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**Recent Earthquake Resistant Design Methods
for Different Types of Foundations**

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**Foundation Engineering Section,
Structure and Bridge Division,
Public Works Research Institute,
Ministry of Construction**

P r e f a c e

There is growing importance attached to the investigations of earthquake resistance of bridge foundations, for the bridges tend to be larger and larger in scale and there is the increasing necessity for building bridges on poor ground according to the needs of road planning and accordingly many earthquake resistant designs have been developed for bridge foundations. The previous earthquake resistant designs were mainly based on the seismic coefficient method in which the external forces generated during earthquake were calculated in terms of static load, from which safety calculations were made to obtain the safety factor. Later, however, as the expressways developed, increased importance is attached to the road alignment and the construction of high bridge piers has been necessitated, which is beyond the previously used seismic coefficient method. As a result, wide use is now being made of the modified seismic coefficient method in which the effects of the seismic responses of the substructures themselves are also taken into consideration.

However, when bridge foundations become larger and have to be constructed on the poor ground, the previously used concept that the foundation structure makes the same responses as the ground is no longer valid and therefore studies are now being made to investigate the seismic responses of the foundation structure itself, considering also some other characteristics of seismic force than acceleration such

as frequency and time duration. Seismic response analysis has been made on many bridges from such a viewpoint, thus more and more clarifying the seismic resistance of bridge foundation structures.

This paper is devoted to the description of actually made earthquake resistant designs of bridge foundations.

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1. Research Results and Summary

1.1 Type of foundation:

Spread foundation (large-scale)

Setting up of earthquake resistant design objectives:

Selection of earthquake calculation method:

- . Even in the case of a large-scale structure, modal analysis by a simple system with two degrees of freedom is possible because the earthquake behavior of the entire foundation can be examined by assuming it as a rigid foundation.
- . Calculations are made by response spectra, treating the maximum response alone.

Modeling for earthquake resistant design:

- . For the purpose of maintaining the continuity between superstructure and substructure, a part of the weight of superstructure is treated as a weight added to substructure and superstructure and substructure are separated.
- . When substructure is under water, dynamic water pressure during earthquake is to be calculated in terms of virtual mass.

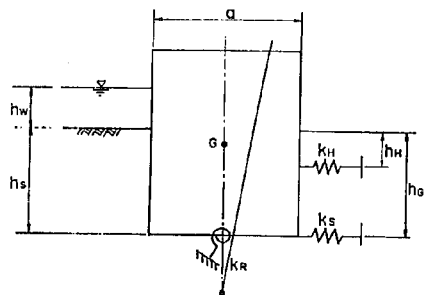
$$M_w = \frac{0.6 \rho A h_w}{(a/h_w)^{1/2}}$$

ρ : bulk density of water

A : sectional area

a : width in the direction of vibration

h_w : depth of water
($0.8 \leq a/h_w \leq 4.0$)



(In the direction of length b)

- . The soil surrounding substructure is to be calculated in terms of virtual mass, assuming it to vibrate together with the structure during earthquake.

$$M_s = \frac{1}{2} (\gamma_s / g) a^2 d$$

γ_s : weight of soil

g : acceleration of gravity

d : thickness of soil layer

- . Earthquake force is to be calculated by the use of response acceleration spectrum (statistical value obtained from observations made in the vicinity).

Related constants:

- . As for the dynamic spring constant of the subgrade, the lowest value of the modulus of deformation of the original position measured by the pressiometer is to be used. The spring constants in the horizontal, vertical, rotational and shearing directions are to be determined as instructed in the "Specification for Highway Bridge Substructure Design (Caisson)".

$$K_H = k_H \cdot (b/D)^{-3/4}, \quad k_H = \frac{1.2}{D} E_p, \quad b : \text{width } (b = \sqrt{A_H})$$

$$K_V = k_V \cdot (a/D)^{-3/4}, \quad k_V = \frac{1.0}{D} E_p, \quad a : \text{length } (a = \sqrt{A_V})$$

$$K_R = K_V \cdot I, \quad I : \text{moment of inertia of the bottom}$$

$$K_S = \lambda K_V, \quad \lambda = \frac{1}{3}$$

- . Or Blot's equation $K_R = \frac{E_R}{(1-\nu^2)k} a^2 b, K_S = \frac{E_R}{2(1+\nu)bk}$

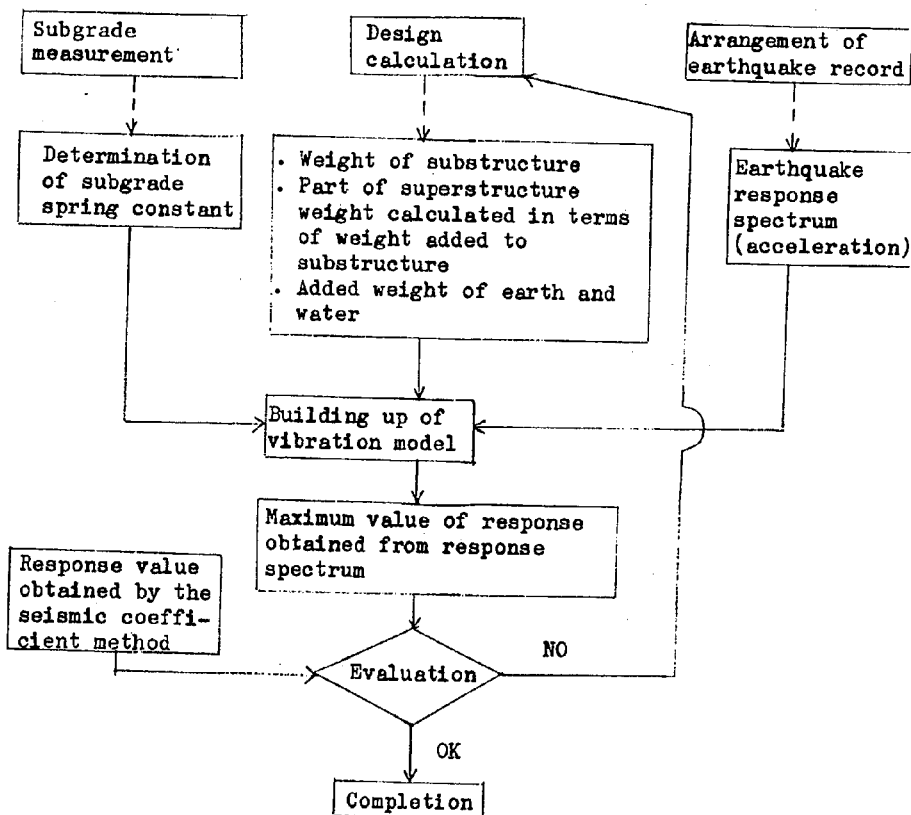
E_R : Voigt's elastic module,

ν : Poisson's ratio

Evaluation of earthquake resistant design calculation and precautions required:

- . The values to be obtained by calculations are natural period, center of rotation, displacement of gravity center and angle of rotation of the model.
- . The above values are used to evaluate the subgrade reaction, distance of eccentricity, bearing power, sliding and tipping.

Flowchart of earthquake resistant design calculation:



1.2 Type of foundation:

File foundation (Multiple-layer shearing model)

Setting up of earthquake resistant design objectives:

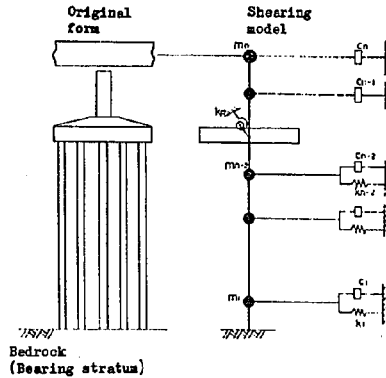
- . When the soft soil stratum extends deep beneath the surface, the ground movement is estimated to be large.
- . Since the structural type and dimensions are fixed, it is necessary for the bearings to be of flexible structures to reduce the section force, and therefore.

Selection of earthquake calculations method:

- . When the piles are likely to vibrate, coupled with the surrounding soil, the ground is to be replaced by a multiple-layer shearing model.
- . When there are no seismic wave records available for the same stratum of soil, the incident wave to the pile bearing stratum is to be estimated from the estimated acceleration of the seismic foundation, using the shearing wave theory.
- . Since the coefficient of subgrade reaction cannot be obtained, the dynamic reaction coefficient is to be obtained by modeling the subgrade by the finite element method.

Modeling for earthquake resistant design:

- . The ground is replaced by shearing models, stratum by stratum.
- . In the case of superstructure, only damping is to be considered, neglecting the reaction characteristic.
- . In the footings, the righting moment of rotation and rotational inertia are to be considered.



- The incident wave to the bearing stratum is to be calculated by means of the following equation, using the laws of reflection and refraction.

$$u_k(t, z) = f_k(t - \frac{z}{V_k}) + g(t - \frac{H_k - z}{V_k})$$

The stratum where the response $U_k(t, z)$ little changes is to be regarded as input bedrock (bearing stratum).

Related constants:

- The spring constant of subgrade is to be estimated, using FEM in plane strain.

$$A = \frac{1}{z} = k - S^T K^{-1} S$$

- As for the added mass, it is assumed that footing or piles have kinetic energy equal to that of the subgrade.

$$m_a = \frac{1}{z^2} X^T M X$$

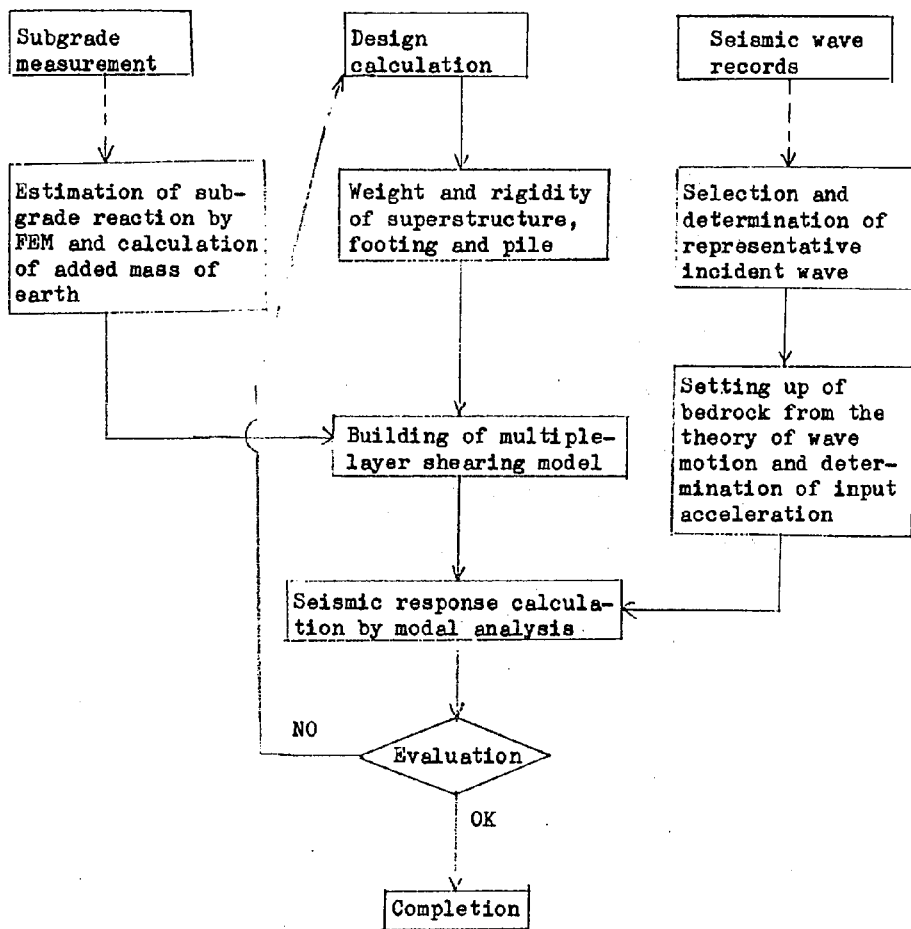
Evaluation of earthquake resistant design calculation and precautions required:

- . In the case of soft ground, there is so great a difference in the acceleration of seismic waves between the bedrock and ground surface that it is necessary to make investigations on the ground responses.
- . The bending moment in a pile is greater in its lower part because the seismic waves are rapidly amplified.

Problems

- . If there is a limit to the application of the theory of shearing wave (multiple-stratum ground, great earthquake).
- . Consideration to the effects of the group of piles on the shearing model.
- . Consideration to the damping mechanism and nonlinearity.

Flowchart of earthquake resistant design calculation:



1.3 Type of foundation:

Caisson foundation

Setting up of earthquake resistant design objectives:

- . In the case of soft ground, there are problems with the caisson behavior during earthquake.
- . Effects of the type of superstructure.

Selection of earthquake calculation method:

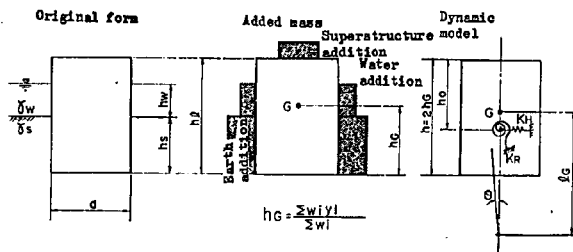
- . As for ground modeling, the model of spring-dashpot system is to be used from a practical viewpoint although there are the wave motion theory and finite element method.
- . Response spectrum analysis by a model with two degrees of freedom, horizontal translation and rocking motion.

Modeling for earthquake resistant design:

- . Effects of superstructure and water are to be treated as added mass.

$$M_w = \frac{0.6 \gamma_w A_w h_w}{(b/h_w)^{3/2}}, \quad M_s = \frac{1}{2} \gamma_s g b^2 d$$

- . When the side ground is very soft, it is to be treated as a plastic state if the horizontal reaction exceeds the allowable bearing capacity.
- . In the case of a large caisson, the response spectrum of earthquake records in the vicinity is to be used.
- . The maximum design acceleration shall be 180 to 250 gal.



Related constants:

- . It is necessary to examine the subgrade spring constant by a variety of Boussinesq's equation as given below and other methods. (Pauw, Tajimi, Richart)

$$K_v = \frac{E}{I_p(1-\nu^2)B_v\beta}$$

E : Ground deformation modulus = $6E_0 \sim 5E_0$

E_0 : Pressiometer deformation modulus

B_v : Foundation conversion width = $\sqrt{A_v}$

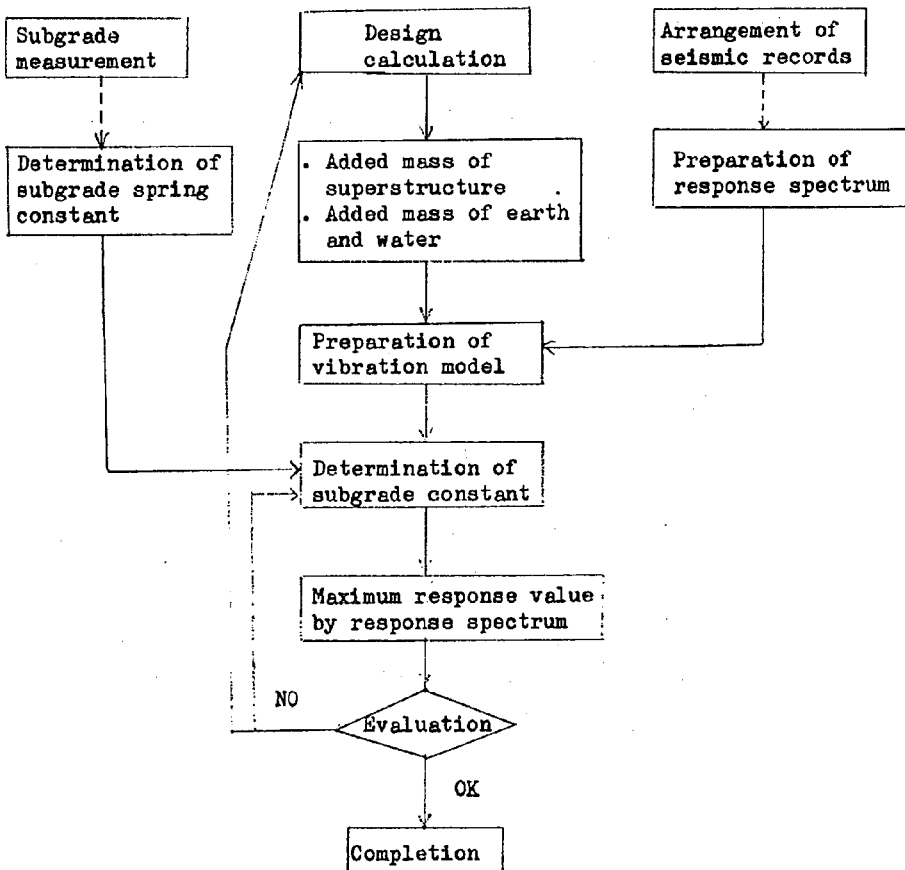
ν : Poisson's ratio I_p : Shape factor

Evaluation of earthquake resistant design calculation and precautions required:

- . There are some problems about the conversion of superstructure by added mass but in general there is not much difference from the results of analysis by the whole system model.
- . The response is governed by the evaluation of the subgrade spring constant. Particularly, the evaluation of the effects of caisson shape.

- . The values to be calculated are the natural period, the horizontal and rotational displacement of gravity center, and maximum acceleration response.
- . Since the substructure response is influenced by the structural type of superstructure, it is necessary to make response analysis at the basic design stage.

Flowchart of earthquake resistant design calculation:



1.4 Type of foundation:

Multi-column foundation

Setting up of earthquake resistant design objectives:

- . Since this is a flexible structure, the elastic behaviors of the foundation and footings are to be investigated.
- . The coupled vibration with the superstructure must be considered.

Selection of earthquake calculation method:

- . It is necessary to investigate the three-dimensional behavior for reasons of topographic inclination, arbitrariness of earthquake input direction, eccentricity of superstructure reaction and asymmetry of the foundation structure. Since it is impossible to estimate the condition in which the foundation and ground are bound together (the spring constant is not uniform), added mass of ground and damping constant, there is the necessity for analysis of three-dimensional vibration.
- . In order to make calculation in terms of a multiple system of material points, modal analysis is to be used.

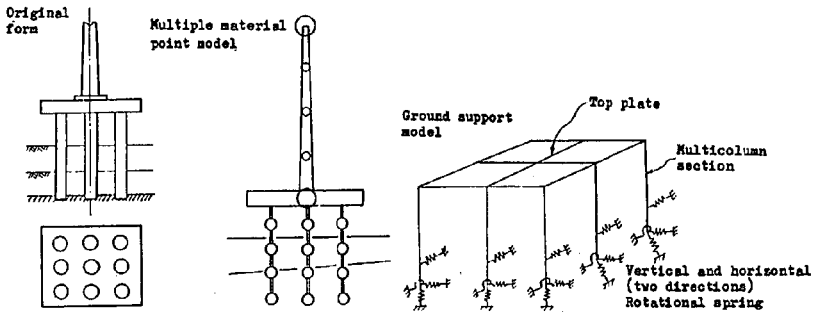
Modeling for earthquake resistant design:

- Considering the three-dimensional vibration, the deformations are to be taken as unknown quantities.

Pier : Horizontal x and y displacements (2)

Footings: Horizontal x and y displacements, torsional angle
 and rotational angle (10)

Superstructure: Horizontal x and y displacements and torsion (4)

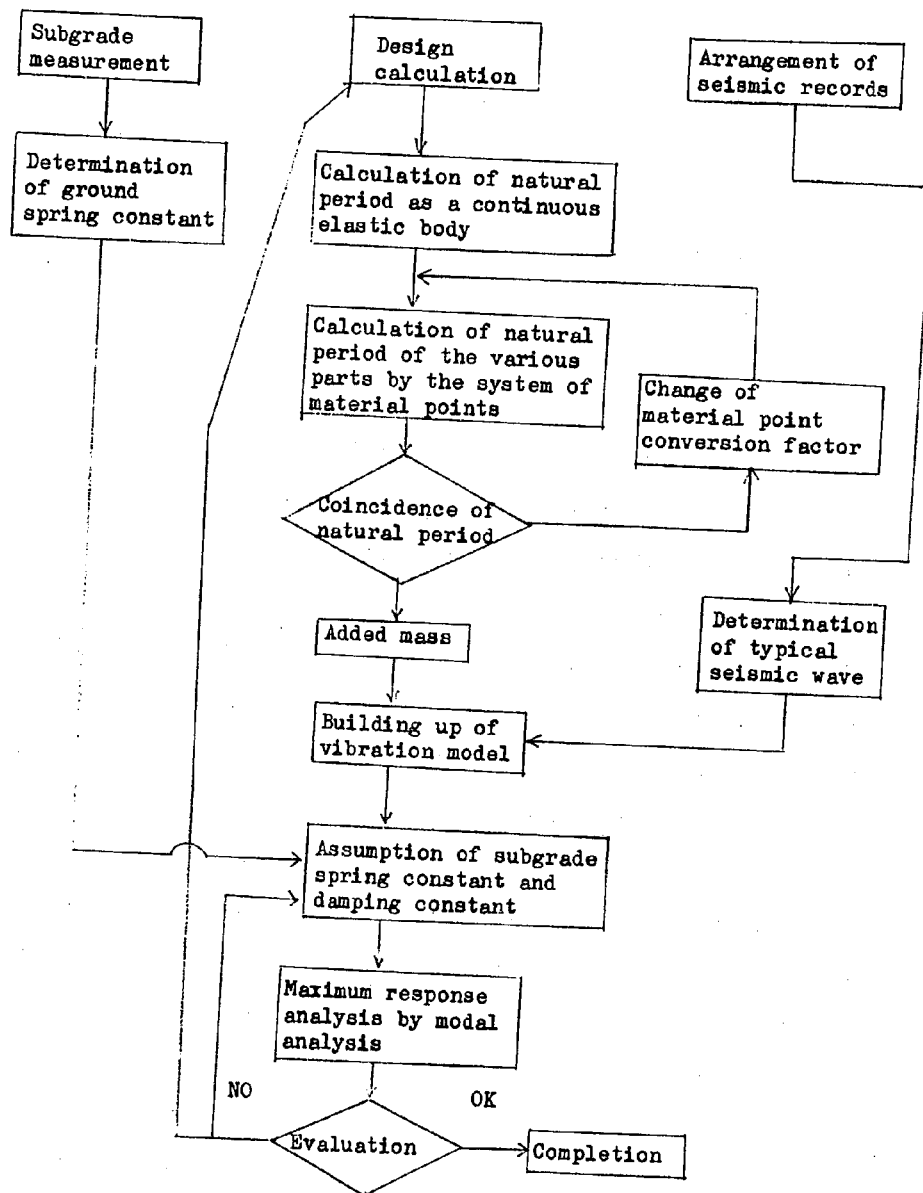


Related constants:

Evaluation of earthquake resistant design calculation and precautions required:

- . Comparison of static calculation and dynamic calculation results.
- . Comparison of calculated values with vibration experimental results.
- . It is necessary to check the responses by varying the subgrade reaction coefficient.
- . Since the foundation has so great influence on the superstructure, its coupling to the superstructure is also investigated.

