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Seismic analysis of a continuous underground deep wall

S.G. Nomachi
Nihon University, Chiba, Japan

K.G. Matsuoka, N. Kishi & K. Kohari
Muroran Institute of Technology, Japan

M. Takahashi
Muroran Br. of Hokkaido Development Bureau, Japan

ABSTRACT: To construct the main tower foundation of suspension bridge (the Swan Bridge), the method using an artificial island and a continuous underground deep wall was applied. Since the firm bed rock lies 73m below the sea level and the continuous underground deep wall become a tall structure with 106m height, stability of that structure for earthquake is discussed.

Numerical analyses are executed by means of finite ring element method applying artificial earthquake with normalized maximum amplitude 60gal. It is clear that bigger axial stress occurs at the location near the lower edge of cylindrical wall of foundation's main body.

1 INTRODUCTION

The Swan Bridge (called Hakucho Ohashi in Japanese) is under construction which is a stiffened suspension bridge of 3 spans with 2 hinges at Muroran Harbor located in the northernmost island of Japan. The main span of the bridge is 720m long and the total length is 1,380m. Since the firm bedrock for the main tower foundation lies 73m below the sea level, the main tower foundation is constructed by the method of continuous underground deep wall in an artificial island as shown in Fig. 1.

The construction sequences are: 1) to build an artificial island of 69m diameter enclosed by steel sheet piles. 2) in the artificial island, to construct a continuous underground deep wall of 37m diameter, 1.5m thick and 106m deep, for earth retention and prevention of underground water entry. 3) to excavate in the inner side of the wall and build downward the cylindrical wall of the foundation's main body. 4) to fix the partition walls of the foundation.

Now the stability situation of the continuous underground deep wall (hereafter, we will name it continuous wall for abbreviation) may need to elucidate during construction of the foundation's main body.

In this paper, the seismic stability of the continuous wall during the construction of the foundation's main body is discussed. The analysis of the dynamic response of the above structure is performed by means of the finite ring element method, in which the artificial island, the continuous wall, a part of the cylindrical wall of foundation and submerged ground are concerned.

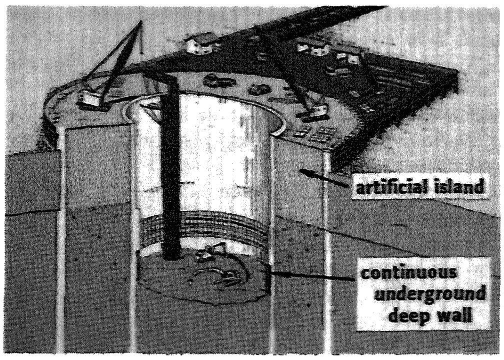

Figure 1. Cross-sectional view of the continuous wall underconstruction.

2 FINITE RING ELEMENT METHOD

In finite ring element having an arbitrary rectangular cross section as shown in Fig. 2, we denote displacements in general coordinates r, z and θ as u, v and w respectively. Likewise the components of forces and displacements at the i-th node are denoted by (f_{ri}, f_{zi}, $f_{\theta i}$) and (d_{ri}, d_{zi}, $d_{\theta i}$). Taking local coordinates (ξ-η) having the ori-

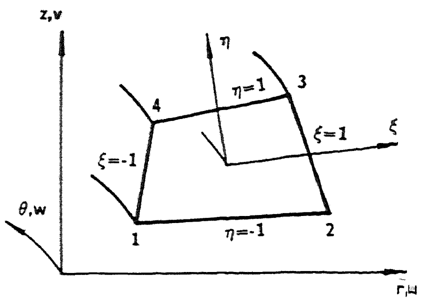

Figure 2. Finite ring element having an arbitrary rectangular cross section.

gin point at gravity center of element, coordinates and displacements in the arbitrary point in element are written as follows:

$$\begin{Bmatrix} r \\ z \end{Bmatrix} = \begin{bmatrix} \{N\}^T & \\ & \{N\}^T \end{bmatrix} \begin{Bmatrix} \{r_i\} \\ \{z_i\} \end{Bmatrix} \qquad ...(1)$$

$$\begin{Bmatrix} u \\ v \\ w \end{Bmatrix} = \begin{bmatrix} \{N\}^T & & \\ & \{N\}^T & \\ & & \{N\}^T \end{bmatrix} \begin{Bmatrix} \{dr_i\} \\ \{dz_i\} \\ \{d\theta_i\} \end{Bmatrix} \qquad ...(2)$$

in which N is shape function defined as Eq.(3)

$$\{N\}^T = \frac{1}{4}[(1-\xi)(1-\eta)\ (1+\xi)(1-\eta)\ (1+\xi)(1+\eta)\ (1-\xi)(1+\eta)] \qquad ...(3)$$

