### Abstract

Because of the increasing demand for curved bridges combined with challenges of design and construction, there is need to improve practices for the efficient design and construction of curved bridges. This report presents the results of two projects aimed at establishing a curved steel bridge research program within The University of Alabama system: UTCA Projects 01223, “Design and Construction of Modern Curved Bridges,” and UTCA Projects 03228, “Stability of Curved Bridges during Construction.” The overall objectives of these projects were to improve design and construction practices, improve economic efficiency, and increase safety by investigating stability issues associated with the construction of horizontally curved bridges. Activities of the project included (1) synthesizing curved bridge stability research and state-of-the-art practice, (2) identifying construction stability research needs through contact with ALDOT, prominent researchers, and industry, and (3) conducting fundamental stability research that will improve curved bridge design and construction methodology. The stability of both I- and box-shaped curved girders was considered. The planning, design, fabrication, and construction of a local curved I-girder bridge flyover was studied to gain a better understanding of practical challenges associated with curved bridge design and construction. Students conducted analytical investigations involving advanced theoretical approaches to solving governing stability equations as well as the use of advanced finite element methods. Several papers and presentations resulted. The project enabled the investigators to participate in national level conferences and meetings. The report provides an in-depth synthesis of literature and current practice related to the design and construction of curved I-girder and box girder bridges and an outline of needs for future research. The research literature collected will facilitate continued research efforts. Additional technical results of the effort have been and will continue to be disseminated through conference presentations and proceedings, journal publications, and student theses and dissertations. The figures are provided separately from the HTML versions of the report; readers are encouraged to download the PDF version for printing the full document.
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Executive Summary

The use of horizontally curved girders in the design of highway bridges and interchanges in large urban areas has increased dramatically in recent years. In fact, nationwide, over one-third of all steel superstructure bridges constructed today are curved. The primary reason for the increase is that curved bridges offer an economical means of satisfying the demand placed on highway structures by predetermined roadway alignment and tight geometric restrictions to maintain required traffic design speeds. In addition, curved bridges result in an aesthetically superior solution that has motivated increased use of designs which utilize curved configurations. There will be a likewise increased need for curved superstructure bridges that will facilitate smooth traffic flow off of interstate highways and other major roadways as the population of Alabama’s major metropolitan areas grow.

Because of the increasing demand for curved bridges combined with challenges of design and construction, there is need to improve practices for the efficient design and construction of curved bridges. This report presents the results of two projects aimed at establishing a curved steel bridge research program within The University of Alabama system: UTCA Projects 01223, “Design and Construction of Modern Curved Bridges,” and UTCA Projects 03228, “Stability of Curved Bridges during Construction.” The overall objectives of these projects were to improve design and construction practices, improve economic efficiency, and increase safety by investigating stability issues associated with the construction of horizontally curved bridges. Activities of the project included (1) synthesizing curved bridge stability research and state-of-the-art practice, (2) identifying construction stability research needs through contact with ALDOT, prominent researchers, and industry, and (3) conducting fundamental stability research that will improve curved bridge design and construction methodology. The stability of both I- and box-shaped curved girders was considered. The planning, design, fabrication, and construction of a local curved I-girder bridge flyover was studied to gain a better understanding of practical challenges associated with curved bridge design and construction. The fabrication of a curved box girder being fabricated at a local plant was also observed and studied. Students conducted analytical investigations involving advanced theoretical approaches to solving governing stability equations as well as the use of advanced finite element methods. Several papers and presentations resulted. The project enabled the investigators to participate in national level conferences and meetings. The report provides an in-depth synthesis of literature and current practice related to the design and construction of curved I-girder and box girder bridges and an outline of needs for future research. The research literature collected will facilitate continued research efforts. Additional technical results of the effort have been and will continue to be disseminated through conference presentations and proceedings, journal publications, and student theses and dissertations. The figures are provided separately from the HTML versions of the report; readers are encouraged to download the PDF version for printing the full document.
1.0 Introduction

1.1 General

The use of horizontally curved girders in the design of highway bridges and interchanges in large urban areas has increased dramatically in recent years. In fact, nationwide, over one-third of all steel superstructure bridges constructed today are curved. The primary reason for the increase is that curved bridges offer an economical means of satisfying the demand placed on highway structures by predetermined roadway alignment and tight geometric restrictions to maintain required traffic design speeds. In addition, curved bridges result in an aesthetically superior solution that has motivated increased use of designs which utilize curved configurations. There will be a likewise increased need for curved superstructure bridges that will facilitate smooth traffic flow off of interstate highways and other major roadways as the population of Alabama’s major metropolitan areas grow.

In the early days of curved bridge design and construction, bridge superstructures supporting curved roadway alignment were comprised of short straight girders linked at the supports. This resulted in inefficient use of very short spans between support piers. As the technology for designing and fabricating curved girders became available, it became possible to design curved bridges with much greater distances between supports. The cost of the curved girder system employing a series of straight girders is high compared to the total cost of the curved girder bridge system using curved girders, as a substantial portion of the substructure that would be necessary for the straight beams can be eliminated. Furthermore, using continuous curved girders permits the use of shallower sections as well as a reduction in the slab overhang of outside girders.

Today, curved girders are widely used in bridge superstructures. The designer has many choices including material (concrete vs. steel), cross section shape (tub girder vs. I-beam), etc. Furthermore, the past three decades have resulted in advances in optimizing curved bridge design, resulting in innovative, aesthetically pleasing structures. However, due to the addition of curvature, the design and construction of bridges becomes immensely more complicated than that of straight bridges. While the girders, stringers, and floor beams of straight bridges can be designed by systematically isolating each member and applying standard loads, curved bridges must be designed with careful consideration to system-wide behavior. In essence, the addition of curvature adds torsion to the system that results in significant warping and distortional stresses within the member cross sections. Furthermore, “secondary members” such as cross frames and diaphragms that provide stability in straight bridges become primary load carrying members in curved bridges.

The “closed” box- or tub-girder cross section resists curvature induced torsion far more efficiently than the “open” I-girder cross section. Although other states have demonstrated substantial economic benefits of using box girder superstructures over I-girder cross sections for
sharp curvatures (radius < 1000 ft), Alabama has not yet used box girders for curved bridge superstructures because of a lack of design firms and construction contractors within the region that have box girder experience. Federally sponsored or coordinated research on box girder stability is limited. The original scope of the FHWA Curved Steel Bridge Research Project included fundamental research on the behavior of both steel I-girder and box girder bridges. However, soon after the initiation of the project in 1992, investigators and FHWA program managers realized that there would not be enough resources to address both I- and box girder research. All of the box girder research tasks were subsequently eliminated. There is still a void in the understanding of curved box girder behavior and a need to improve currently used design and construction processes.

Although the design of curved bridges is much more complex than that of straight bridges, there are no requirements specific to the design of curved bridges integrated into the American Association of State Highway Transportation Officials (AASHTO) Standard Specifications for Highway Bridges. There is, however, the “Guide Specifications for Horizontally Curved Highway Bridges,” which was first adopted in 1981. This “guide” is widely recognized to be outdated, disjointed, and difficult to use.

Another area that has been widely recognized as needing further research is that of lifting and transporting curved girders during construction. By far, the most frequent problems are encountered during construction. The problems are typically more severe and more common than those encountered during the construction of straight bridges. During the construction of straight bridges, girders and stringers are easily erected by one crane using one or two pick-up points, or by two cranes using one pick-up point each. The individual straight girders can simply be set in place with little concern for instability. Lifting and setting presents little difficulty for straight beams where the center of gravity is coincident with the centroidal axis of the beam cross section. However, for horizontally curved girders, the center of gravity is non-coincident with the cross section centroid. Depending on the lifting and support mechanisms used, significant torsional stresses and minor-axis bending stresses may be induced. Therefore, the handling and erection of horizontally curved girders requires engineering expertise beyond that required for the construction of straight bridges. Engineers not experienced in the design of curved bridge systems often make the mistake of assuming that behavior and design is the same as that for straight bridges. Instability during construction can easily translate into unsafe conditions for construction workers, not to mention unforeseen additional costs.

Because of the increasing demand for curved bridges combined with challenges of design and construction, there is need to improve practices for the efficient design and construction of curved bridges. This report presents the results of two projects aimed at establishing a curved steel bridge research program within The University of Alabama system: UTCA Projects 01223, “Design and Construction of Modern Curved Bridges,” and 03228, “Stability of Curved Bridges during Construction.” The report provides an in-depth synthesis of literature and current practice related to the design and construction of curved I-girder and box girder bridges and an outline of needs for future research. Additional technical results of the effort have been and will continue to be disseminated through conference presentations and proceedings, journal publications, and student theses and dissertations.
1.2 Objectives

The overall objectives of this project were to conduct background research and initiate stability investigations that will lead to improved design and construction practices, improved economic efficiency, and increased safety for the design and construction of horizontally curved bridges. Additional goals of the project included:

- Establishing relationships with ALDOT bridge engineers, FHWA program managers, prominent curved bridge researchers, and industry leaders;
- Identifying research needs;
- Identifying technology transfer and continuing education needs of ALDOT and the bridge design industry of the region;
- Contributing to structural stability research needs through graduate student involvement;
- Sponsoring PhD students in the joint UAB-UAH Civil Engineering PhD program;
- Identifying potential sources of future support and conducting the preliminary work needed to demonstrate to potential future sponsors such as ALDOT and FHWA the capabilities of the research team;
- Supporting “faculty development” that will result in a bridge research program within the UA System that will continue to contribute to needs of the Alabama and the nation for many years;
- Disseminating results through conference presentations and nationally and internationally recognized peer reviewed journals.

1.3 Scope and Project Description

1.3.1 Tasks

The four tasks of the project work plan and a synopsis of the accomplishments associated with each are described below. The work was accomplished over a 20-month period beginning January 1, 2003.

TASK 1. Synthesis of current practice

The data-gathering effort involved investigating the current state-of-the-art curved bridge design and construction practice around the country. Particular focus was placed on the design and construction of curved box girder bridges and the erection of both box- and I-girder bridges. The usual avenues of collecting information such as publication database searches and Web-based searches were used. However, the synthesis was largely based on direct contacts by the
investigators. Many states have conducted investigations to improve the design and construction of curved bridges and the resulting reports of such projects often do not show up in the typical library or journal database search. Investigators at the University of Texas, the University of Houston, the University of Pittsburgh, and the Pennsylvania State University, for example, were contacted, as well as design industry experts. Other materials such as workshop materials developed by the Structural Stability Research Council were collected and reviewed. More than 200 articles and reports were collected and reviewed, and a synthesis is provided in subsequent chapters of this report.

**TASK 2. Identification of research needs**

This task involved reviewing recent literature and meeting with research authorities and industry leaders. The objective was to identify needs specific to Alabama as well as areas of national need. Particular focus was placed on identifying research needs associated with the design and construction of curved box girder bridges and stability problems that frequently occur during the erection of both box- and I-girder bridges. A meeting with ALDOT bridge engineers helped to identify how university expertise can help with their needs. The investigators also contacted design firms, manufacturers, and construction companies in the region to establish relationships and to understand their issues regarding curved bridge design and construction. A local curved I-girder bridge project was studied to gain a better understanding of practical challenges associated with fabricating, transporting, and erecting curved girders. The investigators met with FHWA researchers and engineers involved in the FHWA Curved Steel Bridge Research Project at the Turner-Fairbanks Highway Research Laboratory to identify areas of national need and to explore the possibility of future sponsorship. The investigators also met with prominent curved bridge researchers from several universities and companies. The “research needs” effort identified topics suitable for Masters and PhD students with interest in curved bridge stability. It also provided an understanding of potential sources of support for continuing curved bridge research.

**TASK 3. Student-oriented analytical research**

The synthesis and research-needs aspects of this project identified suitable Masters and PhD topics. The analytical research efforts involved graduate students who investigated curved bridge research topics that may lead to improvements in bridge design specifications or improvements in construction practice. The graduate student research topics involve steel composite construction, since that is where the primary focus of current research is directed. Both I- and box girder cross sections were considered. The investigations were analytical in nature, rather than dependent upon laboratory testing. An MS project report and several publications were produced. Two PhD students and a MSCE student continue to work on their topics.
TASK 4. Technology transfer

Ultimately, success of the project depends on the dissemination of the results to stakeholders. Interaction with ALDOT, other researchers, and industry identified the need for guide documents and continuing education courses. In addition to this final report, transfer of all findings to stakeholders will be facilitated through conference presentations and proceedings, journal publications, and student theses and dissertations.

1.3.2 Project Team

The research was directed by Dr. Jim Davidson, an Associate Professor in the Department of Civil and Environmental Engineering at the University of Alabama at Birmingham (UAB). Dr. Davidson was involved in the Federal Highway Administration “Curved Steel Bridge Research Project” (FHWA-CSBRP) while at Auburn University (1992-1996). He is currently the chairman of Task Group 14, “Horizontally Curved Girders,” of the Structural Stability Research Council and has numerous publications related to the behavior of curved bridges. Dr. Davidson was assisted by four civil engineering graduate students: (1) Ramy Abdalla (PhD student), (2) Mahendra Madhavan (PhD student), (3) Lance Osborne (MSCE student), and (4) Toby Leamon (MSCE student). Mr. Osborne completed his MSCE requirements May 2002. Abdalla, Madhavan, and Leamon continue to work on their degree requirements.

1.4 Summary of Accomplishments of the Research Team

This project has resulted in progress towards establishing a curved bridge stability research program within the University of Alabama System. Two PhD students and two MSCE students were recruited into the UAB graduate program as part of this project, and several papers and presentations were produced. A library of relevant papers and research reports required for a long-term graduate research program in this area was collected. The project enabled the investigators to participate in meetings and conferences focused on curved bridge stability and construction issues. An understanding of current practice and research needs, from the national and Alabama Department of Transportation perspectives, was developed.

1.4.1 Stability of Curved I-Girders during Construction

Students involved in I-girder aspects of the project are developing theory-based formulations and analytical models to resolve issues associated with design and construction using curved I-girders. For example, methods used to lift single curved girders onto supports may induce significant distortion, warping, and minor axis bending stresses that were not considered in the design. It may be necessary to lift two girders temporarily braced with cross frames to provide stability during erection. Therefore, strength equations are needed for various lifting scenarios. Also, the cross frames can be subjected to significant forces during the construction of curved I-girder bridges that are not encountered in straight bridges. Engineers need design methodology
for predicting bracing member forces. Transverse and longitudinal stiffeners may be required to prevent unacceptable levels of distortion, and design guidance is needed.

Current focus is on the stability of curved I-girders during construction. A comprehensive synthesis of current practice and research literature related to the construction of curved I-girder bridges is presented in Chapter 3. Curved I-girders, in particular, can be unstable during lifting and transporting unless adequately braced. Problems are common yet rarely documented. However, there are no comprehensive guidelines or recommendations on construction practices, nor is there a published survey or summary of problems encountered by DOTs during construction of curved bridges. Discussions with prominent researchers and FHWA program managers during this project indicated that this is an issue that warrants additional research.

Analytical studies have been focused on the effects of curvature-induced warping on the local buckling of curved I-girder flanges. Several papers and presentations were developed. A PhD dissertation is anticipated to thoroughly address stability issues that are encountered with curved I-shaped plate girders. Advanced theory and finite element methods are being used. The analytical research will culminate with improved criteria suggested for curved bridge design and construction.

Closed-form equations are being developed to predict maximum stresses that occur under various lifting mechanisms used during erection of curved I-girders. The initial phase of this research is focused on the development of equations based on small displacement torsion of thin-walled structures that represent idealized support conditions for lifting of curved I-girders. Later phases of the research will involve the use of finite element analyses to adjust the theory-based equations for deflection amplification and local stress effects.

A local curved bridge construction project, the Galleria I-459 flyover in Hoover, Alabama, was studied from inception and preliminary design through placement of the girders and concrete deck. This study provided a better understanding of the curved I-girder bridge design and construction process. The close proximity of the project to the University of Alabama at Birmingham (UAB) and the ability to safely observe the progression of construction from the Galleria parking deck provided a unique opportunity to document the construction. Many of the persons involved in the flyover construction were interviewed, including ALDOT project managers, contractor project engineers, erectors, fabricators, and others. The fabrication company (Carolina Steel Corporation of Montgomery Alabama) was visited, engineers were interviewed, and digital images were taken to provide an understanding of problems that arise during the curving process. An understanding of challenges while transporting within the fabrication facility and from the fabrication facility to the jobsite was acquired. Erectors were interviewed to gain a practical, real-world perspective on challenges that occur at the jobsite during lifting and placing of the girders. Digital images were taken on a bi-weekly basis for several months to provide a record of the project. All of the results were merged as a comprehensive report (Osborne 2002).
1.4.2 Design and Construction of Curved Box Girder Bridges

Box girder construction practices were investigated and research needs identified. Information was collected on construction projects that involved curved box girders towards the goal of understanding the engineering and economic advantages to using closed cross section girders for curved bridges. Prominent researchers and practitioners were contacted. An understanding of construction procedures and stability issues was developed. Box girder designs were collected from other state DOTs and will be reviewed and used in analytical case studies as part of student research projects. A comprehensive synthesis of current practice and research literature related to the construction of curved box girder bridges is presented in Chapter 4. There were two overall aims of the box girder aspects of the project:

(1) Curved box girder bridges have not been used in Alabama but may offer economic advantages in future highway construction projects. Therefore, one aim is to understand current box girder design and construction practice and how box girder geometries might help Alabama.

(2) Curved box girders were eliminated from the FHWA Curved Steel Bridge Research Project and federally organized and sponsored research projects will likely arise in the future. Therefore, the second aim is to conduct preliminary work needed to be recognized as a leading research program in this area and perhaps open the door towards participation in future federally sponsored research.

Analytical stability research was initiated in this area, with a focus on the effects of distortion on the strength of curved box girders and lateral bracing requirements. Finite element models were developed. Theory-based formulations and analytical models are being developed to resolve issues associated with construction using curved box girders. The analytical research will culminate in a PhD dissertation that will address stability issues that are encountered with curved box-shaped plate girders. An ultimate goal is to develop and recommend guidelines and design criteria.

1.4.3 Outreach and Interaction

The project facilitated valuable interaction with prominent researchers and industry leaders. The investigators met with researchers from the University of Houston, Pennsylvania State University, the University of Pittsburgh, Auburn University, and Georgia Institute of Technology. In addition to academic researchers, the investigators interacted with engineers from the FHWA Turner-Fairbanks Highway Research Center, the Pittsburgh and Tampa offices of HDR Engineering Inc., BSDI Inc., and the American Institute of Steel Construction (AISC).

The transfer of new technologies to Alabama stakeholders can result in tax dollar savings as the region continues to populate and new bridges and highway interchanges are required. For example, curved box girder bridges are not used in Alabama, whereas other states have demonstrated substantial advantages in the use of box girder superstructures for bridges with sharp curvature. Curved box girder construction introduces challenges that are different from I-
girder construction, but in general, the boxes are more stable during construction and provide significantly enhanced efficiency in resisting torsional loads. Academic research can facilitate the transfer of recent advances in box girder construction to Alabama stakeholders, and can provide research-level technical expertise that can help avoid problems. The Team met with ALDOT bridge engineers to discuss their interests and needs regarding curved bridge education and research.

1.5 Report Content and Organization

2.0 General Literature Review and Synthesis

2.1 Origins of Curved Beam Theory

In the early years of modern bridge design, engineers were reluctant to use curved girders in forming curved roadways due to the mathematical complexities associated with the design of such systems. Curved girders are subjected to not only major axis flexural stresses but also to significant torsional stresses, even under gravitational loading alone. In addition, deflection, cross section distortion, and deflection amplification (large displacement) effects are much more pronounced in curved girder systems. The inherent rotation characteristics of horizontally curved girders require that the diaphragms and bracing that are used in straight girder systems simply to prevent premature lateral buckling become very important (primary) load carrying components in curved systems. In the past two decades, the availability of digital computers to carry out the complex structural analysis and design of such girders, along with advancements in fabrication and erection technology, have made horizontally curved girder superstructures a viable and cost efficient option for designers.

Design considerations are different in the curved I-girder design compared to the curved box girder design. The I-girder is an "open" section and is characterized by very low torsional resistance. The twisting of the I-girder results in significant normal stresses in the flanges. Diaphragms and cross frames between girders are an absolute necessity in restraining these stresses to acceptable levels and must be carefully designed with respect to both strength and spacing. The single curved I-girder is inherently unstable and, at typical bridge girder lengths, is very flexible and susceptible to large deformation. Extreme care must be taken in handling and erection. The box girder behaves as a "closed" section with generally improved torsional resistance over the I-section, but with its share of complications in fabrication and erection. As with curved I-girders, internal and external cross frames and diaphragms must be carefully designed to reduce cross section distortion, but external bracing is often removed after the concrete deck has cured.

The earliest theoretical work on curved beam theory is attributed to St. Venant (1843) over 150 years ago. Since then, a number of other European and Japanese researchers have contributed to the analysis of curved beams, as pointed out in a review by McManus et al. (1969). These researchers include Gottfield (1932), Umanskii (1948), Dabrowski (1964, 1965, 1968), Vlasov (1961), Timoshenko (1905), Shimada and Kuranashi (1966), and others. Comprehensive presentations of the basic theory of thin walled beams including flexure, torsion, distortion, and stress distribution is provided in several texts (Nakai and Yoo 1988, Vlasov 1961, Dabrowski 1968, Kollbrunner 1969, Heins 1975).
2.2 Development of Curved Bridge Design Approach

Research prior to the mid-sixties on the behavior of curved girders was generally limited to theoretical work on the linear elastic static behavior of isolated curved members and based on strength of materials assumptions, namely, that the cross section does not distort, that Hooke's law applies, and that small deflection theory applies. Since the mid-sixties an emphasis in curved girder research in the United States and Japan has been placed on the practical use of curved beam theory towards the design of horizontally curved bridges. Theory has been formulated for the horizontally curved girder in the curved bridge system including buckling behavior, large displacement behavior, and ultimate strength behavior.

In 1965, U.S. Steel (Highway 1965) published an approximate procedure called "V-load Analysis" for determining moments and shears in horizontally curved open-framed highway bridges. It is theoretically pure with regard to torsion due only to curvature and load distribution for static equilibrium and works best for framing without composite action. The method does not account for lateral bracing between girders in the plane of the flanges. Accuracy of the method with regard to live load depends upon the ability of the user to assign appropriate load to the girders prior to the V-load analysis. It has been noted that the live load distribution factors used in straight bridge design do not appropriately model the distribution in curved bridges and researchers have proposed equations for curved bridge design (V-Load 1984, Heins and Jin 1984, Brockenbrough 1986).