Flowchart of earthquake resistant design calculation:



1.5 Conclusion:

Earthquake resistant design methods, mainly the modeling techniques for the earthquake resistant design of bridge foundations, have been summarized. As seen from the examples, the objectives of earthquake resistant design mainly concerns the following two cases.

- i) The bridge is very long.
- ii) The ground is poor.

The details of these two items are

- i) a. The effects of the seismic responses of substructure on the superstructure due to the long span of the bridge.
 - b. Dynamic characteristics of the substructure.
- ii) a. The ground behavior during earthquake is not well known.
 - b. The behavior of the foundation structure itself during earthquake.

The current trend is obviously to evaluating the behaviors during earthquake which are difficult to grasp by the seismic coefficient method due to the maximum acceleration, frequency and time of duration, which are the seismic force characteristics, and the response characteristics (natural period) of ground and structures, in order to design structures with a higher degree of safety. However, the behaviors during earthquake have not yet been fully clarified in the analysis of seismic responses.

Many studies involving actual observations and analysis of experimental results have been made about the way the subgrade spring constant, damping constant and added mass are to be treated and increased knowledge will be obtained in the near future. From a methodological standpoint, the

earthquake resistant design calculation has now reached a state capable of making sufficient evaluations required in designing earthquake resistant structures.

2. Actual Cases of Research

(A) Actual conditions of earthquake resistant design calculations

No.	Name of bridge (Managing organ)	Calculations made by	Type of foundation (substructure) Ground	Design seismic intensity	Vibration dynamic model	Analysis method (Conditions)
	Yoneyama-Ohashi (Hokuriku Dist. Construction Bureau)	Hokuriku Dist. Construction Bureau	Spread foundation, high bridge piers	$K_H = 0.2$	Continuous elastic body (dynamic three moment method)	Spectrum method (Public Works Research Institute mean spectrum) Max. input acceleration 0.2 g, Damping coefficient $n/p = 0.02$ & 0.05
	Sakaigawa-bashi (Japan Highway Public Corp.)	Japan Highway Public Corp.	Spread foundation, caisson foundation	$K_H = 0.2$	Continuous elastic body Multiple material points system (41 points)	Spectrum method (P.W.R.I. mean spectrum), Damping coefficient (Substructure 0.02, 0.05, 0.10) Direct integral (El Centro earth- quake waves), max. input accelera- tion 0.2 g
	Amakusa No.4 Bridge (Japan Highway Public Corp.)	Public Works Research Institute, Japan Highway Public Corp.		$K_H = 0.15$	Multiple material points system (34 points)	Direct integral (Taft earthquake waves), max. input acceleration 0.15 g
	Nagoya-Ohashi (Saga Pref.)	Saga Pref.		$K_H = 0.15$	Multiple material points system (14 points)	Direct integral (El Centro, Taft earthquake waves), max. input acceleration 0.15 g
	Showa-Ohashi (Niigata Pref.)	Public Works Research Institute	File foundation, Sand stratum	$K_H = 0.2$	Continuous elastic body (superstructure one point) One-point nonlinear system	Spectrum method (P.W.R.I. mean spectrum, Niigata earthquake waves), max. input acceleration 0.2 g Direct integral (Niigata earth- quake E-W component, Sept., 1964), max. input acceleration 157 gal

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No.	Name of bridge (Managing organ)	Calculations made by	Type of foundation (substructure) Ground	Design seismic intensity	Vibration dynamic model	Analysis method (Conditions)
	Yachiyo-bashi (Niigata City)	Public Works Research Institute		$K_H = 0.2$	Continuous elastic body (superstruc- ture one point)	Spectrum method (P.W.R.I. mean spectrum, Niigata earthquake spectrum) max. input acceleration 0.2 g
	Chidori-bashi (Aichi Pref.)	Aichi Pref.		$K_H = 0.25$	Continuous elastic body (superstruc- ture one point)	Spectrum method (P.W.R.I. mean spectrum), max. input acceleration 0.25 g
	Hirai-bashi (Tokyo Metropo- lis)	Public Works Research Institute, Tokyo Metro- polis	Legged caisson foundation Soft ground	$K_H = 0.3$	Continuous elastic (superstructure one point)	Spectrum method (P.W.R.I. mean spectrum), max. input acceleration 0.3 g
	Sokozawa-bashi (Japan Highway Public Corp.)	Japan Highway Public Corp.	Spread foundation, High piers Hard rock (phyllite)	$K_H = 0.2$ $K_V = 0.1$	Continuous elastic body (superstruc- ture one point)	
	Kanmon-kyo (Japan Highway Public Corp.)	Japan Highway Public Corp.	Spread foundation Weathered bedrock	$K_H = 0.15$	Multiple material points system (2 degrees of free- dom, 65 points in superstructure)	Spectrum method (acceleration spectra of 13 components obtained in hard ground), max. input acceleration 150 gal, direct integral (1962 Kushiro E-W com- ponent earthquake waves)
	Imasaka Kyoryo (Japanese National)	Japanese National Railways	Spread foundation, High Piers, Concrete replacing weathered sand		Multiple material system (2 degrees of freedom, 10 points in superstructure)	Spectrum method

No.	Name of bridge (Managing organ)	Calculations made by	Type of foundation (substructure) Ground	Design seismic intensity	Vibration dynamic model	Analysis method (Conditions)
	Hiroshima-Ohashi (Japan Highway Public Corp.)	Japan Highway Public Corp.	Well foundation, High piers, Sand stratum	$K_H = 0.25$ $K_V = 0.0$	Multiple material points system (caisson $5 \times 5 = 25$, superstructure 24 points)	Spectrum method (P.W.R.I. mean spectrum), max. input accelera- tion 200 gal, damping constant $h = 0.10$
	Urato-Ohashi (Japan Highway Public Corp.)	Japan Highway Public Corp.	Caisson foundation, high piers		Multiple material points system (Pier 18, superstructure 41 points)	Direct integral (El Centro earth- quake wave), max. input acceleration 200 gal
	Shin-1inogawa- bashi (Tohoku Dist. Construc- tion Bureau)	Tohoku Dist. Construction Bureau	Caisson foundation			Fourier spectrum analysis, correlation function analysis
	RC Rahmen Kokakyo (elevated road bridge)	Japanese National Railways	File foundation		Continuous elastic body, Multiple material points system (25 points in the direction of track, 12 points in the direction at right angles to track)	Spectrum method (velocity response spectrum), max. input acceleration 250 gal
	Minato-Ohashi (Osaka City)	Osaka City Osaka Univer- sity	Caisson foundation Soft ground (alluvial clay, gravels)	$K_H = 0.25$	Multiple material points system (equi- valent spring-dash- pot system)	Spectrum method (Honshu-Shikoku Bridge Authority's Earthquake Resistant Design Guide, response spectrum), max. input acceleration 250 gal

No.	Name of bridge (Managing organ)	Calculations made by	Type of foundation (substructure) Ground	Design seismic intensity	Vibration dynamic model	Analysis method (Conditions)
	Ishikarikako-kyo (Hokkaido Development Bureau)	Hokkaido Development Bureau	Steel pile well foundation, Alluvium		Multiple material points system (2, 14 points)	Checking by vibration experiments Damping constant: 0.10 & 0.20
	Hamana Ohashi (Japan Highway Public Corp.)	Japan Highway Public Corp.	Caisson foundation High piers	$K_H = 0.22$ $K_V = 0$	Multiple material points system	Spectrum method (P.W.R.I. mean spectrum), max. input acceleration 200 gal, Direct integral (El Centro, Hachinohe 1968 N-S earthquake waves)
	Sekaisuido-kyo (Japan Highway Public Corp.)	Japan Highway Public Corp.	Pile foundation Soft ground		Multiple material points system (spring-dashpot system)	Direct integral (El Centro earthquake waves), earthquake bedrock 50 gal, surface 180 gal, Damping constant: 0.10 (Measured value: 0.35)
	Tsugaru-Ohashi (Nagadoro-kyo) (Aomori Pref.)	Aomori Pref. Hokkaido Univ.	Pile foundation Soft ground		2-point system (spring-dashpot system)	Fourier spectrum analysis Max. input acceleration 130 gal, damping constant (primary 0.089, secondary 0.588)
	Onaruto-bashi (Honshu-Shikoku Bridge Authority)	Honshu-Shikoku Bridge Authority	Multicolumn foundation, suspension bridge piers	$K_H = 0.22$	Multiple material points system (10 points)	Spectrum method (Honshu-Shikoku Bridge Authority's spectrum)
	Myoko Ohashi (Hokuriku Dist. Construction Bureau)	Hokuriku Dist. Construction Bureau, Public Works Research Institute	Caisson foundation, High piers	$K_H = 0.2$ $K_V = \pm 0.1$	Multiple material points system	Spectrum method, corrected seismic coefficient method, Direct integral (El Centro, Akashi earthquake waves), damping constant $h = 0.02$

No.	Name of bridge (Managing organ)	Calculations made by	Type of foundation (substructure) Ground	Design seismic intensity	Vibration dynamic model	Analysis method (Conditions)
	Hirato Ohashi (Nagasaki Pref.)	Nagasaki Pref.	Spread foundation	$K_H = 0.15$ $K_V = 0.10$	1-point system (2 degrees of freedom) Superstructure 64 points	Spectrum method (P.W.R.I. spectrum, Seinaibashi, Itajima earthquake waves), max. input acceleration 150 gal, damping constant $h = 0.05$ (superstructure 0.02)
	Omishima-bashi (Honshu-Shikoku Bridge Authority)	Honshu-Shikoku Bridge Authority	Spread foundation	$K_H = 0.20$ $K_V = \pm 0.1$	Continuous elastic body	Spectrum method
	Hikawa-bashi (Japan Highway Public Corp.)	Japan Highway Public Corp.	Spread foundation, high piers		Multiple material points system (sub- structure 19 points, superstructure 31 points)	Spectrum method (El Centro, P.W.R.I. mean spectrum), max. input accelera- tion 180 gal, damping constant $h = 0.02$ & 0.05 , direct integral (El Centro)
	Nanpoku Bisan Seto Ohashi (Honshu-Shikoku Bridge Authority)	Honshu-Shikoku Bridge Authority	Caisson foundation		1-point system (two degrees of freedom)	Input acceleration 180 gal
	Hachinohe Ohashi (Aomori Pref.)	Aomori Pref.	File foundation Caisson foundation	$K_H = 0.24$		Max. input acceleration was 252 gal according to the Tokachioki Earthquake incident waves to the bedrock
	Katagami Ohashi (Okayama Pref.)	Okayama Pref.	File foundation			Complied to the Honshu-Shikoku Bridge Authority's Earthquake Resistant Design Guide Checking by vibration experiments (damping constant: 0.05, 0.07 in axial direction, 0.02, 0.08 in the direction at right angles to the axis of the bridge)

No.	Name of bridge (Managing organ)	Calculation made by	Type of foundation (substructure) Ground	Design seismic intensity	Vibration dynamic model	Analysis method (Conditions)
	Nankaku Ohashi Sansaiyama Ohashi (Nagano Pref.)	Nagano Pref. Highway Corp.	Caisson foundation, spread foundation, high piers	$K_H = 0.25$		Spectrum method (P.W.R.I. mean spectrum) Damping constant $h = 0.02$ & 0.05
	Tedorigawa Dam No.3 Bridge (Ishikawa Pref.)	Ishikawa Pref.	Spread foundation, high piers	$K_H = 0.16$ (after alteration $K_H = 0.20$)	Continuous elastic body, Multiple material points system	Spectrum method (P.W.R.I. mean spectrum, El Centro), max. input acceleration 160 gal Damping constant (substructure: $h = 0.05$, superstructure: $h = 0.02$)

(B) Summarized description of actual cases of seismic response analysis

(1) Type of foundation:

Spread foundation (Kanmon-kyo)

Bridge:

3-span, 2-hinge suspension bridge

Bridge length: 1039 m (167.75 + 703.5 + 167.75)

Literature:

"Doboku Gijutsu" (Civil Engineering Technology), Oct. 1970

Location and organization concerned:

Shimonoseki, Yamaguchi Pref. ~ Moji, Fukuoka Pref.

Japan Highway Public Corporation

Purpose of analysis:

Analysis method:

Average response spectrum analysis by a 2-degree of freedom model of motion

Analysis results, evaluation and problems:

- o When the seismic intensity method and the 2-degree of freedom system are compared, the values obtained by the two methods fairly well agreed in the respects of eccentricity, subgrade reaction and sliding safety factor in the case of a large foundation.
- o When calculations are made for the foundation alone, there is not much difference in reaction between the seismic intensity method and the 2-degree of freedom system, therefore, there is no problem

in this respect. However, when the continuity of superstructure and the behavior of structures during earthquake are considered, it would be necessary to take the responses into consideration when making calculations

Stability calculation results of seismic intensity method and 2-degree of freedom system (Bridge axial direction)

Division	Shimonoseki abutment		Moji abutment	
Earthquake-resistant design calculation	Seismic intensity method	2-degree of freedom system	Seismic intensity method	2-degree of freedom system
Ground deformation modulus in the vertical direction (t/m^2)		3.2×10^5		3.2×10^5
Period (sec)		0.56		0.32
Center of rotation (m)		93.8		102.5
Displacement of center of gravity of the model (cm)		1.5		0.46
Angle of rotation of the model (rad)		1.56×10^{-4}		4.53×10^{-5}
Subgrade reaction (t/m^2)	136	140	116	107
Distance of eccentricity (m)	10.8	13.7	10.54	10.62
Consideration to bearing power (t/m^2)	$136 < 320$	$140 < 320$	$116 < 250$	$107 < 250$
Consideration to sliding	$1.6 > 1.5$	$1.51 > 1.5$	$3.3 > 1.5$	$1.6 > 1.5$
Consideration to tipping (m)	$e = 10.8$ $\frac{55}{3} - 10.8$ $> \frac{55}{6}$	$e = 13.7$ $\frac{55}{2} - 13.7$ $> \frac{55}{6}$	$e = 10.54$ $\frac{55}{2} - 10.54$ $> \frac{55}{6}$	$e = 10.62$ $\frac{55}{2} - 10.62$ $> \frac{55}{6}$

Type of ground and evaluation:

- o The foundation rock are so much weathered and cracked that the deformation was determined from the correlation of deformation characteristics obtained by pressiometer observations at site and jack tests.

$$\left(\frac{\text{Deformation modulus obtained from the envelop in jack test}}{\text{Deformation modulus obtained by pressiometer}} \right)$$

$$\times \left(\frac{\text{Deformation modulus obtained by the iteration curve in jack test}}{\text{Deformation modulus obtained by the envelop in jack test}} \right)$$

$$= 5 E_p$$

where

E_p = deformation modulus obtained by pressiometer

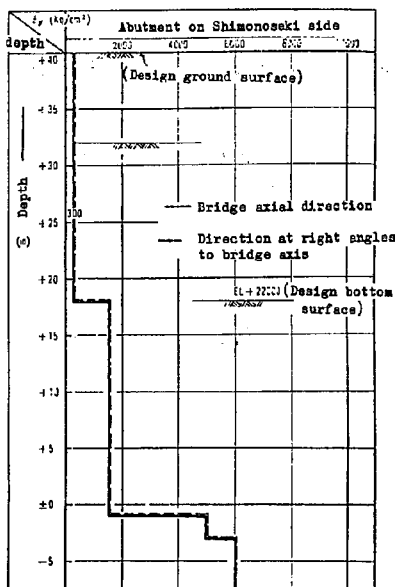


Fig. 2 Abutment base on Shimonoseki side $E_p(\text{kg/cm}^2)$

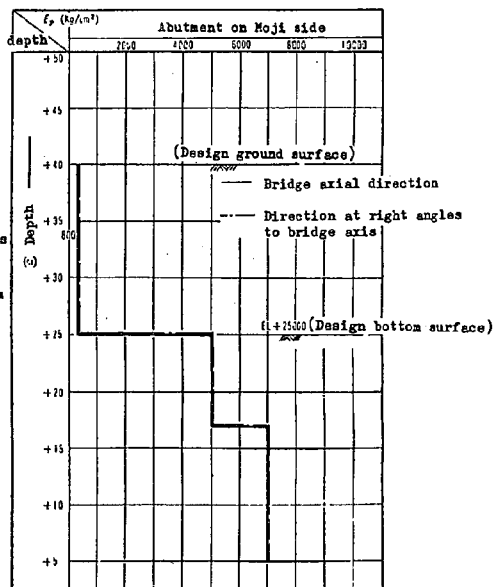


Fig. 4 Abutment base on Moji side $E_p(\text{kg/cm}^2)$

Engineering constants of ground (Bridge axial direction)

I t e m	Symbol	Shimonoseki abutment		Moji abutment	
		Earthquake time	Normal time	Earthquake time	Normal time
Deformation modulus of bearing stratum (t/m^2)	E_v	1.24×10^5	0.62×10^5	3.2×10^5	1.6×10^5
Vertical spring constant of bearing stratum (t/m^3)	K_v	3.23×10^3	1.62×10^3	8.34×10^3	4.17×10^3
Horizontal spring constant on the side (t/m^3)	K_1	$K_1 = 450$	$K_1 = 225$	0	0
Rotational spring constant on the bottom (t/m^3)	K_θ	33.62×10^8	16.81×10^8	86.65×10^8	43.33×10^8
Shearing spring constant on the bottom (t/m^3)	K_s	1.08×10^3	0.54×10^3	2.8×10^3	1.4×10^3
Horizontal frictional coefficient	μ	0.6	0.6	0.6	0.6
Angle of internal friction of rock (degree)	ϕ	30°	30°	30°	30°
Cohesion of rock (t/m^2)	c	6.0	6.0	10.0	10.0

Ground constants in the entire system:

< Shimonoseki side >

$$K_S = 2.62 \times 10^6 \text{ t/m}$$

$$K_{H1} = 0.294 \times 10^6 \text{ t/m}$$

$$K_R = 35.6 \times 10^8 \text{ t}\cdot\text{m}$$

< Moji side >

$$K_S = 7.33 \times 10^6 \text{ t/m}$$

$$K_{H1} = 0.294 \times 10^6 \text{ t/m}$$

$$K_R = 86.73 \times 10^8 \text{ t}\cdot\text{m}$$

o Added mass of water

$$M_v = \frac{0.6P \cdot A \cdot h}{(a/h)^{2/3}}$$

where $0.8 \leq a/h \leq 4.0$

M_v ; virtual mass of water (t)

P ; density of water (t/m^3)

A ; Sectional area (m^2)

a ; width of section perpendicular to the direction of vibration (m)

h ; depth of water (m)

o Added mass of surrounding soil

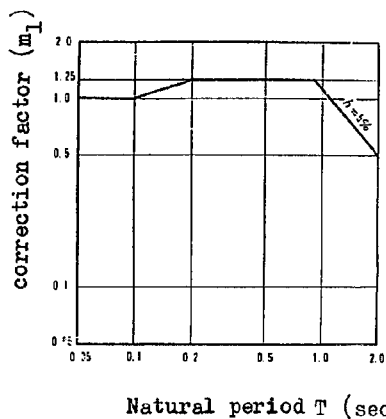
$$M_s = (1/2) \cdot (rs/g) \cdot b^2 \cdot d$$

where M_s ; virtual mass of soil ($\text{t}\cdot\text{sec}^2/\text{m}$)

s ; unit weight of soil (t/m^3)

b ; width of structure perpendicular to the direction of vibration (m)

d ; thickness of soil stratum (m)



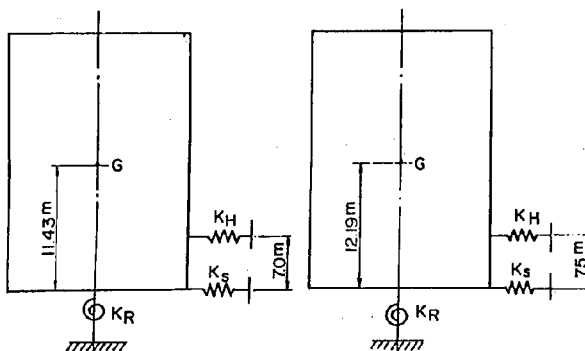
Acceleration response spectrum curve

Original form of foundation and analysis model:

Bridge abutment model

< Shimonoseki side >

< Moji side >



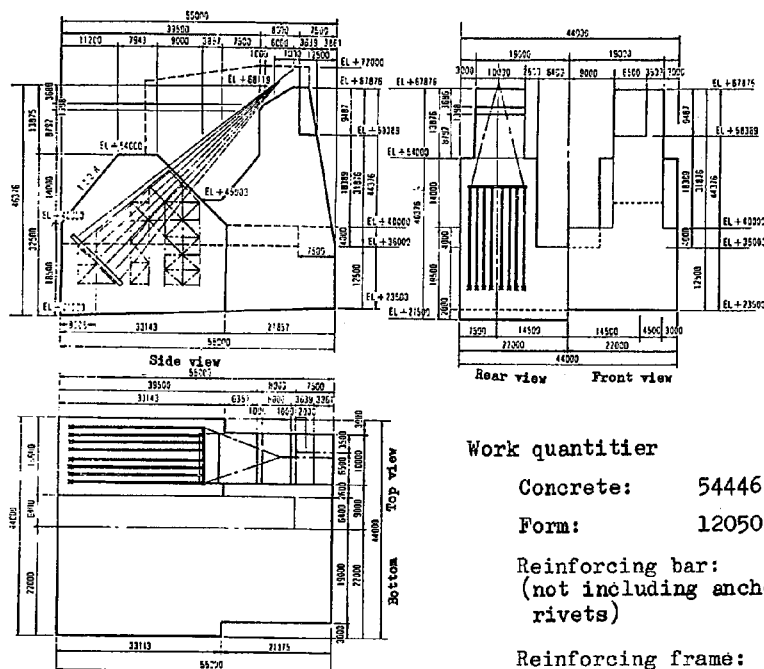
- o The effect of superstructure is to be treated as added mass.
- o Foundation rock is to be calculated in terms of elastic rotational, vertical and shearing spring.
- o The surrounding soil is to be calculated in terms of the spring to cause compressive or tensile deformation.

