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Progrès dans l'optimisation structurale

Fortschritte in der Optimierung von Tragwerken

Progress in Structural Optimization

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

A.B. TEMPLEMAN
Department of Civil Engineering
University of Liverpool
Liverpool, Great Britain

Optimization Concepts & Techniques in Structural Design

In preparing the Introductory Report for Theme II, 'Progress in Structural Optimization' the three reporters each took a specific aspect of the theme. My objective was to provide an introduction to Structural Optimization; to describe its philosophical goals and to outline in brief and simple terms some of the mathematical techniques which are most frequently used to solve structural optimization problems. It was intended to be a 'beginner's guide' to the topic which would be expanded in more detail by the other Reporters and by the authors of papers in the Preliminary Report.

I will not dwell on an introduction to structural optimization but will assume that you are familiar with it from the Introductory Report. Towards the end of the Report I have remarked that by 1970 most of the simple problems of structural optimization had been solved, only the difficult ones being left. I think this point is demonstrated very well by the six papers in the Preliminary Report under Theme IIa. None of them deals with simple, straightforward problems; they all are concerned with difficult aspects of the topic and they all give a very fair indication of the present-day complexities of structural optimization.

I would like to consider first the papers by Anraku and by Balasubramonian and Iyer since they represent the forefront of technologically difficult problems. Anraku is concerned with designing steel frames to withstand dynamic earthquake loadings. Balasubramonian and Iyer are concerned with random variable variable loadings. It is significant that throughout my Introductory Report I have not mentionedloadings such as these. Much of the research done in structural optimization over the last twenty years has considered only deterministic static loads. Only in the last few years have researchers begun to look at optimum design for dynamic or non-deterministic loads. The reason for this is that the complexity of the problems increases dramatically as one moves away from deterministic static loads and this is an area of work which still requires much research. It is also an essential area for future research. Structural design methods and codes of practice are now moving towards a greater recognition of the probabilistic nature of loadings. Limit state concepts in which safety factors against many different possible occurrences are assessed are also becoming widely accepted. If structural optimization is to retain any relevance for the practising engineer it is essential that it too should be able to handle dynamic loads, probabilistic loads and limit state concepts.

The papers by Anraku and by Balasubramonian and Iyer are therefore welcomed because they are pointing in the right direction for the future of practical structural optimization. However, Balasubramonian and Iyer's paper dealing with structural optimization under random loading effects is entirely theoretical. One of the main reasons why structural optimization methods are not now used more widely in practical design is that there has too often been a large gap between what is correct in theory and what works in practice. This is particularly so when using applied probabilistics. There is a world of difference between defining in theoretical terms the probability of failure of a structure and actually evaluating it accurately for a real-world structure. Nevertheless Balasubramonian and Iyer have made a start in rationalising the effects of random loadings.

Anraku's paper deals with optimum design of frames for earthquake loading - once again a technologically complex form of loading. Some codes of practice incorporate requirements for designing against earthquakes and Anraku is to be complimented on attempting to extend structural optimization into this difficult area of work. As an optimization method Anraku has used sequential linear programming. This method is often used when no other method is available or when the problem is very complicated. Unfortunately it is on these highly nonlinear problems that its performance is worst and it is evident from Anraku's paper that he has experienced difficulties with this method. He comments that an accurate analysis of the dynamic loading is essential if the method is to converge and it appears from his Figure 6 that his optimized design violates some design restrictions by as much as 20%. Both these effects are inherent in the sequential linear programming method. Any linearisation of a highly nonlinear model is bound to be both sensitive to error and inaccurate.

The paper by Brozzetti *et al* is a complete contrast to the preceding papers. It is concerned with a very practical, pragmatic approach to using a commercially available computer package program for designing steel structures. In particular they consider the minimum weight design of practical steel frames so as to satisfy a large number of limit state criteria. The paper highlights the philosophical point that structural optimization is not a mathematical discipline but is, and will always continue to be, an engineering discipline. The objective of structural optimization is to produce the best possible engineering structure. Sometimes precise mathematical methods will allow this to be done mathematically but usually the practical limitations of codes of practice, methods of construction and aesthetics.make a completely mathematical formulation of the design problem impossible. Here the expertise of the engineer is essential. Sometimes those researching new structural optimization methods ignore practical considerations or make dubious assumptions in order to force a practical problem into a mathematically amenable form. While this may be possible for research purposes it is not possible for practical design purposes. Practical structural optimization very often has to be an inexact process relying sometimes upon rigorous mathematics, sometimes upon heuristics and always relying upon engineering experience. Brozzetti *et al* do not describe their optimization technique in detail - it seems to be sequential linear programming but coupled with a lot of engineering knowledge in order to produce real-world structural designs. In their paper they demonstrate that in order to produce really economical designs it is necessary to include the nonlinear interactions of axial forces and bending moments in steel framed structures. Very often these interactions are ignored by researchers when studying these structures since they introduce awkward mathematical nonlinearities.

The remaining three papers all deal with almost classical topics in structural optimization. Structural optimization has always been concerned with two basic questions - one practical, the other more theoretical. The practical question is - 'How can I design the most efficient structure to perform a specific task?' The more theoretical question is - 'What are the funadmental laws which govern structural efficiency?' It is important to distinguish between these two questions and theoretical work which attempts to answer the second question should not be criticised because it seems irrelevant to practical design. Work in theoretical structural optimization is important and essential because it adds to our fundamental knowledge of structural behaviour. An increased awareness of why some structures are more efficient than others will eventually benefit practical design engineers but the immediate practical relevance of such work may not be apparent.

Nakamura and Nagase consider the optimum rigid-plastic design of multi-storey plane frames for multiple load cases. In my Introductory Report I mention in Section 4.4 that optimum rigid plastic design can be represented as a linear programming problem. Nakamura and Nagase have done this and have then considered some of the more advanced aspects of linear programming theory using duality theory in order to reduce the size of the problem and solve it rapidly. This area of work, optimum rigid-plastic design is much researched and it can truthfully be said that our knowledge of the mechanics of structures in the plastic régime has been greatly advanced by such work. Nakamura and Nagase have made an important contribution to this topic by considering multiple loading cases and their paper is well worth further study. They do not claim to be able to produce an optimum practical design but their method can be used for rapidly producing an efficient and economical initial design which can then be analysed and modified in minor ways to satisfy engineering criteria. I commend their treatment and uses of duality and I believe their work could be developed to form the basis of really efficient design programs for practical structural design.

The two final papers, one by myself and one by Lipp and Thierauf both deal with the same classical problem. How can one design truss-type structures for minimum weight in the presence of restrictions upon member stresses, nodal displacements, member size limits? The optimum design of trusses has always been a subject of much research for several reasons. First of all trusses are practical engineering structures and so it is a relevant area of work. Secondly, the problem is a nonlinear one of a most interesting mathematical form and thirdly the methods which can be used to design trusses can also, with minimal modifications, be used to optimally design certain classes of more complex finite element plate structures. Perhaps the major difficulty which any optimum truss design method has to face is that of problem size. For each truss member there is usually one variable (the cross-sectional area) for which an optimal value is to be found. Trusses of several hundred members are not uncommon so for these structures the optimum design problem expressed mathematically is nonlinear, has several hundred variables and even more constraints.

A straightforward numerical search for an optimum of such a large problem is not possible as it is wasteful of time and computer resources. Recently engineers have looked more deeply at this problem and have found that by examining the theory of optimality more carefully new, more rapid design methods for trusses can be developed. My own paper explores this topic further and describes how duality principles can be used to develop new design methods. The paper by Lipp and Thierauf is concerned with the same approach - indeed the mathematics of the two papers is remarkably similar. I do not have time in this summary to talk about the differences and similarities in these papers in detail but I would like to add a final comment. In my Introductory Report I mention that duality might prove to be a mathematical concept of great value to those interested in the optimum design of large structures. My own paper reflects this of course but it should be noted that the Lipp-Thierauf paper is also concerned with duality via the Lagrange multiplier technique thus strengthening my earlier opinion.

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Comments by the Author of the Introductory Report

Remarques de l'auteur du rapport introductif

Bemerkungen des Verfassers des Einführungsberichtes

FRED MOSES
Professor of Civil Engineering
Case Western Reserve University
Cleveland
Ohio, USA

System and Geometrical Optimization for Linear and Non-Linear Structural Behaviour

Please accept by apologies for being unable to attend and report personally to this meeting but circumstances beyond my control have led to my absence. I offer my best wishes for the success of this session and I thank in advance Dr. Templeman for substituting on my behalf.

Many discussions at earlier IABSE sessions have considered optimization. Professor Courbon defined optimization as designing and constructing a structure at the lowest cost with the object of fulfilling a well-defined purpose. Cost consideration must be given to safety, service life, maintenance and future adaptability. Within this broad context, the speciality of structural optimization arose to provide specific design purposes and methods which will aid in reaching an optimum structure. Thus, in the same way that matrix methods or finite elements aid in structural analysis, techniques of structural optimization have been developed to improve design procedures. Its applicability depends as much on reducing the ultimate cost of the structure as on savings in time and cost for the design engineer.

Historically, optimization has used simple design rules to check optimum designs. Gradually, more sophisticated mathematical methods applied with computer programs arose to systematically search and locate optimum structures.

A description of formal optimization methods taken from fields of mathematical programming and Operations Research has been presented by Dr. Templeman in his survey paper published in the Introductory Report. Such methods have found widespread application in the design of structural elements which are described by a number of design variables and constraints determined by codes of practice.

Figure 1 of my Introductory Report shows examples of such element designs. There is an example of a welded box girder for which I have had occasion to design large numbers for crane structures. Another example, is the welded plate girder which we designed based on the rather complex provisions for unbraced members in the AISC specifications. Also shown is a prestressed concrete beam with eleven design variables. The element design is controlled by contraints on loading and prestress force and deformations. Other element designs reported include welded columns, stiffened ship plates, shear walls, prestressed plates and reinforced concrete beams.

Element optimization has led to a number of computer programs whose function is to efficiently design a variety of elements and perform the tedious calculations required by the designer in trying to proportion such elements. The programs have usually been based on penalty or geometric programming methods of optimization. Professors Ohkubo and Okumura in their Preliminary Report paper have derived the optimum design of elements such as bridge girders and truss members using the method of sequential linear programming. This was then adapted by them to a branch and bound procedure for solving discrete variables such as steel type and flange thickness. A different approach to the optimization of element, in this case concrete bridges, is presented by Ulizkij and Jegoruschkin. It uses influence factors for predicting the behaviour of the bridge and therefore simplifies subsequent optimisation.

A combination of elements as in a total structural framework requires a different approach to optimization. Any changes in the design on the path to the optimum may subsequently require complete reanalysis of the structure to determine new stresses and deflections. In Figure 2 of my Introductory Report, a grillage is shown in which redistribution of forces occurs following each design change. The optimum design procedure for this case was reported by Moses and Onoda. Other examples of system optimization are statically indeterminate trusses and frames.

A system optimization, to be efficient, requires techniques such as the sequential linear programming shown by Ohkubo and Okumura. It is important that the number of cycles of reanalysis does not become large leading to excessive demands for computer time.

Inclusion of gross geometrical variables of the structures represents an important improvement in the class of problems for which optimization may be applied. Figure 4 of my Introductory Report shows a transmission tower in which the tower shape and location of nodes is permitted to change leading to significant reductions in structural weight. The left figure is the original design while the right is an optimized case. The optimization takes place automatically with a program using methods of minimisation working with respect to the geometric or shape design variables.

Another example of geometric optimization is the arch dam reported by Vitiello and shown in Fig. 5 of my Introductory Report. The mesh shown is the finite element analysis while X_1 - X_4 are the geometric design variables. Such applications show that major improvements in structural efficiency can often come from variations in geometric design variables. This is investigated for arches and suspension bridges by Professor Hirai and Yoshimura in their Preliminary Report.

Form and type of structure represent a high level of optimization for which programs have only recently been attempted. Figure 6 from my Introductory Report paper shows a schematic diagram for optimizing the cost of single storey factory buildings. The variables include structural layout such as bay spacing and also the type of joists, girders, columns and foundation including material type and detailed design variables. The design methods, automatically performed by the computer, can lead to important structural savings and can be updated following changes in individual construction and material costs.

Bomhard in the Preliminary Report shows a comparison of structural form with an illustration of beams, arches and suspensions to cope with long-span structures. Suruga and Maeda have developed a very interesting concept of a decision matrix to compare structural forms for their application to floor systems of long-span bridges. Each type of floor system such as composite girder or orthotropic deck is rated according to cost, construction and performance before a final decision can be made. This leads to a multi-objective criteria for optimization which may have important applications to other examples such as comparing economy with safety. The inclusion of safety directly in the optimization methodology is covered by Tegze and Lenkei with an example of collapse analysis of statically indeterminate plane structures.

The inter-relationship of safety and economy of structure has been recognised by many authors, but more effort is required to bring these factors into both the code specifications and the programs for optimum design.

Comments by the General Reporter

Remarques du rapporteur général

Bemerkungen des Generalberichterstatters

YUKIO MAEDA
Professor of Civil Engineering
Osaka University
Suita, Osaka, Japan

Examples of Computer-Aided Optimal Design of Structures — General Report

The main subject of Theme II is on "Progress in Structural Optimization", which was originally proposed by the Japanese National Group of IABSE, because it was intended to stimulate and encourage Japanese engineers to apply the concept and method of optimization to problems of structural design, since the structural optimization has very recently been introduced into Japan. Along this intention we are very thankful for the three excellent Introductory Reports.

Dr. Gellatly and Mr. Dupree presented a very excellent paper as an introductory report on applied structural optimization in terms of examples of computer-aided optimal design of structures. They covered two different approaches to the optimum design of complex structural systems, emphasizing the practical aspects of design problems intended for producing a useful tool for designers.

The first approach, "Optimality Criteria Approach" will be accepted by designers because of its simplicity and effectiveness. The approach to the weight minimization of fixed-geometry structures with constraints based on the use of optimality criteria, appears to offer considerable advantage over mathematical-programming based methods. At comparative studies, the present method seems to reach a similar or better design in considerably fewer iterations than most numerical search methods with the reduction of computational costs.

They presented five examples, and also Dr. Gellatly discussed this approach at his other paper[1)] at an example of "Twenty-five-bar Transmission Tower" in which, using the current program, convergence was obtained in seven iterations to get its minimum weight, although, using a numerical search method, over one hundred analyses might have been required.

These results are very encouraging us, because they indicate that some, if not all, of the difficulties encountered in large-scale optimization problems for the very large number of variables in finite element representation of real structures, can be eliminated through this type of approach. However, certain problems may still remain to be unsolved, particularly with regard to convergence characteristics.

The second approach is labeled Sieve-Search Procedure developed at Bell Aerospace Company, the guiding philosophy of which is that an optimum system is an optimum arrangement of pre-optimized components. The results obtained from the design studies on high-speed vessels and a design study on a complete bridge structure have indicated that, firstly, the method will permit the full variation of construction method, materials and configuration as well as component sizing, and secondly, this method is also an efficient cost-effective approach to automated optimum design.

Dr. Gellatly and Mr. Dupree suggested finally that the ideal solution for optimization problems would possibly appear to be a combination of the two approaches, in which the sieve-serach defines configuration and non-continuous variables and the optimality criteria method will be used for refinement of the design, expecting a considerable potential for overall system optimization at various design problems.

We have been expecting a number of papers to be presented at the Preliminary Report under the stimulus and for the discussions of the Introductory Reports. For the Sub-Theme IIc, the following five papers have been accepted:

1. The paper presented by Mr. Gurujee

The paper shouldhavebeen discussed at Theme IIa. He proposed a general optimization algorithm for a structure. A structural optimization problem can be generally solved as a sequence of analysis-programming cycles by the mathematical programming. In the optimization process which the author proposed in the form of a chart shown in Fig.1 at the Preliminary Report, p.179, the relation between the changes in the behavior variables due to a specified change in each of the design variables, is found and stored in the form of "Sensitivity Matrix". Then, the programming problem can be solved by using the penalty function method. In this paper, however, he did not show any specific examples to which his proposed method was applied.

2. The paper presented by Prof. Yamada and Mr. Furukawa

They treated the optimal design of a system of tower and pier of a suspension bridge, on the elastic foundation subjected to earthquake ground motion. They showed an example how to combine mathematical programming and dynamic structural analysis through response spectrum for a dynamic loading problem, referring to Figs. 1 and 2 at the Preliminary Report, p.184. To simplify very complicated real dynamic behavior of the system, two design variables were selected: longitudinal width of the pier and stiffness of the tower. A generalized cost was selected as the objective function, and requirements for stress of the tower and displacement of the pier at its top, and buckling of the tower, overturning of the pier, and physical limits, were constraints.

Since the problem is non-linear and undifferential, the Sequential Unconstrained Minimization Technique by Powell's direct search method was applied to optimization, probably because the method is more reliable in terms of guaranteed convergence if the first derivatives or no derivatives are available. At a numerical example, the authors found out that the generalized cost is greatly affected by the modulud of elasticity of the foundation. This problem is overall system optimization of a simple tower-and -pier system. Shape and geometry optimization and combination with detailed element optimization will be a future problem.

3. The paper presented by Prof. Konishi and Prof. Maeda

The paper on "Total Cost Optimum Design of I-Section Girders for Bridge Construction" treated examples of detailed design optimization of main elements of girder bridges. Generally, at the problem of bridges, cost optimization is selected as the objective function, but the cost used to be defined material cost only or material plus overall fabrication cost. At the present paper, the objective function consists of material and fabrication costs, which cover costs of full-scale drawing, machining, shop welding, shop assembly and shop painting base on actual detailed informations obtained at fabricating shops in Japan.

A computer-aided optimum design of girders by the method of "Sequential Linear Programming" was illustrated at I-shaped, deck-type, welded plate girders with five different span lengths, and sixteen design variables including material selection (See Fig.2 at the Preliminary Report, p.192). The influence of material and size selections on the total cost was discussed in detail, to help designers carry out a detailed element design efficiently from the point of optimization, taking into consideration not only material cost, but also shop fabrication cost.

For a specific or individual bridge, it would be required to study on an overall optimization design including transportation and erection costs for a system of main girders, laterals and decks.

4. The paper presented by Professor Schindler

He proposed an optimization method to combine design-oriented approach and computer-oriented approach, in which a designer can search for a range of approximation near an optimum value with a design program, within the capacity of a computer, not spending so much money and time for computer calculation.

He illustrated his method at the optimum design of a railway truss bridge shown in Abb.1 at the Preliminary Report, p.196, taking into account three kinds of deck system , two kinds of steel, two kinds of bridge class, five kinds of span length. The objective function was total steel weight, and the design variables were span length, number of panels, height of the truss, and width of chord members. For various truss heights, steel weights were calculated by a computer with parameters of span length and number of panels. By comparison of each steel weight, the minimum weight was found out for a certain value of span length and of number of panels.

This approach is not straightforward, but rather comparative or selective. Sometimes depending on a problem, this approach may save the time and money for a computer more than mathematical programming methods. This kind of approach could be examined in contrast with a study presented by Prof. Ohkubo at Theme IIb [2] who proposed a sub-optimizing method for trusses.

5. The paper presented by Messrs. Tanaka, Kamemura and Maruyasu

They introduced the total computer-aided design system for girder bridges. which has recently been developed at Nippon Kokan Company, Japan. Automated computer techniques for design have advanced so that various types of detailed element design and selection among alternatives for minimum cost can be carried out. In this sense, the proposed computer system is a well advanced method for automated design of a girder type bridge in its element and overall system.

As the authors pointed out, such a computer program could be used for lowering cost, increasing standardization of elements and also evaluating the effects of changing constraints on weight, cost and behavior. The authors discussed conceptually the interaction between optimum design and automated design, but they did not show concretely with an illustration how to incorporate optimization into the automated design program.

The proposed computer system should be examined in contrast with the flow chart of Sieve-Search Optimization for bridge design proposed by the Introductory Reporters, Dr. Gellatly and Mr. Dupree [3].

As a concluding remark, at the Prepared Discussion nore demonstrations of structural optimization are welcome in terms of examples to encourage designers to utilize optimization techniques at their routine office practice, and also to discuss what kinds of problems have been encountered at practical designs.

References:
1) R.A. Gellatly & L. Berke, "Optimality-criterion-based Algorithms", Optimum Structural Design, ed. by Gallagher & Zienkievicz, John Wiley, 1973, p.44.
2) S. Ohkubo & T. Okumura, "Structural System Optimization Based on Sub-optimizing Method of Member Elements", Prel. Rept. 10th Congress, IABSE,1976.
3) R.A. Gellatly & D.M. Dupree,"Examples of Computer-Aided Optimal Design of Structures", Introductory Rept., 10th Congress of IABSE, 1975.

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Optimum Design of Cable-Stayed Bridges using an Optimality Parameter

Calcul de ponts haubannés à l'aide d'un paramètre d'optimisation

Die Berechnung von Schrägseilbrücken mit einem Optimierungsparameter

HISASHI DAIGUJI
Chief Engineer
Tech. & Eng. Div., Harumoto Iron Works Co., Ltd.
Osaka, Japan

YOSHIKAZU YAMADA
Professor of Civil Engineering
Kyoto University
Kyoto, Japan

1. INTRODUCTION

The optimization discussed in this paper is applied for the design of the overall super-structure of cable-stayed bridges. Then the hierarchy of this study is belonged to category 3 described in the introductory report of the 10th congress by Templeman(1). The optimization method developed here is a kind of the optimality criterion method discussed by Templeman, Gellatly and Dupree(1)(2).

Up to the present many nonlinear programming techniques have been developed and applied for the optimum design of bridge super-structures, but successful applications are very few. Because the fully stressed design for a common type of super-structure such as girder bridge is a convenient design method and gives the satisfactory economical result. Therefore from the practical point of view, the optimum design without considering the price of sub-structure may be important for only some specific type of bridges such as cable-stayed bridges, suspension bridges.

In this study, an optimality condition parameter is obtained by a mean of the numerical calculation and the parameter is used to determine the economically proportional sizes of the cable and girder.

2. PRECONDITIONS FOR THE NUMERICAL PROCEDURE

To determine an optimality parameter by numerical process, the following preconditions must be given.

(1) Utilization of the structural nature of cable-stayed bridges is very important to find out the optimality condition. Fig. 1, 2 show the static behavior of cable-stayed bridges due to dead loads and live loads. These examples show that the each rigidity value of cable and girder is not main factor of changing the section force distribution. It is obvious that the main influence to the girder section force is a rigidity ratio, $\gamma = EG \cdot IG / EC \cdot AC$. where, EG is the modulus of elasticity of girder, IG is the moment of inertia of girder, EC is the modulus of elasticity of cable, AC is the cable area.

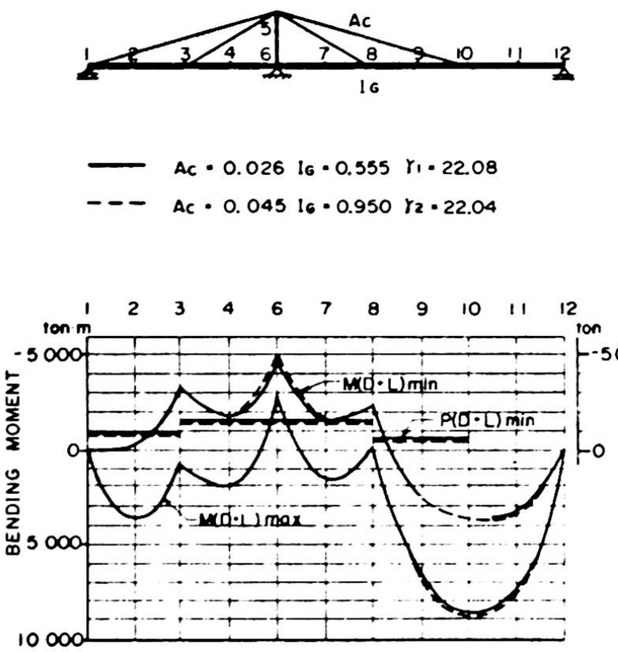

Fig. 1 2 Span Bridge

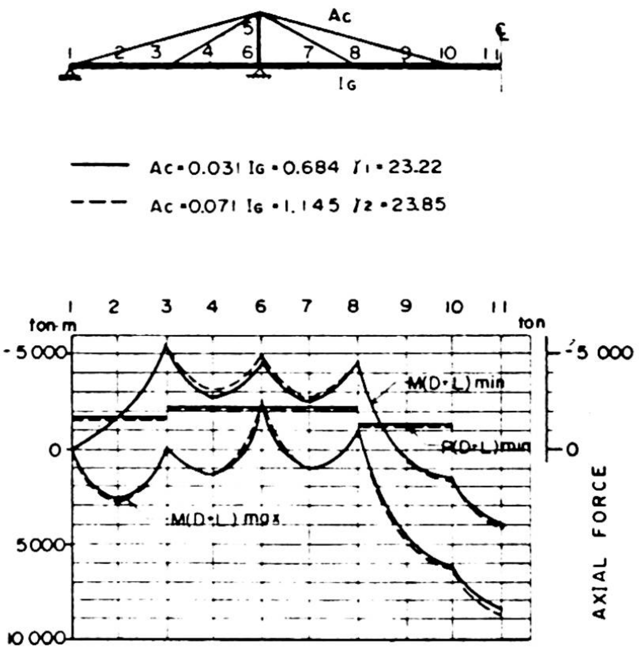

Fig. 2 3 Span Bridge

(2) The price ratio of materials including the cost of fabrication and erection is assumed as follow.

structural steel : high tensile steel : cable = 1 : 1.15 : 2.0

(3) Fig. 3 is the analyzing structural system which rigidity ratio is assumed as:

$\gamma_{try} = EGX_1/ECX_2$ X_1: moment of inertia of girder
X_2: cable area

Fig. 4 is the actual redesign structure which rigidity ratio is expressed by:

$$\gamma_{real} = \frac{EG/NG \cdot \sum_{n=1}^{NG} IGn}{EC/NC \cdot \sum_{n=1}^{NG} ACn}$$

IGn: moment of inertia of girder
ACn: cable area
NC, NG: number of cable and girder

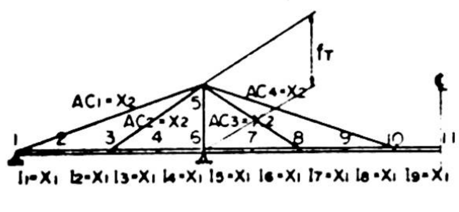

Fig. 3 Assuming Member System

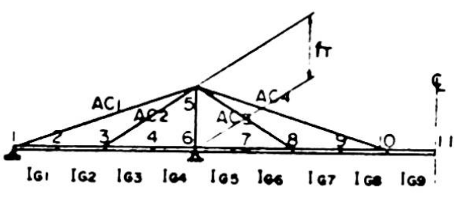

Fig. 4 Actual Member System

The approximate design process must be carried out by keeping the following ctiteria.

$$0.9 < \gamma_{try}/\gamma_{real} < 1.1$$

(4) The section and material compositions of stiffening girder are illustrated in Fig. 5, 6. The price of girder member is determined by the element design based on the fully stressed design.

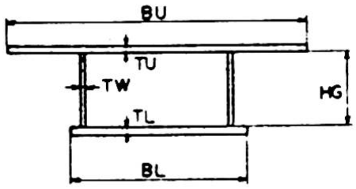

Fig. 5 Girder Section

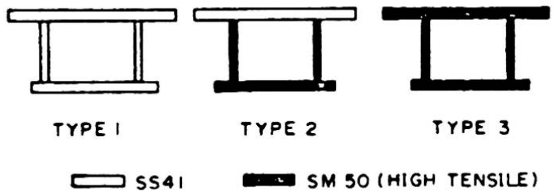

Fig. 6 Material Composition of Girder

3. DETERMINATION OF AN OPTIMALITY PARAMETER

An optimality parameter is determined after carried out the next 2 step procedure.

