MENSHIN DESIGN EXAMPLE OF A HIGHWAY BRIDGE

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ABSTRACT

This paper presents a Menshin design example of a highway bridge based on the Japanese design specifications. The design of Yama-age Bridge which was completed in May 1993 in Tochigi Prefecture is presented as an example. The Yama-age bridge is of 6-span continuous concrete box girder with length of 246.3m. High damping rubber bearings were adopted as Menshin devices.

YAMA-AGE BRIDGE

Fig. 1 and Photo 1 show the Yama-age bridge[1)-3)]. Table 1 summarizes the design outlines of the Yama-age bridge. The superstructure is of 6-span continuous prestressed concrete two-cell box girder with length of 246.3m. The deck width ranging from 6.5m (standard section) to 8.0m (wide section). The abutment is of a inverted T-type reinforced concrete substructure and the piers are of reinforced concrete wall type with rectangular section. The foundations of all substructures are of direct foundation. High damping rubber (HDR) bearing is adopted for the bridge. Since Menshin design is applied only in longitudinal direction, the displacement in transverse direction is restrained by stoppers. Forced excitation test using an eccentric-mass shaker and quick-release jacks was also made for the bridge in December 1992 as shown in **Photo 2**[4)].

The ground condition of the Yama-age bridge consists of sand-gravel layers and slate layer. N-value by the standard penetration test for the sand-gravel layers is ranging from 30 to 50, and that of the slate layer is greater than 50. The ground condition is classified as Class I .

MENSHIN DESIGN OF YAMA–AGE BRIDGE

Design Specifications

The Yama–age Bridge was designed in accordance with the regulations of the "Design Specifications for Highway Bridges[5]". "Guidelines for Design of Menshin Highway Bridges[6]" and "Manual for Menshin Design of Highway Bridges[7)-8)]" were also referred for the Menshin design issues. It should be noted that since the bridge was designed based on the Design Specifications for Highway Bridges, the design lateral force was not reduced in consideration of the damping effect of Menshin bearings.

Menshin Design

Fig. 2 shows the Menshin design flow used for the Yama–age Bridge. In the Menshin design of highway bridges, the Menshin devices are designed by the "Seismic Coefficient Method" and the "Bearing Capacity Method". In both methods, the lateral force is statically applied to the bridge, and the seismic safety is checked based on the allowable stress design approach in the Seismic Coefficient Method and bearing capacity basis considering ductility in the Bearing Capacity Method. Bridges are designed by the Seismic Coefficient Method, and then the ductility is checked for reinforced concrete piers by the Bearing Capacity Method.

Design of Menshin Bearing

The relation between shear modulus of elasticity $G(\gamma)$ and shear strain γ for HDR bearings adopted is shown in **Fig. 3**. The stiffness of the bearings shows the nonlinear characteristics depending on the strain of the bearing. The shear modulus of HDR is given by the following experimental equations.

$$\left.\begin{array}{ll} G(\gamma)=26.3-46.0\gamma+45.7\gamma^2-21.1\gamma^3+3.88\gamma^4 & (0<\gamma\leqq 1.8) \\ G(\gamma)=0.31+6.89\gamma-1.08\gamma^2 & (\gamma>1.8) \end{array}\right\} \quad (1)$$

The design of HDR bearings was made according to the following procedure as :

1) Assume the design displacement of bearings for two levels of seismic lateral forces and compute the shear strain of bearings.

$$\gamma=\frac{u_B}{H_R} \quad (2)$$

where,

γ : shear strain

u_B : design displacement of bearing

H_R : thickness of rubber bearing

2) Compute the shear modulus of elasticity and the equivalent stiffness of bearings.

$$K_B = A_0 \times G(\gamma) \tag{3}$$

where,

$G(\gamma)$: shear modulus of elasticity

K_B : equivalent stiffness

A_0 : sectional area of bearing

3) Compute the natural period of the bridge and the horizontal design lateral force coefficient.

$$K_T = \sum K_B \tag{4}$$

$$T = \frac{2\pi \cdot R_d}{\sqrt{g \cdot K_T}} \tag{5}$$

where,

K_T : total stiffness of bearings

T : natural period

R_d : dead load of superstructure

g : gravity acceleration

4) Compute the displacement of bearings, and compare it with the assumed displacement.

$$u_s = \frac{R_d \cdot k_h}{K_B} \tag{6}$$

where,

k_h : lateral force coefficient

Tables 2 to 4 show the design of HDR bearings of the Yama−age bridge.

Design Lateral Force Coefficient

The bridge was designed based on the Seismic Coefficient Method. Since the seismic design structural unit of the Yama−age bridge is defined as the total bridge system, analytical idealization of the bridge is shown in **Fig. 4**. Bearings and foundations were modeled as equivalent linear spring elements. Natural period of the bridge is computed for the seismic design structural unit as :

$$T = 2.01 \cdot \sqrt{\delta} \tag{7}$$

$$\delta = \frac{\int w(s)\, u(s)^2 ds}{\int w(s)\, u(s) ds} \tag{8}$$

where,

T : natural period

$w(s)$: dead weight of the seismic design structural unit at point "s"

$u(s)$: lateral displacement at point "s" when subjected to $w(s)$ in the direction considered in design

Design seismic coefficient is computed as:

[Seismic Coefficient Method]

$$k_h = c_Z \cdot c_G \cdot c_I \cdot c_T \cdot k_{h0} \tag{9}$$

$$c_T = 1.33 \cdot T^{-2/3} \text{ (Ground Condition : Class I)} \tag{10}$$

where,

k_h : lateral force coefficient

c_Z : modification factor for zone (**Fig. 5**)

c_G : modification factor for ground condition (**Table 5**)

c_I : modification factor for importance (**Table 6**)

c_T : modification factor for structural response (**Table 7**)

k_{h0} : standard design horizontal seismic coefficient (=0.2)

