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STATE HIGHWAY ADMINISTRATION

RESEARCH REPORT

SERVICEABILITY-RELATED ISSUES FOR BRIDGE LIVE LOAD DEFLECTION AND CONSTRUCTION CLOSURE POURS

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16. Abstract

This study investigated the design criteria and practices in an effort to improve the quality of bridge designs in the State of Maryland and beyond. This first criterion investigated was the live load deflection for steel bridges. The second design/construction criterion investigated was designing and detailing bridge deck closure pours. Previous and current practices and future planning on the serviceability of bridges have been documented. State-of-the-practice methods from federal and other state agencies were collected. Three bridges were chosen for refined analyses to investigate the live load deflections. Field measurements for these three bridges were collected from the research team to facilitate this study. Thirty steel girder bridges from the Maryland State Highway Administration's (SHA) inventory were selected for statistical analyses. Steel bridges designed with the live load deflection limit have been evaluated. Closure-pour analyses were conducted by line-girder models, two-dimensional grid models or three-dimensional finite element models. All three methods generate accurate enough camber diagrams to predict differential deflections between stages for straight girder systems, if creep is not considered. Creep effect could be alleviated by proper camber and scheduling on pouring.

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Table of Contents

Table of Contents i

List of Tables iii

List of Figures iv

1. Introduction 1

2. Literature Review 3

    2.1 Serviceability-related to Steel Girder Bridges under Live Load 3

        2.1.1 Live Load Deflection Studies and Design Criteria 3

        2.1.2 Codes and Specifications in Other Countries 7

    2.2 Construction Closure Pours 11

        2.2.1 Federal Highway Administration's (FHWA) Regulation 11

        2.2.2 Other States' Practices 11

        2.2.3 Research and Testing Findings 12

3. Live Load Deflection Validation of Steel Bridges 15

    3.1 Loading considered in the Load and Resistance Factor Design (LRFD) and the Allowable Stress Design (ASD) 15

        3.1.1 Loading considered in the Allowable Stress Design 15

        3.1.2 Loading considered in the Load and Resistance Factor Design 17

        3.1.3 Summary of Loading in Two Methods 18

    3.2 Introduction of Computer Programs used in This Study 18

        MERLIN-DASH 18

        DESCUS-I 19

        CSiBridge 20

    3.3 Three Representative Maryland Bridges 20

4. Refined Analysis of Three Representative Maryland Bridges 22

    4.1 Model Analysis of the I-270 over Middlebrook Road Bridge 22

    4.2 Model Analysis of the Route 1 over Paint Branch Bridge 26

    4.3 Model Analysis of I-95 over Patuxent River Bridge 29

    4.4 Summary of Live Load Deflection Comparison 33

5. Results of 30 Sample Bridges Using the Line-girder Method 37

    5.1 Live Load Deflection Analysis 37

    5.2 Load Rating Analysis 48

6. Construction Closure Pours Case Study 50
6.1 Model analysis of the MD140 over MD27 Bridge 51
6.2 Summary of Displacement Comparison by Refined Analyses....................................56
7. Summary and Conclusion 61
References...64

List of Tables

Table 2.1 Historic Depth-to-Span, D/L, Ratio for Highway Bridges 4
Table 3.1 Multiple Presence Factors in ASD Method 17
Table 3.2 Multiple Presence Factors in the LRFD Method 18
Table 3.3 Loads and Factors considered in ASD and LRFD Methods 18
Table 4.1 Beam Sections 23
Table 4.2 Dead Load Information of Route1 Bridge 27
Table 4.3 Dead Load Information of the I-95 over Patuxent River Bridge 30
Table 4.4 Beam Sections 32
Table 4.5 Live Load Deflection of Three Representative Bridges 34
Table 4.6 Live Load Deflection of Three Bridges with Adjustment 35
Table 5.1 Single-span Bridges Live Load Deflection 39
Table 5.2 Two-span Bridges Live Load Deflection 40
Table 5.3 Three-span Bridges Live Load Deflection 42
Table 5.4 Single-span Bridges Live Load Deflection with Modified Distribution 44
Table 5.5 Two-span Bridges Live Load Deflection with Modified Distribution 45
Table 5.6 Three-span Bridges Live Load Deflection with Modified Distribution 47
Table 6.1 Maximum Displacements Comparison for Normal and Staging Models by DESCUS 57
Table 6.2 Comparison Table between Different Models with Diaphragms 57
Table 6.3 Comparison Table between Different Models without Diaphragms 57

List of Figures

Figure 2.1 First Flexural Frequency versus Static Deflection 8
Figure 2.2 Dynamic Load Allowance 9
Figure 2.3 Deflection Limits for Vibration Controls of Australian Codes 10
Figure 3.1 HS-20 Design Truck 16
Figure 3.2 HS-20 Lane Load and Concentrated Load 16
Figure 4.1 Typical Cross Section of the I-270 over Middlebrook Road Bridge S.B.R 23
Figure 4.2 Girder Elevations 23
Figure 4.3 I-270 over Middlebrook Road Bridge Model in MERLIN-DASH 24
Figure 4.4 DESCUS-I Graphic of the I-270 over Middlebrook Road Bridge 25
Figure 4.5 I-270 over Middlebrook Road Bridge Model in CSiBridge 26
Figure 4.6 Typical Cross Section of the Route 1 over Paint Branch Bridge 27
Figure 4.7 Elevation 27
Figure 4.8 Route 1 over Paint Branch Bridge Model in MERLIN-DASH 28
Figure 4.9 DESCUS-I Graphic of the Route 1 over Paint Branch Bridge 28
Figure 4.10 Route 1 over Paint Branch Bridge Model in CSiBridge 29
Figure 4.11 Typical Cross Section at Mid-span of the I-95 over Patuxent River Bridge 30
Figure 4.12 Partial Girder Elevation-N.B.R 30
Figure 4.13: The I-95 over Patuxent River Bridge Model in MERLIN-DASH 31
Figure 4.14 Girder Elevation of Half of the I-95 over Patuxent River Bridge in DESCUS-I 31
Figure 4.15 Half Framing Plan of the I-95 over Patuxent River Bridge in DESCUS-I 32
Figure 4.16 The I-95 over Patuxent River Bridge Model in CSiBridge 33
Figure 4.17 Distribution of Live Load along the Roadway 35
Figure 5.1 Distribution of Span Length for Sample Bridges 37
Figure 5.2 Deflection vs. Span Length for All Sample Bridges 38
Figure 5.3 Deflection vs. Span Length for Single-span Bridges 39
Figure 5.4 Deflection vs. Span Length for Two-span Bridges 40
Figure 5.5 Deflection vs. Span Length for Three-span Bridges (Side Span) 41
Figure 5.6 Deflection vs. Span Length for Three-span Bridges (Mid Span) 41
Figure 5.7 Deflection vs. Span Length for Single-span Bridges with Modified Distribution 43
Figure 5.8 Deflection vs. Span Length for Two-span Bridges with Modified Distribution 44
Figure 5.9 Deflection vs. Span Length for Three-span Bridges (Side) with Modified Distribution 46
Figure 5.10 Deflection vs. Span Length for Three-span Bridges (Mid) with Modified Distribution 46
Figure 5.11 Allowable Stress Rating 48
Figure 5.12 Load and Resistance Factor Rating 49
Figure 5.13 Rating Factor Ratio (LRFR/ASR) 49
Figure 6.1 Typical Cross Section of Bridge MD140 over MD27 51
Figure 6.2 Girder Elevations (Mid Strip) 52

Figure 6.3 Bridge Pouring Stages and Sequences 52
Figure 6.4 DESCUS Model 53
Figure 6.5 CSiBridge Model for MD140 over MD27 Bridge Isometric View 54
Figure 6.6(a) Time Dependent Concrete Strength in CSiBridge Model (kip/in^2) 55
Figure 6.6(b) Time Dependent Concrete Stiffness in CSiBridge Model (kip/in^2) 55
Figure 6.6(c) Time Dependent Creep Coefficient in CSiBridge Model.................................56
Figure 6.7 Vertical Displacement Results from CSiBridge (with Diaphragm) (inch). 58
Figure 6.8 Vertical Displacement Results from CSiBridge (without Diaphragm) (inch). 58
Figure 6.9 The Creep Growth in Girder 11(Orange) and 12(Green) for with Diaphragms Model (in)..59
Figure 6.10 The Creep Growth in Girder 11(Orange) and 12(Green) without Diaphragms Model (in). 60
Figure 6.11 The Creep Growth in Girder 11(Orange) and 12(Green) without Diaphragms Model (in). 60

SERVICEABILITY-RELATED ISSUES FOR BRIDGE LIVE LOAD DEFLECTION AND CONSTRUCTION CLOSURE POURS

EXECUTIVE SUMMARY

This study investigated the design criteria and practices in an effort to improve the quality of bridge designs in the State of Maryland and beyond. This first criterion investigated was the live load deflection for steel bridges. Since the live load deflection criterion is optional in the AASHTO LRFD Bridge Design Specifications (2014), the Maryland State Highway Administration (SHA) establishes no maximum limit on deflection and leaves the burden on the designers to establish limits. This study developed a menu of criteria that designers can choose from in their bridge designs.

The second design/construction criterion investigated was designing and detailing bridge deck closure pours. A closure pour is a small area of concrete bridge deck that connects two portions of a bridge deck placed in different stages of construction. For staged construction, the designer should consider the deflections of the bridge on either side of the closure pour to ensure proper transverse fitting.

In order to achieve these two objectives, the following tasks were completed:

1) Previous and current practices and future planning on the serviceability of bridges were documented. This study looked at bridges within the short and median span range and selected 30 samples from SHA's inventory; all are steel girder bridges, where the highest live load deflection occurs. Steel bridges designed with the live load deflection limit were evaluated and summarized in this study.
2) The next step was to collect and study state-of-the-practice methods from federal and other state agencies. All available current state-of-the-practice methods from the Federal Highway Administration's regulations, research and testing findings in the past and also the practices from other states were located, collected and listed for study. Three bridges, the I-270 over Middlebrook Road (bridge no. 1504200), Route 1 over Paint Branch (bridge no. 1600400) and I-95 over Patuxent River (bridge no. 1619701) were chosen for refined analyses to investigate the live load deflections. Field measurements for these three bridges were collected from the research team to facilitate this study.
3) Several finite element models, with different software, were developed for the entire bridge to compare the differences in deflection for the bridge model versus the simple single girder analysis traditionally performed by SHA. The two-dimensional grid models and three-dimensional finite element models can be used for the live load deflection analysis as well as the staged construction analysis. In addition to the I-270 over Middlebrook Road (bridge no. 1504200), Route 1 over Paint Branch (bridge no. 1600400)

and I-95 over Patuxent River (bridge no. 1619701) bridges, the MD 140 bridge (bridge no. 6032) was modeled and studied to identify the impacts resulting from different construction methods.

4) A summary of all the work listed above is included. Recommendations associated with precast concrete beam or steel girder construction, complemented with current Maryland practices on live load deflection limit and closure pours, are listed below:

A: Findings associated with bridge live load deflections -

1. Span Length (L)/800 is appropriate for the live load deflection limit for steel bridge design no matter what type of design load or design method is applied. The maximum 1/800 of the span length for general vehicular bridges and 1/1000 of the span length for vehicular bridges with pedestrian traffic are universally accepted criteria for the live load deflection limit.
2. The live load deflection from the HS-25 design truck alone in the Allowable Stress Design (ASD) method (employed by the State of Maryland from 1990 until 2008, the year when the Load and Resistance Factor Design (LRFD) was adopted) is larger than the deflection from the larger of the HL-93 design truck load alone or HL-93design lane load +25% truck load in the LRFD method. Therefore, if the "HS-25 equivalent" truck is required by Maryland for deflection criteria, a factor of 1.25 is suggested for usage in the HL-93 design truck to obtain conservative results. In bridge deflection analysis, the lane load governs for bridges that have a longer span length while the design vehicular load governs for those with shorter spans.
3. Comparing the numeric results from two-dimensional grid models and three-dimensional finite element models, the line girder method proves to be an acceptable application for live load deflection analysis of steel beam/girder bridges with all lanes loaded. Short-term field monitoring using a laser device also found live load deflections are within these limits.
4. AASHTO LRFD Specifications (2014) allows an average value (number of lanes/number of girders) used for the investigation of maximum absolute deflection for straight girder systems with all girders treated as equal. This study found it is generally true for bridges that are not too wide. By using line-girder programs wider-than-three-lane bridges may cause a discrepancy with relatively smaller live load deflection. When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded and all girders can be assumed to deflect equally as stated in the AASHTO LRFD Specifications (2014). The line-girder program with dynamic load allowance and average distribution factor (number of lanes/number of girders) can be used, but the multiple-presence factor should be removed.

B: Findings associated with bridge construction closure pours -

5. The general practice in Maryland is to use a line-girder program to establish the camber diagrams. The result is generally accurate enough and acceptable in practice. Multiple girders with varied girder spacing are grouped into several camber diagrams. It should be

noted that camber diagrams are not grouped based on their tributary widths, but rather, on the narrower girder spacing. In this case some girders may be under-cambered. This usually does not cause problems for one-stage construction, but may cause trouble for multi-stage construction if one side of the closure pour is under-cambered.

6. Multiple camber diagrams can be calculated by the line-girder models, two-dimensional grid models or three-dimensional finite element models. All three methods generate results accurate enough for straight girder systems.
7. Maryland adopted the practice of a minimum closure width of three (3) feet and diaphragms/cross frames in the staging bay of structural steel girders not rigidly connected until later, which are also the research team's recommendation.
8. To investigate the staging effect of a staged-construction, two-dimensional grid models and three-dimensional finite element models are highly recommended. The differential displacement between stages could not be considered in a line-girder model.
9. When comparing the results of the two-dimensional grid model with those of the three-dimensional finite element model, these two methods produced results accurate enough for straight girder systems. However, since the two-dimensional grid model program has to simulate the closure pour by subtracting the deck loads during staged construction, the three-dimensional finite element model can be more closely simulated with a gap on the deck. Also, in the case where the closure pour is significant, the three-dimensional finite element model would provide more accurate results.
10. For further staging analysis, the creep effect of concrete was considered in this study. Two controlled sets of models were studied. One model assumes the diaphragms in the closure pours always connect with girders during the whole staged construction. The other model assumes the diaphragms are disconnected. To investigate the creep effect with concrete slab, two different model analyses from DESCUS and CSiBridge were performed. For the staging analysis of DESCUS, only the time-dependent property of Young's Modulus was considered. However, for the CSiBridge, the staging analysis not only simulated the time-dependent property of Young's Modulus, but also considered the creep effect in concrete.
11. For general bridge with constant girder spacing, due to creep effect, the old stage built in the early stage would deflect more and the displacement gap between stages would increase. However, correct cambers would alleviate creep effect.
12. Creep effect on concrete occurs at an early stage. Due to improper camber, excess loading due to superimposed dead load and live load in the early stage, and creep effect, the new deck can be expected to be higher than the existing deck. Based on the analysis, the MD 140 bridge (bridge no. 6032) is expected to have a two (2) to three (3) inch difference in elevation, which is also reported from the field and this differential displacement between stages could be alleviated by proper camber and scheduling on pouring.
13. To achieve better results during staged construction, a 30-day waiting period is recommended between finishing the new deck pour and starting the closure pour. In this way the creep effect from both the old and new construction stages would enter a steady growth stage and the displacement gap between these two stages would be narrowed.

1. Introduction

This study investigated the design criteria and practices in an effort to improve the quality of bridge designs in the State of Maryland and beyond. This first criterion investigated was the live load deflection for steel bridges. Since the live load deflection criteria in the AASHTO LRFD Bridge Design Specifications (2014) is optional, the Maryland State Highway Administration (SHA) establishes no maximum limit on deflection and leaves the burden on the designers to establish limits. This study developed a menu of criteria that designers could choose from in their bridge designs. The second design/construction criterion investigated was designing and detailing bridge deck closure pours. A closure pour is a small area of concrete bridge deck that connects two portions of a bridge deck placed in different stages of construction. For staged construction, the designer should consider the deflections of the bridge on either side of the closure pour to ensure proper transverse fitting.

In summary, the objectives of this study were to assess and set the criteria on the “optional” live load deflection evaluation specified in the AASHTO LRFD Bridge Design Specifications (2014) and provide recommendations on bridge deck closure pours practices.

In order to achieve these objectives, the following tasks were completed:

1) Previous and current practices and future planning on the serviceability of bridges were documented. This study looked at bridges within the short and median span range and selected 30 samples in the state of Maryland; all are steel girder bridges, where the highest live load deflection occurs. Steel bridges designed with the live load deflection limit were evaluated and summarized in this study.
2) The next step was to collect and study state-of-the-practice methods from federal and other state agencies. All available current state-of-the-practice methods from the Federal Highway Administration’s regulations, research and testing findings in the past and also the practices from other states were located, collected and listed for study. Three bridges, the I-270 over Middlebrook Road (bridge no. 1504200), Route 1 over Paint Branch (bridge no. 1600400), and I-95 over Patuxent River (bridge no. 1619701) were chosen for refined analyses to investigate the live load deflections. Field measurements for these three bridges were collected by the research team to facilitate this study.
3) Several finite element models with different software applications were developed for the entire bridge to compare the differences in deflection for the bridge model versus the simple single girder analysis traditionally performed by SHA. The finite element model can be used for the live load deflection analysis. Then the impacts resulting from different construction methods were studied.
4) A summary of all the work listed above is included. Recommendations associated with steel girder construction, which complemented the current Maryland practices on live load deflection limit and closure pours, are also included in this report.

The focus of this study was about serviceability-related issues of median and short span steel bridges, which mainly includes live load deflection. Chapter Two summarizes the literature and the exploration of serviceability issues for steel bridges. The development of bridge live load deflection limit or criteria in different states and countries were reviewed and discussed.

In Chapter Three, live load deflections of steel bridges were investigated. The methods of both Allowable Stress Design (ASD) from the AASHTO Standard Specifications for Highway Bridge (2002), and Load and Resistance Factor Design (LRFD) from the AASHTO LRFD Bridge Design Specifications (2014), were compared. The comparison includes the types of live load, which are mainly truck loads and lane loads; as well as some important factors such as multiple presence factor and dynamic load allowance (i.e. impact factor).

In Chapter Four, three representative bridge models were analyzed. Each bridge was analyzed with three different programs: MERLIN-DASH, DESCUS-I and CSiBridge. The MERLIN-DASH analyzes the bridge by a line-girder model; DESCUS-I conducts the analysis with two-dimensional grid method and the CSiBridge uses the three-dimensional (3D) finite element method. After an overall comparison of these three representative bridges, about thirty sample bridges from Maryland's Bridge Inventory were collected, modeled, analyzed and tabulated in Chapter Five. Several plots and charts are used to analyze the data and summarize results of the sample bridges.

In Chapter Six, for bridge deck closure pours of one sample bridge in Maryland, two-dimensional grid models created in DESCUS-I and three-dimensional finite element models created in CSiBridge were studied beyond the MERLIN_DASH used in the design stage. The creep effect of concrete was considered during the staging analysis. Finally, in Chapter Seven conclusions of the serviceability-related issue are summarized to complete the study.

2. Literature Review

2.1 Serviceability-related to Steel Girder Bridges under Live Load

2.1.1 Live Load Deflection Studies and Design Criteria

General design principles are detailed in Chapter 2 of the AASHTO LRFD Bridge Design Specifications (2014). Article 2.5.2.6.2 gives the criteria for deflection only as optional; it advises that the maximum deformation of a bridge should not exceed 1/800 of the span length for general vehicular bridges and 1/1000 of the span length for vehicular bridges with pedestrian traffic. The reason for the smaller allowable deflection for the pedestrian bridges is that pedestrians are more sensitive to bridge vibrations than vehicular drivers or passengers. In order to better understand the rationale behind the current AASHTO LRFD deflection limits, identifying how they were developed was of interest in this study The 1/800 span length (L) limit was initially mentioned in the first American Highway Bridge Design Specification in 1953 and has been incorporated in every revision thereafter. The ASCE Committee on Deflection Limitations of Bridges of the Structural Division (1958) reported on their examination of the live load deflection limits and depth-to-span ratio, D/L, which was shown in the 1953 American Association of State Highway Officials (AASHO) Standard Specifications for Highway Bridges. The earliest deflection limits were adopted by the Phoenix Bridge Company in 1871, which limited deflection to 1/1200 of the span length for a train moving at a velocity of 30 miles per hour. The American Railway Engineering Association (AREA) took depth-to-span ratios, which are an indirect method of limiting deflection in the early 1900s; however, the limits were without any basis at that time. The concept of depth-to-span ratios for highway bridges was originally raised in 1913 and adopted by AASHO in 1924.

Vibrations first became an important issue in the 1930s and the Bureau of Public Roads tried to provide a correlation between the vibration problems of bridges and bridge structural properties. They conducted a study that attempted to link the objectionable vibration felt on a sample of bridges built in that era. This study concluded that structures having unacceptable vibrations determined by subjective human response had deflections that exceeded L/800, and this conclusion resulted in the L/800 deflection design limit. Given how old these studies are, information regarding the specifics was not available. However, the bridges included in this early study had wood plank decks, and the superstructure samples were pony trusses, simple beams or pin-connected through-trusses. The Bureau did not incorporate composite girder bridges, which are more popular today. The ASCE Committee in 1958 reviewed the history of the bridge deflection criteria, completed a survey to obtain data on the behavior of bridges and the opinions of bridge designing experts, reviewed field measurements of bridges subjected to moving loads, and gathered information on human perception to vibration. The survey concluded: (1) maximum oscillations occur with passage of medium weight vehicles not heavy vehicles, (2) reports of objectionable vibrations came from continuous span bridges more often than simple span bridges, and (3) there was no defined level of vibration which constituted as being undesirable. Many factors would affect the vibration of the bridge. Some of them are listed here:

- Flexibility and natural frequency of bridge
- Flexibility and natural frequency of vehicle
- Relative weight of vehicles and bridge
- Vehicle speed
- Frequency of load application
- Motion caused by loads in adjacent spans of continuous span structures
- Damping characteristics of bridge and vehicle

The use of depth-to-span ratio, D/L, began in the early 1900s with the AREA stating that pony trusses and plate girders should have a depth no less than 1/10 of the span length. There has been little change with these ratios over the years. The current depth-to-span limits are 1/10 for simple span trusses and 1/30 for simple span rolled shapes and plate girders.

The early specifications for highway bridges used with some changes the depth-to-span ratios from the American Railway Engineering and Maintenance-of-Way Association (AREMA), which was abbreviated to AREA at that time. Table 2.1 shows the limiting D/L ratios that have been incorporated in previous AREA and AASHTO specifications (Taly 1998).