(1) Characteristic Parameter (Step 1)

The basic structure to be effectively prestressed is determined by the grid search procedure, because two design variables are employed for the global system optimization. The cost evaluation is made by the following equation.

$$Z(X_1,X_2) = \sum_{m=1}^{NG} \text{price } G(X_1,X_2) + \sum_{n=1}^{NC} \text{price } C(X_1,X_2)$$

X_1: moment of inertia of stiffening girder
X_2: cable area
price G: price evaluation of girder depend on X_1, X_2
price C: price evaluation of cable depend on X_1, X_2

The characteristic parameter at the grid point is expressed as:

$$KE = EG \cdot IG / EC \cdot AC \cdot HG^2$$

IG: moment of inertia of stiffening girder
AC: cable area
HG: web depth

(2) Determination of an Optimality Parameter (Step 2)

Prestressing forces (external loads) are introduced into the cable of basic structural system determined by above procedure. In this step, prestressing forces are design variables (Fig. 7). In case of two design variables, an optimality parameter minimizing the total cost is also selected among the grid points number of characteristic parameters, and it is expressed by the following nondimensional parameter.

$$KOPT = EG \cdot IGOPT / EC \cdot ACOPT \cdot HGOPT^2$$

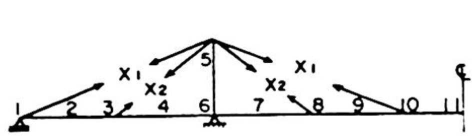

Fig. 7 Prestressing System

4. NUMERICAL MODELS AND RESULTS

Numerical calculation is carried out for nine cases of analyzing models. Structural models of 2-span and 3-span bridges are illustrated in Fig. 8, 9.

The differences of analyzing models are indicated belows.

2-span Bridge

Case	Tower Height	Web Thick	Steel Weight	Other
1	30 m	10 mm	3.3 t/m	
2	30 m	14 mm	3.3 t/m	
3	30 m	10 mm	3.3 t/m	Knie Type
4	30 m	10 mm	5.3 t/m	
5	35 m	10 mm	3.3 t/m	
6	40 m	10 mm	3.3 t/m	

3-span Bridge

Case	Tower Height	Web Thick	KE
7	30 m	10 mm	2.58
8	30 m	14 mm	2.64
9	30 m	14 mm	2.96

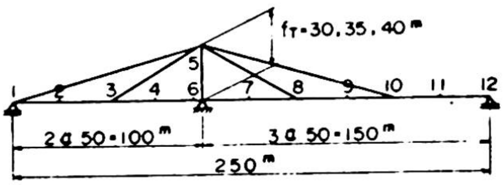

Fig. 8 2 Span Model

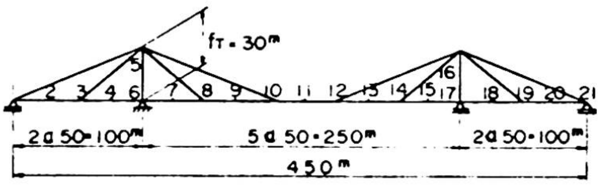

Fig. 9 3 Span Model

Table 1 Numerical Result of Parameters

	CASE	DESIGN STEP		KE PARAMETER	KOPT PARAMETER	
2 SPAN	1	A	FULLY STRESS	4.0 ~ 5.0		
		B	BASIC STRUCTURE	0.92 ~ 7.56		
		C	P.S. ARRANGEMENT	2.47 ~ 2.89	2.55	
	2	B	BASIC STRUCTURE	0.96 ~ 7.88		
		C	P.S. ARRANGEMENT	2.54 ~ 2.96	2.49	
	3	B	BASIC STRUCTURE	0.92 ~ 7.73		
		C	P.S. ARRANGEMENT	2.43 ~ 2.90	2.43	2.48
	4	B	BASIC STRUCTURE	0.92 ~ 7.03		
		C	P.S. ARRANGEMENT	2.53 ~ 2.89	2.55	
	5	B	BASIC STRUCTURE	0.92 ~ 7.05		
		C	P.S. ARRANGEMENT	2.41 ~ 2.89	2.45	
	6	B	BASIC STRUCTURE	0.92 ~ 7.50		
		C	P.S. ARRANGEMENT	2.41 ~ 2.89	2.41	
3 SPAN	7	A	FULLY STRESS	3.22 ~ 4.73		
		B	BASIC STRUCTURE	0.92 ~ 5.67		
		C	P.S. ARRANGEMENT	2.14 ~ 2.58	2.58	
	8	B	BASIC STRUCTURE	0.96 ~ 5.81		2.46
		C	P.S. ARRANGEMENT	2.25 ~ 2.65	2.27	
	9	B	BASIC STRUCTURE	0.96 ~ 5.81		
		C	P.S. ARRANGEMENT	2.53 ~ 2.96	2.53	
TOYOSATO BR.		SPAN 80.5 · 216 · 80.5 (PWS)				2.87
SUEHIRO BR.		SPAN 109 · 250 · 109 (PWS)				2.89
ONOMICHI BR.		SPAN 85 · 215 · 85 (LOCKED-COIL)				2.39

Numerical results of characteristic parameter and optimality parameter are listed in Table 1. The KE parameter of the cable-girder system determined by the fully stressed design is in the small range (3.∿ 5.). On the other hand the KE parameter determined by the approximate design process developed in this study is in the fairly large range (1.∿8.). Prestressing forces are introduced into the suitable basic structure which cable components are not fully stressed. From the results of case 7 ∿ case 9, it is obvious that KE value reduces about 15 percents by introducing the prestressing forces.

5. AN EXAMPLE OF THE DESIGN USING AN OPTIMALITY PARAMETER

The main difference of this method from the usual design method is the use of the parameter KOPT = 2.5 obtained by numerical calculation as shown in Fig. 10.

dead load .... 10.0 t/m
line load .... 50.0 t
uniform load .... 3.5 t
impact 0.2
optimality
  parameter...... KOPT=2.5
assuming rigidity
  girder .... IG=1.0 m^4
  cable .... AC=0.046

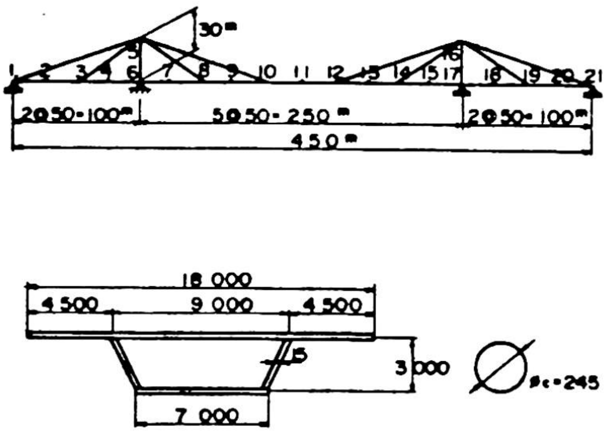

Fig. 10.
Design Conditions
and Basic Dimensions

The optimum bending moment arranged by prestressing forces and the moment inertia of girder members are also illustrated in Fig. 11.

By using the optimality parameter, structural designer can get the reasonable sections of girder and prestressing forces of cable-stayed bridges by one time trial. The assuming rigidity of analyzing system determined by using an optimality parameter is very close to the real rigidity of the final structure.

6. CONCLUSION

Unexperienced structural engineer may feels some difficulties to design the economical cable-stayed bridge. Because allowable stress guarantee the safety of structures, but it does not always guarantee the economical condition. The prices and strength of the cable and steel girder are extremely different. Furthermore the arrangement of the bending stress of the stiffening girder causes the more complicated problem. Therefore the economical criterion for cable-stayed bridges may be important as same as the factor of safety.

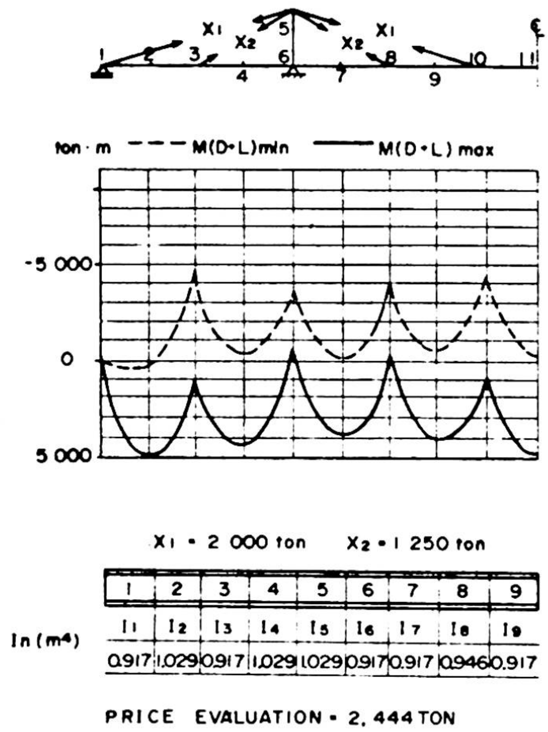

Fig. 11. Optimum Design

Finally the conclusions of the basic study for the optimality criterion method are outlined by the next statements.

(1) The optimality parameter value of the radial type of the cable-stayed bridge based on the price ratio (structural steel : cable = 1 : 2) exists in the range of 2.0 ∿ 3.0.

(2) The moment of inertia of stiffening girder can be considered as uniform along the girder axis by effectively prestressed.

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SUMMARY

A convenient optimization method using an optimality parameter has been discussed. The optimality parameter is based on the structural nature and the approximate design procedure. This parameter is used to determine the optimum rigidity ratio of the cable-girder system after introducing the prestressing forces. The optimum design using an optimality parameter will be easily accepted by structural engineers as the economical criterion.

RESUME

Une méthode pratique d'optimisation à l'aide d'un paramètre d'optimisation est présentée. Le paramètre d'optimisation est obtenu à partir du caractère structural et de la méthode approximative de calcul. Ce paramètre est employé pour déterminer la rigidité la plus favorable du câble et de la poutre après avoir introduit les forces de précontrainte. Le calcul à l'aide d'un paramètre d'optimisation sera accepté par les ingénieurs comme un critère économique.

ZUSAMMENFASSUNG

Eine anwendbare Optimierungsmethode mit einem Optimierungsparameter wird dargestellt. Der Optimierungsparameter wird aufgrund des Tragwerkssystems und der Näherungsberechnungsmethode bestimmt. Mit diesem Parameter werden die günstigsten Seil- und Trägersteifigkeiten unter Berücksichtigung der Vorspannkräfte bestimmt. Die Berechnung mit einem Optimierungsparameter wird von den Ingenieuren zur Steigerung der Wirtschaftlichkeit angenommen werden.

Preponderance of Idealization in Structural Optimization

Prépondérance de l'idéalisation dans les problèmes d'optimisation structurale

Die überragende Bedeutung der Idealisierungen bei der Optimierung von Tragwerken

RENÉ MAQUOI
National Foundation for Belgian Scientific Research
(F.N.R.S.)
Liège, Belgium

JACQUES RONDAL
Assistant
University of Liège
Liège, Belgium

The optimal design of a structure may be divided in two steps. In the first one - the *idealization* - the structural problem is put in following mathematical formulation :

"Find $\vec{X}$ such that :

$$f_k(\vec{X}) < 0 \quad \text{for} \quad k = 1,2,\ldots m \;; \qquad (1)$$
$$h_j(\vec{X}) = 0 \quad \text{for} \quad j = 1,2,\ldots l \;;$$

and :

$$F(\vec{X}) = \text{minimum (maximum)}\text{ "}$$

where $\vec{X}$ is a vector which contains the design variables,

f and h are the constraints of the problem,

and F is the objective function to optimize.

The second step - the *solution process* implies (a) the choice of the solving procedure and (b) the search of the solution of the problem formulated as in (1).

In the opinion of the authors, a good idealization is the basic condition for obtaining a good value of the solution, while a more or less refined mathematical treatment of it plays a rather secondary role |1| .

In many papers of the literature, emphasis is too often brought on the choice of the solution procedure rather than on that of a heuristic which does not modify in anyway the sense of the actual problem.

So long as the structural problem is small - about ten variables and constraints - many methods are available in the literature. However, various numerical experiments have shown that the choice of a method depends on the problem to be solved, for most of the algorithms cannot be used economically in all cases |2|. As a consequence, conclusions concerning the use range and the efficiency of an algorithm for a structural problem can rarely be extended to another one.

If emphasis is almost brought on the idealization, the designer may be sure of obtaining a realistic solution of the problem and, in addition, important simplifications in the mathematical treatment of the second step become possible. Indeed, on one way, a judicious choice of variables or an ingenious variable transformation often enable to present the complex problem in a more simple form, and, on another way, by means of a previous evaluation of the several variables, the designer can establish a hierarchy of the variables and divide the complex problem into smaller ones, which are then more easier to solve quickly.

For example, in |3|, MYLANDER demonstrates that a rather simple variable transformation changes a mathematical non-linear and non-convex problem into a linear programming system. It is worthwhile to recall the following basic non-linear problem which is considered as a very difficult one. The objective function is :

$$f(x) = b_o + a_{o1} x_1 + \left(\sum_{j=2}^{5} a_{oj} x_j\right) x_1 \rightarrow \min$$

subject to constraints :

$$0 \leqslant a_{i1} x_1 + \left(\sum_{j=2}^{5} a_{ij} x_j\right) x_1 \leqslant b_i \quad i = 1,2,3 \qquad (2)$$

$$x_1 \geqslant 0 \; ; \; 1.2 \leqslant x_2 \leqslant 2.4 \; ; \; 20.0 \leqslant x_3 \leqslant 60$$

$$9.0 \leqslant x_4 \leqslant 9.3 \; ; \; 6.5 \leqslant x_5 \leqslant 7.0.$$

where the values of the constants are :

a_{01}	= - 8,720,288.795	a_{21}	= - 155,011.1055
a_{02}	= - 150,512.524	a_{22}	= 4,360.5334
a_{03}	= - 156.695	a_{23}	= 12.9492
a_{04}	= - 476,470.319	a_{24}	= 10,236.8839
a_{05}	= - 729,482.825	a_{25}	= 13,176.7859
a_{11}	= - 145,421.4004	a_{31}	= - 326,669.5059
a_{12}	= 2,931.1506	a_{32}	= 7,390.6840
a_{13}	= - 40.4279	a_{33}	= - 27.8987
a_{14}	= 5,106.1920	a_{34}	= 16,643.0759
a_{15}	= 15,711.3600	a_{35}	= 30,988.1459
b_o	= - 24,345.0	b_2	= 294,000.0
b_1	= 294,000.0	b_3	= 277,200.0

By putting, according to MYLANDER

$$y_i = x_1 \cdot x_i \qquad i = 2, 3, 4, 5$$

and (4)

$$y_1 = x_1$$

above non-linear problem takes following linear formulation :

$$g(y) = b_o + \sum_{j=1}^{5} a_{oj} y_j \rightarrow \min$$

$$0 \leqslant \sum_{j=1}^{5} a_{ij} y_j \leqslant b_i \quad i = 1, 2, 3 \qquad (5)$$

$$y_i \geqslant 0 \qquad i = 1, 2, \ldots, 5$$

$$y_2 - 1.2\, y_1 \geqslant 0 \; ; \quad 2.4\, y_1 - y_2 \geqslant 0$$

$$y_3 - 20.0\, y_1 \geqslant 0 \; ; \quad 60.0\, y_1 - y_3 \geqslant 0$$

$$y_4 - 9.0\, y_1 \geqslant 0 \; ; \quad 9.3\, y_1 - y_4 \geqslant 0$$

$$y_5 - 6.5\, y_1 \geqslant 0 \; ; \quad 7.0\, y_1 - y_5 \geqslant 0.$$

which may directly solved by means of the classical simplex routine.

The optimal solution, obtained after six iterations, is given by :

$$g = -\, 5{,}280{,}344.9$$

$$y_1 = 4.53743 \; ; \quad y_2 = 10.88983 \; ; \quad y_3 = 272.24584 \qquad (6)$$

$$y_4 = 42.19811 \; ; \quad y_5 = 31.76202$$

which in terms of the original variables gives f = - 5,280,344.9

$$x_1 = 4.53743 \; ; \quad x_2 = 2.40000 \; ; \quad x_3 = 60.00000 \qquad (7)$$

$$x_4 = 9.30000 \; ; \quad x_5 = 7.00000.$$

The solution of the original problem by means of non-linear programming methods |4, 5| lead, after a lot of iterations, to values of f which are 2 or 3 % below the true optimum but, in some cases, with value of the variable x_3 which is about 50 % erroneous.

In |6|, the authors show how a suitable choice of the behaviour model for a complex structural design - indeterminate prestressed bridges - leads to a benefit similar to that obtained by MYLANDER.

The idealization of the problem is based on an approach with sensitivity coefficients, as that proposed by GURUJEE |7|, and on a variable transformation; it is then allowed to solve this complex design problem by means of linear programming, without the actual problem be denaturated and taking account of all the technological requirements (cover thickness, anchorage dimensions, redundant effects of prestressing, friction losses, anchorage slippage,...). After the variable transformation, the problem remains partially non-linear but the authors have shown in |8| that the non-linear term, being of the order of 1 % with respect to its corresponding linear component, may be neglected in practice.

The authors would like to conclude by saying that for optimum design, as for all the other engineering activities, mathematics are a good servant but a bad master.

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SUMMARY

In structural optimization problems, it is nearly always observed that, in the search for a realistic solution, the suitability of idealization is more important than the choice of the solving algorithm.

RESUME

Dans les problèmes de dimensionnement optimal, il est généralement constaté que la recherche d'une solution réaliste dépend davantage de l'idéalisation du problème que du choix de l'algorithme de résolution.

ZUSAMMENFASSUNG

Bei der Optimierung von Tragwerken wird allgemein festgestellt, dass die Suche nach einer realistischen Lösung mehr von der Idealisierung des Problems als von der Auswahl des Lösungsalgorithmus abhängt.

Minimum Weight Plastic Design of Regular Rectangular Plane Frames

Calcul plastique pour un poids minimum de cadres plans rectangulaires

Plastische Bemessung auf Minimalgewicht für rechteckige, ebene Rahmen

TSUNEYOSHI NAKAMURA
Associate Professor
Department of Architecture, Kyoto University
Kyoto, Japan

1. INTRODUCTION

Structural engineers have been concerned more with practical computational techniques for optimum structural designs than with theoretical results. On the other hand, scientific investigations on optimality criteria and optimal structures have been carried out mostly by researchers in the field of structural or applied mechanics. These two approaches are mutually compensating in order to develop more rational methods of structural designs.

The introductory report by A.B.Templeman has been primarily concerned with the hierarchy of optimum structural design problems and the corresponding computational techniques. Reference is made in his report to the linear theory of minimum weight plastic design and to the advantage of linear programming. It should also be recognized that the theoretical results on optimality criteria and optimal structures not only have the scientific significance but also lay the foundations and stimulate new ideas for developing practical computational techniques.

The purpose of this discussion is to call attention to the recent results [2-10] by the author and his colleagues on some general solutions derived analytically in closed forms to the problems of minimum weight plastic design of regular rectangular plane frames of practical interest and then to point out the theoretical and practical significances of those solutions.

2. FRAME MOMENT FOR REGULAR RECTANGULAR FRAME

Fig.1 shows a regular rectangular plane frame and one set of vertical and lateral design loads. The geometrical regularity in such a frame not only is reflected in design loads but also characterizes its structural behaviors and optimal plastic designs. In many practical design problems, the story shear resultant increases rapidly from the top floor toward lower stories as compared with the variation of vertical gravity loads. In those countries where frames must withstand against strong-motion earthquakes and strong gusts, *fairly large* lateral design loads are assigned. Under these circumstances, the first step of analytical treatment of a Foulkes-type problem is to assume an extremely deteriorated collapse mecha-

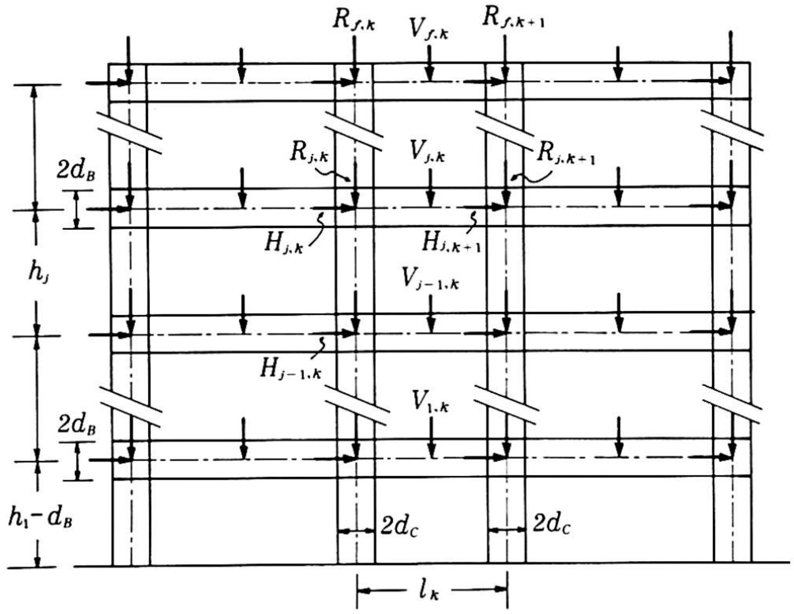
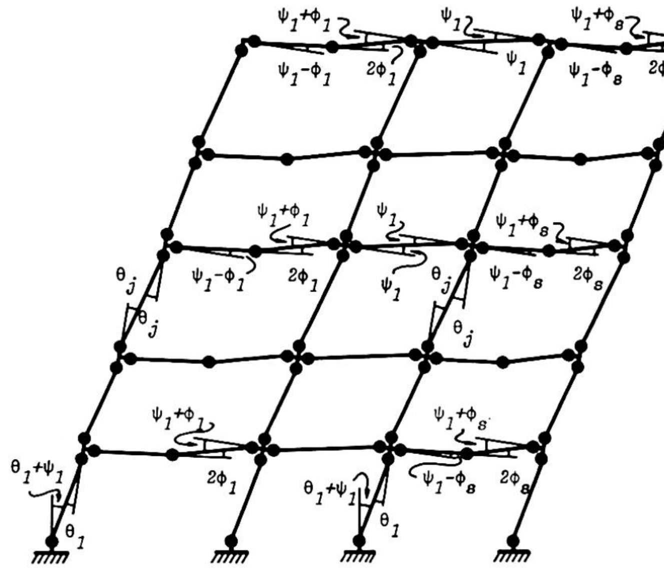


Fig.1
Regular
rectangular
frame



$\psi_1 = \frac{1}{2} l_m \theta$,

$\theta_j = \frac{1}{2} h_j \theta$,

$\phi_k = \frac{1}{2}(l_k - l_m)\theta$,

$\theta_1 = \frac{1}{2}(h_1 - \frac{1}{2} l_m)\theta$,

$\theta > 0$*: a constant,*

for frames satisfying

$2l_m \geqq l_k \geqq l_m$ *and*

$2h_1 \geqq l_m$.

Fig.2
A Foulkes
mechanism
for
regular
rectangular
frame

nism shown in Fig.2, in which *simple* plastic hinges have formed at almost all the potentially critical sections except at the midspan sections of some particular bay(s) to be found as a part of the solution. Fig.3 shows that the corresponding bending moment diagram at plastic collapse may be conceived as the result of two-fold superpositions of decomposed diagrams. Each decomposed diagram is such that the moment equilibrium is maintained at the four corners with the same absolute value in the manner shown in Fig.3. This equal corner moment associated with this elementary moment diagram is called a "frame moment". A restricted minimization may then be carried out analytically in terms of the frame moments, and some statical conditions are derived under which the assumed bending moment diagram corresponds indeed to a general solution. It is then shown that the Foulkes mechanism condition can also be satisfied for a class of frames satisfying two simple geometrical conditions as shown in Fig.2.

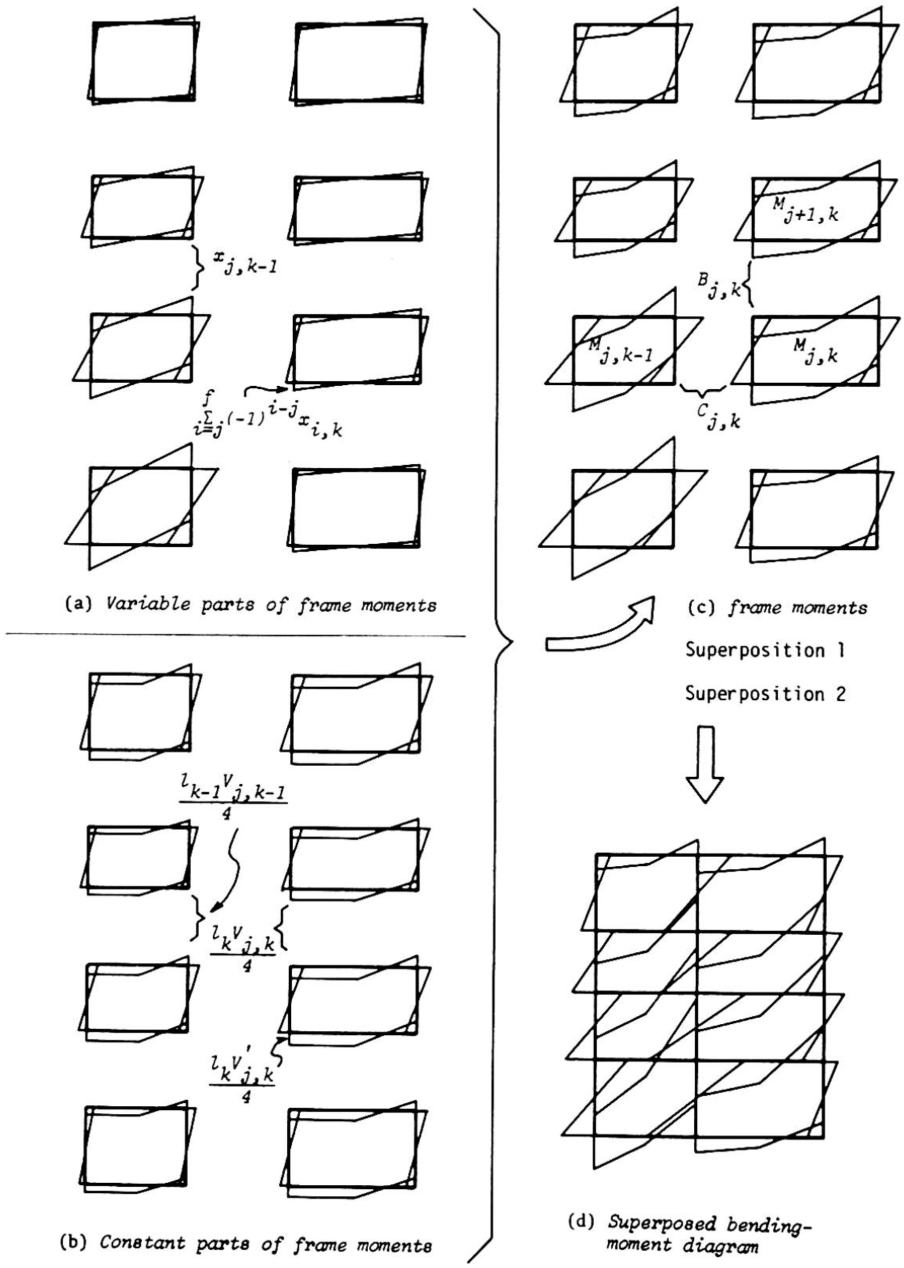


Fig.3 Frame moments and two-fold superposition procedure

3. APPLICATION OF FRAME MOMENT DECOMPOSITION TECHNIQUE

The two general solutions mutually exclusive and compensating on a design chart [2] not only clarify the general features of the classes of the minimum weight designs, but also provide a basis on which some modified general solutions can be derived to problems formulated more realistically by incorporating the axial force-bending moment interaction yield conditions for idealized beams and columns [3] shown in Fig.4. Fig.5 shows a part of modified Foulkes mechanism in a theory [4] in which only the idealized columns are required to satisfy the interaction yield condition shown in Fig.4. The regularity in the frame geometry enables one again to derive the general solutions and the statical and geometrical conditions analytically in closed forms [3, 4].

For the problem where reaction constraints have been incorporated within the framework of Foulkes' theory, a *bay shear distribution law* has been derived in [5] also on the basis of the concept of the frame moment and of the afore-mentioned two-fold superposition procedure.