[Bearing Capacity Method]

$$k_{he} = \frac{k_{hc}}{\sqrt{2 \cdot \mu - 1}} \tag{11}$$

$$k_{hc} = c_Z \cdot c_I \cdot c_R \cdot k_{hc0} \tag{12}$$

$$c_R = 0.876 \cdot T^{-2/3} \text{ (Ground Condition : Class I)} \tag{13}$$

where,

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

k_{hc} : lateral force coefficient for Bearing Capacity Method

c_R : modification factor for structural response (**Table 8**)

μ : allowable ductility factor of reinforced concrete piers

k_{hc0} : standard design horizontal seismic coefficient (=1.0)

The natural period in longitudinal direction is 1.56sec, and that of the transverse direction is 0.194sec. Therefore, the lateral force coefficients are 0.16 and 0.20 in longitudinal and transverse directions, respectively. Since the natural period in longitudinal direction with usual design is 1.075sec, the lateral force coefficient is 0.20. Hence, by adopting the Menshin design, the lateral force is reduced by 20% than the usual design.

Table 9 shows the relative displacement of bearings by the Seismic Coefficient Method. **Table 10** shows the bending moment at the bottom of piers. The bending moment is compared between with and without the Menshin design. **Fig. 6** shows the cross section of pier (P1).

Check of Bearing Capacity for Lateral Force

To prevent a brittle failure such as falling-off of superstructure during large earthquakes, the bearing capacity of the reinforced concrete piers designed by the Seismic Coefficient Method was checked by the Bearing Capacity Method.

The natural period of the bridge is computed using the equivalent stiffness of bearings corresponding to the design displacement and equivalent yielding stiffness of substructures. Since it is 1.77 sec, the lateral force coefficient for the Bearing Capacity

Method k_{hc} is 0.60. **Tables 11** and **12** shows the check results of the bearing capacity of the reinforced concrete piers.

Dynamic Analysis

Dynamic Analysis was made to check the safety of the Yama−age Bridge. the response spectrum analysis and the time history analysis were made. The response spectrum and time history acceleration data of the earthquake ground motion corresponding to the Seismic Coefficient Method was used as an input acceleration. **Table 13** shows the equivalent stiffness and the equivalent damping ratio of HDR bearings.

Since the damping characteristics of the HDR bearings varies depending on the displacement, the following experimental equation on the damping ratio was used.

$$h_B = 0.172 - 0.00693\gamma + 0.00276\gamma^2 - 0.006924\gamma^3 \qquad (14)$$

where,

h_B : effective damping ratio

γ : shear strain of bearing

The damping ratio for the superstructure, substructures, and the foundations were assumed as 3%, 5%, and 10%, respectively. **Table 14** shows the natural period and the damping ratio of an each vibration mode. The 1st to 3rd vibration modes are shown in **Fig. 7**. The 1st mode is a sway mode of the superstructure and this is the most predominant mode in longitudinal direction. The damping ratio of the 1st mode is 14.3%.

Tables 15 and **16** shows the displacement of bearings and sectional forces of the pier bottom computed through the dynamic analyses in comparison with those by the Seismic Coefficient Method. The displacement of bearings and bending moment by the dynamic analyses are less than those by the Seismic Coefficient Method.

Fig. 8 shows the acceleration responses of deck and pier P1 computed through the time history analysis. It is found that the period of the deck response is elongated by adopting the Menshin design and that the deck acceleration is reduced comparing with that of the pier top.

It should be noted here that although the design lateral force was not reduced in consideration of the damping effect by Menshin devices in the design, it is found through the dynamic analyses that the response of the bridge is significantly reduced (by about 35 − 40% than those by the Seismic Coefficient Method) by adopting the Menshin design.

Design Details

According to the design specifications, the falling−off prevention devices are installed for the bridge. **Fig. 9** shows the stopper to prevent excessive displacement of the deck.

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Table 1 Design Outline of Yama-age Bridge

Bridge Type	Prestressed Concrete Post Tensioning
Structure	6-Span Continuous Box Girder
Road Class	Design Vehicle Speed : 60km/h
Bridge Length	246.3m
Span Length	$6 \times 40.8 = 244.8$m
Deck Width	Standard Section : 8.0m(roadway)+2.5m(sidewalk)=10.5m Widening section :11.0m(roadway)+2.5m(sidewalk)=13.5m
Live Load	TL-20
Impact Coefficient	$i=1.0/(20+L)$
Alignment	$R=\infty$ ~ A=240m
Vertical Gradient	1.0% ~ VCL=100m, R=2,500m ~ 5.0%
Cross Slope	1.5%(roadway), 2.0%(sidewalk)
Abutment Skew Angle	90°(A1), 90°27′ 43″ (A2)
Ground Condition	Class : I
Design Lateral Force Coefficient	$k_h=0.16$ (Longitudinal direction) $k_h=0.20$ (Transverse direction)

Table 2 Dimension of HDR Bearings (unit:mm)

Item	A1	P1	P2	P3	P4	P5	A2
Dimension	700×855	950×1500	1030×1580	950×1500	950×1500	950×2500	700×855
Thickness of Rubber Layers	$14.7 \times 14 = 205.8$	$21.0 \times 6 = 126.0$	$34.0 \times 3 = 102.0$	$24.0 \times 5 = 120.0$	$24.0 \times 6 = 144.0$	$18.7 \times 8 = 149.6$	$13.8 \times 19 = 262.2$
Thickness of Insert Plates	$4.2 \times 13 = 54.6$	$4.2 \times 5 = 21.0$	$4.2 \times 2 = 8.4$	$4.2 \times 4 = 16.8$	$4.2 \times 5 = 21.0$	$4.2 \times 7 = 29.4$	$4.2 \times 18 = 75.6$
Height of bearing	260.4	147.0	110.4	136.8	165.0	179.0	337.8

Note)Dimension $A \times B$ represents the widths in longitudinal and transverse directions, respectively.

