

INTRODUCTION

1.1 Main advantages of steel structures

Steel is a universally used material. It is used either separately or combined with another material e.g. reinforced concrete. Its popularity may be attributed to the combined effects of several factors, the most important of which are: it possesses great strength, it exhibits good ductility, its fabrication is easy, and its is relatively cheap. In addition, steel is the ultimate recyclable material. Several advantages are listed in Table 1.1.

Table 1.1. Advantages of steel structures

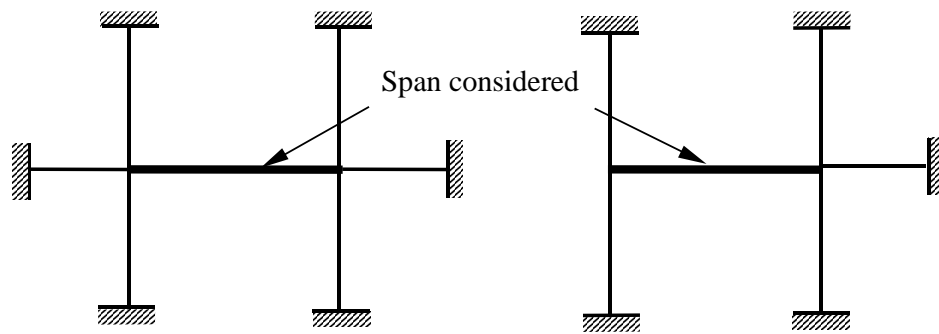
Item	Comments
Ease of erection	No formwork needed and minimum craneage required for the erection, many parts of the structure can be prefabricated away from the site, and it is largely self-supporting during erection.
Modifications	Either extensions or strengthening is relatively straightforward. Possible reuse after a structure is disassembled or scrap value even though not reusable.
Uniformity	The properties of steel do not change appreciably with time, as do those of reinforced-concrete structures.
Low self-weight	Permits large clear spans without intermediate columns
Dimensional control	Prefabrication in the workshop ensures accurate work and quality control.

1.2 Problems associated with the analysis and design of steel frame structures

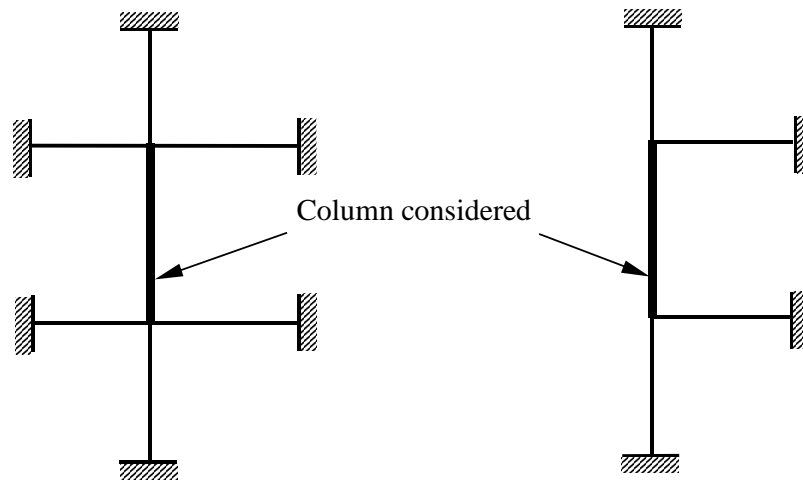
The design of structural steelwork is a process based on many contributing aspects: past experience of successful and unsuccessful construction, laboratory tests and results of research, combining to ensure structures do not fail. Structures can therefore be used efficiently and safely but at the same time must be economically built and maintained. From this it can be understood that the design process must satisfy two conflicting aims—economy and safety. Achieving this compromise is not an easy task, consequently codes of practice have evolved to assist and guide the designer, but different national codes, for example, British and American codes of practice treat the design problem differently. This may be because the behaviour of steelwork frames, for instance, is not well understood because methods of design are still at an elementary stage of development. This may be due to the fact that the problem of design is much less specific than that of analysis.

The question of design or synthesis, involves generating member sizes which are satisfactory in all respects, under all loading conditions. In most cases, an unlimited number of designs will meet these requirements. More realistic designs are based strictly on a trial and error process. The design process starts with the analysis step. After analysing a structure, an initial solution of the member properties are modified and the subsequent solution is re-analysed. At the analysis stage, the British Code of practice (BS 5950: Part I) suggests three methods. Firstly, in order to simplify the analysis to enable hand calculations, sub-frame models given in Figure 1.1 are proposed for beam and column analysis. Alternatively, as a second method, the beam moments may be

obtained by analysing the beams as continuous members over simply supported columns. The third method is to carry out a full analysis of the structure as a whole.



(a) Beam design sub-frames



(b) Column design sub-frames

Figure 1.1. Sub-frames (taken from BS 5950: Part I)

The internal forces and bending moments of each member of the structure can be therefore evaluated. In the second step of design process-checking of ultimate and serviceability criteria, the earliest approaches to elastic design entailed the use of a number of simplifying assumptions. The point of contraflexure occurs at mid-column

height, so that the framework became statically determinate and the sections thus determined uniquely. Another assumption, relating to the stability problem was to approximate the value of the effective buckling length of columns. This approximation, which was included in the old version of the British code of practice titled BS 449, has been extensively revised in to the present version (BS 5950). More details about the design procedure are discussed in Chapter 2.