Rapid advancement in computer technology over the past 20 years has encouraged both theoretical and analytical investigations on many aspects of the behavior of horizontally curved girders. In addition to the general ease in producing numerical solutions to solve complex mathematical relationships, such as in using the finite difference method to provide solutions to insolvable differential equations, this technology has encouraged the development of more general use structural analysis tools, namely, the finite element method. The use of these tools for designing curved bridges has also been encouraged by the requirement that the entire curved bridge superstructure be analyzed as a system. Many software packages have been developed exclusively for the design and analysis of curved bridges, resulting in less use of the V-load method, which was used for over 75 percent of curved bridge design prior to 1973 (AISC 1986). Advanced general-purpose finite element packages such as ADINA, ABAQUS, ANSYS, NASTRAN, etc. have extensive modeling capabilities that can be used in a number of solution types such as linear-elastic static, modal, transient, buckling, geometric nonlinear, and material nonlinear analyses.

In 1969, a comprehensive pooled funds research project sponsored by 25 participating state highway departments was initiated under the direction of the Federal Highway Administration to study the behavior of curved bridges and to develop design requirements. This project, referred to as CURT (Consortium of University Research Teams), was comprised of Carnegie Mellon University, the University of Pennsylvania, the University of Rhode Island, and Syracuse University. Other research was also underway at various universities and research facilities throughout the country. This work was performed throughout the 1970's and resulted in the
AASHTO Guide Specifications for Horizontally Curved Highway Bridges (subsequently referred to as the Guide Specifications), which was officially adopted in 1980.

The CURT project involved: 1) reviewing all published information on the subject of curved bridges; 2) conducting analytical and experimental studies to confirm or supplement this published information and assimilate information from related research programs sponsored by state highway departments; 3) developing simplified analysis and design methods along with supporting computer programs and design aids; and 4) correlating the developed analysis and design methods with analytical and experimental data. The resulting “guide specifications” include Working Stress Design (WSD) provisions, first approved by ballot in November 1976, and Load Factor Design (LFD) provisions, first approved by ballot in October 1979, and deal with both "I" and "box" shape girder bridges. Strength formulations for the web, flanges, and stiffeners are emphasized. Experience indicates that the strength formulations have been at least conservative in that there have been no reported failures of these bridges by overload. The Guide Specifications frequently refer to the design provisions for straight steel girders in the Standard Specifications.

Many theoretical, analytical, and experimental investigations along many areas of curved bridge girder behavior were supported within the CURT program. One of the principal investigators of the strength of curved I-girders was C.G. Culver at Carnegie Mellon University. Culver and co-researchers conducted and presented analytical and experimental research on the lateral instability of horizontally curved members (Culver and McManus 1971), local buckling behavior of curved I-girder compression flanges (Culver and Frampton 1970, Culver and Nasir 1971), bending behavior of curved girder web panels (Culver, Dym, and Brogan 1972a, 1972b), shear behavior of curved girder web panels (Culver, Dym, and Brogan 1972c), and web slenderness and transverse stiffener requirements for curved girders (Culver, Dym, and Uddin 1973). Many of the requirements in the Guide Specifications, including the procedure for determining the allowable flange normal stress in compression, are based on the studies reported by Culver and co-researchers in the early 1970's.

The Guide Specifications in its original form was disjointed and difficult to use. There was significant discontinuity in the compressive strength formulation between compact and noncompact sections. Additionally, the strength predicted by the formulation did not approach that predicted by the formulation for straight girders as the radius of the curved girder approaches infinity. The commentary was incomplete and lacked the detail needed to explain the development of many provisions. Many of the original key references detailing the research were not readily available, nor could they be interpreted without a great deal of study. Given the lack of comprehensible support material available for understanding and clarifying the Guide Specifications, there existed much potential for misinterpretation of many of the provisions.

Changes to the original 1980 Guide Specifications were included in the AASHTO Interim Specifications for the years 1981, 1982, 1984, 1985, 1986, and 1990. A new edition was published in 1993 that included the interim changes, and additional changes, but did not reflect the extensive research on curved girder bridges that was conducted since 1980 or the many important changes in related provisions of the straight girder specifications.
In 1993, NCHRP Project 12-38, "Improved Design Specifications for Horizontally Curved Steel Girder Highway Bridges," was initiated to reorganize the Guide Specifications using the current state of the art information and retaining the Working Stress and Load Factor format. NCHRP Project 12-38 produced Report 424 “Improved Design Specifications for Horizontally Curved Steel Girder Highway Bridges” and a 2003 edition update to the Guide Specifications (Hall and Yoo 1996, Hall et al. 1999). The 2003 Guide Specifications demonstrates the applicability of the guidelines by including design examples for I- and box girders. Currently, NCHRP Project 12-52, “LRFD Specifications for Horizontally Curved Steel Girder Highway Bridges,” is being conducted by Modjeski & Masters to prepare specifications for the design and construction of horizontally curved steel girder bridges (for both I- and box girders) in a calibrated load and resistance factor design (LRFD) format that can be recommended to AASHTO for adoption.

In 1992, a comprehensive research project administered by FHWA, "Curved Steel Bridge Research Project" (referred to as the FHWA-CSBRP), was started based in part on the research needs identified by NCHRP Project 12-38. It was intended that the work done by this FHWA project be used by NCHRP Project 12-52 to write and advance work done by NCHRP Project 12-38 into an LRFD based design and construction specification for horizontally curved steel girder bridges. The goal of the FHWA-CSBRP was to (Yadlosky 1993) "... (A) conduct fundamental research into the structural behavior of curved steel flexural members and bridges for use in developing an LRFD specifications, and (B) address construction issues, in order to provide adequate information to develop and clarify design specifications." The work of this project was organized into many tasks and subtasks and was delegated to companies, universities, and research organizations all over the country. Testing associated with the FHWA-CSBRP has recently been completed, and final reports are being prepared.

The only other bridge design document in the world that specifically addresses curved bridge design is the Japanese “Guidelines for the Design of Horizontally Curved Girder Bridges” by the Hanshin Expressway Public Corporation (Hanshin 1988). Several researchers, including the primary author of this report (Davidson), have demonstrated disparity in the strength formulations between the Japanese and American curved bridge design guides, which further emphasizes the need for additional research. Recent correspondence with Japanese researchers indicates that an updated version of the Hanshin guide is being developed (Madhavan 2004).

### 2.3 Cross Frame Interval Design

Horizontally curved I-girders undergo a coupled lateral bending moment of the top and bottom flange due to curvature referred to as torsional warping moment or "bimoment," which induces warping of the girder cross section. The bimoment is related to the second derivative of the angle of twist (Vlasov 1961, Galambos 1968). Due to this rotation characteristic, the diaphragms and cross frames become major load carrying members. In straight girder bridges, the primary function of the cross frames and diaphragms is to prevent premature lateral buckling of the girder so the cross frame members are designed as secondary members. In curved bridge systems, the cross frames and diaphragms have the added responsibility of restraining the twisting of the
girder, thereby reducing the warping stresses in the flanges and reducing the overall vertical deflection of the system. The spacing interval between cross frames therefore becomes a critical design parameter that limits the warping stress and deflection to acceptable levels. Also, there is growing sentiment in the bridge engineering community to minimize the number of cross frames due to fatigue problems and added cost.

For curved I-girder bridge systems under gravity loading where the rotation of the cross section is restrained by cross frames or diaphragms, the bimoment, and thus lateral bending of the flanges, varies dramatically in magnitude and direction along the span with the peak moments occurring at the cross frame locations. In positive lateral moment regions of the span, such as at the cross frame locations, the bimoment increases the normal compressive stress on the outside of curvature edge of the flange and decreases the stress on the inside. In the intervals between cross frames, the direction of bimoment is reversed and the highest stresses occur on the inside edge of the flanges.

An approach used for developing models of straight bridges has been to idealize the girders and cross frames with an "equivalent" truss system where the flanges and web of the girders are replaced with equivalent truss members. Several studies used this modeling technique for curved I-girders (Heins and Jin 1984, Schelling, et al. 1989). With a system of girders and cross frames modeled in this manner, bending about the two major axes, torsion, and warping are accounted for but distortion of the web is not. A number of studies, however, indicate that distortional deformation may have a considerable effect on deflections and stresses in curved beams (Hikosaka and Takami 1985, Brockenbrough 1986). Prior to the FHWA-CSBRP, analytical design oriented research concerning the cross frame spacing requirements of horizontally curved bridges was limited to publications by Littrell (1985), Yoo and Littrell (1986) and Schelling et al. (1989).

Davidson et al. (1996a, Keller 1996) investigated the cross frame spacing requirements of horizontally curved I-girder bridges as part of the nominal strength research of the FHWA-CSBRP. The focus of the research was to: 1) improve understanding of the contribution of cross frames to the structural integrity of a curved I-girder system; and 2) determine and quantify those parameters which most affect the response of the system. The finite element method was used to create detailed models of horizontally curved steel I-girder bridges connected by cross frames. The effects of a number of parameters on the behavior of the curved girder system were established and compared to the effect of these parameters in straight girder systems. The parameters which most significantly affect the behavior of the systems were determined to be the degree of curvature, span length, and flange width. An equation was developed based on a nonlinear statistical regression to provide a preliminary design limit for cross frame spacing interval. The results of the equation were compared to those results obtained by previous equations developed by various methods for the same purpose.
2.4 Lateral-Torsional Buckling

There have been many theoretical developments pertaining to the elastic lateral-torsional buckling behavior of curved members. A comprehensive review and comparison of these theories was presented by Kang et al. (1992, 1994a, 1994b). Yoo (Yoo 1982c, Rajasekaran and Ramm 1984) used the minimum potential energy principle to obtain solutions for the elastic flexural torsional buckling loads for in-plane and out-of-plane buckling modes of thin walled curved beams that do not undergo local buckling. Yoo and Pfeiffer (1983, Pfeiffer 1981) investigated the elastic buckling behavior of a thin walled curved member through a variational based finite element formulation. Solutions to different cases pertaining to the stability of curved beams were obtained and compared to solutions presented by Timoshenko and Gere (1961), Vlasov (1961), and Culver and McManus (1971) and were shown to be significantly different. The discrepancies were attributed to incorrect formulations in Timoshenko and Vlasov's cases, and to the fact that the governing differential equation was viewed as a deflection amplification problem rather than a classical eigenvalue problem. Later, Yoo and Pfeiffer (1984) presented a solution to the stability of curved beams with in-plane deformation and asserted their earlier conclusion related to the discrepancies with existing solutions including the work of Vacharajittuphahn and Trahair (1975), which is essentially based on Vlasov's formulation. In 1985, Yoo and Carbin conducted a series of laboratory tests on 12 simply supported curved beam specimens subjected to concentrated loads. The length of each specimen was approximately 6.1 m (20 ft) with a subtended angle ranging from 0 to 30 degrees. Specimens were loaded from the top flange in one case and from the bottom flange in another case. All specimens reached their ultimate loads, which were higher than the analytically predicted buckling load values from Yoo and Pfeiffer. Bottom flange loaded specimens had higher ultimate loads and showed lower cross section deformations than top flange loaded specimens.

Papangelis and Trahair (1986 and 1987) examined the work of Yoo and Yoo and Pfeiffer by conducting tests on circular aluminum arches. They concluded that the theoretical loads obtained from the work of Yoo differ substantially from analytical and experimental results of various researchers.

The conflict among these curved beam theories was also discussed in a series of publications by Yang and Kuo (1986, 1987, and 1991, Rajasekaran et al. 1988), who derived the nonlinear differential equation of equilibrium for horizontally curved I-beams by making use of the principle of virtual displacements to establish the equilibrium of a bar in its buckled configuration. Numerical results were obtained and compared with those resulting from Yoo's as well as Vlasov's theories. The authors attributed the discrepancy between their results and Yoo's to the fact that Yoo not only neglected both the radial stress effect and the contribution of shear stresses to the potential energy, but also substituted the curvature terms of the curved beam in the potential energy equation of a straight beam.

Kuo and Yang (1991) further criticized the work of Vlasov and Yoo by solving numerically the buckling problem of a curved beam with a solid cross section under uniform bending in one case and uniform compression in another case.
Based on an elastic buckling finite element study using MSC/NASTRAN, Kang and Yoo (1990) showed that initial curvature and warping do not significantly affect the lateral torsional buckling strength of curved girders with a subtended angle between two adjacent cross frames up to 0.1 radian, the maximum value allowed in the 1993 Guide Specifications. In 1994, Kang and Yoo (1994a and 1994b) presented companion papers on the buckling and large displacement behavior of thin-walled circular beams based on theory derived using the principle of minimum potential energy. Closed form solutions were obtained for limited loading and boundary conditions and used for a comparison to the theory of other researchers.

As part of the CURT project, Culver and McManus (1971, McManus 1971) presented a second order analysis in which the equilibrium equations were formulated based on the deformed structure. Results obtained in this study were compared to those of lateral buckling tests conducted by Mozer et al. (1971, 1975a, 1975b, 1975c). The study recommended a set of formulas that were later adopted into the Guide Specifications. However, there has been much concern for the derivation of these equations with a critical review presented by Hall and Yoo (1995).

Japanese researchers have also conducted research in this area. Nishida et al. (1978) presented work which used the large deflection theory of curved members to derive the approximate critical elastic moment for a horizontally curved beam subjected to two equal end moments. The critical moment approaches that of the straight girder as the radius of curvature approaches infinity in the equations presented.

Kang (1992) demonstrated that a large variation in torsional rigidity ratio has little effect on the critical load ratio of the lateral buckling of horizontally curved girders loaded normal to the plane of curvature and that the subtended angle is the dominating parameter. The results from finite element analyses were used to form a regression equation for the reduction in lateral-torsional buckling of the curved girder over that of the straight (Yoo et al. 1996).

Most of the work described above is of a purely academic nature with little practical analytical results provided on the lateral instability of horizontally curved bridge plate girders. The works by Yoo and co-researchers (Yoo and Pfeifer 1983, 1984, Yoo et al. 1996) are the only investigations published in the public domain that have used the classical eigenvalue approach to provide significant information on the elastic lateral instability of horizontally curved girders. Furthermore, there is much disparity and controversy among various theories on the instability of curved beams presented by the various researchers.

For horizontally curved beams loaded normal to the plane of curvature, the large displacement analysis gives more meaningful design information than the bifurcation buckling analysis. Significant deflections occur in such structures prior to reaching the predicted bifurcation load. The bifurcation buckling load represents an upper bound of elastic instability. Developments on the nonlinear theory of curved beams have been made by many of the researchers discussed above, with a comprehensive review and comparison of these theories by Kang (1992, 1994a). Culver and McManus (1971) used second order analyses and the finite difference technique to solve bifurcation buckling and large deflection problems of curved beams. Their research
became the basis of the ASD strength equations in the Guide Specifications (Task Committee on Curved Girders of ASCE-AASHTO 1977a, 1977b). However, as pointed out by Yoo and Pfeiffer (1983), the critical moment ratio (the ratio of the critical moment of the curved beam to that of the comparable straight beam) derived from their results approaches unity as the subtended angle approaches 90 degrees, which is hardly convincing.

Japanese researchers have also conducted analytical work on the large deflection behavior. Nishida et al. (1978, Fukumoto et al. 1980, Fukumoto and Nishida 1981) provided analytical results on the elastic lateral instability of horizontally curved beams based on large deflection theory, resulting in a predictor equation for critical moment. All of the lateral instability investigations, though, involved the isolated curved member with simplified boundary conditions. Analytical research on the system behavior, including the effects of lateral restraints such as cross frames and diaphragms, is virtually unknown.

2.5 Local Buckling of Curved Girder Flanges

Plate components of the girder in compression must be proportioned to prevent premature local instability. Design specifications stipulate required width-thickness ratios for design of the section as “compact” or “non-compact,” where compact refers to the ability of the section to reach substantial plastic strains prior to local instability. Also, for the design of plate girder webs, the rigidity and placement of longitudinal and transverse stiffeners is always a consideration. The effects of curvature and warping stresses on elastic flange buckling were investigated analytically by Culver and Frampton (1970) and on inelastic flange buckling by Culver and Nasir (1971) and Komatsu et al. (1975, 1981).

Culver (Culver and Frampton 1970) conducted research on the local buckling of the compression flange of curved I-beams in the elastic range. Pre-buckling stresses were determined using the theory of elasticity, which was then used as loading values on the curved flanges. Each half of the flange was treated as a plane stress problem and the governing differential equation was formulated in polar coordinates and solved using the central difference method. The web was simulated with boundary conditions that included a rotational "spring" stiffness at the web line. The results from Culver's research demonstrated that the difference between the buckling behavior of curved plates and rectangular plates in the elastic range is insignificant within the specified limit for curvature.

In a subsequent investigation by Culver (Culver and Nasir 1971), inelastic buckling of curved flanges was investigated. Each half of the curved flange plate was modeled as in the elastic investigation and the inelastic plate stiffnesses were determined from experiment. Residual stresses were also considered. The results showed that the buckling coefficient is a function of the aspect ratio, curvature, and the amount of yielding. However, for curvatures within the design limits stipulated by the Guide Specifications, the effect of curvature was shown to be insignificant.
Kang (Kang and Yoo 1990) conducted an analytical study that examined the allowable flexural stresses permitted by the Guide Specifications using three dimensional modeling of the whole cross section in MSC/NASTRAN. Although the work by Kang was not intended to be an in-depth investigation on the local buckling behavior of curved girders, his results indicated that there may be a significant curvature effect on local buckling. The flange local buckling strength of a curved girder without warping stress applied was shown to decrease the buckling strength by as much as 7 percent compared to the strength of a comparable straight girder and by as much as 15 percent when warping stress is applied.

The Japanese have also conducted analytical research on the local buckling behavior of curved compression flanges and have concluded that the influence of the stress gradient due to warping stress cannot be omitted in evaluating the buckling strength of I-girders with substantial curvature (Nakai et al. 1981, Nakai and Yoo 1988, Fujii and Ohmura 1987). Research was performed using the finite element method with elasto-plastic and large displacement theory on flat plate models without curvature but with applied compressive forces and bending moments to simulate bending and warping stresses. "Simple support" boundary conditions were used to model the junction of the web and flange plate and only half of the flange was modeled. Also, initial deflections and residual compressive stresses were considered (Komatsu et al. 1975, Komatsu and Kitada 1981). The Japanese suggest approximately 30 percent increase in the required curved compression flange thicknesses to eliminate potential local buckling where warping stress is "predominant" (Komatsu and Kitada 1981, Hanshin 1988, Nakai and Yoo 1988, The Japanese Road Association 1990).

Using finite element analyses, Davidson and Yoo (1996b, Wang 1994) concluded that the presence of stress gradient and torsional restraint provided by the web were the dominating parameters affecting the local flange buckling of curved girders and not the actual curvature in flange plate. The results from finite element were used to establish a reduction equation to prevent local buckling before yielding of flanges was developed.

As part of the effort sponsored by UTCA, an improved definition for the effects of curvature-induced warping normal stress on elastic flange buckling behavior was developed (Madhavan and Davidson 2004a, 2004b). The solutions presented were based on energy principles and verified using advanced finite element methods.

2.6 Strength and Stability of Curved Girder Web Panels

2.6.1 Pure Bending

For I-shaped plate girders, the primary role of the web in the region of high moment is to maintain the relative distance between flange plates. Efficient design of plate girders therefore requires that the flange plates carry most of the primary moment and that the web be designed as slender as structurally possible. Because of this, existing web depth thickness limitations, transverse stiffener spacing and rigidity, and longitudinal stiffener location and rigidity for straight girders are largely based on buckling considerations.
The nominal moment strength of slender web plate girders is controlled either by the limit state of yielding at the tension flange or that of buckling at the compression flange. Inelastic behavior of the web is not considered for design purposes. LRFD design philosophy applies a reduction factor to account for the "bend buckling" effect of the slender web and a reduced ability to carry its share of the bending moment. Furthermore, there is post-buckling strength after elastic buckling of the web occurs.

For straight girders, the bifurcation behavior of the web plate is easily and accurately modeled using simplified boundary and loading conditions. However, for curved plate girders, the presence of curvature greatly complicates behavior and design considerations. Curvature induces both warping of the cross section and, more importantly for web considerations, transverse displacement of the web and also causes the longitudinal membrane stresses in the web due to vertical bending to become a nonlinear distribution through the web depth. The magnitude of this transverse displacement and the accompanying plate bending stresses must therefore be considered in the design of curved girders. The membrane stress distribution increases in nonlinearity with increase in curvature, which results in an increase in normal stress in the flanges. This results in a reduction in the moment carrying capacity of the girder.

There have been few investigations on this subject in the U.S. The web slenderness requirements in the Guide Specifications are based upon work done as part of the CURT project during the early- and mid-seventies. Japanese researchers performed both analytical and experimental research on the behavior of the curved web panel, resulting in a formulation for the reduction in strength of the curved plate girder. The reduction represented in U.S. design guides and that suggested by Japanese researchers is based solely on the curvature of the panel and represents a simple regression of analytical data. Also, the research in which design equations were based involved aspect ratios of 1.5 or less and was limited to symmetric sections. Therefore, the application of the resulting reduction equations is very limited. A methodical and theory-based approach to predicting the transverse displacement and accompanying plate bending stresses was unknown prior to the FHWA-CSBRP.

The web slenderness requirements for a curved I-girder presented in the Guide Specifications are based largely on the analytical studies conducted by Culver et al. (1972a, 1972b, 1972c, Brogan 1972). In these studies, the web panel was modeled as a series of isolated elastically supported cylindrical strips subjected to fictitious radial loading and "spring foundation" boundary conditions that simulate the continuous curved plate under bending moment (Wachowiak 1967). The stiffness of the elastic restraints was determined by subdividing the web into unit width vertical strips with length equal to the height of the portion of the web in compression and using both simply supported and fixed ends. The stress state in each cylindrical strip was determined from the total potential energy of a nonlinear arch model using the Rayleigh-Ritz method.

It was emphasized that membrane stresses in the compression region of the curved models were less than that predicted by linear theory and that there was an accompanying increase in flange resultant force. The maximum web bending stress was shown to occur at 0.20\(h\) from the compression flange for the simple support stiffness condition and 0.24\(h\) for the fixed condition,
where “$h$” is the height of the web. It was noted that $0.20h$ would be the optimum position for longitudinal stiffeners in curved girders, which is the same as for straight girders based on stability requirements. From the fixed condition cases it was determined that there was no significant change in the membrane stresses (from free to fixed) but that there was significant effect in the web bending stresses. Numerical results were generated for the reduction in effective moment required to produce initial yield in the flanges based on curvature and web slenderness for an aspect ratio of 1.0 and web to flange area ratio of 2.0. From the results, a maximum reduction of about 13 percent was noted for $a/R = 0.167$ and about 8 percent for $a/R = 0.10$ ($h/t_w = 150$), both of which would correspond to extreme curvature, where “$a$” is the distance between transverse stiffeners, “$t_w$” is the web thickness, and “$R$” is the radius of curvature. To apply the parametric results to developing design criteria for practical curved girders, the deflections and web bending stresses that would occur for the girder with a curvature that corresponds to an initial imperfection out of flatness limit of $D/120$ was used, where “$D$” is the web depth. It was noted that, for a panel with an aspect ratio of 1.0, this would correspond to a curvature of $a/R = 0.067$. The values of moment reduction using this approach were compared with those presented by Basler (Basler and Thurlimann 1961, Vincent 1969).