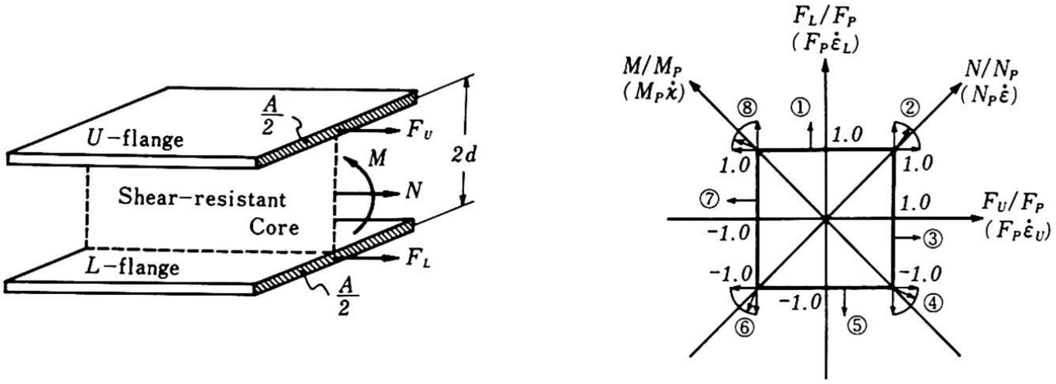


Fig.4 Axial Force-Bending Moment Interaction Yield Condition for an Idealized Sandwich Member

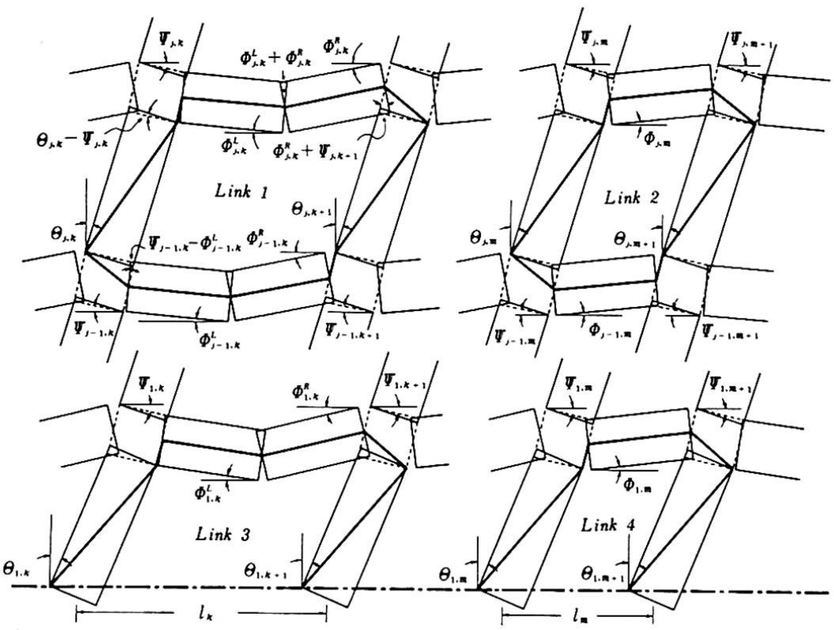


Fig.5
A modified Foulkes mechanism

For the problem of multi-story multi-span frames to be designed for five sets of design loads, a kinematical restricted maximization procedure has been developed in [6] by combining the primal-dual method of LP with a semi-inverse approach similar to [2]. Some general solutions have thereby been derived analytically in closed forms. Fig.6 shows a portal frame obeying an idealized interaction yield condition and subjected to two sets of design loads. Fig.7 shows a fundamental design chart for this frame. This chart together with the theory in [7] constitutes the foundation for a possible analytical attempt of incorporating the result of [3] and [4] in [6].

4. SIGNIFICANCE OF THE CLOSED FORM GENERAL SOLUTIONS

The theoretical significance of these general solutions are now obvious. Each general solution provides a basis for developing practically useful general solutions to problems of more realistic formulations, though some modifications may become necessary for the topmost few stories. The afore-mentioned results may be said to provide ample grounds for the fruitfulness of this successive refining process.

For practical application, these solutions must first be modified for the effect of inelastic stability and member design requirements. The author and his colleagues have already clarified to a certain extent through numerical large-deflection analyses that minimum weight frames can indeed withstand against static al-

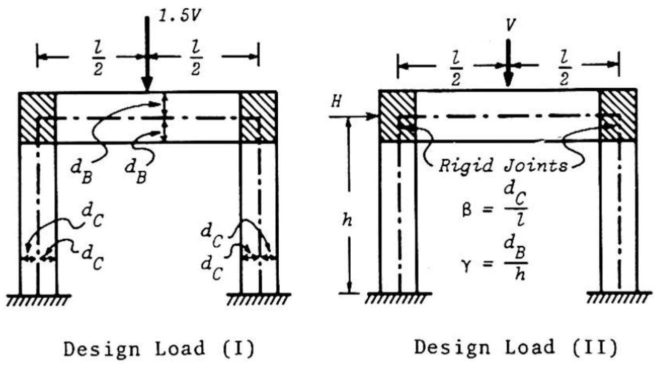


Fig.6
Portal frame
and
the two design
loads

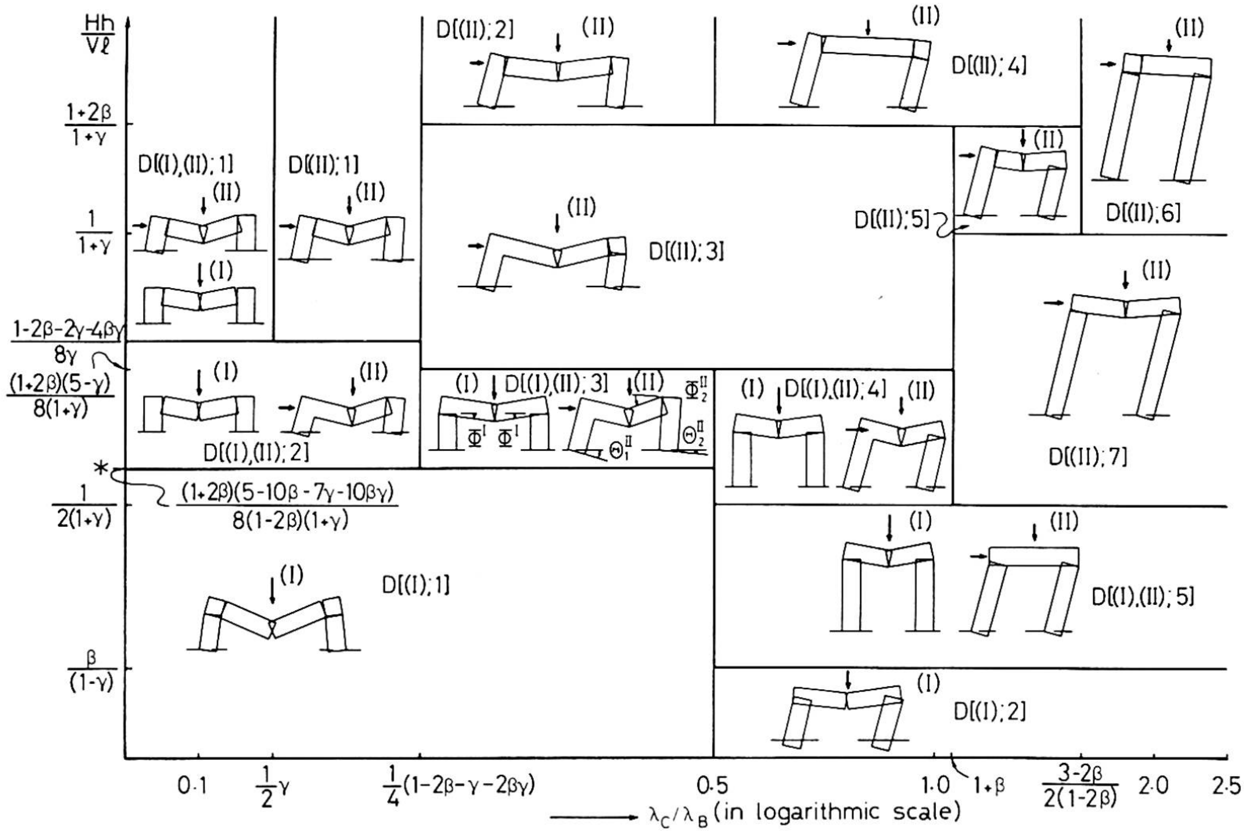


Fig.7 Design chart for a portal frame subjected to two sets of design loads

ternating lateral loads [8] and strong-motion earthquake disturbances [9] and are not particularly imperfection sensitive [10]. It should also be noted that the afore-mentioned solutions are the necessary consequences of the one-sided optimization using an approximate "failure" design criterion aside from the "serviceability" design criterion to be satisfied in practice.

Yet it can be said that the afore-mentioned solutions have the following significances: (i) they clarify the intrinsic features of the minimum weight plastic designs of regular rectangular frames at various levels considerably well; (ii) they will provide good initial solutions, if properly incorporated in a program, to start a numerical search for an optimal solution under additional constraints and may also be utilized as some standards for program verification. (iii) It may be well expected that the closed form solutions will be useful for seeking for optimum span length combinations analytically.

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SUMMARY

The frame moment decomposition technique due to the author and its applications to several more realistically formulated problems have been briefly described. The theoretical and practical significances of the analytical approach to the minimum weight plastic design problems have been explained in reference to the papers by the author.

RESUME

On décrit la méthode de décomposition des moments du cadre, proposée par l'auteur, et ses applications pratiques. La valeur théorique et pratique de cette méthode de calcul plastique, pour un poids minimum, est discutée.

ZUSAMMENFASSUNG

Die vom Autor entwickelte Methode der Momentenzerlegung sowie deren praktische Anwendungsmöglichkeiten werden beschrieben. Der theoretische und praktische Wert dieser Methode der plastischen Bemessung auf Minimalgewicht wird untersucht.

Structural Optimization via Penalty Methods: A New Type of Penalty Function

L'optimisation structurale par les méthodes de pénalisation: un nouveau type de fonction de pénalité

Optimierung von Tragwerken durch Strafmethoden: ein neuer Typ von Straffunktionen

ANTONIO DI CARLO
Associate Professor
University of L'Aquila
L'Aquila, Italy

MAURIZIO DI GIACINTO
Research Student
University of Rome
Rome, Italy

1. Introduction

Sound mathematical idealizations of practical design problems lead as a rule to highly nonlinear, and possibly nonconvex, programming problems.

The main effort in the field of computerized design methods should therefore be concentrated upon the implementation of versatile numerical procedures capable of solving, at least in principle, general mathematical programming problems. It is obvious that particular problems can be solved more cheaply by means of 'ad hoc' techniques exploiting their special properties, but it is the authors' opinion that the general approach should yield the major improvements to structural optimization, at the present stage of its development.

In this note, the attention is focussed on sequential unconstrained minimization techniques, which seem to be among the most interesting approaches for general automated design routines. A new kind of penalty function is introduced, and applied to a typical design problem, with the aim of assessing its capabilities.

2. Mathematical formulation

We consider the following type of problem

$$\begin{aligned} &\text{minimize} && f(x_i) \\ &\text{subject to} && g_j(x_i) \leq 0 \end{aligned} \qquad (i=1,\ \dots,\ n;\ j=1,\ \dots,m) \qquad (1)$$

From problem (1) the following parametric problem is derived

$$\text{minimize} \qquad f(x_i) + \sum_{j=1}^{m} < 1 + g_j(x_i) >^{\alpha} \tag{2}$$

where the symbol $<\cdot>$ has the meaning

$$<\cdot> = \max\,(0, \cdot)$$

and the parameter α ranges over the open interval $(1, +\infty)$.

Each inequality constraint $g \leq 0$ is accounted for by a penalty term

$$p(g) = <1+g>^{\alpha} \tag{3}$$

From fig. 1 it is apparent that function (3) is neither an interior nor an exterior penalty function.

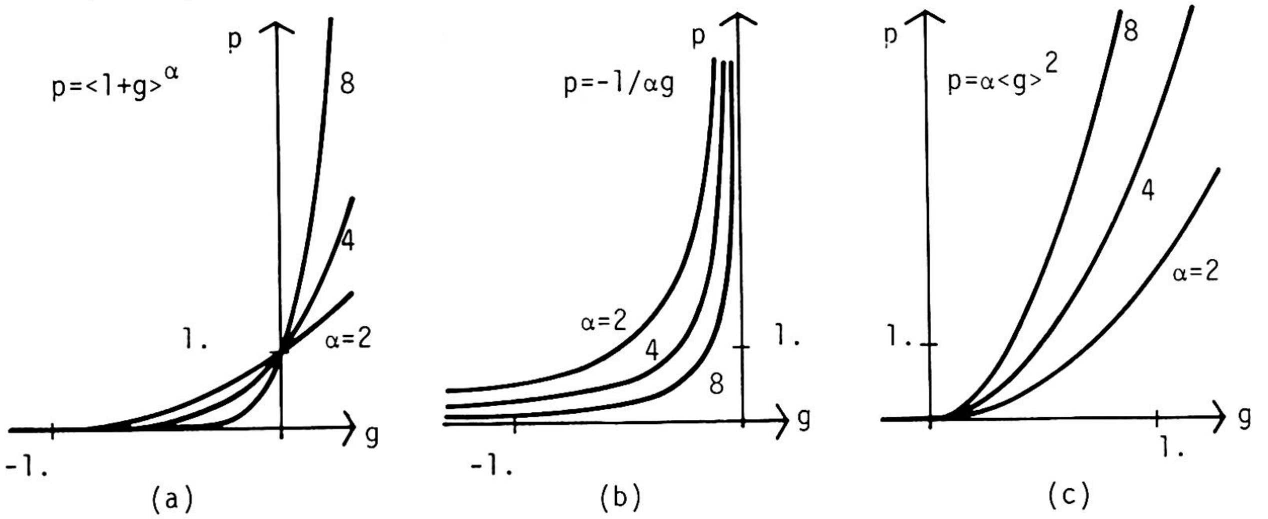

Fig. 1: Proposed penalty function (a) versus interior (b) and exterior (c) penalty functions.

The main properties of formulation (2) may be stated as follows:

i) if problem (1) has a (local) solution, a solution of problem (2) will approach it, when α approaches infinity;

ii) in contrast with interior penalty functions, penalty function (3) is defined over the range $-\infty < g < +\infty$;

iii) in contrast with exterior formulations, formulation (2) yields feasible minima for sufficiently large values of α, i.e. the solution of problem (1) is approached from the inside of its feasible region.

Properties ii) and iii) give an obvious advantage to penalty function (3) over interior and exterior penalty functions, respectively.

3. Allowable stress design of a truss with assigned topology

If a minimum weight design is sought, the objective function is easily ex pressed in terms of member cross-sectional areas and joint coordinates. For each member and each load condition, the following constraints are considered

$$g_+ = \sigma/\sigma^+ - 1 \leq 0$$
$$g_- = \sigma/\sigma^- - 1 \leq 0 \qquad (4)$$

where σ^+ (σ^-) is the allowable tension (compression) stress of the considered member. If member buckling is accounted for, the compression limit σ^- depends on the (minimum) radius of gyration of the member cross-section. For a given type of cross-section, the radius of gyration can usefully be expressed as a function of the area, thus leaving only one design variable for each member. A second set of constraints will impose a minimum admissible value to each area. Displacement constraints may be obviously included.

The major task is to compute the stress and its gradient (the displacement method of analysis is of course preferable). Special attention must be devoted to the fact that stress constraints (4) are not defined over the entire design space: in fact, there exist (unfeasible) designs for which in one or more members the stress grows to infinity. This difficulty can be cured by introducing suitable modifications of the stress constraints (4) outside the feasible region, and by adopting a careful minimization strategy.

4. Numerical results

An algorithm (AUDE) for the numerical solution of automated design problems, based on the described formulation, has been developed. The minimization (2) is performed, for a sequence of suitably increasing (integer) values of α, using the Davidon-Fletcher-Powell method. Two-point cubic fit for successive unidirectional searches is used. Size and geometry variables are treated simultaneously.

The results obtained for a sample design problem, relative to a steel planar truss, are represented in fig.2. The lower chord is assumed to be straight and made up of six bars, long 5 m each. The total span of the upper chord is 30 m also, but its shape is free. All members are tubular, and their thickness is supposed to be adequately represented by the relationship

$$t = 1.5 + 0.02\ D \qquad (t, D \text{ in mm})$$

D being the diameter. Load conditions are specified by a single 10,000 kg concentrated load, moving along the lower chord. The allowable tension stress is assigned a value of 2,400 Kg/cm^2, and the allowable compression stress is computed in terms of the member slenderness ratio according to the Italian Code requirements.

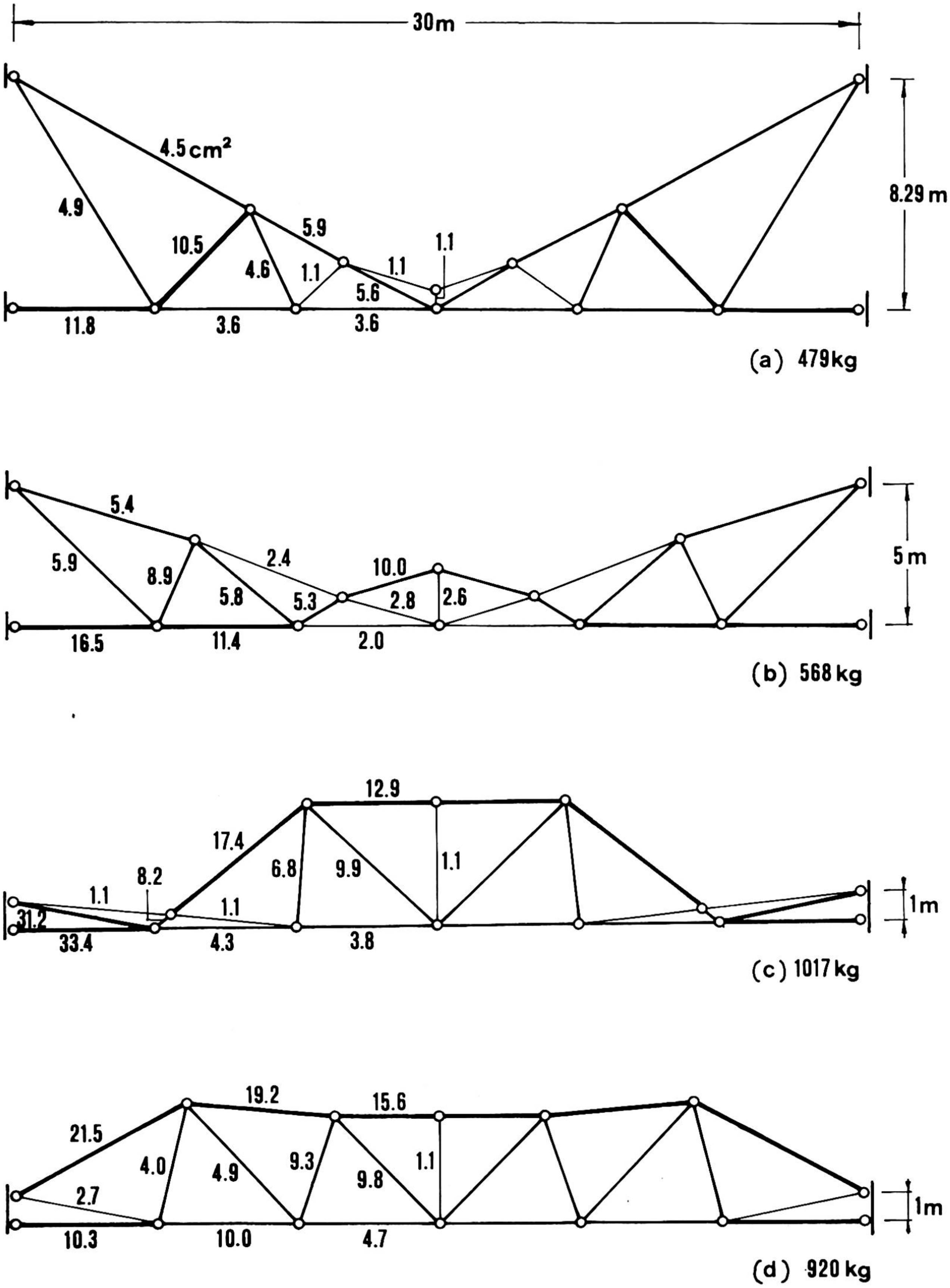

Fig. 2: Truss design

The lower chord should not undergo vertical displacements greater than 1/800 of the span. The truss should be designed for minimum weight.

Taking into account the obvious symmetry of the optimal solution(s), the above stated problem can be treated with 12 size variables, 6 geometry variables, and 3 load conditions. The optimal design obtained by AUDE is depicted in fig. 2a, where the member areas (in cm^2) are also reported. Note that 1.1 cm^2 was the minimum allowable area used in the computation. The weight of the optimum truss is 479 Kg, its height 8.29 m.

If now the distance H between the supports if given a fixed value, the geometry variables reduce to 5, and the optimum weight should obviously increase. Fig. 2b shows the solution obtained for H = 5 m. For H = 1 m two local optima have been detected (figs. 2c, d), the second one being a good candidate for the global solution.

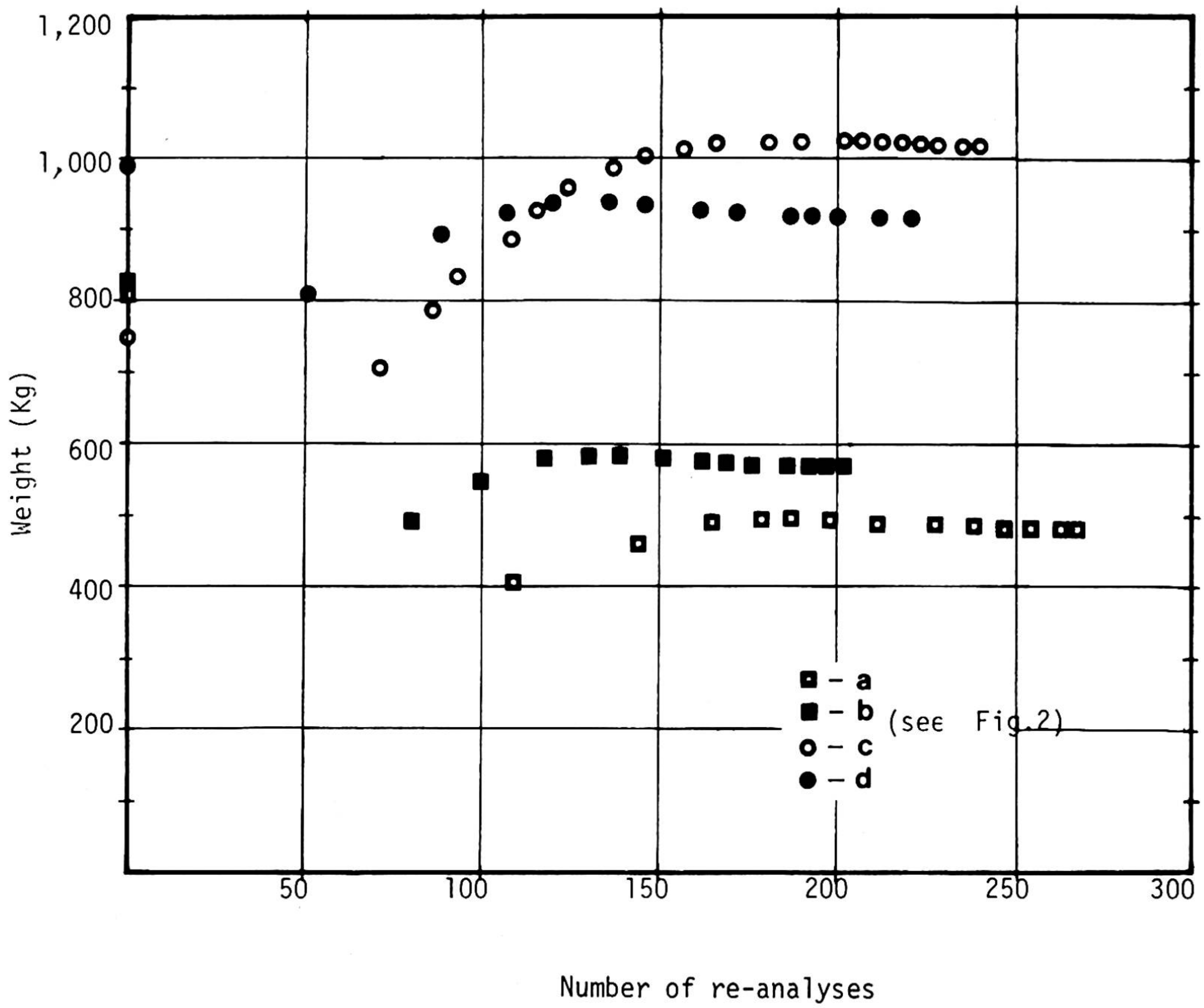

Fig. 3: Minimization trend

In each of these calculations, the parameter α was increased until a value of about 8000, and 200 ÷ 250 re-analyses were performed. Fig. 3 shows the sequences of minima relative to the four cases of fig. 2. As it is seen, after a drastic change on the first response surface, the objective function approaches rather smoothly its asymptotic value.

SUMMARY

An exponential penalty function is introduced and applied to a typical nonlinear and nonconvex design problem. Some results on geometry optimization of plane trusses are presented and discussed.

RESUME

On introduit une fonction de pénalisation exponentielle, et on l'applique à un problème typiquement non linéaire et non convexe d'optimisation structurale. On présente et on discute quelques résultats relatifs à l'optimisation géométrique de structures réticulées planes.

ZUSAMMENFASSUNG

Eine exponentielle Straffunktion wird auf ein typisch nichtlineares und nichtkonvexes Tragwerksproblem angewandt. Einige Ergebnisse über die Optimierung der Geometrie von ebenen Fachwerken werden angegeben und besprochen.

Über die Grundlagen und Methoden der Optimierung

On the Fundamentals and Methods of Optimization

Sur les principes et les méthodes d'optimisation

DIETRICH HARTMANN
Dr.-Ing.
Lehrstuhl für Tragkonstruktionen
Dortmund, BRD

1. Einleitung

Die analytischen Methoden des konstruktiven Ingenieurbaus sind in den letzten Jahren durch verstärkten Einsatz von Digitalrechnern mehr und mehr verfeinert worden. Das dabei erreichte hohe Niveau ist die Folge einer langen Tradition und einer daraus resultierenden Formalisierung der Berechnungsverfahren. Abgesehen von weiteren - sicherlich wichtigen - Verbesserungen (Berücksichtigung nichtlinearen Werkstoffverhaltens und großer Verformungen) sind jedoch keine grundlegend neuen Erkenntnisse mehr zu erwarten. Anders liegen die Verhältnisse bei der Tragwerkssynthese, bei der die Charakteristika des Bauwerkes als Unbekannte betrachtet und im Hinblick auf ein Bewertungskriterium unter Beachtung von technologisch - mechanischen Restriktionen festgelegt oder optimiert werden. Hier befindet man sich erst am Anfang einer Entwicklung, die sicherlich auch Rückwirkung auf die Tragwerksanalyse selbst haben wird.

Innerhalb der Synthese kommt den Optimierungsmethoden tragende Bedeutung zu, da erst mit ihrer Hilfe Syntheseprobleme zu lösen sind. Die Vielzahl der Optimierungen, die nur unzureichend die spezifischen Belange der Ingenieurpraxis berücksichtigen, gibt Veranlassung - in Ergänzung zum Aufsatz von TEMPLEMANN [E, S. 46 ff] einige kritische Anmerkungen zu machen.

2. Kritische Anmerkungen

Zwei Ursachen sind hauptsächlich dafür verantwortlich, daß die Optimierung zu zweifelhaften, da nur akademisch interessanten Lösungen führen kann:

- unrealistisch konzipierte Optimierungsmodelle,
- rein mathematisch orientierte Lösungsverfahren.

Im folgenden soll hierauf kurz eingegangen und einige Anregungen zur Überwindung einer Fehlentwicklung dargelegt werden.

2.1. Konzeption des Optimierungsmodells

Aus der Fülle akademischer Beispiele soll der beidseitig eingespannte Biegeträger minimalen Gewichtes als besonders typisches Beispiel dafür, daß die gefundene "Optimallösung" irrelevant ist, herausgegriffen werden (siehe Bild 1).