Table 3 Design of HDR Bearings by Seismic Coefficient Method

Item	[ ]:Unit	A1	P1	P2	P3	P4	P5	A2
Assumed Displacement	u_s [cm]				7.784			
Thickness of Rubber Bearing	H_R [cm]	20.58	12.60	10.20	12.00	14.40	14.96	26.22
Shear Strain	γ	0.378	0.618	0.763	0.649	0.541	0.520	0.297
Shear Modulus of Elasticity	$G(\gamma)$ [kgf/cm^2]	14.377	10.914	9.749	10.618	11.787	12.050	16.150
Area of Bearing	A_0 [cm^2]	5,872	14,137	16,161	14,137	14,137	14,137	5,872
Equivalent Stiffness	K_B [kgf/cm^2]	4,102	12,245	15,446	12,509	11,571	11,387	3,617
Number of Bearing on Each Substructure					2			
Sum of K_B	K_T [kgf/cm^2]				141,756			
Natural Period	T [sec]				1.337			
Dead load of Superstructure	R_d [kgf]				6,294,200			
Factor for Structural Response	c_T				1.096			
Design Lateral Force Coefficient	k_h				0.18			
Designed Displacement	u_s [cm]				7.770			

Table 4 Design of HDR Bearings by Bearing Capacity Method

Item	[ ]:Unit	A1	P1	P2	P3	P4	P5	A2
Assumed Displacement	u_s [cm]	32.69						
Thickness of Rubber Bearing	H_R [cm]	20.58	12.60	10.20	12.00	14.40	14.96	26.22
Shear Strain	γ	1.588	2.594	3.205	2.724	2.270	2.185	1.247
Shear Modulus of Elasticity	$G(\gamma)$ [kgf/cm^2]	8.674	10.916	11.299	11.064	10.385	10.208	8.469
Area of Bearing	A_0 [cm^2]	5,872	14,137	16,161	14,137	14,137	14,137	5,872
Equivalent Stiffness	K_B [kgf/cm^2]	2,475	12,247	17,902	13,035	10,195	9,647	1,896
Number of Bearing on Each Substructure		2						
Sum of K_B	K_T [kgf/cm^2]	134,794						
Natural Period	T [sec]	1.372						
Dead load of Superstructure	R_d [kgf]	6,294,200						
Factor for Structural Response	c_T	0.700						
Design Lateral Force Coefficient	k_h	0.70						
Designed Displacement	u_s [cm]	32.69						

Table 5 Modification Factor for Ground Condition c_G

Ground Group	I	II	III
c_G	0.8	1.0	1.2

Table 6 Modification Factor for Importance c_I

Group	c_I	Definition
1st class	1.0	Bridges on expressway (limited access highways), general national road and principal prefectural road. Important bridges on general prefectural road and municipal road.
2nd class	0.8	Other than the above

Table 7 Modification Factor for Structural Response c_T

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

Table 8 Modification Factor for Structural Response c_R

Ground Group	Structural Response Coefficient c_R		
Group I	$T\leqq 1.4$ $c_R=0.7$		$1.4<T$ $c_R=0.876T^{-2/3}$
Group II	$T<0.18$ $c_R=1.15T^{1/3}\geqq 0.7$	$0.18\leqq T\leqq 1.6$ $c_R=0.85$	$1.6<T$ $c_R=1.16T^{-2/3}$
Group III	$T<0.29$ $c_R=1.15T^{1/3}\geqq 0.7$	$0.29\leqq T\leqq 2.0$ $c_R=1.00$	$2.0<T$ $c_R=1.59T^{-2/3}$

Table 9 Relative Displacement of Bearings by Seismic Coefficient Method (unit:cm)

Item	A1	P1	P2	P3	P4	P5	A2
Relative Displacement of Bearings	10.04	8.31	5.65	6.33	6.62	7.22	9.75
Design Displacement of Bearings	7.77						

Table 10 Bending Moment at the Bottom of Piers (unit:tf·m)

Item	P1	P2	P3	P4	P5
Menshin Design	4,704.7	3,677.2	2,824.4	2,345.0	2,017.0
Usual Design	4,793.2	5,062.8	4,760.8	4,601.5	4,578.2

Table 11 Check of Bearing Capacity of Piers

Item		Unit	P1	P2	P3	P4	P5
Width of pier		m	8.0	8.0	6.5	6.5	6.5
Thickness of pier		m	2.5	2.2	2.2	2.0	2.0
Longitudinal Reinforcement	Main Reinforcement	mm cm^2	D32−ctc150 1.5 step 1,270.72	D32−ctc150 1.5 step 1,270.72	D29−ctc150 1.5 step 835.12	D29−ctc150 1.5 step 835.12	D29−ctc150 1.0 step 552.46
	Side Reinforcement	mm cm^2	D19−ctc150 138.03	D19−ctc150 126.57	D19−ctc150 114.42	D19−ctc150 108.69	D19−ctc150 108.69
Allowable Ductility Factor μ			4.739	3.683	5.393	4.991	4.984
Equivalent Horizontal Seismic Coefficient k_{he}			0.21	0.24	0.19	0.21	0.20
Bearing Capacity for Shear Failure P_s		tf	1,385.95	1,209.86	995.81	899.18	951.53
Bearing Capacity for Flexural force P_u		tf	378.60	347.17	285.88	278.04	269.70
Bearing Capacity for Lateral Force P_a		tf	378.60	347.17	285.88	278.04	269.70
Inertia Force $P = k_{he} \times w$		tf	348.20	346.58	239.58	245.54	250.82