1.3 The need for design optimization

From the previous section, one can address several difficulties that may face the structural designer when utilising conventional design. Firstly, the skill and experience of the designer, which could lead to completely different designs. Secondly, the complexity of the treated structure makes the difficulty of doing several re-analyses and subsequently redesigns. Thirdly, there is difficulty of handling all possible loading cases. Fourthly, the intended usage of the structure might prevent the designer from achieving economical design. Fifthly, the alternative design and analysis techniques might confuse the designer in choosing the appropriate technique. Therefore, the use of computers has made reliable and accurate analysis much easier, and the speed with which alternative solutions can be analysed makes it possible to achieve more economical designs than were attainable in the past. Design optimization is therefore an interesting research topic, and recommendations for design optimization have been made by design experts among them (MacGinley, 1997 and Adeli, 1994). Design optimization is concerned with the problem of the selection of geometric parameters and mechanical strength properties for the structural elements. This selection consists of a search for the extremal solutions, which satisfy the prescribed criteria, the search being conducted in an objective and rational way, that does not rely on the intuition or special

abilities of the designer. Thus, design optimization takes over that part of the design process, which consists of selecting sizes and subsequently checking that the required criteria have been met. The question arises whether the design optimization field can or should fully replace traditional designing procedures, that is, whether or not the task of optimization is to embrace all structural parameters so that the solution of an optimization problem should be equivalent to obtaining a complete design of a structure. This question will be answered in this research.

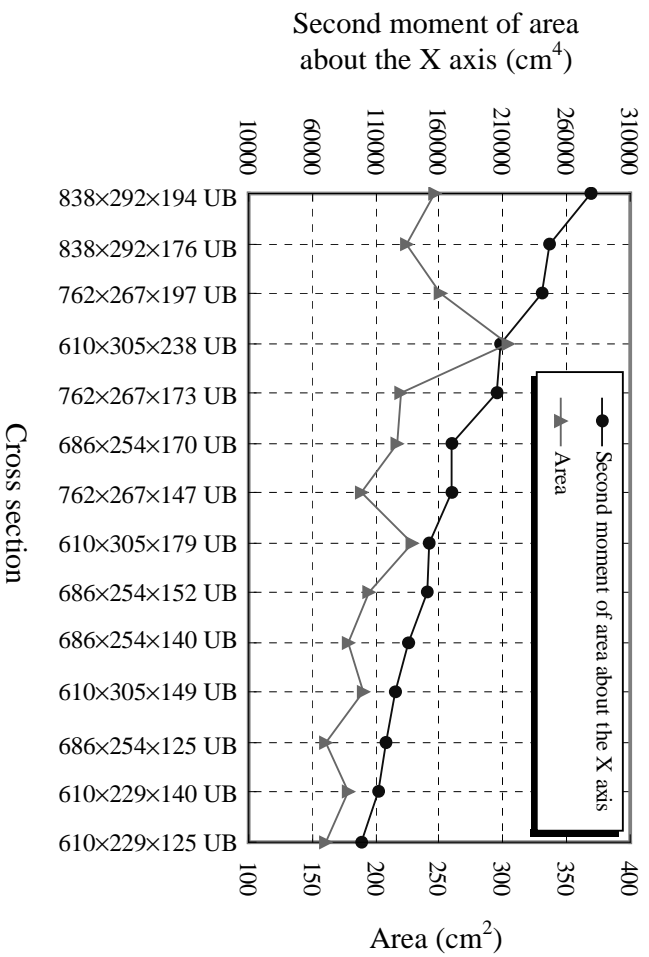
1.4 Main features of a design optimization problem

In the case of simple elements or even the whole structure it is possible and necessary to take into account all requested design criteria in the formulation of an optimization problem. This is exemplified by the classical simple design optimization problem. If the structural steelwork is more complex, however, the problem of including all design criteria in the optimization may prove impossible to solve by gradient-based techniques like linear and non-linear programming algorithms as search methods. In such cases it is advisable to include only some of the parameters in the formulation of the optimization problem. Unfortunately, however, very few physically meaningful problems in structural design, if any, can be formulated directly as linear programming (LP) problems without involving a degree of simplification. Most structural design problems involve highly non-linear constraint and objective function relationships. In addition, the relationships between the design variables, which are the members of the structure are unknown. No one optimization algorithm can possibly be efficient or even successful in all cases of interest. If all design problems involved objective functions that were quadratic, with analytic derivative, then life would be simple (Ghasemi, 1996). However, in design practice, it is well known that many problems are vastly more complicated than this.