Various quantities for the dynamic model
(bridge axial direction)

I t e m	symbol	Shimonoseki abutment	Moji abutment
Length in bridge axial direction (m)	a	55.0	55.0
Position of gravity center of model (m)	h_a	12.645	12.137
Overall height of model(m)	h_o	25.29	24.274
Unit weight of model (t/m^3)	W	3.09	2.6
Virtual mass of model ($t \cdot sec^2/m$)	M	18.2×10^3	14.5×10^3
Rotational inertia with respect to gravity center of model ($t/m \cdot sec^2$)	J	5.56×10^6	4.4×10^6

Shimonoseki side

General view of abutment on Shimonoseki side



Work quantitier

Concrete:	54446.5 m ³
Form:	12050.5 m ²
Reinforcing bar:	346262 kg
(not including anchor bolts and rivets)	
Reinforcing frame:	290423

Material strength

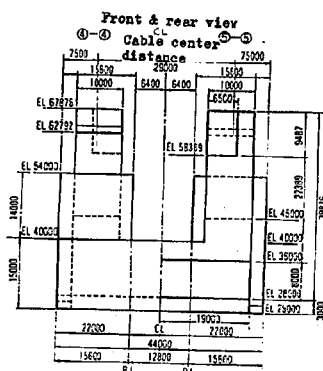
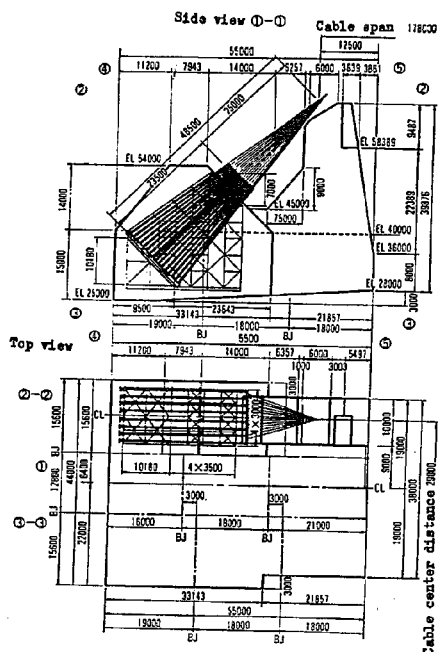
Concrete	δ 91 compressive stress intensity:	240 kg/cm ²
	Allowable bending compressive stress intensity:	80 kg/cm ²
	Maximum aggregate grain size:	50 mm
Reinforcing bar	Deformed bar (SD30):	1800 kg/cm ²
Reinforcing frame	SS41	1400 kg/cm ²

Design conditions

Cable seismic intensity:	14050 t
Design seismic intensity: $K_H = 0.15$ (horizontal), $K_V = 0.075$ (vertical)	
Subgrade reaction coefficient: Earthquake time: $E_V = 1.238 \times 10^5$ t/m ²	
Normal time: $E_V = 0.619$ t/m ²	

Moji side

General view of abutment on Moji side



Work quantities

Excavation	: 103063 m ³
Concrete	: 48370 m ²
Reinforcing bar	: 271 t
Form	: 10584 m ²
Reinforcing frame	: 292 t

Material strength

Concrete	δ91 compressive strength:	240 kg/cm ²
	Allowable bending compressive stress intensity:	80 "
	Maximum aggregate size:	50 mm
Reinforcing bar	Deformed bar:	1800 kg/cm ²
Reinforcing frame	SS-41	1400 "

Design conditions

Cable tensile force: 14050t x 2

Design seismic intensity: $K_H = 0.15$ (horizontal),

$K_V = 0.075$ (vertical)

Deformation modulus of foundation ground:

3.193×10^5 t/m² (earthquake time)

1.597×10^5 t/m² (normal time)

(2) Type of foundation:

Spread foundation (Onaruto-bashi)

Bridge:

Literature:

Onaruto-bashi substructure plan and design, March 1972

Location and Organization concerned:

Honshu-Shikoku Bridge Authority

Purpose of analysis:

Comparison of suspension bridge anchorage earthquake responses

Analysis method:

Mean response spectrum

Analysis results, evaluation and problems:

Type of ground and evaluation:

- o Elastic modulus from Voigt's equation (Earthquake Resistant Design Guide)

$$E_R = 4.5 \times 10^8 \text{ t/m}^2, \quad \nu = 0.25$$

$$\frac{a}{b} = 1.164, \quad k = 1.03$$

$$K_s = 1.118 \times 10^7 \text{ t/m}$$

- o Rotational spring

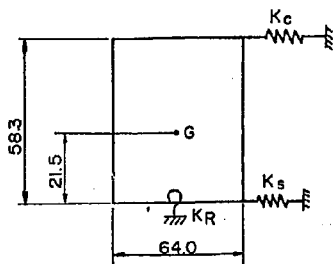
$$K_R = \frac{E_R}{(1-\nu^2)k} a^2 b, \quad \frac{b}{a} = \frac{55}{64} = 0.859, \quad k = 4.08$$

$$K_R = 2.649 \times 10^{10} \text{ tm/rad}$$

- o Bottom shearing spring

$$K_s = K_b a b = \frac{G}{b k} a b = \frac{E_R}{2(1+\nu) b k}$$

Original form of foundation and analysis model:



(Depth 55.0 m)

- o Anchor foundation converted to a rectangular model

$$W_0 = 2.952 \times 10^5 \text{ t}$$

$$x_G = 32.8 \text{ m}$$

$$y_G = 21.5 \text{ m}$$

$$I_G = 1.844 \times 10^8 \text{ tm}^2$$

- o Cable tensile force (one side)

$$2H = 5.1 \times 10^5 \text{ t}$$

$$2T = 5.128 \times 10^5 \text{ t}$$

- o Cable horizontal spring coefficient

$$K_c = \frac{E_c A_c}{PL_s} \sec^2 \alpha = \frac{E_c A_c}{L}$$

$$= \frac{2.0 \times 10^5 \times 0.522}{330} = 3.164 \times 10^4 \text{ t/m} \times 2$$

(3) Type of foundation:

Spread foundation (Inasaka Kyoryo)

Bridge:

3-span continuous box PC girder

Bridge length (57 + 82 + 57)

Literature:

Structural Design Material No. 38, Aug. 1974

Location and Organization concerned:

Sanyo Shinkansen (435.771 Km from Shin-Osaka Station)

Japanese National Railways

Purpose of analysis:

Investigation of vibration behavior of a continuous bridge supported by high piers.

Analysis method:

Modal analysis of 2-degree of freedom rocking vibration and multiple-degree of freedom bending vibration

Analysis results, evaluation and problems:

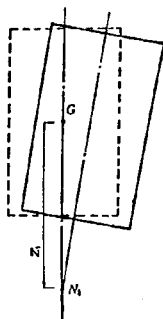
- o The analysis results agreed with the rigid body rocking vibration in the direction at right angles to the bridge axis and with the bending vibration results in the axial direction.
- o The damping constant was 0.05 when the vibration of foundation was predominant and 0.03 when the bending vibration of bridge body was predominant.

(a) Calculation results for the 2-degree of freedom system

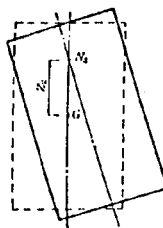
Vibration exciting direction	Thickness of added mass	Spring constant(kg/cm ³)		Damping constant	Natural frequency (Hz)	Displacement of pier crown
		K _H	K _V			
Perpendicular to the track	2 m	2	40	0.05	5.18	0.29
	4 m	2	40	0.05	5.14	0.29
Track	2 m	2	40	0.03	5.13	0.47
	4 m	2	40	0.03	5.28	0.47

(b) Calculation results for the multiple-degree of freedom system

Vibration exciting direction	Thickness of added mass	Spring constant(kg/cm ³)		Damping constant	Natural frequency (Hz)		Pier crown displacement of resonance point (mm)
		K _H	K _V		Primary	Secondary	
Perpendicular to the track	0	2	40	0.03	3.59	17.2	1.74
	2	2	40	0.03	3.58	16.5	1.74

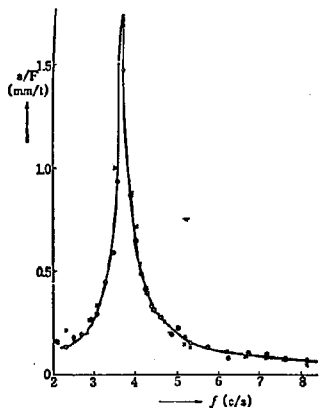


(a) Primary vibration

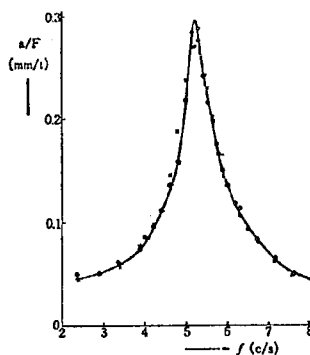


(b) Secondary vibration

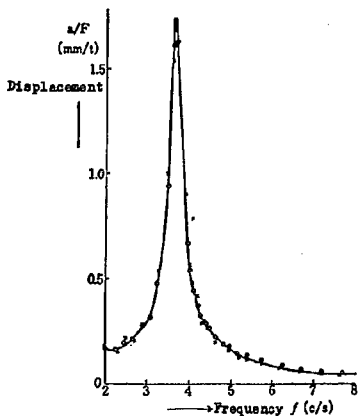
o Experimental results



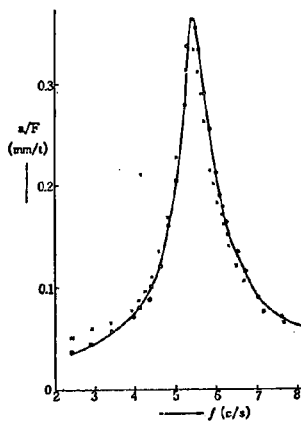
Resonance curve (Test No.1 in the direction of the track, pier ST, No.4)



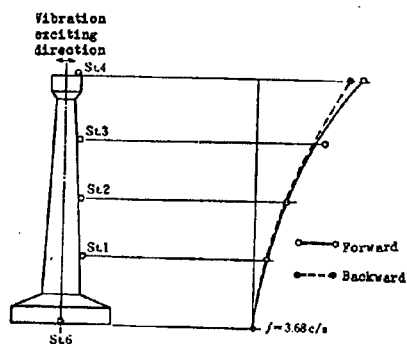
Resonance curve (Test No.2 in the direction perpendicular to the track, pier ST, No.4)



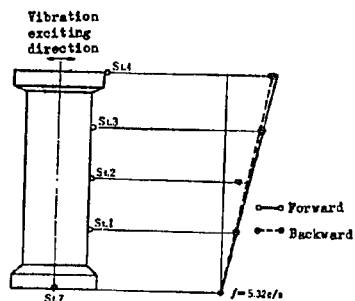
Resonance curve (Test No.2 in the direction of the track, pier ST, No.4)



Resonance curve (Test No.1 in the direction perpendicular to the track, pier ST, No.4)



Mode of vibration in the direction of the track (Test No.2 in the direction of the track)



Mode of vibration in the direction perpendicular to the track (Test No.2 in the direction perpendicular to the track)

Response displacements and damping constants

Test		Point of measurement	Position	Resonance frequency (Hz)	Displacement (mm)	Damping constant
Tests in the direction of the track	No.1	a ₄	Pier head	3.62	1.755	0.025
	No.2	a ₄	"	3.62	1.750	0.037
Tests in the direction perpendicular to the track	No.1	b ₄	"	5.40	0.267	0.057
	No.2	b ₄	"	5.20	0.297	0.066

Type of ground and evaluation:

o Ground spring constants

$$k_h = K_H \cdot A_d$$

$$k_s = \frac{1}{2} K_v \cdot A_b$$

$$k_{\tau} = I_b / K_v$$

A_b : Foundation bottom area

A_d : Foundation side area

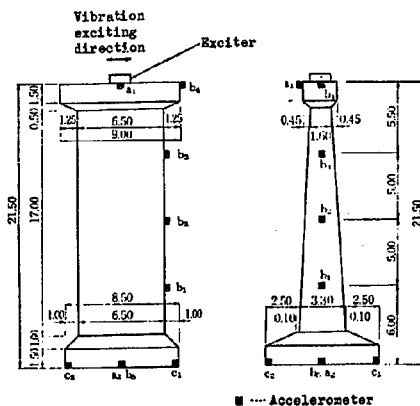
I_b : Moment of inertia of area of the bottom

K_H : Subgrade reaction coefficient (horizontal) = 2 Kg/cm³

K_v : Subgrade reaction coefficient (verticla) = 40 Kg/cm³

Original form of foundation and analysis model:

Original form and accelerometer positions



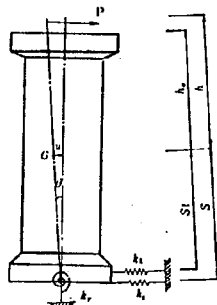
o 2-degree of freedom system

Mass : $16.99 \times 10^2 \text{ Kg} \cdot \text{sec}^2 / \text{cm}$

Gravity center: 6.21 m

Moment of inertia: $9.031 \times 10^7 \text{ Kg} \cdot \text{sec}^2 \cdot \text{cm}$

2-degree of freedom model



G : gravity center

m : mass of rigid body

l : Gravity center diagram of rigid body

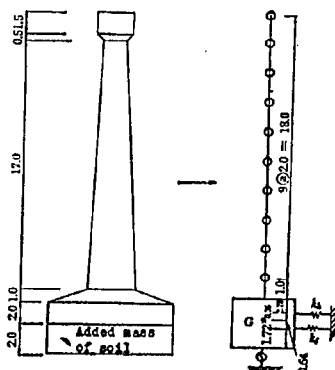
k_h : horizontal spring

k_s : shearing spring

k_r : rotational spring

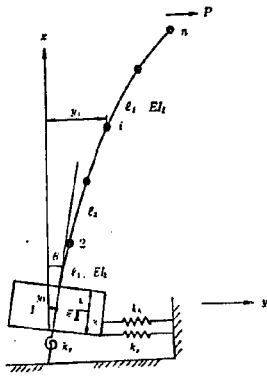
o Surrounding soil is to be considered as added mass

Mass and moment of inertia of area of multiple-degree of freedom system

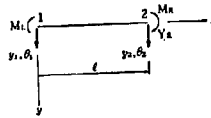


Mass (tsec ² /m)	Moment of inertia of area (m ⁴)
6.658	10.466
8.042	2.662
5.769	3.715
6.633	5.016
7.296	6.591
7.959	8.464
8.662	10.662
9.286	13.211
9.949	16.137
10.610	18.594
Rigid section	
m=79.95	
I=628.8	
k _v =40kg/cm ³	

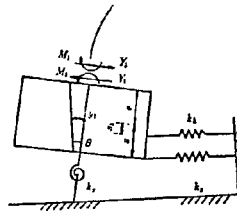
Multiple-degree of freedom model



Dynamic model



Beam balance



Balance with one joint of framework

(4) Type of foundation:

Caisson foundation (Hiroshima Ohashi)

Bridge:

Bridge length 1020 m (348.5 + 261.9 + 351.3)

Literature:

Report on Investigations and studies on Earthquake Resistant Design
of Honshu-Shikoku Bridges, Mar., 1972

Location and organization concerned:

Hiroshima Prefecture

Purpose of analysis:

Because of high bridge piers

Analysis method:

Mean spectrum method, modal analysis

Analysis results, evaluation and problems:

- o The caisson and bridge piers were assumed to be a beam system and this analysis method produced the mode of vibration and values of responses greatly different from when they were assumed to be a rigid body system.
- o There were little difference in the values of responses according to the shape of subgrade reaction coefficients.
The natural period was somewhat longer but the primary equivalent mass increased.
- o It is necessary to proportionally increase the earthquake intensity in a system like this in which the high order mode has a great effect.

Acceleration magnification (P_6 bridge axial direction)

Number of sections						Average
β		0.78	1.8	1.4		
F		24.03	19.23	19.76	-	
Material point	1	1.28	-1.49	0.38	2.00	1.58
	2	0.98	-0.06	-0.71	1.21	
	3	0.78	1.00	-1.21	1.71	
	4	0.48	1.35	-0.75	1.62	
	5	0.234	0.998	0.508	1.14	1.08
	6	0.062	0.410	0.942	1.03	
	7	-0.024	-0.049	0.526	0.53	

Note: The proportional increase coefficient (β) for the horizontal earthquake intensity in the static earthquake calculation was taken at 1.25.

Responses and periods

		P ₁		P ₂		A ₁ ~ P ₁					Remarks
		Bridge axial direction	Perpendicular to bridge axial direction	Bridge axial direction	Perpendicular to bridge axial direction	A ₁	P ₁	P ₂	P ₃	P ₄	
Response	S (cm)	12.8	9.6	12.2	10.7						Bracketed are design values
	S (t)	619 (1200)	757 (727)	427 (437)	580 (601)						
	M (t·m)	16200 (25900)	19200 (14500)	11600 (7660)	17500 (14500)	1610 ()	8550 (5910)	8470 (6050)	13100 (8630)	12100 (8860)	
Period (sec)	First order	1.84 (31 %)	1.12 (33 %)	1.10 (26 %)	1.23 (33 %)	0.93 (14.3%)					Bracketed are equivalent masses
	Second order	0.44 (25 %)	0.30 (26 %)	0.46 (28 %)	0.37 (26 %)	0.86 (6.0%)					
	Third order	0.16 (30 %)	0.15 (30 %)	0.15 (31 %)	0.14 (30 %)	0.83 (11.9%)					

Note) The values of response given in the above table are the maximum values of bridge responses.

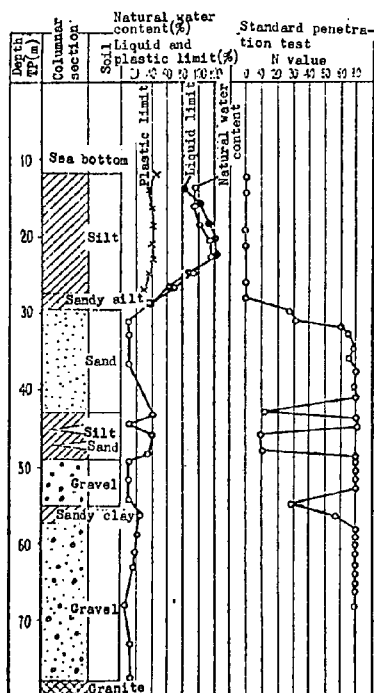
Natural period by ground spring coefficient

			Case A	Case B
Perpendicular to the bridge axial direction	Natural period (sec)	First order	1.23	1.42
		Second order	0.37	0.41
		Third order	0.14	0.15
		Fourth order	0.10	0.10
	Equivalent mass (%)	First order	33	39
		Second order	26	25
		Third order	30	26
		Fourth order	11	10

Case A: Measured values

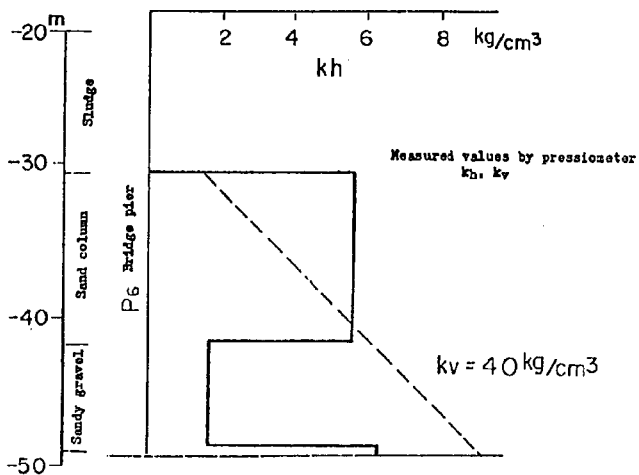
Case B: Hypothetical values

Type of ground and evaluation:



Columnar section near P₅

Distribution K value

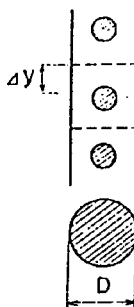


Horizontal spring $K_H = \sum D \cdot \Delta y \cdot K_h$

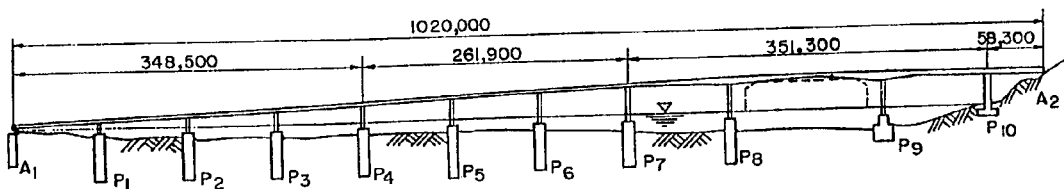
Vertical spring $K_V = \frac{\pi D^2}{4} \cdot k_v$

Rotational spring $K_R = \frac{\pi D^4}{64} \cdot k_r$

Shearing spring $K_S = \frac{1}{4} \cdot K_V$



Original form of foundation and analysis model:



General side view

- o The entire systems of $A_1 \sim P_4$ are for the direction perpendicular to the bridge axis and P_5 and P_6 are for the axial direction and the direction perpendicular to the bridge axis.
- o Damping constant $h = 0.1$
- o As for input, the P.W.R.I mean response spectrum is used and the maximum acceleration is 200 gal.
- o Caisson's lower end is supported by linear spring.
- o Caisson section is also treated as a beam system.
- o Added mass of water and soil ΔW

$$\text{Bridge pier } \Delta W_w = \frac{3}{4} b^2 \cdot h_w \left(1 - \frac{b}{4h_w}\right) \cdot \gamma_w$$

$$\text{Caisson } \Delta W_w = \frac{\pi D^2}{4} \cdot \Delta h \cdot \gamma_w, \quad \Delta W_E = \frac{\pi D^2}{4} \cdot \Delta h \cdot \gamma_E$$

