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Optimal girder bridge study via CASE: Computer-Augmented Structural Engineering methodology

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Purdue University, 1987

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OPTIMAL GIRDER BRIDGE STUDY VIA CASE:
COMPUTER-AUGMENTED STRUCTURAL ENGINEERING METHODOLOGY

A Thesis

Submitted to the Faculty

of

Purdue University

by

Thomas E. Fenske

In Partial Fulfillment of the

Requirements for the Degree

of

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____
Major professor

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To my wife, Suzy.

ACKNOWLEDGMENTS

I wish to express my sincere appreciation to my major professor, Dr. Muzaffer Yener (currently Associate Professor of Civil and Environmental Engineering at Utah State University). Dr. Yener's continued encouragement, support, and guidance is gratefully acknowledged. My gratitude also goes to Dr. Mark Houck for serving as "in-house" Co-Chairman of the examinating committee and overseeing all of the paperwork and to Professors J. T. Gaunt and Martin Corless for giving me the benefit of their time and effort as members of the committee.

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ABSTRACT

Fenske, Thomas E., Ph.D., Purdue University, December 1987. Optimal Girder Bridge Study via CASE: Computer-Augmented Structural Engineering Methodology. Major Professor: Muzaffer Yener.

Fabrication of steel member sections from plate components is becoming much more economical than at any time previously. This dissertation presents the development of a design methodology that includes the structural synthesis process as an integrated component of a Computer-Aided Design and Draft (CADD) system for girder bridge design. The synthesis is based upon minimum cost of the superstructure using unit price values. The constraints are imposed according to the American Association of State Highway and Transportation Officials (AASHTO) specifications with the option of using either the Working Stress Design method or Load Factor Design method. Both the concrete roadway deck and steel girders are considered in the synthesis process. The process allows the steel girders to be fabricated with either stiffened or unstiffened webs, but restricts the synthesis to evaluation of a single depth girder throughout the bridge superstructure. The effect of utilizing fabricated girder sections composed of various plate thicknesses, widths, and depths causes a

variation in section properties throughout the girder. This nonprismatic member effect is included in the analysis and synthesis processes. The nonprismatic element stiffness matrices are derived based upon a classical formulation and employing numeric quadrature techniques. An extremely efficient analytical approach has been developed to perform the complex analysis which results from the traversing vehicle loading.

This CADD methodology has been developed so as to allow extension into a Computer-Aided Manufacturing (CAM) environment. The key factor in the ability to extend this methodology to encompass CAM is based upon the use of fabricated components. This unique formulation, exclusive to CASE, is possible due to the relational database architecture developed for the CASE methodology. The database holds specifically that information required by the fabricator to order material for the manufacturing process, schedule plant operations, operate numeric control machinery, and control shipping and inventory.

I. INTRODUCTION

1.1 General

A recent survey by the Federal Highway Administration (FWHA) [1] has indicated that two highway bridges out of every five in the United States are critically deficient or functionally obsolete. The seriousness of this problem has prompted the development of a federally aided highway program [2]. However, because of economic constraints, priority must be placed on determining which bridges must have immediate replacement and which require only modifications to prolong their service life. Obviously, the cost of replacing every deficient bridge in this country would be prohibitive. Hence, economy must be considered as well as safety in bridge design; both are of paramount importance in bridge evaluations.

To assist the structural engineer in his bridge evaluation, recent attention has focused on computer-aided engineering (CAE) methods [3,4]. However, these methods have had only limited success because they are based upon heuristic or so-called traditional design approaches. Traditional methods of design and fabrication of structural systems, in general, and bridge superstructures, in particular, are changing due to the present economic climate. Fabrication of steel member sections from plate components is becoming much more economical than at any time previously. A prime example of this trend is the "Autofab" [5] structural steel fabrication plants of Europe where the manufacture of certain civil engineering-type structures has been automated. Extension into this automated process possesses enormous potential for cost reduction and improved speed, accuracy, and reliability. However, even in the Autofab environment, a major restriction has hindered the application of Computer-Aided Design and Drafting/Computer-Aided Manufacturing (CADD/CAM) in civil engineering. This restriction is due to the uniqueness of a civil engineering structure as compared to the generally mass-produced structures in the aeronautic, automobile, and appliance industries. In civil engineering-type structures, at present, considerable fabrication and engineering costs result from the need to interpret analysis results, create a design based upon these data, and detail/draft the components of the structure. Clearly, there is a need for a structural design methodology that is reliable plus being both cost-effective and time-efficient.

To address this need, this thesis considers the conceptualization and foundation development of a "rational and systematic" structural design methodology that will be amenable to the CADD/CAM environment for steel girder bridges. The steel girder bridge utilized in this study is illustrated in Figure 1.1. This bridge system was selected for examination because the girder bridge is the most predominant bridge type in the United States. This structural design methodology will hereafter be referred to as CASE, an acronym for Computer-Augmented Structural Engineering. Although this work has been developed for girder bridges (e.g., CASE-GBRIDGE) [6,7], the methodology, with appropriate modifications, may be applied to the design of other structural systems [8,9,10]. To underscore this need for the CASE development, the National Science

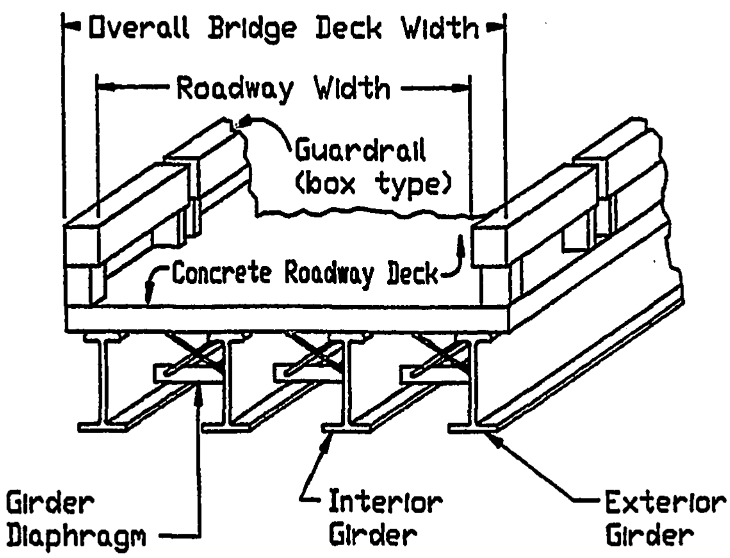


FIGURE 1.1 - Steel Girder Bridge Segment

Foundation [11] has recently stated, "The development of CAD/CAM has more potential to increase productivity than any invention since electricity." As the evolution of automated manufacturing extends into the civil engineering marketplace, an automated design approach must be available.

1.2 The Structural Design Process

The art of structural engineering is that of designing a structure, such as a bridge, spacecraft, or building, to either support or house some specific functional operation or process efficiently and reliably, while simultaneously maintaining the economics of the structure. This structural design process consists of two interdependent components, (1) analysis and (2) synthesis [12]. The determination of whether a given structure will be able to satisfy a given set of functions (loads) is known as the analysis of the structure. On the other hand, the determination of the best possible structure to satisfy the given set of functions is the synthesis (i.e., component selection) of the structure. The design of a structure is partly dependent on the experience gained by the designer from the analysis of similar type structures and partly dependent on economic, sociological, and aesthetic factors. In essence, the goal of all structural engineering design is to obtain an "optimal" structure, as determined by some quantifiable measure.

The optimum structure may be defined as a structure that satisfies all design constraints, such as stress and deflection limitations, imposed by the governing design specification and yields, in general, either minimum cost or weight. Since the structure's stresses and deflections can only be determined from an analysis of an initial trial structure, the design constraints and cost (function) are defined on the basis of these analytical results. The optimal structure is obtained through a search process which minimizes the cost or weight and satisfies all design constraints. A search process based upon a direct analytical optimization procedure can be used only for the simplest of structures possessing very few unknown design variables. For more complicated structures, such as a bridge system, computer automated techniques [13-16] must be applied to obtain the optimum design.

The complexity of a structural system is controlled by the number of and interconnectivity of individual components. Structural systems are physically composed of a large number of components, such as beams and columns, which contribute to the common purpose of the structure. In general, the total structure is designed by investigation of each individual component and verification of the overall structural performance. The process involved in the design of a bridge superstructure is, therefore, highly iterative and requires constant updating of information. The design activities involve many changes in the configuration, primary structure, and material. Thus, it is realized that the structural design process is iterative in nature, requiring data to be continually manipulated between several decision-making processes in order to obtain the final "optimal" design. Therefore, to effectively control the evaluation, storage, and transfer of information in a complex design problem, such as a bridge superstructure, a computer system is required.

It must be noted that the structural design process focusing on analysis and synthesis is only a portion of the total design process. The "total structural design" process consists of conceptualization, analytical evaluation and decision making, and final production segments.

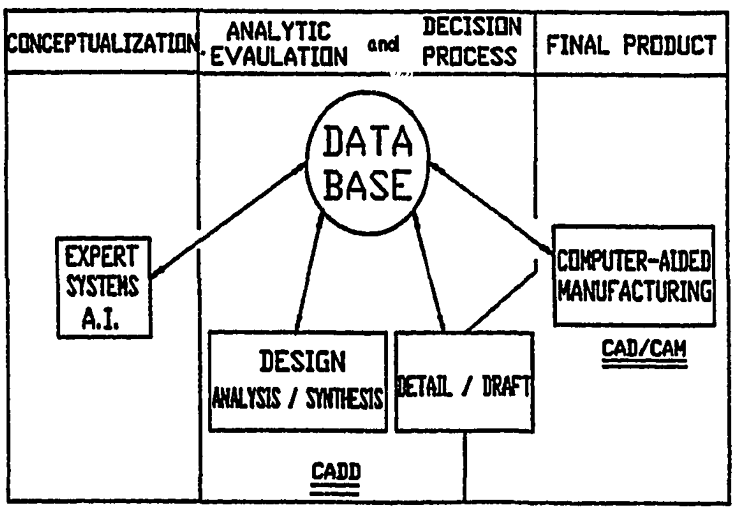


FIGURE 1.2 - Total Design Components

Conceptualization relates to type of structure, such as steel or concrete, framing arrangement, joint fixity, etc. Analytical evaluation and decision making is the analysis/ synthesis process. The final production aspect of the total design process refers to the detailing and fabrication of the structural system. Figure 1.2 illustrates the segments of the total design process. Structural design traditionally has considered only the analytical evaluation segment of the total design process. This traditional design process can be classified into two general catagories: (1) conventional design and (2) computer-assisted design (synthesis); both are usually referred to as Computer-Aided Design (CAD). This ambiguity of nomenclature is due to the fact that both use the computer for analysis; however, only the computer-assisted design method uses the computer to aid in synthesis. Figures 1.3 and 1.4 illustrate the philosophies of conventional and computer-assisted structural design, respectively.

1.3 Computer-Assisted Design

Structural design is based upon an iterative process which attempts to optimize the design by minimizing the cost or weight while maintaining the safety and integrity of the structure. In the conventional structural design procedure, this iterative process is based on a heuristic approach and keeps the engineer busy performing manual calculations when

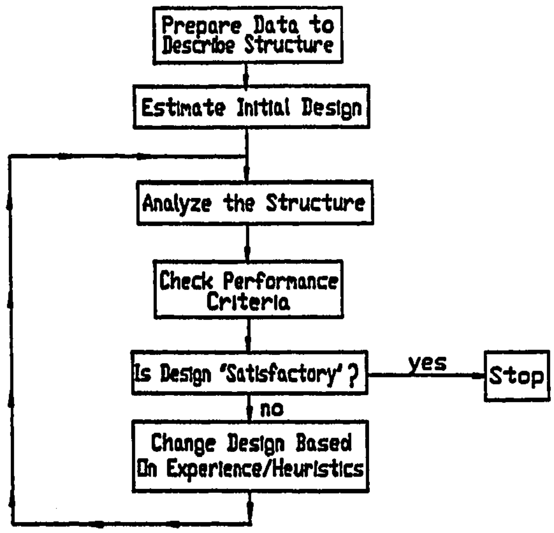


FIGURE 1.3 - Conventional Design Process

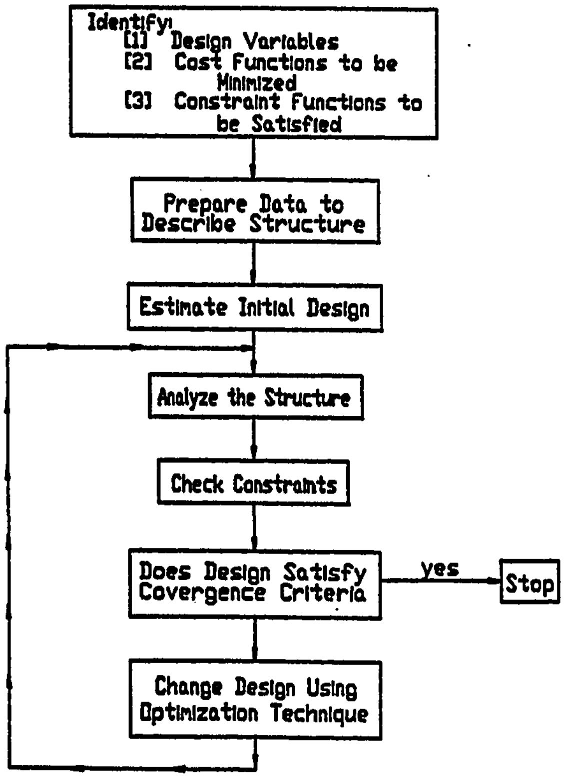


FIGURE 1.4 - Optimal Design Process

considering each design alternative. Contrast this to the computer-assisted design process where the design is changed automatically based on a certain optimality criterion which utilizes a logical procedure to optimize the criterion while satisfying all of the design constraints.

It is apparent, then, that the computer-assisted design procedure is more efficient in dealing with details of design. Multiple and complex constraints are routinely handled and the automated process reduces the total engineering time, which is especially significant in large, redundant structures. In addition, the automated design process forces the designer to identify a set of design variables explicitly, the cost (objective) function to be minimized, and the constraint functions imposed upon the system. This rigorous formulation of the computer-assisted design problem helps the designer to gain a better understanding of the true behavior of the structural system and is generally termed structural synthesis.

While this theory of optimization for structural optimization has been known for more than two decades, its use has been limited in practical design applications. It is often difficult to identify the cost function for the structure and, before automated fabrication, where only specific, hot-rolled sections were available, it was deemed ineffective. Another difficulty of the computer-assisted design process is that it does not include opportunities to vary the conceptuality of the design. An efficient design process would allow the engineer to include his judgment and experience to interpolate intermediate results while utilizing the optimal solution procedures in evaluating the detail aspects of the design, that is, using interactive programming in a computer-assisted design procedure.

The development of the interactive programming mode was required to allow the integration of the various decision making stages into the total structural design process. Interactive programming allows the engineer to direct the program flow by terminal response to intermediate program results. The engineer effectively controls the design process by interpreting the intermediate results and directing the computer through the desired calculations until the finalized design is achieved. The engineer can combine his knowledge, experience, and judgment with the power and speed of the computer, drawing on codes and specifications as well as his experience to formulate all criteria which must be investigated for each structural design. Thus, in the formulation of any "rational and systematic" design procedure, the methodology is mandated to employ computer-assisted design methods and include interactive programming capability.

1.4 CASE: Computer-Augmented Structural Engineering

In the early days of development, the benefit from the computer for structural engineering was primarily in the analysis stage, where the speed and accuracy of the computer was welcome. However, as computer-aided design evolved into traditional and computer-assisted methods; it has become integrated with computer-aided drafting. There are several advantages to using a Computer-Aided Design and Draft (CADD) system versus the traditional methods of design and detailing. Graphics display of the structural configuration and member stresses will allow the engineer to rapidly verify his design by visual inspection. Once the design has been completed, the computer can take over the production of the working drawings. Here again, the designer can use his experience and judgment to ensure that the conversion of the design into working details results in a practical solution. Since incorporating a computer-assisted design methodology into a CADD system would speed the engineering process, allowing the designer more freedom to investigate several alternatives as well as increasing the quality of the drawings and production process, the overall cost will be reduced.

Incorporating computer-assisted design techniques into a "rational and systematic" structural design methodology applicable to girder bridges which are fabricated in a computer-aided manufacturing environment presents special restrictions. Unlike typical mass-produced structures, such as airplanes, automobiles, etc., a distinct feature of all civil engineering type structures is that each is uniquely defined. Each civil engineering type structure (buildings, bridges, etc.) possesses appropriate, individually-defined building width, building height, roadway width, girder spans, design loads, etc. The problem in developing a CADD/CAM structural design approach for this type of structure is in formulating the structural system so as to allow for the integration of the analysis, design, and manufacturing components. An important feature of the CASE methodology is the formulation of the standard analysis member from fabricated component inputs. Fabricated components are detailed parts which represent each component of the structure exactly as it would be manufactured, i.e., plate widths and thicknesses, stiffener sizes, spacing, etc. This formulation process of developing structural analysis members from fabricated component input is described in Chapter 3.

The fabricated component concept is the crucial step in development of a CADD/CAM design methodology. Utilization of fabricated components results in the mathematical model of the structural system used in the design process being composed from the actual structure instead of a simplified representation. Thus, the analytical model actually represents the details of the "true" structure, where the engineer designs and sets the parameters for manufacturing fabricated components from start to finish with information being stored in the computer database.

The CADD portion of the system allows the designer to construct a geometric model, analyze the structure, perform kinematic studies, and produce engineering drawings. The CAM portion of this system allows the user to create numerical instruction for controlling machine tooling and process robots plus allowing coordination of plant operation with a factory management system. Within the near future, using this CADD/CAM procedure will greatly reduce overall cost by simply eliminating needless work and drawings.

In summary, the requirements for CASE to be a "rational and systematic" structural design methodology applicable to a CADD/CAM environment can be briefly stated as:

a) The structural designer is assisted in performing the overall design task, but not allieviated of the design responsibility;

b) The methodology is suitable for interactive programming that includes structural synthesis in a CADD environment and has the ability to extend into the area of CADD/CAM, provided the availability of computer hardware and manufacturing equipment. The essential ingredients for this requirement are:

i) modular development,

ii) database development and management strategies,

iii) conceptualization of new preprocessing and postprocessing methods (fabricated component concept),

iv) efficiency and speed in the design process;

c) The methodology is able to address practical design problems and produce useful practical design results from conceptualization to final product phase.

Note that the methodology is amenable to microcomputer systems and, thus, can be applied to virtually all design environments.

1.5 Limitation of Previous Studies

There have been previous investigations into the area of computer-aided engineering (CAE) and, more specifically, into the subject of plate girders and composite girder bridge systems. All of these previous studies, however, possess serious shortcomings in view of the "rational and systematic" structural design methodology requirements previously stated. The limitations of previous studies will be grouped and briefly examined in this section.

In the area of CAE, there have been a few recent attempts to extend the computer usage into the design process. The aerospace and automobile industries [17,18] have extensively utilized CAE over the past ten years but, as yet, usage for civil engineering type structures, in actuality, has been void. There has been considerable effort in the area of computer-aided conceptualization [19] for building systems but, as yet, no functional methodology has been produced. Some attempts have been made to apply CADD to structural engineering systems. These basically consist of employing a standard finite element analysis package with a preloader for data input. For these systems, the drafting component is the major emphasis, but requires a separate input from the design process. These applications have excluded data transfer between the various analysis/ synthesis/fabrication stages. There have been attempts to allow for data transfer through the design and detailing phases [20]. However, these require the use of overlay graphics and only simulate the actual system by a mathematical model. At the present time, no methodology exists that allows complete transfer of data to be utilized in the "total structural design" process or that can form a basis for a CADD/CAM environment.

Structural synthesis (nonlinear optimization) research has been conducted on welded plate girders to various degrees and with varied success for the past several years. In relation to composite girder bridges, however, these efforts have some major limitations, so as to restrict their applicability and usefulness severely. Virtually all studies have been conducted using weight as the objective function to be minimized, neglecting entirely the fabrication cost. The majority of studies has been restricted to statically determinate structures, thus circumventing the problem of interdependence of optimum member properties and internal distribution of load. The studies that have considered the continuity of the bridge system have restricted themselves to doubly symmetric sections, which are not optimal in composite sections, and have been optimized based upon the section modulus of the restricted section. It must be noted that the optimal moment of inertia of a fully-braced section, which controls the internal load distribution, is different from the optimal section modulus. Another restriction has been the general assumption of constant section properties in the analysis stage, causing inaccurate force distributions.

Furthermore, except for proprietary codes developed by major steel manufacturers, the implementation of the Load Factor Design (LFD) method, in accordance with the American Association of State Highway and Transportation Officials (AASHTO) [21], has not been examined. One reason that the LFD method has not previously been investigated is that the design approach has not been universally accepted by the individual states. The exclusion of other code provisions also have hindered the applicability of several such studies; this includes neglecting lateral bracing conditions and employing constant maximum allowable stress in the optimization process.

There have been several recent investigations into CAD applications to bridge systems. These have basically been limited to assistance in the analysis process and have made no attempt as to synthesis of the structure. These investigations have been conducted with the use of a mainframe computer, although it should be noted that some analysis codes are currently being investigated for possible use on microcomputers. Also, examinations into more reliable analytical models for nonprismatic members have been given considerable attention, but these studies all have been limited to nonprismatic members that can be expressed as some form of a continuous function. In the previous studies, mathematical models are used to approximate the structure, assumptions are made to simplify program development, or restrictions are made regarding the behavior of the system. These techniques greatly reduce or, in some cases, nullify the practical usefulness of the research. The developed CASE methodology significantly reduces the limitations of the previous studies by virtue of its basic formulation. As applied to girder bridges, this methodology includes all pertinent AASHTO specifications and design methods and allows consideration of composite or noncomposite sections, prismatic or nonprismatic beams, and simply supported or continuous spans. Most significantly, the mathematical analysis model formulation used in all previous studies is replaced by a theoretically exact formulation of the system by utilizing fabricated component dimensions and properties as program input. Furthermore, previous to this study, investigations into the interaction of the roadway slab and steel girders, consideration of the CADD/CAM concept, or interactive programming were not examined.

1.6 Objectives of and Scope of Investigation

The purpose of this research is to develop a "rational and systematic" design methodology, CASE, for structural engineering via microcomputer application and to draw conclusions regarding the overall utility of such a methodology as applied to civil engineering type structures fabricated in a CADD/CAM environment.

This research will consider, in particular, a composite girder bridge (GBRIDGE) system; however, it will be applicable to all civil engineering structural framing types. The CASE methodology addresses, in a general sense, the means to include systematically all decision-making components in the total design process. Also, this work will identify and study optimal dimensions of bridges consisting of concrete deck and steel stringers subject to different stress, serviceability, and geometric constraints. The bridges considered in this study are simply-supported single spans and continuous two or three span highway bridges with moderate span lengths which are sufficiently long to require fabrication of the girder section. More specifically, the research project examines:

a) Conceptualization and formulation of a design methodology for civil engineering structural systems that can be utilized in a effective and efficient manner in a CADD/CAM environment:

   i) formulation based upon concept of fabricated components,

   ii) database structure and management,

   iii) modularity programming structure;

b) Addressing specifically the effect of variable plate components and arrangements in the analysis of the superstructures, i.e., account for nonprismatic girders;

c) Implementation of optimal design to girder bridge superstructures, considering both the roadway concrete deck and supporting steel girders;

d) Including consideration of both AASHTO Working Stress Design and Load Factor Design methods into design philosophies.

It is important to note that the CASE methodology was developed for use on a microcomputer, and nonlinear programming methods that are reliable and accurate on microcomputers are implemented in the structural synthesis process. In this study, the computer language used is interpreter BASIC. The effectiveness and efficiency of the

CASE methodology, reflecting the computer limitations, will be evaluated based upon accuracy, reliability, and the ability to verify results since these are the criteria under which structural engineers design.

Due to the limitations of available computer and manufacturing equipment, it is not intended to develop the complete CASE methodology, but rather to formulate the structure so that the utility and practicality of employing the integrated design approach for civil engineering structures in a CADD/CAM environment is fully demonstrated. Only the CADD portion is extensively examined in context of formulation, automated analysis, and synthesis. While it is clearly demonstrated that the CASE methodology is structured such that extension into a CADD/CAM environment can be implemented, the computer implementation and manufacturing numeric control operations of the final product phase are not included within the scope of this research.

1.7 Organization of the Dissertation

Chapter 1 has served as an initial introduction to the CASE methodology; stating why the methodology is needed, the requirements of developing such a methodology, and the advantage of same. The first chapter also has included a brief description of limitations of previous studies plus the scope and objectives of this particular research effort. A synopsis of various aspects of computer-aided engineering, their related progress, and current status is examined in the second chapter. The unique features of the CASE methodology are described in Chapter 3. The formulation of structural analysis members from the particular fabricated components used for girder bridges is illustrated, along with the essential components required for implementation of the methodology, i.e., CASE-GBRIDGE.

The structural design process is composed of analysis and synthesis, and each is examined separately herein. The concepts and formulation employed by the bridge analysis are presented in the Chapter 4, including such topics as investigation of nonprismatic members and the efficient analytic approach for bridge analysis. In Chapter 5, the structural synthesis (nonlinear optimization) of the bridge superstructure is considered. After examination of appropriate methodologies for element component synthesis amenable for use on a microcomputer, the general development of the cost function and constraints is examined. The costs considered are based upon current unit prices presently encountered in a steel fabrication plant environment.

The application of CASE-GBRIDGE is examined in Chapter 6. The modular, interactive nature of the implementation of GBRIDGE is described through the use of an example bridge system. The general utility of the method is reviewed while demonstrating its immediate practical application. In the final chapter, the results and conclusions from implementation of the CASE methodology are considered, along with recommendations for future studies to extend the foundation of CASE developed herein.

II. LITERATURE SURVEY AND HISTORICAL BACKGROUND

2.1 General

The explosion of computer capabilities during the early 1960's encouraged the structural engineer to use the computer in analysis of structures. The analysis and synthesis of a structural system was formally introduced into the structural design process by Lucien Schmit with his theory on structural synthesis [12]. Schmit's research has spawned several studies into various aspects of the design of structural members by computer-assisted methods.

This chapter will examine briefly the background and development of (a) Computer-Aided Engineering, (b) Structural Synthesis, and (c) Bridge Superstructure Design.

2.2 Aspects of Computer-Aided Engineering (CAE)

In the 1960s, engineers began routinely to apply computerized problem-solving methods to a wide variety of structural engineering problems. At that time, technology was limited to unintegrated batch processing and gave little or no attention to efficient management of data. In the 1970s, significant advances were made in computer hardware technology which increased the acceptance and usage of computer methods in structural engineering. These advances included increased computing capabilities, the advent of time shares, allowing a greater number of engineers access to computing equipment, and, most significantly, the development of the microcomputer.

While drastic changes occurred in computer hardware technology, changes in computer programming techniques have come far more slowly. Primarily, advances in applications software for structural engineering have focused on the development of computer graphics systems. However, more recently, several studies have successfully applied computer methods in the area of advanced structural analysis. File management and the extensive use of databases have not been implemented to any significant degree due to the fact that the computer science techniques required to implement such features are beyond the scope of computing knowledge possessed by most engineers. This, in turn, has hindered the development of a truly interactive and integrated analysis and design program.

Computer usage in structural engineering has grown significantly since the early 1970s. At that time, the only common structural engineering applications were analysis (usually restricted to linear elastic models) and detailing of certain repetitive structural components. The work that has been done since that time in CAE for structural engineering has basically concentrated on improvement and expansion of drafting effects. This is mainly a result of the lack of adequate data storage techniques; present methods of data storage and access continue to fall short of the techniques required to effectively expand the use of computers in structural engineering. Current methods generally fall into one of three catagories [22]:

* temporary files: used by many large structural analysis programs (usually unformatted or binary) for segmentation purposes, backup/ restart, or postprocessing;
* explicit interface programs: used for performing the necessary conversions and reformatting when the output of one program serves as input for another;
* text files: used to save both input and output data (usually alphanumeric and formatted).

The first two methods exhibit a total dependence on the application software; the third catagory provides no information to the data management system other than the name of the file.

There have been a few attempts to develop improved data storage techniques for use in structural engineering analysis/design software. Rynearson and Gamel [20] have developed a model called CADS which utilizes a database to share data for multi-discipline coordination of the total design process and to create engineering drawings. A software package developed by the U. S. Army Corps of Engineers [19] is similar. A model developed by White [23] (but not implemented) discusses the integration of CAD/CAE into a larger scheme called CIE (Computer-Integrated Engineering). In this model, the system must be capable of supporting graphics, large-scale design, access to data on an interdisciplinary basis, and interfacing to existing analysis programs, simulation programs, and material control systems.

During the past few years, much work has been done in the area of integrated application software development. Geometric modelling is having a major impact, especially in the automotive and aerospace industries. These two industries have been particularly successful in extending computer usage into the design process; however, as yet, application of these CAE techniques to civil engineering structures has been largely nonexistant.

2.3 Structural Synthesis

2.3.1 Girders

Most research in the area of synthesis of welded plate girder bridges has been related to the determination of the cross-sectional dimensions for given values of bending moment and shear. Razani and Goble [24] described a procedure for minimum cost design of noncomposite symmetrical plate girders for continuous highway bridges using the 1961 AASHTO specifications. The web thickness, web height, and flange width were held constant but the flange thickness was allowed to vary by splicing. The minimum cost design was obtained by using a iterative flange smoothing method to balance material and fabrication costs.

Instead of using mathematical models, Goble and DeSantis [25] used a cost table in a minimum cost design method for composite continuous welded bridge girders. The objective function was the material cost plus the fabrication cost. In this study, the depth of the girder was considered constant. The girder was symmetrical in the negative moment region and, in the positive moment region, where composite action occurs, the top flange width was a fixed percentage of the bottom flange width. The design variables were the top flange thickness, the web thickness, the distance between web splices, and the type of steel at each analysis point along the girder. The design parameters included span length, girder spacing, strength of the concrete, thickness of the concrete slab, effective concrete flange, modular ratio, and the ratio of top flange width to bottom flange width. Using a smoothing technique, Goble and DeSantis determined the optimum number and locations of the web and flange splices. A two-stage grid search was implemented to establish the minimum cost flange width and web height based on the information previously generated on splice points and on the material types. An approximate procedure was used to select a trial section. The bottom flange thickness was determined by designing a flange for a symmetric girder subject to the maximum combination of dead and live load; the top flange was determined by applying dead load only. The smoothing technique and two-stage grid search were then reapplied to improve the design.

There have been several investigations into the synthesis of plate girders that are not part of bridge systems. For example, Holt and Heithecker [26] used calculus to derive formulas for the minimum weight proportions for laterally supported symmetric plate girders without web stiffeners. The three design variables considered in this study were the flange area, thickness of the web, and the height of the web. The proportions of rolled steel beams were compared to the theoretical optimum proportions. Chong [27] applied calculus to derive equations for minimum cost design of unstiffened hybrid beams; the objective function was based on material cost. Equations were developed for optimum web thickness, optimum web height, and optimum flange area. The end result was a uniform girder fully stressed in bending and shear at critical locations. Both of these studies were based upon American Institute of Steel Construction (AISC) specifications for elastic design.

Annamalai, Lewis, and Goldberg [28] developed a computer program for minimum cost design of noncomposite, simply supported plate girders, in accordance with AISC specifications. The program allowed for stiffened and unstiffened girders; only two splices were allowed in each flange. These researchers followed Goble and DeSantis in the cost optimization process, using only the unit cost of steel and a fixed welding cost, but expanded the welding costs to include material cost and labor cost. They also estimated the total cost for various configurations. In this program, the nonlinear programming technique called backtracking was used for the optimization.

Goble and Moses [29] developed a computer program for minimum weight design of symmetrical plate girders with or without adequate lateral support. This research was similar to previous studies except that the minimum weight problem was converted to an unconstrained minimization problem, i.e., application of Segmented Unconstrained Minimization Technique (SUMT). Azad [30] and Vachajitpan and Rockey [31] developed curves for minimum weight design of noncomposite girders.

More recently, the synthesis of welded plate girders has been formulated as a mathematical programming problem in which various programming techniques are used, such as the dynamic programming method used by Azad [32], the penalty function method employed by Sheu [33], and the various methods outlined by Mumuni [34].

2.3.2 Nonlinear Optimization

Structural synthesis has been referred to, in some reports, as nonlinear structural optimization. There are three types of optimization methods which can be applied to structures: (1) graphical, (2) analytical, and (3) numerical. The graphical approach is limited, obviously, to quite simple problems which can be graphed in two-dimensional space and, as a result, is rarely used. The analytical, or classical, method of optimization is based on the condition that the first derivatives of the objective function with respect to the independent variables must vanish at the optimum. Thus, analytical optimization techniques are restricted to appropriate problem types.

Numerical methods, or mathematical programming techniques, are used to optimize structures which can be modeled using finite element methods. One of the significant features of these techniques is that no "a priori" assumptions are made regarding which constraints will be critical at the optimum. Mathematical programming can be divided into two types: linear (LP) and nonlinear (NLP). While linear programming techniques can be applied to structural optimization problems in a few restrictive instances, most frequently, the structural synthesis problem will be nonlinear in nature.

The structural synthesis process seeks to select design variables that yield the "best" member within the limits placed on the structural behavior by the design code specification. The limits are constraints in the optimization process and there are a large number of nonlinear constrained programming methods available. At present, only some of these methods have been successfully applied to structural optimization [29,35]. The most successful of these methods are the transformation or penalty function method, the complex (Box) method, and the enumeration method of backtracking.

A transformation method is any method that solves the constrained nonlinear problem by transforming it into one or more unconstrained optimization problems. This technique is often termed SUMT [36], an acronym for Sequentially Unconstrained Minimization Technique. The various transformation methods include the exterior and interior penalty function methods as well as augmented Lagrangian or multiplier methods. As the constrained problem is transformed into a sequence of unconstrained problems, any of a large number of unconstrained search techniques may be employed. Each successive unconstrained search starts from the solution of the previous search until the desired convergence is attained.

The Box method is a procedure based on the "complex" method developed by M. J. Box [37]. It is a sequential search technique which can be applied effectively to nonlinear programming problems which are subject to nonlinear inequality constraints. (The presence of linear or equality constraints will significantly reduce the effectiveness of this algorithm.) The procedure is intended to find the global minimum (or maximum) due to the fact that the initial sets of points are randomly scattered throughout the feasible region. One advantage to this method is that this search technique requires no derivatives.

The backtracking technique is an enumeration method. These methods have a simple structure and allow for complex coding, code constraints, and design details. They also overcome or bypass many of the problems generated by many other types of mathematical programming techniques. It is important to note that enumeration techniques are applicable to discrete value functions only; furthermore, the only industrial implementation of optimization that has been successfully applied to plate girders is the enumeration method of backtracking [5,38]

The backtrack method [39] solves nonlinear constrained function minimization problems by a systematic search approach. The object of this method is to find a vector of variables $X = \{x_i\}$ $(i=1,2,...,n)$ which will minimize the objective function and also satisfy the design constraints $g_j(x_i) \leq 0$ $(j=1,2,...,p)$. For the variables, series of discrete variables are given in increasing order. Generally, a partial search is performed for each variable and, if the possibilities are exhausted, a backtrack is made and a new partial search is carried out. This procedure can be applied successfully to discrete, nonlinear optimization problems regardless of the complexity of the constraints.

2.4 Bridge Superstructure Design

2.4.1 Analysis Methods and Studies

Classical methods, approximation methods, and numerical methods are the three types of analysis methods applied to civil engineering structures. Classical analysis is based upon the exact solution of the governing differential equations of the system. Classical methods have been applied in a number of studies [40-43]. However, the limitations of these methods, which are applicable only to relatively simple geometry, loading, and boundary conditions, restricts the usefulness to a very narrow range of problems.

More complex problems must be solved using an approximation method or a numerical method. Approximation methods include energy methods, such as the principle of minimum potential energy, variational principles, such as the Galerkin method and the Ritz method, and perturbation methods. However, the application of approximate methods is limited to uncomplicated boundary conditions and simple variation of thickness. It must be remembered that the use of an approximation method will yield just that -- an approximate analysis solution.

The numerical methods of analysis are based on the principles of finite elements and finite differences. Finite element analysis allows for a more exact analysis than can be achieved using the approximation methods and is applicable to a far wider range of problems than either of the other types of analysis methods. Numerical methods have been applied successfully to tapered plates [44], circular plates [45], elements of varying thickness [46-49], and nonprismatic members [50-52].

The two general analysis methods most frequently used in the bridge analysis problem are the flexibility method and Newmark's numerical procedure of successive approximations [53]. Most older programs use the flexibility method with a constant flexural stiffness (EI) to generate influence lines [54]. Busek [55] applied this method in a program for optimizing a rolled section highway bridge girder. Newmark's method is the method most frequently employed in current computer applications due to the ease and simplicity of implementation. Clugh and Biggers [56] used Newmark's method to develop an algorithm for generating stiffness matrices for nonprismatic beam-column members. However, this method is rather inefficient in terms of computer execution time.

Recently, attention has been focused on application of the finite element method to bridge analysis. General purpose finite element programs, such as ANSYS and NASTRAN, have been developed to analyze a broad range of structures and loading types [57]. One of the most common applications of these general purpose programs is in the area of bridge design, particularly three-dimensional analysis of a complete bridge system. An analysis of this type, including cross frames, horizontal bracing, and main longitudinal members, presently is feasible only in special cases due to the considerable set-up time involved. However, significant reductions of overall bridge cost and enhancements to safety can be achieved through such an analysis. These finite element programs are especially helpful in the analysis of special loading conditions, such as thermal, seismic, or wind loads, in determination of realistic lateral distribution factors for girder bridges, in the accurate analysis of curved or skewed bridges, and in the evaluation of alternative load paths involving lateral components. Some customized programs include work by Schelling, Freeman, and Smith [58] and the SIMONS program developed by the State of Wisconsin.

The effect of nonprismatic members has been investigated recently by several researchers. Karabalis and Beskos [59] applied a finite element method for analysis of linear elastic plane structures assuming continuous shape function distribution. Eisenburger [60] derived explicit terms for the stiffness matrices of several common nonprismatic members with the stiffnesses based on the flexibilities of the element. Zochowski and Mizukami [61] and Mikkola and Paavola [62] have also studied the application of finite element methods to nonprismatic structural elements.

2.4.2 Computer Assisted Design Methods

There have been many computer programs developed to take advantage of the speed and efficiency of computers in the analysis of structural systems. Some have been developed by state highway and transportation departments or public educational institutions. Others are proprietary codes developed by private companies. Many such programs exist for the analysis of girder bridge systems.

BRCOM, a microcomputer program for bridge analysis and rating, was a project of the Rural Technology Assistance Program of the Federal Highway Administration. This program analyzes several types of bridge superstructures. The superstructure can be composite or noncomposite; BRCOM also allows consideration of a simple span or continuous span bridge. The program first evaluates the section properties for each element. A force analysis is used to compute the shear, moments, and deflections due to dead load and live load. The analysis is based upon the flexibility method where the variation in member properties are approximated. The shear, bending moment, and deflections are computed for the tenth points along each member span length so as to develop critical load envelopes.

Bethlehem Steel Corporation has developed a proprietary computer program for weight and cost comparison of preliminary design for plate girder stringers in highway bridges [63,64]. This program operates on a supercomputer and can analyze and design either simple spans or continuous spans to a maximum of six. The girders can be unstiffened, transversely stiffened, or have both transverse and longitudinal stiffeners, and can be either hybrid or homogeneous. An important feature of this program is that it includes AASHTO specifications for both Working Stress Design and Load Factor Design.

The Bethlehem program selects an initial web thickness and height. The stiffener spacing then is determined and flanges selected. After the girder geometry has been established, the program considers both material cost and fabrication cost in determining a relative cost factor for each trial girder. The cost analysis considers these costs for flange splices, web splices, web to flange welds, attachment of stiffeners, attachment of shear connectors, radiographic inspection, blast cleaning, and painting. However, it disregards diaphragms, field splices, bearings, and erection costs, among others. After the cost factor is computed, the web thickness is incremented by 1/16 inch for two or three increments with the procedure repeated for each increment. The web height then is incremented by three inches for three increments with the procedure again repeated for each increment. The results include a summary of each trial design with the relative cost index. The most economical designs are elaborated upon with more detailed design information.

USX Corporation (formerly U. S. Steel) has developed a computer program for the analysis and design of plate girder bridges. Starting with a program developed by the Wisconsin Department of Transportation, this program was expanded upon and modified to create the USX program which is called SIMON [65,66]. SIMON follows the 1973 AASHTO specifications and 1974 interim specifications and, like the Bethlehem Steel program, considers both the Working Stress Design and Load Factor Design methods. The program operates on a supercomputer and can evaluate simple spans or up to eight continuous spans. The bridge structure may be composite or noncomposite, and SIMON allows for hybrid girders, ear haunches, and tapered haunches.

SIMON calculates a performance ratio for each of the design criteria; the performance ratio is the calculated value divided by the allowable value. A ratio greater than 1.0 indicates that the design is not acceptable. The program checks the number of trials against a limiting value and compares the girder weight and maximum performance ratio with those calculated in each previous cycle. SIMON can select the lightest weight design for the flange widths and web height specified. The program does not, however, vary the web height to find the minimum weight design. To check other web heights, the program must be re-executed.

The California Department of Transportation has two programs for the analysis of simple span, composite girders: the Composite Girder Design program and Composite Girder Cross Section Analysis/Flange Design program [67]. These programs do not produce an optimum design; however, they will evaluate the minimum weight design for a specified web height and thickness specified.

The Composite Girder Design program designs shear connectors and transverse web stiffeners, calculating moments, shears, required flange areas, and deflections. The results include three sets of curves for the symmetrical half span. The first set of curves includes the required area of the top and bottom flanges and the static moment of the transformed concrete divided by the moment of inertia of the composite section. The second set of curves includes the shear envelope, the stiffener moment of inertia, and the maximum transverse stiffener spacing. The third set shows live load shears and curb railing shears. The Composite Girder Cross Section Analysis/Flange Design program determines the necessary sizes of the top and bottom flanges for a composite girder.

The North Carolina Department of Transportation has developed a computer program for composite plate girders [68] which is similar to the USX SIMON program. This program uses performance ratios to adjust thicknesses of the top and bottom flanges. Changes in flange thickness are made until the lightest weight design is found for the given web height and web height to thickness ratio.

The Virginia Department of Highways and Transportation has a program to analyze and design simple span composite girders [69]. This program uses the 1973 AASHTO specifications, modified to follow the design specifications of the department, and designs homogeneous girders of A36, A588, or A441 steel. In the analysis, the initial trial values default to a minimum flange width of 12 inches and a minimum flange thickness of 3/4 inch if no initial values are specified. The thickness of the flange plates are increased before the width is increased if the girder is found to be overstressed. To account for the dead weight of bracing and other miscellaneous steel, the girder weight is increased by 11 percent for spans up to 150 feet in length and by 18 percent for spans over 150 feet. This program does not produce an optimum bridge design; it selects the minimum weight design for the web height specified.

The Department of Transportation of the State of Georgia has developed a program to analyze simply supported, composite plate girders for highway bridges [70]. The program has provisions for the design of girders using A441, A588, A572, or A514 structural steel but does not use current AASHTO specifications. The Georgia program can select the plates for a composite plate girder but, as with the other programs mentioned here, cannot generate an optimum minimum weight or minimum cost design.

III. THE CASE METHODOLOGY

3.1 General

The need for and requirements of a "rational and systematic" design methodology for civil engineering type structures that is applicable to structural fabrication in a CADD/CAM environment has been detailed in Chapter I. The development of such a methodology possesses enormous potential for cost reduction and improved speed, accuracy and reliability. A major restriction that has hindered this design methodology development is due to the uniqueness of civil engineering structures as compared to the generally mass-produced structures in the aeronautic, automobile, and appliance industries. Additionally, the development of such a methodology has not been previously undertaken because of the comprehensiveness required through several broad scientific areas: structural engineering, computer science, numeric methods, mathematical programming, etc. The development of the CASE methodology eliminates these restrictions.

To demonstrate how the CASE methodology is applicable to steel girder bridges, this chapter considers: (a) Foundation of CASE Methodology, (b) Database Development and

Management Strategy, (c) Interactive Control and Modification, and (d) Programming Modules of CASE.

3.2 Foundation of the CASE Methodology

The development of a "rational and systematic" design methodology that considers CADD/CAM applications for civil engineering structural types presents difficult data modeling problems. The typical structure designed and manufactured in an automated manufacturing environment is generally a "single-item" mass-produced structure. Examples of these types of mass-produced structures are automobiles, airplanes, appliances, etc. In contrast, the typical civil engineering structure is uniquely defined by its individual geometry, loading, members, and material composition. The problem lies in developing an appropriate model for civil engineering structures that will allow for efficient, cost-effective design while simultaneously considering the automation requirements of fabrication. This design/manufacturing data modeling problem is actually a reflection of how the data is implemented into the automated manufacturing system.

In general, the analysis/design process for a mass-produced, "single-item" structure is rigorously performed with only secondary consideration given to design time and cost. This analysis/design process usually results in a prototype structure that is physically tested and, upon verification as the design to be implemented, the resulting design data then are independently introduced into the automated manufacturing system. This automated system operates the numeric control machines, assembly line operations, etc., that mass produce the single-item structure. The overall design cost and time is absorbed in the final product stage without significantly affecting the overall project success or failure. However, in civil engineering structures, the time and cost required for both the analysis/synthesis process and structural fabrication significantly influence the overall project result (success/ failure). This interdependency of both cost and time is due to the fact that the overall project time, from initial conception to delivery of the structural framing, is limited. All costs are reflected in the procedure used in handling data modeling. For civil engineering type structures and, in particular, a girder bridge, the model that is employed in the analysis/design process must be able to translate directly into the graphic display and manufacturing components of a CADD/CAM design system. It is how to define this modeling procedure to allow for the cost-effective and time-efficient design and fabrication of civil engineering type structures that is specifically developed by CASE. The conceptual formulation of CASE is fully illustrated in Figure 3.1. The development of this complete conceputalization of CASE is not included within

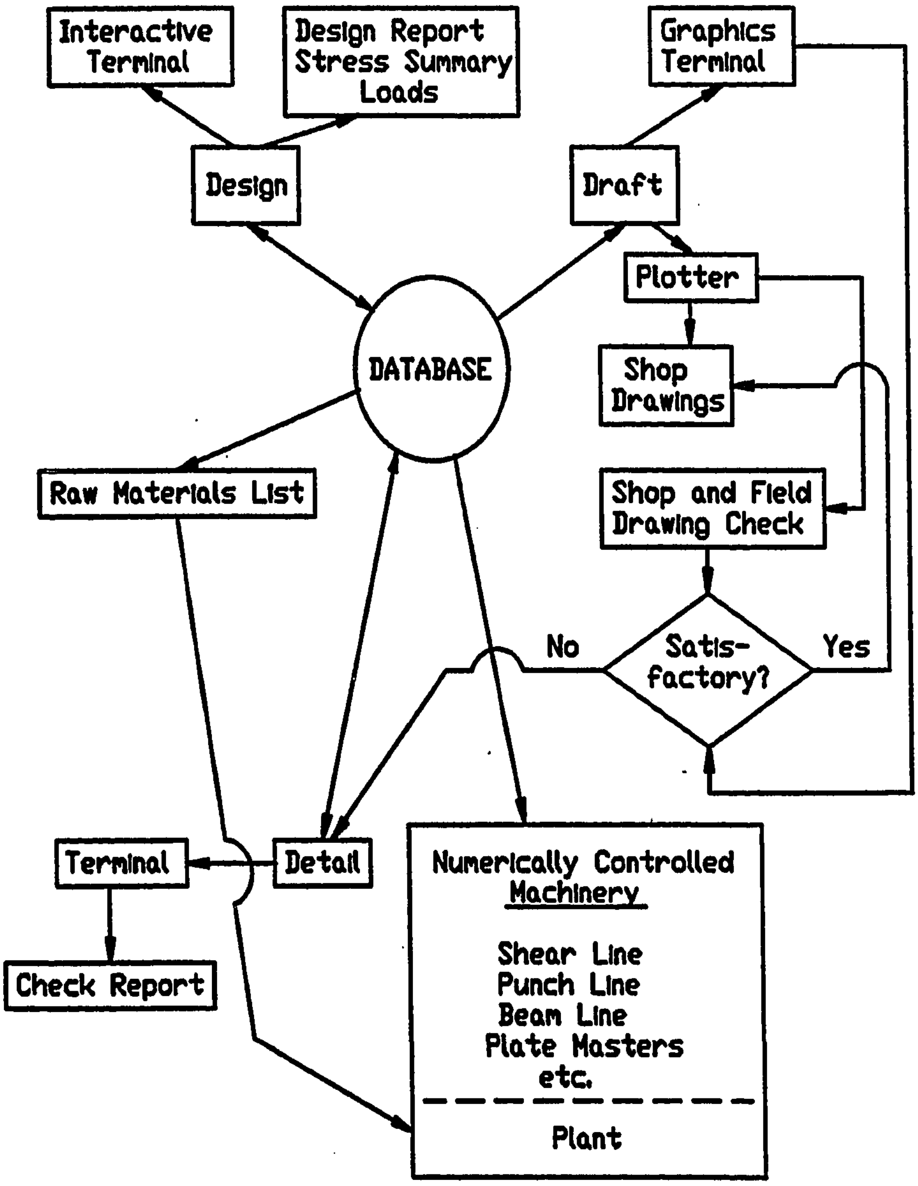


FIGURE 3.1 - Conceptual Formulation of CASE

the scope of this dissertation. As previously stated, this research is a presentation of the concepts and formulation of CASE and lays the foundation for more extensive implementation.

A unique and central feature of the CASE methodology is the formulation of the standard analysis member from fabricated component inputs, where fabricated components are detailed parts which represent each component of the bridge superstructure exactly as it would be manufactured, that is, plate widths and thicknesses, stiffener sizes, spacing, etc. There are three general fabricated component types employed by CASE-GBRIDGE (abutment, intermediate, and pier) to define the theoretical structural member, with the maximum number of parts per individual member limited to four. Regardless of the bridge geometry, each theoretical structural member, that is, each girder, is composed of fabricated components. The limitation of fabricated component lengths are normally controlled by manufacturing and shipping restrictions. The manufacturers' available material cutting shear length usually limits the length of the individual plate changes within a fabricated component. The overall fabricated component length is limited by shipping restrictions that depend upon how the components are shipped from the fabrication plant to the jobsite, i.e., via truck, rail, or barge. An illustration of the three fabricated components and their relationship to the overall bridge system is illustrated in Figure 3.2.

This new method for conceptualization of the bridge superstructure from fabricated components will allow the mathematical model to exactly represent the true structure. This formulation process of determining structural members from component data input is integral to extension into the CADD/CAM concept and is based upon utilization of a fabricated component matrix, Fcomd. In order to describe each individual fabricated component, thirty items are input into Fcomd as shown in Figure 3.3. These items include material properties, sectional dimensions, depth changes, and a variety of additional information. Each component is allowed one inner flange change, one web change, one outer flange change, and one depth change, with all changes currently occurring at the same location. It should be noted that, allowing for either stiffened or unstiffened

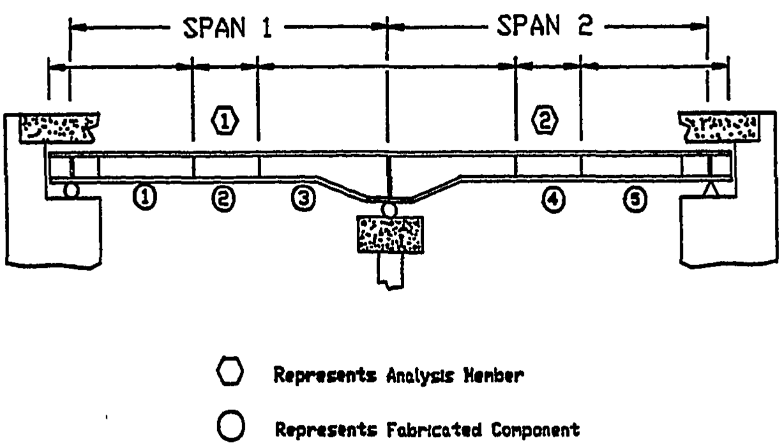
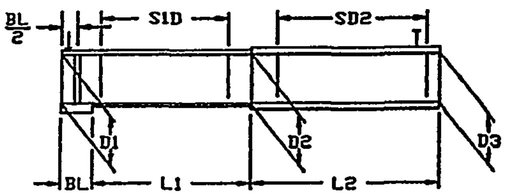
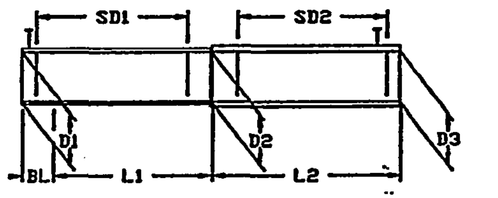


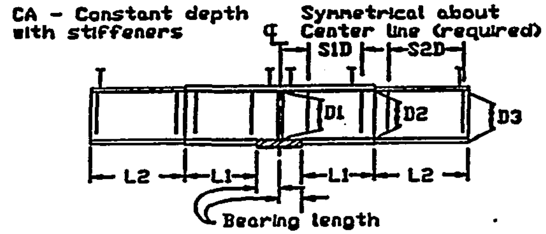
FIGURE 3.2 - Bridge System Composed of Fabricated Components



(a) Composite Action - Abutment Girder (Type 1)



(b) Composite Action - Intermediate Girder (Type 2)



(c) Composite Action - Pier Girder (Type 3)

1	FAB. COMP. NUMBER	= 1
2	FAB. COMP. TYPE	= 1
3	BEARING LENGTH (in)	= 12
4	SEG.-1 LENGTH (L1-ft)	= 20
5	SEG.-2 LENGTH (L2-ft)	= 20
6	START DEPTH (in)	= 70
7	NEXT DEPTH (in)	= 70
8	NEXT DEPTH (in)	= 70
9	THK. TOP FLG. -SEG.1	= .75
10	WID. TOP FLG. -SEG.1	= 14
11	THK. BOTT. FLG. -SEG.1	= .75
12	WID. BOTT. FLG. -SEG.1	= 14
13	WEB THICKNESS -SEG.1	= .5
14	THK. TOP FLG. -SEG.2	= .75
15	WID. TOP FLG. -SEG.2	= 14
16	THK. BOTT. FLG. -SEG. 2	= .75
17	WID. BOTT. FLG. -SEG. 2	= 14
18	WEB THICKNESS -SEG. 2	= .5
19	NUM. OF SIDES FOR STIFF.	= 0
20	STIFFENER THICKNESS (in)	= 0
21	STIFFENER WIDTH (in)	= 0
22	FIRST STIFF. SPACE (in)	= 0
23	STIFF. SPA'G SEG.-1 (in)	= 0
24	STIFF. SPA'G SEG.-2 (in)	= 0
25	SHEAR STUD CODE	= 0
26	STUD SPA'G SEG.-1 (in)	= 0
27	STUD SPA'G SEG.-2 (in)	= 0
28	NUM. OF BEAR'G STIFF.	= 0
29	BEAR'G STIFF. THK. (in)	= 0
30	BEAR'G STIFF. WID. (in)	= 0

1	FAB. COMP. NUMBER	= 2
2	FAB. COMP. TYPE	= 2
3	BEARING LENGTH (in)	= 0
4	SEG.-1 LENGTH (L1-ft)	= 20
5	SEG.-2 LENGTH (L2-ft)	= 20
6	START DEPTH (in)	= 70
7	NEXT DEPTH (in)	= 70
8	NEXT DEPTH (in)	= 70
9	THK. TOP FLG. -SEG.1	= .75
10	WID. TOP FLG. -SEG.1	= 14
11	THK. BOTT. FLG. -SEG.1	= .75
12	WID. BOTT. FLG. -SEG.1	= 14
13	WEB THICKNESS -SEG.1	= .5
14	THK. TOP FLG. -SEG.2	= .75
15	WID. TOP FLG. -SEG.2	= 14
16	THK. BOTT. FLG. -SEG. 2	= .75
17	WID. BOTT. FLG. -SEG. 2	= 14
18	WEB THICKNESS -SEG. 2	= .5
19	NUM. OF SIDES FOR STIFF.	= 0
20	STIFFENER THICKNESS (in)	= 0
21	STIFFENER WIDTH (in)	= 0
22	FIRST STIFF. SPACE (in)	= 0
23	STIFF. SPA'G SEG.-1 (in)	= 0
24	STIFF. SPA'G SEG.-2 (in)	= 0
25	SHEAR STUD CODE	= 0
26	STUD SPA'G SEG.-1 (in)	= 0
27	STUD SPA'G SEG.-2 (in)	= 0
28	NUM. OF BEAR'G STIFF.	= 0
29	BEAR'G STIFF. THK. (in)	= 0
30	BEAR'G STIFF. WID. (in)	= 0

1	FAB. COMP. NUMBER	= 3
2	FAB. COMP. TYPE	= 3
3	BEARING LENGTH (in)	= 12
4	SEG.-1 LENGTH (L1-ft)	= 20
5	SEG.-2 LENGTH (L2-ft)	= 20
6	START DEPTH (in)	= 70
7	NEXT DEPTH (in)	= 70
8	NEXT DEPTH (in)	= 70
9	THK. TOP FLG. -SEG.1	= .75
10	WID. TOP FLG. -SEG.1	= 14
11	THK. BOTT. FLG. -SEG.1	= .75
12	WID. BOTT. FLG. -SEG.1	= 14
13	WEB THICKNESS -SEG.1	= .5
14	THK. TOP FLG. -SEG.2	= .75
15	WID. TOP FLG. -SEG.2	= 14
16	THK. BOTT. FLG. -SEG. 2	= .75
17	WID. BOTT. FLG. -SEG. 2	= 14
18	WEB THICKNESS -SEG. 2	= .5
19	NUM. OF SIDES FOR STIFF.	= 0
20	STIFFENER THICKNESS (in)	= 0
21	STIFFENER WIDTH (in)	= 0
22	FIRST STIFF. SPACE (in)	= 0
23	STIFF. SPA'G SEG.-1 (in)	= 0
24	STIFF. SPA'G SEG.-2 (in)	= 0
25	SHEAR STUD CODE	= 0
26	STUD SPA'G SEG.-1 (in)	= 0
27	STUD SPA'G SEG.-2 (in)	= 0
28	NUM. OF BEAR'G STIFF.	= 0
29	BEAR'G STIFF. THK. (in)	= 0
30	BEAR'G STIFF. WID. (in)	= 0

FIGURE 3.3 - Individual Fabricated Components

girder webs and composite or noncomposite roadway and girder action, there is a total of 244 specific fabricated component combinations that are possible. The important concept is that the data stored in the computer database for each fabricated component represents every piece of information required to physically order the material, fabricate the component, assemble, etc., with no outside intervention or interpretation. Utilizing this formulation process, the "total structural design" becomes a computer-assisted process.

