OPTIMUM SEISMIC DESIGN OF STEEL BRIDGE SYSTEM WITH DOUBLE I-TYPE GIRDERS

Kazuhiro Taniwaki
Professor, Department of Architecture and Environmental Eng., Fukui University of Technology, Gakuen 3-6-1,Fukui 910-8505, Japan
Email: taniwaki@fukui-ut.ac.jp
Kenta Tanaka
Chief Examiner, Chodai Co. Ltd., ootemachi 2-8-4, Naka-ku, Hiroshima, 730-0051, Japan
Email: tanaka-k@chodai.co.jp

Abstract

Recently in Japan, long span steel bridge systems with double I-type girders have been constructed for the reasons of reductions of construction cost and time of construction work, and simplicity of maintenance. This types of bridges have small numbers of girders, therefore, it is inherently difficult to determine the sizes of member elements, such as rubber bearing, RC pier and concrete pile foundation, so as to satisfy the ultimate performance constraints due to devastating earthquakes. In this study, the optimum design method is developed for the seismic design of steel bridge system with double I-type girders in which the design of experiments is successfully utilized in order to estimate the dynamic behaviours and sensitivities of the ultimate performance constraints with respect to the design variables. The optimum solutions of the heights of rubber bearings, cross-sectional dimensions and amount of steel reinforcements for RC piers and the detail of concrete pile foundation are determined by a classical branch and bound method. From the numerical example of five-span continuous steel bridge system with double I-type girders, it is emphasized that the optimum solutions can be obtained quite efficiently by the proposed design method. The effectiveness and efficiency of the optimum design method is also illustrated.

Keywords: optimum seismic design, steel bridge system with double I-type girders, Performance-based design, design of experiments, time-history response analysis

1. INTRODUCTION

In the seismic design of bridge according to the Japanese Specification for Highway Bridges, JSHB (Japan Road Association 2002), the bridge members are not allowed to yield for the frequent earthquakes and those must have the sufficient ultimate dynamic capacities to be able to repair those rapidly after the excitations due to devastating earthquakes.

In the design of bridge system, engineers must consider life cycle cost including initial construction cost and maintenance cost, and simplicity of maintenance. From this viewpoint, long span steel bridge systems with double I-type girders have been constructed recently in Japan. In this type of bridge system the volumes of rubber bearings under the main girders are regulated, therefore the determinations of each member sizes, which ensure the performance specified at the ultimate state due to devastating earthquakes, accompany with difficulty in the seismic design.

In this paper, a rational and efficient optimal performance-based seismic design method is proposed for the bridge system with double I-type girders subjected to devastating earthquakes. The optimization problem is formulated in order to find the heights of rubber bearings, cross-sectional dimensions and amount of steel reinforcements for RC piers, and numbers of piles and the diameters of piles in the cast-in-place concrete pile foundation which minimize the total construction cost.

The design constraints are the relative horizontal displacements for the rubber bearings and the ductile factor for the RC piers and the constraint on the cast-in-place concrete pile foundation specified in JSHB. From the practical design the heights of rubber bearings can take continuous values, but the other variables must be selected from discrete variable sets. The optimization algorithm for the mixed discrete-continuous problems is developed in cooperation with classical branch and bound method (Huang and Arora 1997), dual algorithm and convex approximation (Fleury and Braibant 1986, Taniwaki and Ohkubo 2004), and the design of experiments (Taguchi 1987) in this study. The design of experiments is successfully utilized in order to estimate the dynamic behaviours and sensitivities of the ultimate performance constraints without any time-history response analyses in the optimization process.

The proposed optimal design method is applied to a five-span continuous steel girder bridge system with double I-type girders, and the optimal solutions at various allowable ductility factors of RC pier are discussed to illustrate the rigorousness of the proposed design method. It is also emphasized that the optimum solutions can be obtained efficiently with small numbers of time-history response analyses by introducing the estimation formulae in the design of experiments.

2. OPTIMUM DESIGN FORMULATION AND OPTIMIZATION ALGORITHM

2.1 Design Model

In this study, the five-span continuous steel girder bridge system with double I-type girders shown in Fig.1 is considered in which the superstructure is supported by six rubber bearings, RC piers and the cast-in-place concrete pile foundation. The front and side views of a pier and RC pile foundation are described in Fig.2. The length of piles is 15m and five types of soil conditions in stratum in Fig.3 are considered to calculate the spring constant and the properties of three types of RC pile foundations are summarized in Table 1, in which the construction costs of a pile are assumed to be 65200yen/m^3 for the diameter 1.0m and 73800yen/m^3 for the diameter 1.2m, and the construction costs of footing and form for pile foundation are assumed to be 33500yen/m^3 and 8000yen/m^2, respectively.