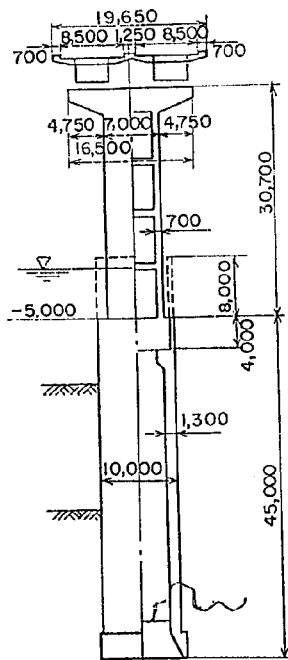
where h_w : water depth

b, D : pier width and diameter

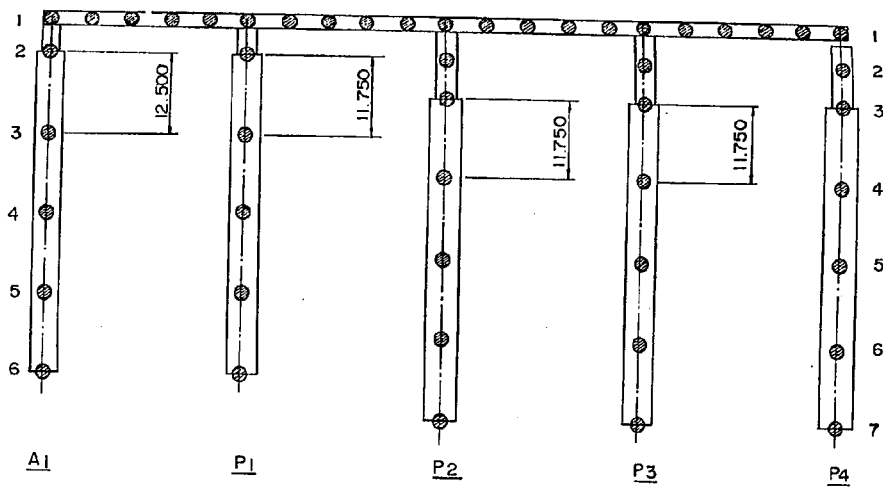
Δh_1 : sliced height

γ_w : 1.03 t/m³

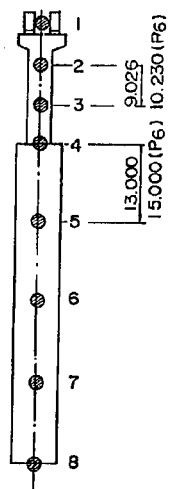
γ_E : 1.4 or 2.0 t/m³



P_6 pier



(a) $A_1 \sim P_4$ system



(b) $P_5 \sim P_6$ system

Dynamic model constants

Material point	(A ₁) ~ P ₁ system			P ₁ (Bracketed are for axial direction)			P ₂ (Bracketed are for axial direction)			Remarks
	Spring × 10 ⁴ (t/m)	Weight (ton)	Moment of inertia	Spring × 10 ⁴ (t/m)	Weight (ton)	Moment of inertia (m ⁴)	Spring × 10 ⁴ (t/m)	Weight (ton)	Moment of inertia (m ⁴)	
1					1468 (3224)			1363 (566.9)		Rotational spring A ₁ = 4.907 × 10 ⁸ (t.m) P ₁ ~ P ₂ = ∞ (∞) P ₁ = 19.64 × 10 ⁴ (∞) P ₂ = 19.64 × ∞ (∞)
2					553.3	↑		325.9	↑	
3					553.3	100.0 (25.01)		350.0	69.31 (20.69)	
4	32.38 (24.50)			0	2070	↓	0	2197	343.7	Superstructure data 17.37m(material point interval) I _a = 3.765 m ⁴ (2 main girders) W _a = 10.37 t/m (2 main girders)
5	80.54 (88.23)			0	3140		0	3646	↓	
6	38.83 (39.47)			43.50	3494		72.50	4235		
7				47.45	3753		95.89	2493		
8				117.49	2202					

(5) Type of foundation:

Caisson foundation (Urato-Ohashi)

Bridge:

PC 5-span continuous box girder bridge

Bridge length 600m (55 + 130 + 230 + 130 + 55) main section

Literature:

Report on Investigations and Studies on Honshu-Shikoku Earthquake

Resistant Bridge Design, Civil Engineering Society, Mar. 1972

Location and organization concerned:

Kochi Prefecture

Japan Highway Public Corporation

Purpose of analysis:

Since this is a high-pier bridge, vibration tests were made and the test results were compared with the results of response analysis.

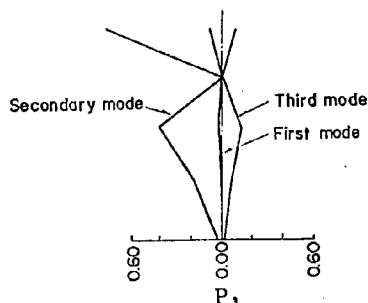
Analysis method:

Modal analysis of the entire bridge system by 59 material points.

Analysis results, evaluation and problems:

- o The dynamic analysis results were 47 - 73 % of static calculation values
- o Since the dynamic analysis showed a greater value for the effective span of the main girder, reinforcing bars are to be increased to increase the strength.
- o Vibration experimental results and calculated values agreed with respect to resonance frequency.
- o The effect of subgrade reaction coefficient was small.

- o The damping constants were appropriately assumed as far as the experimental results were concerned.



Natural vibration modes

Natural frequencies and damping constants

			Experimental value		Measured value		
			f (HZ)	h (%)	f (HZ)	h (%)	K value
P ₂ pier	Bridge axial direction	First order	3.57	2.5	3.83	2.0	45 kg/cm ³
		Second order	13.6	4.0	15.9	"	"
		Third order	-	-	-	"	"
	Perpendi- cular to the axial direction	First order	3.66	1.9	3.66	2.0	45 kg/cm ³
		Second order	14.3	2.7	14.7	"	"
		Third order	-	-	-	"	"
P ₃ pier	Bridge axial direction	First order	2.51	3.2	2.52	2.0	51 kg/cm ³
		Second order	-	-	5.25	"	"
		Third order	10.8	(8)	11.29	"	"
	Perpendi- cular to the axial direction	First order	2.47	3.5	2.47	2.0	51 kg/cm ³
		Second order	-	-	5.23	"	"
		Third order	10.5	(8)	10.91	"	"

Natural periods and responses

	Bridge pier		Main girder		Remarks
	Lower end of P ₂	Lower end of P ₃	On the support of P ₂	On the support of P ₃	
Bending moment (t . m)	57,872 (79,978)	78,623 (167,744)	2,100 (2,500)	2,100 (2,800)	Bracked are statistical calculation results
Shearing force (t)	1,692 (2,721)	2,136 (4,104)	720	690	
Displacement (cm)			Maximum at hinged joint 44.7 (39.8)		
Natural period (sec)	1st order = 2.567, 2nd order = 1.072, 3rd order = 1.925, 4th order = 0.959, 5th order = 0.727				

Comparison of maximum responses of bridge piers

	Fixed foundation		Translation Rotation	
	M (t · m)	S (t)	M (t · m)	S (t)
A ₁	2,380	164	2,504	166
P ₁	8,682	1,428	8,895	527
P ₂	58,740	1,542	57,827	1,692
P ₃	64,352	1,261	78,623	2,136
P ₄	15,490	509	18,586	655
P ₅	6,102	260	9,210	336

Results according to the Earthquake Resistant Design Guide (P₃)

Natural period 2.0 sec

M 113,000 t · m

S 3,180 t

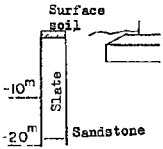
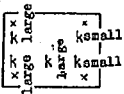
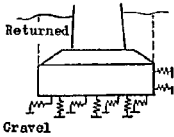
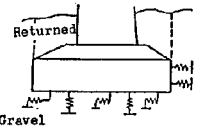
V 14,622 t

where $K_H = 0.2$,

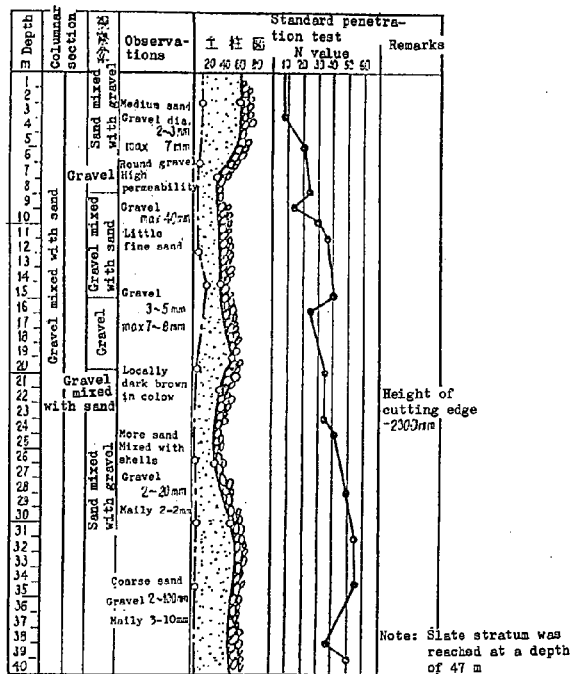
$\beta = 0.21$

Type of ground and evaluation:

Comparison of subgrade reaction coefficients obtained
by various tests and analyses

	Estimated from N value	Plate loading test	Vibration experiment	Dynamic analysis
P ₂ pier			Bridge axial direction $k_h = -$, $k_v = 31$ Perpendicular to axial direction $k_h = -$, $k_v = 45$	$k_h = k_v = 35$
P ₃ pier	Fukuoka-Uto equation $k = 2 \sim 5$ Terzaghi's formula $k = 6 \sim 8$ Hayashi's sim- plified formula $k = 4.5 \sim 6$	Theory of elasticity $k = 5.6 \sim 18$ Calculated in terms of 75cm diameter from experimental formula $k = 6.5 \sim 21$ Calculated in terms of ∞ dia- meter $k = 4.5 \sim 14$	Bridge axial direction $k_h = 5$, $k_v = 5$ Perpendicular to axial direction $k_h = 5$, $k_v = 5$	$k_h = k_v = 15$
		(Urato side) 		

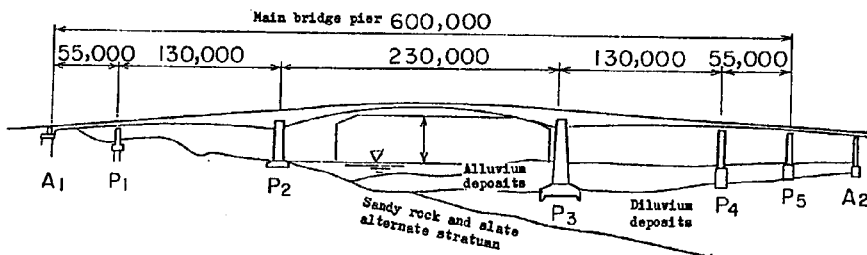
(Note) Unit: (kg/cm³)



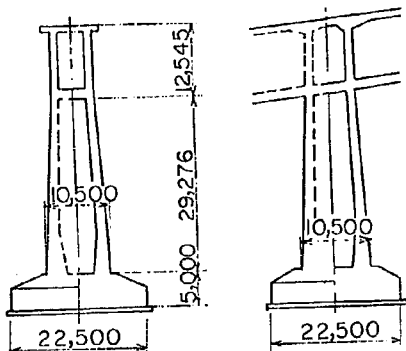
P₃ geologic columnar section

Original form of foundation and analysis model:

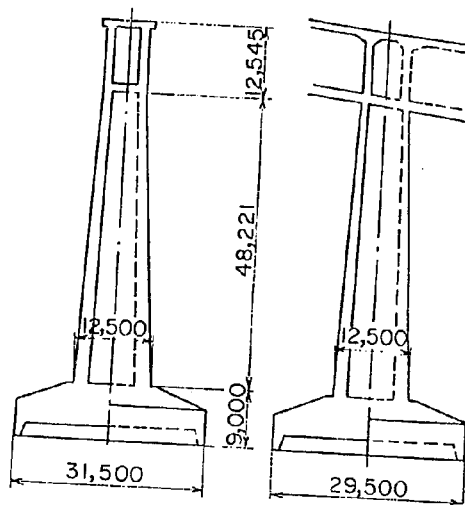
- o The entire bridge system was considered.
- o The damping constant was $h = 0.02$ uniformly for all the modes.
- o As for the input seismic wave, El-Centro (May 1940, N-S, 0.33 g) was used and the maximum acceleration was 200 gal 8 seconds.



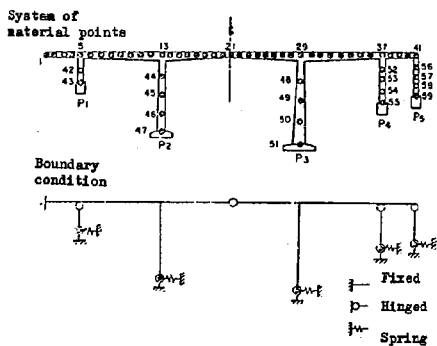
General side view and positional diagram



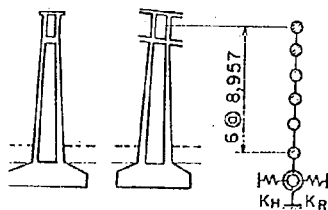
(a) P₂ pier



(b) P₃ pier



Analysis model

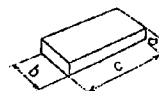


Calculation model

$$K_H = k \cdot a \cdot b + \frac{1}{4} \cdot k \cdot a \cdot b$$

$$K_R = \frac{1}{3} \cdot k \cdot \left(\frac{c}{2}\right)^2 \cdot h \cdot c$$

$$k = 0.15 N$$



(6) Type of foundation:

Caisson foundation (Kanmon-kyo)

Bridge:

Literature:

Civil Engineering Technology, Oct. 1970

Location and organization concerned:

Shimonoseki, Yamaguchi Pref. Moji, Fukuoka Pref.

Japan Highway Public Corporation

Purpose of analysis:

Analysis method:

Mean response spectrum analysis by the model of motion of 2-degree of freedom system

Analysis results, evaluation and problems:

- o In the case of a deep foundation, there arise considerable differences between the seismic coefficient method and the 2-degree of freedom system. The seismic coefficient method produces a greater degree of safety for sliding.

Safety calculation results of seismic coefficient method and
2-degree of freedom system (Bridge axial direction)

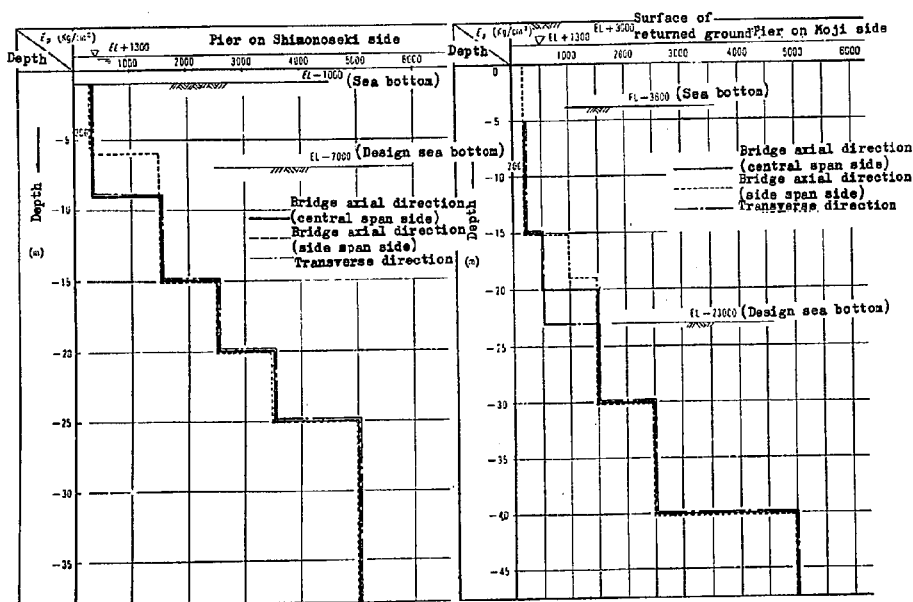
Division	Pier on Shimonoseki side		Pier on Moji side	
	Seismic coefficient method	2-degree of freedom system	Seismic coefficient method	2-degree of freedom system
Vertical modulus of ground deformation (t/m ²)		1.2 x 10 ⁵		1.2 x 10 ⁵
Period (sec)		0.46		0.64
Center of rotation (m)		34.8		26.3
Displacement of the gravity center of model (cm)		1.0		1.63
Angle of rotation of model (rad)		2.87 x 10 ⁻⁴		6.29 x 10 ⁻⁴
Subgrade reaction (t/m ²)	83	128	125	221
Distance of eccentricity (m)	1.92	3.98	2.77	6.63
Consideration to bearing power (t/m ²)	83 < 360	128 < 360	125 < 360	221 < 360
Consideration to sliding	5.4 > 1.5	3.2 > 1.5	5.8 > 1.5	3.4 > 1.5
Consideration to tipping (m)	e = 1.92 $\frac{2e}{3} - 1.92$ $> \frac{2e}{6}$	e = 3.98 $\frac{2e}{3} - 3.98$ $> \frac{2e}{6}$	e = 2.77 $\frac{2e}{3} - 2.77$ $> \frac{2e}{6}$	e = 6.63 $\frac{2e}{3} - 6.63$ $> \frac{2e}{6}$

Type of ground and evaluation:

- o The foundation rock are so much weathered and cracked that the deformation was determined from the correlation of deformation characteristics obtained by pressiometer observations at site and jack tests.

$$\left(\frac{\text{Deformation modulus obtained from the envelop in jack test}}{\text{Deformation modulus obtained by pressiometer}} \right) \times \left(\frac{\text{Deformation modulus obtained by the iteration curve in jack test}}{\text{Deformation modulus obtained by the envelop in jack test}} \right) = 5 E_p$$

where, E_p = deformation modulus obtained by pressiometer



Simonoseki-side pier base
 E_p (kg/cm^2)

Moji-side pier base
 E_p (kg/cm^2)

Engineering ground constants (Bridge axial direction)

I t e m	Symbol	Shimonoseki-side pier		Moji-side pier	
		Earthquake time	Normal time	Earthquake time	Normal time
Modulus of deformation of bearing stratum (t/m ²)	E _v	1.2 × 10 ⁵	0.6 × 10 ⁵	1.2 × 10 ⁵	0.6 × 10 ⁵
Vertical spring constant of bearing stratum (t/m ³)	K _v	5.27 × 10 ³	2.63 × 10 ³	5.3 × 10 ³	2.65 × 10 ³
Horizontal spring constant of the side (t/m ³)	K _i	$\begin{cases} K_1 = 156 \\ K_2 = 470 \\ K_3 = 2340 \end{cases}$	$\begin{cases} K_1 = 78 \\ K_2 = 235 \\ K_3 = 1170 \end{cases}$	$\begin{cases} K_1 = 312 \\ K_2 = 780 \\ K_3 = 2340 \end{cases}$	$\begin{cases} K_1 = 156 \\ K_2 = 390 \\ K_3 = 1170 \end{cases}$
Rotational spring constant of sea bottom (t/m)	K _θ	4.1 × 10 ⁸	2.05 × 10 ⁸	4.2 × 10 ⁸	2.08 × 10 ⁸
Shearing spring constant of sea bottom (t/m ³)	K _s	1.76 × 10 ³	0.88 × 10 ³	1.77 × 10 ³	0.88 × 10 ³
Horizontal frictional coefficient	μ	0.6	0.6	0.6	0.6
Internal friction angle of rock (degree)	φ	35°	35°	30°	30°
Cohesion of rock (t/m ²)	c	13.0	13.0	10.0	10.0

Ground constants in the case of the entire system

< Shimonoseki-side >

$$K_s = K'_s = 1.44 \times 10^6 \text{ t/m}$$

$$K_{H1} = 0.112 \times 10^6 \text{ t/m}$$

$$K'_{H1} = 0.094 \times 10^6, K'_{H2} = 0.281 \times 10^6 \text{ t/m}$$

$$K_R = 4.22 \times 10^8 \text{ t·m}$$

< Moji-side >

$$K_s = K'_s = 1.411 \times 10^6 \text{ t/m}, K_R = 4.137 \times 10^8 \text{ t·m}$$

$$\left. \begin{aligned} K_{H1} &= 0.280 \times 10^6, K_{H2} = 0.156 \times 10^6 \\ K_{H3} &= 0.142 \times 10^6, K'_{H1} = 0.375 \times 10^6 \\ K'_{H2} &= 0.250 \times 10^6, K'_{H3} = 0.2225 \times 10^6 \end{aligned} \right\} \text{ t/m}$$

o Added mass of water

$$M_v = \frac{0.6 P \cdot A \cdot h}{(a/h)^{3/2}}$$

where $0.8 \leq a/h \leq 4.0$

M_v : virtual mass of water (t)

P : density of water (t/m³)

A : sectional area (m²)

a : width of section perpendicular to the direction of vibration (m)

h : depth of water (m)

o Added mass of surrounding soil

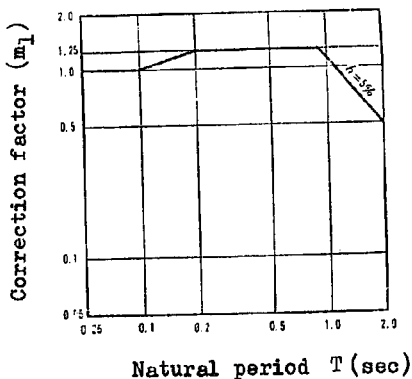
$$M_s = (1/2) \cdot (\gamma_s/g) b^2 \cdot d$$

where M_s : virtual mass of soil (t·sec²/m)

γ_s : unit weight of soil (t/m³)