The behavior of the structural system represented in the analysis and design process is dependent upon the properties of the entire member (girder), not the properties of the individual fabricated components. For the formulation of theoretical structural members, a SPAN matrix is derived from keying off the input of fabricated component types, i.e., abutment, intermediate, or pier girder type. The SPAN matrix is dimensioned such that the number of rows is equal to the total number of theoretical structural members. The number of columns equals four, which represents the maximum number of fabricated components allowed to comprise one girder. Each fabricated component is assigned a number when input. This then is placed into the SPAN matrix in the appropriate location. For example, if a structural member is the third member of a girder and has two parts, 7 and 8, then element SPAN(3,1) equals 7, element SPAN(3,2) equals 8, and elements SPAN (3,3) and SPAN(3,4) are both equal to zero. The control of the CASE methodology is derived from the interaction of the fabricated component types, fabricated component matrix, Fcomd, and the SPAN matrix. These are the central tools of CASE development; however, other aspects of CASE are briefly discussed in Section 3.5.

3.3 Database Development and Management Strategy

A key element in the implementation of computer-assisted techniques to structural engineering problems is the flow of information between program segments. Within the CASE methodology, a new database architecture has been developed which is significantly different in its approach to data management than currently existing data models, including the AISC database for hot-rolled sections. In the following sections, a general overview of database structures is given and the CASE database is discussed in detail.

3.3.1 General Database Planning and Organization

In order to achieve successful implementation of a CADD/CAM system, all interfacing between the various program segments must be dealt with effectively. The efficient structuring of data is critical to both storage needs and execution speed. The most effective way of managing this transfer of information is through the use of a database. In planning and organizing an effective database, a rational procedure must be developed to integrate communicating, processing, and databasing into a coherent, comprehensive information system. An orderly database is comprised of data elements stored in an organized, planned fashion. A properly organized database consists of a set of named database segments in which each segment is a collection of named files. Each file consists of an orderly set of uniform records; each record is composed of a collection of named fields; and so on, to the smallest addressable information element.

The acquisition, storage, processing, retrieval, presentation, and dissemination of information in a manner which will meet the needs of the user are the first basic objectives which must be satisfied in the implementation of a database. The acquisition method employed is probably the single most important factor influencing how much the system will be used. An efficient, easy-to-use method of entering information into the database will be invaluable when considering savings of data input errors and overall modeling time. A particularly user-friendly data input scheme has been developed for use in the CASE methodology. This input scheme is described in depth in Section 6.3.1.

The design information recorded to the database through use of the data input segment must contain not only descriptions of the geometric and physical properties of the elements of a structure but also information about the relationships between various segments. These relationships may be geometric (i.e., geometric and topological data) or physical (i.e., attributes). The geometric and topological data are necessary to define the basic physical element of a structure. The geometry represents the dimensions and spatial location of each element, while the topology describes the connections between the elements. The topological definition of each element must be accompanied by the corresponding geometric description in order to fix its position in space. Attributes define the physical composition of an element and describe its functional characteristics. Physical properties (area, depth, thickness, etc.), response characteristics (force, moments, etc.), and design properties (yield stress, modulus of elasticity, etc.) are entered into the database as attributes.

When considering the relationship between the database and the data processing, i.e., data manipulation, one requirement becomes evident. The database must be available to all applications program modules and no module should have its own database. One of the key objectives of an effective database must be independence. Allowing different applications access to the same data eliminates needless redundancy in the storage of necessary information. This independence allows changes to be made to either the data or the program without requiring changes to the other. Thus, the database need not be altered to accommodate new programs and existing programs are not affected by changes to the data structure.

The objective of data independence is to insulate the applications programs from the data management techniques. Thus, the independent database structure [22] must provide for:

* <u>data definition</u>: defines the database and builds the framework into which the attributes are placed; data definition is performed by the database administrator using a data definition language (DDL).
* <u>data modification</u>: includes insertion, modification, and deletion of data values; performed by the user with a data manipulation language (DML).
* <u>data retrieval</u>: consists of obtaining desired information from the database and includes the ability to search, manipulate, and query without the need to write application programs; performed by the user using a structured query language (SQL).

As a result of this database independence, any application program can communicate with the database using a host language interface. This interface initiates the desired internal operations of the database.

The ease of storage and retrieval of information from the database is influenced largely by the data model chosen for structuring the database. There are three database models currently in common use: the hierarchical model, the network model, and the relational model. The hierarchical and network models are extremely limited in their applicability and will be covered only briefly here. The standard relational model is the basis of the database developed for use in the CASE methodology and will be discussed in depth.

The hierarchical model is a multi-level data model composed of nodes and links in a tree structure as shown in Figure 3.4. The records in the highest level control the records in the intermediate level which, in turn, control the records in the lowest level. In order to access information stored in the lowest level, the application program must search the highest level and, then, the intermediate level before reaching the level at which the desired information is stored.

A network model is also a multi-level database model. However, in the network model, each node may be linked to other nodes in both upward and downward directions, as shown in Figure 3.5. Again, it is apparent that desired information cannot be accessed directly but is obtained by navigating through the database files until the required data is located. It is clear that the hierarchical and network models share some of the characteristics which cause them to be ill suited for unusual or extremely varied data inquiries. In fact, the hierarchical model can be considered to be a special case of the network model.

The standard relational database is different from the hierarchical and network data models in a number of ways. In a relational data model, there are no predetermined paths between files of information. An illustration of a relational model similar to Figures 3.4 and 3.5 would simply be a set of nodes with no links between them. The relational database automatically creates the required links upon demand.

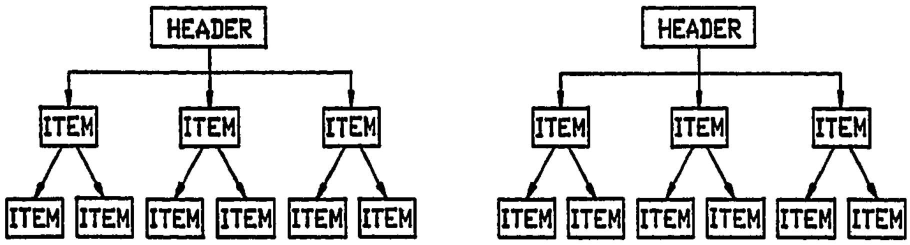


FIGURE 3.4 - Hierarchical Data Model

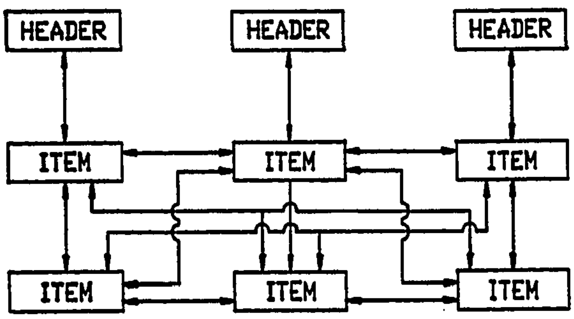


FIGURE 3.5 - Network Data Model

The actual structure of a relational database, however, is not represented by a set of independent nodes. A relational model is a single-level model illustrated by a collection of two-dimensional tables. Each table is called a relation. The rows of a relation are called tuples and the columns are termed attributes. All attribute values are drawn from the same domain, i.e., they are of the same data type. Each tuple represents a distinct entity and contains a value for each attribute. Tuples and attributes have no order and they may be arbitrarily interchanged without changing the data content and/or meaning of the relation. Tuples are accessed by means of a key, a single attribute or group of attributes that uniquely define the tuple. Figure 3.6 illustrates the structure of a relation. A standard notation is used to represent relations. This notation lists the name of the relation which is followed by the attributes of the relation enclosed in parentheses.

Relation Name	Attribute 1	Attribute 2	Attribute 3	Attribute 4	Attribute 5
Tuple 1	Value 1,1	Value 1,2	Value 1,3	Value 1,4	Value 1,5
Tuple 2	Value 2,1	Value 2,2	Value 2,3	Value 2,4	Value 2,5
Tuple 3	Value 3,1	Value 3,2	Value 3,3	Value 3,4	Value 3,5
Tuple 4	Value 4,1	Value 4,2	Value 4,3	Value 4,4	Value 4,5

FIGURE 3.6 - Relational Data Model

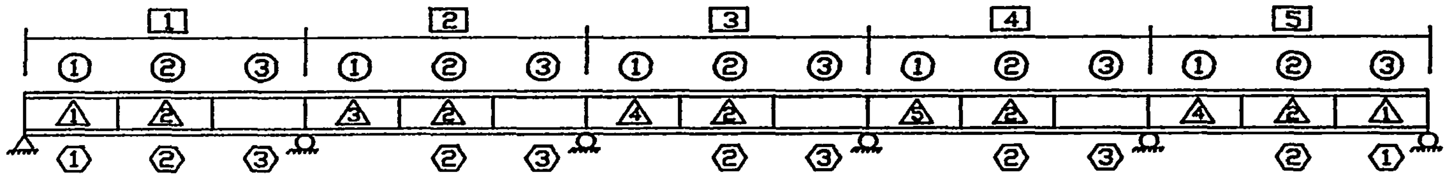
3.3.2 Database for the CASE Methodology

Many of the database facilities discussed in the previous section have been included in development of the CASE database. The database is formulated independently of the other segments developed within the CASE methodology. This, of course, is a primary requirement for supporting the flexible design sequences of CASE where the program segments may interact in a variety of sequences, where multiple iterations must be performed, and where multiple alternative designs may have to be generated and compared.

Information to be stored in the CASE database is represented as either geometric, topological, or attribute data. The attribute data stored in the database are the true physical properties and characteristics of each fabricated component type. The inclusion of attribute data is what takes us beyond simple computer-aided drafting applications; it is the basis for computer-aided analysis/design techniques and allows extension into the computer-aided manufacturing environment. This information allows the design under consideration to be formulated in such a way that the fabrication plant can draw from the design database in order to manufacture the required components for construction and coordinate this with shipping constraints.

While the CASE database is basically a relational-type data model, the architecture of the CASE database has been extended and enhanced beyond the characteristics outlined previously of the standard relational database. The standard relational data model is a single-level model. The CASE database is more complex in that it is a multi-level model. The first level of the database stores the job name and, in the case of GBRIDGE, the number of girders, roadway width, etc., including a relation corresponding to the SPAN matrix. The second level includes a relation analogous to the Fcomd matrix, which includes all of the physical properties and characteristics which completely describe each fabricated component type. The data is stored here when entered during the Data Input segment of the CASE methodology.

Figure 3.7 illustrates the basic link between the SPAN relation and the Fcomd relation levels of the CASE database. For the bridge span indicated, the relation SPAN holds the geographic data; that is, part locations are described by addressing the particular theoretical structural member and, further, the specific fabricated component of those which comprise the member. It is clear, then, from Figure 3.7a, that the first fabricated component of the first theoretical structural member is part number one, the third fabricated component comprising the third theoretical structural member is part number five, and so on. These part numbers are



[N] = Theoretical Structural Member
(N) = Fabricated Component
△N = Part Number

⟨N⟩ = Part Type
1 - abutment
2 - intermediate
3 - pier

(a) Theoretical Bridge Span

Fabricated component / Theoretical Structural Member	#1	#2	#3	#4
#1	1	2	3	0
#2	3	2	4	0
#3	4	2	5	0
#4	5	2	4	0
#5	4	2	1	0

(b) Span Relation

Part Number	Part Type	Web Thickness	Thickness Top Flange	Thickness Bottom Flange	Width Top Flange
1	1	~	~	~	~
2	2	~	~	~	~
3	3	~	~	~	~
4	3	~	~	~	~
5	3	~	~	~	~

(c) Fcomb Relation

FIGURE 3.7 - Illustrative Example of CASE Database

stored in the relation SPAN as shown in Figure 3.7b.

When the user is operating the processing segments of CASE, the data required for the analysis/design is accessed from the database in a two-step process. When part data is requested, the database accesses the SPAN relation, which contains the part number to analyzed. When the part number to be considered has been determined, the application segment in use then accesses the corresponding tuple of the Fcomd relation and loads the data stored in that location into the Fcomd matrix for use in the data manipulation process.

The type of data stored in the CASE database marks an important distinction between the CASE methodology and previous attempts in the area of CADD/CAM when applied to civil engineering type structures. Heretofore, most engineering databases have been structured to store data pertaining to sectional properties as well as the physical characteristics of the theoretical members. A major concept developed within the CASE methodology is to regenerate data when possible and practical, thus eliminating unnecessary storage of data. The CASE database holds specifically that information which must be stored for use by the fabricator when manufacturing the components needed for construction, i.e., the fabricated component data. By storing simple and efficient algorithms, the computer can regenerate section properties, such as moment of inertia, section modulus, cross-sectional area, etc., in a fraction of the time required to search the enormous number of bits of information in the database to find the required values. As an example, the section of the CASE source code used to generate the composite/noncomposite moments of inertia is shown in Figure 3.8.

It is clear that an efficient and carefully structured database is critical to the CADD/CAM process for civil engineering type structures. The data entered into the database must be available to the wide range of analysis and

```
6740 SUB Mom_inertia(Ttf,Wtf,Tbf,Wbf,Tw,Dw,Imna,Nn,Sl,Ip,Areastl)
6750 !*************************************************************************
6760 !
6770 !       SECTION MOMENT OF INERTIA ROUTINE
6780 !
6790 !*************************************************************************
6800 COM /Bxsect/ Gs,Sdl,Haunch,Tcrt,A_cstl,Y_cstl
6810 !
6820 !  DETERMINE EFFECTIVE CONCRETE WIDTH
6830 B=Sl/4
6840 IF Gs<B THEN B=Gs
6850 IF 12*Tcrt<B THEN B=12*Tcrt
6860 !
6870 !  CALCULATE I
6880 Areastl=Ttf*Wtf+Tbf*Wbf+Tw*Dw
6890 Iarea=Areastl
6900 Imom=Ttf*Wtf*(Dw/2+Ttf/2)-Tbf*Wbf*(Dw/2+Tbf/2)
6910 IO=Ttf^3*Wtf/12+Tbf^3*Wbf/12+Dw^3*Tw/12
6920 IF Nn=0 AND A_cstl=0 THEN 7010
6930 IF Nn<>0 THEN
6940   Imom=Imom+B/Nn*Tcrt*(Dw/2+Ttf+Haunch+Tcrt/2)
6950   Iarea=Iarea+B/Nn*Tcrt
6960   IO=IO+(Tcrt^3*B/Nn)/12
6970 ELSE
6980   Imom=Imom+A_cstl*(Dw/2+Ttf+Haunch+Y_cstl)
6990   Iarea=Iarea+A_cstl
7000 END IF     ! since IO=IO+0 for reinforcing stl
7010 Imna=Imom/Iarea
7020 Ad2=Ttf*Wtf*(Dw/2+Ttf/2-Imna)^2+Tbf*Wbf*(Dw/2+Tbf/2+Imna)^2+Dw*Tw*Imna^2
7030 IF Nn=0 AND A_cstl=0 THEN 7090
7040 IF Nn<>0 THEN
7050   Ad2=Ad2+Tcrt*B/Nn*(Dw/2+Ttf+Haunch+Tcrt/2-Imna)^2
7060 ELSE
7070   Ad2=Ad2+A_cstl*(Dw/2+Ttf+Haunch+Y_cstl-Imna)^2
7080 END IF
7090 Ip=IO+Ad2
7100 SUBEXIT
7110 SUBEND
7120 !
```

FIGURE 3.8-Computer Code for Section Properties Calculation

design sequences needed in diverse projects, such as the unique structures encountered in civil and structural engineering. The newly-developed architecture applied to the CASE database has been created specifically to allow the extension from CADD into CAM which has been so long desired. The structure of this database permits the fabricator to obtain the information needed to:

* order material
* schedule plant operations .
* operate numerically-controlled machines
* control shipping and inventory.

The data exchanged and shared among applications is the key to integration. Successful transfer of data between program segments requires that the program segments be interactive and modular. Interactive means that the program has multiple uses of instructions and controls by the user. Modular structure allows for separate segment operation; the program segments can be executed independently or as a custom design package, as required for the given structure under consideration.

3.4 Interactive Control and Modification

The CASE methodology employs interactive programming, thus, allowing the engineer to direct program flow via keyboard response to intermediate program results. In essence, the engineer effectively controls the design process by interpreting intermediate results and directing the computer through the desired calculations until a finalized design is achieved. This process can be contrasted to batch mode processing in which all data is input at the initial stage and is processed continuously until the finalized output is printed and/or displayed, with no allowance for interpretation of intermediate results. If the engineer is not satisfied with the proposed design, the entire batch process must be repeated. However, in certain checking processes, the batch mode is desirable and has been incorporated as an option in the developed superstructure program.

In totally interactive computing, the designer can combine his knowledge, experience, and judgment with the power and speed of the computer. The engineer draws upon codes and specifications as well as his experience and knowledge to formulate all criteria which must be investigated for each design. Operating with a computer equipped with graphics facilities, the designer can engage in a "dialogue" with the computer in such a way that both designer and computer are used to best advantage. As an example, the graphic display of the CASE stress summary will allow the engineer to visually verify the design and, if modifications are required, readily implement them. The modification procedure is rapid since the program development has been modular and structured (in BASIC).

It should be noted that a limitation of implementation of CASE on certain microcomputers is that they cannot match contemporary or older minicomputers or mainframe computers in execution speed. However, when one notes that, in the interactive processing mode, much of the session is run at the speed of response of the designer and not at the speed of execution of the computer, one realizes that the higher computational speed of the large computer is largely untapped. Nevertheless, certain computational aspects are affected by both the available memory size and execution speed of the microcomputer.

3.5 Programming Aspects of CASE Methodology

All programs which perform numeric operations are composed of three distinct phases: preprocessing, processing, and postprocessing. The functional aspects of each are:

preprocessing: the input of data; preliminary sorting and problem formulation

processing: the utilization of this data in program calculations

postprocessing: output of program results

The CASE methodology actually modifies all three phases as normally employed in structural engineering programs.

In the preprocessing phase, input data is garnered from the fabricated components used in the actual structure.

Also, to help in determining whether a cross section is adequate for a given load, analysis points are generated along the member length, or theoretical axis, at key locations. These locations are based on a criteria of material changes and also are located at n/10 of the span length, where n equals 1 to 10. Since the section properties may change at material breakpoints, section points are programmatically generated to include the effect of the true variation of member properties throughout the bridge system. This conceptualization is unique to the CASE methodology. Figure 3.9 illustrates the locations of these points along the span. These analysis points are the basis for the formulation of the processing (analysis) phase.

Analytical investigation of structural systems has been accomplished on microcomputer using traditional matrix finite element methods. However, the CASE methodology, as

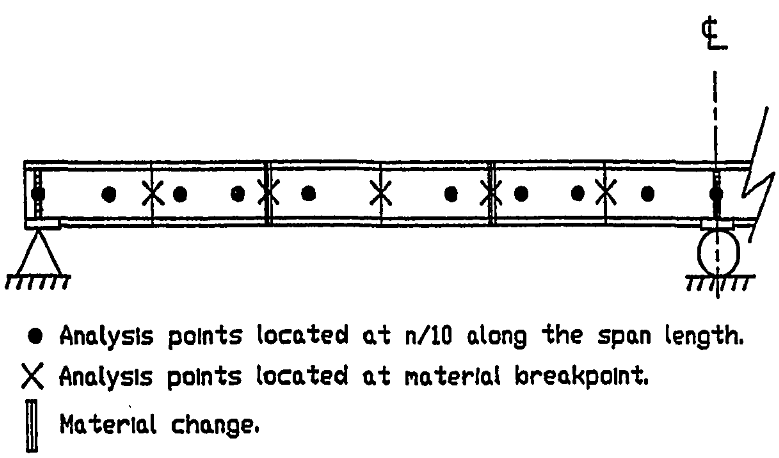


FIGURE 3.9 - Analysis Points

applied to girder bridges, utilizes a theoretically exact formulation of the element stiffness matrix which is the basis of the processing phase. Nonprismatic girders are being designed commonly for highway bridges. Hence, the nonprismatic beam elements are essential for application in the analysis of such girders to produce more accurate analysis for design. The formulation and analysis application of the element stiffness matrix is examined in detail in Chapter 4.

The postprocessing phase consists of output of the information generated in the processing (or analysis) phase. At present, some of the important features of the output of the postprocessing segment include:

* graphical display of actual and allowable stress variation throughout the bridge system
* bridge elevations and plans
* total cost information for the bridge system and optimized bridge sections (including concrete roadway and steel girders)
* hard copy of output, including all bridge information required for geometry, material properties, and cross-sectional sizes.

CASE also allows various levels of output, from a complete report including all available data to the printout of a specific table or graph. In a CADD/CAM environment, this data can be used to create engineering drawings, order fabricated components from inventory, instruct numerically controlled machines to fabricate needed components, etc. As previously discussed, the actual extension of the program beyond CADD is not included in the scope of this research. However, the database has been structured to allow for ready and immediate implementation of computer-aided manufacturing.

IV. BRIDGE ANALYSIS

4.1 General

The analysis of the bridge system is, in itself, a rather complex task given that the girders must be designed for an envelope of shear and moment forces produced from sets of moving loads. These moving loads represent the effect resulting from vehicles traversing the bridge superstructure and the loads are distributed laterally to the supporting girders through the roadway deck slab. This lateral distribution effect to nonprismatic continuous girders occurs simultaneously with the moving loads and is one of the major influences that complicate the bridge analysis.

All methods of structural analysis are concerned essentially with solving the basic equations of equilibrium and compatibility. Direct analytical solutions are limited to cases where the load distribution, section properties, and boundary conditions can be described by simplistic mathematical expressions but, for complex structures, numeric methods generally are more reliable and efficient. In the CASE-GBRIDGE methodology, a modified numerical analysis method of displacement-based finite elements is employed.

This chapter will examine (a) the formulation and development of the nonprismatic element stiffness matrix, (b) the development of the global stiffness equation and influence line generation, and (c) application of AASHTO loading requirements.

4.2 Nonprismatic Element Stiffness Matrix Development

The procedure for applying the standard displacement based finite element method can be stated as [71,72]:

i) The structure is discretized into a finite number of simple geometric subregions, called elements.

ii) The elements are assumed to be interconnected at a discrete number of nodal points situated on the element boundaries. The degree of freedom at the nodes, called nodal displacements (unknowns), normally refer to the displacement at the nodes but can also include other terms such as stresses and strains.

iii) A shape function, in terms of the nodal displacement parameters,is chosen to represent the displacement field within each element. Based on the shape function, a stiffness matrix is written to relate the nodal forces to the nodal displacement parameters. Based upon the applied loading, nodal forces can be formulated and a set of simultaneous nodal force displacement equations generated and solved.

The solution yields the unknown nodal displacements which, through application to the element shape functions, are related to internal member forces and displacement at any specified analysis points. The key in the finite element analysis process is the development and application of an efficient and reliable element stiffness matrix.

Nonprismatic beam elements are essential for accurate and reliable analysis of girder bridge systems since the supporting girders are generally nonprismatic members. This usage of nonprismatic members presents difficulties in employing the traditional finite element method (FEM) analysis approach. As stated, the FEM procedure assumes a continuous shape (displacement) function in formulating the element stiffness matrix. In the case of segmentally nonprismatic beams, however, any approximating shape function which represents the entire girder length is required to be discontinuous. The reasoning for this discontinuity requirement for the shape function can be explained by examining the moment-curvature relationship,

$$\frac{d^2y}{dx^2} = \frac{M_x}{EI_x}$$

in which

y = displacement of the neutral axis

x = location at any point on member x

M_x = moment at location x

I_x = moment of inertia at location x

E = modulus of elasticity at location x.

Analogous to the shape function, which represents the element displacement between nodal degrees of freedom, in this equation, y represents the displacements due to bending of the beam member's neutral axis as a function of the member's length, x. The problem is that on either side of a material change, as shown in Figure 4.1, the internal resisting moment M_x is the same but the member's neutral axis location and moment of inertia are different. Therefore, any analysis formulation must account for this discontinuity. Application of FEM assumes a continuous shape function or interpolating polynomial, however, so the method can only lead to exact answers under conditions of y

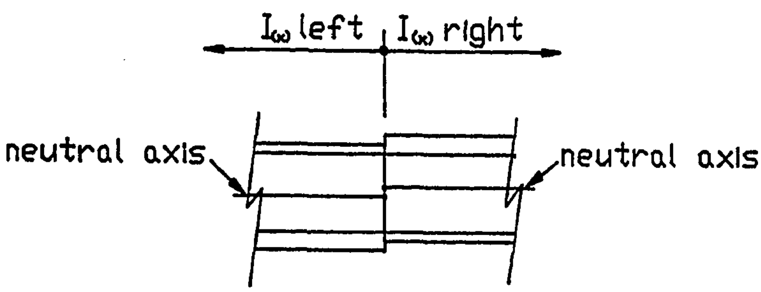


FIGURE 4.1 -- Partial Nonprismatic Girder

being continuous, i.e. for prismatic girders. The problem is that virtually all girder bridges have segmental, nonprismatic supporting girders.

This discontinuity problem can be overcome by application of the traditional FEM method provided that the girder is modelled by a series of prismatic beam elements. Each prismatic segment can utilize a continuous shape function because, for each segment, the neutral axis location remains constant. In other words, for each individual segment of the series beam elements modeling the girder, the flexural rigidity (EI) remains constant, although EI can vary for each segment. This type of formulation for nonprismatic girders requires a large number of prismatic beam elements to be employed to obtain accurate analytical results. This segmental formulation requires a large amount of the available computer RAM (random access memory) and requires considerably longer execution time to solve the resulting increased number of simultaneous equations. When application is intended for a computer system with limitations on available memory and execution speed, this segmental formulation process is obviously unacceptable. Thus, attention must be focused on applications of accurate elements using only a reduced degree of freedom system and avoiding, to some extent, the limitations imposed by the displacement function approach. This problem of nonprismatic element stiffness development is overcome by integration of classical beam theory employing numeric integration and traditional displacement-based finite element analysis.

The nonprismatic element stiffness matrix developed for CASE-GBRIDGE is formulated in two parts; flexural contribution and axial contribution. The related global degrees of freedom are illustrated in Figure 4.2. The flexural contributions to the girder stiffness matrix assume that the girder is bent in a principal plane and the effects of shear deformations can be neglected. Employing these assumptions plus the fact that the angle change between two adjacent cross sections is small after bending has occurred, an efficient procedure for formulating the element stiffness matrix can be utilized. The nonprismatic element formulation process employes the classical analysis approach of superposition in which the indeterminate structure is reduced to a statically stable and determinate structure via removal of redundant end moments M_L and M_R. These redundant end moments then are re-applied and the resulting member end

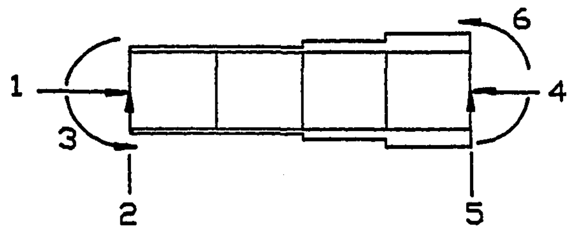


FIGURE 4.2 - Nonprismatic Element Degree of Freedom

rotations are related to the fact that the actual rotations at fixed ends are zero. Manipulating the solution of the resulting simultaneous equation will yield the nonprismatic element stiffness matrix and equivalent nodal forces. The expressions for simple beam member end rotations [77] are obtained by noting that the tangential deviation divided by the member length, L, is equal to the member end slope for small deflections theory, i.e.

$$\theta_L = -\int_0^L \frac{L-x}{L} \frac{M_x dx}{EI_x}$$

$$\theta_R = \int_0^L \frac{x}{L} \frac{M_x dx}{EI_x}$$

where

$\theta_L = \tan\theta_L$ = slope at left end

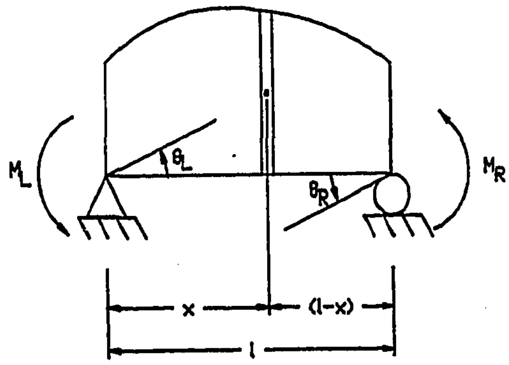
$\theta_R = \tan\theta_R$ = slope at right end

I_x = moment of inertia at reference location x

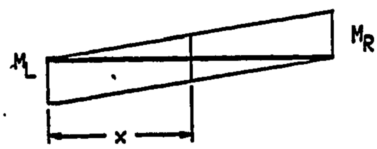
These terms are illustrated in Figure 4.3a.

The moment at any location x in terms of member end moments as defined in Figure 4.3b is given by

$$M_x = [(\frac{M_L + M_R}{L})x - M_L].$$



(a) End Rotation (M_L and M_R removed)



(b) Internal Moment at location x in terms of End Moments M_L and M_R

FIGURE 4.3 - Nonprismatic Element-Flexural Formulation

Substituting this moment expression for M_x into the member end slope expressions and multiplying through the resultant equations by $I_L L/I_L L$ yields the end slopes in terms of basis stiffness. The basis stiffness is given by EI_L , and is defined as the product of the modulus of elasticity, E, and the moment of inertia at the girder's left end, I_L. When the girder is composite, the concrete slab is equated to an equivalent steel contribution by employing the mechanics of materials transform section method. The flexural stiffness coefficients are evaluated by noting that the element stiffness is the resulting member end forces when a unit distortion is applied (all other possible displacements are held to zero). Thus restraining the opposite end rotations and noting that the flexural stiffness is equal to the member end moment divided by the rotation, the flexural stiffness components are given as

$$[S_{e(\text{flexural})}] = \frac{EI_L}{L^3(AC-B^2)}\begin{bmatrix} A & (B)L & -A & (A-B)L \\ (B)L & (C)L^2 & (-B)L & (B-C)L^2 \\ -A & (-B)L & A & [-(A-B)]L \\ (A-B)L & (B-C)L^2 & [-(A-B)]L & [A-2B+C]L^2 \end{bmatrix}$$

where

$$A = I_L \int_0^L \frac{dx}{LI_x} \qquad B = I_L \int_0^L \frac{xdx}{L^2I_x} \qquad C = I_L \int_0^L \frac{x^2dx}{L^3I_x}$$

and I_L = moment of inertia at left end of member

L = length of member

x = variable location along member length

This can be expressed in general terms as:

$$S_{e(\text{flexural})} = \text{FACT}\begin{bmatrix} S_{22} & S_{23} & S_{25} & S_{26} \\ S_{32} & S_{33} & S_{35} & S_{36} \\ S_{52} & S_{53} & S_{55} & S_{56} \\ S_{62} & S_{63} & S_{65} & S_{66} \end{bmatrix}$$

where

$$\text{FACT} = \frac{EI}{L^3}\left[\frac{1}{AC - B^2}\right]$$

and the subscripted values s_{ij} indicate the location in the element 6x6 stiffness matrix where i defines the row and j defines the column. It must be noted that, in evaluating the moment of inertia of composite girders by the transformed section properties approach, three individual conditions are examined based upon loading type. Initially, before the concrete hardens in the composite section, the dead load must be carried by the supporting steel girder alone. Loads which are applied after the concrete slab is in place and hardened are resisted by the composite section. The long term effects of the dead load are based upon the composite section using a modular ratio of three times the initial value to account for concrete creep and shrinkage effects. The live loading condition (short term duration) is based upon consideration of the composite section using a modular ratio of n to account for full composite compressive action of the concrete in evaluating I.

The nonprismatic stiffness coefficients due to flexure have been derived as closed-form integrals in terms of natural or global coordinates. The formal integration of these coefficients is very tedious and susceptible to error and, since each new girder would require individual evaluation, formal integration is neither practical nor efficient for computer implementation. Instead, these integrations must be obtained numerically. All of the numeric integration methods, or quadrature formulas as generally termed, deal with approximating the integral by a weighted sum of the values of the integrand of points on the interval of integration; that is, the quadrature formulas amount to an approximation of the form;

$$I = \int_a^b f(x)dx = \sum_{i=1}^{n} W_i f(x_i)$$

where $a \leq x_1 < x_2 < \ldots < x_n \leq b$

The numbers W_i are termed the "weighting coefficients" and the points x_i at which the function is to be evaluated are generally termed "sampling points." The quadrature formula then can be viewed as the proper selection of particular weighting coefficients and sampling points to numerically approximate the given integral. The technique employed in selecting the weighting coefficient and sampling points is what distinguishes the various numeric integration methods.

One particular method, Gaussian quadrature, allows the sampling points, termed gauss points, to be chosen such that the best possible accuracy is obtained for a specific polynomial order of the function f(x). The method yields "exact" answers for a polynomial function of the order "p" provided the number of gauss points "n" is greater than

$$n = \frac{p+1}{2}$$

The gauss points represent the location of the abscissas at which the polynomial is to be evaluated in local coordinates. The local coordinate system employed by Gaussian quadrature requires the limits of integration to occur between $-1 \leq x \leq 1$. Thus, the integrals must be normalized and a coordinate transformation employed. The Gaussian quadrature formula is

$$I = \int_a^b f(x)dx = \int_{-1}^{1} f(\xi)d\xi = \sum_{i=1}^{n} W(\xi_i)f(\xi_i) \qquad \text{Equation 4.1}$$

The application of Gaussian quadrature to evaluate the nonprismatic element stiffness matrix flexural coefficients is readily accomplished via direct application of Equation 4.1. Referring to Figure 4.4, the location of any point x in the global system can be expressed in terms of local coordinates by

$$x = \xi(\text{linear variation}) + \xi(\text{axis location})$$

$$= \xi\left(\frac{b_i - a_i}{2}\right) + \left(\frac{a_i + b_i}{2}\right)$$

$$x = \frac{\xi(b_i - a_i) + (a_i + b_i)}{2}$$

$$\text{and } dx = \left(\frac{b_i - a_i}{2}\right)d\xi.$$

Employing these transformation relationships, the element coefficients can be readily evaluated by Gaussian quadrature in the local coordinate system. The accuracy of employing this technique is demonstrated by application to two

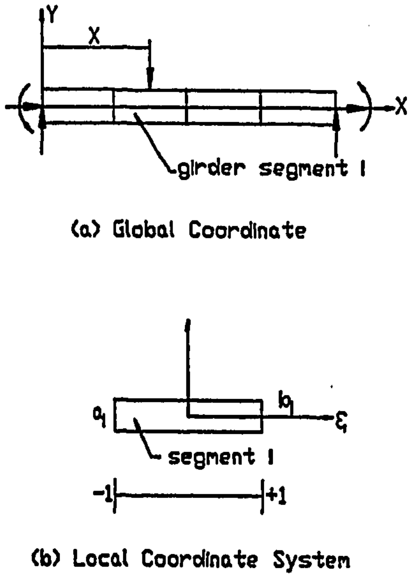


FIGURE 4.4 - Coordinate Transformation

illustrative problems given in Figures 4.5 and 4.6. The final member end moments are evaluated in these problems by a theoretically exact approach (virtual work), Gaussian quadrature, and the traditional FEM segmental beam element method. The results are given in Tables 4.1 and 4.2. In the first problem, all results are identical; however, the segmental beam approach (using three beam segments) required twice the amount of computer memory storage and execution speed in comparison to the CASE-GBRIDGE method. In the variable depth problem, not only did the segmental approach require eight times more memory (15 segments) and execution speed, it was considerably less accurate. In employing Gaussian quadrature to evaluate the nonprismatic flexural

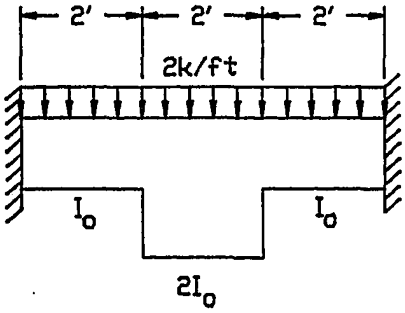


FIGURE 4.5 - Segmental Nonprismatic Beam Example

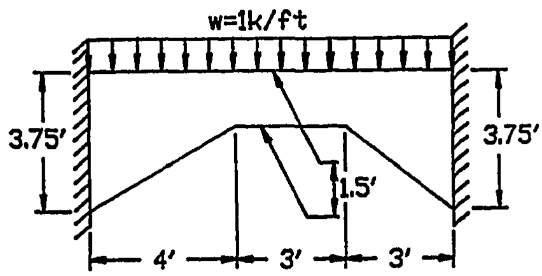


FIGURE 4.6 - Tapered Nonprismatic Beam Example

TABLE 4.1 - Solution Comparison for Segmental Beam

Member End	MEMBER END MOMENTS (ft-k)		
	Theoretically Exact	CASE-Quadrature Solution	Traditional FEM
Left End	5.47	5.47	5.47
Right End	5.47	5.47	5.47

TABLE 4.2 - Solution Comparison for Tapered beam

Member End	MEMBER END MOMENTS (ft-k)		
	Theoretically Exact	CASE-Quadrature Solution	Traditional FEM
Left End	11.37	11.39	11.74
Right End	10.28	10.34	10.71

element coefficients for variable depth girders, the moment of inertia, I_x, initially is evaluated at the gauss point and treated as a constant; then, the remaining numeric integration is performed. The gauss point locations and weighting coefficients are presented in Appendix A, along with the numeric calculations for the illustrated problems and equivalent nodal load information.

The complete nonprismatic element stiffness matrix is obtained by combining the flexural and axial contributions. The axial contribution is based upon standard displacement-based FEM by employing the assumption of centroid segment alignment. The concrete roadway system is neglected in considering axial effect, i.e., only the supporting steel girders are considered to carry axial loads. The variation in the girder cross-sectional area, A_x, can be expressed as

$$A_x = A_L[1 + t_w(h_r - h_L)x] \qquad \text{Equation 4.2}$$

and, again,

$$x = \frac{\xi(b - a) + (a + b)}{2} \quad \text{and} \quad dx = \left(\frac{b - a}{2}\right)d\xi.$$

The element stiffness axial coefficient formulation can be expressed as

$$[S_{e\ axial}] = \int_v [B]^T[E][B]dv$$

$$= \frac{E}{L^2} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \int_V^L dv$$

$$= \frac{E}{L^2} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \int_0^L A_x dx$$

Performing the same transformation for the variation of area in the integral in Equation 4.2 as employed in the flexural coefficient, evaluating and summing the individual contribution results in an effective axial volume of $A_r L$. A_r is the resulting equivalent area and L is the overall member length. Thus, the axial coefficients for the nonprismatic element stiffness matrix are expressed as

$$[S_{e(axial)}] = \frac{A_R E}{L} \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} = \begin{bmatrix} S_{11} & S_{14} \\ S_{41} & S_{44} \end{bmatrix}$$

Assembling both the flexural and axial coefficients into the complete 6x6 nonprismatic element stiffness matrix yields

$$[S_e] = \frac{E}{I} \begin{bmatrix} s_{11} & s_{12} & s_{13} & s_{14} & s_{15} & s_{16} \\ s_{21} & s_{22} & s_{23} & s_{24} & s_{25} & s_{26} \\ s_{31} & s_{32} & s_{33} & s_{34} & s_{35} & s_{36} \\ s_{41} & s_{42} & s_{43} & s_{44} & s_{45} & s_{46} \\ s_{51} & s_{52} & s_{53} & s_{54} & s_{55} & s_{56} \\ s_{61} & s_{62} & s_{63} & s_{64} & s_{65} & s_{66} \end{bmatrix}$$

For the condition involving prismatic members, in which both the cross sectional area and moment of inertia remain constant throughout the span length, the coefficients A, B, and C become 1, 1/2, and 1/3, respectively. Also, A_r becomes A of the section. Under this condition, the nonprismatic element stiffness matrix becomes

$$[S_e] = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} & -\frac{6EI}{L^2} & 0 & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ 0 & \frac{6EI}{L^2} & \frac{2EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{4EI}{L} \end{bmatrix}$$

Thus, application of the CASE-GBRIDGE nonprismatic element stiffness matrix to prismatic members results in precisely the theoretically exact formulation.

4.3 GBRIDGE Analysis Methodology

The basis of the finite element direct stiffness method of analysis employed for GBRIDGE is the relationships at the joints between applied actions and resulting displacements, where joint equilibrium and compatibility must be satisfied. The purpose is indirectly to determine the deflected shape of the structure considering the discretized joints. Action-displacement relationships take the matrix form [A]=[S]*[D], where [A] is the action matrix of applied loads, [S] is the global stiffness matrix based upon the bridge girder section properties, and [D] is the unknown displacement matrix. The global stiffness matrix is a matrix of coefficients that can be considered to represent the actions taking place at a node due to a unit displacement of a member end. Thus, with the actual actions [A] known, the actual displacements [D] due to [A] can be found through matrix manipulation. With the joint displacements known, the internal forces, stresses, and displacements at analysis points can be evaluated by application of superposition through usage of influence lines as described later.

Individual nonprismatic element stiffness matrices contribute to the formulation of the global stiffness matrix, which is used to yield the structural behavior results. GBRIDGE finite element stiffness analysis is based upon planar structure behavior. Planar structures are framed structures whose loading and members all lie in the same plane. Framed structures are systems consisting of members which are long in comparison to their cross section; girder bridges are typical of this. The action-displacement relationship generates linear simultaneous equations, the number of which depend upon the number of spans and joint fixity. The solution method utilized in solving these equations is a banded Cholesky method, employing in-place decomposition. This standard solution procedure requires no elaboration. However, consideration of joint fixity and influence line development will be briefly examined.

In the analytical process, the actual restraints of the structure's joints (support reactions) must be considered; otherwise, the generation of simultaneous equations will be singular. The joint fixity has been accounted for in GBRIDGE by employing a nodal renumbering technique in developing the global stiffness matrix from the contributions of the nonprismatic element stiffness matrices. This technique yields a major advantage over currently employed bridge analysis methods in that it can account for any generalized bridge geometry and constraint condition. In contrast, the generally employed analysis techniques, the flexibility method [56], and Newmark's procedure [53] are generally limited to simply supported bridges having only vertical unknown reaction forces.

The nodal renumbering procedure employed by GBRIDGE is based upon rearrangement of the global stiffness and action matrices. Both the stiffness matrix [S] and the joint load vector [A] are rearranged and partitioned so that terms pertaining to unrestrained degrees of freedom are separated from the restrained terms. This rearrangement has been accomplished by utilizing a displacement index. Revised displacement indices are computed automatically by examining the actual fixity condition for each possible unknown joint displacement. The procedure is as follows: if the global degree of freedom is not restrained, then the displacement index must be reduced by the cumulative number of restraints encountered up to that point. However, if the displacement under consideration is actually restrained, then the displacement index must be updated by the current cumulative restraint number. The appropriate equations are

$$\text{unconstrained:} \quad J_{new} = J_{old} - C_{jo}$$

$$\text{constrained:} \quad J_{new} = n + C_{jo}$$

where

J_{new} = new global degree of freedom (dof) number

j_{old} = old global degree of freedom number

C_{jo} = the accumulative number of joint restraints up to the global dof in question

and n = the total number of unrestrained global degrees of freedom and can be expressed as $n = nd_j n_j - n_r$

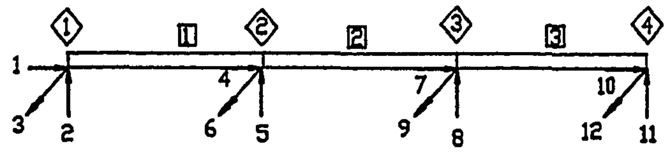
with nd_j = the possible global displacement per joint

n_j = the total number of joints

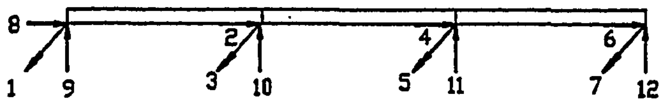
n_r = the total number of joint restraints

The procedure is illustrated in Figure 4.7.

Once the global stiffness matrix has been obtained from the renumbered degree of freedom bridge superstructure,



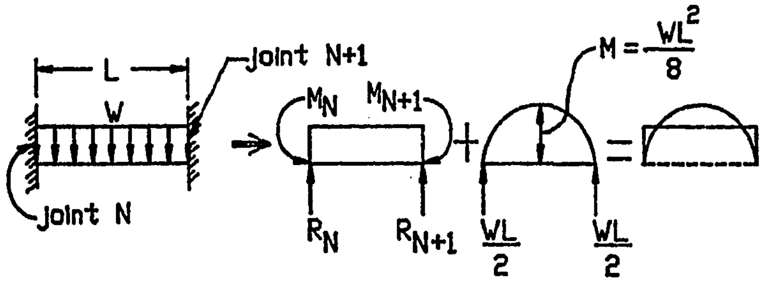
(a) Original Global Degrees of Freedom
□ represents member number
◇ represents joint number



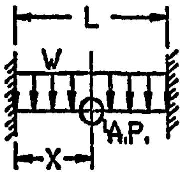
(b) Renumbered Degrees of Freedom

FIGURE 4.7 - Nodal Renumbering Scheme

analysis is performed for both the dead load and superimposed dead load conditions. The loads to be investigated per AASHTO requirements are dead load (DL), superimposed dead load (SDL), live load (LL) and impact (I). For the condition of dead load, only the member end forces are computed, using only the stiffness of the noncomposite girder system. These end forces are stored as negative values and superposition is applied by addition of positive simple span beam moments to produce the actual internal force values at any requested analysis point. This superimposed effect for uniform loads is illustrated in Figure 4.8. Also, a dead load increasing factor is allowed to account for the weight of the welding, secondary weights, etc. The superimposed dead load effects are obtained in a



(a) Superimposed Uniform Load Effect



$$M_{AP} = -M_N + \frac{W}{2}\left[L(x) - x^2\right]$$

$$V_{AP} = R_N + W\left[\frac{L}{2} - x\right]$$

(b) Shear and Moment per Analysis Point

FIGURE 4.8 - Analysis Point Forces for Uniform Loads

similar manner, except that dead load inflection points are utilized in evaluation of each member stiffness matrix. The long term effect of concrete creep and shrinkage is accounted for by increasing the modular ratio by three for composite girder action.

The live load analysis is obtained from utilization of influence lines generated for each girder analysis point. An influence line shows the value of any action (shear, moment, deflection) due to a unit point load moving across the structure. That is, plotting the values of any given action for a specific analysis point as ordinates at all analysis points of application of a unit transverse load creates an influence line for that given action. The influence line to any action actually represents the deflected profile of the structure, to a given scale, produced by an impressed unit distortion in the nature of the action.

To determine the live load influence lines, it is necessary only to obtain the end moments over the supports and apply distribution equations. The determination of member end moments is accomplished by indirectly considering the effects of the fixed end moments for any specific unit loading. Final member end moment equations are developed from an arbitrary application of 1000 ft-k joint moment to each unrestrained rotational degree of freedom. The resulting end moments divided by 1000 are the coefficients that, when multiplied by the fixed end moments, result in the true member end force. The fixed end moments are computed numerically for a unit load placed successively at each analysis point. Utilization of this analysis technique significantly reduces computation time required in evaluation of final member end moments for the multitudes of loadings required since relatively few analyses are performed based upon applied joint moments only. Thus, the true member end forces are obtained for any loading without actually analyzing that loading condition.

Internal forces for the live load condition are evaluated in a fashion similar to evaluating dead load and superimposed dead load internal forces at each analysis point from superposition of member end moments and simple beam effect. When the member end moments (moments over the supports) have been determined, the influence lines for all analysis points along the span can be rapidly computed. These influence lines are obtained by combining the proportional value of the negative moment diagram with the simple beam moment diagram. The moment and shear ordinates of influence lines for a specific analysis point are computed from:

Moment Influence Lines:

$y < kL$

$$M_{AP} = -M_N + [M_{N+1} - (-M_N)] \frac{y}{L} + y(1-k)$$

$y > kL$

$$M_{AP} = -M_N + [M_{N+1} - (-M_N)] \frac{y}{L} + k(1-y)$$

Shear Influence Lines:

$y < kL$

$$V_{AP} = \frac{1}{L} [M_N + M_{N+1}] - (\frac{y}{L})$$

$y > kL$

$$V_{AP} = \frac{1}{L} [M_N + M_{N+1}] - (\frac{L-y}{L})$$

The definition and meaning of these terms are illustrated in Figure 4.9. The influence lines for moments and shears at analysis points in spans other than the span on which the

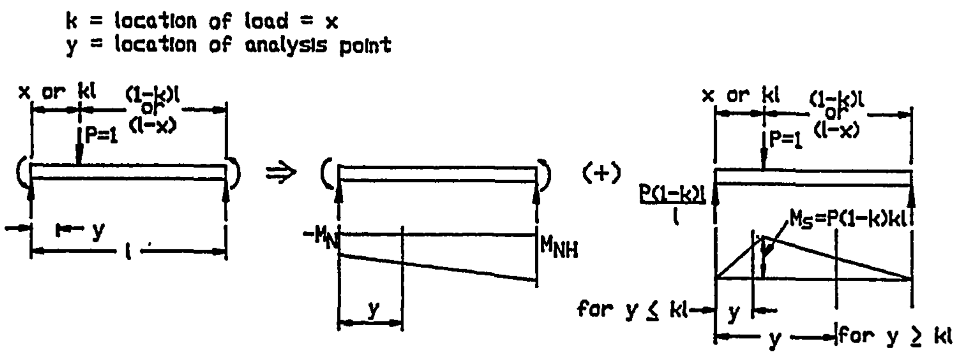


FIGURE 4.9 - Moment Influence Line Generator

unit load is located are directly proportional to the end moments, M_N and M_{N+1}, for that span. Therefore, the last term in the moment and shear influence line equation is omitted when the analysis point under investigation and the loading location are not in the same span. The influence lines are useful for two purposes:

* for determination of the position of live load that will cause the maximum value of the particular function for which the influence line is constructed;
* to compute the value of that particular function with the loading placed for maximum effect.

Typical influence lines for specific analysis points of a three-span continuous bridge are illustrated in Figure 4.10.

4.4 AASHTO Loading Application

The loading used in the CASE methodology, as applied to girder bridges, is based upon the AASHTO 1983 edition of "Standard Specifications for Highway Bridges"[21]. The

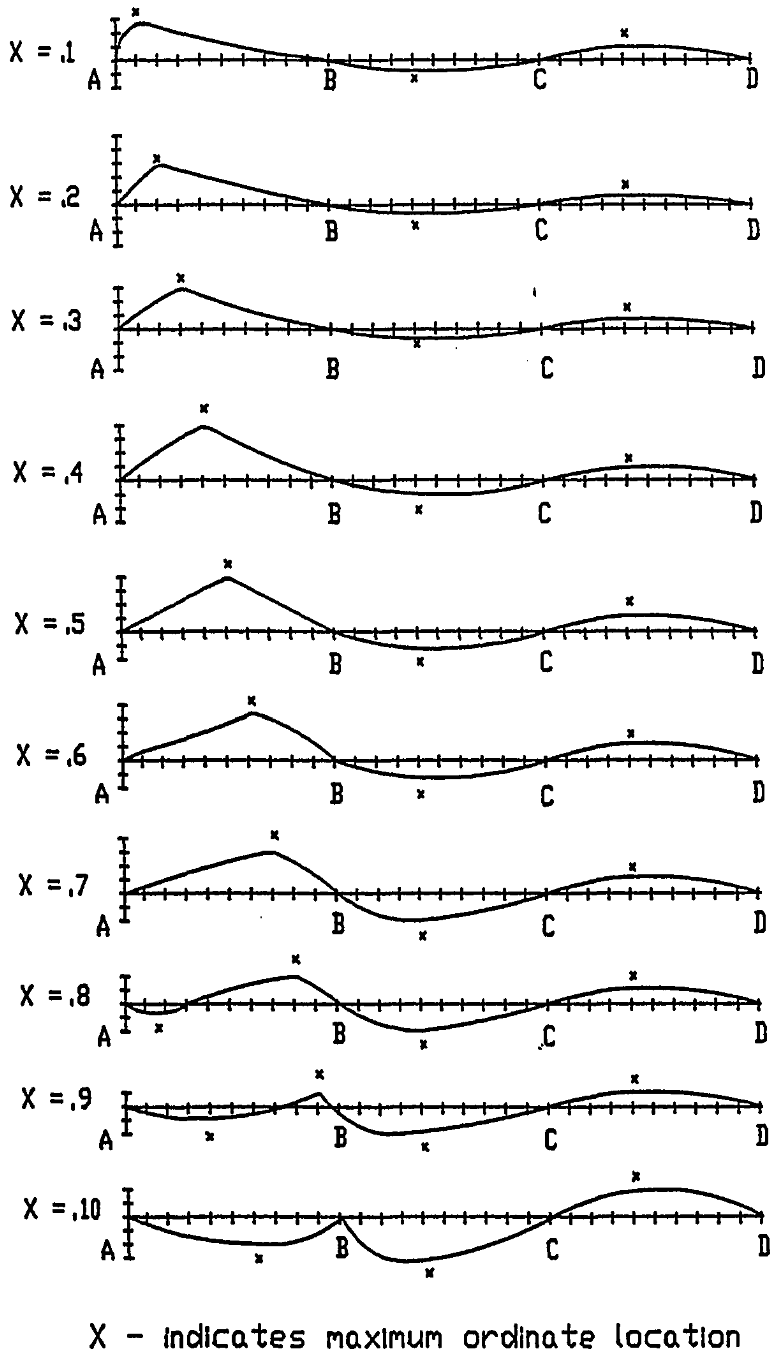
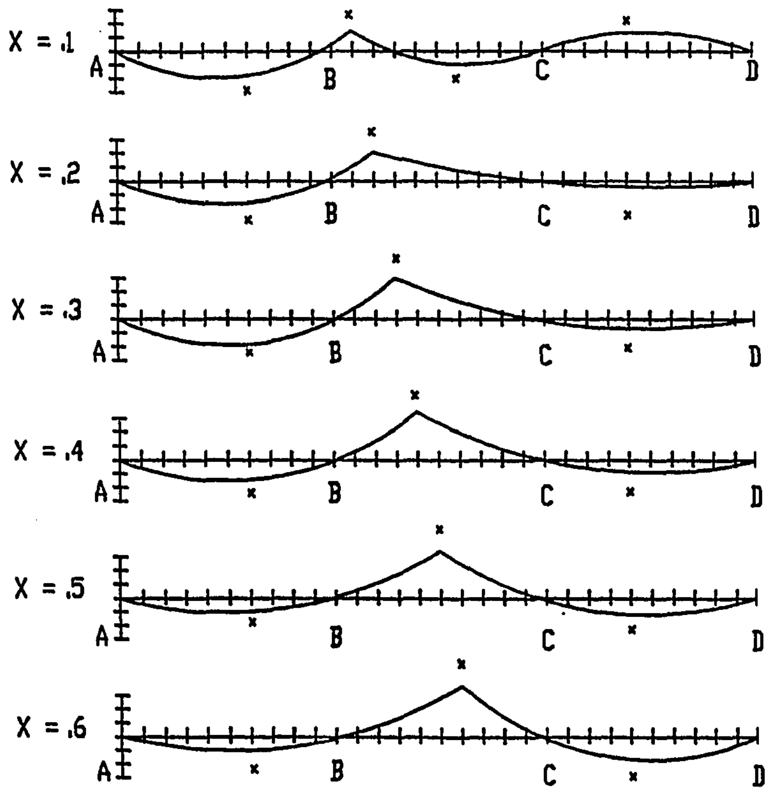


FIGURE 4.10 - Moment and Shear Influence Lines

(a) Moment Influence Lines



X - indicates maximum ordinate location

FIGURE 4.10(con't.)