An extension of this work was published a year later when Culver et al. (1973) checked the accuracy of the isolated elastically supported cylindrical strips by treating the panel as a two-way shell rather than as individual strips. The flange/web intersection was modeled as fixed and the boundaries at the transverse stiffeners were modeled as fixed and simple. Longitudinal stiffeners were modeled with moments of inertias as multiples of the AASHO (1969) values for straight girders at the time. The same geometric parameters as used in the first investigation were used. Using analytical results obtained for the slenderness required to limit the plate bending stresses in the curved panel to those of a flat panel with the maximum allowed out-of-flatness ($a/R = 0.067$) and with $D/t_w = 330$, an equation was developed for curved plate girder web slenderness with one longitudinal stiffener. It was further concluded that if longitudinal stiffeners are located in both the tension and compression regions, the reduction in $D/t_w$ would not be required. For the case of two stiffeners, web bending in both regions is reduced and the web slenderness could be designed as a straight girder panel. This work was continued by Mariani et al. (1973) where the optimum transverse stiffener rigidity was determined analytically.

During almost the same time, Abdel Sayed (1973) studied the pre-buckling and the elastic buckling behavior of curved web panels and proposed approximate conservative equations for estimating the critical load under pure normal loading (stress), pure shear, and combined normal and shear loading. The linear theory of shells was used. The panel was simply supported along all four edges with no torsional rigidity of the flanges provided. The transverse stiffeners were therefore assumed to be rigid in their directions (no strains could be developed along the edges of the panels). The Galerkin method was used to solve the governing differential equations and minimum eigenvalues of the critical load were calculated and presented for a wide range of loading conditions (bending, shear, and combined), aspect ratios, and curvatures. For all cases, it was demonstrated that the critical load is higher for curved panels over the comparable flat panel and increases with increase in curvature.
Daniels et al. (1979a, 1979b, 1979c, 1980a, 1980b, Zettlemoyer et al. 1980) summarized the Lehigh University five year experimental research program on the fatigue behavior of horizontally curved bridges and concluded that the slenderness limits suggested by Culver were too severe. Equations were developed for Load Factor Design and for Allowable Stress Design.

Numerous analytical and experimental works on the subject have been published by Japanese researchers since the end of the CURT project. Mikami and Furunishi presented work in Japanese journals (1980, 1981) and later in the ASCE Journal of Engineering Mechanics (1984) on the nonlinear behavior of cylindrical web panels under bending and combined bending and shear. They analyzed the cylindrical panels based on Washizu's (1975) nonlinear theory of shells. The governing nonlinear differential equations were solved numerically by the finite difference method. Simple support boundary conditions were assumed along the curved boundaries (top and bottom at the flange locations) and both simple and fixed support conditions were used at the straight (vertical) boundaries. The large displacement behavior was demonstrated for a range of geometric properties. Numerical values of the load, deflection, membrane stress, bending stress, and torsional stress were obtained but no equations for design use were presented. Significant conclusions include that: (1) the compressive membrane stress in the circumferential direction decreases with increase in curvature; (2) the panel under combined bending and shear exhibits a lower level of the circumferential membrane stress as compared with the panel under pure bending and as a result, the bending moment carried by the web panel is reduced; and (3) the plate bending stress under combined bending and shear is larger than that under pure bending. No formulations or recommendations for design use were made. The results were briefly compared to those of Culver.

Kuranishi and Hiwatashi (1981, 1983, 1984) used the finite element method to demonstrate the elastic finite displacement behavior of curved I-girder webs under bending using models with and without flange rigidities. Rotation was fixed about the vertical axis at the ends of the panel (transverse stiffener locations). The nonlinear distribution of the membrane stress was noted but appeared significant only for extreme curvature and slenderness. Based on this nonlinear membrane stress distribution, an effective web height was demonstrated. Also, the reduction in bending moment resistance was demonstrated but, for slenderness in the design range, only a small reduction was noted. No formulations or recommendations for direct design use were made.

Fujii and Ohmura (1985) presented research on the nonlinear behavior of curved webs using the finite element method. Models included simple support, fixed support, and flange rigidities at the flange/web boundaries. The large displacement behavior was demonstrated for loads beyond the elastic bifurcation load. Also, the nonlinear membrane stress distribution was demonstrated, but the effect on resistance moment or flange stress increase was not discussed. It was emphasized that the web panel model with no flange rigidity is inadequate in estimating the behavior of the curved panel under significant loading. No specific recommendations or formulations regarding the design of curved I-girder webs were made.

Suetake et al. (1986) examined the influence of flanges on the strength of curved I-girders under bending using the mixed finite element approach. The ends of the panels (transverse stiffener
locations) were modeled as simple supports and various width/thickness ratios for the flanges were modeled. Geometric nonlinear analyses were conducted. Conclusions included that the aspect ratio of the panel was of minor importance and that the influence of the flange rigidity cannot be ignored. Also, observations were made on the torsional buckling behavior of the flanges. No quantitative formulations for design use were recommended.

Nakai et al. (1986) conducted analytical research on the elastic large displacement behavior of curved web plates subjected to bending using the finite element method. The web plate panels were modeled with and without flange rigidity. Models without flange rigidity were modeled as fixed and simple supports. The boundary conditions at the panel ends (transverse stiffener locations) were modeled as simple support. One and two levels of longitudinal stiffeners were also modeled. It was determined that including the flange stiffness is essential to extract reliable results for the behavior of the curved web panels and therefore all parametric results provided are from the use of the flange rigidity models.

It was further shown that increasing curvature has little effect on the resisting moment (less than 10 percent within the range of actual bridge parameters). This was attributed to the fact that the web contributes only a small portion to the flexural resistance compared to the flanges. It was also demonstrated that the maximum web deflection occurs in the vicinity of 0.25h from the compression flange but that this transverse deflection is effectively eliminated when one or two longitudinal stiffeners are present. The effect of curvature on the plate bending stresses was also demonstrated with respect to the effect of web slenderness ratio and curvature.

Web slenderness requirements were formulated by Nakai based on the effects of curvature on displacement and stress and proposed for adoption by the Hanshin Guidelines for the Design of Horizontally Curved Girder Bridges (1988, Kitada et al. 1986). It was suggested that limiting values should be established so that the curved web plate transverse deflection and plate bending stress be limited to the maximum transverse deflection and bending stress that would occur in the straight girder with the same dimensions but, instead of curvature, with a maximum initial deflection of D/250, which is the maximum allowable initial deflection stipulated in the Japanese design code (Japan 1990). From a comparison of the displacement results with the stress results, it was shown that criteria based on the stress requirement would result in a more conservative design. Equations were developed based on regression analyses.

Nakai and co-researchers conducted other research pertaining to the behavior of curved I-girder webs, including a series of experimental research on the behavior of the curved I-girder web under bending, shear, and combined bending and shear (Nakai et al. 1983b, 1984a, 1984b, 1984c, 1985a, 1985b). In 1983, Nakai et al. (1983b) presented the results from eight experimental test specimens under pure bending with slenderness ratios of 178 with no longitudinal stiffeners and one specimen with a longitudinal stiffener and a slenderness of 250. Aspect ratios of 0.5 and 1.0 and radii of curvature of 10 and 30 m were used. It was verified that the ideal buckling phenomenon does not occur in curved panels but rather the out-of-plane displacement of the web plate gradually increases with applied bending moment. It was noted, however, that a critical bending moment could be clearly observed.
Davidson et al. (1999a, 1999b, 2000a, 2000b, 2002, 2004, Ballance 1996) investigated the buckling, finite displacement, and ultimate strength behavior of curved I-girder web panels under pure shear, pure bending, and combined bending and shear using the finite element model. The finite element models confirmed that the elastic buckling stress for the curved panel was higher than that of the comparable flat panel under pure shear. Under pure bending, Davidson et al. (1999a, 1999b) showed that the nonlinear transverse displacement effectively reduces the moment carrying capacity of the curved section over that of the straight. Based on a “lateral pressure” analogy, curvature reduction equations on the design slenderness were developed and suggested. This work was extended to study the effects of longitudinal stiffeners on the strength and stability of curved web panels (2000b).

2.6.2 Pure Shear

The web of a plate girder stiffened by flanges and transverse stiffeners has considerable post buckling strength due to "truss" or "tension field" action from the interaction of the buckled web, flanges, and stiffeners. This ability has been well established and has been used in ASD, LFD, and LRFD format design codes for straight girders. However, the lack of research on curved girders has kept tension field action out of the Guide Specifications.

According to Basler (1961), the ability of a plate girder to behave in manner similar to a truss was recognized as early as 1898. The work of Basler led to a theory that agreed with tests and provides criteria to ensure that truss action can be developed. Considering truss action raises the shear strength from that based on buckling to approach a condition corresponding to shear yield in classical beam theory.

Numerous works have been presented over the years on the bending behavior of cylindrical curved webs, including works by Culver in which the Guide Specifications are based, but very little work has focused on the shear behavior. Early analytical work on the elastic stability of stiffened cylindrical shells subjected to pure shear was conducted by Batdorf (1947a, 1947b) and co-researchers and then by Stein and co-researchers (1949a, 1949b). In these works, equilibrium equations were derived assuming all four edges of the web panel to be simply supported. Equilibrium equations were solved using the finite difference method. Mariani et al. (1973) later extended the work of Stein and Yeager to include the case of the curved plate with multiple stiffeners under pure shear and developed an optimal stiffener spacing criterion to establish stiffener requirements for curved girder webs. From the work of these researchers it is generally agreed that, in the bifurcation sense, the critical load of the curved web panel is greater than that of the straight girder with the same aspect ratio, slenderness ratio, and boundary conditions.

Experimental research on the ultimate and post-buckling reserve strength of curved girders has been conducted by Ilyasevitch and Klujev (1971) and by Mozer et al. (1971) as part of the CURT project. It was observed from these tests that there is a decrease in the post-buckling strength with increase in curvature, although the measured shear strengths are within 10 percent of the ultimate shear strength by straight girder theory, which could be considered to be within the range of acceptable experimental error. Also, the experimental investigation by Mozer et al.
indicated that, in areas of negative bimoment (tending to bend the compression flange inward), the web behaves more like that of a straight girder and can carry the ultimate shear strength predicted for a straight girder with similar proportions.

The Japanese have also conducted a series of experimental tests on the ultimate strength of web panels under pure shear, pure bending, and their combinations (Nakai et al. 1984a, 1984b, 1984c, 1985a, 1985b). Their results agree that curvature has little effect on the elastic critical shear load but that there is some decrease in ultimate strength. Like the Guide Specifications, the Japanese design specifications do not recognize post buckling reserve strength for curved plate girders due to lack of research in this area (Hanshin 1988).

Davidson et al. (1996) presented results on the buckling and ultimate strength of curved web panels in pure shear. The finite element method was used with combined geometric and material nonlinear solution sequences to analyze some typical plate girder web panels of various curvatures. The aspect ratios of the panels were also varied to compare the effects of transverse stiffener spacing for curved panels to that of straight. The results from the buckling analyses agreed with that of previous research whereas the elastic critical load of the curved panel was determined to be greater than that of the comparable flat panel. Furthermore, the combined geometric and material nonlinear analyses revealed no substantial decrease in ultimate strength of the curved panel with respect to that of the straight (flat).

Lee and Yoo (1999) studied the bifurcation buckling and ultimate strength analysis of curved web panels subjected to pure shear using finite element method. The analysis revealed that the curved web panels are capable of developing considerable post buckling strength after the bifurcation point. The results also suggested that straight girder equations developed by Lee et al. (1994, 1996) can be used to effectively predict the shear strength of curved web panels subjected to pure shear.

### 2.6.3 Combined Bending and Shear

The review of research on the subject of curved web panels subjected to pure bending shows that a number of investigations had been performed on the finite displacement behavior (mostly by Japanese researchers) but that very little information of practical design use had been presented. Furthermore, the design formulations that have been presented are solely a function of the curvature of the panel, applicable only to symmetric sections, and applicable to panel aspect ratios no greater than 1.0.

In reality, there will generally be transverse shear present along with the vertical bending of the bridge girder. However, in the majority of straight girder situations, the nominal strength in bending is not influenced by shear, nor is the nominal shear strength influenced by bending. Particularly, in slender webs where “bend-buckling” may occur, the bending stress is redistributed so that the flanges carry an increased share. The shear strength, however, is not reduced as a result of “bend-buckling” because most of the shear strength is from tension field action with only a small contribution from the portion of the web adjacent to the flange. In
stockier webs “bend-buckling” does not occur, but high web shear in combination with bending may cause yielding of the web adjacent to the flange.

No known analytical investigation on the finite displacement behavior of the curved I-girder web panel under combined bending and shear has been published by American researchers. Japanese researchers Mikami and Furunishi (1981) presented work including shear along with bending in Japanese journals and later in the ASCE Journal of Engineering Mechanics (1984), but this work was limited to cases where the applied shear stress equals the applied vertical bending stress at the top of the panels. From these investigations it was concluded that the presence of shear along with bending did adversely affect the moment carrying capacity of the beam but no formulations for design use were presented.

Abdel-Sayed (1973) studied the pre-buckling and elastic buckling behavior of curved web panels under pure bending, pure shear, and combined bending and shear and showed that in all cases the elastic critical load of the curved panel was greater than that of the comparable flat panel. In this investigation, though, the lateral and torsional rigidities of the flanges were not modeled.

Nakai and co-researchers (Nakai et al. 1984a, 1984b, 1985a) conducted experimental studies on the buckling and ultimate strength behavior of the curved I-girder web panels under combined bending and shear. Tests were conducted on 12 specimens. A circular interaction curve was fitted to the buckling values from the tests and interaction curves also resulted for the ultimate strength of the curved girders involving the theoretical nominal strengths for pure shear and pure bending.

Under combined bending and shear, Davidson (2000a) verified that the elastic buckling load under any combination of shear with vertical bending stresses resulted in higher critical loads for the curved panel over that of the straight. It was concluded that the use of design equations presented for pure bending would result in conservative designs up to $V/V_n = 0.6$, where $V$ is the calculated shear force over the web and $V_n$ is the nominal shear resistance defined for pure shear.

2.7 Ultimate Strength of Horizontally Curved Girders

Both analytical and experimental ultimate strength investigations have been made by several researchers. Culver and McManus (1971) studied the inelastic behavior of horizontally curved girders and made design recommendations that were adopted into the LFD portion of the AASHTO Guide Specifications. Yoo and Heins (1972) studied the plastic collapse of horizontally curved girders and presented a yield criterion and design charts along with equations for practical applications. Yang et al. (1988, 1989) presented yield surface formulations for I-sections with nonuniform torsion and bimoments; similar work was presented by Imai and Ohto (1987) in Japan. Pi and Trahair (1994) presented findings on the inelastic behavior of I-beams under combined bending and torsion based on the finite element method. A series of extensive analytical studies that included the effects of large displacement and material nonlinear behavior of horizontally curved beams were performed by Japanese investigators (Yoshida and Imoto 1973, Fukumoto and Nishida 1981, Maegawa and Yoshida 1981, Yoshida and Maegawa 1983,
1984) using the transfer matrix method and assuming ideal elastic perfectly plastic material behavior. Later, similar results were obtained by Lee (1987, 1988) using a flat six degree-of-freedom triangular plate shell finite element developed by Lee. Inelastic large displacement finite element modeling has been used with varying degrees of success in predicting the behavior of curved girders (Tan et al. 1992, Thevendran et al. 1994, Liew et al. 1994).

The allowable normal flange stress in the Guide Specifications for “Allowable Stress Design” are based on the research that Culver (1972) and co-researchers conducted as part of the CURT project. The “Load Factor Design” portion of the Guide Specifications is based on research by Galambos (1978) as an extension of work performed in the CURT project at Carnegie-Mellon University. The parts of the design equations that represent the reduction in strength due to curvature are quite complex and cumbersome.

The Hanshin Expressway Public Corporation proposed an interaction formula for limiting the stresses in horizontally curved I-girders for adoption into its Guidelines for the Design of Horizontally Curved Girder Bridges (Hanshin 1988, Nakai and Yoo 1988). The equations represent interaction for allowable stress in the compression flange including the presence of warping and the reduced lateral buckling strength of the girder due to curvature. It is based on theoretical and experimental research in the elastic range by Nakai and co-researchers (Nakai and Kotoguchi 1983, Nakai, Kitada, and Ohminami 1983).

In research by Fukomoto and Nishida (1981) using the transfer matrix method for both the elastic and inelastic ranges, an approximate ultimate strength formula was presented involving the plastic moment capacity of the section, the elastic buckling moment of the straight beam with the same length and cross section dimensions, and the elastic buckling load of the entire section about the weak axis.

Nakai et al. (1985a) presented an empirical equation for ultimate moment based upon 19 tests in which the elements comprising the cross sections are classified as compact, and the \(a/d\) ratio is less than one (where \(a\) is the distance between transverse stiffeners and \(d\) is girder depth).

In several investigations (Kang 1992, Yoo and Pfeiffer 1983, Yoo, Kang, and Davidson 1996), it was demonstrated that a large variation in the torsional rigidity ratio has little effect on the critical load ratio (curved/straight) for the lateral buckling of horizontally curved girders loaded normal to the plane of curvature and that the subtended angle is the dominating parameter. A curvature reduction formula was derived from a regression of data resulting from an elastic finite element investigation using curved beam elements which include warping (Kang 1992). Although this strength reduction equation was developed based upon an elastic analysis, it was proposed (Yoo, Kang, and Davidson 1996) that the reduction in critical moment of curved girders results from the presence of the rotational component of the girder behavior and likewise there would be a similar reduction in ultimate moment capacity. Previous ultimate strength tests by others on curved I-girders appear to verify this conjecture (Yadlosky 1993, Yoo, Kang, and Davidson 1996).
Yoo and Davidson (1997) presented yield interaction equations that were based on the static equilibrium of the I-shape girder under vertical moment and lateral flange moments resulting from the inherent non-uniform torsion present in the curved open I-section. Equations were presented for singly symmetric composite and non-composite I-shapes for both positive and negative moment zones. Complete plastification for compact sections, partial yield penetration for the compact-flange sections, and initial yield at the flange tip for non-compact sections were considered for a total of 18 interaction cases. A computer program was created and the reduction due to curvature was demonstrated for a number of hypothetical cases.

Davidson and Yoo (2000, 2003a, 2003b) presented the results of finite element models representing a curved three-girder test frame planned under the FHWA-CSBRP experimental phase. The models were used to evaluate the effects of curvature on the bending strength of curved I-girders. Linear elastic static, buckling, and combined material and geometric nonlinear analyses were conducted using models that represent the test frame and component test specimens that will be inserted into it. The results are compared to various predictor equations developed from analytical work by other researchers, including Japanese research not readily available in the U.S. Predictor equations were also compared for such parameters as: warping stress, elastic flange buckling curvature reduction, and curved-web maximum displacement and maximum stress.
3.0 Synthesis of I-girder Research and Current Practice

3.1 Problems Associated with Construction

Curved steel I-girder bridges comprise almost one third of the total steel bridge market in the United States (Burrell et al. 1997). Given the rapid increase in use of curved I-girder bridges, it is essential to understand construction aspects of the bridge. However, research carried out over the past 30 years was primarily focused on strength issues of curved girders, which assumes importance only after the girders have been set in place with deck hardened. Once the deck is cured, stability against lateral torsional buckling and local buckling stability of the top flange is provided. Curved steel I-girder bridges are relatively stiff and strong after the girders have been erected and the concrete deck hardened. However, prior to stabilization and hardening of the concrete deck, the structure may be quite flexible and vulnerable to stability problems during construction (Galambos et al. 1996). Instability in curved steel I-girder bridges generally occurs during lifting and erecting, resulting in excessive displacements and stresses that are typically unaccounted for during the design. As a part of the Federal Highway Administration’s (FHWA) Curved Steel Bridge Research Project (CSBRP), Zureick et al. (1994) and Zureick and Naquib (1999) summarized 3 decades of research accomplished through analytical, experimental and theoretical methods. Of the large number of references collected, very few addressed construction aspects of curved girders.

Problems with curved girders occur from the very beginning. Residual stresses formed during girder fabrication can be significantly greater than those induced to straight plate girders and affect the strength of the girder. Ensuring proper camber is difficult. Transporting and shipping girders with significant curvature becomes problematic. Girders must be carefully restrained to prevent instability during shipping and overhangs must be carefully checked.

Once at the job site, placing the girders becomes cumbersome. Curvature results in lifting procedures different from those typically used in straight bridge erection. Engineers developing erection plans must carefully determine whether to lift the girders singly or two at a time with cross frames attached, whether to lift with one or two cranes, where the pick points should be to avoid instability during lifting, etc. Bridges supporting curved alignment are typically constructed for busy interchanges where construction space and traffic detour time must be minimized, which often affects the design of the construction sequence and methodology.

During the construction of straight bridges, girders and stringers are easily erected by one crane using one or two pick-up points, or by two cranes using one pick-up point each. The individual straight girders can simply be set in place with little concern for instability. Lifting and setting presents less difficulty for straight beams where the center of gravity coincides with the centroidal axis of the beam cross section. For horizontally curved girders, the center of gravity is non-coincident with the cross section centroid. Thus, depending on the lifting and support mechanism used, significant torsional stresses and minor-axis bending stresses may be induced.
Curved I-girders, in particular, are unstable during lifting and transporting unless adequately braced.

Once the girders are in place, elevation and fit-up problems are common. Erectors must often resort to rather severe tactics to get all of the superstructure pieces to fit appropriately. Careful design of temporary shoring is critical. Once all superstructure components are in place but prior to deck hardening, dead loads induce significant torsion and distortion stresses in the girders, which cause instability of the entire system. The designer conceives of the structure as a completed entity, with all elements interacting to resist the loads. Stability of the completed structure depends on the presence of all structural members such as girders, cross frames, concrete deck, and connections. After deck pouring, full resistance of the girder system has not been completely developed as the concrete in the deck slab has not yet hardened and does not have enough resistance to carry lateral forces and vertical loads. Furthermore, the configuration of the incomplete structure is constantly changing during construction, and stability often relies on temporary bracing. The safe assembly of a well-designed framework of girders and cross frames requires awareness that the components by themselves are not necessarily stable. At no stage in the assembly is the factor of safety equal to that of the completed structure unless temporary support is designed carefully to ensure safety. Analysis of the stability requirements for incomplete and constantly changing assemblies presents a challenging problem to regular straight bridges and more so in case of curved ones.

