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II c

Exemples de calculs d'optimisation à l'aide de l'ordinateur

Beispiele des Computer-Einsatzes bei der Optimierung

Examples of Computer-aided optimal Design of Structures

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Structural Optimization through Sensitivity Coefficients

Optimisation des structures au moyen des coefficients de sensitivité

Optimierung der Tragwerke mittels Sensitivitätskoeffizienten

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1. INTRODUCTION

There are basically two approaches to the solution of a structural optimization problem. In one approach which is followed by Schmit [1] and by Schmit and Fox [2], both kinds of variables namely the design variables and the behaviour variables are treated as unknowns in the programming problem. In the other approach which is followed by Romstadt and Wang [3] and by Vanderplaats and Moses [4] the solution procedure consists of a series of analysis-programming cycles. In each programming stage the most recent set of behaviour variables is treated as known and the design variables are treated as unknowns. The advantage of the first approach is that the programming problem is to be solved only once. The size of the programming problem depends on the number of nodes, the number of members and the number of load conditions. Generally for any practical case the size of the problem becomes almost unmanageable. In the second approach though the programming problem has to be solved many times the size of the problem is smaller. Hence the second approach is preferred to the first.

In the present paper the second approach is followed but the solution procedure consists mostly of solving a series of programming problems. It is observed that in the conventional approach (e.g. that followed by Romstadt and Wang [3]) the time required for the solution of a problem increases rapidly with the increase in the statical indeterminancy of the structure. In such cases the proposed approach is more economical.

2. THEORETICAL ANALYSIS

2.1 Formulation and solution

Formulation of the structural optimization problem as a programming problem has been very well brought out by several authors such as Vanderplaats and Moses [4] or by Brown and Ang [5]. It is therefore assumed here that a structural optimization problem can be formulated as the following non-linear programming problem.

$$\left.\begin{array}{l}\text{Minimize } F(X) \\ \text{Subject to } G_j(X,Y) \leq 0 \quad j=1,\ldots,m\end{array}\right\} \qquad (1)$$

where X is a vector of design variables and Y is a vector of behaviour variables. The objective function can be any function which can be expressed as a function of the design variables. Usually weight of the structure is treated as the objective function. The constraints can be any inequalities which have to be satisfied by the structure such as stress limitations, size limitations or the deflection limitations. In this formulation no distinction is made between the constraints and the restraints.

The optimization process (see fig. 1) is started with an initial set of design variables, X_0. The analysis of the structure (by stiffness matrix method) is carried out to give the associated set of behaviour variables Y_0. After this analysis is over it is found out what is the change in each behaviour variable due to a 100 per cent change in each of the design variables. This information is stored in a matrix called sensitivity matrix which is denoted by CH. A general element CH_{ij} of this matrix stands for the change in i^{th} behaviour variable due to a unit change in j^{th} design variable. The structural optimization problem is then formulated in terms of X_0 and Y_0 to yield new set of design variables $\bar{X}_0$ (see block A). The corresponding set of behaviour variables is now found from the matrix equation

$$Y_1 = Y_0 + CHX\,(\bar{X}_0 - X_0) \qquad \text{(see block B)}$$

The next programming cycle then makes use of this vector Y_1 to yield new solution of the design variables $\bar{X}_1$. The process thus continues till the difference between the design vectors obtained from two successive programming cycles is found to be smaller than a predetermined vector ϵ. It is clear that the sensitivity coefficients calculated for the initial design will not be useful if the structure is statically indeterminate to a high degree and the original design has undergone a lot of change. In that case the sensitivity coefficients are recalculated (see block C).

The optimization of the structure is thus consisting of mostly the solution of a series of programming problems.

2.2 Calculation of the Sensitivity Coefficients

With the initial set of design variables the stiffness matrix of the structure is assembled in half-band form. If one design parameter is changed by 100 per cent the new stiffness matrix is obtained by recomputing the element stiffness matrix only for one member and then making the appropriate changes in the overall stiffness matrix. Knowing the original set of displacements, the external forces and the new stiffness matrix; new displacements are computed using Jacobi iteration [6]. With new nodal displacements known the new set of behaviour variables and hence the change in each of them due to 100 per cent change in one design variable is computed. These changes when divided by the original value of the design variables give one column of the sensitivity matrix. Before computing the next column the stiffness matrix is reduced to the original matrix.

2.3 Solution of the Programming Problem

The programming problem stated in (1) is solved by using

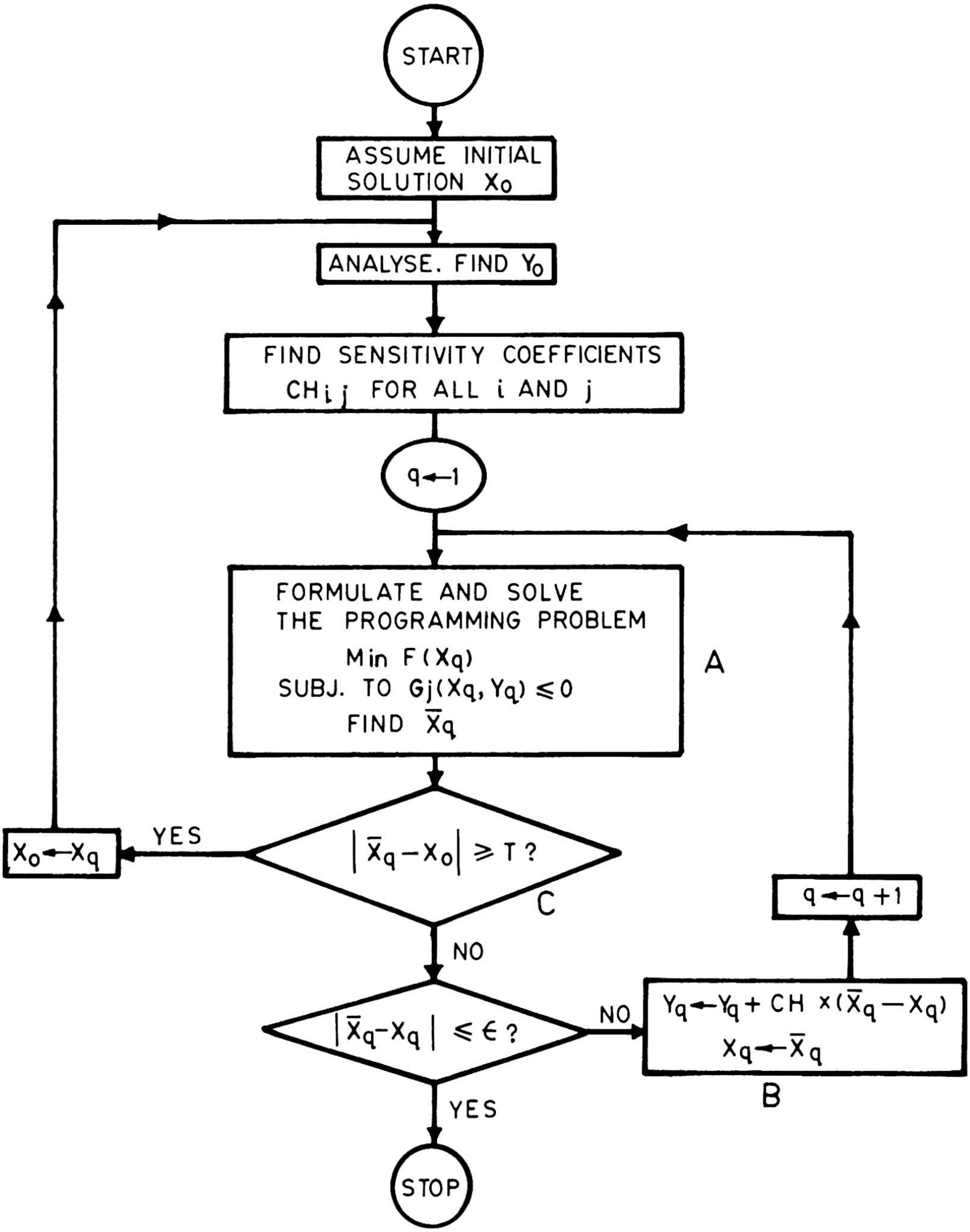

FIG.1. FLOW CHART FOR THE OPTIMIZATION PROCESS

exterior penalty function [6] method which consists of solving a sequence of unconstrained minimization problem:

$$\text{Min } \Phi(X,Y,r) = F(X) + r \langle G_j \rangle^2$$

for increasing values of r which is called penalty. The unconstrained minimization method which is found to be most efficient is the Davidson-Fletcher-Powell [6] method of variable metric and the unidirectional approach that is used is the direct root method. If the number of components in X is large the programming problem given in (1) may not lead to convergence. Hence the vector X is split into subvectors X_1, X_2, ... etc. where each of the subvectors is of a much smaller dimensions than the original vector X [7]. The solution of the programming problem (1) then consists of solving a series of smaller programming problems where only one of the subvectors such as X_1, X_2 are treated as unknowns. The use of exterior penalty function is found to be better when solving such partitioned problems.

3. COMPUTER PROGRAMMING AND NUMERICAL WORK

A general computer program based on the proposed method is written separately for trusses and for frames. The program is written in FØRTRAN IV language and is compatible with IBM 360, CDC 3600, DEC 10 and EC 1030 (called Ryad in some countries) computer systems. Several truss and frame problems for minimum weight design have been solved.

4. CONCLUSIONS

A general optimization algorithm for any structural problem has been suggested.

Acknowledgement

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SUMMARY

Structural optimization problem is generally solved as a sequence of analysis-programming cycles. In this paper it is shown how this problem can be treated as a series of programming problems. The relation between the changes in the behaviour variables due to a specified change in each of the design variables is found and stored in the form of "Sensitivity Matrix". This matrix directly gives the solution corresponding to a given set of design variables. The availability of this matrix dispenses with the frequent analysis.