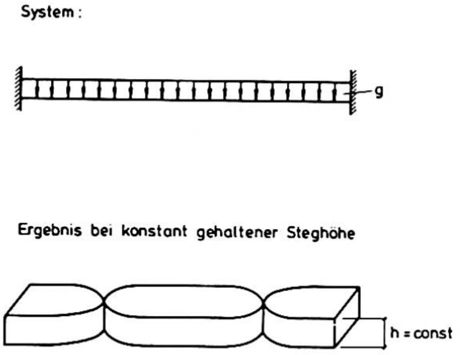

Bild 1: Optimierung des Gewichtes eines beidseitig eingespannten Trägers, ein "akademisches Beispiel" nach Hupfer (1)

Das gefundene Optimum ist die Folge eines schlecht konzipierten (ill - conditioned) Optimierungsmodells, das sich im Normalfall immer aus den drei Elementen

- Optimierungsvariablen
- Optimierungskriterium
- Optimierungsrestriktionen

zusammensetzt. Ist auch nur eine dieser Größen unzutreffend formuliert, zum Beispiel die Zielfunktion, verzerren sich die Optimierungsergebnisse. Viel weitreichendere Folgen stellen sich allerdings ein, wenn Restriktionen fehlen, fehlerbehaftet sind oder nicht in die Form gebracht werden können, wie es das später benutzte Lösungsverfahren verlangt, weil in diesem Fall der Lösungsraum ein völlig anderes Aussehen haben kann als in Wirklichkeit und somit andere Optima möglich sind. So hätte sich bei dem obengenannten Biegeträger mit Sicherheit eine andere Lösung ergeben, wenn technologische Restriktionen (Kontinuität der Kontur) berücksichtigt worden wären.

Will man praxisrelevante Optimierungsergebnisse sicherstellen, müssen alle drei Optimierungselemente ingenieurmäßig aufgebaut sein. Das ist aber im allgemeinen nur dann möglich, wenn man lediglich die Algorithmisierbarkeit der drei Elemente und nicht bestimmte mathematisch erzwungene Ausdrücke fordert.

2.2. Wahl des richtigen Lösungsverfahrens

Viele der vorgeschlagenen, heute gebräuchlichen Lösungsverfahren orientieren sich an der Denkart der Mathematiker, für jeweils eng abgegrenzte Problemklassen Lösungsmethoden zu erstellen, die nur unter bestimmten Voraussetzungen (hinsichtlich Konvexität, Stetigkeit, Differenzierbarkeit und bestimmter Formen der Nichtlinearität, etc.) anwendbar sind. Beispielsweise informiert das Bild 2 über die vielfältigen Varianten allein bei differenzierbarer Zielfunktion. Im Bereich der technischen Optimierung gibt es jedoch nur selten Fälle, die derartig mathematisch klassifizierbar sind. Infolgedessen braucht der Ingenieur eine Art "Generallöser", der universell anwendbar, flexibel und effizient ist. Die Effizienz - ein bislang nicht eindeutig definierter Begriff - soll dabei u.a. danach beurteilt werden,

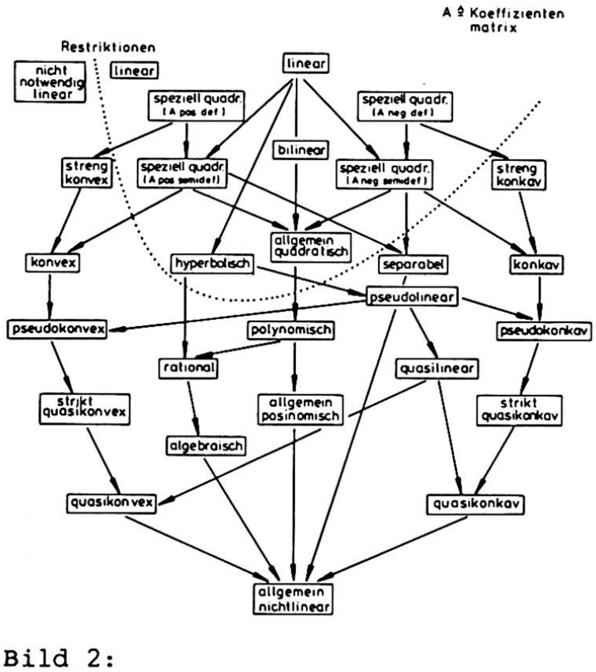

Bild 2:
Klassifikation nichtlinearer differenzierbarer Optimierungsaufgaben
nach Collatz . Wetterling (2)

- wie groß der Arbeitsaufwand zur Anpassung des Lösungsverfahrens an ein beliebig nichtlineares Optimierungsproblem ist,
- wie groß der Arbeitsaufwand zur Anpassung der Zielfunktion und Restriktionen an das Lösungsverfahren ist,
- wie groß der Rechenzeitbedarf (CPU-time) bei bestimmten Testfunktionen ist,
- wie groß der Kernspeicherbedarf ist,
- wie das Konvergenzverhalten bei pathologischen Fällen ist.

Da insbesondere der Arbeitsaufwand zur Anpassung an Gegebenheiten erhebliche Kosten verursacht, muß der universellen Anwendbarkeit eines Lösungsverfahrens - unabhängig von der Klasse des Optimierungsproblems - größter Stellenwert eingeräumt werden. Die meisten der derzeitigen Lösungsverfahren genügen einer solchen Flexibilitätsforderung nicht. Die sogenannte Evolutionsstrategie dagegen [3], [4], die vom Verfasser mit Erfolg bei der Optimierung von Schalentragwerken eingesetzt wurde [4], [5], und im folgenden kurz vorgestellt werden soll, genügt dieser Forderung und kommt - bei entsprechender Weiterentwicklung - dem lang gesuchten "Generallöser" einen Schritt näher.

3. Evolutionsstrategie

Die Evolutionsstrategie ist ein sequentiell arbeitendes, iteratives stochastisches Suchverfahren mit Lernfähigkeit, bei dem die Suchschrittweite den Verhältnissen des jeweiligen Suchraumes angepaßt und selbst optimiert wird. Da die Suchschrittweite eine Zufallsvariable ist, kann man die Strategie als Monte-Carlo-Simulation höherer Stufe bezeichnen, deren Effizienz im Vergleich zu anderen Verfahren sich besonders bei vielen (ab 10 Variablen) bemerkbar macht. Sie darf aber auf keinen Fall mit der eigentlichen Monte-Carlo-Simulation verwechselt werden, weil ihr methodisches Vorgehen erheblich vom bekannten Monte-Carlo-Verfahren abweicht.

Das "Geheimnis" des Erfolgs und der Ausbaufähigkeit dieser Strategie ist darin begründet, daß die innere Logik des Verfahrens Optimierungsmechanismen der biologischen Vererbung, deren optimierender Effekt die Biologie tausendfach beweist, simuliert.

Durch eine einfache Konvergenzregel wird erreicht, daß optimale Fortschrittsgeschwindigkeit erzielt wird. Diese ist dann gegeben, wenn im Durchschnitt nach jeweils 5 zufälligen Suchvorgängen 1 Erfolg (Qualitätsverbesserung) erreicht wird. Andernfalls ist die Suchschrittweite (besser die Streuung der Suchschrittweite) zu vergrößern oder zu verkleinern. (Bild 3 zeigt die prinzipielle Arbeitsweise der Strategie an einem Beispiel).

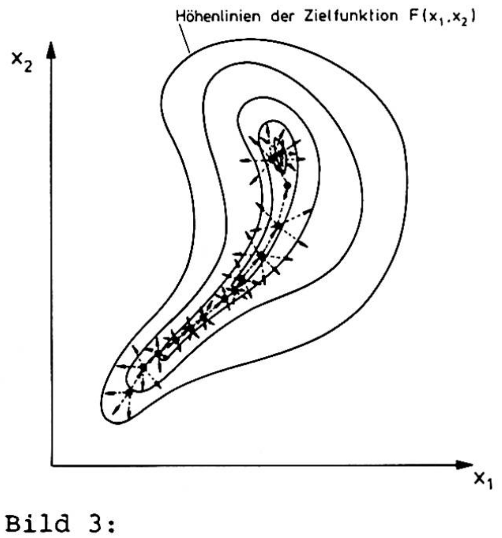

Bild 3:
Arbeitsweise der Evolutionsstrategie an einem zweidimensionalem Beispiel

4. Abschließende Bemerkung

Mit der Evolutionsstrategie besitzt der Ingenieur ein geeignetes Werkzeug, um Optimierungsprobleme im Konstruktiven Ingenieurbau zu lösen. Da die Routine in programmierter Form vorliegt und flexibel anwendbar ist, braucht sich der Benutzer nur noch um das problemabhängige Optimierungsmodell zu kümmern, eine Aufgabe, die jeder Ingenieur ohne große Mühe bewältigen kann.

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ZUSAMMENFASSUNG

Es werden kritische Anmerkungen zum Aufbau eines auf die Praxis ausgerichteten Optimierungsmodells gemacht und die universell anwendbare Evolutionsstrategie wird als Lösungsverfahren technischer Optimierungsprobleme vorgestellt.

SUMMARY

Some critical remarks are made for the establishing of practical optimization models. Furthermore, a generally applicable solution method, the "evolution strategy" is proposed.

RESUME

Des considérations critiques sont faites pour l'établissement de modèles d'optimisation répondant aux besoins de la pratique. La méthode de la "stratégie évolutive" peut être utilisée de façon universelle pour résoudre des problèmes techniques.

Cora's Lesson

La leçon de Cora

Die Lehre von Cora

A.L. BOUMA
Professor of Engineering Mechanics
University of Technology Delft
Delft, the Netherlands

F.P. TOLMAN
Head Software Development
Inst. T.N.O. for Building Materials and Building Structures
Rijswijk, the Netherlands

1. INTRODUCTION.

Since 25 years we have been acquainted with the development of optimization-techniques for the design of structures. The first programs were mainly based on the "mechanism" approach of limit analysis.
After a general matrix theory for structures became available about 1966, design-programs based on the equilibrium method were developed [1].
Formulating a minimum-weight design, based on the equilibrium method, with only stress-limitations as constraint conditions, leads to a problem of linear programming (L.P.).
In practice, however, a design with only stress-limitations is not acceptable. The building codes require a number of additional constraint conditions, for instance with relation to stability and rotation capacity (fig. 3). Often these special conditions are strongly non-linear and they can be linearized only in the neighbourhood of a solution, so that one has to pass on to a non-linear programming technique.

2. CORA.

Recently a design program has been developed in the Netherlands (named CORA) for the design of braced steel frames. The designs meet the requirements by the national authority. Moreover, it is possible to distinguish between welded joint connections with or without stiffeners. Given an accepted input of geometry and loading (fig. 1), the program automatically produces the design of the frames using a sequential linear programming formulation (fig. 2). The introduction of the special constraint conditions means that a very large linear programming problem has to be solved a number of times. The solution has been found in applying a sophisticated L.P. algorithm and in the approximation of the constraint conditions by one plane with the aid of the method of least squares. Instead of five sets of constraint conditions (fig. 4) only one set has to be taken into consideration (fig. 5). It proved that 3 or 4 iterations were sufficient to obtain the theoretically exact values of the solution.

Problems arose from the wish to develop an instrument which will really be used in practice. This means that it should not be too expensive in use and that it should fit realistic structures. The more difficult problems however were formed by the codes themselves. In drawing up these codes the committee has had in mind of course a more or less sensible structural engineer and a proper structure. But in applying these rules and codes in an automated design program irrevocably gaps and inconsistencies prove to be present in the codes. The computer is not a sensible structural engineer and he stumbles in the pitfalls caused by these gaps. Especially a mathematical optimization technique is a master in finding the inadmissible minima, as we noticed to our regrets several times.

3. THE LESSON.

The experience gained with this program has led us to the insight that if, in developing a design-process, one has to make allowance for requirements made by the government or a local authority, the design-program has to be separated from these prescriptions or codes. This applies to computer-aided design and the more so to computer-automated design. The reasons for this opinion - which we believe should be generally accepted - are:

a. The codes and prescriptions contain gaps and inconsistencies, which will always be recognized by the optimization-technique and unfortunately exploited.

b. The programmer who builds the design-program is not allowed to improve these inconsistencies.

c. By integrating the code into the design-program the program becomes dependent on this code. Codes have a temporary character. Adapting a design-program to code-changes will in general be very expensive or even impossible.

d. Working up codes into a form which is understandable by the computer is a lot of work which can best be left to those who draw up the codes instead of to every individual programmer.

4. CONCLUSION.

If - at least for the Building Industry - we want to leave behind us the more or less trivial examples, to proceed with our design-techniques to real life structures, we have to create the possibility to develop C.A.D.-programs that are independent from the codes. Therefore the codes have to be brought into a computer-readible form for instance in the way - indicated by Fenves [3] - by means of Decision Logic Tables. If this effort has led to success, the codes can be changed without consequences for the design-program (of course it will have consequences for the design itself).
Because the programs will be less vulnerable, software development for C.A.D. will become more popular. Two consequences seem to be of particular importance:

a. To develop C.A.D.-programs that can make codes of different countries accessible. This will make the software less dependent and less "national".

b. To study beforehand the effect of proposed code-changes, e.g. on economics or safety.

5. ACKNOWLEDGEMENT.

The CORA-program has been developed in the T.N.O. Institute for Building Materials and Structures, at the request of the Steel Structure Society and with financial aid of the C.I.A.D. (Computer Society of civil engineers). The first author was the chairman of the steering committee, the second author was responsible for the technical contents and the production. Most of the work was carried out by Ir. A.K. de Groot, Ing. G. Kusters and Ir. B. Speelpenning.

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SUMMARY

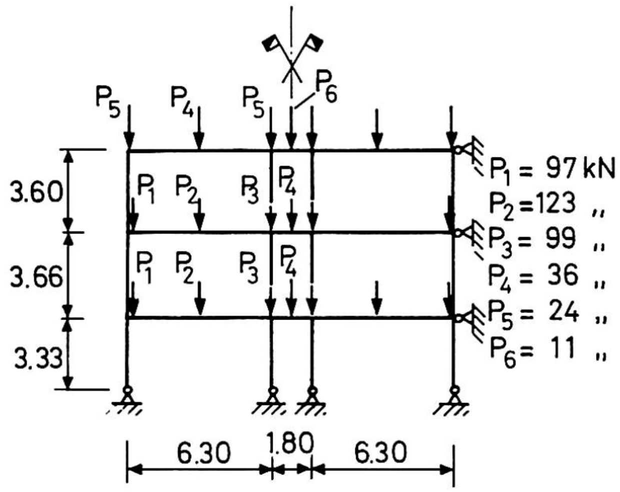
During the last three years a computerprogram (CORA) has been developed in the Netherlands for the design of braced steelframes. The frames designed with this program meet the requirements, made by the national authority. The building code requirements have been integrated into the design program, which has led to a very complex problem. The experience gained by solving this problem, has led to the insight, among other things, that design programs should be separated from rules and requirements in the building codes.

RESUME

Pendant les derniers trois ans un programme d'ordinateur (CORA) a été développé aux Pays-Bas pour le dimensionnement des ossatures en acier. Les ossatures dimensionnées avec ce programme satisfont aux conditions posées par les règles nationales pour les structures en acier. Les règles ont été insérées dans le programme de dimensionnement ce qui a conduit à un problème très compliqué. L'expérience a conduit à la conclusion que les programmes de dimensionnement doivent être séparées des conditions dans les règles ou codes.

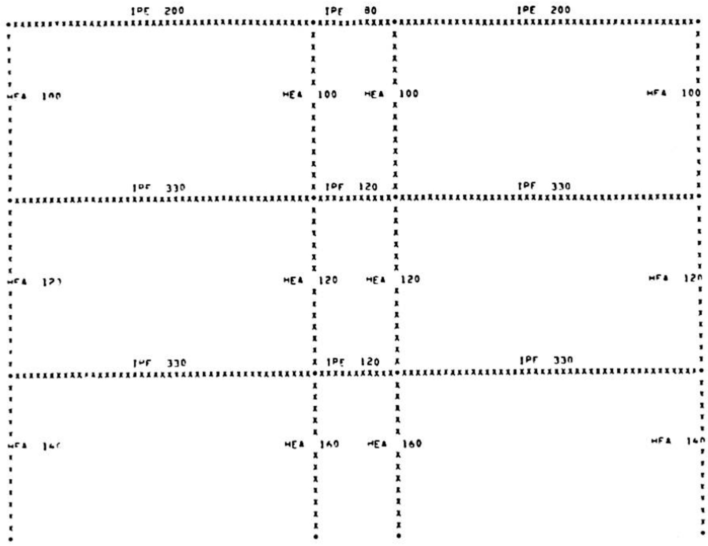
ZUSAMMENFASSUNG

Während der letzten drei Jahre ist in den Niederlanden ein Computerprogramm (CORA) für das Entwerfen von Rahmentragwerke aus Stahl entwickelt worden. Die Tragwerke die mit diesem Programm entworfen sind, erfüllen die Forderungen der nationalen Behörden. Die bautechnischen Anordnungen sind in das Entwurfsprogramm aufgenommen worden, was zu einem sehr komplizierten Problem geführt hat. Die Erfahrungen haben zu der Ansicht geführt, dass Entwurfsprogramme unabhängig und separat von Regeln und Forderungen in bautechnischen Anordnungen sein sollen.



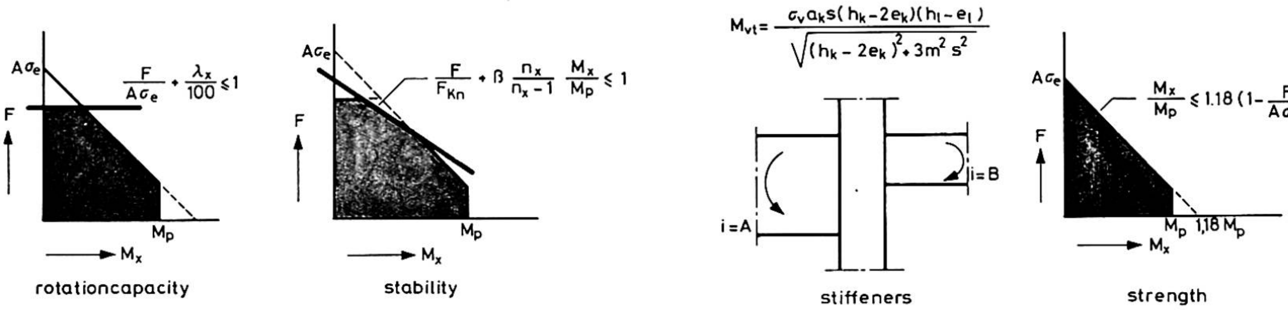
geometry and loading

fig.1



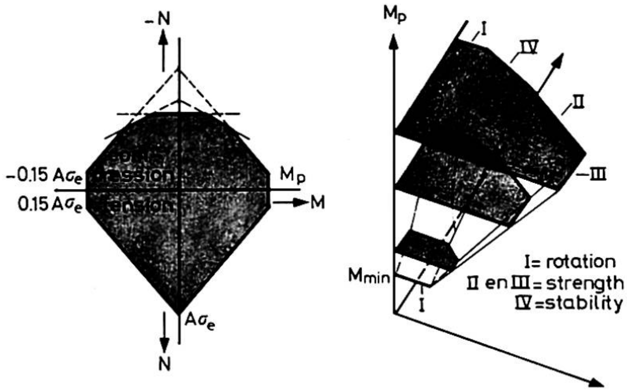
lineprinter plot

fig.2



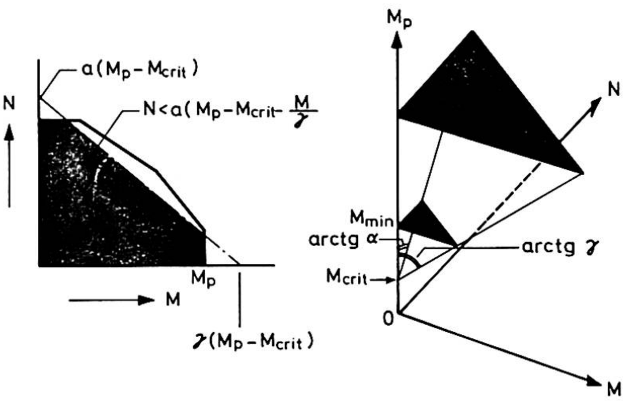
constraints

fig. 3



constraint surface

fig.4



approximate constraint surface

fig.5

Basic Optimum Design Diagrams of Highway Plate Girders

Diagrammes fondamentaux pour le calcul optimum de ponts-poutres à âme pleine

Grunddiagramme für den optimalen Entwurf von Vollwand-Brückenträgern

SADAJI OHKUBO
Associate Professor
Ehime University
Matsuyama, Japan

TOSHIE OKUMURA
Professor Emeritus
University of Tokyo
Tokyo, Japan

1. INTRODUCTION

In the structural system optimization process multistage systematic comparisons and decisions in such as behavior analysis, fabrication, erection, appearance etc. have to be made with regard to a number of optimized solutions of a wide variety of different structural schemes. For the progress of such a design process, the developments of the optimum design programs for specific structures are the essential efforts. The design approaches centered on this problem have been developed principally based upon the use of a combination of mathematical programming or rigorous numerical search techniques and structural analysis method. However the man-machine communication using these sophisticated optimization programs causes numerical complexity and large computer costs considerably in the large scale structural systems, accordingly, to avoid the difficulties and to make a realistic large scale system optimization process reliable, more practical and efficient design approaches have been expected [7,8].

On this problem a simplified optimum design process for minimum cost design of constant-depth highway plate girders is presented in this paper on the basis of the data manipulation of system optimum design diagrams developed by the graphical optimum design method [1,2] and the fundamental characteristics of the diagrams are discussed.

2. DESIGN CONDITIONS

The bridge types to be objective in this paper are 1st class simple span, 2 and 3 span continuous highway welded plate girders with constant web height throughout the bridge length. The range of bridge length, width, span ratio and the number of segments are indicated in Table 1. The arrangements of the main girders and thicknesses of reinforced concrete slabs are assumed as shown in Fig. 1 and Table 2 with due regard to the load distributions and appearances of the bridge superstructures.

The design variables to be determined are the steel type, M, all sectional dimensions, X, and the length, ℓ, to be used for each girder segment.

Design criteria imposed on the steel girder section are constraints on allowable stresses, plate thicknesses for stabilities of the girder and minimum rigidities of vertical and horizontal stiffeners, and which, as well as the displacement constraints of the girder, are taken from the "Specifications for Steel Highway Bridges" [5]. Discrete constraints on commercial availability of plate thicknesses are also considered. The steel types available for the design are assumed as SS41, SM50 and SM58 steels which physical and economical characteristics are described in the Specifications and the "Table of Prime Costs for Steel Highway Bridges" [6] respectively.

The total cost of the girder, TCOST, is assumed to consist of material cost, CM, fabrication cost, CFF, and welding cost, CWF, which are evaluated with reference to Ref. [6].

$$\mathrm{TCOST} = \sum_{i=1}^{NS} \mathrm{COST}_i \times \ell_i$$
$$= \sum_{i=1}^{NS} [\mathrm{CM}_i(X,BP,SP,EX) + \mathrm{CFF}_i(X,KS,CP) + \mathrm{CWF}_i(X,WL,CP)] \times \ell_i \qquad (1)$$

in which NS = number of segments, COST = minimum cost per unit length of girder segment, BP = base price of steel plate per unit weight, SP = extra charge for steel type, EX = extra charge for plate thickness and width, KS = number of workmen to fabricate unit weight of steel plate except welding, CP = daily wage of a workman, WL = welding length in specific size. SP and the charge for pre-heating in welding vary with steel type used.

3. EFFECTS OF PRICE RATIOS

Since the optimum solution and the total cost of a girder with specific bridge length and width are complex functions of the design factors such as X,BP, EX,CP,KS, etc. as described in the previous section, it seems to be considerably difficult to presume the effects to the optimum solutions due to the changes of the design factors. However the ratio of CP to BP may be supposed to be the most effective factors. Consequently at first stage of this study the effects due to the changes of unit price ratios CP/BP to the optimum solutions are investigated.

Table 3 shows the examples of optimum solutions for the girders at various price ratios. As seen clearly in this table the optimum steel type, M, moment of inertia, I, and length, L, to be used for each girder segment are extremely insensitive to a wide variety of the unit price ratios except the case CP/BP=0.0 which is an unconsiderable case in the practical design problems. These phenomena can be seen at every span lengths and bridge types in the minimum cost design of highway plate girders and it has been cleared that the optimum design diagrams presented in the following sections may be applicable for a wide range of the ratios of CP to BP with sufficient accuracy.

4. SL(BL)-OPTIMUM WH RELATIONSHIPS

The minimum total cost of the plate girder with specific SL, BW and NS is changed concavely with web height on the whole, but locally several minimum solutions exist as noted in Ref. [1,2]. The optimum web height, WH_{op}, therefore should be decided by comparing the results in each discrete web

Table 1. Bridge Types, Lengths, Widths, Span Ratios and Segment Numbers

BRIDGE TYPE	BRIDGE LENGTH	BRIDGE WIDTH	SPAN RATIO	SEGMENT NUMBER
SIMPLE SPAN G.	20~40 m	6,7,8,9 m	——	5,7,9
2 SPAN CONT. G.	40~80	6,7,8,9	1.00:1.00	8,10,12,14
3 SPAN CONT. G.	60~120	6,7,8,9	1.00:1.00:1.00 1.00:1.30:1.00	13,15,23

(Unit = m)

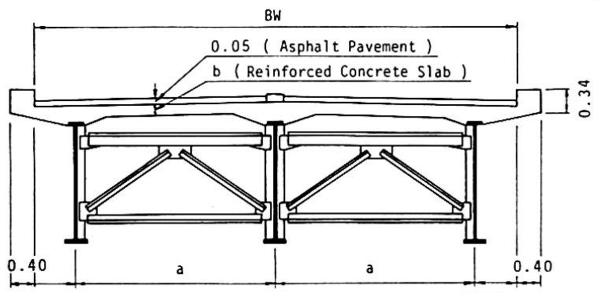
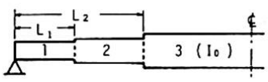


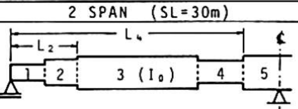
Fig. 1. Cross Section of Girder Bridge

Table 2. Girder Spaces (a) and Thicknesses of Reinforced Concrete Slabs (b)

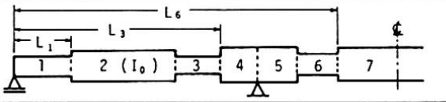
Bridge Width = BW (m)	6.00	7.00	8.00	9.00
a (m)	2.4	2.8	3.3	3.8
b (cm)	19.0	20.0	22.0	23.0

Table 3. Examples of Optimum Solutions for the Girders at various Price Ratios
(BW = 8.00m , WH = 170cm)





CP/BP	1 SPAN (SL=30m) Seg. 1 M_1	I_1/I_0	L_1/BL	Seg. 2 M_2	I_2/I_0	L_2/BL	2 SPAN (SL=30m) Seg. 2 M_2	I_2/I_0	L_2/BL	Seg. 4 M_4	I_4/I_0	L_4/BL
0.0	SS41	0.656	0.098	SM50	0.963	0.228	SS41	0.984	0.097	SS41	0.882	0.447
0.023	SM50	0.501	0.103	50	0.966	0.230	SM50	0.749	0.102	SM50	0.678	0.447
0.045	50	0.504	0.104	50	0.969	0.231	50	0.746	0.101	50	0.708	0.444
0.068	50	0.531	0.110	50	0.964	0.229	50	0.748	0.101	50	0.709	0.444
0.082	50	0.532	0.111	50	0.963	0.229	50	0.752	0.103	50	0.664	0.451
0.098	50	0.533	0.111	50	0.967	0.230	50	0.757	0.104	50	0.665	0.450
0.136	50	0.502	0.103	50	0.946	0.223	50	0.757	0.104	50	0.665	0.451



3 SPAN (BL=80m, η=1.25) Seg. 1 M_1	I_1/I_0	L_1/BL	Seg. 3 M_3	I_3/I_0	L_3/BL	Seg. 7 M_7	I_7/I_0	L_6/BL
SS41	0.564	0.043	SS41	0.725	0.273	SS41	0.969	0.424
SM50	0.566	0.043	SM50	0.755	0.270	SM50	0.976	0.429
50	0.566	0.043	50	0.755	0.270	50	0.976	0.429
50	0.566	0.043	50	0.756	0.270	50	0.977	0.429
50	0.560	0.042	50	0.753	0.269	50	0.974	0.431
50	0.566	0.043	50	0.756	0.270	50	0.977	0.429
50	0.566	0.043	50	0.755	0.270	50	0.978	0.429

BP = Base Price of Steel plate (yen/ton)
CP = Daily Wage of a Workman (yen/(person·day))
M = Opt. Steel Type
I = Opt. Moment of Inertia (cm^4)
L = Opt. Distance of variation of Sectional Dimensions from Left End Support

height. Fig. 2 shows WH_{opt} for every bridge types and bridge widths. They are changed stepwise and keep constant values for fairly wide ranges of span lengths. Discrete web plate thicknesses play an important role in the decision of optimum web heights and the ratios of web heights to thicknesses fairly coincide with the upper limits prescribed by the stability constraints of the girders. It is to be notable furthermore that only 170,200 and 230cm are selected almost as optimum web heights for the plate girders with nonuniform cross sections with a few exceptions of 190cm at the small ranges of 2, 3 span continuous girders.

