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MECHANICS AND DESIGN OF TUBULAR STRUCTURES

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CHAPTER 5

TOPOLOGY OPTIMIZATION OF TUBULAR STRUCTURES

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ABSTRACT

Topology optimization is a great part of structural optimization. Structures should be safe and economic. In most cases these two conflicting aspects can be systematically synthesised by optimum design. Economy is achieved by minimizing the cost function and safety is guaranteed by considering the design constraints.

There are many optimization techniques. The hillclimb and backtrack mathematical programming methods are described giving a detailed flow chart. If a continuous mathematical method is used and discrete series of values are given for variables, the discrete optima can be determined by a complementary discretization, which is also explained. The multiobjective optimization techniques, where more objective functions are given have been shown.

The cost optimization is important in structural design. The welding times of various welding technologies are different. Using the COSTCOMP program we can calculate the welding times. Adding other times, like flattening plates, surface preparation, cutting, electrode changing, deslagging, painting, etc. we can form a complete cost function. Times are usually general, but costs are different in various countries. Introducing the fabrication and material specific cost ratio k_f/k_m between 0 and 2 kg/min, it is possible to build the cost function using times and to work out optimization in different economic conditions. Examples are shown for applications of design of welded box beams and stiffened plates. The fabrication cost percents for welded box beam and stiffened plate are 29 - 35 and 46 - 71% of the total costs, respectively, thus they can have a significant effect on optimum dimensions. The discrete optima depend on the manufacturers, on the k_f/k_m ratio and the welding technologies.

Optimum design problems of tubular trusses are treated. In these applications the discrete variables appear in various forms. In the cost function the material and fabrication (welding) costs are formulated. In the optimization of trusses it is verified that the optimum geometry depends on the profile shape of compression members.

Expert system shells, the Personal Consultant Easy (EASY) and the LEVEL 5 OBJECT (L5O) are used. The connection between the optimization techniques and the expert shells makes it possible to find the best solution among several alternatives. The Rosenbrock Hillclimb procedure is used in L5O and five single-objective and seven multiobjective optimization techniques are used in EASY. The benefits of these systems in the optimum design of belt-conveyor bridges are shown. In an example the main truss girder of a belt conveyor bridge is designed with different heights and different numbers of columns to find the optimal topology. The Eurocode 3 standard is used for the structural analysis.

5.1 INTRODUCTION

People in their everyday life always make optimization on a conscious or a subconscious way „to reach the best, which is possible with the resources available". The consciousness makes the act more efficient. They have always targets to reach and constraints to control them. The birth of optimization methods as mathematical techniques can be dated back to the days of Newton, Lagrange and Cauchy. The further development in optimization was possible by the developments of differential calculus by Newton, Leibnitz, the variational calculus by Bernoulli, Euler, Lagrange and Weierstrass, the introduction of unknown multipliers by Lagrange. The concept of multiobjective optimization was formulated one hundred years ago by Pareto in 1896.

The first written analytical work published on structural optimization was made by Maxwell in 1890, followed by the well known work of Michell in 1904. These works provided theoretical weight minima of trusses, using highly idealised models, but the analytical way of solution of the structural optimization problem is still usable.

During the Second World War and in the late 1940's and the early 1950's the development of optimization concerned to the minimum weight design of aircraft structural components: columns, stiffened panels, subject to compressive loads and to buckling. Digital computers appear in the early 1950's and gave a strong impulse to the application of linear programming techniques. The applications were focused primarily on steel frame structures.

In the late 1950's and 1960's the applications of structural optimization on lightweight structures concentrated to the aircraft and space industries. This time some new optimization techniques have been developed by works of Rosenbrock, Box, Powell. The great development of this period is that the finite element method, which is a powerful tool for analysis of complex structures, has been invented by Zienkiewich and applied by many others for structural analysis.

Modern structural optimization can be dated from the paper of Schmit in 1960, who drew up the role of structural optimization, the hierarchy of analysis and synthesis, the use of mathematical programming techniques to solve the nonlinear inequality constrained problems. The importance of this work is that it proposed a new philosophy of engineering design, the structural synthesis, which clarifies the methodology of optimization.

The optimum design procedure has three main phases as follows:

(1) *preparation*: selection of materials, profiles, type of structure, joints, fabrication technology, erection method, definition of loads, design constraints and objective function(s), definition of the candidate structural versions;

(2) *mathematical phase:* constrained function minimisation by computerised mathematical programming methods;

(3) *evaluation:* selection of the most suitable structural versions adding some heuristical aspects (aesthetics, transportation etc.), investigation of the most significant parameters, sensitivity, working out design rules and incorporation into expert system(s).

Optimum design is important tool for engineers, since it enables them to *achieve significant weight and cost savings* by using mathematical methods and by systematisation of the design process selecting all the important aspects.
In order to make a survey of the most important design problems for welded metal structures we start from the fact that the best way to decrease the weight of a plated structure is *the decrease of plate thicknesses*. In design of thin-walled structures a lot of problems arise as follows:
(1) *fabrication difficulties* caused by residual welding stresses and distortions;
(2) *stability problems*: overall and local buckling phenomena and their interaction;
(3) *high additional stresses due to warping torsion*: it is necessary to apply the strength theory of thin-walled structures;
(4) *high stress concentrations in joints*: danger of *fatigue fracture* in the case of variable loads;
(5) *vibrations* due to low eigenfrequencies of a thin-walled structure: it is necessary to study the *vibration damping methods*;
(6) to avoid buckling and vibration, *stiffeners* should be used and *stiffened plates and shells* should be designed;
(7) determination of the sufficient measure of the decrease of thicknesses by *optimum design*.

The above mentioned aspects emphasise the need to study the optimum design which is the main theme of this course.
The variables in the optimum design of welded metal structures are as follows:
- *in rod structures* dimensions of profiles (widths and thicknesses of plate elements of welded I- and box-beams),
- *in trusses:* co-ordinates of nodes, cross-sectional areas of members, number of members;
- *in stiffened plates and shells:* dimensions of plate or shell elements, number of stiffeners. These variables can be treated as continuous or discrete ones. The dimensions of plate elements or standard profiles can be given by a series of discrete values. In this case we can treat them as discrete variables or as continuous ones and, at the end of optimization we can discretize them by an additional procedure.

Thus, methods of discrete or continuous optimization can be used. The advantage of a method depends on the optimization problem as it will be shown by applications.

In the optimization of a *tubular truss* the overall and local buckling constraints have been important and the optimum structural height (distance between parallel chords) is sought which minimises the whole weight. The overall buckling constraints should be defined according to the Eurocode 3 which considers the effect of initial imperfections of rods, since the calculation with the Euler formula gives errors in the unsafe side.
In the optimization of a *roof truss* it was shown that, due to the differences between the radii of gyration of various profiles used in compression members, the optimum roof slope angle depends on the type of profile. The optima have been found by calculating the weights corresponding to the series of discrete slope angles.

5.2 SINGLEOBJECTIVE OPTIMIZATION TECHNIQUES

5.2.1 Design variables, objective functions, constraints and preassigned parameters

Objective function (more functions at multiobjective optimization), design variables, preassigned parameters and constraints describe an optimization problem.

Design variables and preassigned parameters
The quantities, which describe a structural system can be divided into two groups: preassigned parameters and design variables. The difference between them is that the members of the first group are fixed during the design, the second group is the design variables, which are varied by the optimization algorithm. These parameters can control the geometry of the structures. It is the designer choice, which quantities will be fixed or varied. They can be cross-sectional areas, member sizes, thicknesses, length of structural elements, mechanical or physical properties of the material, number of elements in a structure (topology), shape of the structure, etc.
For example, in the case of a simple tubular beam the quantities are as follows: 1. span length of the beam, 2. sizes and area of the cross section, 3. characteristics of applied materials, 4. loadings, bending moments, 5. shape of the beam, 6. type of supports, end conditions, 7. number of supports etc. Some of them can be design variable, all the others should be preassigned quantities.

Cross-sectional design variables
Size, or dimension variables are the simplest and the most natural design ones. The cross-section area of tension and compression members, the moment of inertia of bent members, or the plate thicknesses can be design variables of this kind. In simple cases a single design variable (i.e. area) is sufficient to describe the cross section, but for a more detailed design several variables may be necessary. For example, if we consider the overall buckling of members, the moment of inertia or the radius of gyration would be also important as design variable. It should emphasise that less variables for the same problem mean considerable advantage in the solution from the optimization point of view. From the analytical point of view, the result can be opposite.

Material design variables
The Young modulus, yield stress, material density, thermal conductivity, specific heat coefficient etc. can be material design variables. These properties has a discrete character, i.e. a choice is to be made only from a discrete set of variables. In most cases the optimization procedure is nondiscrete, so these discrete variables complicate the optimization problem. In this case it is advisable to use discrete optimization techniques like backtrack. For a few number of available materials non-discrete technique would probably be more efficient to perform the optimization separately for each material.

Geometrical variables
Geometrical variables are the span length of a beam, the coordinates of joints in a truss or in a frame. Although many practical structures have geometry, which is selected before

optimization, geometrical variables can be treated by most optimization methods. In general, the geometry of the structure is represented by continuous variables.

Topological variables
Topology means the structural layout, number of supports, number of elements etc. These can be of discrete or continuous type. In truss systems the topology can be optimized automatically if we allow members to reach zero cross section size. The uneconomical members can be eliminated during the optimization process. Integer topological variables can be the number of spans of a bridge, the number of columns supporting a roof system or the number of elements in a grillage system.

Constraints
Behaviour means those quantities that are the results of an analysis, such as forces, stresses, displacements, eigenfrequencies, loss factors etc. These behaviour quantities form usually the constraints. A set of values for the design variables represents a design of the structure. If a design meets all the requirements, it will be called feasible design. The restrictions that must be satisfied in order to produce a feasible design are called constraints. There are two kinds of constraints, explicit and implicit ones.

Explicit constraints
Explicit constraints, which restrict the range of design variables, may be called size constraints or technological constraints. These constraints may be derived from various considerations such as functionality, fabrication, or aesthetics. Thus, a size constraint is a specified limitation, upper or lower bounds on a design variable. Examples of such constraints include minimum slope of a portal frame structure, minimum thickness of a plate, minimum or maximum ratio of a box section height and width, etc.

Implicit constraints
Constraints derived from behaviour requirements are called behavioural constraints. Limitations on the maximum stresses, displacements, or local and overall buckling strength, eigenfrequency, damping are typical examples of behavioural constraints. The behaviour constraints can be regarded as implicit variables. The behavioural constraints are often given by formulae presented in design codes or specifications. Other parts of the behavioural constraints are computed by numerical technique such as FEM. In any case the constraints can be evaluated by analytical technique. From a mathematical point of view, all behavioural constraints may usually be expressed as a set of inequalities.
The constraints may be linear or nonlinear functions of the design variables. These functions may be explicit or implicit in the feasible region X and may be evaluated by analytical or numerical techniques. However, except for special classes of optimization problems, it is important that these functions should be continuous and have continuous first derivatives in X.

Design space, feasible region
We may regard each design variable as one dimension in a design space and any particular set of variables as a point in this space. In cases with two variables the design space

reduces to a plane problem. In the general case of N variables, we have an N-dimensional hyperspace.

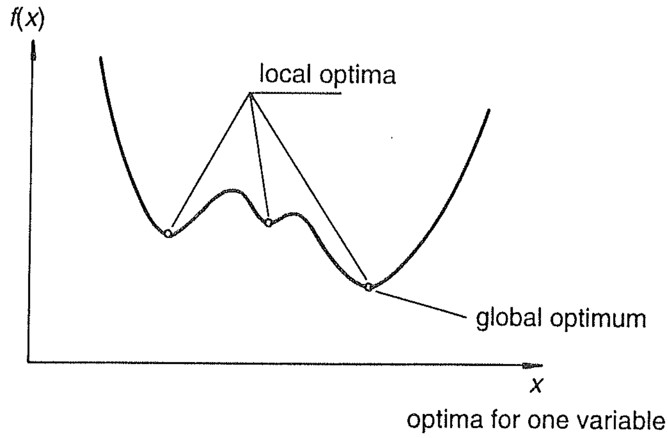


Fig. 5.1 Optima for one variable

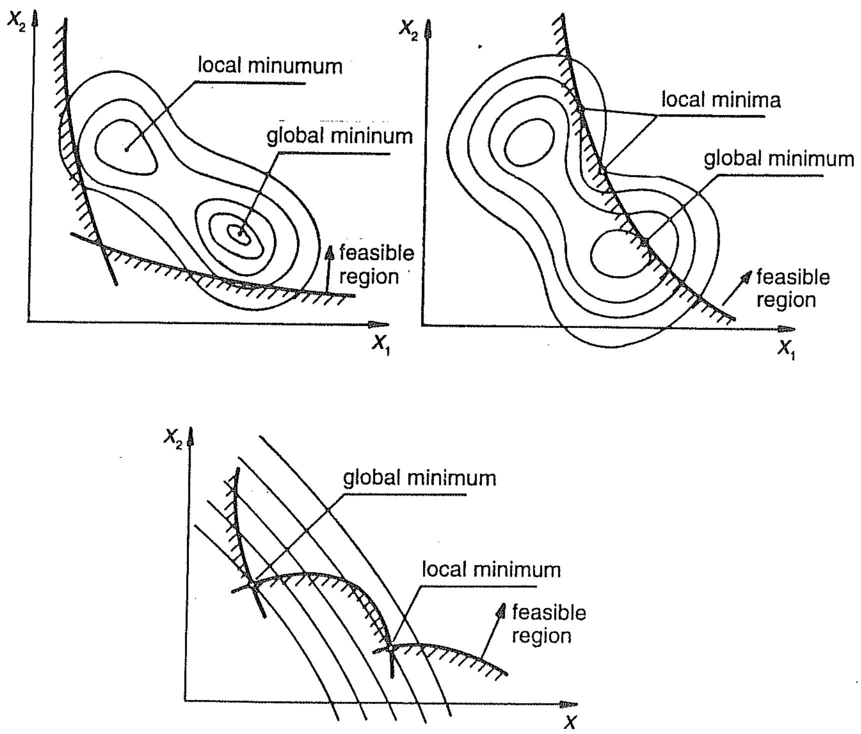


Fig. 5.2 Optima for two variables

Considering only the inequality constraints the set of values of the design variables that satisfy the equation $g_j(x) = 0$ forms a surface in the design space. These are boundary points. This surface cuts the space into two regions: one where $g_j(x) < 0$ (these are interior points) and the other where $g_j(x) > 0$ (these are the exterior points). The set of all feasible designs forms the feasible region. The solution of the constrained optimization problem in most cases lies on the surface. The solution can be local or global optimum (see Fig. 5.1, 5.2).
Any vector x that satisfies both the equality and inequality constraints is called a feasible point or vector. The set of all points which satisfy the constraints constitutes the feasible domain of $f(x)$ and will be represented by X; any point not in X is termed nonfeasible.

Convexity, concavity
It is very important to determine under what condition a local optimum is also a global one. It depends on the form of the feasible region, determined by the constraints. For a convex region the local optimum is a global one, otherwise there are several local optima (Fig.5.3a,b). Convex is a region, if between any two interior points all points are also interior, otherwise the region is non-convex.

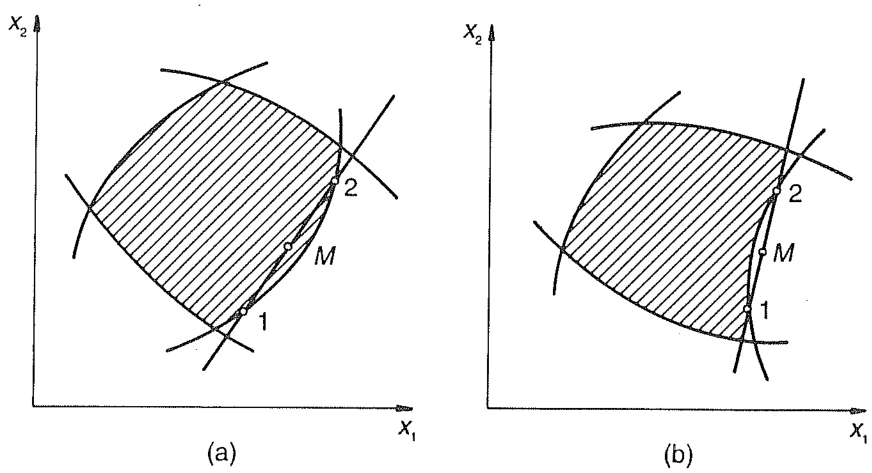


Fig. 5.3a. Convex sets b.) Non-convex sets

The set F (the feasible region) is convex, if for any points x_1, x_2 in the set the line segment joining these points is also in the set. Mathematically the Φ function is convex, if $\Phi(\Theta x_1+(1+\Theta)x_2) \leq \Theta\, \Phi(x_1)+(1-\Theta)\, \Phi(x_2)$ over the feasible domain. Θ is a scalar with the range $0 \leq \Theta \leq 1$. The sets shown in Fig. 5.4a are convex, those in Fig. 5.4b are not. They called non-convex, or concave. No analytical method is to classify a problem as being convex or non-convex.

Objective function
In most practical cases an infinite number of feasible designs exists. In order to find the best one, it is necessary to form a function of the variables to use it for comparison of

design alternatives. The objective function (also termed the cost, or merit function) is the function whose least, or greatest value is sought in an optimization procedure. It is usually a nonlinear function of the variables x, and it may represent the mass, the cost of the structure, or any other function, which extremum can give a possible and useful solution of the problem. The minimization of $f(x)$ is equivalent with the maximization of $-f(x)$.

