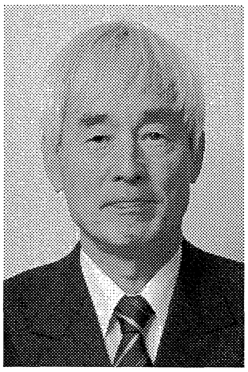
Design and Experimental Performance Evaluation of Lens-type Shear Panel Dampers for Highway Bridge Bearings



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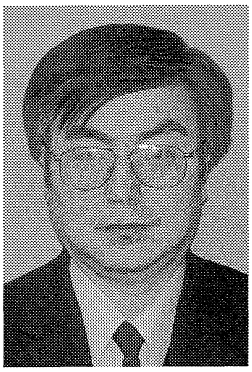

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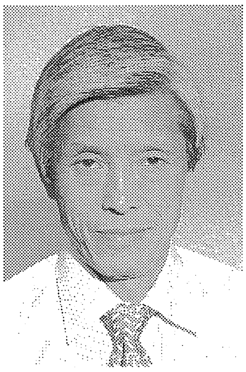

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Abstract

This paper describes lens-type shear damper newly developed for highway bridge bearing[1),2),3),6)]. It utilizes low-yield steel LY100 and concave lens-shape panels. Both properties provide low strength and high ductility which are major requirements for damping devices, then contributes to high energy dissipation against seismic motion. Large deformation due to high speed strain velocity causes steel lens plate heating up to 400~500 centigrade in a moment. Earthquake energy is converted both to strain energy and heat energy. Cumulative deformation capacity of lens identity determines ultimate state of failures associated with strength and life time, dependent on time history of level-2 design earthquake(EQ)[4)]. Fracture is roughly estimated by Miners rule using damage index method. Prediction matches well with testing results. As case study with dampers, dynamic analysis on the existing continuous bridges has been conducted with some design parameters. The base shear acceleration due to level-2 earthquake reduces down to 0.45g～0.5g from 0.8g of lead rubber bearing system. For planning of bridge system with dampers, 1- DOF model is simply useful to roughly know the base shear with dampers at the initial stage of planning only when dead load is known. Design methods and experimental performance evaluation results are reported.

Keywords: Shear Panel Damper, Seismic Design, Bridge Bearing, Dynamic Analysis, Low Yield Steel

1. Introduction

The shear panel damper is developed as a part of function-separated bearing system to serve for lateral seismic loads. The size scale-up ratio from specimen to commercial products is from 0.5 to 0.75 ～ 1.25, proportionally. Stiffness of damper model has great influence on dynamic response associated with resistance versus displacement. Two kinds of damper models, S (stiff)-model and R(regular)-model are specified for design use. The former is for safety evaluation of resistance to design the structural members, and the later is for displacement and fracture to design the damper devices. Large deformation of steel with high speed strain rate falls in crucial fracture problems; one is maximum displacement and traveled pass, and other is cumulative deformation capacity of lens identity. Deformation capacity which is thought to be strain energy capacity, mainly depends on strain rate and magnitude (EQ),stress state and intensity (panel shape) and fracture toughness(LY100).Based upon the fundamentals of lens identity and some design criterion, damage index method and base shear design method are proposed. With some combination of design parameters, several case studies are simulated for their performance evaluation.

2. Lens-type shear panel damper

2-1 Lens-type shear panel damper and half size specimen (Figure-1、Table-1,Table-2)

Figure-1 illustrates the panel details of half size model of prototype for test use. Mechanical properties of shear panel and low yield steel of JFE-LY100 are specified in Table-1and Table-2,both by nominal values. Through a series of experimental works by using half size models, it is found that concave lens shape +low yield steel LY100 provide most effective way to satisfy low strength and high ductility with large energy dissipation[2),6)].

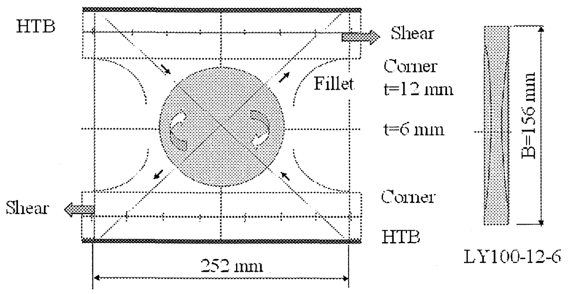


Figure-1 Lens- type shear panel damper: Panel shape and connection

Table-1 Mechanical properties of LY100-12-6

yieldstress (0.2% strain)	80N/mm
yield dislacement (shear strain 3.2%) δy	5mm
yield shearstress $\tau y=\sigma y/\sqrt{3}$	46.2N/mm2
yield strength Qy (at lens center t=6mm)	66.1KN
yield strength Qy (at panel edge,t=12mm)	86.5KN
Max.shear Qmax (at base with fillet)	245KN
Qmax/Qy	2.80～2.87
δmax/δy	8～10

Table -2 Mechanical property of low yield steel (JFE LY100)

steel grade	LY-100
yield strength	80～120 N/mm2
tensile strength	200～300 N/mm2
yield ratio	<60 %
elongation	>50 %
charpy value	27J (at 0°C)

2-2 Lens-type shear panel damper and scale-up products (Figure-2、Table-3)

Based upon the fundamentals of half size model, commercial products are planned to actual service use by scale-up rules. The size scale-up ratio from specimen to commercial products is from 0.5 to 0.75～1.0(full size) ～1.25,proportionally the force scale-up ratio changes from 25tf to 75～100(full size)～150tf per single unit. The mechanical properties and fundamental nominal values for design use are specified in Table-3.It is possible to make thickness of lens panel with LY100 change by 1mm up from 18mm to 30mm. Lens panel name, LY100-t1-t2 means low yield steel of grade 100, thickness t1 at panel edge and t2 at lens center, lens deepness t2/t1 is set up to be 0.5 as optimum size ratio.