RESUME

Le problème d'optimisation des structures est résolu généralement par une série de cycles dans un programme de calcul. On montre comment ce problème peut être traité par une série de problèmes de programmation. La relation entre les changements des variables de comportement et ceux de l'une quelconque des variables du projet est determinée et compilée sous forme de "matrice de sensitivité". Cette matrice donne directement la solution correspondant à un ensemble donné de variables du projet. On peut donc éliminer avec cette matrice de nombreux calculs.

ZUSAMMENFASSUNG

Das Problem der Optimierung von Strukturen wird allgemein als eine Reihe von analytischen Programmierungszyklen gelöst. Im vorgelegten Beitrag wird gezeigt, wie die allgemeine Methode der Optimierung als eine Reihe von Programmierungsproblemen behandelt werden kann. Die Beziehungen zwischen den Verhaltensvariablen und den Entwurfsvariablen werden hergestellt und als "Sensitivitätsmatrix" gespeichert. Aus dieser Matrix ergibt sich direkt die der gegebenen Gruppe von Entwurfsvariablen entsprechende Lösung. Die Benützung dieser Matrix vermeidet eine Wiederholung der Berechnung.

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Earthquake-Resistant Design of the Tower and Pier System of Suspension Bridges

Dimensionnement contre les tremblements de terre du système de pylône et pile des ponts suspendus

Die Erdbebenbemessung des Systems von Pylon mit Sockel bei Hängebrücken

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1. INTRODUCTION

Economical applications of mathematical programming methods in structural optimization are limited to specific structures as mentioned in Introductory Report.[1] In the case of structures with relatively simple and bulky dimensions, the mathematical programming method could be applied efficiently even if the structures are designed under relatively complicated design conditions. Dynamic loading problems are not treated in Introductory Report, and loading conditions appearing in optimal design have been mostly limited to the static ones.

Studies on aseismic design of long-span suspension bridges were carried out for many years in Japan, and the results of investigations were published as the official or individual reports. According to the studies on aseismic design of the suspension bridges, design of the tower and the pier is very important.[2] These parts of the bridge must be investigated as a system because of the interaction of these parts during the earthquake. The tower and pier system of suspension bridges involves rigid, massive, and large pier and relatively flexible and slender tower, so that the system has very complicated interaction.[2] The combination of the methods of mathematical programming and dynamic structural analysis is in fact well suited to the aseismic design of the tower and pier system of suspension bridges.

To formulate earthquake action for aseismic design, the method of response spectrum is employed in the design codes of the long-span suspension bridges in Japan. In this paper, the response spectrum method is mainly applied in the dynamic analysis and design of the system. Another approach based on more probabilistic concepts using power spectrum density of earthquake action and random vibration theory is possible using design constraints for reliability. Some approximation concepts[3] are used to save the computing time and to decrease the design variables in this paper.

2. THE STRUCTURAL SYSTEM

The system to be designed is the tower and pier system of the suspension bridges as shown in Fig.1, height h_T of the tower and h_P of the pier are determined from the environmental attribute of the bridge, and width b_1 of the pier is determined from geometrical relation with the bridge width. The design variables in global sense are, therefore, the longitudinal width b_2 of the pier and the stiffness of the tower. The combination of these two variables induces very complicated dynamic properties of the system.[2)]

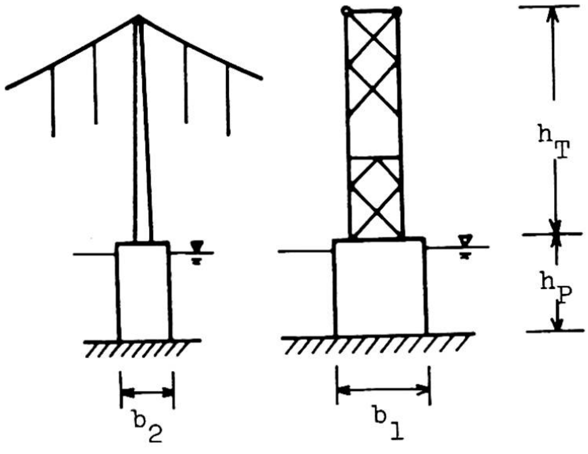

Fig.1 Structural System

3. DYNAMIC ANALYSIS OF THE SYSTEM

Analytical Model The analytical model of the tower and pier system of the suspension bridge treated in this paper is shown in Fig.2. The tower is assumed to be the lumped mass system, and the following assumptions are made:

(1) The foundation has elastic property.
(2) The reaction of the cable at the top of the tower is taken into account by applying the equivalent axial thrust and using an equivalent spring for the cable.[4)]
(3) The pier is assumed to be perfectly rigid and to be a single-degree-of-freedom capable of rocking motion.

Model of Earthquake Excitation Earthquake excitation is represented by response acceleration spectrum. In this study, the standard spectrum as shown in Fig.3 is used which is authorized by Honshu-Shikoku Connection Bridge Authority of Japan. In this figure, the longitudinal axis refers to be response magnification factor β, and standard acceleration in this design is 180 gal.

Dynamic Response Analysis The equation of motion for this multi-degrees-of-freedom-system can be written as:

$$[M]\{\ddot{y}\}+[C]\{\dot{y}\}+[K]\{y\}=-[\bar{M}]\{\ddot{z}\}$$

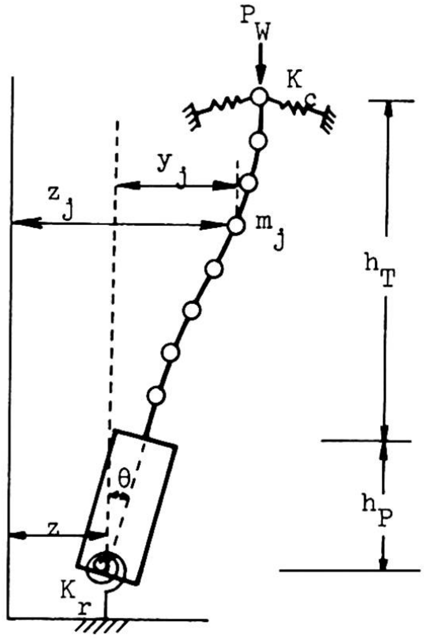

Fig.2 Analytical Model

where [M] is a mass matrix, [C] is a damping matrix, [K] is a stiffness matrix, {y} is a displacement vector, $\{\ddot{z}\}$ is an earthquake acceleration vector. With the aid of modal matrix [Φ] and the generalized displacement vector {q}, where {y}=[Φ]{q}, then the equation of motion rewritten in the following form assuming proportional damping.

$$[I]\{\ddot{q}\}+[\grave{}(2h_i\omega_i)\text{`}]\{\dot{q}\}+[\grave{}(\omega_i)^2\text{`}]\{q\}=-\{P\}$$

where ω_i refers to natural frequency, and h_i is damping constant of i-th mode.

The maximum displacement of point j, $y_{max}^{(j)}$, can be evaluated by root mean square method:

$$y_{max}^{(j)}=\sqrt{\Sigma(F_i\phi_i^{(j)}q_{i\ max})^2}$$

where F_i refers to the participation factor of i-th mode, $\phi_i^{(j)}$ refers to the relative displacement of point j in i-th mode. $q_{i\ max}$ is obtained from response spectrum given in Fig.3:

$$q_{i\ max}=\beta_i\ddot{z}_{max}/\omega_i^2$$

where $\ddot{z}_{max}$ is the maximum earthquake acceleration.

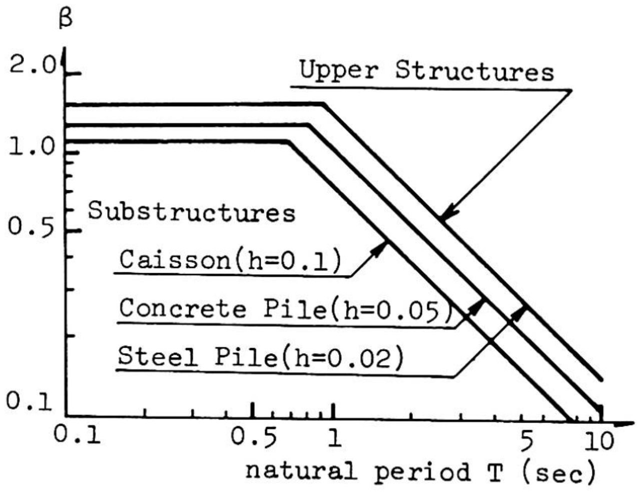

Fig.3 Earthquake Excitation Model

4. DESIGN MODEL

Approximation Concepts of the Tower To save culculation time and to improve reliability of solution, two design variables of the system are selected: One is the moment of inertia of the tower, the other is the longitudinal width of the pier. Other variables of the system are defined by approximation concepts.[3)]

Let I, A and Z refer to the moment of inertia, the cross sectional area and the section modulus respectively, the empirical relation such as following may hold:

$$A=1.21*I^{0.33}$$

$$Z=0.55*I^{0.75}$$

The moment of inertia of the tower can be varied along the height in two ways: One is linearly varied; the other is stepwise varied into two portions. These design models are shown in Fig.4.

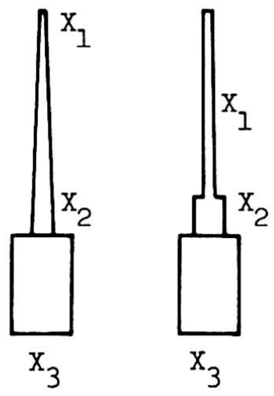

Fig.4 Design Model

Foundation Model The modulus of elasticity of the foundation is denoted by E. In the result of the past studies,[2)] complicated dynamic phenomena due to the foundation condition, width of the pier, and the rigidity of the tower were observed. In the cases where two of the natural frequencies are very close, the coupled vibration of the tower and pier occurs, and the structural systems of such cases should be avoided. In this study, the modulus of elasticity ranges from $10*10^4$ ton/m^2 to $150*10^4$ ton/m^2 taking into account wide variety of foundation conditions.