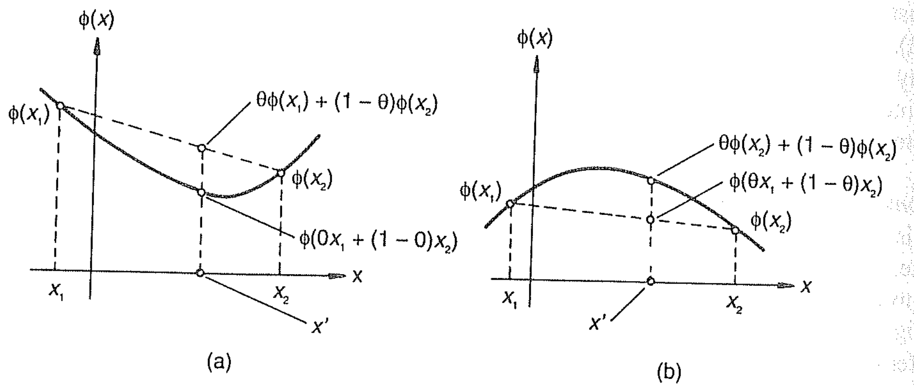


Fig. 5.4 Convex and non-convex function in one variable

The selection of an objective function can be one of the most important decisions in the whole optimum design process. If we choose several objectives to be minimized, we reach the area of multiobjective optimization and the greatest decision is to find in this case the relative importance of the different objective functions. Mass is the most commonly used objective function due to the fact that it is readily quantified, although most optimization methods are not limited to mass minimization. The minimum mass is usually not the cheapest. Cost is of wider practical importance than mass, but it is often difficult to obtain sufficient data for the construction of a real cost function. A general cost function may include the cost of materials, fabrication, welding, painting, maintenance, etc.

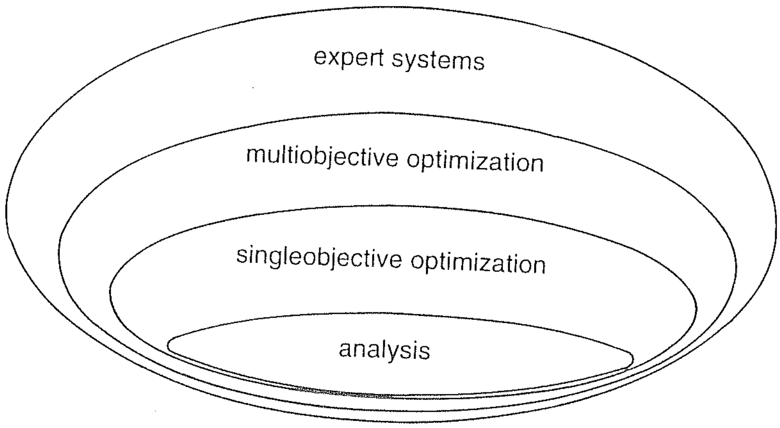


Fig. 5.5 Hierarchy of different design stages

Divisions in optimization techniques

The different single-objective optimization techniques make the designer able to determine the optimum sizes of structures, to get the best solution among several alternatives. The efficiency of these mathematical programming techniques is different. A large number of algorithms have been proposed for the nonlinear programming solution Himmelblau [5.1], Vanderplaats [5.2], Schittkowski et al [5.3]. Each technique has its own advantages and disadvantages, no one algorithm is suitable for all purposes. The choice of a particular algorithm for any situation depends on the problem formulation and the user.

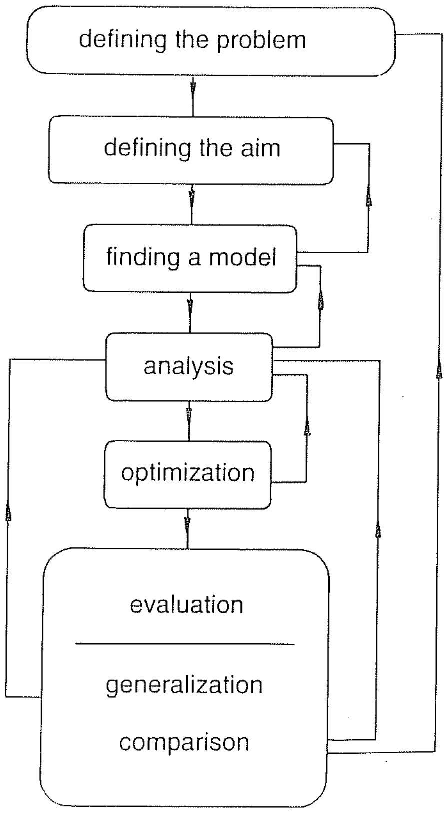


Fig. 5.6 Logical structure of optimum design

The logical structure of the optimum design can be seen in Fig. 5.6. It follows the logical thinking of a human. There are close connections between the different levels. If there are problems finding the optimum, we should go back to the analysis stage. If there are problems in the analysis stage it can be necessary to go back to the modelling stage. A good solution in the optimization is achieved by several loops in the scheme.

The general formulation of a single-criterion nonlinear programming problem is the following:

$$\text{minimize} \qquad f(x) \qquad x = \{x_1, x_2, \ldots, x_N\} \tag{5.1}$$

subject to $g_j(x) \le 0, \quad j = 1,2,\dots,P$ (5.2)

$h_i(x) = 0 \quad i = P+1,\dots,P+M$ (5.3)

f(x) is a multivariable nonlinear function, $g_j(x)$ and $h_i(x)$ are nonlinear inequality and equality constraints respectively.
The optimization models can be very different from each other. Fig. 5.7 shows the main alternatives.

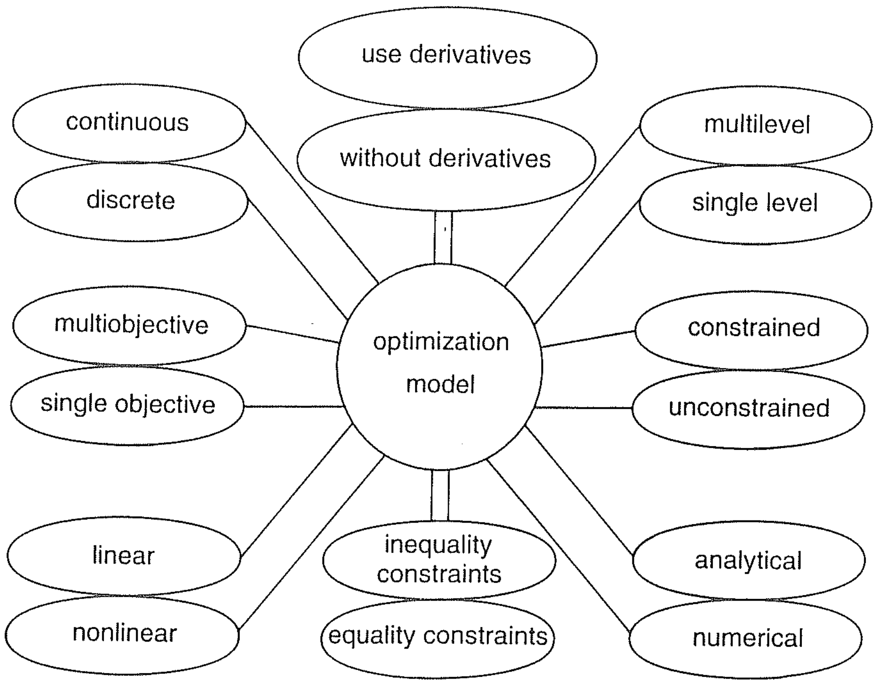


Fig. 5.7 Different optimization models

5.2.2 Methods without derivatives

Optimization techniques, which require the evaluation of function values only during the search, called methods without derivatives (zeroth-order methods). These methods are usually reliable and easy to program. Often can deal effectively with non-convex functions. The price to pay for this generality is that these methods often require thousands of function evaluations to achieve the optimum. Thus these methods can be considered as most useful for problems, in which the function evaluation is not computationally expensive and we can rerun the programs from different points to avoid local optima.

The hillclimb method

This method is a direct search one without derivatives. Rosenbrock's [5.4] method is an iterative procedure that bears some correspondence to the exploratory search of Hooke and

Jeeves [5.5] in that small steps are taken during the search in orthogonal coordinates. However, instead of continually searching the co-ordinates corresponding to the directions of the independent variables, an improvement can be made after one cycle of co-ordinate search by lining the search directions up into an orthogonal system, with the overall step on the previous stage as the first building block for the new search coordinates. Rosenbrock's method locates $x^{(k+1)}$ by successive unidimensional searches from an initial point $x^{(k)}$ along a set of orthonormal directions.
The method is executed as follows:

Minimize the objective function $f(x_i) \rightarrow \min.$

Design constraints are:

$$\begin{array}{llll} \text{explicit} & x_i^L \leq x_i \leq x_i^U & (i = 1,2,...,N), & \\ \text{implicit} & g_j(x_i) \geq 0 & (j = 1;2,..,M). & (5.4) \end{array}$$

(i) Before starting the minimization process, define a set of 'initial' step lengths S_i, to be taken along the search directions M_i, i= 1,2,...,N. The starting point must satisfy the constraints and should not lie in the boundary zones.
(ii) After each function evaluation, the following steps are carried out: Define by f^o the current best objective function value for a point where the constraints are satisfied, and $f(x)$ where in addition to this the boundary zones are not violated. f^o and $f(x)$ are initially set equal to the objective function value at the starting point.
(iii) The first variable x_1 is stepped a distance S_1 parallel to the axis and the function evaluated. If the current point objective function value, f, is worse (greater or less) than f^o or if the constraints are violated, the trial point is a failure and S_1 decreased by a factor β, $0 < \beta \leq 1$, and the direction of movement reversed. If the move is termed a success, S_1 increased by a factor α, $\alpha \geq 1$. The new point is retained, and a success is recorded. The values of α and β are usually taken as 3,0 and 0,5 respectively.
(iv) Continue the search sequentially stepping the variables, x_i, a distance S_i parallel to the axis. The same acceleration or deceleration and reversal procedure is followed for all variables, until at least one step has been successful and one step has failed in each of the N directions. Perturbations are continued sequentially in the search directions until a success is followed by a failure in every direction, at which time the kth stage is terminated. Since an equal value of a function counts as a success, a success is eventually reached in each direction as the multipliers of reduce the magnitude of the step length. The final point obtained becomes the initial point for the succeeding stage $x^{(k+1)} = x^{(k)}$. The normalized direction $S_i^{(k+1)}$ is chosen parallel to $x_0^{(k+1)} - x_0^{(k)}$, and the remaining directions are chosen orthonormal to each other and to $S_i^{(k+1)}$.
(v) Compute the new set of directions $M_{i,j}^{(k)}$ rotating the axes by the following equations. In general, the orthogonal search directions can be expressed as combinations of all the co-ordinates of the independent variables as follows :

$$M_{i,j}^{(k+1)} = \frac{D_{i,j}^{(k)}}{\left[\sum_{l=1}^{n}(D_{i,j}^{(k)})^2\right]^{1/2}} \tag{5.5}$$

where

$$D_{i,1}^{(k)} = A_{i,1}^{(k)} \tag{5.6}$$

$$D_{i,1}^{(k)} = A_{i,1}^{(k)} - \sum_{l=1}^{j-1}\left[(\sum_{n=1}^{j} M_{n,j}^{(k+1)} A_{n,j}^{(k)}) M_{i,j}^{(k+1)}\right] \qquad j = 2,3,...,N \tag{5.7}$$

$$A_{i,j}^{(k)} = \sum_{l=j}^{N} d_i^{(k)} M_{i,l}^{(k)} \qquad i = 1,2,...,N, \quad j = 1,2,...,N \tag{5.8}$$

d_i -sum of distances moved in the i direction since last rotation of axes.

(vi) Search is made in each of the x directions using the new co-ordinate axes. In each x direction the variables are stepped a distance S_j parallel to the axis and the function is evaluated.

$$\text{new } x_i^{(k)} = old\, x_i^{(k)} + S_j^{(k)} * M_{i,j}^{(k)} \tag{5.9}$$

(vii) If the current point lies within a boundary zone, the objective function is modified as follows;

$$f(new) = f(old) - (f(old) - f^*)(3\lambda - 4\lambda^2 + 2\lambda^3) \tag{5.10}$$

where the boundary zones are defined as follows:

$$\lambda = \frac{distance\ into\ boundary\ zone}{width\ of\ boundary\ zone}$$

lower zone: $$\lambda = \frac{x_i^L + (x_i^U - x_i^L) * 10^{-4} - x_i}{(x_i^U - x_i^L) * 10^{-4}} \tag{5.11}$$

upper zone: $$\lambda = \frac{x_i - (x_i^U - (x_i^U - x_i^L) * 10^{-4})}{(x_i^U - x_i^L) * 10^{-4}} \tag{5.12}$$

At the inner edge of the zone, $\lambda = 0$, i.e., the function is unaltered (f(new) $= f$(old)). At the constraints, λ=1, and thus f(new) $= f^*$.

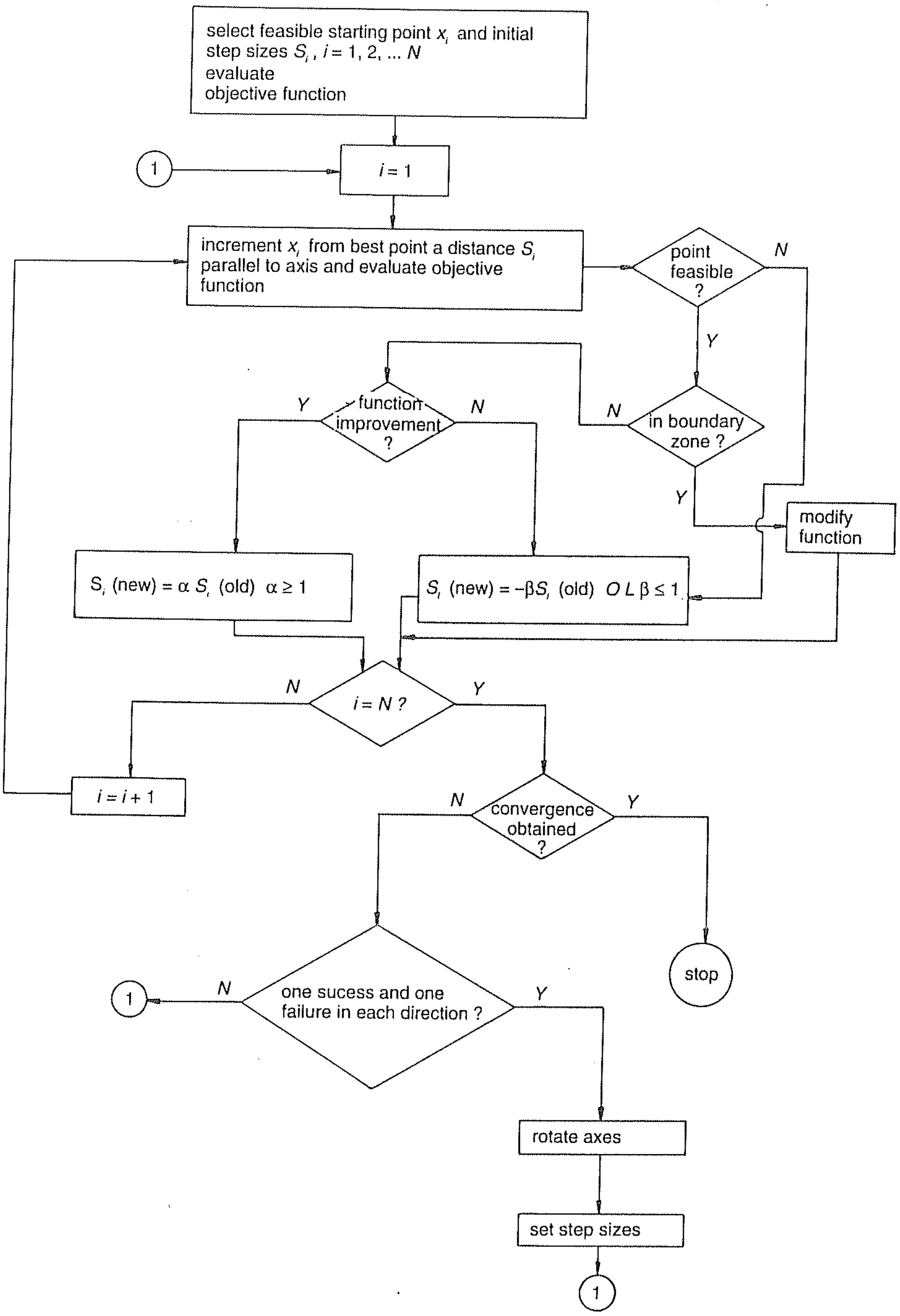
For a function, which improves as the constraint is approached, the modified function has an optimum in the boundary zone.

(viii) f^* is set equal to f^0 if an improvement in the objective function has been obtained without violating the boundary zones or constraints.

(ix) The search procedure to find the continuous values of the variables is terminated when the convergence criterion is satisfied.

(x) The procedure was modified by a secondary search to find the discrete values of the variables. A flow chart illustrating the above procedure is given in Fig. 5.8.

The procedure stops if the convergence criterion or the iteration limit is reached. The procedure is very quick, but it gives usually local optima, so it is advisable to use more



5.8 Flow chart of the hillclimb method

starting points. The Turbo/Borland C version of hillclimb technique can be found in Farkas & Jármai [5.6].

The backtrack method

Backtrack method is a combinatorial programming technique, solves nonlinear constrained function minimisation problems by a systematic search procedure. The advantage of the technique, that it uses only discrete variables, so the solution is usable. The general description of backtrack can be found in the works of Walker [5.7] and Golomb & Baumert [5.8]. This method was applied to welded girder design by Lewis [5.9], Annamalai [5.10] and Farkas & Jármai [5.6].

The general formulation of a single-criterion nonlinear programming problem is the following:

$$\text{minimise} \quad f(x) \quad x_1, x_2, \ldots, x_N \tag{5.13}$$

$$\text{subject to} \quad g_j(x) \le 0, \quad j = 1,2,\ldots,P \tag{5.14}$$

$$h_i(x) = 0 \quad i = P+1,\ldots,P+M \tag{5.15}$$

$f(x)$ is a multivariable nonlinear function, $g_j(x)$ and $h_i(x)$ are nonlinear inequality and equality constraints. The equality constraints should transfer to inequality ones to handle them by the program:

$$\begin{aligned} h_i(x) - \varepsilon &\le 0 \quad i = P+1,\ldots,P+M \\ h_i(x) - \varepsilon &\ge 0 \end{aligned} \tag{5.16}$$

ε is a given small number.

The algorithm is suitable to find optimum of those problems, which are characterized by monotonically increasing or decreasing objective functions. Thus, the optimum solution can be found by increasing or decreasing the variables. Originally the procedure can find the minimum of the problem. If we are looking for maximum, we should introduce - $f(x)$. The time of search is long, because the procedure makes a detailed search.