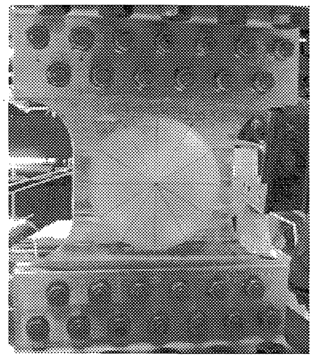


Figure-2 Lens-type shear panel damper: LY100-24-12

2-3 Setting plan to bridge (Figure- 3)

Figure-3 illustrates damper types of single panel and double panels. Double panels are set up with single panel together in parallel, which possesses double capability of single panel. The lower side of panel is tightly fixed to the basement by double array HTB with double angles and the upper side is connected by shear key to the sole plate welded to the bottom flange of bridge. The small clearance at shear key connection allows slight rotation due to live loads and also small slide due to expansion by temperature change. Dampers are available both to simply supported bridge and to continuous span bridge with hinge connection to each pier within limited span length where thermal expansion is well treated.

3 FUNDAMENTALS

3.1 Damper model: Bilinear model with rectangular shape (Figure- 4 , Figure-6)

Figure-4 shows the typical load-displacement hysteretic curves for 30mm constant amplitude under the sinusoidal tests (two cases of slow and T=1sec).The peak load gradually decreases with repeated cycles and the cracking initiates at 7~8 cycles. Figure-6 shows an assumed analytical model, a bilinear model of rectangular shape, where two parameters of Qmax and S1 are defined. The maximum loads, Qmax and Qpeak are determined; Qmax for analytical model denotes the average value of resistance shears, and Qpeak for design use is the highest value among them. Qpeak /Qmax is about 1.04～1.18, both in the static and dynamic tests. S1 is determined from the unloading gradients. On the basis of static and dynamic database, two damper models are proposed.

(1) S-model: Stiff model of hard response. Use for strength design. The values of Qmax-s, Qpeak-s and S1-s are determined to be 245KN, 282KN and 140KN/mm, respectively.

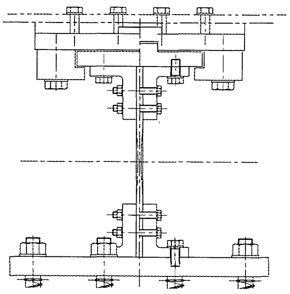
(2) R-model: Regular model of soft response. Use for displacement design and life cycle evaluation

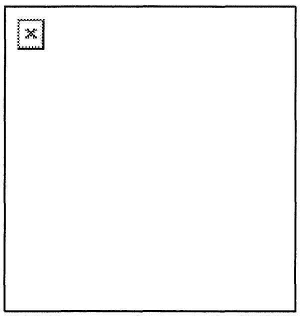
The values of Q max-r, S1-r are set to be 225KN and 134KN/mm respectively, which is equivalent to 92% and 96% values of S-model.

Table-3 Properties of lens-type shear panel dampers: specimen (half size) and scale-up products

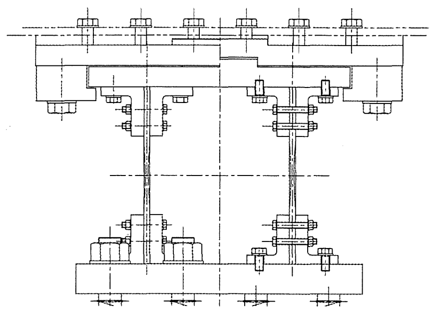
Properties			Symbol	Unit	Specimen	Products				
Specimen & products	standard	product name	strength	tf	25tf	50tf	75tf	100tf	125tf	150tf
		scale	s	t1/24	0.5	0.75	0.875	1	1.125	1.25
		lens-type shear panel	LY100-t1-t2		L-12-6	L-18-9	L-21-10.5	L-24-12	L-27-13.5	L-30-15
Lens	panel size	thickness at edge	t1	mm	12	18	21	24	27	30
		thickness at center	t2	mm	6	9	10.5	12	13.5	15
		diameter	D	mm	130	195.0	227.5	260.0	292.5	325.0
	dimensions	square panel B*B	B	mm	156	234	273	312	351	390
		fillet R	4t1	mm	48	72	84	96	108	120
		thickness ratio	B/t1		13	13	13	13	13	13
Lens properties	strength & displacement	yield strength	Qy	KN	86.49	194.6	264.9	346.0	437.9	540.6
		max./yield ratio	Qmax/Qy		2.83	2.83	2.83	2.83	2.83	2.83
		peak strength	Qpeak	KN	282	635	864	1128	1428	1763
		peak/max. ratio	Qpeak/Qmax		1.15	1.15	1.15	1.15	1.15	1.15
		max.strength (ave.)	Qmax	KN	245	551	750	980	1240	1531
			Qmax	tf	25	56	77	100	127	156
		gradient of unloading	S1	KN/mm	140	210	245	280	315	350
		yield displacement	δy	mm	5	7.5	8.8	10.0	11.3	12.5
	design limit	limit of disp.(max.)	Dmax=7δy	mm	35	52.5	61.25	70	78.75	87.5
		limit of disp.(peak)	Dpeak=8δy	mm	40	60	70	80	90	100
		limit of damage pass	Dtp*	mm	800	1200	1400	1600	1800	2000

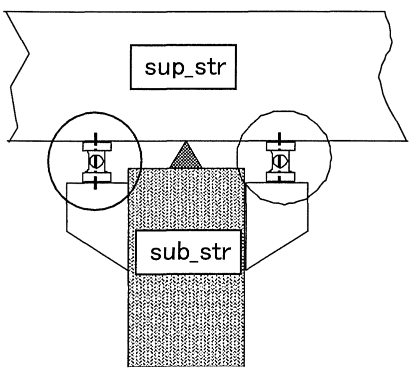
Single type (LY100-24-12)





Double type (2LY100-24-12)





