EARTHQUAKE RESISTANT DESIGN OF BRIDGE SUBSTRUCTURES

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PREFACE

This paper contains the earthquake-resistant portions of drafted Standards for design of concrete and reinforced concrete structures proposed for adoption by the Japanese National Railways. The writer was a member of the Committee selected to compile these Standards. This information is presented on behalf of the chairman of the committee, Dr. T. Yoshida.

The Standards for Designing Concrete and Reinforced Concrete Structures were compiled at the request of the Japanese National Railway by the Japanese Society of Civil Engineers. The Committee consisted of 14 members and finalized these Standards after 2 years of work.

In studying the effect of earthquakes on structures, it was accepted generally at first that the standard values for the acceleration to be used should be 0.2 g horizontally and 0.1 g vertically. The committee has now modified these values as stated herein after careful consideration with Professors R. Takahasi, H. Kawasumi, and K. Kanai of the Earthquake Research Institute, Tokyo University.

Also to be presented briefly will be an outline of the fundamental studies on the effect of earthquakes on bridge substructures as conducted by Prof. Konishi of Kyoto University. Experimental research in this field is also being undertaken now by members of the Earthquake Research Institute, Tokyo University, under the guidance of Prof. Takahasi, and the Institute of Industrial Science, Tokyo University, under the guidance of Prof. S. Okamoto.

STANDARD SPECIFICATIONS OF THE JAPANESE NATIONAL RAILWAYS CONCERNING THE EFFECTS OF EARTHQUAKES ON REINFORCED CONCRETE STRUCTURES

These specifications provide that when the dead load, the earth pressure and the water pressure are considered, the effect of earthquake force shall be taken in consideration also.

The values shown in Table 1 are to be used generally for the value of the horizontal seismic coefficient. The vertical seismic coefficient value to be used is half the value of the horizontal seismic coefficient.

EXPLANATORY NOTES

In the design of a structure, it is assumed that the force determined by use of the horizontal seismic coefficient value given in Table 1 acts statically on the structure, the earth, and etc. This acceleration value

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is assumed as that which will produce a statically applied force necessary to cause that deformation of the structure and foundation equivalent to actual earthquake force. The acceleration value given in Table 1 can be modified by use of Tables 2 and 3, if the designer so desires and only with the approval of the railway authorities, to account for such items as the locality of the structure, the type of site soils, the kind of structure, a balance between strength of structure and stability of structure, and the respective vibrational characteristics for the structure and the foundation.

It can be noted by examination of Tables 2 and 3 for the same region that:

1. The harder the soil structure and the more solid the foundation becomes, the greater the horizontal force acting on the structure becomes from the horizontal acceleration.

2. The more massive the structure, the larger the horizontal force acting on the structure becomes from the horizontal acceleration.

For a slender structure constructed on either diluvial or alluvial foundations, a comparatively large value of acceleration must be used.

Tables 2 and 3 have been prepared based upon the results of study of the Kanto, Imaichi and Fukui earthquakes in Japan and also upon the observations of strong earthquakes reported upon in America.

Before tables 2 and 3 are fully applicable for actual design, there are several items requiring further study. Much of the data contained therein has been concluded on theoretical bases. The lack of data necessary from actual observation of strong earthquakes and of rock formations and knowledge of the vibrational characteristics of structures required, instead, theoretical conclusions.

The seismic coefficient has an almost constant value of 0.35 to 0.25 in areas included in region A. The larger value is adopted in cases where the conditions match those itemized in Tables 2 and 3 and the strength and stability calculations are considered.

In the regions denoted as B or C, the seismic coefficient becomes 3/4 and 1/2, respectively, of the A region value. The seismic coefficients shown in the Tables can be increased or decreased in accordance with the importance of the railway track installations and in accordance with evidence supporting the feasibility of such increase or decrease.

The regional classifications of Table 1 have been compiled on the basis of determinations made by Dr. Kawasumi for the expected values of the maximum seismic coefficient during a 75 year period (1).