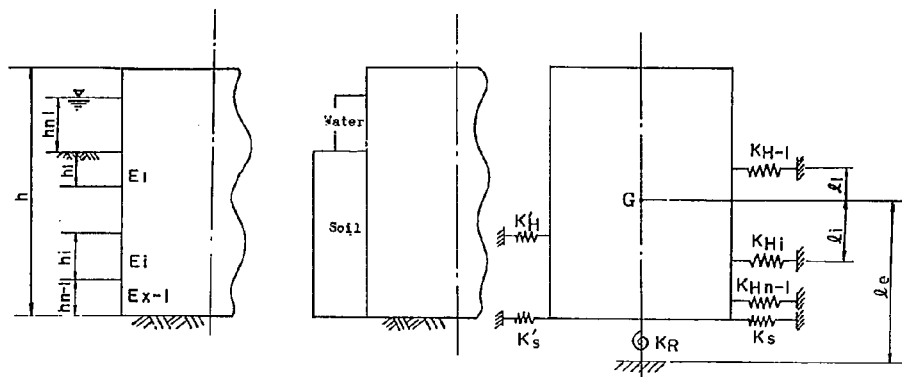
b : width of structure perpendicular to the direction of vibration (m)

d : thickness of soil stratum (m)



Acceleration response spectrum curve

Original form of foundation and analysis model:



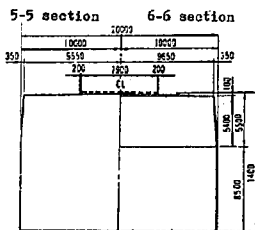
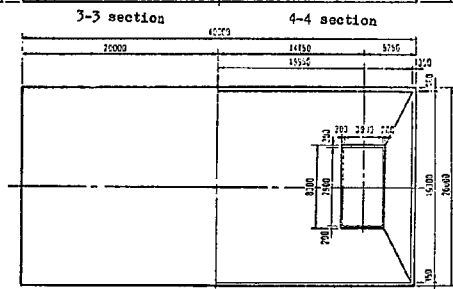
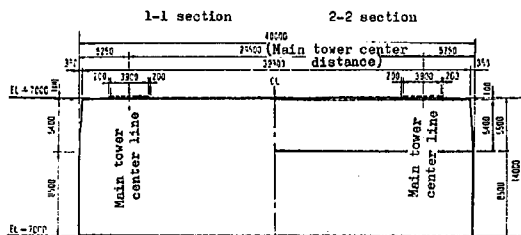
o Shimonoseki-side pier $h/2 = 11.43$ m (in the entire system)

Moji-side pier $h/2 = 16.85$ m (in the entire system)

Various quantities of dynamic model (Bridge axial direction)

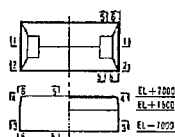
I t e m	Symbol	Shimonoseki-side pier	Moji-side pier
Length in bridge axial direction (m)	a	20.0	20.0
Position of gravity center of model (m)	h_a	9.4	16.55
Overall height of model (m)	h_0	18.8	33.1
Unit weight of model (t/m ³)	W	3.77	3.31
Virtual mass of model (t·sec ² /m)	M	5.7×10^3	8.9×10^3
Rotational inertia with respect to the gravity center of model (t/m·sec ²)	J	3.57×10^5	1.115×10^6

< Shimonoseki side >

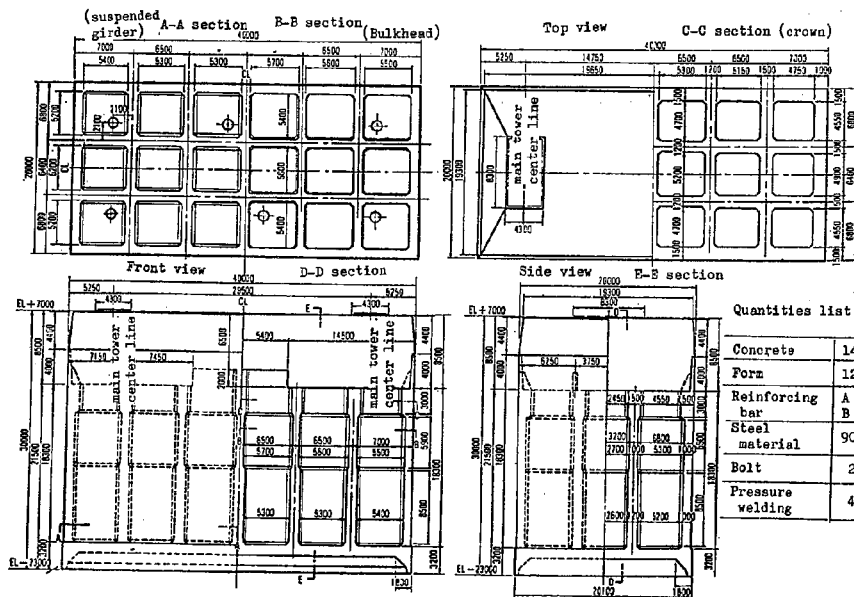


Concrete	11042 m ³
Form	1654 m ² (side form) 277 m ² (partitioned form)
Steel bar	A total 123907 kg B total 747003 kg
Steel material	40200 kg (angle) 1030 kg (plate)
Iron	720 (joint) 895 (cross)
Pressure welding	3420 places (D 29) 6720 places (D 32)

Quantities list



General view of Shimonoseki-side pier



Concrete	14490 m ³
Form	12168 m ²
Reinforcing bar	A total 903873 kg B total 1235127 kg
Steel material	90602 kg
Bolt	2456
Pressure welding	4600 places

< Moji side >

General view of Moji-side pier

(7) Type of foundation:

Caisson foundation (Shin-iinogawa-bashi)

Bridge:

3-span continuous steel bed plate box-girder bridge

Bridge length 441.5 m (2 @ 3 x 73.2)

Literature:

Civil Engineering Society Abstracts of papers

Location and organization concerned:

Kawakita-cho, Tosei-gun, Miyagi Pref. (Kitagami River)

Tohoku District Construction Bureau, Ministry of Construction

Purpose of analysis:

Changes in the vibration behavior of caisson before and after the setting of girders

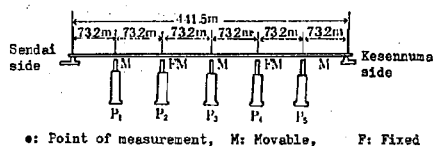
Analysis method:

Fourier spectrum analysis and correlation function analysis

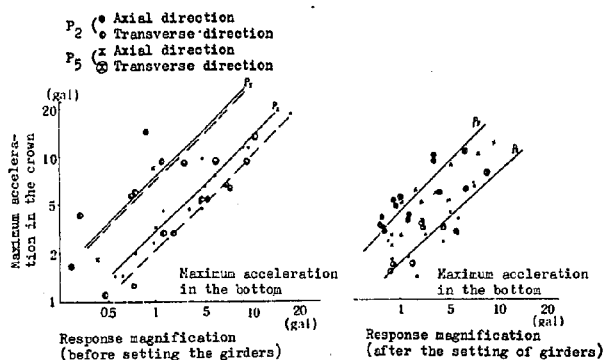
Analysis results, evaluation and problems:

o The period of the caisson became longer after the setting of the girders and the center of rocking of caisson moved upward while the response magnification decreased.

There was not much difference in the damping constant.



Side view of the piers of Shin-iinogawa-bashi bridge



Predominant period

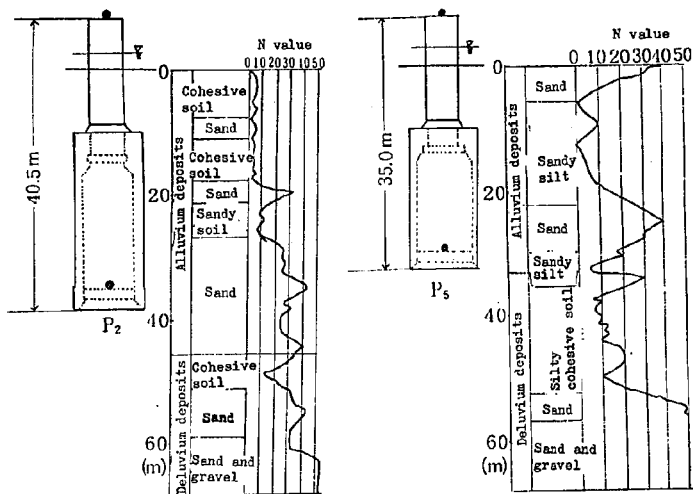
		Component	Bottom	Crown
P_2	Before the setting of girders	Axial direction	<u>0.18 ~ 0.20</u> , 0.40 ~ 0.50	<u>0.18 ~ 0.20</u> , 0.40 ~ 0.50
		Transverse direction	<u>0.18 ~ 0.20</u> , 0.40 ~ 0.50	<u>0.16 ~ 0.20</u> , 0.40 ~ 0.50
		Vertical direction	0.46	0.46
	After the setting of girders	Axial direction	<u>0.18 ~ 0.20</u> , 0.40, 0.95 ~ 1.0	0.18 ~ 0.20, <u>0.40 ~ 0.46</u> , 0.95 ~ 1.0
		Transverse direction	<u>0.16 ~ 0.20</u> , 0.40 ~ 0.45	0.15 ~ 0.20, <u>0.40 ~ 0.46</u>
		Vertical direction	0.20, <u>0.45</u>	0.16 ~ 0.20, 0.40 ~ 0.45
P_5	Before the setting of girders	Axial direction	0.20, 1.0	0.15, <u>0.30</u> , 1.0
		Transverse direction	<u>0.20</u> , 0.90	0.17, <u>0.30</u> , 1.1
		Vertical direction	0.46	0.46
	After the setting of girders	Axial direction	<u>0.17 ~ 0.20</u> , 0.40 ~ 0.46, 0.95 ~ 1.0	0.16 ~ 0.20, <u>0.40 ~ 0.46</u> , 0.95 ~ 1.0
		Transverse direction	<u>0.16 ~ 0.20</u> , 1.0	0.16 ~ 0.20, <u>0.35 ~ 0.42</u> , 1.0
		Vertical direction	0.20, <u>0.50</u>	0.20, <u>0.45 ~ 0.50</u>

Underlined are outstandingly predominant

(Unit: second)

Type of ground and evaluation:

Original form of foundation and analysis model:



A brief description of the caisson foundation (P_2 , P_5) and the surrounding ground composition

(8) Type of foundation:

Caisson foundation (Minato Ohashi)

Bridge:

Cantiler truss bridge

Bridge length 980 m (235 + 510 + 235)

Literature:

Ground and Foundations, Sept. 1975

Civil Engineering Technology, May 1970

Location and organization concerned:

Osaka City Hanshin Expressway Public Corporation

Purpose of analysis:

In the case of a very large caisson, it is desirable to make rational design taking the dynamic characteristics into consideration.

Analysis method:

Direct responses to seismic waves of the equivalent 2-degree of freedom system of ground - foundation by the spring-dashpot system

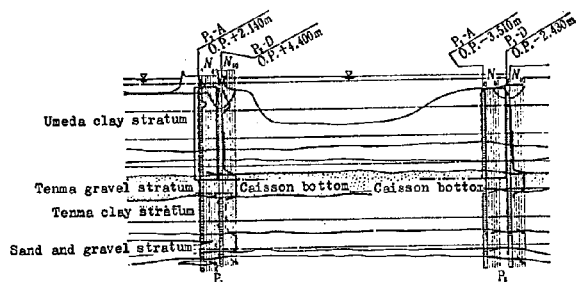
Analysis results, evaluation and problems:

- o The superstructure was assumed to be an added mass contributing to the vibration of the substructure and given to the 2-degree of freedom model of the caisson. In order to check the validity, the superstructure and substructure as a whole was replaced by a multiple-point model. The results were little different from the entire system in the axial direction while left some problems in the transverse direction as shown in the table at the right.

(Damping constant $h = 0.05$)

- 64 -

Type of ground and evaluation:



- o The subgrade spring constant is given by modified Boussinesq formula on the basis of the modulus of deformation obtained by pressiometer.

$$k_v = \frac{E}{I_p (1-\nu^2) B_v \lambda}$$

where k_v : subgrade spring constant (t/m³)

E : modulus of deformation of subgrade (t/m²) = $6E_0$

E_0 : modulus of deformation by pressiometer (t/m²)

B_v : equivalent loading width of foundation (m) = $\sqrt{A_c}$

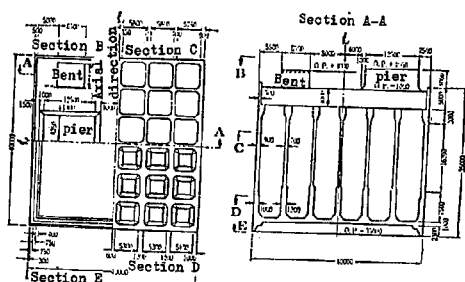
ν : Poisson's ratio

I_p : shape factor, A_c : vertical loading area (m²)

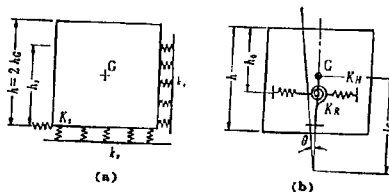
Predominant period of spectrum density

Point of measurement	Measuring component	Predominant period	Max. value
Nanko reclaimed land	H, EW	0.30, 0.50, 0.70	0.70
	H, NS	0.76, 0.98, 1.40	0.76
	V	0.50, 0.68, 1.05	0.68
No.2 jetty	H, EW	0.30, 0.66, 0.05	0.66
	H, NS	0.64, 0.96, 1.40	0.96
	V	0.48, 0.66, 0.82	0.66

Original form of foundation and analysis model:

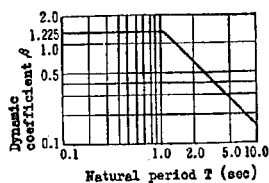


General view of caisson (P_2 caisson)



Spring action of subgrade

- o The effects of superstructure and water were treated as added mass.
- o As for the dynamic coefficients (β), the graph at the right was used.
- o The adjacent ground was so soft that it was assumed to be plastic when the horizontal reaction exceeded the allowable bearing capacity.



Acceleration response spectrum

Vertical load acting on the bottom of caisson

(Unit: ton)

	P ₂ caisson	P ₃ caisson	Remarks
Superstructure load	27,516 (25,357)	27,516 (25,357)	
Dead load	23,198	23,198	
Live load	4,318 (2,159)	4,318 (2,159)	
Pier weight	4,363	6,577	
Caisson weight	61,504	57,957	
Weight of water in caisson	29,598	25,419	
Weight of earth and sand	6,898	-	Earth and sand on top plate
Uplift	-51,912	-50,264	100% considered at below low water level
Total	77,967 (75,808)	67,205 (65,046)	

Bracketed are the values 1/2 of live load, which are for use in the investigation of preconsolidation load.

(9) Type of foundation:

Caisson foundation (Hamana Ohashi)

Bridge:

Central span with central hinge, 5-span continuous rigid frame

Bridge length: 631.8 m (50 + 140 + 240 + 140 + 50)

Literature:

Foundation, Dec. 1974

Location and organization concerned:

Between Shinohara-cho, Hamamatsu City and Nii-machi, Hamana-gun,
Shizuoka Prefecture (Hamana Bypass)

Japan Highway Public Corporation

Purpose of analysis:

Since this is such a long bridge with its central span as long as 240 meters and a relatively long period, analysis was made to compare with the results of the modified seismic coefficient method

Analysis method:

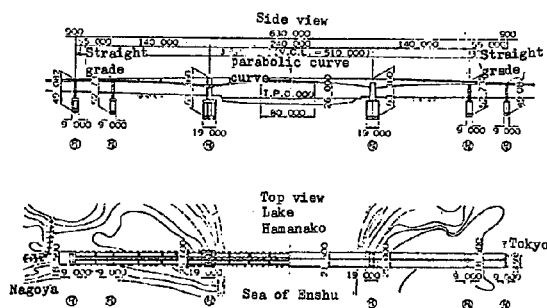
Seismic response analysis (), spectrum mode analysis

Analysis results, evaluation and problems:

- o The maximum response bending moment (shown in the table at below right) of the upper and lower ends of P_3 and P_4 piers were smaller than the results of the modified seismic coefficient method considering the responses by the "Guide for Earthquake Resistant Design for Road Bridges" (Japan Road Association)
- o What govern are the first, third and fifth order vibrations from the spectrum analysis and the comparison of the three subgrade

reaction coefficients little affected the natural period and stimulus coefficient.

- o Since the substructure is greatly influenced by the shape of the superstructure, the former should be designed after the design of the latter has been roughly determined. In the case of a long bridge, it was appropriate to make seismic response analysis in the basic design stage.

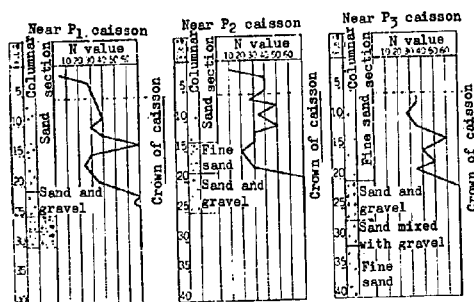


General view

Comparison of numerical values

	Upper end of pier	Lower end of pier
Seismic response analysis	22.000 ^{t.m}	72.000 ^{t.m}
Statistic analysis	26.000	88.000

Type of ground and evaluation:



Geologic columnar section

Subgrade reaction coefficients

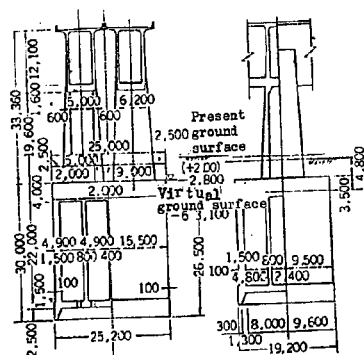
Subgrade reaction coefficient		Value (kg/cm ²)
Horizontal subgrade reaction coefficient	K_H	5.0 (EL-2.8 ~ EL-17.8)
		15.0 (EL-17.8 ~ EL-32.8)
Side shearing subgrade reaction coefficient	K_S	0.5 (EL-2.8 ~ EL-17.8)
		1.5 (EL-17.8 ~ EL-32.8)
Vertical subgrade reaction coefficient	K_V	20.0
Bottom shearing subgrade reaction coefficient	K_B	10.0

o As for the horizontal and vertical subgrade reaction coefficients, reference was made to the "Road Bridge Substructure Design Guide (Caisson)", the theory of elasticity (Pauw, Tajimi, Richart) and Urato Ohashi vibration test data. Respecting test results, the

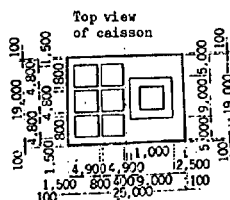
- bottom shearing reaction coefficient was taken at $K_B = 0.5 \cdot K_V$.
- o The side shearing subgrade reaction was taken at $K_S = 1/10 \cdot K_H$ in accordance with the Caisson Guidance.
 - o Three subgrade reaction coefficients ($K_V = 10, 20 \text{ and } 30 \text{ kg/cm}^3$) were used for comparison because it would be dangerous to use any specific value.
 - o The seismic waves of the maximum accelerations recorded in the El Centro (1940) and Hachinohe (1968) earthquakes converted in terms of 200 gal were used in accordance with the "Road Bridge Earthquake Resistant Design Guidance".
- 180 gal in the "Honshu-Shikoku Earthquake Resistant Bridge Design Guide (Draft)".

Original form of foundation and analysis model:

Axial sectional view



P₃ and P₄ bridge piers



(10) Type of foundation:

Caisson foundation (Akashi Straits Bridge)

Bridge:

Literature:

Bridges and Foundations, Sept. 1972

Location and organization concerned:

Honshu-Shikoku Bridge Authority

Purpose of analysis:

Investigations for the type of foundation of suspension bridge tower system

Analysis method:

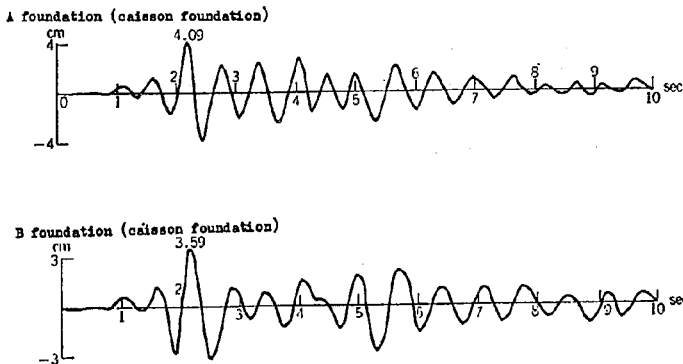
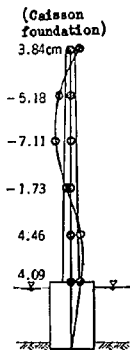
Seismic response analysis of multiple-degree of freedom system

Analysis results, evaluation and problems:

- o In the case of a caisson foundation, the response would vary according to the ground spring but this variation would not be so serious as to influence the design of the foundation.

Maximum response displacement

Displacement response (crown of caisson)

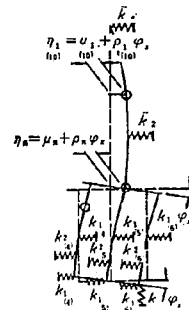


Type of ground and evaluation:

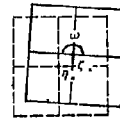
- o The ground was assumed to be of three degrees of softness.

Ground	(1)	(2)	(3)
Modulus of elasticity (t/m^2)	45,000	140,000	300,000
Vertical spring (t/m^3)	3,151	11,670	25,009
Horizontal spring (t/m^3)	6,705	20,870	44,726

Dynamic model



(a) Displacements of tower and pier

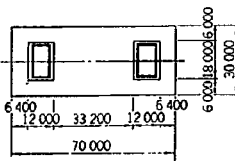
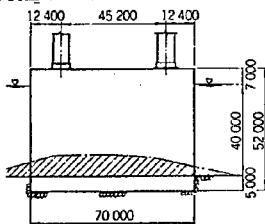


(b) Horizontal displacement of top plate

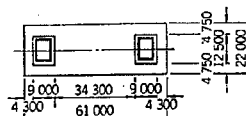
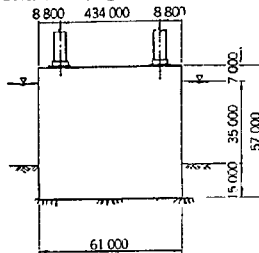
Original form of foundation and analysis model:

- o As for the input, Elcentro NS, $\nu = 0.05$,
1/200 sec was used.