Damping Constant The damping constant is assumed to be 0.1 for the mode where the vibration of the pier is predominant, and to be 0.02 for the vibration of the tower. For the coupling modes 0.05 for both modes is assumed.

5. OPTIMIZATION

Objective Function The generalized cost, W, is selected to be the objective function:

$$W=W_T+k*W_P$$

where W_T represents the weight of the tower and W_P, of the pier, and k refers to the ratio of unit cost of the pier to that of the tower.

Constraints The following constraints are considered:
(1) Stress of the tower shaft does not exceed the given allowable stress defined against earthquakes.
(2) Displacement of the pier top does not exceed the given allowable value.
(3) Tower shaft is safe against buckling.
(4) Pier is safe against overturning.
(5) Other geometric conditions.

Optimization Technique Objective function and constraints obtained in this way become non-linear and undifferential type, so SUMT by Powell's direct search method without differential is employed as optimization technique.

6. NUMERICAL EXAMPLE AND INVESTIGATIONS

As a numerical example, the tower and pier system shown in Fig.5 is considered, and the results of the computation are shown in Table 1,2. These computation were performed using the design model with stepwise varied cross section. In making Table 1, the following data was used:

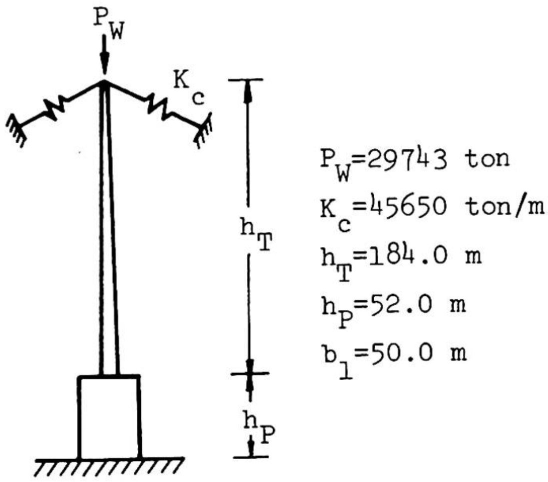

Fig.5 Tower and Pier System

cost ratio 0.2
maximum acceleration .... 180 gal
allowable value of pier top displacement .... 0.05 m
allowable stress of steel .... 37700 ton/m^2

From Table 1, the following investigations may be made:
(1) When elastic modulus of foundation, E, is small, the design of the system is determined only by the displacement constraint at the pier top. When the value of E is large, it tend to be determined by overturning of the pier and buckling of the tower, and the pier width tends to decrease. It shows that the pier width is closely related with E.
(2) The generalized cost is greatly affected by the modulus of elasticity of the foundation. Thus, the investigation of the foundation is very important.
(3) When E is large, the effect of earthquake response tend to decrease, and stiffness of the tower becomes uniform along the height of the tower. From this, when E is large enough, it is not necessary to increase the cross section of the lower part of the tower.

The design is controlled severely by the constraint of the displacement of the pier top in the range of small E. When this constraint is relieved to 0.065 m, the results are shown in Table 2. From these Tables, the following remarks may be made:
(1) In the range of small E, when the constraint of the pier top displacement is relieved slightly, the generalized cost decrease considerably. This result shows that the allowable value of the displacement of the pier top has a significant effect.
(2) In the range of large E, the result is not so affected by the constraint on displacement.

7. CONCLUSION

Table 1

E ($10^4 ton/m^2$)	I (m^4) Upper	I (m^4) Lower	b_2(m)	W	Constraints: Pier (1)	Pier (2)	Tower: Top (3) Base	(4)
10	11.22	62.26	38.97	52146	X			
20	16.52	65.97	24.45	36077	X			
30	5.81	24.90	19.91	27865	X			
50	4.78	21.80	14.37	20896	X	X		X
70	4.75	4.77	14.35	20509		X		X
150	4.75	4.77	14.35	20509		X		X

(1): Displacement of the pier top (2): Overturning
(3): Stress of the tower shaft (4): Buckling

Table 2

E ($10^4 ton/m^2$)	I (m^4) Upper	I (m^4) Lower	b_2(m)	W	Constraints: Pier (1)	Pier (2)	Tower: Top (3) Base	(4)
10	8.15	32.71	28.76	39081	X			
20	13.65	24.43	18.56	27053	X			
30	4.75	8.41	14.54	20842	X			X
50	4.77	4.78	14.35	20511		X		X
70	4.77	4.78	14.35	20511		X		X
150	4.75	4.77	14.35	20508		X		X

(1): Displacement of the pier top (2): Overturning
(3): Stress of the tower shaft (4): Buckling

The optimal design of the tower and pier system on the elastic foundation subjected to earthquake excitation is studied by using response spectrum and modal analysis. Investigation in this study shows that necessity or importance of displacement condition of the pier top must be discussed more precisely from the point of safety of the structure in the range of small E, and that necessity of earthquake-resistant design must be discussed more precisely from the dynamic response of the structure in the range of large E.

PROBABILISTIC APPROACH

Probabilistic approach using power spectrum density for earthquake and based on random vibration theory can be formulated as follows.

Earthquake load is represented by power spectrum density function shown in Fig.6.[5] As earthquake is assumed to have zero mean and to be stationary probabilistic process, variances of the displacement and of the velocity can be evaluated based on the random vibration theory.

Failure probability can be com-

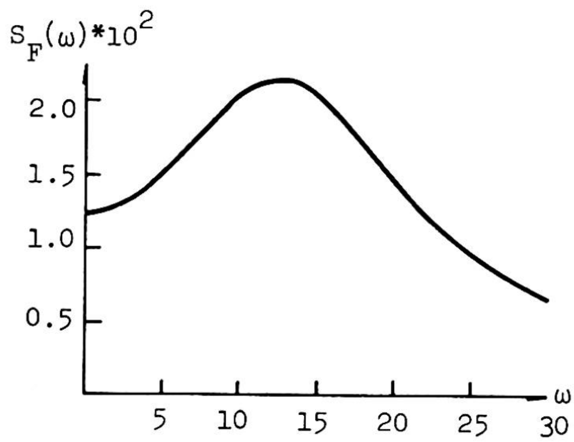

Fig.6 Power Spectrum Density

puted through dynamic reliability theory using displacement and velocity variances. Thus, it is possible to formulate optimization by probabilistic approach using failure probability as constraints.

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SUMMARY

Effective application of the structural optimization method is limited to some specific types of structures in civil engineering structures. In the case of structures with relatively simple and bulky dimensions, the mathematical programming method could be applied efficiently. In this paper, the authors carried out the optimal design of the tower and pier system of suspension bridges on the elastic foundation subjected to earthquake ground motion using response spectrum and dynamic analysis.

RESUME

Une application pratique de la méthode d'optimisation structurale est limitée à certaines structures du génie civil. Dans le cas de structures relativement simples et de grandes dimensions, la méthode de programmation mathématique peut être appliquée efficacement. Dans cet article les auteurs ont fait le calcul d'optimisation du système de pylône et pile des ponts suspendus sur fondation élastique subissant le tremblement de terre, à l'aide d'une analyse dynamique.

ZUSAMMENFASSUNG

Die Anwendung der Tragwerks-Optimierung ist auf einige spezielle Strukturarten im Bauingenieurwesen begrenzt. Bei Strukturen mit einfachen und massigen Abmessungen lässt sich das mathematische Programmierungsverfahren erfolgreich verwenden. In diesem Aufsatz wird eine Optimierung des Pylonsystems auf elastischem Untergrund unter Erdbebenlast entwickelt. Hierbei werden Verhaltensspektren und dynamische Analysen angewendet.

Total Cost Optimum of I-Section Girders

Dimensionnement de poutres à section en I en vue d'un coût total optimal

Optimierung von I-Stahlträgern bezüglich der Totalkosten

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1. INTRODUCTION

The optimum design for I-section girders has been investigated considerably, and most of the investigations aim at a minimum-weight-design or a design including only the cost of shop welding in the fabrication cost. Considering all of the fabrication costs, however, the optimum values of design variables will vary remarkably. Therefore, a total cost optimum design, in which an objective function considers material cost as well as fabrication cost including costs of full-scale-drawing, machining, shop welding, shop assembly and shop painting, has to be done. Since such variables as plate thickness, surface area and weight of members, material grade, etc., are included in fabrication costs, the optimum value of the objective function may not be exactly computed, if some of variables are omitted. Therefore, the design variables to be used in this investigation include almost all dimensions of a cross section.

At the present study, a computer-aided, optimum design for single simply supported girders is carried out by SLP method (Sequence of Linear Programming method).[1),2)] If their upper and lower lateral bracings and sway bracing are designed and their dimensions are determined, it is posible to do an automated design by the use of an automated drawing machine.

2. OPTIMUM DESIGN FOR I-SECTION GIRDERS

Material S, cover plate thickness T_c, cover plate width B_c, flange plate thickness T_f, flange plate width B_f, web plate thickness T_w, web plate height B_w, and segment length of a girder section C_ℓ are selected as design variables. Concerning S, steel of $41kg/mm^2$ in tensile strength is expressed with 4, $50kg/mm^2$ is expressed with 5 and $58kg/mm^2$ with 6, and an intermediate value is set on a continuous function.

