DESIGN EXAMPLE OF A HIGHWAY BRIDGE BASED ON THE MANUAL FOR MENSHIN DESIGN OF HIGHWAY BRIDGES

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ABSTRACT

Presented is a design example of a base isolated bridge with a span length of 40 m and a clear height of 10 m. Design was made in accordance with the Manual of Menshin Design of Highway Bridges. Effect of Menshin design is discussed.

INTRODUCTION

A design was made for the same design condition with the U.S. group for comparing the seismic isolation design of bridges. Two types of Menshin devices were considered in this example calculation. First is the lead rubber bearings[1] (LRB) and the other is the high damping rubber bearing (HDR). Design was made in accordance with the Manual for Menshin Design of Highway Bridges[2] which was compiled in March 1992 as the final accomplishment of the Five Year Research Program between the Public Works Research Institute and 28 private firms on the "Development of Menshin Systems of Bridges". The design is not the detailed design but the rough estimation of approximate size and reinforcement.

DESIGN CONDITION

A bridge with multi-span continuous prestressed concrete girder with a weight of 200 kN/m (20.4 tf/m) as shown in Fig. 1 is considered. Only a single span segment is designed. The span length and the width of the superstructure are assumed as 40 m and 14 m, respectively. The girder depth is 2 m. The clear height is assumed as 10m. Soil condition is assumed as a stiff clay with N-value of the Standard Penetration Test of 30 or more. Thickness of this layer is about 60 m. Those are the design conditions specified for the common condition between U.S. and Japan. Table 1 summarizes those conditions.

Because the soil is of stiff clay with N-value of 30 or more, it may be diluvial clay layer. The soil condition is classified by the characteristics value as defined by

$$T_G = \Sigma \frac{4 H_i}{V_{si}} \tag{1}$$

where,

T_G : characteristics value (sec)

H_i : thickness of i-th sub layer (m)

V_{Si} : shear wave velocity of i-th sub layer (m/sec)

Because the shear wave velocity of the diluvial clayey layer may be over 400 m/sec. the soil condition may be classified as the 1st group in accordance with the "Part V Seismic Design" of the "Design Specifications of Highway Bridges"[2].

The foundation type is therefore assumed as a spread foundation (direct foundation). The bearing capacity, which is the most critical in design of foundation, is assumed as 60 tf/m^2. In actual design, the detailed soil survey is required. Table 2 shows the design condition of the foundation based on the Design Specifications of Highway Bridges. Assuming that the covering depth is 1.5 m, the pier height from the surface of the footing to the pier crest is 11.5 m.

It is assumed that the bridge is located in the seismically active area. And also it is assumed that the bridge is categorized as the "important" bridge.

Then based on the Manual the design lateral force coefficient in the Seismic Coefficient Method is as[2]

$$k_h = c_Z \cdot c_G \cdot c_I \cdot c_T \cdot c_E \cdot k_{hO} \geqq 0.1 \qquad (2)$$

where

$$c_T \cdot c_E \geqq 0.8 \qquad (3)$$

where,

k_h : seismic coefficient in the Seismic Coefficient Method

c_Z : modification factor for zone (= 1.0, seismically active area)

c_G : modification coefficient for ground condition (= 0.8, 1st group)

c_I : modification factor for importance (= 1.0, important bridge)

c_T : modification of structural response, and shall be computed by Table 3

c_E : modification factor for damping ratio of bridge, and shall be determined by Table 4

k_{hO} : standard design lateral force coefficient for Seismic Coefficient Method(= 0.2)

In the Bearing Capacity Method, the lateral force coefficient shall be as[2]

$$k_{he} = \frac{k_{hc}}{\sqrt{2\mu - 1}} \geqq 0.3 \qquad (4)$$

$$k_{hc} = c_Z \cdot c_I \cdot c_R \cdot c_E \cdot k_{hcO} \qquad (5)$$

where,

k_{he} : equivalent lateral force coefficient for Bearing Capacity Method

k_{hc} : lateral force coefficient in the Bearing Capacity Method

c_Z : modification factor for zone (= 1.0, seismically active area)

c_I : modification factor for importance (= 1.0 , important bridge)

c_R : modification factor for structural response, and shall be computed by Table 5

c_E : modification factor for damping ratio of bridge, and shall be obtained by Table 6

k_{hc0} : standard lateral force coefficient for Bearing Capacity Method (= 1.0)

μ : allowable ductility for reinforced concrete piers

The modification coefficients c_E, c_T and c_R have to be determined based on the natural period and damping ratio of the bridge.