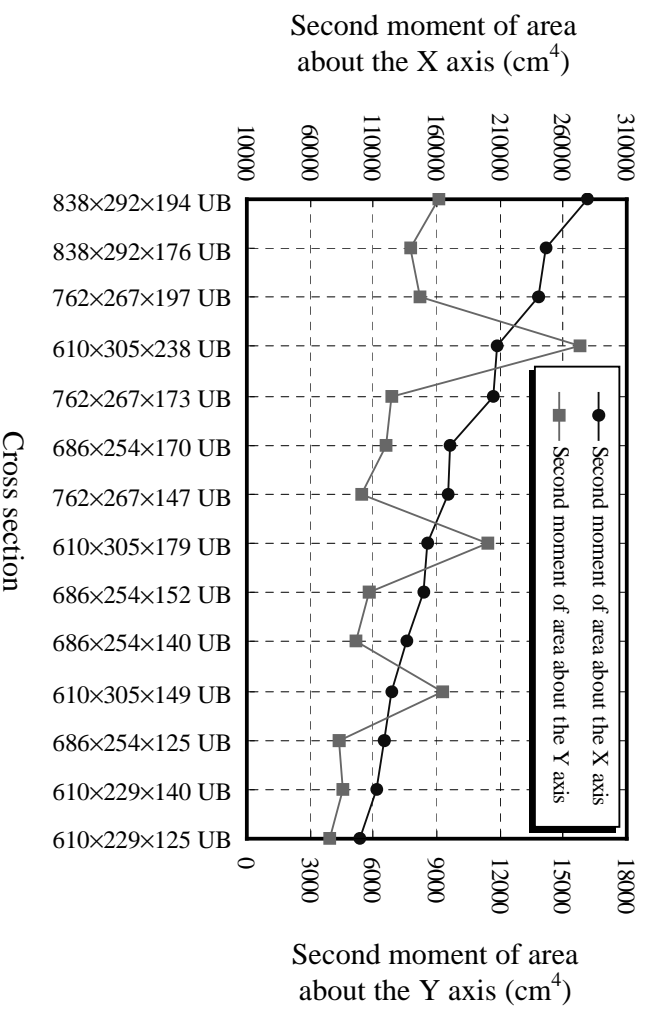
What is worse, often the function we are attempting to optimize is not analytically available. It is only known in the form of a computer code that evaluates the function point by point. To make it clear let us investigate the nature of the design variables, in this case, the properties of sections considering universal beams (UBs), and universal columns (UCs) from British Standard (BS 4). The cross-sectional area, and second moment of areas about the major X axis are depicted in Figures 1.1a and 1.2a for UBs and UCs respectively. While, for the same sections, the second moment of areas about the major X and minor Y axes are displayed in Figures 1.1b and 1.2b for UBs and UCs respectively. In these figures, the sections are shown in descending order with respect to their second moment of area about X axis where the properties of these sections are taken from Steel Construction Institute (1985).

From these figures, it can be concluded that there is no one-to-one relationship between area and second moment of area for the group of sections, UBs or UCs. This makes it difficult to find an accurate relationship between the design variables and objective or constraint functions.

At this stage, having introduced some of the main features of a design problem, it can be concluded that it is necessary to develop a design optimization tool. Evolutionary algorithms such as genetic algorithms, and simulated annealing are suitable to design optimization. Such algorithms do not need any relationship between the constraint function and design variables. They can therefore deal with discrete variables like cross sections from a catalogue and discontinuous domains of the objective function. Furthermore, it is possible to deal with more complex functions.