The constraints contain limit of stress, limit of deflection, limit of plate width to thickness, as specified at the Specifications, limit of flange width to web height, namely $B_f/B_w=1/3\sim1/6$, and upper and lower limits of the values of design variables, which are also used as move limits.

When an allowable tensile stress and an allowable compressive stress of a material are given by σ_{at} and σ_{ac}, respectively, and a ratio of height to thickness of web plate is given by γ, σ_{at}, σ_{ac} and γ are expressed as a function of S as follows:

$$\sigma_{at}=\sigma_{at}(S), \qquad \sigma_{ac}=\sigma_{ac}(S) \quad \cdots\cdots\cdots\cdots(1)$$

$$\gamma=\gamma(S) \quad \cdots\cdots\cdots\cdots(2)$$

Then, an objective function Z is expressed with

$$Z=\sum_j \rho\cdot V_j\cdot C\cdot CM+\sum_j\sum_i H_{ij}\cdot SMH+\sum_k\sum_\ell \tilde{H}_{k\ell}\cdot SMH, \quad \cdots\cdots\cdots\cdots(3)$$

where V_j: volume of the j-th element, ρ: unit weight of steel material, C: coefficient for unit cost of steel material, CM: unit cost of steel material, SMH: unit cost for one man hour work, H_{ij}:work man hour of the i-th manufacturing operation of the j-th element as a function of design variables, $\tilde{H}_{kl}$: work man hour of the k-th manufacturing operation of the l-th element as a fixed value. When, C is considered as a function of T, S and B, and C_1, C_2 and C_3 indicate the case of the function of T, C, the case of the function of S, C and the case of the function of B, C, respectively, the following expressions are obtained:

$$\left.\begin{array}{ll} C_1(T)=0.0348T^2-0.0845T+1.2091 & \\ C_2(S)=0.27S-0.08 & \\ C_3(B)=1.0 & \text{for } B<200(\text{cm}) \\ C_3(B)=1.0+(B-200)/(0.3\ \text{CM})\times 0.01 & \text{for } B\geqq 200(\text{cm}) \end{array}\right\} \cdots\cdots\cdots\cdots (4)$$

where T: plate thickness, B: plate width. H_{ij} can be considered as a function of S, T, W and A_r, where W: weight, A_r: surface area. HA is the coefficient of work man hours depending on S, and the following equation may be obtained:

$$H_{ij}=H_{ij}\times HA(S), \cdots\cdots\cdots\cdots (5)$$

where

$$HA(S)=0.04S^2-0.29S+1.52. \cdots\cdots\cdots\cdots (6)$$

The coefficients of Eqs. (4) and (6) are obtained on the basis of actual examples at a bridge fabricating shop in Japan. In the case of welded joints, the work man hours of butt welded joint, H_{ij}, and fillet welded joint, H'_{ij}, become a function of total welded length, but their calculation is to be made with a ratio, η, of equivalent welded length to 6^{mm} fillet. Assumed as a function of T, H_{ij} and H'_{ij} are calculated by the following equations, that is, in the case of butt welds,

$$H_{ij}=H_{ij}(L_1),\ L_1=L_1\times\eta_1(T),\ \eta_1(T)=1.2T^2+3.8T+1.3 \cdots\cdots\cdots\cdots (7)a$$

and in fillet welds,

$$H'_{ij}=H'_{ij}(L_2),\ L_2=L_2\times\eta_2(T),\ \eta_2(T)=0.0476T^2+0.1952T+0.7572 \cdots\cdots\cdots\cdots (7)b$$

where L_1, η_1: in the case of butt welds, total equivalent welded length and ratio of equivalent welded length, respectively; L_2, η_2: in the case of fillet welds, the same as L_1, η_1. H_{ij} for marking and painting may be considered a function of A_r.

The procedure of this optimum design is shown by a block flow chart as in Fig.1, in which X shows the variables to be computed by the simplex method and X^0 shows their initial values.

At this study, single main girders without lateral bracings and sway bracing are treated, because omitting of the bracings does not affect the optimum value of total cost.

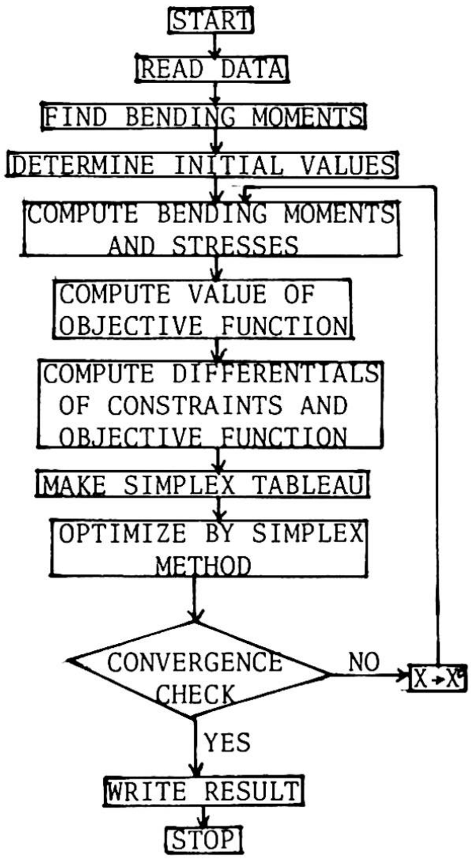

Fig.1. Flow chart of optimum design of girders

3. EXAMPLE OF OPTIMUM DESIGN

3.1 The conditions of design are given as follows:
1) type: I-shaped and deck-type welded railway plate girder, 2) live load: KS18 specified at the Railway Bridge Specifications in Japan, 3) span length: 5 kinds of span length, 16^m, 19^m, 22.3^m, 25.5^m and 30^m, 4) specifications: the Japanese Specifications for Design of Steel Railway Bridges.[3)]

It is assumed that a girder can be provided with three kinds of variation of sections as seen in Fig.2 with NA=2, 3 and 4, in which NA means the number of different girder sections. The upper cover and flange plates are symmetrical with the lower plates.

σ_{at}, σ_{ac} and γ are respectively given at the Specification as:

$$\left.\begin{array}{l}\sigma_{at}=(0.125S^2-0.792S+2.168)\times 1300 \\ \sigma_{ac}=50S^2+50.5S+199-(0.2S^2-1.3S+2.5)\times(\ell/B_f)^2\end{array}\right\} \cdots\cdots\cdots\cdots (1)a$$

Table 1. Comparison of optimum values for span length of 2230^{cm}

SMH (1000 yen)	NA	S_1	S_2	S_3	T_c (cm)	T_{f1} (cm)	T_{f2} (cm)	T_{f3} (cm)	T_w (cm)	B_c (cm)	B_{f1} (cm)	B_{f2} (cm)	B_{f3} (cm)	B_w (cm)	$SLEG_1$ (cm)	$SLEG_2$ (cm)	$SLEG_3$ (cm)	$SLEG_4$ (cm)
0.0	2	4.0			2.00	1.86			1.19	48.1	44.7			183.8	740.2	1115.0		
1.6	2	4.0			1.83	2.09			1.15	43.9	50.2			178.5	672.5	1115.0		
3.2	2	4.0			1.62	2.33			1.11	38.9	56.0			172.4	589.2	1115.0		
0.0	3	4.0	4.0		1.95	2.00	1.40		1.15	46.8	48.0	29.6		177.8	712.6	1027.2	87.8	
1.6	3	4.0	4.0		1.80	2.29	1.40		1.07	43.2	54.9	27.8		166.5	644.1	1027.2	87.8	
3.2	3	4.0	4.0		1.65	2.35	1.40		1.09	39.7	56.4	28.2		169.3	597.6	1027.2	87.8	
0.0	4	4.0	4.0	4.0	1.58	2.29	1.86	1.40	1.14	38.0	54.9	44.7	29.5	177.2	583.0	766.3	150.0	199.1
1.6	4	4.0	4.0	4.0	1.45	2.41	1.83	1.40	1.12	34.9	57.9	44.0	29.0	173.7	533.3	788.9	137.8	188.3
3.2	4	4.0	4.0	4.0	1.30	2.68	1.82	1.40	1.11	31.2	57.3	43.6	28.6	171.8	472.7	800.4	132.3	182.3

SMH (1000 yen)	NA	δ_1 (cm)	δ_2 (cm)	Z (1000 yen)	Weight (ton)	β	σ_{a1} (Kg/cm²)	σ_1 (Kg/cm²)	σ_{a2} (Kg/cm²)	σ_2 (Kg/cm²)	σ_{a3} (Kg/cm²)	σ_3 (Kg/cm²)	σ_{a4} (Kg/cm²)	σ_4 (Kg/cm²)	α	$\frac{SLEG_1}{0.5L}$	$\frac{SLEG_2}{0.5L}$	$\frac{SLEG_3}{0.5L}$
0.0	2	2.21	1.85	288.3	4.92	1.01	1155	1155	1166	1166					12.1	0.664		
1.6	2	2.30	1.93	555.6	4.91	0.97	1170	1170	1174	1174					12.5	0.603		
3.2	2	2.40	2.01	804.9	4.95	0.93	1184	1184	1179	1179					12.9	0.528		
0.0	3	2.30	1.92	284.1	4.83	0.96	1165	1165	1171	1171	1122	517			12.5	0.639	0.921	
1.6	3	2.48	2.08	571.5	4.83	0.88	1182	1182	1178	1178	1111	607			13.4	0.578	0.921	
3.2	3	2.45	2.05	841.0	4.83	0.90	1185	1185	1179	1179	1114	583			13.2	0.536	0.921	
0.0	4	2.34	1.96	270.1	4.58	0.96	1182	1182	1178	1178	1166	1166	1122	1122	12.6	0.523	0.687	0.134
1.6	4	2.40	2.01	578.2	4.57	0.94	1188	1188	1180	1180	1165	1165	1119	1119	12.8	0.478	0.708	0.124
3.2	4	2.42	2.03	873.7	4.58	0.92	1186	1186	1180	1180	1165	1165	1117	1117	13.0	0.424	0.718	0.119

$\gamma=2.5S^2-47.5S+305$ ··(2)a

where ℓ represents a distance between fixed points of a flange plate. If α is assumed to be a value of a span length divided by a web height, the initial values of B_W, B_C, T_C, B_f and T_f are calculated under the assumption of α, but the initial values of S and locations of joint are given as constant values independently of α. Now, the calculation of the initial values by a computer, makes it possible to do an automated design.