(a) Moment Influence Lines

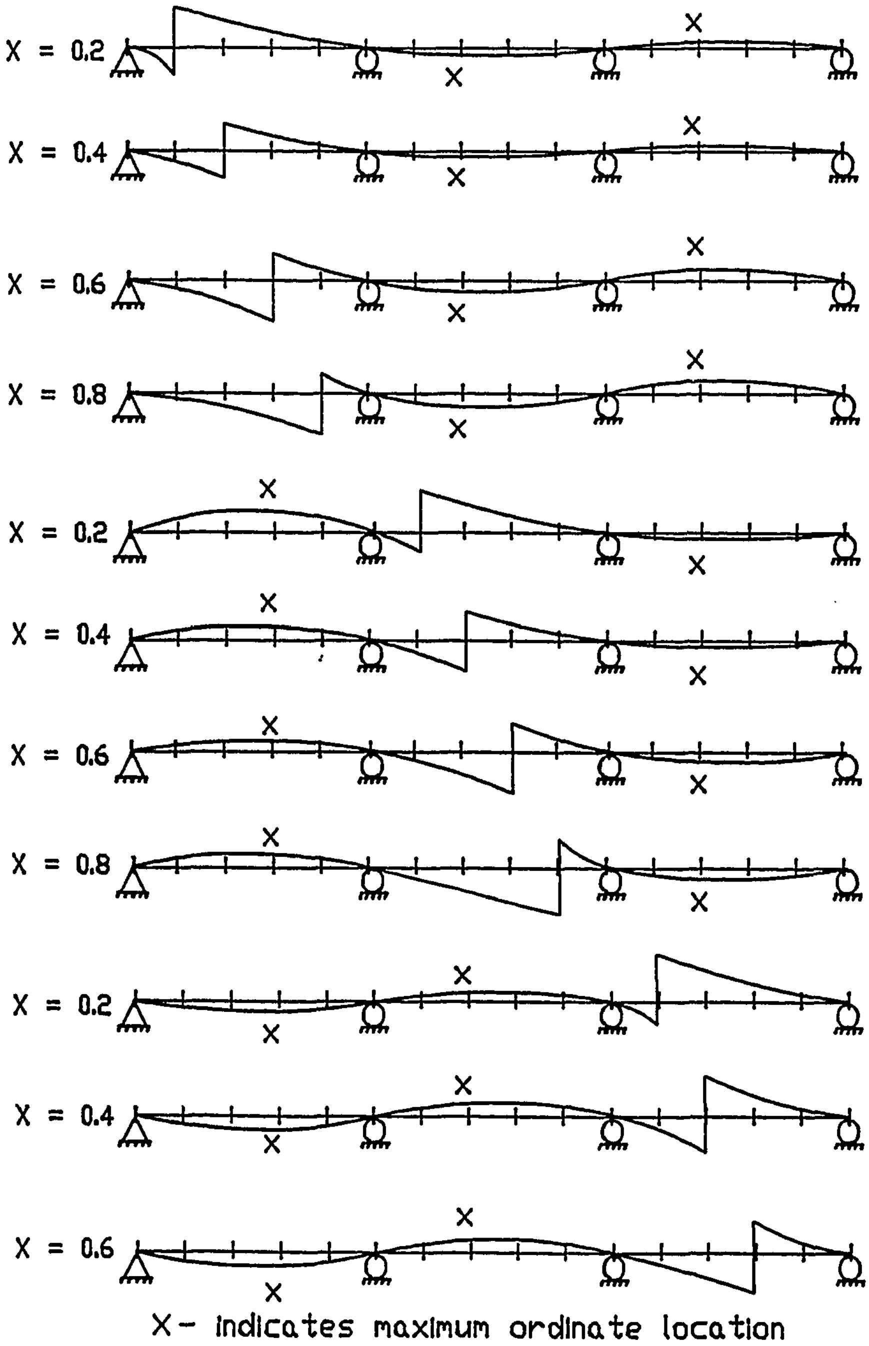


FIGURE 4.10(con't.)

(b) Shear Influence Lines

loading considered to be applied to the superstructure consists of dead load, superimposed dead load, live load, and impact (or dynamic effect of live load). These loading components are applicable to the superstructure concrete roadway deck and supporting steel girder system.

The dead load consists of the weight of the bridge structure plus the weight of all permanently attached items, such as piping, cables, public utilities, etc. In composite girder systems, the loading before the concrete roadway deck has hardened is considered resisted by the girder section only. The superimposed dead load condition is permanent loading that is added to the bridge system after the composite action between the steel girder and concrete roadway has occurred. Weights from guardrailing and future wearing surface are examples of superimposed dead load. The long term effect of creep and shrinkage is accounted for by variation of the concrete modular ratio.

Two systems are specified by AASHTO for the live load condition, H and HS loading. H loading represents truck loads for county roads and state highways. The loading currently used for the federal interstate highway system is HS20-44, which represents a 20-ton truck with a 16-ton semitrailer. A variable axle spacing is included. Truck loadings are concentrated loads. However, uniform loads are also considered in the form of lane loadings. The H and HS truck loadings are physical representations of a fictitious semi-truck loading, whereas the H and HS lane loadings are equivalent loadings representing the effect of a procession of vehicles. The uniform portion of lane loadings may be continuous or discontinuous, as necessary to produce maximum internal forces (moments and shears). These loadings are indicated in Figure 4.11.

The AASHTO bridge specifications are developed so that the analytical investigation and synthesis of the roadway slab and girders are conducted independently. The bridge roadway deck is designed by considering the slab as a one-foot width continuous beam running over supporting girders. Both the positive and negative moments for dead load and superimposed dead load conditions are given by Equation 4.3, representing a simple span moment. Analogously, AASHTO specifications give the live load bending moment for a simple span of one-foot width by Equation 4.4.

$$M_{DL} \text{ OR } M_{SDL} = \pm \frac{WS^2}{8} * CF \qquad \text{Equation 4.3}$$

$$M_{LL} = \pm \frac{(S+2)P}{32} * CF \qquad \text{Equation 4.4}$$

where

S = effective span length (girder spacing in feet)
W = uniformly distributed load (dead load or superimposed dead load)
P = live load wheel load value: 12k for H15 and HS15 loading; 16k for H20 and HS20 loading
CF = continuity factor = 0.8

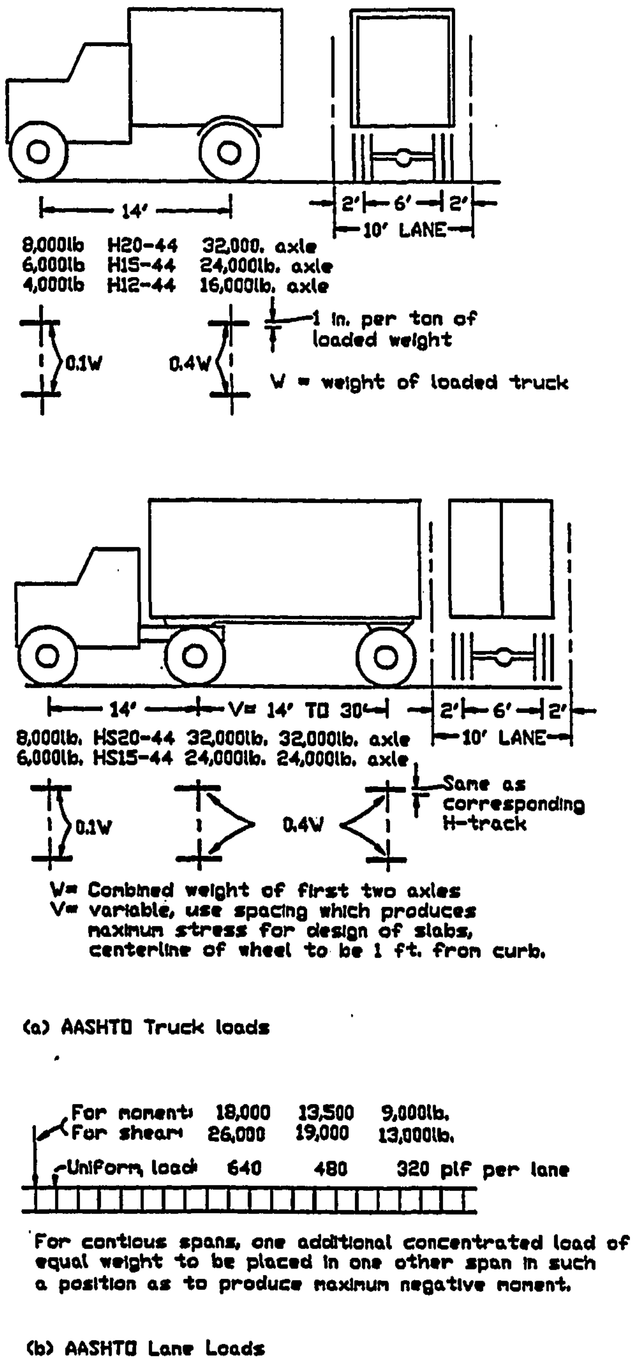


FIGURE 4.11 - AASHTO Truck and Lane Loading

In the slab continuous over three or more supports, the moment, as determined by Equations 4.3 and 4.4, is multiplied by 0.8 for both positive and negative values to account for continuity. Impact is included in the evaluation of the total design moment by the application of Equation 4.5.

$$M_I = 0.3 * M_{LL} \qquad \text{Equation 4.5}$$

Also, AASHTO requires overhang considerations to be included where the slab cantilevers over the outer girder. This negative moment condition, illustrated in Figure 4.12, is accounted for by considering uniform dead load for the slab $[M = (w\ell^2)/2]$ and a concentrated loading effect from guardrail and curbing. The live load is evaluated from Equation 4.6.

$$M_{LL} = \frac{Px}{E} \qquad \text{Equation 4.6}$$

where

P = loading as previously defined

x = distance from center line of girder to concentrated wheel load P

$E = 0.8x + 4.25 < 7.0$

Again, impact is considered from Equation 4.5 in determining the total design moment.

For the supporting girders, there is a distinction between interior and exterior girders. In the case of dead

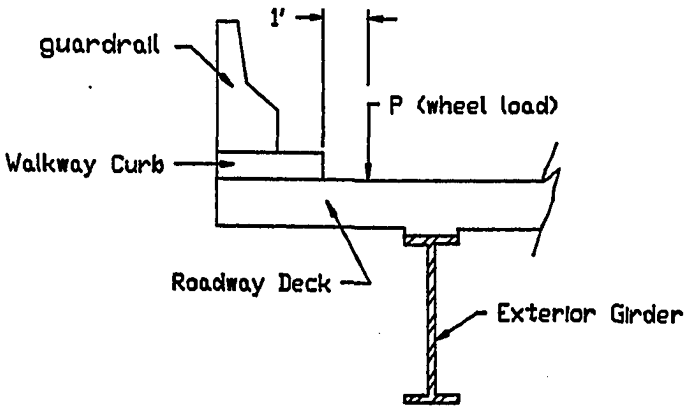


FIGURE 4.12 - Roadway Overhang

load and superimposed dead load analysis, each interior girder carries the weight that is proportional to one-half the distance between girders on each side. Exterior girders carry a portion of the weight from the outside edge of the roadway slab to the midpoint of the span between the exterior girder and the first interior girder. The loading due to curbs, guardrail, and future wearing surface all have been considered to be placed after the roadway slab concrete has hardened and is analyzed based upon equal distribution to all supporting girders. The effect of composite girders has been accounted for as previously indicated.

AASHTO requirements for live loading on bridge girders are directly related to the girder spacing and location. The loads which are applied to each girder depend upon transverse distribution factors, which are based upon supporting girder spacing and are different for the interior and exterior girders. The wheel load distribution equations for interior and exterior girders are given by Equations 4.7 and 4.8, respectively.

Interior girder

$$DF = \frac{S}{5.5} \qquad \text{Equation 4.7}$$

where

DF = distribution factor per wheel loading
S = average girder spacing < 14 feet

Exterior girder

$$DF = \frac{S}{4 + 0.25S} \qquad \text{Equation 4.8}$$

where

S is as previously defined for $6 < S < 14$;
if $S < 6$, Equation 4.6 is used.

GBRIDGE loading is based upon axle loads and, thus, the axle distribution factor is one-half of these wheel load distribution values. The live load applicable to an exterior girder of a bridge designed for two or more lanes of traffic will be slightly less than that for an interior girder. However, AASHTO requires that the exterior girders be designed based upon the maximum external forces attained in any stringer. Therefore, all girders are generally designed as the same section, except where the roadway slab creates a major cantilever over the exterior girder, thus requiring the exterior girder loadings to be greater than that on interior girders. Again, the impact moment is considered as a increase of the live load moment by Equation 4.9.

$$M_{impact} = I * M_u \qquad \text{Equation 4.9}$$

where

$$I = \frac{50}{L + 125} \leq 0.3$$

As previously stated, in the live load analysis segment of GBRIDGE, an extremely efficient technique for generating influence lines has been implemented. The influence lines are diagrams whose ordinates at any analysis point equal the magnitude of some particular function of that structure, such as shear or moment, due to a unit load acting at the location of each ordinate. The first consideration in application of influence lines in live load analysis is whether the loading system consists of concentrated loads or uniformly distributed loads. The value of a structure function due to a series of concentrated loads (i.e., truck loading) is quickly obtained by multiplying each concentrated load magnitude by the corresponding ordinate of that influence line for the function. When the loads are located between analysis point positions, GBRIDGE utilizes a linear interpolation to evaluate the ordinate at the load location. In the case of lane loading (i.e., distributed loading), the value of a structure function may be obtained by multiplying the area of the influence line diagram by the magnitude of the uniform load. However, GBRIDGE utilizes an approximating technique for uniform lane loading. An equivalent concentrated load is generated by multiplying the load magnitude times the contributory width that is located midway between adjacent analysis points. Then, this equivalent concentrated load is treated identically to the concentrated truck loading as shown in Figure 4.13.

The placement of loading for maximum effect is also dependent upon the type of load system. For the lane loading condition, the loading may consist of partial and alternate span loading, whichever creates the greatest effect. GBRIDGE utilizes the sign of the ordinates for investigation of alternate and partial span loading. Also included, as specified by AASHTO, is the effect of one

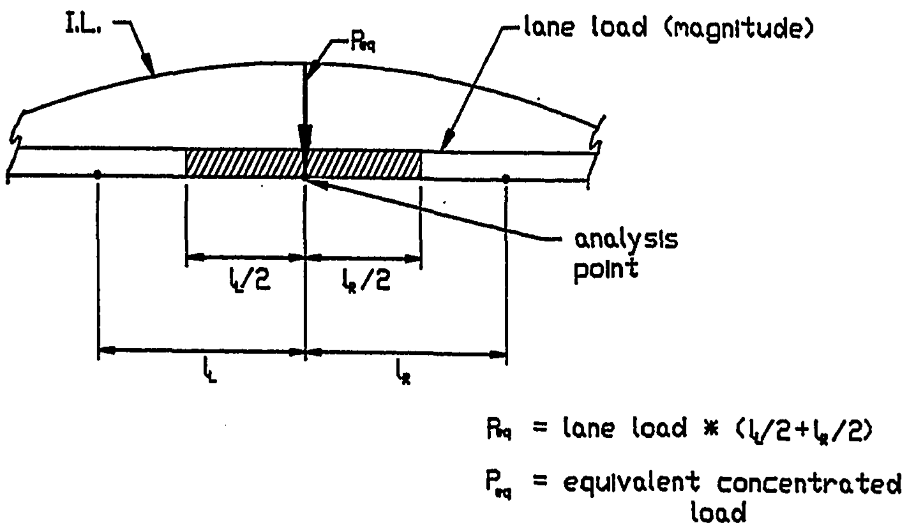


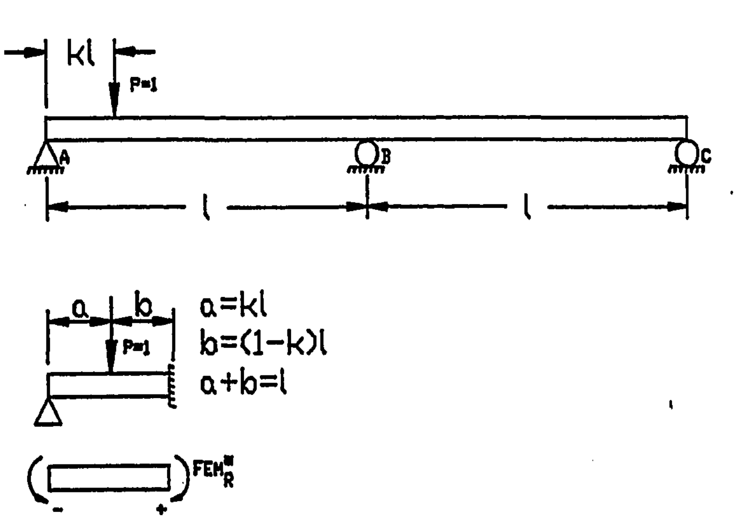
FIGURE 4.13 - Equivalent Loads from Uniform Loading

concentrated load in the investigation of positive moment but, for negative moment locations, a concentrated load is placed in each of the two adjacent spans. No such provision is included by AASHTO for shear and, thus, only a single concentrated load placed at the proper maximum ordinate is used.

The internal forces at each analysis point due to concentrated loads in the truck loading condition are somewhat more complex to evaluate than the lane loading condition. First, consideration must be given to whether the loading is H or HS loading. HS loading has a variable rear axle spacing; this spacing is a required input value and is not varied programmatically in CASE-GBRIDGE. Secondly, the direction of truck travel also must be considered since the axle loadings are variable. GBRIDGE allows for travel from left to right, right to left, or both directions. It should be noted that a symmetric structure will have only symmetric analysis point internal forces when truck traffic is examined for both directions due to the variation in load magnitude. The maximum effect is examined by placing one of the largest wheel loads at the maximum ordinate being considered and linearly interpolating the ordinates at the other wheel loads.

GBRIDGE, besides developing an efficient method of generating influence lines, has incorporated a technique for rapid internal forces evaluation. Again, the maximum effect must consider the possibility of alternate span loading and the total combined loading effect. First, the analysis point location of the maximum ordinate will always be the same location for spans other than the one where the internal forces are being determined. The ordinate values will change but not the location. The reasoning for this is that, in an elastic analysis, the distribution of loading is dependent only upon the member properties. The magnitude of loading only affects the magnitude of the member end moment, not the location of maximum effect. Consider the two span beam system in Figure 4.14. The differentiation of the modified fixed end moment equation, which controls the end moment expressions that are derived for generation of influence lines, yields the location of maximum moment of $k = 0.57735$. This location is synonymous with the maximum ordinate positions, since the influence lines are controlled by the moment expressions.

When the analysis point location and loading effect are in the same span, the internal force evaluation process is dependent upon the analysis point position. If the analysis point is a significant distance from the supports, only one loading effect is required to be investigated because all ordinates are of the same sign. Therefore, the maximum ordinate is at the analysis point that is under investigation for the internal forces. However, if the analysis point is sufficiently close to the supports



$$FEM_R^* = \frac{P_{ab}}{2\ell^2}(a + \ell)$$

Substituting $a = k\ell$ and $b = (1-k)\ell$ and allowing $P = 1$:

$$\text{Modified } FEM_R^* = \frac{P(k\ell)[(1-k)\ell](k\ell+\ell)}{2\ell^2}$$

$$= 1 * \frac{[k(1-k)(1+k)]\ell^3}{2\ell^2}$$

$$= 1 * \frac{[k(1-k)(1+k)]\ell}{2}$$

$$\text{Modified } FEM_R^* = 0.5(k-k^3)\ell$$

$$FEM_{BA} = 0.5(k-k^3)\ell$$

To locate M_{max}:

$$\frac{\partial FEM}{\partial k} = \; = \frac{\partial[0.5k - 0.5k^3\ell]}{\partial k} = 0.5\ell - 1.5k^2\ell$$

$$k^2 = 1/3 \qquad M_{max} = 0.19245\ell$$

$$k = 0.57735$$

FIGURE 4.14 - Two Span Continuous Beam

(< 0.2 or > 0.8), then both positive and negative ordinate values occur and must be checked both for positive and negative loading effects (refer to Figure 4.7).

In an analogous fashion, the shear effects for each analysis point are generated from the influence lines. The procedure for obtaining the maximum shear from all spans employs the same basic procedure as for moments, except both the positive and negative shear effects are examined for absolute maximum shear load. Also, fatigue and shear stud spacing are both dependent upon shear range (i.e., the maximum difference between positive and negative shear forces) which has only slight variation throughout the bridge system as indicated by Figure 4.15.

A point that needs clarification with regard to the analysis process is that of the analysis model. Since the loading conditions of dead load, superimposed dead load and live load are investigated separately but the actual effect is the result of simultaneous action, the analytical model must reflect the true behavior of the bridge system. For composite girders, the global stiffness matrix varies for each analysis component. The reason is that the dead load analysis considers the girder effective only in resisting the loads, whereas the superimposed dead load and live load conditions consider the composite action. Even the superimposed dead load and live load conditions are different due to the fact that the live load is transient

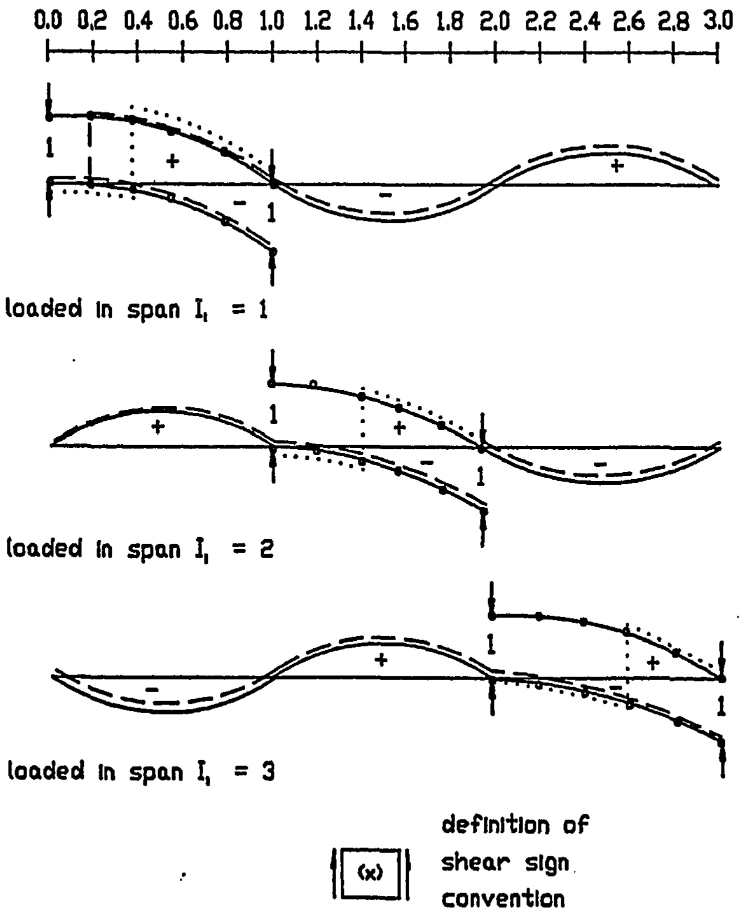


FIGURE 4.15 - Shear Range

and the superimposed dead load is long term; also, both use different material properties. However, both the dead load and the superimposed dead load possess virtually the same inflection points, which separate the positive and negative moment areas. The live load inflection points are not the same. However, since the analysis is to model the behavior of the system and not just formulate equilibrium of the model, GBRIDGE utilizes the dead load inflection points throughout the analysis process in the evaluation of member stiffnesses.

Finally, AASHTO specifies that either the truck loading or lane loading is to be used depending upon which produces the maximum effect. The truck loading condition generally controls up to 134 feet for simple spans and approximately 200 feet for continuous spans. However, only one loading type is examined per analysis run by GBRIDGE. For comparison between lane loading and truck loading, the analysis must be performed twice; each time specifying the desired loading type to be investigated. For the purpose of implementing CASE-GBRIDGE in this study, the only loading considered for the synthesis process described in the next chapter is HS20 truck loading. The overall accuracy of the CASE-GBRIDGE analysis procedure is illustrated in Chapter 7.

V. BRIDGE SYNTHESIS

5.1 General

There are currently two automated (computerized) synthesis approaches for structural design, both are intended to produce systems which are, to some degree, "optimal". The first approach relies on the theory of mathematical programming to determine the optimal design. The structural synthesis process can be described algorithmically and be solved programmatically. The second approach is that of artificial intelligence via expert systems. These systems are suited for problem solving that is judgmental in nature. The structural synthesis approach employed by GBRIDGE is that of mathematical programming.

Examination of (a) structural synthesis methods applicable to automated structural design, (b) explicit formulation of the objective function, and (c) explicit formulation of the design constraints will be included in this chapter.

5.2 Structural Synthesis via Nonlinear Mathematical Programming

The problem of structural synthesis by mathematical programming can be stated as:

Minimize $f(\mathbf{X},\mathbf{D})$ $\quad X = 1,2,\ldots,n$

Subject to $g_j(\mathbf{X},\mathbf{D}) \leq 0$ $\quad j = 1,2,\ldots,m$

$$\mathbf{S}(\mathbf{X})*\{\mathbf{D}\} = \{\mathbf{A}\}$$

where f is an objective (cost) function that is to be minimized, g_j are constraint functions, **X** is a vector of design variables, **D** is an NDOF vector of nodal displacements (where NDOF is the number of degrees of freedom of the structure) and is an implicit function of the design variables, **S(X)** is the stiffness matrix for the structure and is an implicit function of the design variables, **A** is a vector of applied loads, n is the number of design variables, and m is the number of constraints. Such a formulation of the design problem arises naturally when the structure is modeled using finite element methods. The equation $\mathbf{A} = \mathbf{S}(\mathbf{X})\mathbf{D}$ is the formulation of the analysis action-displacement relationship previously described in Chapter 4 and is the interlocking problem between the analysis and synthesis formulation of the structural design application. The features that are central to this formulation are:

a) The constraint functions depend on unknown displacements **D** which are obtained by solving a system of linear action-displacement equations $\mathbf{A} = \mathbf{S}(\mathbf{X})\mathbf{D}$. Thus, the structure of these functions cannot be known beforehand. Specifically, evaluation of functions and their gradients requires solution and differentiation of the results from the linear system of $\mathbf{D} = \mathbf{S}(\mathbf{X})^{-1}\mathbf{A}$;

b) The functions f and g_j are, in general, implicit functions of the design variables. Thus, the objective function $f(\mathbf{X},\mathbf{D})$ depends on the displacement vector **D** which, in turn, depends on **X** through the FEM equation $\mathbf{S}(\mathbf{X})\mathbf{D} = \mathbf{A}$. The presence of such implicit functions contrasts with problems generally encountered in mathematical programming where all functions are usually explicit. In other words, the solution of the action-displacement simultaneous analysis equations yields unknown displacements **D**, but the desired design vector is **X**.

c) The optimal design problem outlined above is highly nonlinear and, in general, non-convex. The nonlinearity and nonconvexity in the functions is primarily due to the implicit nature of the functions.

The various components of the structural synthesis problem now require a more elaborate description. The structural synthesis procedure seeks the selection of design variables, within the limits (constraints) placed on the structural behavior, geometry, or other factors, to achieve its goal of optimality defined by the objective function for specified loading or environmental conditions. The basic features of design variables (and design parameters), constraints (behavior and side), and the objective function combined together form the structural design problem in geometric design space.

The "total" structural framing system can be described by a set of quantities, some of which are viewed as variable during the structural synthesis process. Those quantities defining a structural system that are fixed during the automated design are called preassigned parameters and they are not varied by the mathematical programming algorithm. Those quantities that are not preassigned are called design variables. The preassigned parameters, together with the design variables, will completely describe a design. In general, the design variables are represented by a column vector, X, which specifies a point in design space. The design variables of an optimum structural design problem may consist of member sizes, plate sizes, depths, values of structural configuration, properties of material, or any quantifiable aspect of design.

The constraints generally can be defined as restrictions that must be satisfied in order to produce an acceptable design. Constraints can be grouped in a broad sense as behavior constraints or side constraints. Furthermore, the constraints may take the form of a limitation imposed directly on a variable or variable group (explicit constraint) or may represent a limitation on quantities which depend on design variables that cannot be stated directly (implicit constraints). Behavior constraints, in general, are nonlinear functions of the structural design variables and are derived from the performance or behavior of the structure. These constraints are imposed by the appropriate design specification and limit such items as minimum plate thicknesses, maximum stiffener spacing, allowable stresses based upon design variables, and various other requirements. Side constraints place restrictions on the range of the design variables for reasons such as manufacturing, shipping, aesthetics, that is, for reasons other than the performance of the structure. For use in bridge structures, the behavior constraints will be based upon the specifications of the American Association of State Highway and Transportation Officials (AASHTO), which include both the Working Stress Design (WSD) method and the Load Factor Design (LFD) method.

Within the constraint limits, there usually exists an infinite number of feasible designs (depending upon whether the design variables are continuous or discrete). In order to determine the best or optimal design, it is necessary to formulate a computable function in terms of the design variables, i.e., an objective function. The objective function (also called the merit, criterion, or cost function) is the function whose least value is sought in the structural synthesis process. The selection of the objective function is one of the more important aspects of the automated design process but it can be a rather difficult task. Weight has predominated the studies on structural optimization. However, a general cost function is more realistic for civil engineering structures where manufacturing and construction are a major portion of the total project budget.

5.2.1 GBRIDGE Structural Synthesis Methodology

Having explicitly defined the problem of structural synthesis, description of the methods of solutions incorporated into GBRIDGE will be briefly described.

Appropriate methods for structural synthesis, to a great extent, depend upon the type and detail of the structural member being examined. Most reported methods applied in structural synthesis [20] have been concerned with structural assemblages such as trusses and frames and illustrate methods for finding the minimum weight design that optimally distributes the loading to the various structural framing members. These methods are generally restrictive, in that they consider only a single unknown to describe completely all member properties. The member area or moment of inertia are prime examples of the unknowns assumed to describe the member completely. This type of design variable linkage significantly simplifies the mathematical programming problem in that the structural analysis and synthesis are solved simultaneously. However, this linkage technique usually yields suboptimal designs and sometimes even infeasible designs.

On the other hand, the designer in practice spends a considerable amount of time proportioning structural framing elements of beams, columns, footings, walls, etc. Dimensions of elements, such as depth, plate thickness, plate width, reinforcing steel area, and other details, must be found after the forces on the member are known. The computations are tedious and experience is required to find designs that meet the various code constraints but are not excessively overdesigned. The constraints and cost expressions for CASE-GBRIDGE are set up so that the engineer is allowed to modify, verify, or change these expressions.

In actuality, GBRIDGE employs two separate synthesis techniques, one for the steel girder system and another for the concrete roadway deck. The reasoning for the separation, as described in the preceding chapter, is that separate analytical techniques are required in examining the roadway deck and steel girders. Employing a selection criteria of reliability and accuracy and considering the limitations posed by the computer system, it was deemed that methods that had previously been successfully implemented in similar problems should be employed. The most commonly employed method used in a steel manufacturing environment is the enumeration method of backtracking, whereas the most widely published method applied to civil engineering structures is the interior penalty function method. Thus, the backtracking method is utilized for the steel girder synthesis process and the interior penalty function method is employed to optimize the concrete roadway deck.

5.2.2 Backtrack Method

The structural synthesis process for the steel girders involves the selection of design variables from a set of discrete plate sizes. The plate elements comprising the girder are restricted to specific size limitations due to economy of both production and manufacturing of steel plates. As there is only a finite number of candidates, it may appear advantageous to enumerate or investigate individually all possible designs. However, the number of possible design candidates may be very large and, in fact, complete enumeration is combinatorially explosive. The utility of the backtracking scheme (i.e., implicit enumeration) is that, by examining only a very small subset of the possible combinations, the method implicitly examines all possible combinations.

The structural synthesis task, as related to the steel girder, is to assign plate sizes to each design variable so that the resulting structural system satisfies the requirements of behavior and cost. Once a set of plate elements has been assigned, it is a relatively straightforward process to evaluate behavior and cost. The difficulty lies in the selection of the optimum values. The advantages of using backtracking for structural optimization of discrete variable systems are:

* it possesses a readily implemented structure,
* it allows for very complex cost function, code constraints, and design details, and
* it is exhaustive, thus guaranteeing a <u>global</u> optimum.

Also, only one active variable is assigned at any time in the search process. Thus, the computer storage requirement is minimal and, therefore, ideally suited for implementation on computers with RAM restrictions.

As stated by Golomb and Baumert [73], "The basic idea of the backtrack programming is to build up the sample vector one component at a time and to use modified criterion functions to test whether the vector being formed still has a chance of success. The power of the method is this: If the partial vector $(x_1, x_2, \ldots)$ is already seen to be inherently suboptimal, then

$$\prod_{i=3}^{n} M_i = \frac{M}{M/M_2}$$

possible test vectors may be ruled out in one fell swoop without having to examine them individually."

The modified criterion function may be obtained from the design constraint function and the objective function. The design constraint function reflects the laws of structural mechanics and the design code requirements (in this study, the AASHTO specifications) and can be represented as $f_d(\mathbf{X})$. For the sake of discussion, consider a doubly symmetric noncomposite girder, as shown in Figure 5.1, having the design constraint function expressed as $f_d(d,t_w,b_f,t_f)$. In this function, the design variable vector $\mathbf{X}$ (X_i=1,2,3,4) is represented with d, t_w, b_f, and t_f, where d equals the girder depth, t_w equals the web thickness, b_f equals the flange width, and t_f equals the flange thickness. f_d has the value "true" (1) if the specification requirements are met or "false" (0) if they are failed. The objective function, $f_o(d,t_w,b_f,t_f)$ is the cost of the girder and the function to be minimized. The two functions, f_d and f_o, can be combined into a single

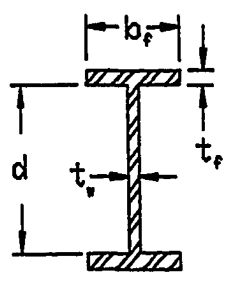


FIGURE 5.1 - Cross Section-Doubly Symmetric Girder

modified criterion function $f_c(d,t_w,b_f,t_f)$ with the value of "true" or "false" by giving the function value "true" only if the girder satisfies the design criterion and, at the same time, the cost is less than the minimum cost ($COST_{min}$) of the least expensive satisfactory section found previously. Initially $COST_{min}$ can be taken as the total cost of the girder by using the largest values of d, t_w, b_f, and t_f to be considered.

In backtrack programming, a trial vector is built up one design variable at a time by testing variable values in a modified criterion function. This modified criterion function is used to determine if a girder cost less than $COST_{min}$ could possibly be obtained with a particular design variable value. As an example, a modified criterion function $f_1(d)$ is used to test values of the girder depth, d_i. Function $f_1(d)$ must be selected to insure that $f_1(d_i) > COST_{min}$ for all discrete values of d_i not equal to d and all t_w, b_f, and t_f. Backtrack programming is effective because all vectors containing d_i can be eliminated if $f_1(d_i) > COST_{min}$. Once a value of d_i has been eliminated, d_{i+1} is tested.

When it has been determined that a value d_i may lead to a lower girder cost than any found previously, another modified criterion function $f_2(d,t_w)$ is used to test whether or not there exists a vector $(d_i,t_w{}^j)$ which gives $f_2(d_i,t_w{}^j) < COST_{min}$. If all the values of t_w are exhausted without the indication of a possible lower cost section, it is necessary to backtrack and to test a new value of d_{i+1} in $f_1(d)$. After a value of t_w^j has been found which gives $f_2(d_i, t_w^j) < COST_{min}$, a third modified criterion function $f_3(d, t_w, b_f)$ is used to test a vector (d_i, t_w^j, b_f^k). Again, if all b_f^k are exhausted without the indication of a possible lower cost section, it is necessary to backtrack to f_2 and test t_w^{j+1}. Finally, the last element, t_f, is added to the vector and values of t_f^l are tested in the criterion function $f(d_i, t_w^j, b_f^k, t_f^l)$.

In general, the procedure of backtracking is a search for a vector of design variables, $X = \{x_i\}$ $(i=1,2,\ldots,n)$ for which the objective function, $f_o(X)$, will be a minimum and simultaneously satisfy all design constraints, $g_j(X_i) \leq 0$ $(j=1,2,\ldots,m)$. In applying backtracking to GBRIDGE, the number of design variables plus the number and type of constraints vary according to the particular girder design formulation. The design formulation allows for either stiffened or unstiffened webs, which controls the number of design variables, n. The number and type of constraints depend upon whether the Working Stress or Load Factor Design philosophy is to be investigated. The design variables for GBRIDGE, in general terms, are represented in a discrete series as:

$$x_{i1}, x_{i2}, \ldots, x_{iu}, \ldots, x_{iTi} \quad (1 < u < T_i)$$

where

i = the particular i^{th} design variable in the objective function ($1 < i < n$, where n is the total number of design variables),

T_i = the total number of discrete plate sizes available for design variable i.

Noting that the evaluation of the cost (objective) function varies with each plate size selection, the steps for implementation of the backtracking method are illustrated in a flowchart in Appendix B.

5.2.3 Interior Penalty Function Method

The interior penalty function method is a transformation method. The term "transformation method" is used to describe any mathematical programming method that solves the constrained optimization problem by transforming it into one or more unconstrained optimization problems. Transformation methods include interior and exterior penalty function methods as well as augmented Lagrangian or multiplier methods.

The transformation approach seeks to transform the constrained problem to a sequence of unconstrained problems. This sequence of unconstrained problems may then be solved by any of a large number of unconstrained search techniques. The transformation or penalty type approach reformulates the constrained nonlinear programming problem to the following form:

$$\text{Minimize } P(X,R) = F(X) + B[R,G(X),H(X)]$$

where $F(X)$ represents the original objective function and B represents the penalty term (sometimes called barrier function) which is a function of the penalty parameters R, the inequality constraints G, and the equality constraints H. The exact way in which the penalty term R is formed defines the particular transformation method. The basic idea behind the transformation methods is to penalize any design which violates one or more of the constraints. However, it must be noted that this is accomplished by drastically distorting the contours of the original objective function which can make the unconstrained search very difficult. To circumvent this difficulty, the original constrained problem is replaced by a sequence of unconstrained problems. At the initial stage, the penalty term is designed so that the original function contours are not altered drastically. Then, at each successive unconstrained iteration, the contours are altered to a greater extent, thus, making each unconstrained minimization more difficult. However, each successive unconstrained search is started from the solution of the preceding stage and the distance traveled from one stage to the next decreases as the number of stages increases. Ideally, the increase in difficulty from stage to stage is offset by the smaller distance traveled so that each stage requires approximately the same computational effort and, thus, overall convergance to the optimal solution is still efficient.

The interior penalty function method is most offen applied for structural synthesis. A major advantage of the interior penalty function method is that one may stop the search at any time and end up with a feasible and, hopefully, usable design. Moreover, the constraints become critical only near the end of the solution process. Thus, instead of taking the optimal design, we can choose a suboptimal, but less critical, design. This is due to the fact that, using the interior penalty function approach, we keep the designs away from the constraint surfaces until final convergence. One drawback is that one must start the solution always with a feasible design but, for structural design, it is usually relatively easy to obtain a feasible point at which to begin. Thus, this shortcoming does not pose serious limitations.

Two points should be considered in practical application of this method to the roadway synthesis in GBRIDGE. First, in most structural design problems, it is relatively easy to find a feasible starting point. For example, we may choose relatively large cross-sectional dimensions which will satisfy stress and displacement requirements. In the GBRIDGE roadway segment, the design variables are always given a preassigned feasible starting vector equivalent to a slab depth of 11 inches and a reinforcing ratio of six-tenths the maximum ($\rho_{b\ max}$). In other design situations, however, it might be more difficult to obtain an initial feasible design. If the starting point x_o violates p constraints, those may be arranged as the first p constraints such that

$$g_p > g_{p-1} > \ldots > g_1 > 0$$

The largest g_p is selected as the objective function for the following problem:

$$g_p(X) \longrightarrow \min$$

$$g_j(X) - g_j(X_o) < 0 \qquad (j=1,2,\ldots,p-1)$$

$$g_j(X) < 0 \qquad (j=p+1,p+2,\ldots,m)$$

This problem is solved by the penalty function method. The search is terminated as soon as the objective function becomes negative, i.e., $g_j(X) < 0$. A new test for feasibility is performed and the process is repeated until all the constraints are satisfied.

Secondly, for decreasing values of R, the minimum of P should converge to a solution of the constrained problem. A simple criterion to check for convergence is to compute

$$\varepsilon_F = \frac{f_{min}(R_{i-1}) - f_{min}(R_i)}{f_{min}(R_i)}$$

and stop where ε_F is smaller than a predetermined value [14]. In the roadway segment of GBRIDGE, convergence is deemed satisfied when $\varepsilon_F < 0.001$ or when a maximum of ten iterations have occured.

Structural synthesis by the interior penalty function method is usually referred to as SUMT (Sequentially Unconstrained Minimization Technique) and, since the interior penalty function method transforms the constrained problem into a sequence of unconstrained problems, the unconstrained searches may be performed by any of a number of methods. The unconstrained search method employed by GBRIDGE is a modified univariate method utilizing a quadratic interpolation polynomial to evaluate the optimal step length. In general, the unconstrained mathematical programming methods (and constrained methods, for that matter) all consist of the same basic philosophy. The algorithms are iterative in nature and require an initial starting vector of design variables, $\mathbf{X}_j$. Then, for each iteration (j=1,2,...,n) of the solution process, $\mathbf{X}_j$ should yield a better solution or, in mathematical terms,

$$f(\mathbf{X}_{j+1}) < f(\mathbf{X}_j)$$

where f represents the objective function being minimized. This reduction process requires sequential minimization of $f(\mathbf{X})$ along successive search directions, $\mathbf{S}_j$; thus,

$$\mathbf{X}_{j+1} = \mathbf{X}_j + \alpha_j \mathbf{S}_j = \mathbf{X}_j + \Delta\mathbf{X}_j$$

This is then a line search in the $\mathbf{S}_j$ direction, which must be a descent value. The amount to move in the $\mathbf{S}_j$ direction is the step length value, α_j, a scalar value. $\mathbf{S}_j$ is determined such that, for some small α value, the design is improved (objective function reduced). Once the direction $\mathbf{S}_j$ is found, a minimization problem using α as a single variable is conducted to determine $\alpha^* = \alpha_j$, the step length to minimize the function in the $\mathbf{S}_j$ direction. The method of choosing the search direction $\mathbf{S}_j$ and the interpolating scheme used find the step size α_j is what distinguishes the various mathematical methods [74]. Similar to the backtracking presentation, the flowchart for the interior penalty function method is also given in Appendix B.

5.3 Explicit Formulation of CASE-GBRIDGE Objective Function

5.3.1 General

The bridge superstructure consists of the supporting girders, roadway deck, and secondary framing as shown in Figure 5.2. The cost function for the entire bridge superstructure expressed in a generic form as

$$F_o = A_{sr}LN_sC_s\gamma_s + N_{sf}C_{sf} + N_{sp}C_{sp} + A_cLC_c + V_{rs}LC_{rs}\gamma_s + C_{pr} + N_{st}C_{st} + A_pC_p$$

where

A_{sr} = cross sectional area of one stringer

L = span length

N_s = number of stringers

C_s = cost of steel per unit weight

γ_s = specific weight of the steel

A_c = area of concrete in cross section of bridge

C_c = cost of concrete per unit volume

V_{rs} = volume of reinforcement per unit length of slab

C_{rs} = cost per unit weight of reinforcing steel

C_{pr} = cost of pavement and railing

N_{sf} = number of stiffeners

C_{sf} = cost of stiffeners

N_{sp} = number of splices

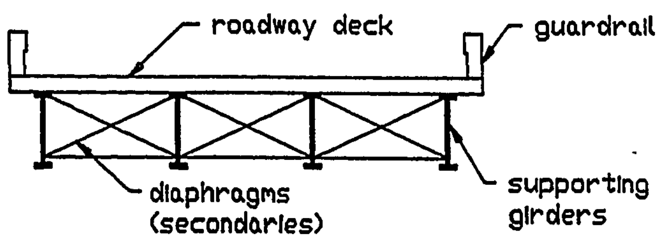
C_{sp} = cost per splice

A_p = area to be painted

C_p = cost per painting

N_{st} = number of shear studs

C_{st} = cost per shear stud



(NON-ISOPARAMETRIC VIEW OF FIGURE 1.1)

FIGURE 5.2 - Cross Section of Bridge Superstructure

The bridge components all interact to help transfer the vehicular loading to the supporting substructure. Although the superstructure's structural performance is dependent upon the integral behavior of all interacting framing, AASHTO specifications consider independently the analysis for the transverse and longitudinal bridge directions. The transverse and longitudinal analysis processes, for the roadway deck and supporting steel girders, respectively, have been described in the preceding chapter.

Considering that the structural analysis procedures are separated, the synthesis process of the total superstructure is accomplished by decomposition of the bridge system into two subproblems: (1) the bridge roadway and (2) the supporting girders. Each subproblem has its own objective function and constraints.

5.3.2 Roadway Deck

The concrete roadway deck consists of the in-place concrete slab, reinforcing steel, guardrailing, and related construction field work as shown in Figure 5.3. Although top and bottom reinforcement is present, the standard beam utilized in the design process of the roadway slab system is considered a one-foot wide, singly-reinforced member without shear reinforcement. This is because the slabs are very shallow (8 to 12 inches) and any compressive steel reinforcement is virtually located at the roadway slab

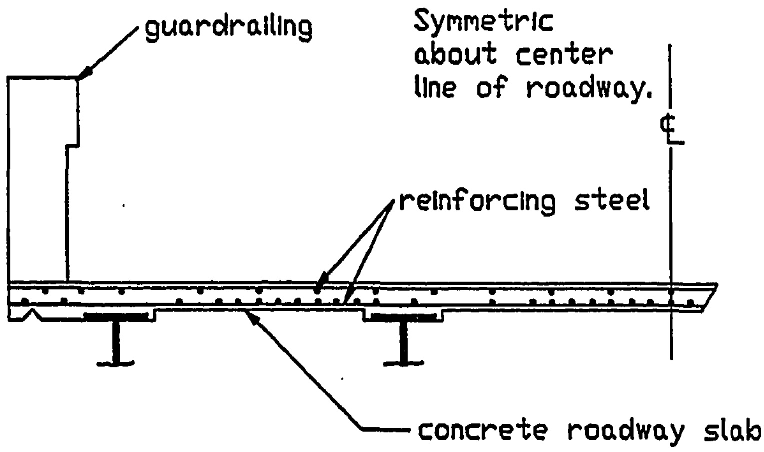


FIGURE 5.3 - Concrete Roadway Section

neutral axis. The synthesis process consists of determining the effective slab depth and reinforcing steel, where the total cost per one foot width of roadway slab is expressed in general terms as:

$$\text{Total cost} = \ell[C_c(bh) + C_sA_s] + C_f(b\ell) + C_{gr}(b)$$

where

C_c = cost of in-place concrete

C_s = cost of reinforcing steel

C_f = cost of forming

C_{gr} = cost of guardrailing

h = overall slab depth

b = one foot width (12")

A_s = area of reinforcing steel

ℓ = overall width of bridge system

Removing the fixed cost and dividing through by the roadway width ℓ allows the objective function to be expressed in terms of cost per lineal foot as:

$$F_o = C_c(bh) + C_s A_s$$

This formulation is allowed since "in all cost optimization programs only the relative cost values are important and not their absolute values. The absolute cost values affect the final value of the objective function but not the optimal value of the design variables." [13]

5.3.2.1 Roadway Unit Costs

Within the roadway segment of CASE-GBRIDGE, default values have been included for unit costs. However, while GBRIDGE is in the interactive mode, the user is given the option of changing any unit cost and, if desired, storing these costs so as to become the future default unit costs. At present, the default unit costs utilized are based upon the 1985 national average of concrete bridge systems as sited by the Concrete Reinforcing Steel Institute (CRSI) publication, "A New Look at Short Span Concrete Bridges - CRSI" [75]. These initial default cost values are:

Variable Costs:

Concrete: \$77.50 per yd^3

Rebar (based upon protective covering):

uncoated - \$0.47 per lb

epoxy coated - \$0.69 per lb

Fixed Costs:

Guardrail cost (cost for single rails):

concrete box railing - \$41.00 per ft

New Jersey railing - \$49.00 per ft

Forming cost: \$13.45 per ft^2

Roadway finish cost: \$10.26 per ft^2

Only the variable unit costs, as applied to the objective function, are used in the synthesis process, but all costs are used in the roadway cost output. The procedure employed by GBRIDGE in recommending reinforcement selection is described in the next chapter.

Referring to Figure 5.4, GBRIDGE always assumes a 2.5 inch reinforcement cover in the formulation of the objective function. In other words, the roadway depth is expressed as

$$h = 2.5 + X_1$$

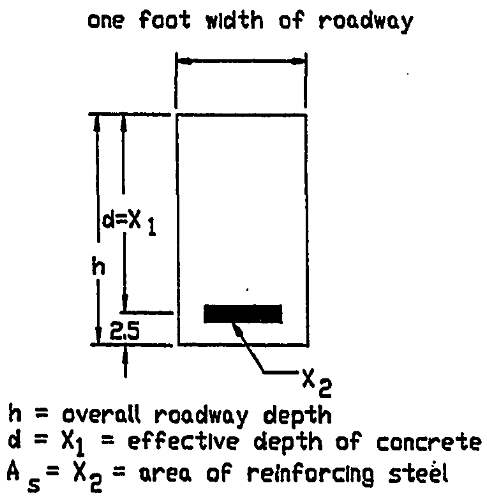


FIGURE 5.4 - Concrete Roadway Analysis/Synthesis Model

where X_1 is the location of the effective depth. Thus, for a twelve-inch wide beam segment, the concrete roadway objective function may be expressed as:

$$F_o(X) = C_c[12*(2.5 + X_1)] + C_s X_2 \qquad \text{Equation 5.1}$$

$$X = (X_1, X_2)$$

or

for uncoated reinforcing steel:

$$F_o(X) = 0.239198X_1 + 1.598X_2 + 0.597994$$

for epoxy-coated reinforcing steel:

$$F_o(X) = 0.239198X_1 + 2.346X_2 + 0.597994$$

where

X_1 = effective depth of reinforcement (in)

X_2 = area of reinforcing steel (in^2)

5.3.3 Steel Girders

For the structural synthesis of steel girders, this investigation considers only the direct manufacturing costs that would affect fabrication in a CADD/CAM environment. In these manufacturing costs, overhead, shipping, and basic erection costs are not considered, except the cost of field splices are included in the total cost output. Also, painting costs are considered a constant and are not used in developing the objective function. The significant cost effects are the direct labor cost and the material cost.

In general, the total cost of the welded plate girder is composed of the material cost of the plate elements, the cost of stiffeners, the cost of splices, and the cost of welding the flanges and web. The bearing stiffeners and shear studs are considered only as design parameters and are used in the total cost evaluation.

5.3.3.1 Steel Girder Objective Function

The supporting steel girder consists of the flange and web plates, stiffeners, shear studs, and related splicing requirements as illustrated in Figure 5.5. The supporting steel girder total cost per fabricated component segment, obtained by separation of the roadway and girder, can be expressed in general terms as

$$\text{Total Cost} = C_1(b_t{*}t_t)LL + C_2(b_b{*}t_b)LL + (C_3+C_4)(d_w{*}t_w)LL + (C_5{*}N_{sf}) + (C_6{*}N_{st}) + C_{sps} + C_{spe} \qquad \text{Equation 5.2}$$

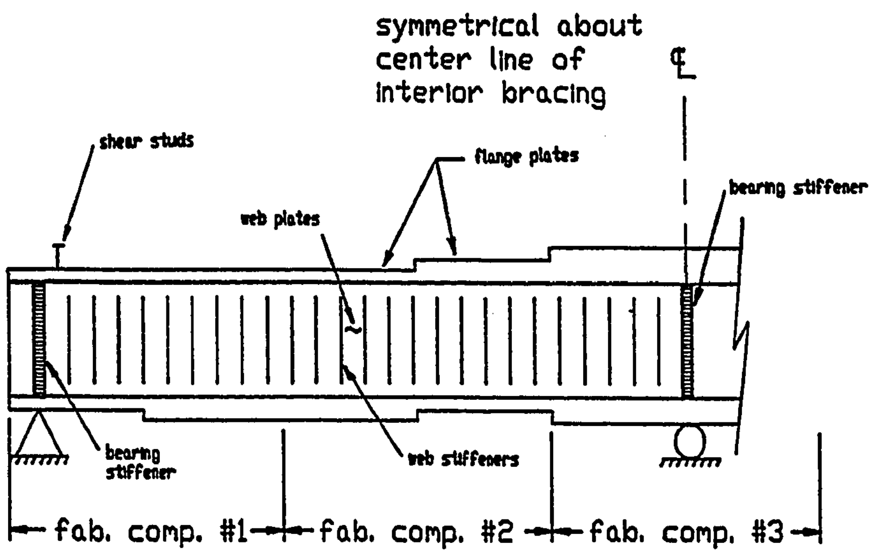


FIGURE 5.5 - Composite, Stiffened Web Girder for Single Span

where

C_1 = material cost per top flange (i.e., 3.2 * cost)

b_t = width of top flange

t_t = thickness of top flange

LL = segment length

C_2 = material cost per bottom flange

b_b = width of bottom flange

t_b = thickness of bottom flange

C_3 = material cost per web

C_4 = fabrication cost for assembly of webs and flanges

d_w = web depth

t_w = web thickness

C_5 = cost per stiffener (material plus fabrication)

N_{sf} = number of stiffeners

C_6 = cost per stud (material plus fabrication)

N_{st} = number of studs

C_{sps} = cost of splice at start end

C_{spe} = cost of splice at terminal end

The splice types and locations are preassigned design parameters to meet shipping and manufacturing restrictions and, along with shear studs, affect only the overall cost. The objective function is expressed in terms of cost per lineal foot by dividing through by the segment length LL, i.e.,

$$F_o(X) = C_1*(b_t*t_t) + C_2*(b_b*t_b) + (C_3+C_4)(dw*t_w) + C_5*(1/d_o)$$

$$(X) = (d_w, t_w, d_o, b_t, t_t, b_b, t_b)$$

where d_o represents the stiffener spacing since the number of stiffeners (NS) equals LL/d_o, and the other variables are as previously defined. A major utility is obtained by formulation of the objective function and related cost in this fashion, i.e., the stiffener spacing for stiffened web girders is evaluated simultaneously with all other design variables. To accomplish this evaluation, it is necessary to relate cost values to girder depth. The various cost components are examined next.

5.3.3.2 Material Cost

The unit cost for steel plates is composed for the base price plus extras. The total price reflects the size, quantity, type, and testing method of the steel plates, i.e.,

Material Cost = Base Price + Size Extra
+ Testing Cost

All material costs are expressed in terms of cents per pound. These costs reflect the actual manufacturing cost of plate girders, and the default values have been obtained by averaging cost data obtained from specific fabricators and steel producers [76,77]. Again, as in the roadway segment, GBRIDGE allows for changes of default cost values in the steel girder synthesis.

5.3.3.2a Base price

The base price reflects the basic unit cost of steel and is a function of the shipping weight. The default base price utilized by GBRIDGE, reflecting delivery cost to the steel fabricator, is $0.2565 per pound.

5.3.3.2b Size extra

The Size Extra is a function of the steel plate size, width, and thickness. It reflects the cost of cutting plates to the desired dimensions, either at the fabrication plant or steel mill. The default size extra prices are given in Table 5.1.

TABLE 5.1 -- Size Extra Costs (cents/lb.)

WIDTH (inches)	5/16	3/8 to 7/16	1/2 to 15/16	1 to 1-1/2	1-9/16 to 2
12-14	5.35	5.35	2.75	2.55	2.15
16-22	4.25	3.80	2.10	1.90	2.10
24-28	2.65	2.20	1.75	1.20	2.00
30-36	2.05	1.60	1.45	1.55	1.95
38-48	1.95	1.50	1.35	1.35	1.90
50-60	1.35	1.00	0.95	1.05	1.60
62-72	1.00	0.75	0.45	0.75	1.30
74-90	0.90	0.65	0.35	0.65	1.15
92-100	1.25	0.90	0.55	0.80	1.30
102-110	1.55	1.15	0.95	1.00	1.35

5.3.3.2c Testing extra

Design specifications may require plates to be subject to impact testing, which is the case for AASHTO specifications. The impact testing is a function of steel grade and plate thickness. The default values employed by GBRIDGE are given in Table 5.2.

TABLE 5.2 -- Impact Testing Cost (cents/lb.)

STI GRADE		THICKNESS				
AASHTO Test @ 40^{O} F	ASTM Designation	5/16 to 1/2	9/16 to 3/4	13/16 to 1	1-1/16 to 1-1/2	1-9/16 to 2
M183	A36	0.55	0.55	0.55	0.85	2.25
M222	A588	5.80	5.80	5.80	5.80	5.90
M223-80	A572	3.05	3.50	3.50	4.35	4.35

5.3.3.3 Stiffener Cost

The stiffeners considered are transverse web stiffeners and bearing stiffeners. The stiffener sizes increase with increasing girder depth and, thus, cost can be expressed in terms of depth. Transverse stiffeners are employed where economical to increase web shear capacity, but bearing stiffeners are required at all supports.