To find the optimum for a single variable many single variable search techniques are available. An efficient and suitable search method is the interval halving procedure. We assume that the objective function is monotonously decreasing, if the variables are decreasing. At the line search, when only one variable is changing, the aim is to find the minimum feasible value of the variable, starting from the maximum value.

The starting point, i.e. the maximum value, should satisfy the constraints. When the investigation shows, that the minimum value satisfies the constraints, then the solution is found. If not, the region is divided into two subregions with the middle value. If the constraints are satisfied with the middle value, then the upper region is feasible, all points there satisfy the constraints. In this case we should investigate the lower region, to find the border between the feasible and unfeasible regions.

Sign [means feasibility, sign { unfeasibility. The halving procedure works as follows:
Assume, that the variable is a thickness given by the following series of discrete values:

```
6    8    10    12    15    18    20    25    30 mm
{...........................................]
```

Furthermore assume that the maximum value is feasible, the minimum is unfeasible. If the middle value is feasible, the region to be investigated is as follows:

6 8 10 12 15

{..]

At the upper part of the region one cannot find any solution, so it is possible only at the lower part. We can leave the upper region without any further calculations. Continuing with the middle point of the lower region, if it is unfeasible, then the remaining region is only one quarter of the original one, after two checks.

10 12 15

{......................]

If the middle point is feasible, then it gives the solution.

12

]

The ratio of the number of total discrete points and checked discrete values is 9 / 4. If we have 1025 discrete values, then this ratio is much better, at the first halving step we can leave 512 discrete numbers without further investigations. The halving procedure stops, if the step length is less, that the distance between two discrete points. The step length should not be uniform between every discrete values, but for practical reasons we usually use a uniform value. The number of discrete values should be 2^k+1, where k is an integer number.

In the case of a completely general series the latter can be completed with the maximum values as follows:

Basic:	1	2	3	4	5	6	7	8	9
Completed:	4	6	8	10	12	14	16	16	16

At the backtrack method the variables are in a vector form $x = \{x_i\}^T$ $(i = 1,...,n)$ for which the objective function $f(x)$ will be a minimum and which will also satisfy the design constraints $g(x) \geq 0$ $(j = 1,...,P)$. For the variables, series of discrete values are given in an increasing order. In special cases the series may be determined by $x_{k,min}$, $x_{k,max}$ and by the constant steps Δx_k between them. The flow chart for backtrack method is given in Fig. 5.9. First a partial search is carried out for each variable and if all variations have been investigated, a backtrack is made and a new partial search is performed on the previous variable. If this variable is the first one: no variations have to be investigated (a number of backtracks have been made), then the process stops. The main phases of the calculation are as follows.

1. With a set of constant values of $x_{i,t}$ $(i = 2,...,n)$ the minimum $x_{i,m}$ value satisfying the design constraints is searched for. The interval halving method can be employed. This method can be employed if the constraints and the objective function are monotonous from the sense of variables.

2. As in the case of the first phase, the halving process is now used with constant values, and the minimum $x_{1,m}$ value, satisfying the design constraints is then determined.

3. The least value $x_{n,m}$ is calculated from the equation relating to the objective function $f(x)$

$$f(x_{1,m},\ldots,x_{n,m}) = f_o$$

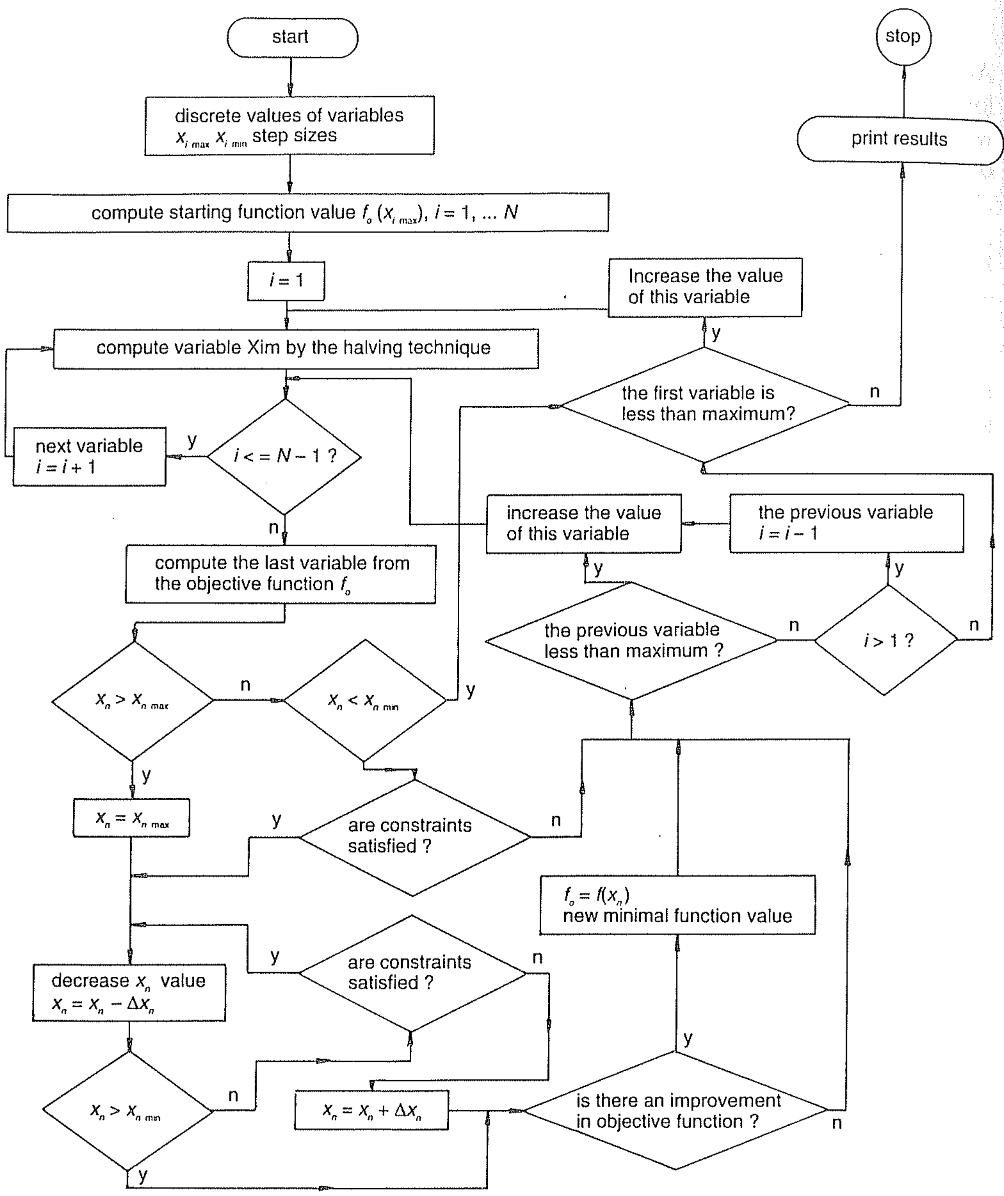


Fig. 5.9 Flow chart of backtrack method

where f is the value of the cost function calculated by inserting the maximum x-values. Regarding the $x_{n,m}$ value, three cases may occur as follows.

(3a) If we decrease x_{n-1} step-by step till it satisfies the constraints or till $x_{n,min}$, the minimal values are reached. If all variations of the x_n value have been investigated, then the program jumps to the x_{n-1} and decreases it step-by step till x satisfies the constraints or till $x_{n-1,\, min}$ are reached.

(3b) If $x_{n,m} < x_{n,1}$, we backtrack to x_{n-1} .

(3c) If $x_{n,m}$ does not satisfy the constraints, we backtrack to $x_{n-1,m}$. If the constraints are satisfied, we continue the calculation according to 3a.

The number of all possible variations is $\prod_{i=1}^{n} t_i$ where t_i is the number of discrete sizes for one variable. However, the method investigates only a relatively small number of these. Since the efficiency of the method depends on many factors (number of unknowns, series of discrete values, position of the optimum values in the series, complexity of the cost function and/or that of the design constraints), it is difficult to predict the run time. The main disadvantage of the method is, that the runtime increases exponentially, if we increase the number of unknowns.

We've made the program in C language modifying the procedure in the sense, that originally the program depended on the number of variables. All variables were computed by the halving procedure except the last one, which was computed from the objective function. The modified version is independent from the number of variables. The Turbo/Borland C version of backtrack method can be found in Farkas & Jármai [5.6]. Advantage of the methods, that it gives discrete values, usually finds global minimum. The disadvantage of the method that it is useful only for few variables because of the long computation time.

5.3 DISCRETIZATION AFTER CONTINUOUS OPTIMIZATION

The number of nondiscrete optimization techniques is more that of discrete one. To make the search more practicable it is advisable to use discrete member sizes. The original program can be extended with a secondary search to find discrete optimum sizes in such a way, that not only the explicit and implicit constraints satisfied are but the merit function takes its minimum as well. It is assumed that the optimum discrete sizes are near to the optimal continuous ones [5.11].

Starting from the optimum continuous values, the secondary search chooses the nearest discrete sizes for each continuous size from the series of discrete values. The number of chosen discrete sizes for one continuous size can be two, three or more. The possible variations can be obtained using binary, ternary or larger systems. In our numerical example we use the binary system, two discrete sizes, upper and lower, belonging to one continuous value. In a binary system number the figure zero means the upper discrete size,

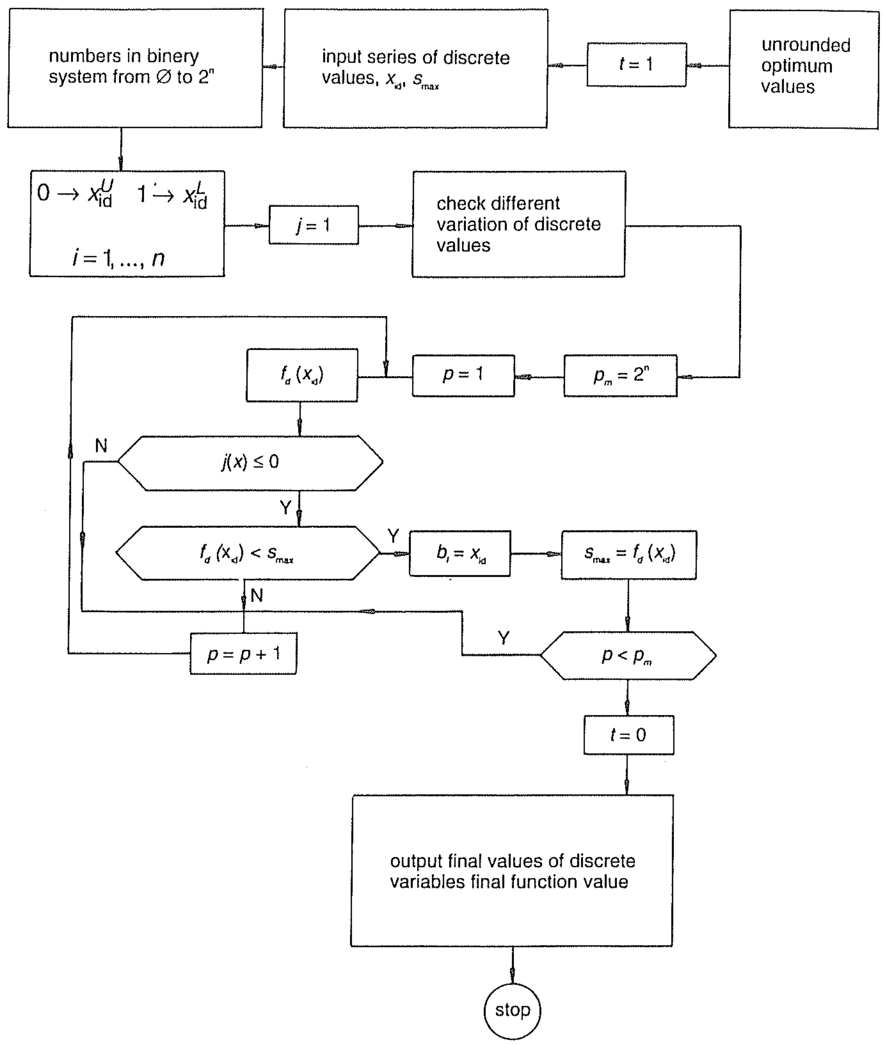


Fig. 5.10 Flow chart of the secondary discretization

the figure one means the lower one. The first $2n$ number in binary system gives the all possible variations. Each variation is tested, whether the explicit and implicit constraints are satisfied, and the optimal values minimising the merit function are determined. (Fig. 5.10).

The unrounded optimum values of fourth variable are as follows:

1^{Lower}	$1^{\text{Unrounded}}$	1^{Upper}
2^{Lower}	$2^{\text{Unrounded}}$	2^{Upper}
3^{Lower}	$3^{\text{Unrounded}}$	3^{Upper}
4^{Lower}	$4^{\text{Unrounded}}$	4^{Upper}

The number 0000 means the lower discrete values of all variables, the number 1111 means the upper discrete values of all variables. The other numbers in the binary system are the variants of the possible discrete solution. One of the tested variants is the solution, giving the minimum objective function value.

5.4 MULTIOBJECTIVE OPTIMIZATION

The first work on multiobjective (multicriterion, vector) optimization was presented by Pareto in 1896 [5.12]. After at least fifty years, problems of multiobjective optimization were again considered by von Neumann and Morgenstern [5.13]. A relatively modern formulation of the multiobjective optimization problem was presented by Zadeh [5.14]. However, wider interest in optimization theory, operations research, and control theory was not taken until the late 1960s, and since then numerous papers have been published on the subject. Most are concerned with the theory and applications of multiobjective decision making from a general viewpoint, and few applications to engineering design can be found. Comprehensive bibliographies on multicriterion decision-making and related areas have been written by Stadler [5.15, 5.16], Cohon [5.17] present the foundations of modern multiobjective optimization.

Almost all decisions are multiobjective. In complex engineering design problems there often exist several non-commensurable objectives which must be considered. Such a situation is formulated as a multiobjective optimization problem in which the designer's goal is to minimise and/or maximise several objective functions simultaneously to get compromise between them.

For decades engineers have employed a single measure, like costs, weights, or benefits, to determine an optimum.

For example design a simple beam with two supports at the end, the cost of the beam can be an objective, to be minimised, but also the rigidity should be maximal, or with another measure the deflection should be minimum. These two objectives: cost and deflection are in conflict. A singleobjective solution cannot be a reasonable one. A compromise is necessary to reduce these two conflicting values as much, as possible. Multiobjective programming techniques the tools doing this. Multiobjective analysis and optimization represent a general philosophy of design. It puts the designer a more useful position of providing decision makers a set of good alternative solutions rather than a single optimal solution.

The most popular criteria used in structural optimization are: minimum mass or cost, maximum stiffness, minimum displacement at specific structural points, maximum frequency of free vibration and so on. These criteria are very often in conflict Osyczka

[5.18, 5.19]. In such cases, it is necessary to formulate a multicriteria optimization problem and look for the set of compromise solutions in the objective space. Next, the so-called preferred solution should be chosen taking into account an additional criterion or using a so-called global criterion like the utility function, distance function or hierarchical method (see Eschenauer et al. [5.20]; Jendo [5.21]; Koski [5.22].

A multicriteria optimization problem can be formulated as follows :

Find x such that

$$f(x^*) = opt\, f(x), \tag{5.17}$$

such that

$$g_j(x) \geq 0 \qquad j = 1,....,P \tag{5.18}$$

$$h_i(x) = 0 \qquad i = P,....,P+Q$$

where x is the vector of decision variables defined in n-dimensional Euclidean space and $f_k(x)$ is a vector function defined in r-dimensional Euclidean space. $g_j(x)$ and $h_i(x)$ are inequality and equality constraints.

The solutions of this problem are the Pareto optima. The definition of this optimum is based upon the intuitive conviction that the point x^* is chosen as the optimal, if no objective can be improved without worsening at least one other objective.

5.4.1 Method of objective weighting

Weighting objectives method

The pure weighting method means to add all the objective functions together using different weighting coefficients for each. It means, that we transform our multicriteria optimization problem to a scalar one by creating one function of the form:

$$f(x) = \sum_{i=1}^{r} w_i f_i(x)$$

$$\text{where } w_i \geq 0 \text{ and } \sum_{i=1}^{r} w_i = 1 \tag{5.19}$$

If we change the weighting coefficients result of solving this model can vary significantly, and depends greatly from the nominal value of the different objective functions.

Normalized objectives method

The normalized objectives method solves the problem of the pure weighting method e.g. at the pure weighting method, the weighting coefficients do not reflect proportionally the relative importance of the objective, because of the great difference on the nominal value of the objective functions. At the normalized weighting method w_i reflect closely the importance of objectives.

$$f(x) = \sum_{i=1}^{r} w_i f_i(x) \,/\, f_i^0$$

where $w_i \geq 0$ and $\sum_{i=1}^{r} w_i = 1$ (5.20)

The condition $f_i^0 \neq 0$ is assumed.

5.4.2 Method of distance functions

Let f^0 be the ideal solution that simultaneously yields minimum values for all criteria. Such a solution does not exist but is introduced in compromise programming as a target or a goal to approach, although impossible to reach (perfection is impossible).
In compromise programming, the "best" or satisfying solution is defined as one that minimises the distance from the set of nondominated solutions to the ideal solution (*). The criterion used in compromise programming is the minimization of the normalized deviation from the ideal solution f^0 measured by the family of L_p metrics defined in several different forms.

This family of L_p metrics indicates how close the satisfying solution is to the ideal solution, and represents the feasible set. In this paper, the satisfying solutions are determined for two particular values of p, namely, $p = 2$ and $p = \infty$ (which correspond to the minimization of the Euclidean and maximum distances, respectively), and are given below. For the case $p = \infty$, the largest deviation is the criterion of comparison and is referred to as min-max criterion.

Global criterion method type I
Global criterion method means that a function which describes a global criterion is a measure of closeness the solution to the ideal vector of f^0. The common form of this function is:

$$f(x) = \sum_{i=1}^{r} \left((f_i^0 - f_i(x)) / f_i^0 \right)^P \qquad P=1,2,3,... \qquad (5.21)$$

It is suggested to use P=2, but other values of P such as 1,3,4, etc. can be used. Naturally the solution obtained will differ greatly according to the value of P chosen, P=1 means a linear correlation, P=2 a quadratic one, etc.