Table 2.1 Historic Depth-to-Span, D/L, Ratio for Highway Bridges

Year(s)	Trusses	Plate Girders	Rolled Beams
AREA			
1913,1924	1/10	1/10	1/12
1907,1911,1915	1/10	1/12	1/12
1919,1921,1950,1953	1/10	1/12	1/15
AASHO (later AASHTO)			
1913,1924	1/10	1/12	1/20
1931	1/10	1/15	1/20
1935,1941,1949,1953,2012	1/10	1/25	1/25

Both AREMA and AASHTO Specifications included statements that required flanges to be strengthened if section depths smaller than those required by the limiting depth-to-span ratio are used.

The use of depth-to-span ratios was primarily to limit deflections but was also driven by economics. The limiting values of depth-to-span ratios have decreased with time while allowable stresses have increased. This would result in shallower sections being used, which would result in larger deflections. This result confused the ASCE Committee on Deflection

Limitations of Bridges of the Structural Division, which was tasked with investigating the origins of the deflection and depth-to-span limits. Furthermore, the committee quoted the 1905 AREA Committee's explanation of their depth-to-span ratios:

> "We established the rule because we could not agree on any. Some of us in designing a girder that is very shallow in proportion to its length decrease the unit stress or increase section according to some rule which we guess at. We put it there so that a man would have a warrant for using whatever he pleased.''

A conclusion was reached in the 1905 report that the two criteria, deflection limit and depth-to-span ratio, are of different origin. The deflection limit is to limit undesired vibration while the depth-to-span ratio is a result of economics. Also, the committee could not provide any recommendations or methods for the best way to limit deflections or vibrations.

In the American practice, the deflection of bridges supporting vehicular traffic is generally limited to the span length divided by 800 (L/800) for simple and continuous spans, and L/300 for cantilever arms. For bridges intended to also carry pedestrian and bicycle traffic the AASHTO specifications have placed further limits. These deflection provisions are very simple to use, but not directly related to the real issue of concern about the vibration response under live load. The deflection and dynamic response both involve the stiffness of the bridge as well as some other parameters, such as the mass, damping and so on.

In the 1970s, Wright and Walker performed a study reviewing the rationality of the deflection limitation provisions and Roeder, et al. revisited the subject decades later in 2002 suggesting that the current AASHTO live load deflection limits L/800 for vehicular traffic bridges and L/1000 for pedestrian are not always sufficient in controlling excessive bridge vibration and should ultimately be removed. Fountain and Thunman conducted a study, which examined live load deflection criteria for steel bridges with concrete decks in 1987. They concluded that AASHTO's live load deflection criteria did not achieve the purported goal for strength, durability, safety, or maintenance of steel bridges. They questioned the AASHTO deflection criteria because the influencing 1930 Bureau of Public Roads study did not incorporate composite girder bridges. In 2007, Barker and Barth compared the procedure in AASHTO LRFD, which should have provided some uniformity in application, to the specific procedures used in several states. They found that there is wide variation in the deflection limit employed by the various states. Of the 47 states reporting deflection limits for bridges without pedestrian access:

- 1 state employs a L/1600 limit,
- 1 state uses a L/1100 limit,
- 5 states employ a L/1000 limit,
- 1 state expresses a preference for L/1000 but requires L/800 limit, and
- 39 states employ a L/800 limit.

Of the states reporting deflection limits for bridges with pedestrian access:

- 1 state employs a L/1600 limit,
- 2 state use a L/1200 limit,
- 1 state employs a L/1100 limit,
- 39 states use a L/1000 limit,
- 3 states employ a L/800 limit.

There is very wide variation in these deflection limits, since the largest deflection limit is twice as large as the smallest deflection limit. Two of the 47 states use the deflection limit as a recommendation rather than a design requirement.

Another problem is the live load that bridge designers use in order to obtain the live load deflections. The AASHTO Specification indicates that deflections due to live load plus impact are to be limited by the deflection limit. Within this context, there is ambiguity in the loads and load combinations that should be used for the deflection calculations, because design live loads are expressed as both individual truck loads and uniform lane loads. The survey showed that the loads used to compute these deflections have even greater variability than observed in the deflection limits.

- 1 state employs the HS-20 truck load only,
- 16 states use the HS-20 truck load plus impact,
- 1 state uses the HS-20 lane load plus impact,
- 1 state uses the HS-20 truck load plus lane load without impact,
- 7 states use the larger deflection caused by either the HS-20 truck load plus impact or the HS-20 lane load plus impact,
- 17 states use the HS-20 truck load plus lane load plus impact,
- 4 states consider deflections due to some form of military or special permit vehicle,
- 8 states use the HS-25 truck load.

It is not easy to compare the variability of the load and the variability of the deflection limit in different states because these are not mutually exclusive. Wisconsin DOT, for instance, uses the smallest deflection limit, but it also employs smaller loads than most other states. However, the relative importance of the lane load and design truck load are possible to be different for long and short span bridges, and so the L/1600 limit used in Wisconsin may be more restrictive for short span bridges. On the contrary, the Wisconsin limit may be a generous deflection limit for very long span bridges, because the truck load becomes relatively smaller with longer bridge spans despite the small deflection limit.

The actual methods used to calculate deflections are not defined in the AASHTO Specifications. Deflection limits are based upon deflections caused by service loads under actual service conditions in typical engineering practice. Load factors or other factors used to modify design loads are not normally used in these deflection calculations, and the actual expected stiffness of the whole structure is also needed for calculation. This is another reason that explains the variability in the application of the deflection limits. Load factors and lane load distribution factors are employed in some states while they are not adopted in others. Lane load distribution factors can significantly affect the magnitude of the loads used to calculate the deflections. The survey shows that 26 states use lane load distribution factors from the AASHTO Standard Specifications in calculating these deflections. Three states report that they use the LRFD lane load distribution factors. Thirteen states indicate that they effectively apply the loads uniformly to the traffic lanes by the AASHTO multiple presence lane load rules. They then compute the deflections of the bridge as a system without any modification for load factors, girder spacing or lane load distribution. These states effectively use an equal distribution of deflection principle. One state uses its own lane load distribution factor that is comparable to system deflection calculations. Several states indicate some flexibility in the calculation method, and a few states indicate a reluctance to permit the bridge deflection limit to control the design. The effect of the lane load distribution factor can be very significant. Depending on the spacing of bridge girders, the load used for bridge deflection calculations can be getting, at most, 100% larger than the load used for states where deflections are computed for the bridge as a system or where the loads are uniformly distributed to girders. Load factors may also be an issue of concern. Five states report that they apply load factors to the load used for the deflection calculation. These load factors also increase the loads used to compute bridge deflections, and they increase the variability in the application of the deflection limit between different states.

Since the 1930s, vibration becoming an important issue for bridge structure, the natural frequency of the bridge was attracting people's attention and becoming more and more of a concern. Some researchers state that bridge vibration is better controlled by a limit based on a dynamic property of the bridge, such as natural frequency rather than deflection limit criteria. Other studies show that the presence of excess vibrations is caused more by the natural frequency of the bridge, vehicle speed, and surface roughness than it is correlated to the deflection. Deflection limits not considering these factors are insufficient in preventing excess vibrations. Different methods and solutions were devised. Even fatigue was treated as a key factor that would generate deflection of bridges, thus the topic of fatigue load has been researched for many years. However, no uniform criterion has been established.

2.1.2 Codes and Specifications in Other Countries

2.1.2.1 Canadian Standards and Ontario Highway Bridge Code

Both the Canadian Standard and the Ontario Highway Bridge Design Code (OHBDC) use a relationship between natural frequency and maximum superstructure static deflection to evaluate the acceptability of a bridge design for the anticipated degree of pedestrian use. Figure 2.1 shows the plot of the first flexural frequency (Hz) versus static deflection (mm) at the edge of the bridge, which the natural frequency is calculated using following equations:

$$f_{obs} = 0.95 f_{cal} + 0.72 \qquad \textit{Eq 2.1}$$

$$f_{cal} = \frac{\pi}{2L^2}\sqrt{\frac{E_b I_b g}{w}} \qquad \textit{Eq 2.2}$$

where $\mathbf{f_{obs}}$ and $\mathbf{f_{cal}}$ are the observed frequency and calculated frequency, respectively. $\mathbf{E_b}$ is the modulus of elasticity of steel, $\mathbf{I_b}$ is the moment of inertial of the beam of cross-section, **g** is the acceleration due to gravity, and **w** is the weight per unit length of the beam. Consistent units must be employed for all variables. This equation was validated for structures with 2 Hz < $\mathbf{f_{cal}}$ < 7 Hz.

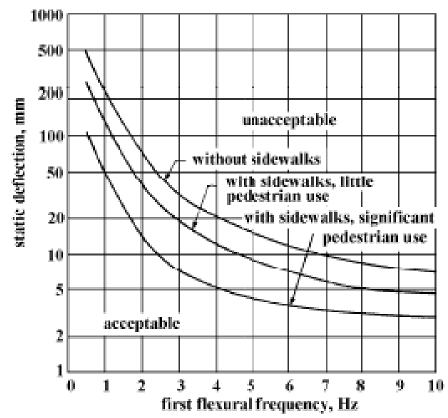

Figure 2.1 First Flexural Frequency versus Static Deflection (Ministry of Transportation, 1991)

More recent studies by Billings conducted over a wide range of bridge types and vehicle loads, loads ranging from 22.5 kip to 135 kips (100 KN to 600 KN), confirm the results of the initial study (Ontario Ministry of Transportation, 1991).

For both the Canadian Standards and the Ontario Code, only one truck is placed at the center of a single traveled lane and the lane load is not considered. The maximum deflection is computed due to factored highway live-load including the dynamic load allowance, and the gross moment of inertia of the cross-sectional area is used (i.e. for composite members, use the actual slab width). For slab-and-girder construction, deflection due to flexure is computed at the closest girder to the specified location if the girder is within 1.5m of that location.

2.1.2.2 Australian Codes

Australian Codes (AUSTROADS, 1992; AUSTRALIAN, 1996) require a similar curve, shown in Figure 2.3, to limit the static deflection as a function of the first mode flexural frequency for road bridges with footways. The serviceability design load of a single T44 Truck, including the same dynamic load allowance as that of OHBDC shown in Figure 2.2, should be positioned along the spans and within a lane to produce the maximum static deflection at a footway. Where the deflection of a road bridge without a public footway complies with the other limits specified in the codes, the vibration behavior of the bridge does not need to be specifically investigated. Where these deflection limits are exceeded, the vibration behavior of the bridge shall be assessed by a rational method, using acceptance criteria appropriate to the structure and its intended use.

Meanwhile, the deflection of highway bridge girders under live load plus dynamic load allowance shall not exceed 1/800 of the span length (AUSTRALIAN, 1996). However, the work (Sergeev and Pressley, 1999) showed that the origin of this live load deflection limit is uncertain. It was originally adopted in earlier versions of the Code, apparently taken from contemporary AASHTO Specifications. In this study, the live load deflection limits for three exiting bridges were investigated and alternative serviceability criteria were proposed. As a result of the combination of both the proposed design live loadings (A160, S1600 and M1600), which are heavier than the original design T44 truck, and the utilization of higher strength steels 50 ksi (350 Mpa and higher), composite bridges were found to be particularly vulnerable to the deflection limits. So, the validity of a live load deflection control criterion was questioned. The Lotus Street Duplication Bridge is a slab on steel I-girder bridge with spans of 153 + 117 feet and the actual L/D equals 23.7, less than the recommended value of 25 for composite girders. However, the live-load deflection limit was exceeded by 12% for 101t Double Bottom Road Train (DBRT) loading and by 45% for M1600 loading. The Mortlock River Bridge is a 6-span continuous composite steel bridge. The deflection under 101T DBRT loading controlled the design and resulted in low L/D ratio of 13.7. The deflection limit L/800 is exceeded under M1600 by 7%. Bridge 1470 is a simply supported composite steel bridge with a span length of 87.4 feet, the deflection under 101T DBRT loading also controlled this design and the M1600 live-load deflection is 44% greater than the L/800 limit. Thus, it is recommended that the Serviceability Limit State Criteria in the Australian Design Code should be optimized by eliminating the artificial live load and placing more emphasis on the elastic response of structures to serviceability loads, namely preventing rapid structure deterioration by controlling crack widths under short term loads and controlling vibration as appropriate to the situation.

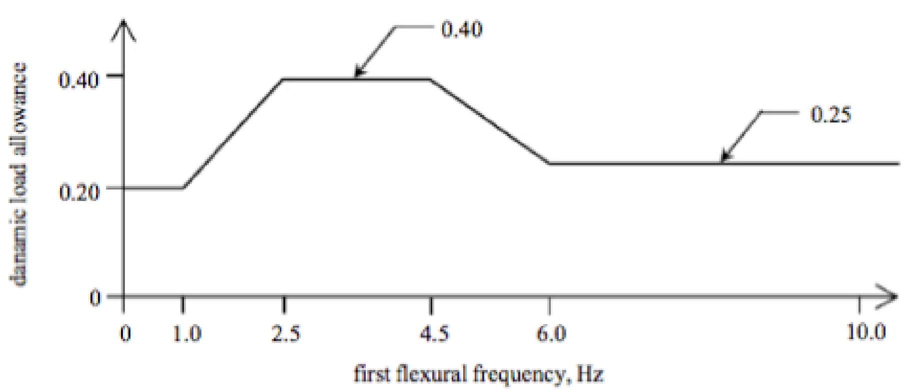

Figure 2.2 Dynamic Load Allowance
(Ministry of Transportation, 1991 and CSA International, 2000)

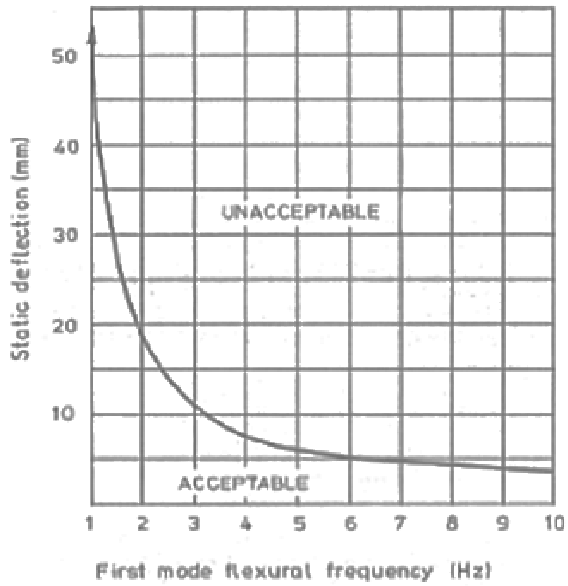

Figure 2.3 Deflection Limits for Vibration Controls of Australian Codes (AUSTROADS, 1992; AUSTRALIAN, 1996)

2.1.2.3 Codes and Specifications of Europe

A brief review of the codes and specifications used in European countries were also examined. Most European Common Market countries base their design specifications upon the Eurocodes (Dorka, 2001). The Eurocodes are only a framework for national standards. Each country must issue a "national application document (NAD)" which specifies the details of their procedures. A Eurocode becomes a design standard only in connection with the respective NAD. Thus, there is considerable variation in the design specifics from country to country in Europe. If an NAD exists for a specific Eurocode, then this design standard is enforced when it is applied to a building or bridge. Often, the old national standards are still valid and are applied. There is a rule though, that the designer cannot mix specifications. The designer must make an initial choice and then use this in all design documents for the structure. However, the full live loads are generally factored with a "vibration factor" to account for extra stresses due to vibrations in European bridge codes. No additional checks (frequency, displacements, etc.) are then required. For long span or slender pedestrian bridges, a frequency and mode shape analysis is usually performed. Special attention is always paid to cables, since vibrations are common, and some European bridges have problems with wind induced cable vibration. Deflection limits are not normally applied in the European bridge design.

2.2 Construction Closure Pours

2.2.1 Federal Highway Administration's (FHWA) Regulation

Staged construction is defined as building a parallel portion of a bridge at a different time in Federal Publication No. FHWA-NHI-08-048 (2007). Closure pours usually are employed when connecting two parts of a bridge in a closure bay. Some owners are unwilling to use closure pours due to unfortunate experiences with them. It is suggested to consider pouring sequence, shrinkage, and proper computation of cambers in order to get very compatible deflections. Another issue that should need attention is a case where some girders are composite when adjacent girders are being decked. When the wet concrete deck is placed on the non-composite steel, a disproportionate portion of this new load is drawn to the stiffer composite girders. To avoid this effect, the cross-frames/diaphragms connected to the two stages might be disconnected, and then a closure pour is used.

2.2.2 Other States' Practices

The California Department of transportation's (Caltrans) Bridge Deck Construction Manual (1991) suggests engineers should pay attention to the type of splice required for widening and closure pours. It claims the top deck must match two existing bridge decks. The Memo to Designers 9-3 Widening Existing Bridges (2010), published by Caltrans, recommends that when the dead load deflection of a bridge widening exceeds ¼ inch, a minimum closure width of 3 feet is utilized to complete the attachment to the existing structure. The memo states that, for precast or steel girders, "a closure pour defers final connection to the existing structure until the deflection from the deck slab weight has occurred; and it provides width to make a smooth transition between differences in final grade that result from design or construction imperfections." For cast-in-place construction the memo states, "good engineering practices dictates that the closure width should relate to the amount of dead load deflection that occurs after the closure is placed, and, closure depth should be kept to a minimum. A minimum closure width of three feet (3 ft.) is recommended." The memo further states that "it is advantageous to delay the placing of the closure pour to reduce the transfer of load to the existing structure, to improve the riding quality of the deck, to lower the stresses in the closure slab and to allow for shortening of prestressed girders."

Similarly, the Nevada Department of Transportation NDOT Structures Manual (2008) also emphasizes the two purposes of a closure: it defers final connection of the stages until after the deflection from deck slab weight has occurred, and it provides the width needed to make a smooth transition between differences in final grades that result from construction tolerances. A minimum closure width of three (3) feet is recommended. Greater closure widths may be required when larger relative dead-load deflections are anticipated. The required width can be estimated by considering the closure pour to be a fixed-fixed beam and by limiting the stresses in the concrete to the cracking stress. The manual also includes the following specifications:

- Stay-in-place forms shall not be used under the closure pour.

- Diaphragms/cross frames in the staging bay of structural steel girders shall not be rigidly connected until after the adjacent stages of the deck have been poured. Construct concrete diaphragms in the staging bay of prestressed concrete girders after adjacent portions of the bridge are complete. The diaphragms may be poured as part of the closure.
- Reinforcing steel between different stages shall not be tied or coupled until after the adjacent stages of the deck have been poured.
- Support the finishing machine on an overhang jack that is connected to the girder loaded by the deck pour. Do not place the finishing machine on a previously poured deck. The bridge designer must indicate in the contract documents that this method of constructing the closure pour is not allowed.

In 2010, Michael Sprinkel, Chris Blevins, Richard E. Weyers, and their group conducted a study that examined the failure and repair of a deck closure pour on Interstate 81in Virginia. They reported that several reinforced concrete decks on I-81 were replaced using the 3-ft wide center closure pour with epoxy coated reinforcement extending from each of the decks in 1992. However, three bridges were observed to be in a near failure or failure state after 17 years. The reason for the failure is mainly due to reinforcement corrosion in the vicinity of the leaking construction joints and transverse cracks. Therefore, this study recommended that expansive deck concrete should be used for closure pours to minimize or prevent the opening of closure pour construction joints and the formation of cracks due to shrinkage of the concrete. Also, placing closure pour construction joints over beams is suggested so that the closure pour is supported by the adjacent beams rather than the transverse reinforcement.

2.2.3 Research and Testing Findings

H. I Hung and Y. H. Chai (2011) recommended shortening closure pour waiting time for bridge construction. Current practice in California requires up to a 60-day waiting period for closure pour after the release of falsework for both staged construction and widening of existing bridges. The relatively long wait time is intended to reduce the stress build-up and mitigate the damage in the bridge deck due to the potential differential displacement between the newly constructed deck and the previously poured deck. However, the current waiting period does not take into account the displacement capacity of the closure slab, which varies depending on the dimensions and reinforcement details, as well as the time-dependent differential displacement that will be imposed on the closure slab. The numerical examples in their study indicate that the current waiting period is conservative, especially in staged construction.

The findings of NCHRP Synthesis 345 (2005) are based on survey questionnaires and interviews sent to steel bridge fabricators, steel bridge erectors and contractors from U. S. states and Canadian provinces. This study reported that problems develop in staged construction as the result of the difference in elevation between the deflected position of members after pouring and the non-deflected position of the members before pouring.

Furthermore, deck alignment, cross-frame connection and girders between two stages all require special considerations. For a successful stage construction, bridge owners developed several different strategies. The first one is using a closure or construction pour between stages. In Ohio, bridge owners use an 800 mm construction closure pour between stages to control delta deflection between stages. In Montana, they use a 600 mm closure pour to deal with the deflection between stages. In Tennessee, they require a closure pour for staged construction. Differential displacements across a staged construction field-cast connection can have a detrimental impact on the performance of the connection. Ben Graybeal (2012) investigated the bond of #4 (#13M) reinforcing bars and found that differential displacements equal to or greater than 0.05 inches (1.27 mm) caused a reduction in the bond strength. Not surprisingly, differential displacements seemed to ream a hole in the embedment material around the reinforcing bar, thus reducing the bond capacity.

Bridge Superstructure Design-MM No. 10 (2001) from the Iowa Department of Transportation placed guidelines when considering closure pours for bridge decks with longitudinal construction joints:

a. If there is more than 2 inches (50 mm) of dead load deflection in the bridge deck, then closure pours should be used.
b. If the staged construction is on a highway system with a high volume of truck traffic (approximately 500 or more trucks per day), then a closure pour should be considered. This will be addressed on a case by case basis.

The closure pour should be wide enough to allow for splicing of the transverse reinforcing steel along with 2 inches (50 mm) of clearance for the end of the bars from the construction joint. The minimum closure pour width should be three feet (900 mm).
Closure pours should be placed in areas with constant cross-slope in the bridge deck. In addition, closure pours over beams should be avoided.

Reasons for closure pours are:

a. For large deflections it may be difficult for the contractor to match up the elevations of the construction joints without a closure pour. Also, it is difficult to tie the reinforcing steel due to the difference in elevations and possible interference with new beam lines.
b. For areas with high truck traffic there can be problems with vibrations due to traffic that could cause poor bonding of the concrete to the reinforcing steel adjacent to the construction joint.

When closure pours are used, follow these guidelines for the different types of bridges:

For Concrete Slab Bridges -
Closure pours are typically not used for continuous concrete slab bridges. This is because the falsework is required to remain under the stage I construction until after the stage II construction has been completed and the falsework is ready for removal. Removing the

falsework at the same time allows the slabs from both stages to deflect under dead load together. This prevents moments from developing in the construction joint due to the slabs deflecting at different times.

For Prestressed Concrete Beam Bridges -

a. For prestressed concrete beam bridges with intermediate concrete diaphragms, the diaphragm shall not be placed in the bay where the closure pour is to be placed.
b. For prestressed concrete beam bridges with steel intermediate diaphragms, the diaphragm bolts used in connecting the channel to the bent plate shall remain loose until the second stage has been poured, then tightened before the closure pour.
c. The abutment and pier diaphragms should be staged with the deck pours and be in place before the closure pour.