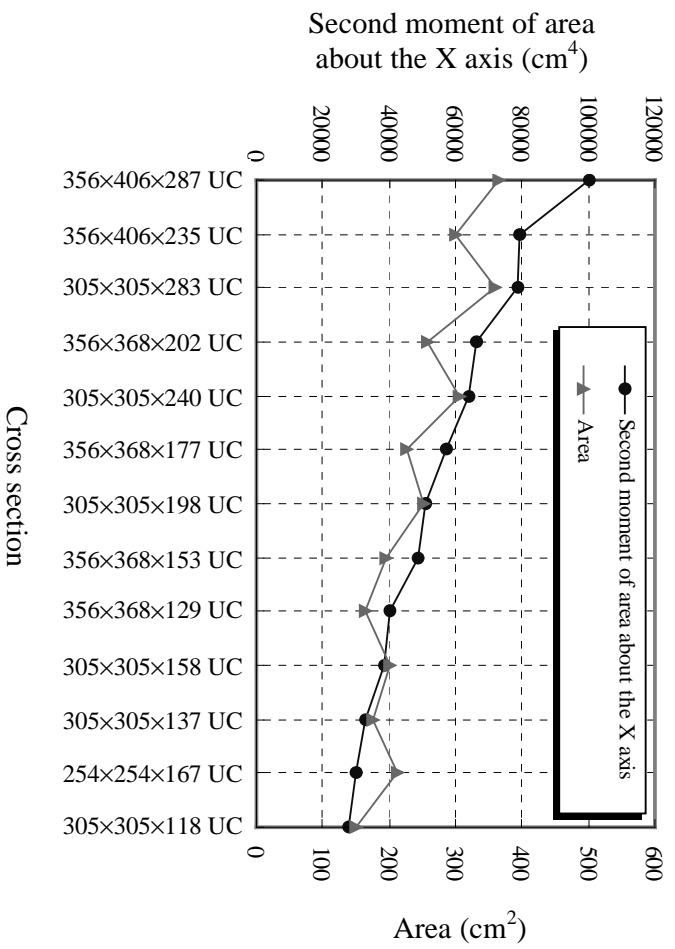


(a) Relationship between the second moment of area about the X axis to area

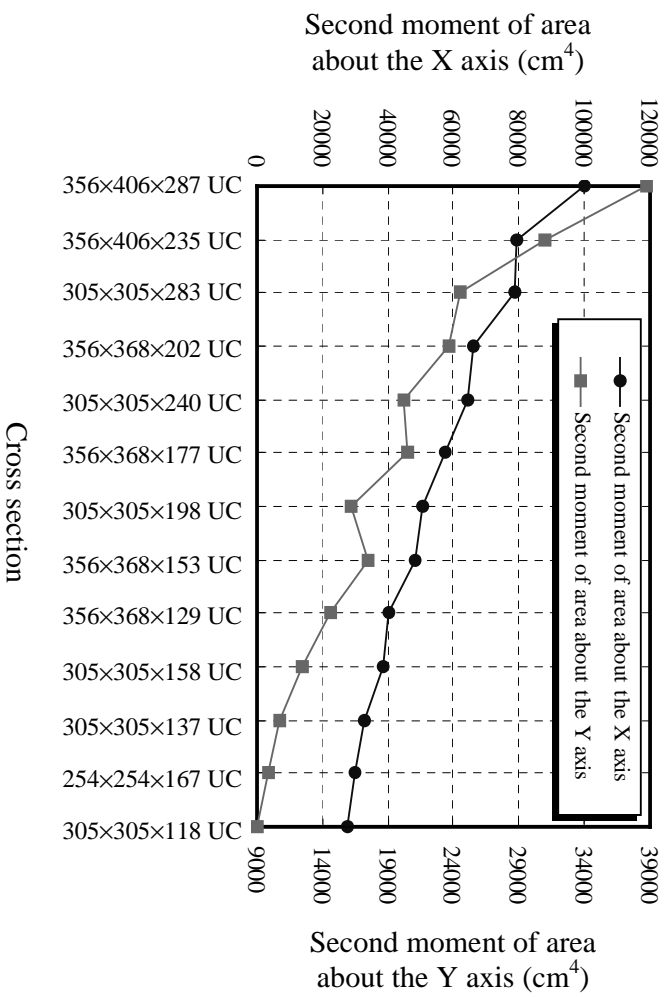


(b) Relationship between the second moment of areas about the X and Y axes

Figure 1.1. Relationship between the cross sectional properties of universal beams (UBs)



(a) Relationship between the second moment of area about the X axis to area



(b) Relationship between the second moment of areas about the X and Y axes

Figure 1.2. Relationship between the cross-sectional properties of universal beams (UCs)

Generally, evolutionary algorithms require more function evaluations than gradient-based methods. The investigation of different techniques is important to speed up these algorithms. The advantages of applying these optimization techniques are discussed in many textbooks such as Arora (1989), Brandt (1989), Haftka and Gurdal (1993), Adeli (1994), Fletcher (1993), and Xie and Steven (1997). These advantages can be summarised as follows:

1. Some optimization techniques do not need any prior information about the objective function or constraint functions.
2. The possibility and flexibility of dealing with complex structures under different loading conditions and constraints.
3. The ability of dealing with sections from standard catalogues classified as discrete design variables.
4. The capability of achieving more than one design solution.
5. The flexibility of formulating the engineer experiences and skills into the design optimization problem. For examples, the number of design variables, the linking of the design variables to the structural members, the acceptance of the design obtained according to the practical experiences of the designer.

1.5 Literature review of steelwork design optimization

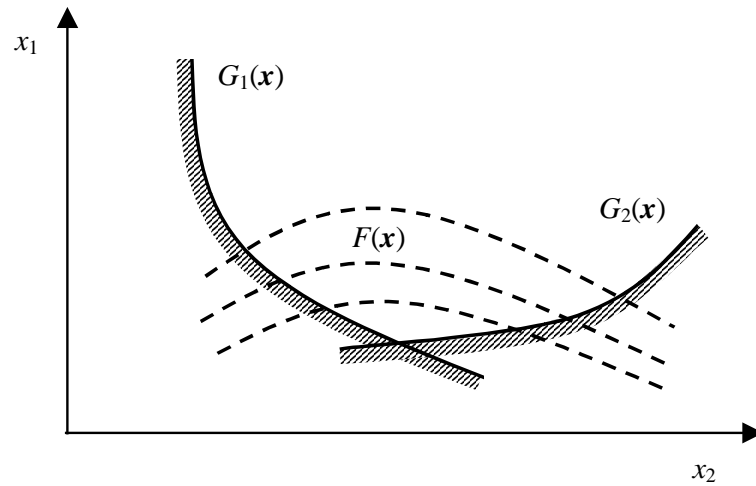
There are many classical references to the solution of large classes of problems, with extensive literature available in the field. Within the context of the present work, several particular areas of interest can be identified relating to the major thematic areas. These areas may be defined from the consideration of optimization techniques with applications covering both closed form and discretized solutions. In general, optimization techniques used in structural engineering design can be categorised into

three distinct approaches: (1) mathematical programming, (2) optimality criteria methods and (3) evolutionary algorithms. Several textbooks discuss these methods, among them Reklaitis *et al.* (1983), Vanderplaats (1984), Arora (1989), Brandt (1989), Kirsch (1993), Fletcher (1993), Haftka and Gurdal (1993), and Adeli (1994). Review papers (Vanderplaats and Thanedar, 1991, Schittkowski *et al.*, 1994, Arora *et al.* 1994, and Huang and Arora, 1995) also illustrate methods applied for mixed discrete-integer-continuous variable nonlinear optimization for structural design applications with the focus on problems having linked discrete variables. These methods as well as the literature review are presented in the following sections.