Global criterion method type II
The deviations in the absolute sense are as follows:

$$L_P(f) = \left[\sum_{i=1}^{k} \left| f^0{}_i - f_i(x) \right|^P \right]^{1/P} \qquad 1 \leq P \leq \infty \qquad (5.22)$$

if P=1 $$L_P(f) = \sum_{i=1}^{k} \left| f^0{}_i - f_i(x) \right| \qquad (5.23)$$

if P=2 $$L_P(f) = \left[\sum_{i=1}^{k} \left| f^0{}_i - f_i(x) \right|^2 \right]^{1/2} \qquad \text{Euclidean metric} \qquad (5.24)$$

if $P=\infty$ $L_P(f) = max|f^o_i - f_i(x)|$ Chebysev metric i=1,...,k (5.25)

Global criterion method type III
Instead deviations in the absolute sense it is recommended to use relative deviations such as

$$L_P(f) = \left[\sum_{i=1}^{r}\left|\frac{f^o_i - f_i(x)}{f^o_i}\right|^P\right]^{1/P} \qquad 1 \le P \le \infty \qquad (5.26)$$

In this case the P has a larger set.

5.4.3 Min-max method

At the min-max method the maximum loss of the collective objective will be minimised. The min-max optimum compares relative deviations from the separately reached minima. The relative deviation can be calculated from

$$z_i'(x) = \frac{|f_i(x) - f^o_i|}{|f^o_i|} \quad \text{or} \quad z_i''(x) = \frac{|f_i(x) - f^o_i|}{|f_i(x)|} \qquad (5.27)$$

If we know the extremes of the objective functions, which can be obtained by solving the optimization problems for each criterion separately, the desirable solution is the one which gives the smallest values of the increments of all the objective functions. The point x^* may be called the best compromise solution considering all the objective functions simultaneously and on equal terms of importance.

$$z_i(x) = max\,\{ z_i'(x), z_i''(x) \} \quad i \in I \qquad (5.28)$$

$$\mu(x^*) = min\,max\,\{ z_i(x) \} \quad x \in X \;\; i \in I \qquad (5.29)$$

where X is the feasible region.

5.4.4 Constrained method

The basis of these methods is the transformation of the vector optimization problem into a sequence of single objective optimization problems by retaining one selected objective as the primary criterion to be optimized and treating the remaining criteria as some predetermined constraints. These constants are then altered within their defined ranges, and the subset of Pareto optima is systematically generated. This approach has gained wide acceptance because it is more practical and rational than the weighting objectives method, if there is a dominant objective exists.

Minimise $f_L(x)$ (5.30)

subject to the constraints $g_j(x) \ge 0$ $j = 1,....,M$ (5.31)

$f_i(x) \ge h_i$ $i = 2,....,Q$

and side bounds on the design variables as

$$x_i^L \le x_i \le x_i^U \quad i = 1,....,N \qquad (5.32)$$

where h_i are the parametrically varied target levels of Q objective functions. Each vector, or constraint set, H of the various h_i, will produce one Pareto solution. As in the weighting method, many different combinations of values for each h_i,, must be examined in turn to generate the entire Pareto set. The constraint method provides direct control of the generation of members of the Pareto set and generally provides an efficient method for defining the shape of the Pareto set. It should be noted that the constraint method does not recognize weak Pareto optima.

After transformation of the vector optimization problem into a scalar optimization problem, the latter may be solved using some appropriate mathematical programming techniques.

5.4.5 Hybrid methods

Weighting global criterion method

The weighting global criterion method is made, by introducing weighting parameters, one could get a great number of Pareto optima with (5.16) (Jármai [5.23]. If we choose P=2, which means the Euclidean distance between Pareto optimum and ideal solution Jármai [5.24]. The coordinates of this distance are weighted by the parameters as follows:

$$L_P(f) = \left[\sum_{i=1}^{r} w_i \left| \frac{f^0{}_i - f_i(x)}{f^0{}_i} \right|^2 \right]^{1/2} \qquad (5.33)$$

where P is the dimension of the function space, x indicates the design variables and X the constraint set, r is the number of objective functions, f^0 is the optimum of the *ith* objective function, and w_i are the weighting factors.

The solution obtained by minimizing Eq. (5.33) differs greatly depending on the value of P chosen.

Weighting min-max method

The weighting min-max method one gets combining the min-max approach with the weighting method, a desired representation of Pareto optimal solutions can be obtained

$$z_i(x) = \max \{ w_i z_i'(x), w_i z_i''(x) \} \quad i \in I \qquad (5.34)$$

Weighting coefficients w_i reflect exactly the priority of the criteria, the relative importance of it. We can get a distributed subset of Pareto optimal solutions.

Selection of the „best" solution

Once a subset of Pareto optima has been generated, the designer has to make an important decision concerning the selection of the best solution from this subset. The selection is not obvious when several conflicting criteria are considered but may be made subjectively by giving preference to one criterion over the others.

5.5. COST CALCULATION OF WELDED STEEL STRUCTURES

In the early stage of optimization the mass of the structures has been minimized. Nowadays there are also some optimization techniques, which can not handle complicated cost functions. To get an economic structure in the period of increasing fabrication costs, one should take into account as many elements of costs as possible. The cost of a structure is the sum of the material, fabrication, transportation, erection and maintenance costs. Fabrication cost elements are the welding-, cutting-, preparation-, assembly-, tacking-, painting costs etc. It is very difficult to obtain such cost factors, which are valid all over the world, because there are great differences among the cost factors in the highly developed and at the developing countries, there are also great differences in the same country among factories, which are highly automated or not. If we choose times, as the basic data of fabrication phases, we can handle this problem. The fabrication time depends on the technological level of the country and the manufacturer, but it is much closer to the real process to calculate with. After computing the necessary time for each fabrication phase one can multiply it by a specific cost factor, which can represent the development level differences.

Although the whole production cost depends on many parameters and it is very difficult to express their effect mathematically, a simplified cost function can serve as a suitable tool for comparisons useful for designers and manufacturers. The artificial intelligence is also applied for cost estimation. In this paper we would like to emphasise the role of fabrication costs, especially the role of welding costs using different welding technologies. We don not consider neither the amortisation, transportation, erection, maintenance costs, nor the variation of exchange rates, etc.

5.5.1 Fabrication costs

The cost function can be expressed as

$$K = K_m + K_f = k_m \rho V + k_f \sum_i T_i \tag{5.35}$$

where K_m and K_f are the material and fabrication costs, respectively, k_m and k_f are the corresponding cost factors, ρ is the material density, V is the volume of the structure, T_i are the production times.

Fabrication times for welding

Eq.(5.35) can be written in the following form

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m}(T_1 + T_2 + T_3 + T_4 + T_5 + T_6 + T_7) \tag{5.36}$$

where

$$T_1 = C_1 \Theta_d \sqrt{\kappa \rho V} \tag{5.37}$$

is the time for preparation, assembly and tacking, Θ_d is a difficulty factor, κ is the number of structural elements to be assembled.

Formula (3) can be derived as follows (Lihtarnikov [5.25]). For a plated structure consisting of κ elements the time of this part of the fabrication is proportional to the perimeter, for the *i*th element it is $T_i = c_1 P_i$. The mass of an element is proportional to the square of the perimeter $G_i = c_2 P_i^2$, thus $P_i = c_3\sqrt{G_i}$ and $T_i = c_4\sqrt{G_i}$. For the total structure, in average, it is $G = \kappa G_i$ and $T_1 = \kappa T_i = c_5\kappa\sqrt{G/\kappa} = c_6\sqrt{G\kappa}$.
Some proposed values for the difficulty factor are summarized in Table 5.1.

Table 5.1 Proposed values for the difficulty factor Θ_d. For skewed angle joints add 1-2 points

Structures	Welds	V-weld 60^0	Fillet weld 90^0
Planar	long welds, flat position	1.0	2.0
Spatial	short welds, plate, flat steel	1.5	2.5
Spatial	U-,L-profiles, tubes	2.0	3.0
Spatial	I-, T-profiles	2.5	4.0

$$T_2 = \sum_i C_{2i} a_{wi}^{1.5} L_{wi} \tag{5.38}$$

is the time of welding, a_{wi} is the weld size, L_{wi} is the weld length, C_{2i} are constants given for different welding technologies. For manual-arc welding $C_2 = 0.8*10^{-3}$ and for CO_2-welding $C_2 = 0.5*10^{-3}$ min/mm$^{2.5}$.

$$T_3 = \sqrt{\Theta_d}\sum_i C_{3i} a_{wi}^{1.5} L_{wi} \tag{5.39}$$

is the time of additional fabrication actions such as changing the electrode, deslagging and chipping. $C_3 = 1.2*10^{-3}$ min/mm$^{2.5}$. Formulae (5.37, 5.38, 5.39) have been proposed by Pahl and Beelich [5.26].

Ott & Hubka [5.27] have proposed that
$C_3 = (0.2\text{-}0.4)C_2$ in average $C_3 = 0.3C_2$. Thus, the modified formula for T_2+T_3 , neglecting $\sqrt{\Theta_d}$, is

$$T_2 + T_3 = 1.3\sum C_{2i} a_{wi}^{1.5} L_{wi} \tag{5.40}$$

The software COSTCOMP [5.28] was developed by the Netherlands Institute of Welding. It gives welding times and costs for different welding technologies [5.29]. Using Eq. (5.36) for T_1, the other times are calculated with a generalized formula

$$T_2 + T_3 = 1.3\sum C_{2i} a_{wi}^{n} L_{wi} \tag{5.41}$$

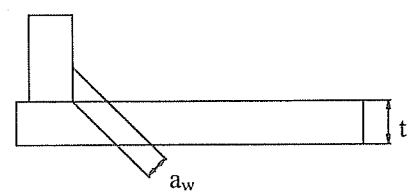
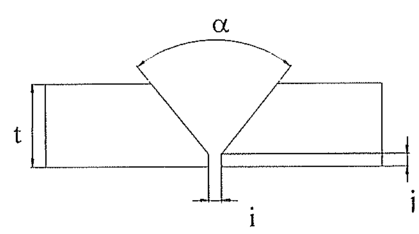
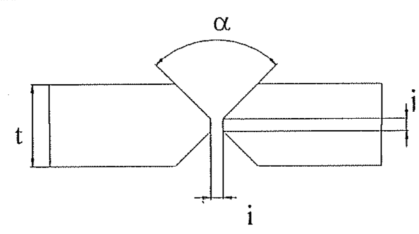
The different welding technologies are as shown in Table 5.2. The weld types are given in Table 5.3.
Using COSTCOMP we have calculated the welding times T_2 (min) in function of weld size a_w (mm) for longitudinal fillet welds (Table A1), for 1/2 V and V butt welds (Table A2), for K and X butt welds (Table A3), for T butt welds (Table A4), for U and double U butt welds (Table A5), in downhand position.

The welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal fillet welds (Table A6) for longitudinal V butt welds (Table A7) in positional welding, which means not downhand, but vertical or overhead positions.

Table 5.2 Applied welding technologies

SMAW	Shielded Metal Arc Welding
SMAW HR	Shielded Metal Arc Welding High Recovery
GMAW-C	Gas Metal Arc Welding with CO_2
GMAW-M	Gas Metal Arc Welding with Mixed Gas
FCAW	Flux Cored Arc Welding
FCAW-MC	Metal Cored Arc Welding
SSFCAW (ISW)	Self Shielded Flux Cored Arc Welding
SAW	Submerged Arc Welding
GTAW	Gas Tungsten Arc Welding

Table 5.3 Different weld types

	**1. Fillet weld** *t=0-15 mm* $a_w \leq 0.7\ t_{min}$
	**2. V butt-weld** *t=4-15 mm* α=40-90° *i=1-2 mm* *j=0-2 mm*
	**3. X butt-weld** *t=10-40 mm* α=40-60° *i=2-3 mm* *j=2-3 mm*

Cost savings can be achieved using a cheaper welding technology, like SAW instead of SMAW or GMAW, if it is possible. Table 6 shows the cost savings for the two different structures and for the five different groups of welding. For welded box beam the cost savings can be 13 %, for stiffened plates the cost savings can be 32 % of the total cost. All compared results are optimized.

5.6.3 Conclusions

a) Cost functions are formulated by means of the COSTCOMP software for longitudinal fillet welds carried out with manual SMAW, semi-automatic GMAW-C and automatic SAW method in downhand position.

b) Using these cost functions the optimal dimensions of a stiffened plate are computed which minimise the total cost and fulfil the design constraints on overall and local buckling.
c) The comparison of optimal solutions shows that significant cost savings may be achieved by using SAW instead of SMAW or GMAW-C.

d) Numerical computations show that the optimal dimensions of a stiffened plate depend on the applied welding method and illustrate the necessity of cooperation between designers and fabricators.

e) Comparison of optimal solutions for minimum weight (k_f/k_m = 0) and minimum cost shows that the fabrication cost affects significantly the optimal dimensions, therefore the consideration of the total cost function results in more economic structural versions.

5.7 OPTIMUM DESIGN OF TUBULAR TRUSSES

5.7.1 Introduction

Authors dealing with the optimum design of metal structures make in some cases simplifications to solve the problems easier. E.g. in the optimization of trusses they neglect the overall buckling of compressed members or use too simple stability constraints such as the Euler buckling curve.
It is well known that the Euler buckling curve neglects the very important effect of initial crookedness and residual stresses caused by fabrication processes (welding, cold-forming). These effects can be described only by a more complicated mathematical form. It will be shown that the use of the Euler buckling curve causes unsafe design, which is not permissible.
Furthermore, the suitable optimum design procedure will be described using all stability constraints necessary for safe design. The case of welded thin-walled tubular trusses is selected for this purpose, in which not only the constraints on overall buckling, but also the

5.5.2 Time for flattening plates

In the catalogue of different companies one can find the times for flattening plates (T_4 [min]) in the function of a plate thickness (t [mm]) and the area of the plate (A_p [mm^2]). The time function can be written in the form:

$$T_4 = \Theta_{de}\left(a_e + b_e t^3 + \frac{1}{a_e t^4}\right)A_p \tag{5.42}$$

where a_e=9.2*10^{-4} [min/mm^2], b_e= 4.15*10^{-7} [min/mm^5], Θ_{de} is the difficulty parameter (Θ_{de}= 1,2 or 3). The difficulty parameter depends on the form of plate.

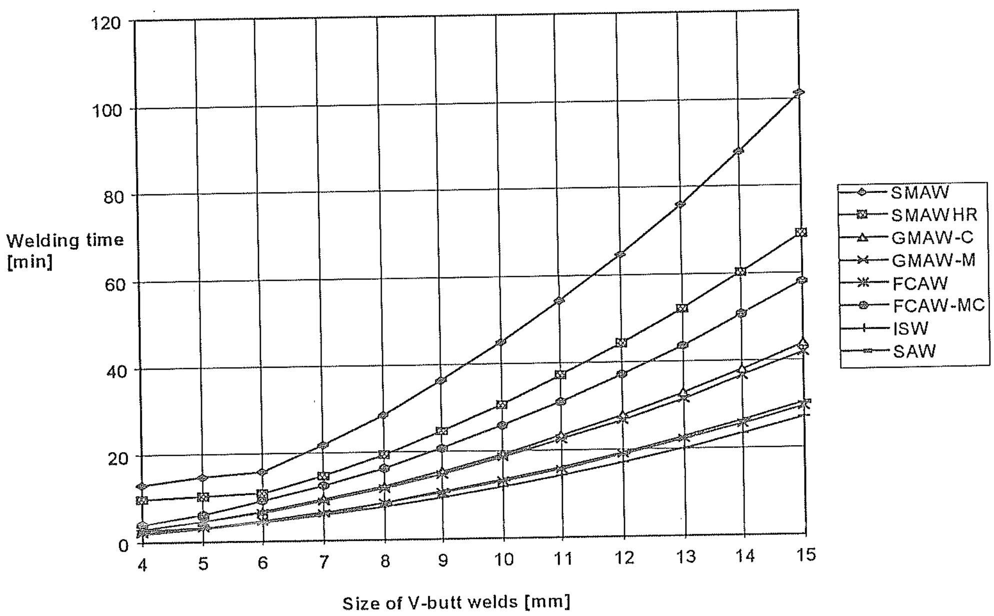


Fig. 5.11 Welding times T_2 (min) in function of weld size a_w (mm) for longitudinal V butt welds downhand position

5.5.3 Surface preparation time

The surface preparation means the surface cleaning, painting costs, ground coat, top coat, sand-spraying, etc.
The surface cleaning time can be defined in the function of the surface area (A_s [mm^2]) as follows:

$$T_5 = \Theta_{ds} a_{sp} A_s \quad (5.43)$$

where $a_{sp} = 3*10^{-6}$ [min/mm^2], Θ_{ds} is a difficulty parameter.

5.5.4 Painting time

The painting means making the ground and the top coat. The painting time can be given in the function of the surface area (A_s [mm^2]) as follows:

$$T_6 = \Theta_{dp}(a_{gc} + a_{tc})A_s \quad (5.44)$$

where $a_{gc} = 3*10^{-6}$ [min/mm^2] , $a_{tc} = 4.15*10^{-6}$ [min/mm^2], Θ_{dp} is a difficulty factor, Θ_{dp}=1,2 or 3 for horizontal, vertical or overhead painting.

5.5.5 Cutting and edge grinding times

The cutting and edge grinding can be made by different technologies, like Acetylene, Stabilized gasmix and Propane with normal and high speed, see Tables A8 and A9.

The cutting time can be calculated also by COSTCOMP. The normal speed acetylene has the highest time and the high speed propane has the smallest cutting time (Fig. 5.12).