Because in seismic design the bridge is critical in longitudinal (bridge axis) direction, major attention was paid in seismic design in longitudinal direction.

DESIGN OF MENSHIN DEVICES

Design Requirements

The design displacement of the Menshin device is an important parameter for designing the Menshin devices, and is defined as

$$u_B = \frac{k_h \cdot W_u}{K_B} \quad \text{(Seismic Coefficient Method)} \qquad (6)$$

$$u_B = \frac{k_{hc} \cdot W_u}{K_B} \quad \text{(Bearing Capacity Method)} \qquad (7)$$

where,

u_B : design displacement of Menshin device (cm)

K_B : equivalent stiffness (kgf/cm^2)

k_h : seismic coefficient in the Seismic Coefficient Method, and shall be computed by Eq. (2)

k_{hc} : lateral force coefficient in the Bearing Capacity Method, and shall be computed by Eq. (5)

W_U : weight of superstructure supported by the Menshin device (kgf)

It should be noted in Eq. (7) that the lateral force coefficient by Eq. (5) is used for design of the Menshin device. Therefore the reduction of lateral force considering the ductility of the pier is not considered for evaluating the lateral force for design of the Menshin device.

Following items are required to be checked for the Menshin devices :

(a) Check against Vertical Pressure

$$\sigma_{max} \leqq \sigma_{max,a} \qquad (8)$$

$$\sigma_{max} - \sigma_{min} \leqq \triangle\sigma_a \quad (9)$$
$$\sigma_{max} = R_{max} / A_{RO} \quad (10)$$
$$\sigma_{min} = R_{min} / A_R \quad (11)$$

where,

- σ_{max} : maximum vertical stress (kgf/cm^2)
- $\sigma_{max,a}$: allowable maximum vertical stress (= 80.0 kgf/cm^2)
- σ_{min} : minimum vertical stress (kgf/cm^2)
- $\triangle\sigma_a$: allowable amplitude of compression stress (= 50 kgf/cm^2)
- R_{max} : maximum vertical force (dead weight + maximum active load)
- R_{min} : minimum vertical force (dead weight + minimum active load)
- A_R : sectional area of Menshin device (cm^2)
- A_{RO} : effective sectional area of deformed Menshin device (cm^2)

(b) Check against Lateral Displacement

$$u_O \leqq \gamma_{Oa} \cdot \Sigma t_e \quad (12)$$
$$u_E \leqq \gamma_{Ea} \cdot \Sigma t_e \quad (13)$$

where,

- u_O : design displacement of Menshin device against normal load such as creep, shrinkage and temperature change (cm)
- γ_{Oa} : allowable shear strain of rubber against normal load (=0.7)
- u_E : design displacement of Menshin device against normal load and seismic effects (cm)
- γ_{Ea} : allowable shear strain of rubber against normal load and seismic effects, and shall be 1.5 in the Seismic Coefficient Method and 2.5 in the Bearing Capacity Method
- Σt_e : total thickness of rubber (cm)

(c) Check against Buckling

$$\sigma_c \leqq \sigma_{cRa} \quad (14)$$

where,

- σ_c : averaged vertical stress (kgf/cm^2)
- σ_{cRa} : allowable vertical stress for safety against buckling (kgf/cm^2), and shall be determined as

$$\sigma_{cRa} = G \;\; S \cdot \alpha / f_S \quad (15)$$

where,

- G : shear modulus of rubber (kgf/cm^2)
- S : coefficient depending on shape, and shall be obtained in the square bearings as

$$S = \frac{A_R}{2(a+b)\; t_e} \quad (16)$$

- A_R : sectional area of rubber (cm^2)
- a, b : effective length and width of rubber (cm)
- t_e : thickness of one rubber layer (cm)

$\alpha = a / \Sigma t_e$ or $b / \Sigma t_e$

Σt_e : total thickness of rubber (cm)
f_s : safety factor (= 2.5)

(d) Check against Local Shear Strain

$$(\gamma_c + \gamma_R + \gamma_s) \leqq \gamma_u / f_s \tag{17}$$

where,

γ_c : local shear strain due to vertical load
γ_R : local shear strain due to rotation of bearing
γ_s : local shear strain due to lateral displacement
γ_u : rupture strain of rubber and shall be determined by Table 7
f_s : safety factor for local shear strain, and is 2.5 for normal load and 1.8 and 1.2 for seismic design by means of the Seismic Coefficient Method and Bearing Capacity Method, respectively