For Steel Girder Bridges -

The bracing in the bay that contains the closure pour is to be installed after the second stage has been poured and prior to placing the closure pour. The bolt holes shall be field drilled in the cross bracing members to provide allowances for “fit up” of the diaphragm. For integral abutments, the same procedure as described for prestressed beams shall be used.

3. Live Load Deflection Validation of Steel Bridges

In the first part of this chapter, live load deflection of steel bridges is studied. Live loads are assumed to consist of gravity loads (vehicular live loads, rail transit loads and pedestrian loads), the dynamic load allowance, centrifugal forces, braking forces and vehicular collision forces; they are considered to be transient loads that are assumed to be applied to the short-term composite section. However, the live loads of interest in this study are only the design truck and lane loads since the primary live loads on bridge spans are due to traffic. The methods of both Allowable Stress Design (ASD) from the AASHTO Standard Specifications for Highway Bridge (2002) and Load and Resistance Factor Design (LRFD) from the AASTHO LRFD Bridge Design Specification (2014) are compared. The comparison includes the types of live load as well as important factors such as multiple presence factor and dynamic load allowance (i.e. impact factor). In the second part, three different programs, MERLIN-DASH, DESCUS-I and CSiBridge, are briefly introduced including analysis principle and procedure. The MERLIN-DASH program analyzes the bridge by a line-girder bridge model and the DESCUS-I program conducts the analysis with two-dimensional grid method. Both are used as design tools by SHA. The CSiBridge uses the three-dimensional finite element method. Three bridges from SHA's bridge inventory were selected to be representative bridges. The full investigation and analyses including the field measured data as well as the model analysis of these three bridges are shown in the next chapter.

3.1 Loading considered in the Load and Resistance Factor Design (LRFD) and the Allowable Stress Design (ASD)

3.1.1 Loading considered in the Allowable Stress Design

In the AASHTO Standard Specifications for Highway Bridge (2002), the Allowable Stress Design (ASD) method has been adopted. The live loads in highways are defined as standard truck and lane loads in ASD.

For the standard truck load, there are four standard classes of highway loading, which are H-20, H-15, HS-20 and HS-15. Loading H-15 of two axles is 75 percent of loading H-20 as well as HS-15 of three axles is 75 percent of loading HS-20. The HS-20 design truck, which is currently used in most states for live load analysis and design, indicates a vehicle with a front tractor axle weighing 8 kips (two sets of wheels, 4 kips on the left and 4 kips on the right), a rear tractor axle weighing 32 kips, and a semitrailer axle weighing 32 kips. The distance between the front wheels and the rear tractor wheels is 14 feet and the distance from this axle to the semitrailer axle is varying from 14 to 30 feet as shown in Figure 3.1.

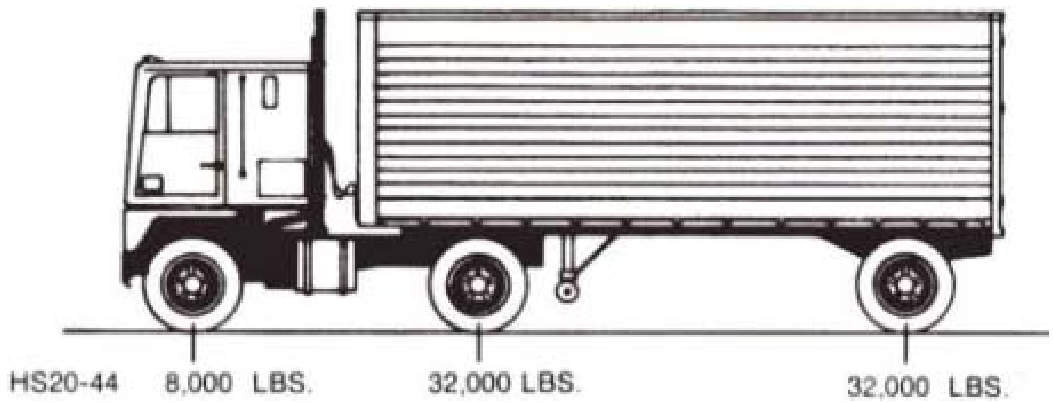

Figure 3.1 HS-20 Design Truck

In addition to the standard truck loadings, the AASHTO specifications also allow the representation of the truck as a single concentrated load and a uniform load. For the HS-20 truck loading, the concentrated load is defined as 18 kips for the moment, and 26 kips for the shear; the uniform load is 0.64 kips per linear foot as shown in Figure 3.2. In this chapter, 18 kips is used because the deflection is related to the moment. Both the concentrated load and uniform load should be considered as uniformly distributed over a 10-foot width on a line normal to the centerline of the lane from the AASHTO specifications.

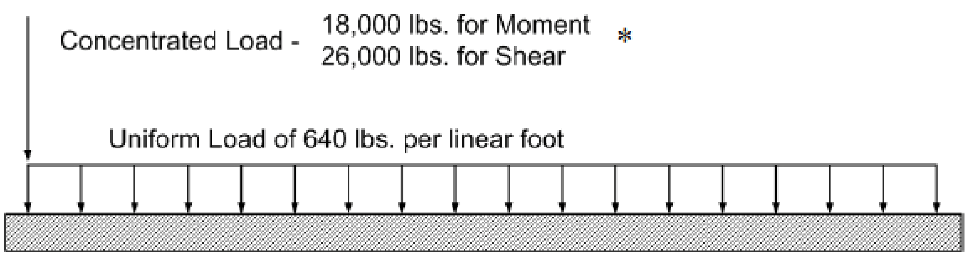

Figure 3.2 HS-20 Lane Load and Concentrated Load

In Maryland, the new design truck loading called 'HS-25' was adopted due to some concern that the HS-20 truck load did not adequately reflect actual conditions. The HS-25 truck load is 25 percent higher than the HS-20 truck load resulting in 10 kips in the front and 40 kips in the back instead of 8 and 32 kips. Also, the concentrated load for moment and shear and the uniform lane load are higher by 25 percent making them 22.5 kips, 32.5 kips and 0.8 kips per linear foot, respectively. The live load deflection result from the worst case scenario of the both truck loading and lane load plus concentrated load would be used during the deflection analysis.

For the live load deflection analysis with ASD method, several factors are used in the calculation of live load deflection. The live loads should be increased for steel bridges to allow for dynamic, vibratory and impact effects. The impact allowance should be determined by the following formula:

$$I = \frac{50}{L + 125} \qquad Eq\ 3.1$$

in which,

I = impact fraction (maximum 30 percent);

L = length in feet of the portion of the span that is loaded to produce the maximum stress in the member.

Another important factor, the multiple-presence-factor, should be used for reduction in load intensity. Based on the AASHTO Standard Specifications (2002), the multiple presence factors are shown in Table 3.1.

Table 3.1 Multiple Presence Factors in ASD Method

Number of lane(s)	Multiple presence factors
One or two lanes	1.00
Three lanes	0.90
Four lanes or more	0.75

3.1.2 Loading considered in the Load and Resistance Factor Design

AASHTO has developed and published the "LRFD Bridge Design Specifications" (the latest being 2014). The FHWA has endorsed the new LRFD method and mandated its adoption for new bridge designs after 2007. The LRFD Specifications use the AASHTO HL-93 truck loading as the design truck loading. The HL-93 designation consists of a "design truck plus design lane load" or "design tandem plus design lane load," whichever produces the worst case. A "HL-93 design truck" is identical to the HS-20 load configurations shown in Figure 3.1. The "design tandem" consists of a pair of 25 kips axles spaced 4.0 feet apart and the transverse spacing of wheels should be taken as 6.0 feet. The design lane load is 0.64 kips per linear foot uniformly distributed in the longitudinal direction of the bridges. The LRFD method used in this study takes the larger deflection resulting from (1) a design truck loading alone and (2) the design lane load plus 25 percent of the design truck load.

The dynamic allowance would be different in this design method from the one in ASD method. The dynamic load allowance in LRFD is a constant of 33 percent only applied to the truck loading. The multiple presence factors are also slightly different from ASD, which are shown in Table 3.2.

Table 3.2 Multiple Presence Factors in the LRFD Method

Number of loaded lanes	Multiple presence factors
1	1.20
2	1.00
3	0.85
>3	0.65

3.1.3 Summary of Loading in Two Methods

A summary of these two methods mentioned previously is shown in Table 3.3.

Table 3.3 Loads and Factors considered in ASD and LRFD Methods

	ASD	LRFD
Loading	larger of $\begin{cases} HS\,25 \text{ truck loading alone} \\ \text{Concentrated Load} + \text{Lane} \end{cases}$	larger of $\begin{cases} HS\,20 \text{ truck loading alone} \\ 25\% \text{ of } HS\,20 \text{ truck} + \text{Lane} \end{cases}$
Dynamic Allowance (Truck only)	$I = \dfrac{50}{L + 125}$	$I = 33\%$ (truck only)
Multiple Presence Factors	$\begin{cases} 1.20 \text{ for one lane} \\ 1.00 \text{ for two lanes} \\ 0.85 \text{ for three lanes} \\ 0.65 \text{ for more lanes} \end{cases}$	$\begin{cases} 1.00 \text{ for one or two lanes} \\ 0.90 \text{ for three lanes} \\ 0.75 \text{ for more lanes} \end{cases}$

3.2 Introduction of Computer Programs used in This Study

The programs used in this study are MERLIN-DASH, DESCUS-I and CSiBridge.

MERLIN-DASH

Design, **A**nalysis and Rating of **S**traig**H**t Girder Bridge Systems (MERLIN-DASH), was developed for bridge design engineers who function in a software production environment. In order to provide a program which would be applicable nationally, MERLIN-DASH was developed to offer the wide range of features and options necessary to meet the demands of universal usage in the analysis, design, and rating of steel and reinforced concrete bridges.

MERLIN-DASH incorporates a flexible sequence of operations initiated with analysis and proceeding, at the user's option, to perform any or all combinations of analysis, design, code check, rating and staging for the AASHTO WSD (or called ASD), LFD or LRFD methods. Generality also extends into the structural model incorporated within MERLIN-DASH. The structural analysis is performed via a series of modular subroutines based on the stiffness method. An extensive mesh generation capability on a line girder allows for the incorporation of fully automated AASHTO Dead Load (DL) and Live Load (LL) sequences. Incorporated within the MERLIN-DASH system are the most general and the widest ranges of capabilities.

In this portion of the study, the rating functions of AASHTO WSD (or called ASD) and LRFD methods have been used for the three representative bridges and the rest of the sample bridges as well. In MERLIN-DASH, the girders or beams of one bridge model are all assigned as the same girder or beam as the interior one in the real bridge. The shoulders of the bridge models are treated as curb to which no design load is assigned. Bracing between girders or beams is not considered in the analysis of this software. The analysis was based on the influence line since this is only in two dimensions. The HS-25 design truck and HL-93 design truck are used as design load for ASD and LRFD methods, respectively.

DESCUS-I

The computer program Design and Analysis of Curved I-Girder Bridge Systems (DESCUS-I), performs the complete analysis and partial design of a straight or horizontally curved bridge composed of flanged steel sections which act either compositely or non-compositely with a concrete deck. The program can be run using either WSD (or called ASD), LFD, or LRFD method. The bridge may be of arbitrary plan configuration and can be continuous and skewed over supports. The girders may have high degree of curvature, may be nonconcentric, bifurcated, and may contain hinges.

The program models the bridge structure as a two-dimensional grid in a stiffness format with three degrees-of-freedom at each nodal point (corresponding torsion, shear, and bending moment). All nodal locations, member connectivity, and section properties are generated internally from basic input. All dead load (DL) computations are performed automatically within the program to satisfy the construction conditions specified by AASHTO. Additional constant dead load is allowed per girder as a special program input option. All live load (LL) computations also are performed automatically, where the AASHTO truck and lane loadings are applied to an influence surface previously generated for the entire bridge. Impact (I) effects also are included per AASHTO recommendations. Up to nine arbitrary trucks can also be specified and analyzed concurrently.

Output contains the positive and negative maximum moments, shears, and torsion along with the corresponding primary and warping stresses for each girder and beam or truss diaphragm element. These maxima are given along with all AASHTO group combinations for DL + LL + I. Also, outputs are deformations along each girder for DL and maximum LL + I, along with the allowable deflections recommended by AASHTO. Finally, various tables are output which yield information on the design of the sections including maximum stresses, allowable stresses, shear ranges, shear connector, and stiffener spacing.

In DESCUS-I the analysis is based on the influence surface instead of the influence line in MERLIN-DASH. Therefore the results from the model built in this program are expected to be more accurate than those in MERLIN-DASH, since the transverse action would affect the whole bridge behavior.

CSiBridge

CSiBridge is a new software for bridge analyses and it provides a way to model the whole bridge in three-dimensions instead of 2-dimensions as in MERLIN-DASH and DESCUS-I. CSiBridge is a module separated from the previous version of SAP2000. Modeling, analysis and design of bridge structures have been integrated into CSiBridge to create the ultimate in computerized engineering tools. Using CSiBridge, engineers can easily define complex bridge geometries, boundary conditions and load cases. The bridge models are defined parametrically, using terms that are familiar to bridge engineers such as layout lines, spans, bearings, abutments, bents, hinges, and post-tensioning. The software creates spine, shell or solid object models that update automatically as the bridge definition parameters are changed. CSiBridge design allows for quick and easy design and retrofitting of steel and concrete bridges. The parametric modeler allows the user to build simple or complex bridge models and to make changes efficiently while maintaining total control over the design process. Lanes and vehicles can be defined quickly and include width effects. Simple and practical charts are available to simulate modeling of construction sequences and scheduling. In addition, AASHTO LRFD design is included so that the user could easily obtain the results that are needed. In this study, four load cases, which are the HS-25 truck and lane load plus concentrated load for the ASD method, as well as the HS-20 truck along and 25 percent of the HS-20 truck load plus lane load for the LRFD method, were applied for a certain bridge. The worst situation in each method was found and analyzed. The analysis type used in this study is ‘moving load’ in CSiBridge since it should be consistent with MERLIN-DASH and DESCUS-I.

Among these three programs, CSiBridge should be able to obtain the most precise results while the MERLIN-DASH is the most convenient program for users to use to model the bridge and do analysis. It is impossible to model all the sample bridges with the accurate but complicated finite element method. Therefore, a few representative bridges need to be selected and analyzed with different programs to examine whether the results from MERLIN-DASH are acceptable.

3.3 Three Representative Maryland Bridges

In order to validate the analysis procedure used in this study, several representative bridges were selected for full investigation. Through the Maryland State Bridge Inventory the most popular design truck load was determined to be the HS-20 truck. Also, in most cases, the number of the spans is one, two, or three and the individual span length is 30 feet to 200 feet.

Based on these considerations, the three bridges selected were I-270 over Middlebrook Road (bridge no. 1504200), Route 1 over Paint Branch (bridge no. 1600400), and the I-95 over Patuxent River (bridge no. 1619701) bridges. The I-270 over Middlebrook Road bridge is a single span straight girder bridge with the span length 140 feet. The Route 1 over Paint Branch bridge is a two-span skewed bridge with two equal 80 foot spans. This bridge is just in front of the University of Maryland at College Park hence it is easy to measure. The I-95 over Patuxent River bridge is located on one of the most important highways on the East Coast of the United States and is a three-span bridge with spans lengths of 165 feet for the two side spans and 180 feet for the center span. The heavy traffic on this bridge also makes it to be worth investigating. All three bridges were designed with the HS-20 design truck load. Field measured deflection data and model analyses, including the two-dimensional model and the three-dimensional finite element model, are presented and comprehensively discussed in the next chapter.

4. Refined Analysis of Three Representative Maryland Bridges

In this chapter, all the three representative bridges were analyzed using MERLIN-DASH, DESCUS-I and CSI Bridge computer programs. In the Allowable Stress Design method, the HS-25 design truck was adopted for the live load deflection although the bridges are designed with the HS-20 loading, because Maryland used HS-25 loading instead of HS-20 loading from the 1990s until 2007 when LRFD was adopted. The HL-93 design truck was used for deflection analysis in the Load and Resistance Factor Design method.

4.1 Model Analysis of the I-270 over Middlebrook Road Bridge

The bridge located on I-270 over Middlebrook Road (bridge no. 1504200) is a single span bridge whose span length is 140 feet. This bridge was built using the HS-20 design load in 1991. There are four lanes in the North Bound Route (N.B.R. to Frederick) and three lanes in the South Bound Route (S.B.R. to Washington). The S.B.R. bridge was modeled and analyzed in this study, therefore the multiple-presence-factor in both the ASD and LRFD methods are 0.85 and 0.9, respectively, according to the summary chart in Section 3.1.3. The typical section of the S.B.R. is shown in Figure 4.1. The roadway width is 57 feet 6.5 inches, including the two shoulders with 10 feet width on the right and 11 feet and 6.5 inches width on the left. There are also two barriers, which are 2 feet wide on the right and 1 foot 5.5 inches on the left. There are eight identical beams beneath the 8.5 inches deep concrete deck slab with the spacing of 7 feet and 11 inches and the two overhangs, 2 feet 3 inches on the left and 3 feet 6 inches on the right, from the very outside of the cross section of the bridge. Two different plate girder sections are used in this bridge; the dimensions and the arrangement of the plate girders could be found in Figure 4.2 ‘girder elevation’. Section 1 is assigned for a 35-foot-long distance from both ends of the bridge and section 2 is for the middle 70 feet. Also, the impact factors for the ASD method were calculated as 18.9% based on equation 3.1 and a constant 33% for the LRFD method. The lateral bracing spacing is 23.33 feet since there are 7 bracings equally spaced between the two ends of the bridge. The steel in this bridge has a yield stress of 50 ksi and the reinforcing steel is Grade 60.

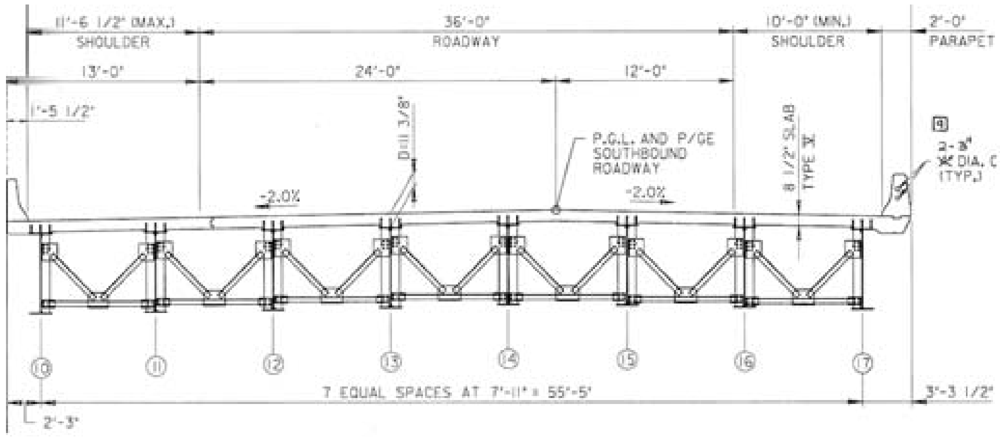

Figure 4.1 Typical Cross Section of the I-270 over Middlebrook Road Bridge S.B.R

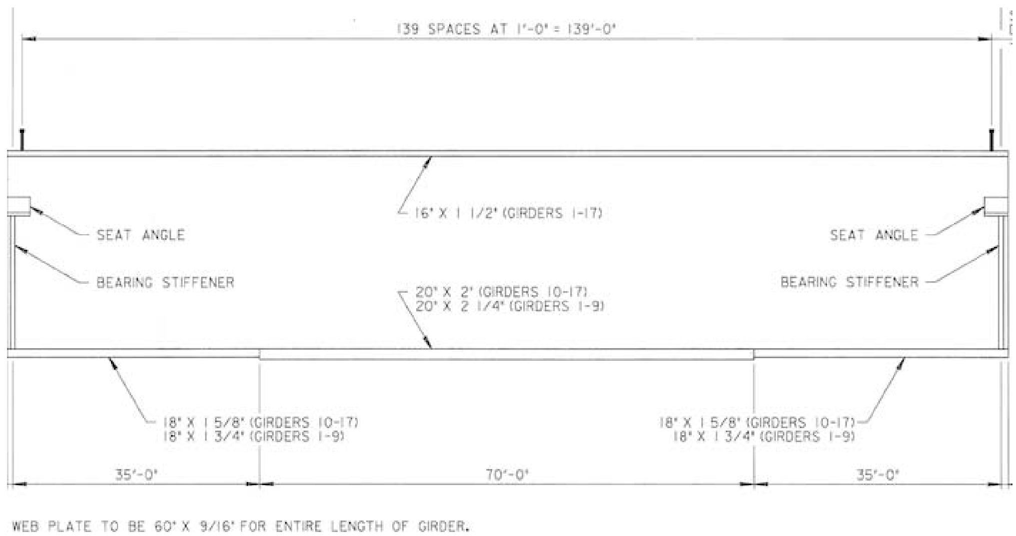

Figure 4.2 Girder Elevations

For the I-270 over Middlebrook Road (bridge no. 1504200) bridge analysis in MERLIN-DASH, each beam is assigned as the plate girder sections detailed in Table 4.1 below. Section 1 is assigned from 0 to 35 feet and 105 to 140 feet, respectively, and section 2 is from 35 to 105 feet, which is in the middle 70 feet of the bridge. The bridge roadway is adjusted to 36 feet by removing the shoulders, and the edge of slab to curb is input as 12.5 feet by averaging the remaining cross section of the bridge once the 36 feet of driving roadway width is removed.

Table 4.1 Beam Sections

	P.G. Web		P.G. Flange			
Unit: inch	Depth	Thick	Top Width	Top Thick	Bottom Width	Bottom Thick
Section1	60	0.5625	16	1.5	18	1.625
Section2	60	0.5625	16	1.5	20	2

The haunch has been defined as 2.875 inches in depth and 16 inches in width. The detail factor for beam has been estimated as 1.05. The span length is 140 feet. The beam spacing and the arrangement of beam sections are easy to follow with the information mentioned above. The slab has the depth of 8.5 inches and the intensity of the dead load per beam was calculated by multiplying the depth of the deck slab with the tributary width, which is the spacing between two adjacent beams, also multiplied by the density of concrete. In this case, it is 0.841 kips/ft.

The arbitrary uniform load per beam in this program consists of dead load including ‘haunch’ and ‘stay-in-place’, and superimposed dead load that usually are ‘future wearing surface’ and ‘barrier’. In the I-270 over Middlebrook Road bridge (bridge no. 1504200), they were determined to be 0.048 kips/ft., 0.022 kips/ft., 0.216 kips/ft. and 0.152 kips/ft., respectively. Diaphragms and stiffeners are neglected in this study since only the moment strength was taken into consideration. Shear is not a concern here. The model built in MERLIN-DASH is shown in Figure 4.3 below.