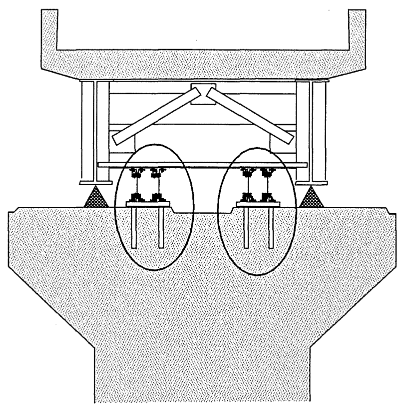


Figure-3 Lens-type shear panel dampers: single and double types: Bearing+Damper

Table-4 Dynamic response with S-model and R-model: Testing, analysis and design reviews

test	damper model		EQ2-2-1	random loading test results			analysis by output data		Effects of f=Qmax/Qo and s to response					
case[5]	type	shear	disp.scale	Max.disp	trav.pass	life cycles	damage pass	life cycles	shear ratio	Max.disp	trav.pass	d.pass	life cycles	
	stiffness	Qmax	s	D	Dtp	(c1+c2)/2	Dtp*	Nf	f	$D.f^2$	$Dtp.f^2$	$Dtp^*.f^4$	Nf/f^4	$Nf.s^2/f^4$
E1	S	245	1	27.6	272	6	125	6.40	1	27.6	272	125	6.40	6.40
E8	R	225	1	33.6	325	4.5	183	4.37	0.918	28.3	274.1	130.2	6.15	6.15
E4	S	245	1.2	33.1	332	4.5	183	4.37	1	33.1	332.0	183.0	4.37	6.30
E7	R	225	1.2	39.2	390.1	3	263	3.04	0.918	33.1	329.0	187.1	4.28	6.16

Nf=800/Dtp*, c1:cycles at crack intiation, c2:cycles at failure, s: displacement amplification factor of EQ2-2-1loading

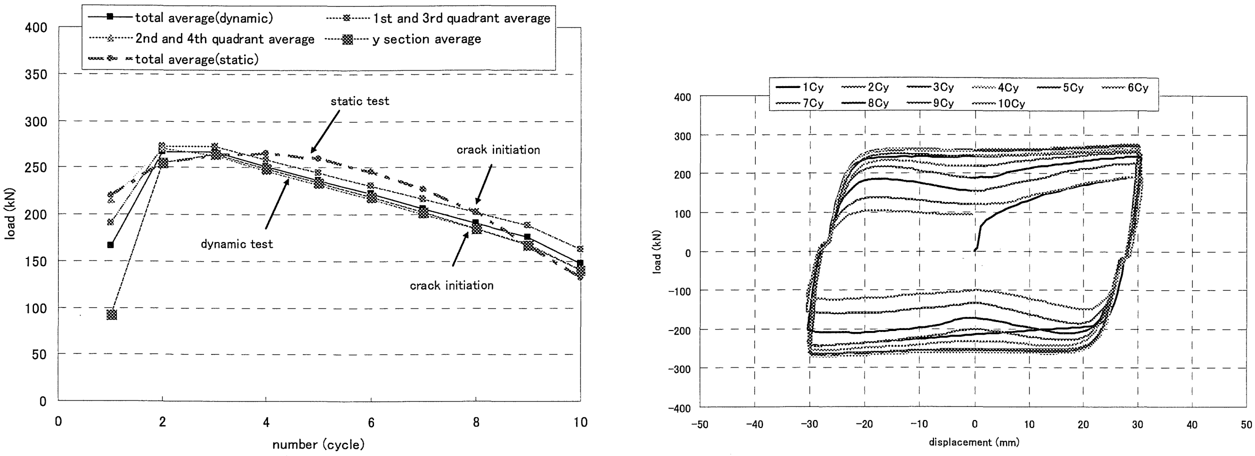


Figure-4 Load versus Displacement and Cycles (by sinusoidal wave test, slow and T=1sec,amplitude 30mm)

Table-5 Gradually incremetal loading tests : cumulative deformation and design limit

mm

Loading	Amplitude	Trav.pass	Damage index (1/Nf)method			Damage pass (Q)method			
	x(mm)	Σ(4x)	$Nf=15100/4x^2$	1/Nf	Σ(1/Nf)	e=x/18.875	e*x	Q=Σ(4e*x)	P=Q/800
δy	5	20	151.0	0.007	0.007	0.265	1.32	5.3	0.007
2δy	10	60	37.8	0.026	0.033	0.530	5.30	26.5	0.033
3δy	15	120	16.8	0.060	0.093	0.795	11.92	74.2	0.093
4δy	20	200	9.4	0.106	0.199	1.060	21.19	158.9	0.199
5δy	25	300	6.0	0.166	0.364	1.325	33.11	291.4	0.364
6δy	30	420	4.2	0.238	0.603	1.589	47.68	482.1	0.603
7δy	35	560	3.1	0.325	0.927	1.854	64.90	741.7	0.927
8δy	40	720	2.4	0.424	1.351	2.119	84.77	1080.8	1.351
9δy	45	900	1.9	0.536	1.887	2.384	107.28	1509.9	1.887
design limit		900			D1<1			800	D2<1

Table-6 Resistance versus displacement: Qpeak, Base shear, Damper model

KN,g

Item	Resistance	Response	Qmax,Qpeak	f=(Q/Qo)	$(Dtp)\ 1/f^2$	$(Dtp^*)\ 1/f^4$
Qpeak	Disp. control loading KN	hard	245∼282	1	1	1.000
(test results)	Force control loading KN	soft	225∼258	0.912	1.203	1.448
Base shear	Base shear acc.0.44g	hard	0.44	1.000	1	1.000
(design)	Base shear acc.0.40g	soft	0.4	0.909	1.235	1.525
Damper model	S-model Qmax KN	hard	245	1.000	1	1.000
(analysis)	R-model Qmax KN	soft	225	0.918	1.186	1.407