(e) Check against Stress of Internal Steel Plate

$$\sigma_s \leqq \sigma_{sa} \tag{18}$$

$$\sigma_s = \sigma_c \frac{t_e}{t_s} \tag{19}$$

where,

σ_s : stress in tension of the plate (kgf/cm^2)
σ_{sa} : allowable stress of steel in tension (kgf/cm^2)
t_s : thickness of steel plate (cm)
σ_c : maximum vertical stress induced in Menshin device (kgf/cm^2)
t_e : thickness of one rubber layer (cm)

Design Requirement of LRB

In the LRB, natural rubber with the shear modulus of 8.0 kgf/cm^2 was adopted. The equivalent stiffness and the equivalent damping ratio is obtained as

$$K_E = \frac{A_R \cdot G \cdot \gamma + A_P \cdot q}{u_{Ee}} \tag{20}$$

$$h_E = \frac{2 Q_d \{u_{Ee} + Q_d / (K_2 - K_1)\}}{\pi u_{Ee} (Q_d + u_{Ee} K_2)} \tag{21}$$

where,

K_E : equivalent stiffness (kgf/cm^2)
h_E : equivalent damping ratio
u_{Ee} : effective design displacement of Menshin device (cm)
Q_d : yield force of lead, and can be determined as

$$Q_d = A_P \cdot q_O \tag{22}$$

A_p : sectional area of lead plug (cm^2)
q_O : yielding shear stress of lead (85 kgf/cm^2)

A_R : sectional area of rubber (cm^2)
G : shear modulus of rubber (kgf/cm^2)
γ : shear strain
q : shear stress of lead, and shall be obtained as

$$q = \begin{cases} -283.6\gamma^2 + 183.8\gamma + 85.0 & (0 \leqq \gamma \leqq 0.5) \\ 28.3\gamma^2 - 128.1\gamma + 163.0 & (0.5 < \gamma \leqq 2.0) \\ 20 & (2.0 < \gamma \leqq 2.5) \end{cases} \tag{23}$$

K_1, K_2 : first and second stiffness of LRB (kgf/cm^2)

The shape of the lead plug needs to be as

$$1.25 \leqq \frac{H_P}{D_P} \leqq 5.1 \tag{24}$$

where,
H_P : height of lead plug (cm)
D_P : diameter of lead plug (cm)

Design Requirements of HDR

In the HDR, the equivalent stiffness and the equivalent damping ratio are evaluated as

$$K_B = \frac{A_R \cdot G(\gamma)}{\Sigma t_e} \tag{25}$$

$$G(\gamma) = \begin{cases} 45.3 - 72.7\gamma_e + 57.3\gamma_e^2 - 19.0\gamma_e^3 + 2.18\gamma_e^4 & (0 < \gamma e \leqq 2.0) \\ 12.2 & (2.0 < \gamma_e \leqq 3.0) \end{cases} \tag{26}$$

$$h_B = 0.154 + 0.0383\gamma_e - 0.0445\gamma_e^2 + 0.00858\gamma_e^3 \quad (0 < \gamma_e \leqq 3.0) \tag{27}$$

where,
K_P : equivalent stiffness (kgf/cm^2)
h_B : equivalent damping ratio
G : shear modulus of the high damping rubber (kgf/cm^2)
A_R : sectional area of rubber (cm^2)
γ_e : effective shear strain
Σt_e : total thickness of rubber

Design of LRB and HDR

The natural period of the Menshin bridge is recommended in the Manual to set about twice as long as the one of the bridge with usual bearing condition (non-Menshin bridge). Because the natural period of the non-Menshin bridge was estimated as 0.58 second, it was aimed to design LRB and HDR so that the natural period of the Menshin bridge becomes approximately 1.1 second. It should be noted that effect of soils was considered in computing the natural period by a set of soil springs as shown in Table 8. Because the soil is stiff, effect of soil was insensitive for the natural period.

Because the equivalent stiffness of LRB and HDR depends on the deformation developed in the device, the natural period of the bridge is not the same depending on the device and the Menshin design method. The natural period of the bridge was evaluated as 1.23 second (LRB) and 0.95 second in the Seismic Coefficient Method, and 1.91 second (LRB) and 1.44 second (HDR) in the Bearing Capacity Method. The damping ratio was evaluated as

$$h = \frac{\Sigma K_{Bi} \cdot u_{Bi}^2 \cdot c_{hi}}{\Sigma K_{Bi} \cdot u_{Bi}^2 \cdot c_i} \tag{28}$$

$$c_{hi} = h_{Bi} + \frac{h_{Pi}}{K_{Pi}} + \frac{h_{Fui}}{K_{Fui}} + \frac{h_{F\theta i} \cdot H^2}{K_{F\theta i}} \tag{29}$$

