1, 1968 from Array by Tamura, et al. in Tokyo

is almost uniform in thickness. The deployment of the seismometers is shown in Fig. 30. Fig. 31 shows acceleration records at the six points located at every 100 meters on the straight line on the ground surface as it is shown in Fig. 30. The earthquake in Fig. 31 was 6.1 in magnitude, the epicentral distance was 52 km and the depth of origin 50 km. The number shown on each records corresponds to the order of the measuring points arranged from one end to the other end. From these records it is noted that the waveforms of relatively longer preiod vibration component are similar at all the measuring points. The displacement records obtained during different earthquakes had the same tendency. It was found that the predominant frequency of 0.8 Hz was corresponding to the observed predominant frequency of the ground. From the cross-correlation between the records, the propagation velocity of the earthquake wave along the surface of the ground was estimated as 2.6 km/s, 2.9 km/s, or more.

(3) Vertical Two Dimensional Array

PHRI and Second Port & Harbour Construction Bureau deployed the seismometers as shown in Fig. 32 at a site in the Tokyo International Airport (Haneda). The array at Haneda, which was constructed as a two dimensional array in 1974, has renewed since 1988 to be three dimensional one at the time of reconstruction of the airport. The properties of waves such as wave velosity, wave direction and so on can be obtained by this array observation. Also, behavior of buried road tunnel under runway is observed. Outline of the road tunnel is shown in Fig. 33 (a). Total length is 812 m. Instruments are deployed at three tunnel sections; A, B and C section. Deployment of instruments at C section is shown in Fig. 33 (b) and total number of instruments is presented at Table 1.

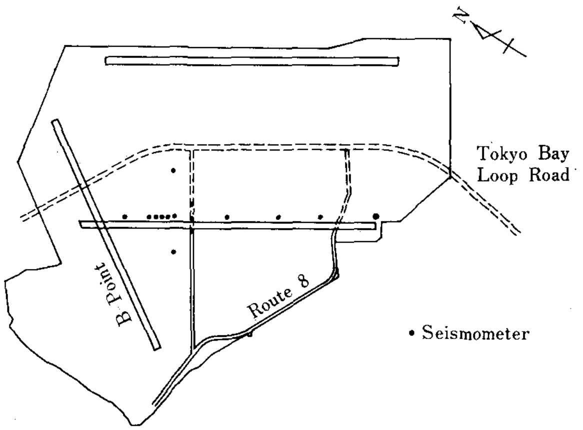


Fig. 32 Seismometer Locations of PHRI in Tokyo International Airport

Fig. 34 shows an example of obtained record in Feb. 1992.

Fig. 35 shows observation systems used at Sodegaura dense array site, about 50 km east of Tokyo, for measuring seismic behavior of embedded pipes. Continuous steel pipe with a diameter of 15 cm and ductile pipes with diameters of 15 cm and 30 cm are embedded 1 m below the ground surface. The length of the continuous steel pipe is 120 m and there is a bend at 20 m from the right end. The length of one element of the ductile pipe is 5 m in the 15 cm diameter pipe and 6 m in the 30 cm diameter pipe. They were jointed together by expansion joints which are capable of absorbing a relative deformation of $\pm 10\%$ of pipe length in the axial direction and a rotation of ± 6 degrees. Stoppers for preventing excessive pulling-out are provided at the expansion joints.

Fig. 36 shows the measured axial strain induced in the steel pipe and ground accelerations developed by an earthquake with epicentral distance of 27 km and an earthquake magnitude of 5.7. The peak ground acceleration was about 34 to 146 cm/sec^2 and the peak pipe strain was about 2.2 to 21.5 μ. It is interesting to note in Fig. 39 that although there are many similarities of the pipe strains between the three locations, the pipe strain are considerably different each other. This clearly shows that the pipe response in axial direction depends considerably on the response of nearby soil.

Fig. 37 shows the axial strain of steel pipe vs. peak ground acceleration relation for the data obtained in the past.

As shown in Fig. 38, the pipe is idealized by a beam elastically supported by springs which represent pipe-soil interaction. Soil response is computed by prescribing the input ground motion at the base rock.

Fig. 39 shows the computed pipe strain in comparison with the measured data. Good correlation can be obtained by assuming the spring stiffness of 100 tf/m^2~300 tf/m^2.

Katayama, et al. established a three dimensional seismometer array, a strain measurement system comprising ground strain transducers and buried pipes on which strain gauges are mounted. Fig. 40 shows the layout of the array and the strain measurement system.

Fig. 41 shows the ground strain observed at the ground strain transducer G3 and the evaluated ground strain. Fig. 42 shows the pipe strain observed at gauge SS2 and the evaluated pipe strain. These strains were evaluated using a triangular element with known displacements at the nodal points. The displacements are obtained by double integration of the acceleration records for the December 17, 1987 earthquake which had a magnitude of 6.7 and an epicentral distance of 58 km.

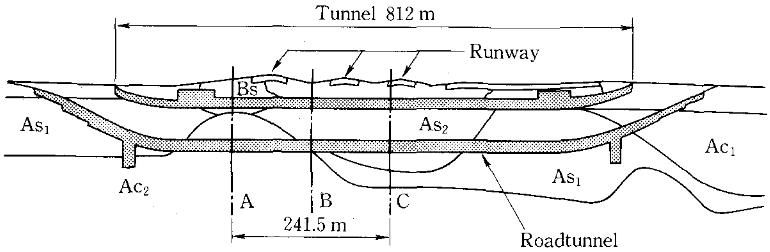


Fig. 33 (a)

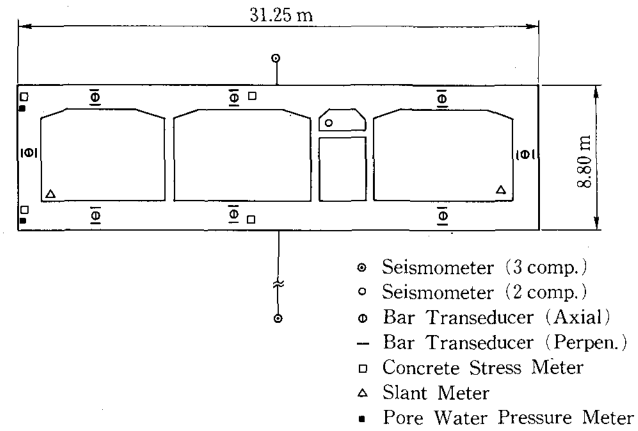


Fig. 33 (b)

5.2 EARTHQUAKE OBSERVATION OF SUBMERGED TUNNELS

Earthquake observations at submerged tunnels began with Haneda Tunnel completed in 1964. Now, at most highway and railway tunnels completed since 1970, systematic