In the southern part of the Kanto area in an A region, there is a location where the expected maximum value of the seismic coefficient is very large, as large as 0.6. Consequently, within this general area it is desirable to adopt as large a value of the seismic coefficient as is practical. Although only the southern portion of Gifu Prefecture is qualified by degree of seismicity to be classified in the A region, the entire prefecture has been so classified.

EARTH PRESSURE AT THE TIME OF AN EARTHQUAKE

Inasmuch as there has been no generally reliable theory developed or adequate actual observed data with respect to the earth pressure at the time of an earthquake, there is no way to pin-point the resultant earthquake effect other than to use the results of experimental tests and what actual observations have been made.

In Mononobe's theory, it is assumed that a wall foundation and the soil wedge are rigid bodies and also that the same amount of the earthquake acceleration acts simultaneously on every part. Although such an assumption seems to be valid at some time, the earth pressure at the time of an earthquake varies according to the complexity of the vibration and the deformation condition of each part. All things considered, the actual value would seem to less than the value calculated using the Mononobe theory.

Before Mononobe formulated his theory, many large retaining walls were designed and constructed taking no earthquake effect into consideration. These walls have withstood successfully many strong earthquakes.

If the lateral earth pressure exerted during an earthquake actually is as large as indicated by theory, these retaining walls and other similar structures should have been overturned in the top forward direction or otherwise badly damaged. No such examples of damage have been seen. Instead, there have been many examples in which the bottom of the wall has been pushed out and the top inclined backward indicating foundation or soil yielding.

Structures standing on weak foundation soil structures usually move in a rocking motion during an earthquake because of many reasons, among which would be the high compressibility of the soil, narrow width of foundation, and etc. However, as the amount of the deformation of stable back fill soil is not usually large, the backfill does not follow the wall inclinations. As a result, a large earth pressure does not act upon the wall except during such times as the wall rocks back and collides with the soil; the effect of which, in some cases, can be considerable.

In the case of a loose sand or very soft clay backfill, any forward inclination of the wall will usually result in immediate forward sliding of the backfill due to both the effect of vibrations and lack of support, with a consequent large pressure effect on the wall.

For a flexible or weak structure such as a sheet pile structure, the vibrational deformation is large and as a result, the resultant earth pressure during an earthquake is considered to be much smaller than for a more rigid structure that rocks. As an example, a model experiment on a quay wall constructed of sheet piling revealed that the earth pressure increases to some degree because of fall-in of the backfill, consolidation, and etc., but that the periodic earth pressure fluctuation corresponding to vibrational deformation is negligible.

If the foundation soil structure rigidity approaches that of rock, or if the foundation has a large width and the structure is a large rigid body, it is considered that a very large instantaneous earth pressure is effected. Generally though, the structure is supposed to rock. To adjudge the expected earth pressure resultant from earthquake action requires consideration of all pertinent factors such as, rigidity of foundation soil structure, width of foundation, rigidity of structure, type and density of back fill and the tendency of this back fill to fall-in or slide.

The following methods of calculating the earth pressure resulting from an earthquake have been used:

1. A method by Dr. T. Sano, identical to that employed in calculating the normal gravitational earth pressure, assumes that the angle of repose of the soil is decreased by use of equation 1.

$$\theta = \tan^{-1} K \qquad \ldots\ldots\ (1)$$

2. A method identical to that employed in calculating the normal gravitational earth pressure except with the following assumptions:

a. The foundation and structure are regarded as one body and are tilted in the critical direction, in respect to the direction of gravity force, an angular amount derived from use of equation 1.

b. Use of equation 2, instead of equation 3 for estimating the weight of the structure (2 and 3).

$$W' = mg(1-K_v)\sec\theta \qquad \ldots\ldots\ (2)$$

$$W = mg \qquad \ldots\ldots\ (3)$$

WATER PRESSURE AT THE TIME OF AN EARTHQUAKE

For formulae and methods for calculating the pressure of water on a bridge pier, a quay wall, and etc., other papers in this Proceedings are to be consulted. In regard to the water pressure on the upstream side of a dam, equations 4, 5 and 6 are given.