Table 12 Relative Displacement of Bearings by Bearing Capacity Method (unit:cm)

	A1	P1	P2	P3	P4	P5	A2
Relative Displacement of Bearing	48.86	30.50	20.08	24.85	28.51	34.06	48.11
Design Displacement of Bearing	32.69						

Table 13 Equivalent Stiffness and Equivalent Damping Ratio of HDR Bearings

Item	[ ]:Unit	A1	P1	P2	P3	P4	P5	A2
Design Displacement	u [cm]	10.04	8.31	5.65	6.33	6.62	7.22	9.75
Area of Bearing	A [cm^2]	5,872	14,137	16,161	14,137	14,137	14,137	5,872
Thickness of Rubber Bearing	H_R [cm]	20.58	12.60	10.20	12.00	14.40	14.96	26.22
Shear Strain	γ	0.488	0.660	0.544	0.528	0.460	0.483	0.372
Shear Modulus of Elasticity	$G(\gamma)$ [kgf/cm^2]	12.505	10.512	11.621	11.955	12.943	12.583	14.503
Equivalent Stiffness	K_B [kgf/cm^2]	3,568	11,805	18,412	14,084	12,698	11,890	3,248
The Effective Damping Ratio	h_B	0.168	0.167	0.168	0.168	0.169	0.169	0.169

Table 14 Natural Period and Damping Ratio

Mode No.	Natural Period (sec)	Damping Ratio
1	1.546	0.143
2	0.451	0.030
3	0.419	0.030
4	0.357	0.030
5	0.302	0.092
6	0.299	0.056
7	0.273	0.115
8	0.259	0.031
9	0.253	0.114
10	0.236	0.031

Table 15 Displacement of Bearing computed by Dynamic Analysis Method (unit:mm)

	Seismic Coefficient Method	Dynamic Analysis	
		Response Spectrum Analysis	Time History Analysis
A1	100.42	65.76	68.72
P1	83.08	55.05	58.70
P2	77.84	36.85	38.82
P3	77.84	41.59	44.74
P4	77.84	43.60	47.14
P5	77.84	47.85	51.32
A2	97.51	63.47	67.77

Table 16 Sectional Forces at Pier Bottom computed by Dynamic Analysis Method

Item		Seismic Coefficient Method	Dynamic analysis	
			Response Spectrum Analysis	Time History Analysis
Bending Moment (tf·m)	A1	3,998.1	828.9	866.4
	P1	4,354.4	2,665.1	2,710.0
	P2	3,652.9	2,325.6	2,665.7
	P3	2,777.8	1,754.7	1,929.7
	P4	2,322.4	1,464.7	1,570.7
	P5	1,992.3	1,234.7	1,272.4
	A2	900.7	514.4	533.4
Shear stress (tf)	A1	426.6	47.3	49.4
	P1	345.1	200.0	186.2
	P2	286.5	165.0	206.9
	P3	238.5	140.8	162.6
	P4	218.3	129.5	146.1
	P5	218.6	128.2	138.2
	A2	146.8	101.6	90.8