The piers are divided into 50 segments in order to calculate the nonlinear dynamic behaviours accurately. The reinforcements in the cross section of piers are arranged in two layers for the bridge direction and one layer for the transverse direction, and the spacing of each reinforcement are fixed at 125mm as shown in Fig.4. With enlargement of cross section, the number of reinforcements is increased so as to keep the spacing of reinforcements. The stiffness of a RC pier is modelled using the

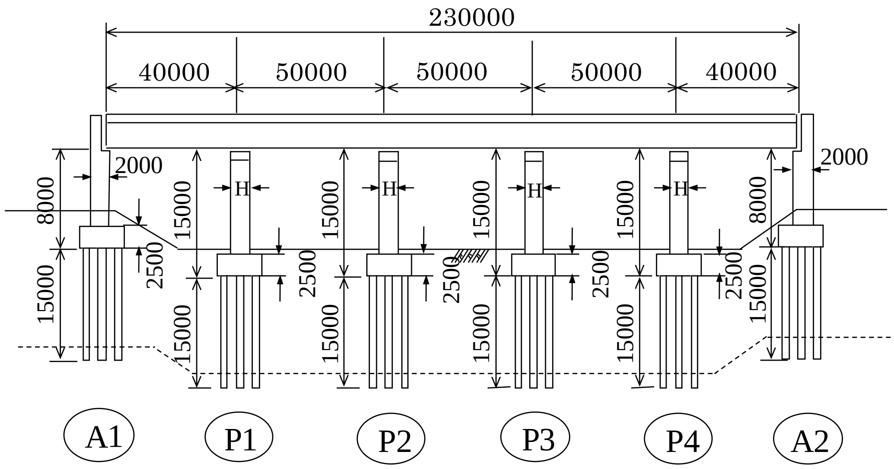


Fig.1 Five-span continuous steel girder bridge system

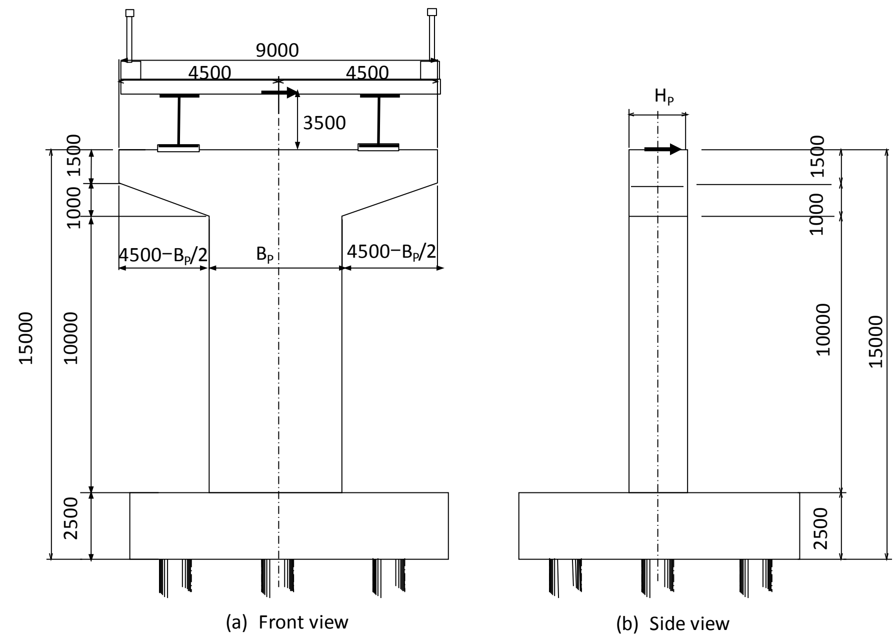
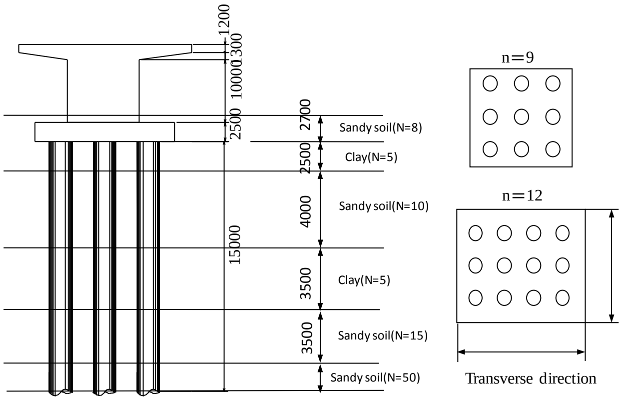