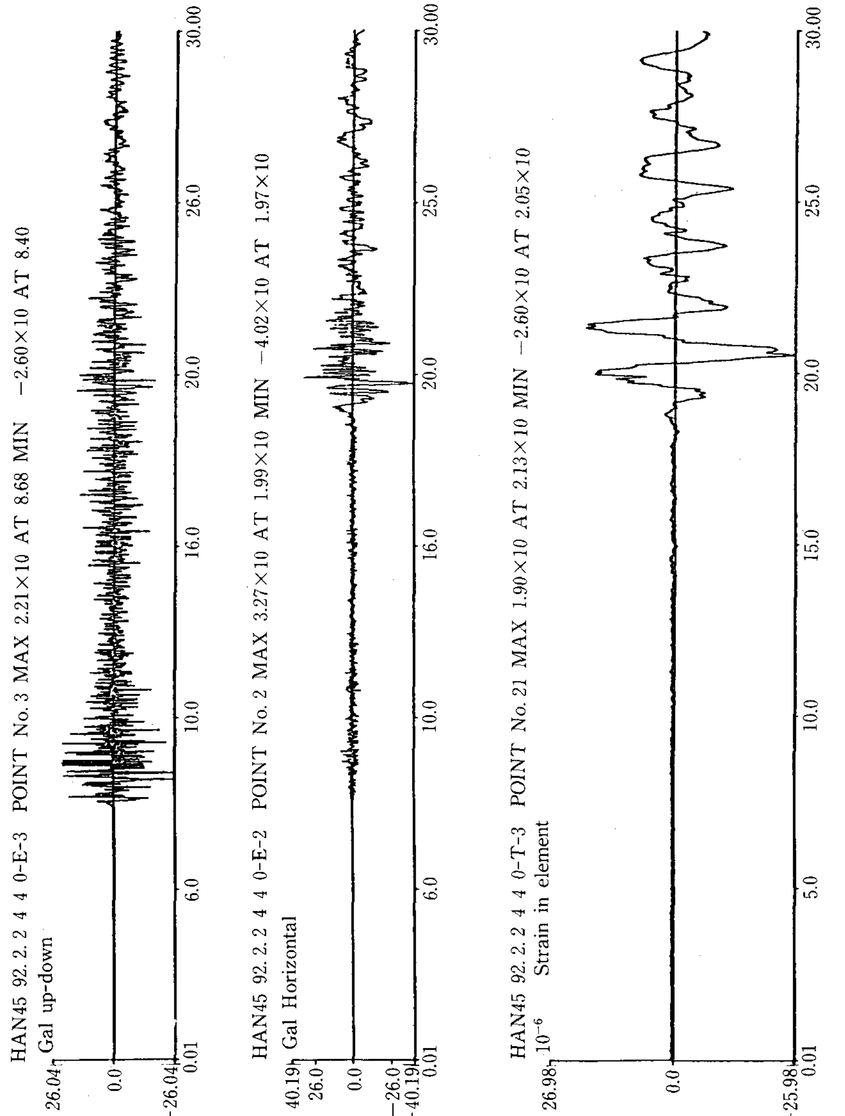
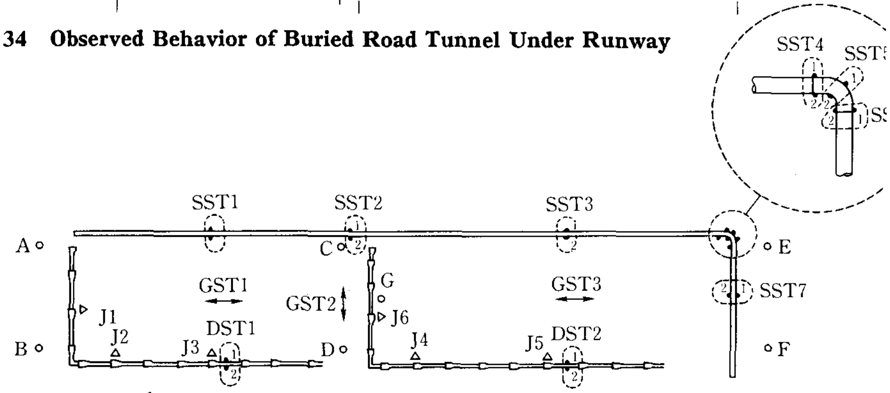


Fig. 34 Observed Behavior of Buried Road Tunnel Under Runway



Legend
- △ Displacement of Joint (J1~J3)
- • Strain of Pipe (SST1~SST7, DST1~DST3)
- ○ Down-hole Accelerometer (A~G)
- — Strain of Ground (GST1~GST3)

Fig. 35 Observation Systems

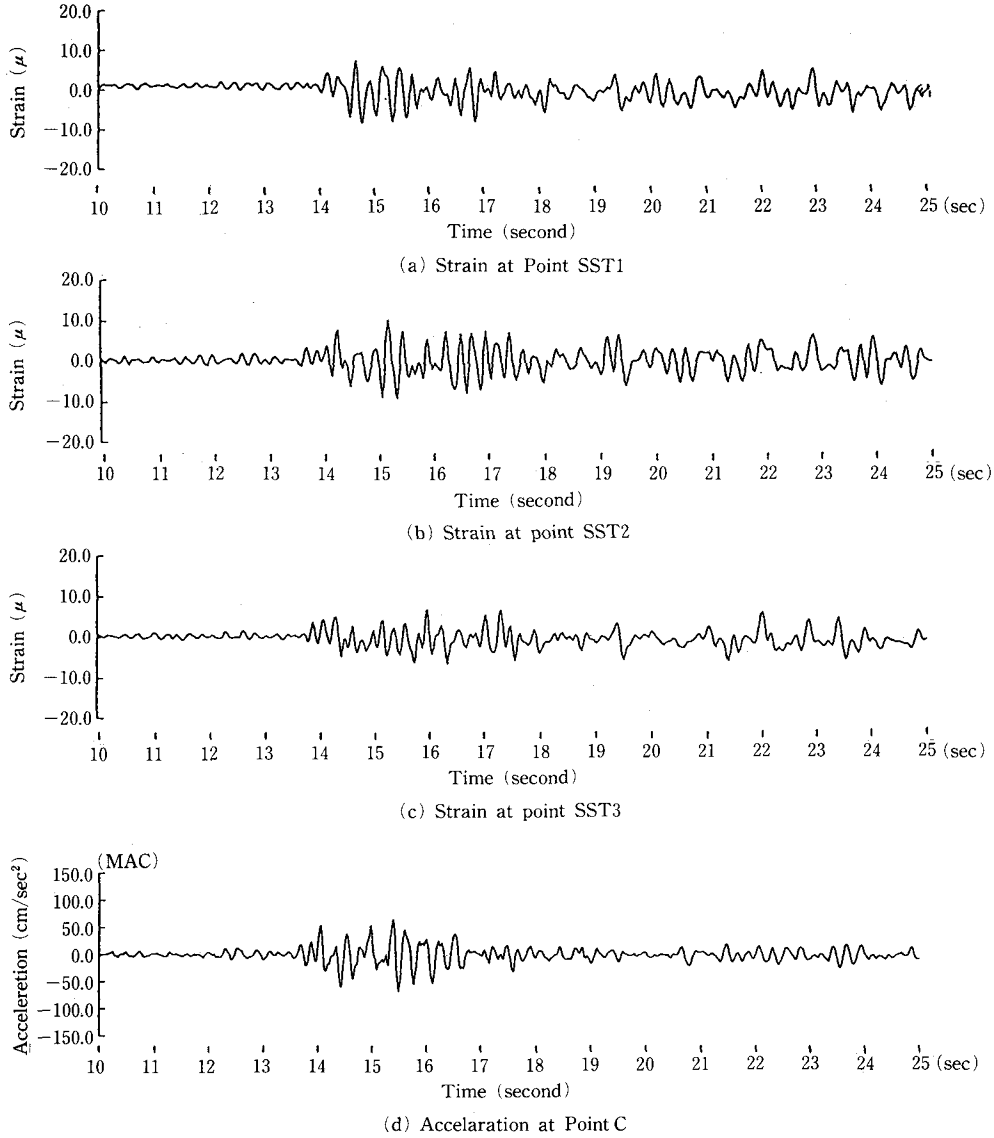


Fig. 36 Measured Axial Strain of Steel Pipe and Ground Acceleration at C Point During EQ13

earthquake observations are being carried out for purposes of maintenance and administration, and for investigation and research concerning behaviors during earthquakes. Observations were started from early stages at Keiyo-line Haneda Tunnel (Tama River), Kinuura Port Tunnel, Tokyo Port Tunnel, Ohgishima Tunnel, and others, the objects of measurements being the behaviors of the tunnels and of ground around the tunnels. Accelerometers are buried in the ground, and accelerometers and displacement meters at the tunnels, while moreover, strain gauges are installed at tunnel walls to measure deformations during earthquakes. There are also cases where relative displacement meters are installed to measure displacements at joints between elements and between elements and ventilation towers.