According to Westergaard (4):

$$\Delta p = 7/8\, KHx \qquad \ldots\ldots\ (4)$$

According to R. Takahasi:

if $T \leqq 2/3\sqrt{H}$, then $\Delta p = 3/4\, K_h T^2$ ...... (5)

if $T \geqq 2/3\sqrt{H}$, then $\Delta p = 1/2\, K_h \sqrt{H}\, T$ ...... (6)

FUNDAMENTAL STUDIES ON THE EFFECTS
OF EARTHQUAKES ON BRIDGE SUBSTRUCTURES

Past earthquakes have caused severe damage to bridges. Surveys have indicated that the primary damage was suffered by the bridge substructure. Herein are described some of the fundamental studies pertinent to earthquake resistant design of bridge substructures.

1. Vibration Tests of Bridge Piers. The natural period, T', and the logarithmic decrement, δ, of piers have been measured. The observed values for the period, $T' = 0.1$ to 0.3 sec, are about three to four times the values computed assuming the piers as cantilevers fixed rigidly at the ground surface. This period increase shows that the degree of fixation at the base cannot be assumed as rigid and is effected by the deformation of the foundation soil structure.

2. Theoretical Analysis of Substructure Vibrations Taking into Account the Deformations of the Foundation Soil Structure.

In Fig. 2, it is assumed that during vibrations, the soil depth AB under the base of the pier is subjected to the horizontal reaction given in equation 7.

$$p = K(x_1) \cdot y_1 \qquad \ldots\ldots\ (7)$$

On the other hand, it has been observed experimentally that the distribution of $K(x_1)$ can be assumed to be triangular, as shown in Fig. 2. Then taking into consideration the deformation of the foundation soil structure, the free period of vibration can be obtained either by use of the frequency equation or the energy method.

Assume that the seismic motion is expressed by equation 8

$$e(t) = e \cdot \cos(\omega t + \phi) \qquad \ldots\ldots\ (8)$$

and is applied to the substructures as shown in Fig. 2. In this case, the theoretical solution of the forced vibration can be solved quite easily. The question that arises is how to assign a value to, N, the repetitions of the seismic motion. Taking $N = \frac{1}{2}$, 1 and 3, the transient solution of the forced vibration are computed by an Analog Computer.

Using the above system of computations, the following typical results were obtained: When the ratio of the primary period, T, of the earthquake motion to the natural period of the pier, T_n, is larger than 2 (which case is assumed to occur in most actual designs), the dynamic effect can be considered statically by increasing the statical value by 20 percent. In this case the factor of the damage level expected is taken as 10 percent.

3. A Proposed Method of Earthquake Resistant Design.

From results of observations, experimental data and experience, the writer proposes the following general method for designing bridge substructures for earthquake resistance:

a. Determine the horizontal seismic intensity, α_o, from the records of past earthquakes for the area.

b. Estimate the values of K_A in Fig. 2 as follows:

(1) For a clay and sand base soil structure:

$$K_A \cong d \qquad \text{....... (9)}$$

(2) For sand and gravel base soil structure

$$K_A \cong 2d \qquad \text{....... (10)}$$

c. Take into account the friction, $\tau = \mu P_o$, acting horizontally on the side surfaces of the pier foundation, where μ is the coefficient of friction varying from 0.3 to 0.5 and P_o is the active earth pressure.

d. Take into account the reaction, $q_1 = K_A z$, acting vertically on the bottom of the well foundation, where z is the vertical displacement of the bottom of the well foundation.

e. Determine the required depth, d, of the pier below the ground surface for the forces as shown in Fig. 3 in such a way that:

(1) The horizontal earth pressure of 1.2 p does not exceed the passive earth pressure of the ground. This includes taking into consideration the equivalent 20 percent increase for the dynamical effect.