5.3.3.3a Transverse stiffeners

The unit cost of the transverse stiffener is reflected by the girder depth and can be expressed by a linear relationship. For a single side stiffener, at 48 inch depth, the cost of the stiffener is nearly \$45.00. This cost includes the cost of material, welding, and labor. At a depth of 120 inches, the single stiffener cost is approximately \$60.00. Utilizing a linear relationship, the unit stiffener cost per inch of depth is given by

$$C_{st} = \{[0.208333(d_w)] + 35\} \ \$/\text{in.}$$

In the synthesis process, GBRIDGE always considers the stiffener on one side only. Figure 5.6 illustrates the linear variation of cost in transverse stiffeners utilized in GBRIDGE synthesis.

5.3.3.3b Bearing stiffeners

Bearing stiffeners, similarly to transverse stiffeners, can be related to girder depth. However, unlike transverse stiffeners, bearing stiffeners are fabricated flush with both flanges (and web) and are placed on both sides of the

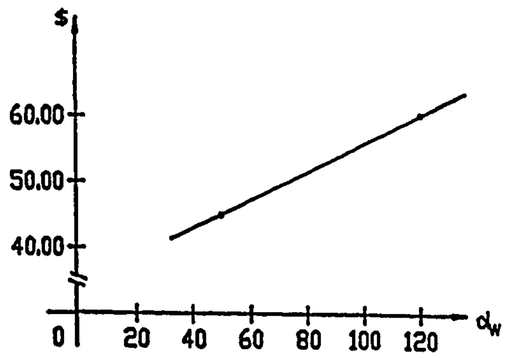


FIGURE 5.6 - Cost Variation in Transverse Stiffeners

web. This is reflected in the higher unit cost. At 48 inch depth, the cost is approximately $100.00, whereas at 120 inch depth, the cost is nearly $135.00. The linear variation is then expressed as

$$C_{bear} = [0.48611(d_w) + 76.667] \ \$/in$$

GBRIDGE uses the bearing stiffener cost only in the total cost evaluation. Figure 5.7 illustrates the linear variation in bearing stiffener cost.

5.3.3.4 Splice Cost

Splice costs are the costs related to interconnecting all of the plate elements into an integral girder shape. The splice consists of the longitudinal connection of the web and flange plates and the transverse butt splice connection at locations of material changes. Two types of transverse butt splices are considered, field splice and

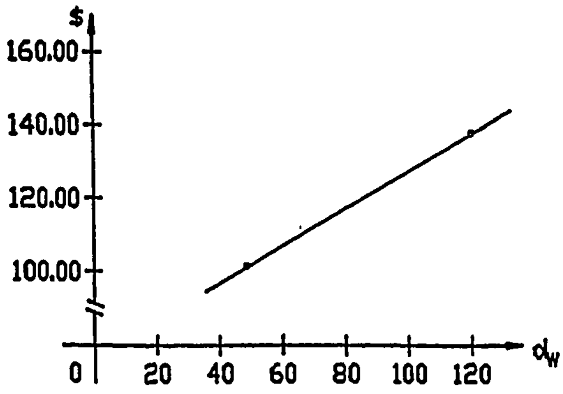


FIGURE 5.7 - Cost Variation in Bearing Stiffeners

shop splice, reflecting the location where the connection occurs. All splices are to be welded. The unit cost reflects the cost of welding material and labor.

5.3.3.4a Web to flange splices

The strength of welding required for connecting the girder's web and flange plates can be evaluated from the standard shear flow formula of mechanics. However, virtually all manufacturing environments require the web-to-flange connection to produce full capacity of the member and, for automated manufacturing facilities employing a production beamline, the cost of web/flange splices are dependent upon web thickness and welding connection type. The standard automated welding types are the full penetration prepared (grooved) butt weld, full penetration squared butt weld, deep penetration fillet weld, and normal fillet weld, as illustrated in Figure 5.8. The most predominant type is the deep penetration fillet weld and, by utilizing only this type in GBRIDGE, the welding cost can be expressed in terms of web thickness only.

The web/flange costs are

$t_w \leq 5/8"$

$$C_{splice} = [0.07(t_w)(16) + 8.05] \ \$/\text{ft.}$$

$t_w \geq 11/16"$

$$C_{splice} = [0.09(t_w)(16) + 10.21] \ \$/\text{ft.}$$

These equations are determined from the cost values of \$8.40/ft for a 5/16 inch web, \$8.75/ft for a 5/8 inch web, \$11.20/ft for a 11/16 inch web, and \$11.65/ft for a one inch thickness. The cost values reflect the fixed cost of set up and labor and the variable welding cost. When the web thickness is 11/16 inch or greater, full capacity welding requires more than a single pass of the beamline welding gun, but the fixed costs are unaltered. These costs are illustrated in Figure 5.9.

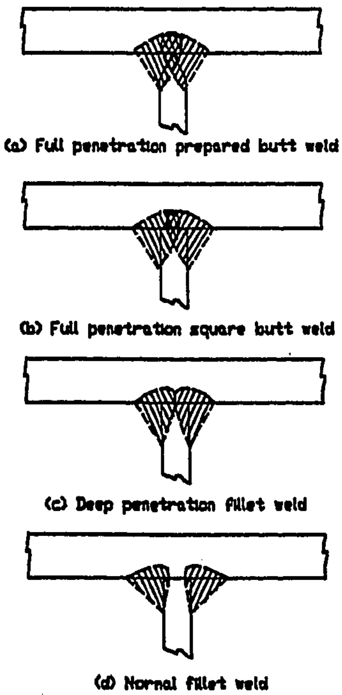


FIGURE 5.8 - Weld Types

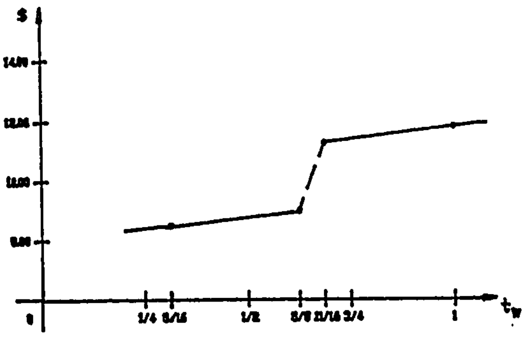


FIGURE 5.9 - Cost Variation in Deep Penetration Fillet Weld

5.3.3.4b Transverse splices

Similarly to the cost of stiffeners, transverse splices can be related to girder depth. All material break change connections, i.e., both shop and field splices, are considered as welded only. The splice cost is composed of the fixed set up cost and the variable welding cost.

For the shop splice, where the girder section changes but the section is to be shipped to the job site as a single unit, the splice costs are approximately $165.00 per 48 inch depth girder and $430.00 for a 120 inch depth girder. Thus, the shop splice cost used in GBRIDGE is given as

$$C_{shop} = 3.75(d_w) \ \$/in$$

This cost variation is illustrated in Figure 5.10. Field splices are considerably more expensive due to increased labor cost and preparation time. The field splice cost employed by GBRIDGE is illustrated in Figure 5.11 and is expressed as

$$C_{field} = 10.5(d_w) \ \$/in$$

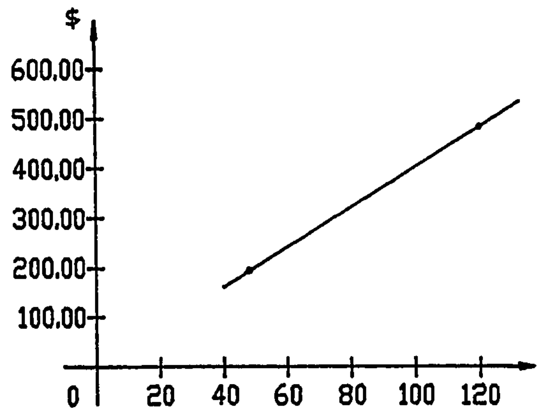


FIGURE 5.10 - Cost Variation in Shop Splice

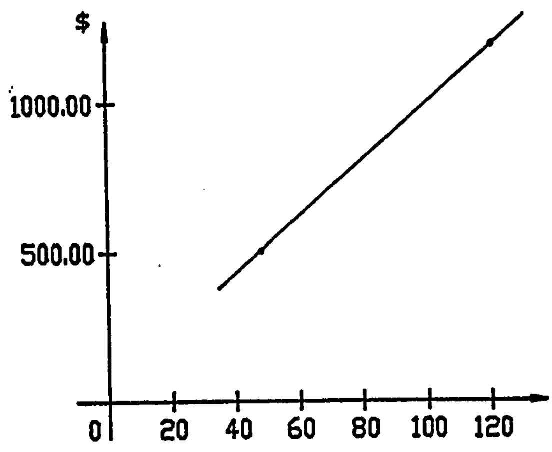


FIGURE 5.11 - Cost Variation in Field Splice

5.3.3.5 Shear Studs Cost

When the steel girder and roadway slab are required to act compositely, shear studs are required to produce the desired integral behavior by transferring horizontal shear from the roadway slab to the steel girder. The cost per individual stud consists of material cost, set up cost and installation cost. These costs, as used by GBRIDGE, are shown in Table 5.3.

TABLE 5.3 -- Shear Stud Cost

stud diameter	4"	height 5"	6"
3/4	0.73	0.76	0.79
7/8	0.75	0.78	0.81
1	0.78	0.81	0.84

5.3.3.6 Cost Versus Weight

From examination of the girder objective function given in Equation 5.2, it is apparent that, for unstiffened girders, minimum weight and minimum cost are synonymous, since the minimum cross-sectional area of a homogeneous section reflects the minimum weight. However, this is not the case when the girder webs are stiffened. The weight of the stiffener is misproportionate to its cost because the stiffeners are extremely labor intensive and, thus, expensive. However, in general, stiffened girders are more cost effective, as shown in Chapter 7.

5.4 General Formulation of Constraints

5.4.1 General

Constraints are the limitations placed upon various aspects of the design so as to apply restrictions to the total design space. As previously indicated, there are two basic constraints types, side constraints and behavior constraints. Side constraints are governed basically by availability of component sizes, whereas the behavior constraints are stress and deflection limitations as specified by AASHTO. In CASE-GBRIDGE, the AASHTO behavior constraints are given for the Load Factor Design (LFD) method as well as the Working Stress Design (WSD) method.

In the traditional WSD method, stress is calculated for service loads and limited to a fraction of the yield or buckling stress for the member under consideration. In the LFD method, as applied to highway girders, structural performance requires the establishment of three different load levels: (1) service load, (2) overload, and (3) maximum load. These load levels correspond to various structural performance requirements. Basically, LFD is based upon the ultimate member capacity and accounts for the difference in load knowledge between the dead load (an accurate evaluation) and live load (a probable approximation), whereas WSD considers a single loading factor of safety. Thus, the important difference between the LFD and WSD methods is that they apply different safety factors in checking the strength of the structure. In the WSD method, a single factor of 1.82 is applied to both the dead and live loads, whereas in the LFD method, factors of 1.3 and 2.17 are applied to the dead and live loads, respectively. In LFD, however, checks for serviceability and overload sometimes control the design because of the lower design factors. The difference between the two design philosophies is illustrated in Table 5.4.

In both WSD and LFD, an elastic analysis is performed, only the resulting effects are multiplied in the LFD approach. In both cases, the dynamic effect of the moving load is included in the design consideration by employing a factored increase of the live load. The amount of increase is dependent upon the superstructure span lengths. The constraints used by GBRIDGE are presented separately for the concrete roadway and steel girders.

TABLE 5.4 -- WSD versus LFD

	WSD	LFD
a) Service Load (D + L)	x	x
b) Overload (D + 1.67L)		x
c) Maximum Load [1.3(D + 1.67L)]		x

5.4.2 Concrete Roadway Constraints

The constraints placed upon the roadway slab by the AASHTO specifications are examined by considering side and behavior constraints separately. Only the effects related to the main reinforcement are examined. The bar selections and distribution are discussed in the next chapter. The main reinforcement in the roadway slab is perpendicular to the traffic (i.e., running across the supporting girders).

5.4.2.1 Roadway Slab Side Constraints

The minimum slab thickness allowed, so that calculations are not required for deflections, is given as

$$h_{min} = \left(\frac{S + 10}{30}\right) * \left(\frac{12"}{1ft}\right) \geq 7"$$

where

S = girder spacing in feet

Thus, the slab depth side constraint is

$$h_{min} - h \leq 0$$

The slab also has a maximum depth limit of 14 inches which is expressed as

$$h - h_{max} \leq 0$$

Limitations are placed upon the size of the reinforcement area to allow for field handling. These rebar size restrictions, in bar size numbers, are

#3 $\leq$ bar size $\leq$ #8

In the bar selection process, the bar size number reflects the bar diameter in 1/8-inch increments, i.e. a #3 bar is a 3/8-inch diameter bar and a #8 bar is one-inch in diameter. GBRIDGE evaluates the reinforcing area and employs this bar size restriction to determine optimal bar sizes and spacing.

5.4.2.2 Roadway Slab Behavior Constraints

Because the basic approach to member safety is different for the WSD and LFD methods, each design philosophy will be examined separately. The service load design approach is based upon a linear stress/strain relationship and examines the actual and allowable stresses of the member. On the other hand, the LFD approach considers only the member's load carrying capacity, i.e., the moment capacity. Both methods, however, are based upon linear elastic behavior.

5.4.2.2a Working Stress Design

The actual stresses calculated at any roadway slab location must be within the allowable stresses defined by AASHTO. The stresses are computed on a service load level. The constraints are expressed in terms of tensile and compression loading as

tension:

$$f_t - F_t \leq 0$$

compression:

$$f_c - F_c \leq 0$$

where capital letters indicate allowable stresses and lower-case letters reflect the actual stresses. The (A) allowable and (B) actual stresses are evaluated from the following formulations.

A) Allowable Stresses

a) Concrete

The permissible extreme fiber stress in compression, F_c, can be expressed as

$$F_c = 0.4f_c' .$$

where f_c' is the ultimate compressive strength of the concrete.

b) Reinforcement

The allowable tensile strength of the reinforcing steel is dependent upon the reinforcing steel yield stress, F_y.

> Grade 40 reinforcement F_t = 20,000 psi

> Grade 60 reinforcement F_t = 24,000 psi

B) Actual Stresses

The actual concrete and rebar tensile stresses are evaluated simultaneously from application of equilibrium and assumed linear behavior. Referring to Figure 5.12a,

$$C = 0.5f_c(12)X$$

$$T = f_tA_s$$

Applying the transformed section method of mechanics of materials, the neutral axis, X, can be computed from the first moment of the tensile and compressive areas as

$$X = \frac{-(nX_2) \pm \sqrt{(nX_2)^2 - 24nX_2X_1}}{12} \geq 0$$

where

$X_2 = A_s$ = area of steel reinforcement

$X_1 = d$ = effective depth of reinforcement, A_s

n = modular ratio (i.e., E_s/E_c)

From the neutral axis location, the moment arm is obtained as

$$\text{arm} = \{X_1 - \frac{X}{3}\}$$

Thus, the tensile and compressive forces for the Working Stress Design moment M_w are

$$T = C = \frac{M_w}{\text{arm}} = \frac{M_w}{\{X_1 - (X/3)\}}$$

Thus, the actual stresses are evaluated as:

a) Concrete

$$f_c = \frac{C}{6X} = \frac{M_w}{6X[X_1 - (X/3)]}$$

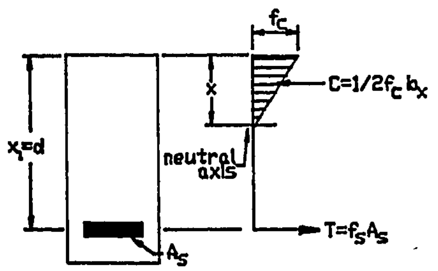
b) Reinforcement

$$f_t = \frac{T}{A_s} = \frac{M_w}{X_2[X_1 - (X/3)]}$$

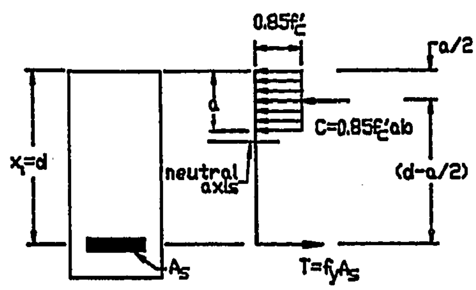
The Working Stress Design moment is comprised of the dead load (M_{DL}), superimposed dead load (M_{SDL}), and live load plus impact (M_{LL+I}) moment contributions and can be expressed as

$$M_w = M_{DL} + M_{SDL} + M_{LL+I}$$

The basic formulation of design roadway moment was presented in Chapter 4, from which the Working Stress Design moment can be expressed as



(a) Working Stress Design Stress Distribution



(b) Load Factor Design Stress Distribution

FIGURE 5.12 - Actual Stress Distributions

$$M_W = [(15X_1+37.5)S^2 + 1.2W_{SDL}S^2 + 0.39(S+2)P] \text{ lb-in}$$

where, as previously defined

S = girder spacing (in feet)

P = wheel load (in pounds)

W_{SDL} = uniformly applied superimposed dead load (lbs/ft)

It is apparent from examining the roadway slab Working Stress Design moment equation plus the objective function and related constraints that the analysis and synthesis for the roadway slab is deterministic. In other words, the synthesis solution simultaneously produces the analytical evaluation. This is true because AASHTO roadway design moments are formulated independently of the finite element based analytical approach. The roadway design moments depend only upon the member depth and girder spacing.

5.4.2.2b Load Factor Design

Although still based upon elastic analysis, the LFD approach compares the factored applied moment to the ultimate usable moment of the roadway slab. CASE-GBRIDGE, for general convenience, considers an analogous stress procedure when utilizing the design check procedure as described in the next chapter. The basic LFD behavior constraint is expressed as

$$M_{fact} - M_{ult} \leq 0$$

The factored applied moment, M_{fact}, is less than the ultimate moment capacity, M_{ult}.

A) Factored Moment

The factored design moment is the greater of the moments produced by factored service moments or the cracked section moment. Thus,

$$\begin{aligned} M_{fact} &= 1.3[M_{DL} + M_{SDL}] + 2.17[M_{LL+I}] \\ &= (19.5X_1+48.75)S^2 + 1.56W_{SDL}S^2 + 0.651(S+2)P \end{aligned}$$

where the variables are the same as defined in the Working Stress segment.

B) Ultimate Usable Design Moment

The ultimate strength design moment, M_{ult}, provided by a member is the nominal strength, M_n, calculated in accordance to strength design philosophy, multiplied by a strength reduction factor, ϕ . Thus, referring to Figure 5.12b, $M_{ult} = \phi M_n = 0.9(M_n)$.

$$M_n = A_s f_y [d - (a/2)]$$

where

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

These variables have all been previously defined, from which the ultimate strength moment can be expressed as

$$M_{ult} = 0.9(M_n) = 0.9 X_1 X_2 f_y - \frac{X_2^2}{22.667} \left(\frac{f_y}{f_c'}\right) f_y$$

To insure a ductile failure mode, the reinforcement ratio must be within the limit $\rho_{min} < \rho < \rho_{max}$. Rho ($\rho$) is defined as the ratio of the area of reinforcing steel to the effective concrete area (b_d), i.e.,

$$\rho = \frac{A_s}{b_d}$$

The reinforcement ratio limitations are defined as

$$\rho_{min} = \frac{200}{f_y}$$

$$\rho_{max} = 0.75\ _b = 0.75\{0.85 \left(\frac{f_c'}{f_y}\right)\left[\frac{87000}{87000 + f_y}\right]\} \doteq (\text{matl } \#)$$

Thus, using the previously defined variable relation, the reinforcing steel limitations can be expressed as

$$X_2 - \frac{2400X_1}{f_y} \geq 0$$

$$12(\text{matl } \#)X_1 - X_2 \geq 0$$

Thus, to insure a ductile failure mode, the reinforcement ratio must be within the limit $\rho_{min} < \rho < \rho_{max}$.

5.4.3 Steel Girders

The steel girders are the main supporting members of the bridge superstructure and, as such, considerable restrictions are placed upon their performance to ensure safety. Similarly to the roadway slab, the girder constraints will be examined in terms of side constraints and behavior constraints being subdivided in accordance with design philosophy.

5.4.3.1 Girder Side Constraints

The Girder Side Constraints are the same for both the WSD method and the LFD method. These side constraints are placed upon the available plate sizes as

$1/2" < t_t, t_b < 2$ by 1/16" increments

$12" < b_t, b_b < 24$ by 2" increments

$5/16" < t_w < 1$ by 1/16" increments

The web depth is limited on plate sizes to

$24" < d_w < 70"$ by 2" increments

The total girder depth is also limited by the AASHTO span/depth ratio and the maximum depth applicable for simple flexure theory. The girder depth, D_G is defined as

$$D_G = d_w + t_t + t_b$$

The limits on the span/depth ratio are

$$\frac{L}{30} < D_G < \frac{L}{15}$$

$$\frac{L}{25} < D_G + T_{con} < \frac{L}{15}$$

where

L = the girder span length (DL analysis inflection point locations)

T_{con} = concrete roadway slab thickness

The stiffener spacing, d_o, is limited to fractions of the actual girder depth, i.e.,

$$0.5d_w \leq d_o \leq 1.5d_w$$

5.4.3.2 Girder Behavior Constraints

The backtracking procedure considers only whether the girder section passes the design criterion for flexure and shear. Thus, for compatibility to code, the constraints are expressed in terms as given by the AASHTO specifications. The variables as applied to GBRIDGE have been given previously.

5.4.3.2a Working Stress Design

In the Working Stress Design approach, the actual stresses are checked against the allowable stresses for the flexure and shear conditions.

A) Bending stress:

$$f_b < F_b = 0.55\ F_y$$

B) Shear stress:

$$f_v < F_v = 0.33\ F_y$$

If $f_v > 0.6\ F_v$, then

$$f_b < \{0.754 - [(0.34 f_v)/F_v]\} F_y$$

C) Compression flange (either top or bottom):

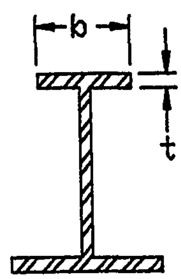
For noncomposite sections,

$$b/t < 3250/\sqrt{f_b} < 24$$

where b is the flange width, t is the flange thickness, and f_b is the calculated maximum compressive stress in psi. For composite sections,

$$b/t < 3860/\sqrt{f_{dll}}$$

where f_{dll} is the top flange compressive stress due to noncomposite dead load.

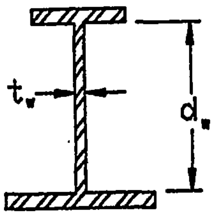


D) Web without Transverse Stiffener

$$t_w > d_w/150$$

$$F_v < (5.625*10^7)/(d_{w/tw})^2 < 0.33F_y$$

where t_w is the web thickness and d_w is the web depth.



E) Web with Transverse Stiffener

$$t_w > d_w\sqrt{f_b}/23{,}000 > D/170$$

$$F_v < F_y/3[c + 0.87(1-c)/\sqrt{1 + (d_o/d_w)^2}]$$

where

$$c = 2.2(10^8)\{[1 + (d_w/d_o)^2]/F_y(d_w/t_w)^2\} < 1$$

$$d_o < 1.5d_w$$

(d_o is the spacing of transverse stiffeners)

F) Web with Transverse and Longitudinal Stiffeners

$$t_w > d_w\sqrt{f_b}/46{,}000 > D/340$$

G) Fatigue Stress

The actual stress ranges should not exceed the allowable fatigue stress ranges given in Table 5.5. In other words, it is required that

$$f_r(\text{actual}) < f_{sr}(\text{allowable})$$

5.4.3.2b Load Factor Design

According to AASHTO specifications, LFD is an alternative method for design of simple and continuous beams and girder structures. It is a method of proportioning structural members for multiples of the design loads.

TABLE 5.5 - Allowable Fatigue Stress Range for Redundant Loads

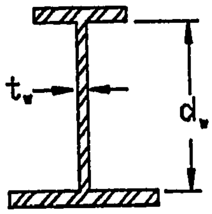
Category (1)	Allowable Range of Stress, F_{sr} (KSI) For 100,000 Cycles (2)	For 500,000 Cycles (3)	For 2,000,000 Cycles (4)	For Over 2,000,000 Cycles (5)
A	60.0	36.0	24.0	24.0
B	45.0	27.5	18.0	16.0
C	32.0	19.0	13.0	10,12[a]
D	27.0	16.0	10.0	7.0
E	21.0	12.5	8.0	5.0
E'	16.0	9.4	5.8	2.6
F	15.0	12.0	9.0	8.0

[a]For Transverse Stiffener Welds on Webs or Flanges

A) Web

$$d_w/t_w < 13,300/\sqrt{F_y}$$

where d_w is the beam depth and t_w is the web thickness.



B) Projecting Compression Flange Element

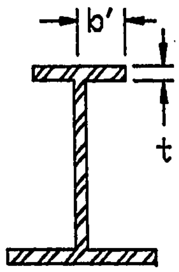
For noncomposite sections

$$b'/t < 1{,}600/\sqrt{F_y}$$

For composite sections

$$b'/t < 2{,}200/\sqrt{1.3F_{dll}}$$

where b' is the projecting flange element width, t is the flange thickness, and F_{dll} is the top flange compressive stress due to noncomposite dead load.



C) Maximum Shear

$$V < V_u$$

where

$$V_u = 0.55F_y(d_w)t_w$$

D) Lateral Bracing (noncomposite sections only)

$$L_b/r_y < 7{,}000/\sqrt{F_y} \quad \text{when } M_2 > 0.7M_1$$

$$L_b/r_y < 12{,}000/\sqrt{F_y} \quad \text{when } M_2 < 0.7M_1$$

where L_b is the distance between two compression flange bracing points, r_y is the radius of gyration with respect to the minor axis, and M_1 and M_2 are the moment components at two adjacent bracing points.

E) Maximum Strength

$$1.3[(D + 5)/3(L + I)] < M_u$$

where M_u is the resultant moment of fully plastic stress distribution acting on a section. It is determined as follows:

$$M_u = F_y Z \qquad \text{(noncomposite sections)}$$

F) Overload Criteria

$$\sigma < 0.80F_y \qquad \text{(composite sections)}$$

$$\sigma < 0.95F_y \qquad \text{(noncomposite sections)}$$

where σ is the maximum bending stress under an overload of $[(D + 5)/3(L + I)]$.

5.4.3.2b(i) Braced noncompact sections

A) Web

For unstiffened sections

$$d_w/t_w < 150$$

For transversely stiffened sections

$$d_w/t_w < 36{,}500/\sqrt{F_y}$$

B) Projecting compression flange element

For noncomposite sections

$$b'/t < (2{,}200/\sqrt{F_y})(\sqrt{M_u}/\sqrt{M})$$

For composite sections

$$b'/t < 2{,}200/\sqrt{1.3F_{dll}}$$

where M_u is the maximum strength (F_yS) and M is the actual moment.

C) Lateral bracing (noncomposite sections only)

$$L_b < 20{,}000{,}000(A_f/F_{yd})$$

where A_f is the cross sectional area of the flange element.

D) Maximum shear

$$V < V_u$$

For unstiffened sections

$$V_u = 3.5Et_w^3/d_w < V_p = 0.58F_y d_w t_w$$

For stiffened sections

$$V_u = V_p[c + 0.87(1-c)/\sqrt{1 + (d_o/d_w)^2}]$$

where E is the elastic modulus of steel in psi and d_o is the distance between two transverse stiffeners. V_p and c are determined as follows:

$$V_p = 0.58F_y d_w t_w$$

$$c = \{18{,}000(t_w/d_w)\sqrt{[1 + (d_w/d_o)^2]/F_y}\} - 0.3 < 1.0$$

E) Maximum strength

$$1.3[(DL + 5/3(LL + I)] < M_u$$

where

$$M_u = F_y S$$

F) Overload criteria

$$\sigma < 0.80F_y \qquad \text{(composite sections)}$$

$$\sigma < 0.95F_y \qquad \text{(noncomposite sections)}$$

where σ is the maximum bending stress under an overload of $[(DL + 5/3(LL + I)]$.

5.4.3.2b(ii) Unbraced sections (noncomposite only)

A) Web

For unstiffened sections

$$d_w/t_w < 150$$

For transversely stiffened sections

$$d_w/t_w < 36,500/\sqrt{F_y}$$

B) Projecting compression flange element

$$b'/t < (2,200/\sqrt{F_y})(\sqrt{M_u}/\sqrt{M})$$

C) Maximum shear

$$V < V_u$$

For unstiffened sections

$$V_u = 3.5Et_w^3/d_w < V_p = 0.58F_yd_wt_w$$

For stiffened sections

$$V_u = V_p[c + 0.87(1-c)/\sqrt{1 + (d_o/d_w)^2}]$$

where E is the elastic modulus of steel in psi and d_o is the distance between two transverse stiffeners. V_p and c are determined as follows:

$$V_p = 0.58F_yd_wt_w$$

$$c = \{18,000(t_w/d_w)[1 + (d_w/d_o)^2]/F_y\} - 0.3 < 1.0$$

D) Maximum strength

$$1.3[(DL + 5/3(LL + I)] < M_u$$

where

$$M_u = F_yS[1 - (3F_y/4\pi^2E)(L_b/b')^2]$$

For unsymmetrical beams, b' is replaced by 0.9b'.

If $V > 0.6V_u$, $M = M_u[1.375 - (0.625V/V_u)]$.

VI. IMPLEMENTATION OF THE CASE METHODOLOGY TO GIRDER BRIDGES

6.1 General

Economic considerations of both engineering and fabrication costs dictate a change in traditional methods of design and fabrication of structural framing systems to a more cost effective and time efficient alternative. To this end, the CASE methodology has been developed herein. However, the various aspects for implementation of this methodology require elaboration, in particular, the implementation to girder bridges, CASE-GBRIDGE.

Within this chapter, the major components of the CASE-GBRIDGE system will be explored. This chapter will consider the following: (a) interactive control and data modification, (b) the significant features of the individual modules of GBRIDGE, and (c) example problems to illustrate the methodology.

6.2 Interactive Control and Modification of CASE-GBRIDGE

The development and implementation of the CASE methodology utilizes interactive programming techniques and modular program structure so as to allow the user the maximum flexibility possible for his particular bridge design needs. The interactive aspect is essential to ensure that the user has complete control over the finalized design. In other words, this ensures that the user is aided by the computer-assisted design capabilities incorporated into GBRIDGE but not relieved of his professional design responsibilities. The operations of the CASE-GBRIDGE program involve the engineer's response via the keyboard to questions, instructions, or requests for data which appear on the terminal screen. Program responses to various levels of input data appear on the screen and the program awaits the user's prompts to verify, modify, or change input before continuing. The user is allowed to review any or all data by either graphical display or tabular alphanumerical screen display before receiving the finalized output from the corresponding printer. Capacity for batch operation processing is also included for certain applications of GBRIDGE.

The interactive nature, when utilized, allows the user to direct the program operations based upon intermediate results. This computational directional control is accomplished through the use of various display menus, with the major programming control generated from the "Master Menu." This master menu allows access to all of the program modules that together comprise the CASE-GBRIDGE system. This menu, illustrated in Figure 6.1, is initially displayed

```
                For Analysis and Optimized Design of
                        Steel Girder Bridges

SELECTION OPTION;  Rotate Control knob to desired
                   Option and press SELECTION (k9 key)

                ==>SELECTION 1- RUN BRIDGE INPUT SEGMENT
                   SELECTION 2- RUN BRIDGE ROADWAY SEGMENT
                   SELECTION 3- RUN BRIDGE GRAPHICS/COST SEGMENT
                   SELECTION 4- RUN BRIDGE ANALYSIS SEGMENT
                   SELECTION 5- RUN BRIDGE SYNTHESIS SEGMENT
                   SELECTION 6- RUN BRIDGE DESIGN CHECK SEGMENT


                |             |             |             |
                |             |             |             |SELECTION
```

FIGURE 6.1 - CASE-GBRIDGE Master Menu

when the program is loaded and returned to after each program segment is executed, when operating in the interactive mode. These program modules are individually examined in the following sections so as to describe their contributions to the CASE-GBRIDGE operations.

6.3 Modules of CASE-GBRIDGE

To effectively examine the features of each individual program module and how it is incorporated within the concepts of CASE, features of each module and its contributions will be examined separately. The modules will be examined in the following order:

(1) Bridge Data Input Module

(2) Bridge Analysis Module

(3) Bridge Design Verification Module

(4) Bridge Roadway Module

(5) Bridge Synthesis Module

(6) Bridge Graphics/Cost Module

The reason for this order in the presentation is that, for the condition of batch processing, the first three modules are automatically executed programmatically. However, after the input segment, when in the interactive mode, the user may access any desired module.

6.3.1 Bridge Data Input Module

The bridge data input segment of GBRIDGE was developed to be user friendly, requiring minimal input to completely specify the bridge system and loading while simultaneously giving a multitude of data input checks for the user to verify, modify, or change data at any stage of program execution. One of the first considerations of the input module is whether existing data previously stored is to be reused or if the user instead wishes to supply completely new input data. The feature allows the user to rapidly review any bridge system previously stored by simply loading the data stored under a given filename. Also, when a new bridge system is to be examined that is similar to a bridge system previously stored, the user simply loads the data by the filename and modifies this data as required to reflect the new bridge system. Actually, the use of the filename is required in various program modules and necessitates that the database be flexible and structured so that the proper information is passed correctly through the program operation. The various layers of the database key off of the filename of the bridge system under consideration.

Screen displays utilizing default values are employed in the input process to minimize the user input effort. Where appropriate, screen displays automatically generate all the initial design parameters by a default mode. For example, such items as design method, loading type, interactive mode, support fixity, etc., are automatically displayed and await the user's response to simply verify or change as desired. After all of the data has been input, whether via existing file data, completely new data, or a combination thereof, all bridge data is displayed for final verification before the user is allowed to exit the input data module.

Queries for input data for the fabricated components is keyed off of individual fabricated component types. Each fabricated component represents exactly the physical part that is to be manufactured, but the structural behavior is dependent upon the overall girder performance, i.e., the structural analysis member. The use of the component type allows for generation of the structural analysis member from the component input data. As previously stated, the generation of these analysis members is accomplished by program recognition that specific fabricated component types occur only at initial and terminal ends of the supporting steel girders. The input process begins with fabricated component data input for an abutment girder and continues until termination is prompted by input of another abutment girder.

6.3.2 Bridge Analysis Module

The bridge analysis segment considers the total loading effect by application of superposition of the individual effects from the dead load, superimposed dead load, and live load conditions as detailed in Chapter 4. As previously stated, the live load condition is obtained by generating influence lines and then applying either the AASHTO truck or lane loading condition. Again, it must be noted that, as presently structured, GBRIDGE does not automatically compare the maximum effect between the truck and lane loading conditions.

6.3.3 Bridge Design Verification Module

This program module evaluates the capability of the girder system to withstand the applied loading as calculated in the analysis segment. To accomplish this evaluation, however, a somewhat obscure point needs clarification. Although the internal moment and shear forces are evaluated at every analysis point, the determination of design parameters, actual and allowable stresses, or factored and

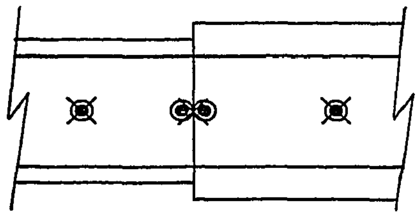


FIGURE 6.2 - Section Properties Points

allowable moments are carried out at section properties points. At least one section property point occurs at every analysis point; however, at the analysis points where changes in material sizes occur, two section properties are assigned, one on each side of the material break as shown in Figure 6.2. Therefore, utilizing the section properties points accounts for the true variation in member properties throughout the girder.

The design verification module controls and directs all of the output of CASE-GBRIDGE, except for that provided from the roadway and graphics/cost modules. Program output for the steel girders is directed to either the screen display or corresponding printer as specified by the user through prompts from the design verification module. Two levels of output are possible, depending upon the amount of details desired. Although the AASHTO design code limitations are the same as may have been employed in the structural synthesis process, they are examined independently within the design verification program segment for each section property point. The output is generated and displayed one girder at a time until all girder members have been examined. The display consists of bridge geometry aspects, member forces and member properties, member design parameters, and a stress summary. This display restriction is due to the RAM limitations.

For the stress display section of the output, a modification has been incorporated into the output for the Load Factor Design condition. This modification allows for the ultimate design moment, M_{ult}, to be expressed as an allowable stress and compared to the factored moment, also expressed in terms of stresses. Actually, both the maximum allowable moment versus the factored moment plus the allowable stresses versus the actual stresses are displayed. The reasoning for this modification is that most engineers are accustomed to performance evaluations based upon stresses but are uncomfortable dealing in member capacities. The modification is accomplished by simply considering the condition under which the maximum strength of the design moment is determined, i.e., the condition used for evaluating M_{ult}. The actual stresses are the factored load conditions divided by the appropriate section properties.

Finally, the design verification module allows for graphical display of the allowable stresses versus actual stresses. This is accomplished by plotting the evaluated actual and allowable stresses at each section property point and interpolating graphically. The graphically displayed stress plots are for stress variations on both the top and bottom girder flange. Again, because of the memory restrictions, these plots are illustrated for each individual member separately. Also, the graphics plots are allowed only in the interactive mode when requested by the user.

6.3.4 Bridge Roadway Module

This module is only accessible when executing GBRIDGE in the interactive mode. The roadway module is, to a certain extent, an independently working program segment employing both its own particular input data and that already input in GBRIDGE. This input within the roadway segment is similar to the bridge input segment in that display screens are utilized which employ default values and allow the user to verify, modify, or change the data. Data which relates both to the supporting girder and the roadway slab, such as girder spacing, number of girders, etc., is passed into the roadway module and cannot be modified while in this segment. Note, however, that the roadway slab thickness is not common to the various individual program modules. This is because of the difference in approaches for the analysis and synthesis of the roadway slab versus the supporting girders.

The analysis can be applied to any specified input or synthesis can be used to assist in designing the roadway slab. As previously indicated, the roadway analysis is deterministic, i.e., the analysis and synthesis evaluations can be performed simultaneously. This is due to the simplified procedure employed by AASHTO for roadway slab moment evaluations. The design variables employed in the cost minimization procedure also completely determine the variables (along with girder spacing) required in the moment evaluation as described in Chapters 4 and 5. Thus, the need to separate the analysis and synthesis is not present and only a single evaluation is required. This condition occurs in all statically determinate systems, or where design variable linkage is utilized to reduce the magnitude of the problem. Thus, the unknowns evaluated in the analysis/synthesis are the unknown design variables and can be evaluated directly, instead of evaluating unknown displacements that result from the FEM approach. Note that the cost values employed have default values; again, the user is to verify, modify, or change the data.

In the roadway slab model of a one-foot width beam, the basic assumption is that the equivalent member could be treated as a singularly reinforced concrete section. The design variables are the area of reinforcing steel and

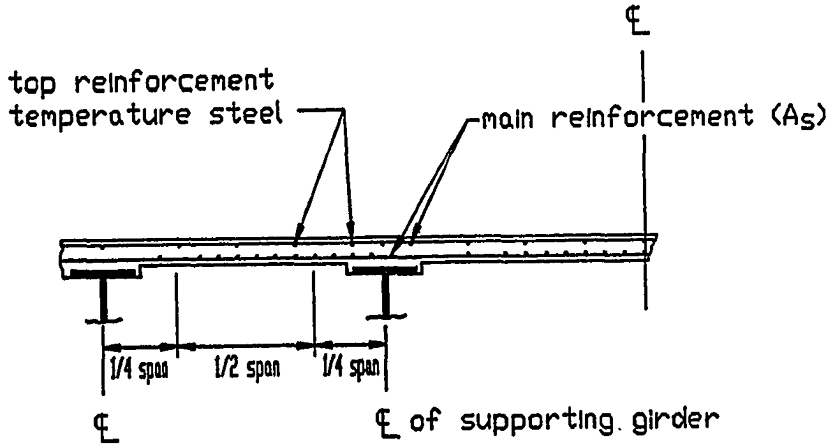


FIGURE 6.3 - Reinforcing Steel Distribution

concrete depth as defined in Chapter 5. These variables only partially describe the roadway system. However, all of the other parameters can be obtained from these. AASHTO specifies that the longitudinal reinforcement be a percentage (200/S < 0.67) of the transverse main steel and, since only positive bending occurs in the longitudinal direction, the longitudinal bending moments result in tension in the lower reinforcing steel. A percentage of the steel is placed at the middle (0.50) and quarter (0.25) as illustrated in Figure 6.3. From these required areas, GBRIDGE selects the minimum bar areas based upon spacings ranging from six inches to twelve inches and bar sizes from #4 bars through #8 bars. The top temperature steel is set at #4 bars at 18-inch spacing, the maximum allowed per AASHTO specifications.

Similar to the design verification module, the roadway module has modified the Load Factor Design philosophy to allow the investigation of member capacity in terms of both moments and stress. From considering the design factored load case only, the allowable stresses are:

Allowable tension stress in reinforcing steel = $0.9f_y$

Allowable concrete compressive strength = $0.765f_c'$

where

f_y = yield strength of rebar

f_c' = 28-day concrete compressive strength

Output is directed from the roadway module to either the display screen or printer as specified by the user. Besides the design parameters, reinforcement distribution, stresses and moments, etc., the roadway costs are also indicated. These costs reflect the in-place cost of a cubic yard of roadway slab plus the total cost per one-foot width of bridge roadway. The one-foot width cost is common to other modules after this program segment has been executed.

6.3.5 Bridge Synthesis Module

Synthesis of the supporting steel girders, unlike the roadway slab synthesis, requires several iterations between the bridge analysis module and the backtracking module. The reasoning, in this case, is that the bridge analysis and synthesis are implicitly interlocked and interdependent as previously described in the "structural synthesis" approach. As each iteration of the analysis cycle occurs for the current bridge element values, the analytical evaluation yields unknown displacements which are related to an internal distribution of loads. The synthesis, based upon this internal loading, effectively restructures the loading distribution and, therefore, changes the analytical results. This fluctuation can be corrected only by reanalysis and new synthesis.

As previously discussed, certain descriptive bridge element values remain constant during the synthesis process and are termed design parameters whereas other values are allowed to vary and are termed design variables. Together, the design parameters and variables fully describe all elements of the structural bridge system. For GBRIDGE, the locations of the field splices are preset by the fabricated component lengths and are generally controlled by shipping constraints. The plate lengths for shop butt splices are preset so as to reflect manufacturing constraints. Thus, the synthesis process is required to select girder depths, plate sizes, and, when applicable, web stiffener spacing. The selection process implemented in the CASE-GBRIDGE operations will be examined next.

The single most critical design variable is the selection of the girder depth. The girder depth effectively controls the member properties utilized in the analysis operation. The resulting analytical internal forces vary throughout the bridge span and, thus, the resulting stress condition varies. The backtracking technique is applicable to a specific force/displacement condition and, if it were applied at each section property point of the girder, each point would yield a different girder depth and plate size. In a study conducted by Knight [78], it was determined that variable depth girders are uneconomical for span lengths less than 400 feet. In accordance with this result and since the vast majority of all bridge spans are less than 400 feet, a constant depth web is adopted for this study.

The depth selection process consists of three analysis/synthesis iterations in which certain predefined analysis points are examined. These locations are at analysis points of absolute maximum moment within each length of fabricated components except at abutment girders where the point is preset to the terminal analysis point. In the initial synthesis, the flange widths are held constant at input values unless the width is less than $[(d_w/4) + 2.5]$ or greater than 24 inches, in which case the plate widths are temporarily set at the minimum or maximum plate widths, 6 or 24 inches, respectively. When plate stiffener spacing are initially input, they are temporarily set equal to the start depth of the individual components. Employing this criteria, the backtracking module is executed for the analysis point location of maximum absolute moment in each fabricated component segment. This application yields a unique depth and thickness for each point. The new girder depth is then ascertained as the weighted average as defined by

$$d_{w(avg)} = \frac{\sum_{i=1}^{n} (t_w)i\,(d_w)i}{\sum_{i=1}^{n} (t_w)i}$$

where n is the number of analysis points and $(d_w)i$ and $(t_w)i$ are the appropriate web depth and thickness, respectively, for section point i. Upon evaluation of the overall bridge girder depth, the analysis members are reformulated and a reanalysis performed to account for the fact that the load distribution has changed. The second synthesis is then performed analogously to the first. Again, a reanalysis is performed before the third synthesis, at which time, all the plate components are allowed to very except, when present, the web stiffener spacing. Upon completion of this iteration, the final girder bridge depth is evaluated and the final iterative reanalysis is performed.

Utilizing a constant girder depth, either determined by the synthesis process just described or input by the user, each plate length segment is examined for maximum positive and negative moment conditions. The backtracking technique is performed, considering all design variables as active, by examining the section properties point having the maximum absolute positive or negative moment. If, within the length segment of the fabricated component, the moment condition changes, then, the section properties points within the component length possessing the largest absolute value of moment with negative sign to the previously evaluated condition is always examined. The resulting variables from the two synthesis are compared with the largest plate elements and smallest stiffener spacing being defined as the final fabricated component values. It must be noted that, for most components, the moment actually does not change sign and, thus, only one backtrack evaluation is required after the girder depth is determined.

After the final selection of the design variables has been accomplished, GBRIDGE then automatically performs another analysis and design check. The design verification module, as previously stated, employs independent constraint checks and affords the user the opportunity to examine the synthesis results either graphically or alphanumerically. The user can then proceed to the final output, manually modify specific design values via application of the bridge input module, or re-execute the backtrack module with the girder depth preselected or algorithmically determined. In any case, the finalized data is only stored if the input module is executed and a filename specified.

6.3.6 Bridge Graphics/Cost Module

To access the bridge graphic/cost module, again, GBRIDGE must be operating in the interactive mode. As the name implies, this module has two primary functions, graphical bridge display and superstructure cost evaluation. The graphical display basically allows the user visual verification of the current bridge system via bridge elevations and/or bridge plans. The display of elevations or plans are independent of each other with the plan display also containing cost information. The elevations indicate span lengths, joint fixity, etc., currently active in the GBRIDGE program. This segment, if utilized after initial input, will serve to insure that all fabricated components have been properly input.

The bridge plan display, when requested, will present a plan view and cross section of the bridge system currently being considered plus yield the cost information pertaining to the particular structure. The cost of the steel girders are re-evaluated based upon current fabricated component information, including both shop and field splice costs. However, costs relating to shipping, erection, painting, diaphragm bracing, etc., are not included, i.e., only basic manufacturing costs are considered. The roadway cost information must be passed from the roadway module, else the cost effect is considered to be zero.

6.4 Implementation of CASE-GBRIDGE

To demonstrate how the various modules of GBRIDGE interact, the methodology is applied to the two-span continuous bridge system shown in Figure 6.4. The girders are considered to be composite in both the positive and negative moment regions and web stiffeners are utilized. The bridge spans are both 100 feet and the WSD philosophy is employed. The girder depth has been preselected to be 48 inches and user interaction has been utilized to obtain the finalized output. The bridge is assumed to be part of an interstate highway and the loading is specified as HS 20-44 truck loading with a fatigue stress cycle of 2,000,000. The roadway width of 28 feet is required for two design traffic lanes and the overall bridge width of 30 feet is used to accommodate guardrails. The girder spacing is defined as 9.25 feet for four girders symmetrically arranged about the bridge centerline. The cost values used for both the concrete roadway and supporting steel girders are the values generated in Chapter 5.

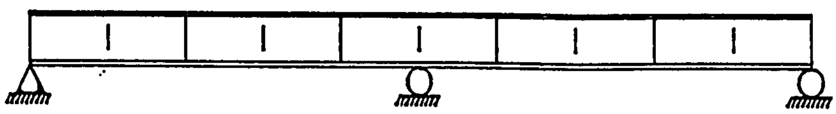


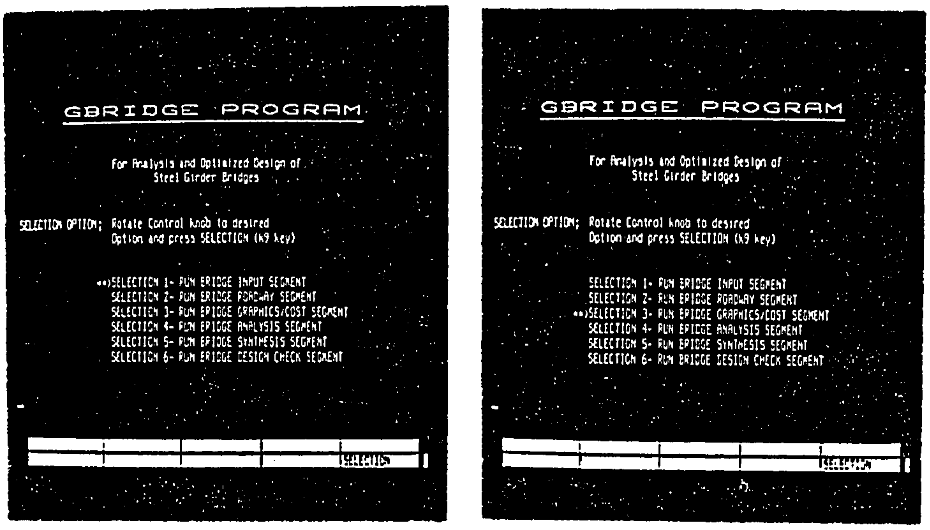
FIGURE 6.4 - Two Span, Continuous Bridge System

The general programmatic operations of the CASE-GBRIDGE methodology are illustrated in Appendix C through a generic pseudo-flowchart listing. Additionally, two finalized outputs of GBRIDGE are presented in Appendix D for the described two-span bridge system considering both the WSD and LFD methods. These reports are generated without utilizing any additional user interaction to modify the bridge design after the final synthesis/analysis operation has been executed. For these bridges, the complete output reports are presented except that the roadway report is only included in the WSD report.

6.4.1 Application of CASE-GBRIDGE

The initial screen display upon executing GBRIDGE is the master menu as illustrated in Figure 6.5a. The master menu serves the purpose of driver module to allow for interaction between the various program segments. User control is via a rotational control knob which allows the user to direct a screen pointer to any desired option, at which time pressing the indicated softkey, and upon user verification, will cause execution of the desired module as illustrated in Figure 6.5b.

The first step in any bridge system investigation is the introduction of bridge system data into the computer. This is accomplished by executing the Bridge Input Segment. The input module initially prompts the user on whether



(a) (b)

FIGURE 6.5 - GBRIDGE Master Menu

existing bridge data currently residing in secondary storage is to be reused or if entirely new bridge data will be input. The bridge data consist of material properties, bridge geometry, design loading, fabricated component data, etc. The input format has been structured to allow the user maximum flexibility and utility by employing multiple prompts and default values. There is a distinction, however, between the basic bridge data and the fabricated component data. The basic bridge data is input once, verified, and eventually reviewed, as illustrated in Figure 6.6. This figure outlines the process for entering new input data. Note that, to change a default value, all that

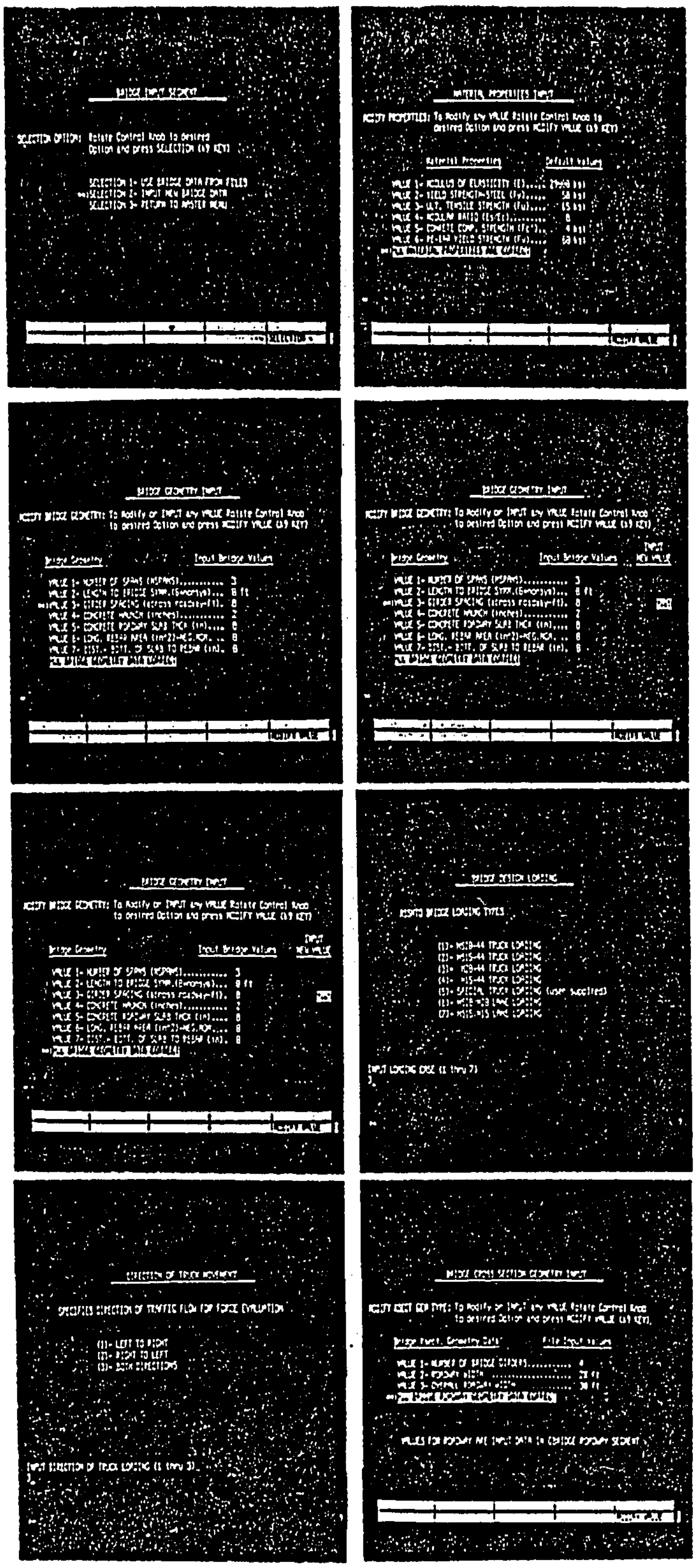


FIGURE 6.6 - Bridge Data Input Segment

is required is simply to move the screen pointer to the specific item to be changed by use of the control knob, pressing the softkey to indicate that this value is to be modified and entering the desired value.

As stated, the basic bridge data is entered only once, whereas the fabricated component data must be entered for each physical component of the bridge system. In other words, the fabricated component data is iteratively input. In the fabricated component section, the initial prompt is used to define the girder type. The only purpose of this selection process is to select the proper data input menu to simplify and reduce the input data requested from the user. This selection process is illustrated in Figure 6.7. The fabricated component data is entered by first selecting the component type and then inputting the requested data. As

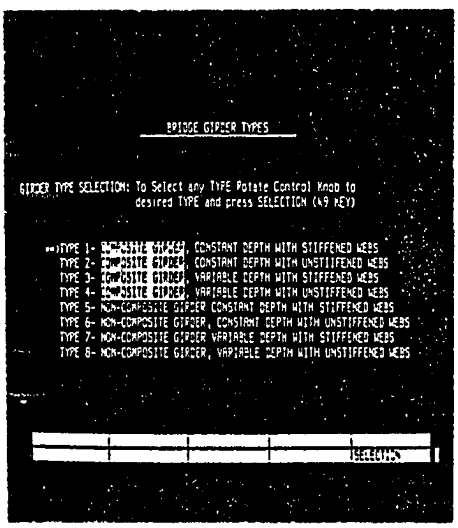
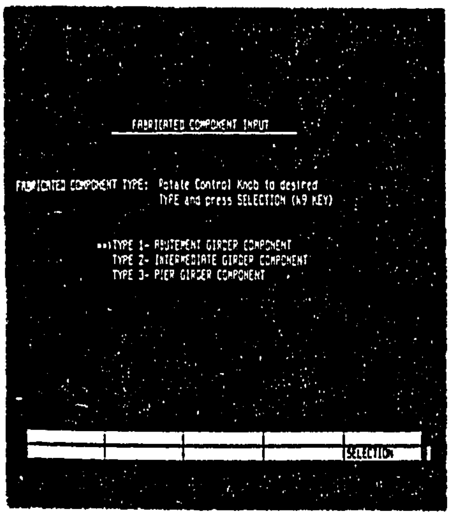


FIGURE 6.7 - Bridge Girder Selection

each new component is input, a number is programmatically assigned as shown in Figure 6.8. This is the number utilized in the SPAN matrix. When a fabricated component is repeated, only the previously assigned component number is required as input.

To serve as a brief explanation of how the fabricated component input process works, consider the input data for the abutment girder. The pointer is set to the currently requested data and awaits user input. Once the input is received, this data is displayed and the screen pointer is moved to the next requested data. This process is repeated until all required data is entered. The program then allows the user to verify that all input data is correct or to modify any particular item. This generalized input process is continued until a second abutment girder is specified.