The construction of horizontally curved girders requires engineering expertise beyond that needed for the construction of straight bridges. Problems are common, yet rarely documented. However, there are no comprehensive guidelines or recommendations on construction practices, nor is there any published survey or summary of problems encountered by state departments of transportation during construction of curved bridges. Advanced analysis methods are rarely used to analyze the bridge for a given erection sequence. Engineers not experienced in the design of curved bridge systems often make the mistake of assuming that behavior and design is the same as that for straight bridges. Instability during construction can easily translate into unsafe conditions for construction workers, not to mention unforeseen additional costs.

Therefore constructability issues must be given due consideration during curved bridge design. The length of the girder sections and location of field splices must be designed with consideration of site conditions and the capabilities of equipment. Also, the designer must provide allowance for additional stresses induced during fabricating, lifting, handling, transporting and erecting of curved girders. In general, all stages of construction before deck hardening must be taken into account during curved bridge design.

The 2003 Guide Specifications requires the contractor to provide a construction plan that details the procedures for fabrication, erection and deck placement. The construction plan may be developed based on the design plans, or developed entirely by the contractor. The contractor’s plan will be considerably more detailed than the construction scheme provided on the design plans and, owing to the complexity of curved bridges during construction, should be stamped by a professional engineer and approved by the owner. As minimum requirements, the construction plan should include (AASHTO 2003):
1. Fabrication procedures, including the method of curving the girders;
2. Shipping weights, lengths, widths, heights, and means of shipping;
3. Erection plan, including the sequence of erection, crane capacities and positions, and the location, capacity and elevation of any temporary supports; and
4. Deck placement sequence, including the time between casts and the magnitude and position of any temporary load required to prevent girder lift-off at bearings.

3.2 Fabricating

An essential fabrication requirement is to ensure that the steel can be assembled in the no load condition by the fabricator, unless specified otherwise in the construction plan. In addition, the flanges and webs fabricated from rolled shapes and plates should be handled in a manner as to prevent visible deformation or incidental damage. Generally there are three methods that can be used to fabricate curved steel I-girders: (1) cut curving, (2) heat curving, and (3) cold curving.

3.2.1 Cut Curving

Cut curving involves flame cutting the flanges to the desired curvature from a standard steel plate. The advantage of this method is that there is no limit on the radius of curvature that can be obtained. This method of fabrication involves careful planning for economical cutting of the flange plates to minimize the amount of scrap generated (Thatcher 1967). In addition, adhering to consistency in plate thickness and steel grades as much as possible in the design allows the fabricator to economize by combining and nesting plates, since the plates must be purchased in minimum widths of 48 in. (Grubb et al. 1996). After the individual flange sections are cut, the required sections that make up the shipping piece are usually spliced together by full penetration butt welds. The web plates cut for camber are held to the required curvature by special fixtures that are tack welded to the flange plates. After tack welding, automatic welding is used to weld the full length of the web/flange intersection. A disadvantage of this method is that special fixtures are required for holding the flange and web during fit-up. After fabricating, the girder is checked for required curvature. Adjustments to required curvature are often made by controlled application of heat.

3.2.2 Heat Curving

Heat curving is the plastic deformation of metals at temperatures above the recrystallization or work hardening range (Wick 1960). Heat curving produces a low resistance to plastic deformation and improved mechanical properties in the steel. It is one of the economical and popular methods of fabricating curved steel I-girders and generally used for longer radii. Heat curving is usually accomplished by simultaneously heating one side of the top and bottom flanges of a fabricated straight I-girder to introduce residual curvature after cooling. The
application of heat can be (1) continuous, (2) strip, or (3) V-type. In continuous heating, the flange edges are heated continuously along their length. In strip heating, the flanges are heated in rectangular strips at regular intervals until the required curvature is attained. In V-type heating, the top and bottom flanges are heated in truncated triangular or wedge shaped areas having their bases along the flange edge and spaced at regular intervals along each flange.

The spacing of the torch and temperature required depends on the amount of curvature to be induced in the girder (California Department of Transportation 2002). The rate of heating along the top and bottom flange is required to progress approximately at the same rate. It is good practice to terminate heating just before the web/flange juncture and not apply directly to web. For girders with radius of curvature of 300 m (985 ft) or more, heating can proceed up to the web/flange juncture; for radius of curvature less than 300 m (985 ft), the heating must be extended beyond the web/flange juncture to a distance equal to 1/8th of the flange or 75 mm (3 in.), whichever is less. In case of V heating, it is necessary that the angle be limited to 30 degrees and the base of the triangle not exceed 254 mm (10 in). Approval of the engineer is required for deviations in fabrication from the above guidelines.

For all three types of heating (continuous, strip and V-type), the edges of the flanges that are heated form the inner (concave) edge of the horizontal curvature after cooling. When the flange thickness exceeds 32 mm (1 ¼ in), the flanges must be heated concurrently on both sides of the flange. The heat curving operation must be conducted with a temperature not exceeding 620º C (1150º F) and be measured by temperature indicating crayons or other suitable means. Once the heat curving operation is completed, the girder is allowed to cool naturally to 315º C (600º F). Artificial cooling methods are employed only after the girder has cooled to 315º C (600º F). Again, any deviation in the fabrication procedure is carried out with the approval of the engineer.

The heat curving operation can be carried out with the web in the vertical or horizontal position. When the web is in the vertical position, care is taken using braces or supports such that the tendency of the girder to deflect laterally during the heat curving process will not cause the girder to overturn. When the web is in the horizontal position, the girder is supported at the ends and at the intermediate supports to obtain uniform curvature. The distance between the intermediate supports is maintained such that the bending stress in the flanges due to the mass of the girder does not exceed 186.2 MPa (27,000 psi). In addition to the girder support at the ends, intermediate safety catch blocks are placed at the midlength of the girder within 50 mm (2 in.) at all times during the heating process to prevent sudden sag due to plastic flange buckling (North Carolina Department of Transportation, 2001).

Intermediate transverse stiffeners can be attached either before or after heat curving. However, the stiffeners are attached only to the web; welding to the flanges is carried out after the required curvature has been obtained. Bearing stiffeners are usually attached after heat curving unless provisions are made for shrinkage of girder components. If longitudinal stiffeners are required, they can be either flame cut or heat curved and then welded to the girder. Cambering the girders is also required before heat curving. The girder webs are cut to the required camber taking into account the allowance for shrinkage due to cutting, welding, and heat curving. Once the heat curving operation is completed and the girders have cooled to a uniform temperature, the girders
are checked for horizontal curvature and vertical camber for final acceptance. Although the procedures for heat curving may look daunting, the advantages of heat curving are significant compared to cut curving in that they produce little or no scrap. In addition, the fixtures that are generally used for straight I-girders can be used for heat curving operations, which eliminates the need for special jigs and fixtures required for the cut curving method.

Recently Sen et al. (2003) studied the applicability of AASHTO (1996) provisions with regard to heat curving I-girders of HPS 485W sections. A three dimensional finite element analyses using NASTRAN was used to assess the applicability of the provisions. Based on the analyses, it was concluded that the girder would develop 50 percent of curvature if the same heat/cool cycle used for Grade 250 steel would be used. This points to a need for using higher temperatures for heat curving of HPS 485W sections and an optimal maximum temperature of 676º C (1250º F) was suggested.

### 3.2.3 Cold Bending

The third method of fabricating a curved I-girder is to cold-bend a fabricated straight I-girder into the required curvature. In the cold-curving process, the straight I-girder is bent plastically to obtain an over bent curvature and released. The relaxed configuration results in the required curvature. While heat curving is currently the most economic method of fabrication, it is a labor intensive process that is not exact. Similarly, cut curving is labor intensive in that handling of curved shapes in the shop is much more expensive than handling a straight girder and usually involves wastage of material (NCHRP 2002). In comparison, cold curving is fast, efficient, and precise, and could possibly be one of the most economical methods of fabricating a curved I-girder. But the possibility of fracture due to localized load effects has raised concerns and has delayed its acceptance for bridge structures (NCHRP 2004). Guidelines and procedures for fabrication are being developed to make use of this technology.

The Transportation Research Board, Committee on Fabrication and Inspection of Metal Structures, identified Cold Curving of Steel Bridge Girders as one of the problems to be tackled (NCHRP 2004a). The Technical Activities Division of the committee has invited research proposals in the area of cold curving. The goal was to advance the practice of cold bending through analysis and appropriate full-scale experimentation (NCHRP 2004b). Numerical and theoretical analyses are necessary to optimize the cold bending process and to establish safe limits on maximum loads, strain, and curvature. The allowable loads should also take into account the fatigue characteristics of the cold curved members when subjected to cyclic loads. Currently, research related to cold bending has been developed through fabricator efforts and not through public funds. Therefore, fabricators who have formulated innovative cold bending systems are unwilling to disclose proprietary information and hence little published information is available on the cold bending process.
3.2.4 Residual Stresses

In general, plate girder fabrication introduces residual stresses and camber loss. This is particularly true for heat curved girders. It is essential to understand the residual stress pattern developed and amount of camber loss. Residual stress is unrelieved stress remaining in a structure or metal part as a result of welding or cutting (Shin and Walter 1981). Residual stresses are created whenever a member is permanently deformed or distorted in a non-uniform manner and persist in a material or a component under uniform temperature in the absence of externally applied loads. Every metal working process induces residual stress. In case of curved girders, residual stresses consist of two parts: one due to manufacturing of the plate girders as straight and the other induced from the curving process. The objective of studying the different types of fabrication procedures is to understand the residual stress distribution pattern induced during the curving process.

In case of cut curving, there is very little information available on the residual stresses developed in flame cut and welded I-section curved girders (Bradford et al. 2001). The fabrication process for a flame cut and welded curved girder is similar to that for a straight girder formed from welding flame-cut plates. Based on the research by Kishima et al. (1969), Culver and Nasir (1969) suggest a flange residual stress pattern for welded I-section as shown in Figure 3-1. Here 2b is flange width. \( R_o, R_w \) and \( R_i \), refers to radius of curvature at the outer flange edge, at web-flange intersection and at inner flange edge. The term \( \sigma_{rc} \) and \( \sigma_{rt} \) refers to residual stress at compression and tension. Note that the residual stress pattern is symmetrical with respect to the web line.

Heat curved girders are obtained by introducing thermal stresses in a fabricated straight girder to begin yielding in the top and bottom flange edge. During cooling, a residual curvature is introduced. In 1968, U.S. Steel initiated a study to investigate heat curving effects on residual stresses, strains, and curvatures (U.S. Steel 1973). The research was carried out by Brockenbrough (1970), who studied analytically and experimentally the residual stress pattern in curved girders fabricated by heat curving. Brockenbrough reported that the magnitude and distribution of heat curved girders is a function of dimensions and material properties of the
straight girder and the heat curving procedure. Since this research showed no damage to the steel and no loss of strength, heat curving has become an accepted means of fabrication. Figure 3-2 shows the flange residual stress pattern of heat curved girders with notation the same as in Figure 3-1.

![Figure 3-2. Residual stress in heat curved girder (Culver and Nasir 1969)](image)

Culver and Nasir (1969) studied the residual stresses developed by cold bending a straight beam by considering the stresses in a beam loaded into the inelastic range. The beam was subjected to a constant moment about the weak axis of sufficient magnitude to cause partial yielding of the flanges of the beam followed by elastic unloading. In addition to residual stress, residual curvature is also developed in the center portion of the beam due to straining in the inelastic range. To obtain the final residual stress, the stress distribution at various stages of the operation was considered. First the residual stresses present (residual stresses in straight girder due to welding) before the bending operation has to be considered. Second, the stresses produced in the inelastic range due to cold bending and finally the residual stresses developed due to elastic unloading of the beam has to be considered. Since the beam unloads elastically, the unloading stress is linearly distributed across the flange width. Depending on the amount of cold bending applied to obtain a desired curvature, two different residual stress patterns develop as shown in Figure 3-3.

Most curved beams in practice have radius of curvatures less than 762 m (2500 ft) and the corresponding residual stress is shown in Figure 3-3c. Based on previous studies, Culver and Nasir (1969) concluded that the amount of initial cooling residual stress does not significantly influence the final stress pattern since the strains due to the applied moment are considerably greater than the yield strain. Therefore the residual stresses present in Figure 3-3c may be valid for both cold bent rolled beams and cold bent girders fabricated from straight welded plates.

To obtain the residual stress pattern for steel with different yield stress, flange width, ratio of web to flange area, a computer program was developed by Culver and Nasir (1969). The average value of the results from the computer program is taken as standard residual stress pattern for cold bent girders as shown in Figure 3-4. Daniels and Batcheler (1979) concluded
that the residual stresses and strains that are developed due to heat curving have little or no effect on the fatigue strength of the girders.

Figure 3-3. Cold bending residual stresses (Culver and Nasir 1969)

Figure 3-4. Standard residual stresses in cold bent girders (Culver and Nasir 1969)
3.2.5 Camber

When a girder is fabricated, the fabricator should have precise information on the amount of camber to be incorporated during fabrication. There are various factors that must be considered before precise information on the amount of camber can be calculated. A loss in camber can occur due to heat curving of the girder, dead load of the girder, reinforcing steel, deck placement, construction loads, and service loads. Also, a loss or gain in camber is possible due to thermal loads, depending on the direction in which the girder is placed. An illustration of noticeable camber in a painted straight girder is provided in Figure 3-5.

Research has been carried out to estimate the loss in camber due to heat curving. AASHTO developed an equation to calculate total amount of camber. Hilton (1984) studied a cambered curved I-girder for a period of seven months subsequent to construction. Based on the studies, Hilton (1984) estimated that the total amount of camber loss was only 13 percent of what the AASHTO equation had predicted (AASHTO 1993) and concluded that loss of camber due to heat curving was insignificant. New York has used thousands of heat-curved girders over the past twenty years without implementing the AASHTO requirement for additional camber. All of their curved and straight girders have additional camber added due to any camber loss from weld shrinkage as standard practice. There have been no adverse affects reported from camber loss due to heat curving (Grubb et al. 1996). Based on experience, Equations 3-1 and 3-2 were introduced as modification factors into the AASHTO camber-loss equation in the 1988 interim specifications.

\[
\frac{305 - R}{259}, \text{ where } R \text{ is the radius of curvature in meters} \quad (3-1)
\]

\[
\left[\frac{1000 - R}{850}\right], \text{ where } R \text{ is the radius of curvature in feet} \quad (3-2)
\]

Camber can be induced by applying heat or the web can be cut out to proper camber before the flanges are welded to the web. The flanges are then fitted to the curved web and welded. Camber is not normally used in horizontally curved I-girders to resist twist and rotation. Since the girders are flexible, adequate bracing will keep the girder from warping. Camber used to offset lateral rotations has generally not been specified.

The residual stresses induced during curving are in addition to the residual stresses induced in a straight girder due to welding of flange plates with the web. Understanding the method of fabrication gives the analyst a correct estimate of residual stress values to be used in the analysis. Therefore, it is important that the construction specifications for curved steel I-girder bridges spell out the method used for fabrication and research should consider residual stress distribution.
3.3 Transporting

An important aspect of curved steel bridge construction lies in proper shipping of the fabricated sections. The steel bridge components can be transported by highways, railways, waterways or a combination of the three, depending on where the bridge is to be constructed.

3.3.1 Highway

Usually, steel bridge components are transported by trucks on public highways. The development of interstate systems has facilitated easy movement and economical transporting of girders over long distances. In addition, it has also increased the size of the pieces that can be transported by truck. Accessibility of the truck to the site on temporary or access roads should be evaluated if transporting by truck is planned. Transporting methods depend on the length and width of the shipping pieces and must be considered in the design. Quite often, the legal limit of shipping and weight is not readily available. However, most states have rules regarding the normal load size and weight permitted on the highway system. Depending on size and weight limitations, escorts or special permits may be required. The following are general guidelines provided for straight girder transportation (Highway Structures Design Handbook 1997):

Length – Shipping pieces (field pieces) up to 45.7 m (150 ft) are possible, and most states allow up to 24.4 m (80 ft) without restrictions.
Height – Bridge or power line clearances govern the height of the load. A 3.7 m (12 ft) load height is a usual clearance limit, but 4.3 m (14 ft) is often possible.

Width - The normal unrestricted load width limitation is 2.4 m (8 ft). Widths up 4.9 m (16 ft) may be possible with permits, escorts, and limits on the time of day or the day of the week of travel. Shipments 6.1 m (20 ft) wide have been known to be accomplished in open country on the Interstate system. However, states would restrict the time of day for transit of a load this size.

Weight – Piece weights of 18.1 metric tons (20 tons) are normal and transportation of loads up to 36.3 metric tons (40 tons) are commonly accomplished with permits. Weights up to 90.7 metric tons (100 tons) are possible but require close cooperation with the state DOT.

Flange Size - A minimum of 360 mm (14 in.) is essential. Increasing the flange width makes shipping and erecting easier and is especially true for curved girders.

3.3.2 Rail

From the standpoint of sizes and weights that can be moved, it is often advantageous to use rail transportation. The height of shipping piece can be permitted up to 4.9 m (16 ft), depending on routing, and 90.7 metric tons (100 ton) weights are possible. The width is usually limited to 2.4 m (8 ft) but after careful review of routing, widths up to 3.7 m (12 ft) may be permitted. Using three or five car units, longer lengths are possible. The site is not usually accessible by railroad and often the pieces must be off-loaded and trucked the final distance.

3.3.3 Waterways

If the fabricator’s plant and erection site have easy access to water on the same or contiguous navigable waterway, then considerable potential is available for transporting via barge. The advantages of this method include larger weight, size, and the potential for subassembly. It is commonly used for major river crossings and, in terms of dollars-per-ton mile, water shipment is the most economical method. It is also feasible to fabricate and assemble entire bridge spans, float them into position, and erect them onto bearings directly from the barge.
The location of splices and the overall geometry of the section can be utilized so that the girder to be shipped will be easier to maneuver. Proper restraint against vertical, longitudinal, and transverse movement must be considered as well as transporting restrictions. For straight girders, the lateral support is provided only at reaction points while curved girders may overhang and additional support must be provided at adequate locations to prevent “roll” over. The use of a dolly and restraining cables used in highway transportation is illustrated in Figure 3-6.

In general, girder sections are assembled in the fabrication yard under no dead load condition before shipping to ensure proper fit-up of girders. It is the responsibility of the fabricator to properly ship such that handling of the girder results in minimal stresses and cross section shape is maintained. The 2003 Guide Specifications suggests that a transportation plan be required by the owner for complex or large structures. The transportation plan should indicate the type of girder supports and locations. Additionally, the type, size, and locations of tie-downs, along with the number of tie-downs to provide adequate redundancy, should be shown.

Other suggestions by the 2003 Guide Specifications are:

1. The shipping pieces should be in the same orientation as in the completed structure.
2. The computed girder stresses due to handling must not exceed the critical stresses specified for noncompact flanges on a single web.
3. Fatigue stresses should not exceed the constant amplitude fatigue threshold as specified in AASHTO LRFD.
4. The supports should ensure that dynamic lateral bending stresses are controlled. Single unbraced I-girders should be cantilevered no more than \( L_c = 43b^{0.25} \) where \( b \) is the minimum flange width in inches.

5. A 100 percent impact allowance should be provided to account for dropping of girders on rigid supports.

6. The temporary stiffening trusses or beams should be specified in the transportation plan.

In essence, more restrictions apply to provide stability to the structure during transporting due to effects of curvature.

The study of literatures in transporting curved I-girders indicates that the method of transporting the girder depends primarily on bridge site location. The mode selected for transporting the girders should ensure that the girders can be easily delivered to the site without deforming the cross section and inducing additional stresses. The mode of transporting the girder can be a combination of highway, railway, and waterway, and limits the girder sections in weight, height, and width. Therefore, shipping methods often drive design decisions and designers should consider these issues.

3.4 Erection

Bridge engineers have been primarily responsible for the structural integrity of the completed structure and not for the partially completed structure or during the “erection stage” of the job (Weinhold 1997). This responsibility is rapidly changing with new specifications requiring the design engineer to investigate the stability of the partially completed structure and is particularly true during erection of curved girders. While straight girders require minimum amounts of bracing and shoring, the lateral stability of the curved girder during the erection is a primary concern.

In general, site conditions dictate how the girders will be delivered and erected. Minimal changes to site conditions can be made but conditions such as overhead power lines, roads, navigable canals, rail roads, streams, river or wetlands often do not allow for adjusting the site conditions (Weinhold 1997). Proper planning during the design stage may eliminate additional costs that may occur due to changes in site condition.

To prevent stresses induced by uneven surfaces, the girders are generally not placed on the ground. Therefore the most cost effective solution for girder erection is to pick the girder directly from the truck after arrival from the fabrication yard. For straight sections, the girders are lifted from the premarked center of gravity and can be easily put into its final position without any false work or temporary support. However, for curved girders, the pickup points are approximately determined by “weighing” a girder section. Compared to straight girders, curved girders require extensive shoring and falsework to support the girder during erection. Also, cranes with higher capacity are required if erecting is carried out with girders in pairs connected to cross frames. Therefore, adequate space for the maneuvering of the cranes and
placement of shoring is much more critical in curved girder bridges. In general, erecting includes lifting of girders, sequencing or site assembly, and erecting cross frames.

3.4.1 Lifting

Overall stability of single long slender girders during lifting is a major concern during the construction of highway bridges. These girders are often erected by one crane using one or two pick-up points, or by using two cranes with one pick-up point each. Lifting of such girders in straight bridge construction presents little difficulty as the center of gravity coincides with the centroidal axis of beam cross section. However, a horizontally curved girder introduces twisting effects during lifting as the center of gravity does not coincide with the centroidal axis of beam cross section. Twisting effects are exacerbated in cases of I-shaped cross sections as they are “open” sections with low torsional rigidity. In addition, depending on the length of the beam, lateral-torsional buckling or significant geometric nonlinear behavior may occur, thus shifting the center of gravity and causing rigid body instability. A typical lifting of a curved girder with a single point and two points are illustrated in Figures 3-7 and 3-8.

Figure 3-7. Crane lifting girder with one lifting point
The calculation of optimum pick points for two lifting points can be obtained by treating the curved girder as a circular arc in plan. Grubb et al. (1996) suggests that approximate pick points can be located at the intersections of the arc with a horizontal line through the center of gravity as shown in Figure 3-9.