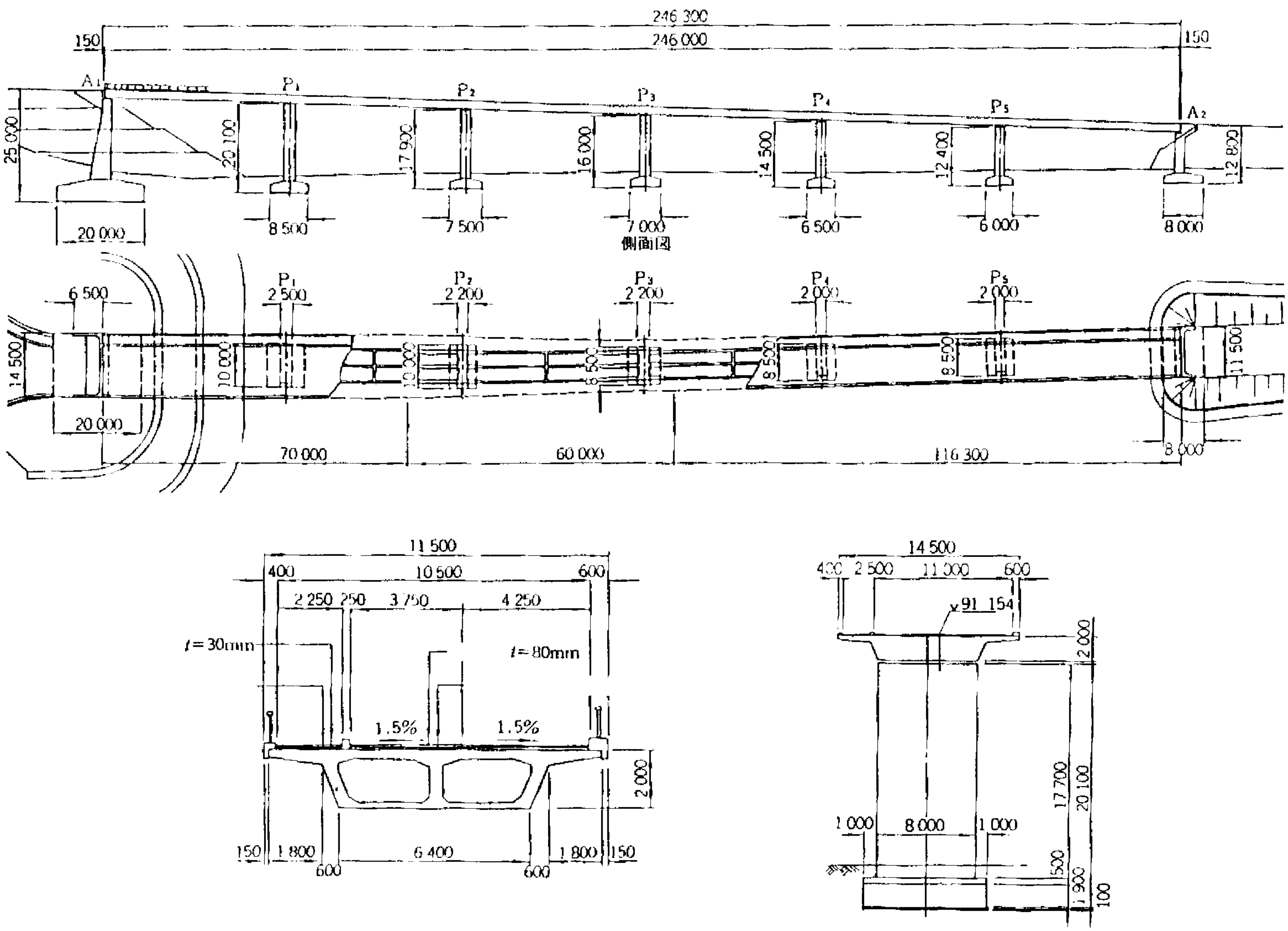

Fig. 1 General View of Yama–age Bridge

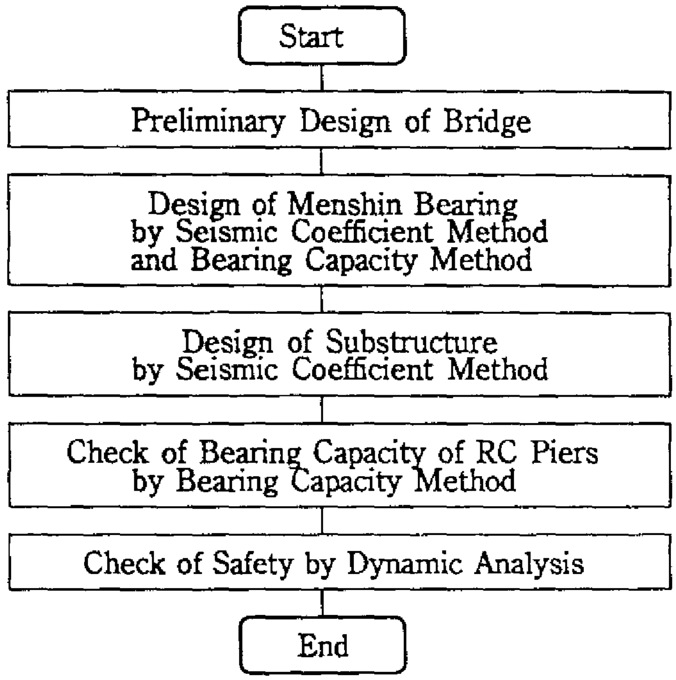

Fig. 2 Menshin Design Flow

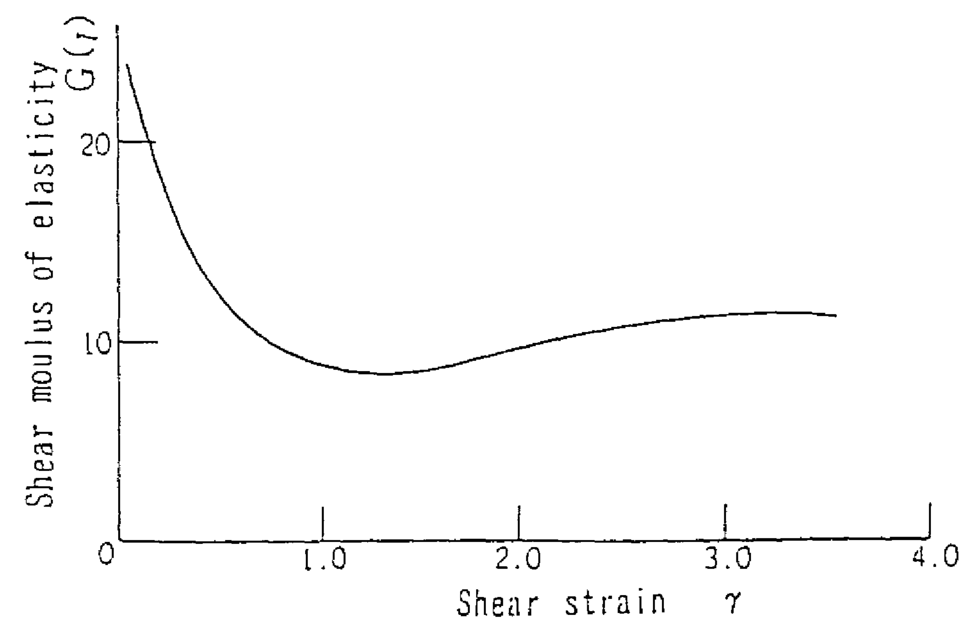