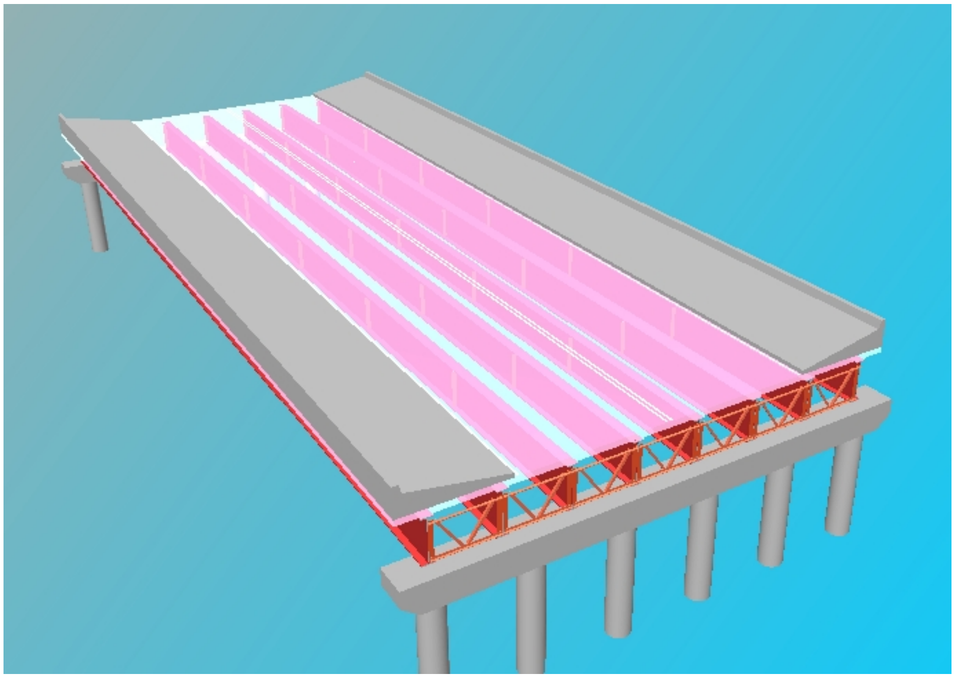

Figure 4.3 I-270 over Middlebrook Road Bridge Model in MERLIN-DASH

The rating function of MERLIN-DASH has been used for both the ASD and LRFD methods. In ASD, the design truck is selected as HS-25 and in LRFD, it is HL-93. The results obtained for live load deflections for this bridge from DASH are 0.821 inches for ASD HS-25 loading with lane load governing and 0.651 inches for LRFD HL-93 loading with lane plus 25% truck governing.

The model of the I-270 over Middlebrook Road bridge (bridge no. 1504200) in DESCUS-I could be more accurate than the model in MERLIN-DASH because the real bridge has a

14-degree skew angle that can only be treated as a straight girder bridge in MERLIN-DASH due to the limitation of the line-girder program. In DESCUS-I, this skew issue can be solved. A V-bracing diaphragm is applied for the bridge and the graphic generated from DESCUS-I is shown in Figure 4.4. Also, the distribution factors can be automatically calculated based on the cross section information pre-input into the program. In the ASD method the design load is still the HS-25 truck and in LRFD it remains the HL-93 truck.

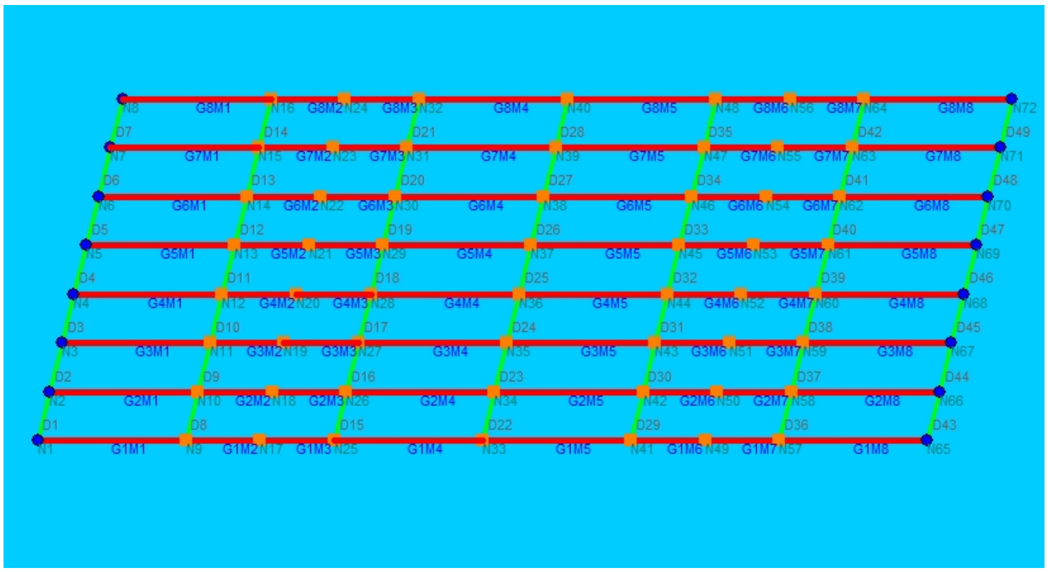

Figure 4.4 DESCUS-I Graphic of the I-270 over Middlebrook Road Bridge

The live load deflections for both design methods were calculated as 1.151 inches with lane load governing in ASD and 1.017 inches with HL-93 governing in LRFD.

In CSiBridge, a three-dimensional finite element model (Figure 4.5) was built and moving load analysis was used for both the ASD and LRFD methods.

The load factors mentioned previously have been manually applied in for both design methods. For the ASD method, the lane load governs and the live load deflection under the HS-25 design truck load is 1.107 inches; for the LRFD method, the truck load alone governs and the live load deflection is 0.984 inches.

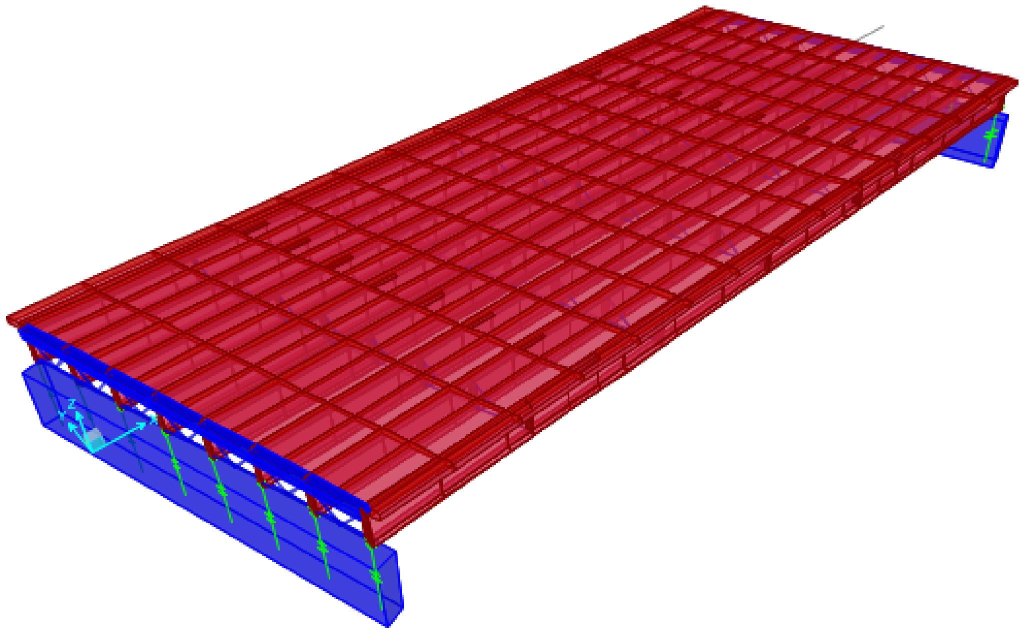

Figure 4.5 I-270 over Middlebrook Road Bridge Model in CSiBridge

4.2 Model Analysis of the Route 1 over Paint Branch Bridge

The Route 1 over Paint Branch bridge (bridge no. 1600400) has two 80-foot-long spans, which makes the impact factor 24.4% (I=50/(80+125)=0.244) for both spans in the ASD method. In the LRFD method, this factor is 33%. The number of lanes along the Route 1 over Paint Branch bridge is also three; hence, the multiple presence factors are the same values as the I-270 over Middlebrook Road bridge (bridge no. 1504200) in both the ASD and LRFD methods, which are 0.85 and 0.9, respectively. The typical cross section shown in Figure 4.6 provides information about this bridge. The roadway for three lanes is 33 feet and the edge of slab to curb is 6 feet. There are six beams with a 7.04-foot distance from each other and the overhang is 3.79 feet. The slab depth is 8.5 inches including 0.5 inches integral wearing surface. The haunch is 2.35 inches deep and 20 inches wide according to the elevation view as shown in Figure 4.7. This figure also shows that two wide flange beams W36x135 and W36x245 were adopted in this bridge. W36x245 is in the middle 50 feet portion of the girder and W36x135 for the remainder of the girder. The structural steel has the yield stress of 50 ksi and the reinforcing steel is Grade 40. The distance between adjacent lateral bracing is 20 feet.

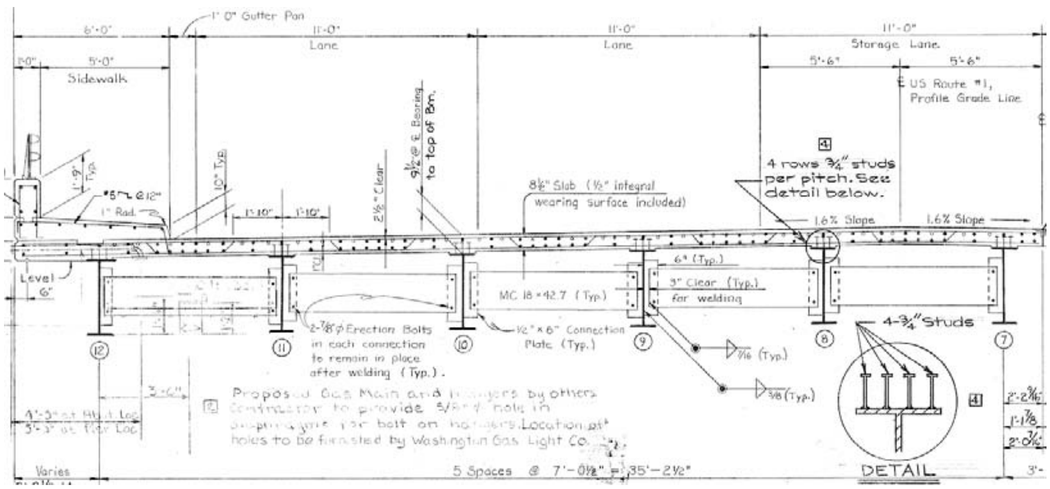

Figure 4.6 Typical Cross Section of the Route 1 over Paint Branch Bridge

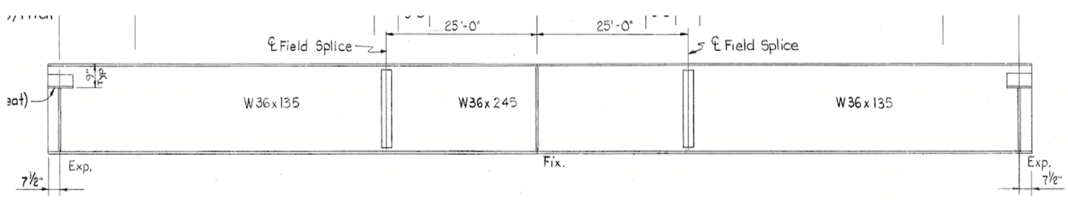

Figure 4.7 Elevation

The Route 1 over Paint Branch bridge (bridge no. 1600400) is a skewed bridge. In MERLIN-DASH, however, this bridge is built to be straight due to line girder limitation. Figure 4.6 only shows half of the bridge roadway so that it is assumed that there is an extra length on the right beyond the overhang of 3.79 feet, same as the left overhang, and 4.15 feet as the sidewalk in average. The dead load information is shown in Table 4.2. The model view of MERLIN-DASH is shown in Figure 4.8.

Table 4.2 Dead Load Information of Route1 Bridge

Slab Loads Per Beam		
Deck Slab		0.748 kips/ft.
Arbitrary Uniform Loads Per Beam		
Dead Load	Haunch	0.021 kips/ft.
	Stay in place	0.049 kips/ft.
Superimposed Dead Load	Future wearing surface	0.176 kips/ft.
	Barriers	0.324 kips/ft.

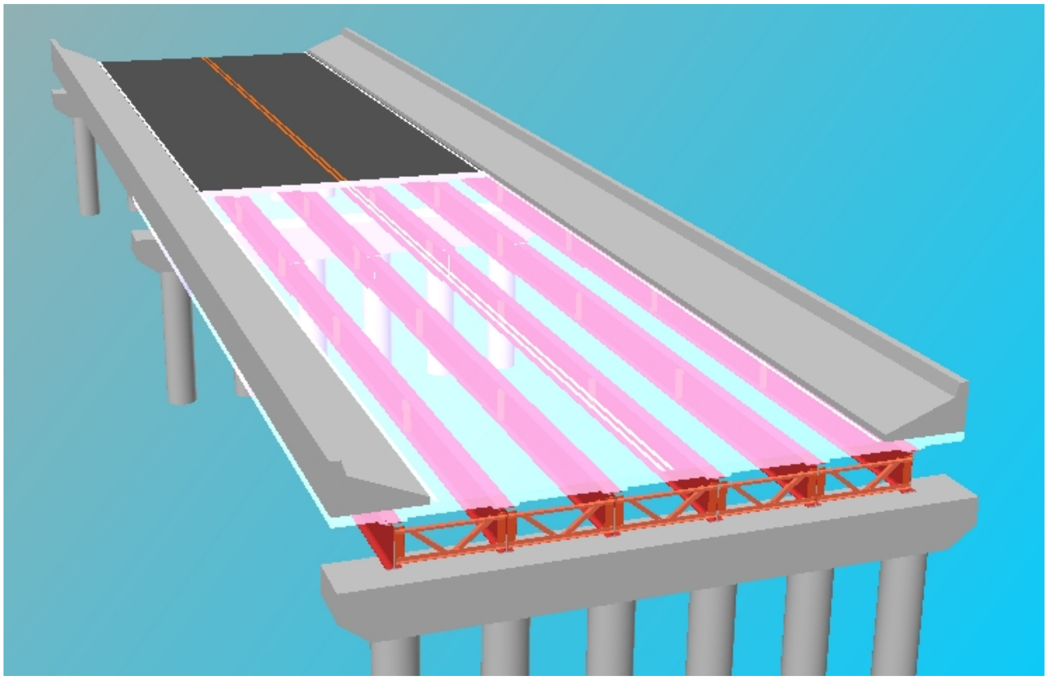

Figure 4.8 Route 1 over Paint Branch Bridge Model in MERLIN-DASH

The live load deflections for this bridge are calculated as 0.85 inches for both spans within the ASD method and 0.687 inches for both spans within the LRFD method. The lane load governs in ASD and HL-93 truck load governs in LRFD.

In DESCUS-I, the 30-degree skew angle is considered. The lateral bracing between two adjacent girders used a steel channel. Analyses were performed for both the ASD and LRFD design options and the live load deflection results were 1.079 inches for the ASD method with the HS-25 truck load governing and 0.918 inches for the LRFD method with HL-93 truck load governing. The bridge framing plan from DESCUS-I is shown in Figure 4.9.

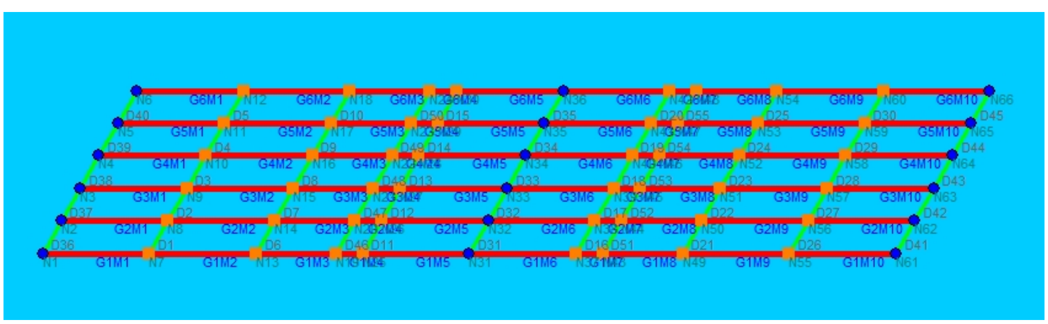

Figure 4.9 DESCUS-I Graphic of the Route 1 over Paint Branch Bridge

In CSiBridge, this bridge is modeled with shell elements as shown in Figure 4.10. For the ASD method, the HS-25 truck load governs and the live load deflections are 1.016 and 0.973 inches for the two respective spans; for the LRFD method, the truck load alone governs and the live load deflections are 0.821 and 0.786 inches for two respective spans.

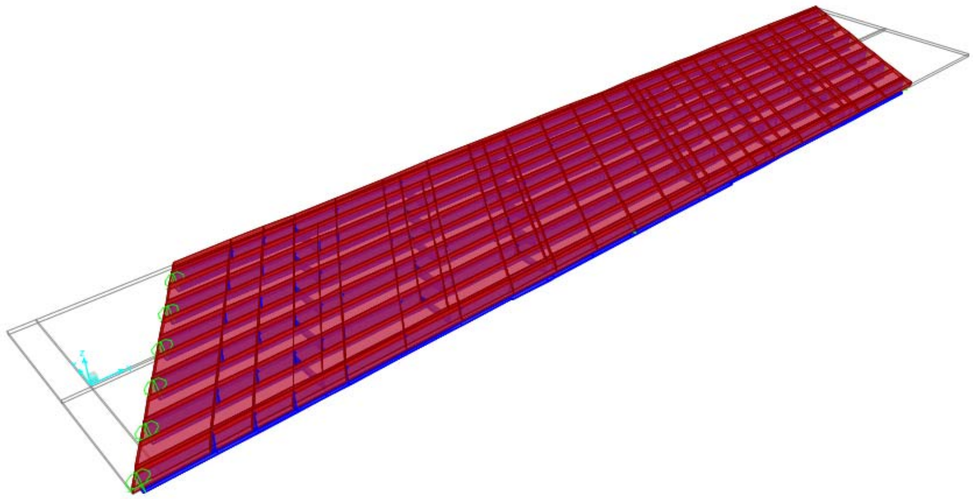

Figure 4.10 Route 1 over Paint Branch Bridge Model in CSiBridge

4.3 Model Analysis of I-95 over Patuxent River Bridge

The I-95 over Patuxent River bridge (bridge no. 1619701) is a three-span steel bridge, which has span lengths of 165 feet, 180 feet and 165 feet, respectively. Therefore, the impact factors in the ASD method are 17.2% (I=50/(165+125)=0.172) for end spans and 16.4% (I=50/(180+125)=0.164) for the middle span. The impact factor is still 33%, same as the previous two bridges in the LRFD method by following the AASHTO LRFD Bridge Design Specifications (2014). There are four lanes on this bridge, so the multiple presence factors are 0.65 in ASD and 0.75 in LRFD.

The I-95 over Patuxent River bridge (bridge no. 1619701) has two vehicle travel directions N.B.R. and S.B.R., whose cross sections are the same but in the opposite directions. A typical cross-section at the mid-span of N.B.R. shown in Figure 4.11 was taken for analysis. The four-lane roadway width is 48 feet and the edge of slab to curb is 11.5 feet including a 10 feet wide shoulder. There are eight girders under 8.5 inches of deep concrete slab in this bridge; the two exterior girders are slightly different from the six interior girders. The girders are at the same spacing of 9 feet 2 inches and the overhang is 3 feet 5 inches. Figure 4.12 only shows half of the girder elevation; the other half is symmetric with this half. In the area of the pier, the web depth of the girder varies in a parabolic shape; therefore, more work needs to be done during modeling of the bridge in each program analysis. The I-95 over Patuxent River bridge was built in the 1970s; hence, the structural steel has the yield stress of 36 ksi.

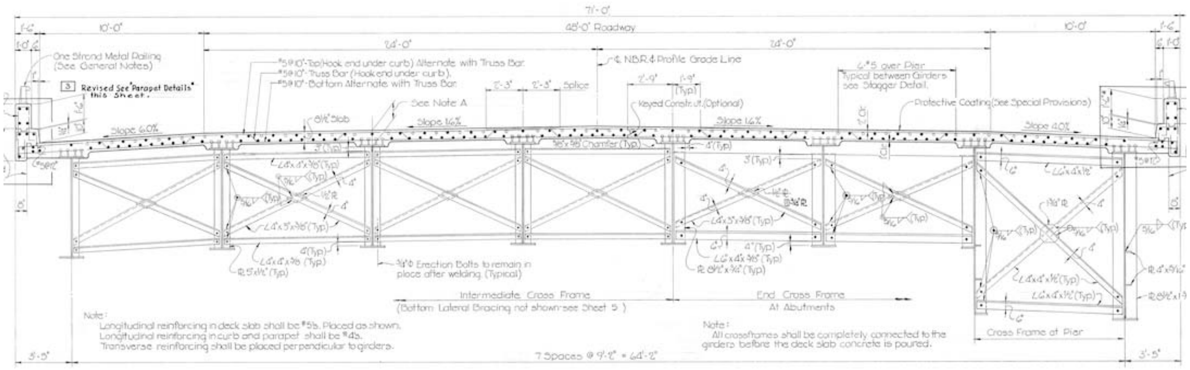

Figure 4.11 Typical Cross Section at Mid-span of the I-95 over Patuxent River Bridge

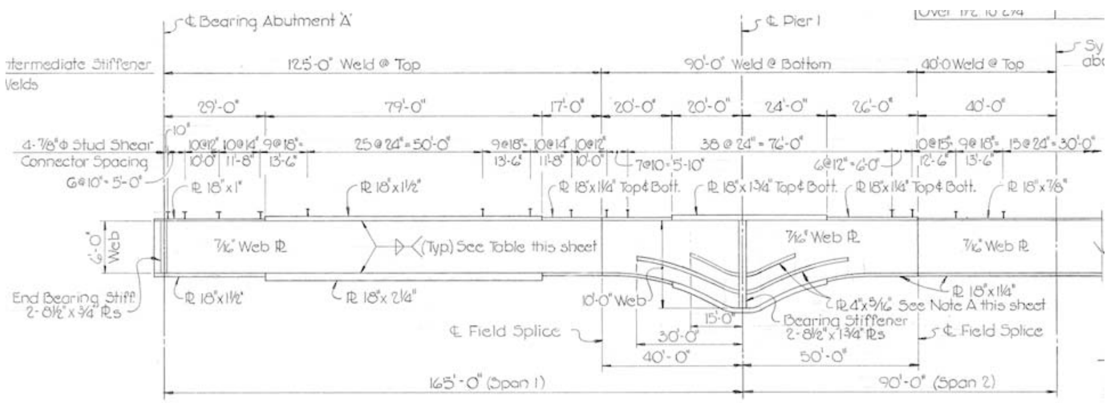

Figure 4.12 Partial Girder Elevation-N.B.R

In MERLIN-DASH, the interior girder sections were used for all eight of the girders and in the area of the parabolic shape defined as concave down. The dead load information is shown in detail in Table 4.3 below. The spacing of the lateral bracing is 23.57 feet for the end spans and 22.5 feet for the middle span. The model built in MERLIN-DASH is shown in Figure 4.13.