Figure 3-9 (Grubb et al. 1996) estimates the approximate pick up points based on the assumption that the section is prismatic. However, bridge girders are often non-prismatic, which creates additional problems for locating the balance points. In practice erectors often “weigh” a piece; the girder may be lifted a few inches and put down repeatedly until the balance points are located. For curved girders, this may take several trials. Wire rope slings or girder clamps are usually used for attaching the free edges of the top flange at the pick up points. Due to the inherent tendency of a curved girder to twist, high stresses may occur at the attach locations. These intense stresses may occur on the inside (concave) of the girder or outside (convex) of the girder, depending on the direction in which the girder rotates. Depending on the length of the beam, different types of lifting and support mechanisms can be used. While longer girders necessitate the use of spreader beams, shorter girders can be lifted with single or double cable slings. In addition, when inclined cables are used, a component of the cable force in the horizontal plane that causes minor axis bending must be taken into account.
Davidson (1996) analyzed the lifting of single girders using the three different types of lifting mechanism illustrated in Figure 3-10. Lifting scheme 1 simulates the girder lifted vertically at the center of the span. Lifting scheme 2 simulates the girder lifted vertically by cables at two locations using a spreader beam. The two locations correspond to the intersection of a line through the center of gravity of the curved I-girder or at quarter points along the span. Lifting scheme 3 simulates the same condition as lifting scheme 2 except that the spreader beam is replaced with inclined cables. The cables are attached to a single lifting point directly above the center of gravity of the curved I-girder or at quarter points along the span. Lengths of the girder were chosen to have a consistent $l/d$ for each lifting scheme, where $l$ is the total length and $d$ is the depth of the girder. An $l/d$ of 20 was used for lifting scheme 1 while an $l/d$ of 30 was used for lifting schemes 2 and 3. Loading of the girder was taken as three times the self weight to enhance nonlinear behavior.

The results from the finite element analyses indicated that lifting scheme 1 is usually effective for lifting short beams with shallow curvature. The girder has a tendency to “roll” over when lifted at the center of span by a single cable as the lifting point does not coincide with the center of gravity. In addition, the localized stress concentrations at the lifting points may present problems as the entire weight of the girder is supported at one location along the span. Lifting scheme 3 presents problems due to significant minor axis bending from inclined cables during lifting. Also, since the cables are attached to the top flange only, the top flange could experience significantly more internal force and moment than the bottom flange. Therefore, this lifting scheme is also not recommended for use except for short spans with shallow curvature. The finite element results indicate that lifting scheme 2 is a much better lifting scheme compared to lifting schemes 1 and 3.
Figure 3-10. Lifting schemes analyzed by Davidson (1996)

Figure 3-11. Lifting of a pair of I-girders using inclined cables
Two parallel girders can be bolted together by the diaphragms or cross frames and lifted as one piece as shown in Figures 3-11 and 3-12. Lifting of girders in pairs help resist wind loads and may save time. However, the ability to lift two girders at once depends on crane capacity. The pick-up points are located near the quarter points along its arc length. In addition to cross frames, another alternative is to add horizontal stiffening trusses to the compression flanges before lifting. Research by Schelling et al. (1989) indicates that horizontally stiffening trusses (lateral bracing) can dramatically increase the stability of curved I-girders during lifting. Additional details of Schelling’s research are provided in Section 3.4.2 (Erecting and Sequencing).

Compression in the bottom flange of the girder at lifting points due to self weight should be considered. If the lifted piece is located in positive bending regions of the finished structure, then the bottom flange is designed for tension. Although this may not be of concern for I-girder flanges, the bottom flanges of box girders should be considered as they are relatively thin and vulnerable to buckling. The designers must be aware of this and should provide nominal stiffening for bottom flanges of box girders to take care of stresses inducing during lifting (Grubb et al. 1996).

### 3.4.2 Erection and Sequencing (Site Assembly)

During lifting and erecting, the possibility of long and slender girders to buckle laterally must be investigated for both straight and curved girders. Stability is achieved by adequate lateral bracing of compression flanges. For straight girders, an approximate determination of the stability of a girder may be made by taking the ratio of the overall length of the girder to the compression flange width. A rule of thumb based on experience (Weinhold 1997) indicates that girders with $l/b < 60$ will be stable during erection ($l$ is the total length of the girder being lifted and $b$ is the top flange width). For $60 < l/b < 80$, stability is questionable, but can be achieved. For $l/b > 80$, the girder will be unstable and will require temporary support. Presently, such
rules of thumb are not available for curved girders. More stringent limitations will be required for curved girders as they have an inherent tendency to “roll over”.

Proper erecting and sequencing of curved girders is essential during construction. The entire placement sequence of the girders and diaphragms or cross frames should be thoroughly planned so that fit-up problems are minimized. The diaphragms or cross frames are bolted between each girder to provide stability and to control deflection of the girders. The fabricator normally assembles the bridge components prior to delivery to the jobsite to ensure that fit-up problems will not occur. Once the girders have been loaded for transporting and unloaded for placement, camber changes may cause further fit-up problems. In addition, the configuration of the partially completed structure must be stable. Unlike straight girders, curved girders depend on adjacent girders for stability (Grubb et al. 1996). With the erection of individual girders, the load transferred between the girders through the cross frame changes along with changes in deflections and girder rotations. Thus, the deflection of the girder at a particular instance during erection depends on the sequencing and connection of a girder to adjacent girders.

The fabricator working with shop drawings may not have thorough knowledge of bridge construction. Therefore it is the designer’s responsibility to consider all factors that affect the erection and fit-up of the girders. Preassembly or girder fit-up done at the fabrication shop before shipping is based on “no-load” conditions. It is the contractor’s responsibility to achieve the same “no-load” condition in the field using appropriate temporary supports (Grubb et al. 1996). Problems arise during erection when field conditions do not represent the shop conditions based on which the fabrication was carried out. Improper shoring leads to immediate deflections and rotations. These effects are larger for long span girders with larger curvatures and further aggravated when supports are skewed. This results in several problems:

1. Difficulty in connecting cross frames;
2. Changes in slopes and elevation at field splices;
3. Changes in final steel elevations resulting in elevation changes for deck forms and screed rails; and
4. Possibility of bearings reaching allowable rotation designed for dead plus live load.

Several approaches can be adopted for erecting and stabilizing curved girders. Grubb et al. (1996) describes three methods for proper erection and stabilization. In the first method, if the crane capacity is available, paired erection is desirable. After erecting the first pair of girders, individual girders can be erected successively and connected to adjacent girders by cross frames. This increases the torsional stiffness thereby adding stability to the system. Cranes with adequate capacity to lift a pair of girders are essential for this method. A second method of erecting each girder is to use one crane to pick up the girder and place it while another crane supports the girder to which it is connected. While both cranes hold their girders, the diaphragms or cross frames are bolted into place. The addition of the second girder and cross bracing between them changes the governing instability mode from flexural torsional buckling of single girder to flexural buckling of two girders acting together as a unit. This method requires adequate area for mobilization of two cranes. A third method is to use temporary false work
towers or bents to shore the girders. Sufficient area for shoring the curved girder is necessary for this method. In general, the method chosen is determined by site conditions, equipment availability, contractor preference, and permission of the engineer.

Currently there are no widely adopted guidelines for erecting curved I-girders. Few researchers have studied the large scale erection behavior of curved I-girders. Important recent studies include Linzell (1999), Galambos et al (1996), Simpson (2000) and Chavel and Earls (2001). Although several CURT era researchers studied the behavior of curved girders through experimental methods, a primary drawback was that they included only small-scale tests of model bridges and of medium-scale models of individual components under idealized loading and boundary conditions. The findings from small scale tests and individual components cannot be extrapolated for actual girders during construction. Based on finite element analyses, Davidson (1996) demonstrated that, unlike straight bridges where each girder can be isolated and analyzed, curved bridge analysis should be carried out based on a system wide behavior. The curved girders depend on the adjacent girders for strength and stability. The Federal Highway Administration realized the lack of definitive research in the area of curved steel bridge and initiated the Curved Steel Bridge Research Project (CSBRP) in 1992. An important task of the CSBRP was the testing of large-scale curved bridge girder sections under realistic boundary conditions.

As a part of the CSBRP, Linzell (1999) conducted a series of studies on an experimental, full-scale curved steel bridge structure. The primary objective of the study was to assess the capability of analytical tools for predicting response during erection. Nine erection studies, examining six different framing plans were carried out. The tests studied different framing plans under self-weight with differing degrees of shoring support that would test the robustness of analysis tools and assess the importance of erection sequence on initial stresses in a curved girder bridge. Based on the studies, Linzell (1999) demonstrated that finite element modeling could accurately predict the behavior that occurred throughout the construction studies. Error was primarily due to heating and forcing one of the girders in the curved girder frame that was incorrectly cambered, resulting in stresses that were unaccounted for in the analytical models. In addition, insight was obtained regarding load redistribution that occurs during curved I-girder bridge construction and subsequent deformations. Further elastic analysis carried out as a part of the CSBRP indicated that, for the completed structure, the final deflected shape and load distribution depends on the erection sequence followed (Duwadi et al. 2000).

Studies were also conducted by the University of Minnesota with the Minnesota Department of Transportation (MNDOT) to understand the behavior of the steel superstructure of a curved I-girder bridge system during all phases of construction. As a part of the study, a field investigation was carried out on a two span continuous horizontally curved I-girder bridge as it was being erected near Minneapolis. The primary objective of the study was to compare the steel superstructure stresses obtained via strain measurements to the predictions of linear elastic software typically used (Galambos et al. 2000). Additionally, the field measurements were compared to a linear elastic analysis program developed specifically for the MNDOT research (Huang 1996).
In general, the construction of the steel superstructure proceeded smoothly. Steel erection was carried out using two 90.7 metric ton (100 ton) cranes, and one 45.4 metric ton (50 ton) crane. The girders were not assembled in pairs, but rather one at a time with one of the cranes utilized to stabilize one girder while cross frames or a second girder was placed. Analysis results correlated well with the field measurements, and showed better correlation as erection of the steel superstructure proceeded. Poor correlation occurred during initial erection stages, which could have been the result of two discrepancies. First, shoring towers modeled as rigid supports in the analytical study did not represent the actual elastic supports. Second, the connection bolts between the cross frames and the girders were not fully tightened and minor fit-up stresses affected the results (Huang 1996).

The research by MNDOT concluded that the performance of steel erection during temporary shoring phases was controlled by stiffness and not strength. The stresses were well below the yield stress throughout construction. Analysis results generally matched field measurements for both stress and deflections. Minor differences were attributed to the improper modeling of temporary shoring supports, erratic effects of warping restraint, minor axis bending, and the unpredictability associated with loose girder-to-cross-frame connections.

Chavel and Earls (2001) evaluated erection of a horizontally curved steel I-girder on the Ford City Bridge in Pennsylvania. ABAQUS was used to recreate the “as-built” erection sequence of the bridge. The analytical model utilizes cross frames detailed for the theoretical no-load case. Observations throughout the analysis of the “as-built” erection sequence indicate that out-of-plane (radial) and vertical displacements remain minimal, with displacements usually less than 25 mm (1 in). In addition, stresses were found to be largest in the top and bottom flanges for each erection stage. The reactions obtained throughout the analytical bridge erection sequence were consistent with engineering judgment.

Chavel and Earls (2001) emphasized the importance of detailing of load carrying members for smooth construction. Using the model developed for the Ford City Bridge, Chavel and Earls report that a difference of two inches in some of the cross frames would have occurred if inconsistent detailing had taken place. A difference in length of two inches can lead to extreme problems with girder and cross frame alignments, resulting in the need for significant forces to be applied to the superstructure during erection in order to bring the bridge components into alignment. This may require cranes with larger capacity, additional shoring, and additional jacking devices. The additional forces required to fit components may be acceptable in some cases; in other cases the erection of bridge might also become extremely complicated, or even impossible.

Two methods are presented by Chavel and Earls (2001) to prevent the problem of inconsistent detailing and to determine cross frame member lengths in curved I-girder bridges:

1. Both the girders and cross frames can be fabricated for the no-load condition and can be achieved by temporary supports in the field. The rotation and displacement that occurs after support removal should be considered a serviceability issue that must be addressed by the design engineer or bridge owner.
2. An innovative method was proposed for the girders and cross frames to be detailed such that the girder webs are out-of-plumb at the no-load condition. The required out-of-plumb (rotation and vertical displacement) are calculated so that the bridge girders will rotate as a rigid body to a vertically web-plumb position as soon as the temporary support is removed due to the application of the structure’s self-weight.

Chavel and Earls (2001) also report that the Ford City Bridge cross frames were detailed incorrectly. The cross frames were detailed so that the girder webs were plumb after the application of concrete deck load, neglecting the self-weight of the steel structure. A significant inconsistency was created when the cross frames were detailed for the web-plumb condition at application of the concrete deck weight only and the girders for the web-plumb condition at no-load. Utilizing a finite element model, it was shown that the cross frame misfits in the order of 32 mm (1.26 in) could be expected; the actual field records indicated 38 mm (1.5 in). In addition, the support reactions in some cases do not follow typical load distribution, and final steel elevations could deviate significantly from expected values, thus causing design changes related to the concrete deck and haunch thicknesses. In essence, the study showed that bridge engineers must pay close attention to the issue of consistent detailing when designing horizontally curved steel I-girder bridges and demonstrates the fact that inconsistent detailing can lead to extreme problems during construction.

Schelling et al. (1989) provided general guidelines for various erection schemes using top and bottom lateral bracing to prevent overstress during construction. The results were based on a parametric study conducted using a 3-dimensional frame that simulated the curved I-girders, cross frames, and lateral bracing system. The model permitted consideration of three moments and three normal forces at each end of each member. This allowed bending about two major axes, torsion and the influence of warping to be incorporated directly into the analysis. The study resulted in equations that define dead load distributions throughout the superstructure. The parameters include span length, radius of curvature, girder spacing, slab thickness, diaphragm spacing, and number of girders. In addition to examining the response of simple span structures, two and three equal span structures were also studied. The basic philosophy behind the top and bottom lateral bracing is to convert the open system into a pseudo-box closed system, thereby improving the torsional resistance of the system. The improved torsional resistance due to the pseudo-box configuration can overcome the rotation tendency of curved I-girders. The parametric study indicates that the dead load distribution factors for simple spans can be applied conservatively to continuous span bridges when the supports are radial and span length ratios are close to unity.
4.0 Synthesis of Box Girder Research and Current Practice

4.1. Background

A box girder is particularly well suited for use in curved bridge systems due to its high torsional rigidity. High torsional rigidity enables box girders to effectively resist the torsional deformations encountered in curved thin-walled beams. There are three box girder configurations commonly used in practice (Figure 4-1). Box girder webs can be vertical or inclined, which reduces the width of the bottom flange.

For I-girder systems, the superelevation can be accounted for by altering the elevation of each individual girder. However, the box girder system is one unit. To design for superelevation, (1) the box girder cross section can be rotated as a whole or (2) the two top flanges can be designed with different elevations while keeping the bottom flange horizontal (Figure 4-2). The second option is more complex to design and fabricate. However, architectural appeal could be the deciding factor.
Figure 4-2. Geometric configurations to account for superelevation in box girder bridges

Since the box girder behaves as a “closed” section with generally improved torsional resistance over the I-section, the distortion of the cross section becomes the primary concern. Internal and external cross frames and diaphragms are designed to reduce cross section distortion and therefore ensure that the section acts as one unit. The bracing connections to the girder can be considered as rigid boundary conditions for parts of the girder that are exposed to higher stresses, such as the outer web or the outer top flange.

4.1.1 Guide Specifications

The 2003 Guide Specifications were developed to incorporate recommendations resulting from the National Cooperative Highway Research Program (NCHRP) Project 12-38 and was intended to reflect the recent research work on curved steel girders and to be consistent with the current straight-girder specifications (AASHTO 2003). The FHWA “Curved Steel Bridge Research Project” was initiated to address the research needs identified by NCHRP Project 12-38. However, the CSBRP was later modified to be limited to curved I-girders. As a result of this change in the project scope, most provisions of the 2003 Guide Specifications that deal with curved box girder are the same as those of the 1993 Guide Specifications, which is essentially the same as the version first adopted in 1981 that were based on research work done in the 70s by the Consortium of Universities Research Team (CURT) project.

The 2003 Guide Specifications did not include major upgrades over the 1993 Guide Specifications except that certain limits previously established for curved I-girders were adopted for box girders. I-girder limits were implemented because applicable research on curved box girders is lacking. Most of the research work on curved box girders utilizes refined numerical methods initiated in the late 1990s. The 1993 Guide Specifications were based on research done decades ago. Recent research efforts have begun to focus on the behavior and design of curved box girders. However, more research is needed to achieve a comprehensive understanding of the behavior and the nominal strength of curved box girders.
The 1993 Guide Specifications included provisions for both the Allowable Stress Design method (ASD) and the Load Factor Design method (LFD). The limit state design method was adopted in the 2003 Guide Specifications. A construction section that includes provisions on fabrication, transportation, erection, and deck sequencing was introduced in the 2003 Guide Specifications. Two design examples were included; one for curved steel I-girder and the other for curved box girder.

The 2003 Guide Specifications reflects classical structure principles including:

1. Structures must satisfy static equilibrium;
2. General stability of the whole structure and the individual components must be ascertained under critical loads;
3. Analyses may be made using small deflection elastic behavior; and
4. Large deflection inelastic analysis behavior is not required but results in improved accuracy.

Analysis of the curved girder systems can be done by approximate or refined methods. Refined methods of analysis, such as finite strip and finite element methods, are recommended by the 2003 Guide Specifications. However, approximate analysis methods can be used when the results from these methods are consistent with the structural principles listed above. The 2003 Guide Specifications allow the designer to determine whether the application of this method is appropriate or not. The M/R approximate method that is described in the commentary is used in the 2003 Guide Specifications curved box girder design example.

The M/R method was introduced by Tung and Fountain (1971) to account for the effect of curvature in simple spans and continuous curved box girders. Tung and Fountain recommended limiting the applicability of this method to situations where the following conditions are met:

1. Girders are concentric;
2. Bearings are not skewed;
3. The span to radius ratio is less than 0.3;
4. Girder depth is less than the width at mid span.

The M/R method is based on the linear behavior of rigid girders while the specifications require checking the limit state for strength on the ultimate loading stage. Moreover, the study by Tung and Fountain states that the effect of curvature on strength and stability must be considered for all curved box girders. However, the absence of specified provisions or guidelines makes it difficult for designers to consider the effects of curvature on strength and stability.

Section 10 of the 2003 Guide Specifications addresses the design of closed box and tub girders. Specifically, this section contains provisions for internal bracing and diaphragms at supports and intermediate diaphragm or cross frames. Also, the strength of box flanges is considered for non-composite and composite sections under tension, compression, and shear.
The strength of the top flanges of box girders is estimated according to the non-compact compression flange provisions. In the curved box design example provided in the 2003 Guide Specifications, a top flange was designed according to the provisions of non-compact flange while the width-to-thickness ratio did not exceed the compactness limit. According to the commentary of the 2003 Guide Specifications, the width-to-thickness limit in this non-compact flange provision was taken as 10 percent less than the AISC Specifications limit to prevent elastic local buckling. This limit was based on experimental research done by Culver and Nasir (1970) and adopted because Davidson et al. (1996) indicated that local elastic buckling might be more critical for curved girder flanges than straight. However, Davidson’s research focused on I-girder geometries, and the flange load and support conditions of box girders are different from those of I-girders. For instance, the stress distribution for box girder flanges are different and inclined webs exerts different forces on the top flanges. Therefore, more curved box girder studies are needed to define the nominal strength of the top flange.

The 1998 AASHTO Standard Specifications do not include provisions for bottom flange plates under a combination of compression and shear due to torsion. The 2003 Guide Specification gives two slenderness ratios, $R_1$ and $R_2$, defined by Equations 4-1 and 4-2. For flanges with slenderness ratio less than $R_1$, the critical stress is estimated base on Huber-von Mises Henky yield criterion. For more slender flanges, the strength is calculated according to the elastic buckling strength given by Timoshenko and Gere (1961) for the long plate with clamped edges under uniform compression and shear.

$$R_1 = \sqrt{\frac{97}{\Delta + \sqrt{\Delta^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}}}$$

Eq. 10-5 of the 2003 Guide Specifications

$$R_2 = \sqrt{\frac{210}{\frac{1}{1.2}\left(\Delta - 0.4 + \sqrt{(\Delta = 0.4)^2 + 4\left(\frac{f_v}{F_y}\right)^2\left(\frac{k}{k_s}\right)^2}\right)}}$$

Eq. 10-7 of the 2003 Guide Specifications

Where

$$F_y = \text{specified minimum yield stress}$$

$$\Delta = \text{reduction factor for maximum stress}$$

$$R_1 = \text{reduction factor for stresses}$$
The 2003 Guide Specifications commentary suggest the use of the beam on elastic foundation analogy (BEF) presented by Wright and Abdel-Samad (1968) for calculating transverse bending stresses for the girders with transverse stiffeners. The limiting value of 20 ksi given by the 2003 Guide Specifications for transverse stresses that is used to design intermediate internal bracing is from the 1993 Guide Specifications. This limit was based on the work of Oleinik and Heins (1974, 1975). Given the power of recent computer modeling applications in design, there is a need to revisit these limits that were based on research work done in the 60s and 70s.

The LFD sections of the 1993 Guide Specifications contain provisions regarding box girder web design that include an equation for calculating inclined web shear stresses. These provisions also require that ultimate shear capacity be calculated according to provisions for I-girders web design. The 2003 Guide Specifications retained most of the 1993 provisions for web design.

### 4.1.2 Design and Analysis of Behavior of Curved Box Girders

Tung and Fountain (1970) developed an approximate method for torsional analysis of curved box girders. This method is similar to the conjugate beam method for calculating beam deflection. The values for moments and torsion from this method were compared to a closed form solution that was referred to as an exact solution. Also, curved girders under uniform vertical load and equilibrium equations that relate shear, moment and torsion were considered (Equations 4-3 through 4-5). The two equations for the moment and torsion (Equations 4-3 and 4-4) were incorporated into a second order differential equation. The solution for this differential equation was considered an exact solution. The method first estimates the moment by neglecting the T/R term, then uses this value for the third equation. In other words, the method solves the problem of coupling Equations 4-4 and 4-5 by neglecting the effect of T when estimating torsion and considers the curved girder as straight under a torsional moment equal to \((M/R - t)\).

\[
\frac{dV}{R d\alpha} = \frac{dV}{dx} = -p 
\tag{4-3}
\]

\[
\frac{dM}{R d\alpha} = \frac{dM}{dx} = \frac{T}{R} + V + \frac{dT}{dx} 
\tag{4-4}
\]

\[
\frac{dT}{R d\alpha} = \frac{dT}{dx} = \pm \frac{M}{R} - t 
\tag{4-5}
\]

where,
$M = \text{bending moment}$

$R = \text{radius of curvature}$

$T = \text{torsional moment}$

$V = \text{shear moment}$

$P = \text{distributed vertical load}$

$T = \text{applied torque}$

$X = \text{independent variable along the longitudinal axis}$

$\alpha = \text{independent angular variable}$

The accuracy of the approximate torsional analysis depends on the accuracy of the bending analysis. Both analyses are affected by the central angle and the bending-torsional stiffness ratio $EI/GJ$. The calculation of cross section properties such as “$I$” and “$J$” for box sections is approximate, especially during the construction phase where the section is not a closed section. This approximate calculation usually assumes a closed section and involves estimating the effects of internal diaphragms and stiffeners.