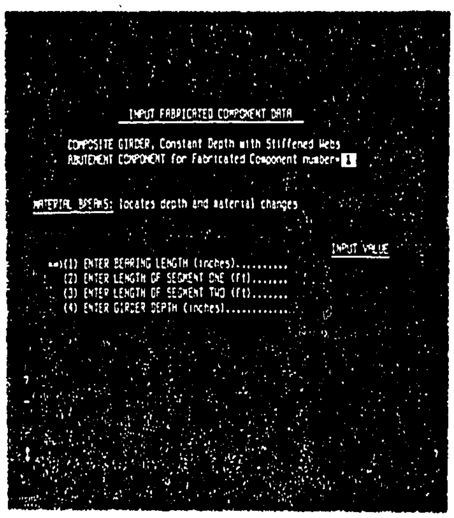


FIGURE 6.8 - Assignment of Component Number

Upon completion of this girder input, the entire bridge steel supporting girder information has been defined. This generalized process is illustrated in Figure 6.9. The second way to enter bridge data is to reload existing bridge data from the secondary storage. This process is

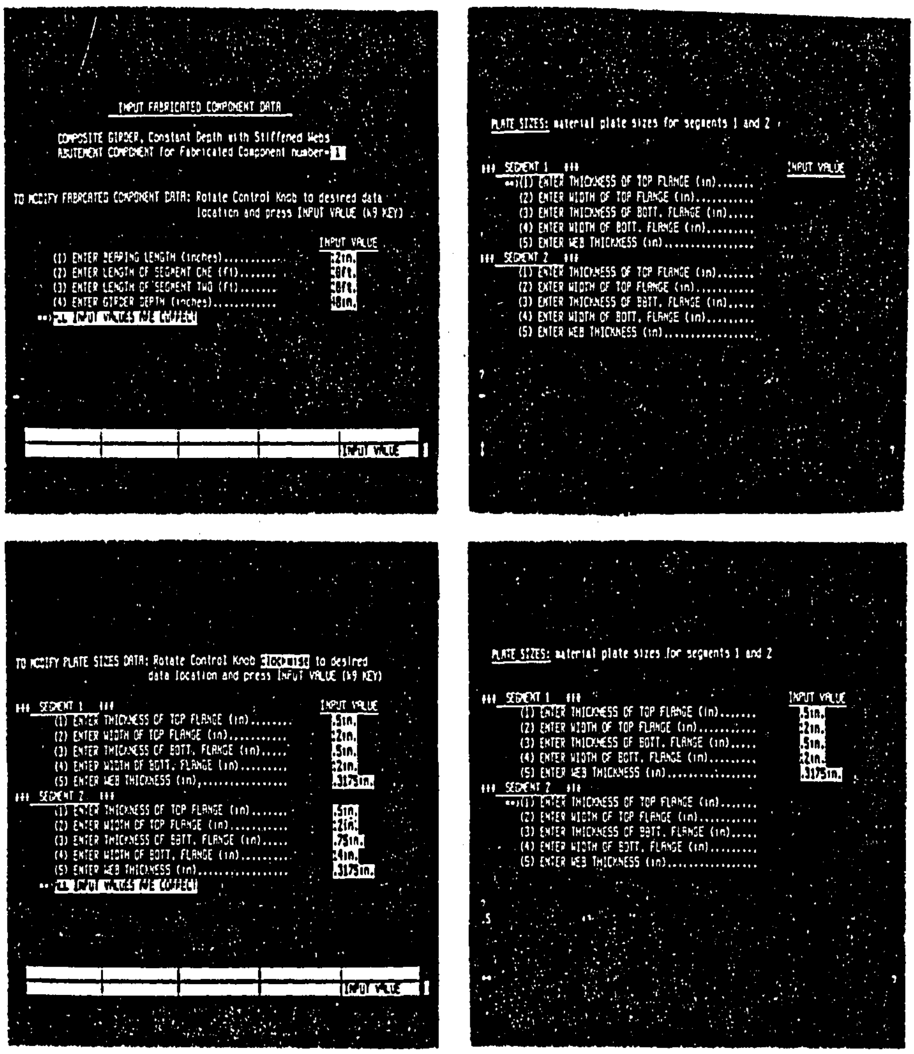


FIGURE 6.9 - Fabricated Component Input

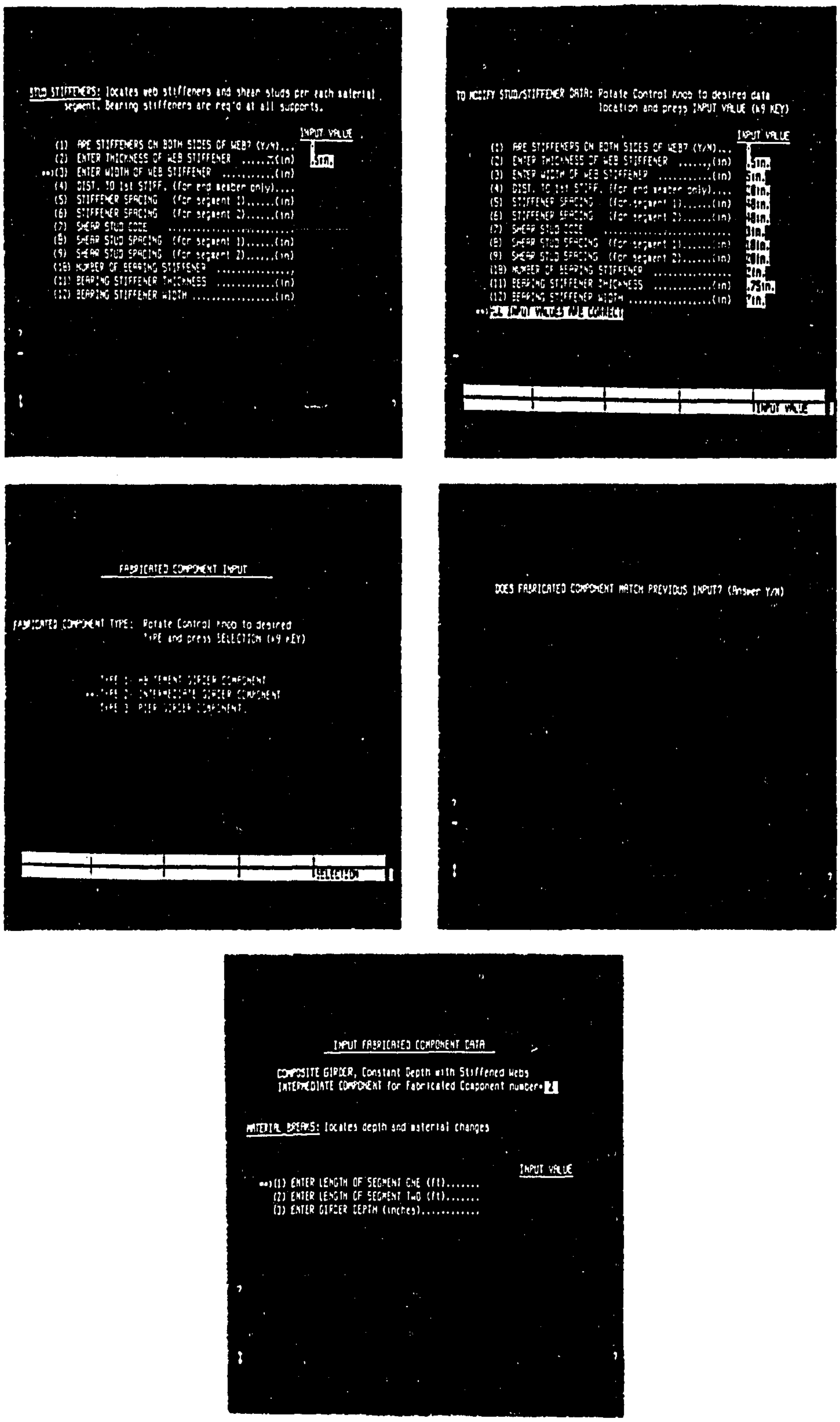
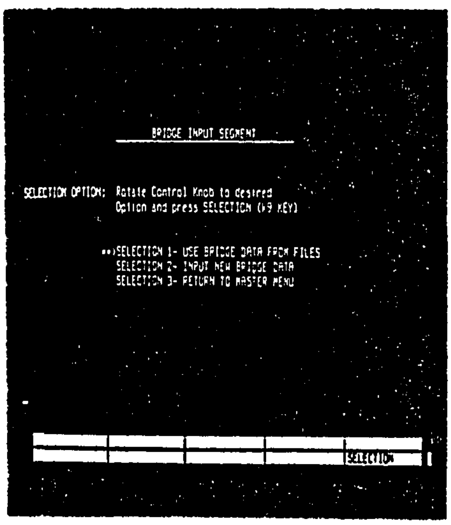


FIGURE 6.9 (con't.)

illustrated in Figure 6.10, where only a filename is required.

Whether input data is reloaded from an existing file or completely new data is input, GBRIDGE requires the user to review all data at this point before continuing. This review process is partially illustrated in Figure 6.11. Once the review is completed, the user has the option of storing this data under any filename he chooses.

At this point, if the input data has specified batch operation, the bridge analysis segment is automatically loaded and executed based upon current steel girder data. This also includes the input for the roadway depth, which is never a preset piece of information. After the analysis, the design verification process is executed and a complete girder stress report is automatically generated at the



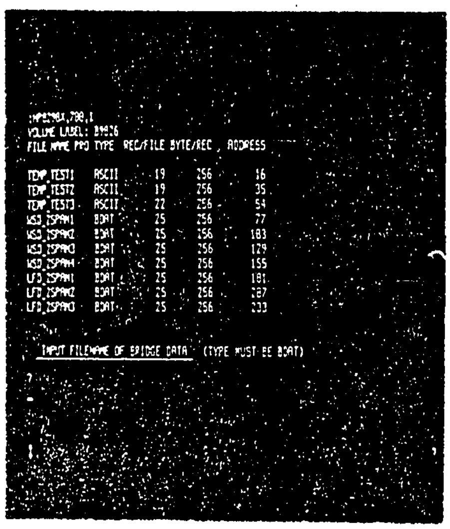


FIGURE 6.10 - Bridge Data Input from Files

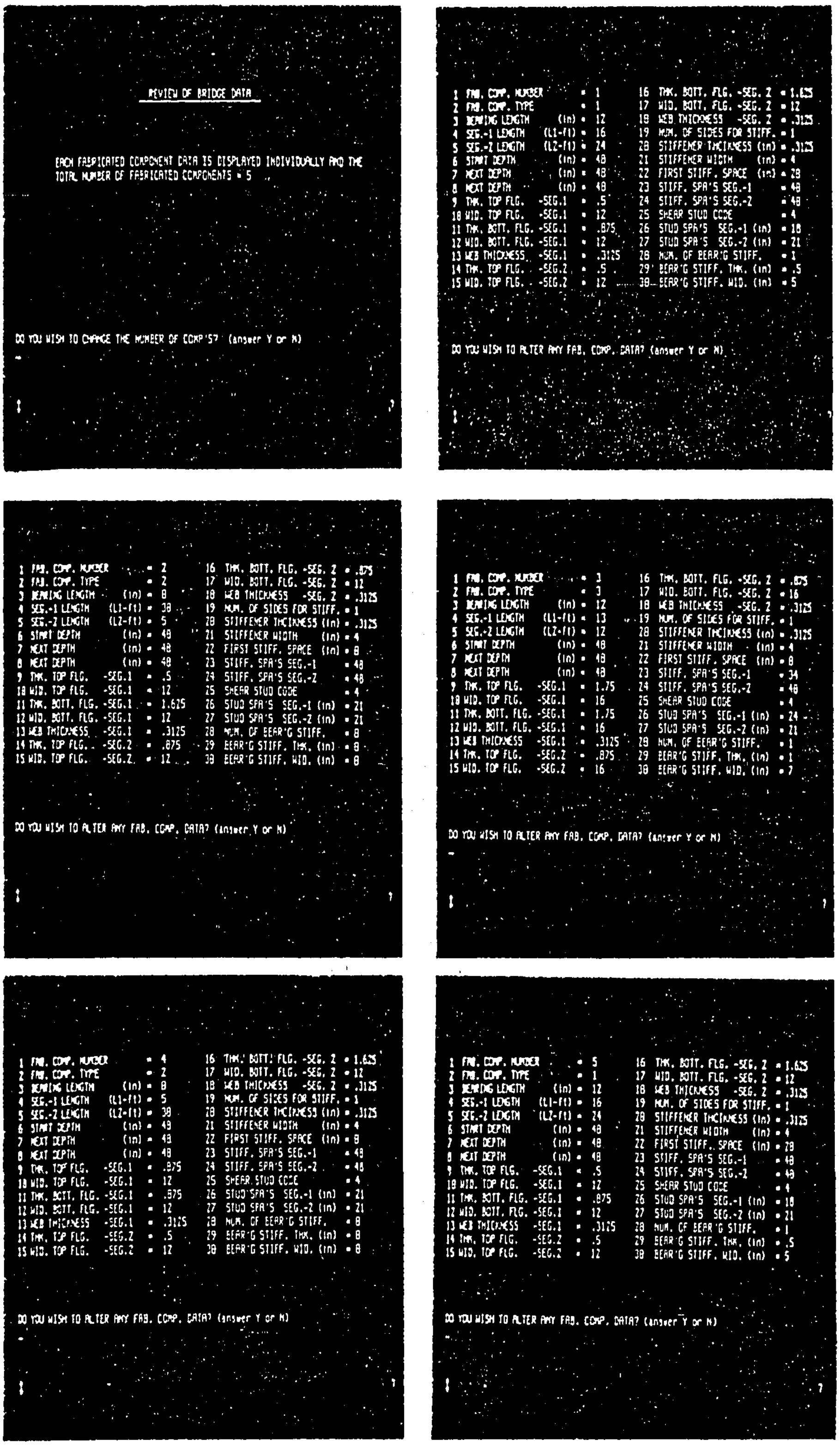
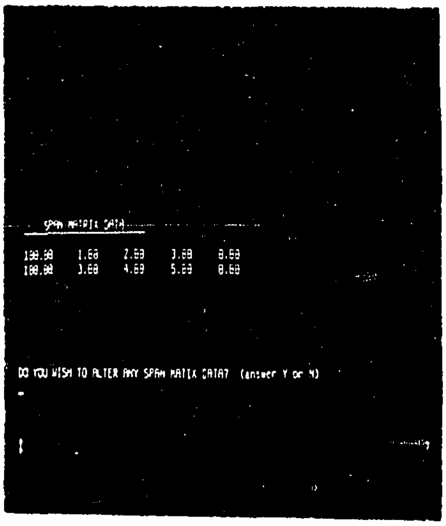
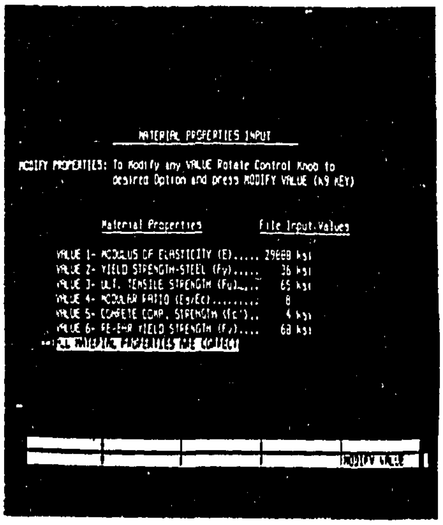
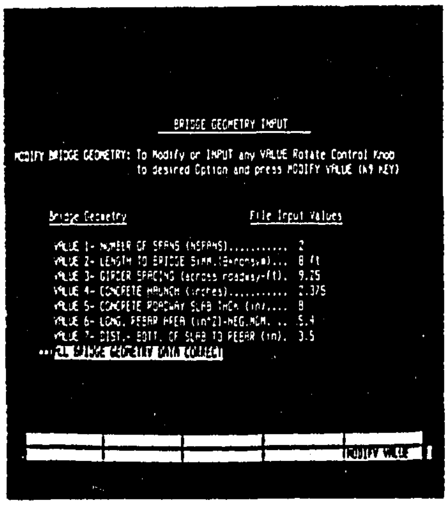
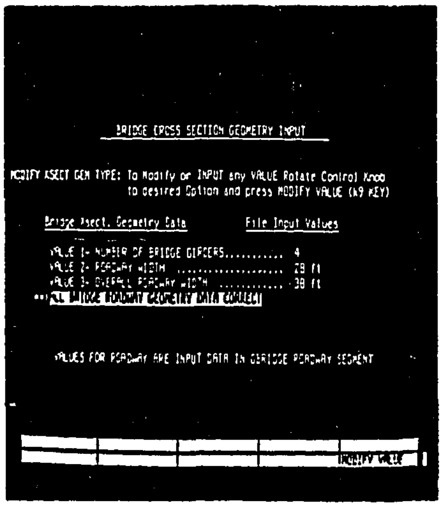


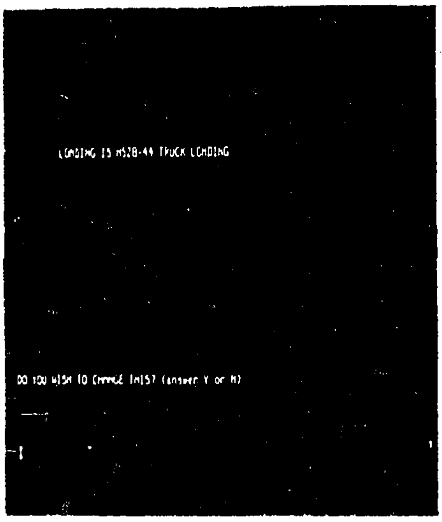
FIGURE 6.11 - Review of Bridge Data











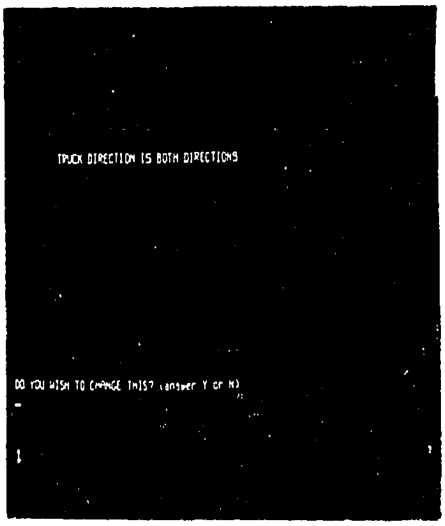
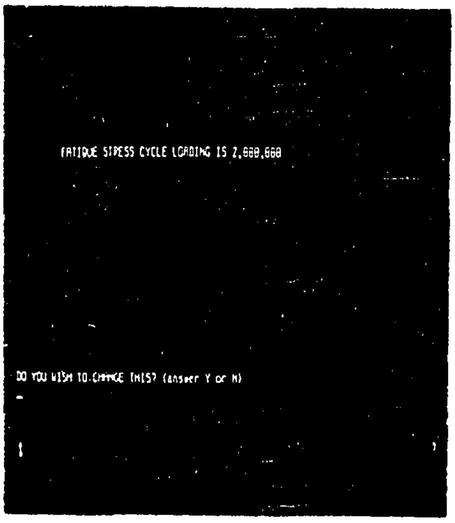


FIGURE 6.11 (con't.)



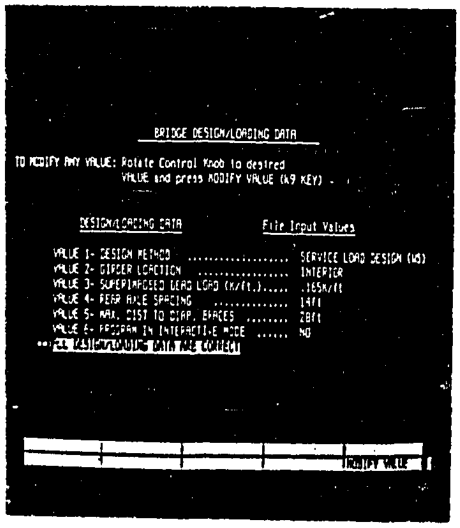


FIGURE 6.11 (con't.)

printer. However, if the input data have specified interactive operation, the control is returned to the master menu and the user then must guide the various CASE-GBRIDGE operations.

If the user prompts the master menu to execute the roadway module, the roadway menu is displayed, as shown in Figure 6.12, and awaits the user's selection as to which operation will be performed. The module is constructed to either analyze a specific roadway system or to perform a simultaneous analysis/synthesis evaluation. Default values are displayed for general roadway information and unit costs, as shown in Figure 6.13a. The user must verify or change this data. For the analysis condition, the bar sizes and spacing are required for the given slab depth. In the synthesis process, these variables are programmatically

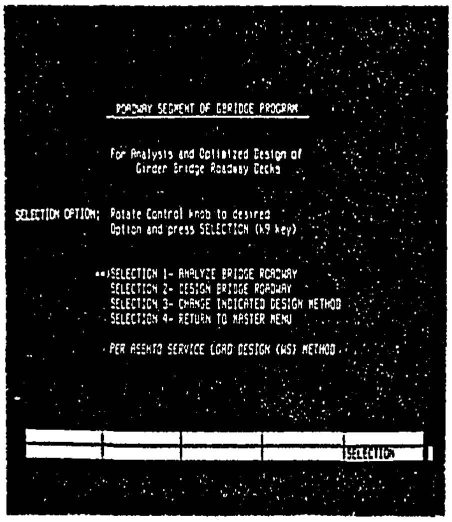
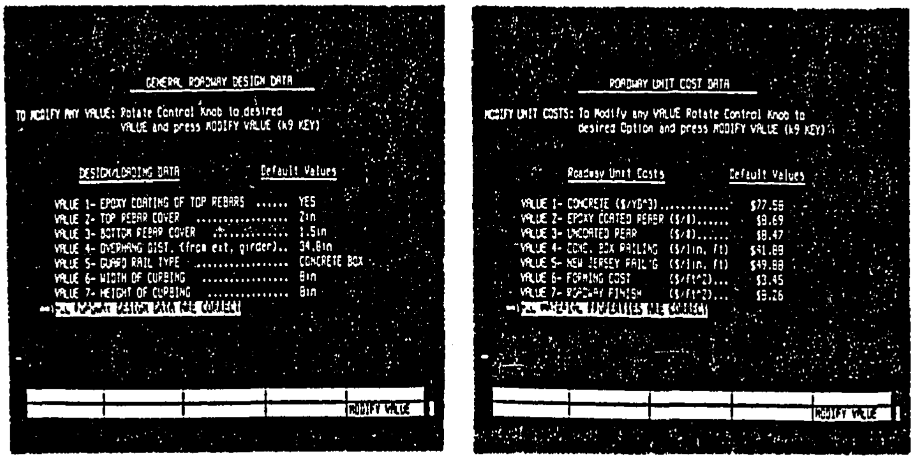


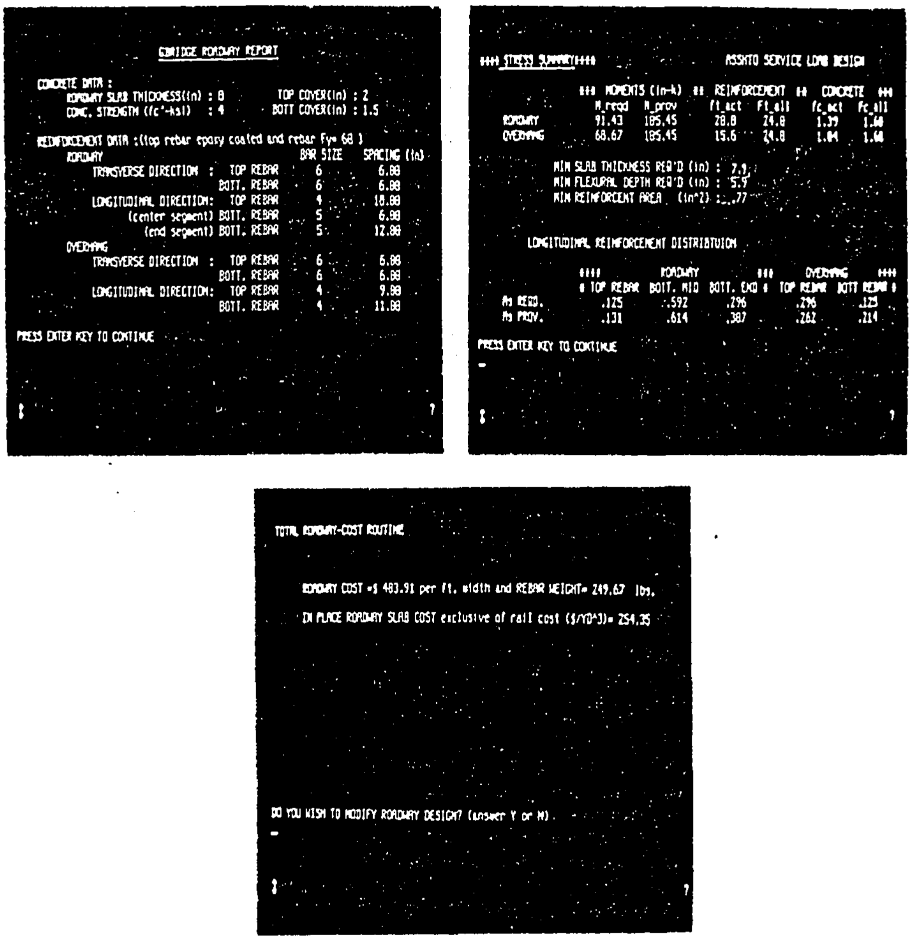
FIGURE 6.12 - Roadway Segment Menu

evaluated as described previously. In either case, a cost evaluation is considered for a one-foot width of slab. Also given is the weight of the rebar and the in-place cost of a cubic yard of concrete. Since the roadway segment operation can be executed independently, it also has an independent output format and the roadway report is generated from this module. The results of utilizing the roadway analysis and synthesis are partially illustrated in Figure 6.13b.

Executing the Bridge Graphics/Cost Segment of GBRIDGE will result in the display shown in Figure 6.14. The bridge elevation can be used to assist in visual verification of bridge geometry. This segment displays the bridge plan and cross section plus gives the superstructure cost information, based upon the current bridge configuration subject to the limitations already discussed, including



(a)



(b)

FIGURE 6.13 - Roadway Segment Operation

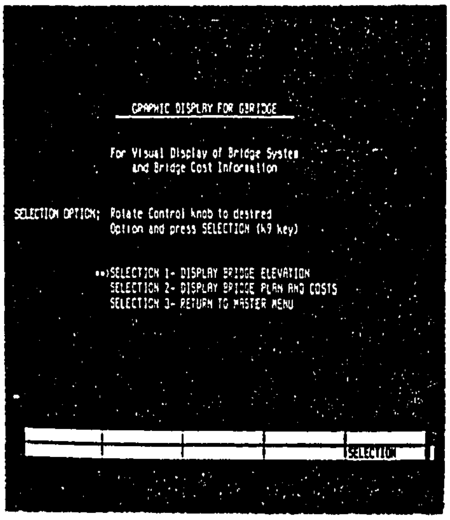


FIGURE 6.14 - Bridge Graphics/Cost Segment Menu

member weights. The roadway segment must be executed before operation of this module; otherwise, the slab cost contributions are considered to be zero. Similar to the roadway condition, this module employs its own output operation. This output reflects the overall superstructure cost and this segment operation is illustrated in Figure 6.15.

The bridge synthesis module may be executed by the user from the master menu after performing an analysis. Initial consideration is given to whether the girder depth is preselected or is to be evaluated programmatically. The default costs are displayed and the user must verify, modify, or change these costs before the synthesis operations are executed. This is shown in Figure 6.16. After the final iteration is performed, a program flag is

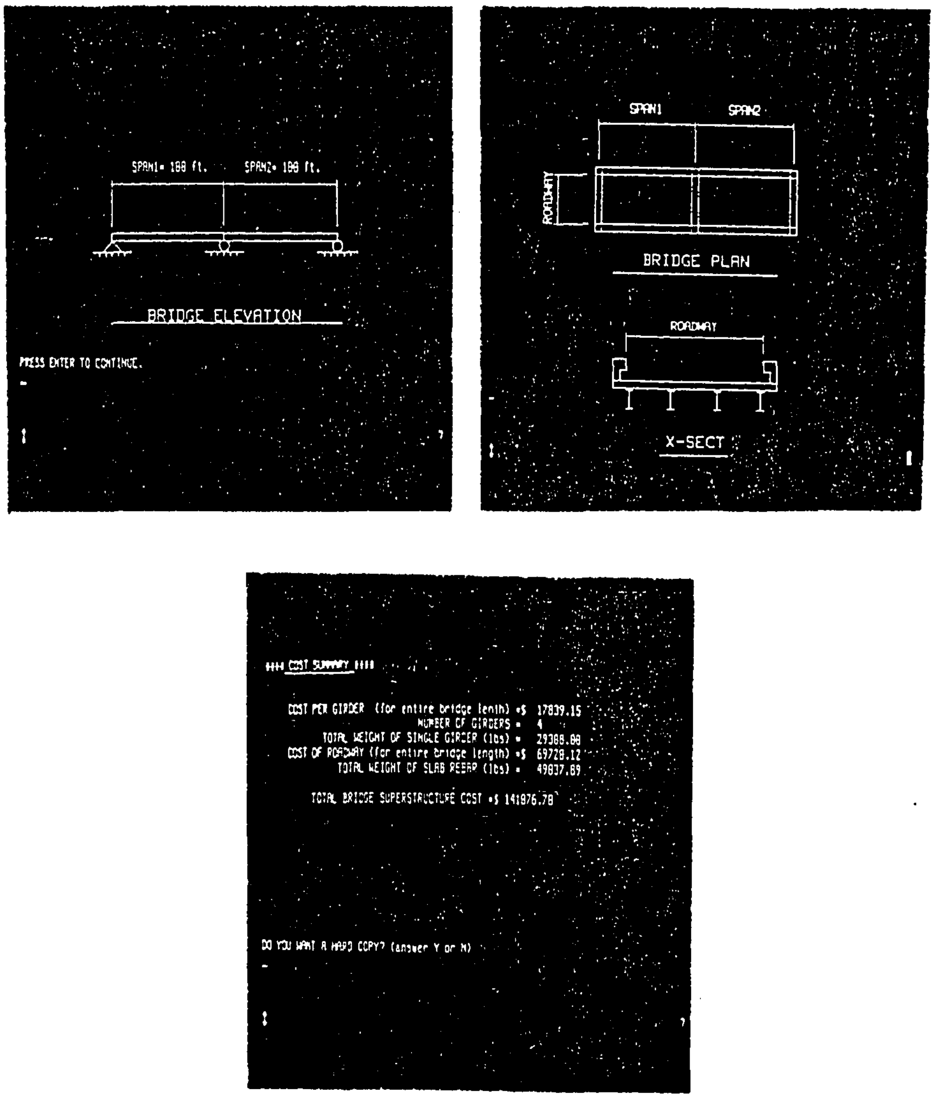


FIGURE 6.15 - Bridge Graphics/Cost Segment Output

set so that an automatic reanalysis is executed along with the design code verification. This is accomplished by resetting the interactive mode to batch processing and performing the analysis. In this manner, the user is required to examine the girder synthesis results.

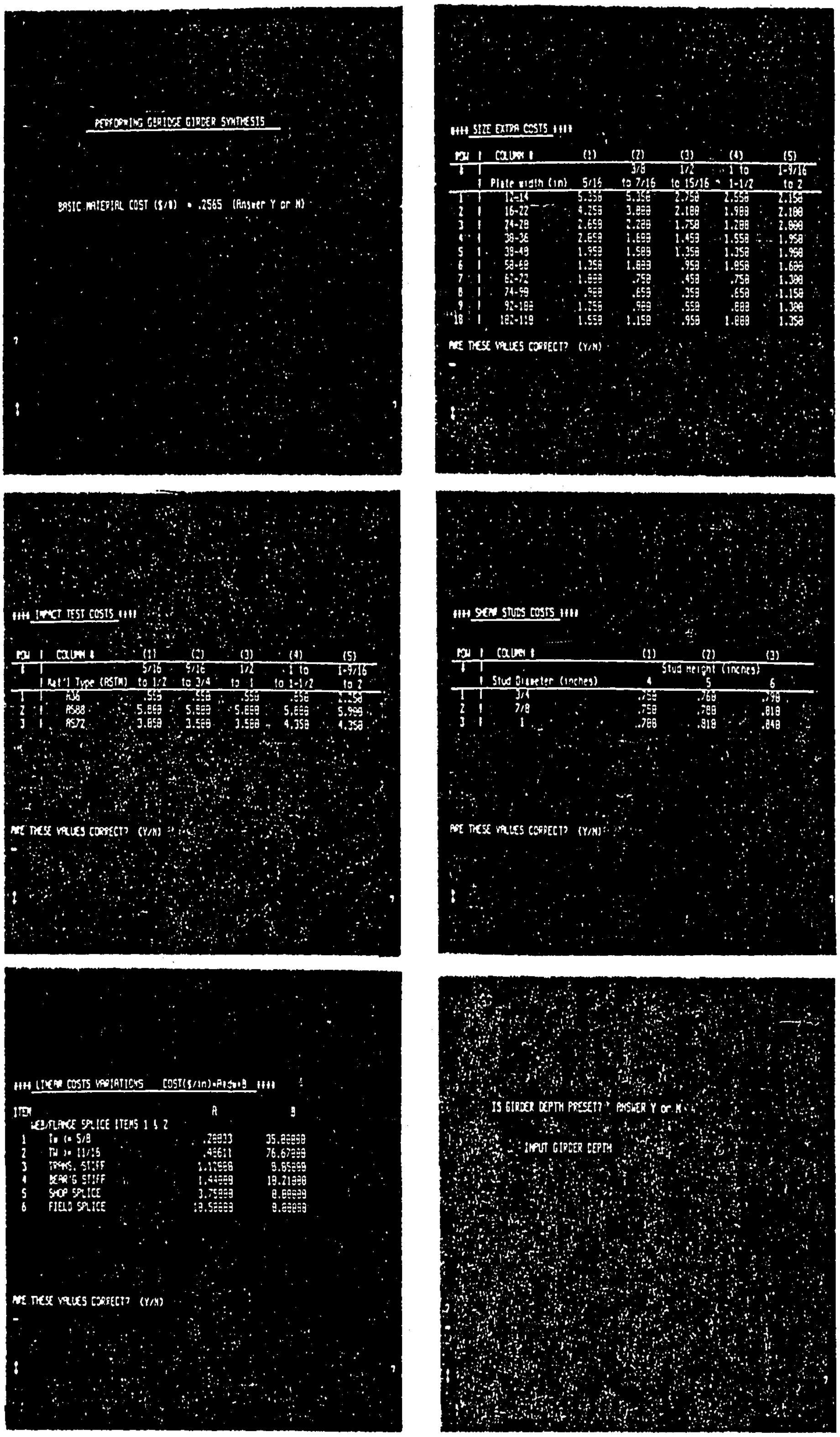


FIGURE 6.16 - Bridge Synthesis Segment

Execution of the Bridge Design Check Segment causes an examination of the current girder system for compliance with the AASHTO code, using the currently specified design method. The validation of the design is accomplished either graphically or in a tabular alphanumeric format. The graphic displays show the allowable and actual flexural stresses for both the top and bottom girder flanges, as shown in Figure 6.17. The alphanumeric results are directed

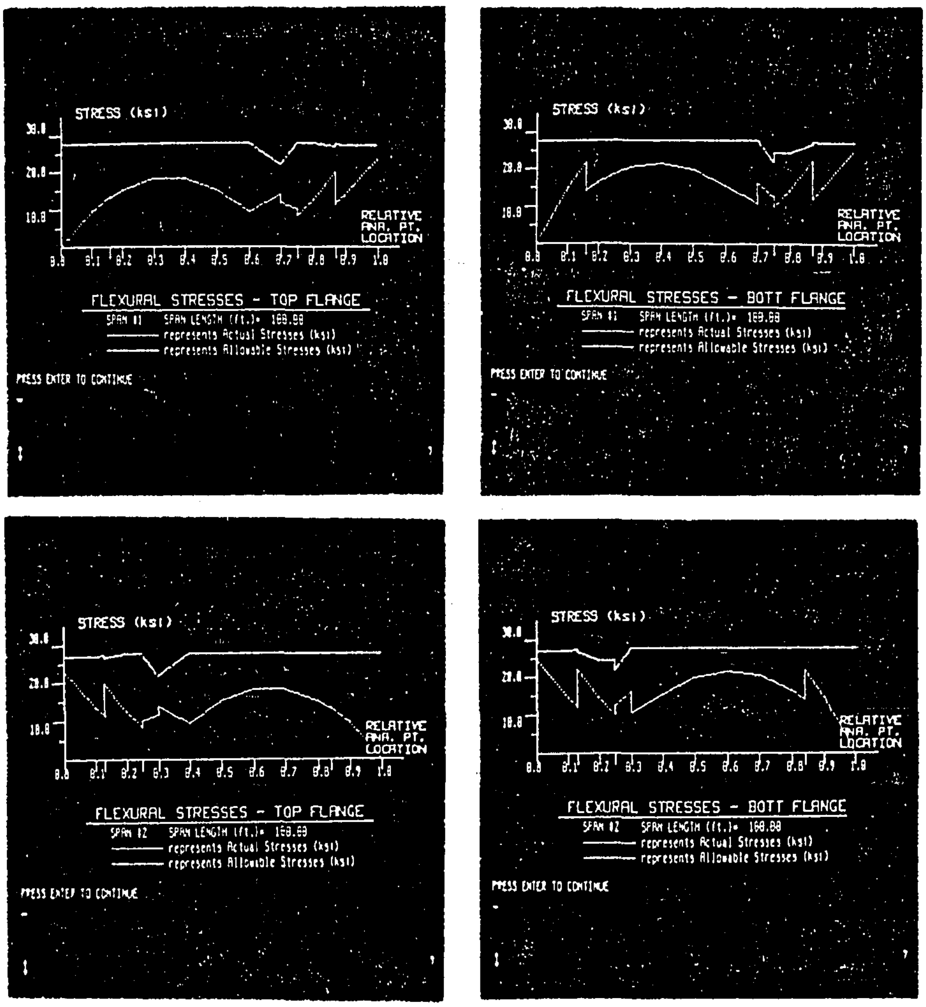
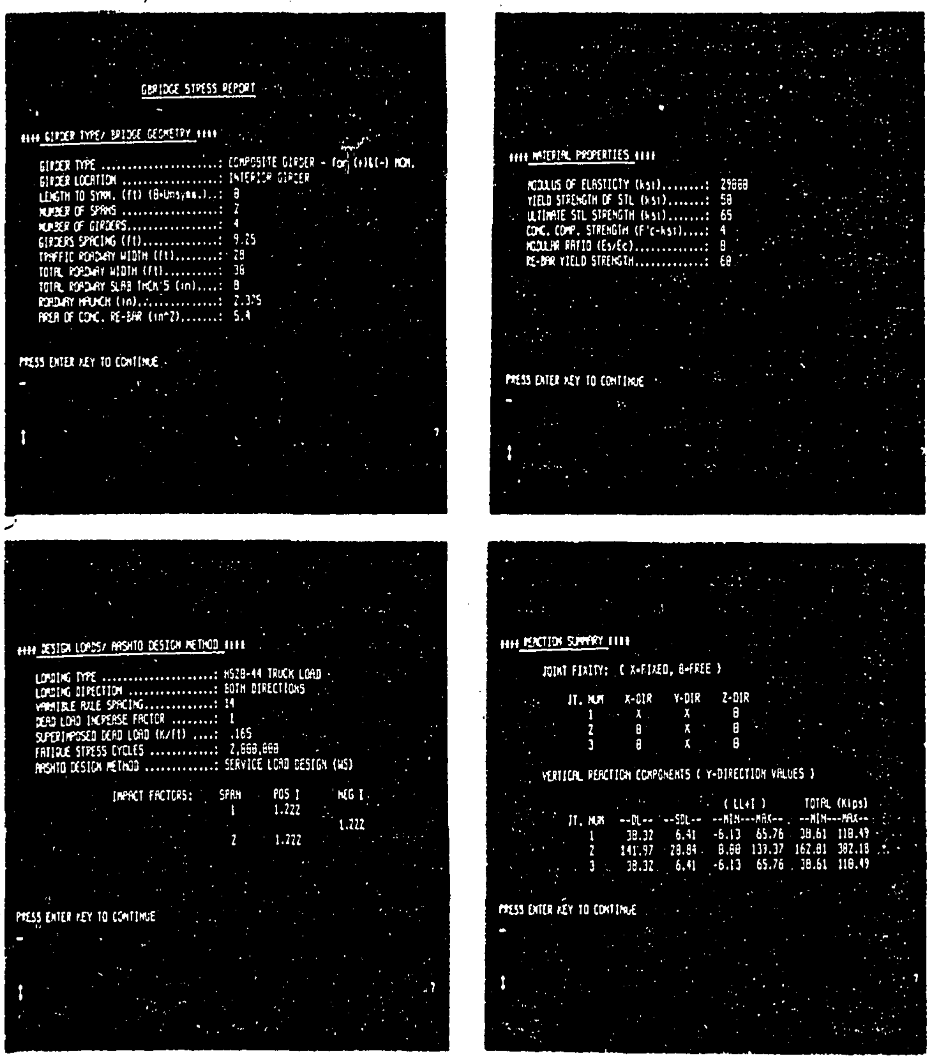


FIGURE 6.17 - Bridge Design Check Segment Graphic Output

to either the screen or printer and have two levels of output. Included are such items as bridge geometry/girder type, material properties, design loads and programmatically evaluated impact factors, joint fixity, etc. A sample output for partial evaluation is shown in Figure 6.18.



<u>FIGURE 6.18 - Bridge Design Check Segment</u>

<u>Alphanumeric Output</u>

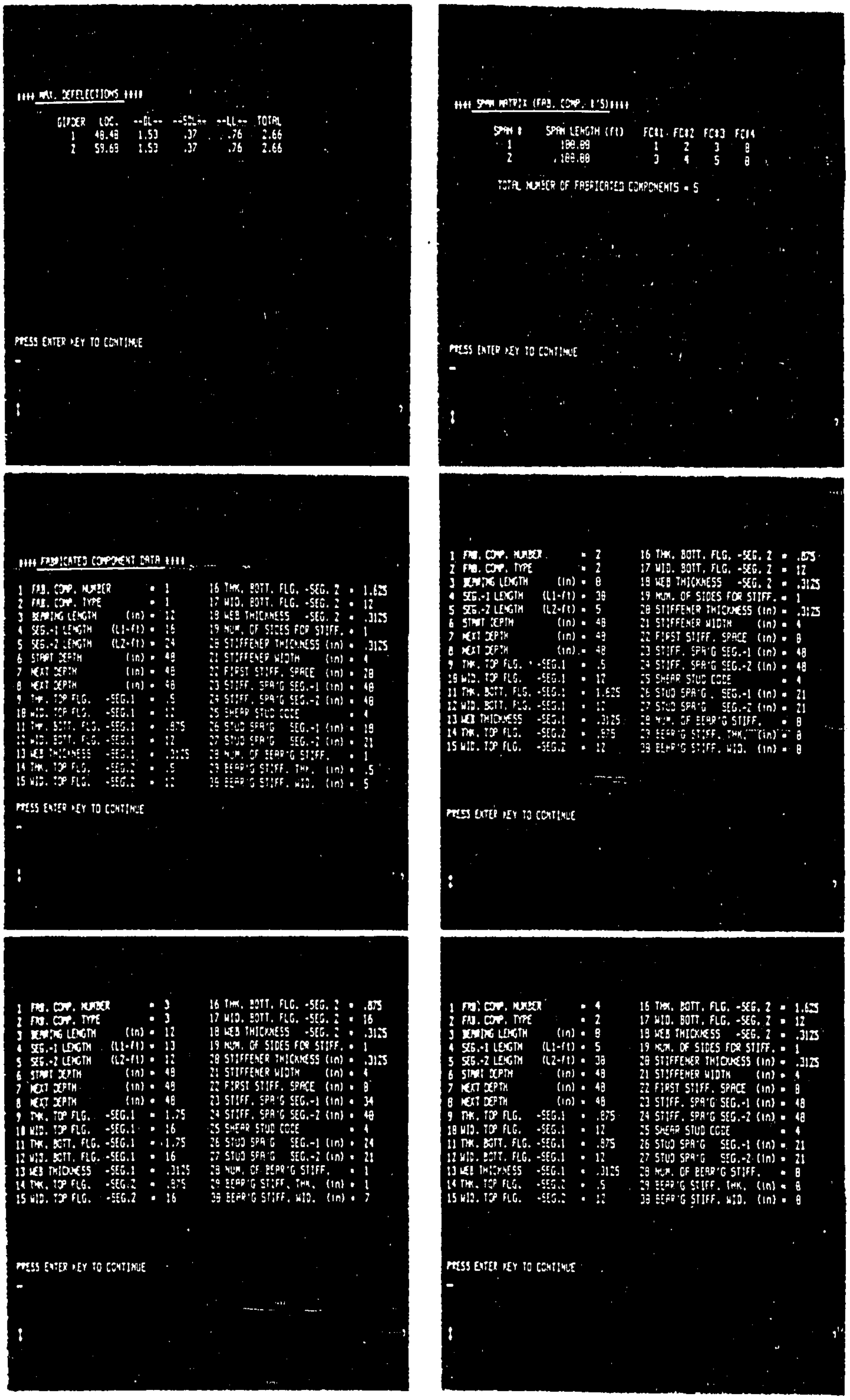
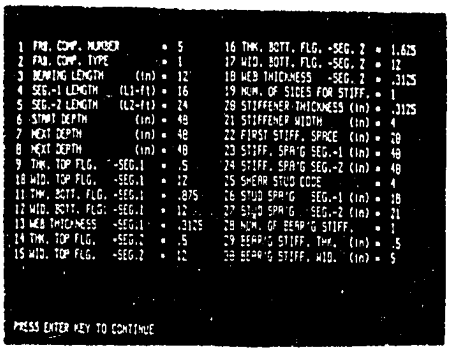
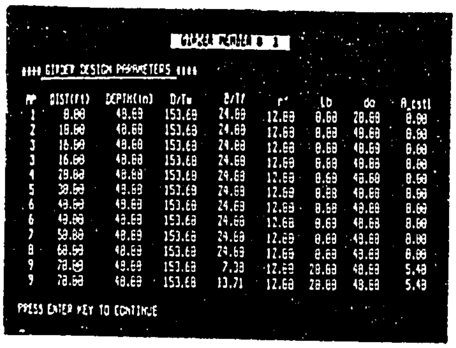


FIGURE 6.18 (con't.)



1 FAB. COMP. NUMBER	= 5	16 THK. BOTT. FLG. -SEG. 2	= 1.625
2 FAB. COMP. TYPE	= 1	17 WID. BOTT. FLG. -SEG. 2	= 12
3 BEARING LENGTH (in)	= 12	18 WEB THICKNESS -SEG. 2	= .3125
4 SEG.-1 LENGTH (L1-ft)	= 16	19 NUM. OF SIDES FOR STIFF.	= 1
5 SEG.-2 LENGTH (L2-ft)	= 24	20 STIFFENER THICKNESS (in)	= .3125
6 START DEPTH (in)	= 48	21 STIFFENER WIDTH (in)	= 4
7 NEXT DEPTH (in)	= 48	22 FIRST STIFF. SPACE (in)	= 20
8 NEXT DEPTH (in)	= 48	23 STIFF. SPA'G SEG.-1 (in)	= 48
9 THK. TOP FLG. -SEG.1	= .5	24 STIFF. SPA'G SEG.-2 (in)	= 48
10 WID. TOP FLG. -SEG.1	= 12	25 SHEAR STUD CODE	= 4
11 THK. BOTT. FLG. -SEG.1	= .875	26 STUD SPA'G SEG.-1 (in)	= 18
12 WID. BOTT. FLG. -SEG.1	= 12	27 STUD SPA'G SEG.-2 (in)	= 21
13 WEB THICKNESS -SEG.1	= .3125	28 NUM. OF BEAR'G STIFF.	= 1
14 THK. TOP FLG. -SEG.2	= .5	29 BEAR'G STIFF. THK. (in)	= .5
15 WID. TOP FLG. -SEG.2	= 12	30 BEAR'G STIFF. WID. (in)	= 5

PRESS ENTER KEY TO CONTINUE

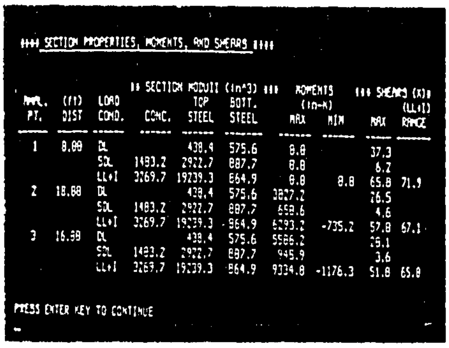


GIRDER NUMBER # 1

GIRDER DESIGN PARAMETERS

MP	DIST(ft)	DEPTH(in)	D/Tw	B/Tf	r'	Lb	do	A_cstl
1	0.00	48.00	153.60	24.00	12.00	0.00	20.00	0.00
2	10.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
3	16.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
3	16.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
4	20.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
5	30.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
6	40.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
6	40.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
7	50.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
8	60.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
9	70.00	48.00	153.60	7.30	12.00	20.00	40.00	5.40
9	70.00	48.00	153.60	13.71	12.00	20.00	40.00	5.40

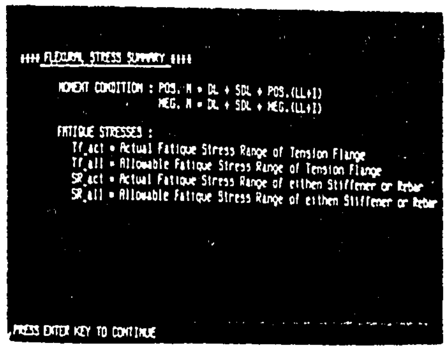
PRESS ENTER KEY TO CONTINUE



SECTION PROPERTIES, MOMENTS, AND SHEARS

ANAL. PT.	(ft) DIST	LOAD COND.	## SECTION MODULI (in^3) ### CONC.	TOP STEEL	BOTT. STEEL	MOMENTS (in-K) MAX	MIN	### SHEARS (K)# MAX	(LL+I) RANGE
1	0.00	DL		430.4	575.6	0.0		37.3	
		SDL	1403.2	2922.7	807.7	0.0		6.2	
		LL+I	3269.7	19239.3	864.9	0.0	0.0	65.0	71.9
2	10.00	DL		430.4	575.6	3027.2		26.5	
		SDL	1403.2	2922.7	807.7	659.6		4.6	
		LL+I	3269.7	19239.3	864.9	6293.2	-735.2	57.0	67.1
3	16.00	DL		430.4	575.6	5586.2		25.1	
		SDL	1403.2	2922.7	807.7	945.9		3.6	
		LL+I	3269.7	19239.3	864.9	9334.0	-1176.3	51.0	65.0

PRESS ENTER KEY TO CONTINUE

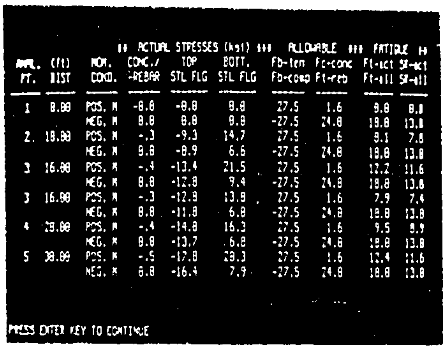


FLEXURAL STRESS SUMMARY

MOMENT CONDITION : POS. M = DL + SDL + POS.(LL+I)
NEG. M = DL + SDL + NEG.(LL+I)

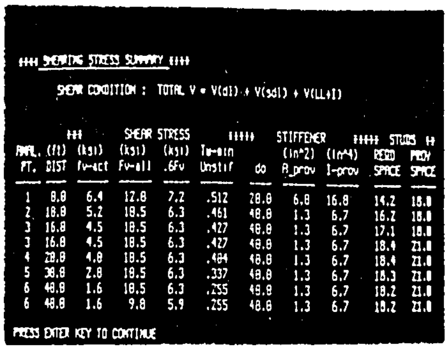
FATIGUE STRESSES :
Tf_act = Actual Fatique Stress Range of Tension Flange
Tf_all = Allowable Fatique Stress Range of Tension Flange
SR_act = Actual Fatique Stress Range of eithen Stiffener or Rebar
SR_all = Allowable Fatique Stress Range of eithen Stiffener or Rebar

PRESS ENTER KEY TO CONTINUE



ANAL. PT.	(ft) DIST	MOM. COND.	## ACTUAL STRESSES (ksi) ### CONC./ -REBAR	TOP STL FLG	BOTT. STL FLG	ALLOWABLE ### Fb-ten Fb-comp	Fc-conc Ft-reb	FATIQUE ## Ft-act Ft-all	SR-act SR-all
1	0.00	POS. M	-0.0	-0.0	0.0	27.5	1.6	0.0	0.0
		NEG. M	0.0	0.0	0.0	-27.5	24.0	18.0	13.0
2.	10.00	POS. M	-.3	-9.3	14.7	27.5	1.6	9.1	7.0
		NEG. M	0.0	-8.9	6.6	-27.5	24.0	18.0	13.0
3	16.00	POS. M	-.4	-13.4	21.5	27.5	1.6	12.2	11.6
		NEG. M	0.0	-12.0	9.4	-27.5	24.0	18.0	13.0
3	16.00	POS. M	-.3	-12.9	13.9	27.5	1.6	7.9	7.4
		NEG. M	0.0	-11.0	6.0	-27.5	24.0	18.0	13.0
4	20.00	POS. M	-.4	-14.0	16.3	27.5	1.6	9.5	8.9
		NEG. M	0.0	-13.7	6.0	-27.5	24.0	18.0	13.0
5	30.00	POS. M	-.5	-17.0	20.3	27.5	1.6	12.4	11.6
		NEG. M	0.0	-16.4	7.9	-27.5	24.0	18.0	13.0

PRESS ENTER KEY TO CONTINUE

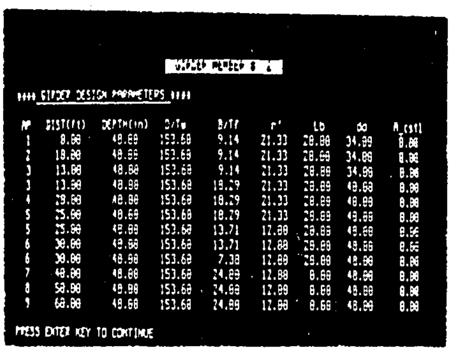


SHEARING STRESS SUMMARY

SHEAR CONDITION : TOTAL V = V(dl) + V(sdl) + V(LL+I)

ANAL. PT.	(ft) DIST	### (ksi) fv-act	SHEAR STRESS (ksi) Fv-all	(ksi) .6Fv	Tw-min Unstif	##### do	STIFFENER (in^2) A_prov	(in^4) I-prov	##### STUDS ## REQD SPACE	PROV SPACE
1	0.0	6.4	12.0	7.2	.512	20.0	6.0	16.0	14.2	18.0
2	10.0	5.2	18.5	6.3	.461	40.0	1.3	6.7	16.2	18.0
3	16.0	4.5	18.5	6.3	.427	40.0	1.3	6.7	17.1	18.0
3	16.0	4.5	18.5	6.3	.427	40.0	1.3	6.7	18.4	21.0
4	20.0	4.0	18.5	6.3	.404	40.0	1.3	6.7	18.4	21.0
5	30.0	2.0	18.5	6.3	.337	40.0	1.3	6.7	18.3	21.0
6	40.0	1.6	18.5	6.3	.255	40.0	1.3	6.7	18.2	21.0
6	40.0	1.6	9.8	5.9	.255	40.0	1.3	6.7	18.2	21.0

PRESS ENTER KEY TO CONTINUE



GIRDER NUMBER # 2

GIRDER DESIGN PARAMETERS

MP	DIST(ft)	DEPTH(in)	D/Tw	B/Tf	r'	Lb	do	A_cstl
1	0.00	48.00	153.60	9.14	21.33	20.00	34.00	0.00
2	10.00	48.00	153.60	9.14	21.33	20.00	34.00	0.00
3	13.00	48.00	153.60	9.14	21.33	20.00	34.00	0.00
3	13.00	48.00	153.60	18.29	21.33	20.00	40.00	0.00
4	20.00	48.00	153.60	18.29	21.33	20.00	40.00	0.00
5	25.00	48.00	153.60	18.29	21.33	20.00	40.00	0.00
5	25.00	48.00	153.60	13.71	12.00	20.00	40.00	0.56
6	30.00	48.00	153.60	13.71	12.00	20.00	40.00	0.65
6	30.00	48.00	153.60	7.30	12.00	20.00	40.00	0.00
7	40.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
8	50.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00
9	60.00	48.00	153.60	24.00	12.00	0.00	40.00	0.00

PRESS ENTER KEY TO CONTINUE

FIGURE 6.18 (con't.)

As stated previously, two complete finalized bridge output reports for the two-span continuous bridge described in Section 6.4 are presented in Appendix D.