The relationships between strain and displacements are

$$\begin{Bmatrix} \varepsilon_r \\ \varepsilon_z \\ \varepsilon_\theta \end{Bmatrix} = \begin{Bmatrix} \dfrac{\partial u}{\partial r} \\ \dfrac{\partial v}{\partial z} \\ \dfrac{u}{r} + \dfrac{1}{r}\dfrac{\partial w}{\partial \theta} \end{Bmatrix} \quad \begin{Bmatrix} \gamma_{rz} \\ \gamma_{r\theta} \\ \gamma_{z\theta} \end{Bmatrix} = \begin{Bmatrix} \dfrac{\partial u}{\partial z} + \dfrac{\partial r}{\partial r} \\ \dfrac{1}{r}\dfrac{\partial u}{\partial \theta} + \dfrac{\partial w}{\partial r} - \dfrac{w}{r} \\ \dfrac{\partial w}{\partial z} + \dfrac{1}{r}\dfrac{\partial r}{\partial \theta} \end{Bmatrix} \qquad ...(4)$$

Performing finite Fourier cosine transformation on ε_r, ε_z, ε_θ and γ_{rz} and sine transformation on $\gamma_{r\theta}$ and $\gamma_{z\theta}$ respectively and introducing [B] and [D] matrices, image functions of strain and stress vectors are formulated as

$$\{\overline{C}_m[\varepsilon]\} = [B]\{\overline{C}_m[d]\}, \quad \{\overline{C}_m[\sigma]\} = [D][B]\{\overline{C}_m[d]\} \qquad ...(5)$$

in which $\{\underline{d}\}$ is nodal displacement vector of element and $C_m[\ ]$ means finite Fourier transformation.

The equation of motion of an element can be derived from the principle of virtual work. Then matrix equation of the whole structure for seismic analysis is obtained as follows:

$$[M]\{\overline{C}_m[\ddot{d}]\} + [K]\{\overline{C}_m[d]\} = -[M]\{\overline{C}_m[\ddot{d}_s]\} - \qquad ...(6)$$

where [M] and [K] are mass and stiffness matrices of the whole structure respectively and $\{\overline{C}_m[\ddot{d}_s]\}$ is image function of acceleration vector in rigid base.

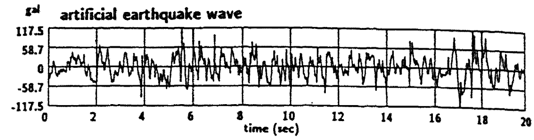

Figure 3. Artificial earthquake wave for bed rock.

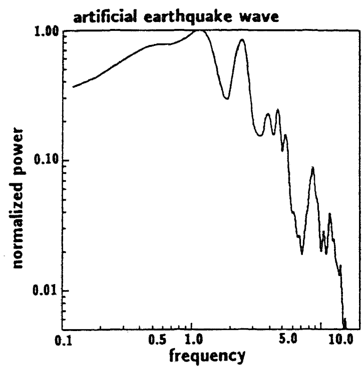

Figure 4. Normalized power spectrum for artificial earthquake wave.

Equation of damping vibration on the r-th normal function ϕ_r is obtained by means of orthogonality of eigen vibration as follows:

$$\overline{C}_m[\ddot{\phi}_r] + 2h_r\omega_r\overline{C}_m[\dot{\phi}_r] + \omega_r^2\overline{C}_m[\phi_r] = \overline{C}_m[P(t)_r] \qquad ...(7)$$

in which suffix r means the r-th eigen vibration and

h_r : damping coefficient

ω_r : eigen angular velocity

$\{X_r\}$: eigen vector of structure

$$\overline{C}_m[p(t)_r] = -\frac{\{x_r\}^T[M]\{\overline{C}_m[\ddot{d}_s]\}}{\{x_r\}^T[M]\{x_r\}}$$

Equation (7) can be solved using Newmark β method (β=1/6). Superimposing responses for all eigen values, characteristics of seismic response of structure in time domain can be obtained.