Table 4.3 Dead Load Information of the I-95 over Patuxent River Bridge

Slab Loads Per Beam		
Deck Slab		0.748 kips/ft.
Arbitrary Uniform Loads Per Beam		
Dead Load	Haunch	0.021 kips/ft.
	Stay in place	0.049 kips/ft.
Superimposed Dead Load	Future wearing surface	0.176 kips/ft.
	Barriers	0.324 kips/ft.

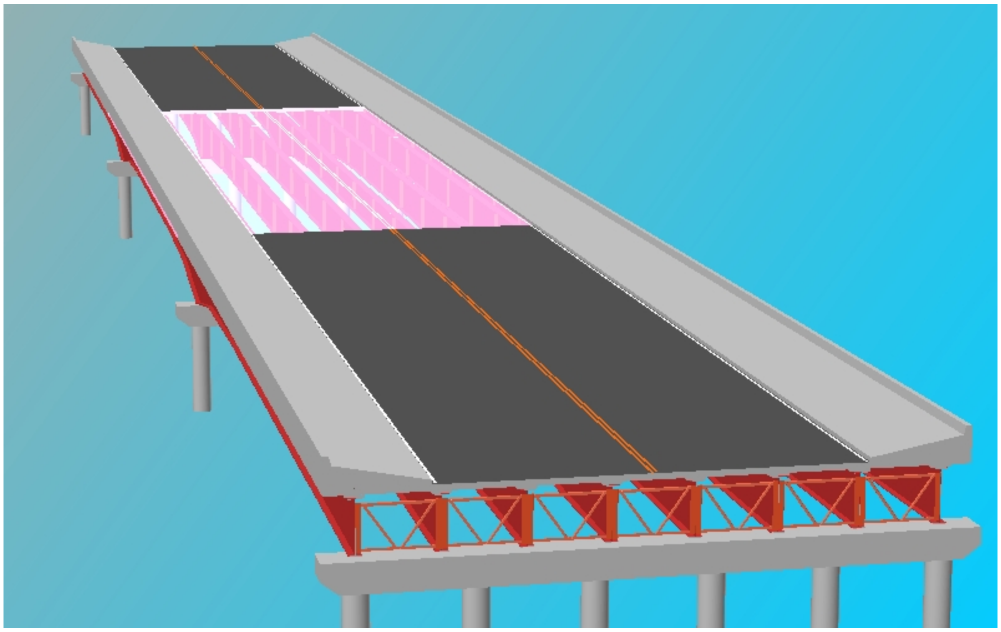

Figure 4.13: The I-95 over Patuxent River Bridge Model in MERLIN-DASH

The live load deflections of the I-95 over Patuxent River bridge (bridge no. 1619701) obtained from MERLIN-DASH rating function are 0.750 inches for end spans and 0.808 inches for the mid span within the ASD method. In the LRFD method, the end span deflection is 0.470 inches and the mid span deflection is 0.527 inches. The lane load governs in ASD and HL-93 design truck load governs in LRFD.

The I-95 over Patuxent River bridge (bridge no. 1619701) is a straight steel girder bridge so skew needs to be considered in the DESCUS-I model. An assumption for parabolic shape in girder elevation was made so that the average depth of the web in the changing area was selected to be the constant; therefore, seven different plate girder sections, according to Figure 4.12, were used for the bridge model in DESCUS-I. As a result, the girder elevation could be modeled, as shown Figure 4.14, and detailed, as listed in Table 4.4. Figure 4.15, is half of the framing plan for the I-95 over Patuxent River bridge from DESCUS-I model graphics.

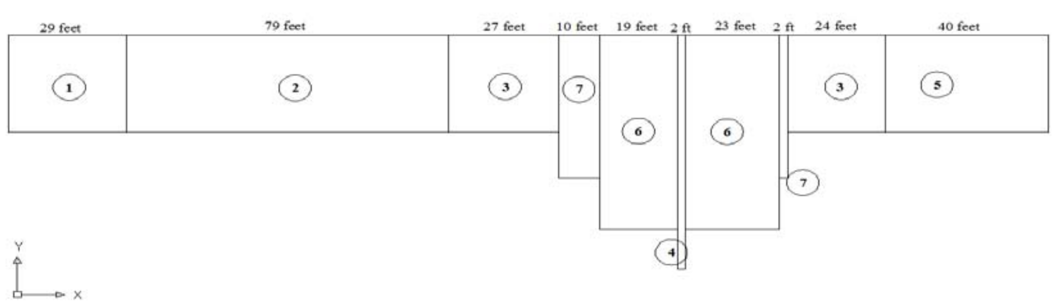

Figure 4.14 Girder Elevation of Half of the I-95 over Patuxent River Bridge in DESCUS-I

Table 4.4 Beam Sections

	P.G. Web		P.G. Flange			
Unit: inch	Depth	Thick	Top Width	Top Thick	Bot Width	Bot Thick
Section1	72	0.4375	18	1	18	1.5
Section2	72	0.4375	18	1.5	18	2.25
Section3	72	0.4375	18	1.25	18	1.25
Section4	120	0.4375	18	1.75	18	1.75
Section5	72	0.4375	18	0.875	18	1.25
Section6	108	0.4375	18	1.75	18	1.75
Section7	84	0.4375	18	1.25	18	1.25

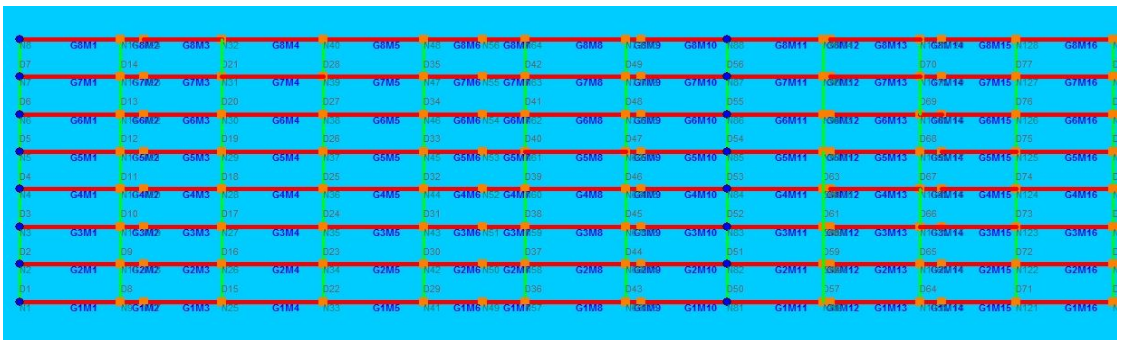

Figure 4.15 Half Framing Plan of the I-95 over Patuxent River Bridge in DESCUS-I

The live load deflections of the I-95 over Patuxent River bridge (bridge no. 1619701) obtained from DESCUS for the ASD Design Method are 1.012 inches for all the spans. In the LRFD method, the end spans' deflection is 0.684 inches and the mid span deflection is 0.732 inches. The lane load governs in ASD and HL-93 design truck load governs in LRFD.

In CSiBridge, this bridge is modeled as the actual one (Figure 4.16). For the ASD method, the lane load governs and the live load deflections are 0.670 inches for end spans and 0.720 inches for the mid span; for the LRFD method, 25 percent truck load plus lane load governs and the live load deflections are 0.930 and 0.933 inches for two end spans and mid span, respectively.

A field bridge test was conducted for the I-95 over Patuxent River bridge (bridge no. 1619701) and a few sensors were used to acquire the live load deflection data. A long-term measurement provided the maximum deflection on the location of 48 feet and 10 inches from the south bridge abutment to be 5.96 mm (0.23 in.) and 4.98 mm (0.20 in.) for the third girder and the second girder, respectively. From the software results, it was found that the measurement data are slightly smaller. This may be due to the fact that the actual vehicles' effect on the bridge did not reach the same high level as the fully loaded assumption used in the program analysis.

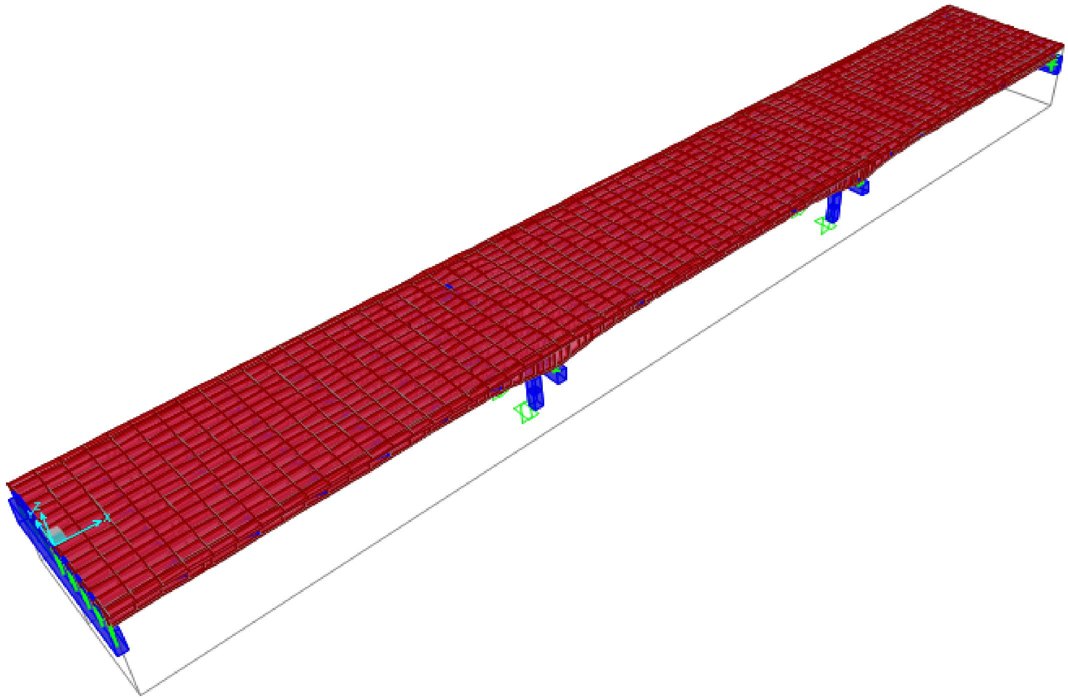

Figure 4.16 The I-95 over Patuxent River Bridge Model in CSiBridge

4.4 Summary of Live Load Deflection Comparison

The live load deflections of representative bridges obtained from three different programs are shown in table 4.5.

Table 4.5 Live Load Deflection of Three Representative Bridges

	LRFD			**ASD**		
I-270 over Middlebrook Road 1 span L=140 ft. 3 lanes	Multiple lane factor **MF=0.85** Impact factor **IF=33%**			**MF=0.9** **IF**=50/(L+125)=**18.9%**		
	DASH	CSI Bridge	DESCUS-I	DASH	CSI Bridge	DESCUS-I
	-0.651	-0.984	-1.017	-0.821	-1.107	-1.151
I-95 over Patuxent River 3 spans L=165-180-165 ft. 4 lanes	**MF=0.65** **IF=33%**			**MF=0.75** **IF**=50/(L+125)=**17.2%** (End span) **IF**=50/(L+125)=**16.4%** (Mid span)		
	DASH	CSI Bridge	DESCUS-I	DASH	CSI Bridge	DESCUS-I
	-0.470(end) -0.527(mid)	-0.670 -0.720	-0.684 -0.732	-0.763 -0.808	-0.93 -0.933	-1.012 -1.012
Route 1 over Paint Branch 2 spans L=80-80 ft. 3 lanes	**MF=0.85** **IF=33%**			**MF=0.9** **IF**=50/(L+125)=**24.4%** (span 1) **IF**=50/(L+125)=**24.4%** (span 2)		
	DASH	CSI Bridge	DESCUS-I	DASH	CSI Bridge	DESCUS-I
	-0.687(span1) -0.687(span2)	-0.821 -0.821	-0.918 -0.918	-0.825(span1) -0.825(span2)	-0.973 -0.973	-1.079 -1.079

According to the AASHTO Standard Specifications for Highway Bridges (2002), Article 10.6.4

> ”When spans have cross-bracing or diaphragms sufficient in depth or strength to ensure lateral distribution of loads, the deflection may be computed for the standard H or HS loading (M or MS) considering all beams or stringers as acting together and having equal deflection.”
>
> “When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally.”

It could be found that the distribution of live load is used as an average value (number of lanes/number of girders) since all the girders are treated as equal. This may cause a discrepancy that the live load deflection is relatively small using line-girder program, which can be explained in Figure 4.17.

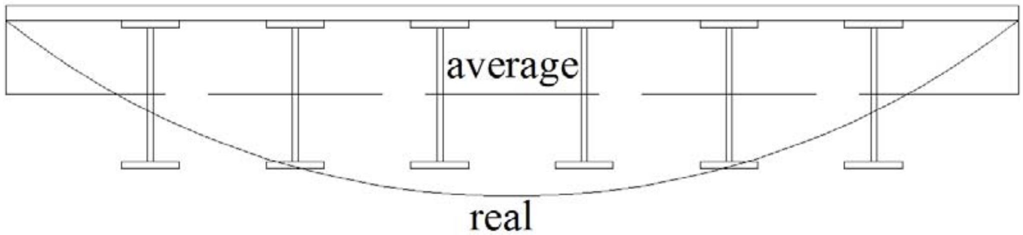

Figure 4.17 Distribution of Live Load along the Roadway

In DASH, the live load distribution factors are using this average value and multiple-presence factor together. Taking I-270 over Middlebrook Road bridge (bridge no. 1504200) as an example, there are three lanes and eight girders so that the distribution factor for ASD is 0.338 and for LRFD is 0.319. This could cause the live load deflection to be very small and then affect the results of analysis. In order to avoid this situation, an adjustment was developed in DASH. The live load distribution factor is defined as the number of the traffic lanes divided by the effective number of girders, which means the girders directly support the lanes but not the curb. For instance, for the I-270 over Middlebrook Road bridge (bridge no. 1504200), the distribution factor for both ASD and LRFD is changed to 0.5. With this idea in mind, the live load deflection for the three representative bridges from DASH becomes higher, which is closer to the actual case and more conservative. With this adjustment made to all of the test bridges, the live load deflections are provided in Table 4.6.

Table 4.6 Live Load Deflection of Three Bridges with Adjustment

	LRFD			**ASD**		
I-270 over Middlebrook Road 1 span L=140 ft. 3 lanes	DASH	CSiBridge	DESCUS	DASH	CSI Bridge	DESCUS
	-1.021	-0.984	-1.017	-1.216	-1.107	-1.151
I-95 over Patuxent River 3 spans L=165-180-165 ft. 4 lanes	DASH	CSiBridge	DESCUS	DASH	CSI Bridge	DESCUS
	-0.723(end) -0.811(mid)	-0.670 -0.720	-0.684 -0.732	-1.017 -1.077	-0.93 -0.933	-1.012 -1.012
Route 1 over Paint Branch 2 spans L=80-80 ft. 3 lanes	DASH	CSiBridge	DESCUS	DASH	CSI Bridge	DESCUS
	-0.970(span1) -0.970(span2)	-0.821 -0.821	-0.918 -0.918	-1.100(span1) -1.100(span2)	-0.973 -0.973	-1.079 -1.079

In Table 4.6, the deflections from DASH are slightly conservative and are acceptable because all of them are under the L/800 criteria. In the next chapter, 30 sample bridges are analyzed with the DASH program.

5. Results of 30 Sample Bridges Using the Line-girder Method

In this chapter, 30 sample bridges from the Maryland Bridge Inventory were modeled in DASH and analyzed for both the ASD and LRFD methods. The Inventory, maintained by SHA, lists the majority of the bridges from the 23 counties in Maryland, including the important information on the bridges, such as location, design load, bridge types and span length, as well as the year when they were built. The design load is typically HS-20. H-20 and HS-25 loads also occupy a certain proportion. Most bridges in this inventory have one span to three spans, and the span lengths range from 30 feet to 300 feet. The sample bridges consist of 10 single-span bridges, 10 double-span bridges and 10 three-span bridges. The following chart shows the length for each span of these bridges. The distribution of the span length of these 30 bridges covers most of the range of the bridge span length in the Inventory; therefore, these 30 bridges could reasonably serve as the sample bridges in this study.

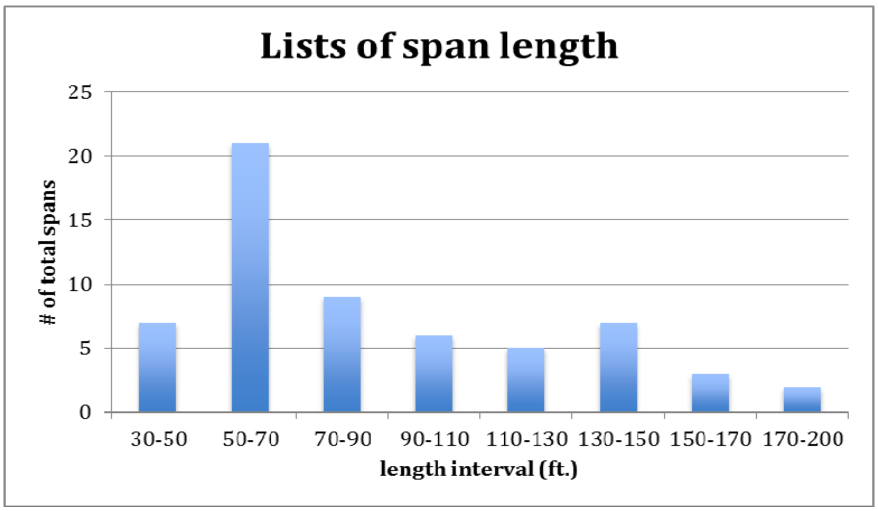

Figure 5.1 Distribution of Span Length for Sample Bridges

5.1 Live Load Deflection Analysis

These bridges were designed by using different truck loads, 17 of them were using the HS-20 truck as the design load, eight of them used H-20, and the remaining four used H-20 because they were built decades ago. The H-20 truck load is similar to HS-20, but only has one axle of wheels in the back. The single line girder method was still used for analysis in this chapter. The live load deflection from the ASD and LRFD methods with two different design trucks for all the bridges were compared. Comparison for single-span bridges, double-spans bridges and three-spans bridges were conducted separately in this section. Since the HL-93 design truck is based on the HS-20 truck, while the HS-25 truck is 25 percent larger than the HS-20, a factor of 1.25 was applied for the live load deflection results from HL-93 in the LRFD method to evaluate how close those results are. The figure of deflection vs. span length for all sample bridges is shown Figure 5.2. It can easily be observed that the live load deflections from HS-25 are larger than ones from HL-93 through the two trend lines since the HS-25 truck is heavier. However, all the bridges with these two different load types have acceptable deflections under the L/800 deflection limit.

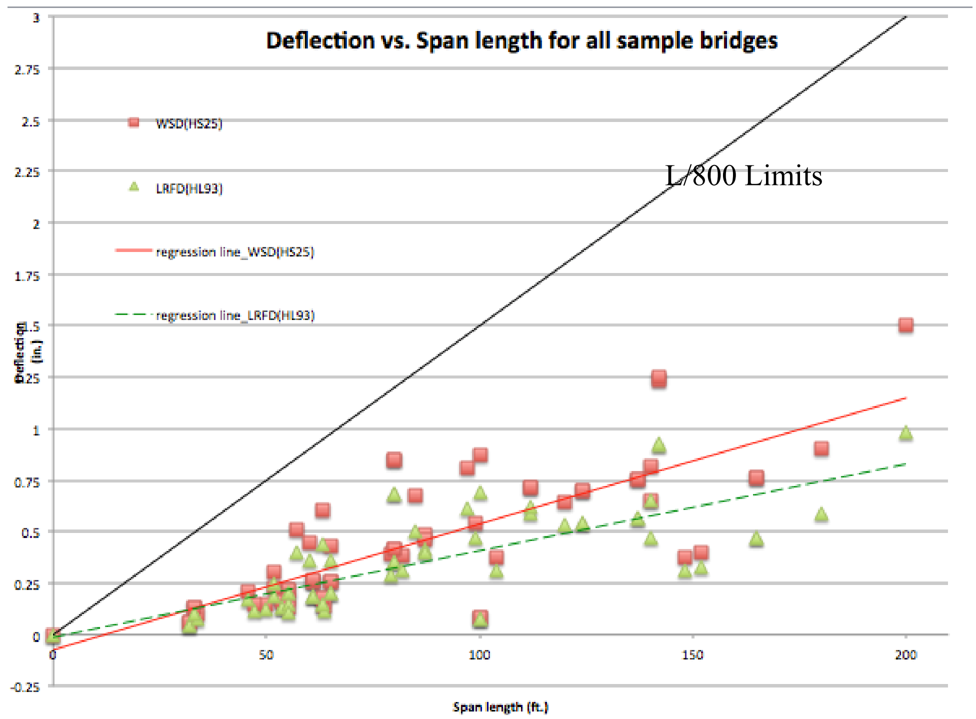

Figure 5.2 Deflection vs. Span Length for All Sample Bridges

Figures 5.3 to 5.6 show the comparison for bridges which have different numbers of spans (single span, two spans and three spans for mid-span and side-span) with different vehicular loads HS-20 for the ASD (or WSD) method and HL-93 for the LRFD method. Another value added to this comparison is the deflection under the HL-93 design truck load multiplied by a factor of 1.25. The trend lines for all these three values are shown in each figure and the tables for live load deflection data are also displayed afterwards (Tables 5.1 to 5.3). In the ASD method, the governing load types for each bridge are also listed in the tables. The capital letter ‘L’ stands for ‘lane load’ and ‘HS’ means the ‘HS-25 design truck’.