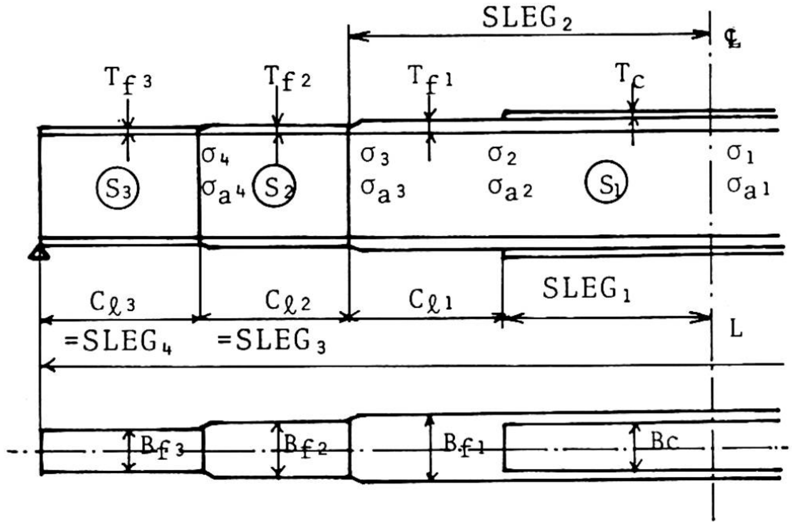

Fig.2. Notations for deck plate girder

3.2 Results of Calculation

As an example of the results of calculation, the case of span length of 22.3^m are summarized in Table 1, in which SLEG: values shown in Fig.1, δ_1:deflections due to live load and dead load at the span center, δ_2: deflections due to live load at the span center, Z: values of objective function, β: coefficients to be given later.

3.3 Discussion

As the result, the followings are discussed:

(1) The materials were considered as the design variables too, but the calculation shows that the case of S=4, namely SS41 steel will give optimum values.

(2) In the case of material cost only, the value of an objective function becomes the cheaper, with an increase of the number of different sections. On the other hand, in the case of material cost and fabrication cost, the value of the objective function becomes the higher and the girder weight becomes the lighter, with an increase of the number of different sections.

(3) Conventionally a web height B_W used to be expressed in terms of the following relation:

$$B_W=\beta\sqrt{\frac{M}{\sigma\cdot T_W}} \qquad \cdots\cdots\cdots\cdots(8)$$

where β: coefficient, M: bending moment. The values of β,calculated by the optimum values, are shown in Table 2. They do not change greatly as to span lengths, but generally become the larger , the longer the span length is.

Table 2. Values of coefficient β

SMH	NA \ L	1600^{cm}	1900^{cm}	2230^{cm}	2550^{cm}	3000^{cm}
0.0	2	0.96	0.97	1.01	1.02	1.00
	3	0.93	0.94	0.96	0.99	1.00
	4	0.92	0.94	0.96	0.98	1.03
1.6	2	0.92	0.94	0.97	0.97	1.01
	3	0.89	0.88	0.88	0.91	1.01
	4	0.89	0.93	0.94	0.93	1.04
3.2	2	0.92	0.93	0.93	0.95	1.02
	3	0.89	0.90	0.90	0.90	1.03
	4	0.87	0.89	0.92	0.94	1.05

(4) The values of SLEG/0.5L are shown in Table 3, where L: span length. In the table, $SLEG_1$ is the shorter, the higher SMH is, while flange plate lengths do not change greatly.

Table 3. Values of SLEG/0.5L

SMH	NA	SLEG \ L	1600^{cm}	1900^{cm}	2230^{cm}	2550^{cm}	3000^{cm}
0.0	2	$SLEG_1$	0.651	0.661	0.664	0.671	0.681
	3	$SLEG_1$	0.625	0.631	0.639	0.645	0.658
	3	$SLEG_2$	0.921	0.921	0.921	0.921	0.921
	4	$SLEG_1$	0.492	0.505	0.523	0.539	0.558
	4	$SLEG_2$	0.668	0.679	0.687	0.696	0.698
	4	$SLEG_3$	0.140	0.140	0.134	0.131	0.117
1.6	2	$SLEG_1$	0.554	0.582	0.603	0.619	0.641
	3	$SLEG_1$	0.537	0.565	0.578	0.594	0.611
	3	$SLEG_2$	0.921	0.921	0.921	0.921	0.921
	4	$SLEG_1$	0.447	0.455	0.478	0.492	0.489
	4	$SLEG_2$	0.700	0.698	0.708	0.715	0.693
	4	$SLEG_3$	0.115	0.124	0.124	0.128	0.118
3.2	2	$SLEG_1$	0.492	0.489	0.528	0.558	0.588
	3	$SLEG_1$	0.485	0.506	0.534	0.548	0.556
	3	$SLEG_2$	0.921	0.921	0.921	0.921	0.921
	4	$SLEG_1$	0.356	0.373	0.424	0.440	0.396
	4	$SLEG_2$	0.694	0.717	0.717	0.707	0.673
	4	$SLEG_3$	0.148	0.138	0.120	0.131	0.143

(5) Except for the value of σ_3 in NA=3, the other maximum working stresses reach up to the full allowable stresses. σ_3 does not become fully-stressed, because the flange plate at this position is determined by its minimum thickness 1.4^{cm} and by its width calculated at $B_f \geqq B_w/6$.
(6) T_c and B_c are the smaller, the higher the fabrication cost is. On the other hand, T_{fi} and B_{fi} are the larger, the higher the fabrication cost is. There is no remarkable difference due to the difference of fabrication cost at the dimension of flange section at the other positions.
(7) The leg length at fillet welds is the smaller and the fabrication cost is the cheaper, the wider the flange plate is and the thinner the flange plate is.
(8) At the present example of design, the optimum dimensions of section for 5-kinds of the span length are calculated, but they can be calculated for the other span lengths by the following procedure. B_w is calculated from Eq. (8) by assuming σ and T_w, and using M and β. Then, the position of joint is obtained from Table 3, and except T_{f2} and B_{f2} in NA=3, the dimension of girder section at the span center and all of the positions of joint can be found by the fully stressed design. However, in NA=3, $T_{f2}=1.4^{cm}$ and $B_{f2}=B_w/6$ or $B_{f2} \geqq 24^{cm}$ are applied.
(9) As seen in the value of β, for the case of material cost only the optimum girder height varies, but for variable unit fabrication costs it does not greatly vary.

4. CONCLUSION

It is indicated that it is possible to carry out the optimum design considering material cost and total shop fabrication cost by means of a program for computer design which is presented at the present study, and it will be possible to extend this program to the computation for a girder with different upper flange section from lower flange section, a composite girder and a continuous girder; and a part of this program has been completed already.

At the present program, transportation and erection costs depending on site conditions are omitted, but in the future, in the case of a specific or individual bridge, it would be necessary to investigate on an overall cost optimum design containing the transportation and erection costs.

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2) Y. Ochi, "Tecnique of Computer Analysis and Its Application", Morikita, Tokyo, 1970, pp. 155-164 (in Japanese).
3) Japan Society of Civil Engineers, "The Specifications for Design of Steel Railway Bridges", 1974, (in Japanese).

SUMMARY

A program of optimum design considering material cost and total shop fabrication cost for the most foundamental I-section girders of steel bridges is presented with design examples. The influence of 16 design variables on the total cost is discussed, to improve the computer-aided automated optimum design.

RESUME

Un programme de dimensionnement optimal tenant compte du coût des matières et des coûts de fabrication est appliqué aux plus élémentaires sections en I des poutres de pont métallique. Des exemples sont donnés. L'influence de 16 variables de dimensionnement sur les coûts totaux est étudié afin d'améliorer le dimensionnement optimal à l'ordinateur.

ZUSAMMENFASSUNG

Es wird ein Programm für die Optimierung von I-Stahlträgern mit Rücksicht auf Material- und Herstellungskosten präsentiert und dessen Anwendung an Beispielen dargelegt. Der Einfluss von 16 Entwurfsvariablen auf die Totalkosten wird untersucht, um die computerunterstützte Entwurfsoptimierung zu verbessern.

Optimierung von Eisenbahnfachwerkbrücken

Optimization of railway truss girder bridges

Optimisation de ponts ferroviaires en treillis

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1. Grundsätzliches

Bei den praktischen Optimierungsaufgaben kann man grundsätzlich drei verschiedene Verfahren anwenden [1] .

Bei dem ersten Verfahren werden einige zweckmässig ausgewählte Varianten durchgearbeitet und die Ergebnisse miteinander verglichen. Ein erfahrener Entwurfsingenieur gewinnt auf diese Weise mit erträglichem Arbeitsaufwand eine ausreichende Übersicht.