Heins (1972) tested a series of box girder models made of plexiglass to measure the angular, radial, and torsional stiffness. The angular stiffness was evaluated using a model of three box girders, the radial stiffness was evaluated using a model of 10 box girders, and the torsional stiffness was evaluated using a model of a single box girder. The models were instrumented extensively to measure deflections. The measured values were compared to analytical values obtained by the slope deflection method. It was assumed that enough bracing was provided to prevent cross section distortion, and hence warping was neglected. It was found that the values for stiffness assumed by analytical methods were highly correlated with the measured values from the test.

Oleinik (1974) developed equations to predict the distortional stresses due to dead and live load. Limiting these distortion stresses to 10 percent of the normal stresses due to bending, new design equations were deduced for the spacing of intermediate diaphragms and implemented in the 1993 Guide Specifications. The primary assumption for this research was that the total normal stresses could be evaluated as the sum of two parts. The first part is the calculation of normal stresses assuming that the cross section does not change and can be evaluated according to Vlasov (1961). The second part is the calculation of normal stress due to the distortion of the cross section and is solved according to Dabrowski (1968). In both parts, the normal stresses are calculated after estimating the deformation of the cross section by solving a differential equation. Oleinik developed a computer program to solve these differential equations based upon the finite difference method.

Yoo et al. (1976) tested a twin curved box girder during construction and in its traffic load bearing configuration. The results of the tests were compared to results from two analytical software programs. The program Curved Single Girder (CURSGL) was used for dead load analysis and the program Curved Bridge System (CURSYS) was used for live load analysis. For the dead load case, the section was analyzed twice; once as an open section and the other as a closed section. It was observed that the actual behavior of the box girder is somewhere in between the two assumptions. Experimental results were within 10 percent of the values.
predicted by analytical methods. While the software gives the average normal stresses at the bottom flanges, the values of the measured normal stresses at the middle of the bottom flanges were smaller than those near the web due to shear lag.

Heins and Oleinik (1976) presented additional details about the computer program used to determine the response curved single box beams with number of interior diaphragms as a parameter. This computer program used the finite difference method to solve the differential equation that describes the behavior of curved box girders (Oleinik and Heins 1974). The program included cross section distortion and the effect of intermediate cross frames. A sample application of the computer program to a single span curved box girder was presented in this publication.

Oleinik and Heins (1975) evaluated the distortional response of curved box girders subjected to dead and live loads and presented the development of the equations for intermediate cross frame spacing design. The computer program presented by Oleinie and Heins (1974) was used to perform a parametric study using several curved box girders. The analyses assumed that cross frames totally restrain the angular distortion at their locations. Members of the cross frames and diaphragms should be rigid enough to validate this assumption. The parametric study concluded that the curved girder would distort significantly under load, and that the stresses due to distortion can be as high as the normal stresses if enough intermediate cross frames or diaphragms were not provided.

Heins (1978) organized and compiled information about box girder bridges and presented correlations for preliminary proportioning of curved and straight box girders. The bridges were divided into three categories. These categories are single span 27.4 to 61 m (90 to 200 ft), two spans 30.5 to 67 m (100 to 220 ft) and three spans 30.5 to 88.4 m (100 to 290 ft). For all the bridges, data regarding the size of the box section were sorted to obtain useful relations for preliminary designs. The data included web thickness, ratio of road width-to-number of boxes, width of the box, bottom flange thickness, and top flange thickness. In this publication, Heins summarized basic formulas for estimating the forces and stresses in straight and curved box girders based on the work of Oleinik and Heins (1974 and 1975) and Heins and Oleinik (1976). Furthermore, Heins synthesized the specifications proposed by the CURT project at that time, which later became the Guide Specifications.

Heins and Lee (1981) presented the results of a two-span curved composite box girder bridge field test in Korea. This bridge was found to be over designed when compared to the AASHTO specifications. The results from the test were compared to results from analytical models that used software programs such as CURSGL, STRUDL and SAP. Approximations were made to account for the effects of the ribs (longitudinal stiffeners) on the top and bottom flanges. The results indicated that warping stresses contributed less than 0.5 percent of the normal stresses. Hence, warping stresses can be neglected due to the high torsional stiffness of the bridge.

Arizumi et al. (1988) tested three models of curved composite box girders. The measurements obtained from the test were compared to the finite strip method and Dabrowski’s distortion theory. The study also compared the values of distortional stresses obtained by Oleinik and
Heins (1975) to values obtained by other simplified methods from the Japanese literature, including Nakai and Murayama (1981) and Sakai and Nagai (1980). Arizumi et al. observed that curved composite box girders without any intermediate stiffeners deform significantly. This deformation causes significant additional stresses and complicates the longitudinal stress distribution in the web, especially for sharp curvature. It was also observed that the stresses in shear studs are affected by torsion as well as shear.

Branco and Green (1982, 1984, 1985) investigated how the bracing systems of an open straight box girder would affect the response of the girder to bending and torsion loading. The response of the open box section was assumed to be of different components and the effects of various parts of the bracing system on these components were examined through experimental tests and analytical finite difference analysis. The response of the box section was assumed to be comprised of two parts. The first part was when the section acts as a rigid section and hence deflects rigidly (longitudinal bending) due to flexure and rotates rigidly (mixed torsion) due to torsion. The second part was due to flexural deformation (bending distortion) and torsional distortion. Bending loading was assumed to cause three different forms of stresses. The first was the regular normal stress that varies linearly along the height of the cross section. The second was the accompanied shear stress distribution. The third was described as spreading stresses (bending distortion), which was analyzed into outward bending of the web, upward bending of the bottom flange, and in-plane bending of the top flanges. Torsion was assumed to be comprised of uniform torsion (St. Venant torsion) and non-uniform (distortion) torsion. The major findings of these studies can be summarized as follows:

1. The ties in the top bracing can reduce bending distortion;
2. Web stiffeners are effective in resisting torsion distortion;
3. Distortional bracing (vertical intermediate cross frames) can be added to resist high concentrated torsional moments;
4. Horizontal bracing of the top flanges reduce warping stresses.

A comprehensive research program sponsored by the Texas Department of Transportation was undertaken at the University of Houston in 1995 and was published in Fan (1999), and Fan and Helwig (1999 and 2002). The research studied the behavior of curved steel trapezoidal box girders through field measurements and analytical studies.

Fan (1999) measured the stresses and studied brace forces during construction of a three-span continuous curved steel box girder. The stresses were recorded at two locations that represent positive and negative moment regions. Measurements were taken at different stages of construction. The experiment allowed Fan to observe the changes in stresses due to erection of different parts of girders and after making splices throughout the stages of slab construction, which includes deck pouring and hardening.

Fan and Helwig (1999) presented the results of a bending behavior study on a trapezoidal box girder system during construction. The finite element results revealed that large forces can develop in the horizontal truss system due to vertical bending of the box girder. Fan and Helwig also investigated the assumption that the top and bottom flanges equally resist the horizontal load
component and found it to be incorrect. Instead, they proposed a method to estimate the forces induced in the top flange horizontal truss members (diagonals and struts) due to two loading cases, vertical bending of box girder and lateral component of applied loadings due to sloping webs, and illustrated the method through a numerical example. The study concluded that for trusses with a single diagonal, the forces induced by vertical bending produce large lateral bending stresses in the top flange.

In a later publication, Fan and Helwig (2002) presented a study of the distortional behavior of box girders with a trapezoidal cross sectional shape through analytical models. In this study, equations for estimating the brace forces in the quasi-closed box girders based on the different distortional components of the torsional loads were used. Finite element analyses were used to verify the brace force values developed in these equations.

Fan (1999) also performed a study on composite box girder bridges under live loading. In this study, truck loads were placed at different locations and the resulting stresses were captured. The stress readings were compared to results from finite elements analysis. Fan concluded that composite action appears to properly adhere to the 1993 Guide Specifications. The study also concluded that curvature did not significantly affect lateral distribution of live loads. Fan recommended performing further studies of the effect of other parameters such as the girder spacing, length, and radius of curvature on the lateral distribution factor.

4.1.3. Ultimate Strength

Yonezawa et al (1977) presented the results of an experimental study of a curved steel box girder under static load. The model was 4.2 m long (13.8 ft) with a radius of curvature of 4 m (13.1 ft) and 0.3 m depth (1 ft). The model had enough lateral bracing to neglect warping and was subjected to two point concentrated loading and deflection. Stresses were measured for the top flange and two webs. It was observed that the web region under compression deflected outward while the region under tension deflected inward. The deflection of the web caused the inside edge of the top to deflect upward in contrast to the deflection of the rest of the top flange, which was downward. The model was tested until collapse to estimate the ultimate strength of the top flanges and the ultimate load was compared to four theoretical methods presented in Japanese literature. It was noted that the ultimate load value from the test was close to the value estimated by only one of these methods. The critical load measured was 27.6 metric tons (30.4 tons). Only one method (25.3 metric tons (27.9 tons)) was close to this value while the other three methods estimated below 10 metric tons (11 tons).

Manko (1984) tested 12 models of straight box girders subjected to 4 types of loading to determine the ultimate load capacity and to investigate the stability of these models in the final stages of loading. Manko chose a closed box model with varying number of stiffeners for the deck plate. The ultimate load was marked by a plastic deformation in the deck or web. The girders lost their load carrying capacity due to local buckling of deck plates, ribs, or web, depending on the type of loading. It was found that both stiffness and load type have great effect on the ultimate load.
According to Salahulddin (1994) a realistic assessment of the strength of box girder bridges can only be made if the design is based on the nonlinear theory of buckling and considers factors such as plastic flow, shear lag, initial geometric imperfection, and residual welding stresses. Salahulddin suggested limits on the initial distortion of box girders and recommended keeping the fabrication tolerance (initial imperfection) 20 percent less than acceptable values by any standard design.

Yabuki et al. (1995) investigated the effect of plate local buckling and distortion on the ultimate strength of curved box girders. A theoretical method for nonlinear analysis of curved box girders was proposed. To account for local buckling, Yabuki et al. adopted a modified stress-strain curve defined by a set of four equations, which were proposed through a regression analysis of the stress-strain data. To account for distortion, a numerical procedure was used to incorporate additional strain at the initial strain stage of the incremental nonlinear analysis process. The deformations due to distortion were evaluated using the beam on elastic foundation (BEF) analogy along with the $M/R$ method. Moreover, two experimental tests on curved box girders were performed. In these experimental tests, a central angle of 30 degrees and web depth-to-width ratio of 4 were used. The width/thickness ratio for web plates was 133 while the width/thickness ratio for compression flanges was 29. Real weld sizes were used to measure the residual stresses. Finally, results from the proposed methods were compared to results from the tests. The comparison showed that the model behaved nonlinearly at a lower loading stage than the proposed method. The test results showed discrepancy of 11 percent and 17 percent while the proposed analysis method showed 1 percent and 2 percent. Although Yabuki et al. considered the agreement between measured and calculated values to be good, they suggested conducting more tests for verification.

Yoo et al. (2001) evaluated the design method of longitudinal stiffeners on box girders. The equations provided by the AASHTO specifications are base on the work of Timoshenko and Gere (1961), and assume that (1) the plate is infinitely long, and (2) the plate buckling coefficient can be less than 4. A regression equation was presented and compared to the AASHTO equations and another equation proposed by Clinton et al. (1986). The regression equation was developed based on three dimensional finite element analyses for several models with various parameters. Yoo suggested using the regression equation presented in the paper to update the conservative AASHTO equation. A numerical example was provided to illustrate the difference.

4.1.4 Web Design

Benussi and Mele (1994) introduced a method for evaluating the shear capacity of box girder webs that have an aspect ratio less than or equal to one. This method estimates the ultimate shear resistance as the sum of reduced critical shear stress of the weakest panel and the tension field stress. The value of the tension field stress is calculated based on the “anchorage length” of diagonal tension bands within the flanges. The method was based on numerical and experimental analyses of 12 laboratory tests. The analyses showed that the stiffness of the flanges has significant effect on the diagonal tension.
Bradford (1990) studied the local buckling of open box sections using the finite strip method. The finite strip models were verified by comparing the results to the values that were obtained through experimental work performed by Rhodes and Harvey (1975) on eccentrically loaded channels. The nonlinear finite strip method was used to study the post-local buckling behavior and to drive moment-curvature relationships for box sections under uniform bending. The results were compared to the results of analyses done by Grave Smith (1972). Based on the results from this study, Bradford introduced an expression for evaluating the local buckling coefficient. It was found that the web stiffener at 15 percent of the web depth below the top flange would increase the buckling coefficient significantly. It was also observed that flexural rigidity decreased by only 10 percent after local buckling. However, deflections of the web were substantial in the post local buckling range.

Bradford and Wong (1992) used the finite strip method to evaluate the local buckling of composite box sections in negative bending. The values obtained from this method were used to develop design charts to determine the buckling coefficient of the web. The design charts illustrate that increasing the flange width would decrease the restraining effect of the flange, and hence lower the buckling coefficient. The charts also showed that, as the neutral axis moves higher, the buckling coefficient decreases.

4.1.5 Other Recent Research

Recent research efforts discuss design and analysis methods of curved box girders, placement of access hatches in steel box girders, distribution factors for live loads, and types of bearings. A brief overview of recent research efforts related to curved box girders is given below.

Sennah et al. (2001) provided a survey of research work on box girder bridge design topics. These topics include box girder bridge configurations, construction issues, load distribution, dynamic response, and ultimate load carrying capacity. The survey reported difficulties encountered in box girder construction such as changes in geometry and excessive rotation of girders before and during the placement of the concrete deck. The primary conclusion regarding the construction and design of curved box girders is that the current North American codes, including AASHTO publications and Canadian codes, as well as published literature, do not provide the design engineer with adequate information on the behavior of unshored straight and curved box girder bridges during construction. Additional research work using 3-D finite element analysis is needed to avoid potential catastrophic failures.

In another review by Sennah et al. (2002), analysis methods used to analyze box girder bridge systems found in the literature were presented. These methods included the grillage-analogy method, folded plate method, finite strip method, and finite element method. The survey summarized research efforts on thin-walled curved beam theory and experimental studies on the elastic response of box girder bridges. Sennah et al. concluded that, although the finite element method seems to be the most costly and time consuming, it is the most detailed and comprehensive method that can overcome the limitations of other simplified methods. Moreover,
issues such as thermal effects are difficult to model accurately, except through a detailed finite element analysis. Both surveys provide a comprehensive list of more than 150 references.

Okiel and El-Tawil (2000 and 2002) carried out a research program to study the behavior, analysis, and design of curved box girders. The research program investigated three issues. A brief description of these issues along with the major conclusions of each is given below.

1. Effect of non-uniform torsion on the behavior and design of existing curved box girders

Okiel and El-Tawil addressed the effect of non-uniform torsion on the behavior and design of existing curved box girders through studying eighteen existing box girder bridges selected from the Florida DOT inventory using software that was developed for the purpose of this research. The program uses two beam elements; one is the general six degrees of freedom beam element, and the other is a seven degree of freedom beam element that can account for warping. The stiffness matrix derived for both elements is limited to small deformations. Neither large deformations nor distortion effects were considered. The results of this program were compared to the results of three analyses. These three analyses include closed form solution for warping, beam element models that accounted for warping using the ABAQUS (1997) finite element software, and full shell element models using SAP2000 (1999). Okiel and El-Tawil concluded that warping has little effect on shear and normal stresses in the structures they studied.

2. Existing distribution factors for curved box girder bridges

Okiel and El-Tawil studied the effect of parameters such as number of girders, number of lanes, girder spacing, span length, and radius of curvature on the distribution factors. Grillage models were used to analyze several bridges. The study concluded that using current distribution factors can lead to errors of up to 25 percent.

3. Placement of access hatches in steel boxes

Access hatches are usually placed in bottom flanges of box girders. Due to the long spans of continuous box girders, the distance between the access hatches sometimes exceeds the safety limits required by rescue workers. Okiel et al. (2000) looked at other locations for placement of the access hatches from a practical point of view, such as at the web and concrete deck. The factors that govern the placement of access hatches and how these factors affect the locations of the hatches were summarized. These factors include strength, feasibility, accessibility, water leakage, impact on traffic, and unauthorized access. They concluded that the bottom flange is the best option for placement of access hatches. They introduced a method to find low stress regions where access hatches can be provided. This method used the program they developed in their research to identify the locations where the maximum stress was less than one-third of the threshold of the element to account for the stress concentration factors for circular holes. The method was verified using detailed finite element models of existing bridges. They studied seven bridges to identify the low stress regions. For the range of structures studied, they found that access hatches can be placed at a distance of 20 percent to 42 percent of the span from the ends that are continuous from both sides and 20 percent to 54 percent of the span for the edge ends.
Bradberry et al. (2002) pointed out that steel trapezoidal box girders were typically supported by pot or disk bearings in Texas, which are typically expensive to fabricate and place. The TDOT sponsored research to study the behavior and design of elastomeric bearings. Elastomeric bearings are preferred by the TDOT because it is more forgiving of placement errors than more sophisticated bearings. Moreover, elastomeric bearings are easy to install, inspect, and replace. TDOT developed design recommendations and procedures for elastomeric bearings to support trapezoidal box girders. These recommendations were implemented on the US 290, IH 35 Interchange in North Austin (Figures 4-3 through 4-6). The study concluded that elastomeric bearings can be used effectively with steel trapezoidal box girders.

Figure 4-3. Direct connector "Y" south abutment end bearing showing longitudinal shear deformation (Bradberry et al. 2002)

Figure 4-4. Direct connector "Y" south abutment end bearing showing transverse shear deformation (Bradberry et al. 2002)
Lauzon and Dewolf (2003) measured the benefits of a monitoring system in terms of financial savings for different types of bridges. They showed that such systems can measure the stresses in steel members or bridge components under service loads and hence can identify causes of cracking and fatigue categories applicable to the structures, which is essential for determining repair alternatives.

Lauzon and Dewolf studied curved, three-span, continuous steel dual box girders. They were contacted for more information about the bridge configuration, the service stresses, and the availability of any further publication. The monitoring program started during the summer of 2003. The curve bridge was a fairly new (10 to 15 years old) one-lane wide twin steel box girder. It was selected for the monitoring program as a representative for that type of bridge, which had been used frequently for interstate interchanges, and also because it was highly visible. The vibration, temperatures, and strains are being monitored. This information will be valuable for verification of finite element models.
4.2. State-of-the-art in Construction of Box Girders

The 2003 Guide Specifications include a section that deals with construction. This section requires that contractors provide a detailed construction plan that meets the following minimum requirements:

1. Fabrication procedures, including the method of curving the girders;
2. Shipping weights, lengths, widths, heights, and shipping methods;
3. Erection plan that includes erection sequence, crane capacities and locations, and temporary supports;
4. Deck placement sequence including time between casts and positions.

Each of these requirements will be discussed in the following paragraphs.

Few publications regarding issues related to construction of curved box girders are available. Some of these publications are summarized in this section while others about specific processes of construction are synthesized in subsequent sections.

Topkaya and Williamson (2003) emphasized the importance of the design for construction loads. Although in most cases these construction calculations are well documented, it is difficult to collect information about the construction stages. In the Topkaya and Williamson study, it was discussed that stresses coming from construction loading can reach up to 60 – 70 percent of the total stresses in the cross section.

Hirasawa et al. (1999), studied curved box girder stability during construction by launching. The launching method is used to erect girders where the girders are required to cross a road or a river and the crane trucks cannot be placed on the erection site. In this method, the girder is pushed from one end to the other while jacked up and down to advance over temporary beams or moving supports. The balance of the girder during this process is very important to get the appropriate reactions at the support points. The balance of the girder is ensured by having a positive vertical reaction at all temporary supports and using supports that can withstand the horizontal reactions. An element stiffness matrix was used to calculate the reactions at the supports of an existing bridge with varying support heights. The girder had a span of 111.7 m (366 ft) and an average radius of curvature of 350 m (1150 ft). A straight beam of 36.1 m (118 ft) was used to launch the girder. Reactions were calculated for two stages of construction due to differences in the elevation of jacking points for different radii of curvature.

Grubb et al. (1996) discussed issues related to construction of curved box girders. These issues were categorized into fabrication, erection, site assembly, and deck placement. Grubb et al. described cut curving and heat curving as the two methods for fabricating curved steel girders. Concerns regarding camber loss due to heat curving were discussed. These concerns were addressed by New York State by providing additional camber in both straight and curved girders to make up for the losses due to weld shrinkage. Another concern was that shipping of curved girders might require additional splices to reduce the size of the shipped pieces.
Issues regarding erecting of box sections were described. The weight of a single piece is usually greater than a comparable I-girder. The additional stiffness that box girders possess makes it more difficult to adjust the girder into position. Grubb et al. stressed the need to prevent distortion by providing adequate bracing and intermediate diaphragms or cross frames. Finally, an equation was given to calculate the center of gravity of curved box girders. Eccentric construction loads can cause the box girder to twist and thus the study recommended adding external diaphragms or cross frames between adjacent girders. On the other hand, eliminating external bracing would reduce maintenance efforts and improve fatigue resistance of the bridge. The use of false work and temporary shoring can help improve fit-up, especially with projects that involve box girders with sharp curvature.

For continuous girders, Grubb et al. indicated that casting first in the positive moment areas of the end spans might be preferred to avoid uplift, especially if the end spans are lightly loaded. Grubb et al. also emphasized caution for the differential deflection that might result from the weight of the finishing equipment. This can lead to pouring extra concrete at certain elevations, and hence, additional loading.

**4.2.1. Fabricating Box Girders**

Box girder fabrication is similar to that of I-girders in many ways; however, special considerations and arrangements apply to box girders to ensure careful layout, fit up, and welding. The 2003 Guide Specifications requires that box flanges be cut-curved. The top flanges of the tub girders can be heat-curved after they are welded to the web. Work sequence may differ from one fabricator to another. This section summarizes fabrication steps.

The research team visited the Carolina Steel Plant in Montgomery, Alabama during December 2003, July 2004, August 2004, and September 2004. Numerous interviews with the engineers and technicians were carried out during these visits. This plant has been fabricating curved steel girders for many years.