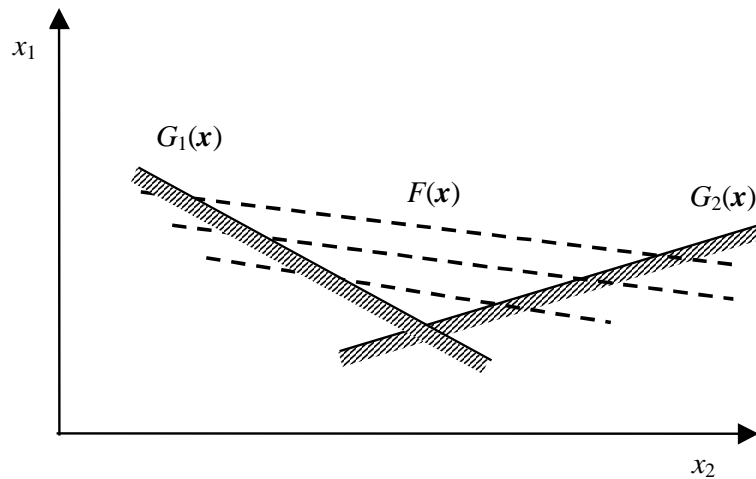
1.5.1 Mathematical programming

Mathematical programming can be subdivided into linear programming (LP) and nonlinear programming (NLP). The major characteristic of LP is that the objective function and the associated constraints are expressed as a linear combination of the design variables. To apply LP techniques to structural design optimization, the relationship between the objective function and the constraints to the design variables has to be linearized. However, when a linear relationship is used to model a nonlinear structural response, errors are inevitable because the actual optimization problem and the linear approximated problem can be graphically represented in Figures 1.3a and 1.3b respectively where $F(\mathbf{x})$ is the objective function with two design variables x_1 and x_2 . Associated constraints are termed $G_1(\mathbf{x})$ and $G_2(\mathbf{x})$. Nonlinear mathematical programming was developed for nonlinear unconstrained optimization problems. The Kuhn-Tucker conditions (Kuhn and Tucker, 1951) provide the necessary conditions for optimal solutions. Direct application of the Kuhn-Tucker conditions is difficult for most design optimization problems. The calculation of gradients and the solutions of the

correlated nonlinear equations prohibit the direct application of that condition for most engineering design problems.



(a) Actual optimization problem



(b) Linear approximated problem

Figure 1.3. Actual and approximated linear optimization problem

Cornell (1966) was the first to tackle the LP technique. He employed a technique to find a minimum weight design of a structural steelwork using elastic analysis. Bigelow and Gaylord (1967) used the LP techniques to find minimum weight applying

plastic design, which included the effects of column axial loading and overall frame instability due to sidesway. Using an array of linear inequalities and checking the corresponding solutions against the yield criterion, they were able to find a minimum weight design for a framework up to four-stories and two-bays.

Using NLP, Brown and Ang (1966), Gallagher and Gellatly (1966), Majid and Elliott (1971) found a minimum weight design for a two-bay four-storey framework. Plamer (1968), Cammaert (1971) and Kirsch *et al.* (1972) utilised dynamic programming in the optimal design of plastic multi-storey frameworks. The technique is based on subdividing the structure into a series of substructures with optimization occurring in sequence through each substructure with respect to other substructures. Each substructure is optimized with respect to its own design variables with the other design variables kept constant. Arora and Govil (1977) argued that this process was inefficient and could lead to non-optimal solutions. As an alternative strategy they proposed a partitioning technique, which enabled all substructures to be considered simultaneously permitting all design variables to vary at the same time. The proposed technique permits small changes in the design variable and utilises a series of Taylor linear expansions on all equations. It is of interest to note that the process is limited to small design changes. Vanderplaats and Sugimoto (1986) described a design program known as automated design synthesis (ADS) based on methods of sequential linear programming (SLP) and sequential quadratic programming (SQP) to find minimum weight design of structures. Karihaloo and Kanagasundram (1989) used the ADS program by Vanderplaats and Sugimoto (1986) to obtain minimum weight designs of plane frames under multiple load systems. Normal stresses, shear stresses, deflections and critical buckling load were limited to prescribed limits under any loading system. In the same year, Gulay and Boduroglu (1989) studied optimum design of frames subjected

to seismic loading. The technique was based on SLP. Constraints were imposed on normal stresses and displacements within prescribed limits. The design variables were the cross sectional areas of the structural elements. Approximate relationships of cross sectional area, radius of gyration, as well as the elastic modulus were used for the standard I sections. Yunkiang (1989) also treated frames with beams subjected to stresses, displacement and buckling limits. The author obtained a minimum weight design using a non-linear programming technique. He concluded that posing the buckling constraints may increase the obtained design weight by 30-50% or even 80% corresponding to the design weight without buckling constraints. Lassen (1993) presented a method for optimal design of frameworks. The approach was based on a reanalysis technique in combination with sub-optimization for each member group in the structure. The sub-optimization was performed using a nonlinear programming algorithm based on SQP. Given a topology of the structure and stress requirements, section dimensions of wide-flange and hollow-section members were treated as design variables and the minimum weight or cost solution was sought. Al-Salloum and Siddiqi (1993) used normal stress as well as nodal displacement limits to the Canadian code of practice to obtain optimum design for steel frameworks. Applying an iterative method and move limits, the minimum weight of steel frames were obtained. They employed different cross sectional relationships of area, second moment of area than those used by Gulay and Boduroglu (1989). Wang and Grandhi (1994) described the applicability of an optimization algorithm using a multi-point spline approximation. The algorithm is based on the function values and first order derivatives of the constraints available at the intermediate points of optimization. The behaviour constraints included stresses, displacements, and local buckling in the optimum design of three-dimensional frame structures. Salajegheh (1995) used approximated element forces for the optimum design