It has been succeeded in recording many earthquakes at the various tunnels up to this time including the recent Izu Oshima Kinkai Earthquake (M=7.0) of 1978. Focusing especially on strains at tunnels during earthquakes, the principal findings from observations until now may be summarized as follows:

1) The strains in a tunnel during earthquake may be broadly divided into strains due to predominant vibrations of the surrounding ground of the tunnel (surface layer ground) caused by body waves, and strains caused by surface waves of longer

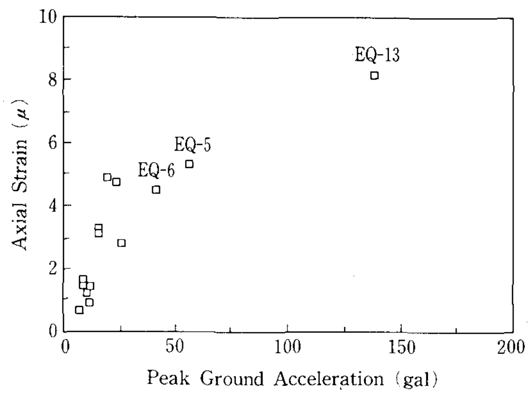


Fig. 37 Axial Strain of Steel Pipe vs. Peak Ground Acceleration Relation

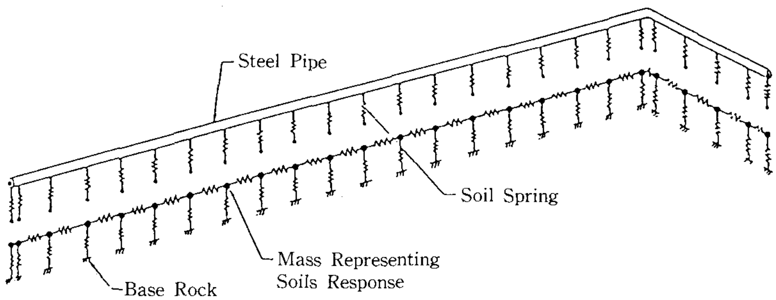


Fig. 38 Analytical Model for Dynamic Response Analysis of Embedded Pipes

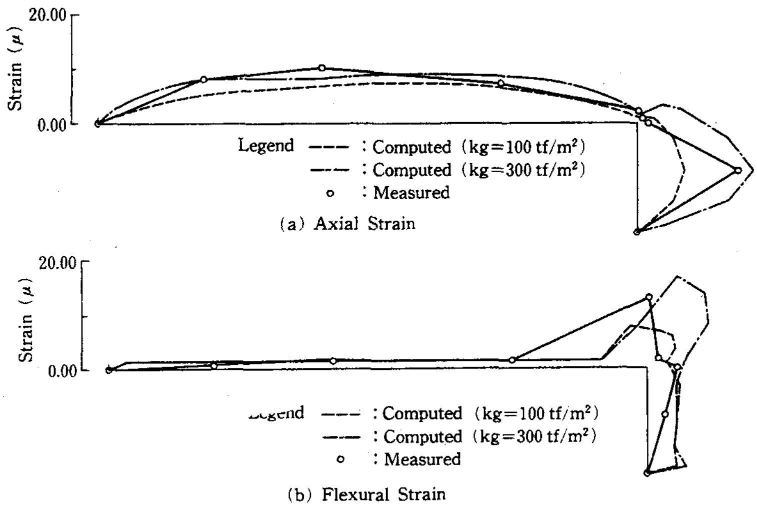


Fig. 39 Best Correlation of Pipe Strain for EQ13

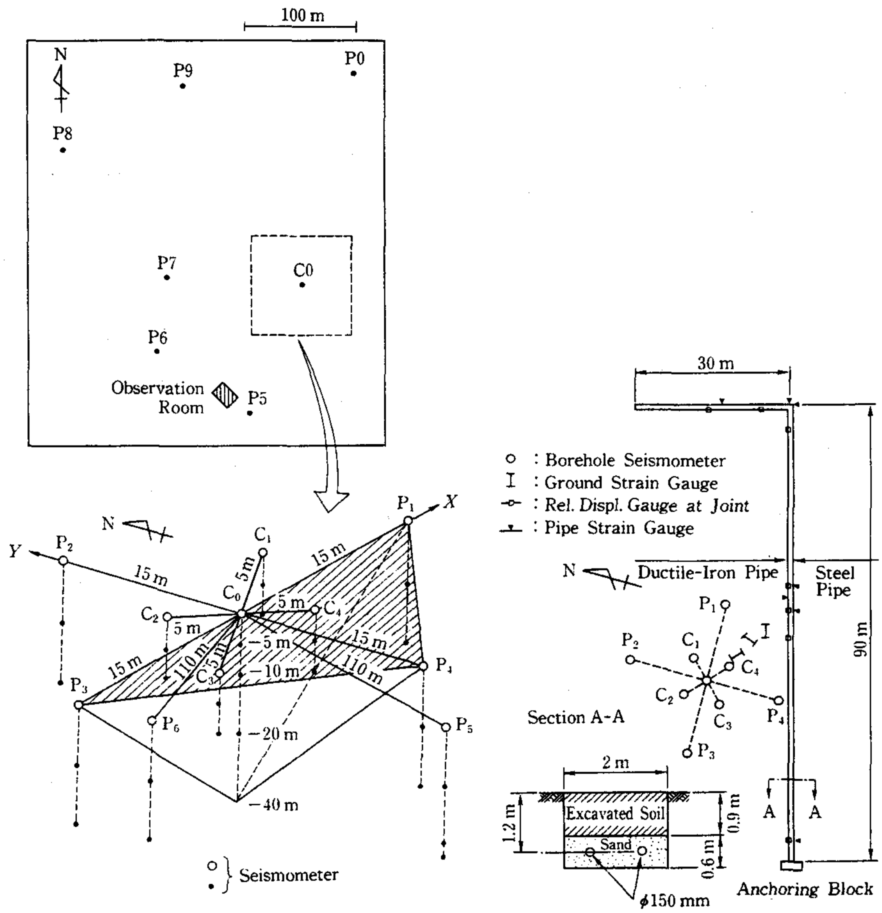


Fig. 40 Layout of Array of Seismometers and Strain Mesurement System of the Institute of Industrial Science, University of Tokyo, in Chiba

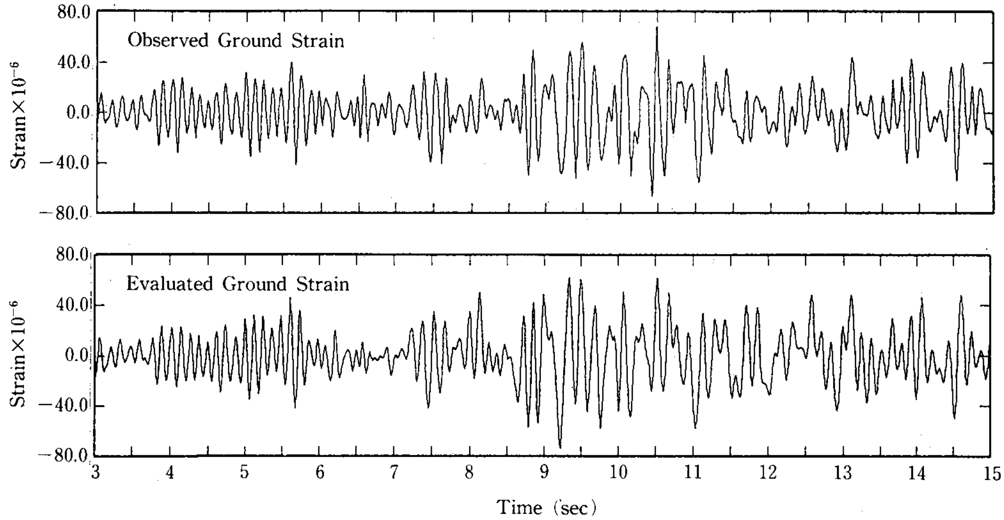


Fig. 41 Observed Ground Strain and Ground Strain Evaluated from Acceleration Records at the Array of Institute of Industrial Science, in Chiba for December 17, 1987 Earthquake