In the first visit, a curved I-girder bridge was being fabricated. The engineers mentioned that the number of curved box girders projects that come to the plant is generally much less than I-girder projects. Only one project has been completed for Florida DOT in the last three years. However, there was another curved steel box girder scheduled for the Florida DOT. The team scheduled additional visits to study the fabrication process.

Fabrication begins with detailing and preparing shop drawings. Curved box girders are typically used for sharp curvature alignments. Combining the horizontal curvature with differences in elevation due to superelevation and camber complicates the detailing process. Inclining the web, which is a common practice with box girders, complicates the detailing even further. The detailer, having the three conditions of horizontal curvature, vertical curvature, and inclination, uses software to calculate the sizes and the coordinates of each piece of steel.
The next step is preparing the steel plates. First, the plates are ordered according to the detailer’s requirements. Steel plates are obtained from large steel producers such as US Steel or International Steel Group (ISG). These plates are shipped to the fabrication plants either by trucks or rail. The plates are usually obtained in widths up to 2.54 m (100 in) and lengths up to 30.5 m (100 ft). After that, the plates are stored at the plant where they can be moved to the shop at the appropriate time. Figure 4-7 shows the arrival of the purchased plates at the plant by rail. Figure 4-8 shows the plates being moved from the storage area to the shop. Figure 4-9 shows plates being unloaded in the working area. These plates are flimsy and require special care during moving. Finally the plates are welded together using full penetration welds to form the big plates that will be cut into girder components. Figure 4-10 shows the full penetration splice between plates used in forming the top flanges.

![Figure 4-7. Purchased plates arrive at a fabrication plant by rail](image-url)
Figure 4-8. Plates moved from the storage area to the shop

Figure 4-9. Moving the plates into the work area
The plates are then laid horizontally on tables. All of the main components (bottom flange, top flanges, and webs) are straightened using a computer-controlled machine. The geometry of these plates can be fed to the machine by computer. The engineers stated that there was a need to modify parts of the software because of the reverse camber in the web due to superelevation. The top flanges are tack welded to the web. The same machine that is used for welding I-girders is used for tack welding the top flange to the web. This machine has cylinders that can align the top flange to the required slope in relation to the web. Figure 4-11 shows the machine that tack welds the top flange and the web; Figure 4-12 illustrates the same machine tack welding the longitudinal stiffener to the bottom flange.

Then, for each side, the top flange and web are continuously welded together. Due to the inclination of the web, there is sometimes a need for specially fabricated inclined jigs and frames to handle the top flange-web piece for welding. Figure 4-13 shows these frames and Figure 4-14 shows the smoothing of the continuous weld of the top flange to the web.

Cross stiffeners and joint plates are prepared separately. The plates are cut and holes are made according to the shop drawing. Figure 4-15 shows the cross frames prepared for welding to the web. These stiffeners are tack welded to the web as shown in Figure 4-16.
Figure 4-11. Machine used to align and tack weld the top flange to the web

Figure 4-12. Tack welding the longitudinal stiffener to the bottom flange
Figure 4-13. Special features and tools used for sloping the web

Figure 4-14. Smoothing the weld of web to flange
Figure 4-15. Cross stiffeners and plates before welding to the web

Figure 4-16. Cross stiffeners are tack welded to the web and the plates are bolted to the flange
Each side, consisting of a top flange, web, and cross stiffeners, can be heat curved to the geometric configuration required by the shop drawing. Heat curving is applied in multiple stages until the desired curvature is achieved. Each side is positioned so that the centerline of the flange plate is horizontal. The flange plates are marked as shown in the Figure 4-17. These parts are deformed by their own weight or by applying pressure in the desired direction. The heat should not be more than 593 °C (1100 °F) and is checked using a special tool and material that dissolves at a prescribed temperature. The heat process is repeated and checked at different locations along the centerline of the top flange until the specified curvature is achieved. The checks can be as simple as measuring the horizontal and vertical displacement from the centerline at locations identified in the shop drawings. Figure 4-18 shows the curved side (web and top flange) after applying the sweep while Figure 4-19 shows the setting of that part of the box to check the sweep and camber. During this process, handling and moving the web and top flange of the box requires special care to avoid excessive deformation. Figure 4-20 illustrates the lifting and moving of the girder webs in the fabrication shop.

All parts were painted before assembly. Only a few touch ups were required in the final stage. Figure 4-21 shows one side of the box girder (stiffened web welded to the top flange) finished and painted before assembly. Cross frames were prepared separately. The plates were cut and welded according to the shop drawing and then stored until they are assembled with the rest of the girder.

The next step is to assemble the girder. This depends on the work sequence and instructions of the shop drawings. Some fabricators prefer welding the two sides (top flange - web piece) to the bottom flange and then attach the cross frames and the top bracings. Other fabricators prefer welding one side to the bottom flange and attaching the cross frames and bracing, then welding the other side. In this “Florida Project,” the two sides of the box girders were tack welded to the bottom flanges first. The cross frames were then bolted to the box and the diaphragms were welded. Figure 4-22 shows tack welding the bottom flange to the stiffened web. Figure 4-23 shows the supports required to keep the web in place before the addition of the cross frames. Figures 4-24 and 4-25 show the bolted connection of the web at the top and bottom chords of the cross frame and Figure 4-26 shows welding the diaphragm to the box girder.
Figure 4-17. Marks for V-shapes where heat will be applied to gain sweep

Figure 4-18. Sweep applied to the box side
Figure 4-19. Setting the box side to check sweep and camber

Figure 4-20. Erecting a side of the curved box
Figure 4-21. Finished and painted side of the girder

Figure 4-22. Tack weld of the web to the bottom flange
Figure 4-23. Vertical and inclined stands supporting the web before adding the cross frames

Figure 4-24. Bolted top chord of the cross frames to the web stiffeners
Figure 4-25. Bolted bottom chord of the cross frames to the web stiffeners

Figure 4-26. Welding the diaphragm to the bottom flange
The frames and jigs used to fabricate curved box girders are moved horizontally and vertically using hydraulic pistons to ensure proper positioning. The arrangement of these tools should accommodate horizontal alignment, slopes, and changes in width. Differences in elevation are achieved using blocking. Moreover, some of these jigs can rotate the girder as shown in Figures 4-27 and 4-28. The collar surrounding the girder rotates around a pin located between the vertical posts. The collar has holes to lift the girder as shown in Figure 4-30.

Figure 4-27. Using jigs and collar to rotate curved box girder to particular a position for shop operation (USS 1978)

Figure 4-28. Using jigs and collar to rotate curved box girder to particular a position for shop operation (USS 1978)
Box girders are quasi-closed sections; hence, these girders might deform excessively under self-weight at the construction site. Many DOTs require shop assembly of a box girder structure. Long continuous girders are often assembled in two or three contiguous shipping pieces. In shop assembly, the bridge system is assembled in a way similar to the projected placement in the field. This process enables checking the splice plates and reaming the connection holes. Figure 4-30 shows the assembly of a curved box girder in the shop; Figure 31 shows the assembly of the whole box girder system in the fabrication yard.

Assembly takes much time and requires a large shop area. Moreover, the process requires checking and accurate measurements of the assembled configuration. Figure 4-32 shows the first stage of building the whole bridge. Figures 4-33 and 4-34 shows the field splice before connection. Figure 4-35 shows how the superelevation is assured during the assembly process and Figure 4-36 shows the preparation for placing the second girder.
Figure 4-31. Preassembly of the girders at the fabrication yard (Chang and James 2002)

Figure 4-32. Starting to build up the assembly
Figure 4-33. Shop assembly of box girders

Figure 4-34. Close up view of the splice location
Figure 4-35. Simulating the superelevation in the shop assembly

Figure 4-36. Set-up for the second set of girders
Finally, the box girder is disassembled into shipping pieces. The pieces go to the cleaning and painting area where they are run through a blast cleaning machine or cleaned by a worker with a blasting hose. Finally the pieces are painted. One primer coat is usually applied; two coats are rarely required.

Erectors sometimes suggest modifying the joints or splice fabrication method to facilitate and speed up the erection process. For example, the original specifications of a curved steel box girder bridge in San Francisco called for an all-welded structure. However, the steel contractor suggested bolted connections to simplify erection (Engineering News Record 1984).

In conclusion, the special geometric configuration associated with curved box girder represents a challenge for the fabricator during the various stages of fabrication. Careful layout, fit-up, welding, handling, and checking are required throughout the process. To minimize welding distortion, a good welding sequence is essential. However, lack of symmetry or geometric inconsistency can lead to significant residual stresses. Also, handling and lifting of the girder components can produce significant distortion. A study of the effects of residual stresses and distortion is needed for curved box girders.

**4.2.2. Transporting Box Girders**

The 2003 Guide Specifications require a detailed plan for transporting heavier, wider, deeper, or longer complex structures. They also require that stresses induced from transporting be less than the critical stresses for non-compact flanges, single web, or box flanges. These stresses are calculated due to self-weight with an allowance for impact of 100 percent.

Barge, rail, or truck, can be used to transport curved box girders from the shop to the site. The location of the bridge or the location of the manufacturing plant can dictate or facilitate the transporting process. If both sites are located on a navigable waterway, barges can be used. Figure 4-37 shows moving a curved box girder from the shop to the waterway; Figure 4-38 shows transporting curved box girders using a barge, which is the most convenient way of shipping as it allows extending the size limits of girders.

Shipping by rail can only be used with straight segments of the girder or with very slight curvature. Figure 4-39 shows a girder being shipped by the rail in the upside down position.

Box girders are typically used for sharp curvature alignments, which can limit the sizes of parts being shipped on trucks and result in increasing the number of field splices. An example of this situation occurred at the Mississippi SPUI project that is described in more detail in Section 4.6. According to Chang and James (2002), five inside pairs of girders were shipped in three segments per span, and were field spliced. However, due to sharp curvature, the pair of outside girders was shipped in five pieces per span.
Figure 4-37. Preparing curved box girder for shipping by barge (Universal Structural Inc. 2003)

Figure 4-38. Using barge to transport curved box girder (Universal Structural Inc. 2003)
Information about the maximum sizes for shipping varies from one erector to another. One erector suggested that parts be shipped by road if the maximum length is within 53.3 m (175 ft), the weight less than 73 metric tons (80 tons), and the height 4.1 m (13.5 ft) on side and 2.9 m (9.5 ft) upright. These requirements vary between states. Figure 4-40 shows a box girder loaded on a truck. However, to be competitive, another fabricator suggested that the length be less than 38.1 m (125 ft), the weight less than 36 metric tons (40 tons), the height less than 2.4 to 2.8 m (8 to 9 ft), and the width less than 3.6 m (12 ft). Chains are usually used to restrain the movement of the box girder on the road.

The transporting process can result in different types of loadings. These loads should be checked to avoid excess stresses and deformations that could result in fit up problems. Using trucks for transporting, the girders usually rest on two points with restraining cables and chains to keep them in position. Figure 4-41 demonstrates the use of trucks. In this situation, the girder is exposed to truck gust loading from other trucks. On the other hand, transporting by barge reduces exposure to vibration and wind loads.

Careful transporting planning saves time and money. As shown in Figure 4-42, many girders can be ready for shipping at one time to minimize the time needed to obtain permits. The advantage of using trucks depends on topography, distance, and route.
Figure 4-40. Box girder being transported by truck (High Steel Structure Inc. 2002)

Figure 4-41. Tub girder on a truck during transporting (High Steel Structure Inc. 2002)
4.2.3. Erecting Box Girders

Curved box girders can be exposed to several lifting mechanisms throughout the course of a project. For example, lifting for shipping in the shop or storing at the site would differ from lifting during bridge assembly. In each case, stresses and deformation due to lifting can be significant and may result in fit-up or alignment problems. Unfortunately, documented information is practically nonexistent about fit-up or alignment problems associated with lifting.

Erection is a very complex process that requires special engineering skills including:

1. An in-depth understanding of the plans and shop drawings;
2. Good knowledge of the tools available on site, the maximum lifting capacity of each crane, and the reach of each crane;
3. Comprehension and appreciation of the site constraints; crane locations and movements must be selected carefully to avoid interference with falsework as well as any other existing structures or traffic;
4. The ability to visualize the whole process;
5. Good analytical abilities to determine the stresses due to each step of the erection process.

Curved box girders are lifted in the fabrication shop from different pick up points. However, lifting for smaller distances in the fabrication shop by overhead cranes has less effect on the curved box girder than lifting on site. Moreover, in the shop there is an opportunity to verify that all dimensions during the assembly of the system before shipping adhere to the drawings. This verification process is difficult to conduct at the construction site. Figure 4-43 and Figure 4-44 show girders lifted in the shop using overhead cranes and different pick-up configurations.
In a paper titled “Erection Engineering for Steel Bridge Superstructures,” Weinhold (1997) listed three types of cranes used for steel erection depending on the size and weight of the girder and site conditions: conventional truck crane with lattice boom, hydraulic crane with telescopic boom, and crawler crane with lattice boom. The first type is the most commonly used crane for construction sites, but it requires assembly time using another crane. The second type is helpful for lifting smaller pieces; it takes less assembly time but has half the lifting capacity and reach capacity of the conventional truck crane. The third type is the most expensive to haul to the site. However, crawler cranes can move while picking up the lifted piece and can rotate 360 degrees while handling lateral loads.

One of the lifting schemes uses one crane with a spreader beam to pick up the girder at two points, as shown in Figure 4-45. Usually these are at the two quarter points. Another scheme involves using two or more cranes. This is used in the final stage of erection to get the system in
place. Figure 4-46 shows a crane used to hold the girder before releasing it on temporary shoring. Figure 4-47 shows cranes holding the girder in its final position.

Figure 4-45. Box girder being lifted using one crane and a spreader beam (Advantage Steel 2000)

Figure 4-46. Erection of curved box girder Garden State Parkway Interchange 159, Bergen County, New Jersey (Modern Steel Construction July 2000)
Erectors should provide a detailed report of erection procedures and falsework. The report should include detailed instructions for erecting each component of the bridge, the consequence of putting different parts into place, and instructions on how to construct field splices. The report should also include stress calculations from lifting at different erection stages.

Use of struts for temporary stability is common. However there are no guidelines for ensuring overall stability. Sometimes counter weighing is required, usually as determined in the field. Counter weights, test weights, or old railroad box girders can be used to maintain overall stability.

Three stability concerns should be addressed; overall stability, local stability of the bottom flange, and lateral and local buckling of top flanges. Overall stability deals with the equilibrium of the girder section as a whole during lifting. After erecting, the bottom flange is exposed to tension; however, during erecting the thin bottom flange is exposed to compression stresses, which can lead to local buckling. The last stability concern is the lateral torsional buckling of the top flange, because the narrow flange can buckle laterally if exposed to high compression stresses. The US Steel Construction Manual (1978) provides an example of stability check.
calculations for straight girders. However, with curved box girders, expressions for stability checks are yet to be adopted.

Curved box girders are characterized by heavy weights and large sizes. Large crane capacity and long reaches are required for lifting such girders. This was evident in the use of a 272 metric ton (300 ton) capacity ringer crane with a 54.9 m (180 ft) boom in a curved steel box girder bridge in San Francisco (Engineering News Record 1984). A 15.4 metric ton (17-ton) spreader frame was used to hoist the boxes.

Erection time is critical because it typically requires traffic disruption. Considerable time savings can be achieved by thoroughly studying different erection scenarios and adopting changes to the original plan (if needed). As pointed out by Deerkoski (1991), it was possible to complete the erection six months ahead of schedule for the I-84 and I-94 intersection project in Hartford, Connecticut by changing the construction sequence.

### 4.2.4 Good Fabrication Practices

In a paper entitled “Twelve Commandments for Economic Steel Box Girders,” Kase (1997) listed 12 good practices to help make the construction of box girder bridges easier, worker friendly, and cost competitive. Some of these practices are intended for closed box girders. However, the same practical information can be useful for curved box girders. These practices are:

1. **Good width:** To help the worker do his job on the box girder a minimum inside dimension of 1.2 m (4 ft) is required for free movement of the worker.
2. **Good height:** It is recommended to be not less than 1.5 m (5 ft) to allow the worker or welder to operate without having to bend.
3. **Two access doors:** This practice is especially important for closed box girders. However, the location of the access doors is important for maintenance.
4. **Abide by OSHA rules and regulations addressing confined space:** when forming a confined space by box girder, consider the requirements for safe entry, testing and monitoring, rescue services.
5. **Include access holes and diaphragm manholes no smaller than 0.81 m X 0.91 m (2 ft-8 in. X 3 ft-0 in.).**
6. **Include manholes concentric in location:** A space of 0.076 m (3 in.) between intermediate diaphragms is good for a welder to fit and weld.
7. **Select corner seams conductive to ease of accomplishment:** Full penetration corner seams from the outside can be used to overcome the confined space restriction.
8. **Allow clearances for personnel to perform quality welds and inspection:** Design should be verified by fabricators before finalizing it to accommodate different personnel and make their tasks easier.
9. **Use full penetration welds:** Box girders are typically composed of heavy plates, which require full penetration welds in some cases. Heavy welding can cause stress concentrations...
and distortion. Details usually require fit-up and can be costly. Early consulting with the fabricator can save effort.

10. Limit the use of wide flange beams to fabricate a box: Exact dimensioning and fabricating of sub-assembled components is essential to achieve good box fit-up.

11. Employ limited and similar cross frames and diaphragms that are easily fabricated and erected: Most of the time box girders are handled through jigs. Manufacturing jigs with different sizes would be costly and time consuming.

12. Do not incorporate fracture critical members (FCM) without forethought: clear instructions and requirements are essential for engineering, detailing, procurement, and quality control.

4.3 Evaluating Design Alternatives

There are several criteria that govern the selection of the curved bridge girder system. The significance of each criterion differs between projects. Sometimes one of these criteria dictates the use of a specific girder system, but most of the time the designer must consider all criteria together to decide on a girder configuration. Few articles were found in the literature that discussed these criteria. Chang and James (2002) explained that the decision to implement a single point urban interchange (SPUI) was the result of carefully weighing costs, capacity, available space, and constructability. Hall (1997) discussed selection factors for girder systems including the structural efficiency, functionality, serviceability, maintainability, aesthetic appeal, buildability, and economy. Some of these criteria are described in the following sections.

4.3.1. Aesthetic appeal

Aesthetic appeal is considered one of the major issues for bridge engineering. Most bridge projects nowadays involve much interaction with the public through public hearings and other means of communication. After citizens understand the need for constructing a new bridge or improving an older bridge or highway, they focus their attention on how the new structure will look and how much it will affect the surrounding environment. Hall (1997) identified form and detail as the two major factors affecting the aesthetics of steel bridges.

4.3.2. Structural efficiency

The primary task of the structural engineer is to design an efficient structure that can pass loads to the supports and ultimately to the foundation. Curved bridges are characterized by large torsional stresses due to curvature that are coupled with bending stresses. Efficient structural components for curved bridges must provide high torsional rigidity as well as flexural rigidity.
4.3.3. Cost

The initial cost can be considered the most important factor in determining the material and the cross section. This includes the cost of superstructure and substructure. In a meeting with the staff of the Alabama Department of Transportation Bridge Bureau, they described the use of prestressed concrete girder systems for spans up to 36.6 to 42.7 m (120 to 140 ft) and the use of steel girders for longer spans.

In an article by Bethlehem Steel and PBQD, Norfolk, VA (1997), the authors emphasized the need to optimize spans for steel bridges by considering the total cost of the structure. The authors introduced a method to calculate the individual superstructure and substructure costs for a varying set of conditions. Substructure costs vary primarily by pier height when their unit costs ($/SF) decrease as the span increases (Figures 4-48 and 4-49).

After calculating the cost of substructure and superstructure, the optimum span length for a given pier height is determined by combining the two sets of curves as shown in Figure 4-50.

The authors concluded that the optimum steel span length depended primarily upon the substructure cost, which in turn is mainly dependent upon pier height. That makes it necessary to analyze both the substructure and superstructure costs to select the most economical steel spans. It was also concluded that the optimum steel span length could be very close to that of the concrete alternative when site conditions do not dictate support locations.

![Figure 4-48. Superstructure cost (Bethlehem Steel and PBQD, Norfolk, VA 1997)](image)
Figure 4-49. Substructure cost (Bethlehem Steel and PBQD, Norfolk, VA 1997)

Figure 4-50. Total cost (Bethlehem Steel and PBQD, Norfolk, VA 1997)
4.3.4. Serviceability

The 2003 Guide Specifications provide serviceability limits to ensure proper performance of the bridge during its design life. These limits are based on experience. The limits given by the 2003 Guide Specifications are deflection and concrete crack control.

4.3.5. Maintainability

State DOTs have programs to inspect and maintain bridges during their lifetime. These programs differ from one state to another. However, the goal of these programs is to ensure the safety of the bridge and maintain it with minimum traffic disruption. Finally, maintainability is measured by how easy and economical it is to maintain bridges.

4.3.6. Buildability

Bridges must be buildable with the existing technology and facilities with minimal disruption to the existing highway system and without unexpected delays and claims (Hall 1997).

4.4 Advantages of Box Girder Superstructures

4.4.1. Aesthetic appeal

Hohmann and Holt (2003) state that box girders have become common in Houston due to their visual appeal. Although the box girder cross section is similar to two I-girders, the box section tends to integrate with the deck to form one element that follows the curved alignment and enhance its appearance (Figure 4-51). Also, one noticeable advantage for the box is the possibility of a wider girder with a shallower depth. This can lead to fewer boxes, with large deck overhangs, and fewer pier shafts (Figure 4-52). Bridge structures usually have high visibility. Therefore, reducing the number of visible appurtenances is another advantage of box girder superstructures. All stiffeners and bracing members can be hidden to maintain a smooth appearance and minimize exposed steel surfaces (Figures 4-51 through 4-55).

A good example of an aesthetically appealing box girder system is the steel box girder bridges of the Greater Toronto International Airport Redevelopment Project where the deck width varies from 12 m (39 ft) to 18.7 m (61 ft) and the two trapezoidal box girders diverge along the length of the bridge to accommodate this widening. Figure 4-55 illustrates how the box girders diverge along the alignment of this project (Ojala 2003).

Another example is the interchange near downtown Shreveport, LA, where visibility made aesthetics a prime consideration (Modern Steel Construction 1991). Thus, box girders were selected for the I-49 and LA 3132 interchange because of their clean appearance.
The aesthetic advantages of using curved box girders was emphasized in the ramps leading to the intermodal transportation complex in Boston. This complex was planned to combine railroad operations, and regional and local bus operations with a link to Logan airport operations. The ramps had high visibility as they are exposed to commuters, thus making aesthetic considerations very important. “The aesthetic goals for the design have resulted in the selection of horizontally curved steel trapezoidal single and multiple box girders made composite with concrete deck. With its smooth, clean and attractive appearance, this type of structure has proven to be aesthetically pleasing to the general public and has the geometric flexibility needed for this complex structure” (Shumway and McLellan 1995).