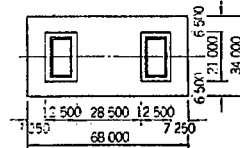
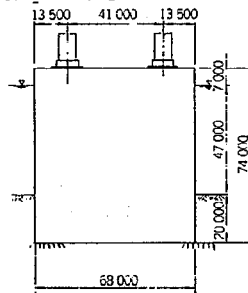
A foundation(rigid) $L = 1500$



B foundation(rigid) $\angle = 860$



```
C foundation(rigid) L=1880
```



(11) Type of foundation:

Multi-column foundation (Akashi Straits Bridge)

Bridge:

Literature:

Bridges and Foundations, Sept. 1972

Location and organization concerned:

Hyogo Prefecture

Honshu-Shikoku Bridge Authority

Purpose of analysis:

Investigations for the type of foundation of suspension bridge
tower system

Analysis method:

Analysis of seismic responses in two directions (X and Y) of a multiple-
degree of freedom system

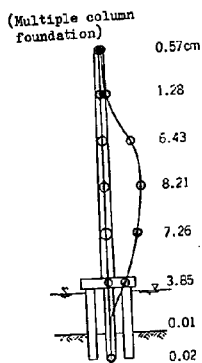
Analysis results, evaluation and problems:

- o Since the multiple column foundation is a flexible structure, it is necessary to take into consideration the elastic deformation of the foundation body in the safety analysis of the foundation during earthquake.
- o There is the necessity for three-dimensional behavior analysis owing to the earth slope, arbitrariness of seismic input direction, eccentricity of superstructure reaction, and asymmetry of foundation structure.
- o The effect of group piles was considered because the contact of

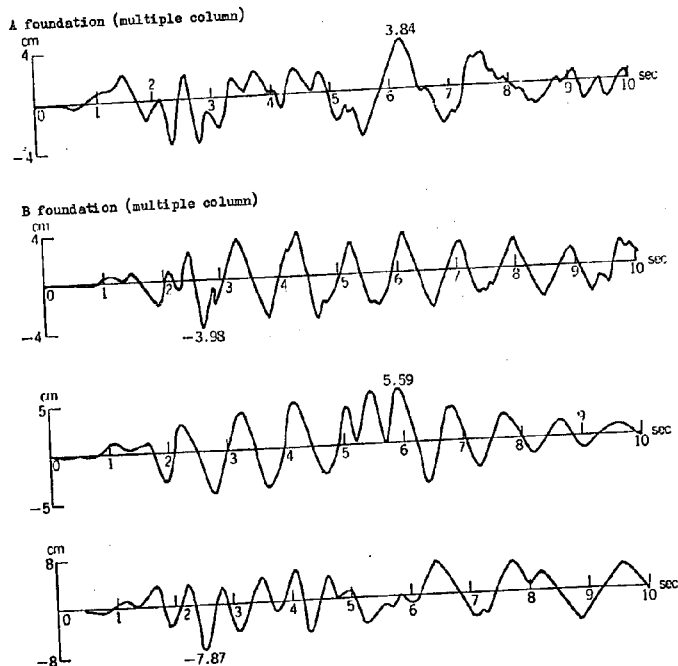
the foundation with the soil was so complicated and it was not so simple to calculate the spring coefficient of each column.

- o There were many uncertainties in the estimation of the virtual mass of ground, damping constant, etc.
- o For the above reasons, analysis was made, assuming the foundation to be a three dimensional rigid frame structure.

Maximum response displacement



Displacement response curve (crown of foundation)



Periods and Deformation

		Rigid foundation			Multiple column foundation		
Stratum		(1)	(2)	(3)	(1)	(2)	(3)
Ed	(t/m ²)	45 000	140 000	300 000	"	"	"
Kv	(t/m ³)	3 751	11 670	25 009	10 700	33 280	71 330
KH	(t/m ³)	6 709	20 870	44 726	12 800	39 810	85 330
Calculated value	Period of foundation (sec)	0.5		0.18	0.45	0.43	0.44
	period of tower (sec)	0.5		1.2	0.46	1.0	1.1
	Horizontal displacement of the crown of foundation (cm)	4.5		0.7	5.3	4.4	4.2
	Maximum horizontal displacement of tower (cm)	18.4		12.0	11.6	11.7	11.7
	Angle of rotation (rad)	0.54x10 ⁻³		0.09x10 ⁻³	0.23x10 ⁻³	0.19x10 ⁻³	0.18x10 ⁻³

- o Three kinds of spring constants (spring ratio) for multiple columns were used but there was almost no effect thereof.
(For foundation A).

Periods and deformations due to nonuniformity of spring constants

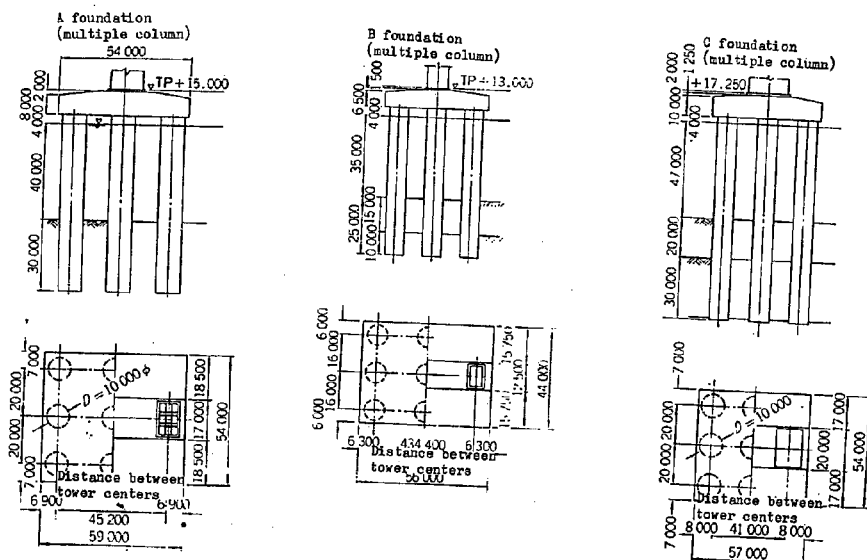
Item	spring distribution ratio	1 : 1 : 1	$\frac{1}{2}$: 1 : 2	1/4 : 1 : 4
Period of foundation (sec)		0.45	0.44	0.45
Period of tower (sec)		0.46	0.48	0.45
Horizontal displacement of crown of foundation (cm)		5.3	5.5	6.2
Maximum horizontal displacement of tower (cm)		11.6	11.9	12.8
Angle of rotation (rad)		0.23 x 10 ⁻³	0.23 x 10 ⁻³	0.27 x 10 ⁻³

Type of ground and evaluation:

- o Three degrees of softness of ground were assumed.

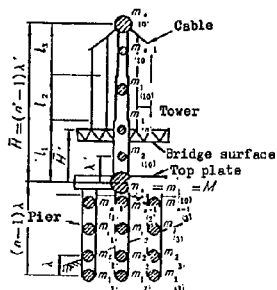
Ground	(1)	(2)	(3)
Modulus of elasticity (t/m ²)	45,000	140,000	300,000
Vertical spring constant (t/m ³)	10,700	33,280	71,330
Horizontal spring constant (t/m ³)	12,800	39,810	85,330

Original form of foundation and analysis model:

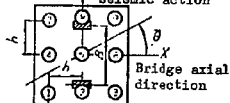


Multiple material points model

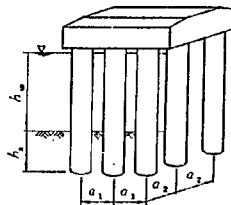
Ground support model



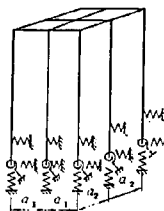
(a) Side view
Direction of seismic action



(b) Top view



(1) Actual multiple
column foundation



(2) Model

- o As for the input, Elcentro NS $\nu = 0.05$ 1/200 sec was used in the bridge axial direction.
- o The rotations in x and y directions at each material points were selected as unknown quantities in the equation of motion for the pier, rotations in x and y directions and torsion for the tower, rotations in x and y directions and torsion for the top plate of foundation. (Pier had 2 degrees of freedom, tower 4 degrees of freedom, and top plate 10 degrees of freedom.)

(12) Type of foundation:

File foundation (Showa Ohashi)

Bridge:

Simple girder type

Bridge length 306.4 m (10 @ 27.64)

Literature:

Bridges and Foundations, Mar. 1970

Location and organization concerned:

Niigata, Niigata Prefecture

Public Works Research Institute, Ministry of Construction

Purpose of analysis:

Comparison of actual earthquake phenomena and seismic response
analysis results

Analysis method:

Response spectrum method, 1-degree of freedom system seismic
response analysis

Analysis results, evaluation and problems:

- o Whether the structural system is linear or nonlinear, the response of the crown of pier remained unchanged during 8 to 9 seconds after the occurrence of earthquake and thereafter it began to change and became larger in the case of linear system.
- o The maximum response of linear system (54.1m) coincided with the maximum response of modal analysis ($2.1 \times 26 = 54$ cm) but it was smaller in the case of nonlinear system (40.1 cm)

o Causes of bridge failure

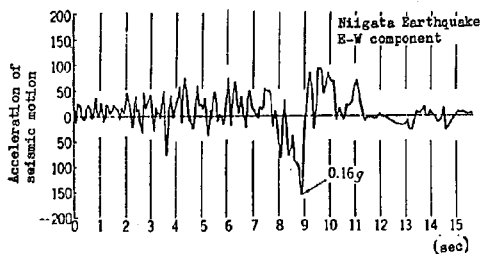
- . Considering the fluidization of ground, the displacement becomes 29 to 54 cm. There is the possibility of the bridge falling when it exceeds 50 cm.
- . When the displacement of the top of pier exceeds 20 cm, the pile would yield and even if the damping effect of the yielding of piles is considered, there would occur a horizontal displacement of 40 cm.
- . Since it is impossible for the piers to vibrate with the same period and phase, the distance between piers would increase to 10 cm or so, thus increasing the possibility of the bridge falling.

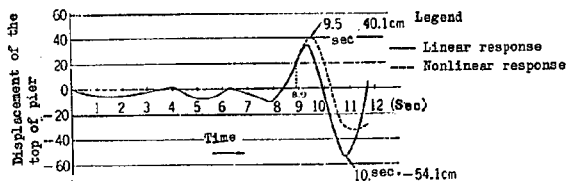
o Precautions for reconstruction of the bridge

- . It is necessary to increase the rigidity and strength of the piers.

Steel pile 600mm $\rightarrow$ 1000 mm or 2 x 1600 mm

Displacement response of the top of pier
(linear, nonlinear)





Momentary changes

- o Natural period with pier pile length (l), pile diameter (D) and subgrade reaction coefficient (K_s): T

$T \sim l$

		$D=0.61m$				
k_s	l	8m	10	12	15	18
1.0 kg/cm ³		2.3 sec	2.9	3.8	4.9	6.2
∞		1.3	1.8	2.4	3.4	4.4

$T \sim D$

$l=8m, k_s=1.0 \text{ kg/cm}^3$

$D(m)$	0.61	0.81	1.02
$T(sec)$	2.3	1.6	1.3

$T \sim k_s$

$D=0.61m, l=8m$

$k_s(kg/cm^3)$	0.1	1.0	10.0	∞
$T(sec)$	3.2	2.3	1.8	1.3

o Comparison of responses by four kinds of spectrum

Relative displacement of the top of pier (x 26cm)

Kind of response spectrum	Pile dia. (D) = Height above ground (l) =		0.61			0.81	1.02	Remarks
	8	10	18	8	8			
(S _A) _{0.2g} = const	Note 1) 1.0	1.6	7.3	0.5	0.3	Note 1) This actual displacement in this case is 26 cm Note 2) Subgrade reaction coefficient K _s = 1 kg/cm ³ Note 3) Damping constant $\frac{n}{p} = 0.02$		
(S _A) _{av.} (earthquake motion 0.2 g)	0.8	1.1	2.2	0.5	0.5			
(S _A) _{E-W} (earthquake motion 0.16 g)	1.3	2.1	5.0	0.7	0.2			
(S _A) _{N-S} (earthquake motion 0.14 g)	1.3	1.4	10.0	0.8	0.3			
Natural period of first order (sec)	2.3	2.9	6.1	1.6	1.25			

Shearing force per pier (t)

$\begin{array}{c} D \text{ (m)} \\ l \text{ (m)} \end{array}$ (S_A)	0.61			0.81	1.02	Remarks
	8	10	18	8	8	
$(S_A)_{0.2g} = \text{const}$	^t 125	^t 130	^t 140	^t 135	^t 145	$K_s = 1 \text{ kg/cm}^3$ $\frac{n}{p} = 0.02$
$(S_A)_{av.}$	105	95	105	155	215	
$(S_A)_{E-W}$	165	180	135	175	120	
$(S_A)_{N-S}$	200	120	230	210	155	

Type of ground and evaluation:

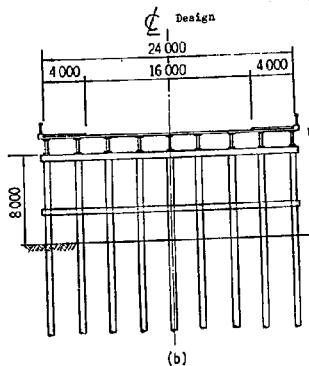
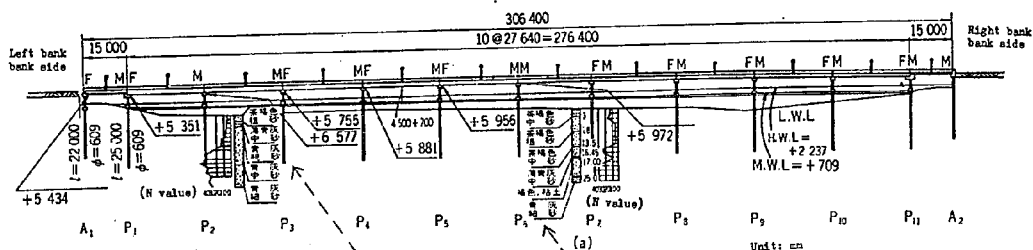
o Subgrade reaction coefficient: K_S

From measurement of actual bridge vibration K_S = 1.0 kg/cm³

Assuming the ground to be perfectly rigid K_S = ∞

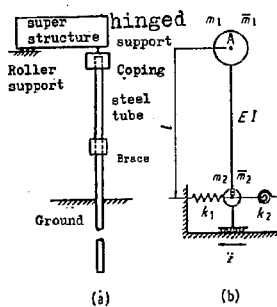
o The effect of liquefaction of surface stratum (during earthquake time) was assumed to be about 2 m.

Original form of foundation and analysis model:

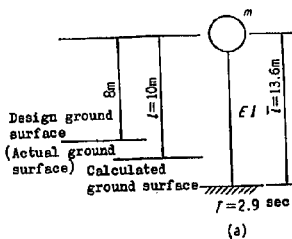
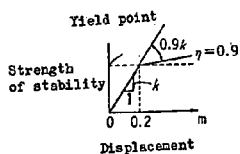


Showa Ohashi
(old)

Brown
coarse sand
Pale bluish gray
medium-grain sand
Bluish gray
coarse sand
Bluish gray
medium-grain sand
Bluish
fine-grain sand



Nonlinear characteristics



o The mass at the point A is a part of the weight of superstructure plus the weight of pier and the mass at the point B assumed to be a part of the weight of pier plus the weight of earth.

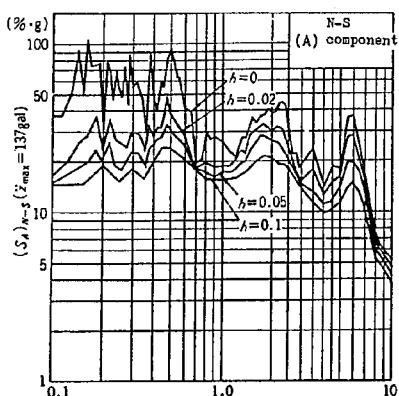
o Various numerical values for the diameters of steel pile foundation.

	ϕ 812.8 mm		ϕ 1,016 mm		ϕ 609.6 mm	
	8 m	18 m	8 m	18 m	8 m	18 m
Ground K_a (t/m)	3.360×10^4	3.36×10^4	4.225×10^4	4.225×10^4	1.71×10^4	1.71×10^4
Point A Weight (t)	50	113	77	174	30	70
Point B Weight (t)	610	670	630	730	590	630
Moment of inertia (t·m sec ²)	7.11	79.33	10.92	120.55	4.98	50.0
Moment of inertia of area (m ⁴)	2.862×10^{-2}	2.862×10^{-2}	5.652×10^{-2}	5.652×10^{-2}	1.188×10^{-2}	1.188×10^{-2}

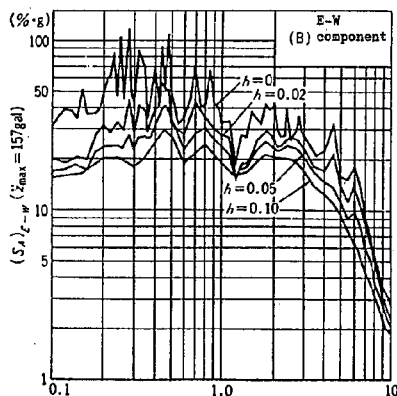
o The following seismic response spectrum curves were used.

- (1) Public Work Research Institute's mean response spectrum curve (seismic motion 0.2 g): $(S_A)_{av}$
- (2) Response spectrum curve by the waveform E-W component of Niigata Earthquake (seismic motion 0.16g): $(S_A)_{E-E}$
- (3) Response spectrum curve by the waveform N-W component of Niigata Earthquake (seismic motion 0.14g): $(S_A)_{N-S}$
- (4) 0.2 g constant response spectrum curve: $(S_A)_{0.2g}$

Response acceleration spectrum curves of Niigata Earthquake records



Natural period T (sec)



Natural period T (sec)

(13) Type of foundation:

Pile foundation (Nagadoro-bashi)

Bridge:

Literature:

Bridges and Foundations, June 1969

Location and organization concerned:

Aomori Prefecture

Public Works Department, Aomori Prefecture

Purpose of analysis:

Seismic responses in the soft ground

Analysis method:

Seismic response calculations by modal analysis

Analysis results, evaluation and problems:

- o In the case of soft ground, when comparing the standard waveform and El Centro waveform, M_{max} became excessively small and H_{max} excessively large, therefore it seems inappropriate to use the short-period EL Centro waveform.
- o H_{max} response by the standard waveform corresponded to the seismic coefficient method ($K_R = 0.13$) and safety was assured because 0.25 was used in the design.
- o The values produced by the modal analysis were close to the values obtained by the standard waveform, showing that this is a highly reliable method of calculation.

Tipping Mmax and horizontal shearing Hmax

	Mmax(kg cm)	Hmax (kg)
(1) Right response calculation (standard waveform)	10.55×10^7	96.4×10^3
(2) Right response calculation (ElCentro waveform)	8.93×10^7	196.9×10^3
(3) Modal analysis calculation (Standard waveform)	11.25×10^7	113.7×10^3
(4) Previous seismic coefficient method (horizontal seismic coefficient: 0.13)	9.92×10^7	126.4×10^3
(5) Previous seismic coefficient method (horizontal seismic coefficient: 0.25)	19.08×10^7	243.0×10^3

Type of ground and evaluation:

- o It was assumed that the soft ground $K_H = 2.0 \text{ Kg/cm}^3$ from the results of the horizontal load test of batter piles (Design of Foundation Piles for Nagadoro Bridge and Load Test, Bridges and Foundation, May 1968).

Original form of foundation and analysis model:

- o Estimation of standard waveform (estimated seismic waves)

The following equation was obtained according to Dr. Kanai's experimental equation for the vibration behavior of earthquake motion".

$$\ddot{x}_{max} = \frac{1}{T} \times \frac{10^{0.41M - P \log x + Q}}{\sqrt{\left\{1 + \left(\frac{T}{T_g}\right)^2\right\} + \left\{\frac{0.2}{\sqrt{T_g}} \left(\frac{T}{T_g}\right)\right\}^2}} \dots\dots\dots(19)$$

where T : period of waves from the base of ground during earthquake

(which varies according to x and M) (sec)

T_g : natural period of ground (sec)

$$P = 1.66 + \frac{3.60}{x}, \quad Q = 0.167 - \frac{1.80}{x}$$

The maximum speed $\dot{Z}_{max}$ and maximum displacement Z_{max} were obtained by the following equation.

$$\dot{z}_{max} = \frac{T}{2\pi} \ddot{z}_{max}, \quad z_{max} = \left(\frac{T}{2\pi}\right)^2 \ddot{z}_{max} \quad \left\{ \dots\dots\dots (20) \right.$$

Equations (19) and (20) are as shown in Fig. 10 (See literature).

When $x = 150$ km and $M = 8$, looking sideways from the $T_g = 0.6$ sec acceleration curve, we find $\ddot{Z}_{max} = 130$ gal (gal = cm/sec²) in the case of (III). Looking at the speed curve, we can estimate $\dot{Z}_{max} = 12.5$ cm/sec and in the case of the displacement curve, it is estimated that $Z_{max} = 1.2$ cm. Thus, we finally decided to take $\ddot{Z}_{max} = 130$ gal as the maximum value of the earthquake motion acceleration which would occur in the surface of the ground of the Nagadoro bridge. Speaking about the waveform of acceleration in this case, we approximately used the sine wave of maximum value $\ddot{Z}_{max} = 130$ gal and period $T_g = 0.6$ sec as in Fig. 11, which was considered as the waveform of acceleration to occur in the surface of the ground of Nagadoro bridge during earthquake. (hereafter this will be called the standard waveform of Nagadoro bridge.)

