THE USE OF GENETIC ALGORITHMS IN THE DESIGN OF CABLE-STAYED BRIDGES

By

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A B S T R A C T

Genetic Algorithms (GAs) are search algorithms based on the principle of natural selection and survival of the fittest to stochastically improve generated initial population of solutions. Only the most promising regions of the search space are enumerated to locate the optimal solution.

The problems covered in this thesis investigates the application of GAs for the determination of worst combinations of loadings and stay removals for large size cable-stayed bridges subject to combinatorial loads such as traffic loads and loads arising from cables out conditions.

Stay removal conditions came out as a result of the obligations imposed by the Department of Transport in Britain that their cable-stayed crossing must be functional under traffic loads with upto two cables missing. This has made the design/analysis of cable-stayed bridges, which is an iterative process, to be extremely complicated and very expensive.

In contrast to many classic optimisation problems which involve solely one load condition without consideration of member failures removals, the main purpose of the thesis is to investigate the potential for the application of GAs to such design situation in which the combinatorial problem of load definitions and stay removals play a particularly important role. For this reason problems associated with highway loading defined in BD37/88 and stay removal conditions will be given special attention in the context of using GAs for the locating of the worst loading/stay removal combinations.

To my late father, mother, brother Mahmoud,
brother Mohammed, and sister Wissal.

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C O N T E N T S

CHAPTER 1

INTRODUCTION

1.1- CABLE-STAYED BRIDGES

Nowadays engineers are stretching their expertise to design and build long bridges more cheaply and quickly than ever before. In November 1991, barely six years after it was first thought of, one of Europe's longest bridges opened across the River Thames at Dartford, about 30 kilometres downstream of the City of London.

The Dartford River Crossing, which is designed to relieve a bottleneck on London's orbital motorway, the M25, puts the first major road bridge over the Thames since Victorian engineers completed Tower Bridge in 1894. The bridge accounts for about three-quarters of the cost of the crossing, which includes two long viaducts, stretched almost 3 kilometres and has a price tag of just £120 million.

The crossing is also the first major highway in Britain to be privatised. This factor alone ensured the development of the most economical structure in the shortest possible time (Neil, 1991).

Cable-stayed bridges and suspension bridges are both forms of suspended crossing, relying on cables above them for support rather than columns below. Although suspended bridges are more complicated to design and build than bridges supported on columns, they can provide long, uninterrupted spans over terrain that is difficult to build.

Suspension bridges provide support by relying on two large cables draped between tall towers, one at each end from which the deck simply hangs, suspended by varying lengths of steel rope. Cable-stayed bridges, by comparison, strung up by diagonal ties to vertical pylons acting as propped cantilevers. Though suspension bridges have the greater capacity to stretch further, they are much more costly.

Until the mid-1980s cable-stayed bridges with spans of more than 350 metres or so were considered. Although main spans breaking the 400-metre barrier had been built, engineers tended to regard these structures as exceptions.

This attitude changed when Peter Taylor, a bridge engineer living in Vancouver, designed a 465-metre cable-stayed span to support a six-lane highway over the Fraser River. Between 1984 and 1986, Taylor broke new ground when he saw through the construction of the composite deck for the world beating Annacis Bridge. Until then, major suspended crossings were either all concrete or all steel. Since then, composite decks have become widely accepted for cable-stayed spans.

Composite decks are a common feature of bridges supported by columns. They are a compromise between concrete, which is cheap but heavy, and steel, which is light but expensive. But for longer spans, where problems of design and construction are more complicated, engineers have been reluctant to mix materials. Instead, they have resorted with a more expensive solution. Mostly they have chosen steel because large concrete decks are costly to support.

1.2- GOING FOR THE RECORD

The new crossing at Dartford will be the second longest bridge of its type in the world, behind the Annacis Bridge. In December 1991, Japanese engineers extended the world record by 25 metres with the completion of a 490-metre cable-stayed span for the Ikuchi Bridge between the island of Honshu and Shikoku. Shortly after that, Norway public roads administration finished the Sharnsundet Bridge about 60 kilometres northeast of Trondheim, with a main span of 530 metres.

Though Norway's record looks set to stand for more than two years, French engineers have already plotted its total eclipse. By mid-1994, the French public highway authority plans to have spanned the mouth of the Seine at Honfleur in Normandy with a cable-stayed deck stretching 856 metres. The Normandy Bridge promises to be such a giant leap in the size of cable-stayed bridges that it has provoked considerable debate through the international construction industry. While many engineers acknowledge that it is possible to design a cable-stayed bridge of this length, few accept it as a practical solution, and will not do so until the project is finished and its accounts audited (Virlogeux, 1993).

Among the greatest problems of building suspended crossings is the difficulty of temporarily stabilising their unfinished decks during construction. As the two halves of the Normandy Bridge approach each other at mid-span, engineers will have to support a pair of flexible, giant cantilevers of concrete and steel, each stretching more than 400 metres from pylons on opposite sides of the river and weighing up to 8400 tonnes.

Construction of the bridge's foundations began in September 1990 after lengthy pre-contract negotiations between the highway authority and a consortium of France's leading construction companies, which had been awarded the job. The two parties were concerned about how they should share the financial and technical risks associated with the project.

COWIconsult, a Danish engineering firm that has been advising the builder of the Normandy Bridge, has designed an even longer cable-stayed bridge. The proposed crossing for the Great Belt Link, a project to connect the island of Zealand and Funen in Denmark and currently Europe's biggest construction job, has a main span of 1204 metres. The design was shelved only because the client decided that navigation access to the Baltic Sea, through the waters of the Great Belt, demanded a clear span of 1600 metres.

The cancellation of the Danish cable-stayed bridge means that Japan is set to challenge the world record at the turn of the century. By 1997, they are due to have completed the Meikouchuou Bridge in Tokyo Bay, with a 590-metre span. And within another two years they expect to have the Tatara Great Bridge between Honshu and Shikoku. The main span of this cable-stayed bridge will stretch 890 metres.

1.3- CHANGE IN ATTITUDE

One of the most distinctive features of civil engineering is that designs have to be carried into effect straight from the drawing board. In a paper marking the 50th anniversary of the notorious collapse of the Tacoma Narrows Bridge in Washington state, Tom Wyatt (1990) reported: ''It is sometimes possible to test elements; it is very rarely possible to test complete structures at full scale. In consequence, innovation is fraught with uncertainty, and if progress is to be made, a client must be persuaded to buy an article unseen.''

The construction industry has been able to extend the boundaries of bridge technology by getting its designers, builders, researchers and computer specialists to work together more closely. In the case of the Dartford bridge, one company, Trafalgar house, not only designed and build the crossing, it also helped to pay for the job (Neil, 1991). This overturns the traditional approach to civil engineering contracts in Britain where the interest of the client were considered to be best served by separating designers, who want the most effective and economical solution, from builders, who want to get the job done as cheaply and as quickly as possible. With experts in different disciplines working on the same side of the contract discussions about what can and cannot be done are more open and teams are more willing to find new ways of doing things.

Technological barriers have also been broken down as firms have traded their experience worldwide. The German pioneers of cable-stayed bridges, the late Hellmut Homberg devised or checked the design of many of the greatest cable-stayed spans. Taylor's firm (Canadian) is advising the builder of Britain's proposed Second Crossing of the River Severn, a cable-stayed bridge with a main span of 456 metres on which work started 1992. In April 1991, Japan invited French and Danish specialists to discuss their work at a seminar in Fukuoka: a paper on the design and construction of the Normandy Bridge was the keynote address.

1.4- DESIGN FOR CABLES OUT CONDITIONS

Suspended highway crossings in Britain must function normally even with one of their cables missing, or with ''one cable out'', as the Department of Transport puts it. This is to ensure that traffic restrictions are not necessary should any cables need to be replaced during the life time of the bridge. As if this were not enough to test the skills of bridge designers, the department has now stipulated that new structures, including the Second Severn Crossing, must be capable of staying in service with two cables out, albeit with the volume of traffic restricted.

Engineers accept that the department, as the client, is entitled to demand whatever it wants. But many recognise that such constraints make it extremely difficult to analyse the structure in order to define the most demanding combination of loads for different parts of the bridge and still come up with an efficient, economical design. ''Frightening'' is how Roger Postlethwaite, one of Britain's leading bridge designers, describes the prospect of analysing all possible combinations of the structure (Neil, 1991).

1.5- MOTIVATION

It is quite clear that nowhere in the construction is the competitive element in design so apparent as it is in the bridge field. It is evidenced by the number of bids submitted based on design alternatives with which contractors participate in design-and-build contracts. Bearing in mind the relatively high cost of preparing a tender in an alternative design or design-and-build situation and the large saving in actual construction cost necessary to cover design fees it must be concluded that the design can have a significantly greater effect on the final price in bridgeworks than in the other forms of construction. It is the horizontal nature of bridge construction which offers greater scope for cost savings through design and consequently the design can have a much greater effect on the cost of construction.

The following factors are some of the reasons which motivated the author to investigate the effects of locating of the worst loading combinations on the design and analysis of cable-stayed highway bridges:

1- The design and analysis of a cable-stayed bridge consist of several stages. The first involves calculations to give preliminary sizes to the deck, pylons, and stays. The second stage, final calculations are prepared, determining the strength and deformations based on the final dimensions including second order effects as well as material non-linearity. Then, the design must be completed with a dynamic analysis. It is often necessary to repeat this process several times in order to achieve good use of the material, particularly so far as the stays are concerned.

2- The analysis and design of cable-stayed highway bridges is a complex problem. The load analysis of these structures subject to BD37/88 UK highway codes requirement is a very demanding task where for each analysis and redesign cycle a large effort is required in order to locate the worst combination of loads. This situation was made even worse by the recent inclusion of two cables out conditions as a design criteria for cable-stayed bridges (Dept. of Transport, UK). All that has made the load analyses completion, for even one structural configuration, to be an expensive task and called upon effective search methods.

3- Cables out conditions and highway loads defined in BD37/88 (1989) are of combinatorial nature and depend on influence line diagrams (ILD's) for the solution process. Problems associated with ILD's could be summarized as: (1) ILDs may be discontinuous at sections under consideration and may have different forms at various portions of the bridge; (2) ILD's ordinates are discrete and it is difficult to express them in closed mathematical form. It can then be concluded that the use of mathematical optimization programming methods is not feasible and alternative methods of search should therefore be investigated. For this type of problems, exhaustive methods and Genetic Algorithms are two optimization strategies which can be used.

4- Combinatorial methods seek the optimum solution by doing an exhaustive search which tries every single possibility in the search space. Although exhaustive methods guarantee the location of optimum solution, for highly complicated structures such as cable-stayed bridges where the final design/analysis is usually achieved after so many iterations, Combinatorial exhaustive methods are considered to be expensive solutions and should be avoided. The recent emergence of Genetic Algorithms (GAs) into structural optimisation and their capabilities of handling discrete optimization problems have made them a fertile ground to be explored.

In essence the thesis investigates the use of GAs for the determination of worst combinations of loadings and stay removals. These worst combinations are themselves affected by the design - the configuration, relative stiffness of the different cable groups, and the extent of prestressing or lack of fit in the cables. Under permanent actions, the cable forces must be adjusted in order to obtain the required bridge profile and the optimum distribution of the internal forces. Under traffic loadings, the deck distributes loads between the stays, which work as elastic supports. For cables-stayed bridges the construction process and sequence is a vital design condition and control of the geometry and dead loads during construction is essential. In any real design all of these aspects, together with temperature effects tolerance and settlement effects, etc., need to be considered. However, the main purpose of the thesis is to investigate the potential for the application of GAs to such design situation in which the combinatorial problem of load definitions and stay removals play a particularly important role in contrast to many classic optimisation problems which involve solely one load condition without consideration of member failures removals. For this reason problems associated with highway loading defined in BD37/88 and stay removal conditions will be given special attention in the context of using GAs for the locating of the worst loading/stay removal combinations. It should be noted that **'optimisation'** is only used in the context of a predefined bridge configuration which could mean a single aspect of variation of cable sizes where any prestressing and/or configuration adjustments are reflected and already been incorporated in the cables forces.

1.6- ORGANISATION OF THE THESIS

In Chapter 2, a review of the parameters which influence the bridge design is presented. A survey of three generations of optimisation techniques (Expert Systems, Mathematical Programming, and Genetic Algorithms) is presented. The application of these methods into structural optimisation together with the strengths and weaknesses of each method is discussed.

In Chapter 3, an introduction of the concept of cable-stayed bridges and the relation between analysis and design is discussed. Also in this Chapter, the modelling of the cable-stayed bridge and the effects of the linear and nonlinear analysis models on the design solution are addressed.

In Chapter 4, the concept of Genetic Algorithms is discussed in detail. Different computational schemes and procedures are described. A general framework for the application of Genetic Algorithms in structural optimisation is presented.

In Chapter 5, parametric studies for the behaviour of cable-stayed bridges subject to traffic loads defined in BD37/88 is presented. Several parameters which governs their applications have been addressed. This chapter presented Genetic Algorithms as optimisation tools for the determination of maximum effects in cable stayed bridges subject to traffic loads defined in BD37/88. The five components of GAs, presented in Chapter 4, have been revisited with the scope of presenting parametric studies on the behaviour of cable-stayed bridges subject to these loads. The solution of the Genetic Algorithm model is based on the use of influence lines diagrams.

In Chapter 6, the behaviour of cable-stayed bridges under one and two cables out conditions is mathematically modelled. This Chapter has also discussed the implications of including cables out conditions as a design criteria on the prospect of analysing all possible loading combinations in order to achieve a functional cable-stayed bridge and called upon effective methods of search. GAs are used for exploring the search space of load combinations for cable-stayed bridges subject to two cables out. Issues confronting the implementation process of GAs have been discussed in details.

In Chapter 7, the limitations of the proposed models, their advantages and shortcomings, and the prospects for their future application are discussed, and the final conclusions are drawn.

CHAPTER 2

BRIDGE DESIGN: LITERATURE REVIEW AND CRITICISMS

2.1- INTRODUCTION

It hardly needs setting down that the bridge designer's aim will always be to achieve a functional structure at a minimum cost. ''Function'' when used in its broadest inference usually means the satisfaction of four important criteria; appearance, safety, durability, and serviceability. Nowadays requirements for the last three items are generally stated with some precision in codified rules on rational considerations of the structure's intended use and environment. The first cannot be taken further than saying the appearance should be aesthetic. But beauty is not restricted to visual qualities, and care is therefore needed. Beauty or aesthetic experience are usually derived from the design considered as a whole.

It is the solution to the achievement of function in each of these four areas which will determine the form of a bridge. The cost should strictly be the lifetime cost, which is difficult to predict accurately at design. However, if the specified durability requirements are met, then the total cost will be greatly influenced by the initial cost, and this in turn is determined largely by quantities of materials and methods of construction. The use of materials and methods of construction may be identified as the principal controls which the designer has over the design. Such a view of the bridge design process is summarized in Fig. 2.1.

Materials and construction method are considered ''controls'' because they may be altered without necessarily changing the function of the bridge or at least without causing a failure to meet the functional requirements. The same two controls apply to many other types of manufacturing activities.

If we accept the wider meaning given to bridge aesthetics that is the achievement of appropriate degree of safety, durability, appearance, and so on, which are all considered to contribute to aesthetic merit, and therefore the character of finished bridges, then by examining the treatment of the two controls, which in turn influence and are influenced by the requirements, then the reasons for some of the differences in style suggested will become more clear. This chapter investigates how these controls, the search for minimum weight and ease of construction, have been handled by engineers in different countries with specific reference being made to the UK and Japan.

2.2- MINIMUM WEIGHT AND FORM

It is not just in civil engineering that the search for minimum weight is a main goal to be achieved. Quantity of material is an important factor in most design fields. Everyone naturally tries to achieve as much as possible using as little as possible. The important question for design criticism is how this aim has been compromised against the achievement of function and appropriate cost, and against the other control, construction method. This is a compromise over which the engineer has primary control; it is one point at which his aesthetics is exposed in the finished product.

In some sort of engineering design weight has a very great influence on serviceability and safety. Aerospace design is the limiting example. The achievement of minimum weight in this field will obviously be more dominant over constructability and appearance than in construction. For bridge design it is also desirable to reduce material weight to a minimum necessary, but this aim will not dominate to the extant that it does in the aircraft design. Because of a reduction in the necessity of minimum weight, there is more design space available, and it might be expected that there will be differences in the degree of importance given to weight reduction in the UK and Japan.

Applied loads will obviously have a major influence on the quantity of material used. Four main loads need to be considered; traffic, wind, temperature and earthquake. The wind and temperature climate in the UK and Japan do not differ significantly. Seismic loading is rarely a governing load case for bridge superstructure, although its presence is a further incentive for the reduction of dead weight. Traffic loads in the two countries are compared in Fig. 2.2 in terms of the specified UDL component of load to be used in bridge design. There is a significant difference, particularly as regards British bridges designed before 1982. However, the post-1982 British loading is quite comparable with the Japanese, and those in the latest revision considerably greater.

Self weight is of great importance for the economy of suspension bridge structures. Current developments in the new Kevlar-type cable materials of very much reduced mass, offer the potential to increase limiting span of suspension bridges from the current 3,000m to around 5,000m (Parsons, 1988). Loads due to self-weight in the towers and cables of suspension bridges typically amount to about 85% of the total. Stiffening girder structures are therefore normally dimensioned to a minimum necessary for distributing load between hangers, and to achieve sufficient torsional stiffness to provide aerodynamic stability.

The box section on the Severn Bridge (UK) allowed sufficient torsional stiffness to be achieved with a box made of thin stiffened plates. Thicknesses were 11.4mm for the upper deck plates, and 9.5mm elsewhere around the box. This, together with the reduction in wind load from the deck, drastically reduced the loading on the towers leading to a great reduction in tower steel.

It is well known that the Severn Bridge has suffered structural damage due to fatigue. Strengthening work has recently been completed to the girder, tower and hangers. The predicted life for the Severn hangers was not really satisfactory since the design was carried out using traffic loading derived from a survey conducted in the early eighties. The traffic volume and proportion of heavy vehicles was very much greater than that which the original designers were asked to consider (Roberts, 1968). Clearly the sizing of elements would have been increased if the bridge were designed for present traffic conditions.

The Severn bridge probably represents the extreme to which lightness was taken by British suspension bridge designers. In general the British designs have been comparatively light, certainly in comparison with Japanese designs.

A direct weight comparison for suspension bridges is less meaningful for cable-stayed bridges. This is so because of the wide variety of cable and tower arrangements in use. The popular use of a small number of stays on early bridges (Wye, Erskine, Lyne, Myton) in the UK has not helped in the achievement of girder structures with minimum weight. It was suggested that the few-stay arrangement was rather a favoured form visually because it gave a simple and minimal appearance. The two recent bridges, Kessock (Japan) and Dartford (UK), have both multiple-stay arrangements, and the minimalism has been expressed more in the girder structures, by using open rather than box sections. On Kessock there was an effort to achieve even greater lightness, which was only forced out by the application of Interim Design and Workmanship Rules following the accidents with box-girder bridges. The designers themselves criticized these rules, making a comparison with the Rees Bridge in Germany which had longer spans and was considerably lighter (Knox et al, 1984).

Certainly Kessock did end up much heavier than previous British examples. It actually achieves a steel weight of $1.21t/m^2$ compared with about $0.67t/m^2$ for Erskine and Wye. Dartford also has a unit weight well under $1.0t/m^2$, so that Kessock should probably be seen as an exception, resulting from conservatism in the wake of box-girder problems. Japanese bridges have shown a wide range of steel weights. Katsushika seems reasonably light at $0.75t/m^2$ when consideration is taken of its curved girder planform.

2.3- CONSTRUCTION PROCESS AND FORM

In the discussion on minimum weight the interdependence of decisions on materials with those on construction method emerges as a major factor. It appears that the ideal of minimum weight is given more importance relative to ease of construction in the British case. Elements and forms determined as a result of construction method become more important in Japanese bridges.

On Severn and Oshima for example, it was the importance attached to construction which influenced the design of the tower plating and horizontal connections. The Severn designers saw the potential for carrying out all jointing work inside as a major benefit, wishing also to eliminate external signs of jointing. Their design however, increased complexities in fabrication. For the Oshima towers emphasis was placed on simplicity in the fabrication of the tower sections, with less concern for external working on site or the visibility of the joints externally in the finished structure. The joints on Oshima have become an important aspect of the visual expression in the bridge. They give a clear statement about the assembly process; it was seen that blocks were brought to site in equal lengths for the main legs (Burden, 1991). Given the taper in leg cross-section this further suggests the favouring of a regularity in length over that of section weight.

Although not directly affecting form, it is perhaps instructive to look at the role played by contractors in the development of new construction methods for suspension bridges. An area where Japanese contractors have been particularly active is in cable erection methods. Aerial spinning was used until 1968 when the Kinpiria Bridge was built using prefabricated parallel wire strands (PPWS). Full-scale adoption of PPWS seems to have been stimulated in Japan by the construction of the Newport Bridge in the USA (Birdsall, 1988). After Kinpira, doubts remained in Japan about the success of the technique. To allow comparison with aerial spinning, the Kamiyoshinogawa Bridge was constructed in 1971 with one main cable aerially spun, the other using PPWS. The superiority of PPWS seems to have been established as a result of this experience. It has been used on almost all subsequent bridges. The Japanese claim time saving, high cable densities and low occurrence of wire crossing as merits of the technique (Konishi, 1980).

2.4- MINIMUM WEIGHT VIA OPTIMISATION TECHNIQUES

The ability of engineers to produce optimal design has been severely limited by the techniques available for design optimisation. Typically, much of the development effort has focused on simulation programs to evaluate design parameters. It is the design, implementation, and evaluation of these programs that most directly call upon the engineer's expertise. One problem currently being addressed is how to use the information provided by the simulation in the iterative process of searching the parameter space for better design. Given an evaluation of a setting of the parameters governing a design, on what basis should the choice be made of the ''best'' parameters to evaluate next? Because design space are combinatorially and the time in which to develop new design is limited, relatively few design points can be evaluated.

2.4.1- SURVEY OF Optimisation TECHNIQUES

Optimisation techniques frequently applied in design are: expert systems, numerical optimisation, and genetic algorithms. The following discussion summarizes the major advantages and disadvantages of each.

2.4.1.1- EXPERT SYSTEMS

Expert systems codify knowledge about a domain in the form of IF THEN rules that are manipulated by forward and backward inference techniques to provide solutions to design problems. Expert systems have been developed in many areas covering wide range of applications. Most of these applications fall into one or more of the following categories (Adeli, 1988): (1) Diagnosis; (2) Design; (3) Data interpratation; (4) Planning; and (5) Education. Expert systems offer advantages in making use of the engineer's domain knowledge; they provide solutions to design problems efficiently; and they explain how these solutions were obtained. They also have disadvantages: they require rules that describe a domain completely; they cannot adapt readily to change within a domain; and they are domain-dependent. Even for a design of minimal complexity, rule completeness is not possible because of the mismatch between the way engineer express their knowledge and the format required by the rule representation in the computer. Knowledge acquisition is always hindered by the inability of humans to express all their knowledge and by the errors introduced as knowledge is transferred from a design engineer to a knowledge engineer (Quinlan, 1987).

In the area of structural engineering several expert system have been developed. Maher (1984) (HI-RISE) and Sriram (1986) (DESTINY) have both developed expert systems for the preliminary design of high rise buildings. Kumar and Topping (1988) developed a rule based expert system (INDEX) for the design of industrial buildings. Adeli and Balasubramanyam (1988) developed an expert system (BTEXPERT) aimed at the optimum detailed design of four types of bridge trusses. Jayasinghe (1992) developed an expert system for the design of prestressed concrete bridges. Cameron and Grierson (1989) developed an expert system for the least-weight design of structural steel buildings.

In the area of structural design Jayasinghe (1992) concluded several characteristics. Expert systems for design (except those concentrating only on conceptual design) try to integrate the priliminary design, detailed design and design evaluation in one expert system, and all expert systems support 'design by repeated analysis' considering structural design as an iterative feedback process. Design modification is carried out using the redesign knowledge incorporated in separate kwoledge modules. Priliminary solutions have been derived on the basis of past experience incorporated as heuristics or as a database in the kwoledge module where the appropriate solution is found by searching through this knowledge. The emphasis placed upon design by repeated analysis in the existing expert systems is clear evidence that knowledge incorporated as heuristics or databases alone is not sufficient to produce good preliminary solutions which lead to efficient designs.

2.4.1.2- NUMERICAL Optimisation

Numerical optimisation algorithms provide a significant capability for the automated optimal structural design problem. Several publications, text books (Haftka 1992, and Templeman 1982), and reviews (Venkayya 1978, Haftka and Granghi 1986, Topping 1983, Kirsh 1989) on the topic have been made.

Numerical optimisation uses gradient approximations to calculate search directions leading to an optimum. Programming methods deal with solution procedures for a certain class of optimisation problems, and especially with the techniques of linear programming, non-linear programming, geometric programming, and dynamic programming. Depending on the type of problem, one or more of these techniques can directly or indirectly be applied.

Very few structural engineering problems can be grouped under linear programming problems since the objective function and/or the constraints are non-linear functions of design variables. However, it is sometimes possible to cast a non-linear programming (NLP) problem into an approximately equivalent linear programming problem (LPP). The application of linear programming techniques was addressed by Pope and Schmit (1971). More recently Rinaldi (1986) (sited in Haftka, 1992) presented a paper on the projective method for linear programming with box type constraints.

Mathematical nonlinear programming algorithms provide a significant capability for the automated optimal structural design problem. Research and development of efficient and reliable algorithms for non-linear programming has recieved considerable attention in recent years. Several publications have added to the generality and versatility of these algorithms in the design of structures subjected to both static and dynamic loads (Hajela and Lamb, 1986). Kirsh and Topping (1992) applied nonlinear programming for minimum weight design of structural topologies. Typical optimum designs require a minimization or maximization of a stated objective and simultaneous satisfaction of several design constraints. The more efficient nonlinear programming for this class of problems are gradient-based, and require at least the first derivative of the objective function and constraints with respect to the design variables (Rao, 1979). Such "hill-climbing" algorithms are extremely efficient in locating a relative optimum closest to the starting point in the design space. In design applications the optimum may be obtained from several initial points in the design space. Even then, no guarantee exists of obtaining the global optimum (Hajela, 1990).

Another technique that can be used for structural optimisation is geometric programming (GP). This technique does not attempt a direct solution of a problem, but, by a series of mathematical transformations, sets up a dual problem in which the constraints are linear. The dual problem is then solved by using the linear programming technique. Geometric programming is best suited for component optimum design where the number of parameters is small in comparison with that of total structure. Hajela (1986) presented a geometric programming strategies for large scale structural synthesis.

The technique of dynamic programming (DP) is used for solving problems in which the objective function, as also the constraint functions, are separable. Unlike other mathematical programming methods DP does not have a simple, rigorous mathematical formulation. DP is a solution philosophy for serial systems rather than a rigorously detailed mathematical technique. If the number of constraints is large, DP requires substantial computational effort.

In summary, the advantages of numerical optimisation are its mathematical underpinnings, its general applicability to engineering designs, and its wide application base. The disadvantages are its inability to exploit domain knowledge, its extreme sensitivity to both problem formulation and algorithm selection, its need for large amounts of computational effort, and its assumptions that the design variables are independent and the parameter space is continuous. Numerical optimisation techniques are good at "exploitation" but not "exploration" of the parameter space. They are successful at exploitation because they focus on the immediate area around the current design point, using local gradient calculations to move to a better design. Since no attempt is made to explore all the regions of the parameter space, however, numerical optimisation can easily be trapped in local optima or by constraints in a region of the parameter space far from the optimal design (Booker, 1987).

2.4.1.3- GENETIC ALGORITHMS

The genetic algorithm is a recent addition to methods of optimisation suitable for use in structural design. Genetic algorithms take an initial set, or population, of design points and manipulate that set with the genetic operators of selection, crossover, and mutation to arrive at an optimal design. Seeding is the process of providing the initial set of design points. Although the seeding is typically performed by random selection, some systems such as EnGENEous (Powell et al, 1989) have used past designs from the parameter space to provide a portion of the initial design set. Genetic algorithms are based on the heuristic assumptions that the best solutions will be found in regions of the parameter space containing a relatively high proportion of good solutions and that these regions can be explored by the genetic operators of selection, crossover, and mutation.

Several researchers have used genetic algorithms to perform structural optimisation. The first application was presented by Goldberg and Samtani (1987) who optimized the member cross-sectional areas of a 10-bar truss. Rajeev and Kirshnamoorthy (1992) extended the work of Goldberg and Samtani by optimizing the member cross-sectional areas of 10-, 25-, and 160-bar trusses where the mutation operator was ignored.

In his early publications Jenkins (1991a,91b,92) presented a plane frame computational environment suitable for the optimum design of structures using genetic algorithms. In these papers Jenkins gave considerations to the basic genetic algorithm operations of selection, crossover, mutation and parameter scaling and to the analysis and design interfaces. In order to verify results produced by genetic algorithms, Jenkins used previously published work where optimum solutions were derived using different methods of optimisation. Examples included: a three-bar truss by Stojanovski and Alekeovski (1986), a thin-walled cross section by Thenvendran (1985), an 18-bar-truss cantilever truss by Campos Schwefel (1989), and a cable-stayed bridge by Hejab (1986). Results were encouraging and have shown that genetic algorithms are actually capable of locating optimum or near optimum solutions with some disadvantages regarding computational time. In order to reduce computational time and search space, Jenkins (1993, 94) proposed a space condensation heuristic as an enhancement of the genetic algorithm. The heuristic uses a structured record of the parameter values selected by the algorithm. Those associated with 'good' solutions are recorded positively and those associated with 'poor' solutions are recorded negatively. The record is then used to progressively reduce the size of the space being searched. Although space condensation heuristic materially assists the genetic algorithm in searching a large combinatorial space, much remains to be done including a fundamental analysis of the mechanics of the 'variable-by-variable' application of the genetic algorithm.

Hajela and Lin (1990, 1992a, 1992b, 1993a,1993b) have published several papers on the use of genetic algorithms in structural optimisation. The application of genetic search in problems with disjoint and nonconvex design spaces was discussed by Hajela (1990). In this paper the cross-sectional dimensions optimisation of a two-beam grillage structure subjected to a uniformly distributed load, a two-element thin walled cantilever torsional rod subjected to a harmonic excitation, and a 10-member truss subject to a sinusoidal load were presented. The paper has also presented the drawbacks in function evaluations necessary to locate an optimum and suggested possible strategies to overcome this limitation. In another paper, Hajela and Lin (1992a, 1992b) proposed a multicriterion optimisation 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. Genetic search strategies in large scale optimisation have also been addressed by Hajela and Lin (1993a). In this paper two advanced search strategies referred to as multistage search and directed crossover have been developed. The multistage genetic search is a specialized strategy for optimizing problems of large dimensionality so as to identify promising regions in the design space in earlier generations of evolution with a relatively smaller population size. Directed crossover is a bitwise generational gradient which is developed to determine which bit string offer the most potential for improving the design fitness. Directed crossover has been shown to be specially effective in problems of high dimensionality. Hajela and Lin (1993b) presented EVOLVE as a multiple strategies genetic based optimisation code. The program included two advanced search techniques (discussed earlier) known as directed crossover and multistage search strategy (Hajela and Lin, 1993a). Also the program included other advanced techniques such as: sharing function implementation, mating restrictions, and automatic encoding and decoding of the design variables. Three case studies were considered. The first is used to demonstrate the effect of sharing function implementation in a multimodal design space. The second, is a rivited lap joint efficiency maximization problem, where the design space consists of a mix of integer and discrete design variables. The third involves the sizing of a 25-bar truss in which multistage genetic search and directed crossover strategies were used. EVOLVE could be considered as a powerful tool which offers flexibility of use in a large number of routine optimisation problems.

Adeli and Cheng (1993) presented a minimum weight design for the optimisation of three space trusses (12-, 25-, and 75-bar trusses) by integrating a genetic algorithm with the penalty-function method. It was shown that, robustness of the integrated genetic algorithm can be improved by employing a variant penalty-function-coefficient strategy where the value of this coefficient is increased after every so many iterations. However, several drawbacks of the proposed approach lacks the following: (1) Convergence is usually achieved with high penalty-function-coefficient which in turn causes ill conditioning in the optimisation process and results in numerical instability or slow convergence; (2) the values of the penalty function coefficient is unknown and it requires many numerical experiments and/or experience in order to choose a suitable value and updating process for the penalty function coefficient. To overcome these shortcomings, Adeli and Cheng (1994a) presented an augmented Lagrangian genetic algorithm for the minimum weight design optimisation of large structures (such as high-rise building structures and space stations with several hundred members) by the hybrid genetic algorithm. Compared with the penalty function-based genetic algorithm (Adeli and Cheng, 1993), only a few additional simple function evaluation were needed in the algorithm. Furthermore, the trial and error process approach for the starting penalty function coefficient and the process of arbitrary adjustments were avoided. The algorithm was shown to be general and can be applied to a broad class of optimisation problems. Adeli and Cheng (1994b) extended their previous work (1993, 1994a) by presenting two concurrent augmented Lagrangian genetic algorithms for optimisation of four large space structures (26- and 35-story high-rise towers, 72-bar truss and a Geodesic dome space truss) utilizing the multiprocessing capabilities of high-performance computers. It was observed that the performance of both algorithms improves with the size of the structure, making them suitable for optimisation of large structures.

Haftka and his colleagues (1993,a,b,c, and d) have published several papers which contributed to the implementation of genetic algorithms in structural optimisation. Haftka et al (1993a) presented the challenges that face the design of composite structures against buckling and proposed a minimum weight design for stiffened panels with buckling constraints using genetic algorithms. Haftka et al (1993b) introduced a binary tree data structures and a local improvement scheme as two approaches for reducing the number of analyses required by a genetic algorithm for the stacking sequence optimisation of composite plates. The local improvement scheme was found to have substantially reduced cost of the genetic optimisation. Haftka and LeRiche (1993c) presented the use of a genetic algorithm to optimize the stacking sequence of a laminated plate. Buckling constraints as well as contiguity and strain constraints were considered. A new genetic operator -permutation- was proposed and shown to be effective in reducing the cost of the genetic search. The capability of genetic algorithms to locate several optima in a single execution have been demonstrated. Haftka et al(1993d) presented the optimal placement of tuning masses, actuators and other peripherals on large space structures using genetic algorithms. An example of minimizing the difference between the two lowest frequencies of a laboratory truss by adding tuning masses was used for demonstrating some of the advantages of genetic algorithms. The relative efficiencies of different codings are compared using the results of a large number of optimisation runs.

Rao et al. (1991) addressed the optimal selection of discrete actuators locations in actively controlled structures via genetic algorithms. Rao (1993) used a genetic algorithm approach for multiobjective optimisation of structures. Sugimoto (1992) applied genetic algorithms for the discrete optimisation of truss structures. Sugimoto et al. (1993) explored the application of genetic algorithm to the multiobjective design of retaining wall structures. Deb (1990) presented a genetic algorithm for the optimal design of a welded beam. Thierauf and Cai (1993) applied genetic algorithms for minimum weight design of 10-, and 200-bar truss. Wang et al. (1993) proposed an efficient genetic algorithm for large scale built-up structural optimisation. Koumousis and Georgiou (1993) applied genetic algorithms in discrete optimisation of steel truss roofs.

Shape and topological optimisation have been addressed by several researchers. Richards and Sheppard (1992) used a classifier system to perform shape optimisation. In their work, the mass of a two-dimemsional structural component under tensile loading was minimized, given a maximum allowable stress constraint. Sandgren et al. (1990) and Jensen (1992) investigated genetic algorithm based topological optimisation of structural components. Sandgren et al. (1990) optimized the cross-section of an automotive body panel, both of which were subject to maximum stress and displacement constraints. Jensen (1992) extended the original approach of Sandgren et al. to include improved bumper beam cross-section results, a multi segmented beam example, a plane-stress, truss-like example, and two automotive body panel examples. Chapman et al. (1993) extended the work carried out by Sandgren et al. (1990) and Jensen (1992) to the structural topology optimisation of the following structures: (1) optimisation of beam cross-section topologies; (2) optimisation of cantilevered plate topologies; (3) efficient techniques for using finite element analysis in a genetic algorithm-based search. Fujita et al. (1993) presented a hybrid approach for optimal nesting using a genetic algorithm and a local minimization algorithm. Sakamoto and Oda (1993) used genetic algorithms for optimal layout design of truss structures.

Grierson (1994) has coupled genetic algorithms together with neural networks to generate an evolutive-cognitive computational model for conceptual design that relies directly on the experience and judgement of the designer to arrive at alternate best-concept solutions. An illustration was made on the conceptual design of a bridge structure. Although the model is still in its early stages of development, it can be applied to find best-concept design solutions in essentially any problem domain since it is not problem specific and it only requires the names of design attributes and their ranges of possible values as input.

Mesh partitioning and mesh generation are another areas where genetic algorithms have been successfully implemented. Topping and Khan (1993) presented an optimisation and artificial intelligence-based approach for solving the mesh partitioning problem in parallel finite element analysis. The coupling of genetic algorithms (used for the optimisation module) and neural networks (as the predictive module) was described. It was shown that a genetic algorithm linked to a neural network predictive module would limit the computational load and the number of design variables for the decomposition problem. Guesta et al. (1993) proposed a triangular mesh optimisation process using genetic algorithms. The main aim is to develop a tool for regenerating meshes, in non-convex two dimensional domain where high degree of automation is required in engineering applications with the finite elements method.

Sathyanarayana et al. (1993) used genetic algorithms for the optimum resources allocation in construction projects. Furata et al. (1993) extended the use of genetic algorithm for the aesthetic design of dam structures. Miles et al. (1993) investigated the use of genetic algorithms in a computer system for a multi-objective engineering problem.

This section has reviewed the application of genetic algorithms in structural optimisation. It shows that genetic algorithms have found wide acceptance and have emerged as effective optimisation tools. Genetic algorithms offer a number of advantages: they search from a set of designs and not from a single design; they are not derivative-based; they work with discrete and continuous parameters; and they explore and exploit the parameter space (Goldberg, 1989). The major disadvantage of this strategy is the computational cost of the large number of runs of the design code needed to evaluate a set of designs for each generation.

2.5- CONCLUSION

This chapter proposed that materials and construction methods are considered as "controls" in the design process. The use of a minimum of materials and provision of a simple construction method would seem to be universal engineering ideals, but the weight given to each will differ depending on cultural and industrial environment.

Minimum weight through optimisation techniques has also been addressed. Expert Systems, Numerical methods, and Genetic Algorithms are three different generations of optimisation strategies which have been reviewed in the context of structural optimisation. Strengths and weaknesses of each strategy have been discussed. Expert systems are domain dependent and cannot adapt readily to change within a domain. Numerical optimisation techniques use local gradient calculations to guide the search through parameter space. They can easily be trapped in local optima and for large scale problems do not work well. Expert systems and Numerical optimisation have fundamental strengths but tend to suffer in constrained optimisation problems, where there are large, nonlinear spaces and there is incomplete domain-dependent knowledge to guide the search. It was shown that is in these situations that the genetic algorithms can be designed to work.

In the next Chapter the problem of cable-stayed bridges is presented while the remainder of the thesis is devoted to the application of genetic algorithms for the optimisation of the worst load combination on cable-stayed bridges due to traffic loads (as defined in BD37/88) and for the cables out conditions.

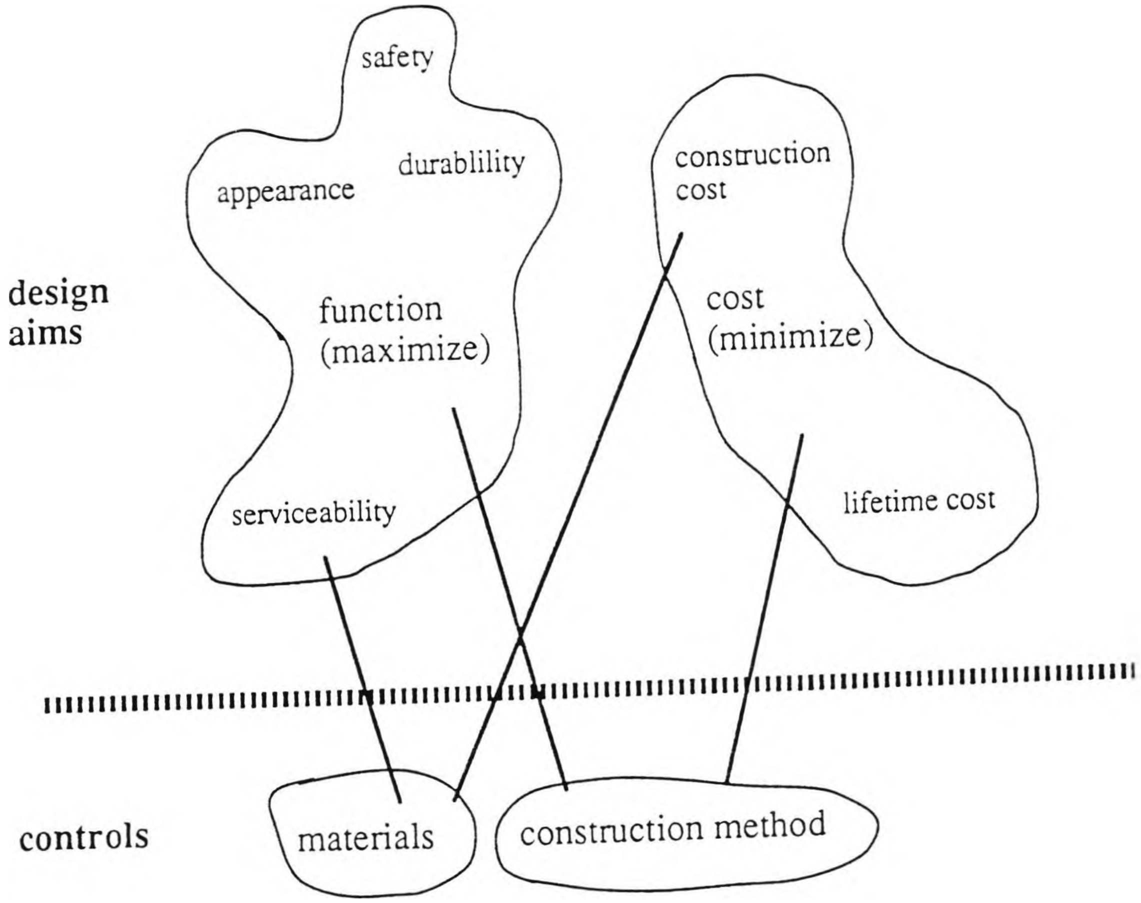


FIGURE 2.1. A REPRESENTATION OF BRIDGE DESIGN SPACE.

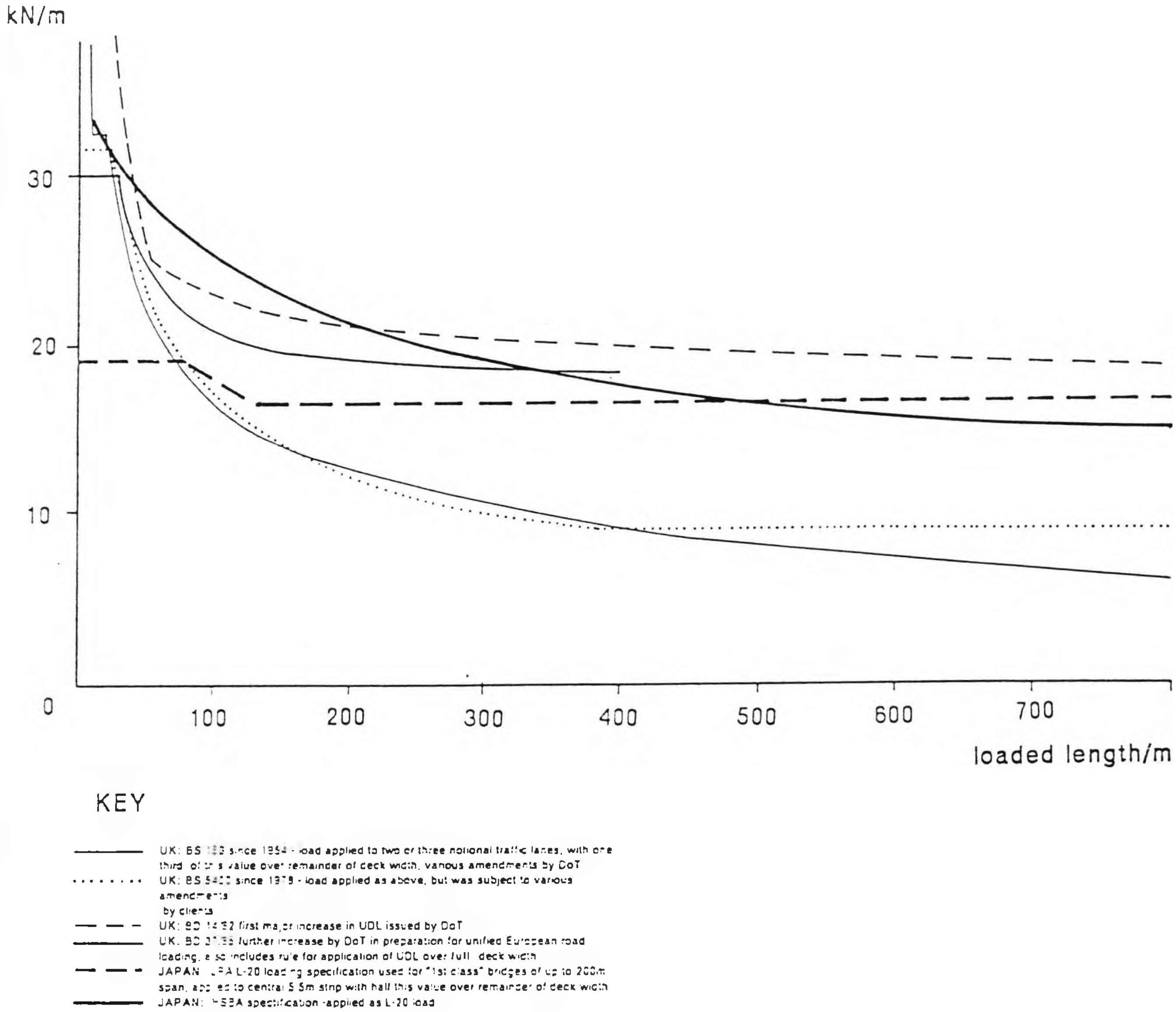


FIGURE 2.2. COMPARISON OF UDL HIGHWAY BRIDGE LOADING SPECIFICATION IN JAPAN AND THE UK (BURDEN, 1991).

TABLE 2.1. PARAMETER R FOR SELECTED JAPANESE AND BRITISH BRIDGES.

name	span/m	height/m	I /m⁴	mass t/m	f_n N/mm²	R
Wve	235	30	1.6	10	450	0.027
Tovosato	216	30	0.94	12	380	0.026

Notes

1. Young's moduli taken as follows: steel 20.5t/mm², locked-coil rope 16.0t/mm², PWS 20.0t/mm²

C H A P T E R 3

OVERALL VIEW OF CABLE-STAYED BRIDGES

3.1- INTRODUCTION

Modern cable stayed bridges began to be erected, mainly in Germany, in the middle of 1950's. Since then cable stayed bridges have become one of the most popular type of bridges, mainly for their economy and aesthetics, and have been erected all over the world ranging from short-span pedestrian bridges to long-span bridges with spans up to 900m.

A key feature of cable stayed bridges is the variety of forms that they may take. The geometry of pylons, composition of cables, geometry of girders, geometry of cable arrangement, number of cable stays, number of spans, and other details can be widely selected and freely combined.

In this chapter a brief description of the geometrical consideration, overall analysis, and loadings on cable stayed bridges is presented.

3.2- TOWERS

The shape of the towers is decisive for the aesthetic expression of cable stayed bridges. Therefore, their design should always be refined, choosing good proportions, tapering the shafts, etc. Nowadays most towers are built in concrete because concrete towers are considerably cheaper than steel towers. Concrete offers more flexibility towards good shaping.

For large spans, up to 500m, free standing towershafts give the best appearance (Fig. 3.1). No transverse bracing is needed if the cables are in a vertical plane and if the centre of gravity of the cross section is close to the plane of the cables.

For high level bridges, a transverse bracing between the tower shafts just below the deck may be needed for the horizontal wind reactions. In bridges with larger spans, the cable anchorages of a semi-fan arrangement may take only a small portion of the tower head. In this case, a transverse bracing bar just at the lower end of the anchor zone allows to spread the shafts a bit so that the deck can pass through without any curtailment (Fig. 3.2). The bracing above the deck level should be slender and should look light between the thin cables.

In regions with strong wind gusts and for very long spans, the A-shaped tower is the optimal solution for appearance and wind stability (Fig. 3.3).

For Bridges with cables in one plane it is best to design a single slender tower in the median strip and protect the cables and the tower with a strong high guard rails. Underneath the box girder, there must be a sufficiently wide pier for bearings to resist the torsional moment of the superstructure (Fig. 3.4). The cables can also be hung up at an A-shaped tower, which may take horizontal cable action at the top if the alignment of the bridge is curved and the anchorages follow the curved line of the median (Fig. 3.5).

Towers inclined backwards (Fig. 3.6) offer neither technical nor economical advantages, but rather a more thrilling appearance. A forward inclination towards the main span causes uneasy feelings of approaching collapse. Towers should normally be vertical.

3.3- CABLES

In cable stayed bridges, the cables play the most important role, structurally and aesthetically. The progress in the cable strength has greatly contributed to the development of cable stayed bridges.

The cables initially used to build cable stayed steel bridges were characteristic of specific countries: spiral ropes in UK (Fig. 3.7a), locked-coil ropes in Germany (Fig. 3.7b), and parallel wire strand cables in Japan (Fig. 3.7c).

As the span of cable stayed bridges increases, the number of cable stays necessarily increases, along with cable tension, the number of ropes per cable, and the complexity of cable anchorage, as well as the difficulty of cable erection. The multiple stay system was adopted to solve these problems, as well as to improve aesthetics and produce such advantages in maintenance as ease of rope replacement. At the same time several types of cables were developed to suit the multiple stay system. These cables have high-fatigue strength anchorages and eliminate or minimize corrosion protection work on site.

In concrete cable stayed bridges, the cables used are mostly steel strands, steel wires, and steel bars for prestressing in common with other prestressed concrete structure. Like cable stayed steel bridges, some cable stayed concrete bridges are constructed of locked-coil ropes. Many of the prestressing cables in cable stayed bridges are advanced versions of conventional post-tensioning tendons and have improved properties required for cable stayed bridges, such as fatigue strength and corrosion protection.

3.4- BRIDGE DECK

Bridge decks are supported on cross beams which in turn are supported on the bridge girder which spans between the main supports.

Bridge decks can be made up of steel or reinforced concrete irrespective of material of girder. In cable stayed bridges the orthotropic steel decks are most popular. The orthotropic steel deck consist of a steel plate supported on rolled steel stiffners. In the case of orthotropic concrete deck, the slab is often supported on precast concrete beams. These stiffners can be of open sections, ie. angles, I channels, or closed sections, i.e. U, V, or Y shapes. The closed section stiffners have higher torsional stiffness. In the case of steel construction, spacing of cross girders varies between 1.8m to 2.5m, when ribs are of open section and between 4.6m to 5.5m, when ribs are of closed section. The thickness of top steel plates varies from 9mm to 20mm depending upon the spacing of the ribs. Various types of deck are shown in Fig. 3.10. For the span range of 60m to 90m it is convenient to use reinforced concrete deck while for higher spans orthotropic steel deck is recommended.

3.5- NUMBER OF CABLES

Most early cable-stayed bridges were of the single, double or triple stay system, meaning that respectively, one, two or three cables were attached to each side of each tower. Nowadays, most major cable-stayed crossings have a large number of cables. This multiple stay system has allowed the construction of thinner decks.

3.6- ARRANGEMENT OF THE STAY CABLES

According to the various longitudinal cable arrangements, cable stayed bridges can be categorized into four basic systems: radial, harp, fan and star arrangements.

3.6.1- RADIAL OR CONVERGING SYSTEM

In this system all cables lead to the top of the tower (Fig. 3.9). Structurally, this arrangement is perhaps the best, as by taking all cables to the top of the tower the maximum inclination to the horizontal is achieved and consequently it needs the smallest amount of steel. The cables carry the maximum component of the dead and live load forces, and the axial component of the force on deck structure is at minimum.

However, where a number of cables are taken to the top of the tower, the cable supports within the tower may be very congested and a considerable vertical force has to be transferred, thus making the detailing rather complex.

3.6.2- HARP OR PARALLEL SYSTEM

In this system the cables are connected to the tower at different heights, and placed parallel to each other (Fig. 3.10). This system may be preferred from an aesthetic point of view. However, the arrangement results in bending moments in the tower. In addition, it is necessary to examine whether the support of the lower cables can be fixed at the tower leg or must be made movable in a horizontal direction.

3.6.3- FAN OR INTERMEDIATE SYSTEM

The fan or intermediate stay cable arrangement represents a modification of the harp system (Fig. 3.11). The forces of the stays remain small so that single ropes could be used. All ropes have fixed connections in the tower.

3.6.4- STAR SYSTEM

The star pattern is an other aesthetically attractive cable arrangement (Fig. 3.12). However, it contradicts the principle that the points of attachment of the cables should be distributed as much as possible along the main girder.

3.7- POSITIONS OF CABLES IN SPACE

There are mainly three ways to position cables in space: double vertical planes system, double inclined planes system and single plane system (Fig. 3.13).

3.7.1- DOUBLE VERTICAL PLANES SYSTEM

Two alternative layouts may be adopted when using this system: The cable anchorages may be situated outside the deck structure, or they may be built inside the main girders.

3.7.2- DOUBLE INCLINED PLANES SYSTEM

In this system the cables run from the edges of the bridge deck to a point above the centre line of the bridge on an A-shaped tower. This system is suitable for very long spans where the A-shaped tower has to be very high and needs the lateral stiffness given by the triangular shape and the frame action.

Joining all cables at the top of the tower has a favourable effect against wind oscillations because it helps in preventing the potentially dangerous torsional movement of the deck.

3.7.3- SINGLE PLANE SYSTEM

Single plane system operates with only one vertical plane of stay cables along the longitudinal axis of the superstructure. In this case all the cables are located in a single vertical strip, which is not used by any form of traffic. This arrangement requires a hollow box main girder with considerable torsional rigidity in order to keep the change of cross-section deformation due to eccentric live load within allowable limits. This system can be used if there is a median space to separate two opposite traffic lanes. In this case no extra width is needed for the tower, and all cables at deck level are protected against accidental impact from vehicles.

3.8- CONSTRUCTION METHODS

The cantilever method is practically the only possible solution for the erection of long spans cable stayed bridges. In this method each new segment is built or installed, and then supported by a new cable or new pairs of cables which balances its weight. However, for medium-span cable stayed bridges this method turns out to be costly. The reason is that the cantilever method involves a great number of successive phases, especially for concrete bridges where the successive length are very short.