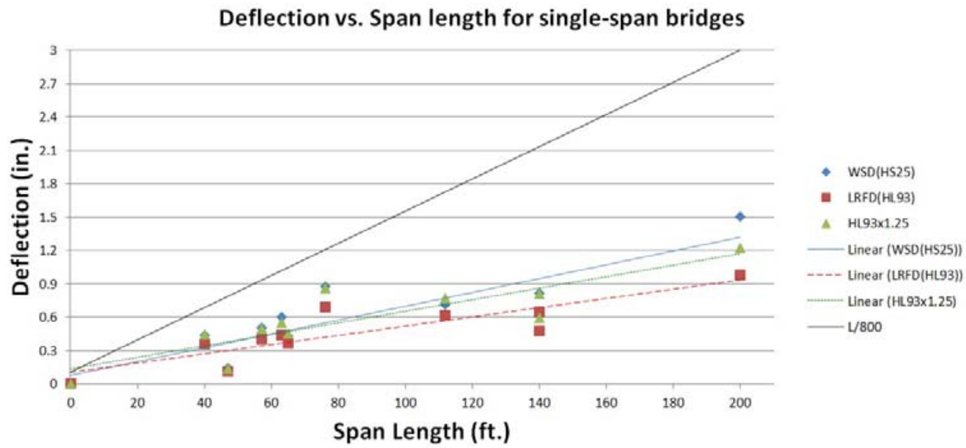

Figure 5.3 Deflection vs. Span Length for Single-span Bridges

Table 5.1 Single-span Bridges Live Load Deflection

Br. #	Span Length	WSD(HS-25) Deflection	WSD(HS-25) Load Type	LRFD(HL-93) Deflection	HL-93x1.25	L/800 (in.)
EB 100'	100	-0.876	HS	-0.693	-0.866	1.500
201200	57	-0.508	HS	-0.401	-0.501	0.855
300700	65	-0.431	HS	-0.363	-0.454	0.975
502703	112	-0.716	HS	-0.619	-0.774	1.680
1000400	63	-0.602	HS	-0.439	-0.549	0.945
1504200	140	-0.821	L	-0.651	-0.814	2.100
1610500	47	-0.144	HS	-0.112	-0.140	0.705
1629400	140	-0.653	L	-0.474	-0.593	2.100
EB 60'	60	-0.443	HS	-0.359	-0.449	0.900
EB 200'	200	-1.505	L	-0.981	-1.226	3.000

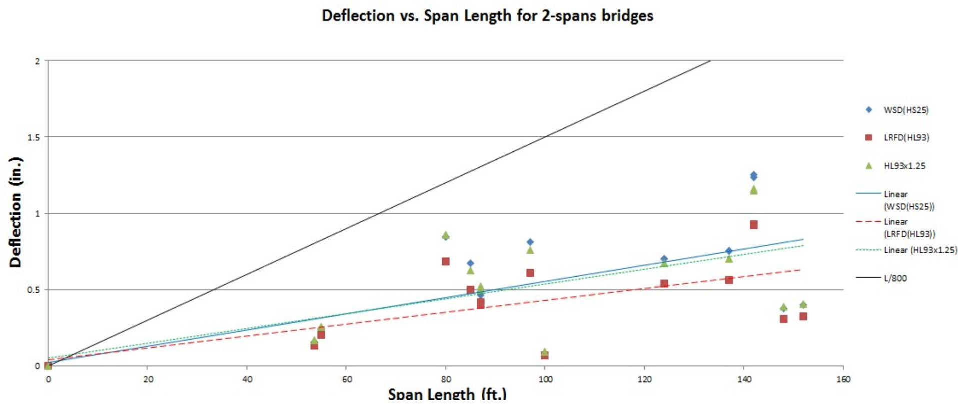

Figure 5.4 Deflection vs. Span Length for Two-span Bridges

Table 5.2 Two-span Bridges Live Load Deflection

Br. #	Span Length	WSD(HS-25) Deflection	WSD(HS-25) Load Type	LRFD(HL-93) Deflection	HL-93x1.25	L/800 (in.)
101900	55	-0.244	HS	-0.203	-0.254	0.825
	55	-0.244		-0.203	-0.254	0.825
206502	124	-0.703	HS	-0.538	-0.673	1.860
	124	-0.703		-0.538	-0.673	1.860
319100	85	-0.674	HS	-0.502	-0.628	1.275
	97	-0.810		-0.609	-0.761	1.455
603200	148	-0.375	L	-0.310	-0.388	2.220
	152	-0.400		-0.325	-0.406	2.280
700300	100	-0.084	HS	-0.073	-0.091	1.500
	100	-0.084		-0.073	-0.091	1.500
1202600	53.5	-0.160	HS	-0.133	-0.166	0.803
	53.5	-0.160		-0.133	-0.166	0.803
1303003	142	-1.235	L	-0.921	-1.151	2.130
	142	-1.251		-0.930	-1.163	2.130
1600400	80	-0.850	HS	-0.687	-0.859	1.200
	80	-0.850		-0.687	-0.859	1.200
BHT87-87	87	-0.487	HS	-0.419	-0.524	1.305
	87	-0.466		-0.401	-0.501	1.305
BHT137-137	137	-0.754	L	-0.564	-0.705	2.055
	137	-0.754		-0.564	-0.705	2.055

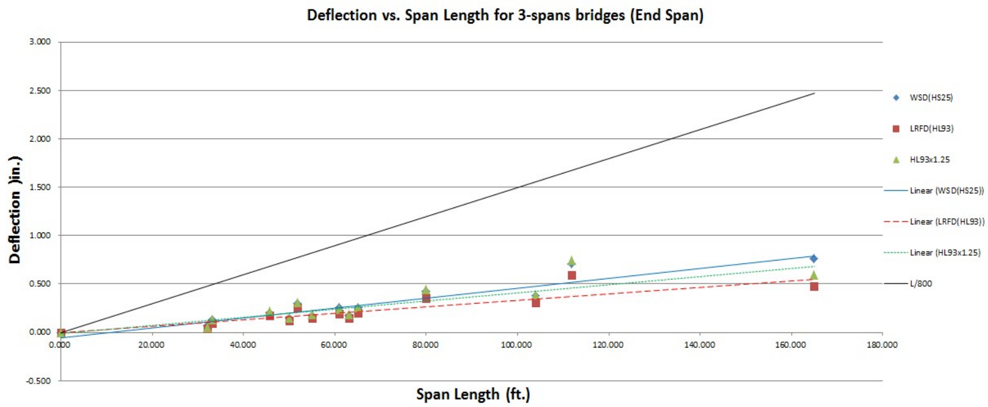

Figure 5.5 Deflection vs. Span Length for Three-span Bridges (Side Span)

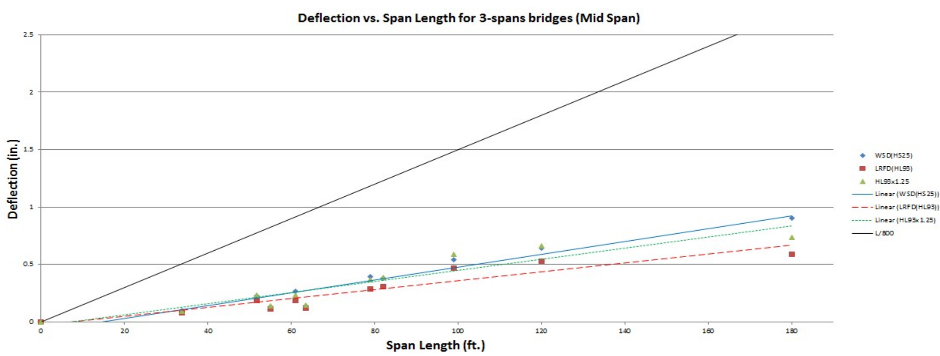

Figure 5.6 Deflection vs. Span Length for Three-span Bridges (Mid Span)

Table 5.3 Three-span Bridges Live Load Deflection

Br. #	Span Length	WSD(HS-25) Deflection	Load Type	LRFD(HL-93) Deflection	HL-93x1.25	L/800 (in.)
107703	104	-0.376	HS	-0.309	-0.386	1.560
	120	-0.641		-0.533	-0.666	1.800
	112	-0.716		-0.591	-0.739	1.680
216901	65	-0.259	HS	-0.206	-0.258	0.975
	82	-0.383		-0.310	-0.388	1.230
	65	-0.259		-0.206	-0.258	0.975
308300	61	-0.259	HS	-0.188	-0.235	0.915
	79	-0.393		-0.291	-0.364	1.185
	61	-0.259		-0.188	-0.235	0.915
318100	32	-0.062	HS	-0.043	-0.054	0.480
	61	-0.264		-0.188	-0.235	0.915
	32	-0.062		-0.043	-0.054	0.480
803700	63.125	-0.176	HS	-0.148	-0.185	0.947
	63.5	-0.138		-0.116	-0.145	0.953
	63.125	-0.176		-0.148	-0.185	0.947
1400501	55	-0.173	HS	-0.144	-0.180	0.825
	55	-0.134		-0.111	-0.139	0.825
	50	-0.145		-0.120	-0.150	0.750
1619701	165	-0.763	L	-0.470	-0.588	2.475
	180	-0.908		-0.587	-0.734	2.700
	165	-0.763		-0.470	-0.588	2.475
1703200	33.167	-0.128	HS	-0.099	-0.124	0.498
	33.75	-0.099		-0.077	-0.096	0.506
	33.167	-0.128		-0.099	-0.124	0.498
2105700	45.75	-0.207	HS	-0.170	-0.213	0.686
	51.667	-0.227		-0.188	-0.235	0.775
	51.75	-0.304		-0.252	-0.315	0.776
BHT80-99-80	80	-0.415	HS	-0.352	-0.440	1.200
	99	-0.543		-0.469	-0.586	1.485
	80	-0.415		-0.352	-0.440	1.200

With the tables and figures shown above, it is easy to see that the lane load governs for the longer spans in length, and the design truck load governs for shorter spans. The live load deflections under the HS-25 truck load are slightly larger than the ones under HL-93 with a factor of 1.25; however, there is no obvious relationship between the live load deflection and the various span lengths. The properties of the bridges themselves, such as different choices of sections for the girders or different design of lateral bracing, would affect the behavior of the bridges under vehicular loads. If the bridges are poorly designed, even a lighter load such as HS-15 generates large deformation.

All the data above are based on the application of live load distribution factor with multiple lane presence together, which were mentioned in Chapter Four. Using this method leads to a relatively small effect on the deflection results, which is not conservative for analyzing bridge behavior given the aforementioned. Therefore, in this section, the modified method of live load distribution was applied, which is 'effective-girder without live load distribution factors' as mentioned in the last part of Chapter Four. The live load deflections of 30 sample bridges under the HS-25 design truck with the ASD method and the HL-93 design truck with the LRFD method as well as the 1.25 times of deflections from HL-93 have been listed and analyzed as the same as the previous analysis. In this case, however, the deflections of the HL-93 truck are getting close to HS-25 and even slightly larger than HS-25 for double-spans bridges according to the results shown in the Figures 5.6 to 5.10 and Tables 5.4 to 5.6, but still, all the deflections are under the L/800 limit. This is because the multiple lane presence factors used in LRFD are smaller than those in the ASD method (see Table 3.3 in Chapter Three).

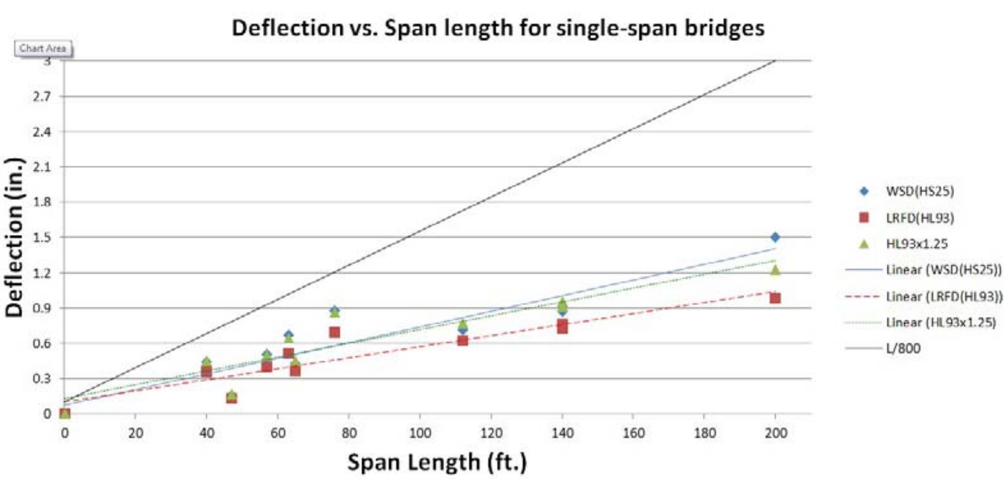

Figure 5.7 Deflection vs. Span Length for Single-span Bridges with Modified Distribution

Table 5.4 Single-span Bridges Live Load Deflection with Modified Distribution

Br. #	Span Length	WSD(HS-25) Deflection	Load Type	LRFD(HL-93) Deflection	HL-93x1.25	L/800 (in.)
EB 100'	100	-0.876	HS	-0.693	-0.866	1.500
201200	57	-0.508	HS	-0.401	-0.501	0.855
300700	65	-0.431	HS	-0.363	-0.454	0.975
502703	112	-0.716	HS	-0.619	-0.774	1.680
1000400	63	-0.669	HS	-0.516	-0.646	0.945
1504200	140	-0.912	L	-0.766	-0.957	2.100
1610500	47	-0.160	HS	-0.132	-0.165	0.705
1629400	140	-0.871	L	-0.729	-0.912	2.100
EB 60'	60	-0.443	HS	-0.359	-0.449	0.900
EB 200'	200	-1.505	L	-0.981	-1.226	3.000

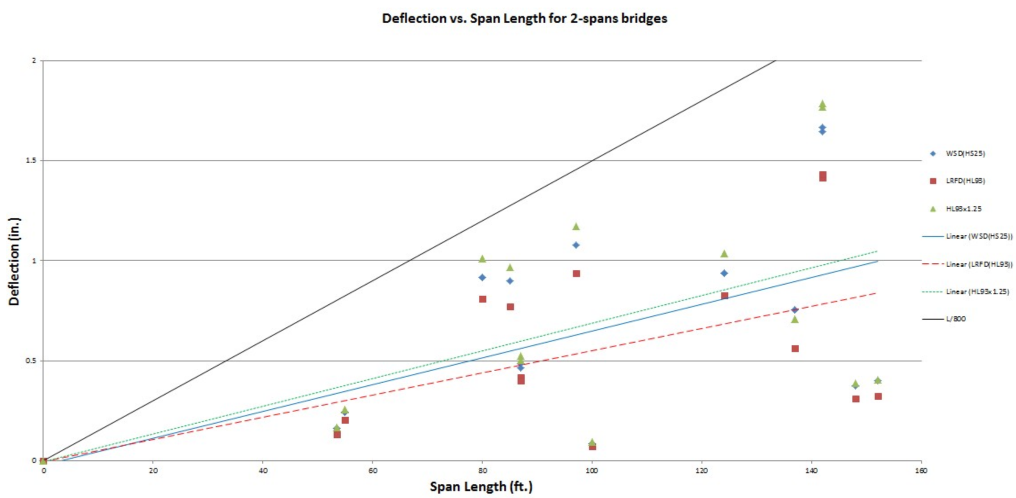

Figure 5.8 Deflection vs. Span Length for Two-span Bridges with Modified Distribution

Table 5.5 Two-span Bridges Live Load Deflection with Modified Distribution

Br. #	Span Length	WSD(HS-25) Deflection	Load Type	LRFD(HL-93) Deflection	HL-93x1.25	L/800 (in.)
101900	55	-0.244	HS	-0.203	-0.254	0.825
	55	-0.244		-0.203	-0.254	0.825
206502	124	-0.937	HS	-0.828	-1.035	1.860
	124	-0.937		-0.828	-1.035	1.860
319100	85	-0.899	HS	-0.772	-0.965	1.275
	97	-1.080		-0.937	-1.171	1.455
603200	148	-0.375	L	-0.310	-0.388	2.220
	152	-0.400		-0.325	-0.406	2.280
700300	100	-0.084	HS	-0.073	-0.091	1.500
	100	-0.084		-0.073	-0.091	1.500
1202600	53.5	-0.160	HS	-0.133	-0.166	0.803
	53.5	-0.160		-0.133	-0.166	0.803
1303003	142	-1.647	L	-1.417	-1.771	2.130
	142	-1.668		-1.431	-1.788	2.130
1600400	80	-0.917	HS	-0.808	-1.010	1.200
	80	-0.917		-0.808	-1.010	1.200
BHT87-87	87	-0.487	HS	-0.419	-0.524	1.305
	87	-0.466		-0.401	-0.501	1.305
BHT137-137	137	-0.754	L	-0.564	-0.705	2.055
	137	-0.754		-0.564	-0.705	2.055

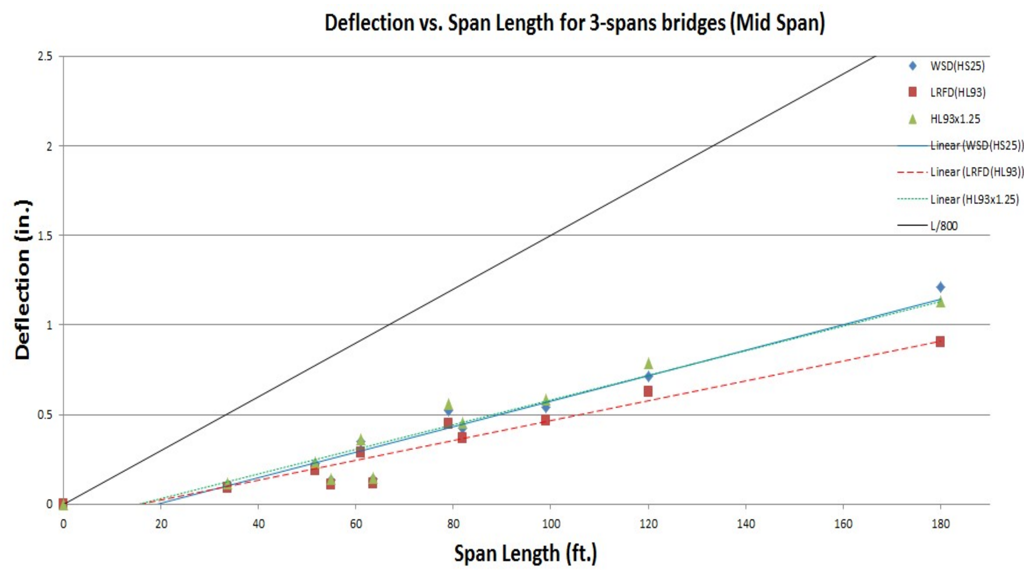

Figure 5.9 Deflection vs. Span Length for Three-span Bridges (Side) with Modified Distribution

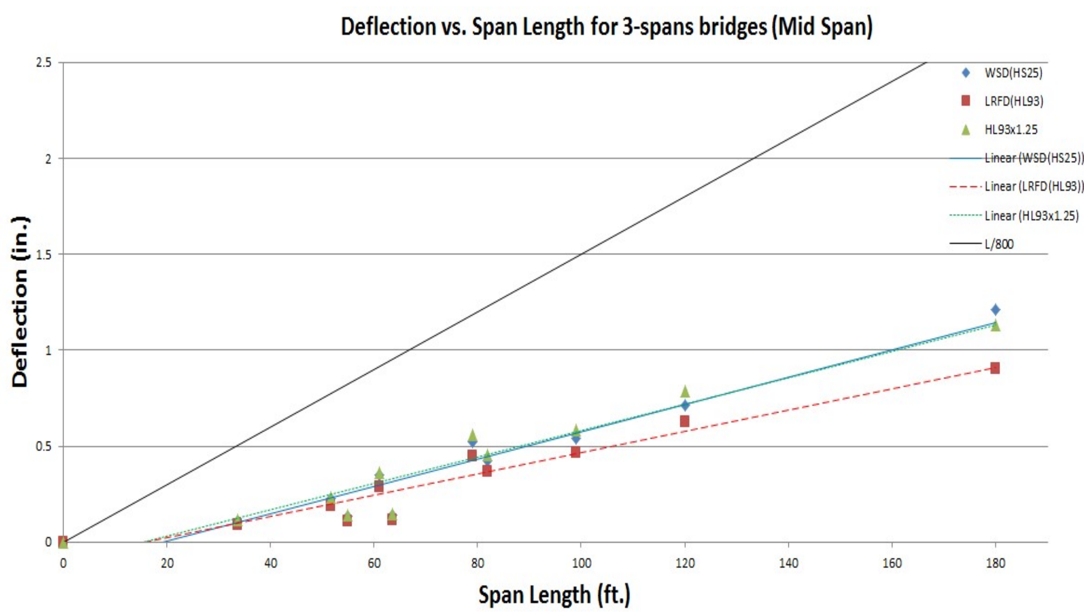

Figure 5.10 Deflection vs. Span Length for Three-span Bridges (Mid) with Modified Distribution

Table 5.6 Three-span Bridges Live Load Deflection with Modified Distribution

Br. #	Span Length	WSD(HS-25) Deflection	WSD(HS-25) Load Type	LRFD(HL-93) Deflection	HL-93x1.25	L/800 (in.)
107703	104	-0.418	HS	-0.364	-0.454	1.560
	120	-0.712		-0.627	-0.784	1.800
	112	-0.796		-0.695	-0.869	1.680
216901	65	-0.288	HS	-0.242	-0.303	0.975
	82	-0.426		-0.365	-0.456	1.230
	65	-0.288		-0.242	-0.303	0.975
308300	61	-0.345	HS	-0.289	-0.362	0.915
	79	-0.524		-0.448	-0.560	1.185
	61	-0.345		-0.289	-0.362	0.915
318100	32	-0.083	HS	-0.066	-0.083	0.480
	61	-0.352		-0.289	-0.362	0.915
	32	-0.083		-0.066	-0.083	0.480
803700	63.125	-0.176	HS	-0.148	-0.185	0.947
	63.5	-0.138		-0.116	-0.145	0.953
	63.125	-0.176		-0.148	-0.185	0.947
1400501	55	-0.173	HS	-0.144	-0.180	0.825
	55	-0.134		-0.111	-0.139	0.825
	50	-0.145		-0.120	-0.150	0.750
1619701	165	-1.017	L	-0.723	-0.904	2.475
	180	-1.211		-0.903	-1.129	2.700
	165	-1.017		-0.723	-0.904	2.475
1703200	33.167	-0.142	HS	-0.116	-0.146	0.498
	33.75	-0.110		-0.091	-0.113	0.506
	33.167	-0.142		-0.116	-0.146	0.498
2105700	45.75	-0.207	HS	-0.170	-0.213	0.686
	51.667	-0.227		-0.188	-0.235	0.775
	51.75	-0.304		-0.252	-0.315	0.776
BHT80-99-80	80	-0.415	HS	-0.352	-0.440	1.200
	99	-0.543		-0.469	-0.586	1.485
	80	-0.415		-0.352	-0.440	1.200

Based on the analyses described above, a conclusion could be reached that lane load governs those bridges having larger span length. The live load deflection under the HS-25 design truck is larger than those under the HL-93 truck. A factor of 1.25 applied for HL-93 would increase the deflection to become the same as HS-25 with or without distribution modification. Therefore, the HL-93 truck load could be adopted as the design vehicular load whether they are old bridges analysis or new bridges design to meet the L/800 limit of live load deflection. A factor of 1.25 is suggested to make the current designs consistent with past practice, safer and to stay on the conservative side.