$$c_i = 1 + \frac{K_{Bi}}{K_{Pi}} + \frac{K_{Bi}}{K_{Fui}} + \frac{K_{Bi} \cdot H^2}{K_{F\theta i}} \tag{30}$$

where

h	: Modal damping ratio of bridge
h_{Bi}	: Damping ratio of i-th damper
h_{Pi}	: Damping ratio of i-th pier/abutment
h_{Fui}	: Damping ratio of i-th foundation associated with translational movement
$h_{F\theta i}$	: Damping ratio of i-th foundation associated with rotation
K_{Pi}	: Equivalent stiffness of i-th pier/abutment
K_{Fui}	: Translational stiffness of i-th foundation
$K_{F\theta i}$	: Rotational stiffness of i-th foundation
u_{Bi}	: Design displacement of i-th menshin device
H	: Height from the bottom of pier to the gravity center of deck

The damping ratio of structural components including the LRB and HDR was assumed as shown in Table 9. The damping ratio of the bridge for the first mode was evaluated by Eq. (28) as 0.21 (LRB) and 0.13 (HDR) in the Seismic Coefficient Method and 0.12 (LRB) and 0.11 (HDR) in the Bearing Capacity Method. The design displacement u_B was 3.1 cm (LRB) and 2.3 cm (HDR) in the Seismic Coefficient Method and 33.0 cm (LRB) and 18.5 cm (HDR) in the Bearing Capacity Method.

The modification factor for damping ratio of bridge c_E, which was evaluated from Tables 4 and 6 based on the above described damping ratio of the bridge, is shown in Table 10. Table 11 shows the modification factor for structural response c_T (Seismic Coefficient Method) and c_R (Bearing Capacity Method). The lateral force coefficient becomes 0.17 (LRB) and 0.18 (HDR) in the Seismic Coefficient, and 0.45 (LRB) and 0.55 (HDR) in the Bearing Capacity Method.

It should be noted that if the non-Menshin bridge has the natural period of 0.58 second, the modification factor for structural response c_T and c_R is

1.25. and that the reduction of the lateral force coefficient by means of the modification factor for damping ratio of bridge c_E can not be considered. A comparison of the lateral force coefficient between the Menshin bridge and the non-Menshin bridge is shown in Table 12. In the Menshin bridge the lateral force coefficient is 15 % (LRB) and 10 % (HDR) in the Seismic Coefficient Method and 36 % (LRB) and 21 % (HDR) in the Bearing Capacity Method less than the non-Menshin bridge.

Fig. 2 shows the LRB and HDR designed based on those conditions. Although there was some interaction between the design of Menshin device and the design of substructure which will be described later, the final shape of the devices determined is presented here. The most critical factor for determining the dimension of the Menshin devices was the check against the lateral displacement in the Bearing Capacity Method by Eq. (13). THe shear strain of the rubber is 250 % and 231 % in LRB and HDR, respectively, while the allowable shear strain γ_{Ba} in Eq. (13) is 250 %.

Table 13 summarizes the design of LRB and HDR.

SEISMIC DESIGN OF SUBSTRUCTURES

Seismic Design of Pier

Seismic design of pier was made by the Seismic Coefficient Method. Fig. 3 shows the analytical model for the pier and the foundation. For designing the pier, the distance in longitudinal direction from the edge of the pier to the front edge of the bearing has to be larger than the following seat length to avoid the spalling of the concrete at the pier crest during an earthquake :

$$S = 20 + 0.5\, l \qquad (31)$$

where,

S : seat length from the edge of pier to the edge of bearing (cm)

l : span lengt (m)

Because the span length is 40 m, 40 cm is required for the seat length S. The seat length has to be provided both side of the bearing in longitudinal direction. Because the length of the Menshin device is 95 cm (LRB) and 89 cm (HDR), two times of the seat length S plus the length of the Menshin device is 175 cm (LRB) and 169 cm (HDR). Therefore, the depth of the pier (length in longitudinal direction) was determined as about 2 m. The width of the pier (length in transverse direction) was determined so that the stress of the concrete and that of the reinforcement become in good proportion.

The strength of the concrete is 240 kgf/cm^2, and the grade of the reinforcement is the SD 345 (deformed bars). As shown in Table 2, the allowable stress is 120 kgf/cm^2 in concrete and 3,000 kgf/cm^2 in the reinforcement.

Table 14 shows the force used for seismic design of pier and the stress computed. Because the bottom of the pier is the most critical in seismic design, the design calculation was made for this section. It can be said that the selected section is in good proportion, because the stress developed in the concrete and the reinforcement is close with their allowable stress and both the concrete and the reinforcements are effectively used.