Fig.2 Front and side views of piers and RC pile foundation

Table 1 Properties of three types of RC piles

Diameter Φ	Number of piles	Width of footing (bridge direction)	Width of footing (transverse direction)	Height of footing	Construction cost (10^3yen)	K_h(kN/m)	$K_{\theta 1}$(kNm/rad) (bridge direction)	$K_{\theta 2}$(kNm/rad) (transverse direction)	S-R spring (kN/rad)	Weight(kN)
1.0m	9	7.0m	7.0m	2.5m	13,466	2212657	23604414	23604414	-3001351	3001.3
1.2m	9	8.4m	8.4m	2.5m	16,544	2762476	38430822	38430822	-4437599	4321.8
1.0m	12	7.0m	9.5m	2.5m	17,965	2950210	31472551	49511633	-4001802	4073.1

Pile construction cost: 65200yen/cm^3 (Diameter1.0m), 73800yen/cm^3 (Diameter1.2m), Form: 8000yen/cm^2, concrete: 18500yen/cm^3



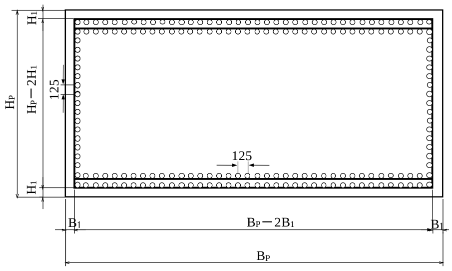
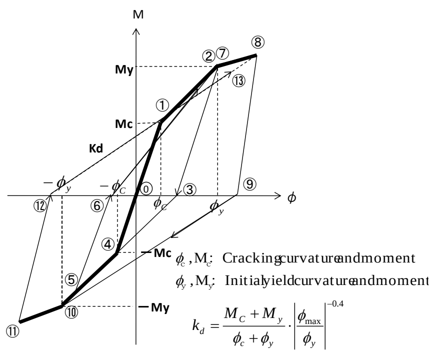


Fig.4 Cross section of a pier

Fig.3 Soil condition



$$k_d = \frac{M_C + M_y}{\phi_c + \phi_y} \cdot \left| \frac{\phi_{max}}{\phi_y} \right|^{-0.4}$$

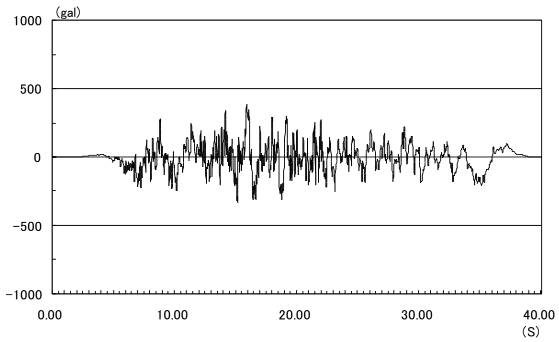


*Fig.5 Trilinear rigidity hysteresis mode*l for *RC pier (Takeda model)*

Fig.6 Acceleration wave motion model

trilinear rigidity reduction type model (Takeda model) shown in Fig.5. The nonlinear behaviours of the bridge system for both the bridge and transverse directions subjected to devastating earthquakes are analyzed precisely by the time-history response analysis using the general purpose nonlinear analysis software, TDAP-III, in which the Type II standard strong acceleration wave motion model in Fig.6 is applied.

2.2 Optimum Design Formulation

In the design of the bridge system, the dimension of superstructure is assumed to be given and widths of rectangular rubber bearings are assumed to be 90cm and 100cm at abutment and piers, respectively. The design variables for rubber bearings are the heights of those at abutment and piers, B_{h1}, B_{h2}

and B_{h3}. For the cast-in-place concrete pile foundations (RC pile foundation) the properties of horizontal and rotation spring constants are calculated considering the numbers of piles and diameters of piles. In this study the horizontal spring constant of RC pile foundation, K_h, which is the common factor to both the bridge and transverse directions for the time-history response analysis, is considered as the design variable. The widths to the bridge and transverse directions and the amount of steel reinforcements in a cross section, H_P, B_P and A_s, are taken into account as the design variables for RC piers. The bridge system shown in Fig.1 is symmetrical about the centreline and the total number of design variables is eleven $(B_{h1}, B_{h2}, B_{h3}, K_{h1}, K_{h2}, H_{P1}, H_{P2}, A_{S1}, A_{S2}, B_{P1}, B_{P2})$. In those design variables, the discrete design variables are expressed as $\mathbf{B_h}, \mathbf{K_h}, \mathbf{H_P}, \mathbf{A_S}, \mathbf{B_P}$. In this study, the discrete design variables $\mathbf{K_h}, \mathbf{H_P}, \mathbf{A_S}$ and $\mathbf{B_P}$ are selected from the following sets;

$$\mathbf{K_h} \in \{2212657(kN/m),\ 2762477,\ 2950210\}$$
$$\mathbf{H_P} \in \{2300(mm),\ 2400,\ 2500,\ 2600,\ 2700\}$$
$$\mathbf{A_S} \in \{642.4(mm^2),\ 794.2, 956.6\}$$
$$\mathbf{B_P} \in \{3500(mm),\ 4000,\ 4500\}$$