3 INPUT EARTHQUAKE WAVE

Dynamic response analysis was executed applying artificial earthquake wave made for bed-rock of the Swan Bridge. In this analysis, since the continuous wall is a temporary structure for construction of the bridge foundation, the wave was normalized with maximum amplitude 60gal. Figures 3 and 4 are the original artificial earthquake wave and the concerning normalized power spectrum respectively. It is seen that the dominant frequencies of this wave are 1.2Hz and 2.7Hz. Numerical analysis was executed up to 20sec with an interval of 0.01sec.

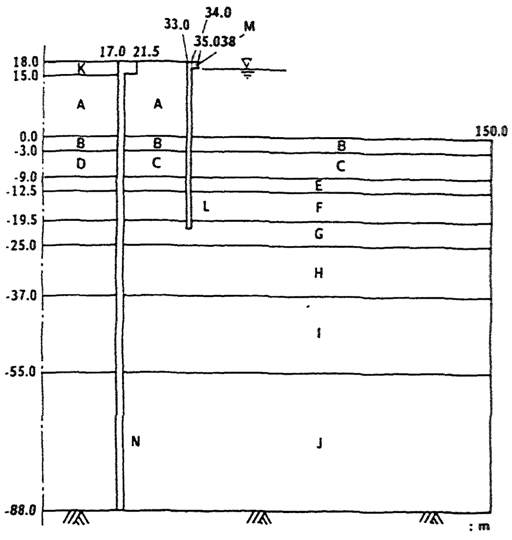

Figure 5. Cross-sectional view of the structure.

Table 1. Material properties.

I.D.	material	E (kgf/cm^2)	ν	γ (tf/m^3)
A	slurry of coal ash	1000.0	0.05	1.66
B	sand	500.0	0.45	1.70
C	silt	100.0	0.45	1.70
D	improved soil	645.0	0.45	1.70
E	sand	820.0	0.45	1.70
F	sand	580.0	0.45	1.80
G	silt	715.0	0.45	1.65
H	sand	2630.0	0.45	1.75
I	tuff silt~sand	7840.0	0.45	1.80
J	tuff sandstone	15600.0	0.30	1.70
K	gravel	1000.0	0.05	1.66
L	sheet pile	20000.0	0.30	0.30
M	ring beam	2.1×10^6	0.30	7.85
N	continuous wall	3.0×10^5	0.20	2.45

4 NUMERICAL ANALYSIS

General cross-sectional view and strata in the area of main tower foundation in this analysis are shown in Fig.5. The radius of the free boundary was assumed as 150m and its width is almost three times of the radius of artificial island. The material properties of each stratum are described in Table 1 in which those values are obtained based on PS measurement.

We considered four construction stages as follows:

1) case 1 : the stage after finishing construction of artificial island surrounded by steel sheet piles and continuous wall up to bed-rock (-88m below under the sea bed)

2) case 2 : the stage in which the inner portion of the continuous wall was excavated up to -11m below under the sea bed

3) case 3 : the stage in which the inner portion of the continuous wall was excavated up to -29m below under the sea bed and inner cylinder was constructed from -5m up to -28m below under the sea bed which is a part of main tower foundation and of 2m thickness.

4) case 4 : the stage in which the inner portion of the continuous wall was excavated up to -58.5m and inner cylinder was constructed up to -51.5m below under the sea bed.

The boundary conditions at the circumference of underground layer considered in this analysis are assumed as free in r direction and fixed in z direction. The elements touching to sea water are added to virtual mass estimated based on Westergaard's empirical equation. Numerical calculations were conducted considering from fundamental vibration up to 30th eigen values and modes with damping factor h=0.10.

5 NUMERICAL RESULTS

Four construction stages of this structure are separately analyzed. Natural periods of 3 fundamental vibrations for each case are listed in Table 2. The value of first mode is almost the same in all cases which is about 1.1sec. Distributions of acceleration and horizontal displacement of the continuous wall along the direction of wall depth are shown in Fig. 6. The maximum acceleration occurs at the top of continuous wall in each case and the response value in case 2 is the biggest one which is 132 gal and more than twice of the maximum acceleration of input wave. Distribution of acceleration gradually decreases with the depth of continuous wall from the top and the value in deeper area of 30m from the sea bed is less than 10gal.

The maximum displacement occurs at the top of continuous wall similar to the response of acceleration. The biggest one among them is 28mm and occurs in the case 1 and the smallest one is 22.6mm, occurs in the case 4. Generally displacement from top up to 30m below under the sea bed is larger with increasing height. It seems that the strata lying below the 25m depth from the sea bed are of stiffer rocks.