The drawbacks of the cantilever method can be summarized as:

i- Design is sophisticated and can only be carried out by specialized consulting firms equipped with appropriate computer facilities;

ii- The large number of phases lengthens the construction time and therefore all the underlying consequences in term of costs;

iii- The large number of phases multiplies the basic operations that are repeated at regular intervals. For example, it is necessary to adjust the tension in cables following the construction of each new segment which often requires the intervention of a highly specialized team each time;

iv- The large number of phases requires the need for numerous geometric verifications, which are very costly and must be repeated at every phase.

For medium-span bridges, this cost is considerable in relative terms and can result in elimination cable stayed solutions in comparison with conventional bridges. For that reason, for the last ten years, alternative methods of construction have been deployed for medium-span cable stayed bridges. These may include; construction on scaffolding or on temporary supports, placement by rotation of already built cable stayed cantilever and finally installation of the deck by the incremental launching method. In these methods all cables can be installed and tensioned in one single operation, which limits material and personnel costs and simplifies design work and geometric control.

3.9- ANALYSIS OF CABLE STAYED BRIDGES

A multiple stay system is a highly redundant system in which the stiffening girder behaves as a continuous beam supported elastically at the point of cable attachments. Except in the case of very simple cable-stayed bridges, computers are unavoidable for the optimum solution of this type of structures.

The analysis of the structure can be divided into three main parts:

i- Global, or general, analysis considering the structure as a whole.

ii- Local analysis, considering in more detail the effects of locally applied loads on the structural elements.

iii- Stress analysis, using the load effects obtained from global and local analyses to calculate stresses in members.

3.10- GLOBAL ANALYSIS

For the global analysis, the cable stayed bridge is divided into its main elements: Deck beam, pylons, piers and cables. Each element is considered as a beam or truss element and idealised as such in a frame model for a global computer analysis of the complete structure. The global analysis of cable stayed bridges should model their behaviour under **linear, nonlinear and dynamic** effects.

For global analysis, the structure is often idealised as a 2D and/or 3D beam model (Fig. 3.14). Several models should be made corresponding to different activities, each activity being specific for a particular loading analysis (linear, nonlinear and dynamic analysis). For these different models care should be taken to obtain internal forces and deformation at the same points of the structure. The models should be sufficiently detailed to obtain results at each point where a stress analysis is to be carried out. Each model should idealise the deck and pylon legs with beam elements with nodes at each cable stay anchorage. Depending upon the activity under consideration, each cable may be modelled either as a cable element or as a truss element with an apparent stiffness due to cable sag.

3.10.1- LINEAR ANALYSIS

In linear analysis displacements are considered small, and the contribution of the member forces to the balance of the external loads is stated in terms of the statical geometry of the structure. However, for cable stayed bridges this assumption has been shown to be approximate and, for large spans, unsafe. When the actual performance of the bridge is analyzed and the final deformed geometry is considered, then the loading moments, deflections and stresses have larger magnitudes if nonlinearity is neglected.

3.10.2- NON-LINEAR ANALYSIS

Although the material in the members of cable stayed bridges behaves in a linear elastic manner, the overall load-displacement relationships for the structure will be nonlinear under normal design loads. The principal sources of non-linearity in concrete cable-stayed bridges are:

a) Long cables sag under their own weight, the amount of sag being a function of the stay tension. Owing to the ratio of live to dead load , this effect will, in general, be relevant only for very long cables. Particular attention must be given to the load cases which significantly unload the stays, where the loss of stiffness of these elements can be relevant.

b) The deck and the pylons are in general subjested to important axial forces. In many cases they consist of slender elements. If deformations are relatively large, second order effects may be important, and the equilibrium conditions should be established for the deformed configuration.

c) Material non-linear effects of the concrete elements can change the relative stiffness of the various elements significantly. It may be necessary to evaluate the effect on the load-bearing behaviour of the structure.

In each case the designer must identify the relevant sources of non-linearity. If necessary, adequate techniques of non-linear analysis must be used.

It should be noted that in Chapters 5 and 6 the real non-linear behaviour, particularly relating to the effects of cable sag variations under varying tensions caused by the live loading have not been accounted for - principally because in this thesis the Genetic Algorithm process is to assist the design stage, not final analysis or optimisation.

3.10.3- DYNAMIC/AERODYNAMIC ANALYSIS

The role of dynamic forces in a cable stayed bridge could be critical in determining the feasibility of the project. There are in general three types of problems: aerodynamic stability, physiological effects on users and safety against seismic loads. The aerodynamic behaviour of this type of structure determines to a great extent its safety. Without damaging the structure, the vibrations due to wind and traffic can cause inconvenience to the users. These physiological effects are generally very subjective experiences. Analysis of all these dynamic effects including seismic load, calls for prior knowledge of the natural frequencies and vibration modes of the structure concerned.

The following sections investigate the modelling of cable-stayed bridges and discuss the effect of the model(s) on the design solution.

3.11- IDEALISED MODELLING OF CABLE-STAYED BRIDGES

3.11.1- INTRODUCTION

Model simulation of a structure consists of idealising it as a system of appropriate members which allow its behaviour to be analysed with sufficient accuracy and with a reasonable amount of calculation.

Depending on the complexity of the structure and the stage the design has reached, very different models may be used. These may be plane or spatial systems, covering the whole structure or only a part, and can consists of a wide range of members. The pylon can generally be represented by "beam" type elements. The same can be said of the deck if this actually behaves as a beam (rigid box section, vertically suspended) and also, in all cases, during the preliminary design stage (study of different layouts) and in consideration of erection (checking of several partial systems). The deck can be represented by "shell" type elements in the main design stages if its behaviour differs largely from that of a beam (lateral suspension, deformable cross-section). Use of "plate" type elements is also possible for the study of local problems, using part models. The cables can also be represented as beams by giving them a very small bending inertia and an idealised modulus of elasticity (Ernst's modulus) which makes it possible to take into account the effects of cable sag. This model simulation is especially possible when dealing with structures where the cables are sufficiently tensioned under permanent loads, so that any compression which may arise under live loads results only in a reduction of the initial tension. There are elements which simulate the actual behaviour of cables and these should be integrated into non-linear programs.

3.11.2- PLANE FRAME MODELS

The behaviour of cable-stayed bridges under the action of live loads is difficult to depict by means of simple, intuitive methods. It is thus an advantage during the initial design stages to have available a simplified design model; for example, a projection of the whole structure on to a plane, where all the elements are represented as beam elements. In such a case, one difficulty lies in the representation of the connections between the pylons and the deck (Fig. 3.14a). Because of the simplicity of the introduction of the data and the speed at which calculations are carried out, this model can not only serve as a basis for choosing the dimensions of the structure, but can also be used to endorse the design concept itself.

Furthermore, when preparing the final calculation, it is possible to work in parallel with the simplified system and a space frame model-the latter being sometimes indispensable. It is then possible to verify the order of magnitude of the results and even to detect any numerical errors arising from the program used or from inadequate simulation. Fig. 3.14a shows a plane, simplified frame of the bridge at Diepoldsau (Walther, 1988), which made it possible to carry out preliminary design up to the tender stage.

Final dimensioning can also be done on the basis of a plane frame model. This particularly applies to structures where the pylons experience no transverse bending under dead weight plus live loads due to traffic (central suspension, lateral suspension, fan pattern with a pylon provided with upper bracing Fig. 3.14b). In this case, the transverse loads on the deck are determined by traditional methods and the forces in the stays are estimated as for simply supported beams.

3.11.3- SPACE FRAME MODELS

In certain cases and in particular for important bridges, it may be necessary to use a space frame model, in order to carry out a more detailed analysis of certain aspects.

Fig. 3.15a shows the model of the Dusseldorf-Flehe Bridge which made possible the calculation of the effects of wind and temperature gradients and the effect of the transverse shape of the pylon.

The steel deck of the Zarate-Brazo Largo Bridge is laterally suspended and carries a carriageway bordered on one side by the a railway line. Major non-linear effects and the asymmetrical nature of the transverse loads and the cross-sections of the stays called for the space frame model shown in Fig. 3.15b.

The final design calculation of the bridge at Diepoldsau (Walther, 1988) necessitated the use of a space frame composed of finite 'shell' type elements (Fig. 3.15c). This structure is the first practical example of the new idea of cable-stayed road bridges, with a slender deck consisting of a single concrete slab, 14.5 m wide and with a mean thickness of 0.48 m. The sapce frame made it possible to calculate the transverse bending of the pylons, the ranges of influence of the forces in the stays and the longitudinal and transverse bendng of the deck.

3.11.4- PARTIAL MODELS

Design of the erection stages is a special application for the use of partial models. Plane systems are generally used, given the large number of different structures to be checked. A check can be made to ensure that, in each one, the erection loads are less than those used in dimensioning; in addition, the tension in each new stay can be checked to obtain the desired deformation.

It is sometimes advisable, with calculation time and economy in mind, to examine special or local problems with the aid of partial models. Figs. 3.16a and 3.16b are, respectively two examples for the Dusseldorf-Flehe Bride of a partial space frame model for the design of the pylon and a grillage of beams for the design of the anchorage of the back-stays. Furthermore, a network of finite elements made it possible to analyse the introduction of the force of a stay in the lower slab of the steel deck.

The longitudinal global analysis may be carried out using a 2D model which represents one plane of cables and half the deck. Allowances should be made to take into account the eccentricity of loading by assuming that each transverse element acts as a simply supported beam between the two planes of cables.

The limits of this analysis should be determined by making a comparison with the results obtained by loading a global 3D model incorporating a deck grillage with slab elements to model shear lag effects. Each longitudinal and transverse truss beam may be idealised as a beam element. Calibration of the transverse beams equivalent to trusses could be done with a detailed 3D model where the concrete deck may be represented by plate elements.

In areas where significant 3D effects may occur, e.g. at pier supports, and for most of the transverse global analysis, a global 3D model should be used.

3.12- LOCAL ANALYSIS

The local analysis should be used for sections where the procedure described above is too general for carrying out stress checks, principally the cable anchorages in the deck and pylons. For sections such as the pylon legs, stresses calculated from load effects obtained from the global analysis should be sufficient to determine member sizes and reinforcement.

For local analysis, a part of the deck or pylon should be analysed in greater detail using a finite element model. Each load case may comprise whatever local loading is directly applied to the section under consideration, together with the general forces and moments resulting from the global analysis for the corresponding load case. These should be applied at the boundaries of the finite element model, i.e. at the ends of the beams or walls and the cable anchorage points.

3.13- STRESS ANALYSIS

Finally, stress analysis should be carried out to check stress levels, using the results obtained from global and local analysis.

3.14- LOADING ON CABLE STAYED BRIDGES

There are wide disparities in national and international codes on the definition of loading on bridges. A clear distinction between the service requirements and those of load-carrying capacity also need to be made.

For service requirements, it is only necessary to consider cases of loads actually likely to be encountered. For major structures, it may be advisable to base the design loads on a probabilistic traffic analysis. There is a wide range of statistical data available for the estimation of probable loads. It is still necessary to keep sight of the fact that permanent loads themselves are often decisive, setting aside the phenomena of vibration and physiological effects on users.

It is not possible to define a theory based on probability, however ambitious it may be, given the fact that the majority of serious accidents causing structures to collapse are due to non-stochastic causes (eg human error). These unforeseeable risks must be covered, at least partly, by the margin of safety. It matters little whether this safety margin is ensured indirectly by overall or partial safety factors, provided that these reduce the risks to an acceptable level.

The general philosophy governing the application of loads is that the worst effects of the loads should be sought. In practice, this implies that the arrangement of the loads on the bridge is dependent upon the load effect and the critical section being considered. In addition, the code requires that when the most severe effect on a structural element can be diminished by the presence of a load on certain portion of the structure, the load is considered to act with its least possible magnitude.

In general loads applicable to long spans cable stayed bridges can be categorized in four groups:

i- Static loading such as dead and superimposed dead loads, live loads and static equivalent of the earthquake loading etc.

ii- Static loading during construction which incudes initial condition of the structure due to the construction and non-linear creep and shrinkage effects.

iii- Dynamic effect of wind turbulence.

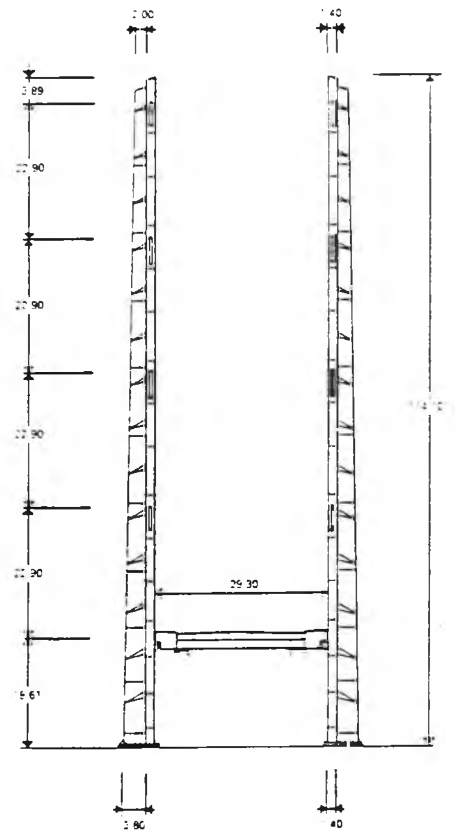
iv- Geometric non-linear effects of large displacements under applied loading.

The loads in (i) and (ii) are usually applied separately to the completed, but undeformed, structure in order to obtain the distribution of load effects according to the relative stiffness of component parts of the structure.

The loads described in (iii) represent the dynamic interaction between the wind and the slenderness of cable stayed bridges. This interaction is very complex due to several parameters such as: excitation arising from vortex shedding, torsional instability and flutter, galloping, wake instability and buffeting by the ambient turbulence in the natural wind. For major structures Wind Tunnel tests are usually carried out to check the aerodynamic response and stability of a cable stayed bridge during erection and after completion.

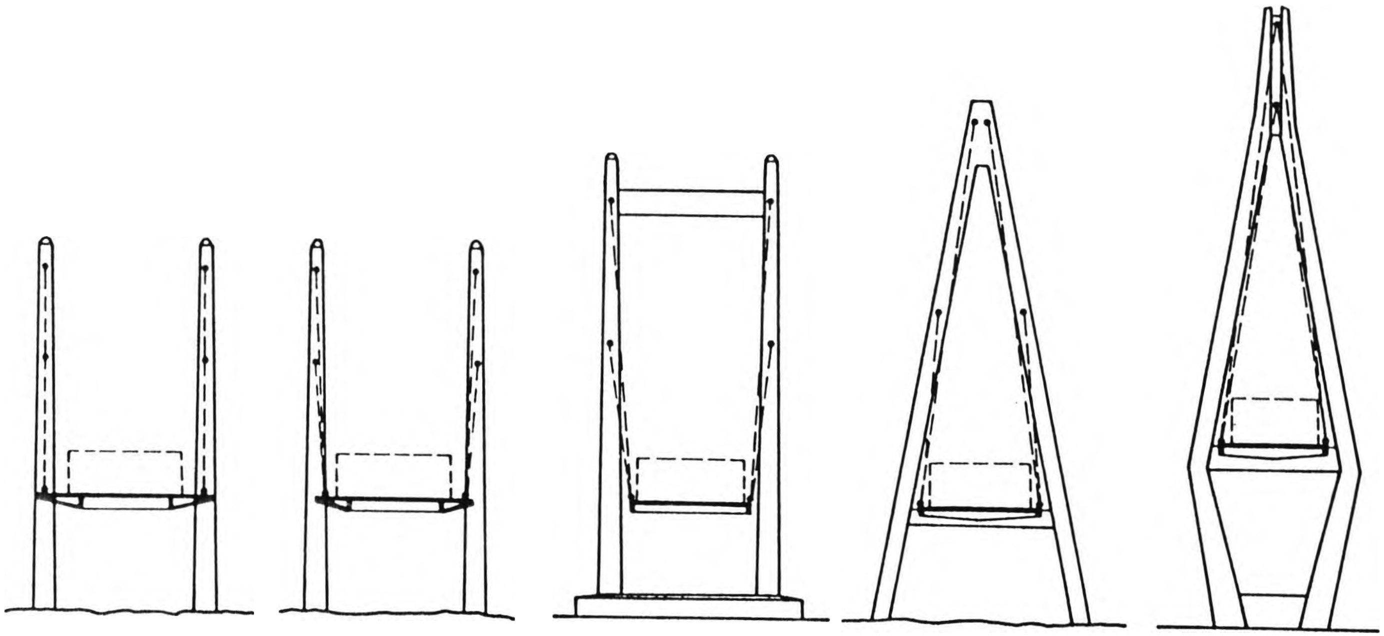
Geometric non-linear effects described in (iv) are considered by applying factors to the load effects calculated in (i). These factors may be calculated using Newton-Raphson type non-linear analysis. Load combinations should be made in advance of their application to the structure and should be multiplied by a partial load factor (γ_{f1}). The non-linear results after convergence should be compared with the linear analysis. A partial additional factor (γ_{f3}) should be applied after calculation of the non-linear effect. The computer model can be either 3D or 2D, depending on the nature of the applied loading.

Different types of loading which have special relevance to cable stayed bridges are discussed in more detail in Appendix D.



FREE STANDING TOWERSHAFT
KNIE BRIDGE, DUSSELDORF,
GERMANY, (WALTHER, 1988)

FIGURE 3.1



(a) (b) (c) (d) (e)

FIGURE 3.2. INFLUENCE OF DECK SIZE ON PYLONS (WALTHER, 1988).

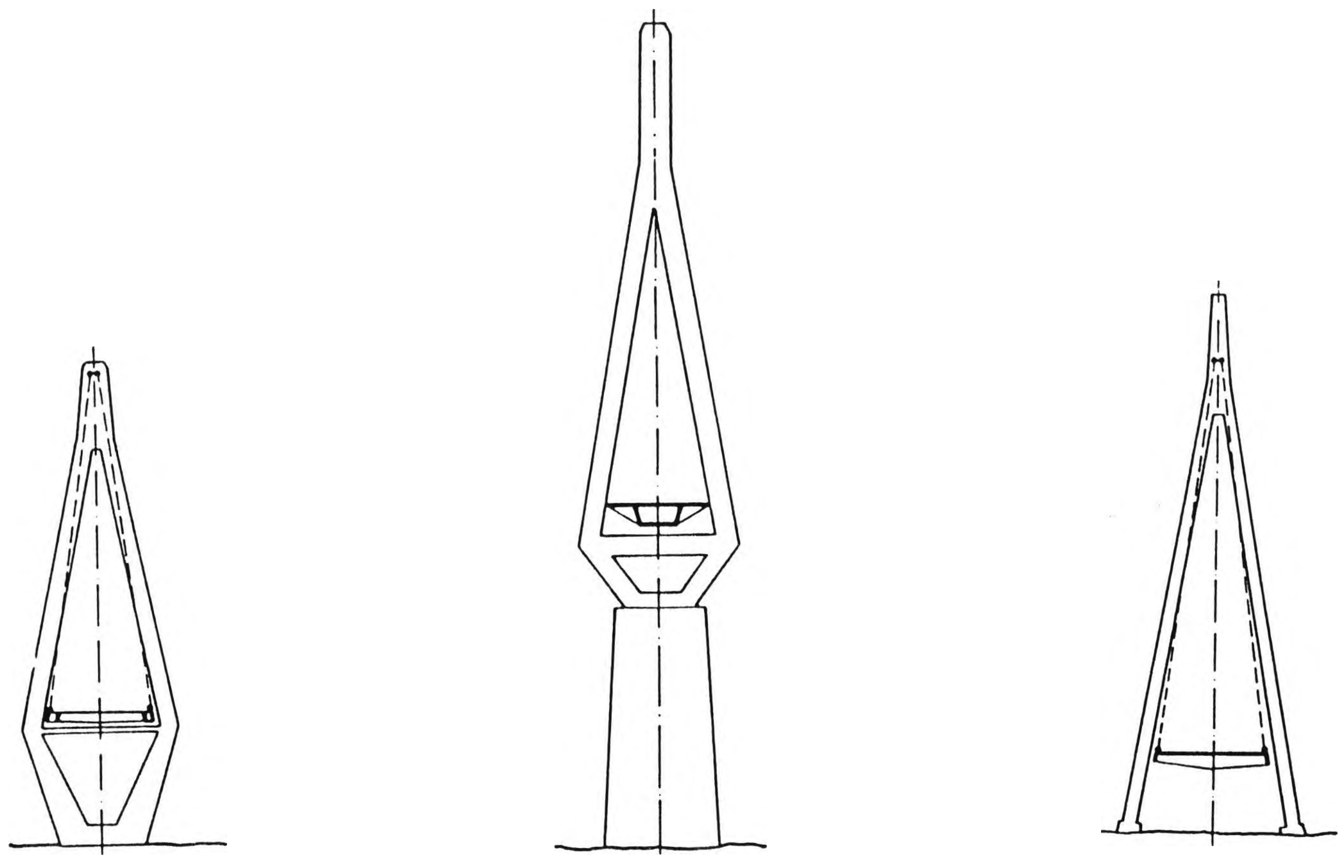
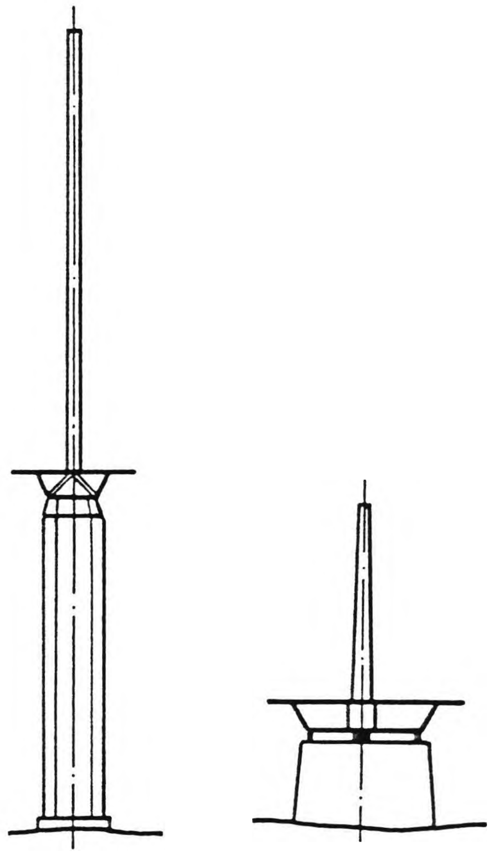
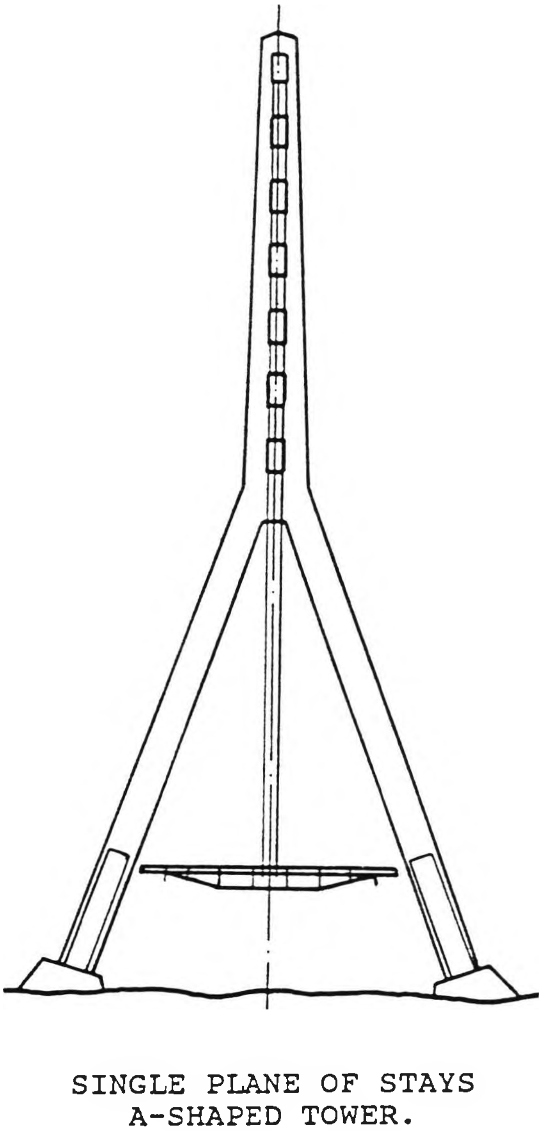


FIGURE 3.3. A-SHAPED PYLONS (WALTHER, 1988).



PYLONS WITH ONE
PLANE OF STAYS.

FIGURE 3.4



SINGLE PLANE OF STAYS
A-SHAPED TOWER.

FIGURE 3.5

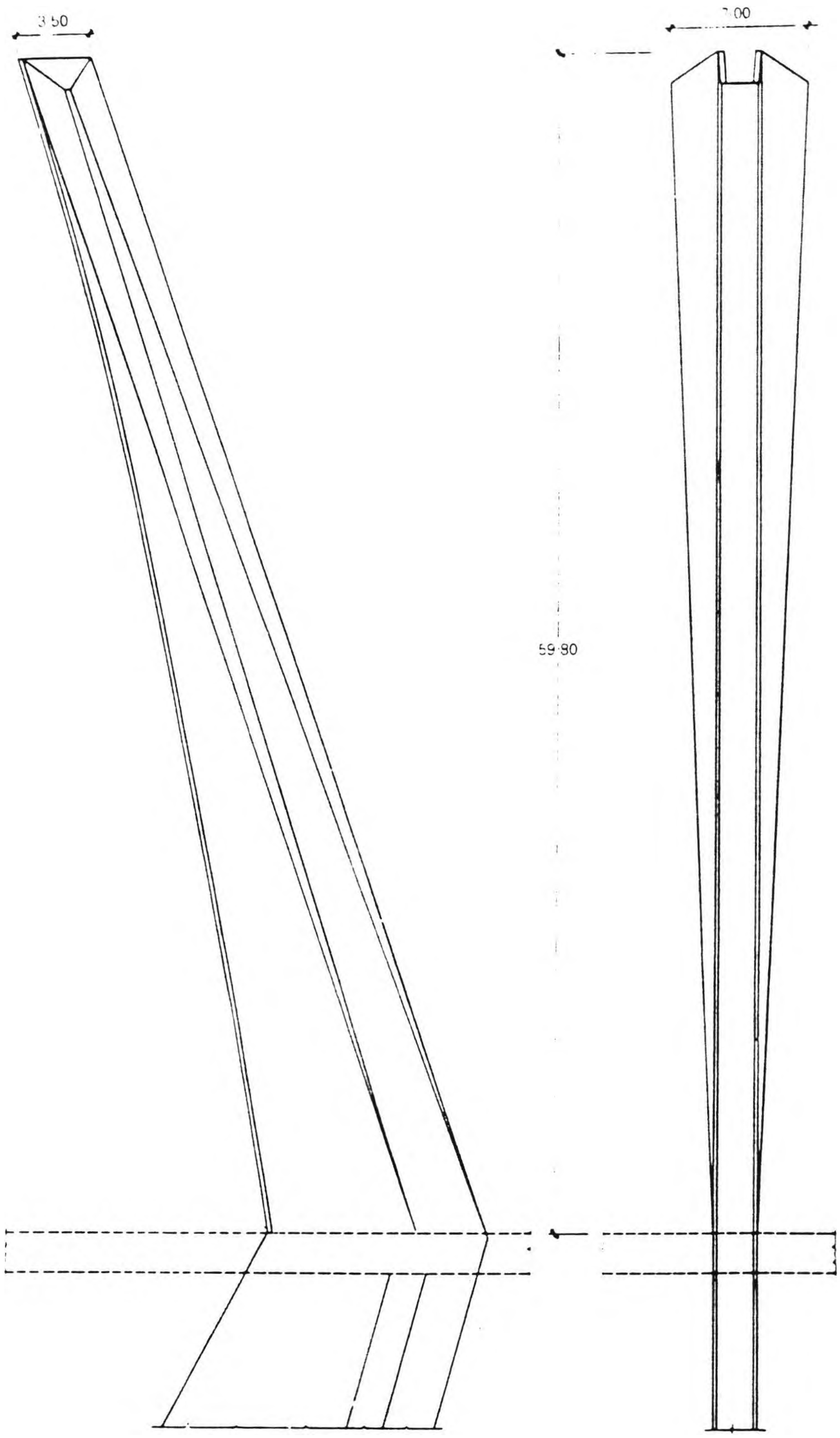
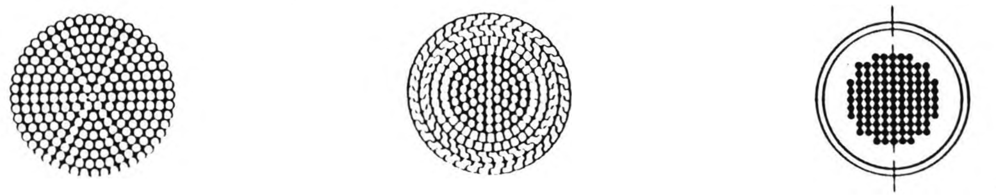


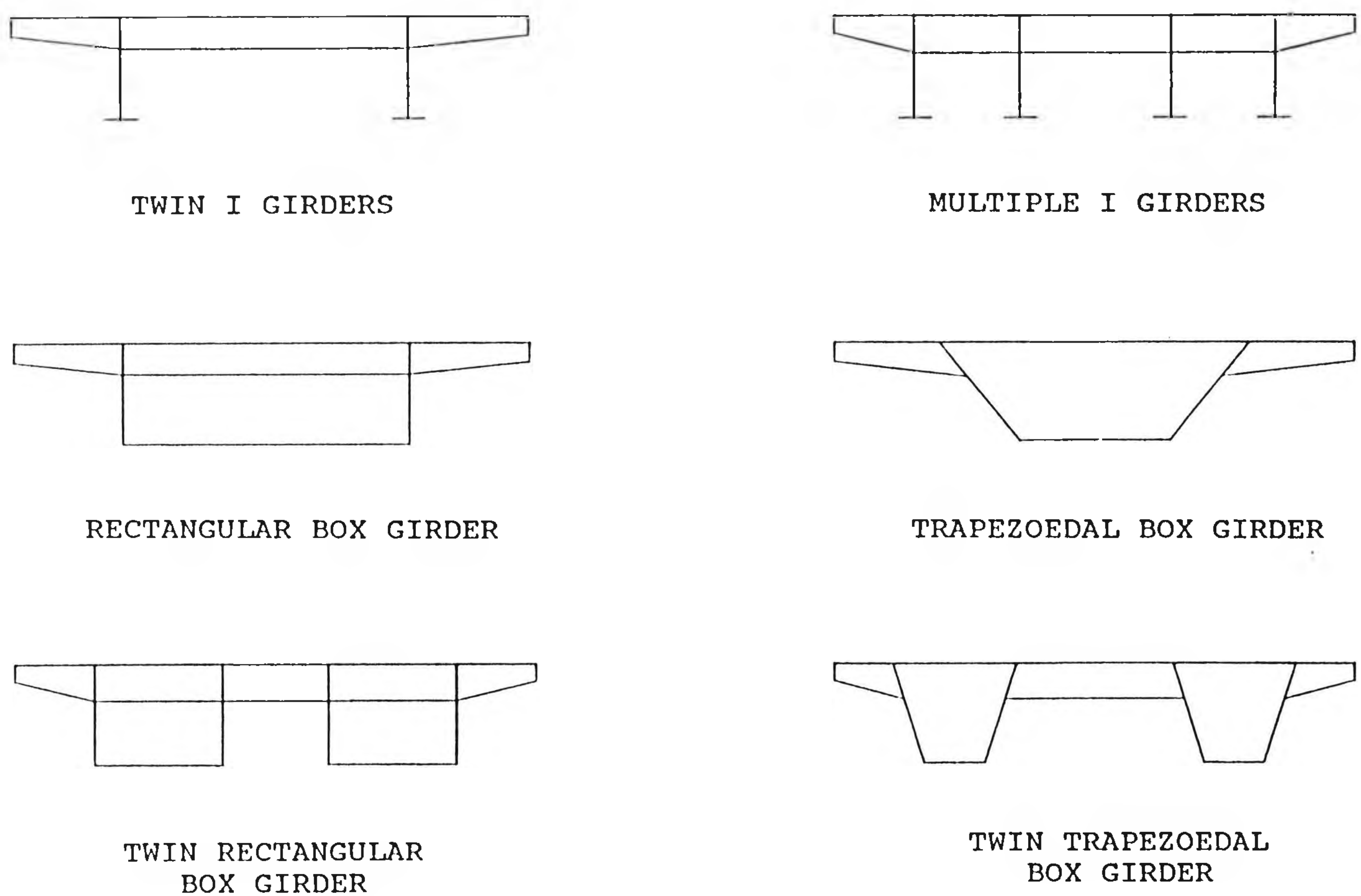
FIGURE 3.6. PYLON OF THE RIO EBRO BRIDGE, SPAIN (WALTHER, 1988).

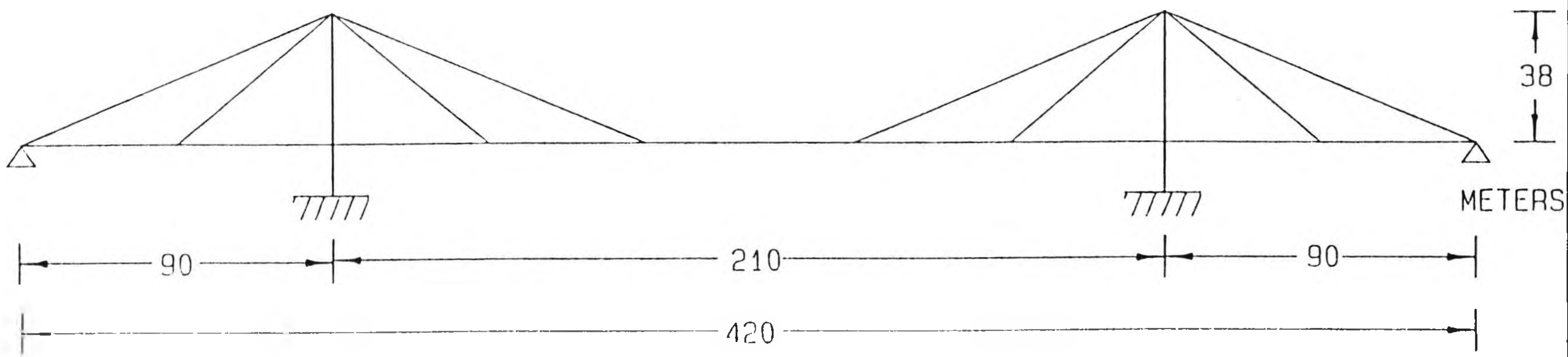


(a) SPIRAL ROPES (b) LOCKED-COIL CABLES (c) PARALLEL WIRE STRANDS CABLES

FIGURE 3.7. TYPES OF CABLES.

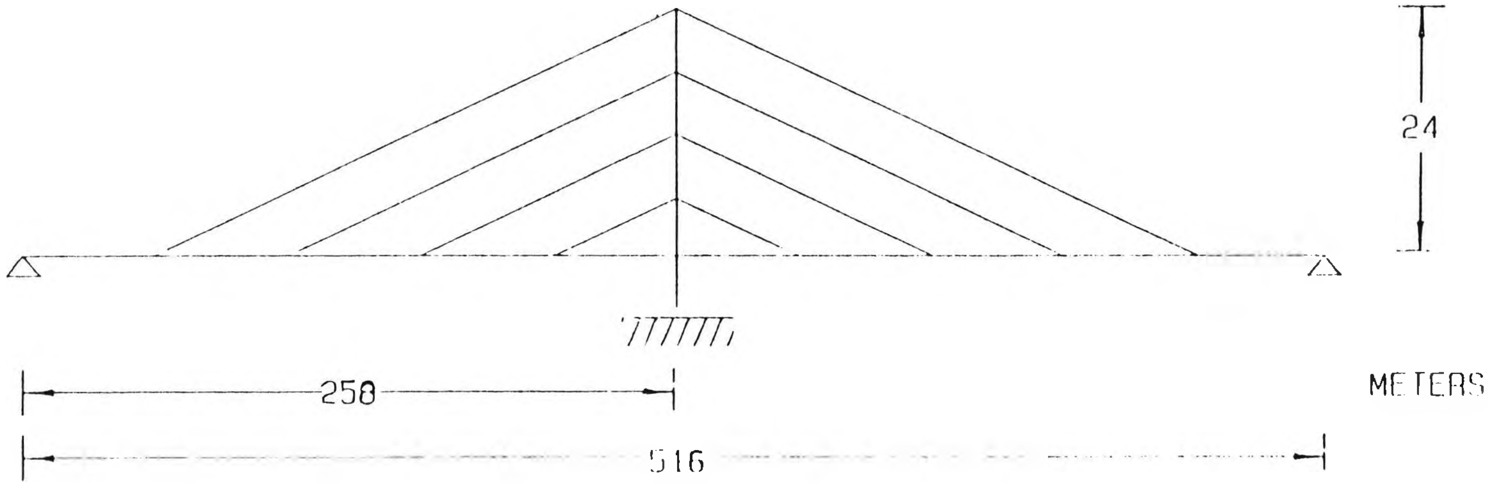
Figure 3.8. BRIDGE GIRDER CROSS-SECTIONS.





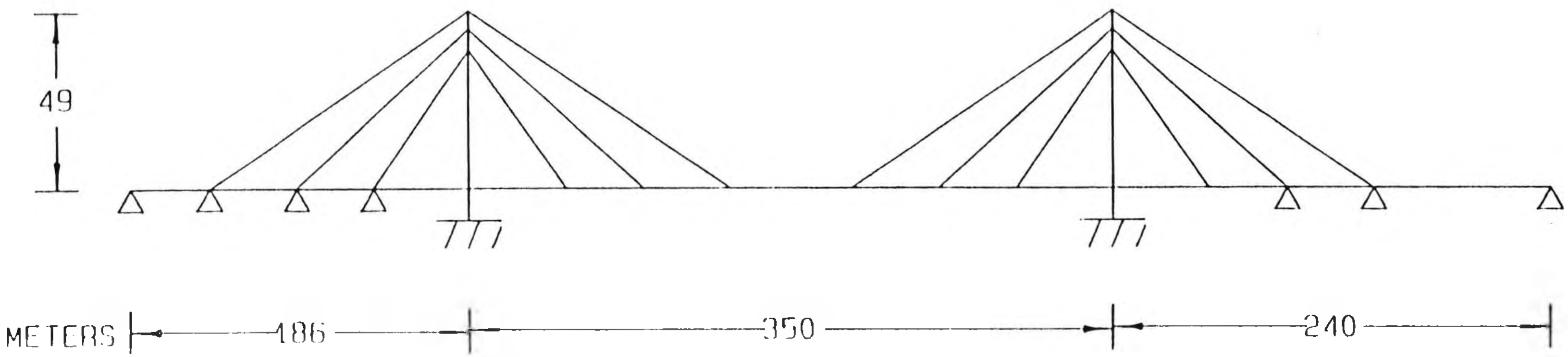
RADIATING SYSTEM

FIGURE 3.9. PAPINEAU BRIDGE, MONTREAL.



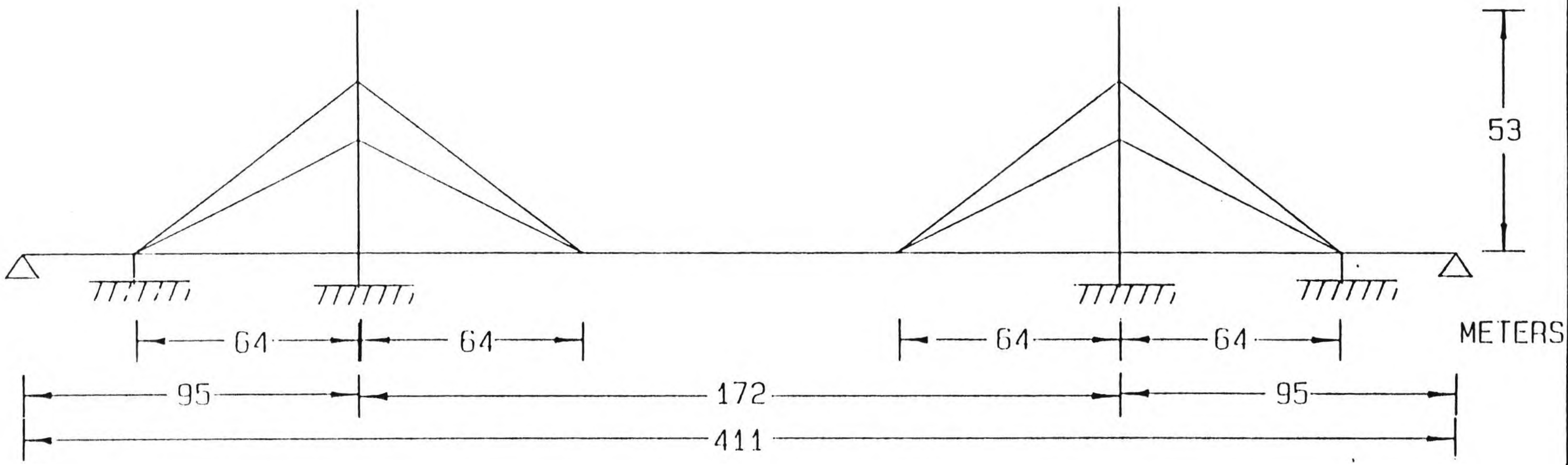
HARP SYSTEM

FIGURE 3.10. OBERKASSELER BRIDGE, DUSSELDORF, GERMANY.



FAN SYSTEM

FIGURE 3.11. DUISBURG-NEUVEMKAM BRIDGE, GERMANY.



STAR SYSTEM

FIGURE 3.12. NORTH ELBE BRIDGE, HAMBURG, GERMANY.

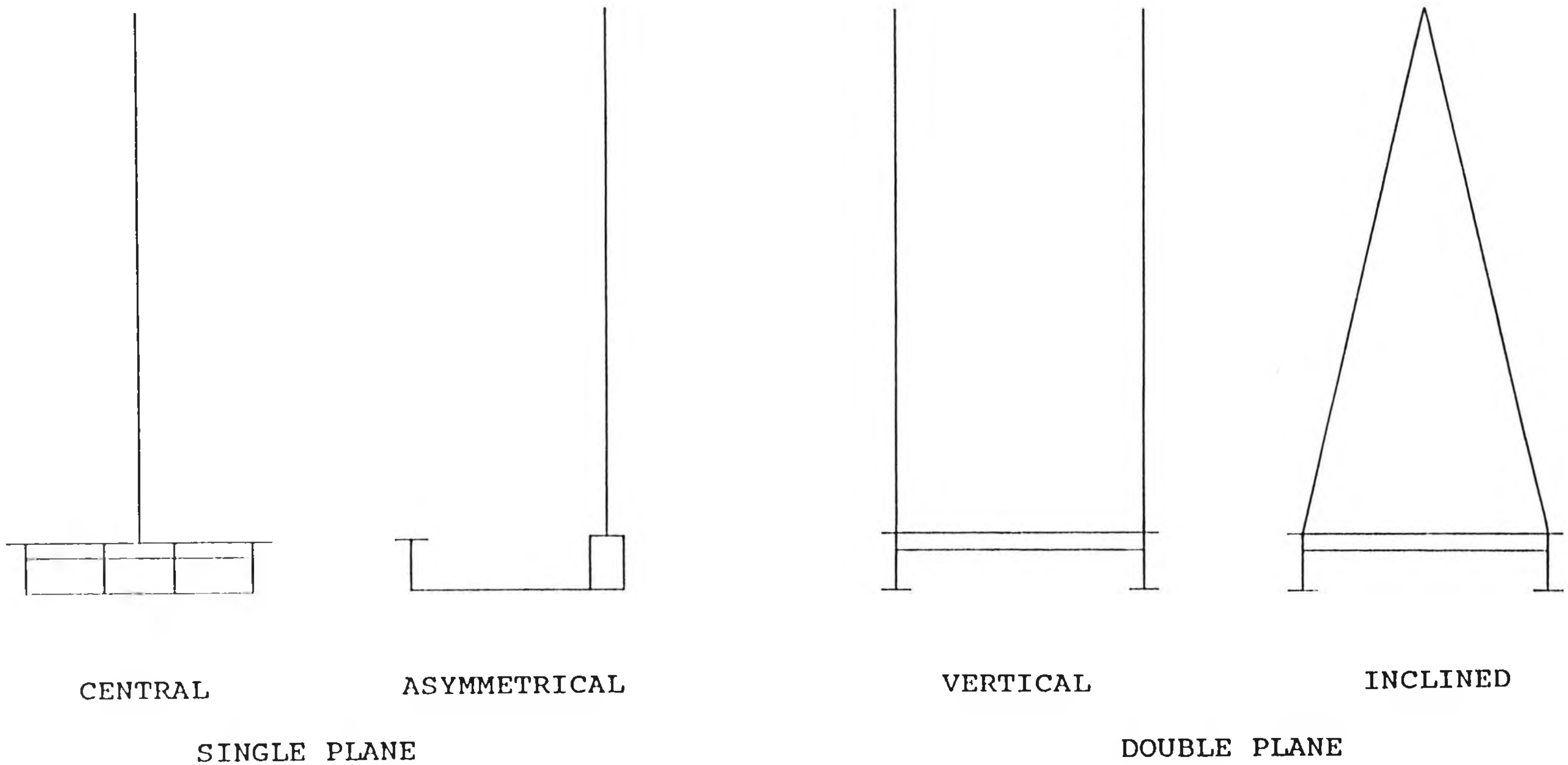
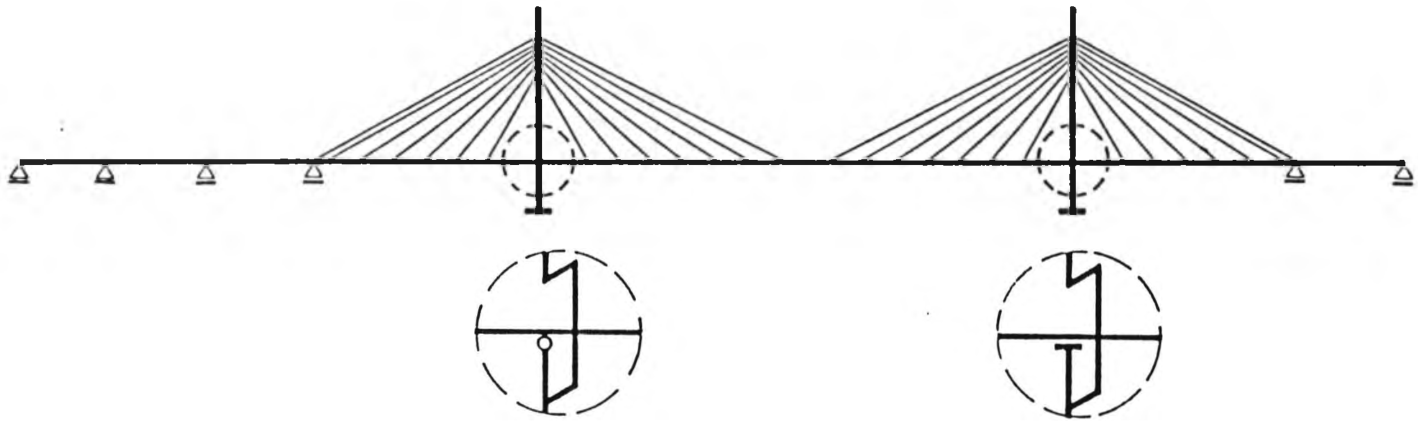
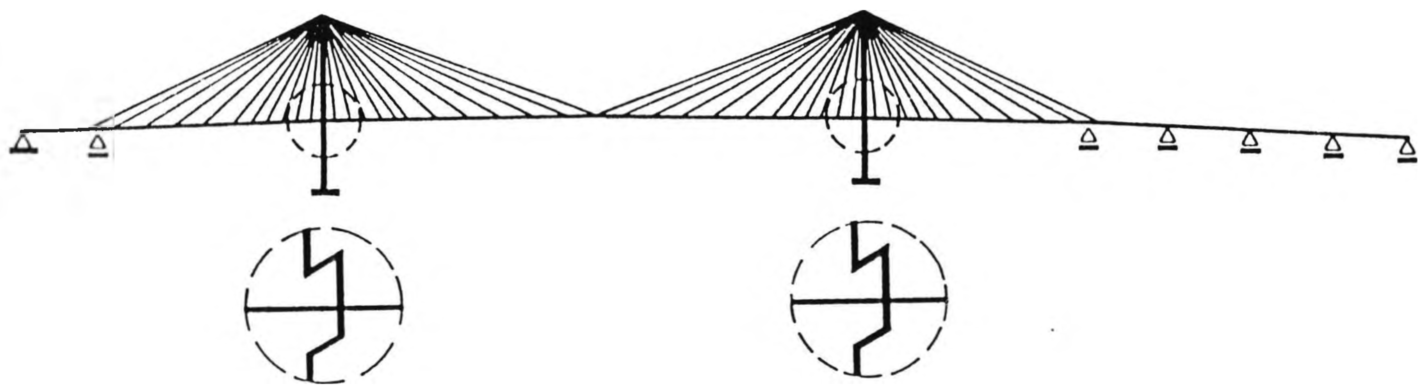


FIGURE 3.13. TRANSVERSE CABLE ARRANGEMENT.

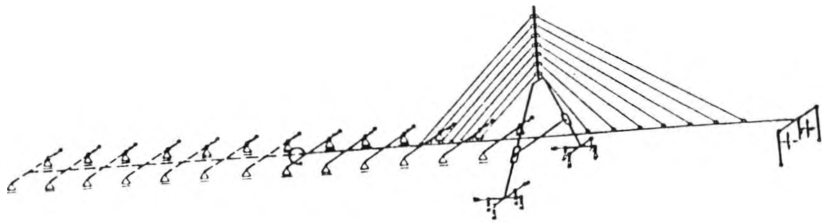


(a) DIEPOLDSAU BRIDGE.

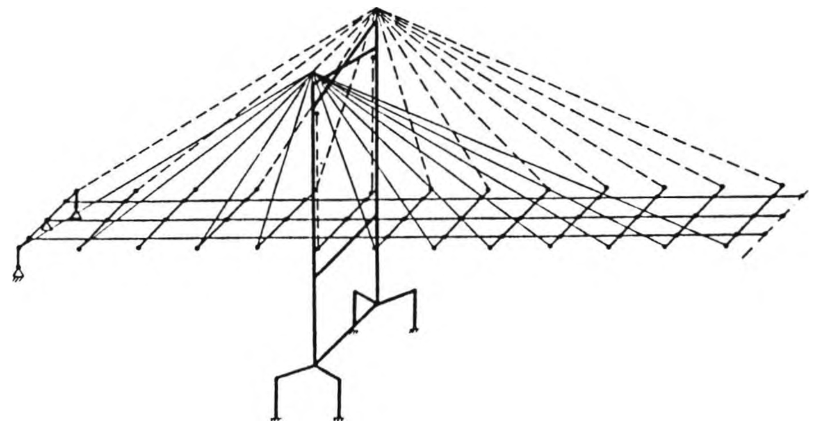


(b) PASCO-KENNEWICK BRIDGE.

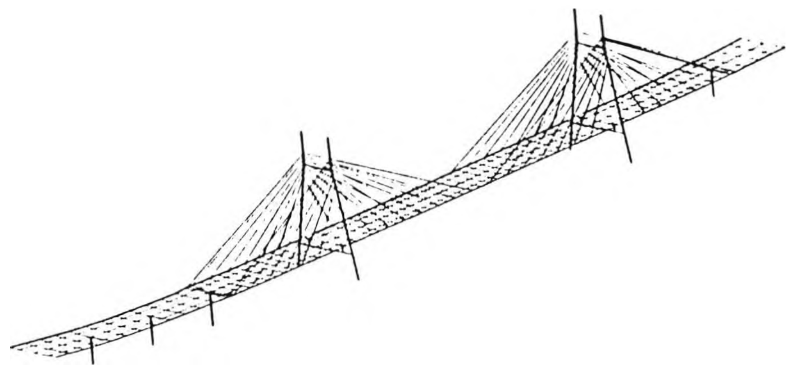
FIGURE 3.14. PLANE FRAME MODELS (WALTHER, 1988).



(a) DUSSELDORF-FLEHE BRIDGE.

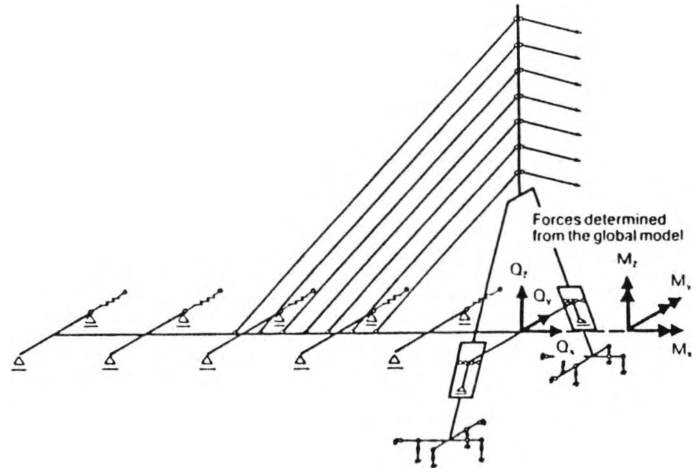


(b) ZARATE-BRAZO LARGO BRIDGE.

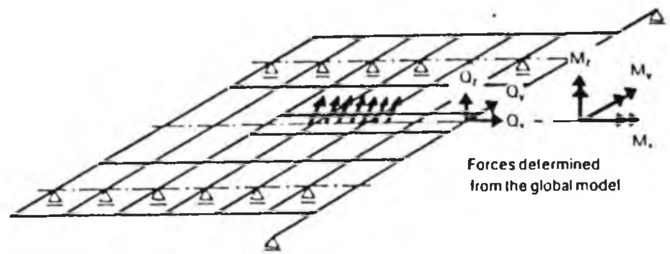


(c) DIEPOLDSAU BRIDGE.

FIGURE 3.15. SPACE FRAME MODELS (WALTHER, 1988).



(a) DUSSELDORF-FELHE BRIDGE: PARTIAL MODEL INTENDED FOR THE DESIGN OF PYLON.



(b) DUSSELDORF-FELHE BRIDGE: PARTIAL MODEL (GRID-WORK MODEL) INTENDED FOR THE DESIGN OF OF THE ANCHORAGE ZONE OF THE STAYS IN THE DECK.

FIGURE 3.16. PARTIAL MODELS (WALTHER, 1988).

CHAPTER 4

OPTIMISATION VIA GENETIC ALGORITHMS

4.1- DESCRIPTION OF GENETIC ALGORITHMS

To get an insight into genetic algorithms, a look at what they are and where they come from is undertaken. The mechanics of the algorithm are presented, with an attempt to gain some idea of why they work. Then a more rigorous explanation of the underlying search processes is presented. Finally, implementations of the algorithm to numerical applications are presented.

4.2- WHAT ARE GENETIC ALGORITHMS?