Bei dem zweiten Verfahren versucht man, die Beziehungen zwischen der vorgegebenen Belastung und Geometrie der Konstruktion einerseits und Konstruktionsabmessungen oder Kosten andererseits mathematisch zu erfassen. Eine ausführliche Beschreibung der dazu anwendbaren mathematischen Methoden ist im Einführungsbericht dargelegt [2]. Jedoch weist dieser mathematischer Weg zwei grundsätzliche Nachteile auf. Durch die unumgängliche Vereinfachung und Idealisierung zu komplizierter mathematischer Beziehungen werden die Ergebnisse in meist unübersehbarer Weise unscharf und gelegentlich sogar fehlerhaft. Ferner ist der praktische Entwurf einer Konstruktion durch die vorgegebene Dispositionsforderungen, das Walzprogramm, die Standartsbestimmungen, verschiedene Konstruktionsrichtlinien und übliche Durchführung der Details usw. weitgehend eingeengt. Die Möglichkeit der Anwendung der allgemeinen mathematischen Methoden [2] , die meist nur durch Einsatz moderner Computer denkbar ist, ist bei praktischen Beispielen oft nicht gegeben.

Daher wurde in letzter Zeit ein drittes Optimierungsverfahren entwickelt [1] , dass die Kapazität moderner Computer in anderer Weise ausnützt und die Vorteile der beiden beschriebenen Methoden vereinigt. Man stellt dabei ein Programm auf, das den Entwurfs- und Bemessungsprozess des untersuchten Konstruktionstypes nachbildet. Dabei ist es nicht schwierig, z.B. die richtigen Werte der Knickzahl, die Abstufung des gültigen Walzprogrammes, verschiedene Richtlinien und Normbestimmungen, übliche Konstruktionsdetails usw. zu berücksichtigen. Durch Variieren der Eingangsparameter stellt man ziemlich leicht den Bereich von optimalen Lösungen fest, die dem angestrebten Minimum der Untersuchten Zielfunktion (Materialverbrauch oder Kosten oder Arbeitsaufwand) nahe liegen.

2. Praktische Anwendung des neuen Verfahrens bei Stahlbrücken

Das neue Verfahren wurde zuerst zur Optimierung der Verbundträger angewandt. Hier hängt der Stahlverbrauch praktisch nur von der Höhe des Trägers und von der Schlankheit seines Steges ab. Daher konnte man hier unter Verbrauch von wenigen Minuten der Computerzeit die optimalen Querschnitte von Eisenbahn- oder Strassenbrücken, für Verbundträger oder auch Verbundkastenträger feststellen.

Bei Fachwerkbrücken war die Anwendung des neuen Optimierungsverfahrens durch die grössere Zahl der Eingangsparameter umständlicher. Es wurden die üblichen Trägerform nach Abb.1, drei Fahrbahntypen (offene Fahrbahn, direkt befahrene mit den Hauptträgern mitwirkende Blechfahrbahn und durchgehendes Schotterbett auf einer mitwirkender Blechfahrbahn), mit geschlossenem oder offenem Brückenquer - schnitt, Ein- und Zweigleisbrücken und wirtschaftliche Kombination der Stahlsorten St 37 und St 52 bis zur Spannweite von L 100 m untersucht. Es hat sich dabei eindeutig ge - zeigt, dass die optimale Trägerform mit dem minimalen Stahlver - brauch, evtl. minimalen Baukosten der tragenden Konstruktion vor allem von der Spannweite L , von der Felderzahl N , von der Trägerhöhe H und von der Höhe v der idealisierten Stabquer - schnitte (Abb.2) abhängt, während der Einfluss der evtl. be - schränkten Konstruktionshöhe des Fahrbahnrostes und der Grösse des Konstruktionbeiwertes vernachlässigbar klein erscheint.

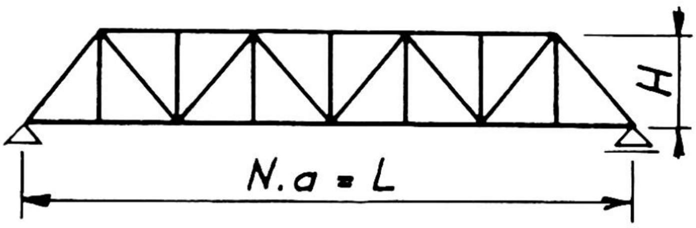

Abb.1

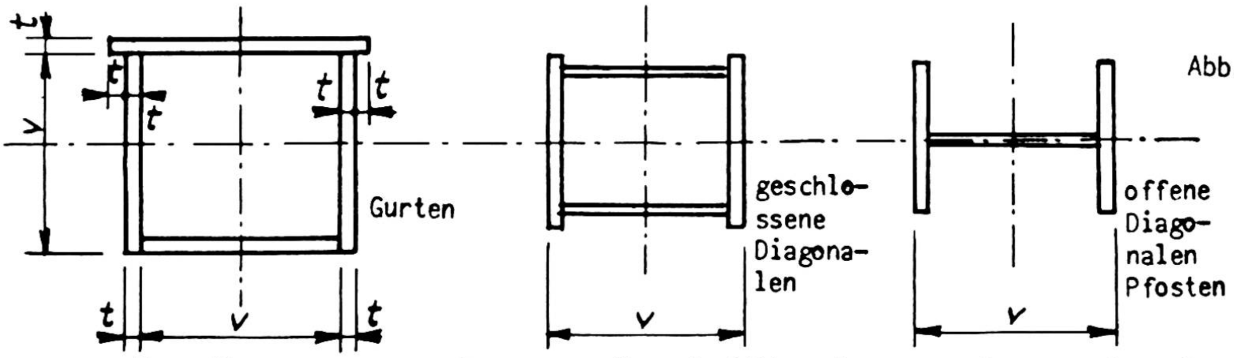

Abb.2

Das Programm wurde so aufgestellt, dass nach der Angabe von L , N und Eigengewicht der Fahrbahn zuerst die geometrische Form für eine ziemlich niedrige Trägerhöhe H berechnet wurde, dann wurden die Längs- und Querträger berechnet und dimensioniert. Nach der Ermittlung von Stabkräften wurden einzelne Stabquerschnitte mit ziemlich kleinem Wert von v dimensioniert. Das resultierende Gesamtgewicht wurde mit dem anfänglichen aus empirischer Formel eingesetzten Wert verglichen; wenn der Unterschied grösser als der vorgegebene Wert war, wurde die ganze Dimensionierung mit korrigierten Werten wiederholt. Zuletzt wurde die Durchbiegung kontrolliert und die Anstrichsfläche festgestellt.

Im weiteren Schritt vergräserte das Computer des Ausgangswert v um Δv, wodurch die Senkung der Zielfunktion Z , d.h.

des Stahlverbrauches oder der Kosten erzielt wurde. Man wiederholte dann die Vergrösserung von v einigemal, bis das Minimum von Z erreicht wurde. Dadurch wurde das Optimum für bestimmte Höhe H festgestellt.

Nachher vergrösserte das Computer den Ausgangswert H um ΔH, wodurch wieder die Senkung des Wertes Z erreicht wurde; diese Iteration wurde so lange wiederholt, bis der Endwert von Z grösser war als sein Wert bei dem ersten Iterationschritt (Abb.3).

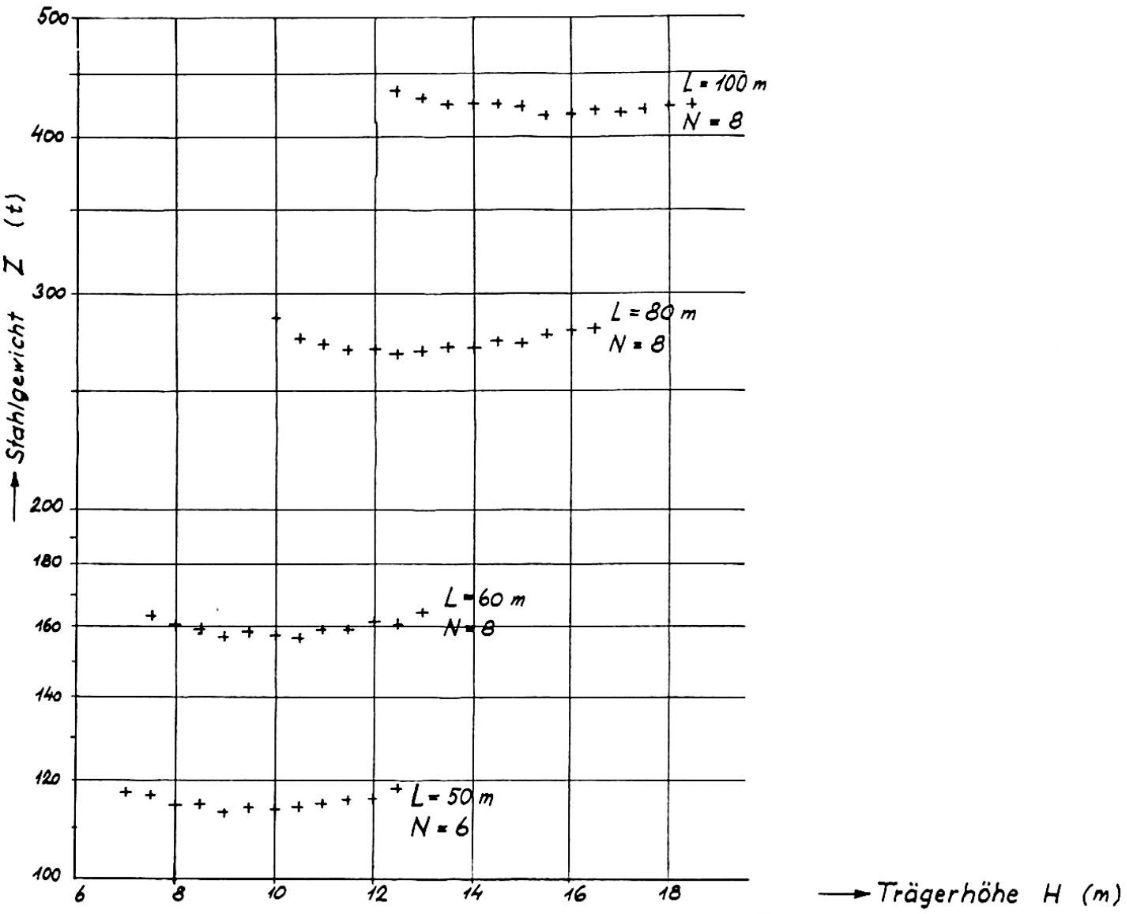

Abb.3

Der Abb. 3 kann man entnehmen, dass die Streung der Werte Z in der Umgebung vom Minimum sehr flach ist und somit die Grösse von Z auf kleine Variationen der Trägerhöhe H nicht empfindlich ist. Deshalb ist es angebracht, nicht von einer optimalen Höhe zu sprechen, sondern von dem Bereich B von optimalen Höhen, dessen Breite durch die Differenz Z festgelegt wird (Abb.4). Zum Beispiel für die Differenz von $\Delta Z = 0{,}02\ Z$ wurden die unteren und oberen Grenzen des Bereiches von optimalen Höhen wie folgt festgestellt :