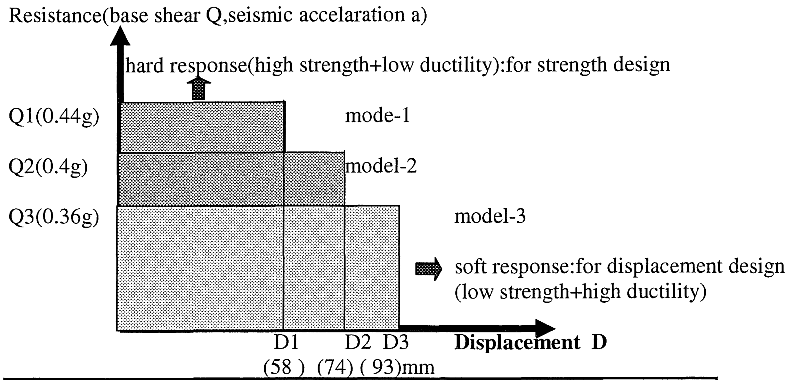
Qpeak/Qmax=1.15 f: base shear ratio

Table-7 Results of dynamic analysis:1-D.O.F

KN, mm, KN/mm

Level-2 EQ	Damper	Panel	Scale	Qmax(KN)	S1	W(KN)	Qmax/W	Qpeak/W	Max.disp	Min.disp	Dtp*	Df	Nf=1/Df
EQ2-2-1	S-model	L-23-11.5	0.958	900.0	268.3	2450	0.367	0.422	9.7	-123.4	425.8	0.278	3.600
		L-24-12	1.000	980.0	280.0	2450	0.400	0.460	7.7	-89.1	239.3	0.150	6.686
		L-25-12.5	1.042	1063.4	291.7	2450	0.434	0.499	5.6	-60.7	136.6	0.082	12.207
	R-model	L-23-11.5	0.958	828.0	257.6	2450	0.338	0.389	16.4	-146.9	682.1	0.445	2.247
		L-24-12	1.000	901.6	268.8	2450	0.368	0.423	9.6	-119.9	405.1	0.253	3.950
		L-25-12.5	1.042	978.3	280.0	2450	0.399	0.459	7.9	-89.1	230.8	0.138	7.223
EQ2-2-2	S-model	L-23-11.5	0.958	900.0	268.3	2450	0.367	0.422	25.5	-54.3	148.6	0.097	10.316
		L-24-12	1.000	980.0	280.0	2450	0.400	0.460	20.7	-51.1	106.2	0.066	15.062
		L-25-12.5	1.042	1063.4	291.7	2450	0.434	0.499	7.0	-44.2	75.9	0.046	21.968
	R-model	L-23-11.5	0.958	828.0	257.6	2450	0.338	0.389	11.9	-70.1	202.1	0.132	7.585
		L-24-12	1.000	901.6	268.8	2450	0.368	0.423	25.4	-54.4	141.4	0.088	11.318
		L-25-12.5	1.042	978.3	280.0	2450	0.399	0.459	21.1	-51.2	102.3	0.061	16.300
EQ2-2-3	S-model	L-23-11.5	0.958	900.0	268.3	2450	0.367	0.422	48.7	-68.7	260.0	0.170	5.896
		L-24-12	1.000	980.0	280.0	2450	0.400	0.460	69.9	-36.0	184.9	0.116	8.653
		L-25-12.5	1.042	1063.4	291.7	2450	0.434	0.499	59.2	-17.4	112.4	0.067	14.839
	R-model	L-23-11.5	0.958	828.0	257.6	2450	0.338	0.389	78.4	-70.0	381.2	0.249	4.021
		L-24-12	1.000	901.6	268.8	2450	0.368	0.423	48.3	-68.6	247.5	0.155	6.465
		L-25-12.5	1.042	978.3	280.0	2450	0.399	0.459	69.6	-36.8	178.5	0.107	9.338

Qpeak/Qmax=1.15



	Base shear	Results of analysis	Estimate by f=Q/Q2	
model	a=Q/W(g)	max.D(mm)	f	$D2/f^2$
model-1	0.44	58	1.1	61.2
model-2	0.4	74	1	74
model-3	0.36	93	0.9	91.4

Figure-5 Concept of base shear design: Hard response and Soft response (Analysis by the 3-continuous span bridge, Figure-7, 8)

3.2 Dynamic response with S-model and R-model: Testing, analysis and design reviews (Table-4)

Table-4 shows the analytical and testing results on the 3-continuos span bridge (Figure-7,8) with S-model and R-model dampers, subjected to level-2 EQ-2-2-1. When base shear ratio f=Qmax-r/ Qmax-s is given to be f=0.918, E1/E8 of max.displacement, traveled pass of moving distance are roughly estimated proportionally to $1/f^2$. When damper stiffness becomes soft, displacement increases as much as double of scale factor f. E4,E7model with EQ amplification factor s=1.2 shows same tendency as E1,E8 model. In both cases, Qmax is kept in constant without changing. Increase in EQ amplification factor s and decrease in stiffness Qmax of dampers causes increase in displacement, dependent on s and f values, where s is equivalent to f^2 as response sensibility factor.

3.3 Cumulative deformation capacity (CDC) and Damage index method [6)]

The displacement capacity which has strong relation to the strain energy capacity depends greatly on the strain rate and seismic magnitude (EQ), the stress states and intensity (panel shape), and the fracture toughness (LY100).As a performance indicator, the cumulative displacement capacity Cdc is used for their evaluation. The relationship between the cumulative displacement capacity (y and Cdc) to the wave amplitude (x) is determined by the experimental results which deal with CDC and the number of cycles to failure Nf versus constant wave amplitude x (5, 10, 15, 20, 30, and 40 mm)[6)].

$$y = 17497x^{-1.0848} \tag{1}$$

$$xy = 15100 \tag{2}$$

Eq.(1) is derived from the test data through regression analysis, and Eq.(2) is a simplified hyperbola of Eq.(1) showing x times y is equal to constant which characterizes lens identity. Based on Miner's rule, Nf and damage accumulated in each cycle Df are given by Eqs.(3),and(4), respectively.

$$N_f = 15100/4x^2 \tag{3}$$

$$D_f = 1/N_f \tag{4}$$

Miner's rule gives the design criteria to failure by Eq. (5).

$$D_1 = \sum (1/N_f) < 1 \tag{5}$$

Nf is referred to number of cycles to failure which indicates life cycles. When a damper is subjected to a harmonic motion with a specified amplitude x=18.875mm, then Cdc is determined to be y=800mm by Eq.(2).The specified amplitude x is set up to be about one half (average value) of maximum response displacement ,7.5δy(37.5mm.).

By using the analytical data of traveled pass Dtp, the damage pass Dtp* is defined by Eq.(6).

$$Dtp^* = \sum (\text{damage pass coefficient e}) \cdot (\text{response amplitude x}) = \sum (4x^2/18.875) \tag{6}$$

where e=x/18.875and Cdc=800mm. Safety of D2 can be evaluated by Eq.(7).

$$D_2 = \sum (Dtp^*/800) < 1 \tag{7}$$

CDC can be evaluated by the two methods: 1) Damage index method by Eq. (3), (4), and (5),and 2) Damage pass method by Eq.(6) and (7). Both results give the same answer exactly, because they stand on the same base of Eq.(2). Damage index method has an advantage to evaluate the damage state without determination of cumulative damage pass limit (Cdc). Table-4 shows count-up data cf=(c1+c2)/2 recorded by tests and Nf by Eq(5).Prediction value Nf matches well with testing data cf. Another trial simulation is shown in Table 5.