of structures. First, the structural responses such as member forces were approximated in terms of some intermediate variables. By substituting these approximate relationships into the original design problem, an explicit nonlinear approximate design task with approximation was achieved. The problem was solved by means of numerical optimization techniques, the results of which were then used for discrete variable optimization. The approximate problem was then converted into a sequence of second level approximation problems of separable form, each of which was solved by a dual strategy with discrete design variables. Simoes (1996) described a program based on LP for the optimum design of steel frameworks accounting for the behaviour of semi-rigid connections. The procedure explicitly considered connections and members by taking connection stiffnesses and member sizes as continuous-valued and discrete-valued design variables, respectively. The optimization algorithm minimised the cost of the connections and members of the structure. Constraints were imposed on stresses and displacement under specified design loads. Hernandez (1998) presented an optimum solution of structures using mathematical programming used for defining the design variables. The author used cross sectional areas of structural elements as design variables, and considered stress constraints and lower limits for the design variables.

1.5.2 Optimality criteria method

The optimality criteria (OC) method was developed from indirectly applying the Kuhn-Tucker conditions of nonlinear mathematical programming combined with Lagrangian multipliers (Arora, 1989). The Lagrangian multipliers are used to include the associated constraints. The OC method is based on continuous design variables. For the case where discrete variables are desired, a two step procedure is typically used. First, the optimisation problem is solved using continuous variables. Second, a set of discrete

values are estimated by matching the values obtained from the continuous solution. The OC method uses a single section property of a structural member as the design variable. All other section properties are expressed as functions of the selected design variable. Different forms of approximation based on curve fitting techniques of one section type are discussed by many authors among them Brown and Ang (1966), Nakamura (1966), Arora *et al.* (1975), Calafell and Willmert (1977), Khan *et al.* (1979), Ding and Esping (1986), Saka (1991), Takewaki *et al.* (1991). Two problems are associated with the discretization of a continuous solution. First, the relationship between the cross-sectional area and other section properties of rolled structural shapes is not one-to-one as discussed in Section 1.4. Consequently, it is difficult to transfer a continuous solution to a discrete one. Second, the selected discrete shapes may result in a frame having a different structural response, which may not satisfy the imposed performance constraints.

Interest was aimed at developing computer codes suitable for design office usage, among them Chang and Liu (1989) who developed computer code named ARS-3 to obtain minimum weight design of plane frames. The code was based on optimality criterion using cross sectional areas as design variables. Xu and Grierson (1993) also introduced a computer-automated method for the optimum design of steel frameworks accounting for the behaviour of semi-rigid connections. The method explicitly accounted for both connections as well as members, by taking connection stiffnesses and member size as continuous-valued and discrete-valued variables in the design process, respectively. A continuous-discrete optimization algorithm was applied to minimise the cost of the connections and members for the structure subject to stresses and displacements constraints.

Another interest aimed to study developed algorithms. This is covered by a great wealth of literature. Cameron *et al.* (1989) proposed a discrete optimization method using sensitivity analysis and an iterative process to obtain minimum weight design of structures using the American Institute of Steel Construction design code named AISC-ASD. The iterative process stopped when there was no change in the structure weight from one design stage to the next. Saka and Hayalioglu (1991) obtained optimum designs for geometrically nonlinear elastic plastic frames. The algorithm was based on coupling the optimality criteria with a large deformation analysis method for elastic-plastic frames. Saka (1991) extended his study to obtain optimum design of steel frames considering displacement and combined stress limitations according to AISC. The same algorithm was used by Hayalioglu and Saka (1992) to obtain optimum design of steel frames, which have members with non-uniform sections. Linear variations of sections were considered. Each member introduced two variables into the design problem provided that they are not linked together. The first was the cross-sectional area at one end and the other was the ratio of areas at both ends. Frbatur and Al-Hussany (1992) proposed a computer program to obtain minimum weight elastic design of steel frames employing the OC. Grierson and Pak (1993) developed a computer-based method for the optimum design of tall steel building frameworks. Specifically, the OC method was applied to minimise the weight of a lateral load-resisting structural system of fixed topology subject to constraints on overall and inter-storey drift. Vanderplaats and Salajegheh (1993) treated the optimum design of structures, where the design variables were chosen from a set of prescribed values. The main idea is to reduce the number of structural analyses in the process of optimization. First the structural responses such as forces and displacements are approximated as functions of cross-sectional properties, thus high quality explicit functions are generated for the responses. Employing these