Stress distribution of σ_z generated in the inner and outer elements of continuous wall are shown in Fig.7 at the occurrence of maximum stress. In cases 1 and 2, the maximum stress occurs at 25m below under the sea bed. Since the bottom edge of steel sheet piles of artificial island are located at the same depth, it seems that dynamic responses may be affected by abrupt change of stiffness. In cases of 3 and 4, bigger bending moments of shell and beam actions are occurred in the area

Table 2. Natural period for the fundamental vibrations.

order / case	1st mode (sec)	2nd mode (sec)	3rd mode (sec)
case-1	1.106	1.038	0.866
case-2	1.112	1.038	0.864
case-3	1.110	1.035	0.865
case-4	1.097	1.016	0.838

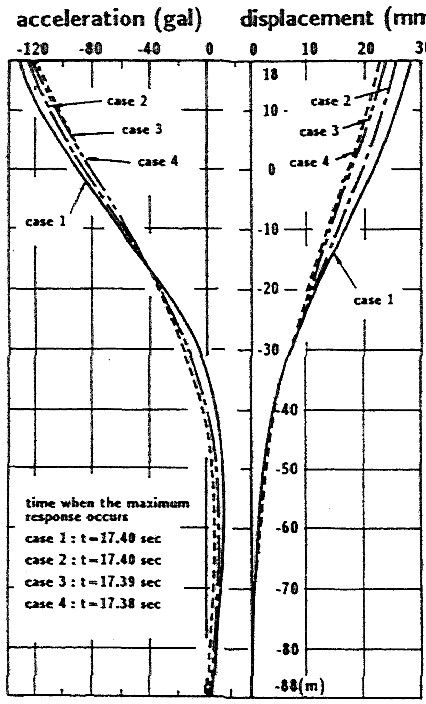

Figure 6. Distributions of acceleration and displacement when the maximum response occurs.

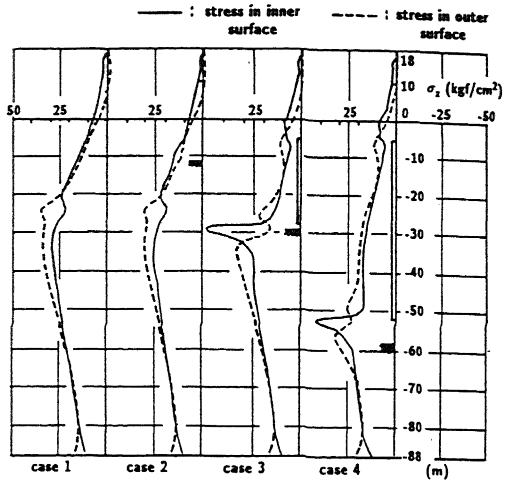

Figure 7. Axial stress distributions in inner and outer surface of continuous wall when the maximum response occurs.

Table 3. The maximum stress components.

	EL. (m)	σ_{z1} kgf/cm^2	σ_{z2} kgf/cm^2	σ_{bb} kgf/cm^2	σ_{bs} kgf/cm^2
case-1	-24.0	21.7	35.0	28.4	6.7
case-2	-23.0	25.5	30.2	27.9	2.4
case-3	-28.5	48.9	16.5	32.7	16.2
case-4	-52.3	42.6	22.4	32.5	10.1

EL. : taking the sea bed as datum
σ_{z1} : axial stress in inner surface
σ_{z2} : axial stress in outer surface
σ_{bb} : bending stress of beam action
σ_{bs} : bending stress of shell action

between the points of excavation and inner cylinder.

The maximum values of σ_z shown in Fig. 7 are tabulated in Table 3. In this table, it is seen that axial stress component which is obtained as beam action is about 30kgf/cm^2 in every case. On the other hand, the maximum bending stress due to shell action is 16.2kgf/cm^2 which is occurred near the edge of inner cylinder only in case 3.

6 CONCLUSIONS

The seismic stability of the continuous underground deep wall, 106m deep, used to construct the main tower foundation of long suspension bridge, is discussed. The numerical analysis is performed by means of finite ring element method considering the interactions among continuous underground deep wall, artificial island, a part of cylindrical wall of foundation and submerged ground.

The results obtained by these calculations are as follows:

1) the fundamental natural periods of all construction stages are almost 1.1sec.
2) The maximum horizontal deformation is less than 30mm atop of structure.
3) The maximum axial bending stress of continuous underground deep wall occurs at lower edge of the cylindrical wall of the foundation's main body, which is 30kgf/cm^2 due to beam action and 16.2kgf/cm^2 due to shell action.

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