After the dimension of the pier and the reinforcement was determined, the check of bearing capacity was made by the Bearing Capacity Method. Some increase of the depth and the reinforcement was required.

Fig. 4 shows the section thus determined. The size off the column is 2.2 m in longitudinal direction and 3 m in transverse direction. When LRB is adopted, 38 reinforcements with the diameter of 29 mm are required along the face in axial direction, and 24 reinforcements with diameter of 19 mm along the face in transverse direction are required. The total area of the reinforcements is 313.0 cm^2, and this corresponds to 0.47 % of the concrete section (main reinforcement ratio = 0.47 %). One the other hand, when HDR was adopted, the diameter of main reinforcement along the face in longitudinal direction were required to increase one grade, and 38 reinforcements with diameter of 32 mm were provided. The reinforcement along the face in transverse direction was the same with the previous case. The total area of the reinforcement is 370.6 cm^2, and the main reinforcement ratio is 0.56 %.

Seismic Design of Foundation

Design of foundation was made by the Seismic Coefficient Method. Safety against sliding, overturning and bearing capacity of supporting soil was checked. Table 15 summarizes the design of the foundation. The size of the footing is 7.5 m x 7.5 m when the LRB was adopted and 7.7 m x 7.7 m when the HDR was adopted.

CONCLUDING REMARKS

The preceding pages presented a design example of a bridge supported by LRB and HDR. By adopting the Menshin devices, 10 % ~ 15 % decrease and 21 %~ 36 % decrease of the lateral force could be made in the Seismic Coefficient Method and the Bearing Capacity Method, respectively. The bridge designed seems in good shape and reasonable.

REFERENCES

1) Robinson, W.H. : Lead-rubber Hysteretic Bearings Suitable for Protecting Structures during Earthquakes, Earthquake Engineering and Structural Dynamics, Vol. 10, 1982
2) Manual for Menshin Design of Highway Bridges, Public Works Research Institute and 28 Private Firms, Bulletin of PWRI, Vol. 59, Public Works Research Institute, March 1992
3) Japan Road Association : Design Specifications of Highway Bridges, 1990

Table 1 Design Conditions

Structure	Multi-span Continuous Prestressed Concrete Girder
Span Length	40m
Width	14m
Clear Heigth	10m
Dead Weight of Superstructure	20.4tf/m (200KN/m)
Soile Condition	N=30, Depth=60m

Table 2 Design Condition for Foundation

Allowable Stress	Steel	σ_{sa}	3,000kgf/cm^2(SD345)
	Concrete	σ_{ca}	120kgf/cm^2(σ ck=240kgf/cm^2)
Allowable Bearing Capacity q_a			60tf/m^2
Safty Factor for Sliding			1.2
Safty for Overturning			Combined force of Axial force and lateral force should be applied within 1/3 of the width of footing

Table 3 Modification Factor for Structural Response c_T

Ground Group	Structural Response Coefficient c_T		
Group I	$T < 0.1$ $c_T = 2.69T^{1/3} \geqq 1.00$	$0.1 \leqq T \leqq 1.1$ $c_T = 1.25$	$1.1 < T$ $c_T = 1.33T^{-2/3}$
Group II	$T < 0.2$ $c_T = 2.15T^{1/3} \geqq 1.00$	$0.2 \leqq T \leqq 1.3$ $c_T = 1.25$	$1.3 < T$ $c_T = 1.49T^{-2/3}$
Group III	$T < 0.34$ $c_T = 1.80T^{1/3} \geqq 1.00$	$0.34 \leqq T \leqq 1.5$ $c_T = 1.25$	$1.5 < T$ $c_T = 1.64T^{-2/3}$

Table 4 Modification Factor for Damping Ratio c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$h \geqq 0.1$	0.9

Table 5 Modification Factor for Structural Response c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T_{EQ} \leqq 1.4$ $c_R = 0.7$		$1.4 < T_{EQ}$ $c_R = 0.876T_{EQ}^{-2/3}$
Group II	$T_{EQ} < 0.18$ $c_R = 1.51T_{EQ}^{1/3} \geqq 0.7$	$0.18 \leqq T_{EQ} \leqq 1.6$ $c_R = 0.85$	$1.6 < T_{EQ}$ $c_R = 1.16T_{EQ}^{-2/3}$
Group III	$T_{EQ} < 0.29$ $c_R = 1.51T_{EQ}^{1/3} \geqq 0.7$	$0.29 \leqq T_{EQ} \leqq 2.0$ $c_R = 1.0$	$2.0 < T_{EQ}$ $c_R = 1.59T_{EQ}^{-2/3}$