Fig. 3 $G(\gamma) \sim \gamma$ Relation of High Damping Rubber

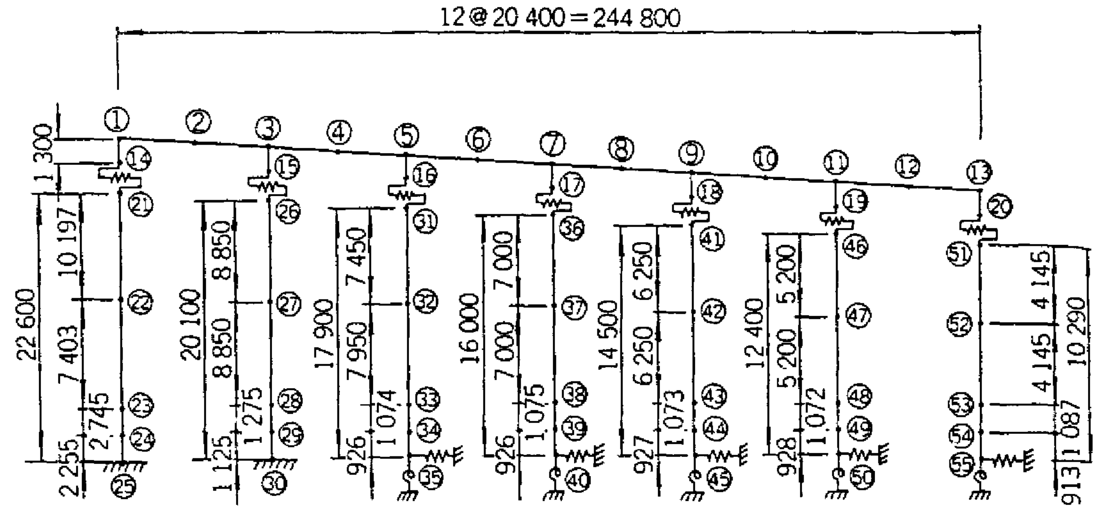

Fig. 4 Analytical Model

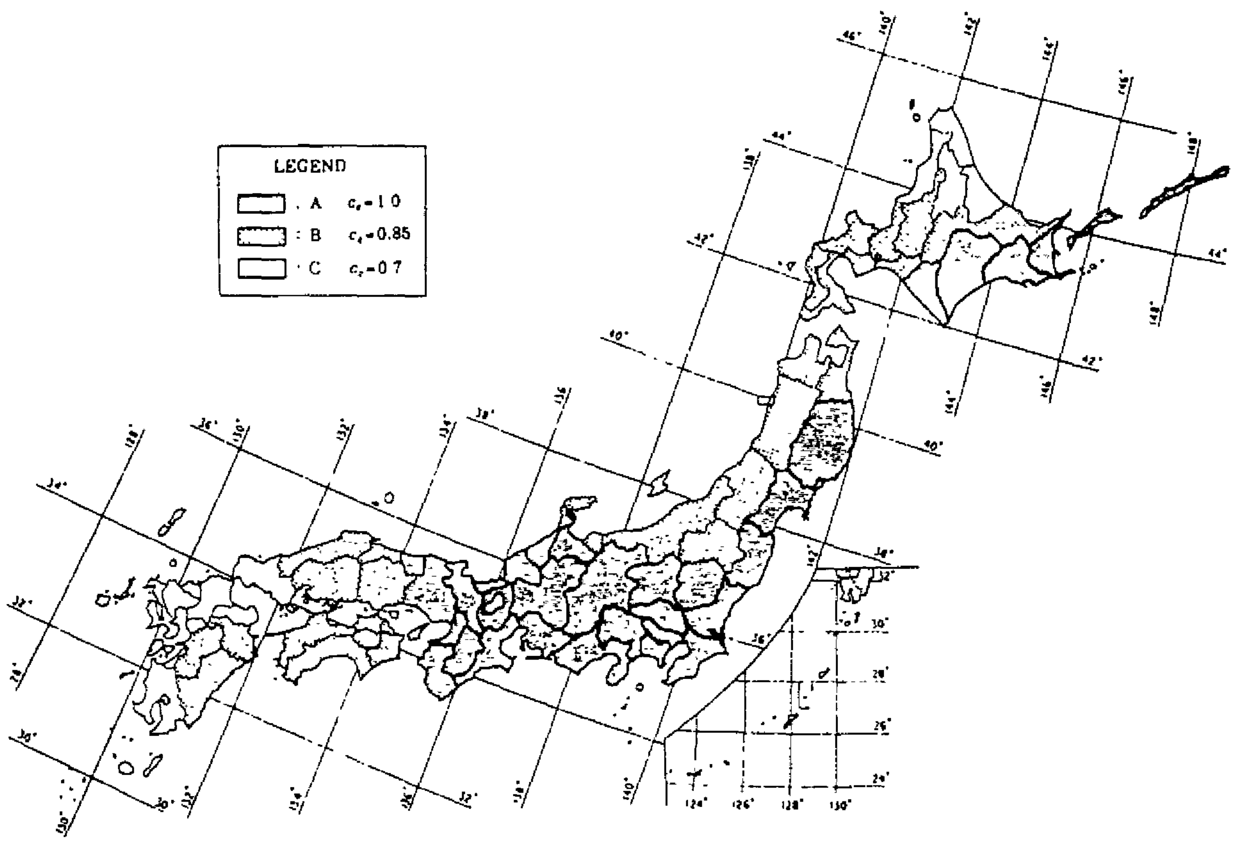

Fig. 5 Modification Factor for Zone c_z

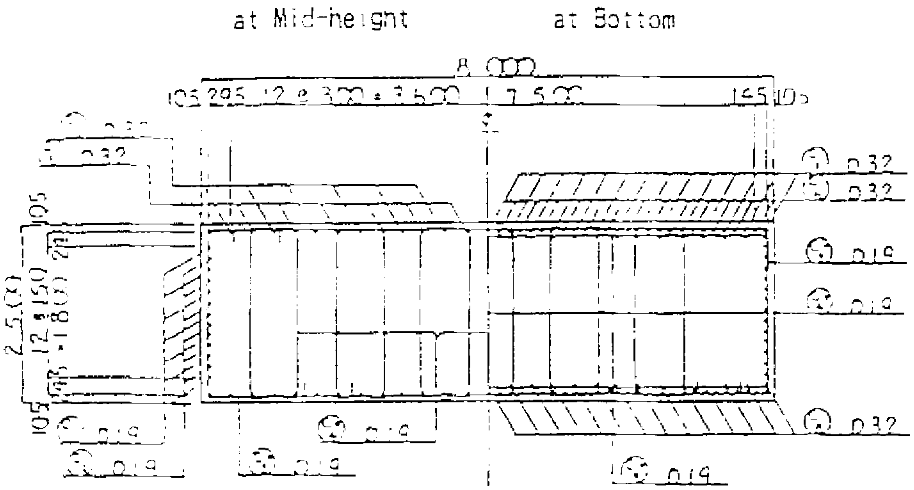