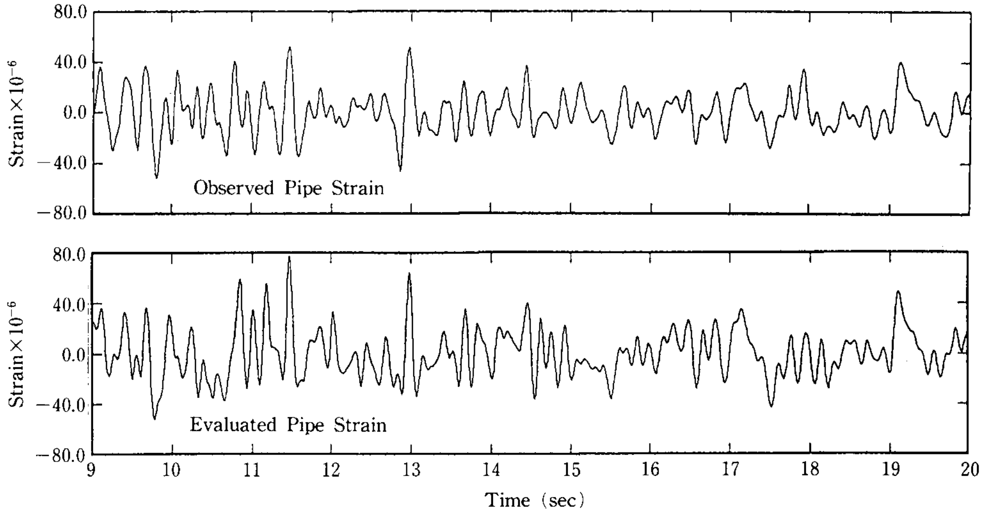


Fig. 42 Observed Pipe Strain and Pipe Strain Evaluated from Acceleration Records at the Array of Institute of Industrial Science, in Chiba for December 17, 1987 Earthquake

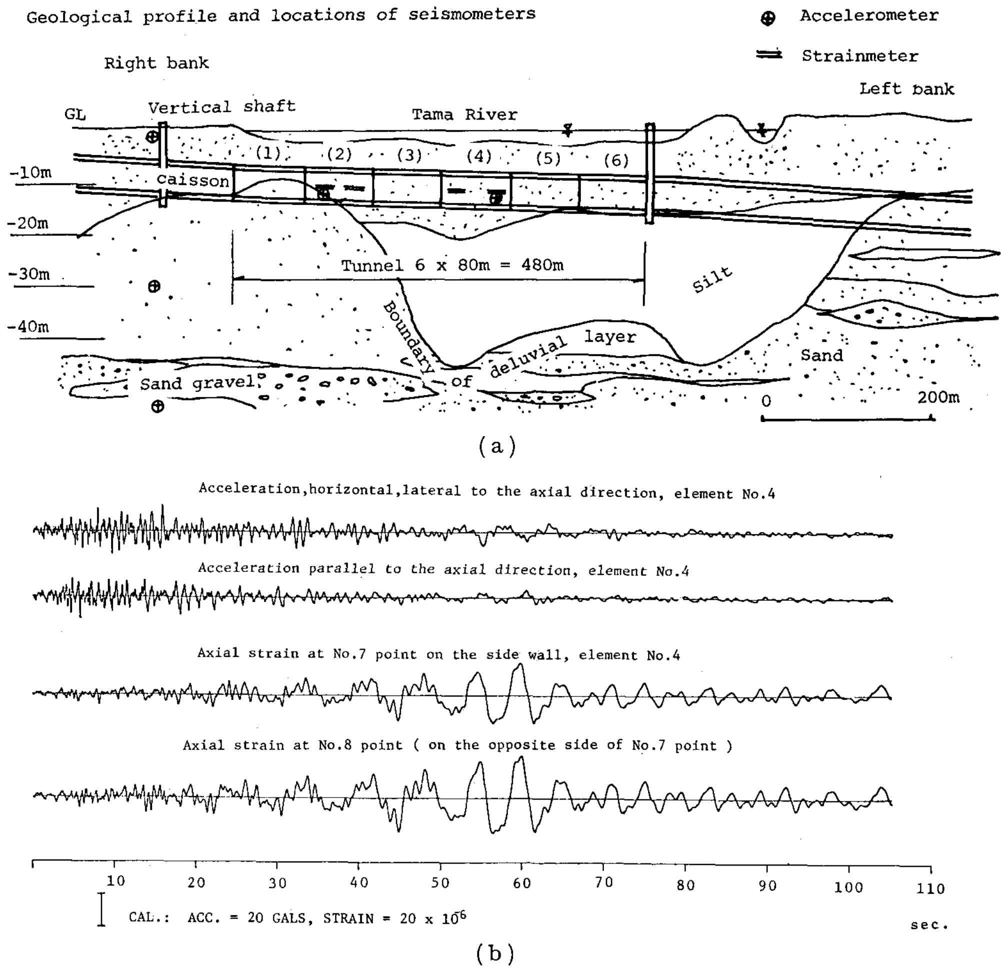


Fig. 43 Earthquake Records at Keiyo-line Haneda Tunnel (Tama River) (Jan. 14, 1978)

periods.

2) The deformation of a tunnel due to the predominant vibrations of the ground is affected by factors such as periodic characteristics of earthquake motions incident at the surface layer ground and the low-order predominant period of the surface layer ground.
3) A tunnel behaves in correspondence to the behavior of the ground during earthquake, and is not seen to indicate self-vibration. This trend may be discerned more clearly in the behavior in the axial direction of the tunnel. However, in case the behavior of the ground changes abruptly in an extremely localized area, the tunnel may show a behavior differeing from the movement of the ground.
4) With respect to deformation of the tunnel, that in the axial direction is generally predominant.

Fig. 43 is the record of the 1978 Izu Oshima Kinkai Earthquake (M=7.0, epicentral distance approximately 100 km) obtained at Keiyo-line Haneda Tunnel (Tama River). One of the features of this earthquake is that distance attenuation of acceleration was prominent and earthquake damage was localized, surface waves of periods about 7 sec seen to be Love waves being predominant at this tunnel. In this figure, strains of the tunnel due to predominant vibrations of the surface layer ground amplified by body waves, and strains due to surfaces waves appearing from after 20 sec can be clearly seen. In looking at the figure, it is necessary to keep in mind that the amplitude of surface waves does not become so large, but acceleration of body waves become large, as the epicentral distance shorter.

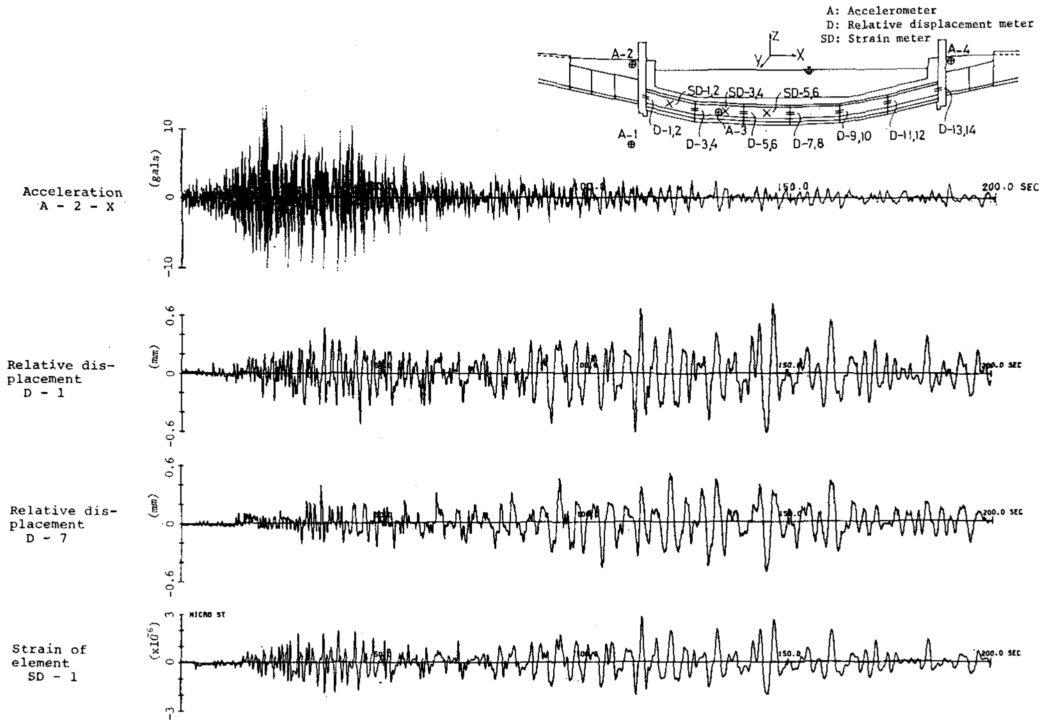


Fig. 44 Eathquake Records at Tokyo Port 2nd Fairway Tunnel (June 23, 1982)

Fig. 44 is the record of the earthquake of June 23, 1982 (M=7.0, epicentral distance 210 km) obtained at Tokyo Port 2nd Fairway Tunnel. The symbols of waveforms, as shown on the longitudinal section of the tunnel, indicate the acceleration at the ground surface, relative displacements of joints at points D_1 and D_7, and the axial-direction strains of the element at points SD-1. In this record also, with approximately 50 sec the boundary, the strains due to predominant vibrations of the ground are main in the first part and those due to surface waves of longer periods at the latter. It can be seen that the strain waveform of the element is composed of two main parts. As for relative displacements of joints, when they are viewed in relation to strains of elements, they indicate large values, which shows the usefulness of the joints.