5.2 Load Rating Analysis

The load rating of the sample bridges based on strength was adopted within the DASH program. Rating methods used in this section are Load and Resistance Factor Rating (LRFR) and Allowable Stress Rating (ASR). The rating level is inventory rating. The HL-93 truck was used in LRFR, same in the previous section, but in ASR the HS-20 truck was adopted since most bridges are designed with the HS-20 truck load. The rating factor was calculated by the simple formula as follows:

$$RF = {}^{(F_b - D)}\!/_{(LL+I)} = \text{Index for live load capacity}$$

where:

$$RF = \text{Rating factor}$$

$$F_b = \text{Allowable stress} = 0.55 F_y R_L$$

$$R_L = \text{reduction factor due to lateral bracing}$$

$$D = \text{dead load stress}$$

$$LL + I = \text{stresses due to live load} + \text{impact}$$

The results of the rating factor for both methods are shown in the following figures (Figures 5.11 to 5.13) and a ratio between LRFR and ASR are also shown in a third figure. The different symbols stand for the design vehicular load for those bridges.

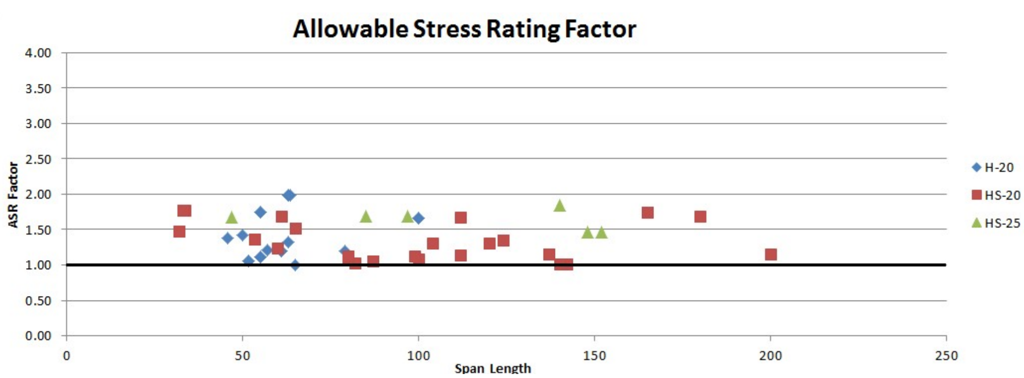

Figure 5.11 Allowable Stress Rating

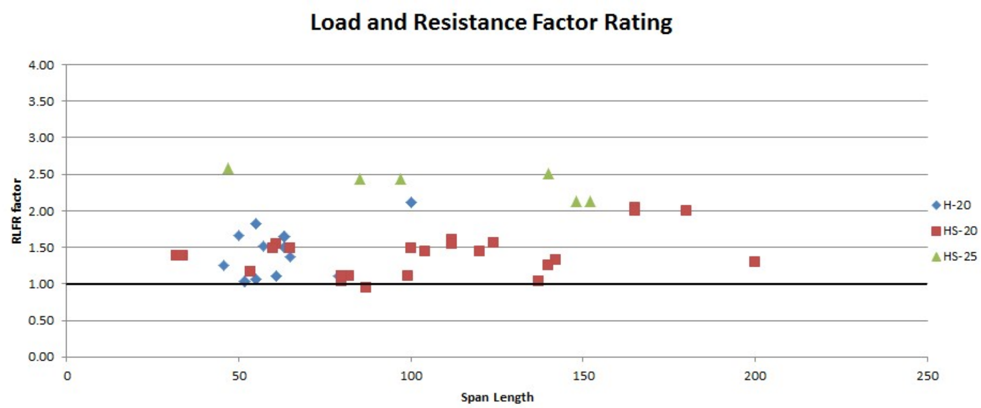

Figure 5.12 Load and Resistance Factor Rating

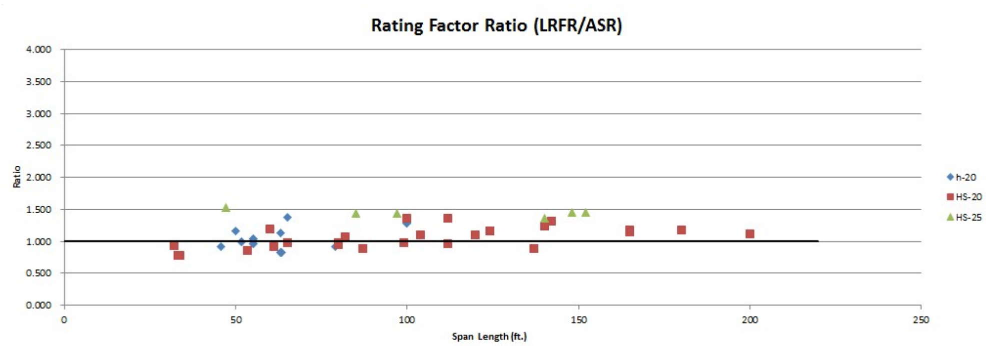

Figure 5.13 Rating Factor Ratio (LRFR/ASR)

From Figures 5.11 to 5.13, the rating factors for almost all the bridges based on both methods could be found to be larger than 1.0, which means the designs are appropriate. It could also noted that the rating factors for the bridges designed in the period of 1990 to 2007 whose design load are HS-25 would have larger values. This is because the lighter truck (HS-20 or HL-93) has been applied in rating for the heavy-truck-design bridges. According to the results, it could be concluded that using the HS-20 or HL-93 truck load is acceptable in design.

6. Construction Closure Pours Case Study

In the closure pour case study, two bridges, MD140 over MD27 (bridge no. 6032) and I-695 over Ingleside Avenue (bridge no. 0312300), were studied. Since the I-695 bridge is a straight bridge with equally spaced girders and no skew and can be easily done within two stages, no serious consequences with one closure pour were found. Therefore, only the MD 140 bridge was studied. During the staged construction of the MD140 over MD27 bridge, the second stage started when the first stage was completed and opened to travel from vehicle traffic. Then, the third stage started afterwards when both stages 1 and 2 completed and opened to travel from vehicle traffic. On the deck pouring sheet prepared by the SHA, the following statements were made about the staged construction as part of the Maryland practice:

1. Closure pours placed after all adjacent pours are in place for a minimum of forty (40) hours.
2. Prior to placing closure pours the contractor shall verify that the girder dead load deflections at girder nos. 5 and 12 (adjacent girders of the new stage) meet the required elevations for the deck construction.
3. If the newly poured section of deck is higher than the required elevation, the contractor shall place a load at midspan of the specified girder to deflect the girder to the required elevation.
4. Place uniform loads over a length of 12 feet and placed symmetrically about the centerline midspan of each girder.
5. The contractor shall not apply these loads until the concrete in the deck slab has reached a minimum of 76% of the specified 28 days compressive strength and as approved by the engineer.
6. The contractor shall submit calculations and details for the loading to the engineer for approval.

During the construction of stage two, it was found that the new deck was higher than stage one. The same phenomenon was also found during the construction of stage 3. Even though some difference was expected, the differential displacements were higher than predicted. Extra effort was then required to try to bring the elevation between stages to the same level. The closure pour was then completed.

There were many factors that could have contributed to the differential displacements between stages. Listed below are just several that the SHA project team brought up:

- Increased deflections resulting from a lower moment of inertia of the girders due to the use of HPS
- Variable girder spacing on either side of the closure pour
- Different girder designs between stages
- Shrinkage of the concrete deck in the first stage.

More detailed analyses were made in the following section to determine the cause of the differential displacements. Instead of shrinkage as stated above, creep effect was considered more influential and was considered in the analysis process. Camber diagrams were not built into the model, but were considered afterwards in order to find the actual differential displacements.

6.1 Model analysis of the MD140 over MD27 Bridge

The MD No. 6032 bridge is located on MD140 over Maryland Midland Railroad, MD27 and West Branch in Carroll County. It is a 300 feet two-span (152 feet and 148 feet) steel bridge that consists of 15 plate girders distributed over 130′ bridge width (Figures 6.1 and 6.2). The girder spacing is 10′ for Girder 1 to 5 in the northbound, 9′-9″ for Girder 11 to 15 in the southbound, 7′-3″ for the rest and 5′ for the median between the northbound and southbound directions. The clear roadway width is 61′-5″ in the northbound direction and 50′-5″ in the southbound direction.

The construction of the bridge was divided into three stages, as shown in Figure 6.3. The first stage covers the middle strip from Girder 6 to Girder 11, the second stage consists of Girder 1 to Girder 5, and the last stage consists of Girder 12 to 15. For each stage, the construction was arranged into three pouring sequences. The first sequence covers the 102 feet in the positive moment area of longer span from the north abutment, the second sequence covers the 99 feet in the positive moment area of short span from the south abutment, and the final sequence is the remainder of the bridge. Between each construction stage, closure pour was adopted. The displacements at the adjacent girders between stages 2 and 3 were studied to evaluate the serviceability of the bridge during construction.

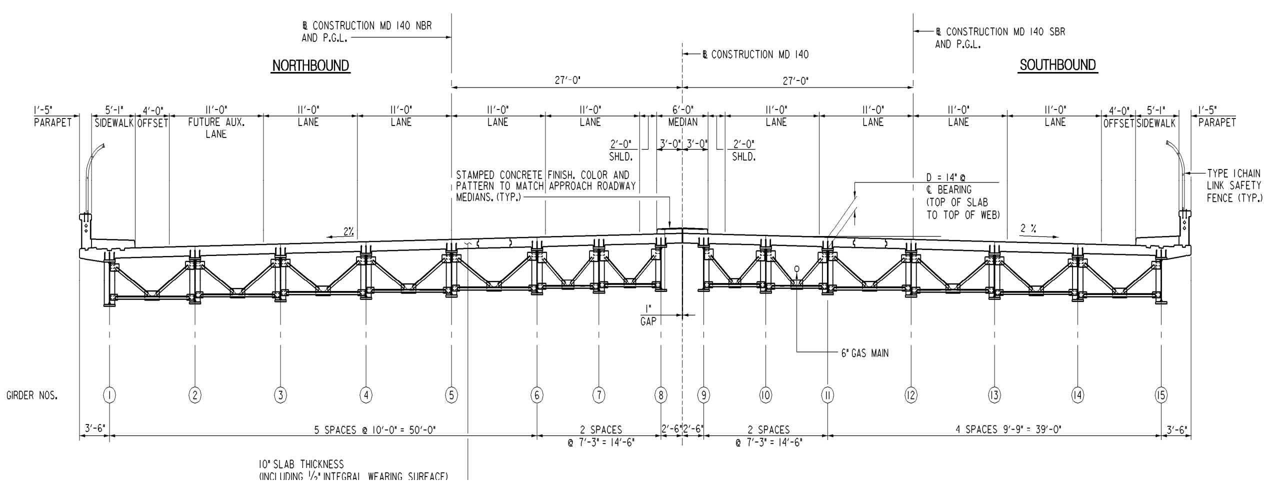

Figure 6.1 Typical Cross Section of Bridge MD140 over MD27

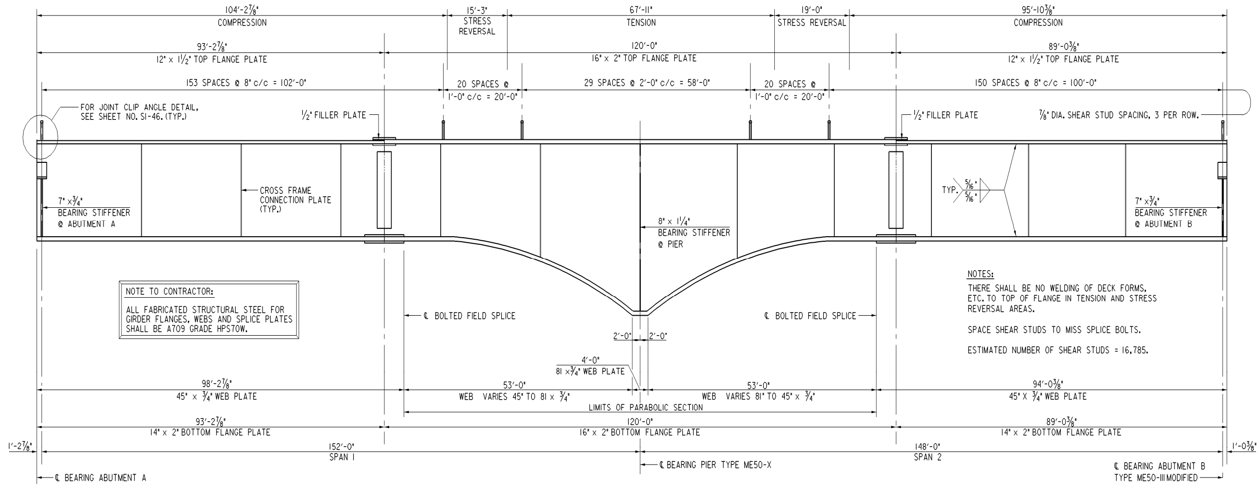

Figure 6.2 Girder Elevations (Mid Strip)

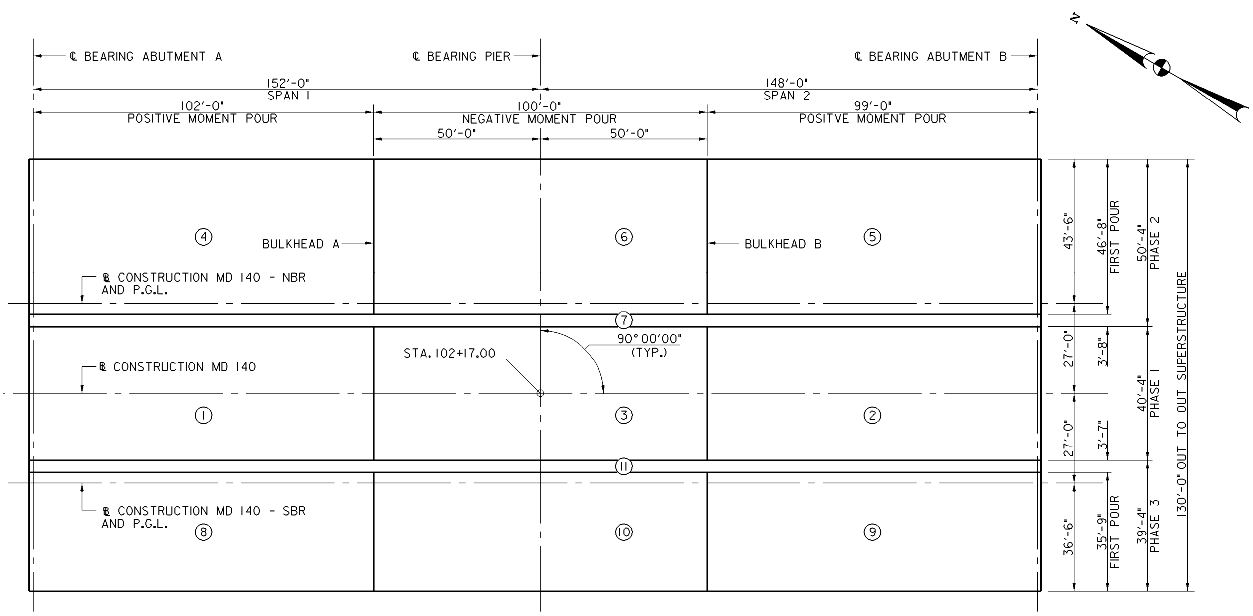

Figure 6.3 Bridge Pouring Stages and Sequences

The bridge was selected for this study because of reported problems associated with closing gaps between stages. The original design used the line-girder program Merlin-DASH to establish the camber diagrams. They were grouped into eight camber diagrams, which are (1) Girder 1, (2) Girders 2-5, (3) Girders 6 & 7, (4) Girder 8, (5) Girder 9, (6) Girder 10 & 11, (7) Girders 12-14, and (8) Girder 15. The girders adjacent to the closure pours are Girders 5 and 6 between stages one and two as well as Girders 11 and 12 between stages two and three. The case study is on the sixth camber diagram group for Girder 11 and the seventh camber diagram group for Girder 12 where the second closure pour is located. Because of the similarity of cambers due to weight of the parapets and sidewalk (ΔS.D.L.) and vertical curvature of the roadway (ΔV.C.) among all girders, only the maximum cambers on the longer span (152′) due to the self-weight of girder (ΔGIRDER) and concrete slab and deck forms (ΔD.L.) of Girders 11 and 12 are compared. From the camber diagram the sums of dead load cambers (ΔGIRDER + ΔD.L.) are 4 3/8″ and 5 9/16″, respectively, with a difference of 1 3/16″ between two girder. However, by viewing the framing plan, it was found that the camber diagrams for Girders 10 and 11 should not be grouped together where the tributary width for Girder 10 is 7′-3″ and that for Girder 11 is 8′-6″ (= 0.5[7′-3″ + 9′-9″]). With the average tributary area considered for Girder 11, the sum of dead load cambers (ΔGIRDER + ΔD.L.) is now 5 1/8″ and the camber difference between Girder 11 and 12 is reduced to 7/16″, instead of the original 1 3/16.″ Therefore, the first priority of matching the elevation between stages is to construct correct camber diagrams with proper tributary area considered.

In order to obtain the global displacement profile under staged construction, the whole bridge was modeled and re-analyzed using both DESCUS-I and CSiBridge computer programs. The MD140 over MD27 Bridge was first constructed as a two-dimensional grid model in DESCUS. In DESCUS, the parabolic sections were simulated using equivalent web depths varying from 45 inches to 81 inches, shown in Figure 6.4. Girders 11 and 12 are the edge girders of each construction stage, and their reactions were monitored in this study.

The girder displacements of the non-composite stage were analyzed at first. Then, the bridge model simulating the construction stages was introduced. To simplify the analysis, in this staging construction model, Girders 1 to 11 were modeled as stage one with fully matured concrete slab, while Girders 12 to 15 were modeled as stage two with concrete slab of one-day age to simulate the stage before closure pours. To discuss the influence of bridge diaphragm during construction, two sets of models were built for comparison. One set has diaphragms between Girders 11 and 12, another set does not.

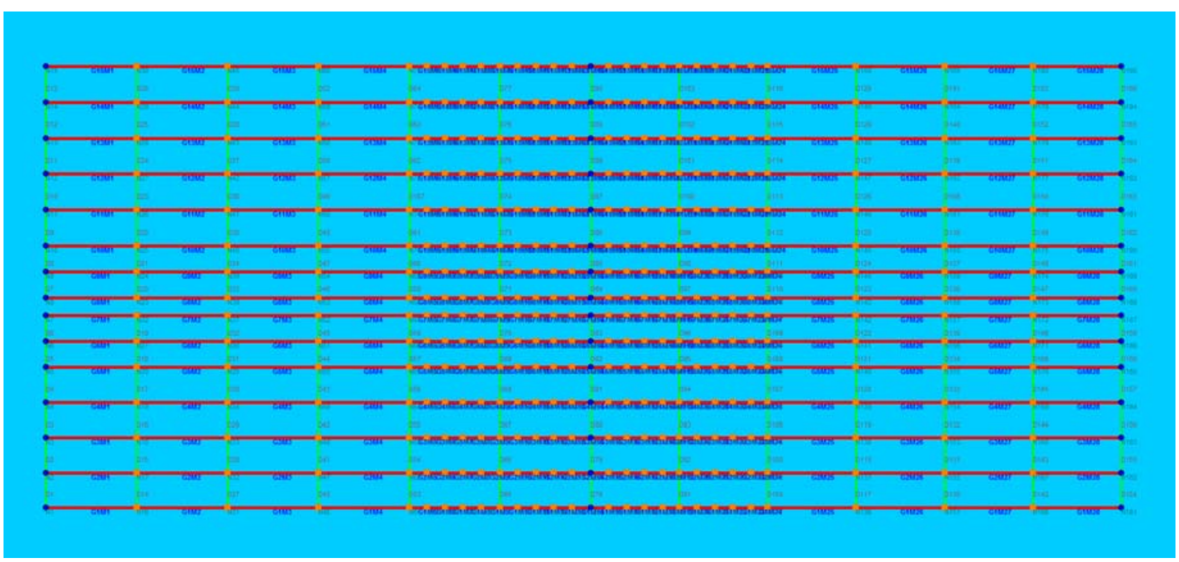

Figure 6.4 DESCUS Model

A three-dimensional global model of the MD 140 bridge (bridge no. 6032) was also established by CSiBridge. To simulate the construction of stages 2 and 3, a gap of closure pour was considered between Girders 11 and 12. Three types of models with different assumptions were built to evaluate the creep effect on the bridge between stages (Figure 6.5).

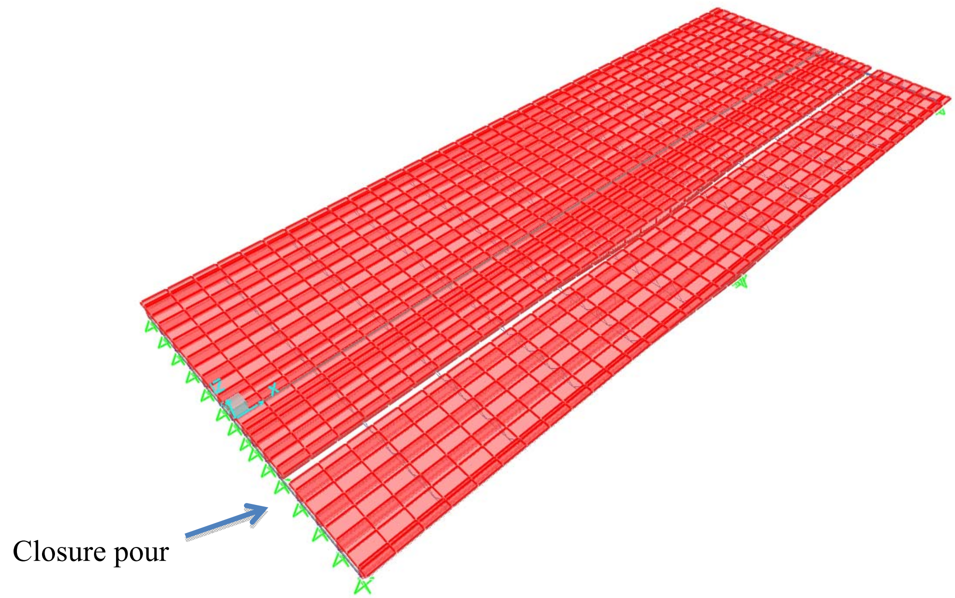

Figure 6.5 CSiBridge Model for MD140 over MD27 Bridge Isometric View

1. The first CSiBridge model was used to simulate non-composite behavior for the early bridge construction. Since in the non-composite period, both bridge girders and slab contribute to the stiffness property of the composite section, the constraint between girder and deck was defined that corresponding joints only share the same vertical displacement.
2. For the second CSiBridge model, Young's module of the bridge slab was set to zero, similar to the assumption of the DESCUS model. Although such a method is not recommended by CSi knowledge base, it can be used to validate the CSiBridge non-composite model and compare later with the staging models.
3. In the third CSiBridge model, time dependent properties of materials and the construction staging were introduced into the bridge model. Concrete compressive strength, stiffness, as well as creep were considered for materials properties (Figure 6.6a-c). The deck construction of Girders 1 to 11 was defined as stage one, construction of Girders 12 to 15 was defined as stage two. A 28-day period was defined between these two stages to allow the concrete from the stage one construction to develop creep. A 3′7″ gap between Girder 11 and 12 was created to represent the closure pour gap during staged construction. The maximum displacement of each girder at 60 feet from the north abutment was monitored and the difference between Girders 11 and 12 (the edge girders of each stage) was also observed and discussed.