Fig. 6 Cross Section and Reinforcement of Pier (P1)

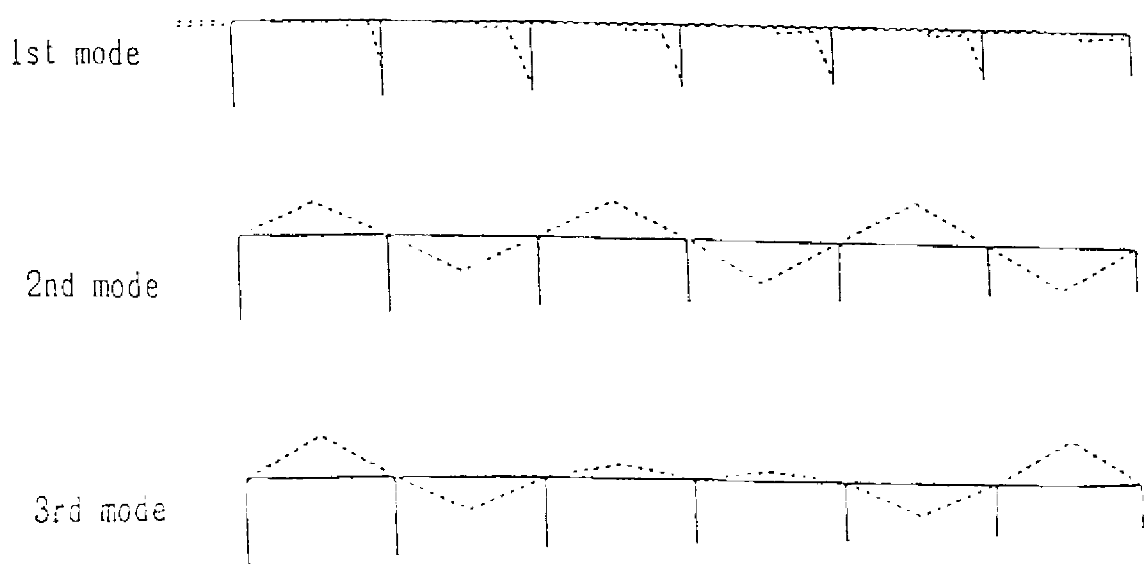

Fig. 7 Vibration Mode Shapes

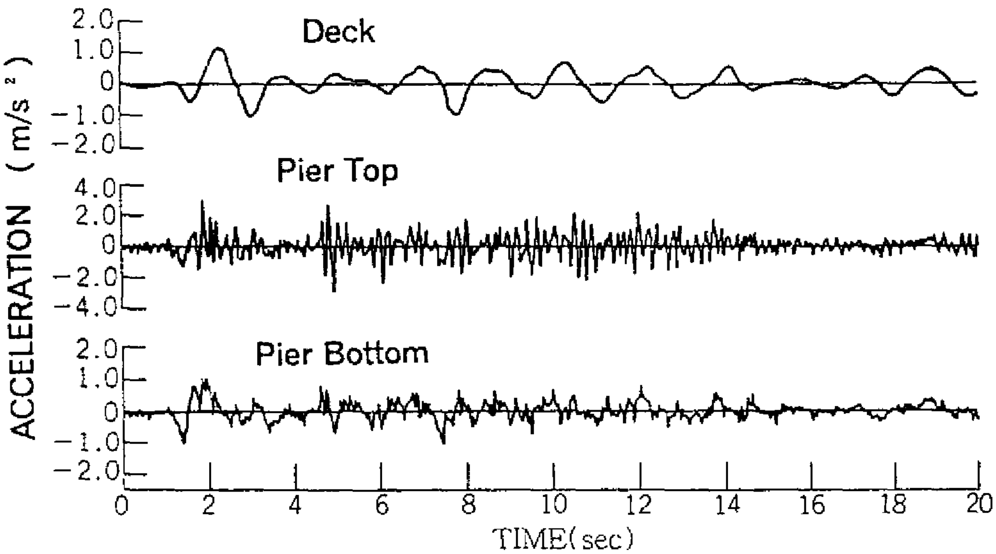

Fig. 8 Acceleration Response of Yama-age Bridge (Pier P1)