GAs have been introduced and developed by John Holland (1975). GAs are a class of stochastic improvement algorithm; they were invented to mimic some of the processes observed in nature. These algorithms solve problems of finding good artificial chromosomes by manipulating the material in the chromosomes blindly. They know nothing about the problems they are solving; the only information they are given is an evaluation of each string they produce, and their only use of that evaluation is to bias the selection of artificial chromosomes so that those with the best evaluations tend to reproduce more often than those with a poor evaluation. In a sense, GAs enforce the survival of the fittest among a population of artificial chromosomes (strings). The algorithms are genetic because the string manipulations employed resemble the mechanics of natural genetics. Every generation, a new set of artificial chromosomes is generated using components (sub-strings) of the fittest of the old generation; an occasional new part is tried for good measure.

GAs are not a simple random walk through some parameter space, GAs can be viewed as parallel search algorithms that efficiently exploit old information to seek in a huge space trial points with above average performance. Indeed, by considering many strings as potential candidate solutions, the risk of getting trapped in a local optimum is greatly reduced.

GAs have been introduced and developed by John Holland (1975) and his students. The main goals of their research have been twofold:

1) to abstract and understand, mathematically, the adaptive processes of natural systems,

2) to design software for artificial systems that retain the important mechanisms of natural systems.

The power of these algorithms is derived from a very simple heuristic assumption: that the best solution will be found in regions of the search space containing relatively high performance of good solutions; and that these regions can be identified by judicious and robust sampling of the space. Holland (1975) showed how simple mathematical models of population genetics can efficiently and implicitly make use of this heuristic. GAs implement these models by iteratively manipulating a population of strings using genetic operators(eg **Selection, Crossover and Mutation**).

GAs are computationally simple and powerful. This is so, because they place a minimum of requirements and restrictions on the user prior to engaging the search procedure. The user simply codes the problem as a finite length string, characterizes the objective or objectives as a black box, then the genetic search takes over, seeking near-optima through the combined action of its operators.

4.3- OVERVIEW OF THE THEORY

In this section the fundamental theorem of GAs, presented by Holland (1975), will be reviewed. More explanations of the theorem can be found in Goldberg (1989).

GAs work on populations of strings (Fig. 4.1a). The theory is based on the concept of schemata (schema in singular). A schema is a similarity template describing a subset of strings with similarities over certain string positions.

The basic structure processed by GAs is the string. Assume that we have a finite binary string of length L_b (i.e. number of bits in the string), and we wish to describe a particular similarity. For instance, consider the two strings St1 and St2, each with a length $L_b = 5$:

$St_1 = 10111$

$St_2 = 11100$

It can be seen that both of the strings have 1's in the first position. Such similarity can be described by for example introducing a star * in all positions where we are not interested in the particular bit value. As a consequence, the similarities in the first and the third positions can be described as follows:

1****   and   **1**

Note that the combined similarity can be described by the string 1*1**, having 1 at the first and third positions respectively. These schemata, or similarity templates, apply not only to strings St1 and St2 but they also describe the subset of strings in each schema. For instance, the schema 1**** describes a subset of $2^4 = 16$ strings, each with a 1 in the first position. The particular schema (1*1**) contains a subset of $2^3 = 8$ strings, each with 1 in the first and the third position. In general, not all schemata are generated equally. Some are more specific than others. Some have defining positions that span a greater or lesser proportion of the string. The specificity of schema h (its number of fixed positions), is called the **order** of schema o(h). For example, o(h = 1*1**) = 2 and o(h = 1****) = 1.

Another factor used in measuring the quality of a schema is its **defining length** d(h), defined as the distance between the outermost defining positions of the schema.

For example, the defining length of any one-bit schema is 0:

d(h = 1****) = d(h = **1**) = 0

For the two-order schema, the d(h = 1*1**) can be computed by substraction the position indices of the outermost defining positions as:

d(h = 1*1**) = 3 - 1 = 2

Using the concepts of order and defining length, the fundamental theorem of GAs, otherwise known as the schema theorem can be written as follows (Goldberg, 1989):

$$n(h,t+1) \geq m(h,t)\ \frac{Fit(h)}{A_{Fit}}\ [\ 1 - p_{crossover} \cdot \frac{d(h)}{L_b} - p_{mutation} \cdot o(h)\ ] \quad (4.1)$$

Where

m(h,t)	=	number of copies of schema h at time t;
m(h,t+1)	=	number of copies of schema h at time t + 1;
AFit	=	Average fitness of the population;
$p_{crossover}$	=	Probability of crossover;
d(h)	=	Defining length of schema h;
L_b	=	length of string St_i
$p_{mutation}$	=	Probability of mutation;
o(h)	=	Order of schema h;
Fit(h)	=	Schema average fitness, defined as:

$$Fit(h) = \frac{\sum_{St_i \in h} Fti(h)}{m(h,t)} \qquad (4.2)$$

The schema average fitness Fit(h) is the average of the fitness values of all strings St_i which currently include the schema h.

Schemata are notational devices for discussing similarities in strings. They also provide the basic means for analysing the performance of GAs.

In Eq. 4.1, $p_{crossover}$ and $p_{mutation}$ refer to the probabilities of applying the genetic operators crossover and mutation respectively. These will be discussed further in the following sections.

The factor multiplying the m(h,t) may be thought of as a growth factor. If it is larger than one, the expected number of schemata h, will continue to grow; otherwise, it can do no more than remain constant in number. It is worth mentioning that Eq. 4.1 holds for all schemata contained in the population. In another words, a simple GA processes all schemata in this manner. Highly fit schemata tend to survive because of the factor $Fit(h)/A_{Fit}$. Short definition lengths are also preferred with a high crossover probability, $p_{crossover}$ (in general close to 1). Moreover, due to the fact that mutation probability, $p_{mutation}$ is often quite small; this has a little effect except on schemata of very high order.

A schema h, is expected to grow in subsequent generations if:

(1) It has above average fitness;
(2) It is relatively short; and
(3) It is of low order.

When all three conditions are met, the schema in question is termed a **building block**. These building blocks are combined and recombined by GAs to seek the best solution.

4.4- GENETIC ALGORITHM ESSENTIALS

This section investigates the mechanics of a simple genetic algorithm. The algorithm does nothing more complex than string copying and partial string swapping.

Genetic algorithms are derived from a simple model of population genetics based on the following assumptions:

(1) Artificial chromosomes, which can undergo genetic transformations are fixed length strings having a finite number of position values (eg 0/1) at each position;

(2) A population contains a finite number of artificial chromosomes; and

(3) Each population individual has a fitness, or relative ability to survive and produce offspring.

Before going into details, it may help to give a brief overview of how GAs work. During each iteration of the algorithm (a generation), the fitness of each individual in the population is determined and strings are stochastically selected to produce offspring according to their relative fitness. Pairs of successful offspring are chosen to mate and produce the offspring of the next generation. Variation is introduced by the use of the genetic operators: Crossover and Mutation. By application of crossover, each offspring draws part of its genetic material from one parent and part from another. Moreover, new genetic material is occasionally introduced through mutation. The artificial chromosomes which survive will, over time, be those which have proved to be the most fit. In other words, the search is directed towards regions containing strings with above average fitness. To be a simple GA which gives good practical results in the sense of Goldberg (1989), a procedure must contain the following types of operators:

1- **Selection;**

2- **Crossover;**

3- **Mutation.**

In order to produce a new population, strings from the current population have to follow a certain procedure inspired from the natural world: First, artificial chromosomes are selected from the current population. Second, they are split up, and recombined and finally 'mutated' to form new chromosomes for the generation that follows.

4.4.1- SELECTION

The first key step in executing a GA is selection. The purpose of this step is to lead the genetic search in a specified direction: regions of high observed average fitness. This concept causes the best chromosomes to proliferate in the future generations and the least fit members to be ruled out. There are many ways to perform selection effectively. One commonly-used and perhaps the easiest technique is **roulette wheel selection** (Fig. 4.1b). The choice of a string in the current population can be obtained by the following procedure:

(a) Compute the total sum of fitnesses of the population strings; call the result population fitness;

(b) Generate j, a random number between 0 and population fitness;

(c) Return the first population member whose fitness, added to the fitnesses of the preceding population strings, is greater than or equal to j.

This procedure is referred to as roulette wheel selection because it can be viewed as allocating pie-shaped slices or segments, on a roulette wheel to population strings, with each slice proportional to the string's fitness. Selection of a particular string from the current population to be a reproduction candidate can be viewed as a spin of the wheel, with the winning string being the one in whose slice the roulette spinner stops. It is worth noting that when using this technique the string fitness values should be positive numbers as they are proportional to the probability of selection.

4.4.2- CROSSOVER

While selection represents a process which retains only the most fit strings of a population for mating, it does not in any way improve the quality of any single string in the population. It is the crossover operator that allows the characteristics of the population strings to be altered. Many GA practitioners believe that crossover is the genetic workhorse, a high performance search technique that acts rapidly to combine what is good in the initial population, and that continues to spread good schemata throughout the population as the GA runs. In fact crossover, which causes long jumps in the search space, is the only operator that is thought to distinguish GAs from all others optimisation algorithms.

Now let us examine how the crossover transform is applied. Again several ways for carrying out this operation are possible. In the conventional approach two strings and a crossing site, called the Crossover-Point (fixed for both of the strings), are generated randomly. Then, position values are swapped between the two strings following the crossover-point, so that two new offspring arise. For simplicity in the following example, each of the two strings used has identical elements (Fig. 4.1c).

There are other several crossover frequently used in GAs (Wang et al, 1993), such as one point crossover, two point crossover, uniform crossover, order-based crossover, crossover combined with mutation and so on.

4.4.3- MUTATION

If selection and crossover provide much of the innovation of the genetic search, then what is the role of the diversity-generating or mutation operator? Mutation is a necessary component of GAs: in the beginning, mutation safeguards the genetic search process from an early loss of valuable genetic material and after substantial convergence it refines solutions after selection and crossover have narrowed the search.

Usually mutation is performed with a low probability rate (e.g. 0.001): thus, when mutation is applied to a binary string during a run of a standard GA, each bit in the string will have a one in one thousand chance of being randomly replaced. If the mutation rate is too high, information dissolves and the process degenerates into a random search (Eigen, 1987). Once again, there are many mutation operators. For example, in a binary-coded GA, one commonly used operator replaces a 1 with a 0 or vice versa if a probability test is passed (Fig. 4.1d).

4.4.4- THE ALGORITHM

The structure of an algorithm that can be applied to a wide range of problem domains is shown in Fig. 4.2 and summarized in the following diagram:

Initialise Population: Randomly generate an initial Population of size Npop

While Not (terminate condition) DO

Compute the Fitness function of each member of the population;

For i = 1 To (Npop/2) Do

Selection: Pick two parents on a roulette wheel basis;

Crossover: Crossover the parents based on crossover probability to produce two new offspring;

Mutation: Mutate each offspring based on mutation probability;

EndFor

END.

4.5- ADVANTAGES OF GENETIC ALGORITHMS

In summary the principle attractions of GAs are:

(a)Globality: the main advantage of this stochastic search is its ability to achieve a near global optimum while most search techniques seek for a local optimum solution. This is due basically to: (1) a parallel search; seeking from a population of points instead of a simple point and (2) the fact that a diverse population of solutions is maintained from generation to generation. Sets of solutions will tend to converge on each local optimum, but these will eventually be left as the overall search identifies new solutions in more profitable regions.

(b)Decision Variable independence: GAs require any continuous valued decision variable to be discretised, for the process of mapping onto a binary string. Yet, they handle integer and discrete valued variables efficiently. Most engineering design problems tend to involve discrete choices such as beam section, which are very much suited to binary representation.

(c)Domain independence: the algorithm works on the coding of a problem, ie each decision variable of the problem is represented by a sub-string of 0's and 1's, so that it is easy to write a general computer program for solving many different optimisation problems.

(d)Non-Linearity: Many conventional optimisation techniques are based on some assumptions like linearity, convexity, differentiability etc. None of these assumptions are needed by GAs. The only requirement is the ability to compute some quality function (fitness function) which may be highly complicated and non-linear.

(e)Flexibility: GAs do not require that the constraints should be expressed explicitly in terms of design variables.

(f)Robustness: as a consequence of the previous advantages, GAs are inherently robust, they can cope with a large spectrum of problems, they can work with highly non-linear problems and they do it in a very efficient manner (Goldberg et al., 1989).

(g)Parallel Nature: Not only are GAs inherently parallel search techniques but also due to the independence of processing every individual solution in the population, computation can be performed in parallel. This implicit parallelism of GAs makes them the most suitable for design optimisation in a parallel computing environment. Attempts have been made for such implementation (Pettey et al., 1987).

4.6- APPLICATION OF A SIMPLE GENETIC ALGORITHM

In the previous sections a simple GA was investigated. Now an application of the algorithm to the minimisation of a simple algebraic function is examined. This example is taken from Bunday and Garside (1987). The problem is to minimise the following function:

Problem GA1:

$$\text{Minimise} \qquad F = 3x_1^2 + 4x_1x_2 + 5x_2^2 \qquad (4.3)$$

$$\text{Subject to} \qquad x_1 \geq 0, \quad x_2 \geq 0, \quad x_1 + x_2 \geq 4 \qquad (4.4)$$

The anticipated solution, as reported in Bundy and Garside (1987), is $F^* = 44$ at $x_1^* = 3$ and $x_2^* = 1$. In order to solve an optimisation problem by GAs, 5 components should be specified:

1) A string representation of the solution to the problem;
2) A way of generating an initial population of solutions;
3) A fitness function measuring the quality of solutions;
4) Genetic operators that improve solutions during the run of the GAs; and
5) Values of the parameter utilised by the genetic search (population size; probabilities of applying operators etc).

4.6.1- STRING REPRESENTATION OF THE SOLUTION

Since standard GAs work on coded variables, a chromosomal representation of the solution is required. Owing to its simplicity, binary coding is adopted. The problem being addressed is the determination of the length of a sub-string (No. of bits), L_s, per decision variable. L_s relies on the type of decision variable involved: continuous, integer or discrete. In general, given a continuous design variable, x_i, which can take any value between a minimum value x_{min} and a maximum value x_{max}, the sub-string length Ls required to a precision of ε, may be estimated from the following expression (Goldberg, 1989):

$$2^{Ls} \geq [(x_{max} - x_{min})/\varepsilon + 1] \qquad (4.5)$$

For problem GA1, if it is assumed that the decision variable x_i, can take any value between 0.0 and 15.0 (inclusive) to a precision of 1.0, then $2^{Ls} \geq 16$. This gives a value of Ls of 4, with the following 16 4-digit combinations of 0 and 1:

1: 0000
2: 0001
3: 0010
4: 0011
5: 0100
6: 0101
7: 0110
8: 0111
9: 1000
10: 1001
11: 1010
12: 1011
13: 1100
14: 1101
15: 1110
16: 1111

The decoding of the strings will produce the corresponding decimal digits which will then represent real values of the variables. This 'parameter' mapping is within the control of the user. The procedures **extract_parm** which removes a sub-string from a full string, and **map_parm** that maps the unsigned integers to the range $[x_{min}, x_{max}]$ presented in Goldberg (1989) are used in this example.

4.6.2- INITIAL POPULATION

Having coded the decision variables x_1 and x_2 as finite-length strings, the initial population of solutions can be set up. It is a common practice when beginning a genetic search to initialise a population of Npop strings by randomly generating bits with equal probability, p_o (eg 0.5), for zero and one. Indeed, a random number is generated between 0 and 1 and compared to p_o. If the generated number is greater than or equal to p_o then the bit value is 1 otherwise 0. If there are Nd decision

variables (x_i, i=1,...,Nd), this process is repeated Nd.Ls.Npop times. Table 4.1 shows a population generated in this way.

4.6.3- FITNESS FUNCTION

Fitness values of solutions are the only information that GAs exploit to move to high performance space regions. To be more precise, a fitness value is used to guide the selection component to choose the most fit strings for crossover and mutation. A fitness value, which must be positive as required by computation of selection probability, expresses the quality or fitness of a solution.

Originally, GAs were designed to deal with maximisation problems. The common practice used to transform a minimisation problem to maximisation problem is to maximise the negative objective function. This approach is not feasible as mentioned earlier. Thus minimisation problems can be solved for example by using a simple device:

$$Fit = Cf - F \qquad (4.6)$$

Where Fit is the fitness function and Cf is a constant large enough to prevent negative values of fitness. For problem GA1, Cf can be estimated by putting both of the variables to their maximum values, 15, in Eq. 4.3. This gives Cf =2700.

Table 4.1 Processing Generation 0
***Constraints violation**

			Generation 0					
St_i		x_1		x_2	F	Fit	Selection Probability $Fit/\Sigma Fit$
1	1111	15	1000	8	1475	1225	0.085
2	1010	10	0111	7	825	1875	0.129
3	0000	0	0010	2	20*	0*	0.000
4	1000	8	1110	14	1620	1080	0.075
5	1100	12	0000	0	432	2268	0.157
6	0000	0	1000	8	320	2380	0.164
7	1110	14	0001	1	649	2051	0.142
8	1100	12	1011	11	1565	1135	0.078
9	1101	13	1110	14	2215	485	0.033
10	0100	4	1010	10	708	1992	0.137
Σ						14491	1

4.6.4- GENETIC OPERATORS

The genetic operators used herein to solve problem GA1 are:

(1) The simple roulette wheel selection;
(2) One point crossover; and
(3) Bit mutation, ie replace a 1 with 0 or vice versa.

4.6.5- PARAMETER VALUES

Hidden behind the conceptual simplicity of GAs, there are a variety of parameters such as population size and probabilities of mutation and crossover. Effective values of these parameters for bit string representation have been intensively studied (De Jong, 1975; Grefenstette, 1986 and Schaffer et al., 1989). Again this problem will be discussed further when the optimisation of two cables out is examined in Chapter 6. Note that the objective of this section is to present a simple illustration of the way in which GAs deal with optimisation problems. For this example the genetic parameters for problem GA1 are:

1. Population size: Npop = 10;
2. Probability of Crossover: pcrossover = 0.8;
3. Probability of mutation: pmutation = 0.03;
4. Number of generations = 50;

4.6.6- IMPLEMENTATION AND RESULTS

The work described in this section was carried out using a PASCAL program, where some of the coded routines were taken from Goldberg's "Simple Genetic Algorithm" (1989).

Table 4.1 shows the binary sub-strings of x_1 and x_2 and their corresponding numerical values, the values of the function F and its corresponding fitness (columns 6 and 7), and finally the probabilities of selection. Note that the maximum fitness (2380) corresponds to the minimum value of the function F of generation 0 (320) has the highest probability of selection (16.4% for St_6) and the worst strings has the lowest probability of selection (0.0% for St_3). A zero in the fitness column indicates that the particular combination of the variables does not satisfy the imposed constraint (Eq. 4.4).

Application of the three genetic operators to the members of generation 0 results in producing new solutions comprising generation 1. The new strings generated and their corresponding function values F and fitnesses are summarised in Table 4.2.

Table 4.2 Results of Generation 1

			Generation 1		
St_i	binary	x_1	x_2	F	Fitness
1	01010100	5	4	235	2465
2	10111001	11	9	1164	1536
3	01100100	6	4	284	2416
4	00111101	3	13	1028	1672
5	01010010	5	2	135	2565
6	11000100	12	4	704	1996
7	01011100	5	12	1035	1665
8	11100001	14	1	649	2051
9	01110100	7	4	339	2361
10	01010011	5	3	180	2520

The optimum solution of problem GA1 ($F^* = 44$ at $x_1^* = 3$ and $x_2^* = 1$) has been found at generation 10 shown in table 4.3.

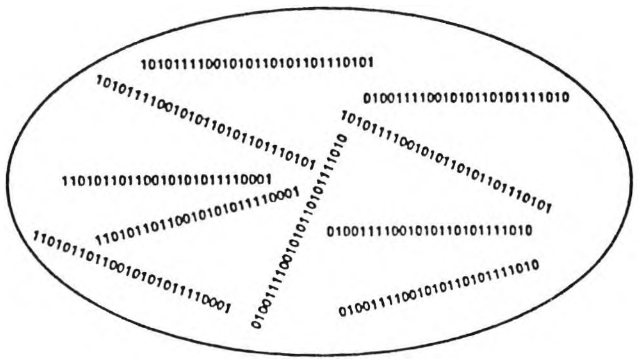
Table 4.3 Results of Generation 10

			Generation 10	
St_1	x_1	x_2	F	Fitness
1	7	3	276	2424
2	6	1	137	2563
3	7	1	180	2520
4	3	13	1028	1672
5	7	2	223	2477
6	6	8	620	2080
7	7	5	412	2288
8	7	1	180	2520
9	3	1	44	2656 (optimum)
10	7	0	147	2553

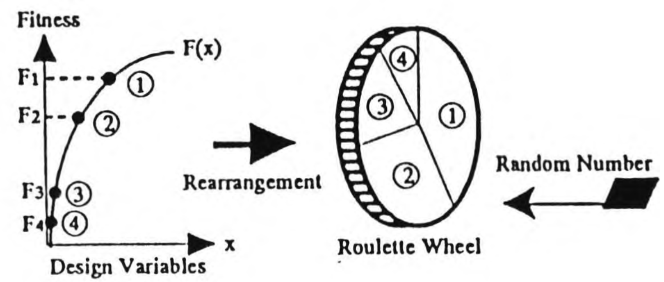
4.7- CONCLUSION

The detailed mechanics of a simple genetic algorithm have been presented. GAs operate on populations of strings. The strings are coded to represent the underlying parameter set. Selection, crossover, and mutation are applied to successive string populations to generate new string populations. The operations performed are simple, string copying and partial string swapping, yet the effect is powerful. A simple genetic algorithm has been introduced to deal with the optimisation of a simple algebraic function with the aim of illustrating both the detail and power of the method.

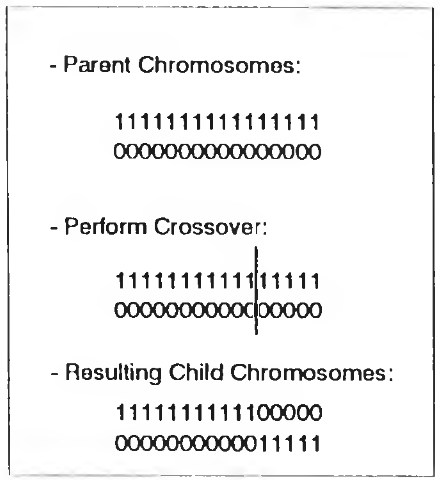
Having introduced, in this Chapter, the basic concepts of GAs and addressing the questions of why and how they work, we can now move into the application of these algorithms in the context of Cable-Stayed Bridges subject to the effect of live loading and stay removal presented in Chapter 5 and 6 respectively.



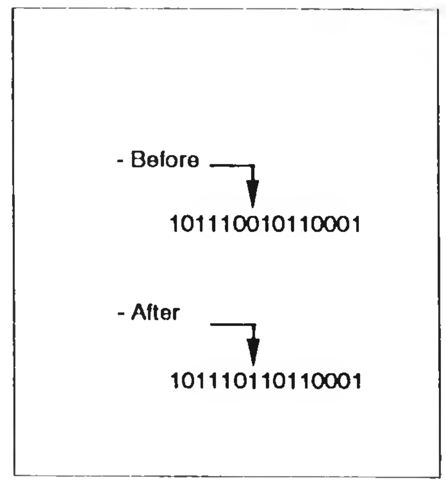
(a) GA population



(b) The roulette wheel selection (Sakamoto, 1993)



(c) Crossover



(d) Mutation

FIGURE 4.1. GENETIC ALGORITHMS OPERATORS.

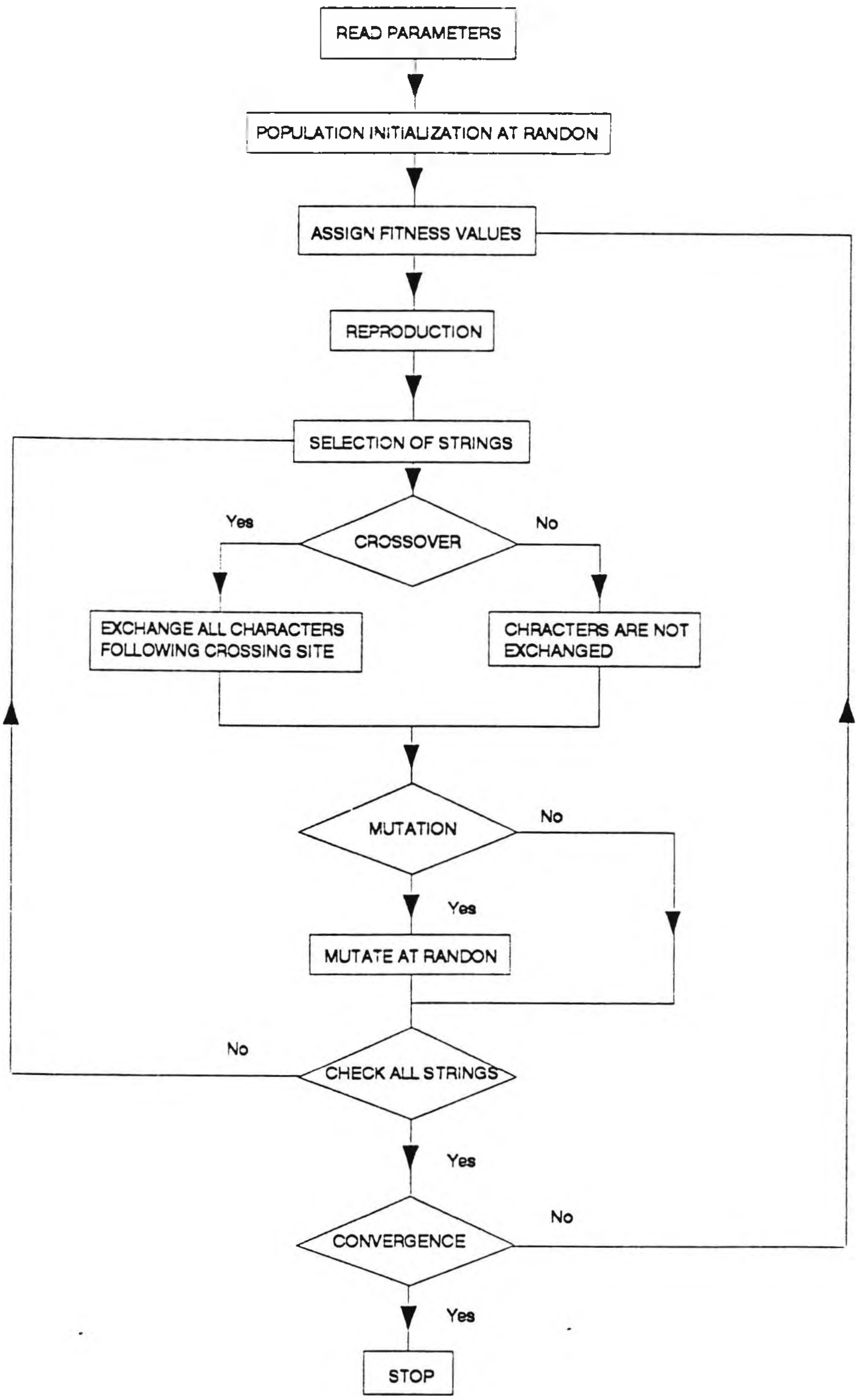


FIGURE 4.2. GENETIC ALGORITHM FLOWCHART.

CHAPTER 5

CABLE-STAYED BRIDGES LIVE LOADING OPTIMISATION VIA GENETIC ALGORITHMS

5.1- INTRODUCTION

Before designing a cable-stayed bridge, it is essential to have an idea about its behaviour under live load. In order to provide the most economical design, the performance of the structure when any the parameters involved in the analysis are changed, should also be understood. It is useful to predict what will happen when changes are performed on the geometry of the structure, the structural form of any of its elements, their mechanical properties, or even on the materials of construction themselves.

The influence of the stiffness parameters of the cables and pylons of cable-stayed bridges on the behaviour of various elements of the bridge superstructure, has been investigated by some authors (Krishna et al. 1985; O'Connor 1971; Podolny et al. 1976). Hegab (1988) presented a comprehensive parametric study. In addition to the stiffness parameters of cables and pylons, Hegab (1988) included the bending stiffness of the bridge girder and the central unsupported length of the main span.

It was concluded (Krishna et al. 1985; Hegab 1988) that any increase in cable stiffness leads to a reduction in girder deflection and to an increase in: (1) The percentage of live load carried by the stay cables of the main span; (2) the axial compressive load in each pylon due to live load; and (3) the pylon displacements and moments. It was also shown that any increase in the bending stiffness of each pylon implies a reduction in the pylon displacements, and therefore, an increase in the tensions in the cables of the loaded span which in turn decreases the girder deflection and increases the pylon base moments. Hegab (1988) showed that any reduction in the bending stiffness of the girder results in an increase in all the parameters discussed above. He also concluded an optimum value of the central unsupported span length for each parameter.

In the reported literature (Krishna et al. 1985; Hegab 1988) only small size cable-stayed bridges were analysed with relatively small number of cables being used . In addition, live loads were of uniform type with constant intensity covering the entire length of the main span of the bridge only.

It should be noted that no attempts have previously been made to study the behaviour of cable-stayed bridges subject to live loads with varying intensity as it varies with different loaded length, (BD37/88, 1989).

Live loads of any type are always associated with Influence line diagrams (ILD's). ILD's provide a systematic procedure for determining how the stress in a given component at a particular point of the bridge varies as the applied load moves on the bridge. The procedure of using ILD's may be divided into two parts, first is the generation of ILD's, then the loading of ILD's in order to determine critical locations, and compute extreme effects.

The problem of loading an influence line with moving loads can be formulated as a mathematical programming problem consisting of optimisation of an objective function subject to a set of constraints. In such formulation, the variables represent the positions of moving load which should be either maximized or minimized; and the constraints represent the spacing between the axles or vehicles and their limitations.

This Chapter presents a parametric study on Multiple-spans cable-stayed bridges subject to live loading as defined in BS5400: part 2 (1978) and modified by the Department of Transport most recent loading directive BD37/88 (1989), (Appendix D). Traffic loads as defined in BD37/88 are of combinatorial type and are very demanding in the number of analyses they require in order to locate the worst effect (section 5.4). It is intended in this Chapter to investigate the various aspects of this loading on the structural behaviour of double-plane radiating type cable-stayed bridges. Subject to BD37/88 traffic loads, the influence of varying cable stiffnesses together with various spans arrangements are considered resulting in a parametric study for cable-stayed bridges of the type small, medium and large size being presented. GAs are used as optimisation tools for the location of worst loading scenarios whereby the solution of the GA model is based on the use of ILD's. A detailed example is analysed in order to demonstrate the different stages of the GA optimisation process and to show that GAs can actually locate the optimum solution. For the purpose of comparing results obtained by GAs, the same example was analysed using a complete enumeration scheme where by an exhaustive search was carried testing every single combination of live loading in order to locate the worst loading scenario which in turn results in the optimum solution being calculated. This solution would provide a guide line whereby the performance of the GAs solutions can be criticized. Different problems facing the GA optimisation process are addressed with various conclusions on the behaviour of cable-stayed bridges subject to BD37/88 traffic loads, are drawn.

5.2- GENERATION OF ILD'S

An influence line is defined as a diagram showing the variation of some behaviour functions of the structure graphically when a unit load moves across the structure. The behaviour functions may include reactions, shear forces, bending moments, axial forces, stresses and deflections. When using computers, ILD for a bridge can be generated simply by placing a unit load vertically downward at every node of the loaded deck of the bridge and carrying out structural analyses. Thus, the series of values of a given behaviour variable obtained by moving the load across the bridge structure yields the ILD for that variable. Fig. 5.2 and 5.3 show some typical ILD's for moment at the centre line of the 24 cables arrangement cable-stayed bridge shown in Fig. 5.1. Discussion on the various aspects of the bridge will be discussed in forthcoming sections.

5.3- PROCEDURE FOR LOADING ILD

Given a particular ILD and the type of loading, the objective of this procedure is to find the maximum and minimum effects. Depending on the type of ILD, the effect can be axial force, shear, moment, reaction or deflection.

The input of this procedure is (1) the ordinate, Y, and the abscissa, X of a given ILD at all the intervals along the deck structure; (2) the type of ILD, i.e., if the given ILD is the axial force, shear, moment, reaction at a support, or deflection, and (3) the applied loading. The output of this procedure is the maximum, minimum, and absolute maximum effect of the influence line.

Once the ILD is known it is possible to determine which portions of the bridge to load in order to produce the maximum effect. This can be achieved by dividing the ILD into a series of positive and negative zones whose areas may be calculated using Gaussian quadrature.

For a uniformly distributed load which does not change with loaded length, all the positive zones should be loaded to get the maximum effect. By adding together the areas of the positive zones, and multiplying by the load intensity, we can determine the maximum effect at the point under consideration. For the negative zones, the minimum effect is determined in a similar way.

For point loads, these should be applied at a locations where the magnitude of the influence line is a maximum or minimum as appropriate.

5.4- BS5400 HIGHWAY LOADING

It is much more complicated to apply 'standard loading', as defined in British Standards or Ministry directives. Highway loadings vary in intensity depending on the length of the bridge that is loaded and the number of highway lanes that are present. The axle spacing of the standard vehicles are often variable, and there are areas of the beam which are not loaded when heavy vehicles are present.

BD37/88 defines two types of highway loading; HA and HB. HA represents the maximum loading which is likely to occur under normal traffic conditions, while HB loading represents a notional abnormal vehicle, carried on four axles.

5.4.1- HA LOADS:

HA loading consists of a uniformly distributed load which is expressed as a load (W) per metre of lane, where W is a function of the loaded length (L), together with a knife edge load. These loadings are shown in Fig. 5.4 for loaded lengths of upto 1600m. For loaded lengths less than 50 m, the increase in the load intensity is obvious. The knife edge load is fixed as 120 KN per lane.

Not all lanes are loaded with the full HA loading. BD37/88 presents an extremely complex specification, in which the proportion of HA on each lane depends on which lane is being considered, the total number of lanes, the width of each lane and the loaded length.

The variation of load intensity with loaded length is the major source of complexity. In areas remote from the point in question, it may be desirable not to load the structure even where the influence line indicates that the structure should be loaded, in order to reduce the loaded length and thus increase the intensity of load in regions where the load is more effective.

There is one further complication. If the zones rise to a sharp peak, the permutation approach is likely to significantly underestimate the true worst effect. For these types of zone, a modified loaded length is used. This is found by dividing the area of the zone by one-half of the maximum height of the zone. This is equivalent to finding the triangle with the same area and height as the true zone, and using its base as the loaded length. It will only be significant for concave zones as shown in Fig. 5.5. The knife edge load is of course applied at the maximum ordinate of the influence line.

The Department of Transport have indicated in BD37/88 that complete zones should be either included or excluded, depending on whether they make things worse or not. The problem reduces to one finding which combination of zones gives the worst effect.

5.4.2- HB LOADS

The HB vehicle is regarded as being carried on 4 axles, and the magnitude of the loading is expressed as a number of units. Each unit causes a load of 10 KN on each axle; the number of unit depend on type of road. Normal roads are designed for 25 units, approx 100 tonnes total load, while motorways are designed for 45 units, 180 tonnes total load. The axle layout is shown in Fig. 5.6. The axle are arranged in two bogies, each with an axle spacing of 1.8m. The bogies can be 6, 11, 16, 21, or 26m apart, depending on which gives the worst effect. Normally, the shortest vehicle will give the worst effect, with the whole vehicle concentrated on the largest zone, but for some shorter structures the worst effect over piers can be achieved by placing the two boogies at the peaks of adjacent zones.

5.4.3- HB LOAD ASSOCIATED WITH HA LOAD

When the HB vehicle is present, the rest of the structure may still carry HA loading. There is no HA for 25m on either side of the HB vehicle in the same lane as the vehicle itself and the knife edge load is not applied in that lane; all other lanes are loaded with both HA UDL and the KEL. The remainder of the lanes carry the maximum HA load, according to the code rules as shown in Fig. 5.7 and Table 5.1.

When considering the lane containing the HB vehicle, certain parts of the lane do not carry HA load. Only the area of zones outside the area displaced by the HB vehicle are used to calculate the effect of that loading. However, the code specifies that the loaded length used to calculate the intensity should include those element of the zones that would have been loaded had the HB vehicle not been present.

5.5- PROBLEM FORMULATION

Any highway bridge may be loaded with various combinations of different types of vehicle, each having a certain number of axles, axle loads, axle spacing, and outstanding regions beyond the exterior axles. The length of any bridge may be occupied with any possible combination of these vehicles. The problem associated with obtaining the extreme effects of the actual vehicles on a given influence line is a complex optimisation problem which can be subdivided into the following two sub-optimisation problems: (1) choosing a combination of vehicles most likely to produce the extreme effects; and (2) determining the exact position and the spacings of the chosen combination of vehicles along the bridge such that the extreme effects are obtained.

This problem can be formulated as a mathematical programming problem where the aim is to find a set of "design-variables" such that a given "objective-function" is maximized or minimized subject to a set of "constraints". In this case, the design variables specify the vehicle combination, their order, and the position of their axles along the girder. The objective function measures the value of the effect of the loading on the influence line, where this value must be either maximized or minimized. The constraints represents the relationship and bounds between the spacing of the axles and the distance between vehicles.

The problem of loading bridges with moving loads depends not only on how many vehicles are on the bridge, but on the location of these vehicles and their arrangement. These computational difficulties have been reflected in various design specifications and have given rise to various simplified loadings such as the HA and HB loading.

The problem of determination of the extreme effects due to HA, HB, and HB with the associated HA loadings over a given influence line are formulated as the following mathematical programming problem. This is achieved by first defining the objective-function which is subject to a set of constraints, and then apply Genetic Algorithms for the optimisation process.

5.5.1- HA LOADING

The coordinates of the points a,b,c,...,f shown in Fig. 5.8(a) are determined where the influence line changes sign. The areas within these points are computed using any available method of numerical integration. The aim is to find W and A so as to maximize or minimize

$$E_{HA} = A.W \qquad 5.1$$

in which E_{HA} is the effect defined as the load intensity multiplied by the areas of the combined zones , $A=\sum A_i$ is the sum of the areas of the combined zones, n is the number of combined zones, and W is the load intensity which is a function of the loaded length L and is given by the equations

$$W=336(1/L)^{0.67} \qquad L <= 50 \text{ m} \qquad 5.2$$

$$W= 36(1/L)^{0.1} \qquad 50 < L < 1600 \text{ m} \qquad 5.3$$

where $L=\sum L_i$ is the loaded length which is the sum of the base lengths of the zones being considered.

In order to decide which zones should be included, one option is to do an exhaustive combinatorial search which explores all possibilities. In this approach each permutation of loads is considered; the loaded length is the sum of the base lengths of the zones being considered, the load intensity is a function of this loaded length, and the effect is the load intensity multiplied by the areas of the included zones, Fig. 5.8 (b). This approach is economical for small to medium size bridges where the number of permutations for all possibilities is relatively small (Appendix E) .

However, for a large size highly redundant multiple-stay system this approach may be uneconomical. This is due to several factors. The static behaviour of such a structure is the result of a complex interaction between several parameters. The shape of ILD's of the forces are not necessary known in advance and they tend to change sign within the span as well as at supports. This results in the number of zones to be combined to be relatively large and the process of using exhaustive combinatorial search to be rather expensive.

5.5.2- HB LOADING

The procedure for obtaining maximum or minimum effects of the HB loading can be illustrated by considering the system of loading shown in Fig. 5.9, consisting of four axles, A, B, C, and D with loads P_A, P_B, P_C, and P_D, respectively. Axle A is assumed to be always located at an interval point along the girder. Axle B is located at station X_B where

$$X_B = X_A + d_1 \qquad 5.4$$

Axle C is located at station X_C, where,

$$X_C = X_B + Z \qquad 5.5$$

and Z is bounded between two limits, $d_{2\,mim}$ and $d_{2\,max}$, as shown in Fig. 5.6. Axle D is located at station X_D, where

$$X_D = X_C + d_3 \qquad 5.6$$

X_A, X_B, X_C and X_D are found by solving the following optimisation problem:

$$\text{Min/Max} \qquad E_{HB} = P_A Y_A + P_B Y_B + P_C Y_C + P_D Y_D \qquad 5.7$$

Subject to:

$$X_B - X_A = d_1 \qquad 5.8$$

$$X_C - X_B \geq d_{2\,min} \qquad 5.9$$

$$X_C - X_B \leq d_{2\,max} \qquad 5.10$$

$$X_D - X_C = d_3 \qquad 5.11$$

In which E_{HB} is the effect. The values X_A, X_B, X_C, and X_D and P_A, P_B, P_C, and P_D are the position and loading of axles A, B, C, and D respectively. The values Y_A, Y_B, Y_C, and Y_D are the ordinate of the influence line at positions X_A, X_B, X_C, and X_D, respectively. The value d_1 and d_2 represent the fixed spacing between axles A and B, and C and D, respectively. The spacing between axles B and C is bounded between an upper limit $d_{3\,max}$ and a lower limit $d_{3\,min}$.

Constraint eqs. 5.8 and 5.11 specify the position of exterior axles, such that in all conditions at least one axle is on the girder.

The search process for the worst position of the HB vehicle can be determined in two stages. In the first, the HB vehicle is placed in a number of positions to find the worst effect. Subsequently, the vehicle moves from this position in steps of ever decreasing size to find the true position where the effect is worst.

In the first phase, the largest ordinate on the influence line is determined. The vehicle is positioned so that each axle is placed at that ordinate in turn (4 positions). This is repeated for each boogie spacing (5 cases). Normally, one of these cases will be close to the worst position and can be used for the second refinement phase.

Once the approximate worst position has been found, the search for the true worst position is done by changing the vehicle position or the boogie spacing by a small amount. Initially, movements of 0.5m are used, but this figure is halved at each stage until the vehicle position and spacing are correct to 1mm.

5.5.3- HB LOAD ASSOCIATED WITH HA LOAD

The aim is to maximize the effect of the HB loading together with the associated HA loading. This is referred to as HA+HB. Fig. 5.10 shows the loading arrangement of the HA+HB loading. The maximum or minimum effects of HA+HB can be obtained by combining the two systems of loading derived earlier on for the HA and HB loadings being applied separately. Therefore the formulation of the HA+HB effect will become

$$E_{HA+HB} = E_{HA} + E_{HB} \qquad 5.12$$

where E_{HA+HB} is the effect of HA+HB loading which is equal to the effect of HA loading, E_{HA} defined in section 5.5.1, being added to the effect of the HB loading, E_{HB} given in section 5.5.2.

For large size bridges, the governing load case would be either HA load being applied separately or HA+HB. The HB load case acting on its own is not usually a governing load case.

The process of combining HA+HB subject to all the constraints mentioned above has made the search space for finding the maximum combination of HB and HA to be enormous. In order to reduce the search space the following simplification can be made. The combination of zones which gives the worst maximum HA loading acting on its own is first calculated. This results in the most two important parameters being calculated, the load intensity W and the corresponding loaded length. The loaded length, which is the sum of the base lengths of the worst combined zones for the HA loading, would restrict the positions of the HB load to be applied within the predefined loaded length, and therefore would result in the first reduction of the search space. While the load intensity W would give the worst value of the associated HA UDL to be used, and therefore reducing the search space for the second time by not calculating the corresponding loaded length and consequently the load intensity for every position of the HB vehicle.

After deciding on the zones to be combined and the associated HA UDL to be used, the HB vehicle is then positioned at every point contained in every zone. This is done by displacing the HA load over the length of the HB vehicle as well as 25 m either sides using the same approach presented earlier on for the HB load till the worst scenario of the maximum HB and HA is found.

From this it can be concluded that the key player in the search process lies in the optimisation of HA loading alone, acting on the bridge. This is true because it produces the worst combination of zones resulting in obtaining the worst loaded length with its corresponding load intensity. These two parameters are then used for the maximization process of HA+HB load.

5.6- HA LOADING OPTIMISATION VIA GENETIC ALGORITHMS

The problem of determination of extreme effects due to HA loading is to solve the following optimisation problem:

Problem GA2:

Minimize / Maximize $E_{HA} = A.W$

Subject to:

$W=336(1/L)^{0.67}$ $L <= 50$ m

$W= 36(1/L)^{0.1}$ $50 < L < 1600$ m

If the ordinates of the influence line can be expressed as a function of abscissa, x, and the constraints had been linear then the problem would have become a constrained optimisation problem. Unfortunately, this is not possible. Indeed, it is generally difficult to express the equation of influence lines in closed mathematical form. The influence lines may be discontinuous at sections under consideration and may have different forms at various portions of the bridge. In such cases the ordinates of the influence lines are discrete and are presented in a tabular or numerical form as shown in Table 5.2 and 5.3. Thus, the use of classical or formal optimisation programming techniques is not generally feasible and stochastic methods of search should be used.

For this particular problem, searching for the optimum solution of HA loading, Combinatorial methods is a common practice (Bridge Engineering Consultancy, UK). Combinatorial methods seek the solution by doing an exhaustive search which tries every single possibility in the search space. Thus, for highly complicated structures such as cable-stayed bridges where the final design/analysis is usually achieved after so many iterations, Combinatorial exhaustive methods would be regarded as rather expensive solutions.

For this reason GAs have been modelled and implemented for the optimisation of the HA loading defined by Problem GA2. It was outlined in Chapter 4 that the application of GAs to optimisation problems requires to specify 5 components. These are:

(1) A string representation of the solution

(2) A way of seeding the initial population

(3) A fitness function

(4) Genetic operators

(5) Values of GAs parameters.

5.6.1- STRING REPRESENTATION

As mentioned earlier, the data presented in Table 5.2 and 5.3, which represent the influence lines, are discrete by nature. Therefore, after the processing stage, the resulting data shown in Table 5.4 are classified to be discrete too. The variables needed for the Genetic Algorithms optimisation of the HA loading are shown in the seventh and fifth columns of Table 5.4. They represent the area and its corresponding base loaded length to be used in the optimisation of problem GA2.

The binary coding, introduced in Chapter 4, will be applied for the coding of problem GA2. Once individual parameters have been coded, the next step would be to assemble them in a string. This is achieved by concatenating the individual parameter codings into a single string. When a particular area is within the string, the bit value is 1 and, 0 vice versa.

Fig. 5.8a shows a typical influence line diagram. The positive region of this influence line consists of 3 zones. The area of these zones are labelled as A1, A2 and A3 while the corresponding loaded length are labelled as L1, L2 and L3. The strings shown in Fig. 5.8b show the binary coding representation. The length of string 'l' depend on the number of zones to be combined. For the strings shown in Fig. 5.8b, 'l' takes the value of 3. Every string represents a solution in the search space. The characteristics of the solution are defined in the position of the genes (bit) in the chromosome (string). The first, second and third gene represents the characteristics of the first, second and third zone, respectively. The characteristics of every zone are known as the area and the loaded length. The notation of '1', '0' at a particular gene in the string means the zone which corresponds to that gene is either included or excluded, respectively. The total area is the summation of the areas of the zones to be permutated. While the loaded length, which is primarily used for the calculation of the load intensity, is the summation of base loaded length of the included zones.

As an example, the string '111' combines of the first, second and third zones. The total area is the sum of all areas and equal to A1+A2+A3. The load intensity W is based on the sum of the base loaded lengths and is equal to L1+L2+L3. The effect, E, is calculated by the multiplying the total area by the load intensity W, and is equal to W.(A1+A2+A3).

Another example is the string '101'. It combines the first and the third zones and excludes the second zone, where the '0' bit is shown. The total area is equal to A1+A3. The load intensity W is based on the loaded length equals to L1+L3. And finally the effect, E, is calculated as W.(A1+A3).

5.6.2- INITIAL POPULATION

Having coded the problem as finite-length binary strings, the initial population of solutions can be set up. It is a common practice when beginning a Genetic Search to initialise a population of Npop strings by randomly generating bits with equal probability p_o (eg 0.5) for 0 and 1. Indeed, a random number is generated between 0 and 1 and compared to p_o. If the generated number is greater than or equal to p_o then bit value is 1, otherwise 0.

5.6.3- FITNESS FUNCTION

It was seen in Chapter 4 that the fitness function returns a measure of how good any encoding solution is. The principle of survival of the fittest within Genetic Algorithms depends upon maximizing a fitness function. Since the maximization and minimization processes of the HA loading are usually performed separately, therefore minimization process can be transformed into maximization by multiplying the objective function by -1.

The HA loading has been formulated in problem GA2 where the fitness is then:

$$Fit = E_{HA} = A.W$$

This fitness relates the effect, E_{HA}, to the area, A, and the load intensity W. Depending on the value of the loaded length, the load intensity W is derived from eq. 5.2 or 5.3 accordingly.

5.6.4- GENETIC OPERATORS

The genetic operators used in the HA loading (problem GA2) are the same as those presented earlier to solve problem GA1. The selection process will be presented via the simple roulette wheel approach with the purpose of providing bias in the population to provide more fit members and to clean the population of less fit members.

The coding adopted for problem GA2 has resulted in strings with relatively small size (section 5.8), therefore, Crossover operator applied here is one-point crossover. Multiple-point crossover will be investigated in Chapter 6 where long strings are used. Mutation used is bit mutation which changes 0 to 1 and vice versa if the probability test is passed.

In brief, the Genetic Operators utilized are :

- Selection
- Crossover
- Mutation

These parameters were discussed in a greater length in Chapter 4.

5.6.5- PARAMETERS VALUES

It has long been acknowledged that the parameters controlling GAs can have a significant impact on their performance. Moreover, GA theory gives little guidance for their proper choice. In this area, three works have been published. De Jong (1975) performed several computational experiments on five minimisation functions, to try to gain some insight into the influence of population size (N_{pop}), probability rate of crossover ($p_{crossover}$) and mutation ($p_{mutation}$) operators, and a number of other parameters, on the efficiency of genetic search. Two performance measures were designed for this purpose: the **on-line performance and the off-line performance.**

The **on-line performance** is simply the mean of all function evaluations up to a given number of trials while the **off-line performance** is the average best fitness of a population up to the given number of trials. Empirical results produced a set of numerical parameters that was generally found to yield the best on-line and off-line performances: N_{pop} = 50-100, $p_{crossover}$=0.60 and $p_{mutation}$=0.001.

Grefenstette (1986) has suggested a more robust approach for the optimal selection of these parameters. Indeed, he has developed a **Meta-Genetic Algorithm** that takes not only the design variables as chromosomal representation but also values of the desired parameters (N_{pop}, $p_{crossover}$, $p_{mutation}$). Applications of these algorithms to the minimization functions used in the above study by De Jong, to generate the best on-line performance, give the following recommended parameters: N_{pop} = 30, $p_{crossover}$=0.95 and $p_{mutation}$=0.01.

Grefenstette's combination of parameter values, which recommended a smaller population size and much higher rates of the genetic operators than De Jong did, have been proven useful across a variety of problem domains (Davis, 1989).

More recently Grefenstette's results were reinforced by the work of Schaffer et al. (1989) that has consumed more than 12 months of CPU time (1.5 CPU years on Sun 3 and VAX machines). They have used De Jong test functions and some additional problems (five other functions) which were complicated and multimodal. They were able to show that robust parameter settings found by their search indicated that good on-line performance can be expected with: N_{pop}= 20-30, $p_{crossover}$=0.75-0.95 and $p_{mutation}$=0.005-0.01.

5.6.6- SENSITIVITY OF PARAMETERS

From the three studies presented above it becomes clear that there is supporting evidence that the reported parameter sets are function independent. Therefore, good parameter values may be taken from the above fairly recent work in this area and the values of the parameters applied are:

N_{pop} = 30;

$p_{crossover}$ = 0.95;

$p_{mutation}$ = 0.01.

These values have been adopted in the parametric study described in section 5.10. However, in section 5.9.1, a smaller value for the population size N_{pop}=10 was selected which allowed us to list the results of input and output files in the thesis so that various aspects of the optimisation process can be shown when judging the strings/solutions produced in each population.

Additional tests on mutation operator have been carried out in section 5.9.2 where the consequence of not including certain loading scenarios in the initial population have been addressed. It was shown that the mutation model is sensitive enough to introduce these scenarios at a latter stage during the GA run.

The use of multiple crossover sites will be examined in section 6.7 of Chapter 6 since the strings formed for this application (sections 5.9 and 5.10) are fairly short and as a result crossover over one site has been considered.

Results reported in sections 5.9 and 5.10 have shown that the GA algorithm does indeed arrive at the desired set of load combinations. Results have been verified and compared through a complete enumeration scheme where it was shown that GA can actually find a global optimum.

5.7- THE GA MODEL ADOPTED

Solution of the problem GA2 can be provided by the following scheme:

```
Algorithm
    Initialise Population:   Randomly generate an initial
                             population of size N_pop
    While Not (terminate condition) Do
        Compute the fitness  function of each string
                             based on HA loading
                             conditions
            For i=1 to (N_pop/2) Do
                Selection    Pick two parents by applying
                             Roulette Wheel selection;
                Crossover    Crossover the parent based
                             on crossover probability to
                             produce new offsprings;
                Mutation     Mutate each offspring (bit
                             mutation) based on mutation
                             probability;
            ENDFOR
    ENDWHILE
```

5.8- BRIDGE SYSTEMS: GEOMETRY AND LAYOUT

The five-spans cable-stayed bridge shown in Fig. 5.1 is geometrically symmetric with a varying total length depending on Inner spans (Li), back spans (Lb), and unsupported span lengths (Lu). Four different spans arrangement are considered covering cable-stayed bridges of the type small, medium, and large size (Fig. 5.1). In the first structural system, inner spans (Li), back spans (Lb), and unsupported span length (Lu) take the values of 100, 40, and 20m, respectively. Spans arrangement for remaining systems are given in Fig. 5.1. It should be noted that for all systems the ratio of inner span to back span is kept constant, (0.40), so that a direct comparison on the behaviour of the different structural system can be made.

For all structures, the stay system consists of 24 cables. The cable system radiates from each pylon top. The points of attachment of the stay cables radiating from each pylon to the main spans and back spans are equidistant, leaving a central unsupported length Lu as shown in Fig. 5.1. The deck elevation is 10m with a total height of each pylon = 46m. The stiffnesses of pylons and girder are as given in Fig. 5.1.

The girders are considered to be continuous at the towers and it is laterally supported at the piers as is passes freely through the pylon legs. Thus, there is no moment transfer between the girders and the pylons. The ends of the cables are anchored at the top the pylon and the pylon are fixed at the base. Axial deformations of the girder and pylons are ignored. All rigid supports of the girder are rollers, except for one hinged bearing to accommodate the length changes due to temperature, as well as to take into account the longitudinal forces due to earthquake, wind, etc. The dimensions and sectional properties of the elements of the bridge with 24 cables per plane are given in Fig. 5.1. The cross-sectional area of cable was varied between $0.005m^2$ and $1.0m^2$, with all cables in a particular case having the same cross-sectional area.

5.9 - IMPLEMENTATION AND RESULTS

This section discuses the implementation of GAs as optimisation tools and how it relates to the analysis of cable-stayed bridges subject to live loads defined in problem GA2.