The bridge system must have sufficient ultimate dynamic capacities for large displacements caused by devastating earthquakes. Therefore, the relative horizontal displacements between superstructure and piers in both the bridge and transverse directions are dealt with as the design constraints, $g_{h1}, g_{h2}, g_{h3}, g_{t1}, g_{t2}, g_{t3}$, for the safety of the rubber bearings. Furthermore, the ductile factors are also dealt with as the design constraints for the RC piers, $g_{\mu 1}, g_{\mu 2}$, so as to ensure the performance at the ultimate state. The equations for design constraints are;

$$g_{h1} = \delta_{h1} - \delta_{a1}(B_{h1}) \le 0 \tag{1}$$
$$g_{h2} = \delta_{h2} - \delta_{a2}(B_{h2}) \le 0 \tag{2}$$
$$g_{h3} = \delta_{h3} - \delta_{a3}(B_{h3}) \le 0 \tag{3}$$
$$g_{t1} = \delta_{t1} - \delta_{a1}(B_{h1}) \le 0 \tag{4}$$
$$g_{t2} = \delta_{t2} - \delta_{a2}(B_{h2}) \le 0 \tag{5}$$
$$g_{t3} = \delta_{t3} - \delta_{a3}(B_{h3}) \le 0 \tag{6}$$
$$g_{\mu 1} = \mu_1 - \mu_a \le 0 \tag{7}$$
$$g_{\mu 2} = \mu_2 - \mu_a \le 0 \tag{8}$$

where δ_{a1}, δ_{a2} and δ_{a3} are the allowable relative horizontal displacements of bearings at abutment and piers, which are given as the products of the heights of bearings B_{h1}, B_{h2} and B_{h3} multiplied by 2.5. The parameter μ is the ductile factor of a pier, which is given by the ratio of working curvature to the yield curvature for the bridge direction. The parameter μ_a is the allowable ductile factor which is assumed considering the condition of construction site for earthquake. If the construction site is the area where the high possibility of devastating earthquakes will be predicted in near future, μ_a should be set at the lower value considering the repair cost after the excitation of earthquake.

In the design of RC pile foundation, the fundamental concept following the JSHB is that RC pile foundation is not allowed to yield when the horizontal ultimate dynamic bearing capacity in the RC pier is not enough large. On the contrary, RC pile foundation is allowed to yield up to the ductile factor 4.0 when the bearing capacity in the RC pier is enough large. This constraint is investigated by analyzing the nonlinear frame structure of RC pile foundation, in which the applied forces V_0,M_0,H_0 are given as the corresponding forces to the horizontal ultimate dynamic bearing capacity for the RC pier.

This constraint of RC pile foundation is quite complex to take into account in the optimization process. Furthermore, the design variable for RC pile foundation depends on the design variables for the sizing variables of RC pile. Therefore, it is difficult to deal with the design variable for RC pile foundation together with the sizing variables of RC pier simultaneously. To simplify the optimum design problem, therefore, it is assumed that the design variable for RC pile foundation is independent, and the constraint on the RC pile foundation is not dealt with in the optimization process. After the determination of optimum solution the constraint on the RC pile foundation is investigated.

The total construction cost minimization problem, which is expressed as the summation of bearing construction cost, $COST_B(\mathbf{B_h})$, foundation construction cost, $COST_F(\mathbf{K_h})$, and pier construction cost, $COST_P(\mathbf{H_P}, \mathbf{A_S}, \mathbf{B_P})$, can be formulated as

find $\mathbf{B_h}, \mathbf{K_h}, \mathbf{H_P}, \mathbf{A_S}, \mathbf{B_P}$ which

minimize

$$COST(\mathbf{B_h}, \mathbf{K_h}, \mathbf{H_P}, \mathbf{A_S}, \mathbf{B_P}) = COST_B(\mathbf{B_h}) + COST_F(\mathbf{K_h}) + COST_P(\mathbf{H_P}, \mathbf{A_S}, \mathbf{B_P}), \tag{9}$$

subject to the constraints in eqs.(1)-(8).