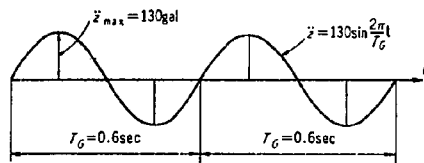
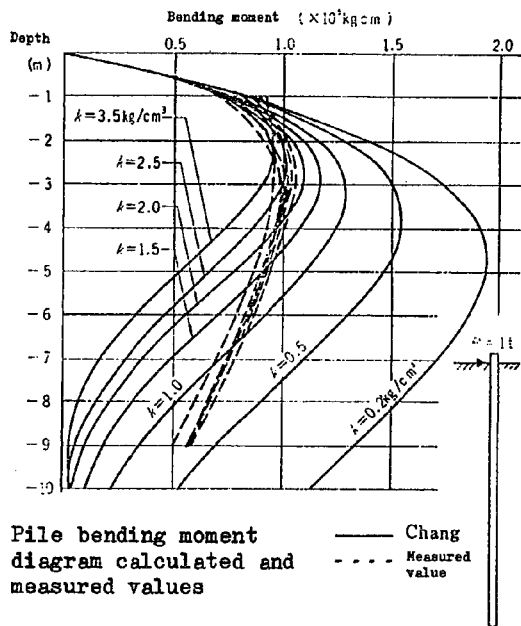


Fig. 11 Standard waveform of Nagadoro bridge

o Estimation of damping constants

As a result of measurements made of the pier P_9 of the Nagadoro bridge, it was found that the natural period $T_0 = 0.25 \text{ sec}$ and damping constant $h_0 = 0.4$ and these measured values were used for the pier P_4 . Since the pier P_4 had the first order natural period $T_1 = 1.12 \text{ sec}$ and the second order period $T_2 = 0.17$, its first order damping constant h_1 and second order damping constant h_2 were

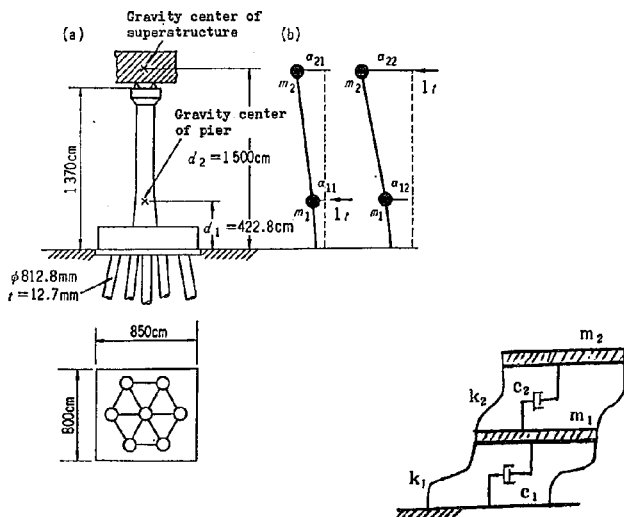
$$\left. \begin{aligned} h_1 &= h_0 \frac{T_0}{T_1} = 0.4 \times \frac{0.25}{1.12} = 0.089, \\ h_2 &= h_0 \frac{T_0}{T_2} = 0.4 \times \frac{0.25}{0.17} = 0.588 \end{aligned} \right\} \dots\dots\dots (30)$$

$\bar{C}_1$ and $\bar{C}_2$ in equation (11) were obtained by equation (12)

(See literature) as follows.

$$\left. \begin{aligned} \bar{C}_1 &= 2h_1 \bar{M}_1 \omega_1 = 2 \times 0.089 \times 4369 \times 5.60 \\ &= 4365 \text{ kg sec/cm} \\ \bar{C}_2 &= 2h_2 \bar{M}_2 \omega_2 = 2 \times 0.588 \times 775 \times 360 \\ &= 32831 \text{ kg sec/cm} \end{aligned} \right\} \dots\dots\dots (31)$$

o Shearing response model of 2-degree of freedom system



Weight of pier 645.420t ($m_1 = 658.6 \text{ Kg sec}^2/\text{cm}$)

One span section of superstructure 326.72t ($m_2 = 333.4 \text{ Kg sec}^2/\text{cm}$)

Spring constant matrix

$$\begin{bmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{bmatrix} = \begin{bmatrix} 728 \times 10^3 & -212 \times 10^3 \\ -212 \times 10^3 & 74 \times 10^3 \end{bmatrix} (\text{Kg/cm})$$

∴ Obtained from Mr. Tamano's equation.

(14) Type of foundation:

Pile foundation (Nagadoro-bashi or Tsugaru Ohashi)

Bridge:

Bridge length 558 m (12 @ 46.55)

Literature:

Bridges and Foundation, Feb., 1971

Location and organization concerned:

Aomori Prefecture

Public Works Department of Aomori Prefecture

Purpose of analysis:

Investigation of vibration response behavior of the entire bridge by artificial earthquake (Soft ground)

Analysis method:

Fourier spectrum analysis

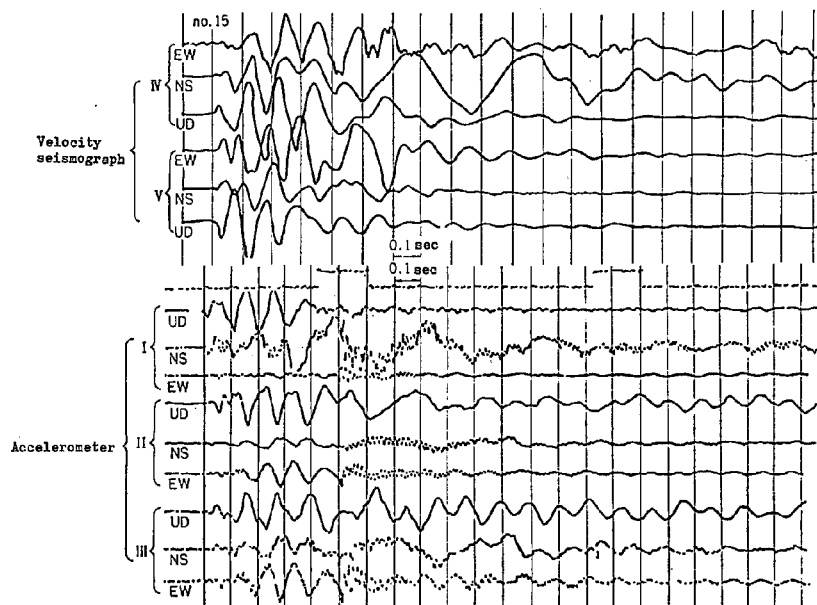
Analysis results, evaluation and problems:

- o The dominant periods of horizontal initial vibration and vertical initial vibration were 0.11 - 0.4 seconds and 0.11 - 0.17 seconds, respectively. The top of pier and girder support were found to behave in the same manner as the surface wave.
- o The dominant periods of the surface waves of the horizontal and vertical vibrations were around 0.6 second and the top of pier and girder support behaved in the same manner as the surface wave in the ground surface.
- o The behavior of the entire bridge during earthquake time can be known by obtaining the dominant period of the ground surface by

an artificial earthquake caused by the explosion of charges.

- o The above is the case in which the piers are low. The situation would be different according to the span length and bridge length.

Artificial earthquake response records



Dominant period

Recording location	Recording component	Dominant period (sec)		Recording location	Recording component	Dominant period (sec)	
		Initial vibrations	Surface waves			Initial vibrations	Surface waves
Ground surface(V)	UD	0.11~0.17	0.7	Center of Bridge girder span (III)	UD		0.3 ~ 0.37
	NS	0.11~0.21	0.6		NS	0.18 ~ 0.40	0.65
	EW	0.10~0.20	0.6		EW	0.10 ~ 0.18	
Pier (IV)	UD	0.12~0.24	0.55 ~ 0.6	Center of Bridge girder span (II)	UD	0.11	0.2 ~ 0.4
	NS	0.11~0.40			NS		
	EW	0.11~0.50			EW		
Girder support(I)	UD	0.11~0.17	0.4 ~ 0.6				
	NS	0.11					
	EW						

Response spectrum

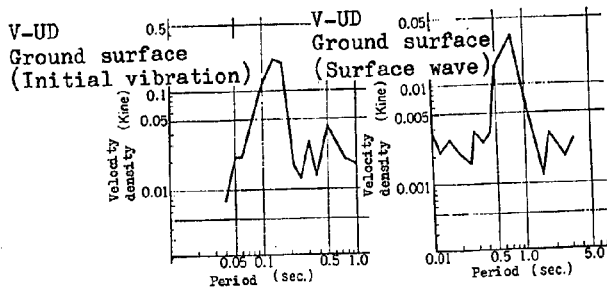


Fig.6

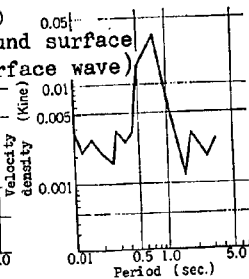


Fig.7

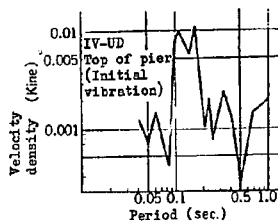


Fig.8

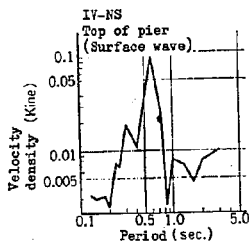


Fig.9

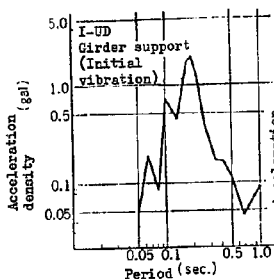


Fig.10

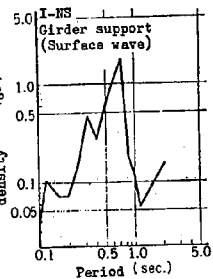
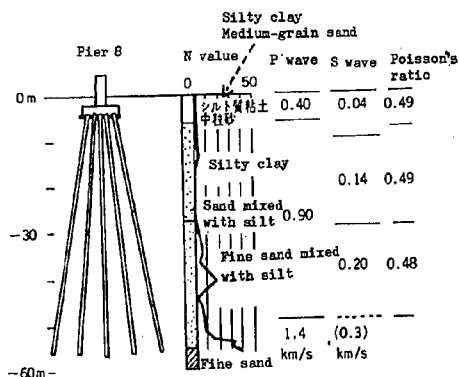


Fig.11

Type of ground and evaluation:



o N value was obtained by boring and Poisson's ratio from the seismic prospecting

$$N = 0 \quad (l \leq 30m)$$

Original form of foundation and analysis model:

o Motion model of 1-degree of freedom system.

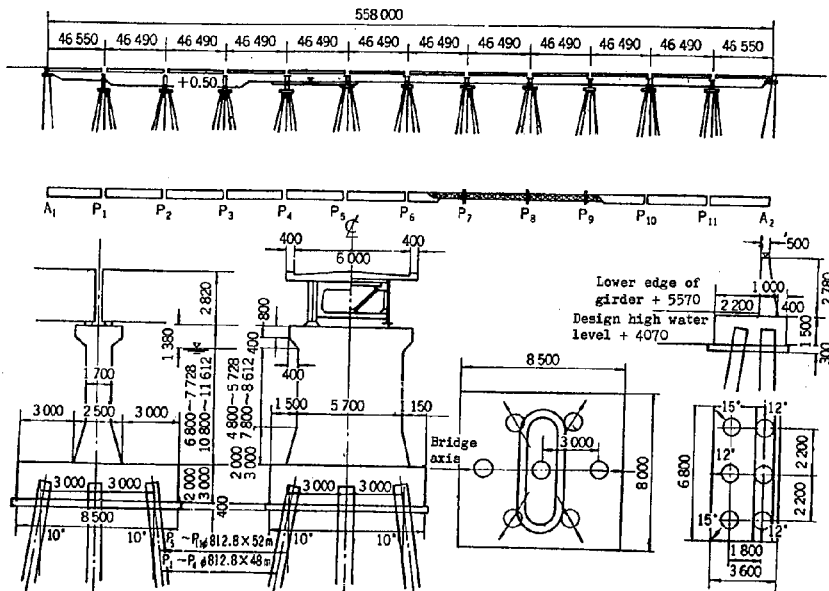
$$\ddot{Z}(t) = -\frac{a_0}{2} + \sum_{n=1}^N C_n \cos(\omega_n t + \varphi_n)$$

$$C_n = \sqrt{a_n^2 + b_n^2}, \quad \omega_n = \frac{2n\pi}{T}$$

$$\varphi_n = \tan^{-1}\left(-\frac{b_n}{a_n}\right)$$

$$a_n = \frac{2}{N} \sum_{i=1}^N \ddot{Z}_i \cos \frac{2\pi}{N} ni$$

$$b_n = \frac{2}{N} \sum_{i=1}^N \ddot{Z}_i \sin \frac{2\pi}{N} ni$$



(15) Type of foundation:

Pile foundation

Bridge:

Bridge length 709.3 m

Literature:

Investigations and Studies on Earthquake Resistant Design (No.2),
Feb., 1970

Expressway Investigation Committee, Bridges and Foundations, June 1972

Location and organization concerned:

Sakaiminato - Mihogaseki, Shimane Prefecture

Japan Highway Public Corporation

Purpose of analysis:

Investigation of seismic behavior of a deep soft stratum

Analysis method:

Analysis of ground and piles as a whole by dashpot shearing motion

Analysis results, evaluation and problems:

- o As for the calculation of incident wave by the theory of shearing wave, the characteristics of surface stratum were eliminated from El Centro NS component and since there little difference between the bases I and II, the incident wave of the base I was used.
- o Seismic wave of 50 gal was assumed for the base but it was amplified to 180 gal in the surface stratum.
- o The stress intensity of the base was safe in the axial direction of the bridge but it sometimes exceeded the allowable limit in the direction perpendicular to the axis of the bridge.

- o The maximum response displacement obtained by the seismic response calculation was larger than that obtained by the seismic coefficient method but it was negligible owing to the ground condition and the size of the pier.

Axial direction: 6.41 cm (2.63 cm)

Transverse direction: 8.75 cm

- o Since the bending moment was large in the transverse direction, it is necessary to make the structure so much flexible.
- o The bending moment of the pile was larger in its lower part but earthquake is sharply amplified in that area by the soft ground.

Problems

- o Waveform and magnitude of incident seismic waves
- o Validity of the theory of shearing wave propagation in the case of multi-stratum ground and great earthquakes.
- o Differences between the theory of wave motion and pressiometer coefficient
- o Damping mechanism and nonlinearity
- o Effect of group piles
- o Stratum inclination

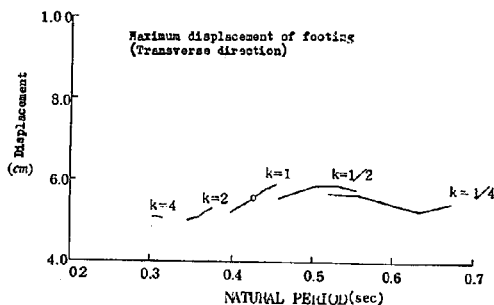
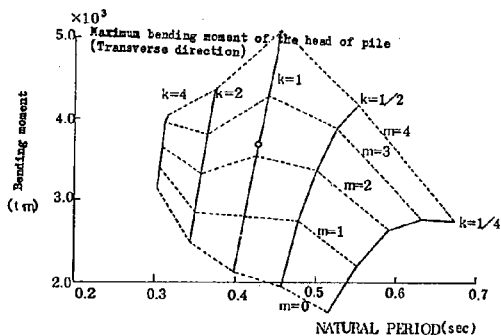
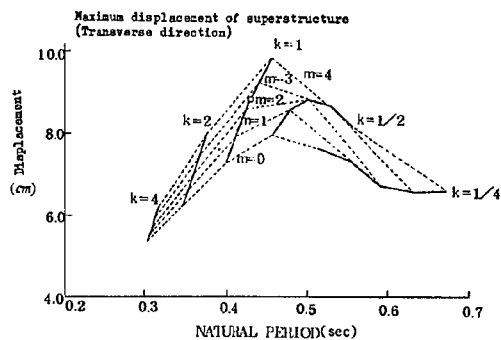
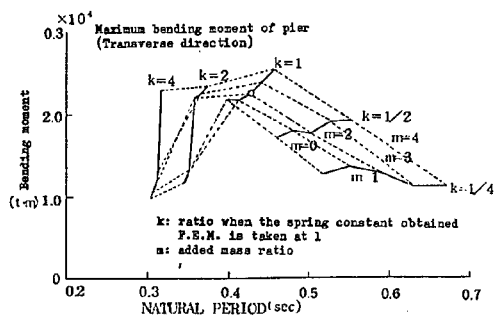
Values of maximum responses

		Axial	Transverse
Displacement (cm)	Superstructure	6.41 cm	8.75 cm
	Footing	5.03 cm	5.58 cm
Bending moment (t.m)	Column	3,860 t.m	22,600 t.m
	Head of pile	3,570 "	3,670 "
Shearing force (t)	Column	160 t	1,640 t
	Head of pile	890 t	1,020 t

Maximum response stress and allowable stress

Section		Axial	Transverse
Pile	Concrete	$\sigma_c = 85 \frac{\text{Kg}}{\text{cm}^2} < \sigma_{ca} = 120$	$\sigma_c = 94 \frac{\text{Kg}}{\text{cm}^2} < \sigma_{ca} = 120$
	Reinforcing bar	$\sigma_s = 933 " < \sigma_{sa} = 2700$	$\sigma_s = 1360 " < \sigma_{sa} = 2700$
Pier	Concrete	$\sigma_c = 453 \frac{\text{Kg}}{\text{cm}^2} < \sigma_{ca} = 120$	$\sigma_c = 142 \frac{\text{Kg}}{\text{cm}^2} > \sigma_{ca} = 120$
	Reinforcing bar	$\sigma_s = 770 " < \sigma_{sa} = 2700$	$\sigma_s = 4021 " > \sigma_{sa} = 2700$

o Responses when added mass and spring constant are changed



Responses when added mass and spring constant are changed

Type of ground and evaluation:

- o Spring constant and added mass are determined by the use of finite elements method
- . Plane strain
- . Triangular element (Displacement function is first order)
- . Footings and piles are rigid and there is neither slipping nor gap between them and the surrounding ground.
- . One-third of the total mass of the element is equally distributed to all the points.

Equation of motion

$$\begin{bmatrix} m & \\ & M \end{bmatrix} \begin{bmatrix} \dot{x} \\ \dot{X} \end{bmatrix} + \begin{bmatrix} k & S \\ S^T & K \end{bmatrix} \begin{bmatrix} x \\ X \end{bmatrix} = \begin{bmatrix} q \\ Q \end{bmatrix}$$

m, M : mass of footing, matrix of total mass of the ground

x, X : displacement of footing, matrix of total displacement of the ground

k, S, S^T, K : Matrix of rigidity

q, Q : external force

If the unit load is exerted on the structural system, from the above equation, we obtain the following equation, thereby to obtain the spring constant k_s .

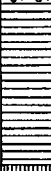
$$\begin{bmatrix} k & S \\ S^T & K \end{bmatrix} \begin{bmatrix} x \\ X \end{bmatrix} = \begin{bmatrix} 1 \\ 0 \end{bmatrix}$$

$$\therefore k_s = \frac{1}{X} = k - S^T K^{-1} S$$

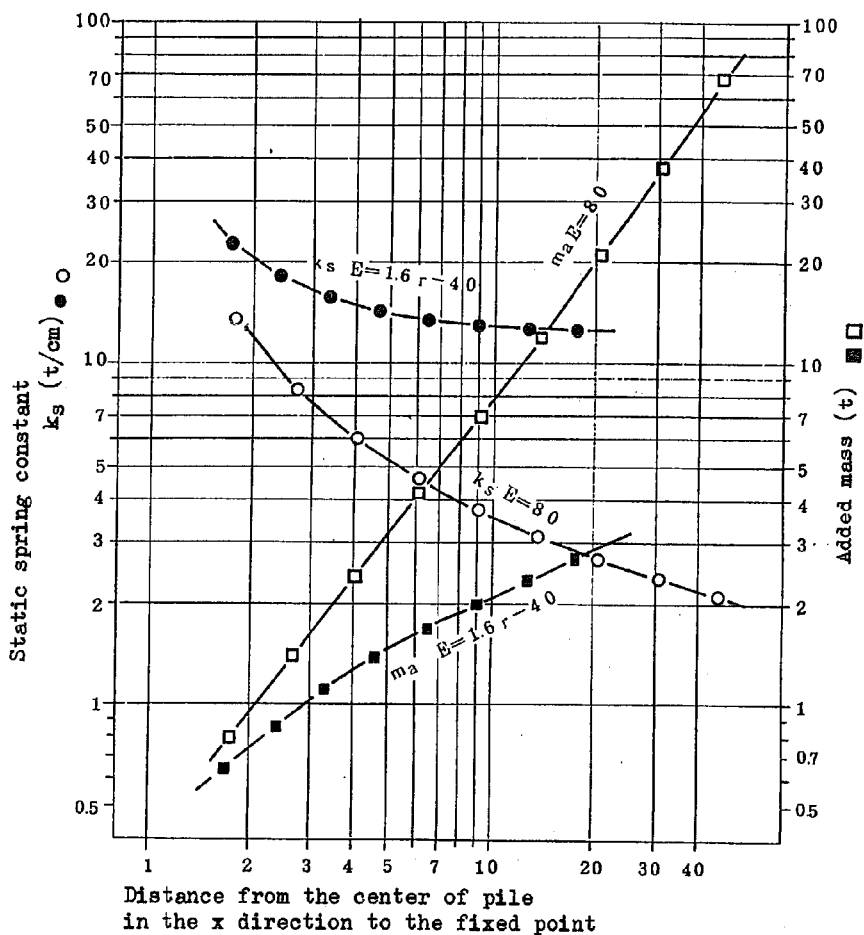
Assuming that the added mass is to be added so that the footings or piles may have the same kinetic energy, the added mass can be obtained from the following equation.

$$\frac{1}{2} m (\omega x)^2 + \frac{1}{2} (\omega X)^T M (\omega X) = \frac{1}{2} (m + m_a) (\omega x)^2$$

$$\therefore m_a = \frac{1}{x^2} X^T M X$$

Stratum	Depth	Ground	Density	Propagation velocity: V Elastic modulus of the ground: E
1	m 4.00	 SAND	t/m^3 1.70	V = 130 m/sec E = 80 kg/cm ²
2	8.00		1.90	
3	11.00		1.75	
4	13.50			
5	20.00	 SILTY	1.65	V = 110 E = 60
6	40.00	 CLAY		
7	49.00	 SAND & GRAVEL	2.00	V = 300 E = 700
8	60.70	 CLAY	1.65	
9		 ROCK	2.00	V = 500 E = 900

Geologic columnar section



Convergence of spring constant and added mass

The figure at the above shows the static spring constant from the center of pile and added mass.

If a wider area of the ground is considered, there is a tendency for the spring constant to decrease and for added mass to increase for the following conceivable reasons.

Three-dimensional expansion of the ground

Treatment of the ground as an elastic body

Damping property of the ground

Here the modulus of elasticity is determined by the following correction to be used in the response calculation.