5.9.1- IMPLEMENTATION FOR CABLE-STAYED BRIDGES

In this section the structural arrangement of the cable-stayed bridge with inner spans, back spans, and central unsupported span length, take the values of 100, 40, and 20m, respectively (Fig. 5.1). The cross-sectional area of cable takes the values of 0.005, 0.01, 0.025, 0.05, 0.075, 0.1, 0.25, 0.5, and 1.0m^2 with all cables in a particular case having the same cross-sectional area. This system is of a small span type cable-stayed bridge, allowing us to demonstrate the working of GAs for ILD's subject to live loading presented in problem GA2. The selected bridge offers the flexibility of having relatively short strings with manageable data files to be able include in the thesis. In section 5.10 only final results of the optimisation process are presented for the remaining systems (structure 2, 3, and 4 of Fig. 5.1) covering cable-stayed bridge of the type medium to large size.

It was reported earlier that behind the conceptual simplicity of GAs, there are a variety of parameters such as population size and probabilities of mutation and crossover. Effective values of these parameters for bit string representation have been studied in section 5.6.5. It should be noted that the objective of this section is to illustrate the working of GAs for the structural analysis optimisation of cable-stayed bridges subject to traffic loads defined in problem GA2 whereby a list of input and output files can be contained in this thesis so that various aspects of the optimisation process can be shown. For this example the genetic parameters given in section 5.6.5 will be used herein with only one alteration being made on the population size, Npop, so that it is reduced from 30 to 10 which allows us to list the results in each population and have an insight look when judging the strings produced in each generation. Therefore for this demonstration the genetic parameters for problem GA2 are taken as:

1. Population size: N_{pop} = 10; as opposed to N_{pop}=30 for section 5.10;
2. Probability of Crossover: $p_{crossover}$ = 0.95;
3. Probability of mutation: $p_{mutation}$ = 0.01;
4. Number of generations = 20;

In this example, the numerical results obtained from the application of GAs on the analysis of the cable-stayed bridge (structure 1) given in Fig. 5.1 is presented. The aim is to find the maximum positive moment at the centre of the bridge subject to traffic loads defined in problem GA2. An exhaustive search was also carried out in order to allow for a direct comparison between the GA and the exhaustive method solution (Appendix B).

Dimensions and structural properties of the cable-stayed bridge used in this example (Structure 1) have been discussed in section 5.8 and are shown in Fig. 5.1.

For this structure, analyses for nine different cables stiffnesses arrangement are carried out. ILD's for moment at the centre of the bridge with cables area taking the values of 0.005, 0.01, 0.025, 0.05, 0.075, 0.1, 0.25, 0.5, and 1.0m^2 with all cables in a particular case having the same cross-sectional are given in table 5.3. ILDs for moments for three different cables area (0.005, 0.075, and 1.0m^2) showing the variation of ILD with cable stiffnesses are shown in Fig. 5.2. It shows that the influence line ordinates decrease as the cable stiffnesses increase resulting in the generation of smaller bending moments at the centre of the bridge.

The discrete ordinates of the influence lines for a 100 kN load travelling across the deck at the centre of the bridge are shown in Table 5.2. The first column shows the intervals of the bridge for the 2D plane frame model. The second column represents the geometrical abscissa of each interval. The third, fourth, and fifth columns represent the discrete ordinates of the influence line for axial force, shear force, and bending moment, where ILD's for moment has been graphically presented in Fig. 5.3.

Given the ILD's, the objective of this procedure is to find the maximum effects of the HA highway loading being applied on the ILD for moment as shown in Fig. 5.3.

The input to this procedure is (1) the abscissa, X, shown in the second column of Table 5.2, of the influence line given at all intervals across the deck; (2) the type of influence lines, which in this case, is ILD for moment, shown in the fifth column of Table 5.2; and (3) the applied loading, which is HA highway loading defined in problem GA2.

Prior to the GAs optimisation process, the data of the influence line shown in Table 5.2 is read and processed and the result is stored in a database shown in Table 5.4. In this table the first column represents the zone number of positive and negative zones where the influence line changes sign. The second column represents the base loaded lengths corresponding to the zones in the first column. The second column in this Table represents the areas of positive and negative zones where the influence line changes sign. The third column identifies the status of every zone as being cusped or not (BD37/88, 1989), corresponding to 'Y', or 'n' respectively. The cusping test (BD37/88, 1989) is carried out by dividing the area of the zone, given in the seventh column, by one-half of the absolute maximum ordinate of the zone. This is equivalent to finding the triangle with the same area and height as the true zone. The base of this triangle, which represents the new loaded length, is then compared with the original loaded length shown in the second column. If the new loaded length is smaller than the original loaded length, then the zone is given the status of a cusped zone, and therefore a value of 'Y' appears in the third column. Otherwise, a value of 'n' appears instead. The degree of the cusping factor is given in the forth column. The fifth column shows the cusped loaded lengths which are later used in the calculation of the load intensity as shown in the sixth column and calculated according to eqs. 5.2 or 5.3. The cusped loaded length is calculated as follows: If the zone is cusped, then the loaded length takes the value of the new loaded length, otherwise the value of the original loaded length holds.

In normal optimisation practice, minimization can be transformed to maximization or vice versa by multiplying the objective function by -1. Equation 5.1 may produce positive or negative values and we are interested in maximizing positive values and minimizing negative values. Minimizing negative values is equivalent to maximizing positive values and consequently the same GA program can perform this.

To deal at the same time with positive and negative values (Table 5.4), it was found that the best way is to penalize the fitness function when it meets a negative in case of maximization and vice versa. Indeed if a maximization is considered, when a negative value is encountered, it is automatically reduced to zero. And as a consequence, the maximum is sought only among the positive values. With minimization, in the event of meeting a positive value its corresponding fitness is zero and the negative fitnesses are multiplied by -1 to transform the minimization to maximization.

Table 5.6 shows a detailed computer output sample for the optimisation process using GAs. This output displays the results of 20 generations. Every generation has a population size of 10 (discussed earlier). Strings consisting of 0's and 1's extend over a length of 7, where there are 7 zones to be permutated (Table 5.4). The 1st, 2nd, 3rd,... up to the 7th positions represent the zones shown in the 1st, 2nd, 3rd,... up to the 7th rows of Table 5.4, respectively. Strings representation and formation have been discussed in section 5.6.1. In table 5.6 every string represents a solution in the search space and they are shown in the second column of Table 5.6. The decoding of every string is translated into fitness, which represents the moment at the centre of the bridge, and is given in the third column of Table 5.6. This fitness represents the value of bending moment which corresponds to the zones to be combined which are contained in its string. The fourth, fifth and sixth columns represent the maximum fitness, the summation of all fitnesses, and the average fitness which they correspond to every generation. A summary of each generation with its corresponding max fitness, summation of all fitnesses, and the fitnesses average reported in each population are shown in Table 5.5.

The **'Strings'** (individuals) consist of 1's and 0's representing the defined variables in the objective function, which in this case are the zones to be permutated. The 1st, 2nd, 3rd,... up to the 7th positions in each string represent the zones shown in the 1st, 2nd, 3rd,... up to the 7th rows of Table 5.4, respectively. Groups are formed , initially at random, to compose families of strings each family containing a single set of parameters representing a solution in the search space. The **'Fitness'** of each group is then evaluated and assessed against the objective function. The strings in the best families are given favourable weightings in a selection process whereby pairs of strings, parents, are chosen, **'Mated'** (combined) by the **'Crossover'** process. An element of **'Mutation'** is introduced, whereby some bits are switched (0 to 1 or 1 to 0), to encourage the development of new genetic material.

After each cycle of selection, crossover and, possibly, mutation, the fitness of each family is again assessed by **'Decoding'**, converting the binary strings to decimal digits, and evaluation of the objective function as defined in eq. 5.1 and shown in the third column of Table 5.5. The cycle then continues into the next generation. As the generations progress, the population gets filled by more fit strings. The process is terminated when the maximum number of generations is reached.

Figs. 5.11 upto 5.16 present graphical display for the performance of GAs in each population through the 20 generation of the GA run which correspond to the data shown in the second and third column of Table 5.5. The initial generation known as generation 0 and another three more generation are shown in Fig. 5.11 while Fig. 5.12 shows the results of generation 4 up to generation 7, and so on, upto Fig. 5.16 which presents results of the last remaining generation only (i.e. generation 20).

The generations history convergence of the entire optimisation process is given in Table 5.5 and is presented in Fig. 5.17. It shows the performance of 20 generations in terms of the maximum and the average fitness found in each generation. It also shows the results of the optimum solution (2895.19 kNm) being calculated using exhaustive search method (Appendix B) which can be considered as a reference to judge the convergence of the GA solution toward the optimum solution.

During the GA run (Fig. 5.17), it is clearly noticed there have been improvement in the overall solution starting from generation 0 upto generation 4 where an optimum solution was detected. A loss in genetic material was reported in generation 9 both in terms of max fitness and the average fitness which is equal to the summation of all fitnesses divided by the population size. However, that loss was later recovered and a steady improvement was then obtained starting from generation 16 upto generation 20 where the optimum solution was located in every generation with good average fitnesses being reported.

The string [0001000] which correspond to the optimum solution for the bending moment at the centre of the bridge can be seen in generation 4, and 15 to 20 of Table 5.6. This string states that the 4th zone of Table 5.4 will give the worst bending moment when considered.

Appendix C1 presents a detailed computer output for the 20 generations compared to the one given in Table 5.6. It shows a record of design load components introduced at the various stages of the GA run together with the decoded value of the effect representing the bending moment in each population, areas and loaded lengths for the zones to be combined as defined in each string. Table 5.6 and Appendix C have both shown that a wide variety of solutions have been considered during the GA run before fixing on the optimum solution calculated through the exhaustive search reported in Appendix B and displayed in Fig. 5.17.

5.9.2- SENSITIVITY OF MUTATION PARAMETER

A special test on the sensitivity of mutation operator to generate certain load scenarios not included in the initial population is presented in Table 5.11. This table presents the output of a GA run consisting of 20 generations where the crossover operator has been disabled by assigning a zero to its probability. The probability of mutation is kept the same as in the previous runs (i.e. 0.01). By doing so new solutions can only be formed through the mutation model. Although the probability of mutation is very low (0.01) the 2nd generation has produced the first new string (shown in ***bold italics***). Subsequent solutions have been reported in generations 3,5,6,7,9,10,12 and 16 where a near optimum solution was actually located in the 9th generation (***[00010100]***). A detailed output for the GA run shown in Table 5.17 can be found in Appendix C2.

In sections 5.9 the working of GAs have been demonstrated as effective optimisation tools for the analysis of cable-stayed bridges subject to combinatorial live loads defined in British Standard (BD37/88, 1989). It has also been shown that the different stages needed for the optimisation process together with deep insight look into every stage of the optimisation process. The optimum solution located by GAs has been checked with the exact solution generated using exhaustive method.

In the following section GAs will be used to carry out a parametric study on the behaviour of cable-stayed bridge subject to various span arrangements and cable stiffnesses (Fig. 5.1) under loading conditions given in BD37/88.

5.10- PARAMETRIC STUDY

In this section a detailed investigation on the effect of cable stiffness on the structural behaviour of cable-stayed bridges subjected to traffic loads defined in BD37/88 is undertaken. Four structures with different spans arrangement (Fig. 5.1), covering cable-stayed bridges with small, medium and long span sizes are investigated. Detailed description of the cable-stayed structures is given in section 5.8. For each structure, nine different sub-structures are considered due to the variation in the cross-sectional area of cables between $0.005m^2$ and $1.0m^2$, with all cables in a particular case having the same cross-sectional area (Tables 5.7 to 5.10). These values do not represent neither the lower nor the upper limits of practical cable sizes, but instead they have been chosen to demonstrate that the behaviour of the cable-stayed bridge becomes insensitive if cables areas exceed certain limits. All structures have 24 cables per plane and the side span to inner span ratio is kept constant (0.4) in order to allow for a direct comparison of the different structures. The girders of the four structures are considered to be continuous at the towers. The ends of the cables are anchored at the top of the pylon and the pylon are fixed at the bases. The dimensions and sectional properties of the elements of different structures are given in Fig. 5.1.

A cable-stayed bridge, in the rigorous sense, behaves nonlinearly when loaded. Nonlinearity (Walther, 1988) is introduced because of: (1) The possibility of large deformations; (2) the P-Δ effect in the towers; (3) axial force-flexure interaction in girders; and (4) sag in cables. The degree of nonlinearity will, nevertheless, vary with the stiffness of the various members and cables and the degree of pretension under selfweight loads. These aspects of the problem have not been included in this parametric study since the analysis presented in this chapter is based on ILDs and is limited to the case of a two dimensional plane frame cable-stayed bridge system which displays geometrical as well as material linearity.

The main objective of this section is to present a parametric study on multiple-span cable-stayed bridges under live loads defined in BD37/88. The influence of the variation of cable stiffnesses together with four different span arrangements are considered. GAs are used for the optimisation process of locating the worst loading scenarios and their working have been fully demonstrated previously in section 5.9. Several issues facing the GA optimisation process have been addressed in section 5.6. In this study GA operators (Crossover, Mutation, Population size) would take the values reported in section 5.6 (i.e. N_{pop}=30, $p_{crossover}$=0.95, $p_{mutation}$=0.01 and number of generations=20).

The maximum values of moments at the centre of the bridge due to the application of HA, HB, and associated HA loadings for the four cable-stayed systems shown in Fig. 5.1 are given in Tables 5.7 to 5.10. Each table corresponds to specified span arrangements referred to in Fig. 5.1 as 'Structure j' where j=1,4, is the structure being studied. For example, Table 5.7 corresponds to structure 1 where spans are as follows:

L_i = Inner span = 100 m
L_b = Back span = 40 m
L_u = Unsupported span length = 20 m

In Table 5.7 the 1st column shows the variation of cable stiffnesses for nine different areas where each area corresponds to a different sub-structure being analysed. For each cable area, moments at the centre of the bridge due to HA, HB, and associated HA have been calculated and are given in the 2nd, 3rd, and 4th column respectively which are graphically displayed in Fig. 5.18a. The 5th, 6th, and 7th columns shows the variation in the ratio of HB/HA, (Associated HA)/HA, and (Associated HA)/HB. While the 8th and the 9th columns represent the moment area under the ILD for the combined zones of HA and associated HA loading. And finally, the 10th and 11th columns show the loaded length and its corresponding UDL for the HA load.

Tables 5.8, 5.9, and 5.10 are similar to Table 5.7 except that they correspond to the 2nd, 3rd, and 4th structures which in turn correspond to different span arrangement (Fig. 5.1).

The variations of moments due to HA, HB, and associated HA loading with cable area for each inner span having the value of 100m, 200m, 300m, and 400m are plotted in Fig. 5.18a to 5.18d. It can be seen that moments generated due to any loading effect have decreased with the increase of cable stiffness. This decrease in moments is sharp in the beginning but become gradual later.

Fig. 5.18a shows the variation of HA, HB, and associated HA with cable area for structure 1 where the inner spans, back spans, and unsupported span length take the value of 100m, 40m, and 20m, respectively. The variation of girder moments due to HA alone and HB loadings have displayed sharp decrease for cable area between $0.005m^2$ and $0.075m^2$. Moments have continued to decrease after cable area of $0.075m^2$ and up to $0.25m^2$ but the rate of decrease become smaller. After cable area of $0.25m^2$ the decrease in moments becomes very small and the curve for HA loading tends to become asymptotic to the cable area axis. For HB loading, moments continue to decrease even beyond a cable area of $0.25m^2$, but the rate of decrease become small. The associated HA loading curves have negligible values compared with HA alone and HB loading.

Figs 5.18b, 5.18c, and 5.18d correspond to the 2nd, 3rd, and 4th structure, shown in Fig. 5.1, with inner spans taking values of 200m, 300m, and 400m respectively. It can be seen that these Figures display a similar behaviour to that of Fig. 5.18a except that the HA loading has increased steadily as the span length keeps increasing whereas in the 400m spans the moment due to HA alone becomes bigger than the moment due to HB loading. It can also be seen that associated HA load have increased considerably for the 300m and 400m span arrangement compared to those of 100m and 200m spans.

Fig. 5.19a shows the moment ratio of HB/HA loading with cable area for the four arrangements. The general trend of this figure show that the ratio of HB/HA decreases as the span length increases. For the 100m span arrangement the moment due to HB loading is more than double that of HA loading. A steady increase in the ratio can be seen upto a cable area of 0.075m², then it starts decreasing, and then becomes almost constant beyond a cable area of 0.5m². Curves for the 200m, 300m, and 400m spans arrangement are similar in that the ratio of HB/HA increases with cable area. For the 200m spans the ratio of HB/HA varies between 1.27 for a cable area of 0.005m² to 1.57 for cable area of 1.0m2. For the 300m spans arrangement the moment due to HA loading exceeded that due to HB loading for cable area between 0.005m² and 0.01m². For cable area above 0.01m² HB loading has a higher effect than the HA loading. However, for the 400m spans arrangement HA loading shows dominance over the HB loading over all cable areas.

In Fig. 5.19b the ratio of variation of (Associated HA)/HA loading with cable area for the four spans arrangements is shown. For the 100m spans the effect of associated HA is small compared to that of HA. Some improvement in the overall ratio of (associated HA)/HA has been reported for the 200m spans. A decrease in the ratio from 0.35 to 0.33 can be seen for cables area ranging from 0.005m² upto 0.05m² after which an increase to upto 0.36 is found. The curves for the 300m and 400m spans have shown a similar behaviour whereby the ration has decreased with increasing cable area of upto 0.025m² after which the ratio has remained almost constant.

5.10.1- PARAMETERS INLUENCING THE OPTIMISATION PROCESS

Parameters which govern the optimisation process of Eq. 5.1 for the HA loading under different span arrangementS are shown in Figs. 5.20. These the area under influence line diagram (ILD), the loaded length, and the UDL corresponding to the loaded length. The effect of cable area on the behaviour of every parameter is examined. It should be noted that these parameters are interrelated and any change in any of them leads to a change in the others. The area under the ILD depends on the loaded length. The bigger the loaded length the bigger the area. However, for the UDL the situation is reversed, the smaller the loaded length the bigger the UDL. Fig. 5.20a shows the variation of the moment area under the influence line with cable area for four different spans arrangement. The four curves which correspond to the 100m, 200m, 300m, and 400m span arrangements exhibit similar performance. It can be seen that moments have decreased with the increase of cable stiffness. This decrease in moments is sharp in the beginning but become gradual later. A similar behaviour for the loaded length can be seen in Fig. 5.20b. For the four curves it is noticed that the change in the loaded length after cable area 0.25 becomes very small. Fig. 5.20c shows the variation of the UDL with cable area. The general trend displayed is that the intensity of UDL has increased with increasing cable stiffnesses. This is true because with stiffer cables, the loaded length become smaller (Fig. 5.20b) and, therefore, the UDL intensity becomes bigger (Eq. 5.2 and 5.3). The rate of increase is noticeable for the 100m span arrangement. This is due to the fact that short loaded lengths of smaller than 50m have been calculated (Fig. 5.20b, and Eq. 5.2 and 5.3). The rate of increase in the UDL intensity have increased from cable area 0.005m^2 to cable area 1.0m^2 by 12.8%, 3.6%, 3.47%, and 3.28% for the 100m, 200m, 300m, and 400m spans arrangement respectively. It can be seen that for the 100m arrangement the rate of increase is high compared to the rest. This is so because the loaded lengths used are fairly small which results in fairly high values of UDL intensity to be encountered (Fig. 5.20b).

5.11- FINAL EXTREME EFFECTS

The extreme effects are maximum and minimum values of reaction at supports, deflection at specified intervals, and bending moment, shear and axial forces at all the intervals along the cable-stayed bridge. The procedure outlined in this chapter can be repeated for any ILD's, and its corresponding extreme effects can be calculated in a similar way.

5.12- CONCLUSIONS

In this Chapter parametric studies for the behaviour of cable-stayed bridges subject to traffic loads defined in BD37/88 have been presented. Live loads of that type are well known for their complexities and their implications on the number of analyses required in order to locate the optimum solution. Several parameters which governs their applications have been discussed and it was shown that the number of permutations and combinations needed for the location of the optimum solution is relatively large even for small size structures.

Traffic loads considered have included three types of live loadings: HA alone, HB alone, and associated HA with HB. The problems associated with the application of each of these loads have been thoroughly discussed. The interrelation between these loads have been also presented.

Difficulties and implications encountered with the calculation of optimum solution using exhaustive search method were discussed and it was concluded that even for a small size bridge the number of analyses required, for the location of optimum solution, using exhaustive methods is large and alternative methods of optimisation should be sought.

Problems associated with ILDs have also been discussed. It was shown that the influence lines may be discontinuous at sections under consideration and may have different forms at various portions of the bridge. It was also shown that the ordinates of the influence lines are discrete and it is difficult to express them in closed mathematical form. It was then concluded that the use of classical optimisation programming techniques is not generally feasible and alternative methods of search should therefore be investigated.

Combinatorial methods, although expensive, are known to be a common practice. They seek the optimum solution by doing an exhaustive search which tries every single possibility in the search space. It was concluded that for highly complicated structures such as cable-stayed bridges where the final design/analysis is usually achieved after so many iterations, Combinatorial exhaustive methods are considered to be expensive solution and Genetic Algorithms (GAs) cuold be used instead.

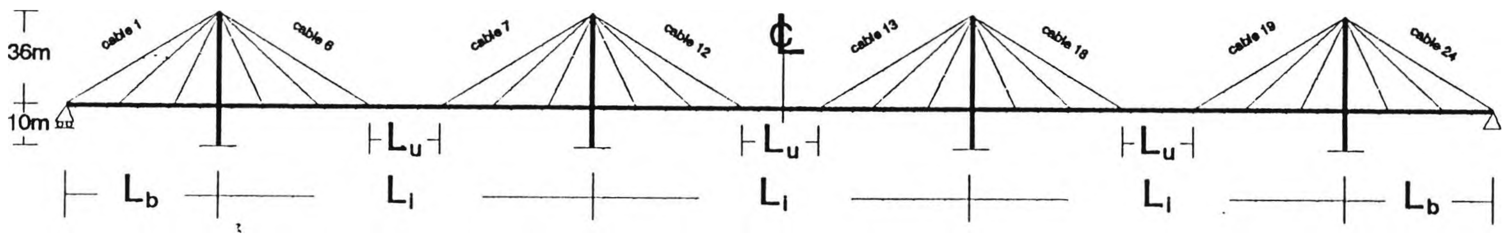
This chapter presented Genetic Algorithms as optimisation tools for the determination of maximum effects in cable stayed bridges subject to traffic loads defined in BD37/88. The five components of GAs, presented in Chapter 4, have been revisited with the scope of presenting parametric studies on the behaviour of cable-stayed bridges subject to these loads. These components can be summarized as: (1) string representation; (2) A way of seeding the initial population; (3) A fitness function; (4) Genetic operators; and (5) Values of GAs parameters.

The working of GAs and the different stages needed for the optimisation process have been demonstrated using a small span size cable-stayed bridge. It was shown that the values of GAs controlling operators (population size, probability of crossover, and probability of mutation) will affect the performance of the GA and its convergence towards the optimum solution. Sensitivity studies for these parameters were presented and their values were concluded. The consequence of not including certain loading scenarios in the initial population have also been addressed and it was shown that the mutation model is sensitive enough to introduce these at a latter stage during the GA run. The use of multiple crossover sites will be examined in Chapter 6 since the strings formed for this application are fairly short and as a result crossover over one site has been considered. A record of different design load components introduced at various stages of the GA run has shown that all feasible regions in the search space have been considered before fixing on an optimum solution. The GA results have been verified and compared by doing a complete enumeration scheme. It was demonstrated that GA can actually find a global optimum.

Having demonstrated the working of GAs on a relatively small cable-stayed bridge, GAs were then used for the optimisation of medium to large size cable-stayed bridges under traffic loads defined in BD37/88. A parametric study on four multi-span cable-stayed bridges with varying spans and cables stiffnesses has been presented. The variations of moments due to HA, HB, and associated HA loading have been discussed. It was shown that moments generated due to any loading effect decrease with the increase of cable stiffness. This decrease in moments is sharp in the beginning but becomes gradual later. It was also shown that moment ratio of HB/HA decrease as the span increases. It was concluded that HB loading will always be a dominating load case for small to medium size bridges while HA loading would govern for large size bridges. Different parameters which govern the optimisation process have also been studied. It was shown that these parameters are interrelated and any change in any of them leads to a change in the others. It was shown that the major source of complexity arises from the variation of the UDL with loaded length. For small to medium size bridges the change in the UDL value with cable stiffnesses was clear while for large size bridge the variation in the UDL became less noticeable. The effect of areas and loaded length have also been studied. It was shown that areas and loaded lengths have both displayed decrease with the increase in cables stiffnesses.

This chapter has presented Genetic Algorithms as efficient optimisation tools for the analysis of cable-stayed bridges subject to moving loads defined in BD37/88. The solution procedure is based on the use of influence lines. The problem has been modelled as a mathematical formulation. The principles outlined in this chapter could easily be modified and applied to any type of structures with linear behaviour and any type of moving loads.

The procedures of applying GAs outlined in this Chapter would be considered as the first foundation towards a much more complicated problem addressing the cables out effect in cable-stayed bridges presented in the next Chapter.



Girder Area = 0.3 m2
Girder Inertia = 0.5 m4
Tower Area = 0.3 m2
Tower Inertia = 0.2 m4

	Structure 1	Structure 2	Structure 3	Structure 4
Ll	100m	200m	300m	400m
Lb	40m	80m	120m	160m
Lu	20m	40m	60m	80m

FIGURE 5.1. MULTIPLE SPANS CABLE-STAYED BRIDGE WITH 24 CABLES ARRANGEMENT.

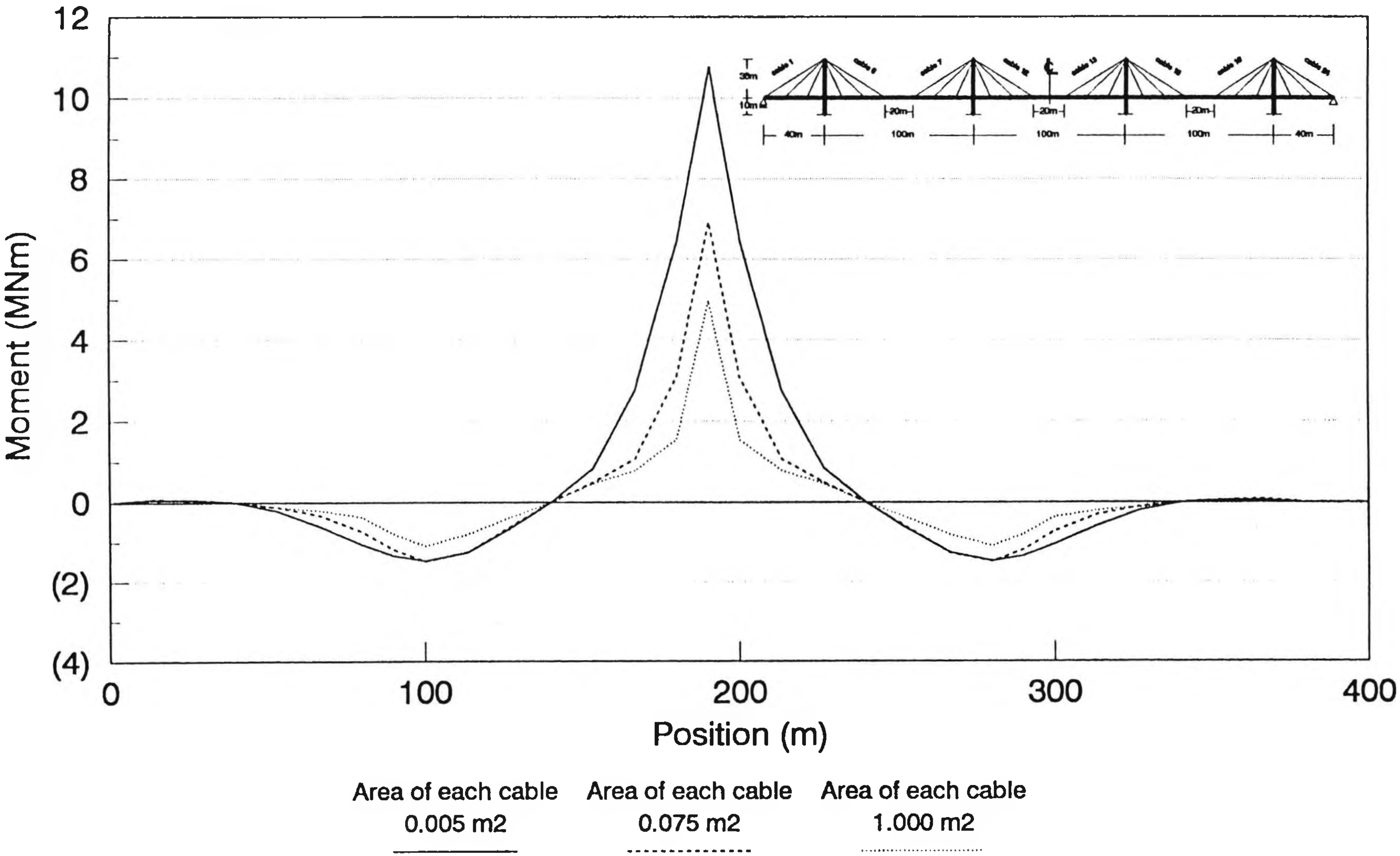
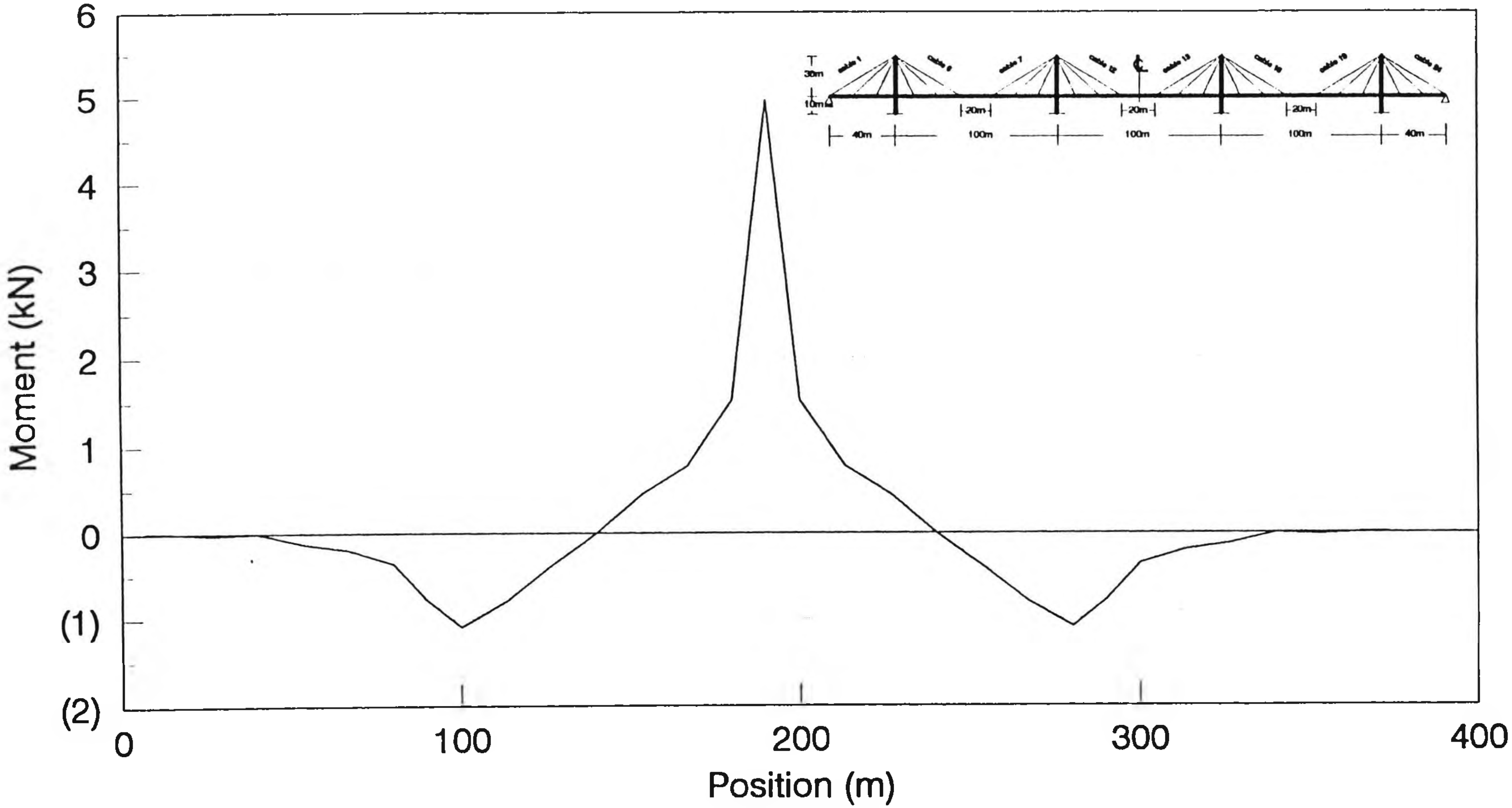


FIGURE 5.2. VARIATION OF ILD FOR MOMENT
WITH CABLES STIFFNESSES.



Area of each cable
1.000 m2

FIGURE 5.3. ILD FOR MOMENT AT THE CENTRE OF THE BRIDGE.

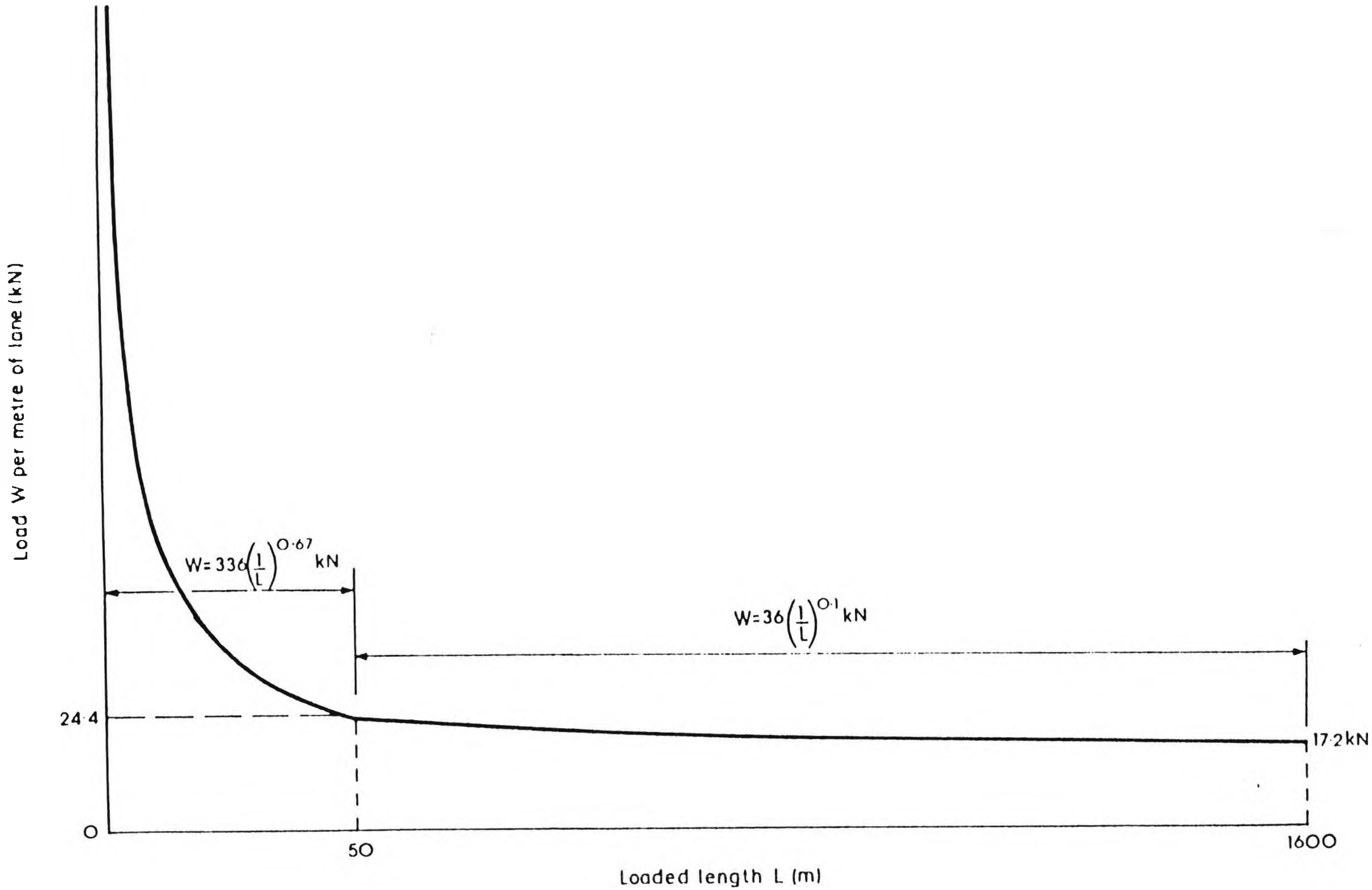
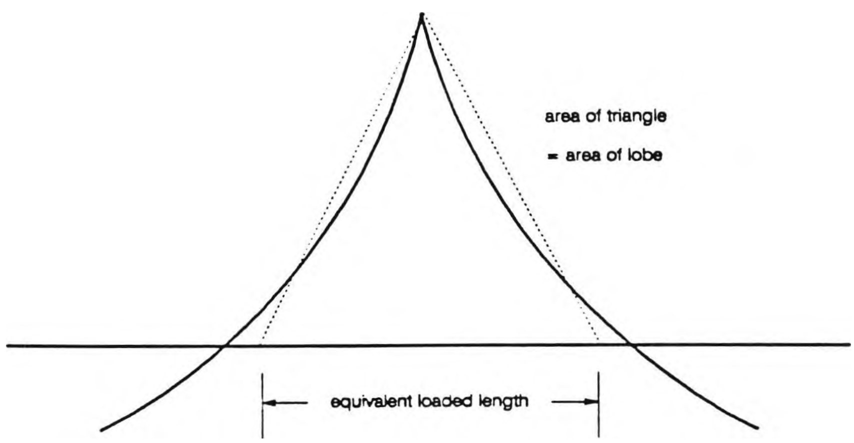
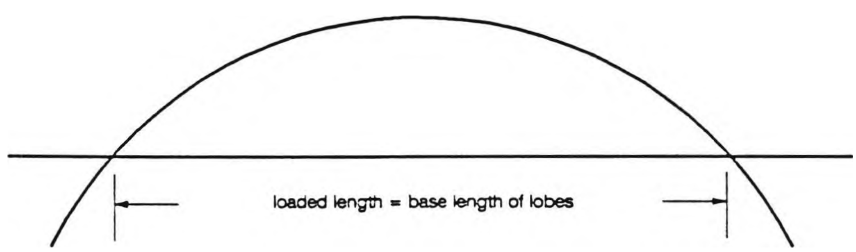


FIGURE 5.4. LOADING CURVE FOR HA UDL (NOT TO SCALE)
(BD 37/88, 1989).

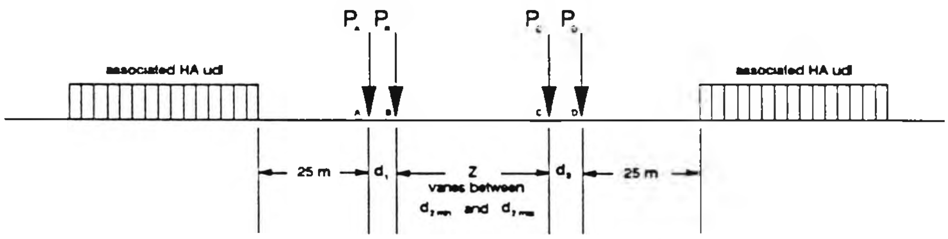


(a) Concave lobes



(b) Convex lobes

FIGURE 5.5. CONCAVE AND CONVEX ZONES.



$P_A = P_B = P_C = P_D$ = 10 kN/unit

$d_1 = d_2$ = 1.8 m

d_{3min} = 6.0 m

$d_{3max} = d_{3min} + 5.0 <= 26.0$ m

FIGURE 5.6. HB VEHICLE AXLE ARRENGEMENT.

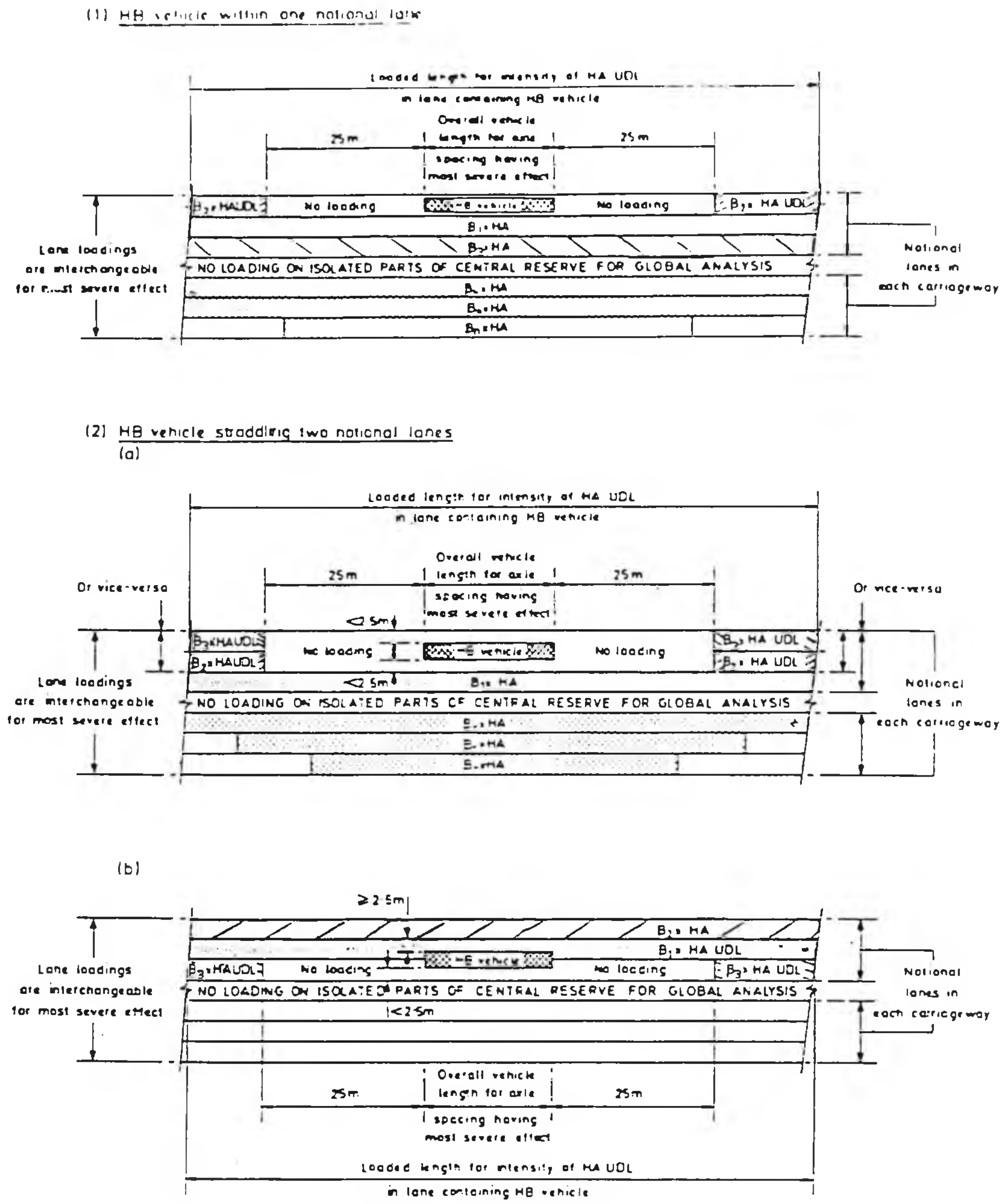
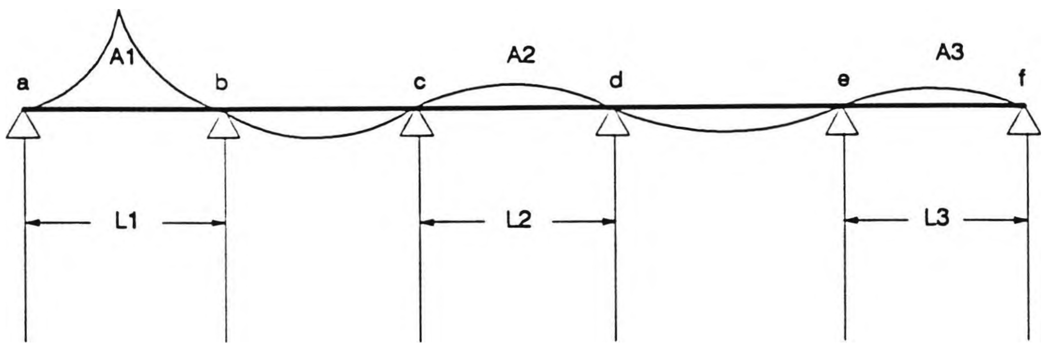


FIGURE 5.7. LOADING SCENARIOS FOR HB LOADING AND ASSOCIATED HA LOADING (BD 37/88, 1989).



(a) INFLUENCE LINE DIAGRAM DEVIDED
POSITIVE AND NEGATIVE LOBES

Permutation				
A1	A2	A3	Loaded Length (L)	Effect. N.W. W(L)
1	1	1	L1 + L2 + L3	W.(A1 + A2 + A3)
1	1	0	L1 + L2	W.(A1 + A2)
1	0	1	L1 + L3	W.(A1 + A3)
1	0	0	L1	W.(A1)
0	1	1	L2 + L3	W.(A2 + A3)
0	1	0	L2	W.(A2)
0	0	1	L3	W.(A3)

(b) PERMUTATIONS AND STRING REPRESENTATION.

FIGURE 5.8. ILD AND STRING REPRESENTATION.

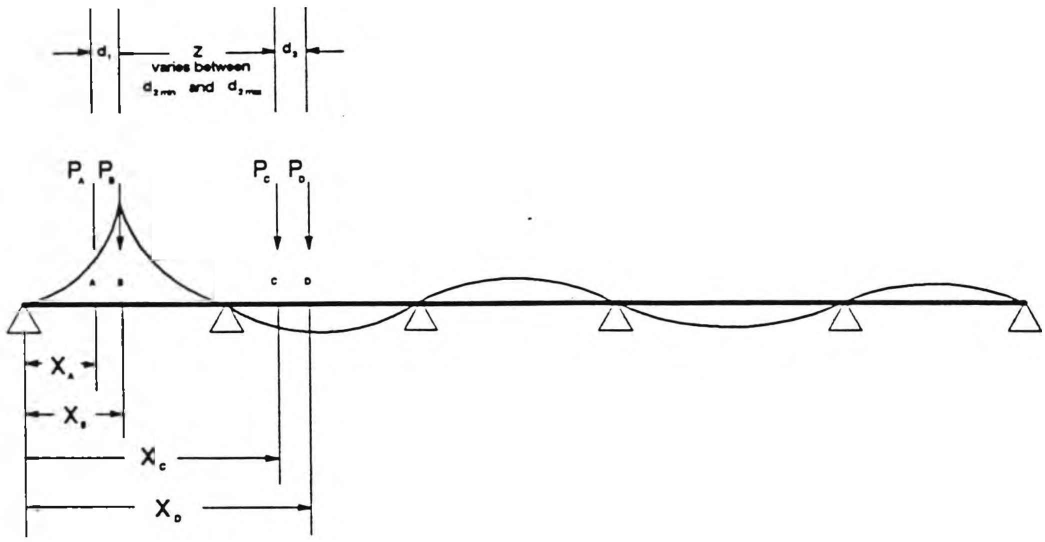


FIGURE 5.9. INFLUENCE LINE LOADED
WITH AN HB VEHICLE.

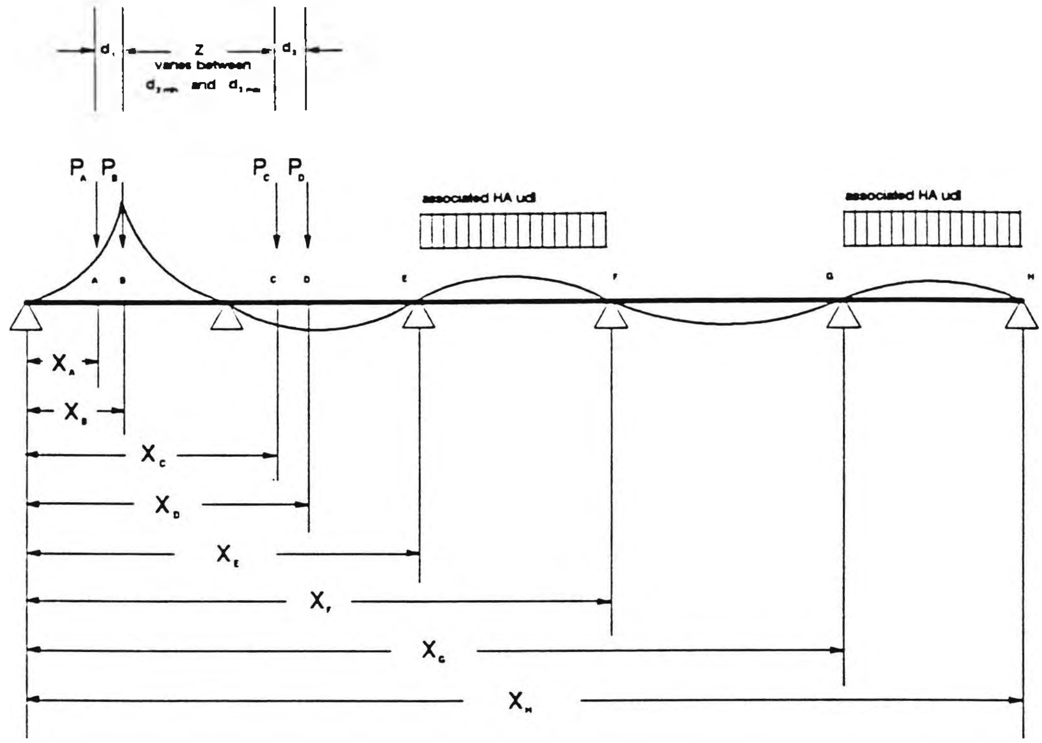


FIGURE 5.10. INFLUENCE LINE LOADED BY HB AND ASSOCIATED HA LOADING.

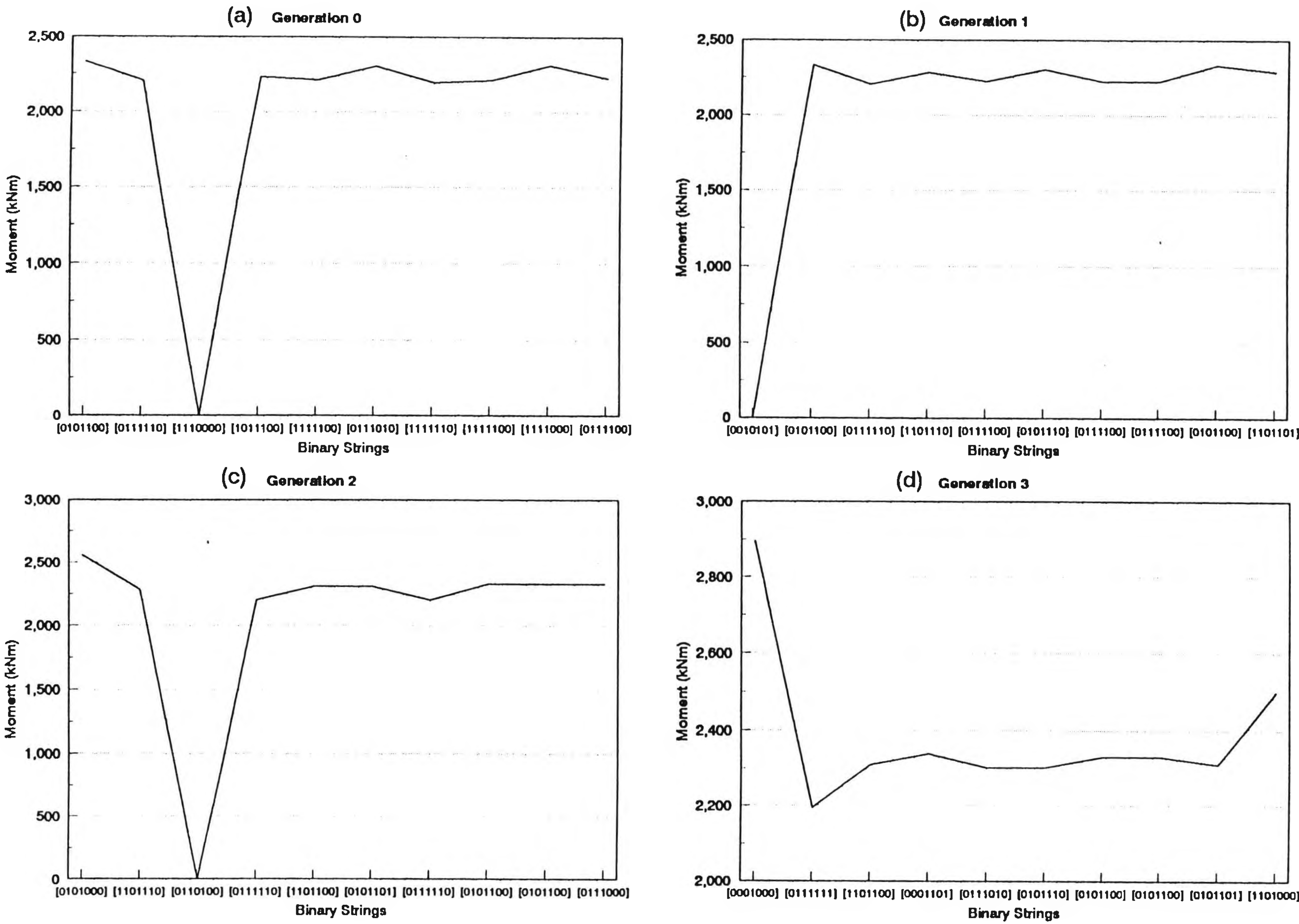


FIGURE 5.11. GENERATIONS (0-3) HISTORY FOR MOMENT AT BRIDGE CENTRE LINE.