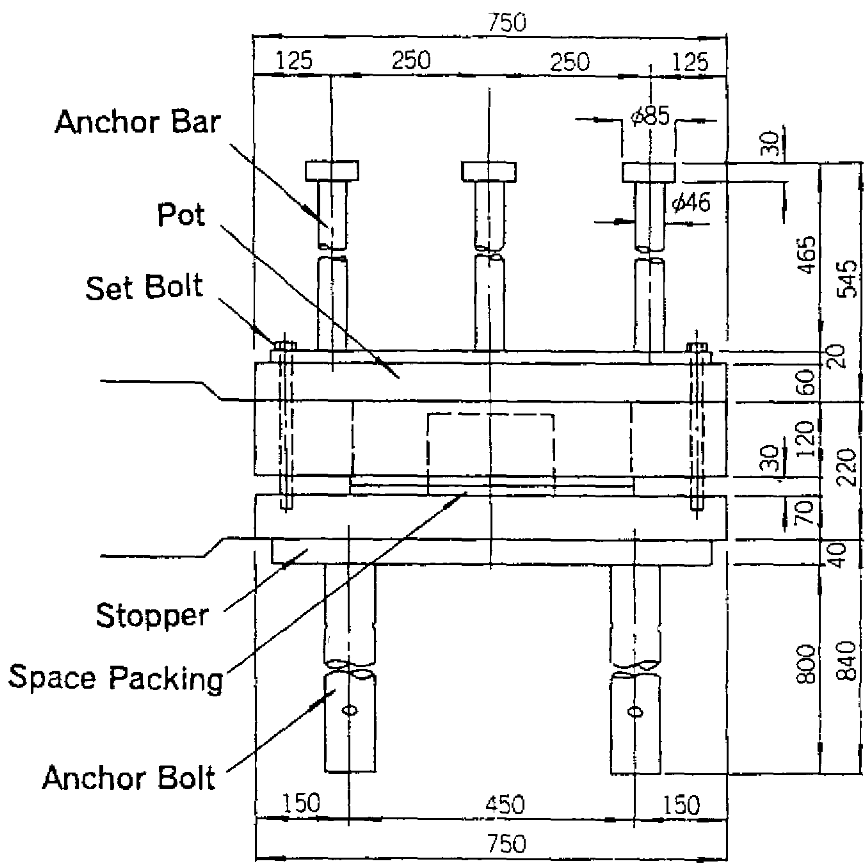

Fig. 9 Stopper

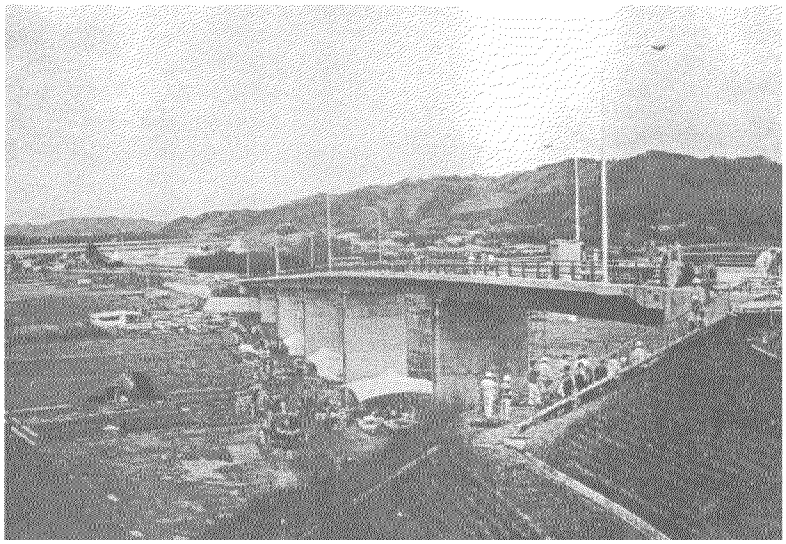

Photo 1 Yama–age Bridge

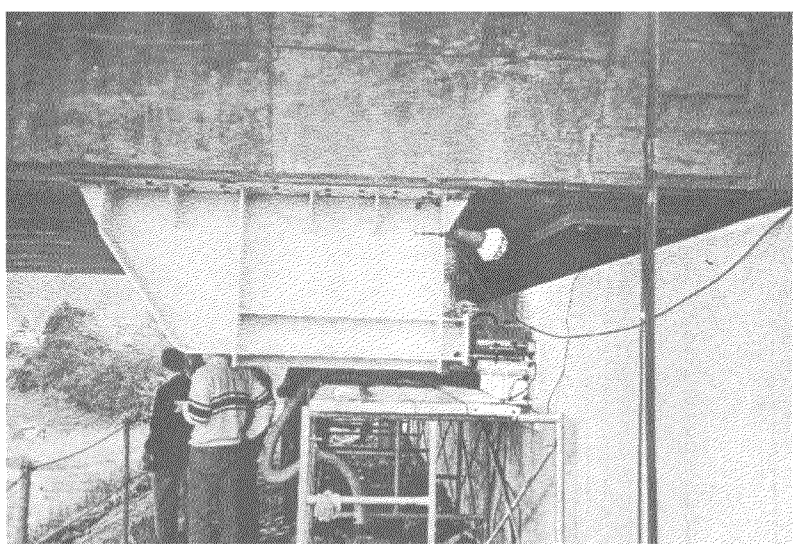

Photo 2 Forced Excitation Test using Quick–Release jacks