approximate expressions in the optimization process, the continuous optimum design can be achieved. Using the results obtained from the continuous variable optimisation and with the help of the approximated responses, a branch and bound method is used to obtain the discrete design values. Al-Salloum (1995) developed an iterative approach based on the fully-stressed design concept employing scaling to find a search path to the optimum design for statically indeterminate steel frames subjected to behavioural constraints on member stresses and nodal displacements, and side constraints on member sizes. Chan *et al.* (1994 and 1995) proposed a computer-based technique based on OC for least-weight design of three-dimensional tall steel building frameworks under multiple lateral loading conditions. A continuous solution was obtained first and then a pseudo-discrete section selection strategy was used to achieve a smooth progressive transition from continuous-variable design to discrete final optimum design. Salajegheh (1996 and 1997) achieved the optimum design of structures by approximating all the structural responses such as member forces, nodal displacements and frequencies. The non-critical constraints were omitted in each design cycle. Then the OC approach was used to solve the resulting approximate design problem. Soegiarso and Adeli (1997) studied the difference of optimum designs obtained from two versions of the design procedure proposed by AISC. These two design procedures are the Load and Resistance Factor Design (LRFD) and the Allowable Stress Design (ASD). The study was based upon using the OC approach. Although the methods of design are LRFD and ASD, the effective length factors were determined as specified by the European Steel Design Code. Saka (1997) discussed the optimum design of steel frames composed of prismatic and/or tapered members. The technique proposed consists of two steps. In the first step, the frame was analysed under the external unit loading for the current values of the design variables. In the second step, that response was used together with the values of

Lagrange multipliers to compute the new values of the depth variables. This process is continued until convergence is obtained. He continued his study with Kameshki in 1998 by discussing the optimum design of unbraced rigid frames using the OC approach. The OC approach was employed to handle sway limitations, and combined stress constraints, which were reduced to non-linear equations of the design variables.

1.5.3 Evolutionary algorithms

As a result of the breakthrough in the development of computer hardware, relatively new techniques known as evolutionary algorithms, such as genetic algorithms and simulated annealing, have been suggested in the structural optimization field. These methods are in the category of stochastic search methods, which are based on natural processes.

Simulated annealing (SA) is a technique that can be used to find a minimum value for continuous-discrete-integer non-linear programming problems (Arora *et al.*, 1994). SA was successfully applied to the design optimization problem by Balling (1991) and May and Balling (1992) who developed and modified a SA strategy for use in the discrete optimization of three-dimensional frames. The analysis has been approximated using the suggestions given by Vanderplaats and Salajegheh (1989). The constraints are the combined stresses of structural members as well as nodal displacements as suggested by AISC.

A genetic algorithm (GA) is a strategy that models the mechanics of genetic evolution (Holland, 1975). The main characteristics of a GA are based on the principles of survival of the fittest and adaptation. The advantages of applying a GA to design optimization of steel frame structures are to include discrete design variables and the open format for constraint statements and multiple load cases. Optimization using GA

has been successfully applied to structural optimization. Applications to structural steelwork can be summarised as follows:

Hajela (1989) was the first to use genetic algorithms to obtain the minimum weight elastic design for a two-beam grillage structure. After that, Hajela (1990) and Hajela and Lin (1992a, 1992b, 1993a, 1993b) published several papers on the use of genetic algorithms in structural optimization. Applications of the genetic search in problems with disjoint and nonconvex design spaces were discussed by Hajela (1990). In this paper the section dimensions of a two-beam grillage structure was optimized. The structure was subjected to a uniformly distributed load. A two-element thin walled cantilever torsional rod subjected to a harmonic excitation, and a 10-member truss subjected to a sinusoidal load were also presented. The paper also discussed the excessive number of function evaluations necessary to locate an optimum and suggested possible strategy to overcome this limitation. In other papers, Hajela and Lin (1992a, 1992b) proposed a multi-criterion optimization strategies for the genetic search in optimal design and have shown that this approach can be applied to problems with a mix of continuous, discrete, and integer design variables and is particularly powerful for problems with known nonconvexities. Large scale optimization problems were also addressed by Hajela and Lin (1993a, 1993b). In these papers two advanced search strategies, referred to as multistage search and directed crossover, were developed. Grierson and Chan (1993) proposed a simple genetic algorithm to obtain minimum weight design of plane single-bay single-storey framework. Depending on the exact fitness and constraints evaluated for a random selection of only one design set of the population, they approximated the value of the fitness and constraints of the rest of the population using sensitivity analysis and adopting a first order Taylor series expansion. Grierson (1996) extended the work by suggesting an idea based on coupling a genetic

algorithm with a neural network to create a computer-automated procedure for conceptual design. Arora and Huang (1996) and Huang and Arora (1997) implemented two techniques to obtain the optimum design of steel frames according to the design procedure requested by AISC. In the first technique, the authors suggested to start the search using the (SQP) method followed by smooth transition to GA. The second technique is similar to the first except that a SA method is used instead of GA. The transition of the design variables from SQP to either GA or SA is performed assuming a region of cross-sectional area and inertia ranged between 30 and 500%. Jenkins (1997) applied GA to find minimum weight design of framed structures. The author used British Standard (BS 5950) but treated the design problem by considering prescribed limits on stresses of 275 N/mm^2 and vertical displacements of nodes without taking into account the formula presented in BS 5950. In this study, different checks required by BS 5950 - shear strength, shear buckling, lateral torsional buckling, horizontal displacements - were not formulated in the design optimization problem. Camp *et al.* (1998) developed a design procedure incorporating a simple GA named FEAPGEN for discrete optimization of plane frameworks. The requirements of the AISC are reflected in the design procedure. Saka (1998) used simple GA for discrete optimum design of grillage problems considering cross-sectional properties of members from a standard set of universal beam sections. Deflection of 25 mm and allowable stress of 275 N/mm^2 were considered as constraints in the formulation of the design problem. Furthermore, in obtaining the response of a 40-member grillage system, the effects of warping and shear were also taken into account.