Table 6 Modification Factor for Damping Ratio c_E

Damping Ratio h	Modification Factor c_E
$h < 0.1$	1.0
$0.1 \leqq h < 0.12$	0.9
$0.12 \leqq h < 0.15$	0.8
$0.15 \leqq h$	0.7

Table 7 Rupture Strain of Rubber

Type of Rubber	Shear Modulus G(kgf/cm^2)	Rupture Strain γ u(%)
Natural Rubber	8.0	500
	10.0	500
	12.0	400
Chloroprene Rubber	8.0	400
	10.0	400

Table 8 Spring Stiffness of Soils

	LRB	HDR
Stiffness for Translation	2.88×10^5 tf/m	2.99×10^5 tf/m
Stiffness for Rotation	4.04×10^6 tf/m	4.51×10^6 tf/m

Table 9 Damping Ratio Assumed in Design for Structural Components

Superstructures			0.03
Menshin Device	SCM	LRB	0.28
		HDR	0.16
	BCM	LRB	0.15
		HDR	0.14
Pier/Columns			0.05
Footing			0.1

Table 10 Modification Factor c_E for the Bridge Designed

	LRB	HRD
SCM	0.9	0.9
BCM	0.8	0.8

Table 11 Modification Factor c_T and c_R for the Bridge Designed

	LRB	HRD
SCM	0.9	1.0
BCM	0.8	0.8

Table 12 Comparison of Lateral Force Coefficient between Menshin Bridge and Non-Menshin Bridge

Design Method	SCM		BCM	
	LRB	HDR	LRB	HDR
① Menshin Design	0.17	0.18	0.45	0.55
② Non-Menshin Design	0.20		0.7	
①／②	85%	90%	64%	79%

Table 13 Design of LRB and HDR

Characteristic Values			LRB		HDR	
			SCM	BCM	SCM	BCM
Lateral Force		k_h	0.17	0.45	0.18	0.55
Natural Period		T(s)	1.23	1.91	0.95	1.44
Design Displacement		u_E(cm)	3.1	33.0	2.3	18.5
Effective Displacement		u_{Ee}(cm)	2.1	23.1	1.6	13.0
Damping Ratio		h_T	0.21	0.12	0.14	0.12
Equivalent Stiffness		K_E(tf/m)	4,775	1,132	6,403	2,374
Equivalent Damping Ratio		h_E	0.28	0.15	0.16	0.14
Lateral Force		F(tf)	105.9	261.4	144.0	448.0
Local Shear Strain	Vertical Strain	γ_C	0.24	1.09	0.60	1.62
	Lateral Strain	γ_S	0.75	2.50	0.28	2.31
	Total Strain	γ_T	0.99	3.59	0.88	3.93

Table 14 Design Force and Check of Seismic Safety of Pier

Characteristics		LRB	HDR
Force Developed at Pier Bottom	Axial Force F	1,062tf	1,062tf
	Bending Moment M	1,564tfm	1,656tfm
	Shear Force Q	136tf	144tf
Diameter of Main Reinforcement and Interval		D29@150mm	D32@150mm
Seismic Design Method	Stress of Concrete σ_c	108kgf/cm^2	106kgf/cm^2
	Stress of Reinforcement σ_s	2,838kgf/cm^2	2,643kgf/cm^2
Bearing Capacity Method	Bearing Capacity considering Ductility	186tf	219tf
	Lateral Force considering Ductility $k_{he} \cdot W$	165tf	203tf

Table 15 Design of Foundation

	LRB	HDR	Allowable Values
Bearing Capacity	54.6tf/m^2	53.2tf/m^2	60.0tf/m^2
Safety Factor for Sliding	6.8	6.6	≧1.2
Safety for Overturning (Eccentricity from Center of Footing)	1.23m	1.28m	≦2.5～2.6