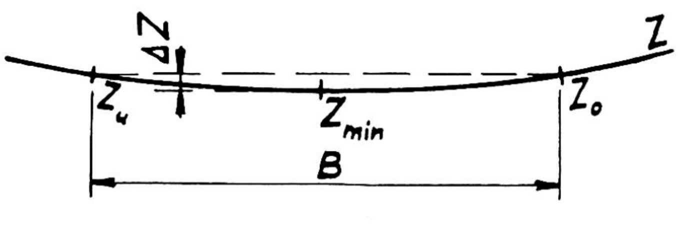

Abb.4

Querschnitt der Brücke	Spannweite L (m)	Feldteilung N (-)	offene Fahrbahn		direkt befahrene Fahrbahn		durchgehendes Schotterbett	
			Grenze des Bereiches der optimalen Höhe H/L					
			untere	obere	untere	obere	untere	obere
offen	50	8	1/8,5	1/6,0	1/9,1	1/6,3	1/9,3	1/5,8
geschlossen	50	8	1/7,1	1/4,9	1/7,4	1/4,9	1/7,5	1/4,8

Was die optimale Kombination des üblichen Stahles St 37 mit Stählen höherer Festigkeit betrifft, ist deren Einsatz nur bei jenen Stäben wirtschaftlich, bei welchen die Stahlverbrauchser - sparnis höher als der zuständige Preisunterschied der fertigen Konstruktion ist. Somit ist es wirtschaftlich, bei Spannweiten von 40 bis 100 m, bei offenen Fahrbahnen beide Gurtungen, bei einer Blechfahrbahn die obere Gurtung und die Längsträger aus St 52 zu entwerfen, sowie auch die "schweren" Diagonalen in der Nähe von Stützen der Brücken mit grösseren Spannweiten.

3. Folgerungen

Es wurde gezeigt und am Beispiel einer Eisenbahnbrücke demonstriert, dass bei den Konstruktionen, deren Kosten nur von wenigen Eingangsparametern abhängen, während viele andere Parameter der Konstruktion mit der Spannweite, mit dem Konstruktionstyp und -zweck zusammenhängen und nicht viel veränderlich erscheinen, vorteilhaft ist, das Berechnungs- und Bemessungsprozess des Entwurfingenieurs in einem Computerprogramm nachzuahmen und den Bereich der optimalen Lösungen durch Variieren der Eingangsparameter festzustellen.

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ZUSAMMENFASSUNG

Es werden drei Optimierungsverfahren definiert und die Anwendung des dritten Verfahrens am Beispiel der Eisenbahnfachwerkbrücken erläutert.

SUMMARY

Three ways of optimization are presented. An application is demonstrated on railway truss girder bridges.

RESUME

On définit trois procédés d'optimisation. L'usage du troisième procédé est démontré pour des ponts ferroviaires en treillis.

Total Computer System for Bridges

Système global pour le projet de ponts au moyen de l'ordinateur

Integrales Computersystem für Brückenentwurf

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1. Optimum design and automated design

Automated design has been studied as a part of automation and labor saving problems by those who are engaged mainly in the practical design work, while the optimum design has been researched and developed by researchers who study mainly the mathematical decision method in connection with this design work. However, it is unreasonable to say that design by the automated design system does not have to be optimum design. If optimum design should be used for a practical application, its concept and method should be used for the automated design system and, therefore, we believe that they should be combined.

Under the present conditions, where the method of optimum design is not employed extensively in practical fields, whether the results of study are adopted or not is decided by designers and, in many cases, the designers modify the results before they utilize them. The practical design work is undisciplined and, in most of cases, constraints and objective function are never represented by well arranged formulas and thus contain many factors which depend upon the man's intuition and, therefore, the CAD system is conveniently used for ensuring a smooth execution of design work. At the present time, the practical method by which we can most probably ensure constant and high quality design is the CAD system which is processed in such a way that the method of optimum design is used for deciding algorism of automated design, allowing the system to supply designers with the data necessary for them to make judgements and, according to such data, the system proceeds on the basis of the man-machine relationship.

Whether a designer can accomplish high quality design by using such a system or not depends on (1) whether the ability of the designer who utilizes this system is proper or not or (2) whether the system can conveniently and quickly supply the required data in an easily usable form and if the system can fully carry out "trial and error" in a short time.

Combination of the above methods is indispensable for the improvement of quality of design and the mathematical decision method is also an indispensable factors.

Even if data of the best quality, when viewed from the standpoint of optimum design, is not supplied from the system, it is expected that the designers may be able to accomplish a design of a considerably high quality, if he can use the system conveniently, which means utilizing both mathematical decision method and the judgement of the designers. Under these conditions, the writers of this report have developed the CAD system for bridge design and used it for practical applications. The following describes the design system of a girder bridge.

2. Design system of girder bridge

2.1 Outline

Most ordinary bridges are of the girder bridge type and, therefore, it is necessary to prepare a system which can be used conveniently and withstand the changes, additions and deletions of shape data, designing conditions, manufacturing conditions, etc.

The overall system consists of four subsystems as shown in Fig.1 which are consistently controlled through the data base. Emphasis has been placed on partial optimizing and data that can be used conveniently and utilized easily by designers.

2.2 ROAD Sub System

This is a universal type system of coordinate calculation. When the form of road, pier layout, main girder and cross beam arrangement are defined, this ROAD Sub System calculates the required values of coordinates. Consequently, the table of values, plan, longitudinal section and cross section are supplied as an output. For the following systems, various figures are filed in phase with each value being taken into consideration.

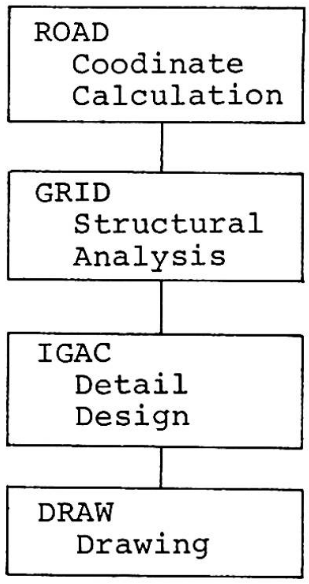

Fig. 1

2.3 GRID Sub System

This system is a structural analysis system which employs a displacement method. When the input of the displacement method is fed independently, the coordinates, stiffness, loads, etc. are mostly fed as input data as far as the GRID is concerned, which is rather complicated for the designers. As for the matters concerning the coodinates, especially, since the results of the above ROAD Sub System are handed from the file, the input load is greatly alleviated.

When girder height is fed into this system as an input, a preliminary analysis is made for a simplified model structure by the stress-method as a preparatory calculation. An assumed stiffness and steel weight are set automatically and, thereafter, the number of input joints is about 200, thus requiring about 20 cards.

2.4 IGAC Sub System

Detailed design is conducted for the main girder section, splices, stiffeners, shear connectors, sway bracings and lateral bracings. As for the coordinates and sectional force, the results of the previous system can be used and, therefore, the designers feed the assembling method of sway bracing and lateral bracing as an input. It is also possible to make various kinds of special designations. Usually, when 10-20 cards are fed as an input, the optimizing process is carried out in the system, one set is decided upon and the design calculation sheet and sectional variation diagrams are produced as an output and filed. However, the designer's personal taste, interchangeability of parts, etc. should also be taken into consideration when the decision is made and, therefore, there arises a demand that some modification should be made after studying the outputs. Meanwhile, questions and modifications can be made by using CRT(IBM 2250).

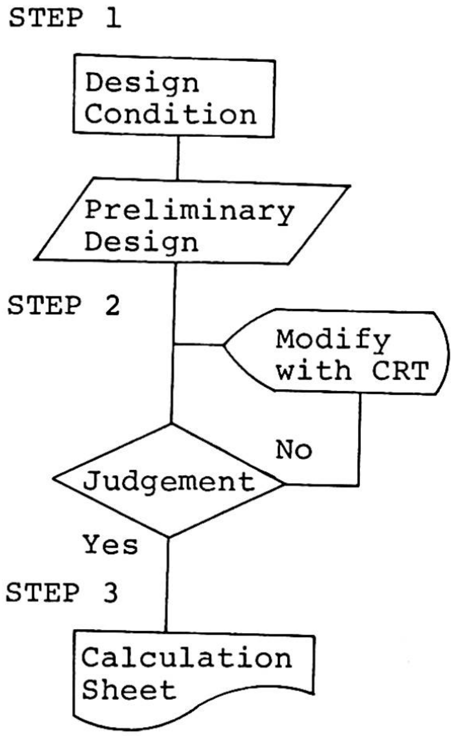

Fig. 2

This system consists of the following three steps;

Step 1; Temporary decision concerning the main girder, cross beam and lateral bracing, preparation of data to be studied(substitute plan included) and filing into Step 2.