(2) The magnitude of 1.2 q is not larger than the allowable bearing capacity value at the base of the well foundation.

f. Determine the dimension of each part of the substructure using the depth, d, determined in step e above.

g. In steps a through f the horizontal seismic intensity, α_o, is assumed as constant as illustrated in Fig. 3. It is more rational however, to consider that in actual earthquake action α_o is not constant along the height of the pier but is proportional to y_1 and y_2 as shown in Fig. 2. In this instance, the steps b through f described above can be applied also.

BIBLIOGRAPHY

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(2) "Seismic Theory of Civil Engineering Construction", by N. Mononobe, Riko-tosho Press (Japanese), 1952.

(3) "Civil Engineering Handbook", by Japanese Society of Civil Engineers, (Japanese), 1954, p 168.

(4) "Water Pressure on Dams during Earthquakes", by H.M. Westergaard, Trans. A.S.C.E., 1933, p 418 - 472.

NOMENCLATURE

			Unit
d	=	Depth of well foundation	m
e	=	Amplitude of seismic motion	m
$e(t)$	=	Periodic function of seismic motion	m
g	=	Acceleration of gravity	m/sec^2
H	=	Depth of water	m
$K(x_i)$	=	Foundation coefficient	kg/cm^3
K_A	=	Foundation coefficient at bottom of well foundation	kg/cm^3
K	=	Overall seismic coefficient	None
K_h	=	Horizontal seismic coefficient	None
K_v	=	Vertical seismic coefficient	None
m	=	Mass of structure	
N	=	Repetition number of seismic motion	None
p	=	Resistant earth pressure	t/m^2
p_o	=	Active earth pressure	t/m^2
Δp	=	Increment of statical water pressure at the time of an earthquake	t/m^2
q	=	Vertical reaction on bottom of well foundation	t/m^2
T	=	Period of seismic motion	sec
T'	=	Natural period of substructure (observed value)	sec
T_n	=	Natural period of substructure (computed value)	sec
x	=	Depth from water surface to base	m
y	=	Horizontal displacement (see Fig. 2)	m
z	=	Vertical displacement of the bottom of well foundation	m
W, W'	=	Weight of structure	kg

			Unit
α_0	=	Horizontal seismic intensity (see Fig. 3)	None
δ	=	Logarithmic decrement of structure	None
ϕ	=	Phase angle of seismic motion	Degree
θ	=	Decrement of angle of repose	Degree
τ	=	Horizontal friction on side surface of well foundation	t/m^2
μ	=	Coefficient of friction between earth and well foundation	None
ω	=	Circular frequency of seismic motion	sec^{-1}