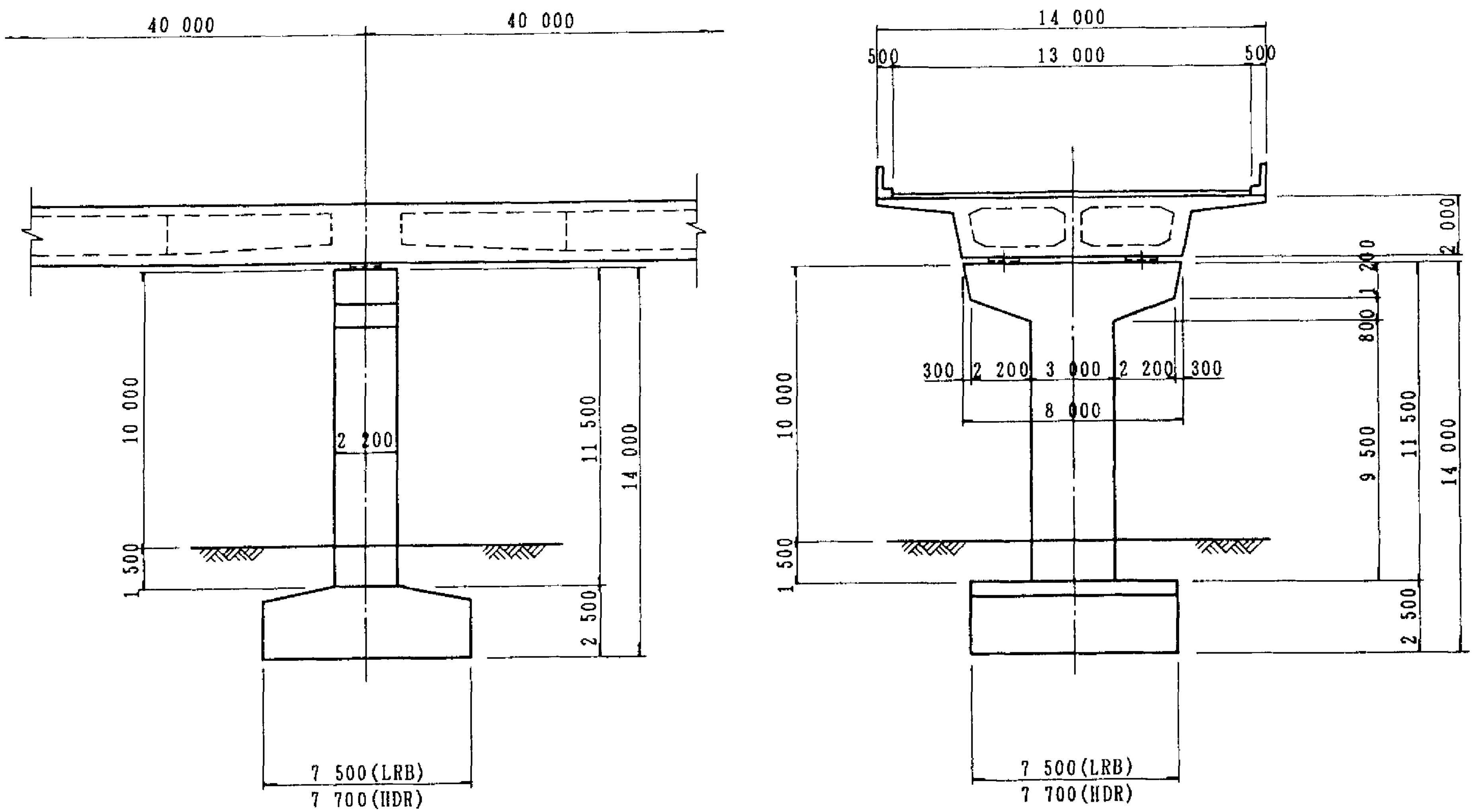

Fig.1 Bridge Considered in Design

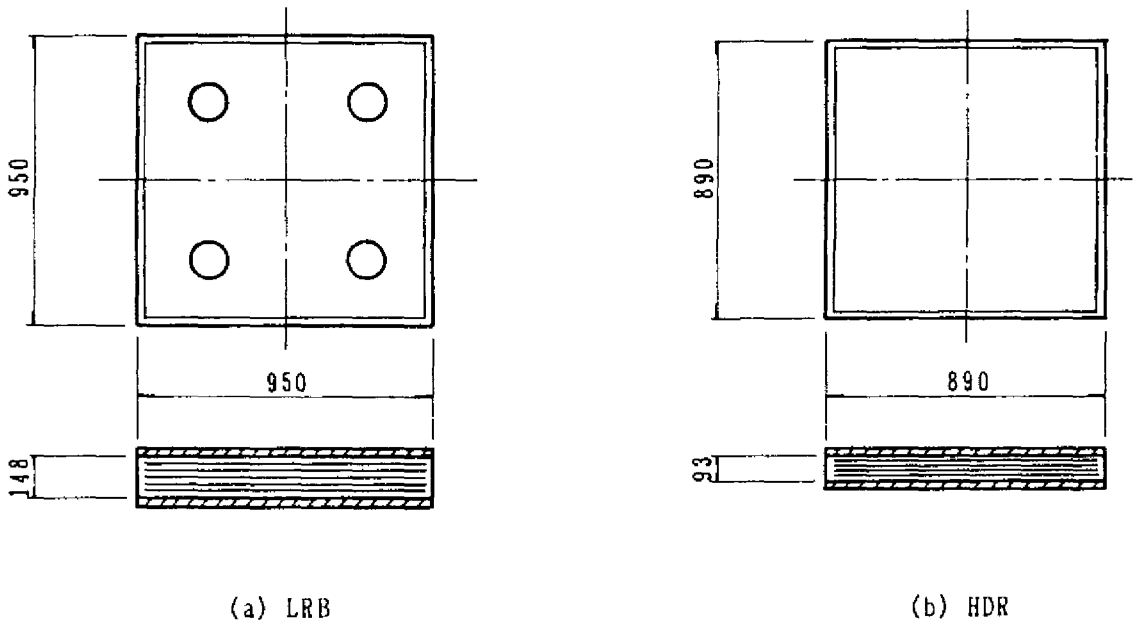

Fig. 2 LRB and HDR