Each model also has two sub-sets, with and without diaphragms between Girders 11 and 12, therefore the effect of diaphragms in staged construction was modeled.

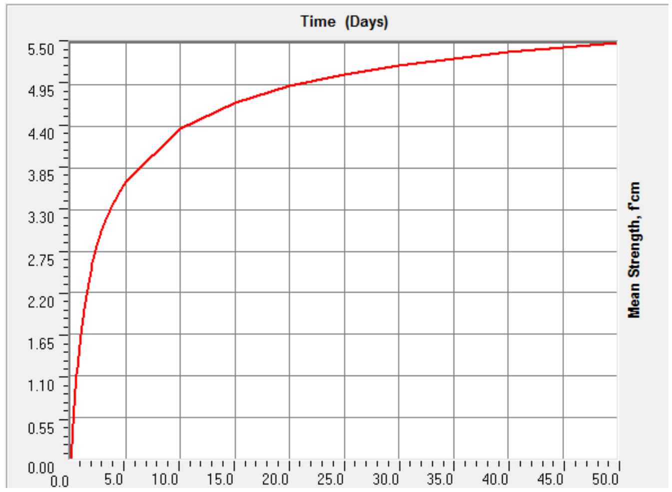

Figure 6.6(a) Time Dependent Concrete Strength in CSiBridge Model (kip/*in*^{2})

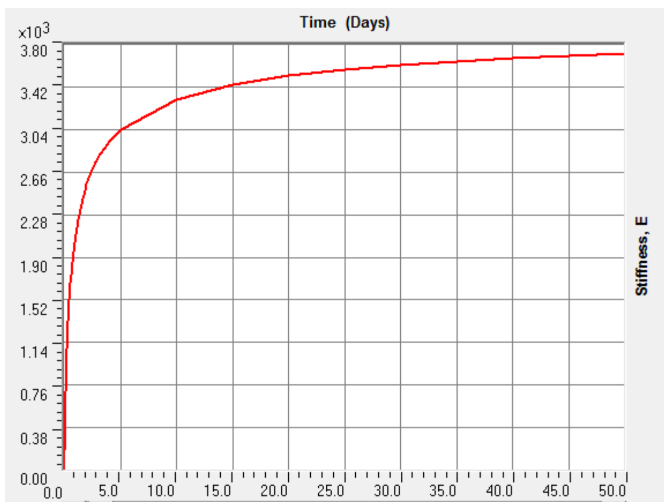

Figure 6.7(b) Time Dependent Concrete Stiffness in CSiBridge Model (kip/*in*^{2})

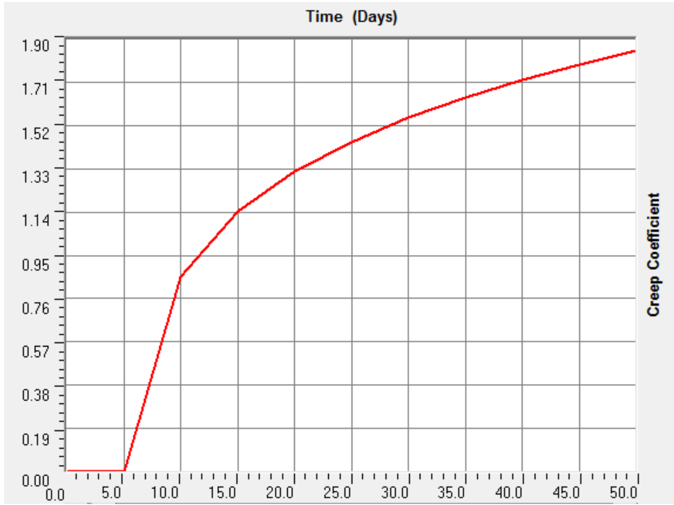

Figure 6.8(c) Time Dependent Creep Coefficient in CSiBridge Model

6.2 Summary of Displacement Comparison by Refined Analyses

The whole bridge was modeled and re-analyzed using both DESCUS-I and CSiBridge computer programs for the purposes of comparing the camber diagrams and staging analysis. The maximum displacements of Girders 11 and 12 from four (4) different models in DESCUS are shown in Table 6.1. For the bridge with diaphragms to simulate the final construction situation, the maximum displacements are 4.69 inches and 3.77 inches at 60 feet and 57 feet from respective long and short span abutments for Girder 11, and 4.84 inches and 3.91 inches at 60 feet and 57 feet from respective long and short span abutments for Girder 12. The displacement difference between Girders 11 and 12 with diaphragm at the long span is only 0.15″ (4.84″ – 4.69″).   For the bridge without diaphragms, which simulate the bridge under staged construction, the maximum displacements are 4.66 inches and 3.75 inches for Girder 11, and 5.18 inches and 4.18 inches for Girder 12. The displacement difference between Girders 11 and 12 without diaphragm at the long span is only 0.52″ (5.18″ – 4.66″), which is very close to the “corrected” differential cambers of 7/16″ established by the line girder program. For each bridge model, the displacements of Girder 12 are always larger than those of Girder 11. It is consistent with the original design by the line-girder program.   The difference is caused by the tributary area of Girder 12 and is larger than the tributary area of Girder 11. Also, there is an increment of differential displacement in the model without diaphragms because the newly constructed concrete slab cannot distribute its load to the whole bridge due to lack of diaphragms.

Table 6.1 Maximum Displacements Comparison for Normal and Staging Models by DESCUS

Maximum displacement for each model (inches)			
	DESCUS	Girder 11 (long span/short span)	Girder 12 (long span/short span)
With diaphragms model	Normal model	4.69/3.77	4.84/3.91
	Stage model	4.38/3.52	4.56/3.68
Without diaphragms model	Normal model	4.66/3.75	5.18/4.18
	Stage model (w/3′-7″ gap)	4.09/3.28	4.68/3.77

Tables 6.2 and 6.3 show the comparison of results between various CSiBridge models and the DESCUS model. By comparing the displacements from the non-composite model of CSiBridge with that from the non-composite model of DESCUS, it can be observed that these two results are very close. More detailed figures (Figures 6.7 and 6.8) show the displacement along the entirety of Girder 11 and Girder 12. The conclusion can be made that the bridge response during non-composite period can be well simulated by both DESCUS and CSiBridge program.

Table 6.2 Comparison Table between Different Models with Diaphragms

With diaphragms	Maximum displacement comparison (@60' from north abutment) (inch)			
		G11	G12	Differential
CSiBridge	Non-composite	4.58	4.69	0.11
	Stage construction (consider closure pour)	4.17	4.24	0.07
DESCUS	Non-composite	4.69	4.84	0.15

Table 6.3 Comparison Table between Different Models without Diaphragms

Without diaphragms	Maximum displacement comparison (@60' from north abutment) (inch)			
		G11	G12	Differential
CSiBridge	Non-composite	4.56	5.21	0.65
	Stage construction (consider closure pour)	4.54	4.66	0.12
DESCUS	Non-composite	4.09	4.68	0.59

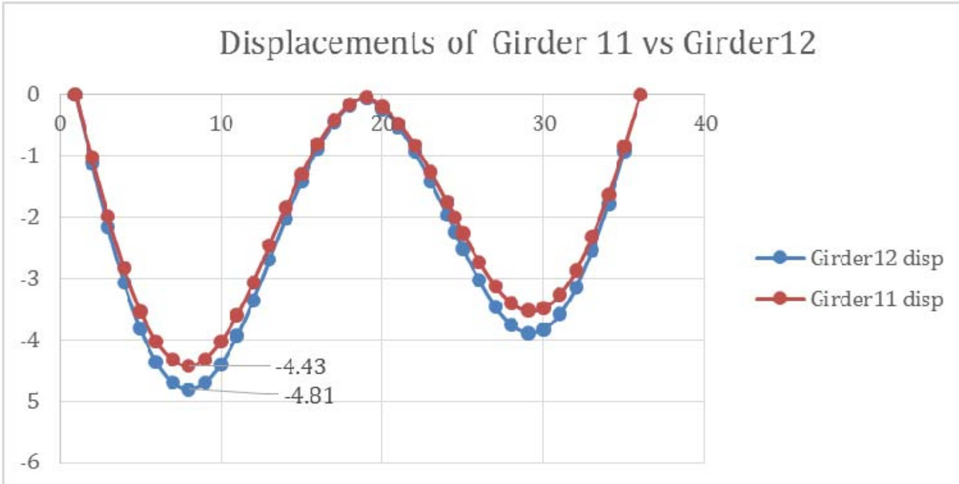

Figure 6.9 Vertical Displacement Results from CSiBridge (with Diaphragm) (inch).

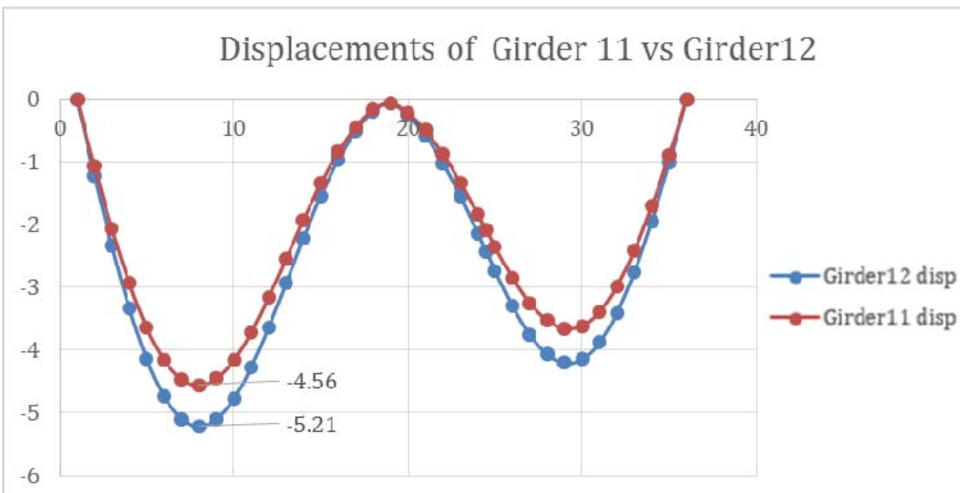

Figure 6.10 Vertical Displacement Results from CSiBridge (without Diaphragm) (inch).

The results from the second CSiBridge model with the Young's module of bridge slab set to zero are compared here with the results from the DESCUS models. The results from the DESCUS and CSiBridge non-composite model are close, with small margins of differences. Such differences exist because in CSiBridge setting concrete Young's module to zero does not necessarily make the bridge non-composite, though in theory these two assumptions should yield the same results.

The results from staged construction models were also compared. The results shown in Tables 6.2 and 6.3 are the final displacements, where the bridge stage one concrete has an age of 31 days and the stage two concrete has the age of 3 days. Since the time dependent properties of material are introduced in this model, the stage one concrete displacement is increased due to the creep effect. It can be observed that the displacement from Girder 11 has a noticeable increase, while the displacement from Girder 12 stays roughly the same (changed slightly due to diaphragm dragging). A detailed investigation about how the maximum displacement developed with time was conducted. The following figures (Figure 6.9 and 6.10) show the creep development for both “with diaphragm” and “without diaphragm” models. The displacement of Girder 11 (shown as a red line) between day 3 to day 30 is slowly increased about 0.2 inches. This displacement increment is the effect from concrete creep behavior. Also, it is noticed that for the “with diaphragm” case, Girder 11 deflects further after the second construction phrase is completed. This is due to the diaphragms connecting the neighbor girder of the next stage helping to distribute the load from the second stage to the first stage. Such behavior is not shown in the “without diaphragm” model.

Comparing the results from the CSiBridge and DESCUS models, some displacement differences are found. The reason is in CSiBridge, a 3′7″ closure pour gap between Girders 11 and 12 was created to model the bridge during construction precisely. The difference due to loading can be narrowed down if the dead loads due to closure pour gap concrete are removed with closure pour gap simulated. However, if the creep effect is considered, the CSiBridge model could generate more realistic results.

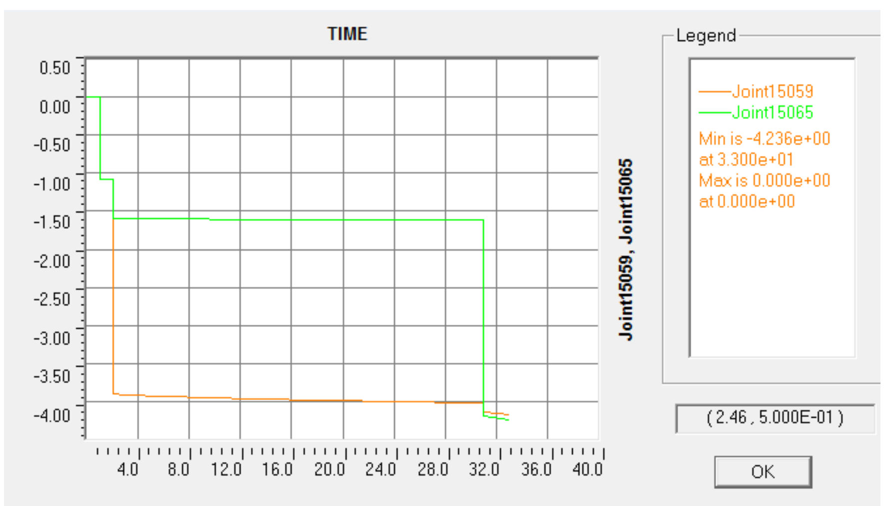

Figure 6.11 The Creep Growth in Girder 11(Orange) and 12(Green) for with Diaphragms Model (in).

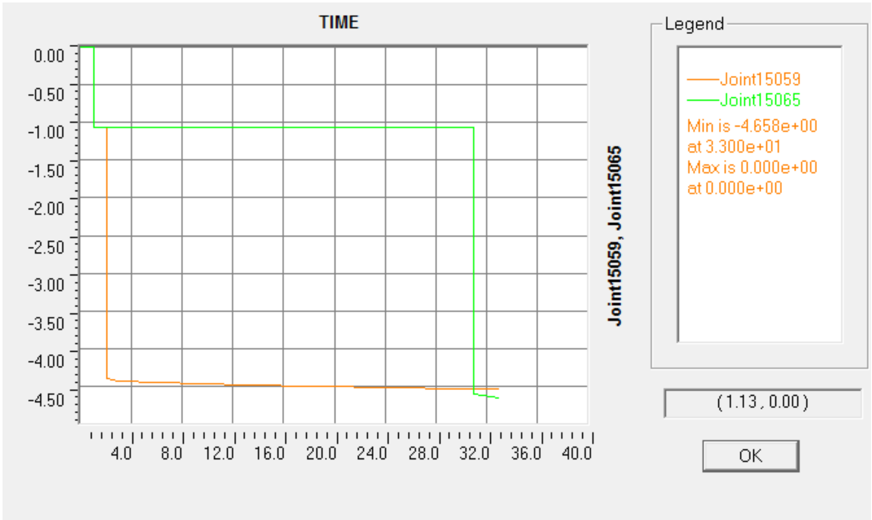

Figure 6.12 The Creep Growth in Girder 11(Orange) and 12(Green) without Diaphragms Model (in).

Figure 6.11 shows the displacement increment for Girder 11 over time from the without diaphragm model. To better represent the real scenario, concrete deck was modeled to carry load start at the age of 7 days. It can be observed that the creep develop relatively fast in the first 30 days (22.63%), after 30 days the displacement growth become steady and gradually reduced. The conclusion can be made that for bridge construction, creep effect has strong impact on bridge during first 30 days.

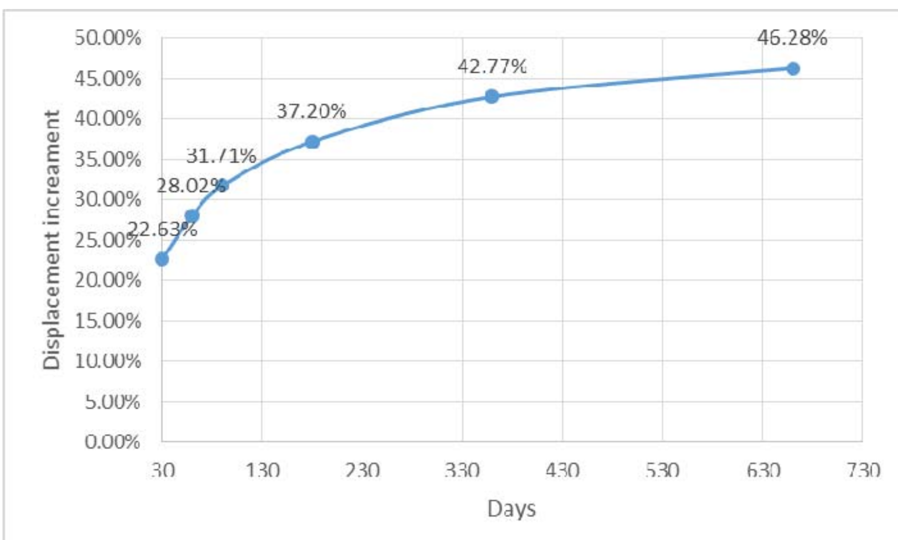

Figure 6.131 Displacement Increment for Girder 11 without Diaphragms Model (in).

7. Summary and Conclusion

As a very important issue in the serviceability of steel bridges, live load deflection still attracts great attention because ensuring safety is always the number one priority in structure design. Bridge structures should be designed with sufficient strength but designers also need to ensure the deflection is within an acceptable range so that drivers or passengers in the vehicles would not believe the bridge is unsafe.

Based on this study, the following conclusions were reached:

A: Findings associated with bridge live load deflections -

1. Span Length (L)/800 is appropriate to be the live load deflection limit for steel bridge design no matter what type of design load or design method is applied. The maximum 1/800 of the span length for general vehicular bridges and 1/1000 of the span length for vehicular bridges with pedestrian traffic are universally accepted criteria for the live load deflection limit.
2. The live load deflection from the HS-25 design truck alone in the ASD method (employed by the State of Maryland from 1990 until 2008 when LRFD was adopted) is larger than the deflection from the larger of the HL-93 design truck load alone or HL-93design lane load +25% truck load in the LRFD method. Therefore, if the "HS-25 equivalent" truck is required by Maryland for deflection criteria, a factor of 1.25 is suggested for usage in the HL-93 design truck to obtain conservative results. In bridge deflection analysis, the lane load governs for bridges that have a longer span length while the design vehicular load governs for those with shorter spans.
3. Comparing the numeric results from two-dimensional grid models and three-dimensional finite element models, the line girder method proves to be an acceptable application for live load deflection analysis of steel beam/girder bridges with all lanes loaded. Short-term field monitoring using laser device also found live load deflections are within these limits.
4. AASHTO LRFD Specifications (2014) allows an average value (number of lanes/number of girders) used for the investigation of maximum absolute deflection for straight girder systems with all girders treated as equal. This study found it is generally true for bridges that are not too wide. Using line – girder programs to analyze wide bridges may result in discrepancies such as relatively smaller live load deflection. When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded and all girders can be assumed to deflect equally as stated in the AASHTO LRFD Specifications (2014). Line-girder program with dynamic load allowance and average distribution factor (number of lanes/number of girders) can be used, but the multiple-presence factor should be removed.

B: Findings associated with bridge construction closure pours -

5. The general practice in Maryland is to use a line-girder program to establish the camber diagrams. The result is generally accurate enough and acceptable in practice. Multiple girders with varied girder spacing are grouped into several camber diagrams. It is noticed that camber diagrams are not grouped based on their tributary widths, but rather, on the narrower girder spacing. In this case some girders may be under-cambered. This does not usually cause problems for one-stage construction, but may cause trouble for staged construction if one side of the closure pour is under-cambered.
6. Multiple camber diagrams can be calculated by the line-girder models, two-dimensional grid model or three-dimensional finite element model. All three methods generate results accurate enough for straight girder systems, if the creep effect is not considered.
7. Maryland adopted the generally recommended practice of a minimum closure width of three (3) feet and diaphragms/cross frames in the staging bay of structural steel girders not rigidly connected until later. Not connected until final pour is a general practice by many states, and is suggested by this study. There is no ill effect for non-connected practice.
8. To investigate the staging effect of staged-construction, two-dimensional grid models and three-dimensional finite element models are highly recommended. The differential displacement between stages could not be considered in a one-dimensional line-girder model.
9. When comparing the results of the two-dimensional grid model with those of the three-dimensional finite element model, these two methods produced results accurate enough for straight girder systems, if the creep effect is not considered. However, since the two-dimensional grid model program has to simulate the closure pour by subtracting the deck loads during staged construction, the three-dimensional finite element model can be more closely simulated with a gap on the deck. Also, in the case where the closure pour is significant, the three-dimensional finite element model would provide more accurate results.
10. For further staging analysis, the creep effect of concrete was considered in this study. Two controlled sets of models were studied. One model assumes the diaphragms in the closure pours always connect with girders during the whole phrase construction. The other model assumes the diaphragms are disconnected. To investigate the creep effect with concrete slab, two different models analyses from DESCUS and CSiBridge were performed. For the staging analysis of DESCUS, only the time-dependent property of Young's Modulus was considered. However, for the CSiBridge, the staging analysis not only simulated the time-dependent property of Young's Modulus, but also considered the creep effect in concrete. If creep effect would be considered, computer programs equipped with creep analysis feature should be adopted.
11. For general bridge with constant girder spacing, due to creep effect, the old stage built in the early stage would deflect more and the displacement gap between stages would increase. However, correct cambers would alleviate creep effect.

12. Creep effect on concrete occurs at an early stage. Due to improper camber, excess loading due to superimposed dead load and live load in the early stage, and the creep effect, the new deck can be expected to be higher than the existing deck. Based on the analysis, the MD 140 bridge (bridge no. 6032) is expected to have a two (2) to three (3) inch difference in elevation, which is also reported from the field and this differential displacement between stages could be alleviated by proper camber and scheduling on pouring.
13. To achieve better result during stage construction, 30-days waiting period is recommended between finishing the new deck pour and starting the closure pour. In this way the creep effect from both the old and new construction stages would enter a steady growth stage and the displacement gap between these two stages would be narrowed.

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