FIGURE CAPTIONS

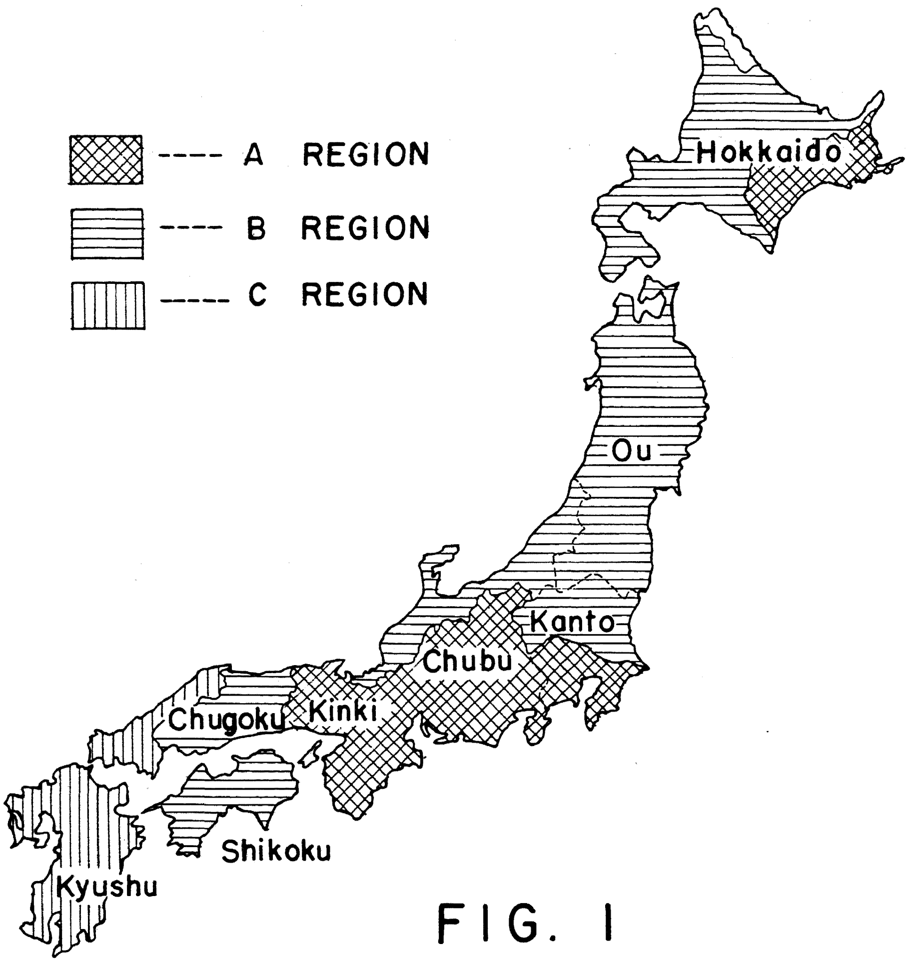
---- A REGION
---- B REGION
---- C REGION
Hokkaido
Ou
Kanto
Chubu
Kinki
Chugoku
Shikoku
Kyushu

FIG. 1

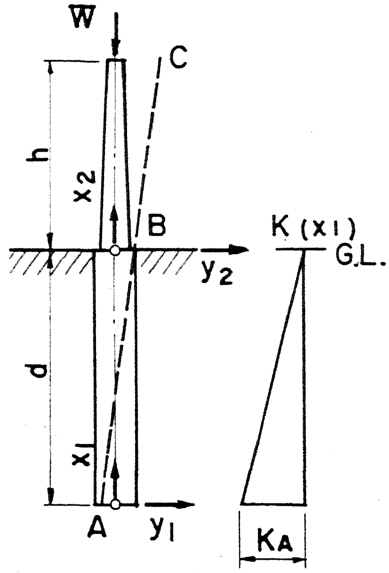
W
C
h
x2
B
K (x1)
G.L.
y2
d
x1
A
y1
KA

FIG. 2

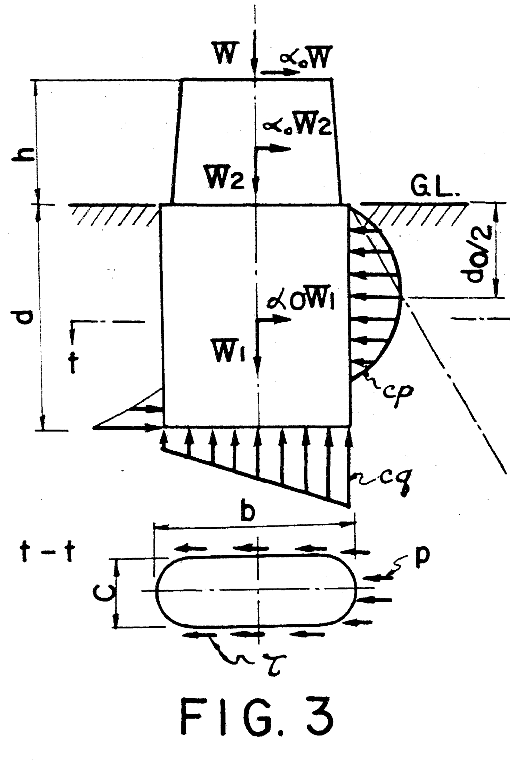
W
α₀W
α₀W2
h
W2
G.L.
d
α₀W1
W1
do/2
t
t
cp
cq
b
t - t
p
c
τ

FIG. 3

Horizontal seismic design coefficients (see Fig. 1).