3.4 Gradually increased displacement tests and evaluation of CDC: design criteria (Table 5)

Table 5 shows the test results for gradually increased displacement history and evaluation of CDC by damage index method and damage pass method. At 7δy, the cumulative damage D1=Σ(1/Nf) becomes 0.927, that is, the D1 value is close to 1 indicating almost failure. In the static test, the max. displacement counts up to 9δy with traveled pass 900mm. In the dynamic test, the estimated max. displacement is reduced to 7δy, where the damaged traveled pass is 741mm, that is, a little below the cumulative displacement limit value of 800mm.

Design criterion can be safely proposed that Ds (static max. displacement), Dd (dynamic max. displacement), Dtp*(damage pass), can be determined to be less than 45mm (9δ y), 35mm (7δy), 800mm, respectively.

4. BASE SHEAR DESIGN: SEISMIC DESIGN WITH DAMPERS

4-1 Outline

Ductility capacity is evaluated in terms of cumulative plastic strain. The effects of dynamic loading were examined in reference to the maximum resistance and ductility capacity on the basis of the experimental works. Two different design approaches are considered in base shear design, one aims at controlling the maximum shear forces transmitted by the dampers to the pies, while the other at controlling the displacement. The former is associated with pier strength and design of the structural members, and the later is associated with ultimate displacement capacity, the post-EQ remaining capacity of life cycles and available joint gaps. The design approaches are empirical based on experimental database. Several design factors are defined as follows:

Basic seismic acceleration
a=Q (base shear force) Q/ W(dead load)
Modified seismic acceleration
a*=q.a, q: resistance factor
Base shear ratio:
f=Q (base shear force) / Qo(basic base shear force)
Displacement ratio:
g= D (displacement)/ Do (basic displacement)

In base shear design with use of shear panel dampers, those factors are correlated with each other.

4-2 Base shear design: Strength design and displacement design (Figure-5)

Concept of base shear design is shown in Figure-5 as resistance (base shear force Q, seismic acceleration a=Q/W) versus displacement. Model-1 indicates hard response with high strength and low ductility, which is for structural design use. Reversely, model-3 indicates soft response with low strength and high ductility, which is for displacement design use. In case of the damper with bilinear model of rectangular shape subjected to random loading, Qmax is kept in constant, whereas displacement changes .Figure-5 also shows results of dynamic analysis on 3 cases with Qmax parameters (0.36g, 0.40g, 0.44g).Maximum displacement D is roughly scaled by 1/f2, where f=0.9,1.0,1.1, respectively.

4-3 Resistance versus displacement: Hard response and soft response (Table-6)

Table-6 shows resistance versus displacement, associated with laboratory testing methods, base shear design and damper models. Each case has hard response with high strength+low ductility and soft response with low strength+high ductility.

Qpeak /Qmax: It is dependent on laboratory testing methods. When dynamic loadings are imposed by displacement control method and force control method, Qpeak falls in different value. The displacement control method restrains input-output displacements by actuators, consequently, response reactions causes resistance changes, reversely the force control method by the facility of turn table, response reactions causes displacement change. Actual responses at site are considered to be close to soft response with semi rigid boundary. Dependent on connection rigidity, resistance factor q= Qpeak/Qmax changes from 1.04 to 1.15.

4-4 Qpeak,Base shear,Damper model : Correlation

with base shear ratio f (Table-6)

Each base shear difference in Table-6 is treated by the same parameter f. Each item has the same level of scale-up factor f=0.9.For safety at design, resistance force and displacement should be evaluated equivalently by the different damper models, S-model and R-model, respectively.

4-5 Base shear design: Design coefficients and design criterion: Qpeak design by Qmax analysis (Table-3)

In principle, by two types of damper models, dynamic analyses should be simulated for strength design and displacement design. Results are modified by several design coefficients (LY100-12-6).

1) Damper model factor (S-model, R-model)
    f=Qmax-s/Qmax-r =245/225 =1.089
2) Displacement amplification factor of EQ
    s=1.0~1.2
3) Resistance factor
    q=Qpeak/Qmax,=1.04~1.15
4) Peak displacement (by static tests)
    Dpeak=8δ y =40mm,
   Max. displacement (by dynamic tests),
    Dmax=7δ y =35mm
   When Dtp* is within the allowable limit 800mm,
    Dpeak/Dmax=1.15 is allowed.
5) Damage index:
    Df=1/Nf<1 (at ultimate state),
    Df<1/3(at service use)
6) Damage pass
    Dtp*<800mm (at ultimate state)
    Dtp*<800/3 mm (at service use)
7) Life cycles (number of cycles to failure)
    Nf>1(at ultimate state)
    Nf>3 (at service use)

4-6 Dynamic analysis by 1-D.O.F : Base shear design by 1-D.O.F model (Figure-6,Table-7)

Figure-6 illustrates 1-D.O.F model for design simulation. For design use, several parameters are considered.

a) Lens panel size: LY100-23-11.5,LY100-24-12,LY100-25-12.5

b) Basic seismic acceleration a=0.338~0.434g, Modified seismic acceleration a*=0.4~0.5g

c) Damper model: S-model, R-model

d) Level-2 EQ: EQ2-2-1.EQ2-2-2, EQ2-2-3 [4)]

For each case with design parameters combination, maximum displacement D, traveled pass Dtp, damage Dtp* and life cycles Nf are shown in Table-7, for design use. Basic seismic acceleration a=0.4~0.5g determines critical values of maximum displacement D and damage pass Dtp*. Nf changes widely from 2.25 to 21.97, dependent on level-2 EQ. In design, average values of 3 waves are evaluated for safety margin.