Fig. 45 is the relation between maximum acceleration and maximum value of axial-direction strains obtained at Keiyo-line Haneda Tunnel (Tama River). This is shown with magnitude as the parameter, and it may be seen that when magnitude is maintained constant, maximum axial strain is not proportional to maximum acceleration.

Fig. 46 shows the relation between epicentral distance and maximum axial strain in case of earthquakes of magnitude 5. The lines in this figure shows the observation results of Kinuura, Tamagawa, Kawasaki, and 2nd Tokyo Port. They roughly indicate the same degrees of distance attenuation.

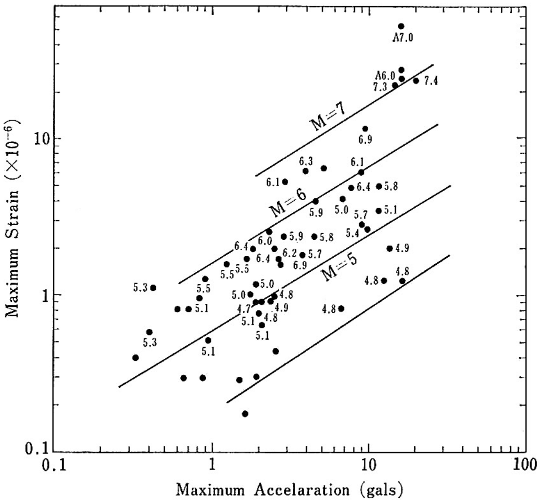


Fig. 45 **Relation Between Maximum Acceleration and Maximum Axial Strains at Keiyo-Line Haneda Tunnel (Tama River)**

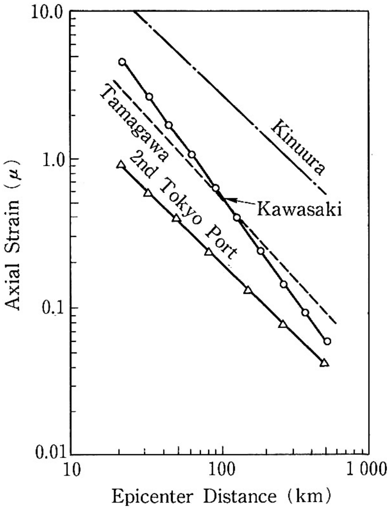


Fig. 46 **Relation between Epicentral Distance and Maximum Axial Strains in Earthquake of Magnitude 5**

5.3 MODEL TEST

Dynamic behaviors of the tunnel and of the ground and their interaction during earthquakes are examined by two types of dynamic tests on the model of the tunnel. One is

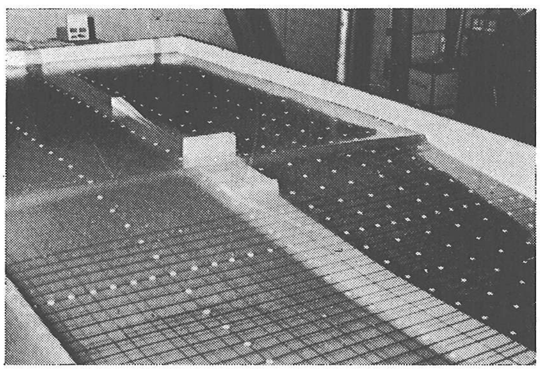


Photo 1

a vibration test of three dimensional model performed in the laboratory. The models are made from a material of low Young's modulus and tested on a shaking table. The behaviors of the tunnel and of the surface layer are studied. Although the tests of this type are usually made in the range of elasticity, results are very useful to get information on the behavior of the tunnel and of the ground in cases of complicated geological condition (Photo 1).

The other type of test is performed on the model tunnel in the field to obtain an information on the interaction between the tunnel and the real ground in the seismic condition. The model of the tunnel is usually made from the vinyl pipe or the steel pipe with a large diameter and is buried in the ground. The ground near the tunnel is excited by a special equipment, weight dropping, explosion and so on.

Main results from these model tests are summarized as follows:

1) The submerged tunnel does not vibrate with its natural vibration but with the natural vibration of the ground. Usually, natural vibration of the ground is not affected by the existence of the tunnel.

2) When two ground conditions of different dynamic characteristics exist along tunnel, the predominant vibration of one ground usually extends to the transition area between these ground conditions and the behavior of the ground is scarcely affected though this depends on the vibration mode. When the tunnel crosses the transition area, it may be deformed both in the axial and in the lateral directions.

3) When the tunnel is connected to another structure with different dynamic characteristics, flexible connections are effective to decrease the stress within the tunnel near the regions of such joints during earthquakes.

In addition, laboratory model tests on the materials used at the joint have been performed for the detailed design of joints. Usually, mechanical properties, workability, and waterproof of elements or materials to be used at the joint, for example, water sealing materiais, were studied by model tests.

5.4 RESPONSE ANALYSIS

From the results of earthquake observations and of dynamic model tests, it has been made clear that the dynamic behavior and the stability of the surrounding are important for earthquake resistant design of submerged tunnels and that the primary factor for the design of tunnels is not the ground acceleration but the ground displacement, especially the relative displacement along the tunnel axis during earthquakes.

Based on these facts, a few of models to calculate seismic response of ground have been proposed. The response of the ground and the tunnels can be calculated in time series by numerical integration. One of the most useful and convenient calculation models is mass-spring model shown in Fig. 47. This model has been used for the design of several submerged tunnels. In the model, the surface layer above baserock is assumed to vibrate in shear mode. The surface layer is divided into a number of slices perpendicular to the tunnel axis. Each slice is represented by an equivalent mass-spring system. The system consists of a mass representing mass of slice and a spring and a dashpot connecting the mass to the baserock. The spring constant of K_3 is determined so as the natural period of the system coincides with the natural period of the first mode of shear vibration of the