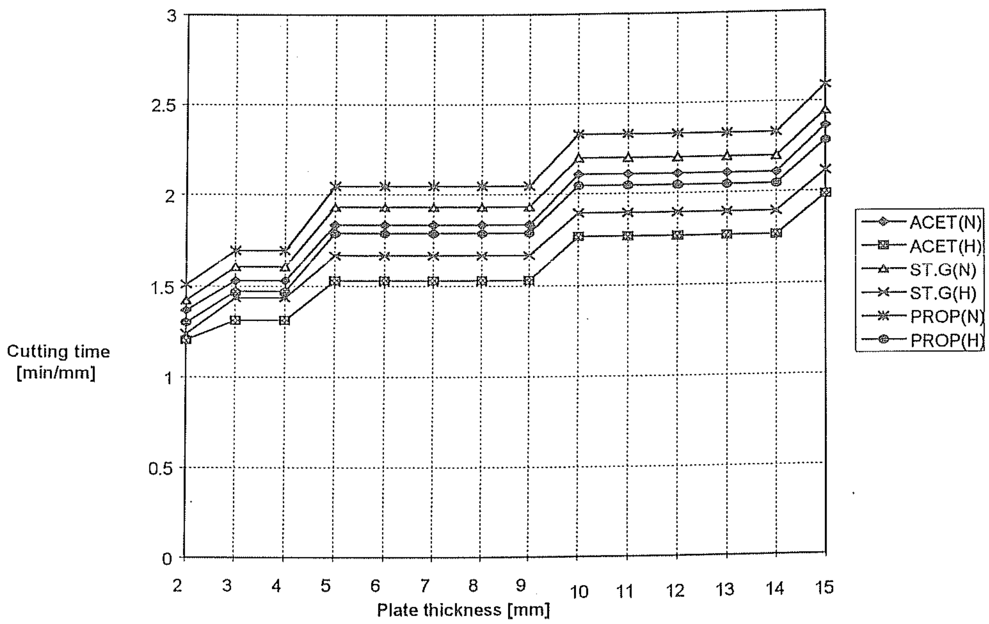


Fig. 5.12 Cutting time of plates in the function of thickness for fillet, T-, V-, 1/2 V butt welds

The cutting cost function can be formulated using Tables A8 and A9 in the function of the thickness (t [mm]) and cutting length (L_c [mm]):

$$T_7 = \sum_i C_{7i} t_i^n L_{ci} \tag{5.45}$$

where t_i the thickness in [mm], L_{ci} is the cutting length in [mm].

5.5.6 Total cost function

The total cost function can be formulated by adding the previous cost functions together.

$$\frac{K}{k_m} = \rho V + \frac{k_f}{k_m}\left(T_1 + T_2 + T_3 + T_4 + T_5 + T_6 + T_7\right) \tag{5.46}$$

Taking k_m = 0.5-1 \$/kg, k_f=0 -1 \$/min. The k_f/k_m ratio varies between 0 - 2 kg/min. If k_f/k_m = 0, then we get the mass minimum. If k_f/k_m = 2.0 it means a very high labour cost (Japan, USA), k_f/k_m = 1.5 and 1.0 means a West European labour cost, k_f/k_m = 0.5 means the labour cost of developing countries.

5.6 NUMERICAL EXAMPLES FOR THE CALCULATION OF COST EFFECTS

We show the cost calculations for two different structures as follows: welded box beams, where the welding cost is smaller and stiffened plates, where the welding cost is larger.

5.6.1 Welded box beam

The beam is simply supported, with uniformly distributed force. Box beam can be a model of welded RHS or CHS tubes. We neglect the effect of transverse diaphragms. The loading of the beam varies in time, the maximum bending moment varies between 0 and M_{max} and the number of cycles is N=2*10^6.
There can be two different weld shapes, such as fillet welds or 1/2 V butt welds.
From the eight different welding technologies we form five groups as follows:

- SMAW,
- SMAW HR,
- FCAW-MC.
- GMAW-C, GMAW-M,
- SAW, SSFCAW (ISW), FCAW,

The welding times in one group are very close to each other.
The cost function to be minimized is, according to Eq. (5.36)

$$\frac{K}{k_m} = \rho LA + \frac{k_f}{k_m}\left(\Theta_d\sqrt{\kappa\rho LA} + 1.3C_2 a_w^n L_w + T_4 + T_5 + T_6 + T_7\right) \tag{5.47}$$

where $A=ht_w+2bt_f$, $\Theta_d = 2$, $\kappa = 4$, $L_w=4L$, ρ=7.85*10^{-6} kg/mm^3.

Constraints according to EUROCODE 3 [5.31]

a) Fatigue constraint

$$\Delta\sigma = \frac{\Delta M}{W_x} \le \frac{\Delta\sigma_c}{\gamma_f} \tag{5.48}$$

where $\Delta M = M_{max}$, $\Delta M = 15*10^8$ Nmm and $\gamma_f = 1.25$ the fatigue safety factor.
For the cycles $N = 2*10^6$ we should choose the stress range $\Delta\sigma_C$ according to the fatigue categories. For longitudinal welds, with start and stop positions (SMAW, GMAW, FCAW) $\Delta\sigma_C = 100$ MPa, while for automated butt welds made from one side with back plates without start and stop positions (SAW) $\Delta\sigma_C = 112$ MPa. The moment of inertia and the section moduli can be calculated as follows

$$I_x = \frac{h^3 t_w}{12} + 2bt_f\left(\frac{h+t_f}{2}\right)^2, \tag{5.49}$$

$$W_x = \frac{I_x}{\left(h+t_f\right)/2} . \tag{5.50}$$

b) Local buckling constraints
For web plates:

$$\frac{t_w}{2} \ge \beta_w h \tag{5.51}$$

$$\beta_w = \frac{1}{124\varepsilon} \tag{5.52}$$

For compressed flanges:

$$t_f \ge \delta_f b \tag{5.53}$$

$$\delta_f = \frac{1}{42\varepsilon} \tag{5.54}$$

In order to avoid too thin flanges we should introduce the following constraint instead of (5.53):

$$t_f \ge 1.2\delta_f b \ . \tag{5.55}$$

In the buckling constraint we should apply the maximal normal stress ($\Delta\sigma$)

$$\varepsilon = \sqrt{\frac{235}{\Delta\sigma / \gamma_f}} \tag{5.56}$$

The unknowns in the optimization are h, $t_w/2$, b and t_f.
The size limits for the unknowns are as follows:

- h = 500 - 1500 mm,
- $t_w/2$ = 5 - 15 mm,
- b = 300 - 1500 mm,
- t_f = 5 - 25 mm.

The elements of cost function for the welded box beam are as follows
Size of welded joint $a_w = t_w/4$ for fillet welds, $a_w = t_w/2$ for 1/2 V butt welds,

Cross section area $A = 2\,h\,t_w/2 + 2\,b\,t_f$
Material cost part $\rho V = \rho LA$
Fabrication costs part $k_f/k_m \sum_i T_i$

$T_1 = C_1 \Theta_d \sqrt{\kappa \rho V}$, where $\rho = 7.85*10^{-6}$, $C_1 = 1$, $\kappa = 4$, $\Theta_d = 2$

$T_2 + T_3 = 1.3 \sum C_{2i} a_{wi}^2 L_{wi}$ where $C_{2i} = 0.7889$ for SMAW, $L_{wi} = 4\,L$

$T_4 = \Theta_{de} \left(a_e + b_e t^3 + \frac{1}{a_e t^4} \right) A_p$ where a_e=9.2*10^{-4}, b_e= 4.15*10^{-7}, $t = t_w/2$, or t_f, A_p $= 2\,hL$ or $2\,bL$

Table 5.4 Optimum rounded sizes of welded box beams in mm with fillet welds and different welding technologies

Welding technology	k_f/k_m	h	$t_w/2$	b	t_f	ρV (kg)	K/k_m (kg)
SMAW	0.0	1420	7	820	12	6211	6211
	0.5	1350	7	825	13	6335	7494
	1.0	1350	7	825	13	6335	8653
	1.5	1295	7	915	13	6581	10076
	2.0	1370	7	810	13	6318	10956
SMAW HR	0.0	1420	7	820	12	6211	6211
	0.5	1350	7	825	13	6335	7335
	1.0	1350	7	825	13	6335	8335
	1.5	1350	7	825	13	6335	9335
	2.0	1370	7	810	13	6318	10320
FCAW-MC	0.0	1420	7	820	12	6211	6211
	0.5	1350	7	825	13	6335	7179
	1.0	1350	7	825	13	6335	8246
	1.5	1350	7	825	13	6335	9168
	2.0	1350	7	825	13	6335	10113
GMAW-C GMAW-M	0.0	1420	7	820	12	6211	6211
	0.5	1355	7	820	13	6326	7198
	1.0	1355	7	820	13	6326	8071
	1.5	1355	7	820	13	6326	8944
	2.0	1355	7	820	13	6326	9816
SAW ISW FCAW	0.0	1420	7	820	12	6211	6211
	0.5	1355	7	820	13	6326	7132
	1.0	1355	7	820	13	6326	7938
	1.5	1355	7	820	13	6326	8744
	2.0	1355	7	820	13	6326	9550

$T_5 = \Theta_{ds} a_{sp} A_s = 5*10^{-7}$ where $a_{sp} = 3*10^{-6}$, $A_s = 2\,hL + 2\,bL$

$T_6 = \Theta_{dp} (a_{gc} + a_{tc}) A_s$ where $a_{gc} = 3*10^{-6}$, $a_{tc} = 4.15*10^{-6}$, $A_s = h\,L + b\,L$

$T_7 = \sum_i C_{7i} t_i^n L_{ci}$ where $C_7 = 1.1388$, $t = t_w/2$ or t_f, $n = 0.25$,

$$L_{ci} = 2(h + L) \text{ or } 2(b + L) \quad (5.57)$$

The optimization is performed by the Rosenbrock's Hillclimb procedure. The computer code run on PC and can calculate both the continuous and discrete values. The discrete optimum sizes of the welded box beams with fillet welds can be found for different welding technologies in Table 5.4.

5.6.2 Welded stiffened plate

The stiffened plates can be a model of stiffened tubes with large diameters, or arches. Since the welding cost is a great part of the total cost, it is economic to optimize these structural components for minimum cost.

The cost function is calculated according to Eq 5.36, where

$A = b_0 t_f + \varphi h_s t_s$, $\delta = 3$, $\kappa = \varphi + 1$, $L_w = 2L\varphi$ and φ is the number of stiffeners.

The stiffeners are welded to the plate by double fillet welds. The welding costs can be calculated for different welding technologies according to Tables A1 - A9.

The main data for the optimization are as follows:

Young modulus of the steel is $E = 2.1*10^5$ MPa, material density is $\rho = 7.85*10^{-6}$ kg/mm^3, Poisson parameter is $\nu = 0.3$, yield stress is $f_y = 235$ MPa, width of the plate is $b_0 = 4200$ mm and the plate length is $L = 4000$ mm.

The compression force is

$$N = f_y b_0 t_{f\,max} = 235 * 4200 * 20 = 1.974 * 10^7 \text{ (N)} \quad (5.58)$$

The independent design variables are as follows: thickness of the plate (t_f), height and thickness of the stiffeners (h_s, t_s) and the number of stiffeners ($\varphi = b_0/a$).

Design constraints

a) Overall buckling design rules, according to API [5.30] for the compressed plate with uniform distance stiffeners.

$$N \le \chi f_y A \quad (5.59)$$

where, χ is the buckling constraints, in the function of the reduced slenderness factor $\bar{\lambda}$:

$$\chi = 1 \quad \text{when} \quad \bar{\lambda} \le 0.5 ,$$
$$\chi = 1.5 - \bar{\lambda} \quad \text{when} \quad 0.5 \le \bar{\lambda} \le 1,$$
$$\chi = \frac{0.5}{\bar{\lambda}} \quad \text{when} \quad \bar{\lambda} \ge 1, \quad (5.60)$$

and

$$\bar{\lambda} = \frac{b_0}{t_f}\sqrt{\frac{12(1-\nu^2)f_y}{E\pi^2 k}}, \quad (5.61)$$

$$k_{\min} = \min(k_F, k_R), \quad (5.62)$$

$$k_R = 4\varphi^2. \quad (5.63)$$

$$k_F = \frac{(1+\alpha^2)^2 + \varphi\gamma}{\alpha^2(1+\varphi\delta_P)} \quad \text{when } \alpha = \frac{L}{b_0} \le 4\sqrt{1+\varphi\gamma} \quad (5.64)$$

and

$$k_F = \frac{2\left(1+\sqrt{1+\varphi\gamma}\right)}{1+\varphi\gamma} \quad \text{when } \alpha \ge 4\sqrt{1+\varphi\gamma} \quad (5.65)$$

where

$$\delta_P = \frac{h_s t_s}{b_0 t_f}, \quad (5.66)$$

$$\gamma = \frac{EI_s}{b_0 D}, \quad (5.67)$$

$$I_s = \frac{h_s^3 t_s}{3}, \quad (5.68)$$

$$D = \frac{Et_f^3}{12(1-\nu^2)}. \quad (5.69)$$

Eq (32) can be rewritten as

$$\gamma = 4(1-\nu^2)\frac{h_s^3 t_s}{b_0 t_f^3} = 3.64\frac{h_s^3 t_s}{b_0 t_f^3}. \quad (5.70)$$

where I_s is the moment of inertia of one stiffener about an axis parallel to the plate surface at the base of the stiffener, D is the torsional stiffness of the main plate.

b) Buckling constraint of the stiffener is:

$$\frac{h_s}{t_s} \le \frac{1}{\beta_s} = 14\sqrt{\frac{235}{f_y}} \quad (5.71)$$

The size constraints for the variables are as follows:

- t_f= 6 - 20 mm,
- h_s= 84 - 280 mm,
- t_s= 6 - 25 mm,
- φ= 4 - 15 mm.

The elements of cost function for the welded stiffened plate are as follows

Size of welded joint $a_w = t_s$

Cross section area $A = b_0 t_f + \varphi h_s t_s$

Material cost $\rho V = \rho L A$

Fabrication costs $k_f/k_m \sum_i T_i$

$T_1 = C_1\Theta_d\sqrt{\kappa\rho V}$, where $\rho = 7.85*10^{-6}$, $C_1 = 1$, $\kappa = \varphi+1$, $\Theta_d = 2$

$T_2 + T_3 = 1.3\sum C_{2i}a_{wi}^2 L_{wi}$ where $C_{2i} = 0.7889$ for SMAW, $L_{wi} = 2\,L\,\varphi$

$T_4 = \Theta_{de}\left(a_e + b_e t^3 + \frac{1}{a_e t^4}\right)A_p$ where $a_e = 9.2*10^{-4}$, $b_e = 4.15*10^{-7}$, $t = t_s$, or t_f,

$A_p = \varphi\, h_s L$ or $b_0 L$

$T_5 = \Theta_{ds}a_{sp}A_s = 5*10^{-7}$ where $a_{sp} = 3*10^{-6}$, $A_s = \varphi\, h_s L + b_0 L$

$T_6 = \Theta_{dp}(a_{gc} + a_{tc})A_s$ where $a_{gc} = 3*10^{-6}$, $a_{tc} = 4.15*10^{-6}$, $A_s = \varphi\, h_s L + b_0 L$

Table 5.5 Optimum rounded sizes of welded stiffened plates in mm with fillet welds using different welding technologies for k_f/k_m=2.0

Welding technology	k_f/k_m	h_s	t_f	φ	t_s	ρV (kg)	K/k_m (kg)
	0.0	210	17	13	11	2737	2737
	0.5	230	17	6	19	3242	6313
SMAW	1.0	235	17	6	19	3258	9409
	1.5	235	17	6	19	3258	12484
	2.0	235	17	6	19	3258	15559
	0.0	210	17	13	11	2737	2737
	0.5	230	17	6	19	3242	5749
SMAW HR	1.0	230	17	6	19	3242	8257
	1.5	230	17	6	19	3242	10764
	2.0	235	17	6	19	3258	13306
	0.0	210	17	13	11	2737	2737
	0.5	230	17	6	19	3242	5553
FCAW-MC	1.0	230	17	6	19	3242	7864
	1.5	230	17	6	19	3242	10175
	2.0	235	17	6	19	3258	12521
	0.0	210	17	13	11	2737	2737
GMAW-C	0.5	230	17	6	19	3242	5299
GMAW-M	1.0	230	17	6	19	3242	7357
	1.5	235	17	6	19	3258	9444
	2.0	230	17	6	19	3242	11471
	0.0	210	17	13	11	2737	2737
SAW	0.5	230	17	6	19	3242	5064
ISW	1.0	230	17	6	19	3242	6886
FCAW	1.5	230	17	6	19	3242	8707
	2.0	235	17	6	19	3258	10564

$$T_7 = \sum_i C_{7i} t_i^n L_{ci} \qquad \text{where } C_7 = 1.1388\,,\ t = t_s \text{ or } t_f,\ n = 0.25,$$

$$L_{ci} = (h_s + L) \text{ or } (b_0 + L) \qquad (5.72)$$

Table 5.5 shows the optimum discrete sizes of the stiffened plate with different welding technologies.

Fig. 5.15 and 5.16 show the distribution of the total cost. The diagrams illustrate that this distribution depends on the welding technologies, the type of welding, the ratio of material and fabrication specific costs and the structure type too.