REGION	LOCALITY	PREFECTURE	HORIZONTAL SEISMIC COEFFICIENT
A	Hokkaido Kanto Chubu Kinki	Nemuro, Kushiro, Tokachi Chiba, Saitama, Tokyo, Kanagawa Yamanashi, Nagano, Shizuoka, Aichi, Gifu Shiga, Kyoto, Hyogo, Mie, Nara, Osaka, Wakayama	0.3
B	Hokkaido Ou Kanto Chubu Chugoku Shikoku	Rumoi, Abashiri, Kamikawa, Sorachi, Ishikari, Shiribeshi, Hidaka, Iburi, Toshima, Hiyama Aomori, Iwate, Akita, Miyagi, Yamagata, Fukushima Ibaraki, Tochigi, Gumma Niigata, Toyama, Ishikawa, Fukui Tottori, Okayama, Hiroshima Kagawa, Tokushima, Ehime, Kochi	0.2
C	Hokkaido Chugoku Kyushu	Soya Shimane, Yamaguchi Oita, Fukuoka, Saga, Nagasaki, Miyazaki, Kumamoto, Kagoshima	0.15

TABLE I

Horizontal seismic coefficient modification table.

Type of Calculation	Strength Calculation of a structure which vibrates freely						Stability calculation of a structure which vibrates freely and strength and stability calculations when the effect of earth pressure is taken into consideration		
	Massive Structure			Slender Structure					
Region / Kind of Found.	A	B	C	A	B	C	A	B	C
Class 1	0.35	0.25	0.20	0.20	0.15	0.10	0.20	0.15	0.10
Class II	0.25	0.15	0.10	0.30	0.20	0.15	0.25	0.20	0.15
Class III	0.15	0.10	0.10	0.30	0.20	0.15	0.30	0.20	0.15
Class IV	0.15	0.10	0.10	0.30	0.20	0.15	0.35	0.25	0.20

TABLE 2

NOTE: The Standard provides that for combined loads, the normal allowable unit stresses for concrete and steel may be increased 1.5 times for combinations of seismic and dead loads and increased 2.0 times for combinations of seismic, dead and live loads.

Classification of foundation soil structures.

Class	Kind of Bed[1]
Class I	The surface is an Alluvium formation 4) 2 meters or less in thickness and directly underneath exists a hard stratum of the third or earlier era extending over a fairly wide area.
Class II	The surface is of the Diluvium formation[2] [3] whose thickness ranges from 3 to 15 meters or of the Alluvium formation whose thickness ranges from 2 to 10 meters.
Class III	The surface is of the Diluvium formation[2] [3] 15 m or more in thickness or of the Alluvium formation [4], whose thickness ranges from 10 to 25 meters.
Class IV	Remarkably soft and weak bed, [5] or the surface is of the Alluvium formation, 25 meters or more in thickness.

TABLE 3

Notes:

1) It is desirable to classify exactly the bed basing it upon consideration of topograph and soil structure and also on field tests. However, a brief standard may be obtained from the above Tables.

2) A Diluvium formation which co-exists with an Alluvium formation is considered equivalent to the Alluvium formation whose thickness is 0.7 times as much as that of the Diluvium formation.

3) The Diluvium formation is a formation which consists of a strata such as gravel, sandy hard clay or loam. This formation often exists in a plateau as the surface stratum. The velocity of the elastic wave (lateral wave) which propagates through the Diluvium formation is approximately 100-150 m/sec.

4) The Alluvium formation is a formation which consists of a strata such as gravel, sandy gravel, and clay, and is still in the course of formation.

If is often found in a canyon or lowlands as the surface stratum.

The velocity of the elastic wave (lateral wave) which propagates through the Alluvium formation is approximately 100 m/sec or less.

5) Remarkably soft and weak bed, means a bed such as marshy ground, saturated sea bed or reclaimed ground composed of these soils.