VII. RESULTS, CONCLUSIONS, AND RECOMMENDATIONS

7.1 General

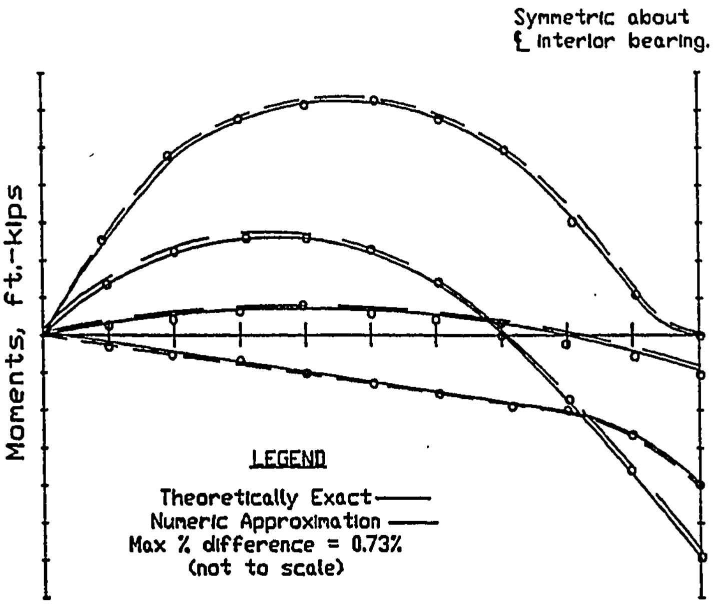
The development and application of the CASE methodology to girder bridges has now been clearly demonstrated. Also, a comprehensive examination of the various aspects required in the implementation of CASE-GBRIDGE has been presented. Within this chapter, the results of the application of CASE-GBRIDGE will be examined along with conclusions therefrom. Also, recommendations for future extension of and enhancements to the CASE methodology are given.

7.2 CASE-GBRIDGE Application Results

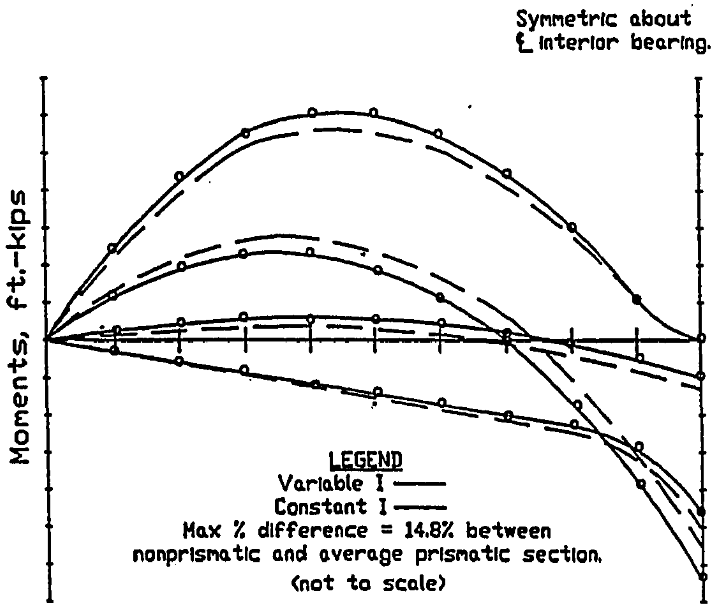
Development and application of CASE-GBRIDGE has demonstrated that a "rational and systematic" methodology can be successfully applied to civil engineering structures, in this particular instance, girder bridges. The essential features required in a structural design methodology as described in Section 1.5 have all been incorporated in the development of CASE and have been described in detail within this dissertation. The formulation of the CASE system, based upon the unique concept of fabricated components, has illustrated via ex-facto evidence that the methodology is immediately applicable to a manufacturing environment capable of fabricating civil engineering type structures.

A comprehensive examination of an analytical bridge evaluation process utilizing nonprismatic member stiffnesses has been presented. The reliability and accuracy of the nonprismatic element has been clearly demonstrated in Chapter 4. The overall accuracy of the GBRIDGE analysis procedure is illustrated in Figure 7.1, where moment diagrams for a composite, two-span girder bridge are presented. These moment diagrams represent a comparison of theoretically exact analysis procedures versus GBRIDGE analysis procedures plus a comparison of nonprismatic versus prismatic member analysis. Examination of the GBRIDGE analysis procedure reveals that a high degree of accuracy is obtained. The overall utility of CASE-GBRIDGE is demonstrated through application of the structural design process to various bridge configurations.

The structural design process, consisting of analysis/ synthesis, has been examined for both stiffened web and unstiffened web composite girder bridges. The AASHTO requirements for both the Working Stress Design method and the Load Factor Design method have been included. To insure uniformity in the application of the CASE methodology in investigating the various bridge arrangements, certain



(a) Theoretically Exact vs. GBRIDGE
Nonprismatic Members



(b) Nonprismatic vs. Prismatic Member

FIGURE 7.1 - Span Moment Variation

bridge parameters were preselected and held constant throughout the examination of each individual bridge.

The preselected values relate to roadway geometry, material properties, and span lengths. Design roadway widths for two-lane highway bridges are required by local and state highway specification to vary between 24 feet and 28 feet and, for this study, the design roadway width was held at a constant 28 feet for all bridges. The overall bridge width has been preset to 30 feet to allow for the guardrails, which have been preselected as concrete-box types. Since application of the AASHTO roadway formulas require that the roadway slab be continuous over three or more spans, consideration is given to investigating bridges with only four or five girders. The concrete roadway haunch and longitudinal area of the reinforcing steel have been preselected as 1-1/2 inches and 5.2 square inches, respectively. These preselected bridge cross-sectional geometric parameters are shown in Figure 7.2. Furthermore, all the material properties and unit costs are considered constant during the bridge system investigation. The material properties used are:

F_y = yield strength of girder steel = 50 ksi

E_s = modulus of elasticity of steel = 29,000 ksi

F_u = ultimate strength of girder steel = 65 days

f_c'= 28-day concrete compressive strength = 4 ksi

N = modular ratio (E_s/E_c) = 8

f_y = rebar yield strength = 60 ksi

The unit costs employed are CASE/GBRIDGE default values that have been derived and detailed in Chapter 5.

The bridge loading considered was HS20-44 truck loading with a 14 foot rear axle spacing. The loading was examined for truck movement in either direction. No increase in the programmatically evaluated dead load was used, but a 20 psf future wearing surface load was assumed. The bridges were all considered as part of the interstate highway system. For this system, a 2,000,000 stress cycles condition was employed in reviewing fatigue stresses. The one-, two-, and three-span bridges examined consisted of span lengths shown

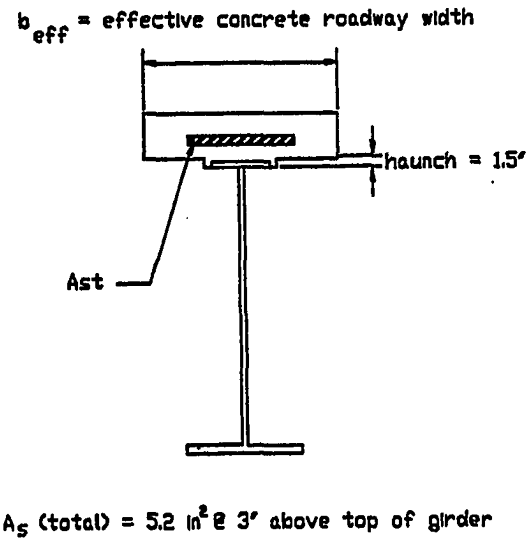


FIGURE 7.2 - Preassigned Girder Cross-Section Parameters

in Table 7.1. The construction sites were considered accessible to truck traffic. Therefore, the maximum fabricated component length was limited to 44 feet to accommodate highway shipping constraints. This shipping constraint is a reflection of the length limit imposed in interstate transport without requiring a special highway

TABLE 7.1 -- Bridge Span Lengths

One-Span	Two-Span	Three-Span
60	80-80	80-120-80
80	100-100	100-140-100
100	120-120	120-160-120

routing permit. The maximum length for any individual plate length is controlled by the manufacturers' plate shear capability. For this study, it was assumed that the maximum plate length was 24 feet. The general arrangement for the fabricated components for the 100-foot single-span bridge, the 100 foot-100 foot two-span bridge, and the 100 foot-140 foot-100 foot three-span bridge are shown in Figure 7.3. The final hardcopy output for the two-span (100'-100') bridge system indicated in Figure 7.3 for both the WSD and LFD approaches are included in Appendix D.

AASHTO specification design requirements for the concrete roadway slab allows the formulation of the analysis and synthesis processes into a single integral problem

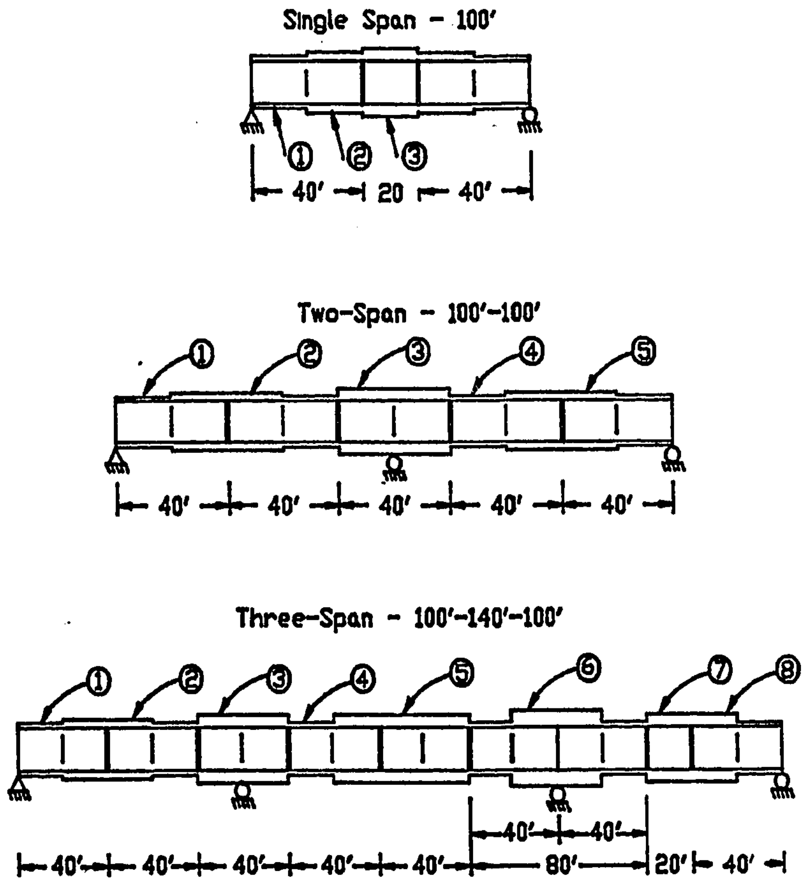


FIGURE 7.3 - One, Two, and Three Span Bridge Arrangements

statement. In other words, employing the two design variables of effective concrete depth ($d = x_1$) and area of steel reinforcement ($A_s = x_2$), the complete analysis and synthesis can be described and performed simultaneously in terms of d and A_s only. All other reinforcement requirements can be related to these values as previously detailed in Chapters 5 and 6. The accuracy of the roadway slab analysis/synthesis can be examined graphically since only two design variables are employed. Allowing the effective roadway depth d to vary between its constraint limits as previously defined, the reinforcing area variable (A_s) can be evaluated directly. The cost then can be readily calculated and plotted as a function of depth. The graphical plot of the optimum roadway slab is shown in Figure 7.4. The cost for variably-spaced girders evaluated from GBRIDGE is presented in Table 7.2, in which the graphical solutions obtained by plotting as in Figure 7.4 coincide with the results obtained from GBRIDGE.

The supporting steel girders, unlike the roadway slab design, cannot combine the analysis and synthesis into a single operation. The reason for this, as previously

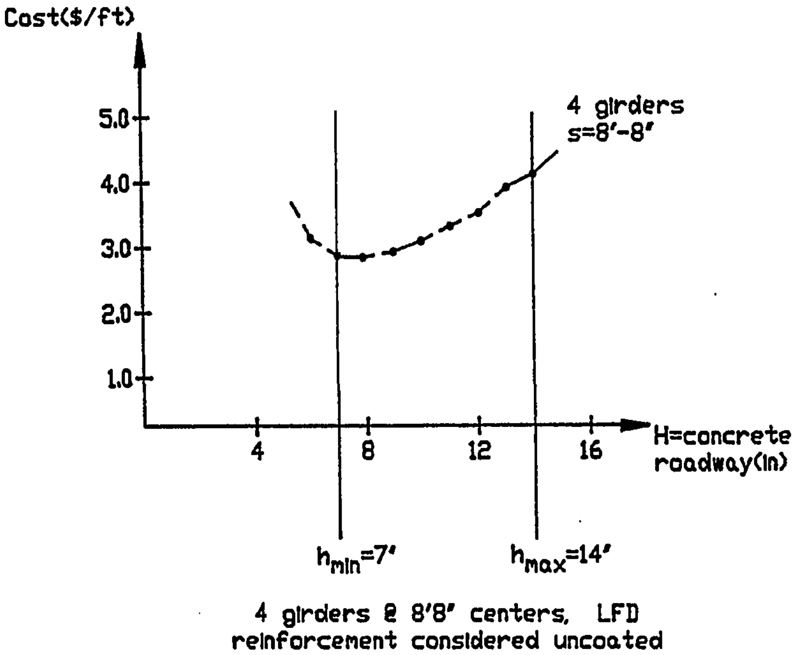


FIGURE 7.4 - Optimal Roadway Slab (1 ft. width)

TABLE 7.2 - Roadway Cost per Girder Spacing

Girder Spacing	Cost ($/ft)	Total Cost ($/ft)
6'8"	$2.54	$18.72
7'4"	2.65	18.83
8'0"	2.75	18.93
8'8"	2.84	19.02
9'4"	2.92	19.10
10'0"	2.99	19.17

reinforcement considered uncoated

discussed in detail, is that the analysis obtains unknown displacements which are dependent upon the overall structural member, but the synthesis employs design variables that represent individual plate elements. These design variables are interlocked with the analysis through the section properties of the overall structural member, but cannot be combined into a single integral formulation.

Thus, an iterative process between girder analysis and synthesis is required to obtain the individual plate components. This iterative procedure has been described fully in Chapter 6. The critical step is the evaluation of the overall girder depth. The accuracy of the weight average approach employed by CASE-GBRIDGE is illustrated in Figure 7.5. The figure shows the cost versus depth relationship of the supporting steel girders and demonstrates that the weighted average approach is both accurate and reliable. The curve is generated from

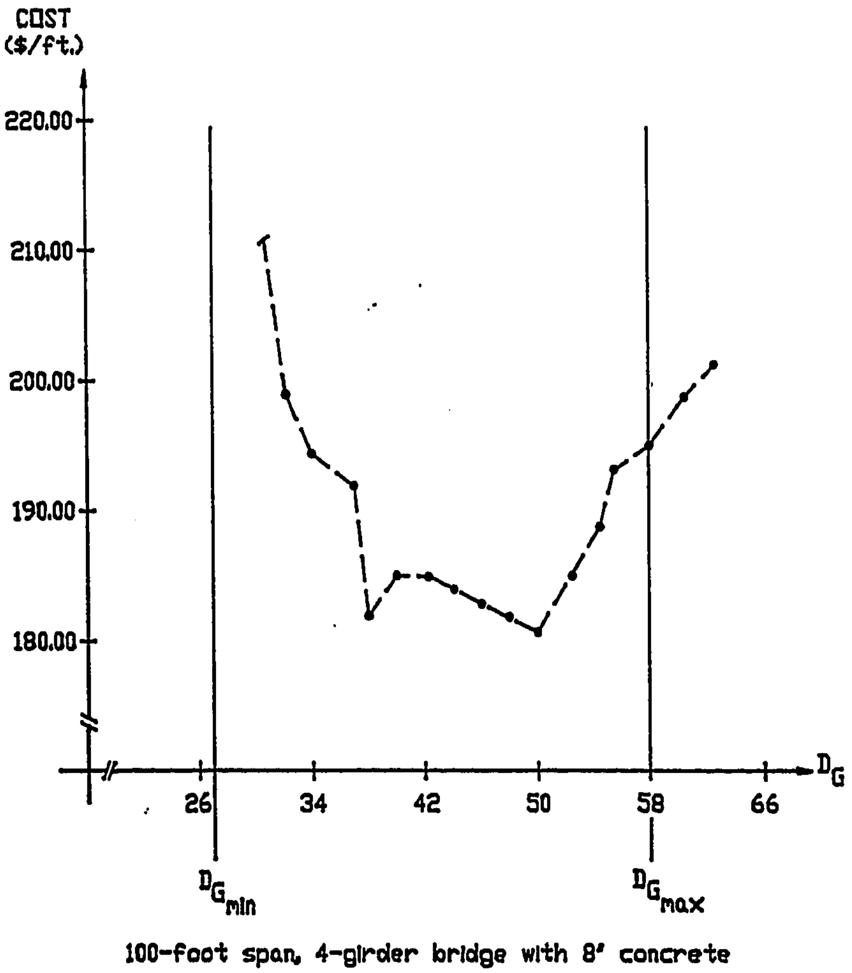


FIGURE 7.5 - Optimal Girder Depth

incrementing preselected girder depths and programmatically evaluating the remaining plate design variables. Since the plates are discrete, the cost objective function is not continuous. This is reflected in the cost versus depth plot.

7.2.1 Cost Analysis Results

The cost evaluation of the specified girder bridges is given in Tables 7.3, 7.4, and 7.5 for the stiffened web one-, two-, and three-span bridge systems, respectively. These tables, however, require explanation so as to understand their true representation. The bridge roadway cost used in the synthesis process, including forming and finish costs, reflect the final in-place roadway cost. These values then reflect all field labor and shipping cost to complete the roadway, including guardrailing at the bridge job site. On the other hand, CASE-GBRIDGE considerations focused on basic direct manufacturing cost in the girder synthesis. Costs relating to shipping, field erection, and diaphragms were not considered. Also, manufacturing costs that are relatively constant, such as painting, were excluded. These additional field costs can be estimated as approximately 0.50 to 1.00 times the direct manufacturing costs. Thus, for comparison, the programmatically evaluated girder cost, which illustrates the variation between WSD and LFD, is presented, but the total superstructure cost is obtained as roadway cost plus 1.75 times the girder cost. The total cost column is then to reflect the in-place superstructure cost, including the field labor costs.

The total cost column in Tables 7.3, 7.4, and 7.5 reflects the completed superstructure cost. Of course, it

TABLE 7.3 - Cost Evaluation for a Single-Span, Four-Girder, Stiffened-Web Bridge

SPAN	Depth				Cost			
	1/25	1/30	h^*_{opt}	Road*	WSD		LFD	
					Gird	Tot	Gird	Tot
60'	28.8	23.5	48.0	20916	13082	43810	12483	42761
80'	38.4	32.0	54.0	27888	24314	70438	22790	67771
100'	48.0	40.0	58.0	34860	40822	106299	36738	99152

TABLE 7.4 - Cost Evaluation for a Two-Span, Four-Girder, Stiffened-Web Bridge

SPAN	1**	Depth				Cost			
		1/25	1/30	h^*_{opt}	Road*	WSD		LFD	
						Gird	Tot	Gird	Tot
80'-80'	58.73	28.2	23.5	48.0	55776	51489	145882	45998	136273
100'-100'	68.58	32.9	27.4	50.0	69720	72333	196303	63017	180000
120'-120'	86.97	41.7	34.8	56.0	83664	89366	240055	79635	223025

*based upon WSD
**1 defined by inflection point locations

TABLE 7.5 - Cost Evaluation of a Three-Span, Four-Girder, Stiffened Web Bridge

SPAN	l^{**}	Depth				Cost			
		1/25	1/30	h^*_{opt}	Road*	WSD		LFD	
						Gird	Tot	Gird	Tot
80'-100'-80'	71.22	34.2	28.5	48.0	97608	90576	256116	78309	234649
100'-140'-100'	78.87	37.9	31.5	52.0	118524	128724	343791	109393	320162
120'-160'-120'	87.65	42.1	35.1	60.0	139440	192412	476161	161119	421398

*based upon WSD
**l defined by inflection point locations

alone does not indicate the cost of the overall bridge system. The overall bridge cost includes the bridge substructure cost plus the approach roadway cost. The substructure cost and roadway cost are generally greater than the superstructure cost. For the specified superstructure, CASE-GBRIDGE assists in the overall bridge cost examination. Thus, to determine the overall optimum bridge system, the cost contributions of the substructure and approach roadway must be considered.

Not only is the number of spans required to be specified in GBRIDGE but also whether the girder bridge has stiffened or unstiffened webs. A cost comparison between the stiffened web and unstiffened web four-girder bridge systems is given in Table 7.6. This table reflects the cost for the single span and two-span bridge systems. The table clearly shows the unstiffened web girders are always more economical for simple span bridges but, as the number and length of the spans increase, stiffened webs become more economical.

It must be noted that, in CASE-GBRIDGE, the synthesis process allows for either stiffened webs or unstiffened webs. If the user selects the girder to be unstiffened, then, only unstiffened girders will be considered in the synthesis. If the selection is for stiffened webs, only stiffened webs are examined. Thus, in this case, the user must be aware that partially stiffened girders may need to

TABLE 7.6 -- Stiffened vs. Unstiffened Web Girders
(*four-girder bridge system, WSD)

Span	Cost (per girder)*	
	Stiffened Web	Unstiffened Web
SS-60	3270	3128
SS-80	6078	5626
SS-100	10205	9072
2S-80/80	12872	15142
2S-100/100	18083	21517
2S-120/120	22341	27033

be examined. This is particularly true when the stiffener spacing specified in the synthesis process is at maximum spacing.

The cost comparisons presented thus far have considered the variable effects of certain girder design components, but all comparisons have been based upon a four-girder bridge cross-section. The reasoning for this is simply that a four-girder bridge is more economical than a five-girder system. As the number of supporting girders increase, the cost of each individual girder and the roadway cost both are reduced, but this reduction is not sufficient to offset the added cost of the extra girder, as shown in Table 7.7. This table reflects the cost for a two-span bridge system. It should be noted, however, that the optimum depth is less for five-girder bridges, and the related approach roadway cost would be affected accordingly. This contribution, along with the additional bearing cost, must be examined outside of CASE-GBRIDGE.

TABLE 7.7 - Comparison of Four and Five Girder Bridges

2-Span Bridges	4-Girder Bridges			5-Girder Bridges		
	d_{opt}	Cost ($) per girder	Total Cost	d_{opt}	Cost ($) per Girder	Total Cost
80'-80'	48.0	12872	51489	42.0	12110	60552
100'-100'	50.0	18083	72333	46.0	16694	83470
120'-120'	56.0	22341	89366	50.0	19199	95997

Girders are stiffened web; WSD
Total Cost reflects girder costs only

One of the reasons why the bridges with the least number of girders are more cost effective is due to the fact that the total concrete roadway cost is relatively insensitive to girder spacing. Actually, if slight overstressing is allowed, the roadway slab for the girder bridges considered could be the same, instead of the increased slab depth that is rigorously required. Employing the default cost value for the 30 foot roadway system, the in-place roadway cost per lineal foot are $348.60 and $331.56 for the four- and five-girder bridge system, respectively. Referring to Figure 7.6, the relative insensitivity of slab cost to girder spacing is illustrated

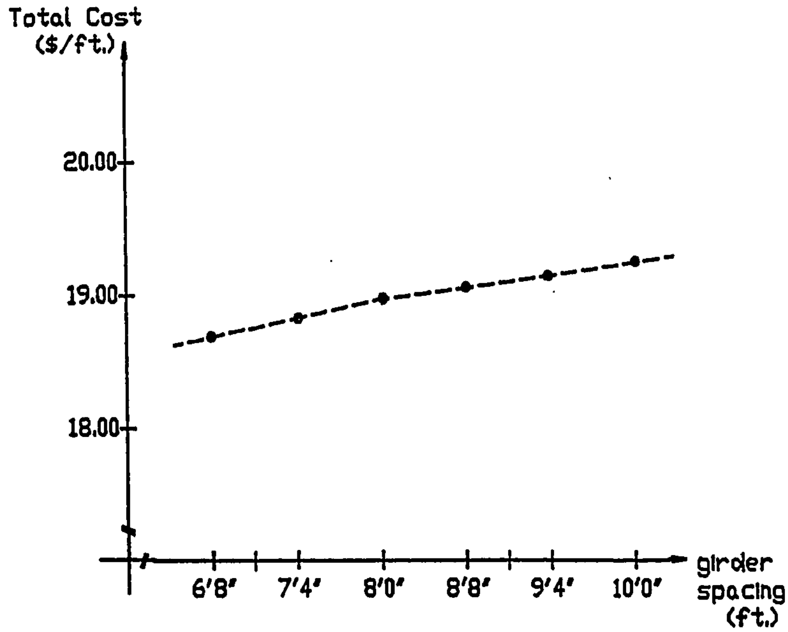


FIGURE 7.6 - Total Roadway Cost per Girder Spacing Variation

using a predefined reinforcing ratio of one-half of the maximum.

7.3 Conclusions

Several conclusions and observations can be drawn from investigation of the development of the CASE-GBRIDGE methodology. They are:

1. CASE Methodology

   (a) The concept of fabricated components, which represents the physical part exactly as it is to be fabricated, can effectively and efficiently be utilized in a design methodology applicable to civil engineering type structures. Usage of fabricated components results in radical modification of traditional preprocessing and postprocessing of general purpose structural engineering programs, but allows for introduction in a CADD/CAM manufacturing environment through application of the system database. This is a significant advantage over currently existing computerized structural design methods.

(b) The use of interactive control (and modular program structure) allows CASE-GBRIDGE to assist the engineer in his bridge evaluation but does not alleviate his design and professional responsibilities. For example, the use of GBRIDGE graphic stress displays and interactive control allows the user to modify the bridge system after synthesis operations are performed to examine, for instance, the economics of a partially stiffened web girder.

(c) Generation of structural analysis members from fabricated component data yields two major advantages for CASE. First, the analytical model utilized in CASE is the "true" structural system and not just a simplistic mathematical representation. Secondly, since the analytical model used is the actual structural system accounting for all material changes, the safety aspects are more rigorously examined since all of the true design parameters are included.

2. Analytical Aspects

(a) All continuous composite girder bridges and bridges composed of girders with material changes are nonprismatic girders and require nonprismatic element stiffness matrices to ensure reliability of the finite element results. The formulation and implementation herein of the nonprismatic element stiffness matrix based upon the classical energy method approach has proven very reliable and accurate. The element uses a reduced six degree of freedom coordinate system. This reduction requires only a minimum of computer memory storage space.

(b) In the analysis process, a unique procedure was utilized in CASE-GBRIDGE that has proven to be very accurate and computationally efficient. The live load investigation uses influence line equations that are in terms of girder end moments. These end moments are determined very rapidly using distribution equations, which reflect the distribution of loading throughout the bridge system. Furthermore, the analysis techniques currently employed in most bridge programs basically consider only hinge supports, but the technique used in GBRIDGE incorporates support conditions which allow for any degree of support fixity to be investigated. The analytical procedure employed would be advantageous to use in any computing environment.

(c) The variation of the moment of inertia of individual members controls the distribution of loading throughout the bridge girders. The major factor influencing the moment of inertia member property and, thus, the member loading, is the girder depth. However, about the optimum section, the girder becomes relatively insensitive to depth variation. Within the limits of this study, it has been observed that, if a girder depth, d_w, between 1/25 and 1/35 of the maximum span length is initially selected, along with selecting the flange widths of $(d_w/4.5) + 1.5$ and $(d_w/4.5) + 3.5$ for the top and bottom flanges of the composite girders (use $(d_w/4) + 2$ in the negative moment area), the load variation throughout the bridge synthesis process is negligible. Actually, for a girder that is close to the optimum section depth, the composite action location is the most crucial factor influencing the member load. GBRIDGE considers the dead load inflection points to define the location for composite action.

3. Synthesis Aspects

(a) Application of structural synthesis has been successfully applied to girder bridges, considering both the roadway deck and supporting girders. It has been observed that the roadway cost is a significant factor in the overall bridge cost, although one which is relatively insensitive. The synthesis process has considered separate objective functions and constraints for the roadway and girders, where the behavior constrains imposed are those given by AASHTO specifications considering both WSD and LFD methods. The side constraints employed reflect the true manufacturing limitations. The cost models developed are reasonably representative of current direct costs (manufacturing and labor).

(b) Two techniques have been used successfully in the synthesis process. The interior penalty function approach has been used for the roadway portion, in which case, the analysis and synthesis are accomplished simultaneously. The supporting girders have been examined through the application of backtracking. Using the backtracking method, which is suitable for nonconvex, nondifferentiable, and discrete-value objective functions, several bridge systems were examined. The backtracking technique employed has proven reliable in the girder synthesis operation, where iteration is required between the analysis and synthesis operations.

(c) Several conclusions and insights can be drawn from the application of CASE-GBRIDGE to the various bridge systems. These can be briefly summarized as:

* Once an approximate depth is obtained such that the distribution of internal girder forces remain constant, the girder depth can vary several inches from the optimum depth without appreciable effect on the girder cost. Preliminary sizes for the girder section have been given herein. The optimum depth is always greater than the AASHTO minimum depth (i.e., considering inflection points as defining span length).
* Under all conditions, girder bridges designed by the Load Factor Design philosophy will be more economical than those designed by the Working Stress Design method. The cost savings vary depending upon the number of spans and the span

length from approximately 5 percent to 20 percent.

* Unstiffened girders are always more economical in simple span bridges. As the number of spans and span lengths increase such that the combined shear and bending effects become significant, web stiffened girders become more economical. Actually, for the span lengths examined, partially stiffened girders are more economical. Furthermore, unstiffened girders possess smaller optimum depths and thicker webs than stiffened girders.
* The use of graphical stress display and interactive programming significantly reduces the engineering time required to review partially stiffened girder bridges.
* The optimum concrete roadway slab is relatively insensitive to supporting girder spacing resulting in the least number of girders (maximum spacing) always being the most economical (within the prescribed AASHTO limitations).
* The roadway cost, although insensitive to girder spacing, is a very significant factor in the overall superstructure cost. The overall superstructure cost, within the examined span

ranges, can be approximated as $40/sq. ft. The overall bridge cost must consider the costs of the substructure and approach roadway.

7.4 Recommendations

CASE-GBRIDGE has satisfied all of the objectives set forth in this investigation, but certain enhancements to and extensions of this methodology are recommended. These are:

(1) Extend the research into the manufacturing aspect of CASE methodology. This initial effort has considered "how to" formulation of a structural engineering methodology that allows a manufacturing program module to extract the required data from the computer database to allow fabrication of individual fabricated components. One of the next steps needs to be the development of the process control programs to execute the various manufacturing aspects.

(2) Extend CASE-GBRIDGE to accommodate more bridge types, e.g., box girder bridges, cable stayed bridges, etc. Also, extend CASE methodology into other civil engineering type structures, such as building systems [9]. This can be accomplished by formulating the required fabricated component, so that the manufacturing function and design are the same.

(3) Modify the analysis module to consider rigid framed bridges (without user intervention). Also, after

transformation to a compiler language, extend CASE-GBRIDGE to automatically compare lane and truck loading conditions. A limitation in CASE-GBRIDGE implementation has been the development computer language. Thus, the transformation of CASE into a compiler computer language, such as FORTRAN, and implementation on a microcomputer containing a math co-processor board is recommended.

(4) Extend GBRIDGE to examine the substructure and approach roadway along with the bridge superstructure. Extensive use of a large project database would be required, beyond that to be utilized in manufacturing.

(5) Examine the dynamics of the bridge system, including structure-soil interaction, via addition of a dynamic analysis module to CASE-GBRIDGE.

(6) Examine the synthesis process through the use of a "rule-based" expert system. Also, extend the graphics applications simultaneously.

7.5 Summary

This dissertation has developed and presented a new and significant concept for the optimal design of highway girder bridges. The need for such a methodology is clear; the requirements for implementation have been outlined and explored in depth.

This research has incorporated unique approaches to several points included in this study. Of primary significance is the concept of fabricated components. This concept allows the analysis model to represent exactly the structure under consideration, eliminating the use of inaccurate mathematical approximations. This concept is essential for the effective and efficient application of CADD/CAM to civil engineering type structures.

Especially important for the efficient flow and sharing of data is the relational database developed herein. This relational database concept utilizes a layered approach which accommodates the fabricated component concept. This database formulation eliminates the need for storage of member section properties. Only the element data required to physically fabricate each individual component are stored within the CASE database.

In this research, an accurate and efficient element stiffness matrix for nonprismatic members has been developed that utilizes numeric quadrature, the accuracy of which was demonstrated through example problems. A stiffness analysis procedure has been formulated employing the nonprismatic element stiffness matrix which includes a rapid load analysis capability based upon described shear and moment influence line equations. As a result, the analysis process uses the true structural variation of cross-section properties, rendering the analysis results more precise.

When an analysis is performed at a particular analysis point, all necessary information is regenerated for each analysis utilizing the member end moment data.

Following discussion of the analysis process, this study describes the interlocking and interdependent nature of the analysis/synthesis process and explicitly formulates the objective functions, for both the roadway slab and the supporting girders, in terms of cost. The girder objective function considers the cost of fabrication and all costs which are related to the variation in girder depth. Both the AASHTO Working Stress Design method and the Load Factor Design method are considered in the synthesis process and constraints based upon these specifications are presented.

Employing either WSD or LFD, the analysis/synthesis process is applied in the selection of the "optimal" design variables. The girders (plate elements) are selected based upon a uniform girder depth that is evaluated based upon a weighted average approach. To ensure adherance to safety requirements, the design checks are performed independently of the structural synthesis process.

As an alternative to the direct structural synthesis approach, the CASE methodology has included a graphics plotting system that graphically displays on the computer terminal the actual and allowable stresses. This can be used in conjunction with the independent cost evaluation segment to rapidly converge to the optimal design variables and/or verify the results obtained from the synthesis. The accuracy and reliability of the CASE-GBRIDGE system is illustrated through application to a wide range of bridge systems. Cost comparison has yielded insight into the contributing factors to bridge economy, along with some general recommended member arrangements.

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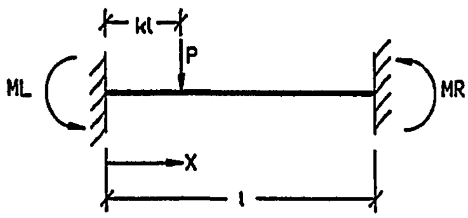
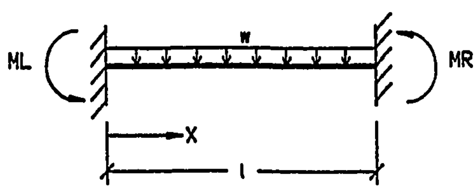
APPENDICES

Appendix A

Formulation of the nonprismatic element stiffness matrix has been presented. However, the bridge analysis procedure is based upon the action-displacement equations to relate member degrees to freedom to member end forces (equivalent nodal forces). These action-displacement equations are obtained by relating the accummulated effects of the element stiffnesses and related member end forces (joint loads). For a concentrated and uniformly distributed load, the equivalent nodal member end forces are given by:



Concentrated Load



Uniform Load

Concentrated Loading:

$$M_L = \frac{-Pk\ell}{(AC-B^2)}[AC - B^2 + kC(F-E) - k^2B(G-F)]$$

$$M_R = \frac{Pk\ell}{(AC-B^2)}[k(C-B)(F-E) + k^2(A-B)(G-F)]$$

Uniform Loading:

$$M_L = \frac{-w\ell^2}{2(AC-B^2)}[BD - C^2]$$

$$M_R = \frac{w\ell^2}{2(AC-B^2)}[AC - B^2 + BD - C^2 - AD + BC]$$

in which A, B, and C are as previously defined and

$$D = I_L \int_\ell \frac{x^3 dx}{L^4 I_x} \qquad E = I_L \int_\ell \frac{dx}{(k\ )I_x}$$

$$F = I_L \int_\ell \frac{x\ dx}{(k\ell)^2 I_x} \qquad G = I_L \int_\ell \frac{x^2 dx}{(k\ell)^3 I_x}$$

Numeric integration is employed in evaluation of the action-displacement equations. The numeric procedure utilized is Gaussian quadrature in which the algebraic expressions are transformed into a summation of numeric products of weighted coefficients evaluated at normalized sampling points (gauss points). The general quadrature expression is given by:

$$I = \int_{-1}^{1} f(x)dx = \sum_{i=1}^{n} W_i * f(\xi_i)$$

where W_i are the weighting coefficients which are multiplied by the function value at the gauss point ξ_i. The gauss weighting coefficient at normalized sampling points are given as:

n	ξi	Wi
1	0	2
2	$-\frac{1}{3}, +\frac{1}{3}$	1,1
3	$-\frac{3}{5}, 0, +\frac{3}{5}$	$\frac{5}{9}, \frac{8}{9}, \frac{5}{9}$

The largest numeric integration occurs for the polynomial expressions of the highest degree, i.e., for the uniformly loaded nonprismatic beams presented in Chapter 4. Consider the segmented nonprismatic beam from Chapter 4, re-illustrated in Figure A.1. The coordinate transformation is given by:

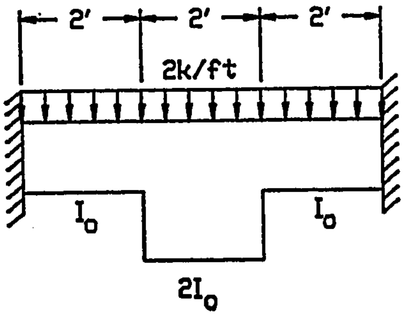


FIGURE A1

$$a(=0) \le x \le b(=L/3) \qquad x = (L/6)(\xi+1)$$

$$dx = (L/6)d\xi$$

$$a(=L/3) \le x \le b(=2L/3) \qquad x = (L/6)(\xi+3)$$

$$dx = (L/6)d\xi$$

$$a(=2L/3) \le x \le b(=L) \qquad x = (L/6)(\xi+5)$$

$$dx = (L/6)d\xi$$

The individual coefficients are determined from:

$$A = I_L \int_0^{\ell} \frac{dx}{LI_x} = \sum_1^3 I_L \int_{-1}^{1} \frac{(L/6)d\xi}{L(I_x)} = \Sigma\, W_i * f(\xi_i) \qquad @\ \xi=0,\ W=2$$

$$= \frac{2}{6} + \frac{2}{12} + \frac{2}{6}$$

$$A = 0.833333$$

$$B = I_L \int_0^{\ell} \frac{x\,dx}{L^2 I_x} = I_L \left[\int_{-1}^{1} \frac{(\xi+1)(L/6)(L/6)d\xi}{L^2 I_x} \right.$$

$$\left. + \int_{-1}^{1} \frac{(\xi+3)(L/6)(L/6)d\xi}{L^2 I_x} + \int_{-1}^{1} \frac{(\xi+5)(L/6)(L/6)d\xi}{L^2 I_x} \right]$$

$$= \Sigma\, W_i * f(\xi_i) \qquad @\ \xi = 0,\ W=2$$

$$= (1)\left(\frac{1}{6}\right)^2(2) + (3)\left(\frac{1}{6}\right)^2\left(\frac{1}{2}\right)^2 + (5)\left(\frac{1}{6}\right)^2(2)$$

$$B = 0.4166667$$

$$C = I_L \int_0^{\ell} \frac{x^2 dx}{L^3 I_x} = I_L \left[\int_{-1}^{1} \frac{(\xi+1)^2(L/6)^2(L/6)d\xi}{L^3 I_x} \right.$$

$$\left. + \int_{-1}^{1} \frac{(\xi+3)^2(L/6)^2(L/6)d\xi}{L^3 I_x} + \int_{-1}^{1} \frac{(\xi+5)^2(L/6)^2(L/6)d\xi}{L^3 I_x} \right]$$

$$= \sum_{i=1}^{2} W_i * f(\xi_i) \qquad @\ \xi = \pm \frac{1}{3},\ W = 1.0$$

$$= \left(\frac{1}{216}\right)[0.42265]^2(1) + \left(\frac{1}{216}\right)[1.57735]^2(1)$$

$$+ \left(\frac{1}{216}\right)[2.42265]^2\left(\frac{1}{2}\right)(1) + \left(\frac{1}{216}\right)[3.57735]^2\left(\frac{1}{2}\right)(1)$$

$$+ \left(\frac{1}{216}\right)[4.42265]^2(1) + \left(\frac{1}{216}\right)[5.57735]^2(1)$$

$$C = 0.29012346$$

$$D = I_L \int_0^{\ell} \frac{x^3 dx}{L^4 I_x} = I_L \left[\int_{-1}^{1} \frac{(\xi+1)^3(L/6)^3(L/6)d\xi}{L^4 I_x} \right.$$

$$\left. + \int_{-1}^{1} \frac{(\xi+3)^3(L/6)^3(L/6)d\xi}{L^4 I_x} + \int_{-1}^{1} \frac{(\xi+5)^3(L/6)^3(L/6)d\xi}{L^4 I_x} \right]$$

$$= \sum_{i=1}^{3} W_i * f(\xi_i) \qquad @\ \xi = -\frac{3}{5}, 0, \frac{3}{5};\ W = \frac{5}{9}, \frac{8}{9}, \frac{5}{9}$$

$$= (\frac{1}{1296})[0.22540]^3(\frac{5}{9}) + (\frac{1}{1296})[1.0000]^3(\frac{8}{9})$$

$$+ (\frac{1}{1296})[1.77459]^3(\frac{5}{9}) + (\frac{1}{1296})[2.22540]^3(\frac{1}{2})(\frac{5}{9})$$

$$+ (\frac{1}{1296})[3.0000]^3(\frac{1}{2})(\frac{8}{9}) + (\frac{1}{1296})[3.77459]^3(\frac{1}{2})(\frac{5}{9})$$

$$+ (\frac{1}{1296})[4.22450]^3(\frac{5}{9}) + (\frac{1}{1296})[5.000]^3(\frac{8}{9})$$

$$+ (\frac{1}{1296})[5.77459]^3(\frac{5}{9})$$

$$D = 0.22685185$$

Using the evaluated coefficients, the equivalent joint load vector is obtained as follows:

$$M_L = -\frac{W\ell^2}{2}\left[\frac{BD-C^2}{AC-B^2}\right]$$

$$= -\frac{W\ell^2}{2}\left[\frac{(0.416667)(0.226850) - (0.290123)^2}{(0.833333)(0.290123) - (0.416667)^2}\right]$$

$$= -\ 0.75925\ W\ell^2$$

$$M_R = \frac{W\ell^2}{2}\left[\frac{AC-B^2+BD-C^2-AD+BC}{AC-B^2}\right]$$

$$= \left[\frac{w\ell^2}{2}\right] \frac{\begin{array}{c}[(0.833333)(0.290123) - (0.416667)^2 \\ + (0.416667)(0.226850) - (0.290123)^2 \\ - (0.833333)(0.226850) + (0.416667)(0.290123)]\end{array}}{(0.833333)(0.290123) - (0.416667)^2}$$

$$= 0.075925\ w\ell^2$$

where, specifying a uniform load of 2 k/ft and a member length of 6 feet, this yields the equivalent load vector of

$$F_o = \begin{bmatrix} 0 \\ 6 \\ -5.4667 \\ 0 \\ 6 \\ 5.4667 \end{bmatrix}$$

These results yield precisely the "exact" theoretical member end forces.

The numeric quadrature procedure is similarly applied to variable depth nonprismatic beam elements. In application of the nonprismatic element to variable depth beams, i.e., when the start and terminal member depths are not the same, the moment of inertia variation must be considered. The variable depth girder of Chapter 4 is re-illustrated in Figure A2. The coordinate transformations are given as:

$$a(=0) \leq x \leq b(=0.4\ell) \qquad x = \frac{\ell}{5}(\xi+1);\ dx = \frac{\ell}{5}\, d\xi$$

$$a(=0.4\ell) \leq x \leq b(=0.7\ell) \qquad x = \frac{\ell}{20}(3\xi+1);\ dx = \frac{3\ell}{20}\, d\xi$$

$$a(=0.7\ell) \leq x \leq b(=\ell) \qquad x = \frac{\ell}{20}(3\xi+1);\ dx = \frac{3\ell}{20}\, d\xi$$

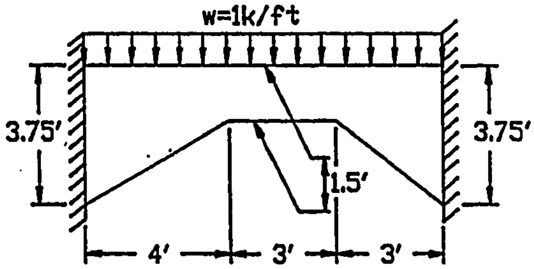


FIGURE A2

The individual coefficients are obtained by accounting for the variation in the members' moment of inertia by evaluation of inertia property at the segmented Gauss points. The coefficients can be evaluated by considering the components of the variable depth girder from Figure A.2. The A coefficient is obtained from:

$$A = I_L \int_0^{\ell} \frac{dx}{LI_x} = I_L \int_0^{.4\ell} \frac{dx}{LI_x} + I_L \int_{.4\ell}^{.7\ell} \frac{dx}{LI_x} + I_L \int_{.7\ell}^{\ell} \frac{dx}{LI_x}$$

$$= I_L \int_{-1}^{1} \frac{Ld\xi}{5LI_x} + I_L \int_{-1}^{1} \frac{3Ld\xi}{20LI_x} + I_L \int_{-1}^{1} \frac{3Ld\xi}{20LI_x}$$

$$= I_L\left[\frac{1}{5Ix_{A1}} + \frac{1}{5Ix_{B1}}\right] + \frac{I_L}{I_C}\left[\frac{6}{20}\right]$$

$$+ I_L\left[\frac{3}{20Ix_{A2}} + \frac{3}{20Ix_{B2}}\right]$$

where the subscribed "I"s refer to moment of inertia at the Gauss points.

$$A = 6.59[0.253] + \frac{6.59}{0.42}[0.3] + 6.59[0.189]$$

$$A = 7.607283$$

The other coefficients are evaluated similarly and are:

$$B = 4.0293866$$

$$C = 2.387009$$

$$D = 1.5226013$$

The equivalent joint load vector is obtained using the evaluated coefficients. The member end moments are obtained as:

$$M_L = - \frac{W\ell^2}{2} \left[\frac{BD-C^2}{AC-B^2}\right] = 11.37 \text{ ft-k}$$

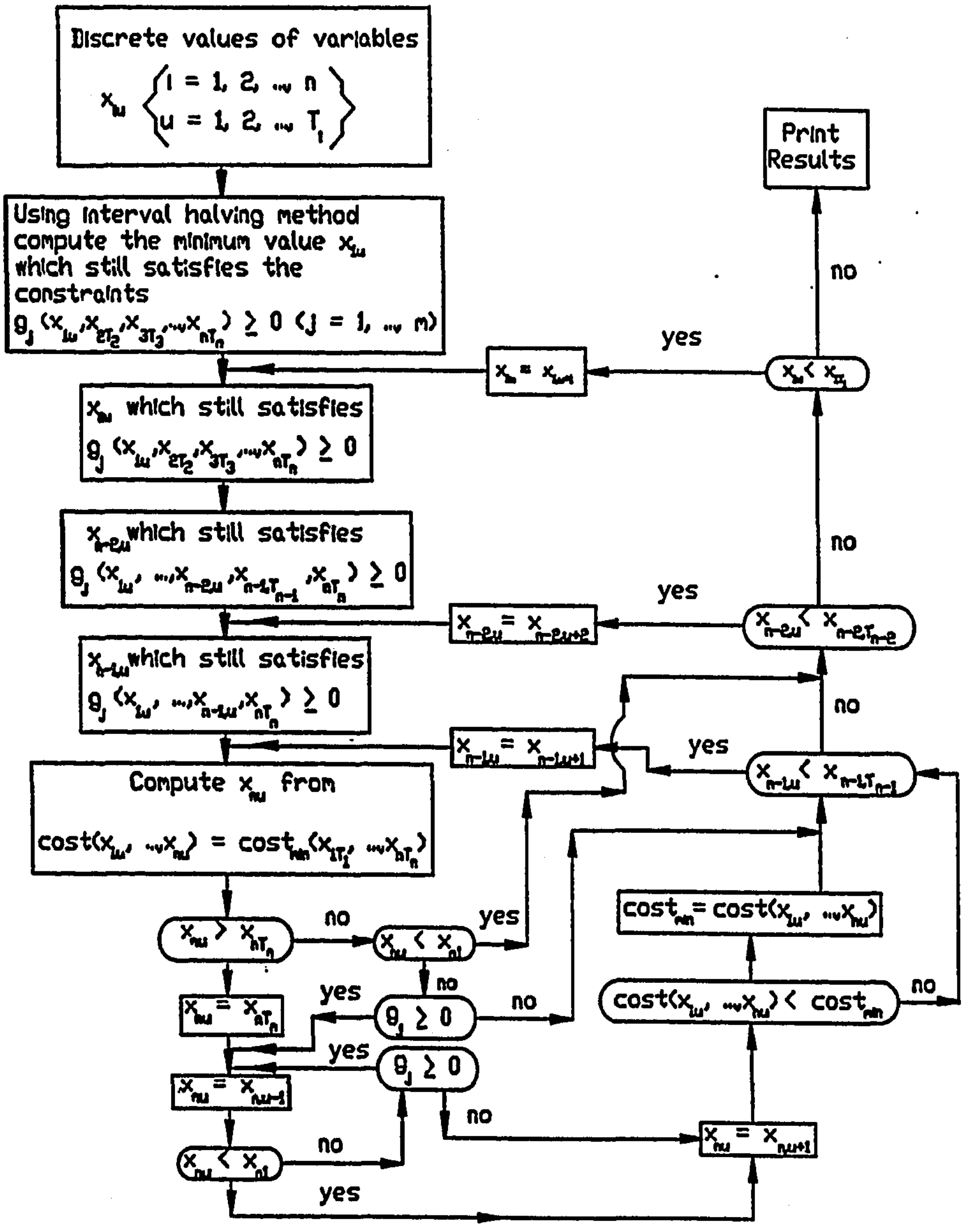
$$M_R = \frac{W\ell^2}{2} \left[\frac{AC-B^2+BD-C^2-AD+BC}{AC-B^2}\right] = 10.28 \text{ ft-k}$$

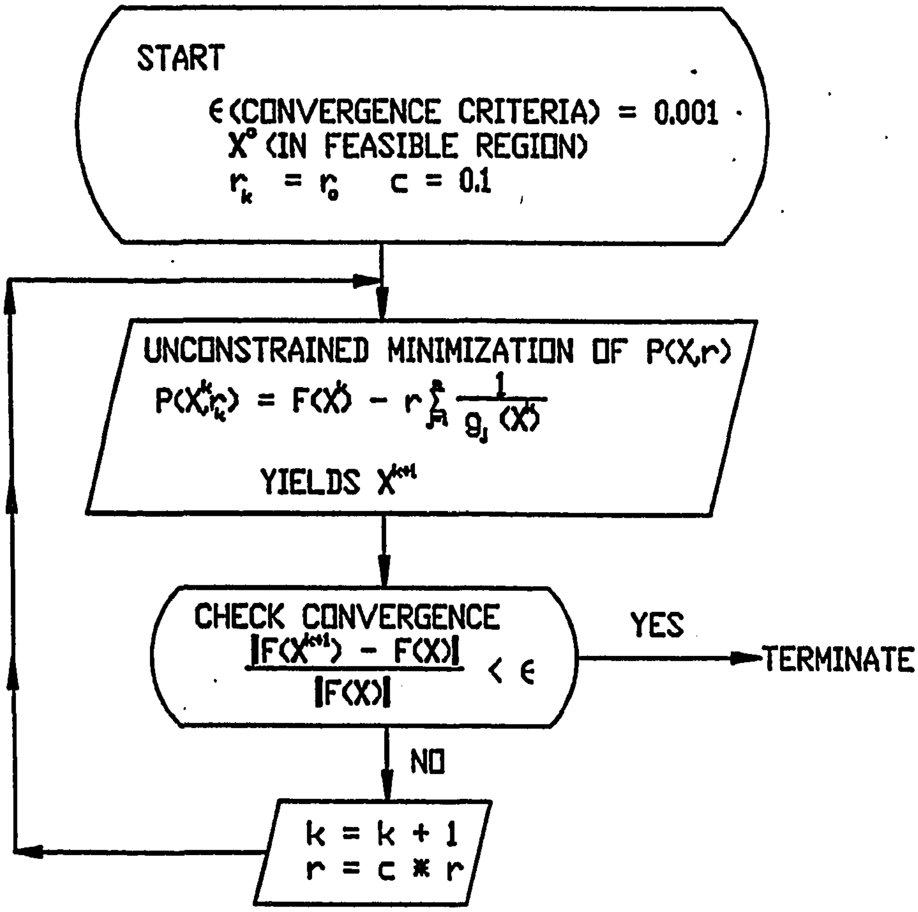
and the resulting joint load vector is given as:

$$F_o = \begin{bmatrix} 0 \\ 5.109 \\ 11.370 \\ 0 \\ 4.891 \\ 10.280 \end{bmatrix}$$

The precision obtained from application of the nonprismatic element numeric quadrature process has been illustrated in Chapter 4 in which the traditional analysis approach to this variable depth girder problem resulted in a 4.2 percent error as compared to only 0.6 percent error obtained from employing the CASE approach. By employing this numeric quadrature procedure for any nonprismatic arrangement, the desired degree of accuracy can be obtained (within numeric truncation limits) by simply increasing the number of sampling points employed.