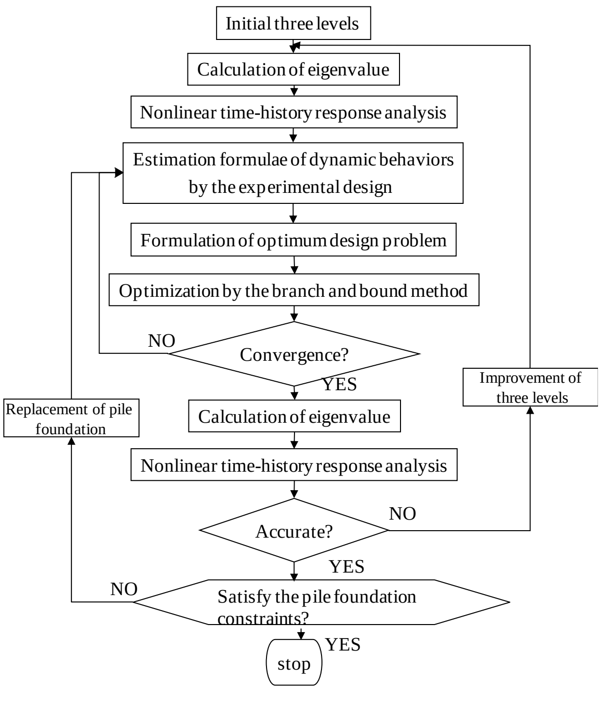


Fig.7 Macro-flow of the proposed optimum design method

Table 2 Orthogonal array table $L_{27}(3^{13})$

	Factor												
No.	1	2	3	4	5	6	7	8	9	10	11	12	13
No.1	1	1	1	1	1	1	1	1	1	1	1	1	1
No.2	1	1	1	1	2	2	2	2	2	2	2	2	2
No.3	1	1	1	1	3	3	3	3	3	3	3	3	3
No.4	1	2	2	2	1	1	1	2	2	2	3	3	3
No.5	1	2	2	2	2	2	2	3	3	3	1	1	1
No.6	1	2	2	2	3	3	3	1	1	1	2	2	2
No.7	1	3	3	3	1	1	1	3	3	3	2	2	2
No.8	1	3	3	3	2	2	2	1	1	1	3	3	3
No.9	1	3	3	3	3	3	3	2	2	2	1	1	1
No.10	2	1	2	3	1	2	3	1	2	3	1	2	3
No.11	2	1	2	3	2	3	1	2	3	1	2	3	1
No.12	2	1	2	3	3	1	2	3	1	2	3	1	2
No.13	2	2	3	1	1	2	3	2	3	1	3	1	2
No.14	2	2	3	1	2	3	1	3	1	2	1	2	3
No.15	2	2	3	1	3	1	2	1	2	3	2	3	1
No.16	2	3	1	2	1	2	3	3	1	2	2	3	1
No.17	2	3	1	2	2	3	1	1	2	3	3	1	2
No.18	2	3	1	2	3	1	2	2	3	1	1	2	3
No.19	3	1	3	2	1	3	2	1	3	2	1	3	2
No.20	3	1	3	2	2	1	3	2	1	3	2	1	3
No.21	3	1	3	2	3	2	1	3	2	1	3	2	1
No.22	3	2	1	3	1	3	2	2	1	3	3	2	1
No.23	3	2	1	3	2	1	3	3	2	1	1	3	2
No.24	3	2	1	3	3	2	1	1	3	2	2	1	3
No.25	3	3	2	1	1	3	2	3	2	1	2	1	3
No.26	3	3	2	1	2	1	3	1	3	2	3	2	1
No.27	3	3	2	1	3	2	1	2	1	3	1	3	2

In the optimum design problem $\mathbf{B_h}$ can take continuous values, but the others must be selected from a list of discrete values. Therefore, the construction cost minimization problem can be expressed as a mixed discrete-continuous problem. Several types of optimization techniques have been developed, and Huang and Arora (1997) investigated the efficiency and reliability of those for discrete and mixed discrete-continuous problems. In this study the optimization problem is solved by the classical branch and bound method with dual algorithm and convex approximation (Fleury and Braibant 1986, Taniwaki and Ohkubo 2004) for the reason that the approach is efficient and reliable for a mixed discrete-continuous problem without any parameters.

2.3 Optimization Algorithm

The macro-flow of the proposed optimization algorithm is depicted in Fig.7. In the optimization process, in general, a number of nonlinear seismic response analyses and sensitivity analyses are necessary to determine the optimal solutions. To avoid these complexity and difficulties and make the optimum design process tremendously efficient, the design of experiments (Taguchi 1987) is applied to introduce the estimation formulae for the dynamic behaviours. The dynamic behaviours and those sensitivities are calculated by using the estimation formulae without analyzing the structure. In the design of experiments, according to the orthogonal array table $L_{27}(3^{13})$ (Taguchi 1987) given in Table 2, the three levels for all design variables are assumed and the twenty seven runs of nonlinear seismic response analyses are carried out using TDAP-III for both the bridge and transverse directions, respectively. The first eleven factors among thirty factors in Table 1 are assigned to the design variables $B_{h1}, B_{h2}, B_{h3}, K_{h1}, K_{h2}, H_{P1}, H_{P2}, A_{S1}, A_{S2}, B_{P1}, B_{P2}$, respectively. Assuming that the intended variable for the *k*th factor is x_k and the mean value of three levels $\hat{x}_{ki}\ (i=1,\cdots,3)$ for the *k*th factor is $\bar{x}_k$, the general form of estimation formula is introduced in the expression of quadratic functions of the design variables given in eqs.(10)-(13).