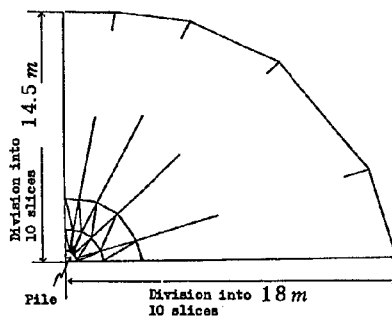
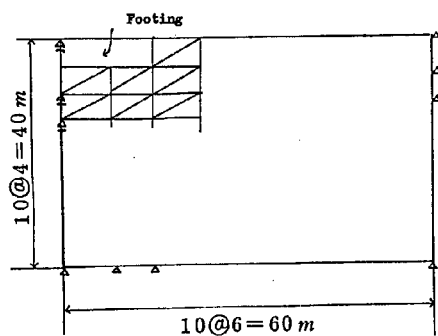
$$E = E_0 (1 + 2r)$$

E_0 : Standard modulus of elasticity

o Comparison of K value (during earthquake)

Design use		2.0 Kg/cm ³
Response calculation	Footing	2.8 "
	Pile	3.3 "
Horizontal test estimation		2.3~2.7 "

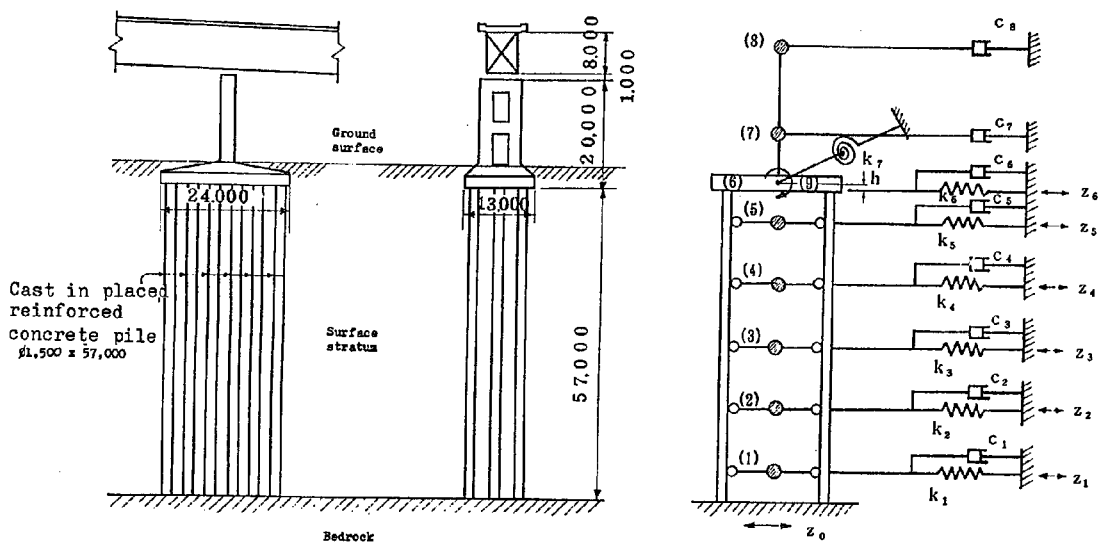
Finite elements model



Original form of foundation and analysis model:

- o Seismic waves are plane waves of shearing type and strike from right below.
- o The surface stratum and base are horizontal and extend boundlessly.
- o It is assumed that there occurs no slipping between strata and all strata are a perfect elastic body.
- o The horizontal force is equally distributed to the 18 piles in a group and they all have the same value of horizontal displacement.
- o The weight of superstructure is concentrated on the piers on both sides, each carrying a half span.
- o The rotation of footing is also considered.
- o The added mass gives the structural body the same amount of energy as the kinetic energy possessed by the ground.
- o As for the shearing wave propagation velocity, density and elastic modulus of the soil, the results of elastic wave test and pressiometer are used. The Poisson's ratio is 0.3 for all the strata.
- o The damping constants of all the standard vibrations are 0.1.
- o The response calculation is made up to the seventh order.
- o The maximum acceleration of incident waves is to be 50 gal.
- o The incident wave shall be the El Centro NS component minus the characteristics of the surface stratum at the point of measurement.

Dynamic model



o The damping constant was 0.35 in the vibration test but 0.10 is used for the sake of safety in the response calculation in the designing.

Spring constant and added mass

		Horizontal spring constant (t/cm)	Added mass ratio	Rotational spring constant (t.cm/sec)	Added rotational inertia ratio	Remarks
Footings	Axial direction	60.80	1.57	10.31×10^7	2.69	1/2 Plane $t = 1 \text{ m}$ (Thickness)
	Transverse direction	48.92	2.22	3.46×10^7	4.31	
Pile		12.29	2.40			1/4 Plane $E_p = 80$ $\gamma = 1.9$ $t = 1 \text{ m}$

Rigidity

	Axial direction	Transverse direction
I_B	$4.46 \times 10^8 \text{ cm}^4$	$4.46 \times 10^8 \text{ cm}^4$
I_N	$1.49 \times 10^{11} \text{ cm}^4$	$4.30 \times 10^{10} \text{ cm}^4$
I_C	$1.64 \times 10^9 \text{ cm}^4$	$1.37 \times 10^{10} \text{ cm}^4$
EI_B	$9.37 \times 10^{13} \text{ Kg cm}^2$	$9.37 \times 10^{13} \text{ Kg cm}^2$
EI_N	$3.13 \times 10^{16} \text{ Kg cm}^2$	$9.03 \times 10^{15} \text{ Kg cm}^2$
EI_C	$3.44 \times 10^{14} \text{ Kg cm}^2$	$2.89 \times 10^{15} \text{ Kg cm}^2$

Weight

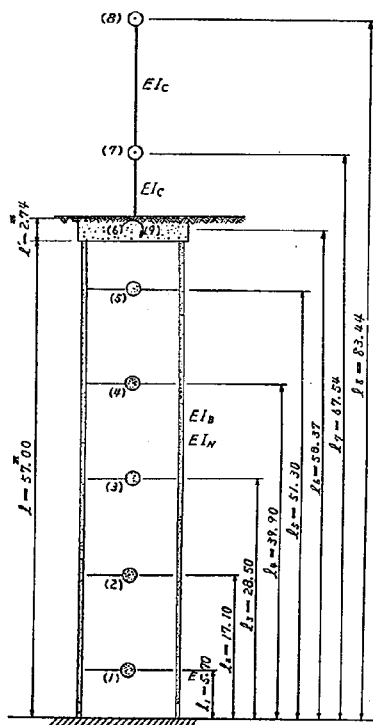
z	Axial direction	Transverse direction
1	2,794 t	2,794 t
2	3,202	3,202
3	2,794	2,794
4	2,794	2,794
5	2,911	2,911
6	5,497	6,888
7	643.5	643.5
8	897.1	897.1
9	I_R	I_R

Rotational inertia

	Axial direction	Transverse direction
I_R	$3,838 \times 10^5 \text{ t-m}^2$	$1,669 \times 10^5 \text{ t-m}^2$

Spring constant

	Axial direction	Transverse direction
K_1	88,270 t/cm	88,270 t/cm
K_2	88,270	88,270
K_3	7,568	7,568
K_4	7,568	7,568
K_5	10,090	10,090
K_6	1,581	2,348
K_R	$2,681 \times 10^9 \text{ t/}$	$1,661 \times 10^9 \text{ t/}$



o Incident wave (Theory of reflection and refraction)

$$u_k(t, z) = f_k(t - \frac{z}{V_k}) + g_k(t - \frac{H_k - z}{V_k})$$

$$f_1(t) = \frac{1}{2} F(t), g_1(t) = \frac{1}{2} F(t)$$

$$f_k(t) = \frac{1 + \alpha_{k-1}}{2} f_{k-1}(t + \frac{H_{k+1}}{V_{k+1}}) + \frac{1 - \alpha_{k-1}}{2} g_{k-1}(t - \frac{H_{k-1}}{V_{k-1}})$$

$$g_k(t) = \frac{1 - \alpha_{k-1}}{2} f_{k-1}(t + \frac{H_{k+1}}{V_{k+1}}) + \frac{1 + \alpha_{k-1}}{2} g_{k-1}(t - \frac{H_{k-1}}{V_{k-1}})$$

$$f_0(t) = \frac{1 + \alpha_n}{2} f_n(t + \frac{H_n}{V_n}) + \frac{1 - \alpha_n}{2} g_n(t - \frac{H_n}{V_n})$$

Uk : response of k stratum

F(t): surface wave

f_k : ascending wave of k stratum

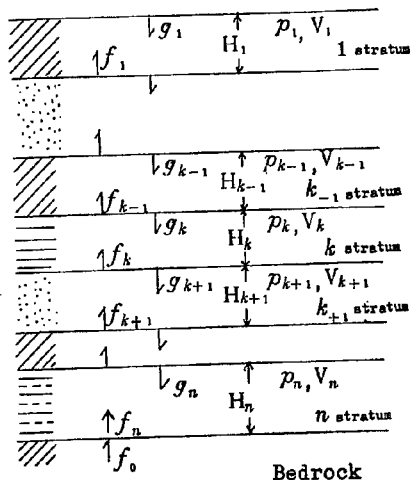
g : descending wave of k stratum

$$\alpha_k = (\rho_k/V_k) / (\rho_{k+1}/V_{k+1})$$

H_k : stratum thickness

ρ_k : stratum density

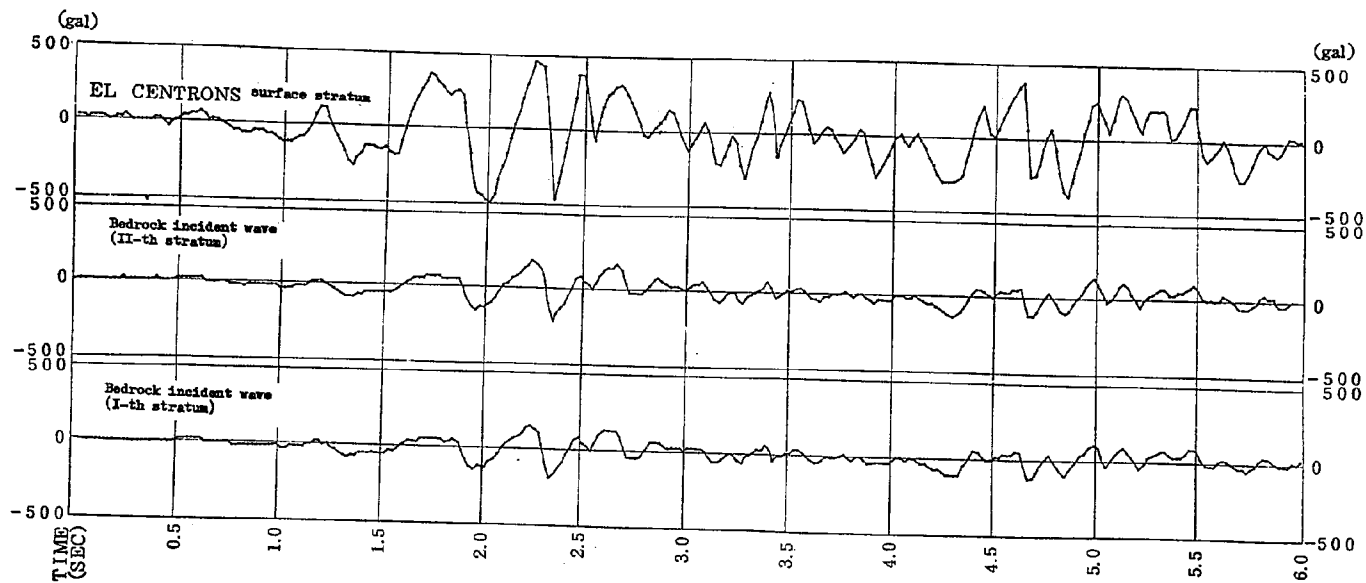
V_k : wave motion velocity in the stratum



The input wave $f_0(t)$ of bedrock when El Centro surface wave was used for surface wave $F(t)$ and the velocities in the various strata when calculated with the second and third strata as bedrock were as shown in the figure at right. The bedrock input waves were as shown in the figure below.

Surface stratum
of El Centro

Stratum	H(m)	$\rho(\text{t/m}^3)$	$V(\text{m/sec})$	Remarks
1	30	2.06	159	
2	305	2.20	856	Bedrock I
3		2.30	1100	Bedrock II



Bedrock incident waves for use in response calculations

(16) Type of foundation:

Sheet pile foundation (Ishikarikako Bridge)

Bridge:

3-span continuous cable stayed bridge,
semi-elliptical steel sheet pile

Bridge length 1,412.7 m (64 + 160 + 64)

Literature:

Foundations, Mar. 1975

Location and organization concerned:

Hokkaido Development Bureau

Purpose of analysis:

Investigation of vibration characteristics of the bridge whose
foundation is high and flexible.

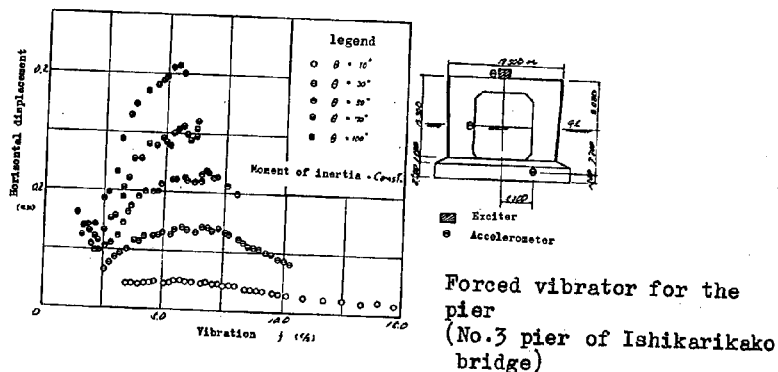
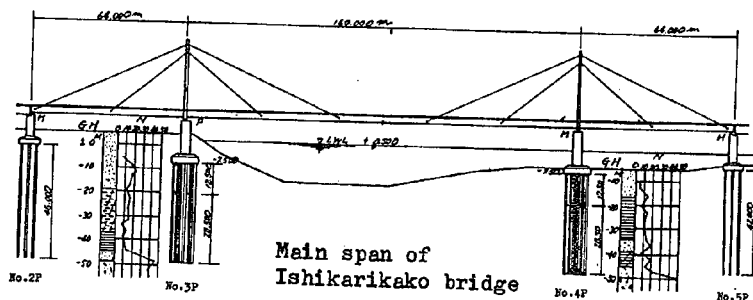
Analysis method:

Multiple material points motion model

Analysis results, evaluation and problems:

- o The dominant period ($T = 0.17$ sec) obtained by the vibration test
after completion of the piers agreed with the calculated value
($T = 0.15$ sec second order vibration).
- o There were problems about the way the material points were taken
after completion of the superstructure, virtual mass, damping
constant and spring constant evaluation.
- o The subgrade reaction coefficient varies so much according to the
displacement of the pile head that it is necessary to redesign
according to the comparison of the hypothetical conditions and the
results.

o The vibration characteristics were not clearly known and therefore the behavior of earth and sand inside the sheet pile well was not known clearly.



Type of ground and evaluation:

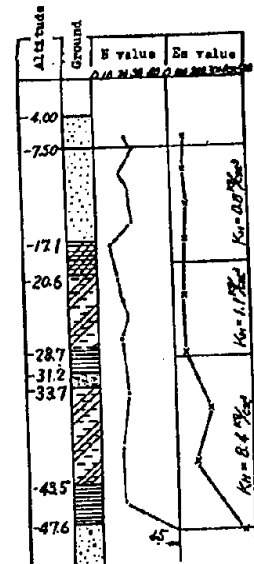
- o The horizontal subgrade reaction coefficient was taken at 80% (0.8 kg/cm³) of the following two values whichever smaller.
- . From the modulus of elasticity by pressiometer test and Louis Ménard equation

$$K_H = 1.05 \text{ kg/cm}^3$$

. $K_H = 1.30 \text{ kg/cm}^3$ which corresponds to the surface deflection in the horizontal loading test.

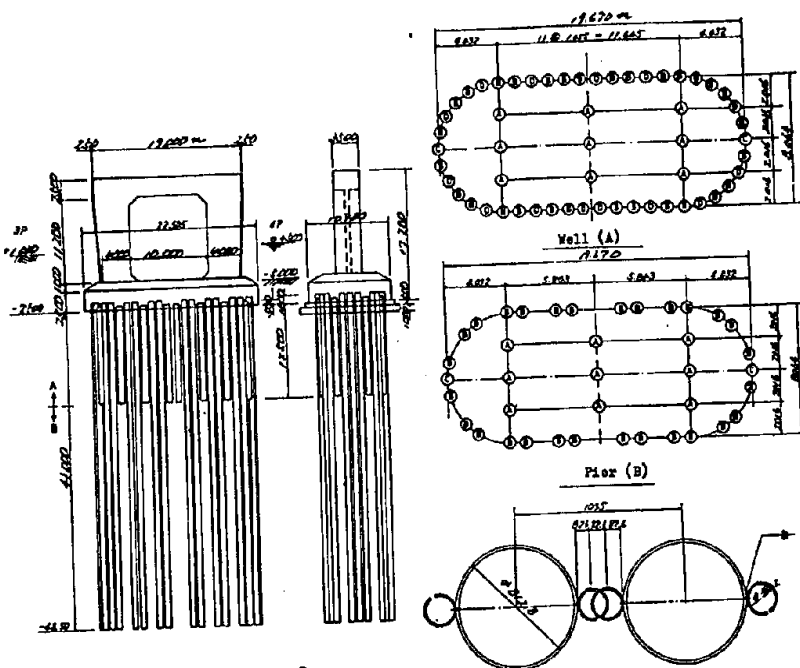
o The vertical reaction coefficient was obtained from the following equation, assuming the pier to be a bearing pile whose effective length was 80% of the length of the pier leg.

$$K_V = \frac{EA}{0.8 L}$$

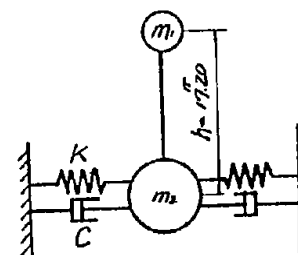


Original form of foundation and analysis model:

o Since the damping constant cannot be determined by numerical analysis, it was assumed to be $\beta = 0.1$ or 0.2 from other experimental values (data obtained when measuring the dominant period of the ground).



Foundation of Ishikarikako bridge (No.3 pier)



Pier vibration waveform

(17) Type of foundation:

Piled rigid-frame foundation

Bridge:

J.N.R. Standard type (35.4 x 10.2 x 21.6)

Literature:

Structural Design Materials No.25, Mar. 1971

Location and organization concerned:

Japanese National Railways

Purpose of analysis:

A high elevated bridge required for topographical reasons

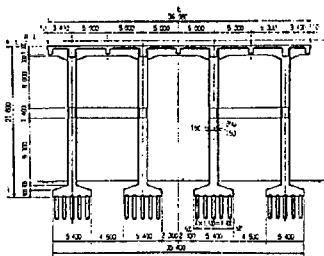
Analysis method:

Modal analysis by multiple material points system

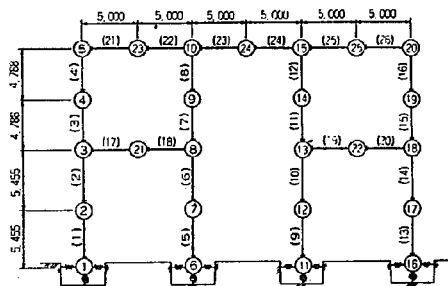
Analysis results, evaluation and problems:

o Damping constants were selected as follows.

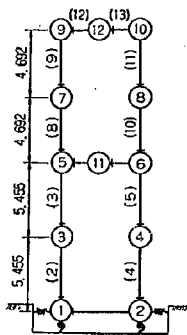
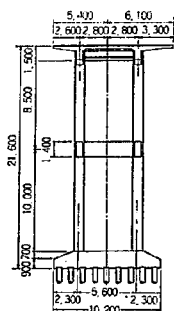
Symbol		A	B	C
Ground condition		Very good	Slightly poor	Very poor
Longitudinal	δ	0.151 ~ 0.188	0.402 ~ 0.427	0.465
	h (%)	2.4 ~ 3.0	6.4 ~ 6.8	7.4
Transverse	δ	0.082 ~ 0.113	0.351	0.446 ~ 0.559
	h (%)	1.3 ~ 1.8	5.6	7.1 ~ 8.9



Original form



Dynamic model



Type of ground and evaluation:

o Ground spring constants

$$k_h = A_o \cdot K_h + 2EI n \beta^2$$

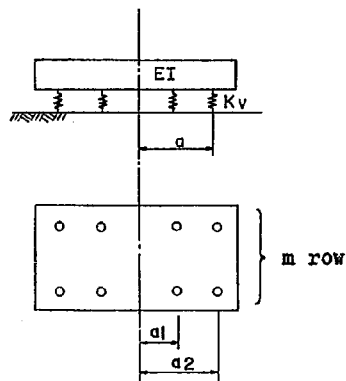
$$k_r = I \cdot K_o + 2m EA (a_1^2 + a_2^2) / \ell$$

$$k_v = A_o \cdot K_v + n EA / \ell$$

Chang's theory was used to give

the group pile effect to the piles.

$$\beta' = \frac{DK_h}{4EI}$$



Original form of foundation and analysis model:

- o The rigid frame perpendicular to the track was modeled with 12 material points and the rigid frame in the direction of the track was modeled with 25 material points.
- o Dead load alone was equally divided into two and concentrated.
- o The degrees of freedom were horizontal, vertical and rotational displacements.
- o The foundation was assumed to be pile foundation.
- o When the rigidity of section decreased, the linear strength of stability was considered (perfect plastic type).
- o There was no phase difference between the ground and the foundation.
- o The input seismic waves were of the following three kinds
 - . El centro, NS, $\alpha_{\max} = 330$ gal, 1940
 - . AKASHI, EW, $\alpha_{\max} = 39$ gal, 1963.3.27 (Echizenoki Earthquake M = 6.9), D = 140 Km
 - . KUSHIRO, NS, $\alpha_{\max} = 220$ gal, (M = 7.0), 1962.4.23, D = 90 Kmand they were calculated in terms of 200 gal.
- o Viscous damping coefficient: C
$$C_{cr} = 2mp, \quad h = C/C_{cr} \quad C = 2mph \quad C_{cr} : \text{Critical damping constant}$$

From the Honshu-Shikoku Bridge Authority's materials, rigid-frame

$$h = 0.05 \text{ and foundation } h = 0.10.$$