5. OPTIMUM M, I, L, SDIM

Optimum M, I and L to be used for the segments of the girder with the optimum web height decided in the previous section may be determined efficiently by using graphical optimization algorithms [1,2]. In the algorithms M_{opt} and I_{opt} for each segment are determined by comparing the minimum costs of unit length of girder segment for every steel type namely SS41, SM50 and SM58 steels which are evaluated straightly with use of the maximum working bending moment, BM, and the relationships between moment of inertias and maximum resisting bending moments, RBM, and minimum costs, COST, of the girder sections which are provided from the suboptimization of girder elements. According as bending moment increases, higher strength

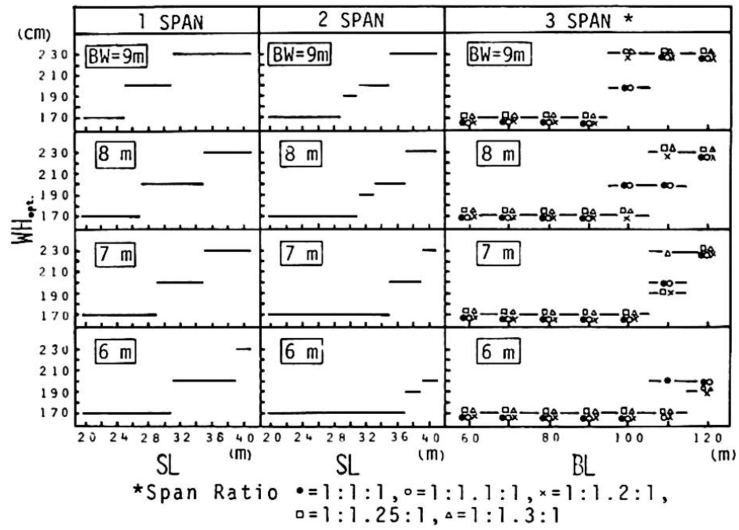


Fig. 2. SL(BL)-$WH_{opt.}$ Relationships for 1,2,3 Span Girders

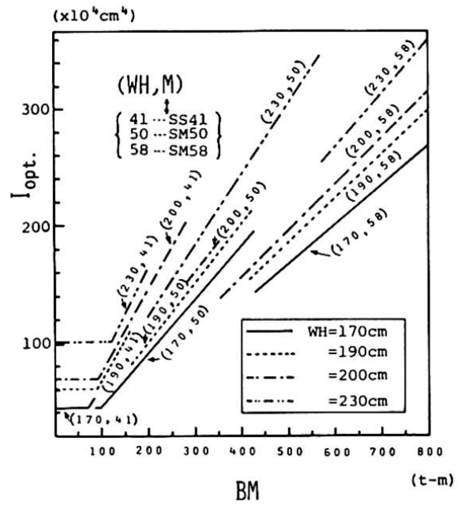


Fig. 3. BM-Opt. M and I Relationships (WH=170,190,200,230cm)

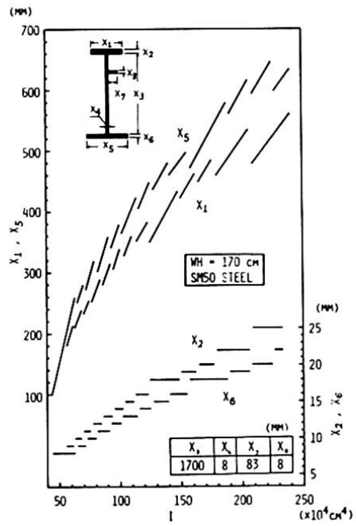


Fig. 4. I-SDIM Relationships for WH=170cm, SM50 Steel

Table 4. Opt. $L_i/SL(BL)$ for the Bridge Types and Number of Segments

Type of Girder	Span Ratio	Number of Seg.	$\xi\ (=L_i/BL)$ (Symmetric)							
SIMPLE SPAN G.		5	0.104	0.238	0.500					
		7	0.061	0.134	0.234	0.500				
		9	0.053	0.118	0.194	0.298	0.500			
2 SPAN CONT.G.		8	0.070	0.323	0.447	0.500				
		10	0.069	0.330	0.434	0.468	0.500			
		12	0.046	0.101	0.302	0.378	0.453	0.500		
		14	0.046	0.101	0.333	0.431	0.457	0.478	0.500	
3 SPAN CONT.G.	1:1.00:1	13	0.049	0.237	0.309	0.334	0.376	0.455	0.500	
		15	0.031	0.067	0.233	0.305	0.334	0.384	0.463	0.500
	1:1.10:1	13	0.047	0.220	0.295	0.323	0.363	0.439	0.500	
		15	0.030	0.066	0.216	0.291	0.323	0.367	0.466	0.500
	1:1.20:1	13	0.046	0.203	0.280	0.313	0.351	0.429	0.500	
		15	0.029	0.064	0.198	0.279	0.313	0.349	0.434	0.500
	1:1.25:1	13	0.046	0.194	0.273	0.308	0.345	0.424	0.500	
		15	0.028	0.062	0.192	0.272	0.308	0.344	0.428	0.500
		23	0.029	0.063	0.180	0.209	0.262	0.287	0.308	0.328
			0.354	0.410	0.441	0.500				
	1:1.30:1	13	0.042	0.188	0.263	0.303	0.340	0.420	0.500	
		15	0.027	0.060	0.183	0.264	0.303	0.340	0.424	0.500

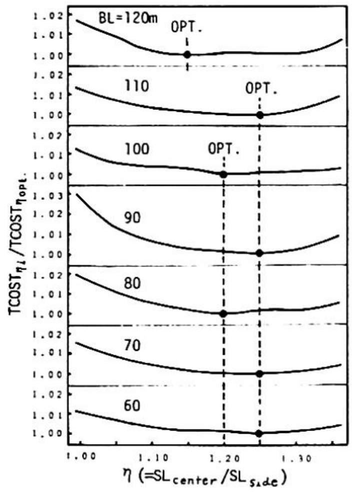


Fig. 5. Span Ratio-TCOST Ratio Relationships for 3 Span Continuous Girders

steels namely from SS41 to SM50 and SM58 are selected as optimum steel types and the corresponding optimum I are therefore varied discretely. The relationships between BM and optimum M, I for WH_{opt} = 170, 190, 200, 230cm are shown in Fig. 3. Horizontal lines of the relationships are caused by the constraints on minimum widths and thicknesses for flange plates. The arrangement of optimum steel types and moment of inertias of the girders therefore differ with values of the maximum bending moment diagrams. On the other hand, the ratios of optimum segment lengths do not change so much with span lengths and web heights, as tabulated in Table 4, they may be represented by a particular set of the ratios for a specific segment number and girder type with quite few exceptions. It has been cleared that the increments of total costs of the girders designed with these segment length ratios are less than 0.3% at most compared with the correct solutions. Optimum sectional dimensions, SDIM, of the girder segments can be decided directly from the moment of inertia and optimum sectional dimension diagrams for the web height and steel type as shown in Fig. 4 for WH=170cm and M=SM50 steel. Flange widths X_2 and X_6 are increased like as sawteeth with discrete changes of flange plate thicknesses as seen in the figure.

6. SPAN RATIO-TCOST RELATIONSHIPS OF 3 SPAN CONTINUOUS GIRDERS

For the 3 span continuous girder design, the variation in the minimum total cost with span ratio may be one of the interesting features and the relations for the girders with nonuniform cross sections are presented in Fig. 5. According to this result the most economical span ratio for plate girder superstructures with nonuniform cross sections is scarcely varied with bridge lengths and it may be decided as almost 1.20∼1.25. It should be emphasized also, however, the fact that the relative differences of total costs in the range of span ratio from 1.10 to 1.35 are considerably small as less than 0.7%. On the contrary, the optimum span ratio for the girder with uniform cross section reduces with bridge lengths from 1.10 to 0.95 for bridge length 60m∼120m and the relative differences of total costs are changed so much as the order of several percents.

7. ABM-OPTIMUM WH, M, I, SW RELATIONSHIPS FOR THE GIRDER WITH UNIFORM CROSS SECTION

Optimum solutions of the girders with uniform cross section may be decided directly such as example A or B in Fig. 6 by using the relationships between absolute maximum bending moment, ABM, and optimum design variables, WH, M, I and SW without reference to the bridge types. Different from the cases of girders with nonuniform cross sections, in which only one optimum solution is obtained for each span length, two optimum solutions, one for SM50 steel another for SM58 steel, which give the same minimum total cost are existed in a wide range of absolute maximum bending moments. Optimum web heights for the girder with uniform cross section differ with optimum steel types and they are decided almost as 170, 240, 270cm for SM50 steel and 200, 230, 260cm for SM58 steel.

8. DESIGN PROCEDURE USING OPTIMUM DESIGN DIAGRAMS

By using the design diagrams stated above as data banks, the minimum cost design of highway plate girders may be carried out simply, and the design procedures are shown in Fig. 7. The design process begins by choice of the segment number, NS, of the girder. The optimum web height and segment lengths are decided from SL(BL)-WH_{opt} and SL-L/BL relationships respectively. In the next place assumptions are made of moment of inertia and girder weights at each segment, the girder is then analyzed. The optimum steel type, moment of inertia and weight to resist the maximum bending moment at each segment are decided straightly from BM-Opt. M, I, SW relationships for the optimum web height. The girder analysis should be repeated until maximum bending moment, namely optimum M, I, SW, are converged. Optimum sectional dimensions for each segment are decided derectly from I-SDIM relationships for the web height and steel type. The total cost of the optimized girder is computed by using I-COST relationships or eq.(1). The entire looping is then carried out again for other NS if necessary. The girders with arbitrary web heights may be designed similarly without the selections of WH_{opt}.

9. CONCLUSIONS

This paper has dealt with fundamental optimum design diagrams effective to the minimum cost design of constant depth highway plate girders and an efficient systematic optimum design process based on the data manipulation of the design diagrams has been proposed.

The design diagrams described herein in detail are only that for the optimum web heights at each span length, but the diagrams for other considerable web heights have been developed as well. It has been confirmed also that the diagrams are applicable with accuracy to a wide range of unit price ratios of materials to workmen.

By using the optimum design diagrams as data banks, the minimum cost design of highway plate girders may be carried out by the simple

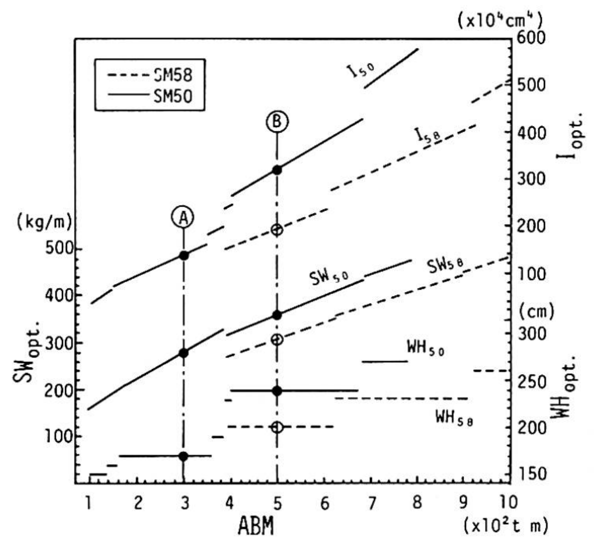


Fig. 6. ABM-Opt. WH,M,I,SW Relationships for the Girder with Uniform Cross Section

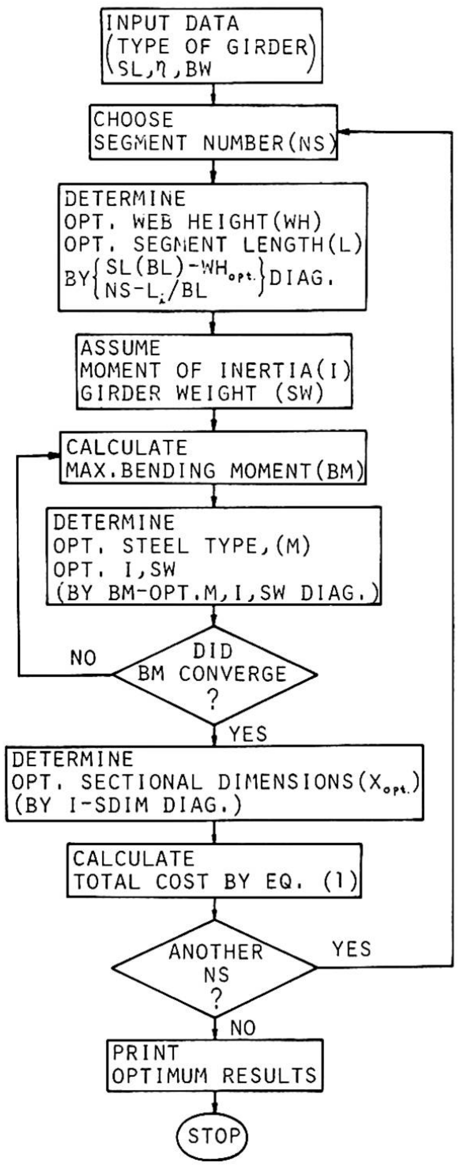


Fig. 7. Flow Chart of the Design Procedures

design procedures. The main computation required for the design is girder analysis only and most part of the design decisions can be made straightly from the diagrams.

The design procedures proposed in this paper may be utilized as one of the element design programs for specific structures in a general purpose system optimization program for highway bridges.

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SUMMARY

The basic optimum design diagrams are presented for the purpose of minimum cost design of 1 to 3 span highway plate girders; the fundamental characteristics of the diagrams are clarified. By using the design diagrams as data banks, minimum cost design of highway plate girders may be carried out in detail by a simple design procedure. The design procedure can be utilized as one of the element design programs in a general purpose system optimization program for highway bridges.

RESUME

On présente les diagrammes fondamentaux et leurs caractéristiques essentielles, pour un calcul, avec frais minimum, d'un pont-route à poutres à âme pleine de 1 à 3 travées. L'usage de ces diagrammes comme banque de données permet de faire le calcul détaillé selon un procédé simple, qui peut être utilisé comme un des programmes élémentaires d'un système général d'optimisation pour les ponts-routes.

ZUSAMMENFASSUNG

Grunddiagramme des optimalen Entwurfes werden mit ihren Charakteristiken dargestellt. Sie erlauben eine Berechnung mit Minimalkosten von 1 bis 3 feldrigen Strassenbrücken aus Stahl-Vollwandträgern. Diese als Datenbank verwendeten Diagramme gestatten ein Minimalkosten-Entwurf bis ins Detail. Das Verfahren lässt sich als eines der elementaren Entwurfsprogramme im allgemeinen System-Optimierungs-Programm für Strassenbrücken benutzen.

A Contribution to the Optimum Design of Prestressed Plane Cable Structures

Une contribution au calcul optimal de structures planes de câbles prétendues

Ein Beitrag zur Optimierung von ebenen vorgespannten Seiltragwerken

FABIO SELLERI
Professore di Scienza delle Costruzioni, Facoltà di Architettura
Università di Roma
Roma, Italia

A research for the optimum design of prestressed plane cable structures is here biefly referred.

The starting point are P. Pedersen's (1) and G.N. Vanderplaats and F. Moses's (2) papers, where trusses are optimized when cross-sectional areas and joint coordinates are the design variables. The techniques here used must be substantially changed to be applied to prestressed cable structures. This is due to initial prestress (the necessary prestress to grant the wanted design requirements) which is to be kept constant under different loading conditions while cross-sectional areas and the geometry of the structure change. This need really complicates the computational process since, keeping the initial prestress constant, big changes in bar forces correspond to small displacements in joints.

Two kinds of prestressed plane cable structures have been optimized: a standard scheme[1] [Fig. 1,a] and a new type of prestressed cable-stayed structure[2] [Fig. 1,b] whose good static and dynamic behaviour had been shown in a previous work (3).

Both types of structures satisfy the necessary and sufficient conditions for fully stressed design, when at least two load systems are considered. Two uniformly distributed loads, a downward load (p_{max}) and an upward load (p_{min}), are here considered besides prestressing.

The problems relating to prestressed cable structures are of non-linear type. Yet, linear theory has been used because the very many checks carried on by the non-linear theory (3),(4) showed that results from linear analysis are valid, at least as far as localization of good structural parameters is concerned.

1. Henceforth indicated by TSHS (two surface hanging structure).

2. Abbreviated by PCSS .

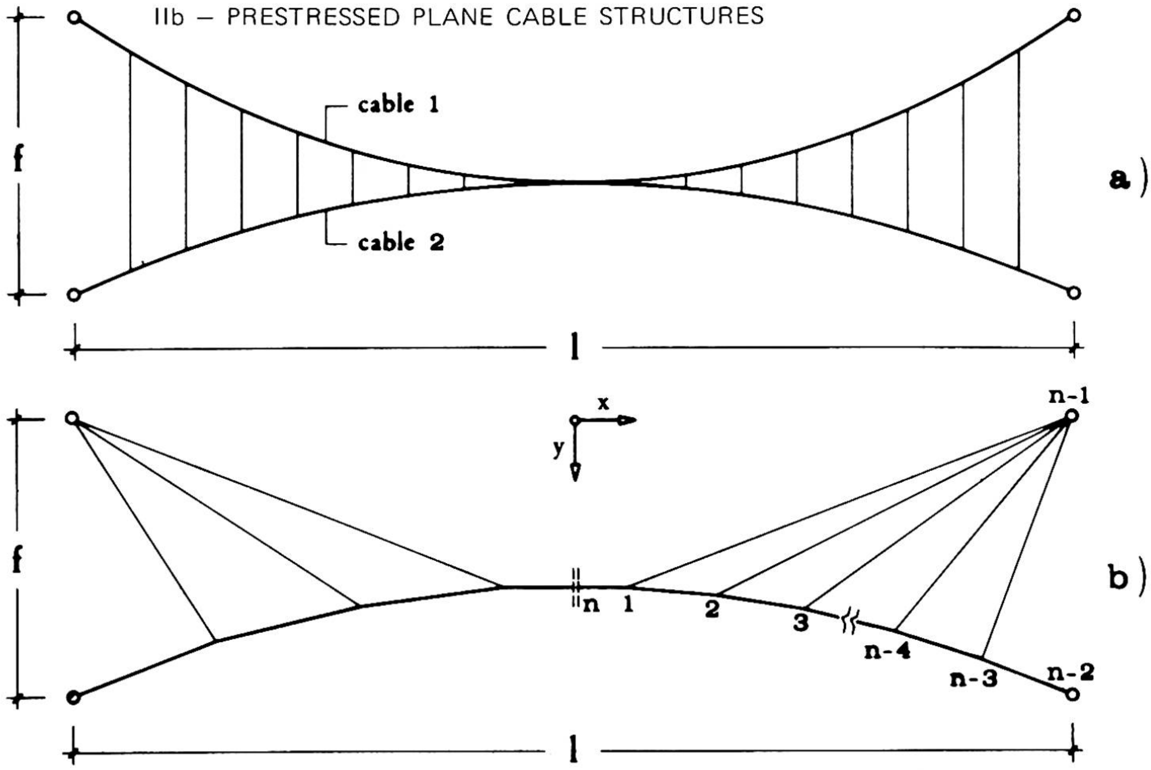

Fig. 1

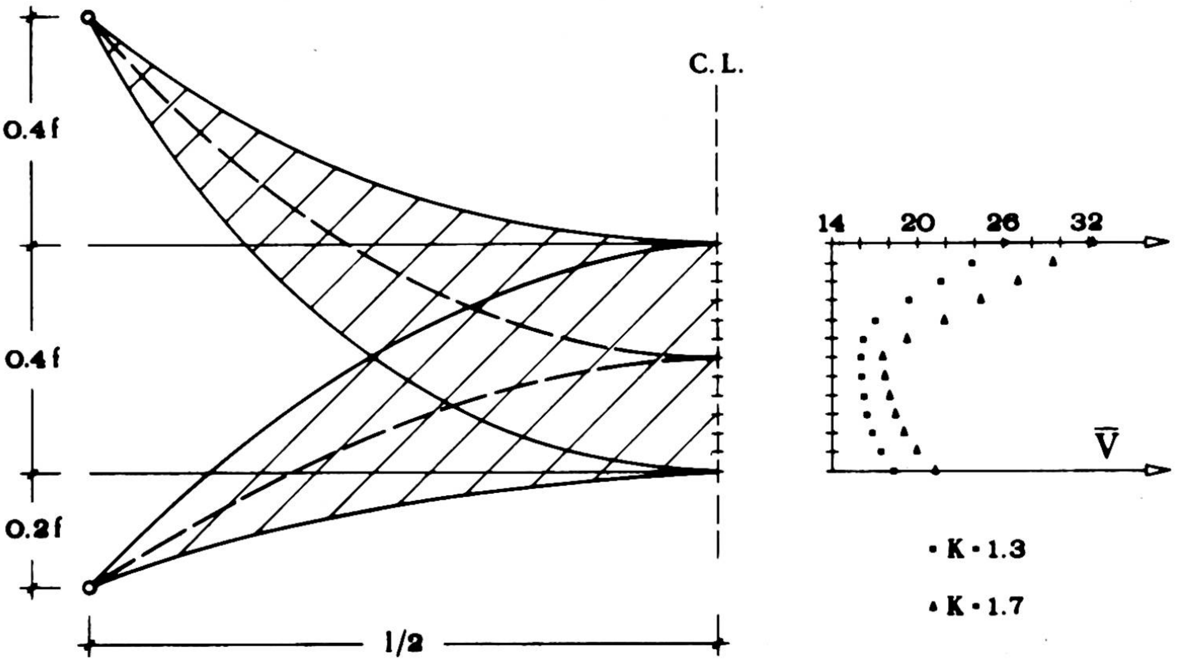

Fig. 2

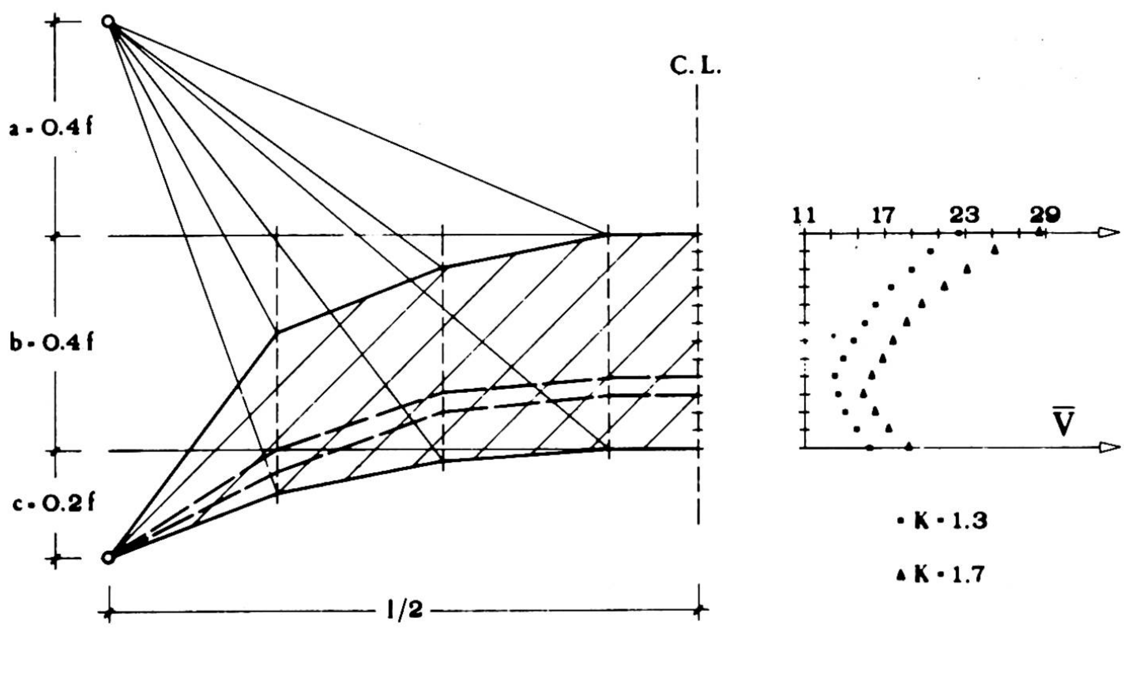

Fig. 3

The TSHS are studied with continuous approach: the relevant computer program (5) allows direct calculation of the areas of main cables and connecting roads once the geometry is fixed.

The PCSS are calculated by a discrete approach, specifically by displacement method: bar cross-sectional areas, when the geometry is fixed, are calculated by successive iterations (6) with a somewhat rapid convergence.

As an exemplification some results are graphically reported in Fig. 2 and in Fig. 3.

In either cases $f/l = 0.10$, $\bar{p} = |p_{max}| / |p_{min}| = 5.00$, $K = 1.3 \div 1.7$ (K is a number which quantifies the initial prestress) and $\bar{V} = (V\sigma_a)/(|p_{min}| l^2)$, where V is the volume of structure and σ_a is the allowable normal stress.

As it known, the fully stressed design of redundant structures coincides with the optimum design in peculiar situations only, which do not occur here.

The global optimum is determined by a process of successive iterations similar to Pedersen's (1). For each iteration, the problem of moving the design variables (cross-sectional areas and joint displacements) is worked out as a Linear Programming problem. Moreover, thanks to preliminary researches developed by fully stressed design, which gives solutions quite near to the global optimum, local minima can be avoided.

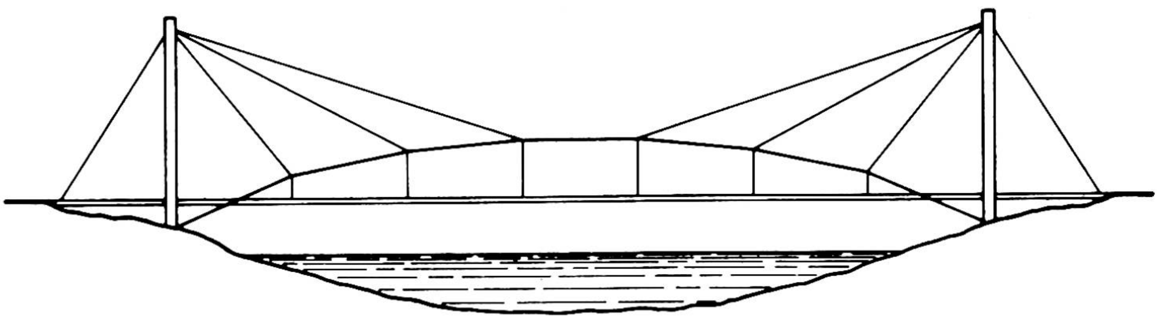

Fig. 4

The optimization process has emphasized the advantages of PCSS over TSHS when volume is the merit function. On the other hand, PCSS have a much better static and dynamic behaviour than TSHS, as it has been largely demonstrated (3), (4).

The extension of the proposed scheme [Fig. 1,b] to long span bridges [see Fig. 4] seems therefore reasonable. To this aim, it is necessary to conform the computational technique already formulated. This will be the purpose of future researches, where the effects of the moving loads and of the wind will be specifically considered.

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SUMMARY

The optimization of prestressed plane cable structures is presented and in particular the optimum design of a new kind of structure, whose good static and dynamic behaviour has already been reported on. Some conclusions and suggestions for future developments are given.

RESUME

On présente l'optimisation de structures planes de câbles prétendues et particulièrement le calcul optimal d'une nouvelle structure, dont une étude a déjà souligné le bon comportement statique et dynamique. Quelques conclusions et idées sont présentées pour de futurs développements.

ZUSAMMENFASSUNG

Es wird über die Optimierung von ebenen vorgespannten Seiltragwerken berichtet und besonders über die optimale Geometrie eines neuartigen Tragwerks; das gute statische und dynamische Verhalten dieses Tragwerks wurde schon früher hervorgehoben. Einige Bemerkungen über die zukünftigen Entwicklungsmöglichkeiten beschliessen diesen Beitrag.

Optimierung der Abmessungen vorgespannter Stahlvollwandträger

Optimization of Dimensions of Prestressed Steel Girders

Optimisation des dimensions de poutres métalliques précontraintes

PAVEL FERJENČIK
Dozent, Dipl.-Ing., CSc.
Fakultät für Bauwesen der Slowakischen Technischen Hochschule
Bratislava, CSSR

Stahlvollwandträger, die durch ein gerades hochfestes Zugband in der Ebene des Untergurtes vorgespannt sind, bilden das meistverwendete Grundelement vorgespannter Stahlkonstruktionen.