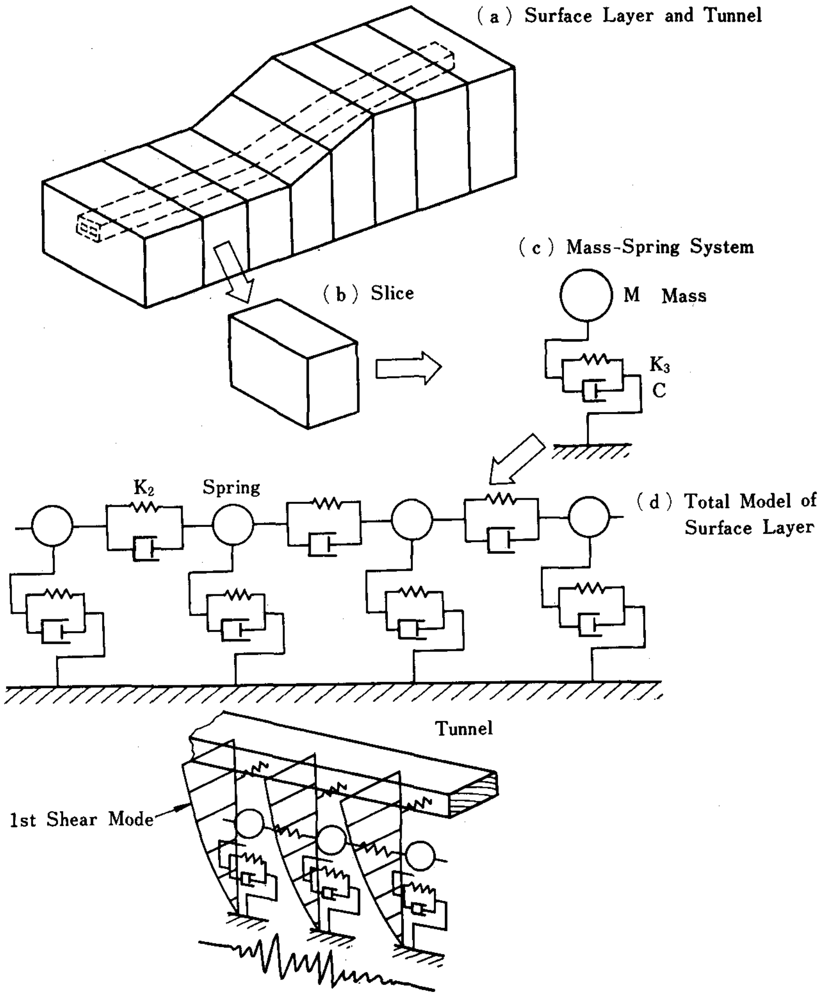


Fig. 47 Mass-Spring Model

slice. Then the neighboring masses are connected each other along the tunnel axis by springs and dashpots (Fig. 47). The spring constant of K_2 is related to the push-pull resistance to the axial relative displacement between adjacent ground slices or the shear resistance to the lateral relative displacement. The spring constants have to be changed depending on direction in which the response is going to be calculated. With the model, response of the surface layer can be calculated. The tunnel is assumed as a beam supported by springs representing soil rigidity. One end of each spring is considered to displace same as the displacement of the ground calculated by the total model of surface layer. In this way seismic response of the tunnel can be calculated. The property of this model is confirmed by the comparison between the calculation results and field observation results (Fig. 48). The effect of the flexible joints or the effect of the existance of the ventilation towers can be estimated by this model.

The recent research works have developed the above mass-spring model

① The rigidity and damping properties of the surface ground change during the vibration of the ground. ② The earthquake waves propagate apparently along the tunnel axis according to the field observations. ③ The sliding occurs between the tunnel and the ambient ground when the large friction force is applied on the surface of the tunnel.

These properties can be taken into consideration in the mass-spring model and the finite element model. The two mass-spring system: the slice of surface layer is replaced

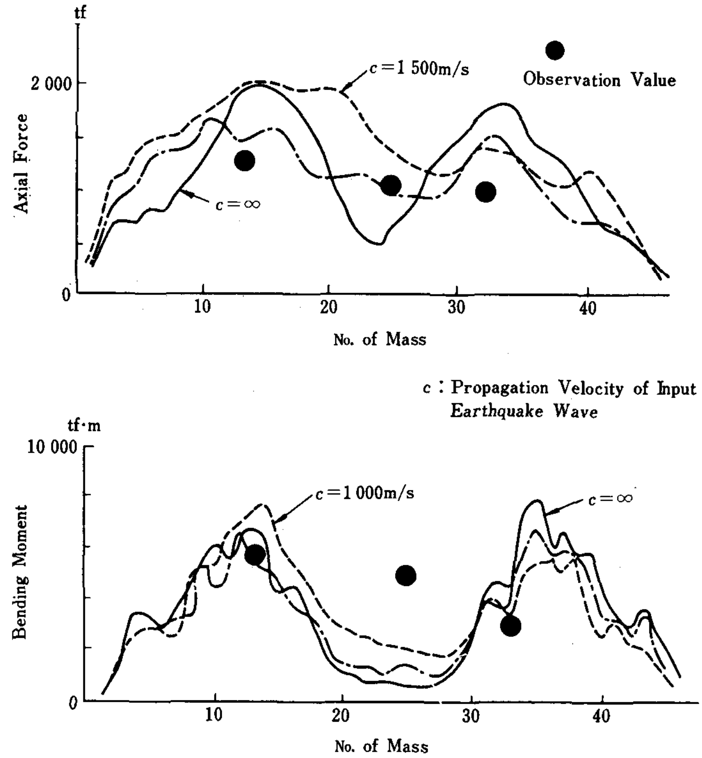


Fig. 48 Caluclation Result and Observation Result

by two masses are proposed. The second shear mode is taken into this model. Mass spring system of these dimensions are also proposed. The response of the ground and the tunnels can be estimated more precisely than two dimensional mass-spring system.

The another useful calculation model is the finite element model. Block elements or two dimensional plain-strain elements are used for the ground and beam elements are used for the tunnel. The response of the submerged tunnel and the surface layer are calculated at the same time. Large capacity and long time are required when a lot of finite elements are used for the calculation model, therefore, a few devices were proposed to reduce the degrees of freedom in the model. As the computers with large capacity are widely used today, the calculation by the finite element model which consists a large number of nodes and elements have gradually become possible.

According to recent research works, surface waves besides body waves play important roles on the vibration of the tunnel and the ground. Some researcher try to develop the calculation model considered surface waves. It will be possible to estimate the vibration property of the submerged tunnels due to surface waves in the near future.

Calculation model above mentioned is effective for the surface layer where the soil condition and topography are almost uniform. For complicated surface layer in a three dimensional expanse such as a drawn valley or basin, three dimensional expanse should be considered in the calculation model. The mass-spring model shown in Fig. 47 was extended to estimation of three-dimensional soil-tunnel interaction[1]. A scheme of the model is illustrated in Fig. 49. An alluvial deposit on an undulating bed rock is divided into vertical soil columns (Fig. 49 (a)). and each column is replaced by a one-lumped-mass-spring system taking account its fundamental mode of shear vibration. Then a net of finite elements is used to link these oscillators together to be a model of the quasi-three-dimensional alluvial surface layer (Fig. 49 (b)). The finite elements represent the equivalent column-column interaction. O. Kiyomiya applied this calculation model into the seismic design of the planning submerged tunnel which will be constructed in a drawn valley. Fig. 50 shows the element mesh of the ground. Fig. 51 shows the calculation results of time history as to ground horizontal displacement and sectional forces in the submerged tunnel. Fig. 52 shows the differences of distribution of force along tunnel axis among three dimensional model, two dimensional model and seismic deformation model.