$$y = b_0 + \sum_{k=1}^{m} b_{k1} z_k + \sum_{k=1}^{m} b_{k2}(-M_{k2}^2 - M_{k3} z_k + M_{k2} z_k^2)\,, \tag{10}$$

where

$$M_{ki} = \frac{1}{3}(\hat{z}_{k1}^i + \hat{z}_{k2}^i + \hat{z}_{k3}^i) \quad (k=1,\cdots,11), \tag{11}$$

$$\hat{z}_{ki} = \hat{x}_{ki} - \bar{x}_k \quad (i=1,\cdots,3)\ \ (k=1,\cdots,m), \tag{12}$$

$$z_k = x_k - \bar{x}_k\,,\ (k=1,\cdots,m) \tag{13}$$

m is the number of design variables (= 11). The estimated values of b_0, b_{k1} and b_{k2} in eq.(10) are given as

$$\hat{b}_0 = \frac{1}{rS_1}\sum_{i=1}^{3} T_{1i}\,, \tag{14}$$

$$\hat{b}_{ki} = \frac{1}{rS_k}\sum_{j=1}^{3} W_{kj} T_{kj} \quad (i=1,2), \tag{15}$$

where

$$S_k = \sum_{i=1}^{3} W_{ki}^2\,, \tag{16}$$

r is the number of runs with the level $\hat{x}_{ki}$ (= 9). T_{ki} is the summation of results by the design of experiments with the level of $\hat{x}_{ki}$. W_{ki} is the value of coefficient with respect to b_{k1} and b_{k2} obtained by replacing z_k with $\hat{z}_{ki}$ in eq.(10). Namely, the values are given by $W_{ki} = \hat{z}_{ki}$ and $W_{ki} = -M_{k2}^2 - M_{k3}\hat{z}_{ki} + M_{k2}\hat{z}_{ki}^2$.

Table 3 Level in the optimization process

Level	B_{h1} (cm) (spring constant(kN/m))		B_{h1}, B_{h2} (cm) (spring constant(kN/m))		k_{h1}, k_{h2}(kN/m)		H_{P1}, H_{P2}(mm)		A_{S1}, A_{S2} (mm^2)		B_{P1}, B_{P2}(mm)	
1	B_{h1}	12.0(13500)	B_{h2}	12.0(16667)	k_{h1}	2212657	H_{P1}	2300.0	A_{S1}	642.4	B_{P1}	3500
			B_{h3}	12.0(16667)	k_{h2}	2212657	H_{P2}	2300.0	A_{S2}	642.4	B_{P2}	3500
2	B_{h1}	14.0(11571)	B_{h2}	13.0(15385)	k_{h1}	2762476	H_{P1}	2500.0	A_{S1}	794.2	B_{P1}	4000
			B_{h3}	13.0(15385)	k_{h2}	2762476	H_{P2}	2500.0	A_{S2}	794.2	B_{P2}	4000
3	B_{h1}	16.0(10125)	B_{h2}	14.0(14286)	k_{h1}	2950210	H_{P1}	2700.0	A_{S1}	956.6	B_{P1}	4500
			B_{h3}	14.0(14286)	k_{h2}	2950210	H_{P2}	2700.0	A_{S2}	956.6	B_{P2}	4500

After the determination of optimum solutions the design constraints with the estimation formulae are examined by re-analyzing the bridge system. In case that the design constraints violate the allowable limit, the three levels for all design variables and estimation formulae for dynamic behaviours are improved and the minimum cost design problem is re-solved. This optimization process is iterated until the relative errors between the estimated design constrains and the exact ones satisfy the allowable limit. After the determination of design variables the constraint of RC pile foundation is investigated. If the constraint is violated, the RC pile foundation is replaced with the larger one and the bridge system is re-optimized.

3. DESIGN EXAMPLES

The proposed optimal design method is applied to the five-span continuous steel girder bridge system shown in Fig.1 and the optimal solutions for $\mu_a = 1.5$, 2.0 and 2.5 are discussed. The unit cost of rubber is 45yen/cm^3. The construction costs of concrete, form and reinforcement for piers are assumed to be 18500yen/m^3, 8000yen/m^2 and 120000yen/tf, respectively. The levels for $\mu_a = 1.5$, 2.0 and 2.5 are set as shown in Table 3. In the optimization process, the lower and upper limits for discrete design variables are set at the minimum and maximum values of the three levels. The optimum solutions of design variables, ultimate bending moments of piers, feasibilities of the design constraints for μ_a =1.5, 2.0 and 2.5 are summarized in Table 4.