Bei der Ableitung rechnerischer Beziehungen für den optimalen Entwurf des durch gerades Zugband vorgespannten Vollwandträgers ist es notwendig, die Beziehungen unter einzelnen Querschnittscharakteristiken des Trägers zu kennen. Mit genügender Genauigkeit ist es möglich, alle Querschnittsgrössen in Abhängigkeit von der Trägerquerschnittsfläche F unter Anwendung folgender drei Parameter auszudrücken: Trägerasymmetrie $A = W_{x1}/W_{x2} = h_2/h_1$, Stegbeiwert $\varphi_{st} = F_{st}/F$, Stegschlankheit $\lambda = h/d$.

Bei der Ableitung der Beziehungen für den optimalen Entwurf des vorgespannten Vollwandträgers wird vom Festigkeitsgesichtspunkt ausgegangen. Die grösste Tragfähigkeit hat der Träger mit der optimalen Asymmetrie, in dem /bei verschiedenen Belastungsstadien/ möglichst viele Stellen ausgenützt sind, bei voll ausgenütztem Zugband - Bild 1.

In der Tabelle I. sind übersichtlich die Grundbeziehungen für die Berechnung der geometrischen Grössen und der Kraftgrössen des asymmetrischen Querschnittes I, beim optimalen Entwurf des vorgespannten Trägers angeführt. Die optimale Zugbandlänge ℓ_v folgt aus den Festigkeitsbedingungen des Trägers an den Stellen des Zugbandanfangs und Zugbandendes.

Als erste sind die Lösungsergebnisse des vorgespannten Trägers mit freier /nicht vorgeschriebener/ Untergurtfläche F_2 angeführt, d. h. die Fläche F_2 ist ausser Festigkeitsbedingung durch nichts beschränkt. Der Entwurf des vorgespannten Vollwandträgers wird beschleunigt, wenn die Bemessungsbeiwerte φ, φ_1, φ_{st}, φ_2, φ_v und ψ_v /Tabelle I./ und das Verhältnis ℓ_v/ℓ berechnet und vereinfacht werden in der Form eines Berechnungshilfsmittels für oft vorkommende Grundbelastungen. Einige Typen aus den Hilfsmitteln für den Entwurf sind auf den Bildern 2a und 2b angeführt. Teil des Hilfsmittels zur Bestimmung der Zugbandlänge ℓ_v ist am Bild 3.

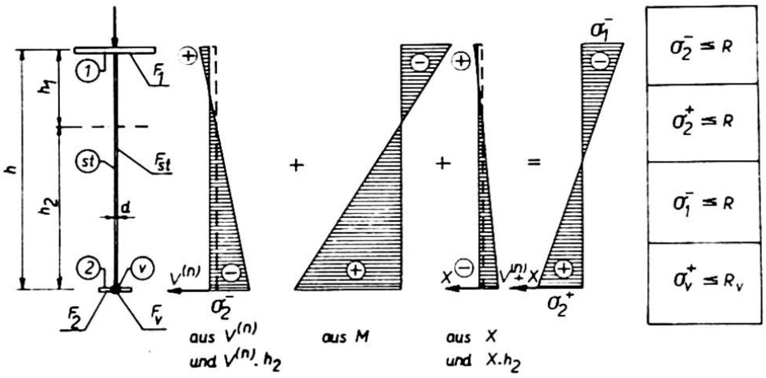

Bild 1. Spannungsbild bei einstufiger Vorspannung. $V^{/n/}$ - Normwert der Vorspannkraft; M - maximales Berechnungsbiegemoment aus der Belastung am Träger ohne Zugband; X - statisch unbestimmte Komponente der Zugbandkraft aus der Berechnungsbelastung; R - Berechnungsbeanspruchung /Index v ist für das Zugbandmaterial/

Tabelle I.

$F = \varphi \sqrt[3]{\frac{M^2}{R^2 \lambda}}$	$\varphi = \sqrt[3]{\frac{1}{C^2}}$;
	$C = \sqrt{\varphi_{st} \frac{3A - \varphi_{st}(A+1)}{3(A+1)}}$;
$F_1 = \varphi_1 F$	$\varphi_1 = \frac{A}{A+1} - \frac{\varphi_{st}}{2}$;
$F_{st} = F \varphi_{st}$	$\varphi_{st} = \frac{2[1-\varphi_2(A+1)]}{A+1}$,
$F_2 = \varphi_2 F$	$\varphi_2 = \frac{1}{A+1} - \frac{\varphi_{st}}{2}$;
$F_v = F \frac{R}{R_v} \varphi_v$	$\varphi_v = D\left(1 - \frac{n_{vd}}{n_{vh}}\right) + \frac{A-1}{A+1}$;
	$D = \frac{6A - \varphi_{st}(A+1)^2}{(A+1)[6A - \varphi_{st}(A+1)]}$;
$V^{(n)} = FR\psi_v$	$\psi_v = \frac{D}{n_{vh}}$

Auf den Bildern 2a und 2b ist der Bereich mit $F_2 < 0{,}04.F$ bei den betreffenden Bemessungsbeiwerten gestrichelt angezeigt; es hat keinen Zweck $F_2 < 0{,}04.F$ zu entwerfen. In diesem Falle wird die Grösse der Trägeruntergurtfläche F_2 durch die Beziehung

$$F_2 = \varphi_2 F = /\frac{1}{A+1} - \frac{\varphi_{st}}{2}/ F$$

gebunden /vorgeschrieben/.

Die Berechnung der notwendigen Bemessungsbeiwerte für diesen zweiten Fall, d. i. vorgespannter Träger mit vorgeschriebener /gebundener/ Untergurtfläche F_2, ist ähnlich wie im ersten Fall, aber es muss auch die oben angeführte Bedingung erfüllt sein.

Wie man aus der Beziehung für φ_{st} /Tabelle I./ sieht, befindet sich in ihr nur eine Unbekannte und zwar Querschnittsasymmetrie A, da φ_2 gewählt wird; es bleibt nur der Beiwert A zu berechnen. Durch Vergleich der Beziehungen für die Berechnung der Kraft X im Zugband, die aus den Festigkeits- und Verformungsbedingungen bestimmt sind, erhalten wir schliesslich die Beziehung

$$\frac{\varrho}{1 + (A-1)\dfrac{6A - 2[1 - \varphi_2(A+1)]}{6A - (A+1)\,2[1 - \varphi_2(A+1)]} - \dfrac{n_{vd}}{n_{vh}}} - \alpha \frac{A\{6A - 4[1 - \varphi_2(A+1)]\}}{\{6A - 2[1-\varphi_2(A+1)](A+1)\}\cdot\left\{(A-1)\dfrac{6A - 2(1-\varphi_2(A+1)]}{6A - 2[1-\varphi_2(A+1)](A+1)} - \dfrac{n_{vd}}{n_{vh}}\right\}} = 0$$

aus der die optimale Asymmetrie A in Abhängigkeit vom den gewählten Parameter E, E_v, R, R_v, φ_2, n_{vd}/n_{vh} und von der Belastungsart bestimmt wird; n_v ist der Koeffizient der Vorspanngenauigkeit /$n_{vd} \leq 1{,}0$ - im allgemeinen 0,9, $n_{vh} \geq 1{,}0$ - im allgemeinen 1,1/, $\varrho = ER_v/E_vR$. Nach Bestimmung des Wertes A werden mit seiner Hilfe

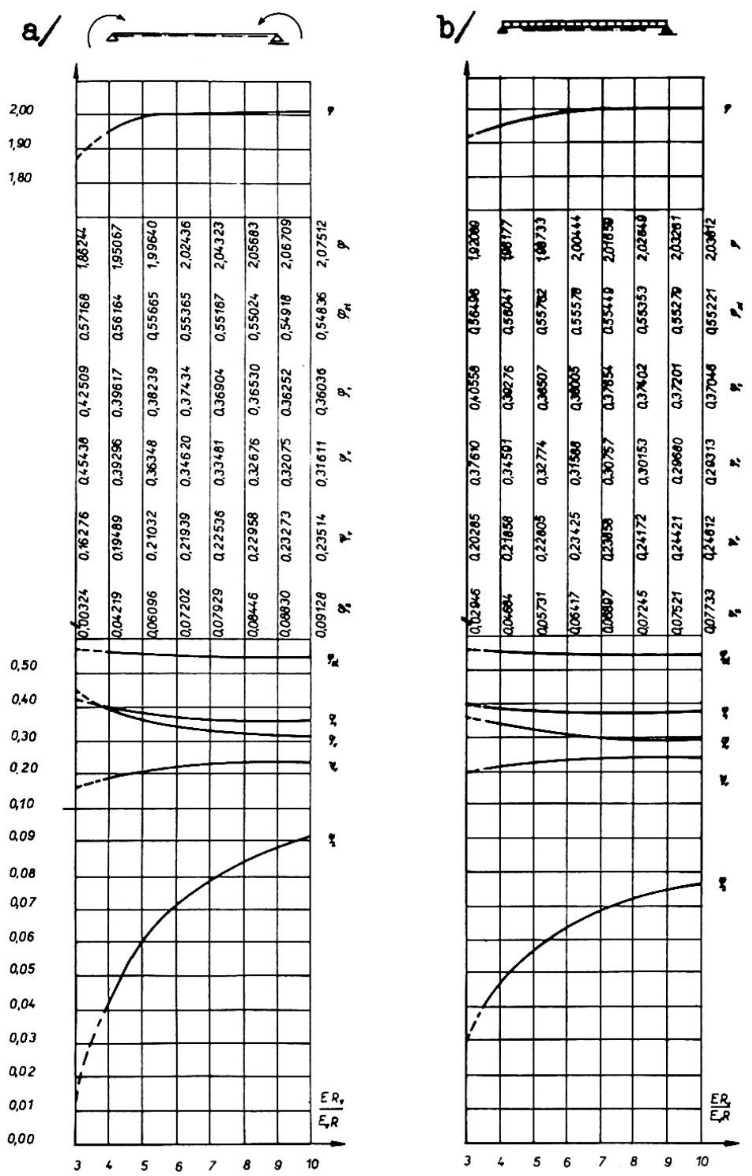

Bild 2. Bemessungsbeiwerte für den Entwurf des vorgespannten Vollwandträgers: a/ $R_1 = R_2 = R$, $n_{vh} = 1,1$ und $n_{vd} = 0,9$; b/ $R_1 = R_2 = R$ und $n_{vh} = n_{vd} = 1,0$

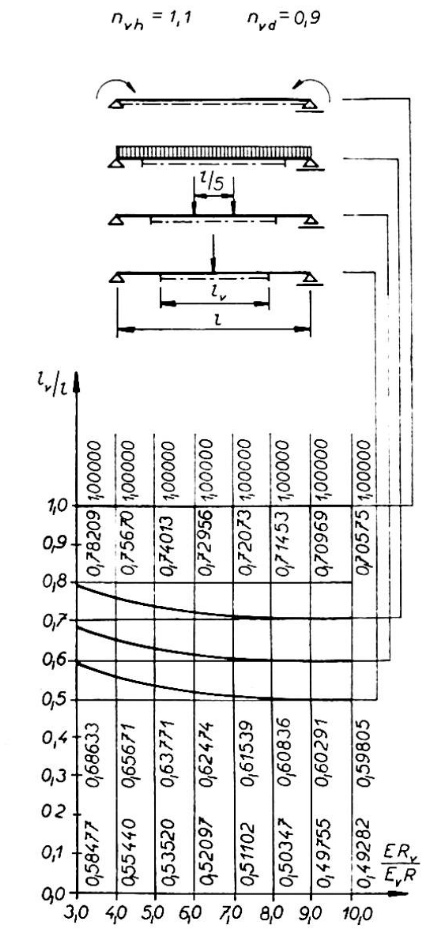

Bild 3. Verhältnislängen ℓ_v/ℓ ; die Zahlenwerte entsprechen den zugehörigen Belastungen in der Reihenfolge wie die Belastung angeführt ist $/R_1 = R_2 = R/$

alle weiteren Beiwerte berechnet, die für den optimalen Entwurf des vorgespannten Vollwandträgers gebraucht werden, und zwar φ, φ_1, φ_{st}, φ_v, ψ_v und ℓ_v/ℓ .

Als Beispiel ist aus der grossen Zahl der Hilfsmittel nur eines angeführt /Bild 4/, mit der grafischen Darstellung und den Zahlenwerten der Bemessungsbeiwerte, die für den optimalen Entwurf des Stahlträgers, der mit geradem Zugband vorgespannt ist, gebraucht werden; der Wert φ_2 ist gewählt. Aus der grossen Zahl tabellarischer Hilfsmittel ist nur eine in der Tabelle II. angeführt.

Der Berechnungsvorgang beim Entwurf ist derselbe wie bei den Trägern mit freier Untergurtfläche; unterschiedlich ist dabei, dass im zweiten Fall φ_2 beim Entwurf gewählt wird.

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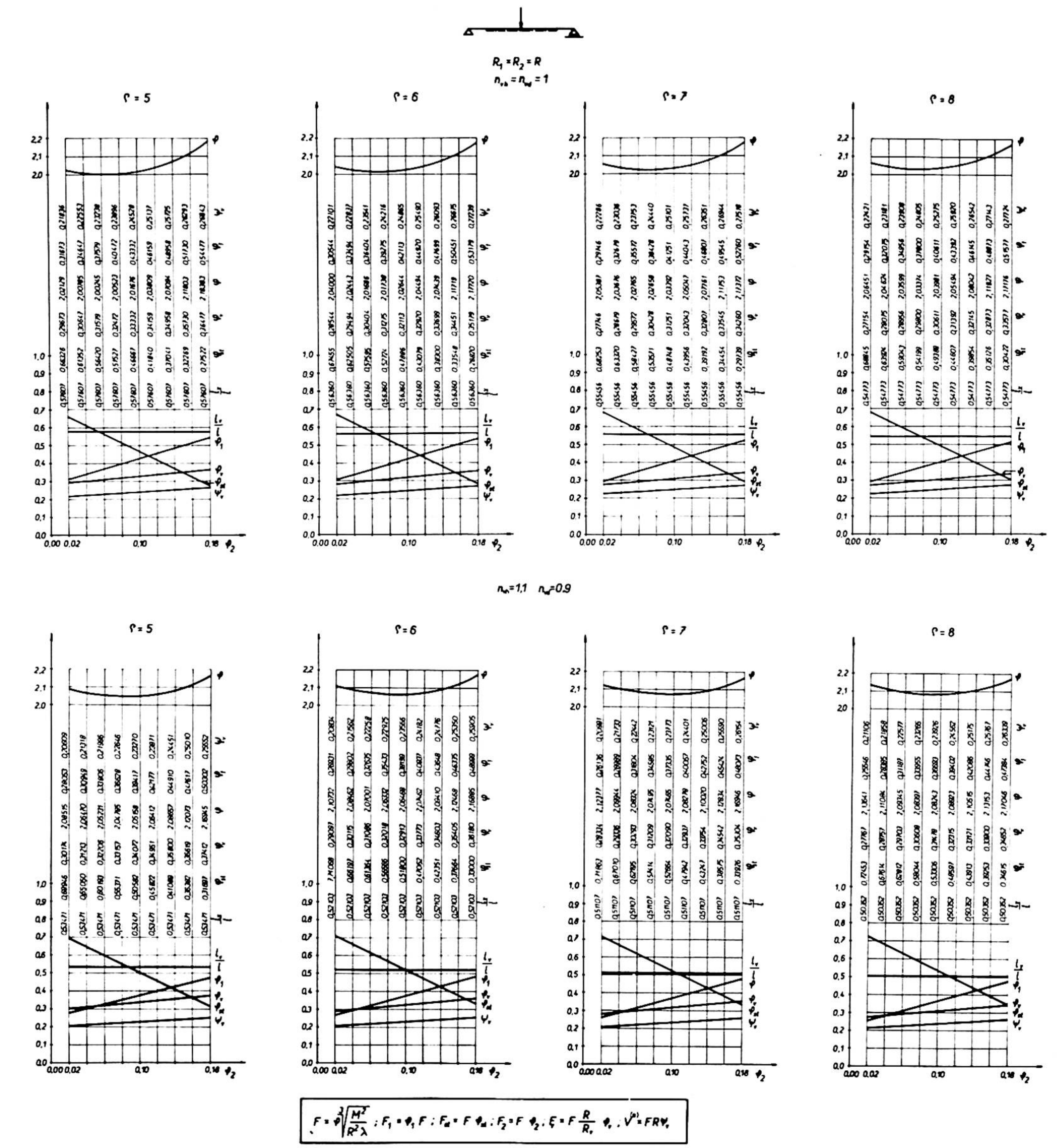

Bild 4. Bemessungsbeiwerte und Beziehungen für den Entwurf des vorgespannten Stahlträgers; Wert φ_2 ist gewählt

III. Meždunarodnaja konferencija po predvariteľno naprjažennym metalličeskim konstrukcijam. Doklady - tom I., Leningrad 1971.
/2/ Ferjenčík, P.: Navrhovanie a realizácie predpätých kovových plnostenných nosníkov. Bratislava, KKDK SvF SVŠT, 1972.
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Tabelle II. Hilfs- und Ergebnisbeiwerte für den Entwurf des vorgespannten Stahlträgers; Wert φ_2 ist gewählt. $R_1 = R_2 = R$ und $n_{vh} = n_{vd} = 1,0$

ϱ	φ_2	A	α	φ_{st}	φ_v	C	D	φ	φ_1	ψ_l	ψ_v
	0,02	2,97355	1,00000	0,46332	0,49667	0,40425	0,16555	1,82907	0,51667	1,00000	0,16555
	0,04	3,07985	1,00000	0,41021	0,50978	0,39591	0,16992	1,85466	0,54978	1,00000	0,16992
	0,06	3,18765	1,00000	0,35759	0,52240	0,38391	0,17413	1,89311	0,58240	1,00000	0,17413
	0,08	3,29714	1,00000	0,30542	0,53457	0,36777	0,17819	1,94808	0,61457	1,00000	0,17819
3	0,10	3,40850	1,00000	0,25366	0,54633	0,34682	0,18211	2,02579	0,64633	1,00000	0,18211
	0,12	3,52193	1,00000	0,20228	0,55771	0,31997	0,18590	2,13758	0,67771	1,00000	0,18590
	0,14	3,63762	1,00000	0,15125	0,56874	0,28544	0,18958	2,30666	0,70874	1,00000	0,18958
	0,16	3,75573	1,00000	0,10054	0,57945	0,23978	0,19315	2,59090	0,73945	1,00000	0,19315
	0,18	3,87646	1,00000	0,05013	0,58986	0,17424	0,19662	3,20545	0,76986	1,00000	0,19662
	0,02	2,27839	1,00000	0,57005	0,38994	0,38125	0,19497	1,90192	0,40994	1,00000	0,19497
	0,04	2,34116	1,00000	0,51859	0,40140	0,38011	0,20070	1,90571	0,44140	1,00000	0,20070
	0,06	2,40367	1,00000	0,46760	0,41239	0,37632	0,20619	1,91848	0,47239	1,00000	0,20619
	0,08	2,46602	1,00000	0,41702	0,42297	0,36969	0,21148	1,94136	0,50297	1,00000	0,21148
4	0,10	2,52830	1,00000	0,36684	0,43315	0,35995	0,21657	1,97622	0,53315	1,00000	0,21657
	0,12	2,59060	1,00000	0,31700	0,44299	0,34673	0,22149	2,02614	0,56299	1,00000	0,22149
	0,14	2,65297	1,00000	0,26749	0,45250	0,32950	0,22625	2,09617	0,59250	1,00000	0,22625
	0,16	2,71548	1,00000	0,21828	0,46171	0,30746	0,23085	2,19515	0,62171	1,00000	0,23085
	0,18	2,77820	1,00000	0,16935	0,47064	0,27937	0,23532	2,33997	0,65064	1,00000	0,23532
	0,02	2,05107	1,00000	0,61550	0,34449	0,36644	0,20869	1,95282	0,36449	1,00000	0,20669
	0,04	2,10151	1,00000	0,56484	0,35515	0,36773	0,21309	1,94824	0,39515	1,00000	0,21309
	0,06	2,15142	1,00000	0,51463	0,36536	0,36668	0,21921	1,95197	0,42536	1,00000	0,21921
	0,08	2,20089	1,00000	0,46482	0,37517	0,36314	0,22510	1,96461	0,45517	1,00000	0,22510
5	0,10	2,25000	1,00000	0,41538	0,38461	0,36695	0,23076	1,98727	0,48461	1,00000	0,23076
	0,12	2,29880	1,00000	0,36627	0,39372	0,34785	0,23623	2,02178	0,51372	1,00000	0,23623
	0,14	2,34737	1,00000	0,31748	0,40251	0,33549	0,24150	2,07112	0,54251	1,00000	0,24150
	0,16	2,39574	1,00000	0,26897	0,41102	0,31939	0,24861	2,14015	0,57102	1,00000	0,24661
	0,18	2,44396	1,00000	0,22072	0,41927	0,29883	0,25156	2,23725	0,59927	1,00000	0,25156
	0,02	1,93850	1,00000	0,64061	0,31938	0,35709	0,21292	1,98675	0,33938	1,00000	0,21292
	0,04	1,98316	1,00000	0,59042	0,32957	0,35958	0,21971	1,97755	0,36957	1,00000	0,21971
	0,06	2,02719	1,00000	0,54067	0,33932	0,35988	0,22621	1,97647	0,39932	1,00000	0,22621
	0,08	2,07069	1,00000	0,49131	0,34868	0,35787	0,23245	1,98385	0,42868	1,00000	0,23245
6	0,10	2,11374	1,00000	0,44231	0,35768	0,35341	0,23845	2,00050	0,45768	1,00000	0,23845
	0,12	2,15638	1,00000	0,39363	0,36636	0,34630	0,24424	2,02779	0,48636	1,00000	0,24424
	0,14	2,19868	1,00000	0,34525	0,37474	0,33626	0,24982	2,06796	0,51474	1,00000	0,24982
	0,16	2,24067	1,00000	0,29715	0,38284	0,32291	0,25523	2,12459	0,54284	1,00000	0,25532
	0,18	2,28241	1,00000	0,24930	0,39069	0,30569	0,26046	2,20363	0,57069	1,00000	0,26046
	0,02	1,87138	1,00000	0,65652	0,30347	0,35075	0,21876	2,01061	0,32347	1,00000	0,21676
	0,04	1,91266	1,00000	0,60665	0,31334	0,35396	0,22381	1,99844	0,35334	1,00000	0,22381
	0,06	1,95330	1,00000	0,55720	0,32279	0,35506	0,23056	1,99432	0,38279	1,00000	0,23056
	0,08	1,99337	1,00000	0,50814	0,33185	0,35395	0,23704	1,99846	0,41185	1,00000	0,23704
7	0,10	2,03293	1,00000	0,45942	0,34057	0,35052	0,24326	2,01149	0,44057	1,00000	0,24326
	0,12	2,07206	1,00000	0,41102	0,34897	0,34458	0,24926	2,03455	0,46897	1,00000	0,24926
	0,14	2,11079	1,00000	0,36292	0,35707	0,33589	0,25505	2,06949	0,49707	1,00000	0,25505
	0,16	2,14917	1,00000	0,31508	0,36491	0,32412	0,26065	2,11929	0,52491	1,00000	0,26065
	0,18	2,18724	1,00000	0,26750	0,37249	0,30881	0,26606	2,18877	0,55249	1,00000	0,26606
	0,02	1,82682	1,00000	0,66750	0,29249	0,34620	0,21936	2,02820	0,31249	1,00000	0,21936
	0,04	1,86591	1,00000	0,61785	0,30214	0,34988	0,22660	2,01397	0,34214	1,00000	0,22660
	0,06	1,90433	1,00000	0,56862	0,31137	0,35150	0,23352	2,00775	0,37137	1,00000	0,23352
	0,08	1,94216	1,00000	0,51977	0,32022	0,35100	0,24017	2,00968	0,40022	1,00000	0,24017
8	0,10	1,97948	1,00000	0,47125	0,32874	0,34824	0,24855	2,02028	0,42874	1,00000	0,24655
	0,12	2,01632	1,00000	0,42305	0,33694	0,34306	0,25270	2,04053	0,45694	1,00000	0,25270
	0,14	2,05274	1,00000	0,37514	0,34485	0,33526	0,25863	2,07208	0,48485	1,00000	0,25863
	0,16	2,08880	1,00000	0,32750	0,35249	0,32452	0,26437	2,11754	0,51249	1,00000	0,26437
	0,18	2,12451	1,00000	0,28009	0,35990	0,31044	0,26992	2,18110	0,53990	1,00000	0,26992
	0,02	1,79509	1,00000	0,67553	0,28446	0,34277	0,22124	2 04169	0,30446	1,00000	0,22124
	0,04	1,83262	1,00000	0,62605	0,29394	0,34678	0,22862	2 02592	0,33394	1,00000	0,22862
	0,06	1,86950	1,00000	0,57698	0,30301	0,34879	0,23567	2 01816	0,36301	1,00000	0,23567
	0,08	1,90576	1,00000	0,52828	0,31171	0,34870	0,24244	2 01849	0,39171	1,00000	0,24244
9	0,10	1,94149	1,00000	0,47992	0,32007	0,34642	0,24894	2 02733	0,42007	1,00000	0,24894
	0,12	1,97674	1,00000	0,43187	0,32812	0,34179	0,25520	2 04559	0,44812	1,00000	0,25520
	0,14	2,01156	1,00000	0,38410	0,33589	0,33461	0,26124	2 07476	0,47589	1,00000	0,26124
	0,16	2,04599	1,00000	0,33660	0,34339	0,32460	0,26708	2 11720	0,50339	1,00000	0,26708
	0,18	2,08006	1,00000	0,28933	0,35066	0,31138	0,27273	2 17672	0,53066	1,00000	0,27273
	0,02	1,77135	1,00000	0,68166	0,27833	0,34011	0,22266	2 05234	0,29833	1,00000	0,22266
	0,04	1,80773	1,00000	0,63231	0,28768	0,34436	0,23014	2 03540	0,32768	1,00000	0,23014
	0,06	1,84345	1,00000	0,58337	0,29662	0,34665	0,23730	2 02646	0,35662	1,00000	0,23730
	0,08	1,87855	1,00000	0,53479	0,30520	0,34688	0,24416	2 02556	0,38520	1,00000	0,24416
10	0,10	1,91311	1,00000	0,48655	0,31344	0,34495	0,25075	2 03308	0,41344	1,00000	0,25075
	0,12	1,94718	1,00000	0,43861	0,32138	0,34073	0,25710	2 04984	0,44138	1,00000	0,25710
	0,14	1,98082	1,00000	0,39095	0,32904	0,33401	0,26323	2 07723	0,46904	1,00000	0,26323
	0,16	2.01405	1,00000	0,34355	0,33644	0,32454	0,26915	2 11746	0,49644	1,00000	0,26915
	0,18	2,04692	1,00000	0,29640	0,34359	0,31195	0,27487	2 17405	0,52359	1,00000	0,27487

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ZUSAMMENFASSUNG

Der Aufsatz befasst sich mit der Optimierung der Abmessungen vorgespannter Stahlvollwandträger. Angeführt werden Berechnungshilfsmittel für den Entwurf, wobei die Trägeruntergurtfläche F_2 im ersten Fall frei und im zweiten Fall gebunden ist.

SUMMARY

This article deals with the optimization of dimensions of prestressed steel girders. Calculation means are given for the design, whereas in the first case the area of the lower flange F_2 is assumed to be free and in the second case it is assumed to be given.

RESUME

L'article traite de l'optimisation des dimensions des poutres métalliques précontraintes. Des tables auxiliaires de calcul sont présentées, avec la section de la semelle inférieure F_2 libre en premier cas et liée en deuxième cas.

Optimization of the Prestressing in the Cables of a Cable-Stayed Bridge

Optimisation de la précontrainte dans les câbles d'un pont à haubans

Optimierung der Vorspannung in den Kabeln einer Schrägseilbrücke

DIETHELM FEDER
Dr.-Ing.
M.A.N.
Gustavsburg, GFR

1. Reasons for prestressing

It is well known that the cables of a cable-stayed bridge have to be prestressed in order to utilize the high strength of the cables and to reduce the bending moments in the main girder.

The degree of prestressing required depends on the structural system of the bridge, the relative stiffness of the cables and the girder, and the ratio of live load to dead load. Finding the most economic relation between the longitudinal stiffness of the cables and the bending stiffness of the main girder is a complicated optimization problem in itself.