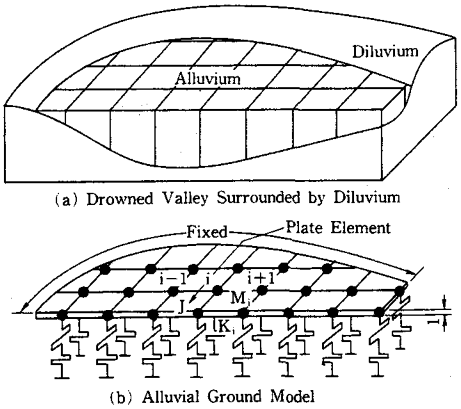


Fig. 49 Quasi-three-dimensional ground model

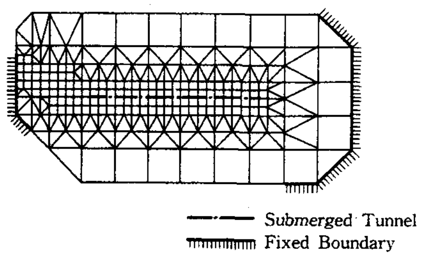


Fig. 50 Element mesh of ground

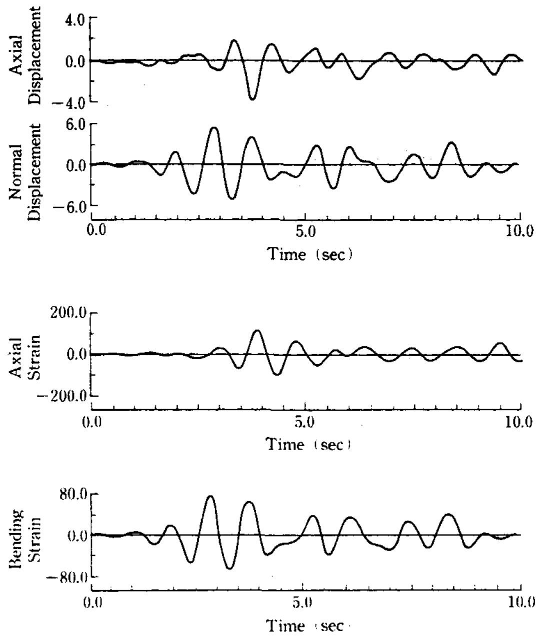


Fig. 51 Calculation Result

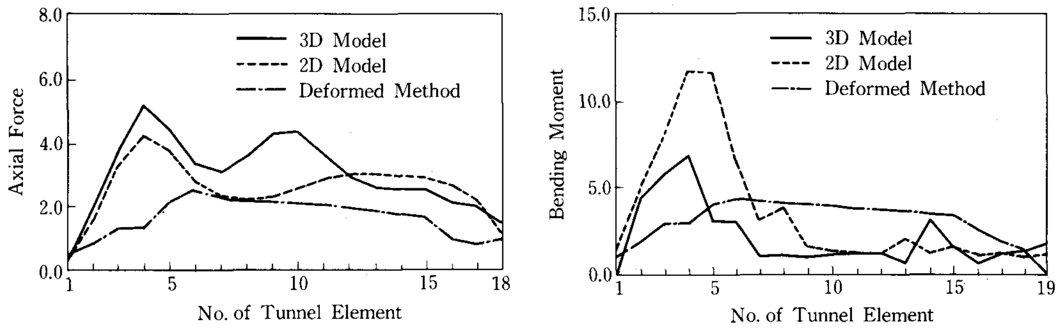


Fig. 52 Distribution of Forces

5.5 FLEXIBLE JOINT

The submerged tunnel is generally situated in relatively soft ground. Therefore, since a rigid joint is often disadvantageous in consideration of earthquake and ground settlement, a flexible joint is sometimes used. For this reason, various studies and analyses have been conducted. Examples of flexible joints are shown in the followings.

5.5.1 Tokyo Port Tunnel

For all the joints between the elements of Tokyo Port Tunnel (refer to P. 194), the flexible structure was adopted to reduce the impact of earthquake and to enhance the adaptability to un-uniform settlement. And for the joint between the shaft and the element, a movable mechanism was also employed, because the shaft and the element behave in different manner.

In designing the flexible joint between the elements, an Ω shaped steel plate was adopted. This is because ① the joint is expected to be deformed to a limited range and ② has the spring ability allowing sufficient deformation at the time of earthquake, and also absorbs the deformation due to the variation of temperature and ③ can resist the reaction of the compressed rubber gasket.

The joint consists of ① rubber gasket for resisting compressive force, ② Ω shaped steel plates for resisting tensile force, ③ and shear key for resisting vertical and horizontal relative displacement. The details are shown in Fig. 53.

And, since the joint between the shaft and the element needs the function mentioned above, the joint using the rubber gasket and PC. cable was employed. (Fig. 54)

5.5.2 Tokyo Port 2nd Fairway Tunnel

The Tokyo Port 2nd Fairway Tunnel is a four lane vehicular tunnel connecting two reclaimed lands in Tokyo Bay. (refer to p. 201)

In deciding the joint structure of this tunnel, the following three cases were examined.

Case 1 All joints are rigid.

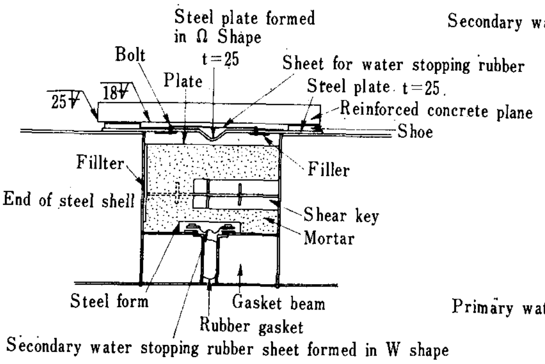


Fig. 53

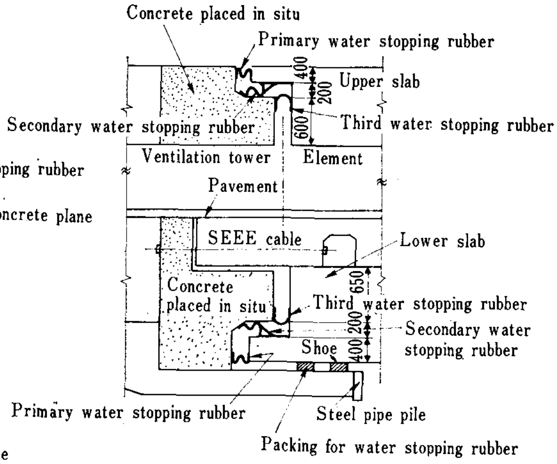


Fig. 54

Case 2 All joints between the elements are rigid.
The joint between the shaft and the element is flexible.

Case 3 All joints are flexible.

Based on the result of earthquake response analysis of these three cases, Case-3 was adopted, because it showed considerable effect, especially concerning the decrease of axial force.

When the designing of the joint was carried out, the following three items were mainly taken into account.

The joint ① has the same strength and durability as the body of the element.
② can be installed from the inside of the element as much as possible.
③ has the structure permitting the decrease of sectional force in the longitudinal direction.

The structure of the joint is shown in Fig. 55.

This joint principally consists of 1) rubber gasket as compression spring element, 2) PC cable as tensile spring element and 3) steel and reinforced concrete shear key resisting vertical and horizontal displacement respectively.

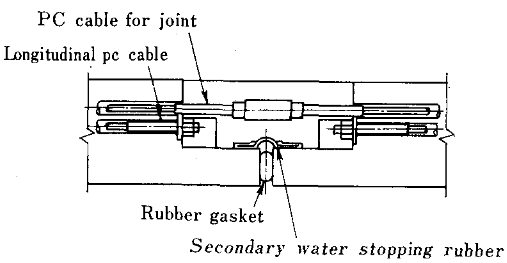


Fig. 55

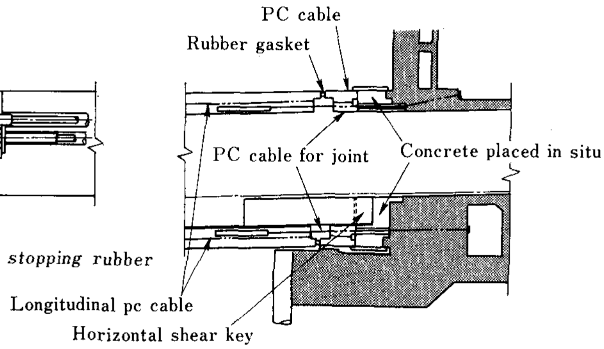


Fig. 56

And, the flexible joint was built in the final element just before its end, prior to its sinking, and the rigid structure was used for the joint between the final element and the shaft. (Fig. 56)

5.5.3 Daiba Tunnel

Daiba Tunnel is a double-tracked railway tunnel with a total length of 5.7 Km located in the Tokyo Port area. Of this tunnel, the length of the submerged part is 672 m. This section was constructed in very soft ground with the both ends in reclaimed lands, the ground settlement due to consolidation was progressing. And the final settlement at the floor level of the tunnel was estimated to be 1.1 m at the time of construction.

For this reason, various types of joint were studied. As a result, the flexible joint was adopted, taking the large ground settlement, the earthquake resistance and construction cost into consideration.