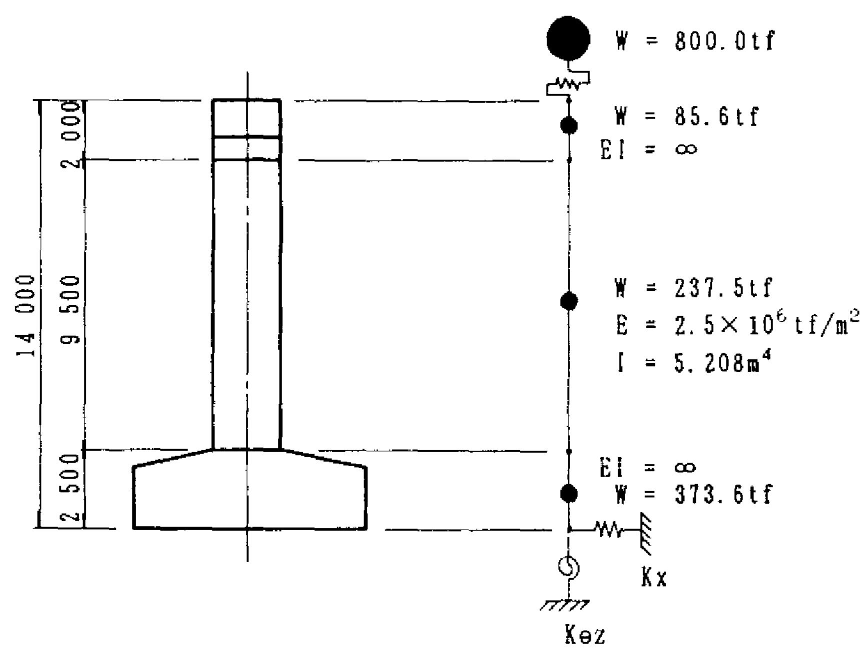

Fig. 3 Analytical Model in Design

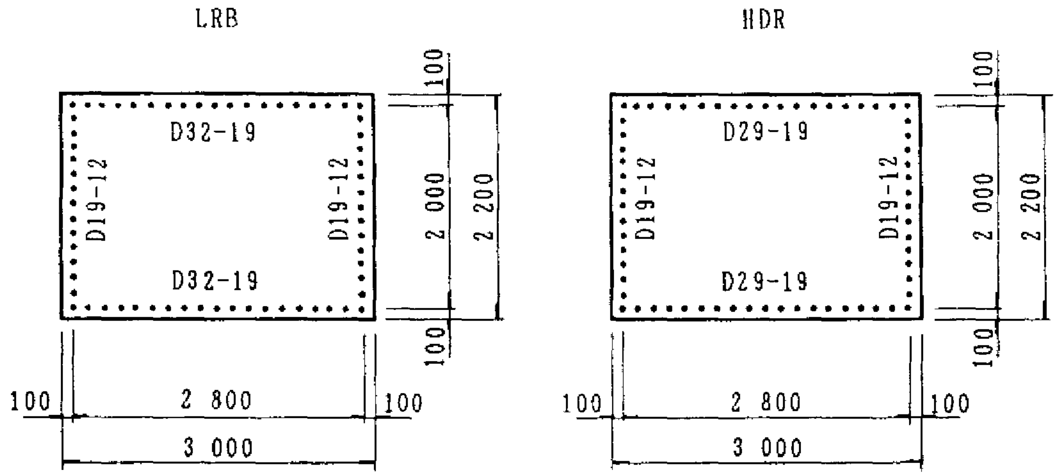

Fig.4 Section of Pier and Main Reinforcement