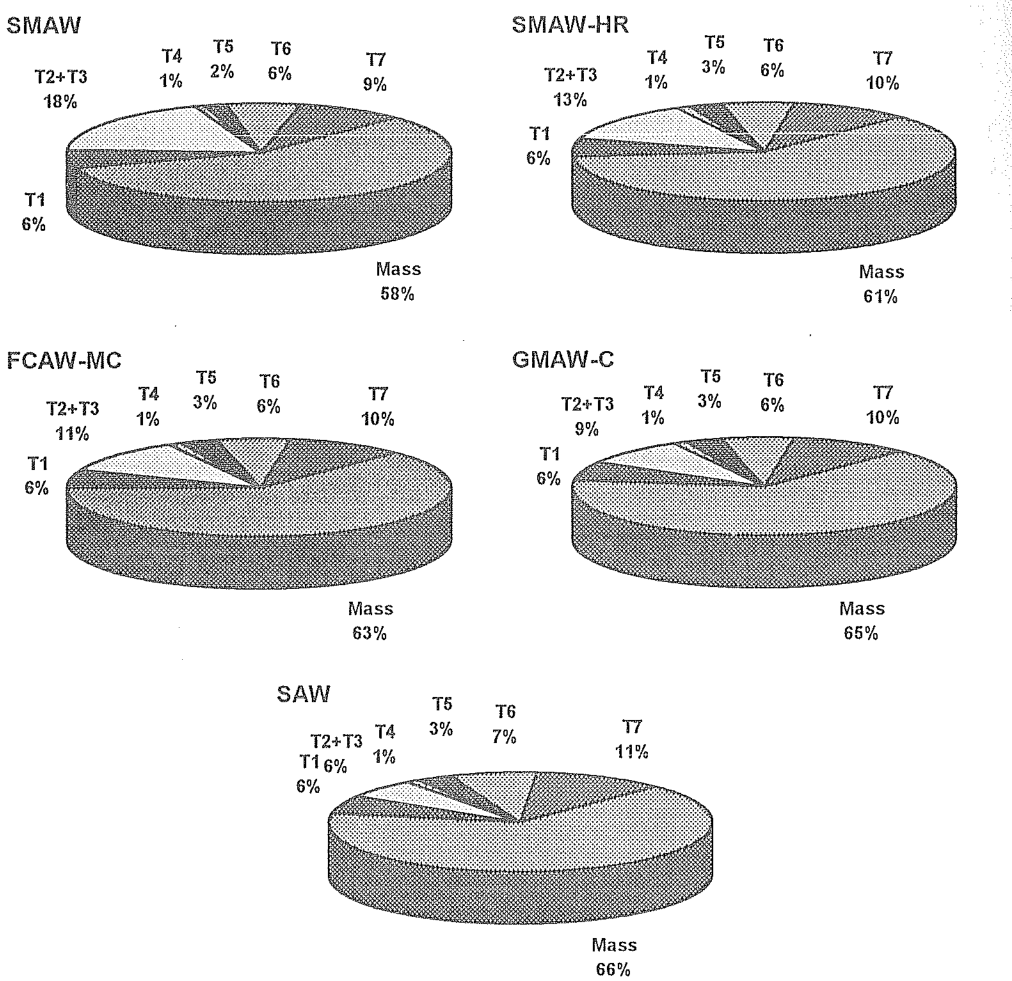


Fig. 5.13 The total cost distribution of the welded box beam with fillet welds using different welding technologies for k_f/k_m=2.0

The welding technologies in Figures 5.15 and 5.16 are given in decreasing order relating to the welding time and cost. The differences are great among them. The welding time and cost is the greatest for SMAW, the quickest and cheapest are the SAW, FCAW and ISW. For stiffened plates using SMAW 46% of the total cost is the welding cost, using SAW, this is only 20%.

Comparing the two structures the fabrication costs of stiffened plates are larger, than those of welded box beams. The reason is that stiffened plates contain more elements, which need more welding.

In the case of welded box beam (Table 5.4), using k_f/k_m=2.0 ratio, for SMAW the mass of the structure is ρLA = 6318 kg. The fabrication cost is 100 (10956-6318) / 10956 = 43 % of the total cost. The mass of stiffened plate is ρLA = 3258 kg (Table 4), the fabrication cost is 100 (15559-3258) / 15559 = 79 % of the total cost.

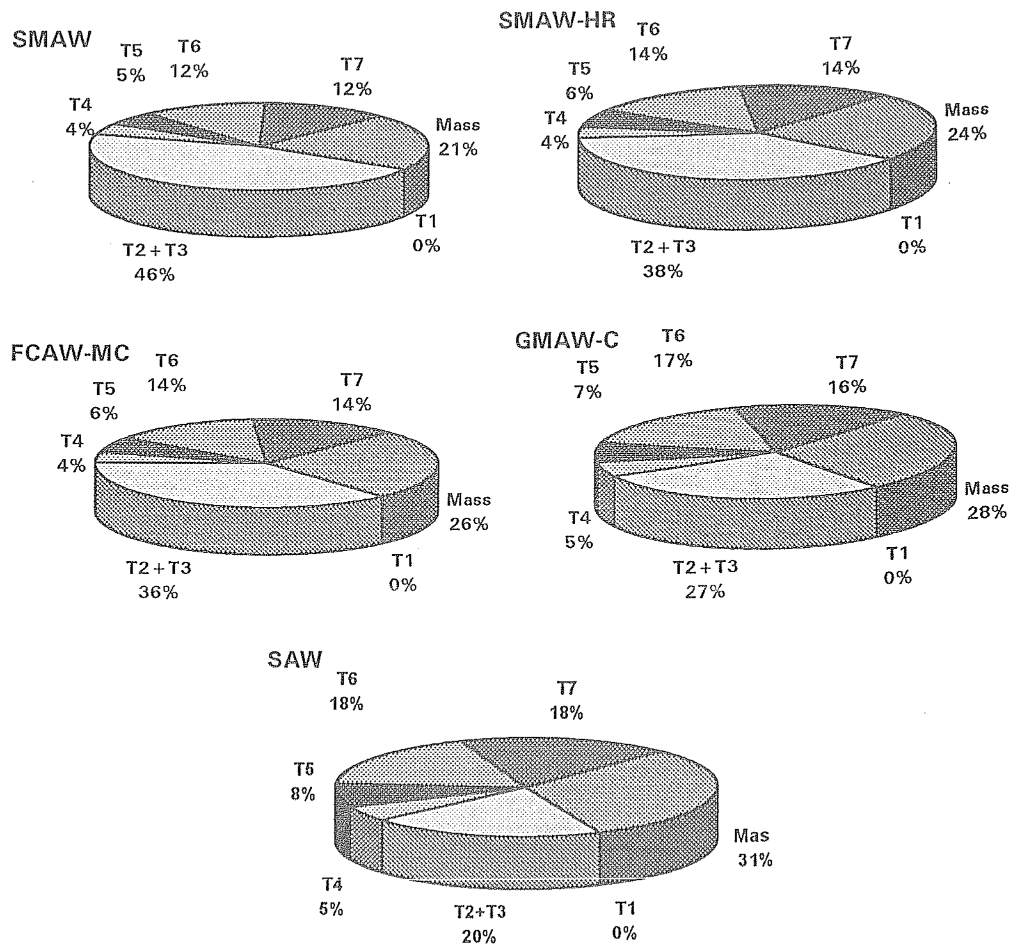
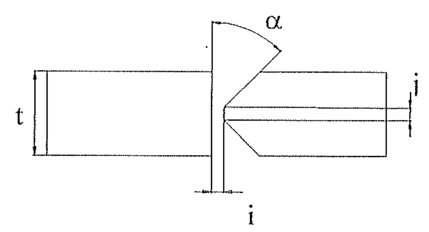
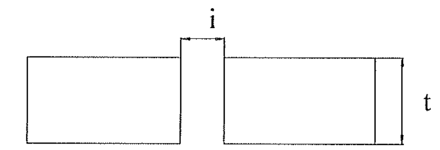
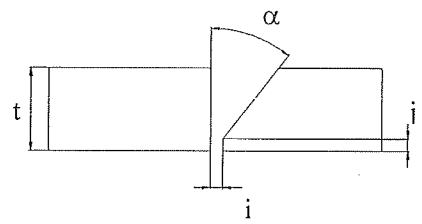
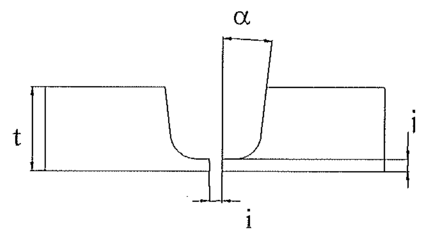
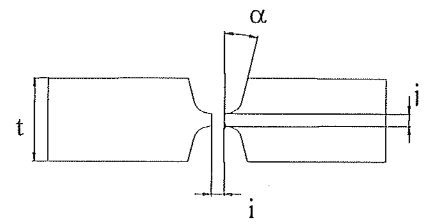


Fig. 5.14 The total cost distribution of the welded stiffened plate with fillet welds using different welding technologies for k_f/k_m=2.0

	**4. K butt-weld** t=10-40 mm α=40-60° i=0-3 mm j=2-3 mm
	**5. T butt-weld** t=2-8 mm $i=t/2$
	**6. 1/2 V butt-weld** t=4-15 mm α=40-60° i=0-2 mm j=0-2 mm
	**7. U butt-weld** t=20-40 mm α=10-20° i=2-3 mm j=2-3 mm
	**8. Double U butt-weld** t=20-40 mm α=10-20° i=2-3 mm j=2-3 mm

In Fig. 5.11 and Tables in Appendix data are given for eight welding techniques and for different weld types.

Fig. 5.11 shows, that the welding times for longitudinal V butt welds in decreasing order is the highest for SMAW, SMAW-HR, GMAW-C, GMAW-M, FCAW, FCAW-MC, ISW and the lowest for SAW. The order is the same for different weld types (Tables A1 - A7).

constraints on local buckling of plate elements should be considered. The consideration of all important constraints will be illustrated by a numerical example of a simple tubular truss welded from CHS rods.

Table 5.6 Cost savings using different welding technologies

Welding technology k_f/k_m=2.0	Welded box beam		Stiffened plate	
	Total cost	Cost savings in %	Total cost	Cost savings in %
SMAW	10956	0	15559	0
SMAW-HR	10320	6	13305	14
FCAW-MC	10113	8	12521	20
GMAW-C	9816	10	11471	27
SAW	9550	13	10560	32

5.7.2 Numerical example of a tubular truss

In order to illustrate the role of stability constraints we select a simple planar, statically determinate, K-type truss with parallel chords and gap joints, welded from CHS rods (Fig. 5.17). In the optimum design the optimal distance of chords h is sought which minimises the total volume of the structure and the dimensions of rods fulfil the design constraints. The structural members are divided to 4 groups of equal cross-section as follows: 1 - lower chord, 2 - upper chord, 3 - compression braces, 4 - tension braces.

According to DIN 2448 [5.32] and DIN 2458 [5.33] the available CHS have the following dimensions (discrete values):

d = 133, 139.7, 152.4, 159, 168.3, 177.8, 193.7, 219.1, 244.5, 273, 298.5, 323.9
t = 2.9, 3.2, 3.6, 4, 4.5, 5, 5.6, 6.3, 7.1, 8, 8.8, 10.

All members are made from steel Fe 510 with ultimate strength f_u = 510 MPa and yield stress f_y = 355 MPa.
The load is shown in Fig. 5.17, the factored value of the static forces is F = 200 kN. Calculate the required cross-sections for various values of $\omega = h/a_o$ to select the ω_{opt} which minimises the total volume V. The numbering relates to groups of members of equal cross-section. The variables are as follows: d_i and t_i (i=1,2,3,4). The objective function is expressed as

$$\frac{V}{2\pi a_o} = 5(d_1 - t_1)t_1 + 4(d_2 - t_2)t_2 + 3\sqrt{\omega^2 + 1}(d_3 - t_3)t_3 + 2\sqrt{\omega^2 + 1}(d_4 - t_4)t_4 \quad (5.73)$$

The constraints are as follows.

Local buckling constraints for all sections according to Wardenier et al. [5.34] are

$$d_i / t_i \le 50 \quad (5.74)$$

Stress constraint for tension members are

$$\frac{S_{1max}}{\pi(d_1-t_1)t_1} \le \frac{f_y}{\gamma_{Mo}}; \quad S_{1max} = \frac{6.5F}{\omega}; \quad \gamma_{Mo} = 1.1 \tag{5.75}$$

$$\frac{S_{4max}}{\pi(d_4-t_4)t_4} \le \frac{f_y}{\gamma_{Mo}}; \quad S_{4max} = \frac{1.5F}{\omega}\sqrt{\omega^2+1} \tag{5.76}$$

Overall buckling constraints for compression members according to EC 3. are as follows

Upper chord: $$\frac{S_{2max}}{\pi(d_2-t_2)t_2} \le \frac{\chi_2 f_y}{\gamma_{M1}}; \quad S_{2max} = \frac{6F}{\omega}; \quad \gamma_{M1} = 1.1 \tag{5.77}$$

$$\chi_2 = \frac{1}{\phi_2 + \sqrt{\phi_2^2 - \bar{\lambda}_2^2}}; \quad \phi_2 = 0.5\left[1 + 0.34\left(\bar{\lambda}_2 - 0.2\right) + \bar{\lambda}_2^2\right] \tag{5.78}$$

$$\bar{\lambda}_2 = \frac{\lambda_2}{\lambda_E} = \frac{K_2 L_2}{\lambda_E r_2} = \frac{0.9 * 2a_o\sqrt{8}}{\lambda_E(d_2-t_2)}. \tag{5.79}$$

With $E = 2.1 \cdot 10^5$ MPa and $f_y = 355$ MPa $\lambda_E = \pi\sqrt{E/f_y} = 76.4091$.

$K_2 = 0.9$ is the end restraint factor according to Rondal et al. [5.35], $r_2 = (d_2 - t_2) / \sqrt{8}$ is the radius of gyration.

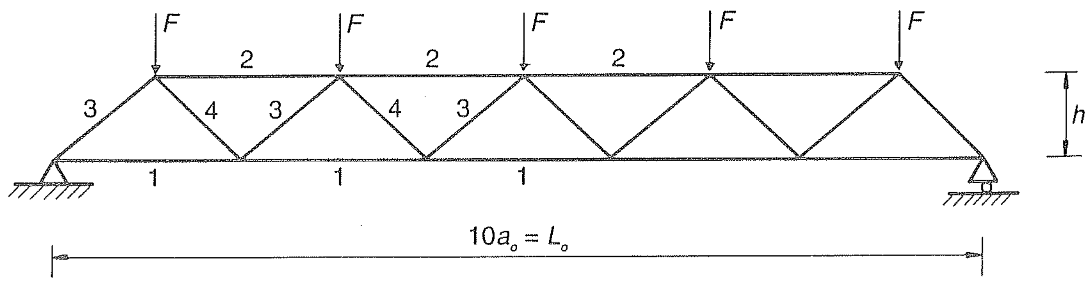


Fig. 5.17 Planar truss with parallel chords

Compression braces:

$$\frac{S_{3max}}{\pi(d_3-t_3)t_3} \le \frac{\chi_3 f_y}{\gamma_{M1}}; \quad S_{3max} = \frac{2.5F}{\omega}\sqrt{\omega^2+1} \tag{5.80}$$

$$\chi_3 = \frac{1}{\phi_3 + \sqrt{\phi_3^2 - \bar{\lambda}_3^2}}; \quad \phi_3 = 0.5\left[1 + 0.34\left(\bar{\lambda}_3 - 0.2\right) + \bar{\lambda}_3^2\right] \tag{5.81}$$

$$\bar{\lambda}_3 = \frac{\lambda_3}{\lambda_E} = \frac{K_3 L_3}{\lambda_E r_3} = \frac{0.75a_o\sqrt{\omega^2+1}\sqrt{8}}{\lambda_E(d_3-t_3)} \tag{5.82}$$

In order to ease the fabrication the diameter of braces should be smaller, than those of chords:

$$d_3 = 0{,}92d_1;\ \ d_3 \le 0.92d_2;\ \ d_4 \le 0.92d_1;\ \ d_4 \le 0.92d_2 \quad (5.83)$$

Prescription for the joint eccentricity to avoid too large additional bending moment in the vicinity of nodes is as follows (Fig. 5.18):

$$e \le 0.25d_1;\ \ e \le 0.25d_2 \quad (5.84)$$

The eccentricity can be expressed by d_i, angle θ and gap parts g_3 and g_4 as follows:

$$\tan\theta = \frac{e + d_1/2}{g_3 + d_3/(2\sin\theta)} \quad or \quad \tan\theta = \frac{e + d_1/2}{g_4 + d_4/(2\sin\theta)} \quad (5.85)$$

Assuming that

$$g_3 = g_4 = 0.05\,d_1 \ \text{ or } \ 0.05\,d_2 \quad (5.86)$$

the geometry constraints can be given by: $\frac{d_3}{2}\sqrt{\omega^2+1} + d_1(0.05\omega - 0.75) \le 0$ (5.87)

and

$$\frac{d_3}{2}\sqrt{\omega^2+1} + d_2(0.05\omega - 0.75) \le 0 \quad (5.88)$$

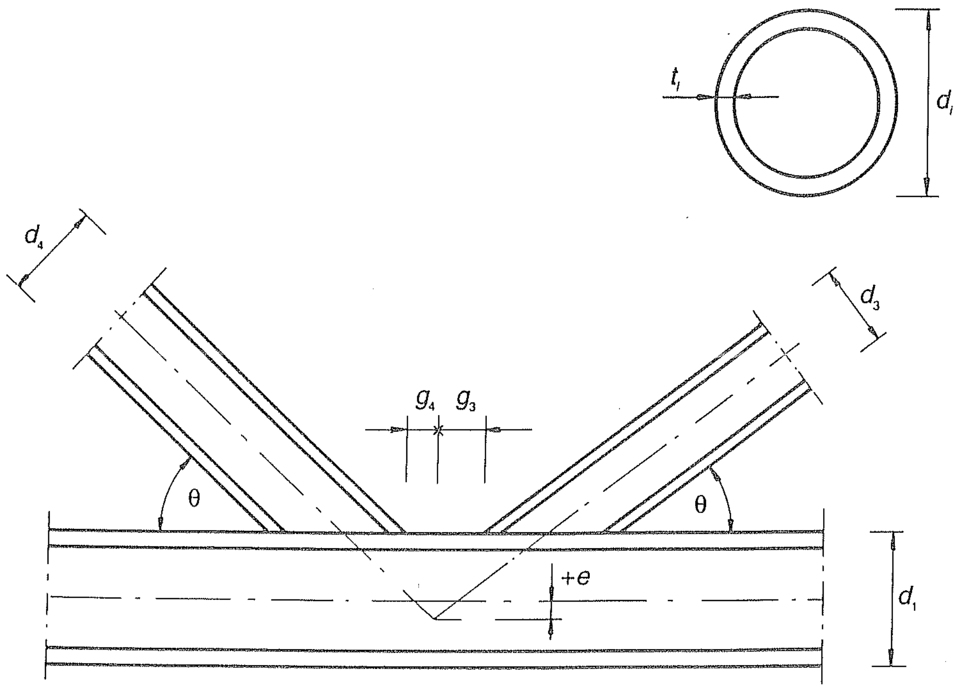


Fig. 5.18 K-type gap joint with eccentricity e

Constraint on static strength of welded joints between chords and braces according to EC 3 is

$$\sqrt{\sigma_{\perp}^2+3\left(\tau_{\perp}^2+\tau_{II}^2\right)} \leq f_u/\left(\beta_w \gamma_{MW}\right) \tag{5.89}$$

$f_u = 510\ MPa, \quad \beta_w = 0.9, \quad \gamma_{Mw} = 1.25.$

From the force S in a brace the following stress components arise in welds:

$$\sigma_{\perp} = \tau_{\perp} = \frac{S\sin\theta}{\pi d a_w} \cdot \frac{\sqrt{2}}{2}; \quad \tau_{II} = \frac{S\cos\theta}{\pi d a_w} \tag{5.90}$$

where a_w is the fillet weld dimension. Substituting Eq. (5.90) into Eq. (5.89) we get

$$\frac{S}{\pi d a_w}\sqrt{\frac{2\omega^2+3}{\omega^2+1}} \leq 453 \text{ MPa} \tag{5.91}$$

For the maximal value of a_w the corresponding brace thickness can be taken. This constraint should be fulfilled for S_3 and S_4.