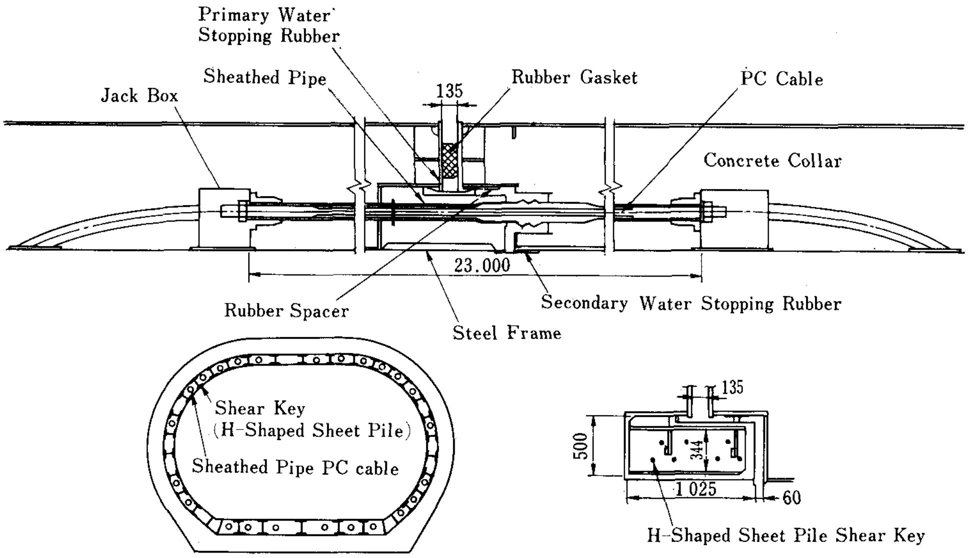


Fig. 57

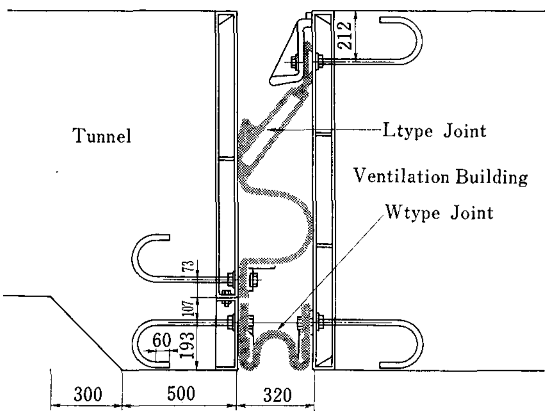


Fig. 58

In order to satisfy the abovementioned function, the behavior of the joint was investigated by settlement analysis of the ground and the tunnel and the seismic response analysis, and the function was finally confirmed by model test. As a result, the joint structure was decided as shown in Fig. 57. The joint consists of rubbergasket for resisting compressive force, PC cable for resisting tensile force, and shear key for restricting relative displacement.

As for the elements at both ends, the flexible joint similar to the joint between the elements was built in advance 5 m from the element end, and the rigid structure was adopted for the joint between the caisson and the edge element.

5.5.4 Kawasaki Port Tunnel

The ventilation buildin g is connected with the tunnel by the flexible joint so as to decrease large sectional force during earthquakes. The flexible joint is used for the ground settlement and displacement due to the earthquakes. This flexible joint consists of W-type rubber gasket and L-type rubber gasket as shown in Fig. 59. The allowable displacement of the flexible joint is ± 15 cm for longitudinal direction, ± 10 cm for transverse direction, and $+5 \sim -20$ cm for vertical direction. The water proof of the rubber gasket is efficient for water pressure within 30 kgf/cm^2. The displacement of the flexible joint is measured every month. The flexible joint has no large displacement according to the measurement result.

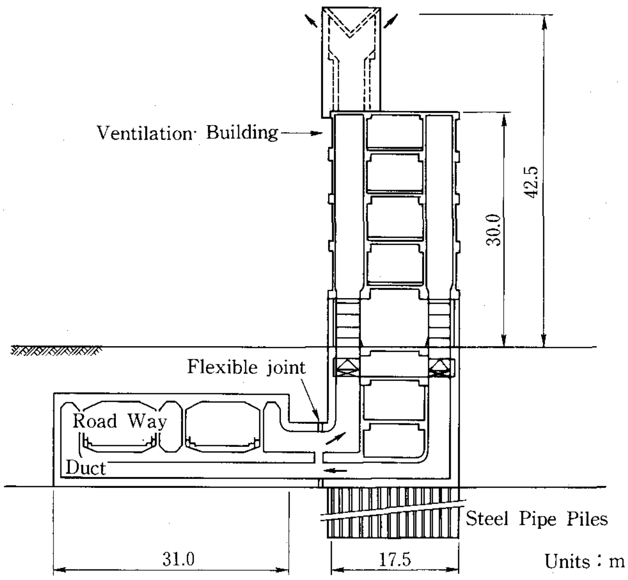


Fig. 59 VENTILATION BUILDING

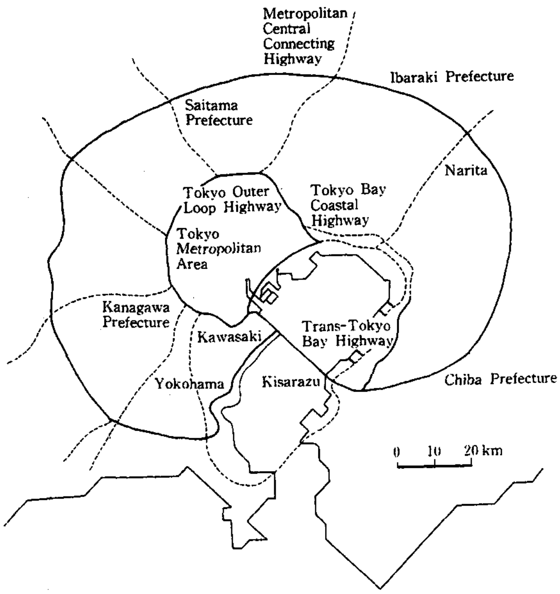


Fig. 1 Tokyo Bay area

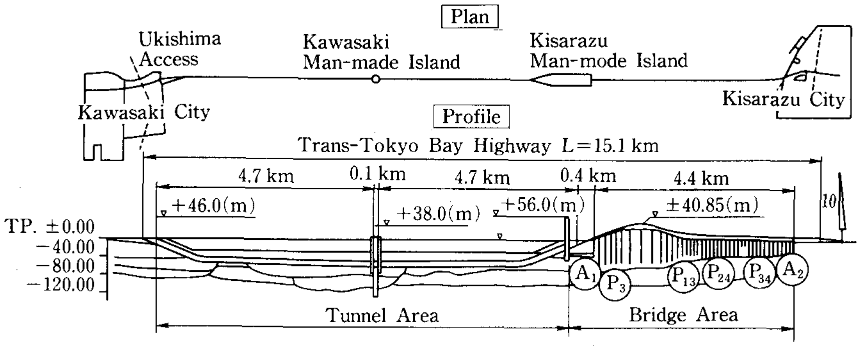


Fig. 2 Project plan and profile

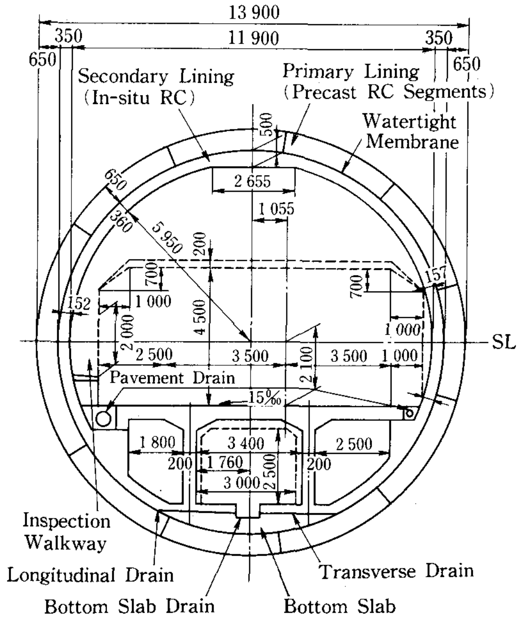


Fig. 3 Tunnel Configuration

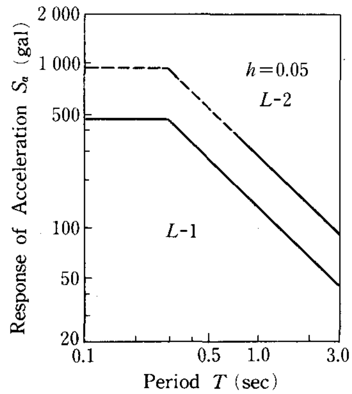


Fig. 4 Spectrums of L1 and L2

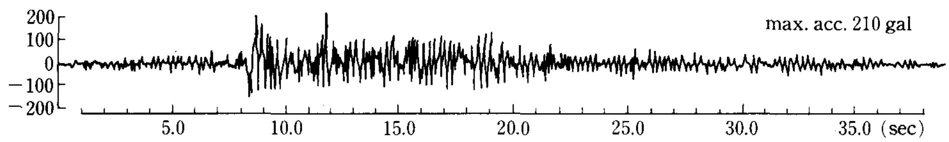


Fig. 5 Synthesized earthquake Wave L1

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