4-7 Displacement design: Evaluation of D, Dtp*, Nf by R-model (Table-4, Table-8)

Table-8 shows displacement D, traveled pass Dtp and damage pass Dtp* of 1-D.O.F model, based on the results of dynamic analysis (Table-7).It is important to analyze and pick up the wave amplitudes correctly and exactly from the random time history response. An amplitude of random vibration wave is so determined to be the distance between a top point of velocity zero and a bottom point of velocity zero where the wave velocity returns reversely that velocity response curves are required together with displacement response curves to analyze the data correctly. Dtp is the moving distance of response in which noise are cut off, whereas Dtp* is damaged distance which are proportional to squre of each amplitude. It is clear that big difference of Dtp* exists between EQ2-2-1,EQ2-2-2 and EQ2-2-3,and S-model and R-model. Average values of Dtp* with S-model are 277,110,186mm for EQ2-2-1,EQ2-2-2 and EQ2-2-3,respectively.Average values of 3 waves are188,285mm for S-model and R-model..

Table-8 shows effects of base shear ratio f to displacements. When the base shear ratio f (Qmax/Q0) is given, dynamic responses of displacement D, traveled pass Dtp, damage pass Dtp* are estimated to be proportional to $1/f^2$,$1/f^2$ and $1/f^4$,respectively.

Table-4 shows effects of f and s to response. In each case of E1,E8,E4,E7, $Nf.s^2/f^4$ converges to the original value of Nf=6.40 of E1,where f=1,s=1.

5. NUMERICAL ANALYSIS: 3-CONTINUOUS SPAN BRIDGE

5-1Analytical model:3 continuous span bridge, superstructure+pier+foundation(Figure-7,Figure-8)

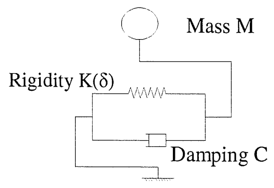
An analytical model with steel bridges, steel pylons of concrete casting inside and steel piles is illustrated in Figure-7.Dimensions, member properties and dead weight are roughly described[5)]. For case studies, 3 types of bearing, elastomeric bearing (EB),base isolation bearing(IB)and damper bearing(DB) are prepared with four sets for each support. Linear or bilinear models of each bearing are shown in Figure-8.The bridge is supported by bearings with hinge connection against seismic forces.

5-2 Case study-1: Bearing types and damping effects (Elastomer, Base isolator, damper) (Figure-9)

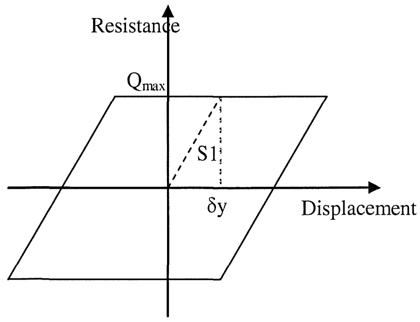
Case-1 (Elastomer): Conventional bearing system provides large displacement of 196mm(at P2) and large base shear acceleration of 0.79g almost without damping effect.

Case-2 (Isolator): Base isolation system provides large displacement of 261mm (at P1), 161mm(at P2) and reduced lateral forces of 0.602g as counter effects.

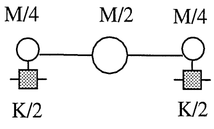
Case-3 (Elastomer (P1,P4)+damper(P2,P3)): It is combined use with EB and DB (LY100-27-13.5), movable at end supports (P1,P4) due to temperature expansion. It provides small displacement of 43mm (at P2, P3) and 155mm (at P1, P4), totally reduced base shear acceleration of 0.496g at P2.



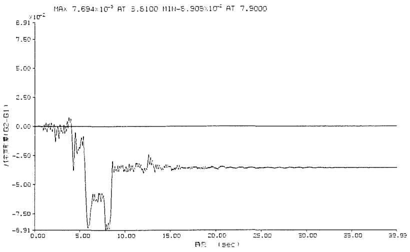
1-DOF damper model



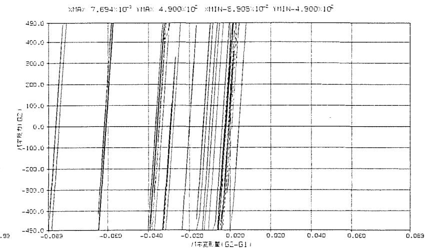
Bilinear model with rectangular shape



Analytical model



Time history of displacement



Time history of resistance

Figure-6 Analytical model of 1-DOF and response

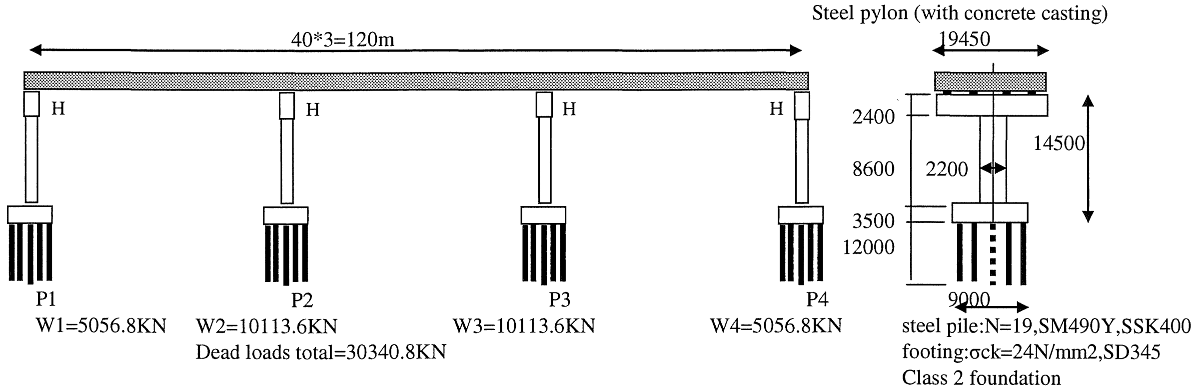
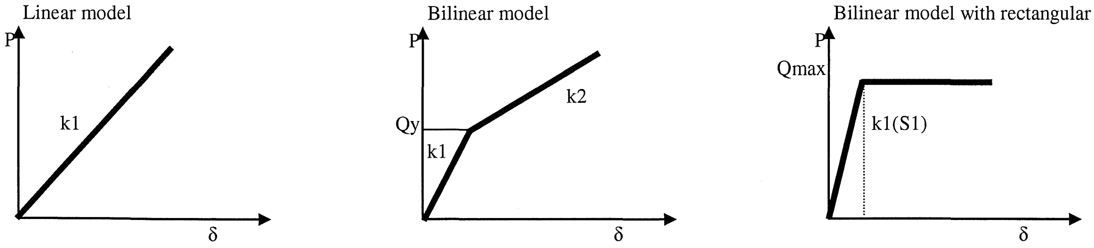