For the node strength the following constraints should be fulfilled (Wardenier et al. [5.34]).

Constraints on chord plastification.

In the joint of rods 1 and 3:

$$S_{3max} \leq S_{31}^* = \frac{f_y t_1^2}{\sin\theta}\left(1.8+10.2\frac{d_3}{d_1}\right) f_1\left(\gamma_1, g_1'\right) \tag{5.92}$$

$$f_1\left(\gamma_1, g_1'\right) = \gamma_1^{0.2}\left[1+\frac{0.024\gamma_1^{1.2}}{exp\left(0.5g_1'-1.33\right)+1}\right]; \quad \gamma_1 = \frac{d_1}{2t_1} \tag{5.93}$$

$$g_1' = g_1/t_1; \quad \textit{we assume that} \quad g_1 = g_3 + g_4 = 0.1 d_1 \tag{5.94}$$

Constraints on chord plastification for joints of rods 1 - 4, 2 - 3 and 2 - 4 can be formulated similarly to Eq. (5.92-5.94), therefore these constraints are not detailed here.

Constraints on punching shear.

In the joint of rods 2 and 3:

$$S_{3max} \leq \frac{f_y}{\sqrt{3}} t_2 \pi d_3 \frac{1+\sin\theta}{2\sin^2\theta} \tag{5.95}$$

Note that the constraint on punching shear was in our calculations always passive, so it is not necessary to investigate it for other joints.

For the computations the Rosenbrock's hillclimb mathematical programming method has been used treating the unknowns as continuous variables. After the determination of the optimal dimensions the discrete optima have been found by using an additional search. The results are summarized in Table 5.7.

The optimal value is $\omega = 1.1$, the difference between the best and worst solution in the range of $\omega = 0.8 - 1.4$ is 100/(28704 - 21063)/21063 = 36%. The checks of constraints are summarized in Table 5.8.

Table 5.7 Optimal discrete dimensions [mm] and $V/(2\pi a_0)$ - values [mm^2] for various $\omega = h/a_0$ - values.

$\omega = h/a_0$	0.8	0.9	1.0	1.1	1.2	1.3	1.4
d_1/t_1	244.5/8	244.5/8	244.5/8	219.1/8	273/8	273/8	298.5/8.8
d_2/t_2	273/8	244.5/8	244.5/8	219.1/8.8	273/8	273/8	298.5/8.8
d_3/t_3	219.1/4.5	219.1/4.5	219.1/4.5	193.7/4.5	219.1/4.5	219.1/4.5	293.7/4.5
d_4/t_4	159/3.6	152.4/3.6	152.4/3.2	152.4/3.2	139.7/3.2	139.7/3.2	139.7/2.9
$V/(2\pi a_0)$	23083	22367	22475	21063	24970	25264	28704

It can be seen that the overall buckling constraint is always active, the local buckling constraint is passive only for chord 2, since for thickness t_2 the chord plastification is governing. Thus, it can be stated that the effect of stability constraints in the optimum design of tubular trusses is significant.

5.7.3 Conclusions

It is shown that the use of the Euler buckling curve instead of the EC 3 overall buckling formula causes 19 - 35% error in the unsafe side in the most important slenderness range of 38 - 89, so it should not be used in the optimization of tubular trusses. The application of limiting tube local slenderness $d/t = 10$ instead of 50 leads to uneconomic solutions.

Table 5.8 Check of the constraints for the optimal solution $\omega = 1.1$

Constraint	Dimen-sion	Eq. (4.7)	Rod 1	Rod 2	Rod 3	Rod 4	Remarks
Local buckling	-	(13)	27<50	25<50	43<50	48<50	active for rods 3, 4
Tensile stress	MPa	(14) (15)	223<323	-	-	270<323	near active for rod 4
Overall buckling	MPa	(16) (17)	-	188<204	240<261	-	active for rods 2, 3
Fabrication	mm	(18)	-	-	194<202	152<202	active for rod 3
Eccen-tricity	mm	(22) (23)	-	-	-8.32	-	near active for rods 1,2,3
Weld strength	MPa	(26)	-	-	368<453	414<453	near active for rod 4
Chord plasti-fication	kN	(27)	-	-	642<713	405<586	active for rods 3-1
Punching shear	kN	(28)	-	-	642<1744	-	passive

The significant role of the stability constraints in the optimum design of tubular trusses is illustrated by a numerical example. In this optimum design procedure the dimensions of CHS truss members and the optimal distance of chords are determined which give the minimum volume (weight) of the structure and fulfil the design constraints. The constraints relate to the overall buckling of compression members, to the joint eccentricity and static strength of joints. For the final optimal version realistic available discrete tube dimensions are determined.

5.7.4 Design of a roof truss

The design method described in Chapter 6 is applied for compression members of a statically determinate roof truss with non-parallel chords to illustrate the savings in weight in the case of trusses by using CHS or SHS instead of double-angle sections.
Consider the truss shown in Fig. 5.19. Four different cross-sections (1-4) are designed for each case. To find the optimal truss height (h) or the optimal slope angle of the upper chord, the truss is designed for heights h=2.5, 3.5, 4.5, 6.0 and 7.5 m (corresponding slope angles are $(4.76°, 9.46°, 14.04°, 20.56°\ and\ 26.56°)$.

In the design of CHS and SHS struts, section properties of the ISO/DIS 4019.2 as well as the tables given by Dutta and Würker [5.36] (DIN 2448, DIN 2458, DIN 59411) have been used. The results of the calculations are summarized in Table 5.19 and Fig.5.9.

Table 5.9 Total volumes of trusses of various heights

Height h (m)	Slope angle βo	Section	Upper chord 1	Lower chord 2	Outside columns 3	Braces 4	Total volume 0^{-7} (mm^3)
		CHS	152.4/2.9	133/3.2	108/2.3	108/2.3	10.72
2.5	4.76	SHS	115/3.2	110/3	70/3.2	70/3.2	11.04
		angles	2x80x8	2x50x7	2x50x6	2x70x6	18.14
		CHS	152.4/2.3	139.7/2.3	101.6/2	101.6/2	9.24
3.5	9.46	SHS	115/2.6	80/3.2	70/2.6	70/2.6	9.71
		angles	2x70x7	2x50x5	2x50x6	2x55x6	15.36
		CHS	139.7/2	127/2	101.6/2	101.6/2	8.95
4.5	14.04	SHS	90/3	80/2.6	70/2.6	70/2.6	9.80
		angles	2x65x7	2x45x5	2x50x6	2x60x6	17.18
		CHS	127/2	101.6/2	101.6/2	88.9/2	8.79
6.0	20.56	SHS	90/2.6	90/2	70/2.6	70/2.6	10.45
		angles	2x70x6	2x40x4	2x50x6	2x60x6	18.87
7.5	26.56	CHS	114.3/2	88.9/2.3	101.6/2	88.9/1.8	8.84

It can be seen that the optimal truss height (slope angle) giving the minimal total volume of the structure depends on the cross-sectional shape. In the investigated numerical example the optimal slope angles are as follows:

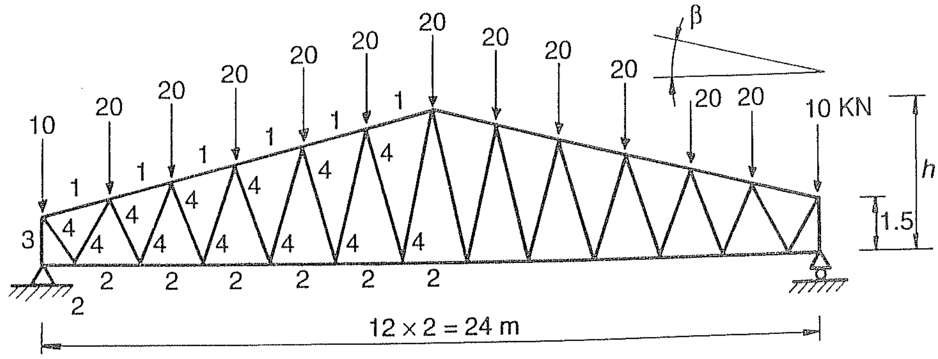


Fig. 5.9 Numerical example of a roof truss. 1 - section for upper chord, 2 - section for lower chord, 3 - section for outside columns, 4 - section for braces. The height h varies with the slope angle β of the upper chord.

For double-angle sections
$\beta \cong 10°$, for SHS $\cong 12°$ and for CHS $\cong 20°$.

The savings in weight by using CHS or SHS instead of double-angle sections are 41-53 % or 39-45 %, respectively e.g. 100 (18.14 - 10.72 / 18.14) = 41 % etc.

These differences are larger than the difference between the material costs of CHS, SHS and rolled angles, thus material cost savings can also be achieved.

Note that the sensitivity of the volume functions for CHS and SHS is relatively small, but the difference between the volumes for the heights h = 2.5 and the optimal h_{opt} = 6 m (for CHS) is 100 (10.72 - 8.79 / 10.72) = 18 %, so, for economic design, it is important to choose the optimal truss slope.

5.7.5 Conclusions

The overall buckling strength of concentrically compressed CHS and SHS struts is much larger than that of double-angle section struts, therefore significant savings in weight and material cost can be achieved by using CHS and SHS instead of double-angle sections in compressed struts and trusses.
By using the limiting local slendernesses and the relationships between the radius of gyration and cross-sectional area, design diagrams are given for the calculation of the required cross-sectional area in function of the compressive force and strut length.
The illustrative numerical example of a roof truss, constructed from CHS, SHS or double-angle sections shows that the optimal geometry of the truss depends on the cross-sectional

shape of compression members. This conclusion is important, since this aspect has not been pointed out till now in the optimum design of trusses.

5.8 EXPERT SYSTEMS

5.8.1 Introduction

Computer programs using AI techniques to assist people in solving difficult problems involving knowledge, heuristics and decision-making are called expert systems. Artificial Intelligence techniques are the best utilized in identifying and evaluating design alternatives and their relevant constraints while leaving the important design decisions to the human designer. The emerging fields of AI and knowledge engineering offer means to carry out qualitative reasoning on computers. There were some attempts to connect the expert systems and structural optimization [5.37]. One of them is an expert system for finding the optimum geometry of steel bridges [5.38].
The connection between single- and multiobjective optimization made it possible in the structural optimization to form a decision support system. In the multiobjective optimization several so called weighting coefficients serve for the designer to give relative importance of the objective functions [5.11,5.12]. The decision support systems (DSS) and the expert systems (ES) are close together, but it is necessary to build an inference engine. The key concept in our approach is to give the user control of important design decisions. Therefore, our approach in applying AI to engineering design is to use AI techniques for keeping track of all design alternatives and constraints, for evaluating the performance of the proposed design by means of a numerical model, and for helping to formulate the optimization problem.

The human designer evaluates the information and advices given by the computer, assesses whether significant constraints or alternatives have been overlooked, decides on alternatives, and makes relevant design decisions.
Depending on the application, an expert system can perform ten type of projects as follows: interpretation, prediction, diagnosis, design, planning, monitoring, debugging, repair, instruction, control. We have used the expert systems for design of structures.

5.8.2 Components of an expert system

The three basic components of an expert system are
- the knowledge base,
- the inference engine,
- the user interface.

There are three main streams in expert systems
- rule-based expert systems can be backward or forward chaining,
- object-oriented systems,

- hybrid systems, which combine object-oriented techniques with rule-based ones (Harmon 1990), (Dym 1991), (Garrett 1990).
- EDA/SQL interface to relational and non-relational databases,
- Rdb/SQL interface to VAX RDB/VMS databases, and
- own worksheet handling system (similar to LOTUS 123).

5.8.3 Overview of Personal Consultant Easy [5.39]

EASY is an EMYCIN-like program developed by Texas Instruments to run on PC-s. Facts are represented as object-attribute-value triplets with accompanying confidence factors. Production rules represent heuristic knowledge. EASY can build systems of up to about 400 rules. A rule tests the value of an O-A-V fact and concludes about other facts. The inference engine is a simple back-chainer.
Control is governed primarily by the order of clauses in the rules. Uncertain information is marked by confidence factors ranging from 0 to 100. EASY accepts unknown as an answer to its questions and continues to reason with available information. Explanation facilities in the program as well as trace functions are used for knowledge base debugging. EASY uses questions to prompt the designer to enter the initial information into a knowledge base. The tool provides several programming aids for debugging.
EASY is implemented in IQLISP. Sources of data can be other language programs or procedures such as FORTRAN, C, C++, data bases such as dBase, LOTUS. The program has some graphics functions as well (DR HALO). The tool uses an Abbreviated Rule Language (ARL) to write the rules.

5.8.4 Overview of Level 5 Object

LEVEL 5 OBJECT (LO5) [5.40] is an object-oriented expert system development and delivery environment. It provides an interactive, windows-based user interface integrated with Production Rule Language (PRL), the development language used to create L5O knowledge bases. The PRL Syntax Section provides syntax diagrams to follow logically when writing a knowledge base. System classes are automatically built by L5O when a new knowledge base is created, thereby providing built-in logic and object tools. The developer can use system classes in their default states or customize them. In this way, the developer can control devices, files, database interactions and the inferencing and windowing environments.
The most remarkable tools of LO5 are:
- object oriented programming (OOP),
- relational database handling (RDB),
- computer aided software engineering (CASE) and
- graphical development system.

The most remarkable tools of LO5 for IBM compatible PCs are:
- Microsoft Windows,
- programming with an object-oriented language (Borland C++),

- direct connection with dBase,
- direct connection with the fourth generation FOCUS data handling system, offers means to carry out qualitative reasoning on computers. Advanced programs that can solve a variety of new problems based on stored knowledge without being reprogrammed, are called knowledge-based systems. If their level of competence approaches that of human experts, they become expert systems, which is the popular name for all knowledge systems, even if they do not deserve the name.

AI techniques provide powerful symbolic computation and reasoning facilities that accommodate intuitive knowledge used by experienced designer. AI techniques, knowledge engineering in particular, can be used in conjunction with numerical programs to serve as an interface between the alternatives and constraints and the designer. AI should be used in the following context [5.41].

- to track the available design alternatives and relevant constraints and to infer candidate modifications in order to improve the design,
- to observe the relationship - intuitive or numerical - between specifications and decision variables, and to give advice on how to formulate the problem for optimization, in particular, to identify the limiting constraints and specifications.

Using LO5 there are two ways of developing programmes: they can be generated either by word processors or in the developing environment. Taking these capabilities into account, L5O was found suitable for development of expert systems for structural engineering.

There are a great number of expert shells available such as ART (Automated Reasoning Tool, Inference Corporation), KEE (Intellicorp), Intelligence Compiler (Intelligence Ware Inc.), Symbologic Adept (Symbologic Corporation), GURU (Micro Data Base Systems), etc. They are available on APOLLO or SUN workstations or on PC-s [5.42].

We have developed the optimization package on PC and we have found the previously described two softwares to be efficient expert shells, so we have made our development using these tools.

The aim was to develop an expert system, which is able to find the optimum version of belt-conveyor bridges due to different geometry, loading, steel grades and design codes. The different variants can be seen in Fig. 5.20. The truss structures can be constructed with four or three chords. The belt-conveyor can be placed on or in the bridge. Instead of a truss structure a tube or a stiffened shell can be used as the main girder.

The total number of variants is about 14000 and it can be increased if we take into account other aspects and constraints in a modular way.

The decision support system, which was connected to the expert one, contains five various single-objective and seven various multiobjective optimization techniques. These techniques are able to solve nonlinear optimization problems with practical nonlinear inequality constraints.

The DSS could contain finite element procedures to compute the mechanical behaviour of the structures. The DSS is described in [5.23].

We have investigated only some variations and our aim is to build the program according to Fig. 5.20 in all details.