Here, only a short description is given of how prestress in the cables was optimized for a given structural system. The aim was to get a uniform distribution of moments in the main girder without overloading the cables.

2. The Deggenau cable-stayed bridge

The bridge in question carries a motorway across the Danube in Eastern Bavaria and has two spans of unequal length, i.e. 290 m and 145 m (Fig. 1). It has one central tower and the large opening is back-stayed by three cables to the small opening and mainly to the eastern abutment. The main girder has a two cell box-section of 14 x 4.5 m with cantilevering deck supported by struts.

The cables are composed of spiral strands arranged in three layers in a rectangular pattern (Fig. 2), for more details about this bridge see [1].

3. Optimality conditions

In static analysis the bridge was treated as a space frame in linear elastic theory with the aid of a standard computer program.

The loading case "end displacement" of a certain cable then delivers a complete set of influence coefficients reflecting the effect of prestressing. Thus, from the loading cases "end displacement" of all the cables one can easily obtain the coefficients for the value of any internal force or moment due to prestressing:

$$S_i = \sum_k a_{ik} X_k$$

with S_i = internal force or moment
a_{ik} = influence coefficient
X_k = prestressing (end displacement of cable k)

The optimality criteria to be considered were the following (see also Fig. 3):

a) The tensions in the cables were not to exceed the maximum allowable values and should not become negative.

b) The moments in the main girder were not to exceed the strength of the girder required for cantilever erection.

c) The moments at the tower base were to be kept to a minimum.

With 6 different cables, 6 positive and 5 negative limit moments for the main girder, and two limit moments for the tower base this gives a set of 25 inequality conditions for the 6 unknown cable end displacements.

4. Practical solution

Assuming that the system would behave inherently reasonable, the number of inequalities was reduced to 19 by dropping the condition that cable tensions should not become negative.

A further reduction was possible due to the fact that positive bending moments in the girder between cable points were not critical.

It was then tried to solve the remaining set of 13 inequalities by means of a standard computer program for linear programming. As it turned out this program failed to give a reasonable solution because, being geared to economic problems only, it could not handle negative values on the right hand side. Merely changing signs all through the equations concerned did not help.

Since there was neither time nor a specialist available to overcome these difficulties, the number of variables and restrictions was further reduced by engineering judgement. The remaining set of 10 inequalities with 5 unknowns was handled as overdetermined system of linear equations by a computer program producing a least squares solution. This approach gave surprisingly satisfactory results for all the values that were to be optimized.

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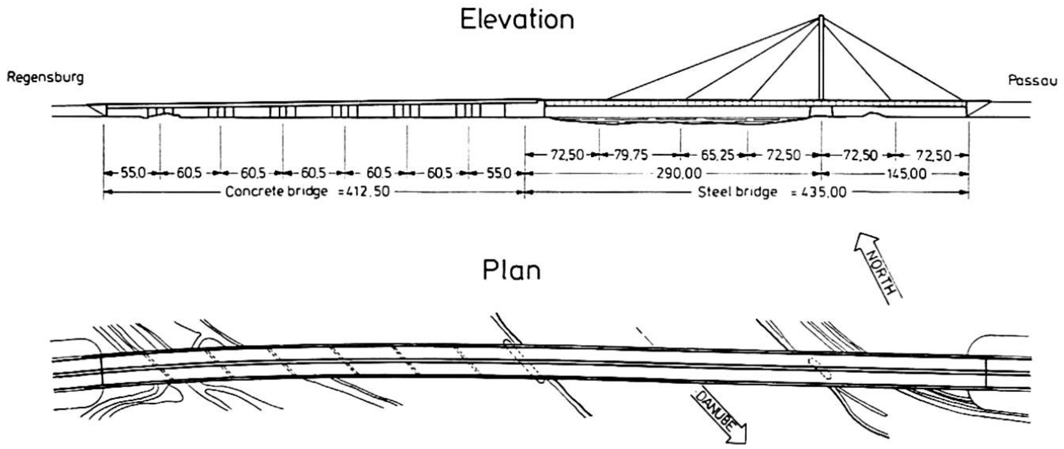

Fig. 1: Deggenau cable-stayed bridge

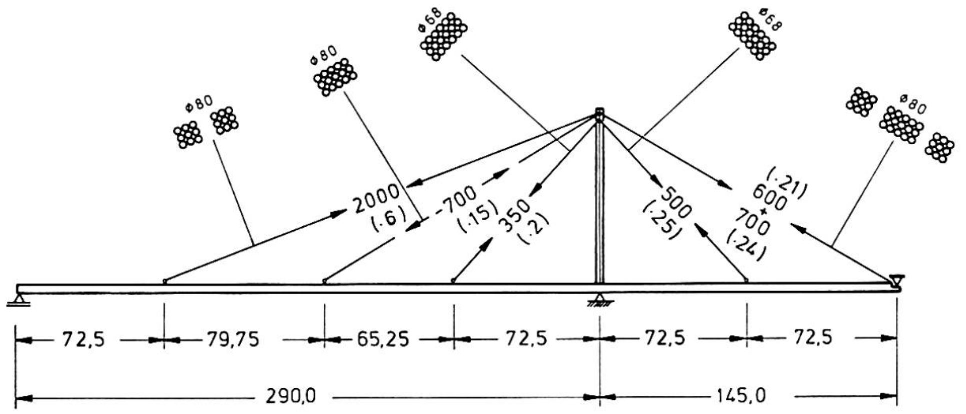

Fig. 2: Cable sections and prestressing:
prestress [kN], (cable end displacements [m])

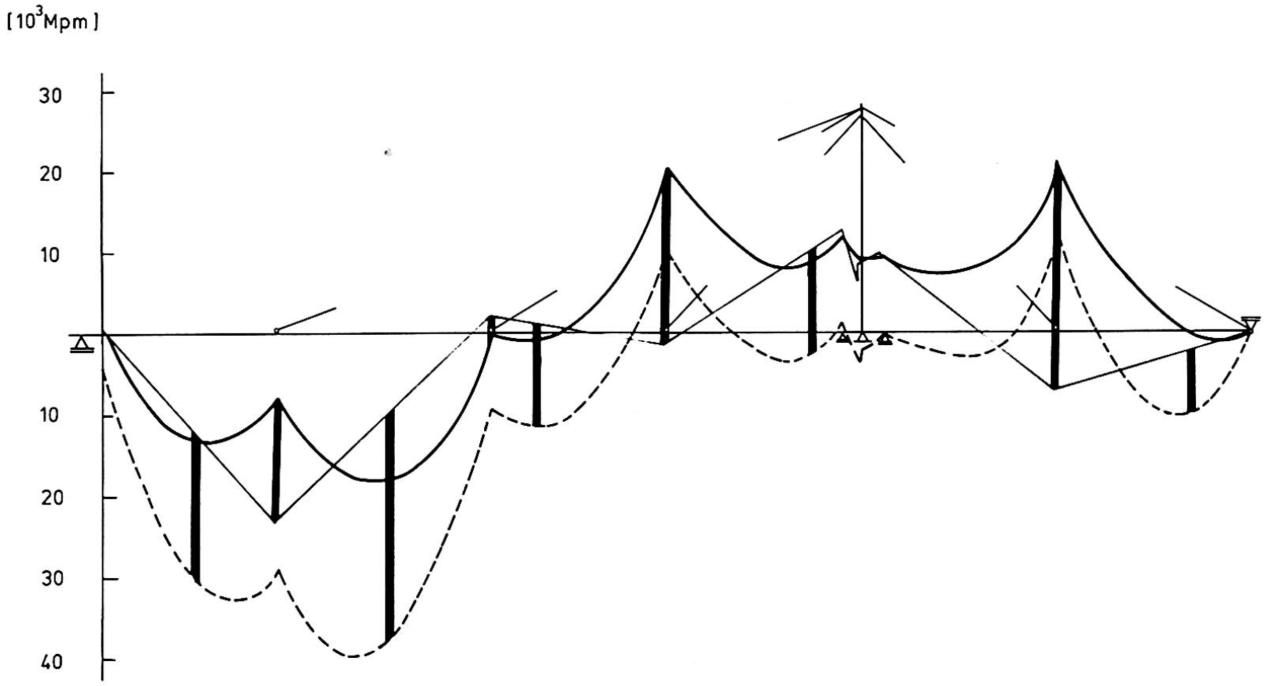

Fig. 3: Optimality criteria and moment distribution
in main girder

SUMMARY

The paper deals with an optimization that has been used in the design of a cable-stayed bridge. The optimality criteria are briefly discussed, difficulties encountered are described, and the engineering approach applied for the solution is outlined.

RESUME

L'article traite de l'optimisation retenue pour le calcul d'un pont à haubans. Les critères d'optimisation sont discutés, les difficultées rencontrées sont décrites, et la méthode d'ingénieur employée est présentée.

ZUSAMMENFASSUNG

Der Beitrag befasst sich mit einer Optimierung, die bei der Berechnung einer Schrägseilbrücke benutzt wurde. Es werden kurz die Optimalitätskriterien diskutiert, aufgetretene Schwierigkeiten werden beschrieben, und das für die Lösung angewendete ingenieurmässige Vorgehen wird umrissen.

Optimierung elastischer, ebener Rahmentragwerke

Optimization of Elastic Plane Frames

Optimisation des cadres plans élastiques

B. CICHOCKI
Dipl.-Ing. Dr. techn.
Institut für Stahlbeton- und Massivbau, TU Graz
Graz, Oesterreich

1. Einführung

Die Optimierung der Tragwerke wurde im Einführungsbericht /1/ in 4 Kategorien eingeteilt. Der vorliegende Bericht beschreibt ein Verfahren, das der 3. Kategorie angehört und gleichzeitig die Aufgabestellung der 4. Kategorie weitgehend erfüllt. Ebene Rahmentragwerke mit linear-elastischem Last-Verformungszusammenhang sollen bei gegebenen Geometrie-, Lagerbedingungen und Belastungszuständen so dimensioniert werden, daß ein Optimalitätskriterium für das gesamte Tragwerk erfüllt und die einzelnen Elemente des diskretisierten Tragwerkes für sich optimal gestaltet sind.

2. Zielfunktion

Mit den Querschnittsflächen A_i als Variablen ist die Zielfunktion für minimales Gewicht oder minimale Kosten zu formulieren. Voraussetzung ist eine stetige und stetig differenzierbare Funktion.

$$f_{(A_i)} \Rightarrow \min \qquad i = 1.....n \qquad (1)$$

3. Optimaler Einzelquerschnitt

Der optimale Einzelquerschnitt ist durch die Erzielung des erforderlichen Biegewiderstandes bei minimaler Querschnittsfläche gekennzeichnet. Die Zusammenhänge zwischen Trägheitsmoment I, Widerstandsmoment W, Bauhöhe h und optimaler Querschnittsfläche sind nach Festlegung der Querschnittsform durch Differentialrechnung zu ermitteln. Unter Verwendung üblicher Näherungen ergibt sich beispielsweise am doppelt-symmetrischen I-Profil, dessen Stegverhältnis $\lambda=h/t$ den Erfordernissen der Stegbeulung entsprechend vorgewählt wird

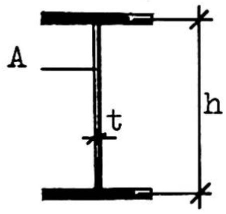

Abb. 1. Bezeichnungen am I-Querschnitt

$$\left.\begin{aligned} I &= \frac{\lambda}{12} \cdot A^2 \\ W &= \left(\sqrt[3]{\frac{16}{3\lambda}} + \sqrt[3]{\frac{2}{3\lambda}} \right)^{-3/2} \cdot A^{3/2} \\ h &= \sqrt{\frac{\lambda}{2}} \cdot A^{1/2} \end{aligned}\right\} \quad (2)$$

Bei einfach-symmetrischen Profilen sind die Gurtflächen als getrennte Variablen einzuführen.

4. Restriktionssystem

Die nicht-linearen Restriktionen für die Variablen A_i sind durch die Forderung gegeben, daß äußere Schnittgrößen an keiner Stelle des Tragwerkes die inneren Widerstände übersteigen dürfen.

$$\left.\begin{aligned} |M^{ä}| &\leq |M^{i}| \\ |N^{ä}| &\leq |N^{i}| \end{aligned}\right\} \quad (3)$$

Der Einfluß der Querkräfte bleibt unberücksichtigt.
Als Vorsorge gegen Stabilitätsversagen des Systems werden die äußeren Schnittgrößen unter Berücksichtigung von Vorverformungen und ungewollten Lastausmitten im Traglastzustand nach Theorie II. Ordnung errechnet.

Bei Verwendung der verallgemeinerten Deformationsmethode wird die globale Steifigkeitsmatrix $[S]$ durch Summation aus den ins globale System rotierten Stabsteifigkeitsmatrizen $[L]$ gewonnen.

$$[L] = [L^{I}] + [L^{II}] \quad (4)$$

Die Matrix I. Ordnung enthält die Variablen A_i, die Matrix II. Ordnung die unbekannten Stablängskräfte N. In der Iterationsrechnung dürfen – mit Rücksicht auf die geringe Änderung der Normalkräfte bei Variation der Steifigkeiten – die Längskräfte des vorhergehenden Iterationsschrittes als Konstantwerte verwendet werden.

Die Knotenverformungen $\{v\}$ unter der Belastung $\{P\}$ sind durch

$$\{v\} = [S]^{-1} \cdot \{P\} \quad (5)$$

gegeben. Die Änderung einer Verformung v_k kann im Bereich kleiner Steifigkeitsänderungen genügend genau durch

$$\Delta v_k = \text{grad}\ v_k\ \{\Delta A\} \quad (6)$$

angenähert werden, wobei grad v_k aus (5) numerisch ermittelt wird. Hierbei kann vorteilhaft von der Näherung

$$[S + \Delta S_i]^{-1} \doteq ([I] - [S]^{-1}\ [\Delta S_i]) \cdot [S]^{-1} \quad (7)$$

Gebrauch gemacht werden /2/, in der $[I]$ die Einheitsmatrix und $[\Delta S_i]$ die Änderung der globalen Steifigkeitsmatrix durch Änderung des Querschnittes A_i bedeuten. Die Matrix $[S]$ muß in jedem Iterationsschritt nur einmal invertiert werden.

Sind $\{w\}$ die aus $\{v\}$ durch Zuordnung und Rotation ins lokale Koordinatensystem gewonnen Stabendverformungen, so gilt für die Schnittgrößen

$$\{M^{ä}\} = [L]\ \{w\} \quad (8)$$

und bei Variation der Flächen A_i

$$\{M^{ä}\} = [L + \Delta L]\ \{w + \Delta w\} \leq \{M^{i}\} \quad (9)$$

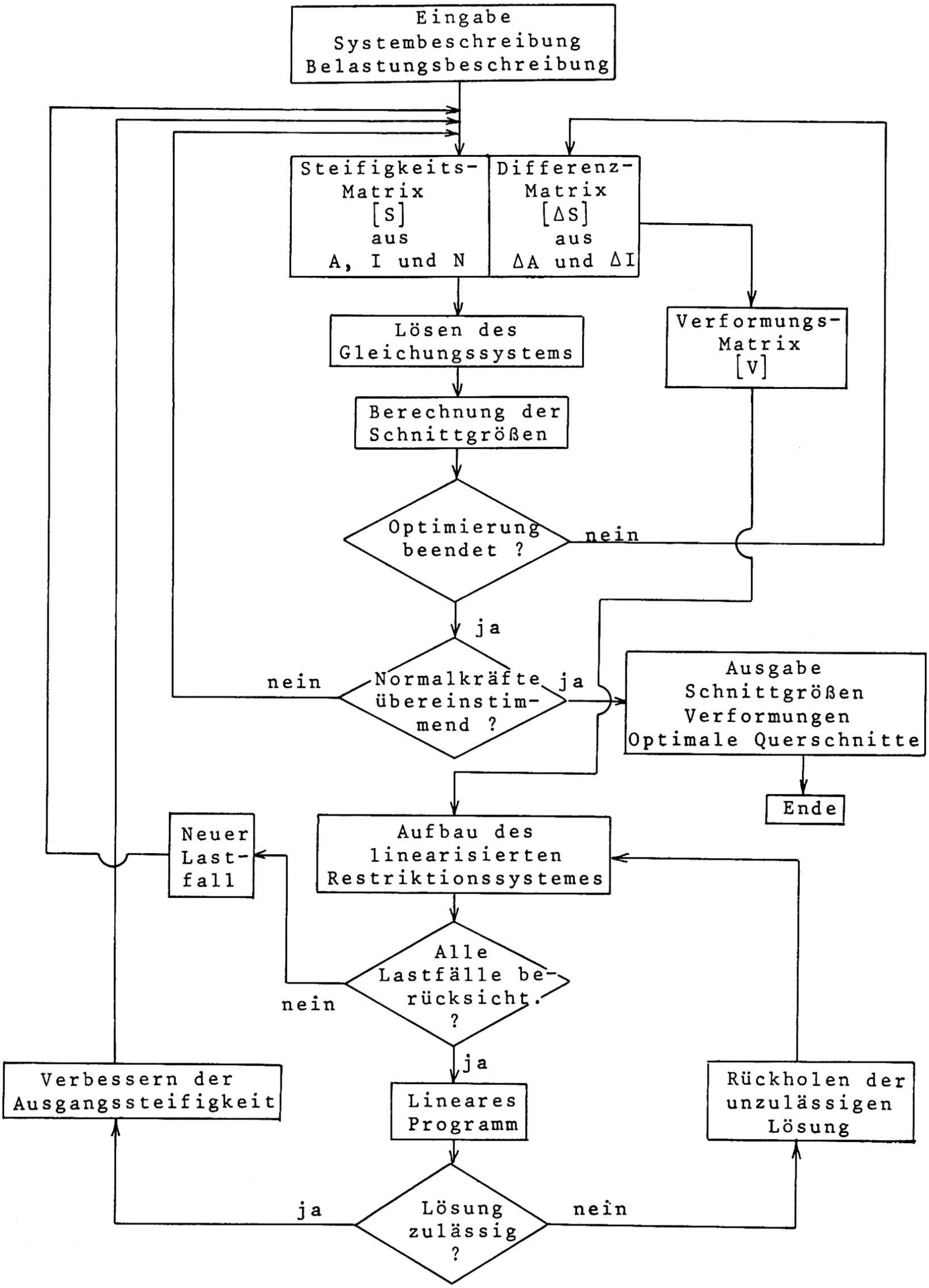

Abb. 2. Schematisches Flußdiagramm

Gleichung (9) an allen Stabenden und für alle Lastfälle formuliert liefert das Restriktionssystem für die Variablen A_i.

5. Mathematische Optimierung

Die Lösung der nicht-linearen Optimierungsaufgabe erfolgt mittels der "Methode der approximierenden Optimierung" /3/, einer Variante der Sequentiellen Linearen Programmierung. Zielfunktion und Restriktionen werden in einem zulässigen Startpunkt $\{A_o\}$ linearisiert und mittels Simplex-Algorithmus die Lösung des Linearen Programmes bestimmt. Der Lösungspunkt $\{\bar{A}\}$, der wegen des Linearisierungsfehlers außerhalb des zulässigen Raumes liegt, wird mit der Vektoraddition

$$\{A^*\} = \frac{1}{2^t} \cdot \{\bar{A}\} + \frac{2^t-1}{2^t} \cdot \{A_o\} \tag{10}$$

rückgeführt. Die Potenz t wird beginnend mit t=0 solange um 1 erhöht, bis $\{A^*\}$ in den zulässigen Grundbereich zu liegen kommt.

Mit den verbesserten Werten für die Querschnittsflächen werden verbesserte Verformungsgradienten errechnet und ein neuerlicher Iterationszyklus durchlaufen.

Der Berechnungsablauf ist an einem schematischen Flußdiagramm erläutert. (Abb.2)

Das Konvergenzverhalten wurde an verschiedenen kleineren Beispielen untersucht. Die Verbesserung des Wertes der Zielfunktion nach 5 Iterationsschritten lag zumeist unter 1 %.

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ZUSAMMENFASSUNG

Der Bericht behandelt die Optimierung elastischer, ebener Rahmentragwerke. Die Lösung des nicht-linearen Programmes erfolgt mit der "Methode der approximierenden Optimierung", einer Variante der sequentiellen linearen Programmierung.

SUMMARY

The paper examines the optimization of elastic plane frames. The solution of the non-linear programming is obtained by the method "Methode der approximierenden Optimierung", a kind of sequential linear programming.

RESUME

L'article traite de l'optimisation des cadres plans élastiques. La solution pour la programmation non-linéaire est obtenue à l'aide de la "Methode der approximierenden Optimierung", une variante de la programmation linéaire séquentielle.

An Optimality Criterion Method for Composite Bridge Deck Design

Une méthode basée sur le critère de l'optimalité pour le calcul du tablier composite d'un pont

Eine Optimierungsmethode für die Berechnung von Verbunddecken im Brückenbau

T.A.I. AKEJU
Lecturer, Department of Civil Engineering
University of Lagos
Lagos, Nigeria

Gellatly and Dupree (1) have discussed some of the important limitations which arise from application of mathematical programming techniques to structural problems. The need amongst others for the evaluation of derivatives of objective functions and constraints in most mathematical programming methods often leads to the expenditure of large computer time for the solution of realistic size structures, whose accurate description depend on large numbers of design variables. Whilst recognising that some new developments are directed towards ameliorating some of these problems, there is yet no indications that all of them have been solved.

On the other hand the broad group of methods, classed as optimality criterion approach, employ completely different techniques from those of mathematical programming and are thereby free from the weaknesses of the latter, although they have their peculiar shortcomings. In this discussion an example of the application of the method to the optimum cost design of composite bridge deck is presented.

In developing an optimality criterion approach for this problem, it is considered necessary to introduce the following simplifying assumptions:

1. The width of the bridge deck is fixed and the configurations of the deck are as shown in Figure 1.

2. For the steel girders, only universal beam sections with tabulated section properties from manufacturers are used.

3. The design of shear connectors is not considered although it is assumed that adequate shear connectors are provided between the slab and the beam to make it possible to use transformed section theory.

4. Shored construction is assumed to reduce the number of load cases to be considered.

The characteristics of the objective function are studied (2) through the aid of a computer program written to design the deck to the requirements of the British Code for Composite Construction (CP 117 1967 Part 2) and for Design of Steel Girder Bridges (BS 153 1966 Part 3 & 4).

Amongst the characteristics studied are:

1. The variation of deck cost with depth of slab for different cost of concrete, μ for a given girder. A typical example Figure 2 shows that the cost of concrete has very little influence on the cost of the deck.

2. The variation of deck cost with depth of slab for different cost of steel, λ. Figure 3 shows that the cost of girder has a predominant effect on the cost of the deck.

3. The variation of the deck cost with depth of slab for girders of the same serial size but of different weights. Figure 4 for four 914mm x 305mm girders shows that if a feasible design region exists with the possibility of choice of girders, the optimum design is obtained for the girder with the lowest value of weight per unit length.

These points dictate the mode of procedure for the optimization process. It is considered reasonable from the relatively minor contribution of the slab cost to choose, as a first approach, a slab depth based only on satisfaction of strength and deflection constraints. The table of girder section properties is ordered with respect to the weight per unit length to create a logical direction of search. A direct search method (3) with variable travel steps, controlled by a sensitivity device, is used to determine the optimum girder.

One of the main points to be emphasised here is the fact that the assumptions stated above, though restrictive, are necessary for application of the concept of optimality criteria. Secondly the knowledge of the characteristics of the problem is the principal factor responsible for the easy development of the simple direct search scheme. Lastly the problem falls into the first class of hierarchy discussed by Gellatly and Dupree (1) and further illustrates the point that optimality criterion methods are very efficient at dealing with only one or two of the hierarchy at a time.

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To be published.

APPENDIX 1 - NOTATION

C_c Cost of concrete per cubic metre.

C_r Cost of reinforcement per cubic metre.

C_g Cost of girder per 100 kilogram.

$\lambda = C_g/C_c$

$\mu = C_r/C_c$

SUMMARY

An example of the application of an optimality criterion method to the design of composite bridge deck for minimum cost is presented. Simplifying assumptions which facilitated the application of the method are discussed. It is argued that a study of the characteristics of the problem contributed immensely to the development of a simple direct search scheme for the optimal solution.

RESUME

On donne un exemple de l'application d'une méthode basée sur le critère de l'optimalité pour le calcul du tablier composite d'un pont,pour des frais minima. Puis des hypothèses simplificatrices qui ont facilité l'application de cette méthode sont considérées. Il est montré qu'une étude des caractéristiques du problème a contribué énormément au développement de combinaisons simples et directes pour la recherche d'une solution optimale.

ZUSAMMENFASSUNG

Ein Beispiel für die Anwendung einer Optimierungsmethode zur Berechnung von Verbunddecken mit minimalen Kosten im Brückenbau wird angegeben. Vereinfachende Voraussetzungen werden diskutiert, welche die Anwendung der Methode erleichtern. Das Studium der Problemscharakteristiken hat sehr viel dazu beigetragen, dass ein einfacher und direkter Lösungsweg gefunden werden konnte.