Figure-7 Bridge model for analysis: 3 continuous span bridge (width 19.45m)

Elastomer bearing(EB) | Base isolation bearing(IB) | Lens type shear panel damper(DB)

Linear model | Bilinear model | Bilinear model with rectangular shape



K1=10043KN/m
N=4*4

P1,P4: k1=28580/m,K2=2707KN/m,Qy=232KN
P2,P3: k1=750520/m,K2=7129KN/m,Qy=436KN

k1=280000KN/m,Qmax=980KN(S-model)
k1=268000KN/m,Qmax=900KN(R-model)
(LY100-24-12)

Figure-8 Analytical model of bearings

Table-8 Results of dynamic analysis (1-D.O.F): Displacement D,Dtp, Dtp* by EQ2-2-1,EQ2-2-2,EQ2-2-3 (mm)

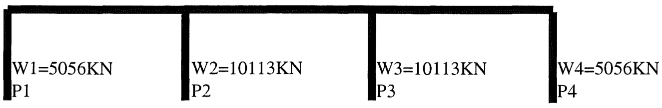
Effects of base shear ratio f to displacements, and average of 3 waves [4)]

mm

	Damper				Results by dynamic analysis				Effects of base shear ratio f to displacements			
Level-2 EQ	Damper	panel	scale	Qmax(KN)	D(amplitude)	Dtp	Dtp*	Dtp*/Dtp	f=Qmax/Q0	f^2.Dtp	f^4.Dtp*	f^2.D
EQ2-2-1	S-model	L-23-11.5	0.958	900.0	66.6	952.5	425.8	0.447	0.918	803.4	302.9	56.1
		L-24-12	1.000	980.0	48.4	882.8	239.3	0.271	1.000	882.8	239.3	48.4
		L-25-12.5	1.042	1063.4	33.1	848.5	136.6	0.161	1.085	999.0	189.3	39.0
		Average	1.0	981.1	49.3	894.6	267.2	0.293	1.00	896.7	243.9	47.8
	R-model	L-23-11.5	0.958	828.0	81.6	1105.5	682.1	0.617	0.918	932.2	485.0	68.8
		L-24-12	1.000	901.6	64.7	948.5	405.1	0.427	1.000	948.3	404.9	64.7
		L-25-12.5	1.042	978.3	48.5	874.6	230.8	0.264	1.085	1029.5	319.8	57.1
		Average	1.0	902.6	65.0	976.2	439.3	0.436	1.00	978.2	403.2	63.6
EQ2-2-2	S-model	L-23-11.5	0.958	900.0	39.9	528.7	148.6	0.281	0.918	445.9	105.7	33.7
		L-24-12	1.000	980.0	35.9	490.3	106.2	0.217	1.000	490.3	106.2	35.9
		L-25-12.5	1.042	1063.4	25.6	442.2	75.9	0.172	1.085	520.7	105.2	30.1
		Average	1.0	981.1	33.8	487.1	110.2	0.223	1.00	488.2	105.7	33.2
	R-model	L-23-11.5	0.958	828.0	41.0	603.6	202.1	0.335	0.918	509.0	143.7	34.6
		L-24-12	1.000	901.6	39.9	527.1	141.4	0.268	1.000	527.0	141.3	39.9
		L-25-12.5	1.042	978.3	36.2	481.4	102.3	0.212	1.085	566.6	141.7	42.6
		Average	1.0	902.6	39.0	537.4	148.6	0.272	1.00	538.5	142.2	39.0
EQ2-2-3	S-model	L-23-11.5	0.958	900.0	58.7	763.3	260.0	0.341	0.918	643.8	184.9	49.5
		L-24-12	1.000	980.0	52.9	710.1	184.9	0.260	1.000	710.1	184.9	52.9
		L-25-12.5	1.042	1063.4	38.3	656.3	112.4	0.171	1.085	772.7	155.7	45.1
		Average	1.0	981.1	50.0	709.9	185.7	0.257	1.00	711.5	175.2	49.2
	R-model	L-23-11.5	0.958	828.0	74.2	871.4	381.2	0.437	0.918	734.8	271.1	62.6
		L-24-12	1.000	901.6	58.5	759.1	247.5	0.326	1.000	759.0	247.4	58.5
		L-25-12.5	1.042	978.3	53.2	699.8	178.5	0.255	1.085	823.8	247.4	62.6
		Average	1.0	902.6	61.9	776.8	269.1	0.340	1.00	778.4	255.3	61.2
Average of 3 waves	S-model		1.0	981.1	44.4	697.2	187.7	0.258	1.0	698.8	174.9	43.4
	R-model		1.0	902.6	55.3	763.4	285.7	0.349	1.0	765.0	266.9	54.6

D=(max.disp+min.disp)/2

3-continuous span bridge model(40m*3=120m) (Figure7,8)



total weight W=30340KN

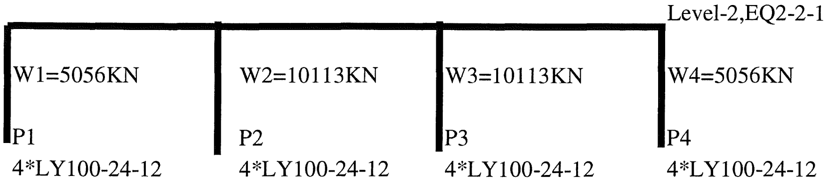
mm

	Bearing		P1	P2	P3	P4	Amax(g)	Apeak(g)	remarks
Case-1	Elastomer	number	4EB	4*EB	4*EB	4*EB			convensional method
	EB	disp.	196	196	197	195	0.789		soft rigidity+large displacement
Case-2	Isolator	number	4*IB	4*IB	4*IB	4*IB			base isolator system
	I B	dis.	261	161	161	271	0.602		long period+large displacement
Case-3	ED+DB	number	4*EB	4*DB	4*DB	4*GB			combined use,movable at end support
	ED+DB	disp.	155	43	43	155	0.496	0.572	due to temperature expansion
Case-4	Damper	number	4*DB	4*DB	4*DB	4*DB			damper system,small disp with large
	DB	disp.	64	55	59	53	0.388	0.446	energy dissipation