In the case of $\mu_a = 1.5$, B_{h2} is the lower limit and B_{h3} is nearly the lower limit. According to the flow-chart in Fig.7 the RC pile foundation is replaced by the largest one which indicates the highest cost. The pile foundations for P_1 and P_2 yield for the bridge directions of P_1 and P_2, and the transverse direction of P_1. However, all constraints on RC pile foundation are satisfied. The estimation formulae of relative displacements can be introduced quite accurately by the design of experiments within 1.0 percent differences compared with that obtained by analysis. On the contrary, the ductile factors in P_1 and P_2 by the estimation formulae are 22.5 percent and 16 percent larger than those by analysis, respectively. At the optimum solution g_{h1} and g_{t2} are active.

In the case of $\mu_a = 2.0$, B_{h1} is almost the same as that of the case of $\mu_a = 1.5$. B_{h2} and B_{h3} are the lower limit. The ultimate bending moments in P_1 and P_2 are smaller than those of $\mu_a = 1.5$ and the pile foundations for P_1 and P_2 do not yield for all directions of P_1 and P_2. All constraints are

Table 4 Optimum solutions for $\mu_a = 1.5$, 2.0 and 2.5

Allowable ductile factors		1.5		2.0		2.5	
B_{h1}(cm) (kN/m)		14.670 (11043)		14.709 (11014)		14.603 (11094)	
B_{h2}(cm) (kN/m)		12.000 (16667)		12.000 (16667)		12.000 (16667)	
B_{h3}(cm) (kN/m)		12.232 (16351)		12.000 (16667)		12.000 (16667)	
K_{h1}(kN/m) (φ, n)		2950210 (ϕ =1.0m, n=12)		2950210 (ϕ =1.0m, n=12)		2762476 (ϕ =1.2m, n=9)	
K_{h2}(kN/m) (φ, n)		2950210 (ϕ =1.0m, n=12)		2950210 (ϕ =1.0m, n=12)		2762476 (ϕ =1.2m, n=9)	
P1	H_{P1}(mm)	2600		2600		2600	
	A_{S1}(mm^2)	956.6		794.2		794.6	
	B_{P1}(mm)	4000		4000		3500	
	Muh_1(kNm)	68173		58594		53148	
	Mut_1(kNm)	99284		86428		68371	
P2	H_{P2}(mm)	2700		2700		2600	
	A_{S2}(mm^2)	956.6		794.2		794.2	
	B_{P2}(mm)	3500		3500		3500	
	Muh_2(kNm)	66287		57199		53457	
	Mut_2(kNm)	80332		70061		68712	
constraint g_{h1}		D.exp.*	1.000	D.exp.*	1.000	D.exp.*	0.997
		Anal**	0.997	Anal**	0.994	Anal**	1.000
constraint g_{h2}		D.exp.*	0.929	D.exp.*	0.907	D.exp.*	0.894
		Anal**	0.920	Anal**	0.905	Anal**	0.891
constraint g_{h3}		D.exp.*	0.914	D.exp.*	0.909	D.exp.*	0.899
		Anal**	0.912	Anal**	0.901	Anal**	0.894
constraint g_{t1}		D.exp.*	0.892	D.exp.*	0.881	D.exp.*	0.910
		Anal**	0.886	Anal**	0.878	Anal**	0.910
constraint g_{t2}		D.exp.*	0.986	D.exp.*	0.973	D.exp.*	0.970
		Anal**	0.980	Anal**	0.972	Anal**	0.970
constraint g_{t3}		D.exp.*	0.951	D.exp.*	0.953	D.exp.*	0.950
		Anal**	0.944	Anal**	0.950	Anal**	0.947
constraint $g_{\mu1}$		D.exp.*	0.968	D.exp.*	0.953	D.exp.*	0.903
		Anal**	0.743	Anal**	0.866	Anal**	0.970
constraint $g_{\mu2}$		D.exp.*	0.991	D.exp.*	0.957	D.exp.*	0.799
		Anal**	0.831	Anal**	0.985	Anal**	0.949
Yield constraints of pile foundation for bridge direction		P_1: yield, $Pu_1 > 1.5KhcW_1$ satisfy all constraints P_2: yield, $Pu_2 > 1.5KhcW_2$ satisfy all constraints		P_1: not yield P_2: not yield		P_1: not yield P_2: not yield	
Yield constraints of pile foundation for transverse direction		P_1: yield, $Pu_1 > 1.5KhcW_1$ satisfy all constraints P_2: not yield		P_1: not yield P_2: not yield		P_1: not yield P_2: not yield	
Total cost (10^6		163.694		161.693		154.167	

D.exp.* : Feasibility of design constraints with the estimation formulae by the design of expe
Anal** : Feasibility of design constraints using exact behaviors by analysis

estimated accurately by the design of experiments within 8.7 percent difference compared with that obtained by analysis. At the optimum solution, g_{h1}, g_{t2}, g_{t2} and $g_{\mu 2}$ are active. We can observe 1.2 percent reduction in the total cost compared to that in the case of $\mu_a = 1.5$.