Step 2; Question and modification by using CRT device. Filing into Step 3.

Step 3; Preparing a design calculation sheet. Filing into DRAW Sub-System.

Step 2 is provided with the CRT pictures of sections, splices, stiffeners, shear connecters, cross beams and lateral bracings. In one particular section, for example;

a. What kind of section can be made if the material at a certain location is changed?
b. What will be the best section if this location is moved 30cm?
c. What will be the thickness of plate when the upper flange width is changed to 50cm?

Various questions such as are listed above are given and if the answers from the system are accepted, the files are renewed accordingly and, thus, the design is modified continuously. Then, the final results are filed for the DRAW Sub System of design drawing.

The final stiffness is filed and the GRID can be reopened by using the file.

Fig.3 An example of the CRT pictures

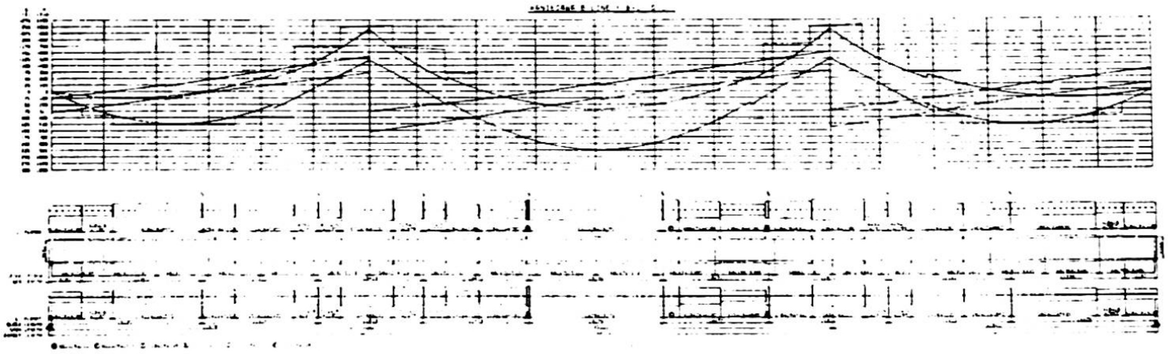

Fig.4 Sectional variation diagram

2.5 DRAW Sub System

This is a system which is used for deciding the details of structures and for making drawings. The results of design calculation are insufficient when they are used as the data for making drawings and, therefore, the mode of structure should be decided in detail. In many cases, however, the details of structure are different depending on each customer. Because of these reasons, the details of structure most often used to meet the standards and design requirements of customers are stored in the system, thus expanding the range of applications. The input designates the items which change the standard of the systm . As for the coordinates, the file is used as a reference and, therefore, the designated item is usually represented by about 10 cards. The outputs are; main girder, cross beam, lateral bracing, detailed design drawing, diagram and the list of steel materials, welding lengths, painting area, etc.

For making the drawings, COM(Computer Output Microfilming) of CALCOMP CO. is used. Unlike the plotter or the drafter in which a pen moves mechanically, this COM is so designed that the locus of an electronic beam is traced on film. One drawing is completed in about five seconds and the operating cost is also very low.

3. Postscript

With this system, the fundamental design(deciding girder arrangement, girder height, etc.) is made after full "traial and error" by means of the ROAD and GRID and, then, the detailed design made by IGAC system is corrected by means of CRT and drawings are made by DRAW. The fundamental design and detailed design are separated, but when the optimum property of design is taken into consideration, we do not believe that there will be much trouble in the actual application if the fundamental values are properly selected.

When this system is used, one designer can complete within one week about 50 drawings, material lists and design calculation sheets for a bridge constructed with five main girders and three span-continuous I girders. Only girder bridges, are described in this report.

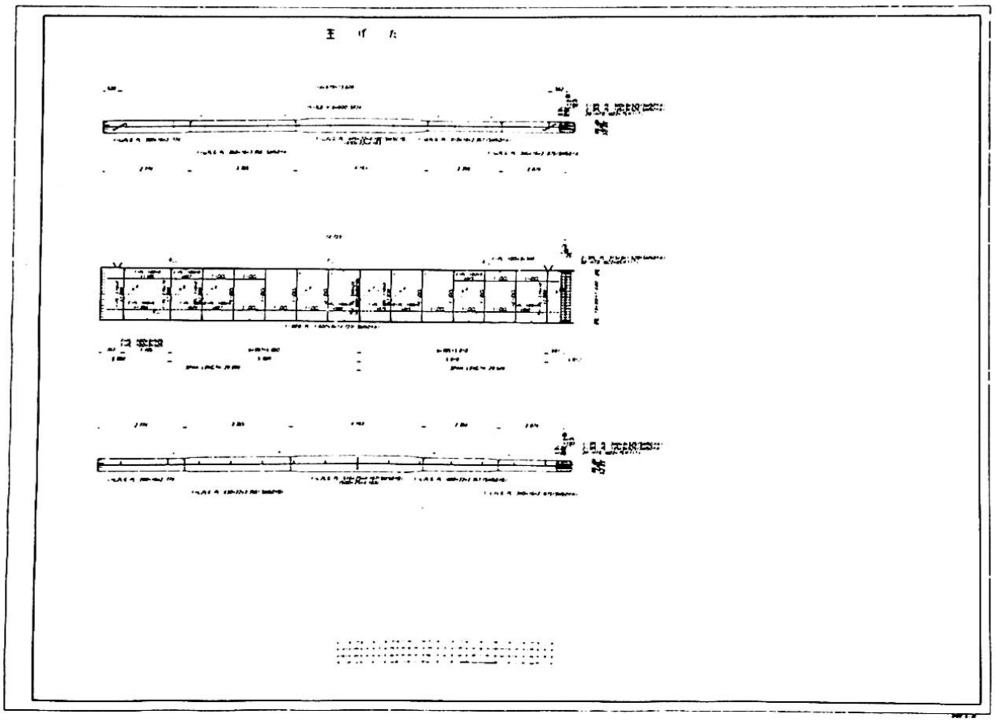

Fig.5 A drawing of main girder by COM

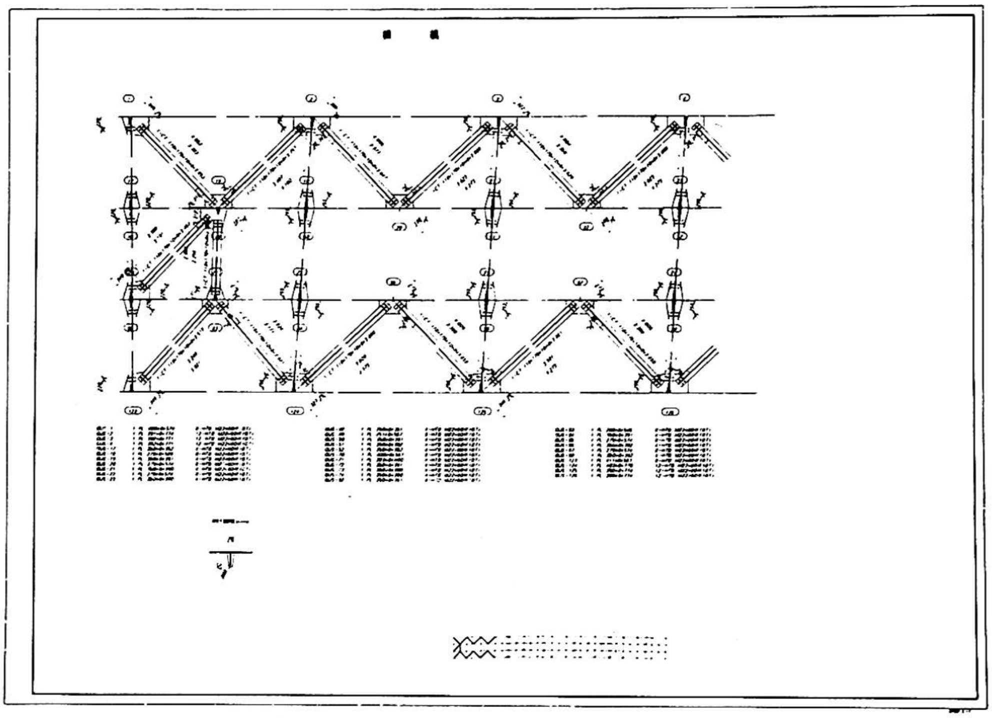

Fig.6 A drawing of lateral bracings by COM

4. REFERENCE

1. Ueno, Mikami, Tanaka, Kamemura and Maruyasu: "Total Computer System for Bridge Design and Loft" Civil Engineering Society Bulletin, No.65, Feb. 1974

2. Sasaki, Sugawara, Tanaka, Shoji, Kamemura, Oishi, Maruyasu and Mayuzumi: "Total Computer System for Bridges", Nippon Kokan Technical Report Overseas, No.21, Dec. 1975

SUMMARY

A "Total Computer System for Bridges" has been developed, which is aimed at combining the optimum design and the judgement of designers. This system has already been used for actual applications and has produced good results. This report introduces design of a girder bridge in the overall system.

RESUME

Un système global pour le projet de ponts au moyen de l'ordinateur a été développé. Ce système combine le calcul optimal et le jugement de l'ingénieur. Il est déjà utilisé en pratique et donne des résultats excellents. Cet article présente la partie du projet de pont en poutres dans le système global.

ZUSAMMENFASSUNG

Ein totales Computersystem für einen optimierten Brückenentwurf wird entwickelt. Das System verbindet die Absichten des Entwurfes mit einer optimalen Problemlösung; in praktischer Anwendung hat das System bereits gute Resultate geliefert. Am Entwurf von Brückenträgern stellt der vorliegende Beitrag einen Teil des gesamten Computersystems vor.