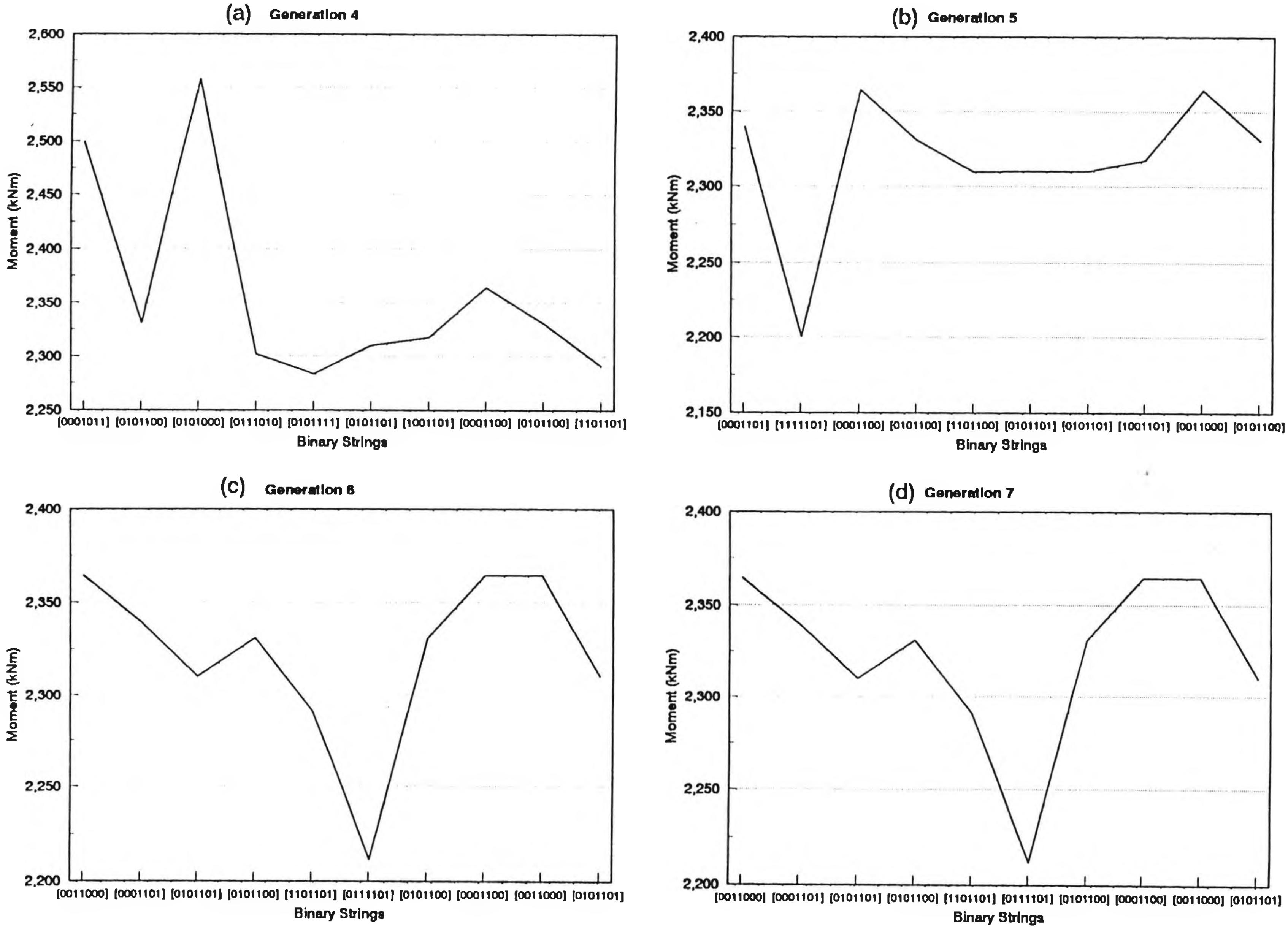


FIGURE 5.12. GENERATIONS (4-7) HISTORY FOR MOMENT AT BRIDGE CENTRE LINE.

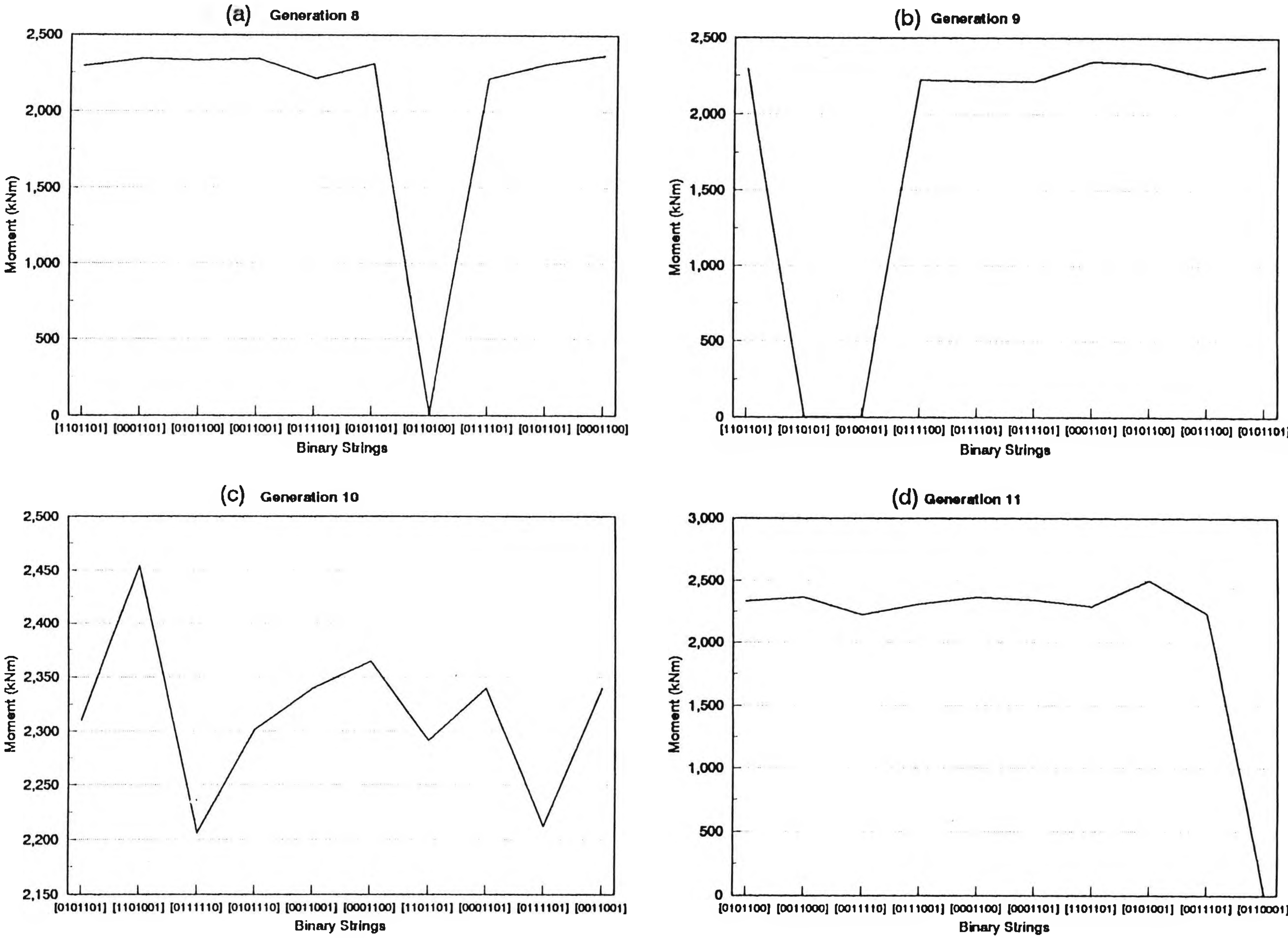


FIGURE 5.13. GENERATIONS (8-11) HISTORY FOR MOMENT AT BRIDGE CENTRE LINE.

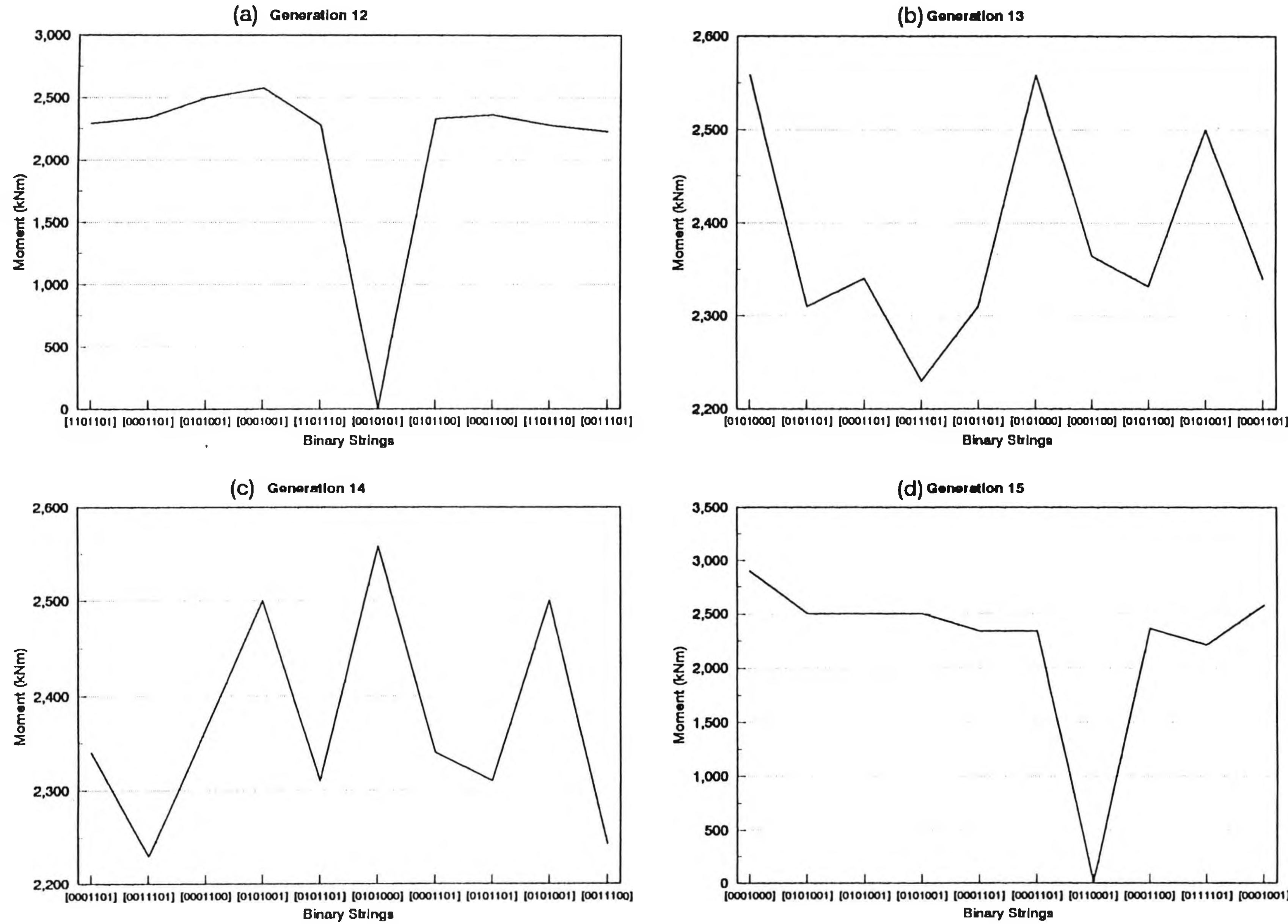


FIGURE 5.14. GENERATIONS (12-15) HISTORY FOR MOMENT AT BRIDGE CENTRE LINE.

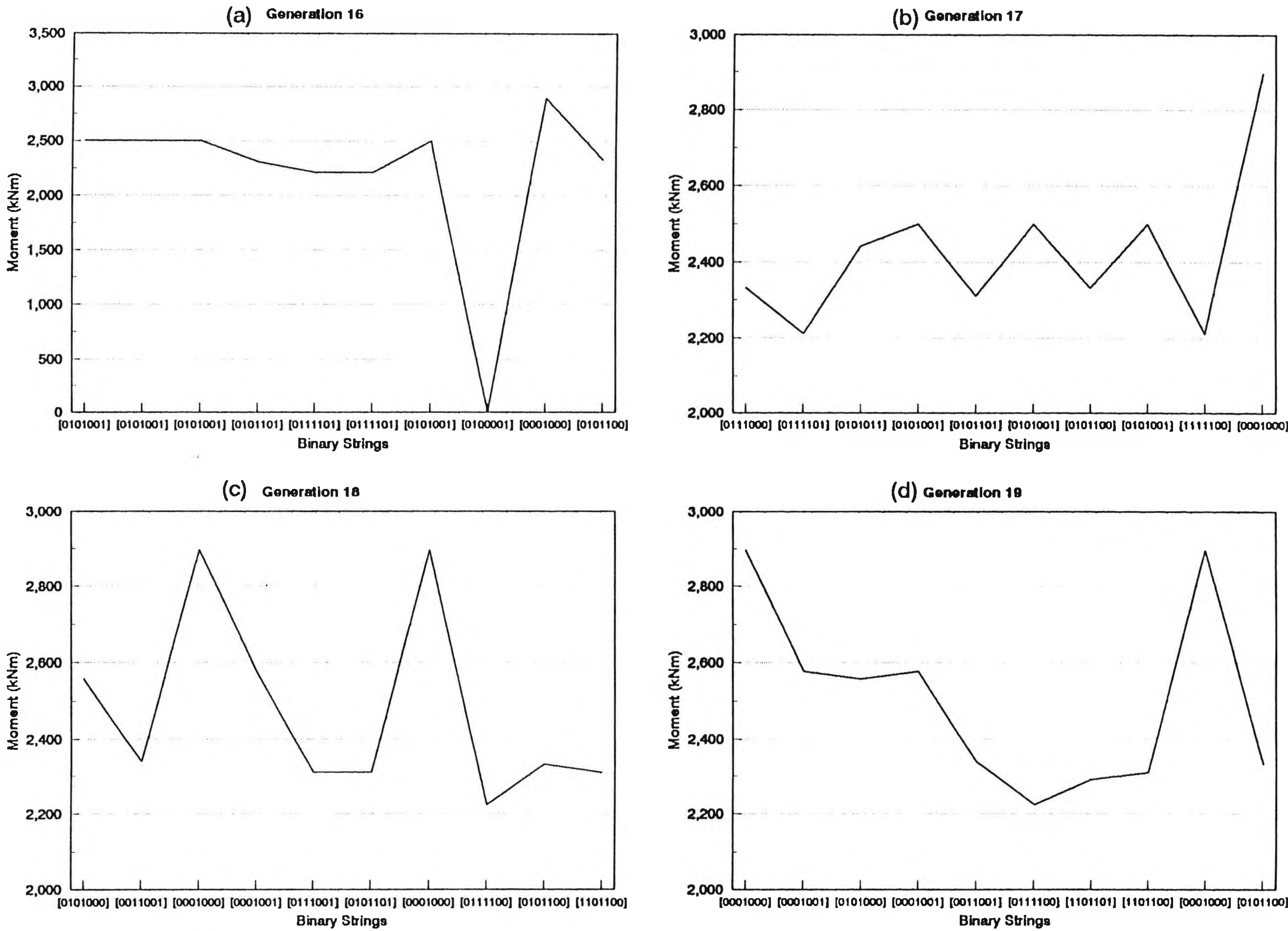


FIGURE 5.15. GENERATIONS (16-19) HISTORY FOR MOMENT AT BRIDGE CENTRE LINE.

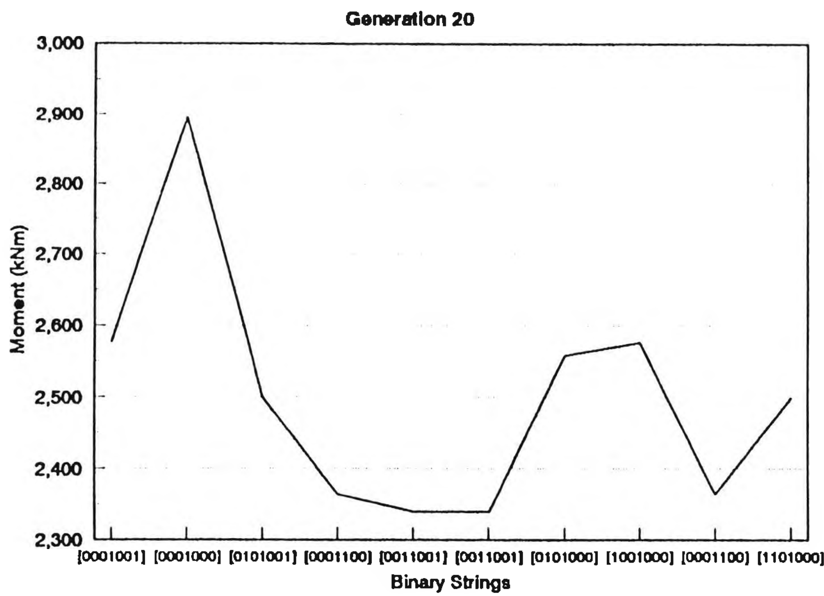


FIGURE 5.16. GENERATION (20) HISTORY FOR MOMENT AT BRIDGE CENTRE LINE.

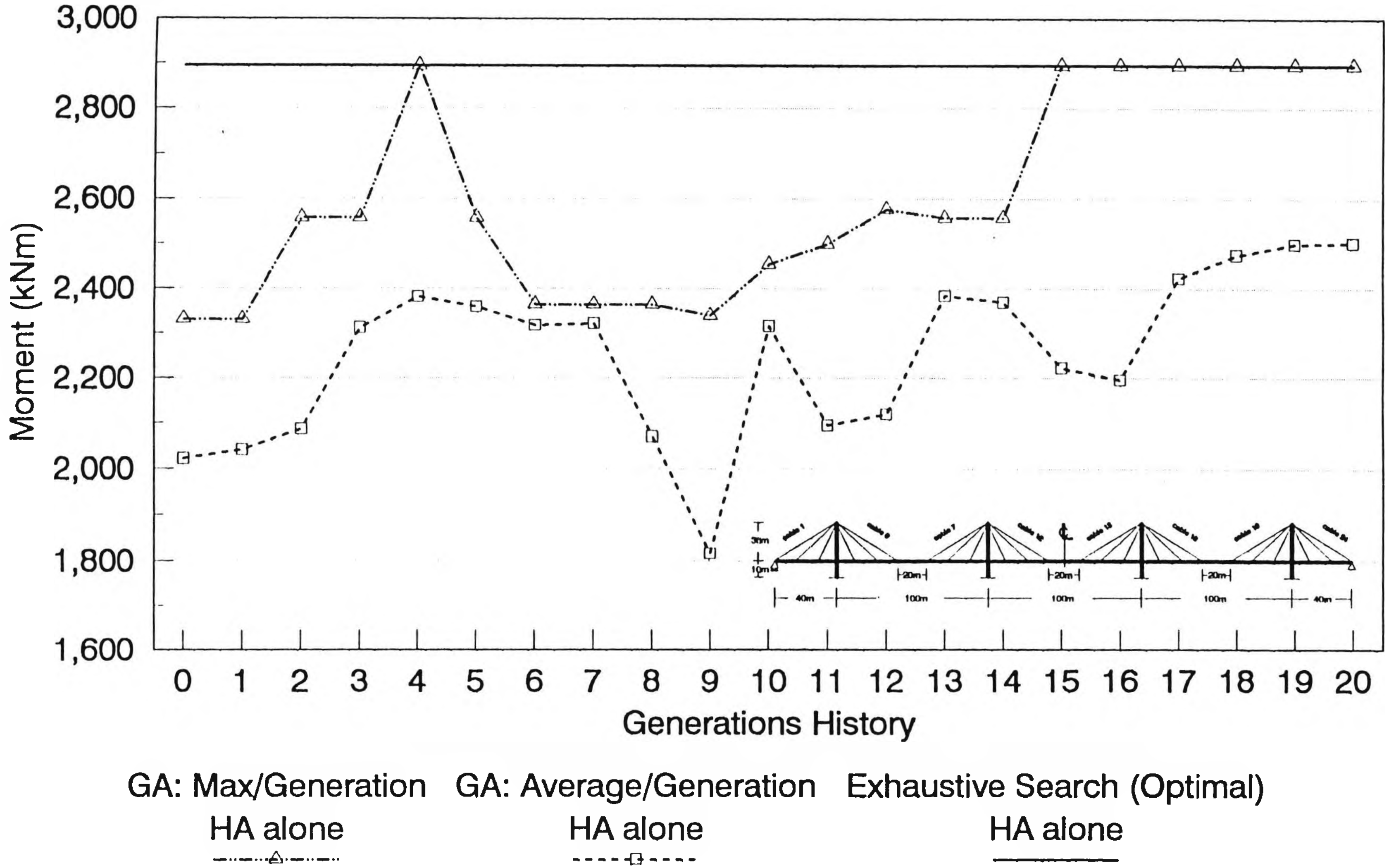


FIGURE 5.17. GENERATIONS HISTORY FOR MOMENT
AT BRIDGE CENTRE LINE.

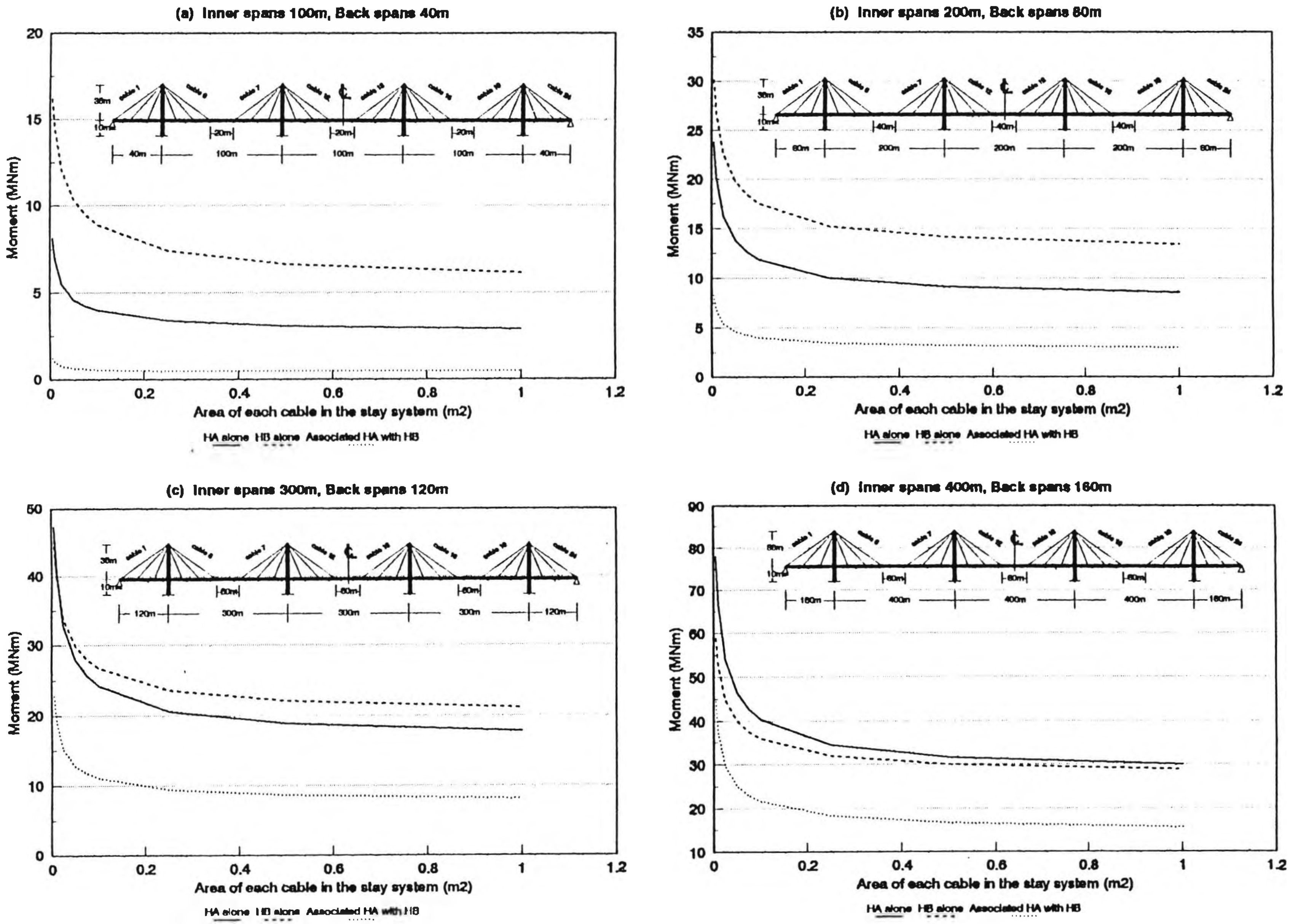


FIGURE 5.18. VARIATION IN MOMENTS AT BRIDGE CENTRE LINE DUE TO BD37/88 TRAFFIC LOADS.

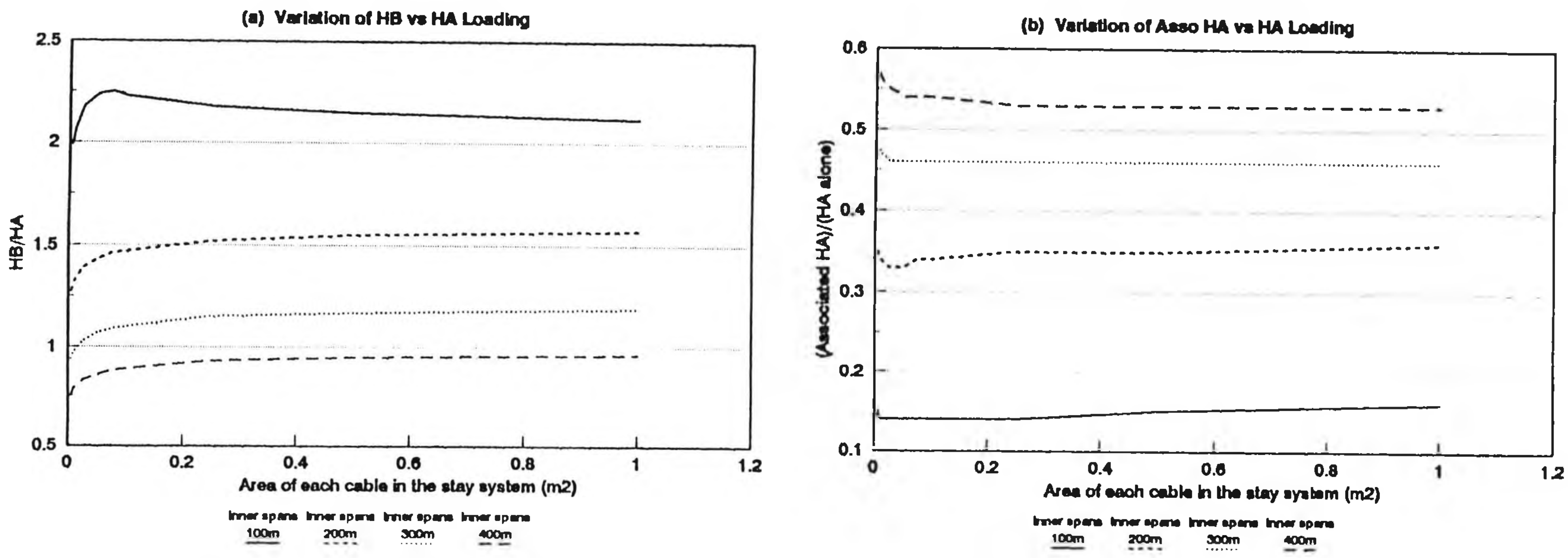


FIGURE 5.19. VARIATION IN LIVE LOAD RATIO FOR FOUR CABLE-STAYED BRIDGES WITH DIFFERENT SPANS ARRANGEMENT.

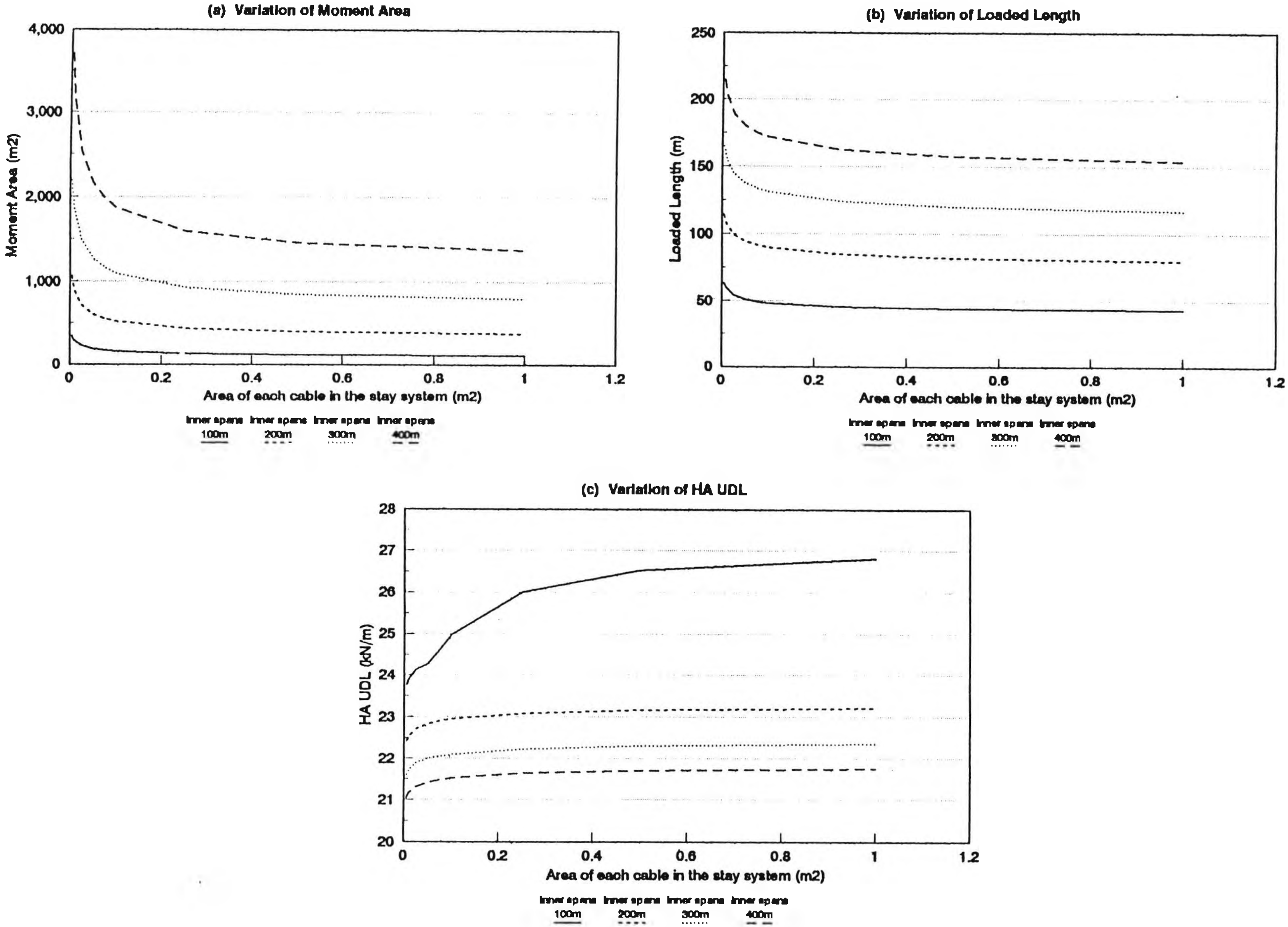


FIGURE 5.20. VARIATION OF HA LOADING PARAMETERS FOR FOUR CABLE-STAYED BRIDGES WITH DIFFERENT SPANS ARRANGEMENT.

TABLE 5.1. HA LANE FACTORS (BD 37/88, 1989).

Loaded length L (m)	First lane factor β_1	Second lane factor β_2	Third lane factor β_3	Fourth & subsequent lane factor β_n
$0 < L \leqslant 20$	α_1	α_1	0.6	$0.6\ \alpha_1$
$20 < L \leqslant 40$	α_2	α_2	0.6	$0.6\ \alpha_2$
$40 < L \leqslant 50$	1.0	1.0	0.6	0.6
$50 < L \leqslant 112$ $N < 6$	1.0	$\frac{7.1}{\sqrt{L}}$	0.6	0.6
$50 < L < 112$ $N \geqslant 6$	1.0	1.0	0.6	0.6
$N > 112$ $N < 6$	1.0	0.67	0.6	0.6
$L > 112$ $N \geqslant 6$	1.0	1.0	0.6	0.6

NOTE 1, $\alpha_1 = 0.274\ b_L$ and cannot exceed 1.0

$$\alpha_2 = 0.0137\ [\ b_L\ (40-L) + 3.65\ (L-20)\ ]$$

where b_L is the notional lane width (m)

NOTE 2, N shall be used to determine which set of HA lane factors is to be applied for loaded lengths in excess of 50m. The value of N is to be taken as the total number of notional lanes on the bridge (this shall include all the lanes for dual carriageway roads) except that for a bridge carrying one-way traffic only, the value of N shall be taken as twice the number of notional lanes on the bridge.

TABLE 5.2. ILD's OUTPUT FILE.

		ILD at Bridge Centre Line Spans (Inner,Back) = (100,40m) Area of Each Cable 1.000 m2		
Joint	X-COOR	Axial	Shear	Moment
1	0.000	0.000	0.000	0.000
2	13.333	0.015	-0.002	0.010
3	26.666	0.012	0.003	-0.015
4	40.000	0.000	0.000	0.000
5	53.333	-0.042	0.024	-0.117
6	66.667	-0.105	0.040	-0.190
7	80.000	-0.191	0.073	-0.352
8	90.000	-0.266	0.164	-0.783
9	100.000	-0.288	0.228	-1.092
10	113.333	-0.184	0.164	-0.797
11	126.666	-0.084	0.075	-0.377
12	140.000	0.000	0.000	0.000
13	153.333	0.064	-0.102	0.449
14	166.667	0.077	-0.194	0.774
15	180.000	0.064	-0.310	1.528
16	190.000	0.059	-0.500	4.962
16	190.000	0.059	0.500	4.962
17	200.000	0.065	0.310	1.528
18	213.333	0.077	0.194	0.774
19	226.666	0.068	0.102	0.449
20	240.000	0.000	0.000	0.000
21	253.333	-0.081	-0.075	-0.377
22	266.667	-0.167	-0.164	-0.796
23	280.000	-0.250	-0.228	-1.090
24	290.000	-0.196	-0.164	-0.780
25	300.000	-0.104	-0.073	-0.348
26	313.333	-0.048	-0.040	-0.188
27	326.666	-0.014	-0.024	-0.116
28	340.000	0.000	0.000	0.000
29	353.333	-0.008	-0.003	-0.015
30	366.667	0.000	0.002	0.009
31	380.000	0.000	0.000	0.000

TABLE 5.3. VARIATION OF ILD FOR MOMENT AT THE CENTRE OF THE BRIDGE WITH CABLES AREA.
(BRIDGE WITH 24 CABLES)
(INNER SPANS=100m, BACK SPANS=40m)

		Area of each cable (m2)								
Joint	X-COOR	0.005	0.010	0.025	0.050	0.075	0.100	0.250	0.500	1.000
1	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	13.333	0.064	0.083	0.097	0.093	0.084	0.076	0.045	0.025	0.010
3	26.666	0.065	0.077	0.080	0.070	0.059	0.050	0.019	-0.001	-0.015
4	40.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
5	53.333	-0.202	-0.192	-0.149	-0.121	-0.112	-0.109	-0.109	-0.113	-0.117
6	66.667	-0.584	-0.572	-0.460	-0.359	-0.308	-0.278	-0.216	-0.196	-0.190
7	80.000	-1.044	-1.081	-0.965	-0.815	-0.722	-0.659	-0.490	-0.405	-0.352
8	90.000	-1.340	-1.440	-1.391	-1.268	-1.181	-1.119	-0.942	-0.845	-0.783
9	100.000	-1.475	-1.620	-1.640	-1.556	-1.485	-1.430	-1.259	-1.159	-1.092
10	113.333	-1.258	-1.380	-1.410	-1.339	-1.268	-1.210	-1.014	-0.888	-0.797
11	126.666	-0.654	-0.692	-0.695	-0.657	-0.621	-0.591	-0.487	-0.422	-0.377
12	140.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
13	153.333	0.828	0.657	0.517	0.483	0.479	0.479	0.475	0.463	0.449
14	166.667	2.733	2.167	1.541	1.204	1.057	0.974	0.813	0.775	0.774
15	180.000	6.444	5.497	4.300	3.494	3.071	2.795	2.092	1.745	1.528
16	190.000	10.757	9.711	8.360	7.420	6.914	6.580	5.706	5.254	4.962
17	200.000	6.448	5.500	4.303	3.496	3.072	2.796	2.093	1.745	1.528
18	213.333	2.741	2.173	1.546	1.207	1.059	0.975	0.813	0.775	0.774
19	226.666	0.836	0.663	0.521	0.486	0.482	0.481	0.476	0.463	0.449
20	240.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
21	253.333	-0.661	-0.697	-0.699	-0.660	-0.623	-0.592	-0.488	-0.422	-0.377
22	266.667	-1.264	-1.384	-1.413	-1.340	-1.269	-1.210	-1.013	-0.887	-0.796
23	280.000	-1.476	-1.621	-1.639	-1.555	-1.483	-1.428	-1.257	-1.157	-1.090
24	290.000	-1.336	-1.436	-1.387	-1.263	-1.176	-1.114	-0.937	-0.841	-0.780
25	300.000	-1.035	-1.072	-0.958	-0.807	-0.715	-0.652	-0.485	-0.400	-0.348
26	313.333	-0.570	-0.561	-0.451	-0.351	-0.301	-0.272	-0.211	-0.193	-0.188
27	326.666	-0.191	-0.182	-0.141	-0.115	-0.107	-0.105	-0.107	-0.111	-0.116
28	340.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
29	353.333	0.055	0.069	0.074	0.065	0.055	0.047	0.017	-0.002	-0.015
30	366.667	0.057	0.077	0.093	0.090	0.082	0.074	0.044	0.024	0.009
31	380.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

TABLE 5.4. INFLUENCE LINE DISCRETE PARAMETERS FOR HA LOADING.

Zone	Length (m)	Cusp Test	Cusp Factor	Cusp Length	HA(UDL) (kN/m)	Marea (m2)
1	18.148	N	1.000	18.148	48.186	0.135
2	21.852	N	1.000	21.852	42.548	-0.157
3	100.000	N	1.000	100.000	22.715	-44.673
4	100.000	Y	0.435	43.500	26.826	107.926
5	100.000	N	1.000	100.000	22.715	-44.513
6	22.156	N	1.000	22.156	42.156	-0.175
7	17.844	N	1.000	17.844	48.735	0.126

TABLE 5.5. GAs GENERATIONS HISTORY.

GENERATION	MAX (kNm)	SUM (kNm)	AVERAGE (kNm)
0	2331.29	20224.01	2022.40
1	2331.29	20419.35	2041.93
2	2558.04	20867.78	2086.78
3	2558.04	23123.43	2312.34
4	2895.19	23816.49	2381.65
5	2558.04	23589.74	2358.97
6	2364.54	23180.45	2318.04
7	2364.54	23219.48	2321.95
8	2364.54	20710.66	2071.07
9	2339.72	18168.01	1816.80
10	2454.15	23159.25	2315.92
11	2499.94	20957.36	2095.74
12	2577.26	21204.09	2120.41
13	2558.04	23841.31	2384.13
14	2558.04	23694.71	2369.47
15	2895.19	22230.86	2223.09
16	2895.19	21963.63	2196.36
17	2895.19	24232.91	2423.29
18	2895.19	24750.75	2475.08
19	2895.19	24998.93	2499.89
20	2895.19	25014.39	2501.44

TABLE 5.6. GAs OUTPUT RUN.

Popsize = 10
Maxgen = 20
Pcross = 0.95
Pmut = 0.010

		STRING	FITNESS	MAX	SUM	AVERAGE
GENERATION 0:						
(Fig. 5.11a)	1)	[0101100]	2331.29			
	2)	[0111110]	2205.87			
	3)	[1110000]	3.00			
	4)	[1011100]	2229.50			
	5)	[1111100]	2211.70			
	6)	[0111010]	2302.18			
	7)	[1111110]	2195.14			
	8)	[1111100]	2211.70			
	9)	[1111000]	2310.01			
	10)	[0111100]	2223.61			
				2331.29	20224.01	2022.40
GENERATION 1:						
(Fig. 5.11b)	1)	[0010101]	2.69			
	2)	[0101100]	2331.29			
	3)	[0111110]	2205.87			
	4)	[1101110]	2283.85			
	5)	[0111100]	2223.61			
	6)	[0101110]	2302.18			
	7)	[0111100]	2223.61			
	8)	[0111100]	2223.61			
	9)	[0101100]	2331.29			
	10)	[1101101]	2291.35			
				2331.29	20419.35	2041.93
GENERATION 2:						
(Fig. 5.11c)	1)	[0101000]	2558.04			
	2)	[1101110]	2283.85			
	3)	[0110100]	0.06			
	4)	[0111110]	2205.87			
	5)	[1101100]	2310.01			
	6)	[0101101]	2310.22			
	7)	[0111110]	2205.87			
	8)	[0101100]	2331.29			
	9)	[0101100]	2331.29			
	10)	[0111000]	2331.29			
				2558.04	20867.78	2086.78

		STRING	FITNESS	MAX	SUM	AVERAGE
GENERATION 3: (Fig. 5.11d)	1)	[0101100]	2331.29			
	2)	[0101000]	2558.04			
	3)	[0101100]	2331.29			
	4)	[1101101]	2291.35			
	5)	[0101100]	2331.29			
	6)	[0111000]	2331.29			
	7)	[0111110]	2205.87			
	8)	[0111110]	2205.87			
	9)	[0101100]	2331.29			
	10)	[0111110]	2205.87			
				2558.04	23123.43	2312.34
GENERATION 4: (Fig. 5.12a)	1)	[0001000]	2895.19			
	2)	[0111111]	2195.19			
	3)	[1101100]	2310.01			
	4)	[0001101]	2339.72			
	5)	[0111010]	2302.18			
	6)	[0101110]	2302.18			
	7)	[0101100]	2331.29			
	8)	[0101100]	2331.29			
	9)	[0101101]	2310.22			
	10)	[1101000]	2499.23			
				2895.19	23816.49	2381.65
GENERATION 5: (Fig. 5.12b)	1)	[0001011]	2499.03			
	2)	[0101100]	2331.29			
	3)	[0101000]	2558.04			
	4)	[0111010]	2302.18			
	5)	[0101111]	2284.01			
	6)	[0101101]	2310.22			
	7)	[1001101]	2317.80			
	8)	[0001100]	2364.54			
	9)	[0101100]	2331.29			
	10)	[1101101]	2291.35			
				2558.04	23589.74	2358.97
GENERATION 6: (Fig. 5.12c)	1)	[0001101]	2339.72			
	2)	[1111101]	2200.81			
	3)	[0001100]	2364.54			
	4)	[0101100]	2331.29			
	5)	[1101100]	2310.01			
	6)	[0101101]	2310.22			
	7)	[0101101]	2310.22			
	8)	[1001101]	2317.80			
	9)	[0011000]	2364.54			
	10)	[0101100]	2331.29			
				2364.54	23180.45	2318.04

		STRING	FITNESS	MAX	SUM	AVERAGE
GENERATION 7: (Fig. 5.12d)	1)	[0011000]	2364.54			
	2)	[0001101]	2339.72			
	3)	[0101101]	2310.22			
	4)	[0101100]	2331.29			
	5)	[1101101]	2291.35			
	6)	[0111101]	2211.77			
	7)	[0101100]	2331.29			
	8)	[0001100]	2364.54			
	9)	[0011000]	2364.54			
	10)	[0101101]	2310.22			
				2364.54	23219.48	2321.95
GENERATION 8: (Fig. 5.13a)	1)	[1101101]	2291.35			
	2)	[0001101]	2339.72			
	3)	[0101100]	2331.29			
	4)	[0011001]	2339.72			
	5)	[0111101]	2211.77			
	6)	[0101101]	2310.22			
	7)	[0110100]	0.06			
	8)	[0111101]	2211.77			
	9)	[0101101]	2310.22			
	10)	[0001100]	2364.54			
				2364.54	20710.66	2071.07
GENERATION 9: (Fig. 5.13b)	1)	[1101101]	2291.35			
	2)	[0110101]	2.69			
	3)	[0100101]	2.82			
	4)	[0111100]	2223.61			
	5)	[0111101]	2211.77			
	6)	[0111101]	2211.77			
	7)	[0001101]	2339.72			
	8)	[0101100]	2331.29			
	9)	[0011100]	2242.78			
	10)	[0101101]	2310.22			
				2339.72	18168.01	1816.80
GENERATION 10: (Fig. 5.13c)	1)	[0101101]	2310.22			
	2)	[1101001]	2454.15			
	3)	[0111110]	2205.87			
	4)	[0101110]	2302.18			
	5)	[0011001]	2339.72			
	6)	[0001100]	2364.54			
	7)	[1101101]	2291.35			
	8)	[0001101]	2339.72			
	9)	[0111101]	2211.77			
	10)	[0011001]	2339.72			
				2454.15	23159.25	2315.92

		STRING	FITNESS	MAX	SUM	AVERAGE
GENERATION 11:						
(Fig. 5.13d)	1)	[0101100]	2331.29			
	2)	[0011000]	2364.54			
	3)	[0011110]	2223.35			
	4)	[0111001]	2310.22			
	5)	[0001100]	2364.54			
	6)	[0001101]	2339.72			
	7)	[1101101]	2291.35			
	8)	[0101001]	2499.94			
	9)	[0011101]	2229.58			
	10)	[0110001]	2.82			
				2499.94	20957.36	2095.74
GENERATION 12:						
(Fig. 5.14a)	1)	[1101101]	2291.35			
	2)	[0001101]	2339.72			
	3)	[0101001]	2499.94			
	4)	[0001001]	2577.26			
	5)	[1101110]	2283.85			
	6)	[0010101]	2.69			
	7)	[0101100]	2331.29			
	8)	[0001100]	2364.54			
	9)	[1101110]	2283.85			
	10)	[0011101]	2229.58			
				2577.26	21204.09	2120.41
GENERATION 13:						
(Fig. 5.14b)	1)	[0101000]	2558.04			
	2)	[0101101]	2310.22			
	3)	[0001101]	2339.72			
	4)	[0011101]	2229.58			
	5)	[0101101]	2310.22			
	6)	[0101000]	2558.04			
	7)	[0001100]	2364.54			
	8)	[0101100]	2331.29			
	9)	[0101001]	2499.94			
	10)	[0001101]	2339.72			
				2558.04	23841.31	2384.13
GENERATION 14:						
(Fig. 5.14c)	1)	[0001101]	2339.72			
	2)	[0011101]	2229.58			
	3)	[0001100]	2364.54			
	4)	[0101001]	2499.94			
	5)	[0101101]	2310.22			
	6)	[0101000]	2558.04			
	7)	[0001101]	2339.72			
	8)	[0101101]	2310.22			
	9)	[0101001]	2499.94			
	10)	[0011100]	2242.78			
				2558.04	23694.71	2369.47

		STRING	FITNESS	MAX	SUM	AVERAGE
GENERATION 15: (Fig. 5.14d)	1)	[0001000]	2895.19			
	2)	[0101001]	2499.94			
	3)	[0101001]	2499.94			
	4)	[0101001]	2499.94			
	5)	[0001101]	2339.72			
	6)	[0001101]	2339.72			
	7)	[0110001]	2.82			
	8)	[0001100]	2364.54			
	9)	[0111101]	2211.77			
	10)	[0001001]	2577.26			
				2895.19	22230.86	2223.09
GENERATION 16: (Fig. 5.15a)	1)	[0101001]	2499.94			
	2)	[0101001]	2499.94			
	3)	[0101001]	2499.94			
	4)	[0101101]	2310.22			
	5)	[0111101]	2211.77			
	6)	[0111101]	2211.77			
	7)	[0101001]	2499.94			
	8)	[0100001]	3.63			
	9)	[0001000]	2895.19			
	10)	[0101100]	2331.29			
				2895.19	21963.63	2196.36
GENERATION 17: (Fig. 5.15b)	1)	[0111000]	2331.29			
	2)	[0111101]	2211.77			
	3)	[0101011]	2441.63			
	4)	[0101001]	2499.94			
	5)	[0101101]	2310.22			
	6)	[0101001]	2499.94			
	7)	[0101100]	2331.29			
	8)	[0101001]	2499.94			
	9)	[1111100]	2211.70			
	10)	[0001000]	2895.19			
				2895.19	24232.91	2423.29
GENERATION 18: (Fig. 5.15c)	1)	[0101000]	2558.04			
	2)	[0011001]	2339.72			
	3)	[0001000]	2895.19			
	4)	[0001001]	2577.26			
	5)	[0111001]	2310.22			
	6)	[0101101]	2310.22			
	7)	[0001000]	2895.19			
	8)	[0111100]	2223.61			
	9)	[0101100]	2331.29			
	10)	[1101100]	2310.01			
				2895.19	24750.75	2475.08

		STRING	FITNESS	MAX	SUM	AVERAGE
GENERATION 19: (Fig. 5.15d)	1)	[0001000]	2895.19			
	2)	[0001001]	2577.26			
	3)	[0101000]	2558.04			
	4)	[0001001]	2577.26			
	5)	[0011001]	2339.72			
	6)	[0111100]	2223.61			
	7)	[1101101]	2291.35			
	8)	[1101100]	2310.01			
	9)	[0001000]	2895.19			
	10)	[0101100]	2331.29			
				2895.19	24998.93	2499.89
GENERATION 20: (Fig. 5.16)	1)	[0001001]	2577.26			
	2)	[0001000]	2895.19			
	3)	[0101001]	2499.94			
	4)	[0001100]	2364.54			
	5)	[0011001]	2339.72			
	6)	[0011001]	2339.72			
	7)	[0101000]	2558.04			
	8)	[1001000]	2576.19			
	9)	[0001100]	2364.54			
	10)	[1101000]	2499.23			
				2895.19	25014.39	2501.44

TABLE 5.7. MOMENT AT THE CENTRE OF THE BRIDGE.
(INNER SPANS=100m, BACK SPANS=40m)

Area of each cable (m2)	HA (MNm)	HB (MNm)	Asso HA (MNm)	HB/HA	Asso/HA	Asso/HB	Area HA (m2)	Area Asso HA (m2)	Loaded Length HA (m)	UDL HA (kN/m)
0.005	8.11	16.17	1.25	1.99	0.15	0.08	341.26	50.70	63.45	23.77
0.010	6.92	14.33	0.99	2.07	0.14	0.07	289.20	45.00	59.56	23.92
0.025	5.49	11.96	0.77	2.18	0.14	0.06	227.61	34.95	54.46	24.14
0.050	4.61	10.33	0.65	2.24	0.14	0.06	189.80	29.39	51.16	24.28
0.075	4.21	9.46	0.59	2.25	0.14	0.06	171.03	26.72	49.48	24.61
0.100	3.98	8.88	0.55	2.23	0.14	0.06	159.20	25.02	48.38	24.98
0.250	3.38	7.39	0.49	2.18	0.14	0.07	129.95	18.83	45.55	26.01
0.500	3.08	6.62	0.47	2.15	0.15	0.07	116.16	17.87	44.22	26.53
1.000	2.90	6.13	0.47	2.12	0.16	0.08	107.93	17.43	43.50	26.82

TABLE 5.8. MOMENT AT THE CENTRE OF THE BRIDGE.
(INNER SPANS=200m, BACK SPANS=80m)

Area of each cable (m2)	HA (MNm)	HB (MNm)	Asso HA (MNm)	HB/HA	Asso/HA	Asso/HB	Area HA (m2)	Area Asso HA (m2)	Loaded Length HA (m)	UDL HA (kN/m)
0.005	23.79	30.17	8.28	1.27	0.35	0.27	1061.27	369.28	114.05	22.42
0.010	20.23	26.79	6.83	1.32	0.34	0.25	897.10	302.69	107.32	22.55
0.025	16.29	22.64	5.42	1.39	0.33	0.24	716.85	238.61	99.64	22.72
0.050	13.86	19.87	4.64	1.43	0.33	0.23	606.58	203.32	94.53	22.84
0.075	12.65	18.45	4.27	1.46	0.34	0.23	552.32	186.56	91.78	22.91
0.100	11.90	17.55	4.05	1.47	0.34	0.23	518.40	176.25	89.95	22.96
0.250	10.04	15.26	3.49	1.52	0.35	0.23	434.91	151.05	84.94	23.09
0.500	9.14	14.14	3.22	1.55	0.35	0.23	394.63	138.90	82.16	23.17
1.000	8.59	13.45	3.05	1.57	0.36	0.23	370.00	131.40	80.29	23.22

TABLE 5.9. MOMENT AT THE CENTRE OF THE BRIDGE.
(INNER SPANS=300m, BACK SPANS=120m)

Area of each cable (m2)	HA (MNm)	HB (MNm)	Asso HA (MNm)	HB/HA	Asso/HA	Asso/HB	Area HA (m2)	Area Asso HA (m2)	Loaded Length HA (m)	UDL HA (kN/m)
0.005	47.31	44.47	22.79	0.94	0.48	0.51	2189.22	1054.72	164.72	21.61
0.010	40.37	39.62	18.98	0.98	0.47	0.48	1857.75	873.30	155.58	21.73
0.025	32.68	33.75	15.06	1.03	0.46	0.45	1492.95	688.10	144.92	21.89
0.050	27.98	29.92	12.81	1.07	0.46	0.43	1271.80	582.08	137.74	22.00
0.075	25.66	27.97	11.73	1.09	0.46	0.42	1163.17	531.53	133.88	22.06
0.100	24.20	26.73	11.06	1.10	0.46	0.41	1094.95	500.20	131.29	22.10
0.250	20.58	23.59	9.41	1.15	0.46	0.40	925.78	423.22	124.05	22.23
0.500	18.83	22.07	8.61	1.17	0.46	0.39	844.29	386.17	119.97	22.31
1.000	17.77	21.15	8.13	1.19	0.46	0.38	794.99	363.70	117.25	22.36

TABLE 5.10. MOMENT AT THE CENTRE OF THE BRIDGE.
(INNER SPANS=400m, BACK SPANS=160m)

Area of each cable (m2)	HA (MNm)	HB (MNm)	Asso HA (MNm)	HB/HA	Asso/HA	Asso/HB	Area HA (m2)	Area Asso HA (m2)	Loaded Length HA (m)	UDL HA (kN/m)
0.005	78.06	58.76	44.59	0.75	0.57	0.76	3709.00	2118.83	214.79	21.04
0.010	66.62	52.44	37.22	0.79	0.56	0.71	3147.00	1758.70	202.94	21.16
0.025	54.02	44.88	29.51	0.83	0.55	0.66	2534.80	1384.70	189.07	21.31
0.050	46.44	40.02	25.07	0.86	0.54	0.63	2168.20	1170.60	179.92	21.42
0.075	42.71	37.54	22.94	0.88	0.54	0.61	1988.50	1068.20	175.04	21.48
0.100	40.36	35.96	21.61	0.89	0.54	0.60	1875.40	1004.30	171.76	21.52
0.250	34.48	31.96	18.31	0.93	0.53	0.57	1594.00	846.30	162.50	21.64
0.500	31.66	30.00	16.72	0.95	0.53	0.56	1458.30	770.10	157.30	21.70
1.000	29.95	28.90	15.75	0.96	0.53	0.54	1376.60	724.00	153.83	21.75

P crossover = 0.0
P mutation = 0.01

TABLE 5.11. GA MUTATION SENSITIVITY TEST

	GENERATION										
STRING #	0	1	2	3	4	5	6	7	8	9	10
1	[0111100]	[0111101]	***[0101111]***	[0111101]	[0111101]	[0111101]	[0001011]	[0001011]	[1001011]	[0001011]	***[0011010]***
2	[0111101]	[0111100]	[0001011]	[0111101]	[0111100]	[0111101]	[0111101]	[0001011]	[0001011]	[1001011]	[0001011]
3	[0111101]	[0111101]	[0111101]	[0111100]	[0111101]	[0001011]	[0001011]	[0111101]	[0001011]	***[0001010]***	[1001011]
4	[0111101]	[0001011]	[0111101]	[0111101]	[0111100]	[0111101]	[0111101]	[1001011]	[0001011]	[0001011]	[0001011]
5	[0111100]	[0111101]	[0111100]	[0111101]	[0001011]	***[1111100]***	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]
6	[0001111]	[0111101]	[0111100]	[0111100]	[0111101]	[0001011]	[0001011]	[0001011]	[0001011]	[1001011]	[0001011]
7	[0111101]	[0111100]	[0111101]	[0111100]	[0111101]	[0111101]	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]
8	[0111101]	[0111100]	[0111101]	***[0000011]***	[0001011]	[0001011]	[0111101]	[0001011]	[1001011]	[0001011]	[0001011]
9	[0111100]	[0111100]	[0111101]	[0001011]	[0111101]	[0001011]	[0111101]	[0001011]	[0001011]	[0001011]	[0001011]
10	[0001011]	[0001111]	[0111100]	[0111101]	[0001011]	[0111101]	***[1001011]***	***[0110101]***	[0001011]	[0001011]	[0001011]

	GENERATION										
STRING #	10	11	12	13	14	15	16	17	18	19	20
1	[0011010]	[1001011]	[0001011]	[0011110]	[0001011]	[0001011]	***[0101011]***	[0001011]	[0011010]	[0011010]	[0001011]
2	[0001011]	[0011010]	[0001011]	[0011110]	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]	[0011010]
3	[1001011]	[0011010]	***[0011110]***	[0001111]	[0001011]	[0001011]	[0001111]	[0011010]	[0011010]	[0011010]	[0011010]
4	[0001011]	[0001011]	[0011010]	[0011010]	[0011110]	[0001011]	[0001011]	[1001011]	[0001011]	[0011010]	[0011010]
5	[0001011]	[0011010]	[0001011]	[0001011]	[0011110]	[0011010]	[0001011]	[1001011]	[0011010]	[0011010]	[0011010]
6	[0001011]	[0001011]	[0001011]	[0011110]	[0011010]	[0011110]	[0011010]	[0101011]	[0101011]	[0001011]	[0011010]
7	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]	[0011010]	[0001011]	[0001011]	[0001011]
8	[0001011]	[0001011]	[0001011]	[0001011]	[0001111]	[0001011]	[0011010]	[0011010]	[0001011]	[0011010]	[0101011]
9	[0001011]	[0001011]	[0011010]	[0001011]	[0011110]	[0001111]	[1001011]	[0001011]	[0011010]	[0011010]	[0011010]
10	[0001011]	[0001011]	[0011010]	[0001011]	[0001011]	[0001011]	[0001011]	[0001011]	[0011010]	[0101011]	[0011010]

C H A P T E R   6

CABLE-STAYED BRIDGES SUBJECT TO CABLES OUT CONDITIONS

6.1- INTRODUCTION

Nowadays, Highway authorities are imposing new obligations in the design of cable-stayed crossings. In Britain, suspended highway crossings must function normally even with one or two of their cables missing. This is to ensure that traffic restrictions are not necessary should any cables need to be replaced during the lifetime of the bridge.