Figure 4-51. Steel ramp near Seattle (Hall 1997)

Figure 4.52. Steel box ramp near Miami (Hall 1997)
Figure 4-53. Curved box girder bridge in Chullora, Sydney, Australia (One Steel 2003)

Figure 4-54. Curved box girder Bridge in Chullora, Sydney, Australia (One Steel 2003)
4.4.2. Structural Efficiency

Trapezoidal box steel girders with concrete decks are a structurally efficient alternative for resisting bending in curved bridges. This configuration allows the use of steel through the bottom flange in areas where stresses are largely tension, and the concrete in the composite section for compression. In addition, the trapezoidal shape minimizes steel by having the width in the bottom flanges less than at the top.

Torsional rigidity is an important criterion for efficient structure design of curved bridges. “The closed box section of the box in the completed bridge has a torsional stiffness that may be 100 to more than 1000 times the stiffness of a comparable I-girder section,” (Fan and Helwig 1999). This makes the box section the preferable cross section when torsional stresses control the design, especially with sharp curvature and large spans. Yet, box sections can provide stability for widely spaced girders and permit large deck overhangs. The inherent torsional rigidity of curved steel boxes permits shipping and erecting without external supports.

4.4.3. Cost

Surtees and Tordoff (1977) introduced an optimization method to appraise and estimate costs of different layouts of box girders with concrete deck. The optimization method was based on the selection of a set of design variables to get a minimum weight and cost. Eleven design variables were selected including stiffener spacing, width and thickness of the top flange, bottom flange, web, and deck. These variables were calculated for different designs according to the British
standards. Bending and shear stresses were calculated according to elastic theory with linear strain distribution, including the effects of shear lag, pure torsion, and flexural distortion. It was found that minimum cost and minimum weight solutions are significantly different. If cost is considered the primary factor, it can lead to heavier designs; if weight is considered the primary factor, it leads to more expensive designs.

Price (1993) highlighted some of the new trends and innovations that can enhance many aspects of box girder bridges such as design, durability, constructability, and costs. Price discussed three of these trends and innovations: reduction in the number of boxes, post-tensioning of concrete deck, and construction technology. It was shown that design for fewer boxes can reduce the total live load for a given cross section after applying the live load distribution given by AASHTO. The use of post-tension concrete in bridges in Ontario, Canada, was presented as a solution for increasing the deck spans due to the use of fewer boxes. Incremental launching was presented as a proven solution where there are site constraints that prevent the use of conventional erection techniques.

Fabrication cost per pound of steel box girders is known to be higher than for I-girders. Thus, in order to use economical box girders, the total cost should be less than the equivalent I-girder bridge. Hall (1997) suggested measures that would result in economical designs for box girder bridges including using minimum number of girders, using one pier shaft per box, and using single box cross sections for ramps with large deck overhangs. The savings in girder and substructure costs usually exceed any added deck cost.

A complete cost estimate report must include the cost of all the components of the bridge. In an excel spreadsheet found on the Florida DOT web page, costs associated with the substructure and costs associated with the superstructure were considered separately. The substructure part had the unit prices for alternatives such as pre-stressed concrete piling, steel piling, or drilled shaft. The superstructure contained unit prices for bearing materials, steel bridge girders, pre-stressed concrete girders, and cast-in-place superstructure concrete. In this spreadsheet, the material costs for a curved box girder was about 25 percent more than the comparable I-girder. Furthermore, the curved steel girder was generally about 15 percent more expensive than straight. However, to get accurate comparisons between design alternatives, the total weights must be calculated and the total cost including the cost of the substructure must be evaluated. This spreadsheet was developed to comply with the “Structures Manual” (FDOT 2003).

4.4.4 Serviceability

One box girder is usually used in place of two I-girders. For I-girder systems, the outermost girder is subjected to heavier load. Therefore the serviceability limits, which include limiting the deflection, might lead to a larger section. On the other hand, in the case of box girder superstructures, the box resists the load as one unit and thus can be less affected by the serviceability limit.
4.4.5. Maintainability

The intermediate bracing of the box girder is hidden inside the box. Hence, the area of the exposed steel is reduced over that of the I-girder system. Minimizing the area of exposed steel increases the resistance to corrosion and reduces the cost of repainting. Also, it eliminates many parts that attract debris, which can retain moisture. According to Hall (1997), this is especially beneficial for weathering steel applications.

4.4.6. Buildability

Wide curved box girders can be shipped in halves to overcome the shipping size limitation through highways and rail. This can be done if the girders are designed with a bolted seam along the bottom flange. The advance in fabrication technologies, including machines that cut and drill holes according to computer software programs, allows more accurate bolt hole locations.

In a bridge in San Francisco, the selection of a steel box girder was controlled by a requirement that no shoring interfere with trains (Engineering News Record 1984). The choice of box girders for this project also fit the sharp curvature route that was selected to gain the necessary 7 m (23 ft) clearance at a maximum grade of 6 percent.

4.5 Limitations of Using Box Girders

The box girder consists of an open section “quasi-closed” during transporting, erecting, and constructing. The low torsional stiffness causes major concerns and requires careful design during the early stages of construction when the steel section may be subjected to large torque. Hall (1997) states that internal cross bracing caused fatigue distress in box girders when forces were not properly considered and a load path was not properly designed to transmit cross frame forces to the girder flanges. Fatigue distress can cause fatigue cracks that are expensive to repair and affect the serviceability of the bridge. Curved box girder sizes can sometimes represent a challenge in shipping. Transporting by highways limits the weight, length and width.

4.6. Examples of Box Girder Bridge Construction

In this section, example projects are presented to illustrate the various factors involved in selecting and designing box girder bridge superstructures. In the second example, “Mississippi Single Point Urban Interchange,” detailed information is given about the structure and the cross sections that provide an idea about the sizes and construction issues encountered.

King County Mt. Si Bridge Replacement, WA
The King County Road Services Division described public meetings organized to gain comment on plans for replacing the narrow, deteriorating Mount Si Bridge across the Middle Fork of the
Snoqualmie River in King County, Washington. The replacement design memorandum (ABKJ Inc. 2001) defined the goals of the project as:

1. Construct a new bridge structure designed to current criteria.
2. Improve motorist and pedestrian safety on the bridge and roadway approaches.
3. Maintain traffic at an acceptable level of service throughout construction.
4. Minimize changes to, or enhance, the existing bridge hydraulic characteristics.
5. Minimize right-of-way acquisition and other impacts to adjacent residents.
6. Minimize impacts to the natural environment.

Six alternatives allowed the use of the existing bridge during construction to keep the traffic moving, which was one of the project goals. Four of the main span alignments were straight with minor curves for the approach spans, and only one curved alignment with no approach spans was considered. The cost per square foot ranged from $175/ft² to $300/ft². The curved steel plate girder (Alternative 4A) cost $230/ft² and the curved steel box girder (Alternative 4B) cost $240/ft². According to the memorandum, the curved steel plate girder alternative was recommended. This alternative had less right of way impact, the lowest project cost estimate, and mitigated environmental impacts, making this option one of the most attractive. From this example, one can notice how different factors can dictate the final decision for the structural system. Also, differences in costs between the curved steel plate girder and curved box girder were less than 5 percent. Cost seems to be the deciding factor after considering the other factors.

**Mississippi Single Point Urban Interchange**

This structure links Interstate 55 and Mississippi Route 463 in a bow tie shape. The owner of the project was the Mississippi Department of Transportation. The prime design consultant for this project was the Jackson office of the Pittsburgh-based Michael Baker Corporation. The office of Tensor Engineering of Indian Harbor Beach, Florida, performed the detailing of shop drawings (Chang and James 2002).

The structure consisted of two sides (Figure 4-56). Each side had six curved box girders with a curvature that ranged up to 45 degrees. The central girder was straight. The outer-most girder had the sharpest curvature. The designer chose curved steel girders instead of short straight diagonal girders to accommodate the curve design and minimize traffic disruption. The box cross section was chosen to utilize the high torsional rigidity for such sharp curvature. The designer analyzed the structure with three-dimensional finite element software.

According to Chang and James (2002), the girders were 30 m (98 ft) long, 1.53 m (5 ft) deep, 2.44 m (8 ft) wide at the top, and 1.75 m (6 ft) wide at the bottom. Due to the sharp curvature of the outside girders, they were shipped in five pieces. The inner girders were shipped in three pieces. Thus, the girders with sharp curvatures required more splices.

The diaphragms at the supports were steel plates while the intermediate cross frames were K-frames. The number of cross frames per girder increased with the increase in curvature. While only 10 bolted K cross frames were used for the straight girders in the center, 20 bolted K cross frames were used for the outer girders.
Garden State Parkway Interchange 159 Bergen County, New Jersey
This bridge provided a connection between the southbound Garden State Parkway and Eastbound I-80 to relieve severe peak hour congestion. The project owner was the New Jersey Highway Authority; the designer was Parsons Brinckerhoff; the steel fabricator was Tampa Steel Erecting; the steel detailer was Tensor Engineering; and the Steel Erector was Archer Steel Construction (Modern Steel Construction 2000).

The structure consisted of a cast-in-place concrete deck that was supported on a monocell trapezoidal box girder that extended over eight spans with a total length of 266.7 m (875 ft) and a radius of 70.1 m (230 ft). The two end spans were 24 m (78 ft, 9 in) and 26.1 m (85 ft and 9 in) configurations and the six interior spans were 36.1 m (118 ft, 5 in).

The box cross section was determined to be the most structurally efficient geometric configuration for such sharp curvature. The width at the top of the flange was 3.8 m (12.5 ft). Adding two deck overhangs of 2.2 m (7.25 ft) enabled the designer to have only one box for a roadway width of 7.3 m (24 ft). Using one box girder required one pier for each support, which made the box girder solution economically competitive. The architecture of the piers was enhanced to improve the appearance of the structure. To account for maintainability, stainless steel reinforcement was used for the deck to extend its life.
Interchange near the Intermodal Transportation Complex in Boston

A curved steel box girder was the ideal solution for the south bay interchange that connects I-93 and I-90 in Boston. Shumway and McLellan (1995) showed how the use of curved steel trapezoidal box girders overcame the very restrictive site constraints and provided an aesthetically pleasing structure adjacent to Boston’s south station transportation center. The structure merged two ramps into one structure and then branched into three ramps within a distance of 103.6 m (340 ft). These restrictions on the horizontal layout, along with the requirements for vertical clearance represented a challenge for the designer. Moreover, the required connection to the bus terminal that was under construction added to site constraints and required a design that was easy to fabricate and erect. The published drawings of the project layout showed that the girders had various tied radii of curvature that range from approximately 36.6 m (120 ft) to 60.7 m (200 ft). The design included the use of only two sizes for the web plates. These sizes were selected to avoid using transverse and longitudinal stiffeners for most of the box girders to ease fabrication and lower cost. Unstiffened bottom flanges were used with a maximum unsupported slenderness limited to 120. Removal of the exterior diaphragms between boxes was evaluated, but it was decided to leave the diaphragms in place.

4.7 Optimizing Design of Box girders

Design of curved box girders includes selecting parameters such as span length, depth, width, and thickness. Some of these factors can be controlled by site conditions. Others must be optimized to get the most economical box configuration. Heins (1981) and later Fan (1999) analyzed torsion as two components, Saint Venant and distortional torsion. These two components are directly proportional to the width and inversely proportion to the depth. Thus, the width-to-depth ratio of the box becomes a very important parameter in optimizing the design.

Article 12.2 of the 2003 Guide Specifications limits the span-to-depth ratio to 25 for simple spans and uses modification factors of 0.9 and 0.8 for the length of continuous span from one end and two ends. This ratio is preferred for limiting the deflection and is used in the example presented in the 2003 Guide Specifications as a starting estimate for the depth. The following is a discussion of guidelines for good design practices for the web, flanges, and cross frames.

Web Design

Straight box girder webs can be “unstiffened” up to a slenderness ratio of 150. Due to the lack of tests for slenderness ratios greater than 70, the 2003 Guide Specifications reduce the slenderness limit for curved girders to 100 for a radius less than 213 m (700 ft). The permitted web slenderness increases linearly to 150 at a radius of 609 m (2000 ft). These limits are given as follows:

For  \( R \leq 700 \) (Equation 6-1 of the 2003 Guide Specifications)
\[
\frac{D}{t_w} \leq 100
\]  \hspace{1cm} (4-6)

For \( R > 700 \) (Equation 6-2 of the 2003 Guide Specifications)

\[
\frac{D}{t_w} \leq 100 + 0.038(R - 700) \leq 150
\]  \hspace{1cm} (4-7)

where,

\begin{align*}
D &= \text{distance along the web between flanges (in.)} \\
t_w &= \text{web thickness (in.)} \\
R &= \text{minimum girder radius in the panel (ft)}
\end{align*}

The decision about whether to stiffen a thin web or to use thicker webs with no stiffeners can contribute to cost savings. Up-to-date knowledge of the plate size availability from the producers along with the prices can help make the right selection for the web thickness. Heins (1981) reported that, due to more intense fabrication requirements, stiffeners cost up to six to eight times the web material on a weight basis. He recommended starting with appropriate thickness and selecting a web depth that is maximum for the web thickness chosen because plate material comes in discrete thicknesses. Heins also suggested that the limit of 150 for webs penalizes high-strength webs by not permitting the designer to use the available shear strength. That is why the use of hybrid flange-web girders is economical.

**Flange Design**

Heins (1981) reported that the most common type of longitudinal flange stiffener is an inverted tee section, which provides the desired stiffness with minimum material. This type of longitudinal stiffener was found common in the examples collected by the research team. The symmetry in the tee section reduces the tendency for lateral torsional buckling compared to unsymmetrical cross sections that can be used as stiffeners. The 2003 Guide Specification does not stipulate a flange slenderness limit for the use of longitudinal stiffeners. However, according to Hall (1978), many designers use longitudinal stiffeners when this ratio exceeds 45.

**Diaphragms and Cross Frames**

Solid diaphragms and K-type cross frames composed of angles are used to control distortion of bridge box girders. The 1993 edition of the Guide Specifications provided an equation for diaphragm spacing that was based on the work of Oliniek (1978). This equation was assumed to limit the distortion stresses to 10 percent of the bending stresses. The 2003 Guide Specifications provides the 10 percent limit, but leaves it up to the designer to calculate the distortion stresses. The 2003 Guide Specifications also limits the spacing to a maximum of 9.1 m (30 ft) and the
bending stress to 138 Mpa (20 ksi) at the limit state. Generally, it is good design practice to include a large number of diaphragms or cross frames with less stiffness than a few very rigid diaphragms (Heins 1981), as rigid diaphragms can produce large local forces on the flanges and webs.

4.8 Research Needs

4.8.1. Curved Box Girder Behavior

In two papers on analysis and design of curved box girders, Sennah et. al. (2001, 2002) concluded that the finite element method is the most general and comprehensive technique among the refined methods for analysis of box girder bridges. He also concluded “the current North America codes as well as the published literature do not provide the design engineer with adequate information on the behavior of unshored straight and curved box girder bridges during construction phase.” He recommended studying the following items:

1. Behavior of straight and curved box girders under construction;
2. Load distribution for curved box girder for different conditions of loading, cross sections, supports;
3. Impact factors for curved cellular bridges;
4. Fatigue response of curved box girders; and
5. Simple expressions for load carrying capacity in straight and curved box girders.

4.8.2. Local Stability

The diaphragms and cross frames are used to retain the cross section of the box and minimize distortion effects. The forces in these cross frames are usually small for straight box girders. However, these forces can be significant for curved girders. Research is needed to address the effect of these forces on the local stability of box webs and flanges.

Heins et al. (1981) reported that there is no consideration in AASHTO for overall buckling of the longitudinal stiffeners as a column. He added that this mode of failure is usually not critical because of the short negative moment regions. This is the case for simply supported straight girders. However, for continuous curved box girders and for curved box girder during erection, the negative moment extends for longer regions and hence guidelines are needed for checking local stability at these regions.

4.8.3. Global Stability

According to Section C10.4.2.4 of the 2003 Guide Specifications, the strength of slender compression flanges is based on elastic buckling of an infinitely long plate with clamped edges subjected to a uniform in-plane compressive stress and shear stress as calculated by classical
torsion theory presented by Timoshenko and Gere (1961). Numerical investigation is essential to update the strength of these slender flanges under compression for curved girders.

4.8.4. Nonlinear Behavior

Fan (1999) noticed that the stress values at the critical sections above the support for continuous curved box girders deviate from those predicted by beam theory. He recommended further study for the nonlinear stress distribution. He also recommended studying the stability of the bottom flange in the negative moment area above the support because it is under a very complex state of stress.

4.8.5. Lifting of Curved Box Girders

Grubb et al. (1996) provided a formula to determine the location of the center of gravity. Studying the different lifting scenarios of curved box girder and the associated stresses with each case is very important. Lifting from quarter points will result in compressive stresses in the bottom flange, which might not have been designed for compression. Hence studying stability of the curved box segments during lifting is also needed.
5.0 Conclusions and Recommendations

A synthesis of curved steel bridge research literature and current practice was developed. Both I-girder and box girder superstructure configurations were considered. More than 200 literature sources were collected, reviewed, analyzed, and integrated into the synthesis. The Chapter 2 component of the synthesis presented an overview of general stability research related to the behavior of curved girders and the development history of the Guide Specifications for Horizontally Curved Bridges. Chapter 2 was logically organized by behavioral issues: cross frame spacing, lateral-torsional buckling, flange plate stability, web panel stability, and ultimate strength. Chapter 3 synthesized stability research and current practice related to the construction of curved I-girder bridges. Challenges associated with fabricating, transporting, and erecting were emphasized. Chapter 4 synthesized stability research and current practice related to the stability and construction of curved box girder bridges. As with Chapter 3, challenges associated with fabricating, transporting, and erecting the steel frame were outlined. Additionally, practical considerations involved in the design selection process such as aesthetic appeal, structural efficiency, cost, serviceability, maintainability, and buildability were discussed. Several construction projects were also reviewed.

A local curved bridge construction project was studied from inception and preliminary design through placement of the girders and concrete deck. The study provided a thorough understanding of the curved I-girder bridge design and construction process. The close proximity of the project to UAB and the ability to safely observe the progression of construction provided a unique opportunity to document a large-scale curved bridge construction project. Many of the persons involved in the flyover construction were interviewed, including ALDOT project managers, contractor project engineers, erectors, fabricators, and others. The fabrication company was visited, engineers were interviewed, and digital images were taken to provide an understanding of problems that arise during the curving process. An understanding of challenges while transporting within the fabrication facility and from the fabrication facility to the jobsite was acquired. Erectors were interviewed to gain a practical, real-world perspective on challenges that occur at the jobsite during lifting and placing of the girders. Digital images were taken on a bi-weekly basis for several months to provide a record of the project. The results from this study were used in Chapter 3, and a separate report on the construction project was developed (Osborne 2002).

Similarly, the fabrication of steel box girders was studied for a curved box girder bridge that will be constructed in Florida to gain an understanding of practical challenges associated with the fabrication of curved box girders. Numerous interviews with engineers and technicians were carried out during the visits, and the results were integrated into Chapter 4. The team plans to continue this aspect of the study as the construction project progresses.

From a national perspective, the design, fabrication, and construction of curved I-girder bridges is more mature than that of curved box girder bridges. However, significant research challenges remain for curved I-girders. The AASHTO Guide Specifications for Horizontally Curved
Highway Bridges is widely recognized to be outdated, disjointed, and difficult to use, so strength and stability research is needed to improve design formulations. Due primarily to weak torsional rigidity and a very complex distribution of stresses over the cross section, problems associated with fabricating, transporting, and erecting curved girders are more prevalent in curved I-girder construction than in straight bridge construction. Engineers not experienced in the design of curved bridge systems often make the mistake of assuming that behavior and design is the same as that for straight bridges. Instability during construction can easily translate into unsafe conditions for construction workers, not to mention unforeseen additional costs. Even though problems are common, there are no comprehensive guidelines specific to the construction of curved bridges. Based upon the review of literature and interaction with industry leaders and prominent curved bridge researchers, the following were identified as discrete stability research topics in need of further investigation.

- Effects of curvature on curved I-girder flange stability, including elastic behavior, post-buckling behavior, and ultimate strength.

- Prediction of distortion induced web stresses that result from curvature for use in fatigue calculations.

- Effects of curvature on the lateral-torsional elastic buckling behavior of curved I-girders in the braced frame superstructure.

- Guidelines for deck placement sequencing to minimize construction stresses, fit-up problems, and erection stability problems for curved I-girders.

- Maximum stress predictor equations and guidelines for transporting and lifting single curved I-girders during construction.

- Simplified ultimate strength predictors for transition into LRFD format specifications.

- Requirements for transverse and longitudinal stiffener spacing and rigidity for curved plate girders based upon curvature induced distortion.

- Plate girder web panel slenderness requirements for regions of pure shear and combined bending and shear.

- Internal and external cross frame, diaphragm, and lateral bracing spacing and rigidity requirements for curved box girders, during construction and in load bearing configurations.

- Guidelines for deck placement sequencing to minimize construction stresses, fit-up problems, and stability problems for curved box girders.
• Guidelines for temporary shoring to minimize stresses, fit-up problems, and stability problems for curved box girders.

• Guidelines for transporting and erecting curved box girders.

• Equations to predict effects of distortion on the strength of curved box girders.

• Ultimate strength behavior of curved box girders.

There is need for research that involves partnerships of federal, state, academic, and industry to solve problems associated with the design and construction of curved bridges. Through interaction with industry leaders and leading researchers, this project identified many areas of stability research need. However, funding opportunities are limited. The Federal Highway Administration Curved Steel Bridge Project (FHWA-CSBRP) being conducted at the Turner-Fairbanks Highway Research Laboratory in McLean, Virginia, has been ongoing since 1992 and is the only large-scale federally sponsored and coordinated curved bridge project today. Currently, FHWA is wrapping up this program and synthesizing the results. After the FHWA-CSBRP is finalized, there may be a shift in interest towards curved box girder bridge research that will offer the opportunity for researchers who are not currently sponsored. There may be other avenues for stability funding such as through the American Iron and Steel Institute (AISI), the Steel Bridge Alliance, AISC, and others. Funding from agencies such as the National Science Foundation is not likely due to the applied nature of the research.

ALDOT has not yet taken advantage of enhanced torsional resistance and the aesthetic appeal provided by box girders and depends on contractors to do the construction engineering of curved I-girder bridges. No severe problems have been encountered with the construction of curved bridges in Alabama. There are upcoming curved bridge construction projects in Alabama that will involve significant curvature and there may be future interest in sponsoring erection stability projects and projects that may look into box girder options. This may translate into opportunities for this program to assist ALDOT in the near future.
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