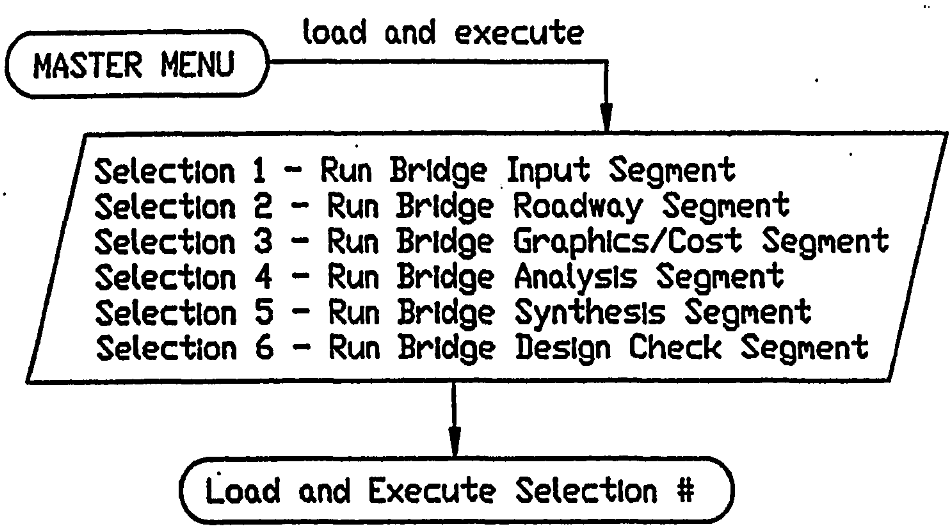
Appendix B

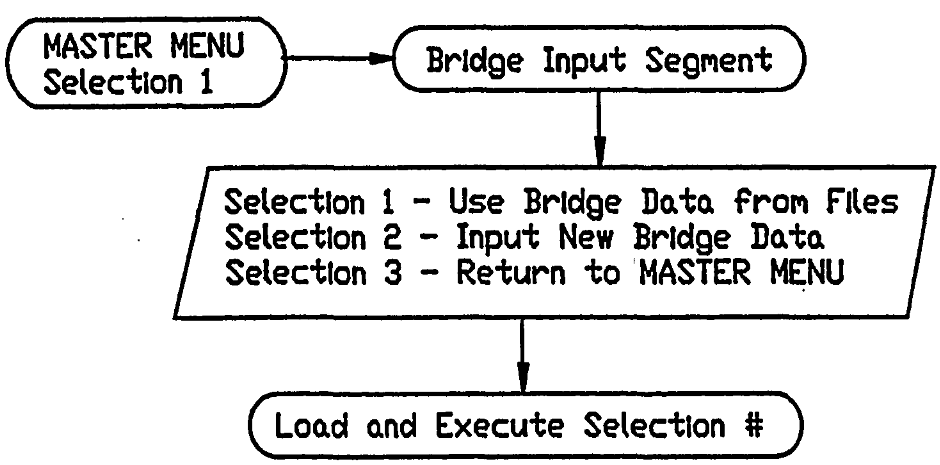

Discrete values of variables
x_{iu} $\left\{ \begin{matrix} i = 1, 2, \ldots, n \\ u = 1, 2, \ldots, T_i \end{matrix} \right\}$
Using interval halving method compute the minimum value x_{1u} which still satisfies the constraints
$g_j(x_{1u}, x_{2T_2}, x_{3T_3}, \ldots, x_{nT_n}) \geq 0$ $(j = 1, \ldots, m)$
x_{2u} which still satisfies
$g_j(x_{1u}, x_{2T_2}, x_{3T_3}, \ldots, x_{nT_n}) \geq 0$
$x_{n-2,u}$ which still satisfies
$g_j(x_{1u}, \ldots, x_{n-2,u}, x_{n-1,T_{n-1}}, x_{nT_n}) \geq 0$
$x_{n-1,u}$ which still satisfies
$g_j(x_{1u}, \ldots, x_{n-1,u}, x_{nT_n}) \geq 0$
Compute x_{nu} from
$cost(x_{1u}, \ldots, x_{nu}) = cost_{min}(x_{1T_1}, \ldots, x_{nT_n})$
Print Results
no
yes
$x_{1u} = x_{1,u+1}$
$x_{1u} < x_{1T_1}$
no
yes
$x_{n-2,u} = x_{n-2,u+2}$
$x_{n-2,u} < x_{n-2,T_{n-2}}$
no
yes
$x_{n-1,u} = x_{n-1,u+1}$
$x_{n-1,u} < x_{n-1,T_{n-1}}$
$x_{nu} > x_{nT_n}$
no
$x_{nu} < x_{n1}$
yes
$cost_{min} = cost(x_{1u}, \ldots, x_{nu})$
$x_{nu} = x_{nT_n}$
yes
$g_j \geq 0$
no
no
$cost(x_{1u}, \ldots, x_{nu}) < cost_{min}$
no
yes
$g_j \geq 0$
$x_{nu} = x_{n,u-1}$
no
$x_{nu} = x_{n,u+1}$
$x_{nu} < x_{n1}$
no
yes

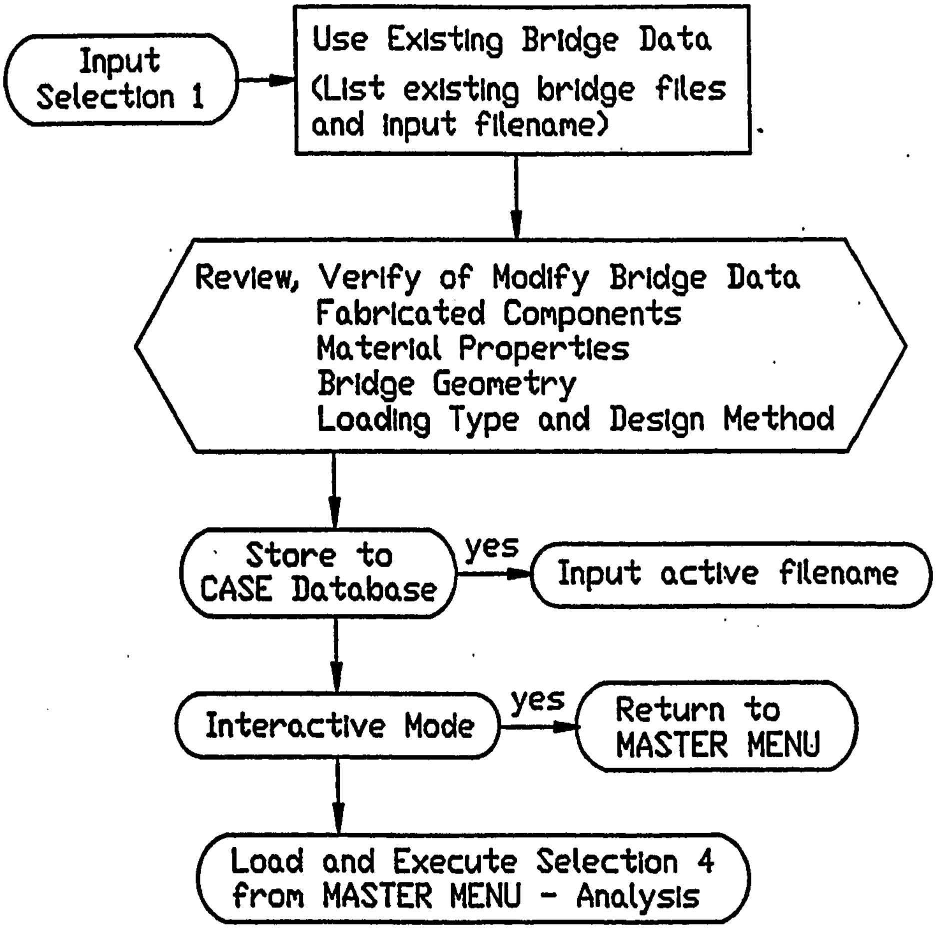

START
ϵ (CONVERGENCE CRITERIA) = 0.001
X° (IN FEASIBLE REGION)
r_k = r_0 c = 0.1
UNCONSTRAINED MINIMIZATION OF P(X,r)
YIELDS X^(k+1)
CHECK CONVERGENCE
YES
TERMINATE
NO
k = k + 1
r = c * r

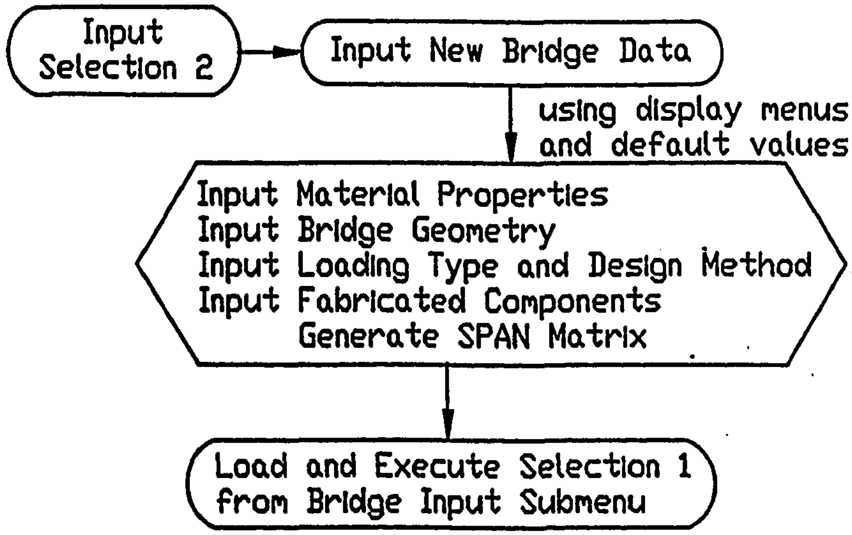
Appendix C

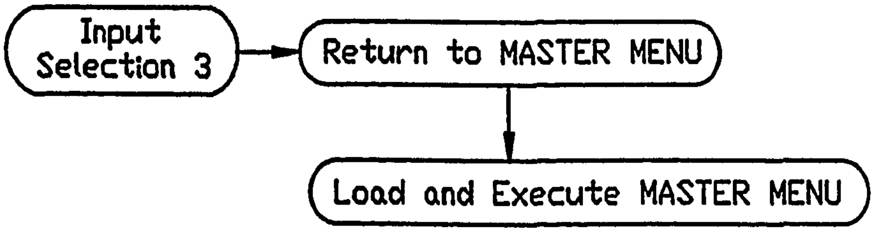
GBRIDGE - GENERAL FLOW

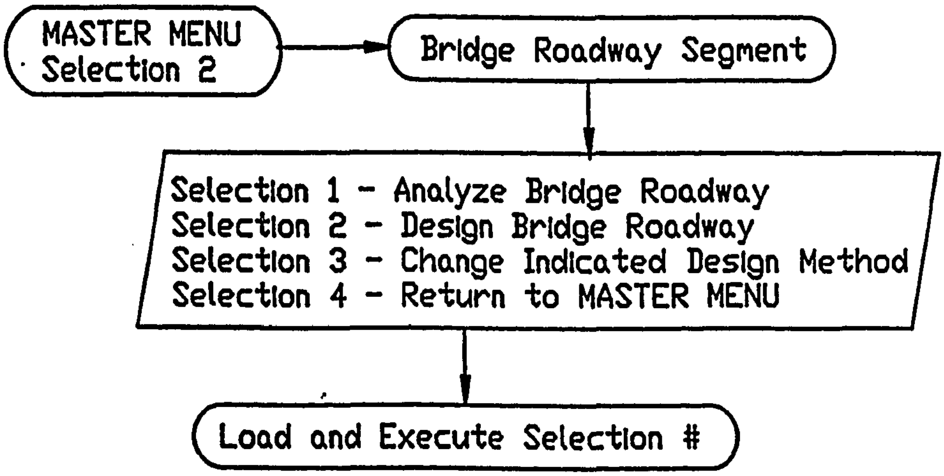


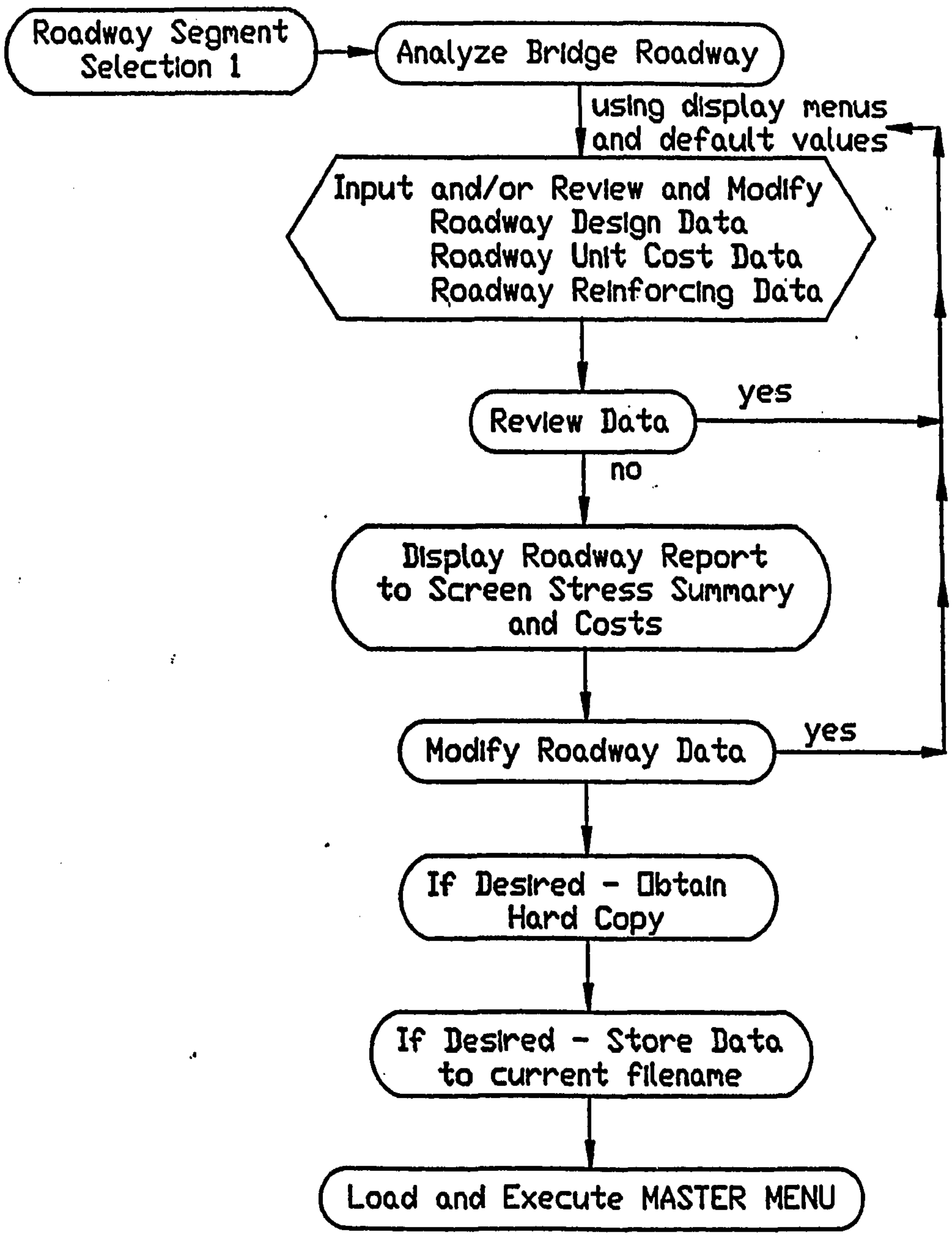



Input
Selection 1
Use Existing Bridge Data
(List existing bridge files
and input filename)
Review, Verify of Modify Bridge Data
Fabricated Components
Material Properties
Bridge Geometry
Loading Type and Design Method
Store to
CASE Database
yes
Input active filename
Interactive Mode
yes
Return to
MASTER MENU
Load and Execute Selection 4
from MASTER MENU - Analysis

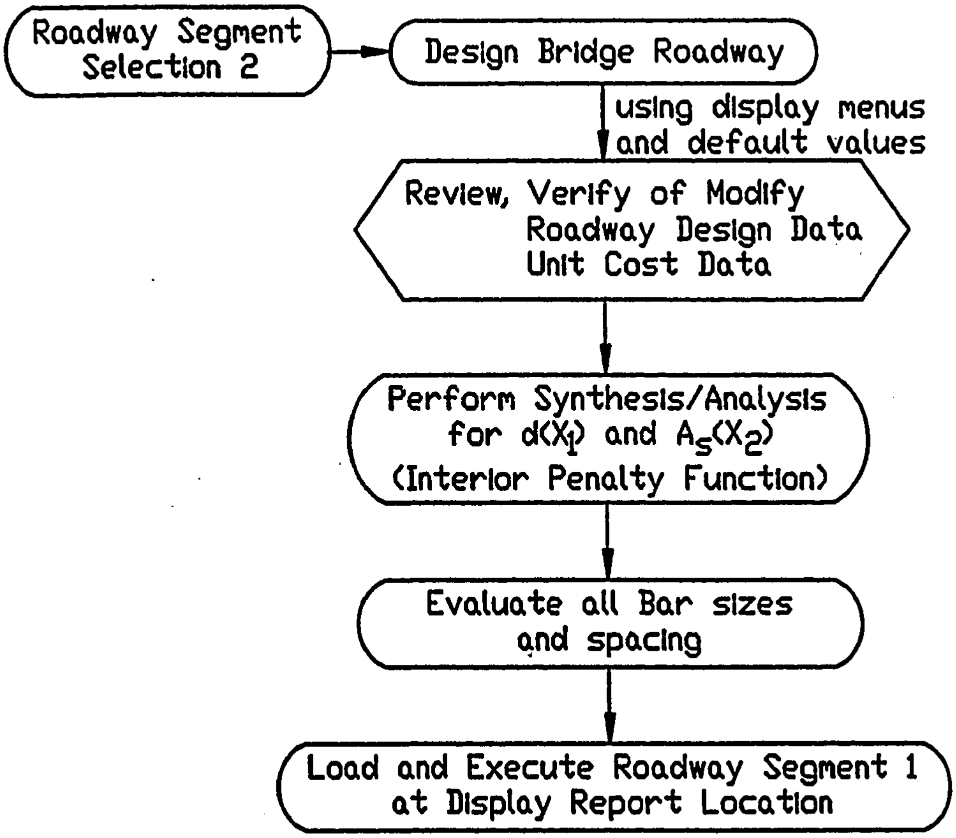

Input
Selection 2
Input New Bridge Data
using display menus
and default values
Input Material Properties
Input Bridge Geometry
Input Loading Type and Design Method
Input Fabricated Components
Generate SPAN Matrix
Load and Execute Selection 1
from Bridge Input Submenu

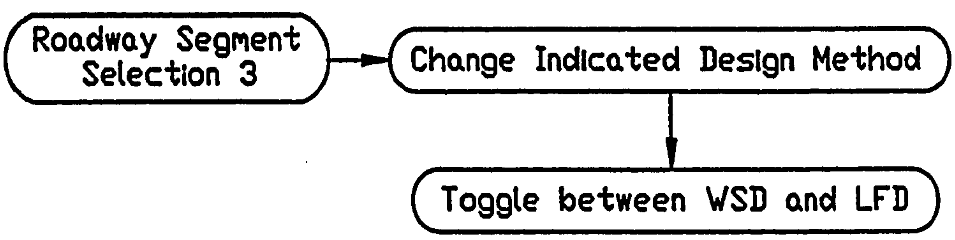

Input
Selection 3
Return to MASTER MENU
Load and Execute MASTER MENU

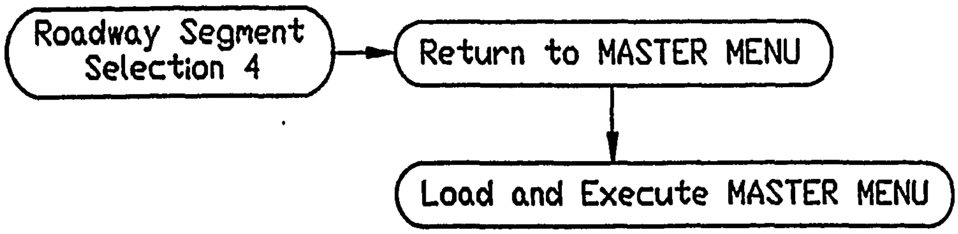

MASTER MENU
Selection 2
Bridge Roadway Segment
Selection 1 - Analyze Bridge Roadway
Selection 2 - Design Bridge Roadway
Selection 3 - Change Indicated Design Method
Selection 4 - Return to MASTER MENU
Load and Execute Selection #

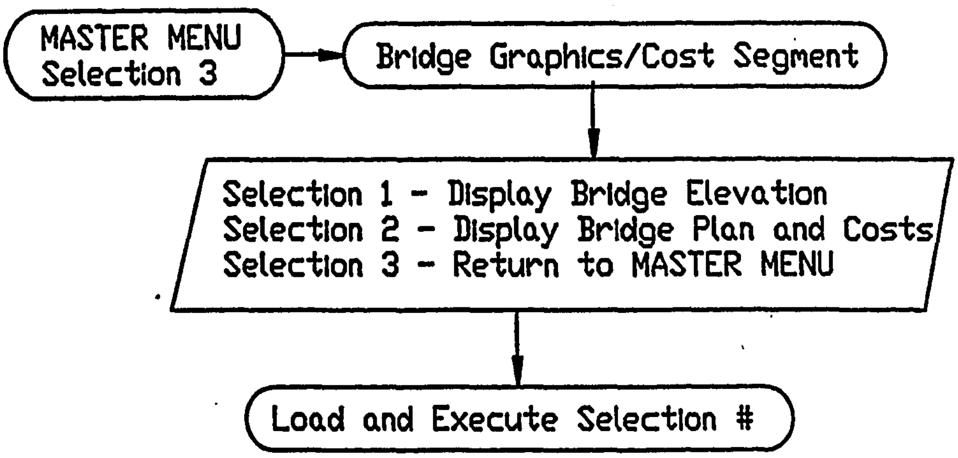


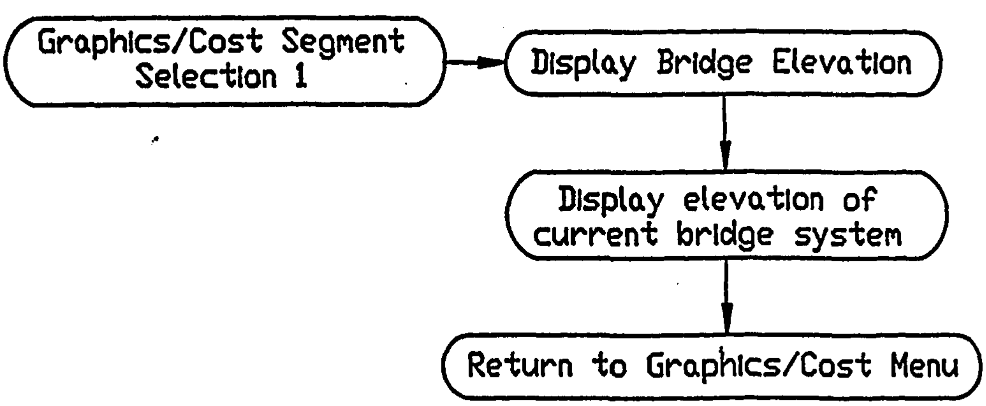
Roadway Segment Selection 1
Analyze Bridge Roadway
using display menus and default values
Input and/or Review and Modify Roadway Design Data Roadway Unit Cost Data Roadway Reinforcing Data
Review Data
yes
no
Display Roadway Report to Screen Stress Summary and Costs
Modify Roadway Data
yes
If Desired - Obtain Hard Copy
If Desired - Store Data to current filename
Load and Execute MASTER MENU

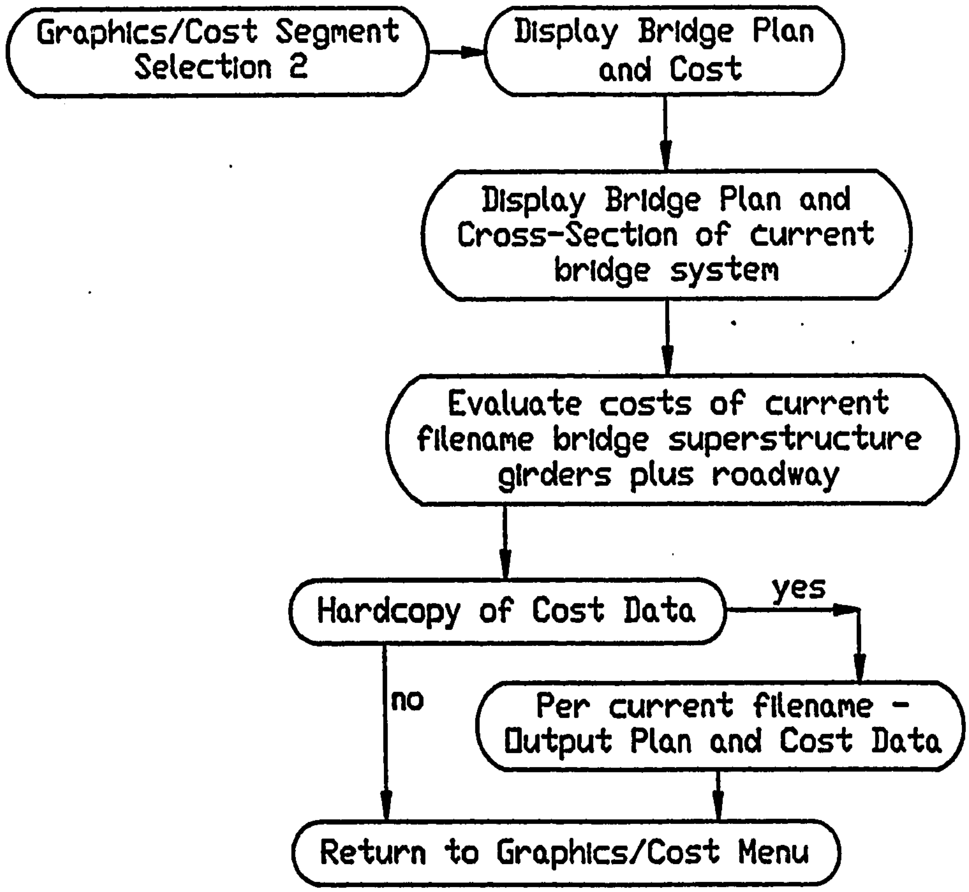

Roadway Segment Selection 2
Design Bridge Roadway
using display menus and default values
Review, Verify of Modify
Roadway Design Data
Unit Cost Data
Perform Synthesis/Analysis for $d(X_1)$ and $A_s(X_2)$
(Interior Penalty Function)
Evaluate all Bar sizes and spacing
Load and Execute Roadway Segment 1 at Display Report Location

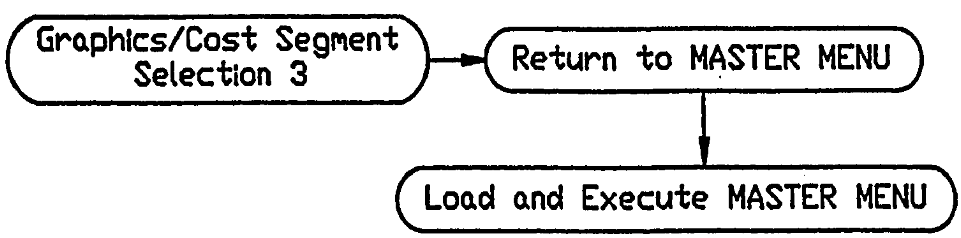

Roadway Segment Selection 3
Change Indicated Design Method
Toggle between WSD and LFD

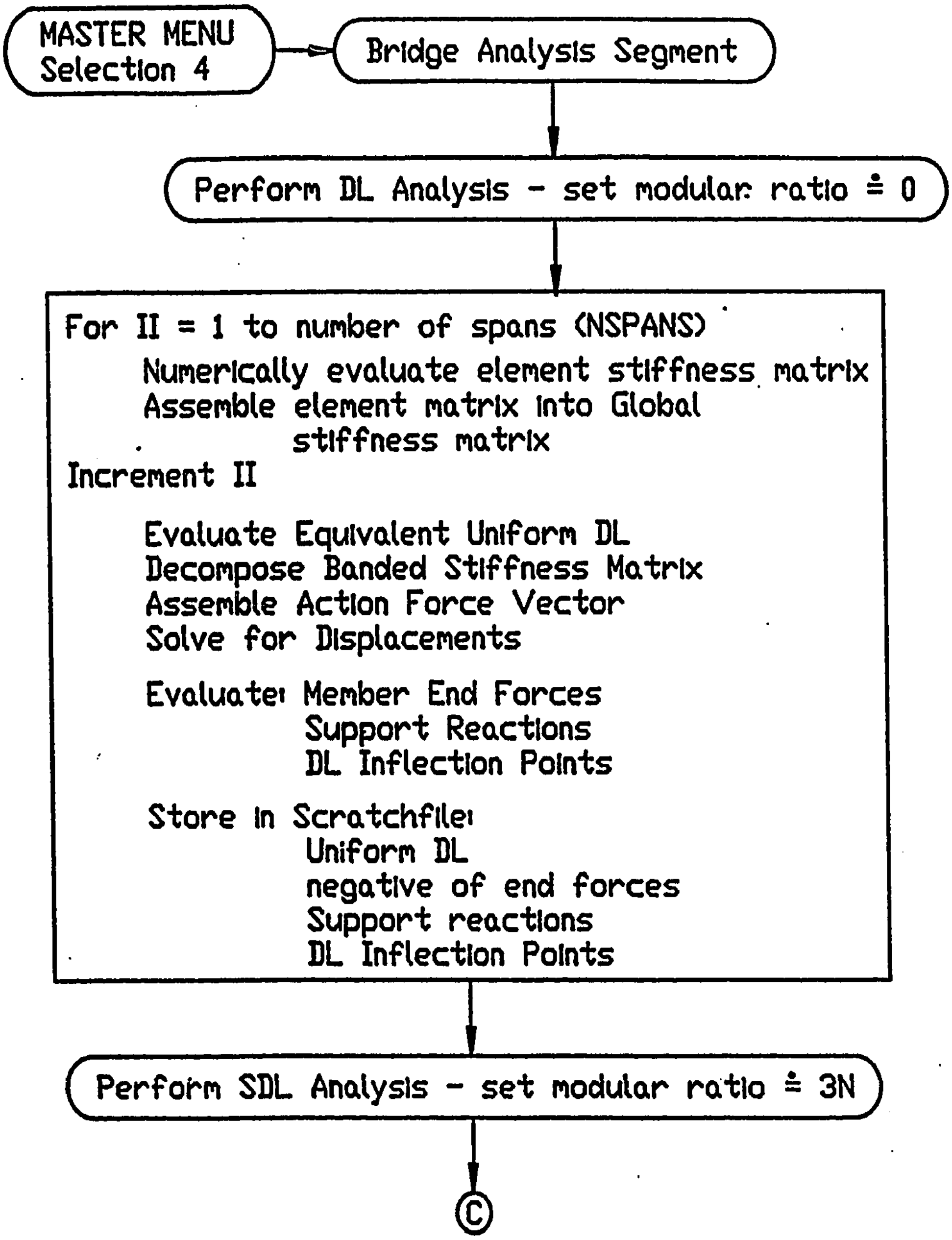

Roadway Segment
Selection 4
Return to MASTER MENU
Load and Execute MASTER MENU


MASTER MENU
Selection 3
Bridge Graphics/Cost Segment
Selection 1 - Display Bridge Elevation
Selection 2 - Display Bridge Plan and Costs
Selection 3 - Return to MASTER MENU
Load and Execute Selection #


Graphics/Cost Segment
Selection 1
Display Bridge Elevation
Display elevation of
current bridge system
Return to Graphics/Cost Menu


Graphics/Cost Segment Selection 2
Display Bridge Plan and Cost
Display Bridge Plan and Cross-Section of current bridge system
Evaluate costs of current filename bridge superstructure girders plus roadway
Hardcopy of Cost Data
yes
no
Per current filename - Output Plan and Cost Data
Return to Graphics/Cost Menu


Graphics/Cost Segment Selection 3
Return to MASTER MENU
Load and Execute MASTER MENU


MASTER MENU
Selection 4
Bridge Analysis Segment
Perform DL Analysis - set modular ratio ≐ 0
For II = 1 to number of spans (NSPANS)
Numerically evaluate element stiffness matrix
Assemble element matrix into Global
stiffness matrix
Increment II
Evaluate Equivalent Uniform DL
Decompose Banded Stiffness Matrix
Assemble Action Force Vector
Solve for Displacements
Evaluate: Member End Forces
Support Reactions
DL Inflection Points
Store in Scratchfile:
Uniform DL
negative of end forces
Support reactions
DL Inflection Points
Perform SDL Analysis - set modular ratio ≐ 3N
C

C

For II = 1 to NSPANS
Using DL inflection points and composite action condition
Numerically evaluate element stiffness matrix
Assemble element matrix into Global stiffness matrix
Increment II

Decompose Banded Stiffness Matrix
Assemble Action Force Vector
Solve for Displacements

Evaluate: Member End Forces
Support Reactions

Store in Scratchfile:
SDL (superimposed dead load)
negative of end forces
Support reactions

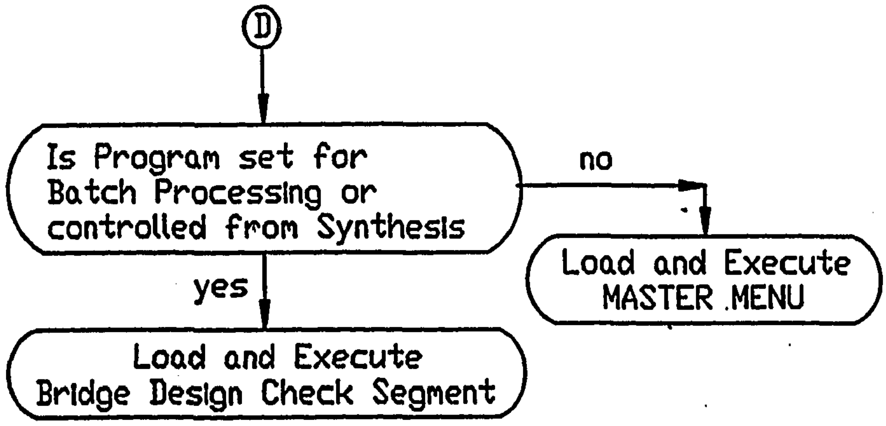
Perform LL Analysis - modular ratio ≐ N

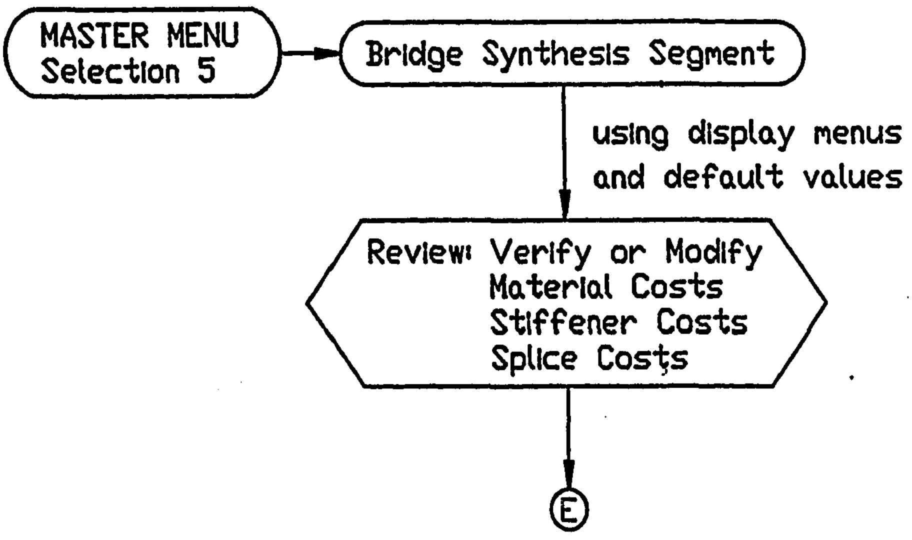
For II = 1 to NSPANS
Using DL inflection points and composite action condition
Numerically evaluate element stiffness matrix
Assemble element stiffness matrix into Global Stiffness Matrix
Increment II

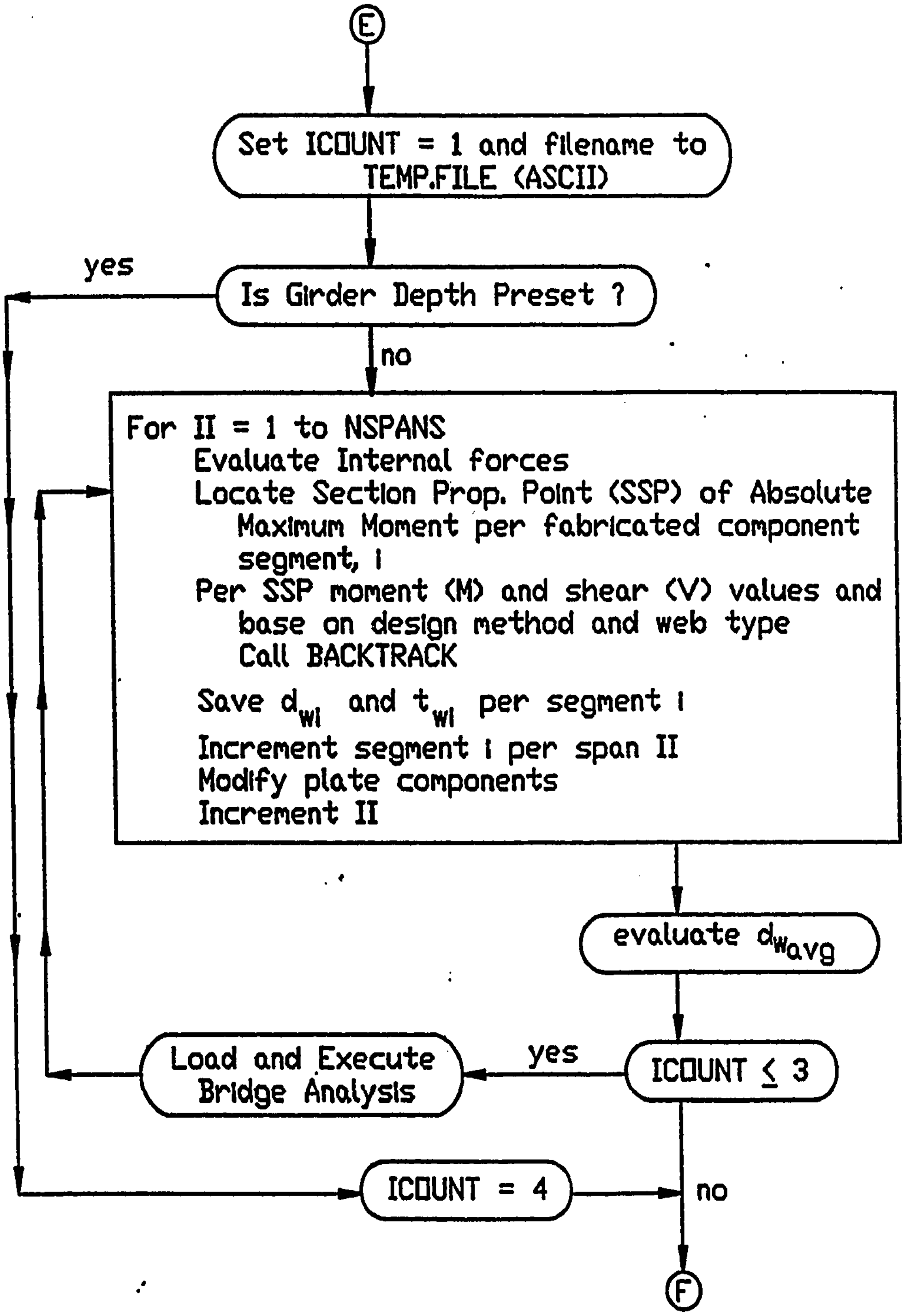
Decompose Banded Stiffness Matrix
Apply Joint Moments and evaluate End Moment Values

Store in Scratchfile:
End Moment Values
Support Reactions

D


D
Is Program set for Batch Processing or controlled from Synthesis
no
Load and Execute MASTER .MENU
yes
Load and Execute Bridge Design Check Segment


MASTER MENU Selection 5
Bridge Synthesis Segment
using display menus and default values
Review: Verify or Modify Material Costs Stiffener Costs Splice Costs
E


E
Set ICOUNT = 1 and filename to TEMP.FILE (ASCII)
yes
Is Girder Depth Preset ?
no
For II = 1 to NSPANS
Evaluate Internal forces
Locate Section Prop. Point (SSP) of Absolute Maximum Moment per fabricated component segment, i
Per SSP moment (M) and shear (V) values and base on design method and web type
Call BACKTRACK
Save d_{wi} and t_{wi} per segment i
Increment segment i per span II
Modify plate components
Increment II
evaluate $d_{w_{avg}}$
Load and Execute Bridge Analysis
yes
ICOUNT ≤ 3
ICOUNT = 4
no
F

F

d_W = depthset or $d_{W_{avg}}$

For II = 1 to NSPANS
Evaluate Internal forces
Locate SSP of Absolute maximum moment of segment I
Per SSP M and V value, call BACKTRACK
If moment changes sign in segment I,
   then per SSP with max M (and related V)
   of opposite sign, call BACKTRACK
   COMPARE +M and -M results and
   set plate sizes as largest plates
Increment segment I per span II
Increment II

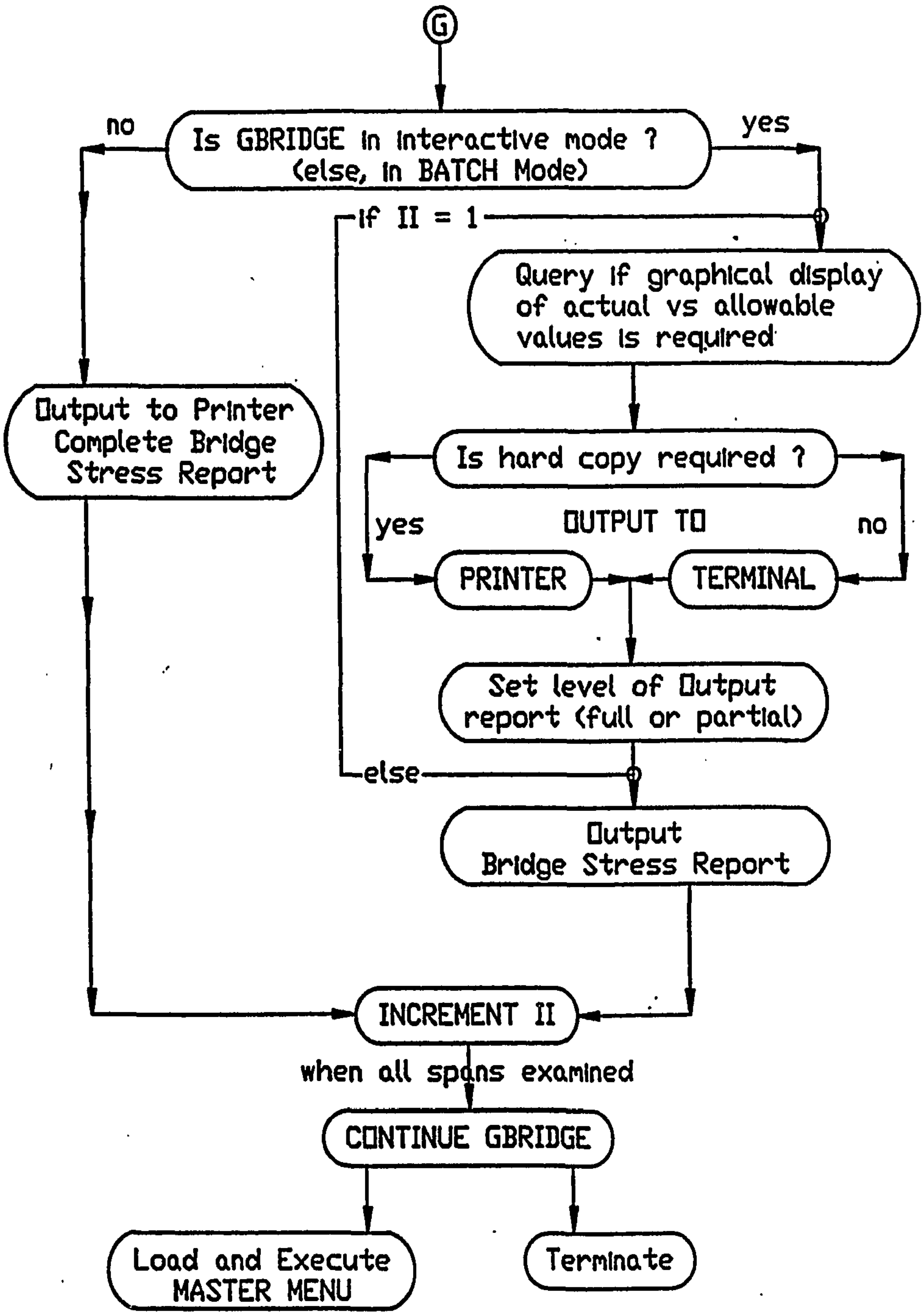
Load and Execute Bridge Analysis

MASTER MENU
Selection 6

Bridge Design Check Segment

Per Current filename
For II = 1 to NSPANS
   Evaluate Internal forces Per Sect. Prop. Point (SPP)
      and stresses/or factored forces
   Per Specified AASHTO Design Method
      evaluate Allowable values per SPP

G

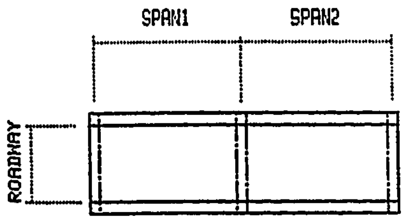

G
Is GBRIDGE in interactive mode ?
(else, in BATCH Mode)
no
yes
if II = 1
Query if graphical display
of actual vs allowable
values is required
Output to Printer
Complete Bridge
Stress Report
Is hard copy required ?
yes
OUTPUT TO
no
PRINTER
TERMINAL
Set level of Output
report (full or partial)
else
Output
Bridge Stress Report
INCREMENT II
when all spans examined
CONTINUE GBRIDGE
Load and Execute
MASTER MENU
Terminate

Appendix D

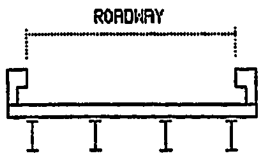
GBRIDGE COST REPORT

BRIDGE FILE NAME :WSD_2SPAN1
REPORT DATE :

**** BRIDGE SYSTEM ****



BRIDGE PLAN



X-SECT

**** COST SUMMARY ****

COST PER GIRDER (for entire bridge lenth) =$ 18083.26
NUMBER OF GIRDERS = 4
TOTAL WEIGHT OF SINGLE GIRDER (lbs) = 31025.00
COST OF ROADWAY (for entire bridge length) =$ 69720.12
TOTAL WEIGHT OF SLAB REBAR (lbs) = 49037.89

TOTAL BRIDGE SUPERSTRUCTURE COST =$ 142053.16

GBRIDGE ROADWAY REPORT

BRIDGE FILE NAME :WSD_2SPAN1
REPORT DATE :

**** ROADWAY DESIGN DATA ****

CONCRETE DATA :

ROADWAY SLAB THICKNESS(in)	: 8	TOP COVER(in)	: 2
CONC. STRENGTH (fc'-ksi)	: 4	BOTT COVER(in)	: 1.5

REINFORCEMENT DATA :(top rebar epoxy coated and rebar Fy= 60)

			BAR SIZE	SPACING (in)
ROADWAY				
TRANSVERSE DIRECTION :		TOP REBAR	6	6.00
		BOTT. REBAR	6	6.00
LONGITUDINAL DIRECTION:		TOP REBAR	4	18.00
	(center segment)	BOTT. REBAR	5	6.00
	(end segment)	BOTT. REBAR	5	12.00
OVERHANG				
TRANSVERSE DIRECTION :		TOP REBAR	6	6.00
		BOTT. REBAR	6	6.00
LONGITUDINAL DIRECTION:		TOP REBAR	4	9.00
		BOTT. REBAR	4	11.00

BRIDGE OVERHANG DATA :

OVERHANG DIST.(in)	: 34.00	CURB WIDTH(in)	: 0.0
GUARD RAIL TYPE	: CONCRETE BOX	CURB HEIGHT(in)	: 0.0

**** STRESS SUMMARY ****

ASSHTO SERVICE LOAD DESIGN

	*** MOMENTS (in-k)		** REINFORCEMENT **		CONCRETE ***	
	M_reqd	M_prov	ft_act	Ft_all	fc_act	Fc_all
ROADWAY	91.43	105.45	20.8	24.0	1.39	1.60
OVERHANG	68.67	105.45	15.6	24.0	1.04	1.60

MIM SLAB THICKNESS REQ'D (in) : 7.9
MIN FLEXURAL DEPTH REQ'D (in) : 5.9
MIN REINFORCENT AREA (in^2) : .77

LONGITUDINAL REINFORCEMENT DISTRIBTUION

	****	ROADWAY	***	OVERHANG	****
	* TOP REBAR	BOTT. MID	BOTT. END *	TOP REBAR	BOTT REBAR *
As REQD.	.125	.592	.296	.296	.125
As PROV.	.131	.614	.307	.262	.214

GBRIDGE STRESS REPORT

BRIDGE FILE NAME :WSD_2SPAN1
REPORT DATE :

**** GIRDER TYPE/ BRIDGE GEOMETRY ****

GIRDER TYPE ....: COMPOSITE GIRDER - for (+)&(-) MOM.
GIRDER LOCATION ....: INTERIOR GIRDER
LENGTH TO SYMM. (ft) (0=Unsymm.)..: 100
NUMBER OF SPANS ....: 2
NUMBER OF GIRDERS.................: 4
GIRDERS SPACING (ft)..............: 9.25
TRAFFIC ROADWAY WIDTH (ft)........: 28
TOTAL ROADWAY WIDTH (ft)..........: 30
TOTAL ROADWAY SLAB THCK'S (in)....: 8
ROADWAY HAUNCH (in)...............: 2.375
AREA OF CONC. RE-BAR (in^2).......: 5.4

**** MATERIAL PROPERTIES ****

MODULUS OF ELASTICTY (ksi)........: 29000
YIELD STRENGTH OF STL (ksi).......: 50
ULTIMATE STL STRENGTH (ksi).......: 65
CONC. COMP. STRENGTH (F'c-ksi)....: 4
MODULAR RATIO (Es/Ec).............: 8
RE-BAR YIELD STRENGTH.............: 60

**** DESIGN LOADS/ AASHTO DESIGN METHOD ****

LOADING TYPE ....: HS20-44 TRUCK LOAD
LOADING DIRECTION ....: BOTH DIRECTIONS
VARAIBLE AXLE SPACING.............: 14
DEAD LOAD INCREASE FACTOR ....: 1
SUPERIMPOSED DEAD LOAD (K/ft) ....: .165
FATIQUE STRESS CYCLES ....: 2,000,000
AASHTO DESIGN METHOD ....: SERVICE LOAD DESIGN (WS)

IMPACT FACTORS:	SPAN	POS I	NEG I
	1	1.222	
			1.222
	2	1.222	

**** REACTION SUMMARY ****

JOINT FIXITY: (X=FIXED, O=FREE)

JT. NUM	X-DIR	Y-DIR	Z-DIR
1	X	X	O
2	O	X	O
3	O	X	O

VERTICAL REACTION COMPONENTS (Y-DIRECTION VALUES)

			(LL+I)		TOTAL (Kips)	
JT. NUM	--DL--	--SDL--	--MIN-	--MAX--	--MIN-	--MAX--
1	38.10	6.34	-6.38	65.68	38.07	110.13
2	144.17	20.98	0.00	139.27	165.14	304.41
3	38.10	6.34	-6.38	65.68	38.07	110.13

**** MAX. DEFLECTIONS ****

GIRDER	LOC.	--DL--	--SDL--	--LL--	TOTAL
1	40.43	1.50	.36	.75	2.61
2	59.57	1.50	.36	.75	2.61

**** SPANS MATRIX (FAB. COMP. #'S) ****

SPAN #	SPAN LENGTH (ft)	FC#1	FC#2	FC#3	FC#4
1	100.00	1	2	3	0
2	100.00	3	4	5	0

TOTAL NUMBER OF FABRICATED COMPONENTS = 5

**** FABRICATED COMPONENT DATA ****

1	FAB. COMP. NUMBER	=	1	16	THK. BOTT. FLG. -SEG. 2	=	1.625
2	FAB. COMP. TYPE	=	1	17	WID. BOTT. FLG. -SEG. 2	=	12
3	BEARING LENGTH (in)	=	12	18	WEB THICKNESS -SEG. 2	=	.3125
4	SEG.-1 LENGTH (L1-ft)	=	16	19	NUM. OF SIDES FOR STIFF.	=	1
5	SEG.-2 LENGTH (L2-ft)	=	24	20	STIFFENER THICKNESS (in)	=	.3125
6	START DEPTH (in)	=	50	21	STIFFENER WIDTH (in)	=	4
7	NEXT DEPTH (in)	=	50	22	FIRST STIFF. SPACE (in)	=	20
8	NEXT DEPTH (in)	=	50	23	STIFF. SPA'G SEG.-1 (in)	=	40
9	THK. TOP FLG. -SEG.1	=	.5	24	STIFF. SPA'G SEG.-2 (in)	=	50
10	WID. TOP FLG. -SEG.1	=	12	25	SHEAR STUD CODE	=	4
11	THK. BOTT. FLG. -SEG.1	=	.875	26	STUD SPA'G SEG.-1 (in)	=	18
12	WID. BOTT. FLG. -SEG.1	=	12	27	STUD SPA'G SEG.-2 (in)	=	24
13	WEB THICKNESS -SEG.1	=	.3125	28	NUM. OF BEAR'G STIFF.	=	1
14	THK. TOP FLG. -SEG.2	=	.5	29	BEAR'G STIFF. THK. (in)	=	.5
15	WID. TOP FLG. -SEG.2	=	12	30	BEAR'G STIFF. WID. (in)	=	5

1	FAB. COMP. NUMBER	=	2	16	THK. BOTT. FLG. -SEG. 2	=	.875
2	FAB. COMP. TYPE	=	2	17	WID. BOTT. FLG. -SEG. 2	=	16
3	BEARING LENGTH (in)	=	0	18	WEB THICKNESS -SEG. 2	=	.3125
4	SEG.-1 LENGTH (L1-ft)	=	24	19	NUM. OF SIDES FOR STIFF.	=	1
5	SEG.-2 LENGTH (L2-ft)	=	16	20	STIFFENER THICKNESS (in)	=	.3125
6	START DEPTH (in)	=	50	21	STIFFENER WIDTH (in)	=	4
7	NEXT DEPTH (in)	=	50	22	FIRST STIFF. SPACE (in)	=	0
8	NEXT DEPTH (in)	=	50	23	STIFF. SPA'G SEG.-1 (in)	=	50
9	THK. TOP FLG. -SEG.1	=	.5	24	STIFF. SPA'G SEG.-2 (in)	=	40
10	WID. TOP FLG. -SEG.1	=	12	25	SHEAR STUD CODE	=	4
11	THK. BOTT. FLG. -SEG.1	=	1.625	26	STUD SPA'G SEG.-1 (in)	=	24
12	WID. BOTT. FLG. -SEG.1	=	12	27	STUD SPA'G SEG.-2 (in)	=	24
13	WEB THICKNESS -SEG.1	=	.3125	28	NUM. OF BEAR'G STIFF.	=	0
14	THK. TOP FLG. -SEG.2	=	.875	29	BEAR'G STIFF. THK. (in)	=	0
15	WID. TOP FLG. -SEG.2	=	12	30	BEAR'G STIFF. WID. (in)	=	0

1	FAB. COMP. NUMBER	=	3	16	THK. BOTT. FLG. -SEG. 2	=	0
2	FAB. COMP. TYPE	=	3	17	WID. BOTT. FLG. -SEG. 2	=	0
3	BEARING LENGTH (in)	=	12	18	WEB THICKNESS -SEG. 2	=	0
4	SEG.-1 LENGTH (L1-ft)	=	20	19	NUM. OF SIDES FOR STIFF.	=	1
5	SEG.-2 LENGTH (L2-ft)	=	0	20	STIFFENER THICKNESS (in)	=	.3125
6	START DEPTH (in)	=	50	21	STIFFENER WIDTH (in)	=	4
7	NEXT DEPTH (in)	=	50	22	FIRST STIFF. SPACE (in)	=	0
8	NEXT DEPTH (in)	=	50	23	STIFF. SPA'G SEG.-1 (in)	=	30
9	THK. TOP FLG. -SEG.1	=	1.75	24	STIFF. SPA'G SEG.-2 (in)	=	0
10	WID. TOP FLG. -SEG.1	=	16	25	SHEAR STUD CODE	=	4
11	THK. BOTT. FLG. -SEG.1	=	1.75	26	STUD SPA'G SEG.-1 (in)	=	21
12	WID. BOTT. FLG. -SEG.1	=	16	27	STUD SPA'G SEG.-2 (in)	=	0
13	WEB THICKNESS -SEG.1	=	.3125	28	NUM. OF BEAR'G STIFF.	=	1
14	THK. TOP FLG. -SEG.2	=	0	29	BEAR'G STIFF. THK. (in)	=	1
15	WID. TOP FLG. -SEG.2	=	0	30	BEAR'G STIFF. WID. (in)	=	7

```
1  FAB. COMP. NUMBER           =  4
2  FAB. COMP. TYPE             =  2
3  BEARING LENGTH       (in)   =  0
4  SEG.-1 LENGTH     (L1-ft)   =  16
5  SEG.-2 LENGTH     (L2-ft)   =  24
6  START DEPTH          (in)   =  50
7  NEXT DEPTH           (in)   =  50
8  NEXT DEPTH           (in)   =  50
9  THK. TOP FLG.    -SEG.1     =  .875
10 WID. TOP FLG.    -SEG.1     =  12
11 THK. BOTT. FLG. -SEG.1      =  .875
12 WID. BOTT. FLG. -SEG.1      =  16
13 WEB THICKNESS    -SEG.1     =  .3125
14 THK. TOP FLG.    -SEG.2     =  .5
15 WID. TOP FLG.    -SEG.2     =  12
16 THK. BOTT. FLG. -SEG. 2     =  1.625
17 WID. BOTT. FLG. -SEG. 2     =  12
18 WEB THICKNESS    -SEG. 2    =  .3125
19 NUM. OF SIDES FOR STIFF.    =  1
20 STIFFENER THICKNESS (in)    =  .3125
21 STIFFENER WIDTH     (in)    =  4
22 FIRST STIFF. SPACE  (in)    =  0
23 STIFF. SPA'G SEG.-1 (in)    =  40
24 STIFF. SPA'G SEG.-2 (in)    =  50
25 SHEAR STUD CODE             =  4
26 STUD SPA'G   SEG.-1 (in)    =  24
27 STUD SPA'G   SEG.-2 (in)    =  24
28 NUM. OF BEAR'G STIFF.       =  0
29 BEAR'G STIFF. THK.  (in)    =  0
30 BEAR'G STIFF. WID.  (in)    =  0
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```
1  FAB. COMP. NUMBER           =  5
2  FAB. COMP. TYPE             =  1
3  BEARING LENGTH       (in)   =  12
4  SEG.-1 LENGTH     (L1-ft)   =  16
5  SEG.-2 LENGTH     (L2-ft)   =  24
6  START DEPTH          (in)   =  50
7  NEXT DEPTH           (in)   =  50
8  NEXT DEPTH           (in)   =  50
9  THK. TOP FLG.    -SEG.1     =  .5
10 WID. TOP FLG.    -SEG.1     =  12
11 THK. BOTT. FLG. -SEG.1      =  .875
12 WID. BOTT. FLG. -SEG.1      =  12
13 WEB THICKNESS    -SEG.1     =  .3125
14 THK. TOP FLG.    -SEG.2     =  .5
15 WID. TOP FLG.    -SEG.2     =  12
16 THK. BOTT. FLG. -SEG. 2     =  1.625
17 WID. BOTT. FLG. -SEG. 2     =  12
18 WEB THICKNESS    -SEG. 2    =  .3125
19 NUM. OF SIDES FOR STIFF.    =  1
20 STIFFENER THICKNESS (in)    =  .3125
21 STIFFENER WIDTH     (in)    =  4
22 FIRST STIFF. SPACE  (in)    =  20
23 STIFF. SPA'G SEG.-1 (in)    =  40
24 STIFF. SPA'G SEG.-2 (in)    =  50
25 SHEAR STUD CODE             =  4
26 STUD SPA'G   SEG.-1 (in)    =  18
27 STUD SPA'G   SEG.-2 (in)    =  24
28 NUM. OF BEAR'G STIFF.       =  1
29 BEAR'G STIFF. THK.  (in)    =  .5
30 BEAR'G STIFF. WID.  (in)    =  5
```

```
**************************
*                        *
*   GIRDER MEMBER # 1    *
*                        *
**************************
```