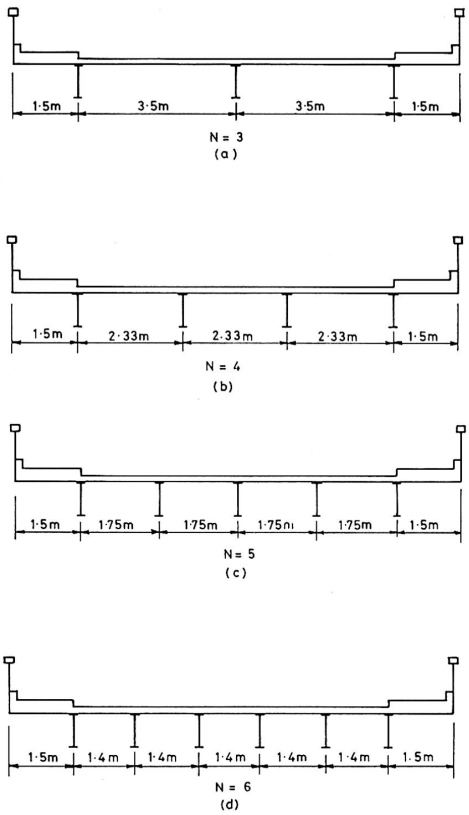

FIGURE 1

BRIDGE DECK CONFIGURATION

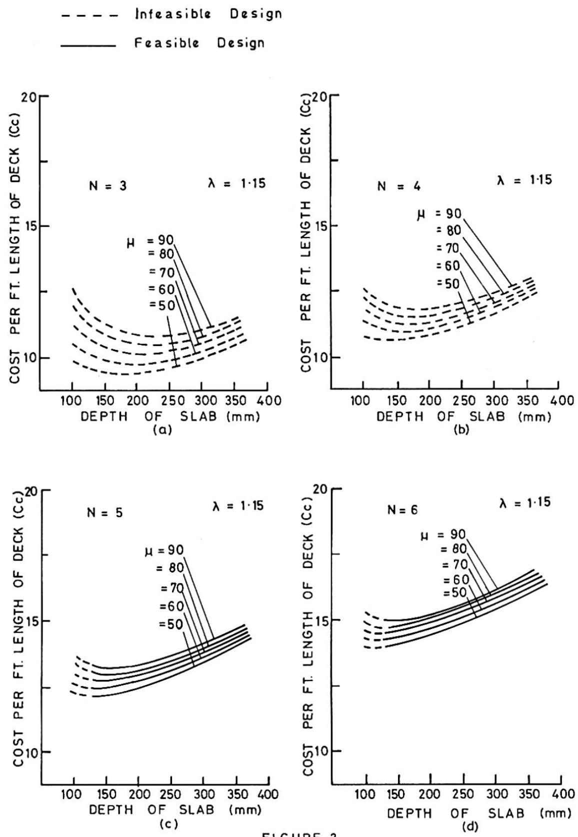

FIGURE 2

VARIATION OF DECK COST WITH DEPTH OF SLAB FOR DIFFERENT μ (762mm x 267mm x 194kg UB)

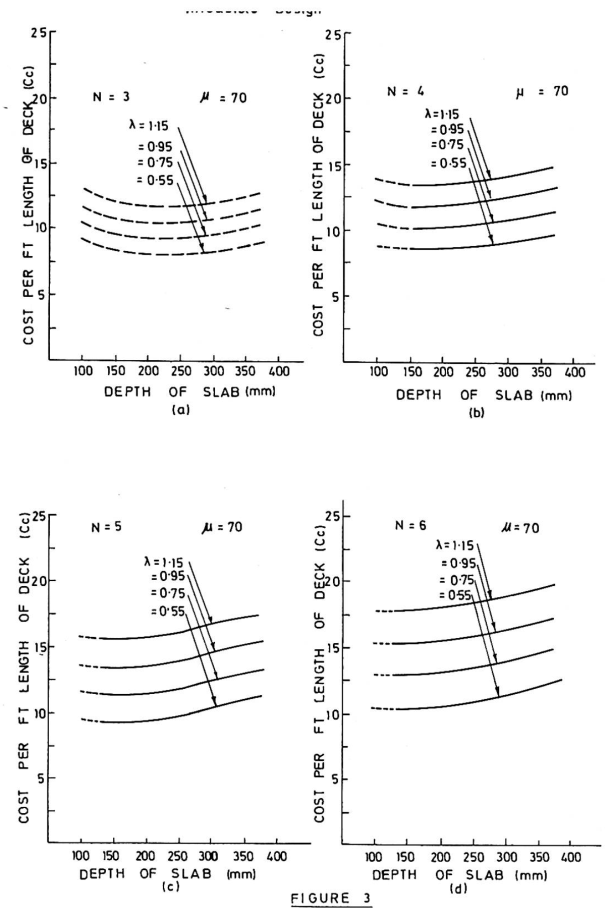

FIGURE 3

VARIATION OF DECK COST WITH DEPTH OF SLAB FOR DIFFERENT λ (914mm x 305mm x 224kg UB)

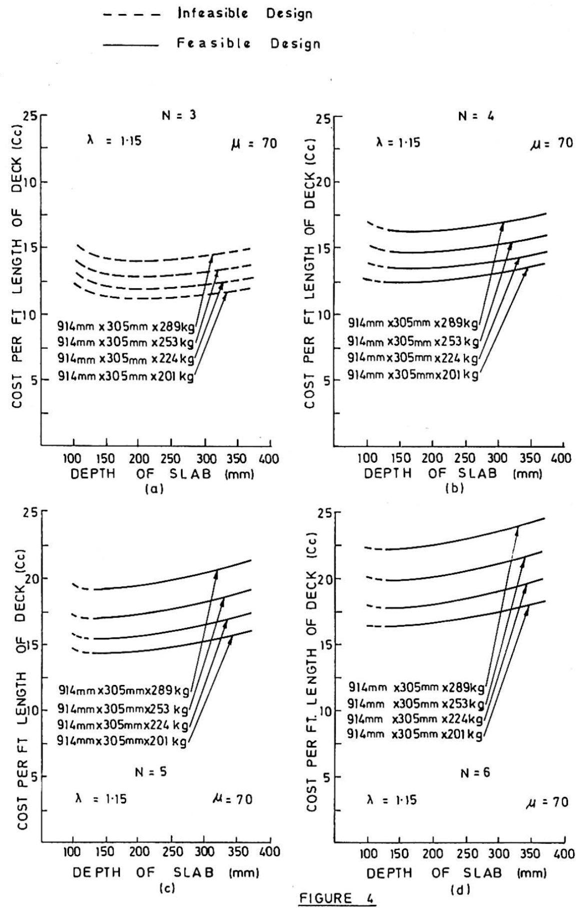

FIGURE 4

VARIATION OF DECK COST WITH DEPTH OF SLAB FOR GIRDERS OF SAME SERIAL SIZE

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Über die praktische Behandlung von Optimierungsaufgaben

On the Treatment of Optimization Problems for Better Use

Le traitement pratique des problèmes d'optimisation

NIKOLA S. DIMITROV
Prof. Dr.-Ing.
Institut für Tragkonstruktionen und konstruktives Entwerfen
Stuttgart, BRD

Mit Hilfe moderner Computer kann man heute ungleich komlexere Probleme angehen, als es zu den Zeiten der mechanischen Rechenautomaten möglich war. Diese Entwicklung ist natürlich zu begrüßen, leider hat sie auch eine Kehrseite. Neben den großen Vorteilen des Gebrauchs von EDV-Anlagen, wie

- hohe Kapazität
- schnelle Bearbeitungszeit
- Erledigung lästiger Routinearbeit

werden auch Nachteile sichtbar.

So verführt die Rechengeschwindigkeit der Automaten dazu, ein anstehendes Problem "simpel" zu lösen, z.B. unbekannte Funktionen durch einstellige Treppenfunktionen anstatt mehrstelliger Verfahren, welche bei finiten Elementen eine Menge von Unbekannten ausschalten, oder daß viele Optimierungsaufgaben durch ungezählte Iterationen gelöst werden, ohne sich ernsthaft mit der Möglichkeit einer direkten Lösung zu befassen.

Weiterhin wird durch die Maschinen die Suche nach einfacheren oder effizienteren Lösungswegen gebremst. Man hat ja ein Programm, wieso soll man nach einer neuen Methode zur Vereinfachung suchen?

So hängt dann leider nur zu oft die Arbeit des entwerfenden Ingenieurs von der Verfügbarkeit eines Programms, bzw. einer Rechenanlage ab. Nicht jeder ist aber in der glücklichen Lage, an ein Rechenzentrum angeschlossen zu sein. Es sollte daher, bei positivster Wertung der bisherigen Erfolge der EDV-Anlagen, nicht jener Grenzbereich vergessen werden, wo ein Problem nach entsprechender Behandlung auch mit einem Tischrechner, statt einer größeren Maschine gelöst werden könnte.

Bei solchen Forschungen kann der Ingenieur in den Entwurfsbüros am meisten profitieren. Er wäre dann imstande, noch mehr Probleme selbständig zu lösen, und nur noch bei wenigen Aufgaben auf große Rechenanlagen angewiesen.

An einem einfachen Beispiel soll demonstriert werden, wie sich aus einem anfänglichen Computerprogramm ein mit Tischrechnern lösbarer Algorithmus entwickelte.

Die Aufgabe ist es, einen Holz-Leimschichtträger als Einfeldträger so zu bemessen, daß er unter allen möglichen Rechteckquerschnitten die kleinste Fläche hat. Siehe Fig.1 u. Fig.2 .

Die Zielfunktion, sowie vier der Randbedingungen sind nichtlinear. In der ersten Bearbeitungsstufe wurde ein iteratives Programm mit Nullstellensuche nach Newton formuliert, welches den jeweils optimalen Punkt lieferte. Diesem Programm entspricht etwa die konventionelle zeichnerische Lösung. Siehe Fig.3 .

Zielfunktion:

$$Z: \quad F = b \cdot h = \alpha \cdot h^2 \longrightarrow \text{Minimum !}$$

Nebenbedingungen:

(1): $\text{vorh } \tau - \text{zul } \tau \leqq 0 \longrightarrow \text{erf } h_1 = \sqrt[2]{qe \cdot l} \cdot P_1$

(2): $\text{vorh } \sigma - \text{zul } \sigma_B \leqq 0 \longrightarrow \text{erf } h_2 = \sqrt[3]{qe \cdot l^2} \cdot P_2$

(3): $\text{vorh } f - \text{zul } f \leqq 0 \longrightarrow \text{erf } h_3 = \sqrt[4]{qe \cdot l^3 \cdot n_f} \cdot P_3$ $(\text{zul } f = l/n_f)$

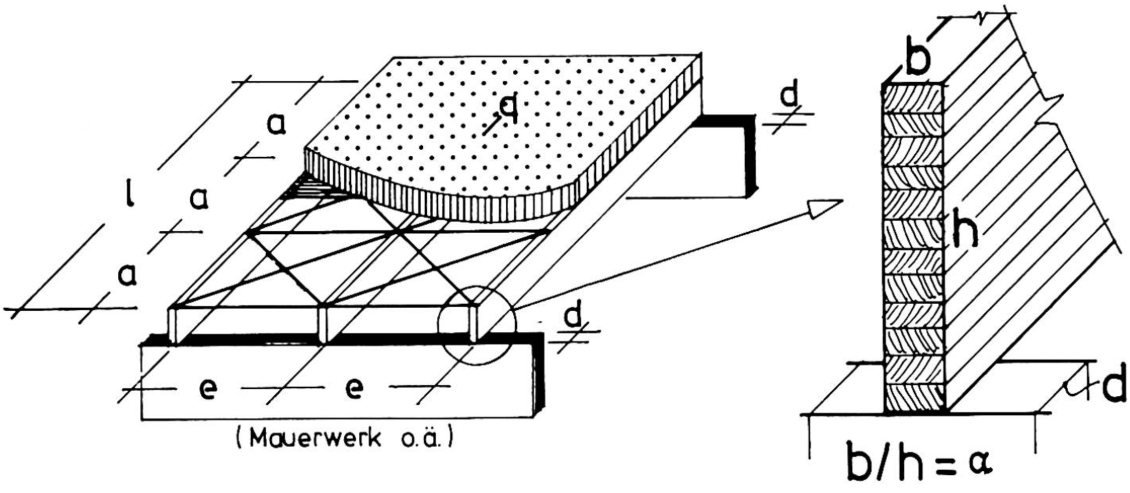
(4): $\nu_{kipp} \cdot \text{vorh } \sigma_B - \sigma_{kipp} \leqq 0 \longrightarrow \text{erf } h_4 = \sqrt[4]{qe \cdot l^3 \cdot \frac{1}{n_a}} \cdot P_4$ $(\text{vorh } a = l/n_a)$

(5): $\text{vorh } \sigma_{D\perp} - \text{zul } \sigma_{D\perp} \leqq 0 \longrightarrow \text{erf } h_5 = qe \cdot l \cdot \frac{1}{d} \cdot P_5$

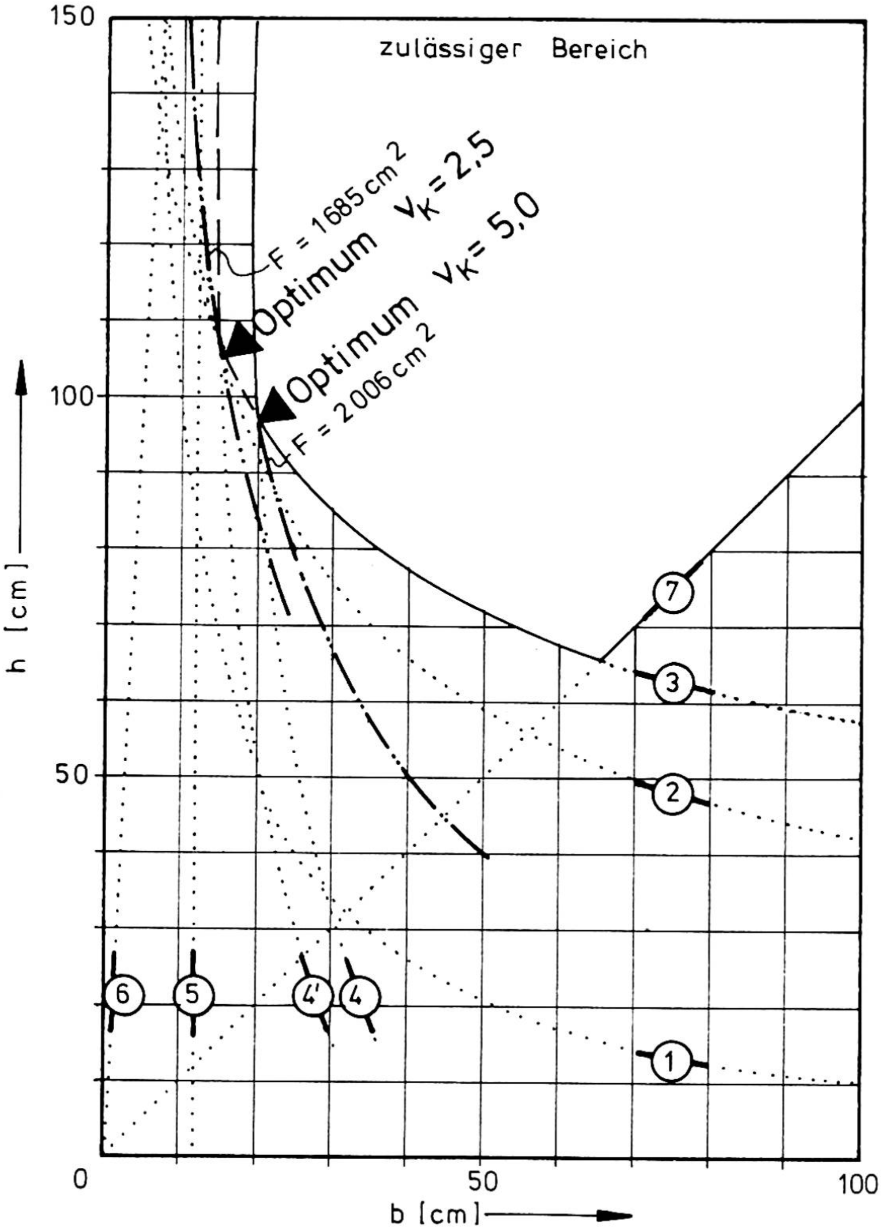
(6): $h/b - 25 \leqq 0 \longrightarrow \min \alpha = 0{,}04$

(7): $1 - h/b \leqq 0 \longrightarrow \max \alpha = 1{,}0$

Figur 1.: Zielfunktion und Nebenbedingungen



Figur 2.: Schema der Aufgabenstellung



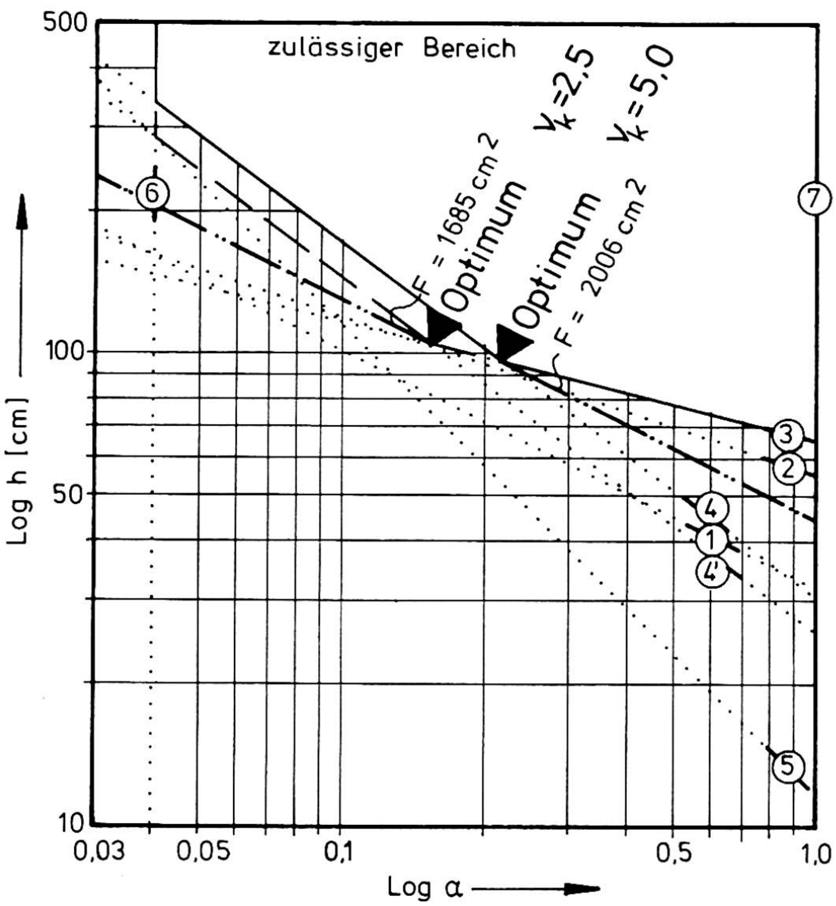
Figur 3.: Konventionelle zeichnerische Lösung

Das Beispiel ist für einen Brettschichtträger aus Nadelholz GKL I mit einer Spannweite von 20m, einer Last von 8,125kN/m, einer freien Kipplänge von 5m und einer zulässigen Durchbiegung von l/2oo gerechnet.

Werden keine ungewollten Ausmitten in der Berechnung berücksichtigt, sollte man mit einer Kippsicherheit von 5,0 rechnen, anderenfalls mit der Kippsicherheit $v_{kipp} = 2,5$. Dies ist beim Gebrauch der Tabellen zu beachten.

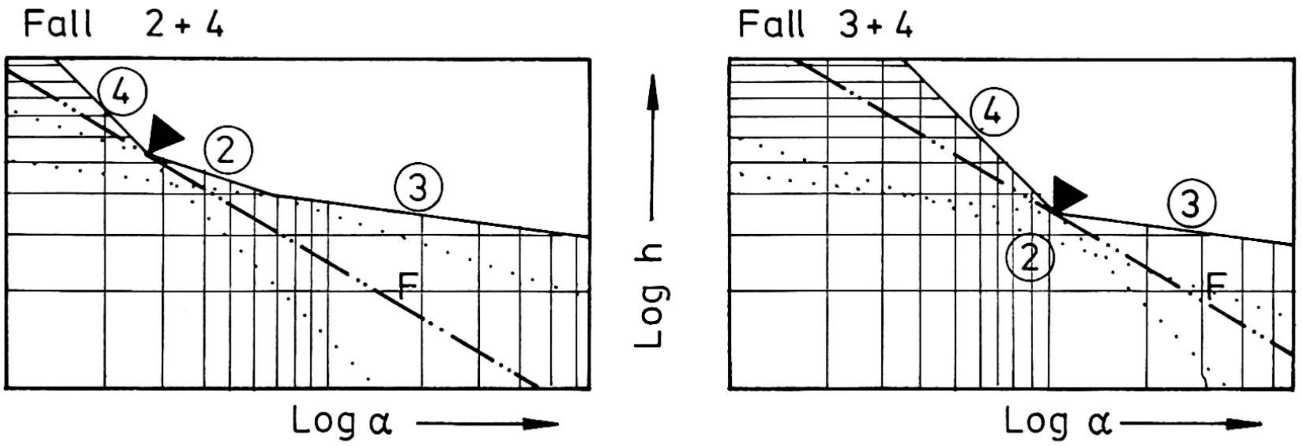
Die zeichnerische Darstellung ist umständlich und wegen der Nichtlinearitäten ungenau. Daher bestand die zweite Bearbeitungsstufe in der Einführung des logarithmischen Maßstabes, wodurch die Funktionen im praktisch vorkommenden Bereich linearisiert werden. So konnte ein Nomogramm entwickelt werden, welches die Suche nach dem optimalen Querschnitt innerhalb eines gegebenen Sortiments oder nach einer ganzzahligen Lösung sehr erleichtert.

Mit den Gleichungen aus Fig.2 und den Faktoren aus Tab.1 berechnet man für jede Randbedingung jeweils die erforderliche Querschnittshöhe für die Verhältnisse $b/h = \alpha = 0,03$ und $\alpha = 1,0$, trägt diese Werte auf Logarithmenpapier auf, verbindet zusammengehörige Punkte mit einem Lineal und hat so den Lösungsbereich. Die Zielfunktion F verläuft parallel zur Randbedingung (1) und wächst nach oben an. Siehe Fig.4. .



Figur 4.: Nomogramm

Für die Fälle, bei denen Ganzzahligkeit keine Bedingung ist und bei denen die Randbedingungen (1) und (5) vernachlässigbar sind, kann noch ein weiterer Bearbeitungsschritt gemacht werden. Durch Vereinfachung der Randbedingung (4) (Fehler auf der sicheren Seite und kleiner 5%) kann die kontinuierliche Lösung direkt angegeben werden, wenn festgestellt ist, welcher Fall maßgebend ist: 2+4 Biegespannung + Kippen, oder 3+4 Verformung + Kippen. Siehe Fig.5. .



$$h = H \sqrt[5]{(qe)^2 \; l^3} \qquad \text{Gl.1a}$$

$$b = B \sqrt[5]{qe \; l^4} \qquad \text{Gl.1b}$$

$$h = H \sqrt[4]{qe \; l^3} \qquad \text{Gl.2a}$$

$$b = B \sqrt[4]{qe \; l^3} \qquad \text{Gl.2b}$$

Figur 5.: Unterscheidung praktisch vorkommender Fälle

Hierzu berechnet man nach Gl.1a und Gl.2a die für den jeweiligen Fall erforderliche Querschnittshöhe h. Der Fall mit dem größeren Wert für h ist maßgebend und optimal. Die zugehörige Querschnittsbreite errechnet sich dann nach Gl.1b bzw. Gl.2b .

Die Faktoren H und B sind in Tabelle 2 zusammengestellt. Setzt man die Spannweite in m und die Last in kN/m ein, ergeben sich die Querschnittsabmessungen in cm.

So ist aus einem Programm für einen Kleincomputer ein einfacher Algorithmus für die Praxis geworden.

Europ. Nadelholz	GKL	P_1	P_2	P_3	P_4 V_{kipp}=2,5	P_4 V_{kipp}=5,0	P_5	α =
Kantholz	I		3.864 12.436					
	II	2.887 16.667	4.217 13.572	1.118 2.686	2.304 31.956	2.739 38.002	0.025 0.833	1,0 0,03
	III		4.750 15.286					
Leimquerschnitt	I	2.500 14.434	3.770 12.132	1.092 2.623	2.276 31.577	2.707 37.552	0.025 0.833	1,0 0,03
	II		4.085 13.148					

Tabelle 1: Faktoren zum Nomogramm

n_a =		1	2	3	4	5	6	7	8	9	10	
2+4	I	4.913	5.644	6.121	6.483	6.779	7.031	7.251	7.447	7.625	7.787	H
		2.219	1.682	1.430	1.275	1.166	1.084	1.019	0.966	0.922	0.883	B
	II	5.678	6.523	7.074	7.493	7.835	8.126	8.380	8.607	8.812	8.999	H
		2.115	1.603	1.363	1.215	1.111	1.033	0.971	0.920	0.878	0.842	B
3+4	200	5.056	5.514	5.800	6.013	6.183	6.325	6.448	6.557	6.654	6.742	H
		2.198	1.695	1.456	1.307	1.202	1.123	1.060	1.008	0.964	0.927	B
	300	5.886	6.419	6.753	7.000	7.198	7.364	7.507	7.634	7.747	7.850	H
		2.089	1.611	1.384	1.242	1.143	1.067	1.007	0.958	0.917	0.881	B

Tabelle 2a: Faktoren H und B für eine Kippsicherheit von 5,0

n_a =		1	2	3	4	5	6	7	8	9	10	
2+4	I	5.644	6.483	7.031	7.447	7.787	8.076	8.329	8.555	8.759	8.945	H
		1.682	1.275	1.084	0.966	0.883	0.821	0.772	0.732	0.698	0.670	B
	II	6.523	7.493	8.126	8.607	9.000	9.334	9.626	9.887	10.122	10.338	H
		1.603	1.215	1.033	0.920	0.842	0.783	0.736	0.698	0.665	0.638	B
3+4	200	5.514	6.013	6.325	6.557	6.742	6.898	7.032	7.150	7.256	7.353	H
		1.695	1.307	1.122	1.008	0.927	0.866	0.817	0.777	0.744	0.715	B
	300	6.419	7.000	7.364	7.634	7.850	8.031	8.187	8.325	8.448	8.560	H
		1.611	1.242	1.067	0.958	0.881	0.823	0.777	0.739	0.707	0.679	B

Tabelle 2b: Faktoren H und B für eine Kippsicherheit von 2,5

ZUSAMMENFASSUNG

An einem einfachen Beispiel wird demonstriert, wie aus einem Computerprogramm ein mit Tischrechnern lösbarer Algorithmus entwickelt werken kann.

SUMMARY

It is shown, how from an initial computer program a simple algorithm for table-top calculators can be developed.

RESUME

On montre à l'aide d'un exemple simple, la façon dont un algorithme soluble par calculateur de poche a pu être développé à partir d'un programme d'ordinateur.

Synthesis and Conclusions

Synthèse et conclusions

Synthese und Schlussfolgerungen

A.B. TEMPLEMAN
Department of Civil Engineering
University of Liverpool
Liverpool, Great Britain

Several interesting points have emerged from the papers in the Preliminary Report and from the deliberations at this session. First of all it can be seen that work in Structural Optimization is being carried out at very many different technical levels. Some of the work which has been described can only be classified as purely theoretical research. Other papers have described attempts to apply some of the advanced methods of structural optimization to practical design problems. A few contributions have described purely practical approaches to structural optimum design which use none of the advanced methods now available.

This very wide range of papers demonstrates one common theme. As structural engineers we are all interested in designing better structures. The ultimate goal of any designer must surely be to produce the best possible design for a structure to fulfill prescribed tasks. We are all seeking the best solution and consequently our work itself is an optimum-seeking process.

There are probably many engineers today who believe that there is no necessity for designers to concern themselves with some of the complicated mathematical methods which have been developed for structural optimization. They will probably argue that a good designer with great experience will naturally produce good designs which are closely optimal, therefore there is no need to use complex numerical techniques. To these people I would like to say that fifteen years ago the same things were being said about the new computer-oriented analysis methods such as matrix methods and finite-element techniques. Now there are few projects which do not at some stage use computer-based analysis methods to verify the suitability of a design. The computer is now an established engineering tool and we should make maximum use of it to help in design. Some people seem to fear that the computer might take over entirely if it is allowed to enter into the design process thus reducing the designer to the status of a computer operator. This fear is totally irrational - the computer is merely a servant carrying out such tasks as are assigned to it. It has no intuition or innovative capacity and these are the essential attributes of a good engineer which cannot be reduced to mathematical form. In structural engineering the experience and skill of a designer will always be required. The increasing use of computers for optimum design can only add to the status of the designer by making his efforts consistently better.

It came as a pleasant surprise to me that IABSE had planned to run this structural optimization session at the Tokyo Congress. For the practicing engineer structural optimization is still largely a thing of the future. For twenty years structural optimization has been an area of intensive research and only recently, within the last five years or so, some of this research has begun to percolate through to practical design engineers in a useable form. Some of this practical work we have seen during this session but the overwhelming tone of the discussions and papers has been that of research. The topic is not yet sufficiently developed for large scale practical use in everyday structural design. More research is necessary and more practical applications need to be studied in detail. Structural optimization still remains a thing for the future but the signs are that this future is not now very far away.

I cannot comment in detail on all this session's papers so I will restrict myself to comments upon a few. Firstly a very important point was made by Professor Massonnet on behalf of Dr. Maquoi. The use of the correct problem model is essential in optimization if realistic results are to be obtained quickly. It is seldom stressed that structural optimization is a two stage process. Firstly, a real-world problem must be modelled in mathematical form and secondly this mathematical problem must be solved. Often the first stage is the more difficult and, as Dr. Maquoi suggests, it is vitally important to get the right relationship between the real-world structure and its mathematical idealisation.

Dr. Hartmann's paper demonstrates this point very well. He shows a design for an encastré beam which, although optimal, is a totally impracticable design. Here the idealisation is incorrect and constraints to ensure a practical solution should have been included but were omitted. Unfortunately he ascribes the impracticable design to the use of mathematical optimization methods - the second stage of the process - and suggests that because mathematical methods produced this crazy design they should be discarded in favour of random search techniques. The logic here is surely wrong. It is not the mathematical techniques which are incorrect but the idealisation of the practical design problem

The papers by Di Carlo and Selleri are both interesting and worthy of further study. Mixed penalty functions as described by Di Carlo are not new, of course, but the specific application is interesting, demonstrating the existence of local optima. The problem of how to find the global optimum from several possible local optima is still an area for future research. Selleri's paper on cable structures and the papers by Daiguji and Feder on cable-stayed, prestressed structures reflect a growing interest in design problems associated with cables and similar work has been done on these topics in the USA and Great Britain.

Several of the papers presented for this Theme have used Linear Programming either straightforwardly or as a sequential iterative technique for design problems. To those people meeting structural optimization for the first time it might possibly appear that Linear Programming is a very powerful, widely applicable method for many structural design problems. I would like to set the record straight by stating that this is not so. Linear programming is a very simple method to use and consequently it is frequently employed to try to solve problems for which it is unsuitable. I will not repeat my warning as given in the Introductory Report, but will merely say that Linear Programming frequently causes more computational problems than it solves when applied to non-linear problems. Most structural optimization problems are non-linear.

I hope this session has given some idea of the great interest which now exists Worldwide in Structural Optimization. The Japanese National Group of IABSE quite rightly recognised the important place which structural optimization is likely to hold in structural design in the future and they showed remarkable foresight in suggesting it as a Theme for this Congress. I believe that at future Congresses much more will be heard of structural optimization and that in years to come it will become an established structural design tool. IABSE will then deserve much credit for stimulating discussions and interest in this topic at their 10th Congress.