It was reported in Chapter 1 that the Department of Transport, as the client, is entitled to demand whatever it wants. But many recognise that such constraints make it extremely difficult to analyse the structure in order to define the most demanding combination of loads for different parts of the bridge and still come up with an efficient, economical design. The prospect of analysing all the possible loading combinations of the structure under two cables out would increase the cost of analysis by many folds (Neil, 1991).

In this chapter the effect of stay removal in cable-stayed bridges is investigated. The first part deals with a mathematical derivation which models the effect of stay removal, up to two cables, on forces and moments locked in the load-bearing components of the structure. In the second part GAs are applied for the optimisation process of the model being derived in the first part.

The analysis discussed in this chapter is limited to the case of a two dimensional plane frame cable-stayed bridge system and is based on the assumption that the cable-stay bridge displays geometrical as well as material linearity, therefore, the principle of superposition applies.

6.2- GENERAL

A multiple-stay bridge is a highly redundant system. The static behaviour of such structure is the result of a complex interaction between several parameters. The paths of the forces are dictated to a great extent by the relative stiffnesses of the load-bearing elements. Of these elements, the cables play the most important role in the structural behaviour of the bridge.

The structural analysis of cables in a cable-stayed bridge is based on the assumption that under any loading condition the cables should always be in tension. This means that the cables should possess reasonable reserves in tension, which should be greater than the possible compressive forces which may originate at certain position of loading. This can be achieved by applying post-tensioning or prestressing forces to the stay system. This assumption permits cables to be considered as rigid bars, stressed by tension and providing the geometrical stability of the cable system under arbitrary loading.

The philosophy of designing bridges is based on the assumption that the worst load combination should be sought. Since cables play the most important role in the structural behaviour of a cable stayed bridge, stay removal will have an adverse effect on forces and moments for which the structure is designed. This adverse effect can be understood if a relation between forces, moments and the tension forces in the cables can be made.

During erection the effect of stay removal is not considered. At this stage, erection loads are predefined and can be monitored. The effect of stay removal starts taking place when the bridge is in service. Stay removal can be categorized into two groups: (a) Controlled stay removal; and; (b) Accidental stay removal.

Controlled stay removal applies to the situation of replacing any cables during the life time of the bridge. In this case the stay removal condition is usually limited to 'one cable out' only and no restriction on the traffic are to be made except in a few situations. For the Second Severn Crossing (U.K.), the traffic in the eight notional lanes is reduced to six notional lanes with the prevention of the HB vehicle from passing through. For the River Dee Crossing (U.K.), The traffic in the four notional lanes is reduced to two national lanes with no restriction on the HB vehicle being made. Highway authorities usually decide on the type of restriction to be imposed.

Accidental stay removal applies in the case of an accidental event resulting in the sudden removal of one or two cables of the stay system at the same time. In this case the bridge should still be in service with some restriction being made on the volume of traffic loads. In the case of Second Severn Crossing, the live load is reduced to 10%. The dynamic effect of the sudden removal of one or two cables is a major problem which will dramatically affect the force(s) in the removed cable(s). For the Second Severn Crossing, the dynamic effect have been transformed into a static effect by applying a load factor of 2.0 to the forces in the cables which have been accidently snapped.

The analysis of a cable-stayed bridge under the cable out conditions may be divided into several parts. In the first part, bending moments, axial and shear forces due to applied loads with all cables in are first determined. In the second part, the post tensioning forces in the cables required to reduce to specified values the stresses and strains determined in stage one are calculated. In the third part, the total forces in the cables due to applied loads and post-tensioning forces are summed up. Finally, the forces in cables calculated from the third part are then used in the maximization process of cable out conditions.

The adverse effect of stay removal on forces and moments depends mainly on the tension force which develops in the cable prior to its removal. The bigger the force, the higher the effect.

Appendix D presents a list of the different type of loads applicable to cable-stayed bridges. On application, every load has a contribution toward the overall force or tension which develops in each cable of the stay system. The assumption of geometrical and material linearity has enabled us to study the effect of forces in the cables based on the principle of generation of influence lines.

For cable-stayed bridges, two types of influence lines can be generated. The first type models the linear behaviour of the bridges under a unit load travelling across the deck. This type has been discussed in chapter 5. The second type is used for the determination of post-tensioning forces in order to achieve a partial balancing due to applied loads. Troitsky (1972) presented a method for the calculation of post-tensioning forces, due to dead load only, based on the reduction of bending moments. In this chapter removing all cables, one at a time, and applying a unit load in the direction of the cable will be called 'Influence lines for cable out effect'. These influence lines give the effect of removing one cable and replacing it with a unit load on forces and moments in the structure.

6.3- INFLUENCE LINES FOR CABLE OUT EFFECT

The generation of influence lines for cable out effect is a straight forward process. Consider the cable-stayed bridge given in Fig. 6.2a where the stay system (for simplification reasons) consists of twelve cables only.

The process starts by removing cable 1 from the original structure given in Fig. 6.2a and the substructure shown in Fig. 6.2 c is obtained. In this substructure, the bending moments, axial and shear forces at all member ends due to a unit load applied along cable 1 are determined. Figs. 6.2d to 6.2p represent the substructures obtained for cable j (j=2,...,12) due to the removal of cable j and the application of a unit load, in the direction of that cable, instead.

Once the structural analysis for all cases is carried out this results in the influence lines for cable out effect of forces and moments at member ends to be generated. Figs. 6.3 (a-c) shows samples of influence lines for cable out effect for moment and cable forces for the bridge structure given in Appendix A.

6.4- ONE CABLE OUT

In this section the effect of one cable out is presented for bending moment only as the same procedures apply for axial, shear, deflection, etc.

The maximization effect of bending moment under one cable out conditions can be summarized in the following steps. First, forces in cables due to applied loads with all cables in are first determined, Fig. 6.4. Second, influence line for cable out effect for moment (Fig. 6.3a), for the section being investigated, is then calculated. Then the cable which results in the worst effect, when removed, on the bending moment is selected. This can be achieved by multiplying the force due to applied load, calculated in stage one, by the moment at that section due to the removal of that cable and applying unit force instead. This process is repeated for all cables in the stay system. The cable which gives the worst effect is then selected for removal. Finally, the bending moment due to the removal of that cable is added up to the initially calculated moments with no cable out.

6.5- TWO CABLES OUT

In the previous sections cable out effect was discussed in the context of one cable out and the generation of influence lines for cable out effect. In this section the effect of removing two cables, at the same time, on bending moment at an arbitrary point is investigated.

The effect of removing two cables may be divided into two parts. In the first part, the relation between the tension in any two cables selected for removal on forces and moments need to be derived. In the second part, the selection process of the two cables which give the worst effect, when removed, need to be optimised.

The relation between the tension forces, which develop in the two cables prior to their removal, and bending moment at an arbitrary point 'a' on the bridge may be represented as a geometrical series. This is to cater for the fact that each of the two cables has an adverse effect on each other resulting in the original tension force in that cable to be increased due to the removal of the other cable and vice versa. This would result in a new theoretical tension force in the cable which will have an adverse effect on the tension force in the other cable when removed resulting in the development of a new theoretical tension force in this cable. the repetition of this process can be presented as a geometrical series.

In brief, the effect of removing two cables on bending moment can be summarised in the following steps. The effect of two cables out on the cables is first determined. This would results in the theoretical force in any two cables to be evaluated. Then, the effect of these theoretical forces on bending moment is then calculated. Finally, the optimisation process for the selection process of the worst two cables selected for removal would then follow.

In this section the geometrical series which models the conditions of two cables out is derived. Then, GAs are applied for optimizing the selection process whereby the worst two cables, selected for removal, will be concluded.

6.5.1- TWO CABLES OUT: MODEL DERIVATION

At the initial stage no cable is taken out. Given the initial bending moment at an arbitrary point 'a', denoted by $M_{a(o)}$, the initial forces in cables i and j, selected for removal, given by $F_{i(o)}$ and $F_{j(o)}$, respectively. The following notation will be used:

U_i: Unit load, say 1000 kN, applied at cable i after it is being removed;

U_j: Unit load, say 1000 kN, applied at cable j after it is being removed;

M_a^i: moment at point a due to cable i removed and unit load, U_i, applied;

M_a^j: moment at point a due to cable j removed and unit load, U_j, applied;

μ_a^i: (moment at point a due to cable i removed and unit load applied) ÷ unit load, defined as $\mu_a^i=M_a^i/U_i$;

μ_a^j: (moment at point a due to cable j removed and unit load applied) ÷ unit load, defined as $\mu_a^j=M_a^j/U_j$;

$F_{i(o)}$: force in cable i at the initial stage (i.e. all cables are in);

$F_{j(o)}$: force in cable j at the initial stage (i.e. all cables are in);

F_i^j: force in cable i due to cable j ($j \neq i$) replaced by a unit load U_j;

F_j^i: force in cable j due to cable i ($j \neq i$) replaced by a unit load U_i;

Φ_j^i: (force in cable j due to cable i ($j \neq i$) replaced by a unit load) ÷ unit load, defined as $\Phi_j^i = F_j^i / U_i$;

Φ_i^j: (force in cable i due to cable j ($j \neq i$) replaced by a unit load) ÷ unit load, defined as $\Phi_i^j = F_i^j / U_j$;

The process of taking two cables out, in a successive order, is an iterative process. At every iteration the relation between the selected cables forces and bending moment are considered.

Let k be the number of iterations to be carried out where k takes the values 0,1,2,3,..., and k=0 represents the initial state (i.e. no cable out).

k=1: Cable i is out:

Force in cable i at the initial stage is $F_{i(o)}$

Force in cable j at the initial stage is $F_{j(o)}$

Moment at point a due to cable i out:

$$M_{a(1)}^i = F_{i(o)} \mu_a^i$$

Force in cable j due to cable i out:

$$F_{j(1)}^i = F_{i(o)} \Phi_j^i$$

Total force in cable j:

$$F_{j(1)} = F_{j(0)} + F_{j(1)}{}^{i} = F_{j(0)} + F_{i(0)}\Phi_j{}^{i}$$

Total moment at point a:

$$M_{a(1)} = M_{a(0)} + M_{a(1)}{}^{i} = M_{a(0)} + F_{i(0)}\mu_a{}^{i}$$

k=2: Cable j is out:

Force in cable i from iteration k=1 $F_{i(0)}$

Force in cable j from iteration k=1 $F_{j(1)}$

Moment at point a due to cable j out:

$$M_{a(2)}{}^{j} = F_{j(1)}\mu_a{}^{j}$$

Force in cable i due to cable j out:

$$F_{i(2)}{}^{j} = F_{j(1)}\Phi_i{}^{j}$$

Total force in cable i:

$$F_{i(2)}=F_{i(0)}+F_{i(2)}{}^{j}=F_{i(0)}+F_{j(1)}\Phi_i{}^{j}=F_{i(0)}+(F_{j(0)}+F_{i(0)}\Phi_j{}^{i})\Phi_i{}^{j}$$

Total moment at point a:

$$M_{a(2)}=M_{a(1)}+M_{a(2)}{}^{j}=M_{a(0)}+F_{i(0)}\mu_a{}^{i}+(F_{j(0)}+F_{i(0)}\Phi_j{}^{i})\mu_a{}^{j}$$

At iteration step k+1 we consider the effect on the following quantities by removing the present forces in cables i and j noted by $F_{i(k)}$ and $F_{j(k)}$ respectively and applying a unit load instead where k takes the values 0,1,2,3,..., and for k=0 the initial values are used:

Moment at point a due to cable i out:

$$M_{a(k)} = F_{i(k)}\mu_a{}^i \tag{6.1}$$

Theoretical force in cable j due to cable i out:

$$F_{j(k+1)}{}^i = F_{i(k)}\Phi_j{}^i \tag{6.2}$$

Total force in cable j:

$$F_{j(k+1)} = F_{j(k)} + F_{j(k+1)}{}^i \tag{6.3}$$

Total moment at point a:

$$M_{a(k+1)} = M_{a(o)} + M_{a(k)}{}^i + M_{a(k)}{}^j \tag{6.4}$$

Introducing Eq. (6.2) into Eq. (6.3), it can be seen that all these quantities at iteration step k+1 can be calculated for once the quantities $F_{i(k)}$ and $F_{j(k)}$ are known. Since $F_{i(k)}$ and $F_{j(k)}$ describe the total force in cables i and j respectively at iteration step k, they can be calculated from Eq. (6.3) by making use of Eq. (6.2) as follows:

$$\begin{aligned} F_{i(k)} &= F_{i(k-1)} + F_{j(k-1)}\Phi_i{}^j \\ &= F_{i(k-1)} + F_{j(k-2)}\Phi_i{}^j + F_{i(k-2)}\Phi_j{}^i\Phi_i{}^j \end{aligned} \tag{6.5a}$$

$$\begin{aligned} F_{j(k)} &= F_{j(k-1)} + F_{i(k-1)}\Phi_j{}^i \\ &= F_{j(k-1)} + F_{i(k-2)}\Phi_j{}^i + F_{j(k-2)}\Phi_i{}^j\Phi_j{}^i \end{aligned} \tag{6.5b}$$

Applying the recurrence formulae given in Eqs. (6.5a) and (6.5b) over and over again until the forces at the initial stage, $F_{i(o)}$ and $F_{j(o)}$, are reached yield:

$$F_{i(k)} = F_{i(0)} \sum_{m=0}^{[\frac{k}{2}]} (\Phi_i^j \Phi_j^i)^m + F_{j(0)} \Phi_i^j \sum_{m=0}^{[\frac{k-1}{2}]} (\Phi_i^j \Phi_j^i)^m \qquad (6.6a)$$

$$F_{j(k)} = F_{j(0)} \sum_{m=0}^{[\frac{k}{2}]} (\Phi_i^j \Phi_j^i)^m + F_{i(0)} \Phi_j^i \sum_{m=0}^{[\frac{k-1}{2}]} (\Phi_i^j \Phi_j^i)^m \qquad (6.6b)$$

where [x] denotes the largest integer number i with $i \leq x$ (for example, [1.2]=1 and [-0.3] = -1).

considering the case for k in the limit of infinity, Eqs. (6.6a) and (6.6b) yield

$$F_{i(\infty)} = (F_{i(0)} + F_{j(0)} \Phi_i^j) \sum_{m=0}^{\infty} (\Phi_i^j \Phi_j^i)^m \qquad (6.7a)$$

$$F_{j(\infty)} = (F_{j(0)} + F_{i(0)} \Phi_j^i) \sum_{m=0}^{\infty} (\Phi_i^j \Phi_j^i)^m \qquad (6.7b)$$

In practice $|\Phi_i^j|$ and $|\Phi_j^i|$ are both < 1 and, as a consequence, $|\Phi_i^j \Phi_j^i| < 1$ is always ensured. Hence, the geometric series on the right-hand side of eq. (6.7) converges. The total force in cable i due to force in cable j being removed and a unit load applied instead of, which is given by eq. (6.7), now, finally, takes the simplified form:

$$F_{i(\infty)} = \frac{F_{i(0)} + F_{j(0)}\Phi_i^j}{1 - \Phi_i^j \Phi_j^i} \qquad (6.8a)$$

$$F_{j(\infty)} = \frac{F_{j(0)} + F_{i(0)}\Phi_j^i}{1 - \Phi_i^j \Phi_j^i} \qquad (6.8b)$$

This result in eqs. (6.8a) and (6.8b) can now be applied to eqs. (6.1), (6.2) and (6.4) yielding for the total moment in the limit of infinity:

$$M_{a(\infty)} = M_{a(0)} + \frac{(F_{i(0)} + F_{j(0)}\Phi_i^j)}{1 - \Phi_i^j \Phi_j^i}\mu_a^i + \frac{(F_{j(0)} + F_{i(0)}\Phi_j^i)}{1 - \Phi_i^j \Phi_j^i}\mu_a^j \qquad (6.9)$$

Eq. 6.9 models the relation, for two cables out conditions, between cable forces (selected for removal) and the bending moment at an arbitrary point in the bridge. In this relation the two cables, selected for removal, are referred to as cable i and cable j. The determination of cable i and cable j and their corresponding adverse effect is an optimisation task. Several parameters contribute to the relation defined in eq. 6.9 and they can be obtained through three different stages.

The first stage is called the initial stage where no cable is taken out. At this stage the worst load scenario which results in the worst moment and cables forces, denoted by $M_{a(o)}$, $F_{i(o)}$ and $F_{j(o)}$, for all cables in the bridge is investigated. Appendix D presents the method used for the calculation of these moment and forces in accordance with BD 37/88.

In the second stage one cable out condition for moment and cables forces is then carried out. This is achieved by removing all the cables, one at a time, and applying unit load instead. At this stage the parameters which represent the influence lines for one cable out effect, known as μ_a^i, μ_a^j, Φ_i^j and Φ_j^i, are generated.

In the final stage the parameters $M_{a(o)}$, $F_{i(o)}$, $F_{j(o)}$, μ_a^i, μ_a^j, Φ_i^j and Φ_j^i are incorporated into the framework of eq. 6.9 resulting in the effect of removing cables i and j, at the same time, on bending moment to be evaluated.

The procedure for the calculation of cables out conditions presented for bending moment in eq. 6.9 can be applied to any other effect (i.e. axial, shear, deflection, ... etc). This is achieved by replacing the parameters related to bending moment with the parameters which correspond to the other effect.

6.5.2- SUMMARY OF CALCULATION OF WORST CABLES OUT CONDITIONS

(1) Find the worst load case with no cable out;

(2) Calculate the associate cable forces $F_{i(o)}$ in cable i, where i=1,..,n (n=number of cables in the stay system);

(3) For one cable out maximise the product $F_{i(o)}$ M_a^i/U_i, where M_a^i is the load effect on the point under consideration due to cable i being removed and unit load being applied instead.

(4) For two cables out the equation for M_a the moment on the point under consideration for two cables out is given in eq. 6.9 and should therefore be maximised.

Care must be taken to ensure the force in a cable is not maximised by taking itself out.

6.5.3- TWO CABLES OUT: SELECTION

A multiple-stay bridge is a highly redundant system. The static behaviour of such structure is the result of a complex interaction between several parameters. The paths of the forces are dictated to a great extent by the relative stiffnesses of the load-bearing elements. The shape of influence lines of forces and moments are not necessary known in advance. The final refined models are usually concluded after many cycles. For the Second Severn Crossing (UK), final models have been reached after carrying out six different revisions. In every revision, 2D and 3D models were set up modelling the linear, non-linear and dynamic behaviour of the bridge. All this has made the process of analysing cable-stayed bridges, even without taking cable(s) out effects into consideration, to be a very complicated process indeed.

The global analysis of any cable stayed bridge is usually carried out by dividing it into its main elements (deck beam, pylons, piers and cables). Each element is considered as a beam or truss element and idealised as such in a frame model for a global computer analysis of the complete structure. The structure is usually idealised as 2D and/or 3D models. The models should be sufficiently detailed to obtain results at any point on the bridge. As an example, the 2D plane frame computer model of the Second Severn Crossing (UK) consists of 3000 structural members, 120 of which are cables. For every member the following scenarios should be investigated:

1- Max +ve bending and its associated shear and axial

2- Max +ve shear and its associated axial and bending

3- Max +ve axial and its associated bending and shear

4- Max -ve bending and its associated shear and axial

5- Max -ve shear and its associated axial and bending

6- Max -ve axial and its associated bending and shear.

The calculation of any max value (bending, shear or axial) depends on the type of loads to consider. In cable-stayed bridges loads can be defined as standard and non-standard loadings.

Standard loadings usually belong to one family known as "Code of Practice". In this research, the British Highway code of practice BS5400 in its latest loading directive BD 37/88 has been adopted. Non-standard loadings covers aspects of loadings which have not been discussed in the highway loading standards whereby stay removal is an example of this type of loads.

The general philosophy governing the application of loads on highway bridges is that the worst effects should be sought. In practice, this implies that the arrangement of the loads on the bridge is dependent upon the load effect being considered, and the critical section being studied. In addition, any code of practice usually requires that when the most severe effect on a structural element can be diminished by the presence of a load on certain portion of the structure, the load is considered to act with its least possible magnitude.

Appendix D presents the main guide lines of the method being used for the calculation of any max value in relation to the code of practice being adopted (i.e. BD 37/88).

Excluding the cable(s) out effect(s), the optimum solution of a cable-stayed bridge is usually achieved after many iterations. In every iteration the optimum solution is one of many solutions. Appendix E presents formulae which relate possible solutions with respect to combinations and permutations. Every permutation / combination represents a solution in the search space which models certain aspects of the bridge. The optimum solution should represent the best solution which models the behaviour of the bridge with respect to minimum-cost and high performance. This results in the search area to be explored to be very large indeed. Moreover, the introduction of cables out conditions into the analysis/design of cable-stayed bridges has even made the search space to increase by manyfold for reasons that are explained later.

The design/check for cable out condition has resulted in the generation of another type of influence lines referred to as "Influence Lines for Cable Out Effect". It is generally difficult to express the shape of influence lines in a closed mathematical form. The influence lines may be discontinuous at sections under consideration and may have different forms at various portions of the bridge. In such cases the ordinates of the influence lines are discrete and therefore stochastic search methods may be suitable for the location of the optimum solution.

For this particular problem, searching for the optimum solution, Combinatorial methods could be an option. In engineering consultancies, it is a common practice to resort to combinatorial methods. Chapter 5 reported various problems which face the optimisation process using exhaustive methods. Thus, for cable-stayed bridges where the final design/analysis is usually achieved after several cycles, exhaustive combinatorial methods are regarded to be very expensive solutions indeed.

The design/check of cable-stayed bridges for cable out conditions has made the situation even worse. The process of selecting the worst two cables for removal is an optimisation task which requires plenty of computational time. The size of search space to explore is related to the number of permutations and combinations needed for selecting the worst two cables for removal. The reader may refer to Appendix E to have a feel for the size of search space (i.e. the number of possible solutions).

For example, in a stay system consisting of 120 cables for the Second Severn Crossing UK, the search space to be explored for the one maximum scenario, out of the six scenarios mentioned previously, consists of 7140 different permutations (Appendix E) needed for every load case from the list of load cases mentioned in Appendix D. In each permutation (i.e. selection of two cables for removal), the corresponding tension which develops in the other cables is calculated and then maximised in accordance to the framework of eq. 6.9. This I hope gives a feel to the reader of the size of search space to be explored for a 2D plane frame consisting of 3000 members, not to mention the 3D effect.

So far, It has been concluded that optimum solution of cable-stayed bridges, subject to standard and non-standard loading with a special reference being made to cable out conditions, is a very complex process indeed. Thus, it can be concluded that the search space to be enumerated is huge and conventional optimisation methods may not appropriate for this task. This is due to the complexity of modelling the ILD as a closed mathematical formulation. On the other hand, exhaustive combinatorial methods require a great deal of computational time for the fact that every single possible solution should be tried in order to locate the optimum solution.

For these reasons GAs would fill in the gap for the optimisation process of cable-stayed bridges under cables out conditions. GAs offer a direct search approach whereby only the promising regions of the search space are explored. This is so, because Genetic Algorithms are based on the principles of natural selection and survival of the fittest.

In the current Chapter, the framework presented in Chapter 4 will be effectively used for the optimisation process of cable-stayed bridges subject to the non-standard loadings with a reference being made to stay removal loads only.

6.5.4- SELECTION OF CABLES OUT VIA GENETIC ALGORITHMS

The strategy outlined in Chapters 4 and 5 for the application of GAs to optimisation problems is also adopted herein. The 5 GA components required are:

(1) A string representation of the solution
(2) A way of seeding the initial population
(3) A fitness function
(4) Genetic operators
(5) Values of GAs parameters.

Before going any further into GA components, it is useful to highlight the desirable features the computational environment should have.

To permit some structural generalization in the study, the computational environment is based on the usual interpretation of the 2D plane frame computer models type structure having three degree of freedom (two translations and one rotation) at each node.

6.5.4.1- STRING REPRESENTATION

It was stressed earlier that GAs can handle continuous, Integer and discrete problem efficiently. Since the problem of cables out is integer, the number of cables to be removed remain integer but coded in finite-length strings.

Given a structure having a total number of cables equals to Nc, each of these cables can be represented in the form of a sub-string. One simple way of doing this is the use of the classical binary coding. In this case the length Ls of any binary sub-string has to be determined. Ls may be estimated from the following equation (Goldberg, 1989):

$$2^{Ls} \geq [(n_{max} - n_{min})/\varepsilon + 1] \qquad (6.10)$$

where n_{min} = minimum number of cables

n_{max} = maximum number of cables

ε = precision; $\varepsilon = 1$ for integer variables (Lin and Hajela, 1992).

6.5.4.2- INITIAL POPULATION

For a population of size Npop, the initial set of strings may be generated at random. For binary coding this may be reached by tossing a fair coin as described in section 5.6.2.

6.5.4.3- FITNESS FUNCTION

The mathematical formulation of Eq. 6.9 already developed in section 6.5.1 presents the framework for the fitness function required by GA model to guide the process of selection. Eq. 6.9 models the relation, for two cables out conditions, between cable forces, selected for removal, and bending moment at any arbitrary point in the bridge structure.

The aim of this optimisation task is to find the two cables, referred to as cable i and cable j, when removed will have an adverse effect on bending moments so this effect can be made maximum or minimum. The parameters known as $M_{a(0)}$, $F_{i(0)}$, $F_{j(0)}$, μ_a^i, μ_a^j, Φ_i^j and Φ_j^i are the main variables, when gathered into the framework of eq. 6.9 result in the effect (fitness) of removing cables i and j simultaneously on bending moment to be evaluated.

In many cases optimisation studies are naturally formulated as minimization problems. The GA depends upon maximising a fitness function. As a result, a way must be found to transform a minimization problem to a non-negative maximization problem.

In normal optimisation practice, minimization can be transformed to maximization or vice versa by multiplying the objective function by -1.

Eq. 6.9 may produce positive or negative values and we are interested in maximizing positive values and minimizing negative values. Minimizing negative values is equivalent to maximizing positive values and consequently the same GA program can perform this.

To distinguish between maximization & minimization an integer variable has been introduced. It is equal to 1 with maximization & to 2 in the opposite case. The user has to specify at the beginning of the run the value of this variable.

To deal at the same time with positive and negative values, it was found that the best way is to penalize the fitness function when it meets a negative in case of maximization and vice versa. Indeed if a maximization is considered, when a negative value is encountered, it is automatically reduced to zero. And as a consequence, the maximum is sought only among the positive values. With minimization, in the event of meeting a positive value its corresponding fitness is zero and the negative fitnesses are multiplied by -1 to transform the minimization to maximization. Consequently, the quality or fitness of a solution is given by the following equation:

$$Fit = | M_{a(\infty)} | \qquad (6.11)$$

Fit must be a positive expression as required by the computation of selection probability.

6.5.4.4- GENETIC OPERATORS & PARAMETERS VALUES

Here again, the genetic operators used for solving problem GA1 and GA2 presented in Chapters 4 and 5 respectively are also used in this Chapter. These operators can be summarized as:

(1) Selection
(2) Crossover
(3) Mutation

The reader may refer to Chapters 4 and 5 for more explanations about these operators and their parameters values.

6.5.4.5- DESIGN & ANALYSIS INTERFACE

The provision of design data in the form of an accessible data base which contains the parametric data of the structure is an essential part of the optimisation process. These data are written into text files which are then accessed when required. Data bases, directories, and Subdirectories which correspond to the parameters $M_{a(o)}$, $F_{1(o)}$, $F_{3(o)}$, μ_a^{i}, μ_a^{j}, Φ_i^{j} and Φ_j^{i} being generated during the two stages discussed earlier are explained to a greater depth in Appendix F.

6.6- SOFTWARE

Two computer programs called GAcable1 and GAcalbe2 were developed and programmed during this project. GAcable1 deals with one cable out while GAcable2 addresses the problem of two cables out.

Several procedures have been written. For more details the reader may refer to Appendix F. Some procedures were developed to prevent 2 events:

1- Case of having the same cables for the member considered when dealing with the problem of 2 cables out. The member considered could be a deck, a pylon or a cable section. In these procedures simple tests were written to prevent the situation of removing the same cable twice. In effect once this event occurs one of the cables is kept, the other one is selected randomly.

2- Case of having identical Nb of both cables and member when the member is a cable section. A boolean (logic) variable is used which takes true if the cable number is the same as the cable section. When it is true it means that this cable has to be discarded by assigning zero to its fitness value. This is to ensure that the effect of that cable is not maximised by taking itself out.

With these two tests the cables have to be different whatever the member & have to be different to the member with any cable section.

6.7- CASE STUDY

The structure shown in Appendix A (Fig. A1) is used to illustrate the optimisation of a 2D plane frame cable-stayed bridge model under the two cables out conditions. The structure is a medium size asymmetric cable-stayed bridge with an overall length of 954m. The main span consists of an insitu reinforced and prestressed concrete deck supported by 38 high tensile cables from a single A frame, insitu concrete tower. In this bridge the stay system consists of 38 cables (Nc=38) named as cable k where k $\in$ [8001-8038]. Information regarding design criteria and methods of analyses that govern this bridge can be found in Appendix A.

Parameters which govern cables out effect are: $M_{a(o)}$, $F_{i(o)}$, $F_{j(o)}$, μ_a^i, μ_a^j, Φ_i^j and Φ_j^i. Each parameter has a direct contribution to the optimisation process of cables out effect as defined in eq. 6.9. Prior to the selection of cables i and j for removal, two distinct stages need to be determined. The first stage, known as the initial stage (i.e. no cable is taken out), parameters corresponding to this stage are $M_{a(o)}$, $F_{i(o)}$, and $F_{j(o)}$ and are calculated according to BD 37/88 (Appendix D). In the second stage influence lines for one cable out effect are generated resulting in the parameters μ_a^i, μ_a^j, Φ_i^j and Φ_j^i to be evaluated.

Parametric data associated with cables forces at the initial stage (i.e. all cables in), $F_{i(o)}$ and $F_{j(o)}$, are shown in table 6.1. The design criteria for these forces is given in Appendix A. The 1st column shows the 38 cables, cable 8001 to 8038, which constitute the stay system. The 2nd and 3rd, 4th and 5th, 6th and 7th columns represent cable forces, $F_{i(o)}$ (i=8001,..,8038), subject to combinations 4-1, 4-2, and 4-3 for both adverse and relieving effects, respectively. Combination 4-1 is related to considering live loads with higher load factor and excludes the application of wind loads and temperatures. Combination 4-2 applies to live loads combined together with wind effects. For the 2D analysis only two types wind loads apply; vertical and longitudinal wind. In the case of 3D analysis a third component of transverse wind load would apply too. Combination 4-3 applies for temperature effect together with live loads. Two types of temperature apply; restrained to movement and temperature difference. It should be noted for every combination (4-1 to 4-3) two sets of output is generated known as full effect and reduced effect. Cables forces for combination 4-1 under full effect and reduced effect are shown in Fig. 6.4. This figure shows the variation in tension for all cables (8001 to 80038) in the bridge prior to the removal of any cable. The reader may refer to Appendix D which gives more explanation on load combinations according to BD37/88 which are of relevance to the design criteria specified in Appendix A.

Table 6.2 shows parametric data associated with effect of removing all cables, one at a time, and applying 100kN load instead, resulting in the parameter μ_a^i being evaluated. The parameter μ_a^i represents the effect of removing cable i shown in the 1st column on axial, shear, and moment given in the 2nd, 3rd, and 4th columns, respectively for location (a) on the bridge, which in this case represents member 2700 shown in Fig. 6. 3(a). This figure shows the variation in moment at point 2700 due to the removal of cable i (i=8001 to 8038) one at a time and replace it with a 100 kN axial load instead.

Table 6.3 and 6.4 present samples of the effect of cable removal for two selected cables (8029 and 8030) resulting in the parameter Φ_i^j being evaluated. The parameter Φ_i^j represents the effect of removing cable j (shown in column 1, where j=8001,..,8038), on the force in cable i (i=8029 for Table 6.3) and applying 100kN load instead (see Fig. 6.3 b-c).

Having completed the pre-processing stage of calculating different parameters ($F_{i(o)}$, $F_{j(o)}$, μ_a^i, μ_a^j, Φ_i^j and Φ_j^i) which govern the cables out conditions, then the optimisation process of the two cables (i and j) selected for removal starts taking place.

Table 6.7 shows a typical generation produced by the GA run. The 1st column represents string number. The 2nd column shows the binary string coding which when decoded results in the fitness to be evaluated which is shown in the 3rd column. The 4th and 5th columns represent the two cables i and j selected for removal with their corresponding cables forces at the initial stage given in the 6th and 7th columns respectively. Penalties are applied wherever a zero fitness is encountered in the decoded string. This is done by penalizing the fitness function when it meets a negative value in case of maximization and vice versa. If a maximization is considered, when a negative value is encountered, it is automatically reduced to zero. As a consequence, the maximum is sought only among the positive values.

In order to check the convergence and the capabilities of GAs in locating global optimum a complete enumeration scheme was carried out. For member 2700 (see Fig. A1, Appendix A) an exhaustive search has been undertaken and the optimum solution based on the design criteria given in Appendix A detected that cables 8029 and 8030 would have the worst effect, on bending moment at section 2700, when removed resulting in the optimum value of moment at that section due to cables removal to be 54958.6 kNm.

Fig. 6.5 and 6.6 show the GAs history of 20 generations each having a population size of 30 for maximum and average fitnesses in every generation. Each figure contains runs where initial population has been generated in two different ways. The first way randomly generates the initial population by the GA program while the second uses a biased initial population whereby initial solution(s) are externally forced into the GA run. In both figures results obtained from GAs are compared with optimum solution generated through the exhaustive search.

Fig. 6.5a shows the maximum fitnesses for two GA runs where initial population has been randomly generated. In the first run Single crossover was used and it showed that the starting initial solution found through the random generation of initial population has located a maximum fitness of 30499.4. A very small increase was reported in the 1st generation. From the 2nd through the 6th generation a considerable increase was reported till the the optimum solution was located in the 7th generation. After that near optimum solutions were reported in the remaining generations. When double crossover is used the situation is rather different where big improvement was reported in the next generation (generation 0) where near optimal solution was located and the properties of that solution was then carried over through the remaining generations. Although using single crossover has located a global optimum, the double crossover has displayed a better performance with respect to average results/population (see Fig. 6.6a).

Figs. 6.5b and 6.6b show results for several tests where initial population was biasly forced into the GA run. The aim is to demonstrate that GA is capable of exploring the search space and locating the optimum solution no matter how weak the initial starting solutions are. For this reason three different runs with different biased initial populations were carried out using single and double crossover sites.

The initial population in the first run consists of 30 strings where every string when decoded results in cables 8003 and 8004 to be removed. These two cables have a very small effect (418.9, 2nd column Table 6.6) when selected for removal compared to the optimum solution (54958.6, Fig. 6.5b) of cables 8029 and 8030 when removed. Fig. 6.5b shows that improvement has been reported in following generation (31082.6) to the initial population and near optimum solution was located in the 6th generation (52740.9) and finally the optimum solution was actually found in the 18th generation (54958.6). The second run has used cables 8007 and 8003 to constitute the initial population. This run has displayed better results in locating the optimum solution as well as in relation to the overall average performance of each generation (Fig. 6.5b and 6.6b). The third run has used a mixture of cables to constitute the initial population (shown in Table 6.9) has actually located the optimum solution in the 11th generation of the GA run.

It should be noted that double crossover was used in the first and the second run while single crossover was applied in the third run. GA runs with double crossover have displayed a relatively better performance than those with a single crossover (see Fig. 6.6b).

Figs. 6.5 and 6.6 have shown results for maximum and average fitnesses found in every generation for the optimisation process of the bridge shown in Fig. A1 subject to two cables out conditions. For member 2700, runs have been made for a population size of 30, and terminating then after 20 generations have been processed. The probabilities of crossover and mutation have taken the values of 0.95 and 0.01, respectively for reasons that are explained in section 5.6.5. Optimum solution characteristics resulted in cables 8029 and 8030 to have the worst effect, on bending moment at section 2700, when removed. Forces in the cables were taken from load combination 4-1 for the full adverse effect shown in column 2 of table 6.1. The best found string is [110011100011] when decoded results in the parameters $F_{8030(o)}$ and $F_{8029(o)}$ representing cables forces at the initial stage (i.e. all cables in) to take the values of -7937kN and -7592kN, respectively. As a consequence, parameters μ_{2700}^{8030}, μ_{2700}^{8029}, Φ_{8030}^{8029} and Φ_{8029}^{8030} taking the values of -1.9919m, -3.9587m, 0.1670, and 0.1583, respectively where:

$F_{8030(o)}$ Represents tension force in cable 8030 at the initial stage (i.e. with all cables in).

$F_{8029(o)}$ Represents tension force in cable 8029 at the initial stage (i.e. with all cables in).

μ_{2700}^{8030}: Represents effect of removing cable 8030 and applying unit load instead on bending moment at section 2700.

μ_{2700}^{8029}: Represents effect of removing cable 8029 and applying unit load instead on bending moment at section 2700.

Φ_{8030}^{8029}: Represents effect of removing cable 8029 and applying unit load instead on the tension force in cable 8030.

Φ_{8029}^{8030}: Represents effect of removing cable 8030 and applying unit load instead on the tension force in cable 8029.

The decoding of the best found string and its associated parameters has resulted in the fitness which represents the effect of removing cables 8029 and 8030 on bending moment at section 2700 to be 54956 kNm. These results are shown in table 6.8.

6.7- CONCLUSIONS

The design and analysis of cable-stayed bridges subject to stay removal has been the main problem addressed in this Chapter. It was shown that cables play a decisive role in the structural behaviour of cable stayed bridges and as a result stay removal will have a severe effect on forces and moments for the structural components of the bridge. This adverse effect can be understood if a relation between forces, moments and the tensile forces in the cables can be established.

Complications associated with controlled and accidental stay removal have been addressed. It was shown that controlled stay removal is associated with one cable out while accidental stay removal may take place over one and two cables out. Derivation of the mathematical statement which models the behaviour of cable-stayed bridges subject to one and two cables out was investigated. In this derivation relation between cable forces prior to their removal and bending moment were made. It was also shown that this derivation is general and can be applied to any effect (i.e. axial and shear force).

The global design and analysis of cable-stayed bridges was then addressed. It was shown that the design and analysis of this type of structures is an iterative process and for every modification in the bridge geometry a complete cycle of design and analysis is required.

It was also shown that design and analysis of cable-stayed bridges in the light of two cables out has increased the search space to be explored by many times. As a consequence, this has resulted in the cost of analysis to increase dramatically and called upon effective methods of optimisation where the number of analyses required can be brought to a minimum.

Several issues which confront the optimisation process of two cables out were addressed. It was shown that the selection of the two cables for removal is a complicated optimisation task and of combinatorial nature where the number of permutations and combinations needed for stay removal require a great deal of computations. It was concluded that numerical conventional optimisation methods are not appropriate for this task and exhaustive combinatorial methods are very expensive solutions. As a consequence, Genetic Algorithms (GAs) are used instead.

Mathematical formulations which govern the behaviour of cable-stayed bridges under cables out conditions were readjusted to fit the framework of applying Genetic Algorithms for the structural optimisation process.

The working of GAs and the different stages needed for the optimisation process have been thoroughly discussed previously in Chapters 4 and 5. Several issues confronting the optimisation of two cables out via GAs were discussed. These included: discretization and coding, fitness mapping, constraints, genetic algorithm parameters, design interface, and analysis interface.

In this Chapter additional sensitivity tests to those presented in Chapter 4 and 5 were carried out. The use of single and double crossover sites have been examined and results showed that GAs have displayed better performance with double crossover sites. A thorough investigation on the starting solutions defined in the initial population were presented. Two methods for seeding the initial population into the GA run were presented where random and bias generation of the initial population have been used.

Several GA runs were made where strings forming the initial population have been forced to actually start from one location which is far from the optimum solution. It was shown that GAs have explored all feasible regions before an optimum solution was reached. This has been verified by undertaking a complete enumeration scheme. It was demonstrated that GA is actually capable of locating global optimum.

In brief this Chapter has modelled the behaviour of cable-stayed bridges under cables out conditions. It has also discussed their implications on the prospect of analysing all possible loading combinations on the number of analyses required in order to achieve a functional cable-stayed bridge and called upon effective methods of optimisation. It was shown that GAs are powerful and practical optimisation tools for this problem and for the design and analysis of cable-stayed bridges.

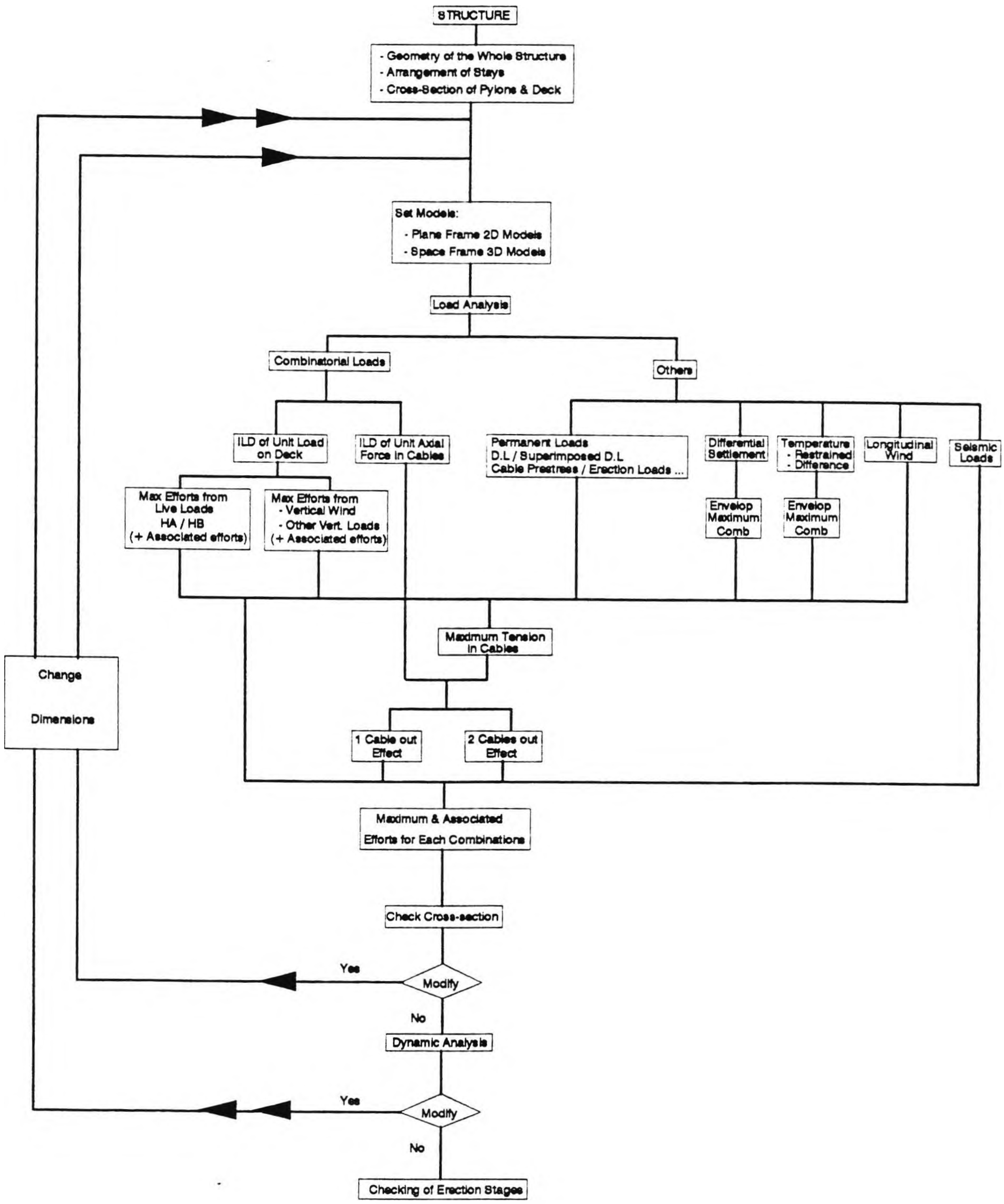


FIGURE 6.1. DESIGN AND ANALYSIS CONCEPT OF CABLE-STAYED BRIDGES SUBJECT TO CABLES OUT CONDITIONS.

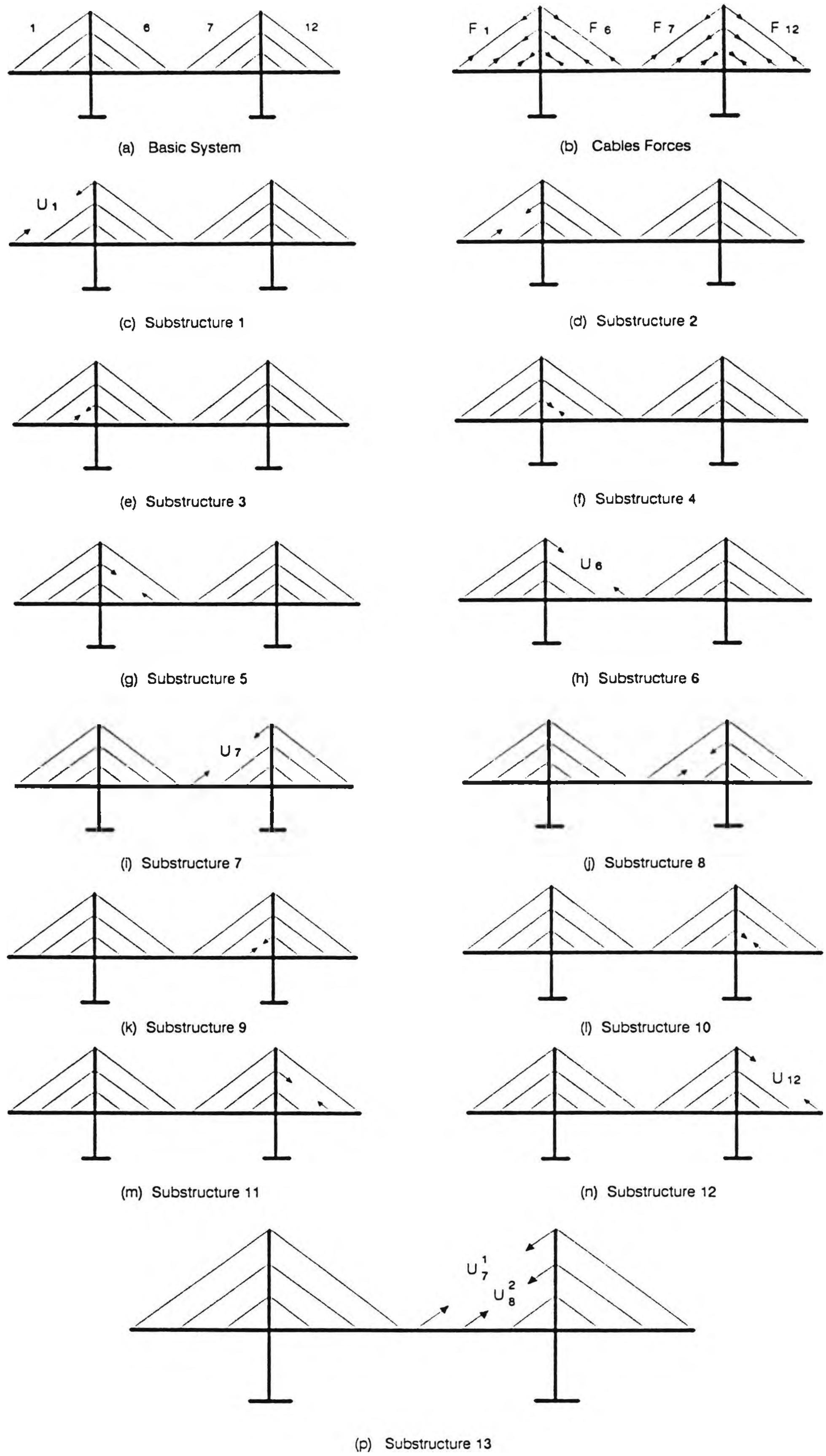
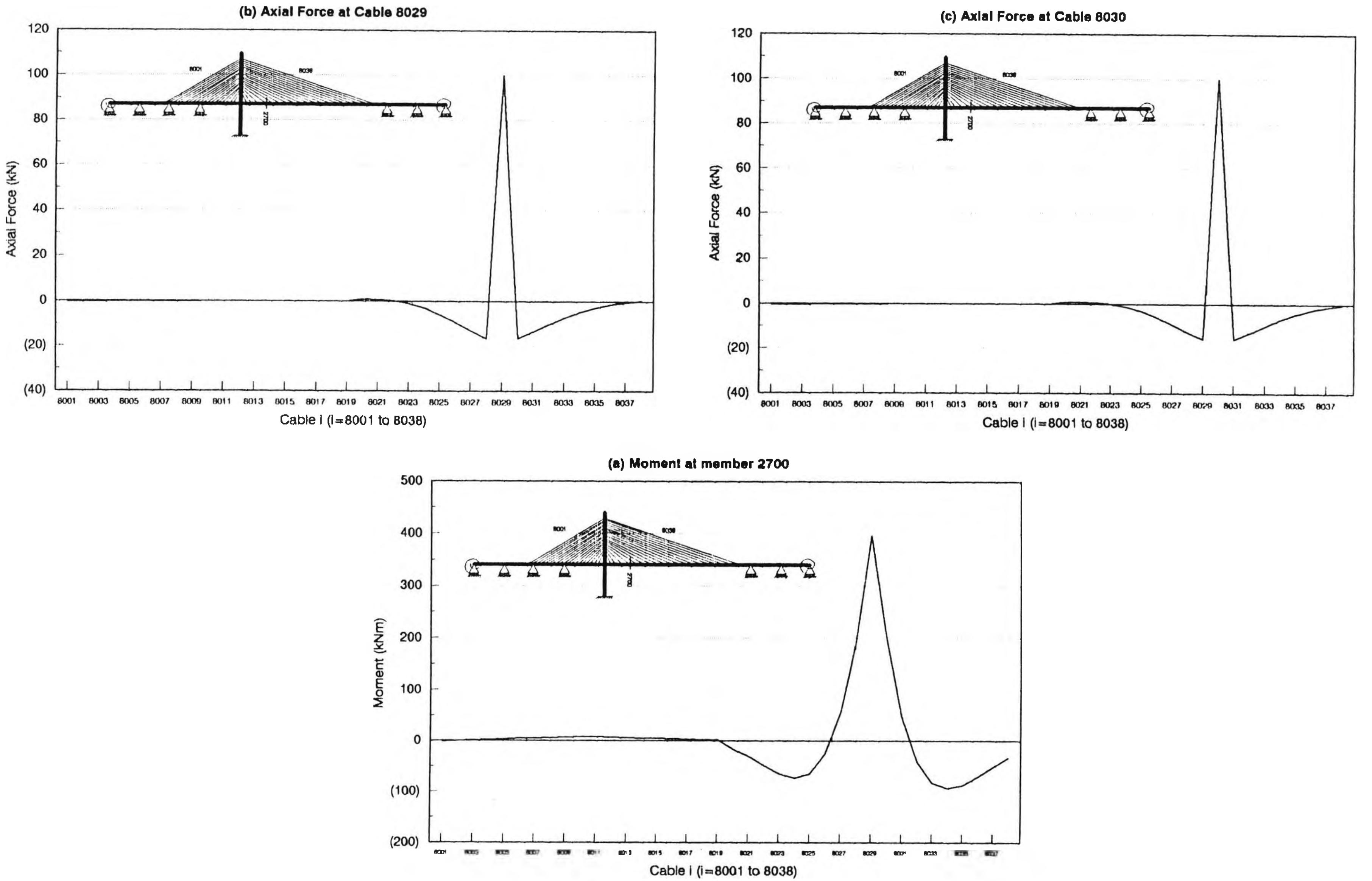


FIGURE 6.2. CABLE STAYED BRIDGE UNDER DIFFERENT STAY CONDITIONS.