1.6 Goals of research

The background of the present research is based on the investigation carried out by Mahfouz (1993) and Mahfouz *et al.* (1994a, 1994b) in which the ideal distribution of a steel frame material was investigated to obtain its maximum buckling strength. The best location of stiff columns in multi-bay single-storey, multi-storey single-bay and multi-bay multi-storey frameworks permitted and prevented from sway was also studied. The authors used a method of analysis based on trial and error method in the evaluation of the elastic critical load in which step-by-step increments of a column material are used. The study provided a background to the present research in which more practical design problems have been investigated employing the design requirements as recommended by BS 5950. Objectives for the present research have therefore been established to satisfy the following goals:

1. Develop, implement and fully test algorithms for the design optimization of steel frame structures. These algorithms have the following characteristics and features:
 - a) require a minimum amount of auxiliary information to guide the search,
 - b) deal with practical manufactured sections from the standard catalogues such as BS 4, BS 4848,
 - c) include requirements for real design problems as specified BS 5950,
 - d) able to obtain more than one feasible solution accepted by practising engineers and
 - e) allow the user to incorporate the designer practical aspects.
2. Develop, implement and test algorithms for the design of steel frame structures according to BS 5950.

3. Study the stability concept of steel frame structures. This requires developing of a FORTRAN code based on a more accurate method of analysis for the evaluation of the critical buckling length of columns in sway and non-sway frames.
4. Study the effect of the accuracy of determining the effective buckling length on the optimization process and the obtained optimum designs of 2D and 3D steel frame structures.

Also a further major set of objective of this thesis is to apply reliable, creative and efficient computational tools for structural analysis and design optimization of steel frame structures by interacting the advances made in computer-aided structural analysis, and genetic algorithms so that a powerful analysis and design tool for 2D and 3D steelwork can be developed to assist structural engineers in designing efficient structures and provide better way of understanding of the structural behaviour.

1.7 Organisation of the thesis

The thesis consists of ten chapters and three appendices. These can be summarised as follows:

Chapter 1 provides an introduction, raising the problems associated with the analysis and design of steel frame structures. The main features of a design optimization problem are then discussed. The literature review of the design optimisation of steel frameworks is also demonstrated. Finally, the aim of research and overview of the thesis are presented.

Chapter 2 describes in details the design procedure as required by the British Codes of Practice. The chapter ends with describing methods used, in the present study, to represent the charts of the effective length factor of column in sway and non-sway frames into a computer code. Here, analytical solutions are obtained using regression

analysis based on Statistical Package for Social Sciences (SPSS) and genetic programming methodology.

Chapter 3 starts with studying the stability concept, historical background about the stability problem of steel frames. It then discusses the methods of analysis, which are used for the determination of the effective buckling length of columns in sway and non-sway frameworks. A FORTRAN code based on the direct method is developed and verified using results from the literature and the finite element package (ANSYS).

In Chapter 4, the stability concept of steel frame structures has been applied where the general forms of the critical buckling load of five frameworks are obtained using the direct method of analysis. The finite element package (ANSYS) is also used to verify the results obtained from these forms. Comparisons have been made between the effective buckling length factor calculated by the more accurate analysis (direct method or finite element method) and that determined by the approach presented by BS 5950. Finally, four questions are raised, and the answers will be given in the following chapters.

Chapter 5 introduces main features and formulation of the design optimisation problem. It also discusses the most frequently used methods of design optimization. Here, the concept of genetic algorithm is described in detail.

Chapter 6 describes the developed GA, new implementations, tuning of genetic algorithm parameters and comprehensive tests are presented. Different crossover operators are implemented and also tested. Comparisons between results obtained by the developed algorithm and those described in the literature or those obtained when using the ANSYS optimization methodology are also presented.

Chapter 7 is concerned with assessing the potential of the developed genetic algorithm to the treatment of complicated structural problems. Here, the maximum ratio between

the effective buckling length factor determined by the approach presented by BS 5950 and the calculated by using the finite element analysis is investigated. Three problems are formulated and solutions are obtained for different examples.

Chapter 8 presents a genetic algorithm based technique for the design optimization of multi-storey steel frame structures according to BS 5950. In this chapter, it has been proven that the developed genetic algorithm linked to design rules to BS 5950 can successfully be incorporated in design optimization in which the structural members are required to be selected from the available catalogue and the design should satisfy a practical design situation. In the formulation of the optimization problem, the objective function is the total weight of the structure and constraints are imposed on the design criteria as required by the British codes of practice (BS 5950 and BS 6399). The design variables are selected from a catalogue (British Standard BS 4). In addition, to study the effect of the accuracy of determining the effective buckling length on the optimization process and the obtained optimum designs, three approaches for determining its value are considered. Finally, In order to verify the results achieved by applying the developed Fortran code for design optimization, the CSC (1998) software is used.

Chapter 9 extends the study of chapter 8 to the discrete optimum design of three-dimensional (3D) steel frame structures using the modified genetic algorithm (GA). Following the design procedure of steel structures to BS 5950, the minimum weight design of 3D steel frame structures is presented where the most unfavourable loading cases are considered. The CSC (1998) software is also used to verify results.

Chapter 10 shows the main achievements, conclusions followed by recommendations for several promising area of further work.

Appendix A describes the stability functions utilised for the stability analysis.

Appendix B comprises a description of a sample of the ANSYS data file for the stability analysis.

Appendix C contains summary of the developed subroutines for the design of 2D and 3D steel frame structures to BS 5950.