In the case of $\mu_a = 2.5$, B_{h1} is also nearly the same as those for other cases. B_{h2} and B_{h3} are also determined by the lower limit. The ultimate bending moments in P_1 and P_2 are smaller than those of $\mu_a = 2.0$. The second largest pile foundations for P_1 and P_2 are selected and those foundations do not yield for all directions of P_1 and P_2. The constraints of relative displacements are estimated accurately by the design of experiments within 0.5 percent difference compared with that obtained by analysis. The optimum solution is feasible, however the ductile factors in P_1 and P_2 by the estimation formulae are 6.7 percent and 15 percent smaller than those by analysis, respectively. At the optimum solution, g_{h1}, g_{t2}, $g_{\mu 1}$ and $g_{\mu 2}$ are active. We can observe 4.65 percent reduction in the total cost can be observed compared to that in the case of $\mu_a = 2.0$.

From the numerical example, it is clear that the constraints of relative displacements are estimated quite accurately by the design of experiments. The constraints of ductile factor can be estimated comparatively accurately near the mean value of ductile factors obtained by twenty seven runs shown in Table 2. However, as the ductile factor is different from the mean value the estimation of ductile factor become inaccurate.

4. CONCLUSIONS

The following conclusions can be drawn from this study:

1) The proposed optimal design method can determine the heights of rubber bearings, cross-sectional dimensions and amount of steel reinforcements for RC piers, and numbers and diameters of piles for bridge system with double I-type girders rigorously and efficiently.

2) By applying the design of experiments, the estimation formulae for the relative horizontal displacements to the bridge and transverse directions can be introduced accurately with small number of nonlinear seismic response analyses. The accuracy of the estimation formulae is excellent within 1.0 percent difference between the exact behaviours and estimated ones.

3) The constraints of ductile factor can be estimated comparatively accurately near the mean value of ductile factors obtained by twenty seven runs. However, as the ductile factor is different from the mean value the estimation of ductile factor become inaccurate. Therefore, we need to pay attention in the applicable range of the estimation formulae for ductile factor in the optimization process.

4) The heights of rubber bearing at abutment are nearly the same for any allowable ductile factors. Almost all heights of rubber bearings at piers are determined the lower limit. The optimum solutions are determined by the constraints of g_{h1}, g_{t2}, $g_{\mu 1}$ and $g_{\mu 2}$.

5) In the proposed design process, the constraint on the RC pile foundation is not dealt with in the optimization process and, then, the RC pile foundation is replaced with the larger one so as to satisfy the constraint on the RC pile foundation. This design process can simplify the optimization algorithm greatly.

Acknowledgements

Part of this work has been supported by FUT Research Promotion Fund.

REFERENCES

Fleury,C. and Braibant,V.(1986), "Structural optimization: A new dual method using mixed variables", *Int. J. Num. Meth. Eng.*, Vol.23, 409-428

G. Taguchi, (1987), *System of Experimental Design*, New York, UNIPUB/Krass International Publications, Volume 1 & 2

Huang,M., Arora,J.S., (1997), "Optimal design with discrete variables: some numerical examples", *Int. J. Num. Meth. Eng.*, Vol.40, 165-188

Japan Road Association, (2002), *Specification for Highway bridges, Part V seismic design*, Maruzen, Tokyo (in Japanese)

Ohkubo,S., Tanaka,K., Watanabe,T., Yoshitake,R.(2002), "Optumal design method for seismic-isolation bridge systems subject to huge earthquakes", *Journals of JSCE.*, No.703/I-59, 67-82 (in Japanese)

Ohkubo,S., Tanaka,K., Kadota,K.(2002), "Minimum cost design method for seismic-isolation bridge systems considering initial construction cost and repair cost caused by a huge earthquake", *Journals of JSCE*., No.710/I-60, 191-208 (in Japanese)

Tanaka,K., Taniwaki,K., Ohkubo,S. (2007), "Optimal performance-based design of seismic–isolation bridge systems", *Third Asian-Pacific Congress on Computational Mechanics, APCOM'07, 12pages in CD-ROM*

Taniwaki,K. and Ohkubo,S. (2004), "Optimal synthesis method for transmission tower truss structures subjected to static and seismic loads", *Int. J. Struct. Multidisc. Optim.* Vol.26, 441-454

TDAP III, ARK Information Systems, INC., home page ; http://www.ark-info-sys.co.jp/en/index.html