FIGURE 6.3. ONE CABLE OUT INFLUENCE LINE FOR SOME SELECTED MEMBERS (100kN LOAD APPLIED).



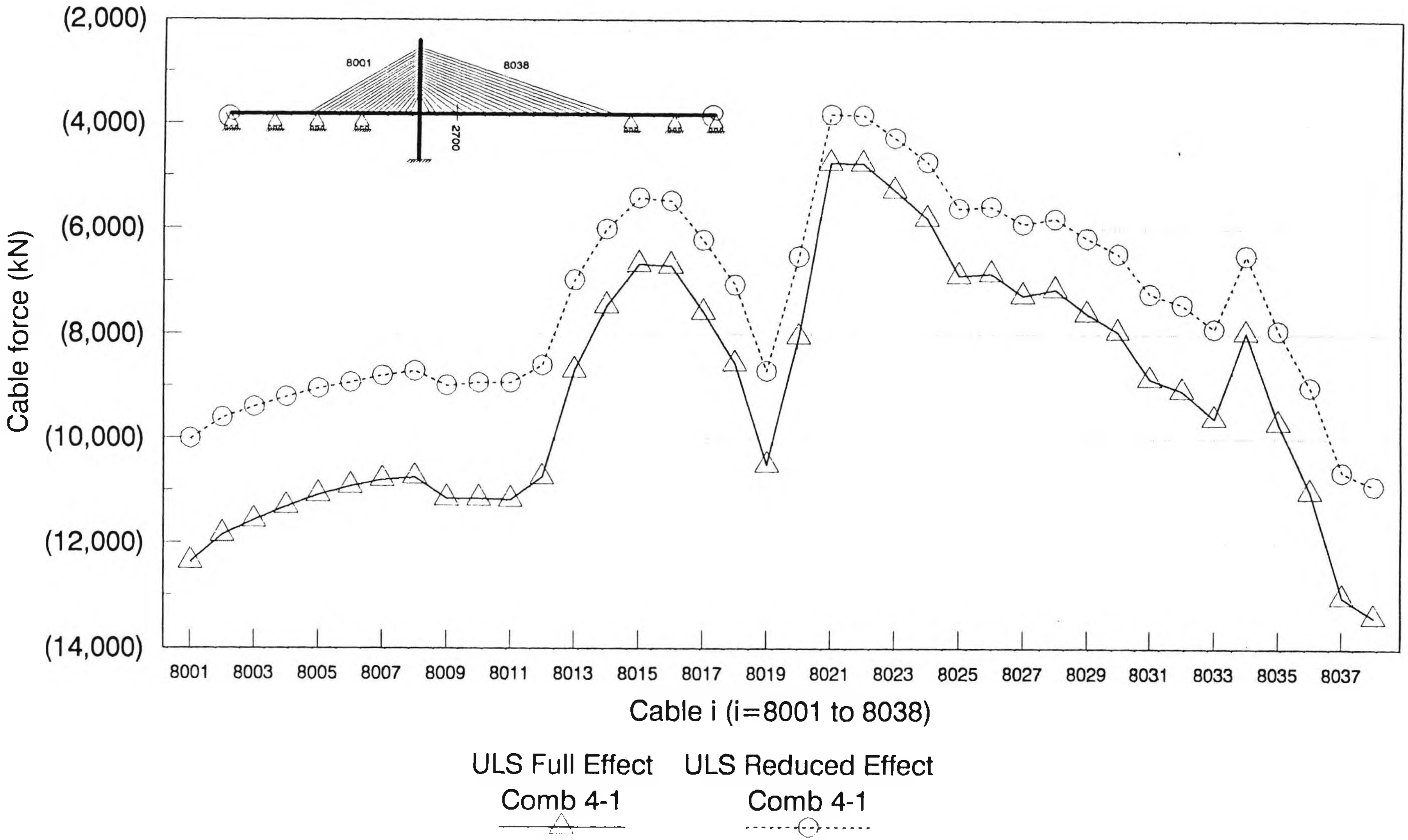


FIGURE 6.4. FORCES DEVELOPED IN CABLES AT THE INITIAL STAGE (i.e. NO CABLE OUT).

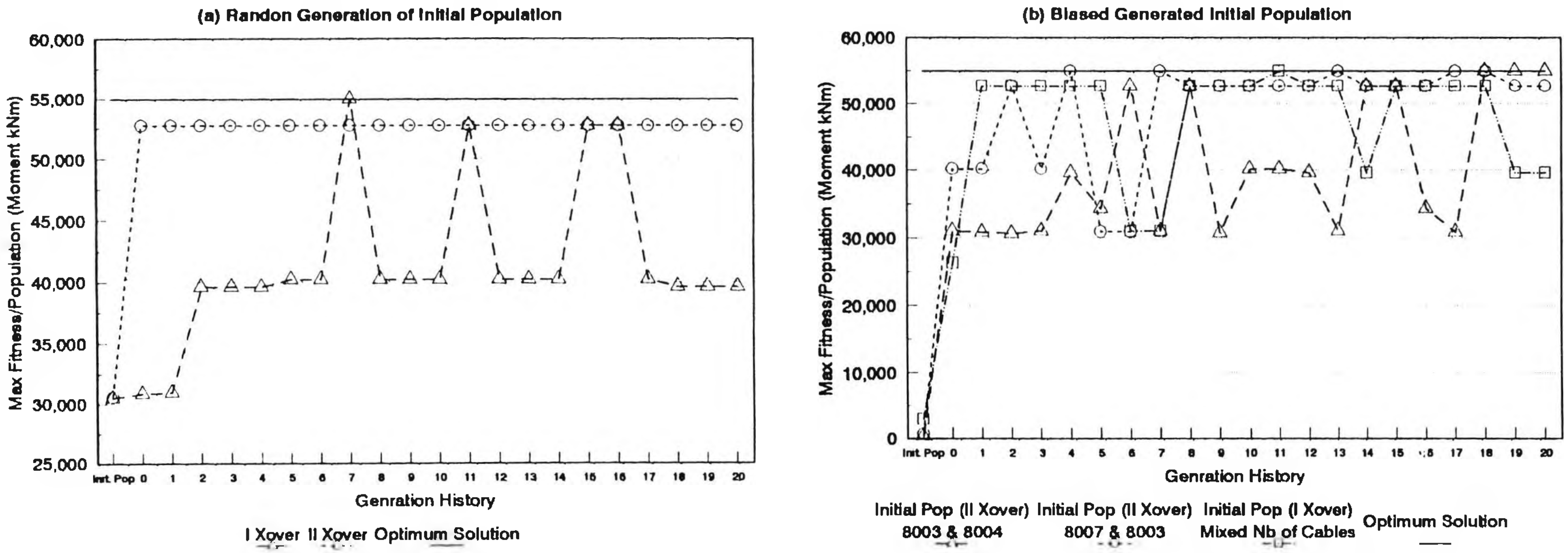


FIGURE 6.5. MAX FITNESSES GAs GENERATIONS HISTORY FOR TWO CABLES OUT EFFECT ON MOMENT AT POINT 2700.

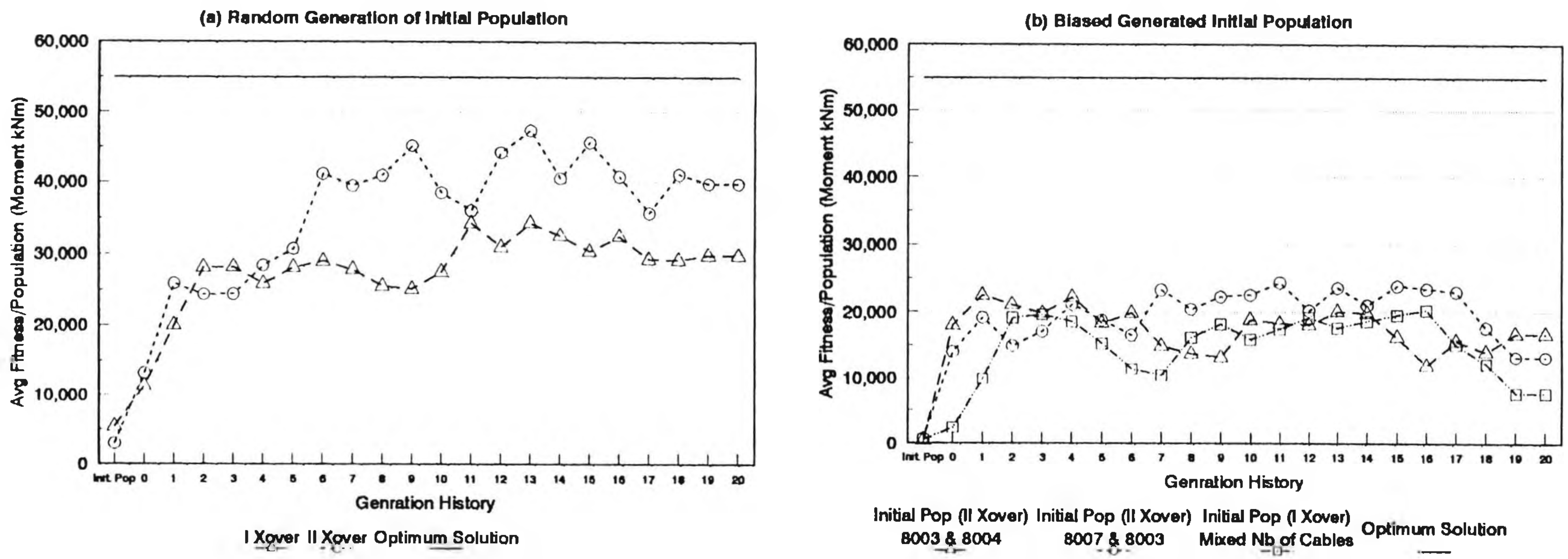


FIGURE 6.6. AVERAGE FITNESSES/POPULATION GAs GENERATIONS HISTORY FOR TWO CABLES OUT EFFECT ON MOMENT AT POINT 2700.

TABLE 6.1. CABLES FORCES AT THE INITIAL STAGE (i.e. NO CABLE OUT).

	COMB 4-1		COMB 4-2		COMB 4-3	
CABLE	ULS FULL	ULS RED	ULS FULL	ULS RED	ULS FULL	ULS RED
8001	-12370	-10009	-12446	-9904	-12498	-9851
8002	-11848	-9611	-11927	-9505	-11959	-9472
8003	-11574	-9403	-11658	-9293	-11674	-9276
8004	-11314	-9209	-11403	-9094	-11405	-9091
8005	-11090	-9048	-11184	-8928	-11176	-8935
8006	-10926	-8932	-11027	-8805	-11011	-8820
8007	-10795	-8800	-10904	-8664	-10881	-8685
8008	-10747	-8711	-10864	-8566	-10836	-8591
8009	-11157	-8986	-11288	-8826	-11254	-8856
8010	-11154	-8932	-11291	-8764	-11253	-8799
8011	-11178	-8930	-11318	-8758	-11274	-8799
8012	-10745	-8594	-10878	-8432	-10833	-8473
8013	-8688	-6971	-8791	-6846	-8754	-6880
8014	-7458	-6011	-7539	-5912	-7510	-5939
8015	-6676	-5413	-6742	-5332	-6720	-5353
8016	-6706	-5471	-6760	-5402	-6747	-5414
8017	-7578	-6220	-7624	-6159	-7626	-6157
8018	-8559	-7057	-8598	-7002	-8619	-6981
8019	-10502	-8697	-10542	-8640	-10592	-8590
8020	-8040	-6521	-8072	-6469	-8130	-6410
8021	-4740	-3810	-4774	-3755	-4762	-3767
8022	-4753	-3829	-4792	-3766	-4743	-3815
8023	-5259	-4261	-5302	-4193	-5233	-4262
8024	-5800	-4720	-5846	-4649	-5783	-4712
8025	-6881	-5607	-6935	-5522	-6868	-5590
8026	-6839	-5572	-6895	-5485	-6830	-5550
8027	-7256	-5907	-7318	-5811	-7251	-5878
8028	-7140	-5806	-7204	-5708	-7138	-5774
8029	-7592	-6173	-7660	-6068	-7590	-6138
8030	-7937	-6463	-8005	-6359	-7932	-6432
8031	-8849	-7219	-8920	-7110	-8834	-7198
8032	-9074	-7422	-9141	-7319	-9040	-7420
8033	-9608	-7880	-9672	-7782	-9601	-7852
8034	-7961	-6498	-8016	-6414	-8003	-6427
8035	-9689	-7913	-9737	-7840	-9804	-7771
8036	-11012	-8997	-11044	-8947	-11210	-8780
8037	-13025	-10633	-13048	-10601	-13331	-10316
8038	-13396	-10892	-13425	-10866	-13799	-10488

TABLE 6.2. AXIAL, SHEAR, AND MOMENT AT MMBER 2700 DUE TO CABLE i REMOVED AND 100kN LOAD APPLIED.

CABLE	AXIAL	SHEAR	MOMENT
8001	-15.3865	0.9745	-1.4436
8002	-13.6607	0.8699	-0.1571
8003	-12.1372	0.7780	1.0716
8004	-10.7966	0.6965	2.2648
8005	-9.6450	0.6254	3.4228
8006	-8.6802	0.5635	4.5253
8007	-7.8883	0.5092	5.5320
8008	-7.2405	0.4605	6.3880
8009	-6.7051	0.4158	7.0472
8010	-6.1748	0.3698	7.4022
8011	-5.5762	0.3198	7.4006
8012	-4.8805	0.2661	7.0332
8013	-4.1062	0.2104	6.3554
8014	-3.3601	0.1585	5.5677
8015	-2.6568	0.1102	4.7331
8016	-2.0263	0.0666	3.9364
8017	-1.4788	0.0271	3.2247
8018	-0.9428	-0.0143	2.6360
8019	-0.4721	-0.0544	2.1816
8020	-1.5208	1.0640	-16.8725
8021	-2.3320	1.4819	-30.2891
8022	-2.9338	1.8205	-48.2776
8023	-2.7762	1.7222	-64.9354
8024	-1.1821	0.8380	-73.1214
8025	2.5486	-1.2308	-65.1548
8026	8.9753	-4.8226	-24.9286
8027	18.6473	-10.2519	58.1984
8028	31.0248	-17.2382	195.2222
8029	45.7975	-25.6191	395.8657
8030	-38.3693	28.6726	199.1874
8031	-26.7526	19.1497	46.5436
8032	-18.4557	11.4647	-41.0771
8033	-14.5822	5.9688	-82.5469
8034	-14.9105	2.2492	-93.2180
8035	-19.5254	-0.0121	-87.2935
8036	-27.3218	-1.2353	-70.7803
8037	-37.9313	-1.7575	-51.5237
8038	-49.7520	-1.8433	-32.4305

TABLE 6.3. AXIAL FORCES AT CABLE 8029 DUE TO CABLE i REMOVED AND 100kN LOAD APPLIED.

CABLE	AXIAL	SHEAR	MOMENT
8001	-0.6019	0.0000	0.0018
8002	-0.5226	-0.0001	0.0012
8003	-0.4513	-0.0001	0.0012
8004	-0.3872	-0.0001	0.0006
8005	-0.3294	-0.0001	0.0006
8006	-0.2792	0.0000	0.0006
8007	-0.2354	0.0000	0.0000
8008	-0.1991	0.0000	-0.0006
8009	-0.1699	0.0000	-0.0006
8010	-0.1467	0.0001	-0.0006
8011	-0.1286	0.0001	-0.0006
8012	-0.1158	0.0001	-0.0006
8013	-0.1059	0.0001	-0.0007
8014	-0.0993	0.0001	-0.0008
8015	-0.0951	0.0001	-0.0009
8016	-0.0944	0.0000	-0.0009
8017	-0.0985	0.0000	-0.0009
8018	-0.1036	0.0001	-0.0008
8019	-0.1137	0.0001	-0.0007
8020	0.8175	0.0000	0.0010
8021	0.6546	0.0000	-0.0011
8022	0.0824	0.0000	-0.0011
8023	-1.1462	0.0001	-0.0022
8024	-3.1591	0.0001	-0.0032
8025	-6.1466	0.0001	-0.0047
8026	-9.6678	0.0001	-0.0056
8027	-13.5818	0.0001	-0.0060
8028	-16.7237	0.0001	-0.0051
8029	100.0000	0.0001	-0.0016
8030	-16.7050	-0.0001	0.0024
8031	-13.8473	-0.0001	0.0045
8032	-10.4415	-0.0001	0.0045
8033	-7.3590	-0.0001	0.0043
8034	-4.7509	-0.0001	0.0035
8035	-2.8065	-0.0001	0.0026
8036	-1.3773	-0.0001	0.0013
8037	-0.4391	0.0000	0.0006
8038	0.1357	0.0000	-0.0006

TABLE 6.4. AXIAL FORCES AT CABLE 8030 DUE TO CABLE i REMOVED AND 100kN LOAD APPLIED.

CABLE	AXIAL	SHEAR	MOMENT
8001	-0.5894	0.0000	0.0018
8002	-0.5071	-0.0001	0.0012
8003	-0.4323	-0.0001	0.0012
8004	-0.3648	-0.0001	0.0006
8005	-0.3046	-0.0001	0.0006
8006	-0.2509	0.0000	0.0006
8007	-0.2055	0.0000	0.0000
8008	-0.1687	0.0000	-0.0006
8009	-0.1407	0.0001	-0.0006
8010	-0.1210	0.0001	-0.0006
8011	-0.1075	0.0001	-0.0006
8012	-0.0961	0.0001	-0.0006
8013	-0.0867	0.0001	-0.0007
8014	-0.0786	0.0001	-0.0008
8015	-0.0736	0.0000	-0.0009
8016	-0.0699	0.0000	-0.0009
8017	-0.0700	0.0000	-0.0009
8018	-0.0711	0.0000	-0.0008
8019	-0.0750	0.0001	-0.0007
8020	0.6950	0.0000	0.0010
8021	0.7255	0.0000	0.0000
8022	0.5338	0.0000	-0.0011
8023	-0.1009	0.0001	-0.0011
8024	-1.3320	0.0001	-0.0022
8025	-3.3608	0.0001	-0.0038
8026	-6.0588	0.0001	-0.0047
8027	-9.4412	0.0001	-0.0052
8028	-12.8579	0.0001	-0.0060
8029	-15.8262	0.0001	-0.0049
8030	100.0000	0.0001	-0.0016
8031	-15.8491	-0.0001	0.0023
8032	-13.0722	-0.0001	0.0037
8033	-10.0059	-0.0001	0.0043
8034	-7.0564	-0.0001	0.0035
8035	-4.6630	-0.0001	0.0032
8036	-2.7345	-0.0001	0.0019
8037	-1.3583	0.0000	0.0012
8038	-0.4242	0.0000	-0.0006

TABLE 6.5. MAX AND AVERAGE FITNESSES FOR RANDOMELY GENERATED INITIAL POPULATION.

	Single Xover		Double Xover	
Generation	Max	Average	Max	Average
Init. Pop	30499.4	5421.5	30580.0	3045.0
0	30860.4	11341.1	***52740.9**	13209.5
1	30946.2	20031.8	52740.9	25925.2
2	39614.6	28152.4	52740.9	24380.4
3	39614.6	28182.3	52740.9	24423.7
4	39614.6	25892.5	52740.9	28431.1
5	40225.5	28170.0	52740.9	30753.1
6	40225.5	29004.9	52740.9	41192.2
7	***54958.6**	27846.7	52740.9	39514.8
8	40225.5	25541.8	52740.9	40966.6
9	40225.5	25102.0	52740.9	45181.3
10	40225.5	27506.8	52740.9	38540.5
11	52740.9	34292.1	52740.9	36059.0
12	40225.5	31019.8	52740.9	44237.7
13	40225.5	34322.4	52740.9	47367.5
14	40225.5	32580.6	52740.9	40595.5
15	52740.9	30402.3	52740.9	45668.9
16	52740.9	32490.3	52740.9	40846.8
17	40225.5	29278.7	52740.9	35756.0
18	39614.6	29203.0	52740.9	41153.5
19	39614.6	29811.6	52740.9	39830.8
20	39614.6	29811.6	52740.9	39830.8

* *Two Cables Out 8029 & 8030 (optimum)*

** *Two Cables Out 8028 & 8029*

TABLE 6.6. MAX AND AVERAGE FITNESSES FOR BIASLY GENERATED INITIAL POPULATION.

	Bias Init-Pop 8003 & 8004		Bias Init-Pop 8007 & 8003		Bias Init-Pop Mixed cables	
	Double Xover		Double Xover		Single Xover	
Generation	Max	Average	Max	Average	Max	Average
Init. Pop	418.9	418.9	784.7	784.7	3012.4	566.5
0	31082.6	18040.2	40225.5	14044.6	26375.3	2390.6
1	30917.2	22569.4	40225.5	19161.4	52740.9	9886.6
2	30671.1	21116.0	52740.9	14970.4	52740.9	19142.2
3	31082.6	19846.2	40225.5	17149.9	52740.9	19647.3
4	39614.6	22286.3	54958.6	21175.4	52740.9	18615.6
5	34324.9	18379.2	30939.8	18814.1	52740.9	15291.7
6	52740.9	19898.8	30946.2	16591.9	31082.6	11443.8
7	31082.6	14948.1	***54958.6**	23349.0	31082.6	10487.7
8	52740.9	13794.1	52740.9	20475.0	52740.9	16248.8
9	30827.1	13212.5	52740.9	22327.0	52740.9	18273.0
10	40225.5	18849.3	52740.9	22615.3	52740.8	15957.7
11	40225.5	18316.0	52740.9	24464.9	***54958.6**	17497.0
12	39614.6	18162.2	52740.9	20317.9	52740.9	19229.5
13	31082.6	20126.3	54958.6	23704.8	52740.9	17671.4
14	52740.9	19919.3	52740.9	21082.6	39614.6	18690.3
15	52740.9	16293.0	52740.9	23979.3	52740.9	19617.9
16	34324.9	11918.7	52740.9	23538.0	52740.9	20327.8
17	30860.4	15664.6	54958.6	23093.6	52740.9	15187.6
18	***54958.6**	14071.1	54958.6	17759.5	52740.9	12213.0
19	54958.6	16790.9	52740.9	13257.8	39614.6	7656.0
20	54958.6	16790.9	52740.9	13257.8	39614.6	7656.0

**** Two Cables Out 8029 & 8030 (optimum)***

Table 6.7. Genetic Algorithms results for two cables out conditions.

----- Population Result @ Generation 3 -----

	String	Fitness	Cab1	cab2	F1	F2
1)	[110010000011]	30860.4	8012	8029	-10745.0	-7592.0
2)	[111111010011]	9400.6	8037	8030	-13025.0	-7937.0
3)	[111100000001]	1020.3	8009	8019	-11157.0	-10502.0
4)	[011101010010]	14794.4	8028	8011	-7140.0	-11178.0
5)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0
6)	[110011000010]	16663.5	8030	8010	-7937.0	-11154.0
7)	[110011110111]	7959.7	8030	8035	-7937.0	-9689.0
8)	[110011111010]	16237.0	8030	8014	-7937.0	-7458.0
9)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0
10)	[111111000011]	23554.8	8037	8029	-13025.0	-7592.0
11)	[111111000011]	23554.8	8037	8029	-13025.0	-7592.0
12)	[111111000011]	23554.8	8037	8029	-13025.0	-7592.0
13)	[101111000011]	22799.2	8036	8029	-11012.0	-7592.0
14)	[111111110011]	9400.6	8037	8030	-13025.0	-7937.0
15)	[110011110010]	16586.9	8030	8012	-7937.0	-10745.0
16)	[111101001011]	20386.0	8028	8031	-7140.0	-8849.0
17)	[011101000011]	52740.9	8028	8029	-7140.0	-7592.0
18)	[111110000011]	30331.1	8019	8029	-10502.0	-7592.0
19)	[111101000011]	52740.9	8028	8029	-7140.0	-7592.0
20)	[111101000011]	52740.9	8028	8029	-7140.0	-7592.0
21)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0
22)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0
23)	[110011100011]	54958.6	8030	8029	-7937.0	-7592.0
24)	[111110000011]	30331.1	8019	8029	-10502.0	-7592.0
25)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0
26)	[101101101010]	4780.5	8027	8013	-7256.0	-8688.0
27)	[111101100011]	52740.9	8028	8029	-7140.0	-7592.0
28)	[110010000011]	30860.4	8012	8029	-10745.0	-7592.0
29)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0
30)	[110010100011]	30860.4	8012	8029	-10745.0	-7592.0

Generation Maximum = 54958.6 kNm
Generation Sum = 815021.0 kNm
Generation Average = 27167.4 kNm

Table 6.8. Two cables out best Generation optimization results via Gaenetic Algorithms

* Member = 27002700.CAB
* Number of Cables = 38
* Number of Parameters = 2
* Min Parameter = 8001
* Max Parameter = 8038
* COMB (4 - 1)
* Full COMB
* Optimisation Over Moment
* Maximization Problem
* Popsize = 30
* Maxgen = 20
* Pcross = 0.95 ... Two Point Crossover!
* Pmutation = 0.01

THE BEST SOLUTION SO FAR HAS BEEN FOUND AT GENERATION 3 :

THE TWO CABLES TO TAKE OUT ARE : 8030 AND 8029

$F_{8030(0)}$ = -7937 kN

$F_{8029(0)}$ = -7592 kN

μ_{2700}^{8030} = -1.9919 m

μ_{2700}^{8029} = -3.957 m

Φ_{8030}^{8029} = 0.1670

Φ_{8029}^{8030} = 0.1583

THE BEST STRING AND ITS CORRESPONDING FITNESS ARE:

[110011100011] -----> FITNESS = 54958 kNM

TABLE 6.9. BIASED INITIAL POPULATIONS DECODED STRINGS.

	INITIAL POPULATION 1		INITIAL POPULATION 2		INITIAL POPULATION 3	
STRING #	CABLE 1	CABLE 2	CABLE 1	CABLE 2	CABLE 1	CABLE 2
1	8003	8004	8007	8003	8001	8002
2	8003	8004	8007	8003	8003	8004
3	8003	8004	8007	8003	8005	8006
4	8003	8004	8007	8003	8007	8008
5	8003	8004	8007	8003	8009	8010
6	8003	8004	8007	8003	8011	8012
7	8003	8004	8007	8003	8013	8014
8	8003	8004	8007	8003	8015	8016
9	8003	8004	8007	8003	8017	8018
10	8003	8004	8007	8003	8017	8019
11	8003	8004	8007	8003	8020	8021
12	8003	8004	8007	8003	8022	8023
13	8003	8004	8007	8003	8024	8025
14	8003	8004	8007	8003	8026	8027
15	8003	8004	8007	8003	8001	8002
16	8003	8004	8007	8003	8002	8004
17	8003	8004	8007	8003	8032	8033
18	8003	8004	8007	8003	8034	8035
19	8003	8004	8007	8003	8036	8037
20	8003	8004	8007	8003	8038	8001
21	8003	8004	8007	8003	8002	8037
22	8003	8004	8007	8003	8003	8036
23	8003	8004	8007	8003	8004	8035
24	8003	8004	8007	8003	8005	8034
25	8003	8004	8007	8003	8006	8009
26	8003	8004	8007	8003	8007	8010
27	8003	8004	8007	8003	8001	8010
28	8003	8004	8007	8003	8011	8015
29	8003	8004	8007	8003	8017	8021
30	8003	8004	8007	8003	8025	8035

CHAPTER 7

CONCLUSIONS AND RECOMMENDATION FOR FUTURE RESEARCH

This thesis has focused on the application of GAs to design situation in which the combinatorial problem of load definitions and stay removals play a particularly important role in the design of cables-stayed bridges. The first three Chapters dealt with cable-stayed bridges as a design concept and addressed most areas that contribute to the overall design and analysis of these structures. The remaining Chapters (4,5 and 6) have presented problems associated with highway loading defined in BD37/88 and stay removal conditions in the context of using GAs for the determination of the worst loading and stay removal combinations.

7.1- CABLES-STAYED BRIDGES AS A DESIGN CONCEPT

Review of parameters which influence bridge design have been presented in Chapters 2 and 3. It was shown that the use of a minimum of materials and provision of a simple construction method would seem to be universal engineering ideals, but the weight given to each will differ depending on cultural and industrial environment. The key features and the modelling of cable-stayed bridges have also been discussed. The iterative nature of the design and analysis process was discussed and it was shown that a large effort is required for each analysis and redesign cycle and called upon effective methods of optimisation.

7.2- OPTIMISATION METHODS

In Chapter 2, available methods of structural optimisation have also been reviewed. Expert Systems, Mathematical Programming, and GAs have been presented as three optimisation techniques. The application of these methods into structural optimisation together with the strengths and weaknesses of each method has been discussed.

7.2.1- EXPERT SYSTEMS

It was shown that expert systems for design, except those concentrating only on conceptual design, try to integrate the preliminary design, detailed design and design evaluation in one expert system, and all expert systems support 'design by repeated analysis' considering structural design as an iterative feedback process. It was shown that the emphasis placed upon design by repeated analysis in the existing expert systems is clear evidence that knowledge incorporated as heuristics or databases alone is not sufficient to produce good preliminary solutions which lead to efficient designs.

7.2.2- MATHEMATICAL PROGRAMMING

Numerical optimisation techniques focus on the immediate area around the current design point, using local gradient calculations to move to a better design. Since no attempt is made to explore all the regions of the parameter space numerical optimisation can easily be trapped in local optima or by constraints in a region of the parameter space far from the optimal design.

7.2.3- GENETIC ALGORITHMS

Chapter 2 and 4 introduced the basic concepts of GAs and addressed the key questions of how and why they work. GAs operate on populations of strings where the strings are coded to represent the underlying parameter set. Selection, crossover, and mutation are applied to successive string populations to generate new string populations. The operations performed are simple, string copying and partial string swapping, yet the effect is powerful. A thorough review on the application of GAs in the structural optimisation has been presented where it was shown that GAs are emerging as powerful optimisation tools and would be well suited for the determination of worst loading combinations and stay removal conditions for cable-stayed bridges presented in chapters 5 and 6.

7.3- PARAMETRIC STUDY FOR THE WORST LOADING PATTERN DETERMINATION FOR CABLES-STAYED BRIDGE VIA GAs

Chapter 5 presented a parametric study for the behaviour of Multiple-spans cable-stayed bridges subject to live loading as defined in BD37/88. Live loads of that type are well known for their complexities and their implications on the number of analyses required in order to locate the worst loading pattern. Several parameters which governs their applications have been discussed and it was shown that the number of permutations and combinations needed for the location of the optimum solution is relatively large even for small size structures. GAs are used as optimisation tools for the location of worst loading scenarios whereby the solution of the GA model was based on the use of ILD's. A small detailed example was studied in order to demonstrate the different stages of the GA optimisation process and to show that GAs can actually locate an optimum solution. This have been verified and compared through a complete enumeration scheme where it was shown that GAs are actually capable of finding a global optimum.

7.3.1- SENSITIVITY ANALYSIS OF GAs CONTROLLING PARAMETERS AND VERIFICATION OF RESULTS

Different problems facing the GA optimisation process were addressed. Sensitivity studies regarding the values of GAs controlling operators (population size, probability of crossover, and probability of mutation) have been presented. The consequence of not including certain loading scenarios in the initial population have also been addressed and it was shown that the mutation model is sensitive enough to introduce these loading scenarios at a latter stage during the GA run. The GA results have been verified and compared through a complete enumeration scheme where it was shown that GA can actually find a global optimum.

7.3.2- PARAMETRIC STUDY AND DISCUSSION OF RESULTS

Having demonstrated the working of GAs on a relatively small cable-stayed bridge, GAs were then used to investigate the structural behaviour of medium to large size cable-stayed bridges under traffic loads defined in BD37/88. A parametric study on four multi-spans cable-stayed bridges with varying spans length and cables stiffnesses has been presented. The variations of moments due to HA, HB, and associated HA loading have been discussed. It was shown that moments generated due to any loading effect have decreased with the increase of cable stiffness. It was concluded that HB loading will always be a dominant load case for small to medium size bridges while HA loading would govern for large size bridges. The variation of UDL with loaded length was shown to be the major source of complexity where for small to medium size bridges the change in the UDL value with cable stiffnesses was clear while for large size bridge the variation in the UDL became less noticeable. The effect of areas and loaded length have also been studied. It was shown that areas and loaded lengths have both displayed decrease with the increase in cables stiffnesses.

7.4- APPLICATION OF GAs FOR THE DETERMINATION OF WORST CABLES OUT CONDITIONS

Chapter 6 has extended the application of GAs to a much more complicated problem addressing "The analysis of cable-stayed bridges subject to cables out conditions". First, the mathematical modelling of cable-stayed bridges behaviour under one and two cables out conditions was presented, followed by the selection of worst cables out conditions using GAs.

7.4.1- MODEL DERIVATION OF CABLES OUT CONDITIONS

The adverse effect of stay removal on the moments and forces can be understood if a relation between forces, moments and the tensile forces in cables can be made. Derivation of the mathematical statement which models the behaviour of cable-stayed bridges subject to one and two cables out was investigated where the relation between cable forces prior to their removal and bending moment were made. Stay removal has been categorized into two groups; controlled and accidental stay removal. It was shown that accidental stay removal may result in one or two cables being removed while controlled stay removal is usually associated with the removal one cable only. The implications of including cables out conditions as a design criteria for cable-stayed bridges has been discussed. It was shown that the prospect of analysing load combinations in the light of two cables out has increased the search space to be explored by many fold and called upon effective methods of optimisation. Several issues which confront the selection process of two cables out were addressed. It was concluded that conventional numerical methods are not appropriate for this task and exhaustive combinatorial methods are very expensive solutions to carry out and, as a result, GAs are likely to be best suited for this task.

7.4.2- RESULTS VERIFICATION AND ADDITIONAL SENSITIVITY TESTS ON CROSSOVER (SINGLE AND MULTIPLE) AND INITIAL POPULATION

Several issues confronting the determination process of two cables out via GAs were discussed. Single and multiple crossover were considered and it was shown that GAs have displayed better performance with double crossover sites. Additional sensitivity tests to those presented in Chapter 4 and 5 have also been presented. Several tests on the starting solutions defined in the initial population was carried out. Two methods for seeding the initial population were checked using random and biased generation of the strings which constitute the initial population. The GA results have been verified and compared by undertaking a complete enumeration scheme. It was shown that GAs are indeed capable of exploring all feasible regions before actually fixing on an optimum solution.

7.5- RECOMMENDATION FOR FUTURE RESEARCH

In this thesis GAs have been used to investigate worst combinations of loadings and stay removals. These worst combinations are themselves affected by the configuration, relative stiffness of the different cable groups, and the extent of prestressing. Additionally, the construction process and sequence is a vital design condition for cable stayed bridges in particular. All of these aspects, together with temperature effects tolerance and settlement effects, etc., need to be considered. To incorporate design into the process this would result in an iterative process which may require repeated processing of the design analysis. Further iteration may be required if the prototype design fails to satisfy some of the design specifications. However, the main purpose of the thesis is to investigate the potential for the application of GAs to such design situation in which the combinatorial problem of load definitions and stay removals play a particularly important role in contrast to many classic optimisation problems which involve solely one load condition without consideration of member failures removals.

It might be highly doubtful whether all, or even a few, of the design aspects could be efficiently incorporated in a single optimisation strategy. A unique combination of expert systems, numerical optimisation, and GAs would offer all the advantages. Expert systems could well handle conceptual design and specifications checking. GAs would be best suited for global exploration and exploitation of the feasible regions in the design space. Numerical methods would perform well in the fine tuning of the final results. This approach is very general and allows the engineer to concentrate on the design problem without having to worry about the selection and the tuning of optimisation algorithms. It is this perhaps which can provide a useful area of future research investigation.

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APPENDIX A

DESIGN STATEMENT

This Appendix describes the design statement the cable-stayed bridge, shown in Fig. A1, used as case study for the research undertaken in this thesis.

A.1.1- TYPE OF HIGHWAY

Dual 2-lane carriageway all purpose road

A.1.2- PERMITTED TRAFFIC SPEED

Over - 113 kph (70 mph)
Under - not applicable

A.2- PROPOSED STRUCTURE

A.2.1- DESCRIPTION OF STRUCTURE

The main bridge is an asymmetric cable-stayed structure linked to approach embankments via multi-span viaducts. Overall length is 954 meters.

A.2.2- STRUCTURAL TYPE

A.2.2.1- SUPERSTRUCTURE

The main span consists of an insitu reinforced and prestressed concrete deck supported by high tensile cables from a single A frame, insitu concrete tower. In form the main span comprises longitudinal edge beams housing the cable anchorages linked together with insitu post-tensioned transverse ribs carrying an insitu concrete deck slab. Structural action is primarily transverse spanning.

The anchor spans will be solid reinforced concrete with voids introduced where detailed design quantifies the balance required for the main span. Prestressing will be introduced where economic. Structural action is primarily transverse spanning.

The approach spans will be post-tensioned insitu concrete multicell box-sections. Structural action is primarily longitudinal spanning.

A.2.2.2- SUBSTRUCTURE

The abutments will be insitu reinforced concrete of the 'spill through' type.The viaduct piers will be rectangular columns insitu reinforced concrete. The anchor span piers (tension/compression) will be rectangular columns in insitu prestressed concrete. The main tower will be a hollow box section insitu reinforced concrete structure.

A.2.3- FOUNDATION TYPE

The foundation type will be piled utilising large diameter bored piles on the east bank and driven precast concrete or Continuous Flight Auger bored piles on the west bank, acting in combined skin friction and end bearing.

A.2.4- SPAN ARRANGEMENTS

The bridge will be a 20 span structure of total length 954m. Span lengths are 30m, 6 at 40m, 194m, 60m, 10 at 40m, 30m.

A.2.5- ARTICULATION ARRANGEMENTS

Expansion joints will be incorporated at each end of the bridge. The deck will be supported at the intermediate piers and at abutments on sliding and sliding guided bearings. The structure will be fully continuous fixed at the main tower. Expansion joints will be provided at each abutment. The out of balance vertical loads generated by the asymmetric nature of the main spans will be restrained by built-in tension piers. These will be designed to accommodate horizontal movements by flexure. See Fig. 1 for articulation diagram.

A.3- DESIGN CRITERIA

A.3.1- LIVE LOADING

A.3.1.1- HA LOADING

Full HA loading

A.3.1.2- HB LOADING

45 units

A.3.1.3- FOOTWAY LIVE-LOADING

On both footways to BD 37/88

A.3.1.4- ANY SPECIAL LOADING AND PROPOSED METHODS OF DEALING WITH ASPECTS NOT COVERED BY STANDARDS

A.3.1.4.1- TEMPERATURE EFFECTS ON CABLES STAYS AND TOWER

Temperature difference between cables and deck of 10 degrees Celsius will be allowed for.

Temperature difference between any two opposite sides of the tower of 10 degrees Celsius will be allowed for, with appropriate gradients for concrete box section.

A.3.1.4.2- SEISMIC LOAD

Seismic forces of 0.06g of the mass of the structure will be applied as horizontal forces in any direction.

Seismic forces will be considered in Combinations with permanent loads only. γ_{f1} = 1.3 (ULS) 1.0 (SLS).

A.3.1.4.3- STAY REMOVAL

The design will allow for accidental removal of any one stay cable with 10% live load under Combination 4 at the ultimate limit state only.

The design will allow for removal of any one stay cable for replacement with full live load on the opposite carriageway under Combinations 1,2 and 3 at both serviceability and ultimate limit states.

A.3.1.4.4- STATIC WIND LOADING

The design mean hourly speed will be taken from BD 37/88 for the completed structure. For the erection stage a return period of 50 years will be assumed.

A.3.1.4.5- ERECTION LOADING

The following non-standard effects will be allowed for in the design for the cantilever erection stages:

Wind loading on the superstructure and on a fully enclosed construction gantry of depth 6 metres by 16 metres, centred on the end of a completed stayed segment.

A construction gantry of 300 kN centred at the tip of a completed stayed segment.

A uniformly distributed superimposed dead load of 10kN per metre of deck from the tower to represent construction materials.

A live load of 300 kN at any position on the cantilever to represent construction plant.

A.4- STRUCTURAL ANALYSIS

A.4.1- Methods of analysis proposed for superstructure, substructure and foundations. Description and diagram of idealised structure to be used for analysis. Assumptions intended for calculation of structural element stiffnesses.

A.4.1.1- STAYED SPANS - STATIC

The cable stayed portion of the structure will be analysed as a plane frame using an elastic analysis program such as LEAP5 (DTp Ref 253C). The proposed idealisation is illustrated in Fig. 2.

Transverse bending and shear, together with distribution of loading for application to the plane frame will be determined via a grillage analysis (program GLAP (DTp Ref. 213C) or LEAP5 or similar). Stiffness will be assigned to each grillage element to model the member it represents in accordance with the procedures outlined in 'Bridge Deck Behaviour' by E.C. Hambly. Vertical springs will be incorporated at stay locations of stiffness appropriate to the stay bearing represented. Fig. 3 illustrates the idealisation.

Additional non-linear forces due to displacements within the structural system will be incorporated within the plane frame analysis. Time dependent deformations such as shrinkage and creep will also be taken into consideration. The non-linear behaviour of the cables will be incorporated using the Ernst modification equation.

A.4.1.1.4- APPROACH SPANS - STATIC

A grillage using elastic analysis (program GLAP or LEAP5 or similar) will be carried out to ascertain the distribution of longitudinal and transverse bending and shears developed within the approach viaducts for all loadings.

Stiffness will be assigned to each grillage element to model the member it represents in accordance with the procedures outlined in 'Bridge Deck Behaviour' by E.C Hambly. Fig. 4 illustrates the proposed grillage idealisation.

Continuous Beam analysis will be carried out to determine worst live load positions and for stress calculations. The distribution of forces obtained from the grillage will be applied as factors to the beam analysis.

A.4.1.2- SUBSTRUCTURE

A.4.1.2.1- PIERS

Piers will be analysed as columns subject to biaxial bending using a program such as COLDES2 (DTp Ref. 249C). The pile/pile cap system, under the action of the forces transferred from the columns, will be analysed as plane/space frames using an elastic analysis program such as GLAP (Dtp Ref. 213C) or LEAP5 or similar. Group behaviour will be assessed using a program as PGROUP (DTp Ref. 195C).

A.4.1.2.2- TOWER

The tower will be analysed as a space frame subject to biaxial bending, torsions and axial loads derived from the analyses described in 4.1.1. Local stresses at anchorage points will be determined by finite element analysis using LEAP5 (DTp Ref. 263C) or LUSAS or similar.

Dynamic analysis of the tower for wind loading during construction will be carried out using program LUSAS.

A.4.1.2.3- ABUTMENTS

The abutment beam will be analysed for acting soil pressures, braking forces and imposed forces from the superstructure. Granular backfill will be specified for behind the abutment beam, whereas the legs will be designed for pressures from general embankment fill. Side friction on the legs will be allowed for by doubling the actual width.

A.4.1.3- FOUNDATIONS

The piled foundations will be analysed by program PGROUP (DTp Ref. 195C), and by elastic frame analysis using a program such as LEAP5 or GLAP.

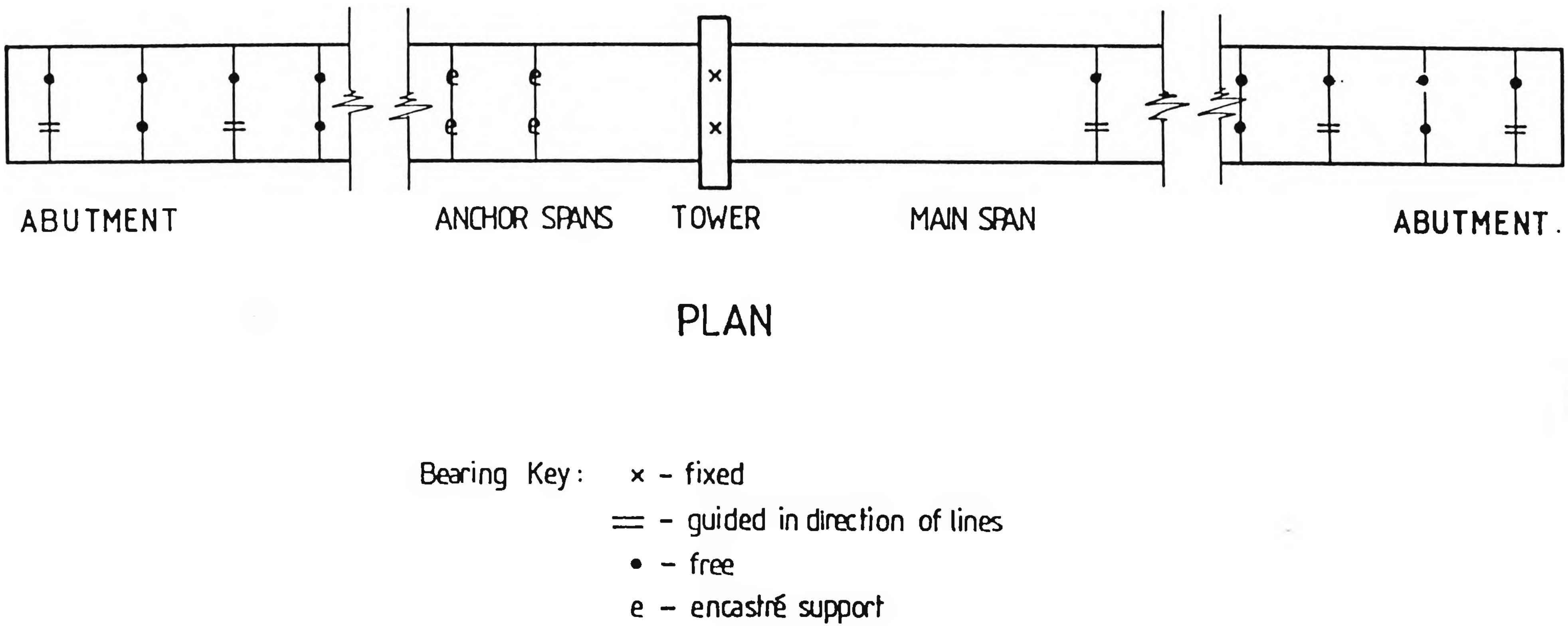
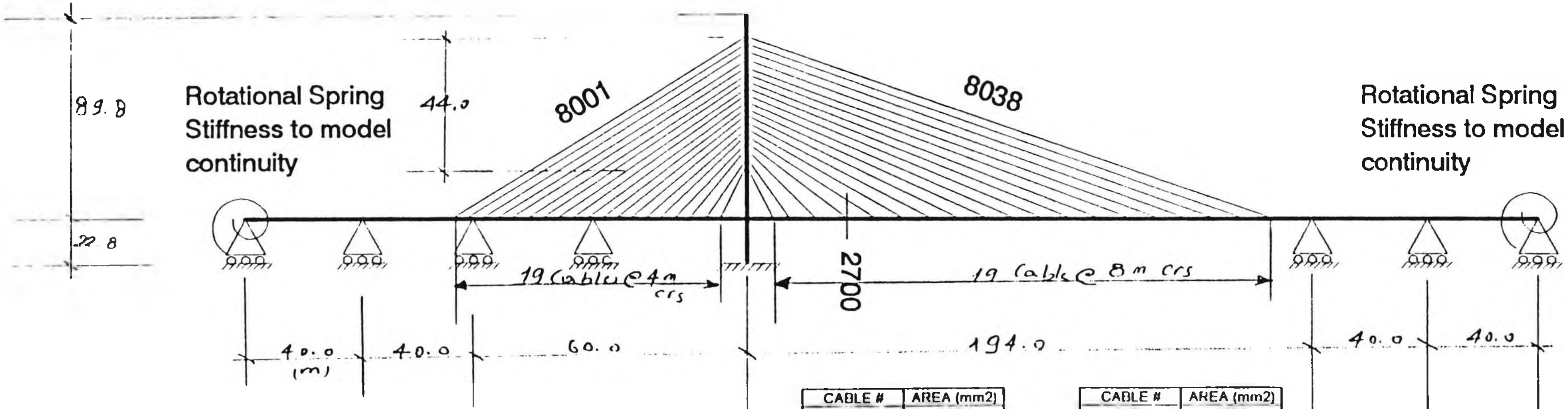


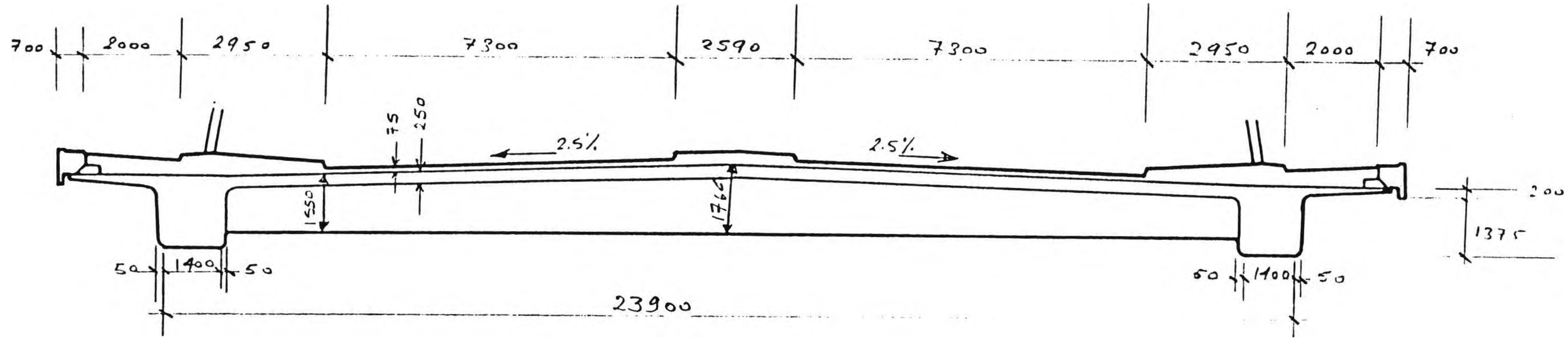
FIGURE A.1. ARTICUALTION ARRANGEMENT.



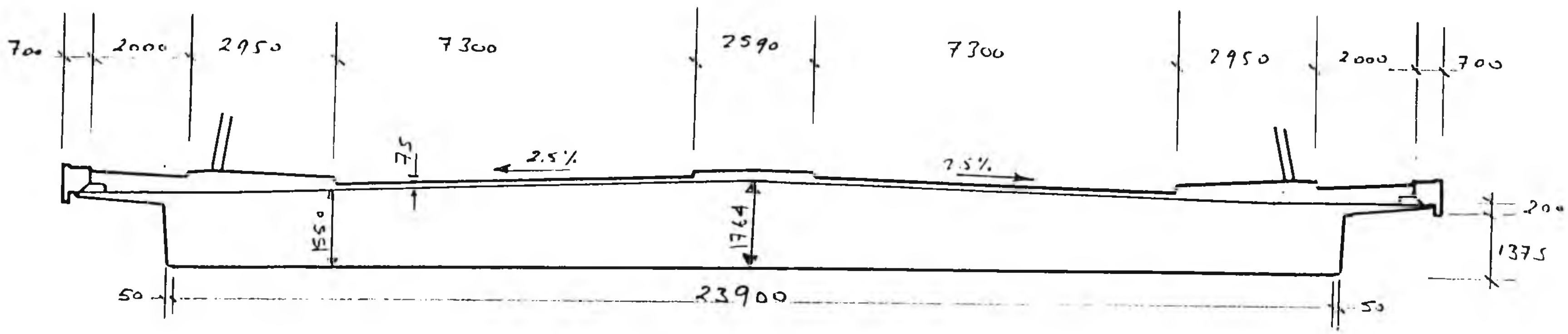
CABLE #	AREA (mm2)
8001	8100
8002	6450
8003	5550
8004	4950
8005	4950
8006	5550
8007	6450
8008	7950
8009	8250
8010	8250
8011	8250
8012	7950
8013	7950
8014	7950
8015	7950
8016	7950
8017	7950
8018	7950
8019	8100

CABLE #	AREA (mm2)
8020	6000
8021	4800
8022	4800
8023	4800
8024	4800
8025	5550
8026	5550
8027	6000
8028	6000
8029	6300
8030	6300
8031	6750
8032	6750
8033	7050
8034	7050
8035	7800
8036	7800
8037	8250
8038	8250

FIGURE A.2. STRUCTURE LONGITUDINAL CROSS SECTION.

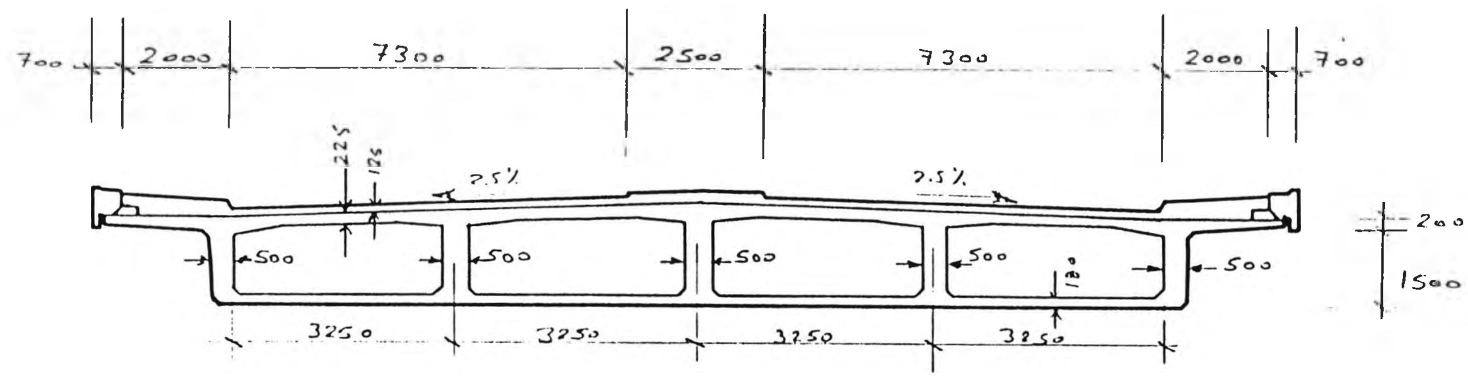


Main Span



Anchor Span

FIGURE A.3. STRUCTURE DECK CROSS SECTION (MAIN SPAN & ANCHOR SPAN).