SPAN LENGTH (ft)= 100.00
INFLECTION POINTS (ft)=   0.00   68.58

**** GIRDER DESIGN PARAMETERS ****

AP	DIST(ft)	DEPTH(in)	D/Tw	B/Tf	r'	Lb	do	A_cstl
1	0.00	50.00	160.00	24.00	12.00	0.00	20.00	0.00
2	10.00	50.00	160.00	24.00	12.00	0.00	40.00	0.00
3	16.00	50.00	160.00	24.00	12.00	0.00	40.00	0.00
3	16.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
4	20.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
5	30.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
6	40.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
6	40.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
7	50.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
8	60.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
9	64.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
9	64.00	50.00	160.00	13.71	12.00	0.00	40.00	0.00
10	70.00	50.00	160.00	18.29	21.33	20.00	40.00	5.40
11	80.00	50.00	160.00	18.29	21.33	20.00	40.00	5.40
11	80.00	50.00	160.00	9.14	21.33	20.00	30.00	5.40
12	90.00	50.00	160.00	9.14	21.33	20.00	30.00	5.40
13	100.00	50.00	160.00	9.14	21.33	20.00	30.00	5.40

**** SECTION PROPERTIES, MOMENTS, AND SHEARS ****

ANAL. PT.	(ft) DIST	LOAD COND.	** SECTION MODUII (in^3) *** CONC.	TOP STEEL	BOTT. STEEL	MOMENTS (in-K) MAX	MIM	*** SHEARS (K)* MAX	(LL+I) RANGE
1	0.00	DL		462.6	604.2	-0.0		37.0	
		SDL	1552.7	2966.2	845.5	0.0		6.2	
		LL+I	3432.6	17568.8	905.9	0.0	-0.0	65.7	72.1
2	10.00	DL		462.6	604.2	3794.5		26.2	
		SDL	1552.7	2966.2	845.5	642.2		4.5	
		LL+I	3432.6	17568.8	905.9	6279.0	-765.0	56.8	67.2
3	16.00	DL		462.6	604.2	5449.0		19.7	
		SDL	1552.7	2966.2	845.5	932.4		3.5	
		LL+I	3432.6	17568.8	905.9	9304.7	-1224.0	51.6	65.1
3	16.00	DL		501.2	944.5	5449.0		19.7	
		SDL	1744.9	2851.3	1304.6	932.4		3.5	
		LL+I	4008.7	11241.2	1393.9	9304.7	-1224.0	51.6	65.1
4	20.00	DL		501.2	944.5	6292.7		15.4	
		SDL	1744.9	2851.3	1304.6	1086.3		2.9	
		LL+I	4008.7	11241.2	1393.9	11142.2	-1530.0	48.2	64.8
5	30.00	DL		501.2	944.5	7494.6		4.6	
		SDL	1744.9	2851.3	1304.6	1332.5		1.2	
		LL+I	4008.7	11241.2	1393.9	14314.8	-2295.0	39.7	64.4
6	40.00	DL		501.2	944.5	7400.2		-6.2	
		SDL	1744.9	2851.3	1304.6	1380.6		-.4	
		LL+I	4008.7	11241.2	1393.9	15551.8	-3060.0	-33.4	64.9
6	40.00	DL		501.2	944.5	7400.2		-6.2	
		SDL	1744.9	2851.3	1304.6	1380.6		-.4	
		LL+I	4008.7	11241.2	1393.9	15551.8	-3060.0	-33.4	64.9
7	50.00	DL		501.2	944.5	6009.4		-17.0	
		SDL	1744.9	2851.3	1304.6	1230.8		-2.1	
		LL+I	4008.7	11241.2	1393.9	15314.3	-3825.0	-41.8	65.5
8	60.00	DL		501.2	944.5	3322.4		-27.8	
		SDL	1744.9	2851.3	1304.6	882.9		-3.7	
		LL+I	4008.7	11241.2	1393.9	13766.5	-4590.1	-49.9	66.5
9	64.00	DL		501.2	944.5	1884.7		-32.1	
		SDL	1744.9	2851.3	1304.6	688.3		-4.4	
		LL+I	4008.7	11241.2	1393.9	12690.3	-4896.1	-53.1	67.1
9	64.00	DL		673.2	799.5	1884.7		-32.1	
		SDL	1736.0	3143.1	1033.0	688.3		-4.4	
		LL+I	3746.0	14013.6	1105.9	12690.3	-4896.1	-53.1	67.1
10	70.00	DL		673.2	799.5	-660.9		-38.6	
		SDL	815.1	1014.1	881.5	337.1		-5.4	
		LL+I	815.1	1014.1	881.5	10709.9	-5355.1	-57.7	68.0
11	80.00	DL		673.2	799.5	-5940.5		-49.4	
		SDL	815.1	1014.1	881.5	-406.8		-7.0	
		LL+I	815.1	1014.1	881.5	6602.1	-6120.1	-64.8	69.7
11	80.00	DL		1523.8	1523.8	-5940.5		-49.4	
		SDL	1519.8	1884.8	1587.9	-406.8		-7.0	
		LL+I	1519.8	1884.8	1587.9	6602.1	-6120.1	-64.8	69.7
12	90.00	DL		1523.8	1523.8	-12516.4		-60.2	
		SDL	1519.8	1884.8	1587.9	-1348.6		-8.7	
		LL+I	1519.8	1884.8	1587.9	2006.4	-9041.5	-70.9	71.6
13	100.00	DL		1523.8	1523.8	-20388.7		-71.0	
		SDL	1519.8	1884.8	1587.9	-2488.5		-10.3	
		LL+I	1519.8	1884.8	1587.9	0.0	-15300.2	-76.0	76.0

**** FLEXURAL STRESS SUMMARY ****

MOMENT CONDITION : POS. M = DL + SDL + POS.(LL+I)
NEG. M = DL + SDL + NEG.(LL+I)

FATIQUE STRESSES :
Tf_act = Actual Fatique Stress Range of Tension Flange
Tf_all = Allowable Fatique Stress Range of Tension Flange
SR_act = Actual Fatique Stress Range of eithen Stiffener or Rebar
SR_all = Allowable Fatique Stress Range of eithen Stiffener or Rebar

			** ACTUAL STRESSES (ksi) ***			ALLOWABLE ***		FATIQUE **	
ANAL. PT.	(ft) DIST	MOM. COND.	CONC./ -REBAR	TOP STL FLG	BOTT. STL FLG	Fb-ten Fb-comp	Fc-conc Ft-reb	Ft-act Ft-all	SR-act SR-all
1	0.00	POS. M	0.0	0.0	-0.0	27.5	1.6	0.0	0.0
		NEG. M	0.0	0.0	-0.0	-27.5	24.0	18.0	13.0
2	10.00	POS. M	-.2	-8.8	14.0	27.5	1.6	7.8	7.4
		NEG. M	0.0	-8.4	6.2	-27.5	24.0	18.0	13.0
3	16.00	POS. M	-.4	-12.6	20.4	27.5	1.6	11.6	11.1
		NEG. M	0.0	-12.0	8.8	-27.5	24.0	18.0	13.0
3	16.00	POS. M	-.3	-12.0	13.2	27.5	1.6	7.6	7.1
		NEG. M	0.0	-11.1	5.6	-27.5	24.0	18.0	13.0
4	20.00	POS. M	-.4	-13.9	15.5	27.5	1.6	9.1	8.5
		NEG. M	0.0	-12.8	6.4	-27.5	24.0	18.0	13.0
5	30.00	POS. M	-.5	-16.7	19.2	27.5	1.6	11.9	11.2
		NEG. M	0.0	-15.2	7.3	-27.5	24.0	18.0	13.0
6	40.00	POS. M	-.5	-16.6	20.0	27.5	1.6	13.4	12.5
		NEG. M	0.0	-15.0	6.7	-27.5	24.0	18.0	13.0
6	40.00	POS. M	-.5	-16.6	20.0	27.5	1.6	13.4	12.5
		NEG. M	0.0	-15.0	6.7	-27.5	24.0	18.0	13.0
7	50.00	POS. M	-.5	-13.8	18.3	27.5	1.6	13.7	12.9
		NEG. M	0.0	-12.1	4.6	-27.5	24.0	18.0	13.0
8	60.00	POS. M	-.5	-8.2	14.1	27.5	1.6	13.2	12.4
		NEG. M	0.0	-6.5	.9	-27.5	24.0	18.0	13.0
9	64.00	POS. M	-.4	-5.1	11.6	27.5	1.6	12.6	11.8
		NEG. M	0.0	-3.6	-1.0	-27.5	24.0	18.0	13.0
9	64.00	POS. M	-.4	-3.9	14.5	27.5	1.6	15.9	15.2
		NEG. M	0.0	-2.7	-1.4	-27.5	24.0	18.0	13.0
10	70.00	POS. M	0.0	-9.9	11.7	27.5	1.6	18.2	0.0
		NEG. M	6.2	5.9	-6.5	-24.3	24.0	18.0	20.0
11	80.00	POS. M	0.0	2.7	-.4	27.5	1.6	14.4	0.0
		NEG. M	8.0	15.3	-14.8	-24.3	24.0	18.0	20.0
11	80.00	POS. M	0.0	.6	0.0	27.5	1.6	8.0	0.0
		NEG. M	4.3	7.4	-8.0	-24.3	24.0	18.0	20.0
12	90.00	POS. M	0.0	7.9	-7.8	27.5	1.6	7.0	0.0
		NEG. M	6.8	13.7	-14.8	-24.3	24.0	18.0	20.0
13	100.00	POS. M	0.0	14.7	-14.9	27.4	1.6	9.6	0.0
		NEG. M	11.7	22.8	-24.6	-27.4	24.0	18.0	20.0

**** SHEARING STRESS SUMMARY ****

SHEAR CONDITION : TOTAL V = V(dl) + V(sdl) + V(LL+I)

	***		SHEAR STRESS		*****		STIFFENER		***** STUDS	**
ANAL.	(ft)	(ksi)	(ksi)	(ksi)	Tw-min		(in^2)	(in^4)	REQD	PROV
PT.	DIST	fv-act	Fv-all	.6Fv	Unstif	do	A_prov	I-prov	SPACE	SPACE
1	0.0	7.0	16.7	10.0	.557	20.0	-1.0	-1.0	13.0	18.0
2	10.0	5.6	13.7	8.2	.500	40.0	1.3	6.7	13.9	18.0
3	16.0	4.8	13.7	8.2	.462	40.0	1.3	6.7	14.4	18.0
3	16.0	4.8	12.5	7.5	.462	50.0	1.3	6.7	15.4	24.0
4	20.0	4.3	12.5	7.5	.435	50.0	1.3	6.7	15.4	24.0
5	30.0	2.9	12.5	7.5	.360	50.0	1.3	6.7	15.5	24.0
6	40.0	-2.6	12.5	7.5	.337	50.0	1.3	6.7	15.4	24.0
6	40.0	-2.6	12.5	7.5	.337	50.0	1.3	6.7	15.4	24.0
7	50.0	-3.9	12.5	7.5	.416	50.0	1.3	6.7	15.3	24.0
8	60.0	-5.2	12.5	7.5	.482	50.0	1.3	6.7	15.0	24.0
9	64.0	-5.7	12.5	7.5	.505	50.0	1.3	6.7	14.9	24.0
9	64.0	-5.7	13.7	8.2	.505	40.0	1.3	6.7	14.6	24.0
10	70.0	-6.5	13.7	8.2	.538	40.0	1.3	6.7	24.0	24.0
11	80.0	-7.8	13.7	8.2	.588	40.0	1.3	6.7	24.0	24.0
11	80.0	-7.8	15.2	9.1	.588	30.0	1.3	6.7	24.0	21.0
12	90.0	-8.9	15.2	9.1	.631	30.0	1.3	6.7	24.0	21.0
13	100.0	-10.1	16.7	10.0	.669	30.0	15.8	23.4	24.0	21.0

NOTE! - @ bearing locations : A_prov = actual bearing stress
I_prov = allowable bearing stress

MIM. TOTAL NUMBER OF SHEAR STUDS REQUIRED = 141
STUD DATA : NUMBER OF STUDS PER ROW = 3
STUD DIAMETER = .875
STUD HEIGHT = 4

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*                        *
*   GIRDER MEMBER # 2    *
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SPAN LENGTH (ft)= 100.00
INFLECTION POINTS (ft)= 31.42 100.00

**** GIRDER DESIGN PARAMETERS ****

AP	DIST(ft)	DEPTH(in)	D/Tw	B/Tf	r'	Lb	do	A_cstl
1	0.00	50.00	160.00	9.14	21.33	20.00	30.00	5.40
2	10.00	50.00	160.00	9.14	21.33	20.00	30.00	5.40
3	20.00	50.00	160.00	9.14	21.33	20.00	30.00	5.40
3	20.00	50.00	160.00	18.29	21.33	20.00	40.00	5.40
4	30.00	50.00	160.00	18.29	21.33	20.00	40.00	5.40
5	36.00	50.00	160.00	13.71	12.00	0.00	40.00	0.00
5	36.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
6	40.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
7	50.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
8	60.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
8	60.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
9	70.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
10	80.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
11	84.00	50.00	160.00	24.00	12.00	0.00	50.00	0.00
11	84.00	50.00	160.00	24.00	12.00	0.00	40.00	0.00
12	90.00	50.00	160.00	24.00	12.00	0.00	40.00	0.00
13	100.00	50.00	160.00	24.00	12.00	0.00	20.00	0.00

**** SECTION PROPERTIES, MOMENTS, AND SHEARS ****

ANAL. PT.	(ft) DIST	LOAD COND.	** SECTION MODUII (in^3) *** CONC.	TOP STEEL	BOTT. STEEL	MOMENTS (in-K) MAX	MIM	*** SHEARS (K)* MAX	(LL+I) RANGE
1	0.00	DL		1523.8	1523.8	-20388.7		71.0	
		SDL	1519.8	1884.8	1587.9	-2488.5		10.3	
		LL+I	1519.8	1884.8	1587.9	0.0	-15300.2	76.0	76.0
2	10.00	DL		1523.8	1523.8	-12516.4		60.2	
		SDL	1519.8	1884.8	1587.9	-1348.6		8.7	
		LL+I	1519.8	1884.8	1587.9	2006.4	-9041.5	70.9	71.6
3	20.00	DL		1523.8	1523.8	-5940.5		49.4	
		SDL	1519.8	1884.8	1587.9	-406.8		7.0	
		LL+I	1519.8	1884.8	1587.9	6602.1	-6120.1	64.8	69.7
3	20.00	DL		673.2	799.5	-5940.5		49.4	
		SDL	815.1	1014.1	881.5	-406.8		7.0	
		LL+I	815.1	1014.1	881.5	6602.1	-6120.1	64.8	69.7
4	30.00	DL		673.2	799.5	-660.9		38.6	
		SDL	815.1	1014.1	881.5	337.1		5.4	
		LL+I	815.1	1014.1	881.5	10709.9	-5355.1	57.7	68.0
5	36.00	DL		673.2	799.5	1884.7		32.1	
		SDL	1736.0	3143.1	1033.0	688.3		4.4	
		LL+I	3746.0	14013.6	1105.9	12690.3	-4896.1	53.1	67.1
5	36.00	DL		501.2	944.5	1884.7		32.1	
		SDL	1744.9	2851.3	1304.6	688.3		4.4	
		LL+I	4008.7	11241.2	1393.9	12690.3	-4896.1	53.1	67.1
6	40.00	DL		501.2	944.5	3322.4		27.8	
		SDL	1744.9	2851.3	1304.6	882.9		3.7	
		LL+I	4008.7	11241.2	1393.9	13766.5	-4590.1	49.9	66.5
7	50.00	DL		501.2	944.5	6009.4		17.0	
		SDL	1744.9	2851.3	1304.6	1230.8		2.1	
		LL+I	4008.7	11241.2	1393.9	15314.3	-3825.0	41.8	65.5
8	60.00	DL		501.2	944.5	7400.2		6.2	
		SDL	1744.9	2851.3	1304.6	1380.6		.4	
		LL+I	4008.7	11241.2	1393.9	15551.8	-3060.0	33.4	64.9
8	60.00	DL		501.2	944.5	7400.2		6.2	
		SDL	1744.9	2851.3	1304.6	1380.6		.4	
		LL+I	4008.7	11241.2	1393.9	15551.8	-3060.0	33.4	64.9
9	70.00	DL		501.2	944.5	7494.6		-4.6	
		SDL	1744.9	2851.3	1304.6	1332.5		-1.2	
		LL+I	4008.7	11241.2	1393.9	14314.8	-2295.0	-39.7	64.4
10	80.00	DL		501.2	944.5	6292.7		-15.4	
		SDL	1744.9	2851.3	1304.6	1086.3		-2.9	
		LL+I	4008.7	11241.2	1393.9	11142.2	-1530.0	-48.2	64.8
11	84.00	DL		501.2	944.5	5449.0		-19.7	
		SDL	1744.9	2851.3	1304.6	932.4		-3.5	
		LL+I	4008.7	11241.2	1393.9	9304.7	-1224.0	-51.6	65.1
11	84.00	DL		462.6	604.2	5449.0		-19.7	
		SDL	1552.7	2966.2	845.5	932.4		-3.5	
		LL+I	3432.6	17568.8	905.9	9304.7	-1224.0	-51.6	65.1
12	90.00	DL		462.6	604.2	3794.5		-26.2	
		SDL	1552.7	2966.2	845.5	642.2		-4.5	
		LL+I	3432.6	17568.8	905.9	6279.0	-765.0	-56.8	67.2
13	100.00	DL		462.6	604.2	-0.0		-37.0	
		SDL	1552.7	2966.2	845.5	-0.0		-6.2	
		LL+I	3432.6	17568.8	905.9	0.0	0.0	-65.7	72.1

**** FLEXURAL STRESS SUMMARY ****

MOMENT CONDITION : POS. M = DL + SDL + POS.(LL+I)
NEG. M = DL + SDL + NEG.(LL+I)

FATIQUE STRESSES :
Tf_act = Actual Fatique Stress Range of Tension Flange
Tf_all = Allowable Fatique Stress Range of Tension Flange
SR_act = Actual Fatique Stress Range of eithen Stiffener or Rebar
SR_all = Allowable Fatique Stress Range of eithen Stiffener or Rebar

			** ACTUAL STRESSES (ksi) ***			ALLOWABLE	***	FATIQUE	**
ANAL. PT.	(ft) DIST	MOM. COND.	CONC./ -REBAR	TOP STL FLG	BOTT. STL FLG	Fb-ten Fb-comp	Fc-conc Ft-reb	Ft-act Ft-all	SR-act SR-all
1	0.00	POS. M	0.0	14.7	-14.9	27.4	1.6	9.6	0.0
		NEG. M	11.7	22.8	-24.6	-27.4	24.0	18.0	20.0
2	10.00	POS. M	0.0	7.9	-7.8	27.5	1.6	7.0	0.0
		NEG. M	6.8	13.7	-14.8	-24.3	24.0	18.0	20.0
3	20.00	POS. M	0.0	.6	0.0	27.5	1.6	8.0	0.0
		NEG. M	4.3	7.4	-8.0	-24.3	24.0	18.0	20.0
3	20.00	POS. M	0.0	2.7	-.4	27.5	1.6	14.4	0.0
		NEG. M	8.0	15.3	-14.8	-24.3	24.0	18.0	20.0
4	30.00	POS. M	0.0	-9.9	11.7	27.5	1.6	18.2	0.0
		NEG. M	6.2	5.9	-6.5	-24.3	24.0	18.0	20.0
5	36.00	POS. M	-.4	-3.9	14.5	27.5	1.6	15.9	15.2
		NEG. M	0.0	-2.7	-1.4	-27.5	24.0	18.0	13.0
5	36.00	POS. M	-.4	-5.1	11.6	27.5	1.6	12.6	11.8
		NEG. M	0.0	-3.6	-1.0	-27.5	24.0	18.0	13.0
6	40.00	POS. M	-.5	-8.2	14.1	27.5	1.6	13.2	12.4
		NEG. M	0.0	-6.5	.9	-27.5	24.0	18.0	13.0
7	50.00	POS. M	-.5	-13.8	18.3	27.5	1.6	13.7	12.9
		NEG. M	0.0	-12.1	4.6	-27.5	24.0	18.0	13.0
8	60.00	POS. M	-.5	-16.6	20.0	27.5	1.6	13.4	12.5
		NEG. M	0.0	-15.0	6.7	-27.5	24.0	18.0	13.0
8	60.00	POS. M	-.5	-16.6	20.0	27.5	1.6	13.4	12.5
		NEG. M	0.0	-15.0	6.7	-27.5	24.0	18.0	13.0
9	70.00	POS. M	-.5	-16.7	19.2	27.5	1.6	11.9	11.2
		NEG. M	0.0	-15.2	7.3	-27.5	24.0	18.0	13.0
10	80.00	POS. M	-.4	-13.9	15.5	27.5	1.6	9.1	8.5
		NEG. M	0.0	-12.8	6.4	-27.5	24.0	18.0	13.0
11	84.00	POS. M	-.3	-12.0	13.2	27.5	1.6	7.6	7.1
		NEG. M	0.0	-11.1	5.6	-27.5	24.0	18.0	13.0
11	84.00	POS. M	-.4	-12.6	20.4	27.5	1.6	11.6	11.1
		NEG. M	0.0	-12.0	8.8	-27.5	24.0	18.0	13.0
12	90.00	POS. M	-.2	-8.8	14.0	27.5	1.6	7.8	7.4
		NEG. M	0.0	-8.4	6.2	-27.5	24.0	18.0	13.0
13	100.00	POS. M	0.0	0.0	-0.0	27.5	1.6	0.0	0.0
		NEG. M	0.0	0.0	-0.0	-27.5	24.0	18.0	13.0

**** SHEARING STRESS SUMMARY ****

SHEAR CONDITION : TOTAL V = V(dl) + V(sdl) + V(LL+I)

	***		SHEAR STRESS		*****		STIFFENER	*****	STUDS	**
ANAL. PT.	(ft) DIST	(ksi) fv-act	(ksi) Fv-all	(ksi) .6Fv	Tw-min Unstif	do	(in^2) A_prov	(in^4) I-prov	REQD SPACE	PROV SPACE
1	0.0	10.1	16.7	10.0	.669	30.0	15.8	23.4	24.0	21.0
2	10.0	8.9	15.2	9.1	.631	30.0	1.3	6.7	24.0	21.0
3	20.0	7.8	15.2	9.1	.588	30.0	1.3	6.7	24.0	21.0
3	20.0	7.8	13.7	8.2	.588	40.0	1.3	6.7	24.0	24.0
4	30.0	6.5	13.7	8.2	.538	40.0	1.3	6.7	24.0	24.0
5	36.0	5.7	13.7	8.2	.505	40.0	1.3	6.7	14.6	24.0
5	36.0	5.7	12.5	7.5	.505	50.0	1.3	6.7	14.9	24.0
6	40.0	5.2	12.5	7.5	.482	50.0	1.3	6.7	15.0	24.0
7	50.0	3.9	12.5	7.5	.416	50.0	1.3	6.7	15.3	24.0
8	60.0	2.6	12.5	7.5	.337	50.0	1.3	6.7	15.4	24.0
8	60.0	2.6	12.5	7.5	.337	50.0	1.3	6.7	15.4	24.0
9	70.0	-2.9	12.5	7.5	.360	50.0	1.3	6.7	15.5	24.0
10	80.0	-4.3	12.5	7.5	.435	50.0	1.3	6.7	15.4	24.0
11	84.0	-4.8	12.5	7.5	.462	50.0	1.3	6.7	15.4	24.0
11	84.0	-4.8	13.7	8.2	.462	40.0	1.3	6.7	14.4	18.0
12	90.0	-5.6	13.7	8.2	.500	40.0	1.3	6.7	13.9	18.0
13	100.0	-7.0	16.7	10.0	.557	20.0	-1.0	-1.0	13.0	18.0

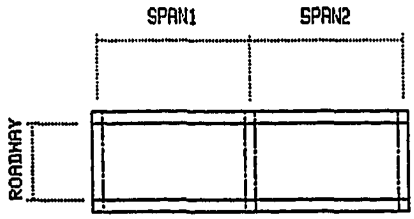
NOTE! - @ bearing locations : A_prov = actual bearing stress
I_prov = allowable bearing stress

MIM. TOTAL NUMBER OF SHEAR STUDS REQUIRED = 151
STUD DATA : NUMBER OF STUDS PER ROW = 3
STUD DIAMETER = .875
STUD HEIGHT = 4

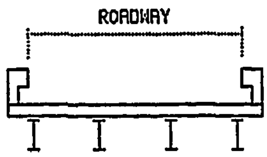
GBRIDGE COST REPORT

BRIDGE FILE NAME :LFD_2SPAN3
REPORT DATE :

**** BRIDGE SYSTEM ****



BRIDGE PLAN



X-SECT

**** COST SUMMARY ****

COST PER GIRDER (for entire bridge lenth) =$	15754.32
NUMBER OF GIRDERS =	4
TOTAL WEIGHT OF SINGLE GIRDER (lbs) =	26730.80
COST OF ROADWAY (for entire bridge length) =$	69720.12
TOTAL WEIGHT OF SLAB REBAR (lbs) =	49037.89

TOTAL BRIDGE SUPERSTRUCTURE COST =$ 132737.40

GBRIDGE STRESS REPORT

BRIDGE FILE NAME :LFD_2SPAN3
REPORT DATE :

**** GIRDER TYPE/ BRIDGE GEOMETRY ****

GIRDER TYPE ....: COMPOSITE GIRDER - for (+)&(-) MOM.
GIRDER LOCATION ....: INTERIOR GIRDER
LENGTH TO SYMM. (ft) (0=Unsymm.)..: 100
NUMBER OF SPANS ....: 2
NUMBER OF GIRDERS.................: 4
GIRDERS SPACING (ft)..............: 9.25
TRAFFIC ROADWAY WIDTH (ft)........: 28
TOTAL ROADWAY WIDTH (ft)..........: 30
TOTAL ROADWAY SLAB THCK'S (in)....: 8
ROADWAY HAUNCH (in)...............: 2.375
AREA OF CONC. RE-BAR (in^2).......: 5.4

**** MATERIAL PROPERTIES ****

MODULUS OF ELASTICTY (ksi)........: 29000
YIELD STRENGTH OF STL (ksi).......: 50
ULTIMATE STL STRENGTH (ksi).......: 65
CONC. COMP. STRENGTH (F'c-ksi)....: 4
MODULAR RATIO (Es/Ec).............: 8
RE-BAR YIELD STRENGTH.............: 60

**** DESIGN LOADS/ AASHTO DESIGN METHOD ****

LOADING TYPE ....: HS20-44 TRUCK LOAD
LOADING DIRECTION ....: BOTH DIRECTIONS
VARAIBLE AXLE SPACING.............: 14
DEAD LOAD INCREASE FACTOR ....: 1
SUPERIMPOSED DEAD LOAD (K/ft) ....: .165
FATIQUE STRESS CYCLES ....: 2,000,000
AASHTO DESIGN METHOD ....: LOAD FACTOR DESIGN (LFD)

IMPACT FACTORS:	SPAN	POS I	NEG I
	1	1.222	
			1.222
	2	1.222	

**** REACTION SUMMARY ****

JOINT FIXITY: (X=FIXED, O=FREE)

JT. NUM	X-DIR	Y-DIR	Z-DIR
1	X	X	O
2	O	X	O
3	O	X	O

VERTICAL REACTION COMPONENTS (Y-DIRECTION VALUES)

			(LL+I)		TOTAL (Kips)	
JT. NUM	--DL--	--SDL--	--MIN-	--MAX--	--MIN-	--MAX--
1	38.31	6.46	-5.96	65.75	38.81	110.52
2	139.12	20.75	0.00	139.03	159.87	298.91
3	38.31	6.46	-5.96	65.75	38.81	110.52

**** MAX. DEFLECTIONS ****

GIRDER	LOC.	--DL--	--SDL--	--LL--	TOTAL
1	40.53	1.67	.37	.76	2.81
2	59.47	1.67	.37	.76	2.81

**** SPANS MATRIX (FAB. COMP. #'S) ****

SPAN #	SPAN LENGTH (ft)	FC#1	FC#2	FC#3	FC#4
1	100.00	1	2	3	0
2	100.00	3	4	5	0

TOTAL NUMBER OF FABRICATED COMPONENTS = 5

**** FABRICATED COMPONENT DATA ****

1 FAB. COMP. NUMBER	= 1	16 THK. BOTT. FLG. -SEG. 2	= 1.25
2 FAB. COMP. TYPE	= 1	17 WID. BOTT. FLG. -SEG. 2	= 12
3 BEARING LENGTH (in)	= 12	18 WEB THICKNESS -SEG. 2	= .3125
4 SEG.-1 LENGTH (L1-ft)	= 16	19 NUM. OF SIDES FOR STIFF.	= 1
5 SEG.-2 LENGTH (L2-ft)	= 24	20 STIFFENER THICKNESS (in)	= .3125
6 START DEPTH (in)	= 50	21 STIFFENER WIDTH (in)	= 4
7 NEXT DEPTH (in)	= 50	22 FIRST STIFF. SPACE (in)	= 20
8 NEXT DEPTH (in)	= 50	23 STIFF. SPA'G SEG.-1 (in)	= 75
9 THK. TOP FLG. -SEG.1	= .5	24 STIFF. SPA'G SEG.-2 (in)	= 75
10 WID. TOP FLG. -SEG.1	= 12	25 SHEAR STUD CODE	= 4
11 THK. BOTT. FLG. -SEG.1	= .5	26 STUD SPA'G SEG.-1 (in)	= 18
12 WID. BOTT. FLG. -SEG.1	= 12	27 STUD SPA'G SEG.-2 (in)	= 24
13 WEB THICKNESS -SEG.1	= .3125	28 NUM. OF BEAR'G STIFF.	= 1
14 THK. TOP FLG. -SEG.2	= .5	29 BEAR'G STIFF. THK. (in)	= .75
15 WID. TOP FLG. -SEG.2	= 12	30 BEAR'G STIFF. WID. (in)	= 5

1 FAB. COMP. NUMBER	= 2	16 THK. BOTT. FLG. -SEG. 2	= 1.25
2 FAB. COMP. TYPE	= 2	17 WID. BOTT. FLG. -SEG. 2	= 12
3 BEARING LENGTH (in)	= 0	18 WEB THICKNESS -SEG. 2	= .3125
4 SEG.-1 LENGTH (L1-ft)	= 24	19 NUM. OF SIDES FOR STIFF.	= 1
5 SEG.-2 LENGTH (L2-ft)	= 16	20 STIFFENER THICKNESS (in)	= .3125
6 START DEPTH (in)	= 50	21 STIFFENER WIDTH (in)	= 4
7 NEXT DEPTH (in)	= 50	22 FIRST STIFF. SPACE (in)	= 0
8 NEXT DEPTH (in)	= 50	23 STIFF. SPA'G SEG.-1 (in)	= 75
9 THK. TOP FLG. -SEG.1	= .5	24 STIFF. SPA'G SEG.-2 (in)	= 75
10 WID. TOP FLG. -SEG.1	= 12	25 SHEAR STUD CODE	= 4
11 THK. BOTT. FLG. -SEG.1	= 1.25	26 STUD SPA'G SEG.-1 (in)	= 24
12 WID. BOTT. FLG. -SEG.1	= 12	27 STUD SPA'G SEG.-2 (in)	= 24
13 WEB THICKNESS -SEG.1	= .3125	28 NUM. OF BEAR'G STIFF.	= 0
14 THK. TOP FLG. -SEG.2	= .5	29 BEAR'G STIFF. THK. (in)	= 0
15 WID. TOP FLG. -SEG.2	= 12	30 BEAR'G STIFF. WID. (in)	= 0

1 FAB. COMP. NUMBER	= 3	16 THK. BOTT. FLG. -SEG. 2	= 0
2 FAB. COMP. TYPE	= 3	17 WID. BOTT. FLG. -SEG. 2	= 0
3 BEARING LENGTH (in)	= 12	18 WEB THICKNESS -SEG. 2	= 0
4 SEG.-1 LENGTH (L1-ft)	= 20	19 NUM. OF SIDES FOR STIFF.	= 1
5 SEG.-2 LENGTH (L2-ft)	= 0	20 STIFFENER THICKNESS (in)	= .3125
6 START DEPTH (in)	= 50	21 STIFFENER WIDTH (in)	= 4
7 NEXT DEPTH (in)	= 50	22 FIRST STIFF. SPACE (in)	= 0
8 NEXT DEPTH (in)	= 50	23 STIFF. SPA'G SEG.-1 (in)	= 40
9 THK. TOP FLG. -SEG.1	= 1.25	24 STIFF. SPA'G SEG.-2 (in)	= 0
10 WID. TOP FLG. -SEG.1	= 14	25 SHEAR STUD CODE	= 4
11 THK. BOTT. FLG. -SEG.1	= 1.375	26 STUD SPA'G SEG.-1 (in)	= 21
12 WID. BOTT. FLG. -SEG.1	= 14	27 STUD SPA'G SEG.-2 (in)	= 0
13 WEB THICKNESS -SEG.1	= .375	28 NUM. OF BEAR'G STIFF.	= 1
14 THK. TOP FLG. -SEG.2	= 0	29 BEAR'G STIFF. THK. (in)	= 1
15 WID. TOP FLG. -SEG.2	= 0	30 BEAR'G STIFF. WID. (in)	= 7

1	FAB. COMP. NUMBER	=	4
2	FAB. COMP. TYPE	=	2
3	BEARING LENGTH (in)	=	0
4	SEG.-1 LENGTH (L1-ft)	=	16
5	SEG.-2 LENGTH (L2-ft)	=	24
6	START DEPTH (in)	=	50
7	NEXT DEPTH (in)	=	50
8	NEXT DEPTH (in)	=	50
9	THK. TOP FLG. -SEG.1	=	.5
10	WID. TOP FLG. -SEG.1	=	12
11	THK. BOTT. FLG. -SEG.1	=	1.25
12	WID. BOTT. FLG. -SEG.1	=	12
13	WEB THICKNESS -SEG.1	=	.3125
14	THK. TOP FLG. -SEG.2	=	.5
15	WID. TOP FLG. -SEG.2	=	12
16	THK. BOTT. FLG. -SEG. 2	=	1.25
17	WID. BOTT. FLG. -SEG. 2	=	12
18	WEB THICKNESS -SEG. 2	=	.3125
19	NUM. OF SIDES FOR STIFF.	=	1
20	STIFFENER THICKNESS (in)	=	.3125
21	STIFFENER WIDTH (in)	=	4
22	FIRST STIFF. SPACE (in)	=	0
23	STIFF. SPA'G SEG.-1 (in)	=	75
24	STIFF. SPA'G SEG.-2 (in)	=	75
25	SHEAR STUD CODE	=	4
26	STUD SPA'G SEG.-1 (in)	=	24
27	STUD SPA'G SEG.-2 (in)	=	24
28	NUM. OF BEAR'G STIFF.	=	0
29	BEAR'G STIFF. THK. (in)	=	0
30	BEAR'G STIFF. WID. (in)	=	0

1	FAB. COMP. NUMBER	=	5
2	FAB. COMP. TYPE	=	1
3	BEARING LENGTH (in)	=	12
4	SEG.-1 LENGTH (L1-ft)	=	16
5	SEG.-2 LENGTH (L2-ft)	=	24
6	START DEPTH (in)	=	50
7	NEXT DEPTH (in)	=	50
8	NEXT DEPTH (in)	=	50
9	THK. TOP FLG. -SEG.1	=	.5
10	WID. TOP FLG. -SEG.1	=	12
11	THK. BOTT. FLG. -SEG.1	=	.5
12	WID. BOTT. FLG. -SEG.1	=	12
13	WEB THICKNESS -SEG.1	=	.3125
14	THK. TOP FLG. -SEG.2	=	.5
15	WID. TOP FLG. -SEG.2	=	12
16	THK. BOTT. FLG. -SEG. 2	=	1.25
17	WID. BOTT. FLG. -SEG. 2	=	12
18	WEB THICKNESS -SEG. 2	=	.3125
19	NUM. OF SIDES FOR STIFF.	=	1
20	STIFFENER THICKNESS (in)	=	.3125
21	STIFFENER WIDTH (in)	=	4
22	FIRST STIFF. SPACE (in)	=	20
23	STIFF. SPA'G SEG.-1 (in)	=	75
24	STIFF. SPA'G SEG.-2 (in)	=	75
25	SHEAR STUD CODE	=	4
26	STUD SPA'G SEG.-1 (in)	=	18
27	STUD SPA'G SEG.-2 (in)	=	24
28	NUM. OF BEAR'G STIFF.	=	1
29	BEAR'G STIFF. THK. (in)	=	.75
30	BEAR'G STIFF. WID. (in)	=	5

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*                        *
*   GIRDER MEMBER # 1    *
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SPAN LENGTH (ft)= 100.00
INFLECTION POINTS (ft)=   0.00   70.42

**** GIRDER DESIGN PARAMETERS ****

AP	DIST(ft)	DEPTH(in)	D/Tw	b'/Tf	Af/FyD	Lb	do	A_cstl
1	0.00	50.00	160.00	11.69	0.00	0.00	20.00	0.00
2	10.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
3	16.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
3	16.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
4	20.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
5	30.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
6	40.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
6	40.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
7	50.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
8	60.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
9	64.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
9	64.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
10	70.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
11	80.00	50.00	160.00	4.68	.01	20.00	75.00	5.40
11	80.00	50.00	133.33	4.95	.01	20.00	40.00	5.40
12	90.00	50.00	133.33	4.95	.01	20.00	40.00	5.40
13	100.00	50.00	133.33	4.95	.01	20.00	40.00	5.40

**** FLEXURAL STRESS SUMMARY ****

MOMENT CONDITION : POS. M =[DL + SDL + 5/3*POS.(LL+I)]
NEG. M =[DL + SDL + 5/3*NEG.(LL+I)]

FATIQUE STRESSES :
Tf_act = Actual Fatique Stress Range of Tension Flange
Tf_all = Allowable Fatique Stress Range of Tension Flange
SR_act = Actual Fatique Stress Range of eithen Stiffener or Rebar
SR_all = Allowable Fatique Stress Range of eithen Stiffener or Rebar

			** FACTORED STRESSES (ksi) ***			ALLOWABLE ***		FATIQUE **	
ANAL. PT.	(ft) DIST	MOM. COND.	CONC./ -REBAR	TOP STL FLG	BOTT. STL FLG	Fb-ten Fb-comp	Fc-conc Ft-reb	Ft-act Ft-all	SR-act SR-all
1	0.00	POS. M	-0.0	-0.0	0.0	50.0	3.4	0.0	0.0
		NEG. M	0.0	0.0	0.0	-50.0	60.0	18.0	13.0
2	10.00	POS. M	-.6	-12.2	33.7	50.0	3.4	10.6	10.2
		NEG. M	0.0	-11.9	10.7	-50.0	60.0	18.0	13.0
3	16.00	POS. M	-.9	-17.7	49.4	50.0	3.4	15.9	15.3
		NEG. M	0.0	-17.1	15.1	-50.0	60.0	18.0	13.0
3	16.00	POS. M	-.7	-16.8	28.0	50.0	3.4	9.1	8.6
		NEG. M	0.0	-15.1	8.3	-50.0	60.0	18.0	13.0
4	20.00	POS. M	-.8	-19.5	33.1	50.0	3.4	11.0	10.4
		NEG. M	0.0	-17.4	9.4	-50.0	60.0	18.0	13.0
5	30.00	POS. M	-1.1	-23.6	41.7	50.0	3.4	14.4	13.6
		NEG. M	0.0	-20.9	10.5	-50.0	60.0	18.0	13.0
6	40.00	POS. M	-1.2	-24.0	44.3	50.0	3.4	16.1	15.3
		NEG. M	0.0	-20.9	9.3	-50.0	60.0	18.0	13.0
6	40.00	POS. M	-1.2	-24.0	44.3	50.0	3.4	16.1	15.3
		NEG. M	0.0	-20.9	9.3	-50.0	60.0	18.0	13.0
7	50.00	POS. M	-1.2	-20.5	41.7	50.0	3.4	16.6	15.7
		NEG. M	0.0	-17.4	5.7	-50.0	60.0	18.0	13.0
8	60.00	POS. M	-1.0	-13.4	34.3	50.0	3.4	15.9	15.1
		NEG. M	0.0	-10.4	-.3	-50.0	60.0	18.0	13.0
9	64.00	POS. M	-1.0	-9.5	29.8	50.0	3.4	15.3	14.5
		NEG. M	0.0	-6.6	-3.3	-50.0	60.0	18.0	13.0
9	64.00	POS. M	-1.0	-9.5	29.8	50.0	3.4	15.3	14.5
		NEG. M	0.0	-6.6	-3.3	-50.0	60.0	18.0	13.0
10	70.00	POS. M	-.8	-2.6	21.7	49.6	3.4	14.0	13.2
		NEG. M	0.0	.1	-8.6	-49.6	60.0	18.0	13.0
11	80.00	POS. M	0.0	-4.8	8.2	46.2	3.4	14.1	0.0
		NEG. M	19.0	28.6	-22.4	-46.2	60.0	18.0	20.0
11	80.00	POS. M	0.0	-4.5	6.7	50.0	3.4	10.8	0.0
		NEG. M	11.5	15.3	-16.6	-50.0	60.0	18.0	20.0
12	90.00	POS. M	0.0	11.7	-10.5	50.0	3.4	8.9	0.0
		NEG. M	17.4	28.0	-29.7	-50.0	60.0	18.0	20.0
13	100.00	POS. M	0.0	25.7	-24.8	50.0	3.4	12.2	0.0
		NEG. M	30.6	48.1	-51.2	-50.0	60.0	18.0	20.0

**** SHEARING STRESS SUMMARY ****

SHEAR CONDITION : TOTAL V = V(dl) + V(sdl) + V(LL+I) .

	***		SHEAR STRESS		*****		STIFFENER		***** STUDS	**
ANAL.	(ft)	(ksi)	(ksi)	(ksi)	Tw-min		(in^2)	(in^4)	REQD	PROV
PT.	DIST	fv-act	Fv-all	.6Fv	Unstif	do	A_prov	I-prov	SPACE	SPACE
1	0.0	7.0	16.7	10.0	.558	20.0	9.3	23.2	12.2	18.0
2	10.0	5.7	10.2	6.1	.502	75.0	1.3	6.7	13.1	18.0
3	16.0	4.8	10.2	6.1	.465	75.0	1.3	6.7	13.5	18.0
3	16.0	4.8	10.2	6.1	.465	75.0	1.3	6.7	15.0	24.0
4	20.0	4.3	10.2	6.1	.438	75.0	1.3	6.7	15.1	24.0
5	30.0	3.0	10.2	6.1	.365	75.0	1.3	6.7	15.2	24.0
6	40.0	-2.4	10.2	6.1	.329	75.0	1.3	6.7	15.1	24.0
6	40.0	-2.4	10.2	6.1	.329	75.0	1.3	6.7	15.1	24.0
7	50.0	-3.8	10.2	6.1	.409	75.0	1.3	6.7	14.9	24.0
8	60.0	-5.1	10.2	6.1	.474	75.0	1.3	6.7	14.7	24.0
9	64.0	-5.6	10.2	6.1	.498	75.0	1.3	6.7	14.6	24.0
9	64.0	-5.6	10.2	6.1	.498	75.0	1.3	6.7	14.6	24.0
10	70.0	-6.3	10.2	6.1	.531	75.0	1.3	6.7	14.4	24.0
11	80.0	-7.6	10.2	6.1	.580	75.0	1.3	6.7	24.0	24.0
11	80.0	-6.3	14.7	8.8	.530	40.0	1.3	6.7	24.0	21.0
12	90.0	-7.3	14.7	8.8	.570	40.0	1.3	6.7	24.0	21.0
13	100.0	-8.2	16.7	10.0	.605	40.0	16.5	23.4	24.0	21.0

NOTE! - @ bearing locations : A_prov = actual bearing stress
I_prov = allowable bearing stress

MIM. TOTAL NUMBER OF SHEAR STUDS REQUIRED = 126
STUD DATA : NUMBER OF STUDS PER ROW = 3
STUD DIAMETER = .875
STUD HEIGHT = 4

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*                       *
*   GIRDER MEMBER # 2   *
*                       *
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SPAN LENGTH (ft)= 100.00
INFLECTION POINTS (ft)= 29.58 100.00

**** GIRDER DESIGN PARAMETERS ****

AP	DIST(ft)	DEPTH(in)	D/Tw	b'/Tf	Af/FyD	Lb	do	A_cstl
1	0.00	50.00	133.33	4.95	.01	20.00	40.00	5.40
2	10.00	50.00	133.33	4.95	.01	20.00	40.00	5.40
3	20.00	50.00	133.33	4.95	.01	20.00	40.00	5.40
3	20.00	50.00	160.00	4.68	.01	20.00	75.00	5.40
4	30.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
5	36.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
5	36.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
6	40.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
7	50.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
8	60.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
8	60.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
9	70.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
10	80.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
11	84.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
11	84.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
12	90.00	50.00	160.00	11.69	0.00	0.00	75.00	0.00
13	100.00	50.00	160.00	11.69	0.00	0.00	20.00	0.00

**** SECTION PROPERTIES, MOMENTS, AND SHEARS ****

			** SECTION	MODULI (in^3)	***	MOMENTS		*** SHEARS	(K)*
ANAL.	(ft)	LOAD		TOP	BOTT.	(in-K)			(LL+I)
PT.	DIST	COND.	CONC.	STEEL	STEEL	MAX	MIM	MAX	RANGE
1	0.00	DL		1036.4	1098.7	-18750.5		68.5	
		SDL	1112.5	1383.0	1173.7	-2350.5		10.2	
		LL+I	1112.5	1383.0	1173.7	0.0	-14296.3	75.5	75.5
2	10.00	DL		1036.4	1098.7	-11164.6		57.9	
		SDL	1112.5	1383.0	1173.7	-1224.4		8.6	
		LL+I	1112.5	1383.0	1173.7	2217.1	-8216.9	70.2	71.0
3	20.00	DL		1036.4	1098.7	-4847.7		47.4	
		SDL	1112.5	1383.0	1173.7	-296.4		6.9	
		LL+I	1112.5	1383.0	1173.7	6929.1	-5718.5	64.0	69.1
3	20.00	DL		485.0	776.8	-4847.7		47.4	
		SDL	673.6	820.2	895.4	-296.4		6.9	
		LL+I	673.6	820.2	895.4	6929.1	-5718.5	64.0	69.1
4	30.00	DL		485.0	776.8	200.1		36.8	
		SDL	1663.1	2892.9	1076.0	433.7		5.3	
		LL+I	3758.5	13079.8	1150.3	11067.0	-5003.7	56.8	67.4
5	36.00	DL		485.0	776.8	2619.6		30.4	
		SDL	1663.1	2892.9	1076.0	776.6		4.3	
		LL+I	3758.5	13079.8	1150.3	13011.5	-4574.8	52.3	66.6
5	36.00	DL		485.0	776.8	2619.6		30.4	
		SDL	1663.1	2892.9	1076.0	776.6		4.3	
		LL+I	3758.5	13079.8	1150.3	13011.5	-4574.8	52.3	66.6
6	40.00	DL		485.0	776.8	3978.7		26.2	
		SDL	1663.1	2892.9	1076.0	965.7		3.6	
		LL+I	3758.5	13079.8	1150.3	14057.3	-4288.9	49.1	66.1
7	50.00	DL		485.0	776.8	6488.3		15.6	
		SDL	1663.1	2892.9	1076.0	1299.8		2.0	
		LL+I	3758.5	13079.8	1150.3	15524.3	-3574.1	41.1	65.2
8	60.00	DL		485.0	776.8	7728.8		5.0	
		SDL	1663.1	2892.9	1076.0	1435.8		.3	
		LL+I	3758.5	13079.8	1150.3	15717.5	-2859.3	32.7	64.6
8	60.00	DL		485.0	776.8	7728.8		5.0	
		SDL	1663.1	2892.9	1076.0	1435.8		.3	
		LL+I	3758.5	13079.8	1150.3	15717.5	-2859.3	32.7	64.6
9	70.00	DL		485.0	776.8	7700.3		-5.5	
		SDL	1663.1	2892.9	1076.0	1373.9		-1.3	
		LL+I	3758.5	13079.8	1150.3	14412.1	-2144.4	-40.0	64.1
10	80.00	DL		485.0	776.8	6402.6		-16.1	
		SDL	1663.1	2892.9	1076.0	1113.9		-3.0	
		LL+I	3758.5	13079.8	1150.3	11188.2	-1429.6	-48.4	64.6
11	84.00	DL		485.0	776.8	5528.2		-20.3	
		SDL	1663.1	2892.9	1076.0	954.5		-3.7	
		LL+I	3758.5	13079.8	1150.3	9335.4	-1143.7	-51.8	64.8
11	84.00	DL		427.7	427.7	5528.2		-20.3	
		SDL	1392.8	3114.5	613.5	954.5		-3.7	
		LL+I	2986.1	44419.7	660.9	9335.4	-1143.7	-51.8	64.8
12	90.00	DL		427.7	427.7	3835.8		-26.7	
		SDL	1392.8	3114.5	613.5	656.0		-4.6	
		LL+I	2986.1	44419.7	660.9	6292.8	-714.8	-57.0	66.9
13	100.00	DL		427.7	427.7	-0.0		-37.3	
		SDL	1392.8	3114.5	613.5	-0.0		-6.3	
		LL+I	2986.1	44419.7	660.9	0.0	0.0	-65.8	71.7

**** FLEXURAL STRESS SUMMARY ****

MOMENT CONDITION : POS. M =[DL + SDL + 5/3*POS.(LL+I)]
NEG. M =[DL + SDL + 5/3*NEG.(LL+I)]

FATIQUE STRESSES :
Tf_act = Actual Fatique Stress Range of Tension Flange
Tf_all = Allowable Fatique Stress Range of Tension Flange
SR_act = Actual Fatique Stress Range of eithen Stiffener or Rebar
SR_all = Allowable Fatique Stress Range of eithen Stiffener or Rebar

			** FACTORED STRESSES (ksi)			*** ALLOWABLE	***	FATIQUE	**
ANAL. PT.	(ft) DIST	MOM. COND.	CONC./ -REBAR	TOP STL FLG	BOTT. STL FLG	Fb-ten Fb-comp	Fc-conc Ft-reb	Ft-act Ft-all	SR-act SR-all
1	0.00	POS. M	0.0	25.7	-24.8	50.0	3.4	12.2	0.0
		NEG. M	30.6	48.1	-51.2	-50.0	60.0	18.0	20.0
2	10.00	POS. M	0.0	11.7	-10.5	50.0	3.4	8.9	0.0
		NEG. M	17.4	28.0	-29.7	-50.0	60.0	18.0	20.0
3	20.00	POS. M	0.0	-4.5	6.7	50.0	3.4	10.8	0.0
		NEG. M	11.5	15.3	-16.6	-50.0	60.0	18.0	20.0
3	20.00	POS. M	0.0	-4.8	8.2	46.2	3.4	14.1	0.0
		NEG. M	19.0	28.6	-22.4	-46.2	60.0	18.0	20.0
4	30.00	POS. M	-.8	-2.6	21.7	49.6	3.4	14.0	13.2
		NEG. M	0.0	.1	-8.6	-49.6	60.0	18.0	13.0
5	36.00	POS. M	-1.0	-9.5	29.8	50.0	3.4	15.3	14.5
		NEG. M	0.0	-6.6	-3.3	-50.0	60.0	18.0	13.0
5	36.00	POS. M	-1.0	-9.5	29.8	50.0	3.4	15.3	14.5
		NEG. M	0.0	-6.6	-3.3	-50.0	60.0	18.0	13.0
6	40.00	POS. M	-1.0	-13.4	34.3	50.0	3.4	15.9	15.1
		NEG. M	0.0	-10.4	-.3	-50.0	60.0	18.0	13.0
7	50.00	POS. M	-1.2	-20.5	41.7	50.0	3.4	16.6	15.7
		NEG. M	0.0	-17.4	5.7	-50.0	60.0	18.0	13.0
8	60.00	POS. M	-1.2	-24.0	44.3	50.0	3.4	16.1	15.3
		NEG. M	0.0	-20.9	9.3	-50.0	60.0	18.0	13.0
8	60.00	POS. M	-1.2	-24.0	44.3	50.0	3.4	16.1	15.3
		NEG. M	0.0	-20.9	9.3	-50.0	60.0	18.0	13.0
9	70.00	POS. M	-1.1	-23.6	41.7	50.0	3.4	14.4	13.6
		NEG. M	0.0	-20.9	10.5	-50.0	60.0	18.0	13.0
10	80.00	POS. M	-.8	-19.5	33.1	50.0	3.4	11.0	10.4
		NEG. M	0.0	-17.4	9.4	-50.0	60.0	18.0	13.0
11	84.00	POS. M	-.7	-16.8	28.0	50.0	3.4	9.1	8.6
		NEG. M	0.0	-15.1	8.3	-50.0	60.0	18.0	13.0
11	84.00	POS. M	-.9	-17.7	49.4	50.0	3.4	15.9	15.3
		NEG. M	0.0	-17.1	15.1	-50.0	60.0	18.0	13.0
12	90.00	POS. M	-.6	-12.2	33.7	50.0	3.4	10.6	10.2
		NEG. M	0.0	-11.9	10.7	-50.0	60.0	18.0	13.0
13	100.00	POS. M	0.0	0.0	-0.0	50.0	3.4	0.0	0.0
		NEG. M	0.0	0.0	-0.0	-50.0	60.0	18.0	13.0

**** SHEARING STRESS SUMMARY ****

SHEAR CONDITION : TOTAL V = V(dl) + V(sdl) + V(LL+I)

	***		SHEAR STRESS		*****		STIFFENER		***** STUDS	**
ANAL. PT.	(ft) DIST	(ksi) fv-act	(ksi) Fv-all	(ksi) .6Fv	Tw-min Unstif	do	(in^2) A_prov	(in^4) I-prov	REQD SPACE	PROV SPACE
1	0.0	8.2	16.7	10.0	.605	40.0	16.5	23.4	24.0	21.0
2	10.0	7.3	14.7	8.8	.570	40.0	1.3	6.7	24.0	21.0
3	20.0	6.3	14.7	8.8	.530	40.0	1.3	6.7	24.0	21.0
3	20.0	7.6	10.2	6.1	.580	75.0	1.3	6.7	24.0	24.0
4	30.0	6.3	10.2	6.1	.531	75.0	1.3	6.7	14.4	24.0
5	36.0	5.6	10.2	6.1	.498	75.0	1.3	6.7	14.6	24.0
5	36.0	5.6	10.2	6.1	.498	75.0	1.3	6.7	14.6	24.0
6	40.0	5.1	10.2	6.1	.474	75.0	1.3	6.7	14.7	24.0
7	50.0	3.8	10.2	6.1	.409	75.0	1.3	6.7	14.9	24.0
8	60.0	2.4	10.2	6.1	.329	75.0	1.3	6.7	15.1	24.0
8	60.0	2.4	10.2	6.1	.329	75.0	1.3	6.7	15.1	24.0
9	70.0	-3.0	10.2	6.1	.365	75.0	1.3	6.7	15.2	24.0
10	80.0	-4.3	10.2	6.1	.438	75.0	1.3	6.7	15.1	24.0
11	84.0	-4.8	10.2	6.1	.465	75.0	1.3	6.7	15.0	24.0
11	84.0	-4.8	10.2	6.1	.465	75.0	1.3	6.7	13.5	18.0
12	90.0	-5.7	10.2	6.1	.502	75.0	1.3	6.7	13.1	18.0
13	100.0	-7.0	16.7	10.0	.558	20.0	9.3	23.2	12.2	18.0

NOTE! - @ bearing locations : A_prov = actual bearing stress
I_prov = allowable bearing stress

MIM. TOTAL NUMBER OF SHEAR STUDS REQUIRED = 137
STUD DATA : NUMBER OF STUDS PER ROW = 3
STUD DIAMETER = .875
STUD HEIGHT = 4

VITA

VITA

Thomas E. Fenske was born July 14, 1949, in Kansas City, Missouri. He graduated Cum Laude from the University of Missouri-Rolla in May, 1976, with a Bachelor of Science degree in Civil Engineering. He received the Master of Science degree in Civil Engineering from the University of Missouri-Columbia in 1982. Mr. Fenske began his study at Purdue University in August of 1984. While at Purdue, he served as a graduate teaching assistant and was selected for membership in Tau Beta Pi, the engineering honor society. He also is a member of Chi Epsilon, the civil engineering honor society.

Mr. Fenske has worked as a practicing engineer in both the public and private sectors. He is currently an Assistant Professor of Civil Engineering in the Speed Scientific School of the University of Louisville, Louisville, Kentucky, and is Vice-President and a principal shareholder in CAAD, Inc., a civil/structural engineering consulting firm. Mr. Fenske is a registered professional civil engineer and a registered professional structural engineer in the Commonwealth of Kentucky and is a member of several professional organizations.