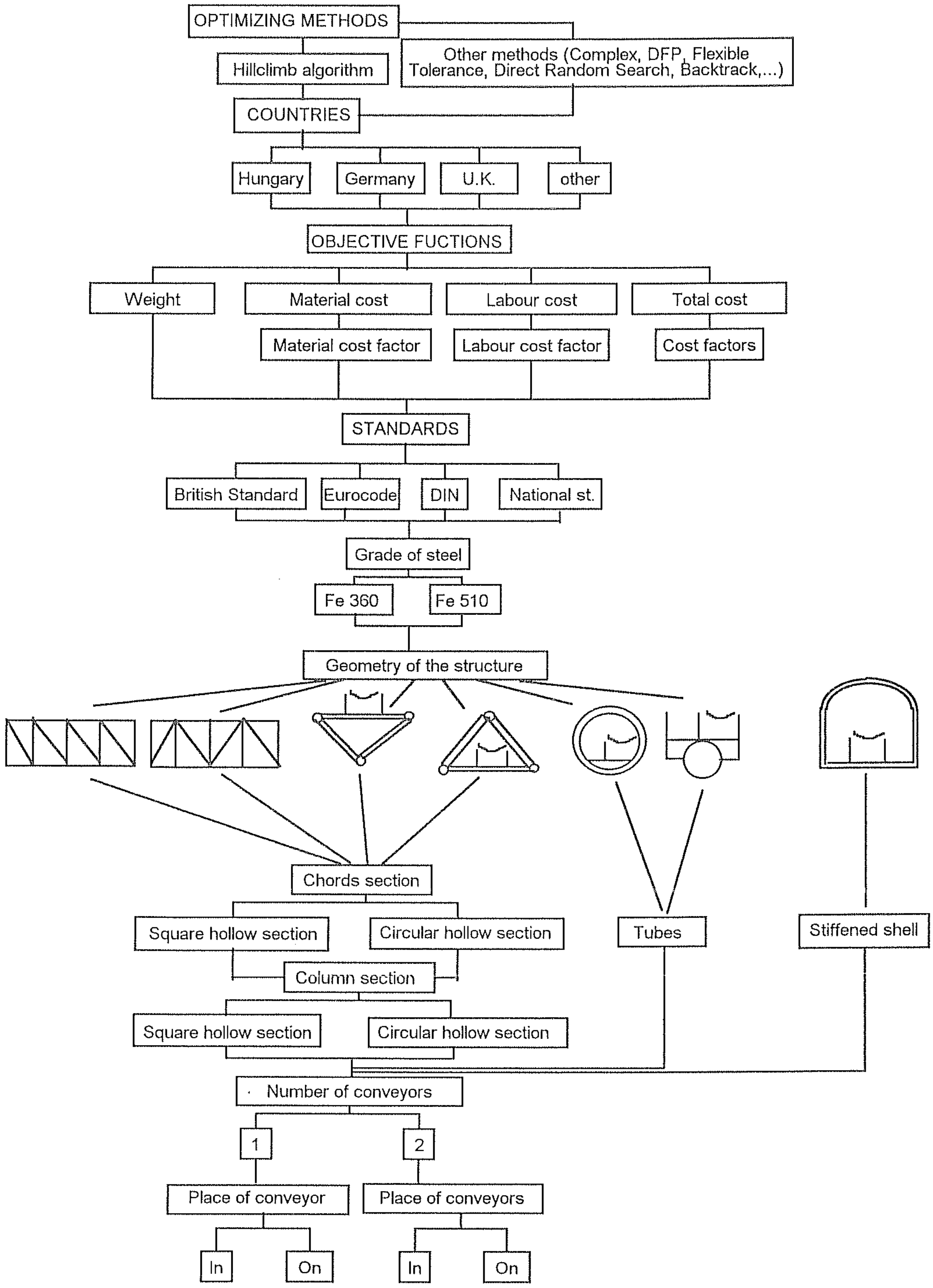


Fig. 5.20 Logical construction of the expert system in LO5 for belt-conveyor bridges

5.8.5 Numerical example using expert system for topology optimization

To illustrate numerically the effect of some structural parameters on the minimum weight design of tubular trusses for belt-conveyor bridges, the optimum topology is sought for a simply supported N-type truss (Fig. 5.21). The belt-conveyor is placed inside the bridge. The total volumes of the planar main truss girder are calculated for three values of node distance 'a'.
The span length is kept constant $L_o = 30$ m. For each 'a' the ratio $\omega = h/a$ is varied and the optimal ω giving the minimum volume is determined.
The loads are as follows:
Uniformly distributed vertical loads for a main truss girder:

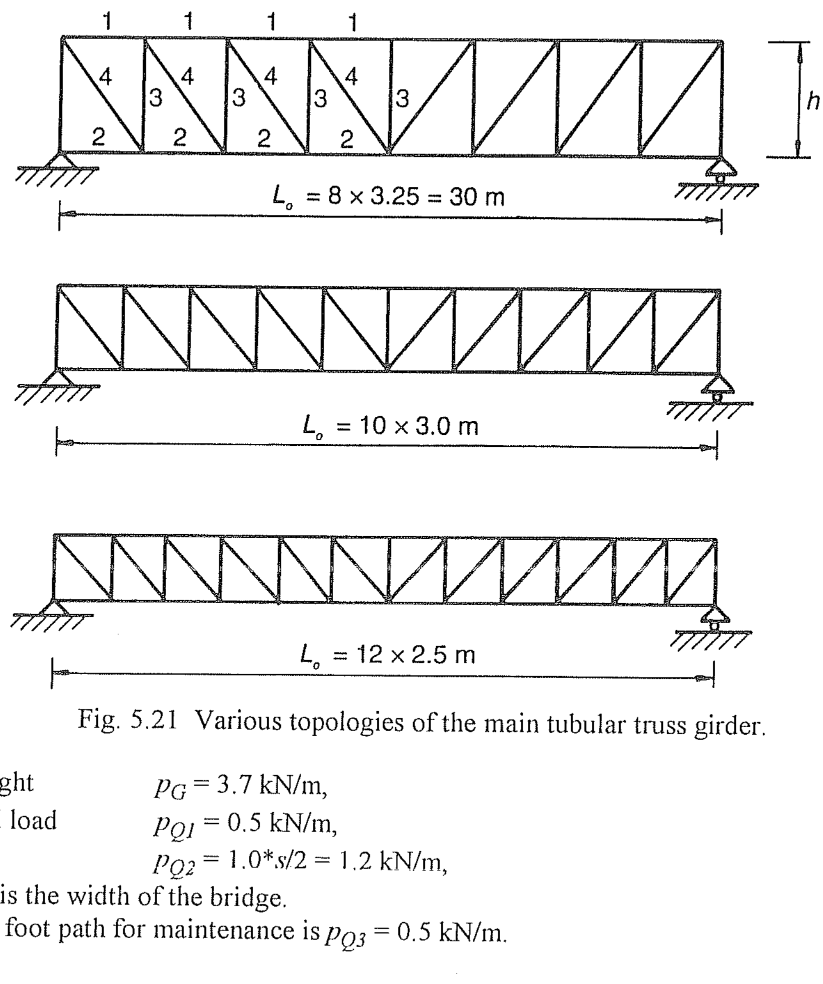


Fig. 5.21 Various topologies of the main tubular truss girder.

self-weight $p_G = 3.7$ kN/m,
imposed load $p_{Q1} = 0.5$ kN/m,
snow $p_{Q2} = 1.0 * s/2 = 1.2$ kN/m,
where s is the width of the bridge.
Load on foot path for maintenance is $p_{Q3} = 0.5$ kN/m.

Factored vertical load with safety factors according to the Eurocode 3 is

$p_v = \gamma_G p_G + 0.9\ \gamma_Q\ (p_{Q1} + p_{Q2} + p_{Q3}) = 1.35*3.7 + 0.9*1.5\ (0.5 + 1.2 + 0.5) = 7.965$ kN/m (5.96)

Horizontal wind load for one horizontal wind girder is

$$p_{wo} = 0.8\ h/2 \quad (5.97)$$

the safety factor is γ_W = 1.5, the factor for simultaneous effects is 0.9, then the factored horizontal load is

$$p_w = 0.9*1.5*0.8\ h/2 \quad (5.98)$$

Four different square hollow sections are considered: for upper chord(1), lower chord(2), inside columns(3) and diagonal braces (4). The outside columns are not treated, since they should be constructed as transverse frames and designed also for bending moments caused by the horizontal wind load.

The maximal forces in truss members are as follows:

Upper chord: $N = N_v + N_h$ (compression)

Force from vertical load $N_v = \dfrac{p_v L_o^2}{8h}$ (5.99)

and from horizontal load $N_h = \dfrac{p_w L_o^2}{8s}$ (5.100)

Lower chord: $N = N_v + N_h$ (tension)

and from the horizontal load $N_h = \dfrac{p_w L_o^2}{8s}$ (5.101)

The forces from the vertical load are given in Table 5.10.

Table 5.10 N_v forces from the vertical load p_v

a	lower chord	inside columns	diagonals
$L_o/8$	$7.5\ p_v a/\omega$	$2.5\ p_v a$	$4\ p_v a\sqrt{1+\omega^2}\ /\ \omega$
$L_o/10$	$12\ p_v a/\omega$	$3.5\ p_v a$	$5\ p_v a\sqrt{1+\omega^2}\ /\ \omega$
$L_o/12$	$17.5\ p_v a/\omega$	$4.5\ p_v a$	$6\ p_v a\sqrt{1+\omega^2}\ /\ \omega$

The compressed members are designed for overall buckling according to Eurocode 3 [5.31] using the buckling curve b for SHS struts and the limiting local slenderness according to CIDECT [5.43] $\delta_{SL} = (b/t)_{\lim} = 35$ for steel of yield stress f_y = 235 MPa. It is shown in our another paper [5.44] that the relationship between the radius of gyration (r) and the cross-sectional area (A) can be expressed as

$$r = a_S\sqrt{A} = \sqrt{\frac{\delta_{SL}}{24}}\sqrt{A} = 1.2076\sqrt{A} \quad (5.102)$$

The design formula for overall buckling is

$$\frac{N}{A} \le \chi f_y , \quad \frac{1}{\chi} = \Phi + \sqrt{\Phi^2 - \bar{\lambda}^2} \tag{5.103}$$

$$\Phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2] \tag{5.104}$$

$$\bar{\lambda} = \frac{\lambda}{\lambda_E} = \frac{KL}{r\lambda_E} , \quad \lambda_E = \pi\sqrt{\frac{E}{f_y}} \tag{5.105}$$

Using the symbols

$$c_o = \frac{100K}{\lambda_E}, \; x = \frac{10^4 N}{L^2}, \; y = \frac{10^4 A}{L^2} \tag{5.106}$$

the design formula can be written as

$$\frac{x}{f_y} \le \frac{y}{\Phi + \sqrt{\Phi^2 - \frac{c_o}{a_S^2 y}}} \tag{5.107}$$

$$\Phi = 0.5[1 + \alpha(\frac{c_o}{a_S \sqrt{y}} - 0.2) + \frac{c_o^2}{a_S^2 y}] \tag{5.108}$$

where L is the strut length, $\alpha = 0.34$ for buckling curve b, K is the end restraint factor for chord members $K = 0.9$, for inside columns $K = 0.75$. For f_y=235 MPa it is $\lambda_E = 93.91$. For a given compressive force N and strut length L (or x) the required cross-sectional area (or y) can be calculated by using a computer program.
The results of calculations are summarized in Table 5.11 and in Fig. 5.22. It can be seen that the optimal $\omega = h/a$ ratios are different for various a-values. The absolute optimum is $\omega = 1$ for $a = L_o/8$.

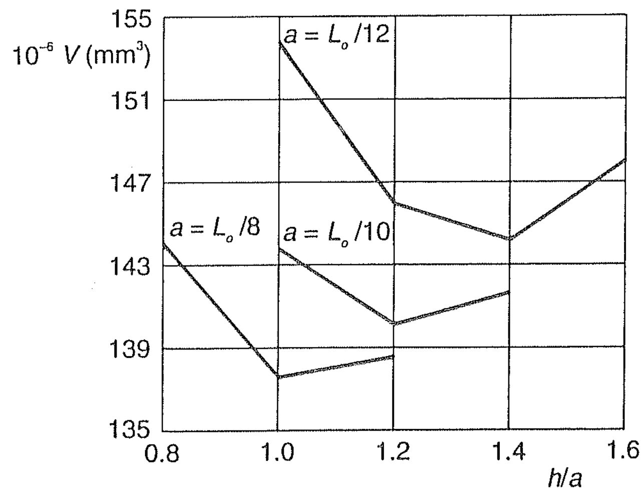


Fig. 5.22 Total volumes of a main girder of belt-conveyor bridge constructed from SHS members.

Table 5.11 Total volumes of a main girder of belt-conveyor bridge constructed from SHS members
$10^{-6}\ V\ (mm^3)$.

$\omega = h/a$	0.8	1.0	1.2	1.4	1.6
$a=L_o/8$	144.1	137.7	138.6		
$a=L_o/10$		143.8	140.2	141.7	
$a=L_o/12$		153.9	146.1	144.1	148.0

Although the sensitivity of the volume function is small, the difference between the V-values for $\omega = 1$ is 100*(153.9 - 137.7)/153.9 = 10 %, thus, 10% savings in weight can be achieved by using the optimal $a = L_o / 8$ version instead of $a = L_o / 12$ version. This optimal version is also advantageous regarding the fabrication costs, since it is constructed with less number of nodes.

5.8.6 Conclusions

The main differences using the EASY and the L5O expert shells were, that in EASY all values for the computation should be given in advance, so the program goes on a given way bordering by the rules, but LO5 asks for the unknowns during the computation, it knows what to ask for, easier to jump from one level to another on the rules' tree and the optimization part is built into the expert shell. It means that the second expert system is much close to the original aim of artificial intelligence.
The numerical example shows that it is important to include the optimum design in an expert system to make it possible to select the most suitable structural versions.

ACKNOWLEDGEMENTS

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Appendix for Chapter 5

Table A1 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal fillet welds downhand position

Welding technology	a_w [mm]	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	0-15	$0.7889a_w^2$
SMAW HR	0-15	$0.5390a_w^2$
GMAW-C	0-15	$0.3394a_w^2$
GMAW-M	0-15	$0.3258a_w^2$
FCAW	0-15	$0.2302a_w^2$
FCAW-MC	0-15	$0.4520a_w^2$
SSFCAW (ISW)	0-15	$0.2090a_w^2$
SAW	0-15	$0.2349a_w^2$

Table A2 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal 1/2 V and V butt welds downhand position

Welding technology	a_w [mm]		$10^3 T_2 = 10^3 C_2 a_w^n$		$10^3 T_2 = 10^3 C_2 a_w^n$	
SMAW	4-6	6-15	$3.13a_w$	$0.5214a_w^2$	$2.7a_w$	$0.45a_w^2$
SMAW HR	4-6	6-15	$2.14a_w$	$0.3567a_w^2$	$1.8462a_w$	$0.3077a_w^2$
GMAW-C	4-15		$0.2245a_w^2$		$0.1939a_w^2$	
GMAW-M	4-15		$0.2157a_w^2$		$0.1861a_w^2$	
FCAW	4-15		$0.1520a_w^2$		$0.1311a_w^2$	
FCAW-MC	4-15		$0.2993a_w^2$		$0.2582a_w^2$	
SSFCAW (ISW)	4-15		$0.1384a_w^2$		$0.1194a_w^2$	
SAW	4-15		$0.1559a_w^2$		$0.1346a_w^2$	

Table A3 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal K and X butt welds downhand position

		K butt welds	X butt welds
Welding technology	a_w [mm]	$10^3 T_2 = 10^3 C_2 a_w^n$	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	10-40	$0.3539 a_w^{1.9349}$	$0.3451 a_w^{1.9041}$
SMAW HR	10-40	$0.2419 a_w^{1.9346}$	$0.2363 a_w^{1.9037}$
GMAW-C	10-40	$0.1520 a_w^{1.9358}$	$0.1496 a_w^{1.9029}$
GMAW-M	10-40	$0.1462 a_w^{1.9354}$	$0.1433 a_w^{1.9035}$
FCAW	10-40	$0.1032 a_w^{1.9351}$	$0.1013 a_w^{1.9028}$
FCAW-MC	10-40	$0.2030 a_w^{1.9351}$	$0.1987 a_w^{1.9038}$
SSFCAW (ISW)	10-40	$0.0937 a_w^{1.9357}$	$0.0924 a_w^{1.9022}$
SAW	10-40	$0.1053 a_w^{1.9362}$	$0.1033 a_w^{1.9040}$

Table A4 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal T butt welds downhand position

Welding technology	a_w [mm]	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	2-8	$\left(0.1211 - 0.00473 a_w^{1.3538}\right)^{-1}$
SMAW HR	2-8	$0.2155 a_w^2 + 2.1485$
GMAW-C	2-8	$0.2189 a_w^{1.8443}$
GMAW-M	2-8	$0.2221 a_w^{1.8176}$
FCAW	2-8	$0.1006 a_w^2 + 0.4247$
FCAW-MC	2-8	$0.2065 a_w^2 + 0.4405$
SSFCAW (ISW)	2-8	$0.0918 a_w^2 + 0.3791$
SAW	2-8	$0.01066 a_w^3 + 1.698$

Table A5 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal U and double U butt welds downhand position

Welding technology	a_w [mm]	U butt welds $10^3 T_2 = 10^3 C_2 a_w^n$	double U butt welds $10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	20-40	$2.2326a_w^{1.4650}$	$1.8195a_w^{1.3692}$
SMAW HR	20-40	$1.5280a_w^{1.4646}$	$1.2461a_w^{1.3686}$
GMAW-C	20-40	$0.9642a_w^{1.4649}$	$0.7865a_w^{1.3688}$
GMAW-M	20-40	$1.6489a_w^{1.4652}$	$0.7526a_w^{1.3698}$
FCAW	20-40	$0.6514a_w^{1.4654}$	$0.5334a_w^{1.3681}$
FCAW-MC	20-40	$1.2833a_w^{1.4652}$	$1.0462a_w^{1.3694}$
SSFCAW (ISW)	20-40	$0.5962a_w^{1.4638}$	$0.4824a_w^{1.3725}$
SAW	20-40	$0.6702a_w^{1.4642}$	$0.5461a_w^{1.3682}$

Table A6 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal fillet welds in positional welding

Welding technology	a_w [mm]	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	0-15	$1.6670a_w^2$
GMAW-C	0-15	$0.4930a_w^2$

Table A7 Welding times T_2 (min/mm) in function of weld size a_w (mm) for longitudinal V butt welds in positional welding

Welding technology	a_w [mm]	$10^3 T_2 = 10^3 C_2 a_w^n$
SMAW	4-15	$0.9518a_w^2$
GMAW-C	4-15	$0.2814a_w^2$

It should be noted that in values for SAW a multiplying factor of 1.7 is considered since in COSTCOMP different cost factors are given for various welding methods.

Table A8 Cutting time of plates, T_7 (min/mm) in function of weld size a_w (mm) for fillet for longitudinal fillet welds and T-, V-, 1/2 V butt welds

Cutting technology	Thickness t [mm]	$10^3 T_7 = 10^3 C_7 t^n$
Acetylene (normal speed)	2-15	$1.1388t^{0.25}$
Acetylene (high speed)	2-15	$0.9561t^{0.25}$
Stabilized gasmix (normal speed)	2-15	$1.1906t^{0.25}$
Stabilized gasmix (high speed)	2-15	$1.0858t^{0.2261}$
Propane (normal speed)	2-15	$1.2941t^{0.2381}$
Propane (high speed)	2-15	$1.1051t^{0.25}$

Table A9 Cutting time of plates, T_7 (min/mm) in function of weld size a_w (mm) for fillet for longitudinal X- and K butt welds

Cutting technology	Thickness t [mm]	$10^3 T_7 = 10^3 C_7 t^n$
Acetylene (normal speed)	10-40	$0.8529t^{0.3643}$
Acetylene (high speed)	10-40	$0.6911t^{0.3803}$
Stabilized gasmix (normal speed)	10-40	$0.8991t^{0.3597}$
Stabilized gasmix (high speed)	10-40	$0.6415t^{0.4367}$
Propane (normal speed)	10-40	$0.9565t^{0.3583}$
Propane (high speed)	10-40	$0.7870t^{0.3825}$