Approach Span

FIGURE A.4. STRUCTURE DECK CROSS SECTION (APPROACH SPAN).

A P P E N D I X   B

EXHAUSTIVE SEARCH SOLUTION FOR MOMENT AT BRIDGE CENTRE LINE

MOMENT AND SHEAR INFLUENCE LINES AT JOINT 16
(BRIDGE CENTRE LINE)

JOINT	X(m)	MOMENT
1	0.000	+0.000000
2	13.333	+0.009719
3	26.666	-0.014506
4	40.000	+0.000000
5	53.333	-0.116784
6	66.667	-0.190429
7	80.000	-0.352064
8	90.000	-0.782916
9	100.000	-1.091670
10	113.333	-0.797062
11	126.666	-0.376777
12	140.000	+0.000000
13	153.333	+0.449080
14	166.667	+0.773845
15	180.000	+1.528131
16	190.000	+4.961754
17	200.000	+1.528159
18	213.333	+0.773877
19	226.666	+0.449251
20	240.000	+0.000000
21	253.333	-0.376591
22	266.667	-0.796302
23	280.000	-1.089996
24	290.000	-0.779831
25	300.000	-0.348194
26	313.333	-0.187901
27	326.666	-0.115545
28	340.000	+0.000000
29	353.333	-0.015387
30	366.667	+0.009041
31	380.000	+0.000000

1. HA UDL LOADING ANALYSIS FOR BENDING MOMENT

Zone	Length (m)	Cusp Factor	Marea (m2)
1	18.1483	1.0000	+0.1347
2	21.8517	1.0000	-0.1565
3	100.0000	1.0000	-44.6728
4	100.0000	0.4350	+107.9264
5	100.0000	1.0000	-44.5132
6	22.1557	1.0000	-0.1745
7	17.8443	1.0000	+0.1262

		MAXIMUM	MINIMUM
Loaded Length	(m)	43.5033	200.0000
Load Intensity	(kN/m)	26.8244	21.1933
Moment Area	(m2)	+107.9264	-89.1860
MOMENT	(mN-m)	+2.8951	-1.8901

2. FOOTPATH LOADING ANALYSIS FOR BENDING MOMENT

		MAXIMUM	MINIMUM
Loaded Length	(m)	43.5033	200.0000
Load Intensity	(kN/m)	4.2782	2.2546
Moment Area	(m2)	+107.9264	-89.1860
MOMENT	(mN-m)	+0.4617	-0.2011

3. HA KEL LOADING ANALYSIS FOR BENDING MOMENT

	MAXIMUM	MINIMUM
LOCATION (m)	+190.0000	+101.0026
MOMENT (mN-m)	+0.5954	-0.1314

4. HB VEHICLE LOADING ANALYSIS FOR BENDING MOMENT

	MAXIMUM	MINIMUM
LOCATION (m)	+193.0000	+101.4088
AXLE SPACING (m)	+6.0000	+6.0000
MOMENT (MN-m)	+6.1266	-1.8877

5. ASSOCIATED HA UDL ANALYSIS FOR BENDING MOMENT

Zone	Length (m)	Marea (m2)
1	18.1483	+0.1347
2	21.8517	-0.1565
3	100.0000	-44.6728
4	40.4000	+17.4297
5	100.0000	-44.5132
6	22.1557	-0.1745
7	17.8443	+0.1262

Zone	Length (m)	Marea (m2)
1	18.1483	+0.1347
2	21.8517	-0.1565
3	40.4000	-5.9225
4	100.0000	+107.9264
5	100.0000	-44.5132
6	22.1557	-0.1745
7	17.8443	+0.1262

		MAXIMUM	MINIMUM
Loaded Length	(m)	43.5033	200.0000
Load Intensity	(kN/m)	26.8244	21.1933
Moment Area	(m2)	+17.4297	-50.4357
MOMENT	(mN-m)	+0.4675	-1.0689

A P P E N D I X C1

DETAILED GENETIC ALGORRITHMS OUTPUT RUN

Popsize	=	10
Maxgen	=	20
Pcross	=	0.95
Pmut	=	0.01

----- Population Result @ Generation 0 -----

1)	[0101100]	2331.2871
2)	[0111110]	2205.8693
3)	[1110000]	3.0023
4)	[1011100]	2229.4951
5)	[1111100]	2211.7041
6)	[0111010]	2302.1775
7)	[1111110]	2195.1444
8)	[1111100]	2211.7041
9)	[1111000]	2310.0145
10)	[0111100]	2223.6092

A	L
0.00100	21.85170
107.92600	43.50000
0.00100	100.00000

Total A	=	107.92800
Total L	=	165.35170
Total C	=	21.60039

Generation Maximum = 2331.2871 @ Generation 0

----- Population Result @ Generation 1 -----

1)	[0010101]	2.6939
2)	[0101100]	2331.2871
3)	[0111110]	2205.8693
4)	[1101110]	2283.8536
5)	[0111100]	2223.6092
6)	[0101110]	2302.1775
7)	[0111100]	2223.6092
8)	[0111100]	2223.6092
9)	[0101100]	2331.2871
10)	[1101101]	2291.3492

A	L
0.00100	21.85170
107.92600	43.50000
0.00100	100.00000

Total A = 107.92800
Total L = 165.35170
Total W = 21.60039

Generation Maximum = 2331.2871 @ Generation 1

----- Population Result @ Generation 2 -----

1)	[0101000]	2558.0376
2)	[1101110]	2283.8536
3)	[0110100]	0.0629
4)	[0111110]	2205.8693
5)	[1101100]	2310.0145
6)	[0101101]	2310.2158
7)	[0111110]	2205.8693
8)	[0101100]	2331.2871
9)	[0101100]	2331.2871
10)	[0111000]	2331.2871

A	L
0.00100	21.85170
107.92600	43.50000

Total A = 107.92700
Total L = 65.35170
Total W = 23.70155

Generation Maximum = 2558.0376 @ Generation 2

----- Population Result @ Generation 3 -----

1)	[0101100]	2331.2871
2)	[0101000]	2558.0376
3)	[0101100]	2331.2871
4)	[1101101]	2291.3492
5)	[0101100]	2331.2871
6)	[0111000]	2331.2871
7)	[0111110]	2205.8693
8)	[0111110]	2205.8693
9)	[0101100]	2331.2871
10)	[0111110]	2205.8693

A	L
0.00100	21.85170
107.92600	43.50000

Total A = 107.92700
Total L = 65.35170
Total W = 23.70155

Generation Maximum = 2558.0376 @ Generation 3

----- Population Result @ Generation 4 -----

1)	[0001000]	2895.1932
2)	[0111111]	2195.1902
3)	[1101100]	2310.0145
4)	[0001101]	2339.7248
5)	[0111010]	2302.1775
6)	[0101110]	2302.1775
7)	[0101100]	2331.2871
8)	[0101100]	2331.2871
9)	[0101101]	2310.2158
10)	[1101000]	2499.2267

A	L
107.92600	43.50000

Total A = 107.92600
Total L = 43.50000
Total W = 26.82573

Generation Maximum = 2895.1932 @ Generation 4

----- Population Result @ Generation 5 -----

1)	[0001011]	2499.0301
2)	[0101100]	2331.2871
3)	[0101000]	2558.0376
4)	[0111010]	2302.1775
5)	[0101111]	2284.0118
6)	[0101101]	2310.2158
7)	[1001101]	2317.8031
8)	[0001100]	2364.5441
9)	[0101100]	2331.2871
10)	[1101101]	2291.3492

A	L
0.00100	21.85170
107.92600	43.50000

Total A = 107.92700
Total L = 65.35170
Total W = 23.70155

Generation Maximum = 2558.0376 @ Generation 5

----- Population Result @ Generation 6 -----

1)	[0001101]	2339.7248
2)	[1111101]	2200.8119
3)	[0001100]	2364.5441
4)	[0101100]	2331.2871
5)	[1101100]	2310.0145
6)	[0101101]	2310.2158
7)	[0101101]	2310.2158
8)	[1001101]	2317.8031
9)	[0011000]	2364.5441
10)	[0101100]	2331.2871

A	L
0.00100	100.00000
107.92600	43.50000

Total A = 107.92700
Total L = 143.50000
Total W = 21.90874

Generation Maximum = 2364.5441 @ Generation 6

----- Population Result @ Generation 7 -----

1)	[0011000]	2364.5441
2)	[0001101]	2339.7248
3)	[0101101]	2310.2158
4)	[0101100]	2331.2871
5)	[1101101]	2291.3492
6)	[0111101]	2211.7674
7)	[0101100]	2331.2871
8)	[0001100]	2364.5441
9)	[0011000]	2364.5441
10)	[0101101]	2310.2158

A	L
0.00100	100.00000
107.92600	43.50000

Total A = 107.92700
Total L = 143.50000
Total W = 21.90874

Generation Maximum = 2364.5441 @ Generation 7

----- Population Result @ Generation 8 -----

1)	[1101101]	2291.3492
2)	[0001101]	2339.7248
3)	[0101100]	2331.2871
4)	[0011001]	2339.7248
5)	[0111101]	2211.7674
6)	[0101101]	2310.2158
7)	[0110100]	0.0629
8)	[0111101]	2211.7674
9)	[0101101]	2310.2158
10)	[0001100]	2364.5441

A	L
107.92600	43.50000
0.00100	100.00000

Total A = 107.92700
Total L = 143.50000
Total W = 21.90874

Generation Maximum = 2364.5441 @ Generation 8

----- Population Result @ Generation 9 -----

1)	[1101101]	2291.3492
2)	[0110101]	2.6891
3)	[0100101]	2.8163
4)	[0111100]	2223.6092
5)	[0111101]	2211.7674
6)	[0111101]	2211.7674
7)	[0001101]	2339.7248
8)	[0101100]	2331.2871
9)	[0011100]	2242.7803
10)	[0101101]	2310.2158

A	L
107.92600	43.50000
0.00100	100.00000
0.12620	17.84430

Total A = 108.05320
Total L = 161.34430
Total W = 21.65345

Generation Maximum = 2339.7248 @ Generation 9

----- Population Result @ Generation 10 -----

1)	[0101101]	2310.2158
2)	[1101001]	2454.1509
3)	[0111110]	2205.8693
4)	[0101110]	2302.1775
5)	[0011001]	2339.7248
6)	[0001100]	2364.5441
7)	[1101101]	2291.3492
8)	[0001101]	2339.7248
9)	[0111101]	2211.7674
10)	[0011001]	2339.7248

A	L
0.13470	18.14830
0.00100	21.85170
107.92600	43.50000
0.12620	17.84430

Total A = 108.18790
Total L = 101.34430
Total W = 22.68415

Generation Maximum = 2454.1509 @ Generation 10

----- Population Result @ Generation 11 -----

1)	[0101100]	2331.2871
2)	[0011000]	2364.5441
3)	[0011110]	2223.3546
4)	[0111001]	2310.2158
5)	[0001100]	2364.5441
6)	[0001101]	2339.7248
7)	[1101101]	2291.3492
8)	[0101001]	2499.9417
9)	[0011101]	2229.5789
10)	[0110001]	2.8163

A	L
0.00100	21.85170
107.92600	43.50000
0.12620	17.84430

Total A = 108.05320
Total L = 83.19600
Total W = 23.13621

Generation Maximum = 2499.9417 @ Generation 11

----- Population Result @ Generation 12 -----

1)	[1101101]	2291.3492
2)	[0001101]	2339.7248
3)	[0101001]	2499.9417
4)	[0001001]	2577.2627
5)	[1101110]	2283.8536
6)	[0010101]	2.6939
7)	[0101100]	2331.2871
8)	[0001100]	2364.5441
9)	[1101110]	2283.8536
10)	[0011101]	2229.5789

A	L
107.92600	43.50000
0.12620	17.84430

Total A = 108.05220
Total L = 61.34430
Total W = 23.85202

Generation Maximum = 2577.2627 @ Generation 12

----- Population Result @ Generation 13 -----

1)	[0101000]	2558.0376
2)	[0101101]	2310.2158
3)	[0001101]	2339.7248
4)	[0011101]	2229.5789
5)	[0101101]	2310.2158
6)	[0101000]	2558.0376
7)	[0001100]	2364.5441
8)	[0101100]	2331.2871
9)	[0101001]	2499.9417
10)	[0001101]	2339.7248

A	L
0.00100	21.85170
107.92600	43.50000

Total A = 107.92700
Total L = 65.35170
Total W = 23.70155

Generation Maximum = 2558.0376 @ Generation 13

----- Population Result @ Generation 14 -----

1)	[0001101]	2339.7248
2)	[0011101]	2229.5789
3)	[0001100]	2364.5441
4)	[0101001]	2499.9417
5)	[0101101]	2310.2158
6)	[0101000]	2558.0376
7)	[0001101]	2339.7248
8)	[0101101]	2310.2158
9)	[0101001]	2499.9417
10)	[0011100]	2242.7803

A	L
0.00100	21.85170
107.92600	43.50000

Total A = 107.92700
Total L = 65.35170
Total W = 23.70155

Generation Maximum = 2558.0376 @ Generation 14

----- Population Result @ Generation 15 -----

1)	[0001000]	2895.1932
2)	[0101001]	2499.9417
3)	[0101001]	2499.9417
4)	[0101001]	2499.9417
5)	[0001101]	2339.7248
6)	[0001101]	2339.7248
7)	[0110001]	2.8163
8)	[0001100]	2364.5441
9)	[0111101]	2211.7674
10)	[0001001]	2577.2627

A	L
107.92600	43.50000

Total A = 107.92600
Total L = 43.50000
Total W = 26.82573

Generation Maximum = 2895.1932 @ Generation 15

----- Population Result @ Generation 16 -----

1)	[0101001]	2499.9417
2)	[0101001]	2499.9417
3)	[0101001]	2499.9417
4)	[0101101]	2310.2158
5)	[0111101]	2211.7674
6)	[0111101]	2211.7674
7)	[0101001]	2499.9417
8)	[0100001]	3.6280
9)	[0001000]	2895.1932
10)	[0101100]	2331.2871

A	L
107.92600	43.50000

Total A = 107.92600
Total L = 43.50000
Total W = 26.82573

Generation Maximum = 2895.1932 @ Generation 16

----- Population Result @ Generation 17 -----

1)	[0111000]	2331.2871
2)	[0111101]	2211.7674
3)	[0101011]	2441.6308
4)	[0101001]	2499.9417
5)	[0101101]	2310.2158
6)	[0101001]	2499.9417
7)	[0101100]	2331.2871
8)	[0101001]	2499.9417
9)	[1111100]	2211.7041
10)	[0001000]	2895.1932

A	L
107.92600	43.50000

Total A = 107.92600
Total L = 43.50000
Total W = 26.82573

Generation Maximum = 2895.1932 @ Generation 17

----- Population Result @ Generation 18 -----

1)	[0101000]	2558.0376
2)	[0011001]	2339.7248
3)	[0001000]	2895.1932
4)	[0001001]	2577.2627
5)	[0111001]	2310.2158
6)	[0101101]	2310.2158
7)	[0001000]	2895.1932
8)	[0111100]	2223.6092
9)	[0101100]	2331.2871
10)	[1101100]	2310.0145

A	L
107.92600	43.50000

Total A = 107.92600
Total L = 43.50000
Total W = 26.82573

Generation Maximum = 2895.1932 @ Generation 18

----- Population Result @ Generation 19 -----

1)	[0001000]	2895.1932
2)	[0001001]	2577.2627
3)	[0101000]	2558.0376
4)	[0001001]	2577.2627
5)	[0011001]	2339.7248
6)	[0111100]	2223.6092
7)	[1101101]	2291.3492
8)	[1101100]	2310.0145
9)	[0001000]	2895.1932
10)	[0101100]	2331.2871

A	L
107.92600	43.50000

Total A = 107.92600
Total L = 43.50000
Total W = 26.82573

Generation Maximum = 2895.1932 @ Generation 19

----- Population Result @ Generation 20 -----

1)	[0001001]	2577.2627
2)	[1101100]	2310.0145
3)	[0101001]	2499.9417
4)	[0001100]	2364.5441
5)	[0011001]	2339.7248
6)	[0011001]	2339.7248
7)	[0101000]	2558.0376
8)	[1001000]	2576.1916
9)	[0001100]	2364.5441
10)	[1101000]	2499.2267

A	L
107.92600	43.50000
0.12620	17.84430

Total A = 108.05220
Total L = 61.34430
Total W = 23.85202

Generation Maximum = 2577.2627 @ Generation 20

The Best Solution So Far has been found @ Generation 4 :

```
[0001000] --------> FITNESS =    2895.1932

              A                      L

         107.92600             43.50000

Total A =                 107.92600
Total L =                  43.50000
Total W =                  26.82573
```

A P P E N D I X C2

DETAILED GENETIC ALGORRITHMS OUTPUT RUN
WITH CROSSOVER OPERATOR DISABLED

* Random Initial Population
* Popsize = 10
* Maxgen = 20
* Pcross = 0.00 ... One Point Crossover!
* Pmutation = 0.01

----- Population Result @ Generation 0 -----

	String	Fitness	Total A	Total L	W
1)	[0111100]	2223.6092	107.929	265.352	20.603
2)	[0111101]	2211.7674	108.055	283.196	20.469
3)	[0111101]	2211.7674	108.055	283.196	20.469
4)	[0111101]	2211.7674	108.055	283.196	20.469
5)	[0111100]	2223.6092	107.929	265.352	20.603
6)	[0001111]	2309.8328	108.054	183.500	21.377
7)	[0111101]	2211.7674	108.055	283.196	20.469
8)	[0111101]	2211.7674	108.055	283.196	20.469
9)	[0111100]	2223.6092	107.929	265.352	20.603
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2253.9
Generation Sum = 22538.5
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 1 -----

	String	Fitness	Total A	Total L	W
1)	[0111101]	2211.7674	108.055	283.196	20.469
2)	[0111100]	2223.6092	107.929	265.352	20.603
3)	[0111101]	2211.7674	108.055	283.196	20.469
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0111101]	2211.7674	108.055	283.196	20.469
6)	[0111101]	2211.7674	108.055	283.196	20.469
7)	[0111100]	2223.6092	107.929	265.352	20.603
8)	[0111100]	2223.6092	107.929	265.352	20.603
9)	[0111100]	2223.6092	107.929	265.352	20.603
10)	[0001111]	2309.8328	108.054	183.500	21.377

Generation Maximum = 2499.0
Generation Average = 2255.0
Generation Sum = 22550.4
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 2 -----

	String	Fitness	Total A	Total L	W
1)	[0101111]	2284.0118	108.055	205.352	21.137
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0111101]	2211.7674	108.055	283.196	20.469
4)	[0111101]	2211.7674	108.055	283.196	20.469
5)	[0111100]	2223.6092	107.929	265.352	20.603
6)	[0111100]	2223.6092	107.929	265.352	20.603
7)	[0111101]	2211.7674	108.055	283.196	20.469
8)	[0111101]	2211.7674	108.055	283.196	20.469
9)	[0111101]	2211.7674	108.055	283.196	20.469
10)	[0111100]	2223.6092	107.929	265.352	20.603

Generation Maximum = 2499.0
Generation Average = 2251.3
Generation Sum = 22512.7
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 3 -----

	String	Fitness	Total A	Total L	W
1)	[0111101]	2211.7674	108.055	283.196	20.469
2)	[0111101]	2211.7674	108.055	283.196	20.469
3)	[0111100]	2223.6092	107.929	265.352	20.603
4)	[0111101]	2211.7674	108.055	283.196	20.469
5)	[0111101]	2211.7674	108.055	283.196	20.469
6)	[0111100]	2223.6092	107.929	265.352	20.603
7)	[0111100]	2223.6092	107.929	265.352	20.603
8)	[0000011]	3.6095	0.127	40.000	28.377
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0111101]	2211.7674	108.055	283.196	20.469

Generation Maximum = 2499.0
Generation Average = 2023.2
Generation Sum = 20232.3
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 4 -----

	String	Fitness	Total A	Total L	W
1)	[0111101]	2211.7674	108.055	283.196	20.469
2)	[0111100]	2223.6092	107.929	265.352	20.603
3)	[0111101]	2211.7674	108.055	283.196	20.469
4)	[0111100]	2223.6092	107.929	265.352	20.603
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0111101]	2211.7674	108.055	283.196	20.469
7)	[0111101]	2211.7674	108.055	283.196	20.469
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0111101]	2211.7674	108.055	283.196	20.469
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2300.3
Generation Sum = 23003.1
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 5 -----

	String	Fitness	Total A	Total L	W
1)	[0111101]	2211.7674	108.055	283.196	20.469
2)	[0111101]	2211.7674	108.055	283.196	20.469
3)	[0001011]	2499.0301	108.053	83.500	23.128
4)	[0111101]	2211.7674	108.055	283.196	20.469
5)	[1111100]	2211.7041	108.064	283.500	20.467
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0111101]	2211.7674	108.055	283.196	20.469
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0111101]	2211.7674	108.055	283.196	20.469

Generation Maximum = 2499.0
Generation Average = 2326.7
Generation Sum = 23266.7
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 6 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0111101]	2211.7674	108.055	283.196	20.469
3)	[0001011]	2499.0301	108.053	83.500	23.128
4)	[0111101]	2211.7674	108.055	283.196	20.469
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0111101]	2211.7674	108.055	283.196	20.469
9)	[0111101]	2211.7674	108.055	283.196	20.469
10)	[1001011]	2453.4159	108.188	101.648	22.677

Generation Maximum = 2499.0
Generation Average = 2379.6
Generation Sum = 23795.6
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 7 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0111101]	2211.7674	108.055	283.196	20.469
4)	[1001011]	2453.4159	108.188	101.648	22.677
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0110101]	2.6891	0.129	239.696	20.813

Generation Maximum = 2499.0
Generation Average = 2216.1
Generation Sum = 22161.1
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 8 -----

	String	Fitness	Total A	Total L	W
1)	[1001011]	2453.4159	108.188	101.648	22.677
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0001011]	2499.0301	108.053	83.500	23.128
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[1001011]	2453.4159	108.188	101.648	22.677
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2489.9
Generation Sum = 24899.1
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 9 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[1001011]	2453.4159	108.188	101.648	22.677
3)	[0001010]	2556.8507	107.927	65.656	23.691
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[1001011]	2453.4159	108.188	101.648	22.677
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2556.9
Generation Average = 2495.7
Generation Sum = 24956.9
Generation Best String = [0001010]

A	L
107.9	43.5
0.0	22.2

Total A = 107.9
Total L = 65.7
Total W = 23.7

----- Population Result @ Generation 10 -----

	String	Fitness	Total A	Total L	W
1)	[0011010]	2330.8589	107.928	165.656	21.596
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[1001011]	2453.4159	108.188	101.648	22.677
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2477.7
Generation Sum = 24776.5
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 11 -----

	String	Fitness	Total A	Total L	W
1)	[1001011]	2453.4159	108.188	101.648	22.677
2)	[0011010]	2330.8589	107.928	165.656	21.596
3)	[0011010]	2330.8589	107.928	165.656	21.596
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0011010]	2330.8589	107.928	165.656	21.596
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2444.0
Generation Sum = 24440.2
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 12 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0011110]	2223.3546	107.929	265.656	20.600
4)	[0011010]	2330.8589	107.928	165.656	21.596
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0011010]	2330.8589	107.928	165.656	21.596
10)	[0011010]	2330.8589	107.928	165.656	21.596

Generation Maximum = 2499.0
Generation Average = 2421.0
Generation Sum = 24210.1
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 13 -----

	String	Fitness	Total A	Total L	W
1)	[0011110]	2223.3546	107.929	265.656	20.600
2)	[0011110]	2223.3546	107.929	265.656	20.600
3)	[0001111]	2309.8328	108.054	183.500	21.377
4)	[0011010]	2330.8589	107.928	165.656	21.596
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0011110]	2223.3546	107.929	265.656	20.600
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2380.6
Generation Sum = 23805.9
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 14 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0001011]	2499.0301	108.053	83.500	23.128
4)	[0011110]	2223.3546	107.929	265.656	20.600
5)	[0011110]	2223.3546	107.929	265.656	20.600
6)	[0011010]	2330.8589	107.928	165.656	21.596
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001111]	2309.8328	108.054	183.500	21.377
9)	[0011110]	2223.3546	107.929	265.656	20.600
10)	[0001011]	2499.0301	108.053	83.500	23.128

```
Generation Maximum      =        2499.0
Generation Average      =        2380.6
Generation Sum          =       23805.9
Generation Best String =      [0001011]

        A        L

      107.9     43.5
        0.0     22.2
        0.1     17.8


Total A  =   108.1
Total L  =    83.5
Total W  =    23.1
```

----- Population Result @ Generation 15 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0001011]	2499.0301	108.053	83.500	23.128
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0011010]	2330.8589	107.928	165.656	21.596
6)	[0011110]	2223.3546	107.929	265.656	20.600
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0001111]	2309.8328	108.054	183.500	21.377
10)	[0001011]	2499.0301	108.053	83.500	23.128

```
Generation Maximum      =        2499.0
Generation Average      =        2435.7
Generation Sum          =       24357.3
Generation Best String =      [0001011]

        A        L

      107.9     43.5
        0.0     22.2
        0.1     17.8


Total A  =   108.1
Total L  =    83.5
Total W  =    23.1
```

----- Population Result @ Generation 16 -----

	String	Fitness	Total A	Total L	W
1)	[0101011]	2441.6308	108.054	105.352	22.596
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0001111]	2309.8328	108.054	183.500	21.377
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0001011]	2499.0301	108.053	83.500	23.128
6)	[0011010]	2330.8589	107.928	165.656	21.596
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0011010]	2330.8589	107.928	165.656	21.596
9)	[1001011]	2453.4159	108.188	101.648	22.677
10)	[0001011]	2499.0301	108.053	83.500	23.128

Generation Maximum = 2499.0
Generation Average = 2436.2
Generation Sum = 24361.7
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 17 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0011010]	2330.8589	107.928	165.656	21.596
4)	[1001011]	2453.4159	108.188	101.648	22.677
5)	[1001011]	2453.4159	108.188	101.648	22.677
6)	[0101011]	2441.6308	108.054	105.352	22.596
7)	[0011010]	2330.8589	107.928	165.656	21.596
8)	[0011010]	2330.8589	107.928	165.656	21.596
9)	[0001011]	2499.0301	108.053	83.500	23.128
10)	[0001011]	2499.0301	108.053	83.500	23.128

```
Generation Maximum      =          2499.0
Generation Average      =          2433.7
Generation Sum          =         24337.2
Generation Best String  =        [0001011]
```

A	L
107.9	43.5
0.0	22.2
0.1	17.8

```
Total A  =   108.1
Total L  =    83.5
Total W  =    23.1
```

----- Population Result @ Generation 18 -----

	String	Fitness	Total A	Total L	W
1)	[0011010]	2330.8589	107.928	165.656	21.596
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0011010]	2330.8589	107.928	165.656	21.596
4)	[0001011]	2499.0301	108.053	83.500	23.128
5)	[0011010]	2330.8589	107.928	165.656	21.596
6)	[0101011]	2441.6308	108.054	105.352	22.596
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0001011]	2499.0301	108.053	83.500	23.128
9)	[0011010]	2330.8589	107.928	165.656	21.596
10)	[0011010]	2330.8589	107.928	165.656	21.596

```
Generation Maximum      =          2499.0
Generation Average      =          2409.2
Generation Sum          =         24092.0
Generation Best String  =        [0001011]
```

A	L
107.9	43.5
0.0	22.2
0.1	17.8

```
Total A  =   108.1
Total L  =    83.5
Total W  =    23.1
```

----- Population Result @ Generation 19 -----

	String	Fitness	Total A	Total L	W
1)	[0011010]	2330.8589	107.928	165.656	21.596
2)	[0001011]	2499.0301	108.053	83.500	23.128
3)	[0011010]	2330.8589	107.928	165.656	21.596
4)	[0011010]	2330.8589	107.928	165.656	21.596
5)	[0011010]	2330.8589	107.928	165.656	21.596
6)	[0001011]	2499.0301	108.053	83.500	23.128
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0011010]	2330.8589	107.928	165.656	21.596
9)	[0011010]	2330.8589	107.928	165.656	21.596
10)	[0101011]	2441.6308	108.054	105.352	22.596

Generation Maximum = 2499.0
Generation Average = 2392.4
Generation Sum = 23923.9
Generation Best String = [0001011]

A	L
107.9	43.5
0.0	22.2
0.1	17.8

Total A = 108.1
Total L = 83.5
Total W = 23.1

----- Population Result @ Generation 20 -----

	String	Fitness	Total A	Total L	W
1)	[0001011]	2499.0301	108.053	83.500	23.128
2)	[0011010]	2330.8589	107.928	165.656	21.596
3)	[0011010]	2330.8589	107.928	165.656	21.596
4)	[0011010]	2330.8589	107.928	165.656	21.596
5)	[0011010]	2330.8589	107.928	165.656	21.596
6)	[0011010]	2330.8589	107.928	165.656	21.596
7)	[0001011]	2499.0301	108.053	83.500	23.128
8)	[0101011]	2441.6308	108.054	105.352	22.596
9)	[0011010]	2330.8589	107.928	165.656	21.596
10)	[0011010]	2330.8589	107.928	165.656	21.596

```
Generation Maximum      =          2499.0
Generation Average      =          2375.6
Generation Sum          =         23755.7
Generation Best String  =         [0001011]

        A         L

      107.9     43.5
        0.0     22.2
        0.1     17.8


Total A  =   108.1
Total L  =    83.5
Total W  =    23.1


The Best Solution So Far has been found  @ Generation  9:


[0001010]  -------->  FITNESS  =          2556.9

        A         L

      107.9     43.5
        0.0     22.2


Total A  =   107.9
Total L  =    65.7
Total W  =    23.7
```

A P P E N D I X   D

LOADING FAMILY APPLICABLE TO CABLE-STAYED BRIDGES

D.1- INTRODUCTION

Loads on cable stayed bridges can be defined as standard and non-standard loadings. Standard loadings include all possible loadings specified by a code of practice. In this thesis reference is made to the British Highway code of practice BS5400 in its latest loading directive BD 37/88 (1989). Non-standard loadings include loadings which are not covered by the code of practice such as; cable out conditions, dynamic wind effect, ship impact loads, etc.

In this Appendix standard and non-standard loadings are briefly described.

D.2- STANDARD LOADINGS

Standard loadings as defined in BD 37/88 are of two types; namely, permanent and transient loadings.

D.2.1- PERMANENT LOADS

Permanent loads may include

Dead loads,
Superimposed dead load,
Differential settlement,
Creep and shrinkage.

DEAD LOAD

Dead load calculation should be based on the geometry of the structure. Depending on the degree of reinforcement within the member under consideration, the nominal unit weight of mass, reinforced and internally prestressed concrete is shown in table D.1.

Table D.1

Percentage of steel, by volume, per m^3	0%	2%	4%	6%	8%	10%
Nominal unit weight kN/m^3	24.4	25.4	26.5	27.5	28.6	29.6

The unit weight of structural steelwork is usually taken as 77 kN/m^3. In the preliminary design stage the unit mass of various sizes of stay cable may be taken as follows:

37 x 15.2 mm Superstrand : 57 kg/m
44 x 15.2 mm Superstrand : 69.2 kg/m
51 x 15.2 mm Superstrand : 82 kg/m

These values should be reviewed during the design once a stay system has been selected.

SUPERIMPOSED DEAD LOADS (SDL)

SDL-Deck surfacing comprises footway, cycleway and carriageway surfacing and verge/median strip plain concrete. The waterproof membrane is usually considered as carriageway surfacing in calculating loads and no deductions are usually made for ducts within the verges or median strip. All other SDL are defined as other loads and these may include: concrete ballast, wind shielding and parapets, lighting columns, safety fences, compressed air line, water mains (charged), sign and signal gantries and gantry bases, access railway track and support steelwork if there is any, maintenance gantry track and support steelwork, drainage piping (fully charged), HV and LV electrical cables and racking and any other deck furniture.

DIFFERENTIAL SETTLEMENT

BD 37/88 does not give much guidance on differential settlement and leaves the decision to the designer. Depending on the geotechnical characteristics of the soil, the following approach has been used on the Second Severn Crossing (Design statement, 1991) where five different scenarios were considered (Fig. D.1):

Case 1. Differential settlement between any two piers equals to 25mm.

Case 2. Differential settlement between any pylon and adjacent pier or pylon equals to 25mm.

Case 3. Differential settlement between abutments and any pier equals to 25mm.

Case 4. Differential settlement between two foundations (where present) at any one pier equals 10mm.

Case 5. Differential settlement between diagonally opposite pier columns at any three adjacent piers equals to 5mm.

CREEP AND SHRINKAGE

Creep and shrinkage should be taken into account where deflections are important and in the design of the articulation of the bridge. BS5400 (part 4, 1990) specifies the method of calculation for deformations associated with creep and shrinkage.

D.2.2- TRANSIENT LOADS

BD 37/88 defines transient loads as any loads other than the permanent loads referred to above: these consist of

Static effect of wind loads,
Temperature loads,
Erection loads,
Primary and secondary highway loadings,
Footway and cycle track loadings.

WIND LOAD

The static application of wind loading with respect to BD 37/88 is complicated and requires engineering judgement. The calculation procedure can be summarized as follow:

The mean hourly wind speed is first obtained. The maximum and the minimum gust speeds for the cases of with or without live load acting on the bridge are then calculated. The gust speed are obtained by multiplying the mean hourly speed by a number of factors. After determining the vertical, longitudinal and transverse wind they should be combined as follows:

1. Transverse alone

2. Transverse ± vertical

3. Longitudinal alone

4. 50% transverse ± longitudinal ± 50% vertical

TEMPERATURE

BD 37/88 considers two aspects of temperature loading, namely, the restraint to the overall bridge movement due to temperature range, and the effects of temperature differences through the depth of the bridge.

Temperature range for a particular bridge is obtained by first determining the maximum and minimum shade air temperature, for the location of the bridge, from isotherms plotted on maps for the British Isles. An adjustment should also be carried out for the height above mean sea level. The minimum and maximum effective temperature of the bridge can then be obtained from tables which relate shade air temperature to effective bridge temperature. The latter can be thought of as the temperature which controls the overall longitudinal expansion or contraction of the bridge.

Due to the variations in solar radiation and the relatively small thermal conductivity of concrete, severe nonlinear temperature differences occur through the depth of a bridge. The positive temperature differences represents the heat gain through the top surface, while the negative temperature differences represents the heat loss from the top surface.

ERECTION LOAD

Loading due to erection methods, sequence and equipment should be determined from the construction methods defined for the bridge.

PRIMARY AND SECONDARY HIGHWAY LOADINGS

Primary highway loading are vertical live loads, whereas the secondary highway loadings are the live loads due to changes in speed or direction of the primary live load.

BD 37/88 classifies primary live loads into two groups: namely, HA and HB loading. HA loading is a formula loading which is intended to represent normal actual vehicle loading. The HA loading consists of either a uniformly distributed load plus a knife edge load or a single wheel load. HB loading is intended to represent an abnormally heavy vehicle being carried on 4 axles. The magnitude of HB loading is expressed as a number of units, each unit causes a load of 10 kN on each axle; the number of units depend on the type of road.

While secondary live loadings are defined in BD 37/88 as loads parallel or transverse to the carriageway due to change in speed or direction of the traffic, namely, centrifugal load, longitudinal braking, skidding, collision with parapets, collision with supports and fatigue and dynamic loading.

FOOTWAY OR CYCLE TRACK LOADING

The footway or cycleway loading in highway bridges is a uniformly distributed load of the value Kx5.0. Where K is constant less than unity and depends: on the value of the UDL and the loaded length of HA loading, the width of the footway/cycleway, and the number of notional lanes.

D.3- NON-STANDARD LOADING

Non-standard loading covers aspect of loadings which are not covered by the code of practice. In Britain, for instance, the Department of Transport requires their cable stayed crossing to be functional under cable(s) out conditions. Another set of non-standard loads may include prestress cable loads, seismic load, dynamic wind loads, ship impact loading, loads due to extreme still water levels etc.

CABLE OUT CONDITIONS

Cable out conditions mean the functionality of the cable stayed bridge with one or more of its cables missing. Two categories of stay removal are associated with cable out conditions. The first is the controlled removal of, usually, one cable, while the second is known as the accidental removal of, usually one or more, cable(s). Chapter 8 is devoted for cables out conditions.

CABLE PRESTRESS LOADS

The load effects in the cables in the permanent load condition are controlled by introducing a deliberate lack of fit to the structure. The load effect due to the deliberate lack of fit is defined as that component of the total permanent load effect in any cable which is not caused by the dead load and superimposed dead load.

SEISMIC LOADING

In the UK, seismic forces may be calculated as 0.06g of the component masses of the structure affected. They may be applied as horizontal static forces in any direction at the level appropriate to the structural and permanent superimposed mass above the section considered. Seismic forces are considered as coexistent with dead load and superimposed dead load including creep and shrinkage, differential settlement, prestress loads, and with stream flow loads but not with live, temperature, wind or shipping impact loads. Bearings, movement joints and structural elements should be designed to accommodate movements resulting from the application of the seismic forces.

DYNAMIC WIND LOAD

Cable stayed bridges are prone to several forms of aerodynamic excitation which may result in motions in isolated vertical bending or torsional modes or, more rarely, in coupled vertical bending-torsional modes.

SHIP IMPACT LOADING

When ship impact loading is considered the pier and pylon bases are designed to resist direct ship impact forces. Ship impact load are considered as dynamic loads, coexistent with dead and superimposed dead loads and stream flow loads.

D.4- Load Combinations

In BD 37/88 there are three principal and two secondary combinations of loads.

D.4.1- Combination 1

The loads to be considered are the permanent loads plus the appropriate primary live loads for highway and footway or cycle track bridges.

D.4.2- Combination 2

The loads to be considered are those of combination 1 plus wind loading plus erection loads when appropriate.

D.4.3- Combination 3

The loads to be considered are those of combination 1 plus those arising from restrained of movements, due to temperature range and differential temperature distributions, plus erection loads when appropriate.

D.4.4- Combination 4

The loads to be considered for highway bridges are the permanent loads plus a secondary live load with its associated primary live load.

The loads to be considered for footway or cycle track bridges are the permanent loads plus the secondary live load of a vehicle colliding with a support.

D.4.5- Combination 5

The loads to be considered are permanent loads plus the loads due to friction at the bearings.

D.6- Partial safety factors

The values of the partial safety factor γ_{f1} to be applied at the ultimate limit and serviceability limit states for the various load combinations are given in table D.1. The individual values are not discussed at this juncture, but the following general points should be noted:

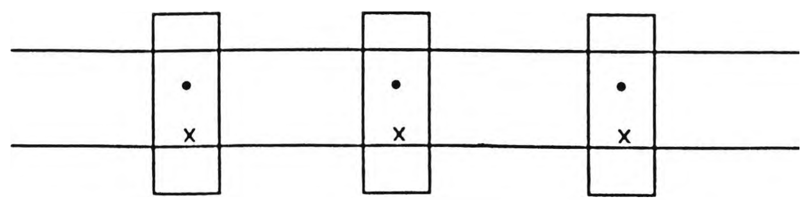
1- Larger values are specified for the ultimate than for the serviceability limit state.

2- The values less for reasonably well defined loads, such as dead load, than for more variable loads, such as live or superimposed dead load. Hence the greater uncertainty associated with latter loads is reflected in the values of the partial safety factors.

3- The value for a live load, such as HA load, is less when the load is combined with other loads, such as wind load in combination 2 or temperature loading in combination 3, than when it acts alone, as in combination 1. This is because of the reduced probability that a number of loads acting together will all their nominal values simultaneously. This fact is allowed for by the partial safety factor γ_{f2} which is a component of γ_{f1}.

4- A value of unity is specified for certain loads when this would result in a more sever effect.

5- The values for dead and superimposed dead load at the ultimate limit state can be different to the tabulated values, as is discussed later when these loads are considered in more detail.



• = pier column, no settlement
x = pier column, 5mm settlement

FIGURE D.1. PROPPOSED DIFFERENTIAL SETTLEMENT, SECOND SEVERN CROSSING BRIDGE, UK.

TABLE D.1. LOADS TO BE TAKEN IN EACH COMBINATION WITH APPROPRIATE γ_{FL} (BD 37/88, 1989)

Clause number	Load	Limit state	γ_{fL} to be considered in combination 1	2	3	4	5
5.1	Dead: steel	ULS*	1.05	1.05	1.05	1.05	1.05
		SLS	1.00	1.00	1.00	1.00	1.00
	concrete	ULS*	1.15	1.15	1.15	1.15	1.15
		SLS	1.00	1.00	1.00	1.00	1.00
5.2	Superimposed dead: deck surfacing	ULS+	1.75	1.75	1.75	1.75	1.75
		SLS+	1.20	1.20	1.20	1.20	1.20
	other loads	ULS	1.20	1.20	1.20	1.20	1.20
		SLS	1.00	1.00	1.00	1.00	1.00
5.1.2.2 & 5.2.2.2	Reduced load factor for dead & superimposed dead load where this has a more severe total effect	ULS	1.00	1.00	1.00	1.00	1.00
5.3	Wind: during erection	ULS		1.10			
		SLS		1.00			
	with dead plus superimposed dead load only, & for members primarily resisting wind loads	ULS		1.40			
		SLS		1.00			
	with dead plus superimposed dead plus other appropriate combination 2 loads	ULS		1.10			
		SLS		1.00			
	relieving effect of wind	ULS		1.00			
		SLS		1.00			
5.4	Temperature: restraint to movement, except frictional	ULS			1.30		
		SLS			1.00		
	frictional bearing restraint	ULS					1.30
		SLS					1.00
	effect of temperature difference	ULS			1.00		
		SLS			0.80		
5.6	Differential settlement	ULS	1.20	1.20	1.20	1.20	1.20
		SLS	1.00	1.00	1.00	1.00	1.00
5.7	Exceptional loads		to be assessed and agreed between the engineer & the appropriate authority				
5.8	Earth pressure: retained fill and/or live load — vertical loads	ULS	1.20	1.20	1.20	1.20	1.20
		SLS	1.00	1.00	1.00	1.00	1.00
	non-vertical loads	ULS	1.50	1.50	1.50	1.50	1.50
		SLS	1.00	1.00	1.00	1.00	1.00
	relieving effect	ULS	1.00	1.00	1.00	1.00	1.00
5.9	Erection: temporary loads	ULS		1.15	1.15		
		SLS		1.00	1.00		
6.2	Highway bridges live loading: HA alone	ULS	1.50	1.25	1.25		
		SLS	1.20	1.00	1.00		
6.3	HA with HB or HB alone	ULS	1.30	1.10	1.10		
		SLS	1.10	1.00	1.00		
6.5	footway & cycle track loading	ULS	1.50	1.25	1.25		
		SLS	1.00	1.00	1.00		
6.6	accidental wheel loading **	ULS	1.50				
		SLS	1.20				

A P P E N D I X   E

PERMUTATIONS AND COMBINATIONS

Understanding the fundamental mathematics of Genetic Algorithms is not difficult but it does require a solid grounding in finite sets, combinatorial counting, and elementary probability. In appendix the counting principles, permutations, and combinations are briefly discussed.

E.1- Counting

The ability to count exact quantities of patterns, classifications, or distinct grouping is an abstract art form that falls under the heading of **Combinatoris** or **Combinatorial analysis.** Most of the results of combinatorial analysis derive from a simple fact, the so-called **Counting Principle:**

> With two experiments M (with m outcomes) and N (with n outcomes), there are m.n total possible outcomes of the compound experiment MN.

E.2- Permutations

A permutation is an ordered arrangement of a set of different items. For example, consider the six arrangements of the three letters A, B, and C which are enumerated as follows:

ABC, ACB, BAC, BCA, CAB, CBA.

More generally, to count the permutations of n unique items, we start with n options for our choice of the first object and lose one degree of freedom after each succeeding choice. Therefore, by the counting principle the total number of permutations of n objects is counted as follows:

$$\text{Number of permutations of } n \text{ objects} = n(n-1)(n-2)..2.1 = n!$$

In general, there are n! (read n factorial) permutations of n unique items.

Sometimes the total number of partial orderings of n objects may be required. Suppose the number of unique orderings of r objects chosen from a set of n objects is needed. The result and count the number of permutations of n objects taken r at a time, can be generalized symbolically P(n,r) (read n permute r) with the computation:

$$P(n,r) = n(n-1)(n-2)...(n-r+1), \text{ r factors.}$$

Using factorial notation, the expression can be written in a more compact form:

$$P(n,r) = n(n-1)(n-2)...(n-r+1) = n!/(n-r)!$$

E.3- Combinations

Sometimes the number of unique grouping of objects irrespective of their ordering is of interest. For example, consider the number of unique orderings of three letters taken two at a time:

AB, AC, BA, BC, CA, CB.

There are clearly $3!/(3-2)! = 6$ such orderings; however, if we wish to count the number of pairs where the order of the pairs is unimportant, then we must divide the number of permutations by the number of duplicates. Since the number of duplicates is equal to the number of orderings of the r objects, the number of combinations among n objects taken r at a time, symbolically $C(n,r)$ (read n choose r) is simply the number of permutations $P(n,r)$ divided by the number of duplicates:

$$C(n,r) = n!/(n-r)!r!$$

A P P E N D I X   F

GAs COMPUTER PROGRAM STRUCTURE

This Appendix presents the structure of the computer program developed for the optimization of cable-stayed bridges using Genetic Algorithms.

Problems with Genetic Algorithms are:

- Fitness function
- Coding
- Decoding
- Sensitivity analysis of some GA parameters such as:

  Nb of generation / Nb of individuals per population,

  Probabilities of Crossover, Mutation & Selection ... etc.

F.1.1- FITNESS FUNCTION

The fitness function for the optimization of cable-stayed bridges under two cables out conditions is given in chapter 8 eq. 8.9.

F.1.2- CODING

Several ways can solve this problem, but the best one is the Multiparameter, mapped, fixed point coding (MC) (Goldberg, 1989). In this optimisation problem, the objective is to know the cable(s) (1 or 2) causing the worst load on the bridge. These cables chosen randomly among the 38 cables in this problem (8001 - 8038). If one wish to use the MC, he has to specify

1- Nb of parameter in our case Nb of cables (total)
2- Length of each parameter (ie Nb of bits in one parameter)
3- Minimum & Maximum parameters.

In this optimisation problem, the parameters are the cables, the minimum & maximum parameters are respectively 8001 & 8038, the length of parameters can vary; ex. 9, 10, 20 bits of 0 & 1 constructing each parameter and the total Nb of parameters is equal to 1 or 2 (one cable out, two cables out).

F.1.3- DECODING

Procedures Extract_parm & Decode_parm can do this task and knowing a string of 0/1 the corresponding cable can be found via calling these two procedures. However, they have been slightly modified to prevent 2 events:

1- case of having the same cables for the member considered when dealing with the problem of 2 cables out. The member considered could be bridge section or cable section. You will see in this peace of the program (Decode_parm) that simple tests are written to prevent this situation. In effect once this event occurs one of the cables is kept, the other one is selected randomly.

2- Case of having identical Nb of both cable and member when the member is a cable section. A boolean (logic) variable called Fito is used which takes true if the cable number is the same as the cable section. When it is true it means that this cable has to be discarded by assigning zero to its fitness value. Fito takes False in the opposite case.

With these two tests, the cables have to be different whatever the member & have to be different to the member with any cable section.

F.1.4- SENSITIVITY ANALYSIS OF THE GA PARAMETERS

A sensitivity analysis of the variables was used to decide parameter string lengths, the more sensitive variables being mapped at finer intervals by using longer strings. The GA program showed good convergence when using a population size of 50 with probabilities of 1.0 and 0.5 for crossover and mutation GA parameters, respectively.

F.2- MAXIMIZATION / MINIMIZATION

Fitness functions values may be positive or negative and we are interested in maximizing positive values and minimizing negative values. Minimizing negative values is equivalent to maximizing positive values and consequently the same GA program can perform this.

To distinguish between maximization & minimization, an integer variable called Max_Min has been introduced. It is equal to 1 with maximization & to 2 in the opposite case. The user has to specify at the beginning of the run the value of Max_Min.
To deal at the same time with positive and negative values, it was found the best way is to penalize the fitness function when it meets a negative in case of maximization and vice versa. Indeed if a maximization is considered, when a negative value is encountered, it is automatically reduced to zero. As a

consequence, the space becomes homogeneous and the maximum is sought only among the positive values. With minimization, in the event of meeting a positive value its corresponding fitness is zero and the negative fitnesses are multiplied by -1 to transform the minimization to maximization.

F.3- DATA FILES

First forces file is supplied by the user and then read (eg. aulsha.c2).

Second the user follows the optimization scenario by specifying the load (comb), the full or reduced forces, the maximization problem over axial/shear/bend and supplies the member. In case of data entered via batch file, all these data are written in a global file and then read by the program. The last thing entered by the user is the name of the output file.

Third, cable files (2 files for each cable) are all opened, read, and closed automatically. Their information is stored in program variables.

It should be noted that all cable files have to be in the same directory.

F.4- PROGRAMS WRITTEN

2 classes of programs are written for each problem (1 & 2 cables out). An introduction of the variables & parameters used is supplied for each program at the top of the program as comment (between { & }). The first class use the screen as a device to introduce the optimization scenario. While the second class use batch files to input the optimization scenario.

F.5- PROCEDURES FOR ONE CABLE OUT

All_Forces_In:

gives Fi all Fi i:1 to 38
Fi is stored in Fful [k,i] or Fred [k,i] depending on the loading chosen (Full or Reduced)
k: loading condition k=1 to 3
i: stands for cable Nb

F_Full_Red:

For the cable chosen, it gives you F1 depending on Full/Reduced conditions.

Enter_Member:

Member is specified by the user.
Member is considered as a string variable composed of 8 characters once entered, the corresponding Axial, Shear and Bending for each cable are stored from the corresponding file.

Open_cable_File:

All cable forces & Moment are red automatically. They are stored in the following variables
Cable_Axial [i,j] i=1 to 38; j=1 to 38
Cable_Shear [i,j]
Cable_moment [i,j]
All the files containing effects on Axial/Shear/Moment of cables by removing any cable are read by this procedure & stored in the above variables.

Alpha:

Allows the determination of Alpha

One_Out_Fitness:

Allows the computation of the Fitness function. Negative fitnesses are reduced to zero with maximization problem. Positive fitnesses are reduced to zero with minimization problem and negative fitnesses are weighted by -1 to apply GA.

Function_Decode:

Decodes strings as an unsigned integer.

Extract_Parm:

Allows the extraction of a substring (one parameter) from a full string.

Function_Map_Parm:

Maps a binary integer to range [Minparameter,Maxparameter].

Decode_Parms:

Gives you the cables out (see Decoding).

Fi_Alpha:

Knowing the cable out, the fitness function can be determined if Fito = False as mentioned in Decoding. The Fitness function is stored in FitnessF.

GA_Parametres:

Initialize some parameters to zero such as Gen, bmax, Nmutation ... etc

Maximization_Minimization:

Choice of loading condition : COMB 1 to 3.
Specify the parameters to be optimized: Axial/Shear/Bend.
Specify the Optimization Problem: Maximization or Minimization.
Specify full or reduced option.

Generation:

Gives the fitness value stored in FitnessF.

6- Procedures for one cable out (see listing)

Fi_Fj:

Allows the determination of F1 & F2 for the full & Reduced load.

Alpha_Beta:

Determines alpha & Beta stored respectively in A & B without their multiplication by -1/100.

Delta_Gamma:

Gives delta & Gamma stored respectively in D & G without their multiplication by -1/100.

Two_Out_Fitness:

Gives the fitness function stored in the variable FitnessF. Allows the determination of the contribution of each cable in the value of the fitness which are func1 and func2 for the cable 1 & 2 respectively.

Decode_Parms:

Same as 1 cable out but in this case 2 cables are considered (see Decoding).

A little detail for the case of having 2 identical cables: I keep the first cable which is stored in the variable Parms[1]. Int_Parameter. The second parameter (stored in Parms[2].Int_Parameter) is changed randomly but without being the same as cable1. ie the value of cable1 is discarded. 3 cases are obtained:

cable2 = 8001,	the random choice of cable 2 is between [8002,8038]
cable2 = 8038,	the random choice is between [8001,8037]
8001 < cable2 < 8038	The random choice is between [8001,cable2-1] or [cable2+1,8038]

If any of the cables is equal to the Member Fito = true meaning that this fitness will be reduced to zero.

cable1_cable2:

Just put the two 2 cables found in the variables cable1 & cable2.

FiFj_AlphaBetaDeltaGamma:

groups the procedures allowing the determination of F1,F2,alpha,beta,delta,gamma,func1,func2 and the fitness value FitnessF in case of Fito = false otherwise FitnessF=0 (Fito=true, see Decode_Parms).

Generation:

After selecting randomly 2 strings on the basis of the Stochastic Remainder method, the new strings have to undergo the Crossover Operator. The results are a New Population. For each individual of this New Population the Decode_Parms procedure is applied giving the 2cables cable1 and cable2. Knowing cable1 and cable2 the fitness function can be obtained & the Process continue until the convergence criteria are met.

F.7- DIRECTORIES/SUBDIRECTORIES/DATA FILES

Member files, cable files and all Forces in files can be stored in any subdirectory you wish but you have to pay attention to the **Assign** command and **FinfFirst** command.

Findfirst allows you to find the file corresponding to the Member you have entered, for this we have to write correctly the subdirectory where this file can be found.

Finffirst is encountered in Enter_Member & Open_Cables_Files procedures (see these procedures)

The same remark is kept for the commend Assign.

For the procedure All_Forces_In the corresponding data can be put in any subdirectory.

F.8- PRINCIPAL VARIABLES

Forces:

When all Cables in (Data file example aulsh4.c2). These forces are parametered by 2 things:

1- Load: Comb 4-i, i=1 to 3
2- Full or Reduced case

The variables used are:

Fful [k,cable]
k=1 to max_comb (=3)
cable = 8001 to 8038

Member:

For each Member considered we have to know the 3 parameters: Axial, shear & bend corresponding to each cable the variables are:

Member_axial[i], Member_shear[i] & Member_Moment[i]
i=1 to 38

Cables' data

The variables used for determining Delta & gamma are:

```
Cable_Axial  [i,j]
Cable_Shear  [i,j]      i=1 to 38
Cable_Moment [i,j]      j=1 to 38
```

Alpha, Beta, Delta, Gamma:

correspond to Alpha, Beta, Delta & Gamma, respectively.

Func1 & Func2:

The contribution of each Cable to the fitness

```
Func1 = alpha(F1+gamma*F2)/1-gamma*delta
Func2 = beta(F2+delta*F1)/1-gamma*delta
```

FitnessF:

is the summation of Func1 & Func2

Cable1:

Equal to Parms[1].Int_Parameter

Cable2:

Equal to Parms[2].Int_Parameter

F.9- PROGRAMS LISTING

Listing of programs and procedures may be found in the accompanied floppy disk at the end of the thesis.