Damper (DB with S-model):LY100-27-13.5(case3),LY100-24-12(case-4) Amax,Apeak:seismic accelaration at P2

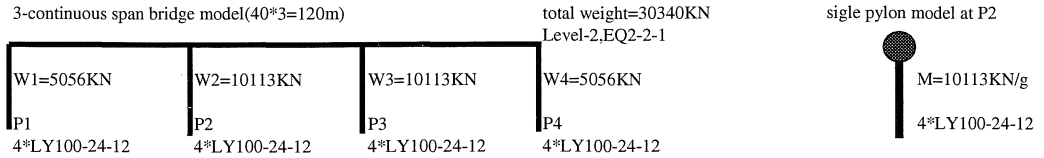
Figure-9 Results of dynamic analysis with various types of bearings
Case study 1: Comparison with elastomer (EB), base isolator (IB) and damper (DB)

3-continuous span bridgemodel(40*3=120m) total weight=30340KN
Level-2,EQ2-2-1



Foundation	bearing	Displacement	m		Bending M at pilon		Resist.	Seismic acc.
Class(1,2,3)		girder	bearing	pilon top	At base	curvature	Q(KN)	a=Q/W2
hard rock	Elastomer	0.285	0.17	0.158	67485	0.0126	6830	0.675
(class-1)	damper	0.136	0.051	0.096	62300	0.0056	3920	0.388
medium layor	Elastomer	0.425	0.196	0.284	68593	0.0142	7891	0.780
(class-2)	damper	0.167	0.055	0.138	63027	0.0066	3920	0.388
soft layor	Elastomer	0.417	0.177	0.27	67570	0.0128	7100	0.702
(class-3)	damper	0.213	0.048	0.176	64157	0.0081	3920	0.388

Figure-10 Case study-2 : Foundation rigidity (Class-1, 2, 3 foundations): response at P2



m, KN*m, KN

		EQ1-2-1		EQ1-2-2		EQ1-2-3		EQ2-2-1		EQ2-2-2		EQ2-2-3	
		T	S	T	S	T	S	T	S	T	S	T	S
Disp.	girder	0.105	0.107	0.108	0.112	0.106	0.176	0.167	0.32	0.165	0.229	0.174	0.259
	bearing	0.021	0.023	0.031	0.036	0.025	0.085	0.055	0.108	0.038	0.086	0.053	0.089
	pilon top	0.092	0.095	0.105	0.104	0.09	0.109	0.138	0.233	0.147	0.188	0.14	0.187
B.moment	at base	59727	60140	60922	61001	59617	61513	63027	67873	63154	65770	63307	65718
	curvature	0.002	0.0026	0.0037	0.0038	0.0019	0.0045	0.0066	0.0132	0.0067	0.0103	0.0069	0.0102
Resistance	Q	3920	3920	3920	3920	3920	3920	3920	3920	3920	3920	3920	3920
	B.S.acc	0.388	0.388	0.388	0.388	0.388	0.388	0.388	0.388	0.388	0.388	0.388	0.388

T:analysis by 3-continuos span bridge model (responce at P2), S:analysis by single pilon model(simplified model)

Figure-11 Case study-3 :Dynamic analysis by exact model and simplified model

Case-4(Damper):Damper system(LY100-24-12) provides small displacement of 55mm(at P2) and reduced base shear acceleration 0.388g(at P2) with large energy dissipation. Four dampers arrangement at P1, P4 contributes to base shear reduction at P2,P3 with desirable seismic loads distribution.

5-3 Case study-2: Foundation rigidity （Class1,2,3), 0n soft layer and hard rock (Figure-10)

Foundation rigidity classes are provided by design code, Class1 (hard rock),class2 (medium layer) and class3(soft layer). Analytical results of bridges with dampers (DB) and elastomers (EB), displacement, bending moment and resistance at p2 are shown in Figure-10. Displacements at girders and pylon tops vary from 136,167,213mm, and 96, 138, 176mm, respectively, proportionally to rigidity change from hard to soft foundation. On the contrary, displacements at dampers are almost kept in constant about 48~55mm, with the same resistance .Since damper stiffness is relatively rigid more than that of piers and foundation, dynamic sensibility to foundation rigidity is thought to be substantially small.

5-4 Case study-3: Dynamic analysis by exact models and single pylon model (Figire-11)

At the initial stage of damper plan, rough estimate design methods are required in a global sense.

Figure-11 compares the exact analytical results with rough estimate by use of simplified model, subjected to level-1(EQ1-2-1,EQ1-2-2,EQ1-2-3) and level-2(EQ2-2-1,EQ2-2-2,EQ2-2-3) design EQ in the codes. A simplified model is created at P2 partially, in a form of simply cantilever column, independent from other portions. When subjected to level-1 EQ, no difference is observed between exact and simplified model, on the other hand level-2 EQ makes big difference in about twice displacement. When subjected to level-2 big EQ , total seismic base shear is shared by each supports equally, and seismic loads are distributed without concentration to rigid piers. Even though a simplified model provides rough estimate with safety side, finally exact analysis will be required.

6. CONCLUSIONS

1. Lens-type shear panel damper is developed as a part of function-separated bearing system to serve for lateral seismic loads, and it provides easy maintenance with panel parts change once being damaged.
2. Base shear design method is proposed based on damper model with bilinear model with rectangular shape. A simple model of 1-D.O.F provides principal and practical data to design use. Base shear acceleration of the bridge with shear dampers goes down to 0.4~0.5g from 0.78g of elastomeric bearings and from 0.6g of base isolation system.
3. Large deformation of low-yield steel with high speed strain rate causes two crucial problems;
1) cumulative deformation capacity against fracture, and
2) energy dissipation by heat transfer.
Base shear design should evaluate resistance versus displacement and life cycles precisely for safety and serviceability.
4. Modified seismic acceleration design (MSAD) methods is simply proposed based on the dampers identity of bilinear model with rectangular shape. MSAD is composed of two parts: strength design for structural members and displacement design for fracture evaluation of the dampers..

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