Discrepancies in Shear Strength of Prestressed Beams with Different Specifications

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Catherine French, Principal Investigator
Department of Civil Engineering
University of Minnesota

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16. Abstract (Limit: 250 words)

Although Mn/DOT inspection reports indicate that prestressed concrete bridge girders in service do not show signs of shear distress, girders rated with the *Virtis*-BRASS rating tool and Load Factor Rating (LFR) have indicated that a number of the girders have capacities lower than design level capacities. One of the reasons for the discrepancy was suspected to be conservatism of the rating methods (i.e., LFR). Other suspected reasons included potential flaws in the rating tools used by Mn/DOT (i.e., *Virtis*-BRASS software) including neglecting possible additional shear capacity parameters (e.g., end blocks). As a consequence, the rating methods have made it difficult to discern the cases for which shear capacity may be a real concern. In order to identify the reasons for the discrepancies and inconsistency in rating results relative to observed performance of the prestressed bridge girders, an analytical research program was conducted. The report provides a brief description of the models that provide the basis for the AASHTO shear design provisions and descriptions of the provisions through the 2002 AASHTO Standard specifications. This is followed by a description of the *Virtis*-BRASS rating tool, which was verified with example bridges provided by Mn/DOT. To investigate prestressed bridge girders within the inventory that might be most at risk for being undercapacity for shear, 54 girders were selected from the inventory for further evaluation. Some of the 54 girders were found to have larger stirrup spacings than required at the time of design. These girders were subsequently rated and evaluated per the 2002 AASHTO Standard Specifications to determine the adequacy of the designs based on the LFR inventory and operating rating methods. Potential sources for increased shear capacity were identified and reviewed.

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Final Report

Prepared by:

Ozer Dereli
Carol Shield
Catherine French

Department of Civil Engineering
University of Minnesota

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St. Paul, MN 55118

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Executive Summary

According to the Minnesota Department of Transportation (Mn/DOT), the first prestressed concrete bridge was built in the state in 1955. Over the years the percentage of prestressed concrete bridges has grown rapidly, as these systems have proven to perform well. The prestressed concrete bridges have been long lasting and have required little maintenance. Currently, Mn/DOT oversees more than 1,200 prestressed concrete bridges, approximately 900 of which were designed according to guidance from the 1979 Interim AASHTO Specification or earlier.

Although Mn/DOT inspection reports indicate that prestressed concrete bridge girders in service do not show signs of shear distress, girders rated with the *Virtis*-BRASS rating tool and Load Factor Rating (LFR) have indicated that a number of the girders have capacities lower than design level capacities. One of the reasons for the discrepancy was suspected to be conservatism of the rating methods (i.e., LFR). Other suspected reasons included potential flaws in the rating tools used by Mn/DOT (i.e., *Virtis*-BRASS software) including neglecting possible additional shear capacity parameters (e.g., end blocks). As a consequence, the rating methods have made it difficult to discern the cases for which shear capacity may be a real concern. In order to identify the reasons for the discrepancies and inconsistency in rating results relative to observed performance of the prestressed bridge girders, an analytical research program was conducted.

The report provides a brief description of the models that provide the basis for the AASHTO shear design provisions and descriptions of the provisions through the 2002 AASHTO Standard specifications. This is followed by a description of the *Virtis*-BRASS rating tool, which was verified with example bridges provided by Mn/DOT. To investigate prestressed bridge girders within the inventory that might be most at risk for being undercapacity for shear, 54 girders were selected from the inventory for further evaluation. Some of the 54 girders were found to have larger stirrup spacings than required at the time of design. These girders were subsequently rated and evaluated per the 2002 AASHTO Standard Specifications to determine the adequacy of the designs based on the LFR inventory and operating rating methods. Potential sources for increased shear capacity were identified and reviewed. The following paragraphs summarize these sections of the report and the findings of the investigation.

The *Virtis*-BRASS rating tool was verified using example bridges provided by Mn/DOT. One significant error related to the calculation of concrete resistance to web-shear of girders near end regions was found. The *Virtis*-BRASS software evaluated the compression in the concrete at the wrong location for use in the concrete contribution to web-shear when the centroid of the composite section was above the web-flange intersection. This error was found to conservatively cause the shear inventory rating factors to be underestimated by up to 25 percent at the critical section for shear, i.e., at "*h*/2" away from the face of the support, according to the 2002 AASHTO Standard Specifications.

From the results of NCHRP Project 12-61, *Simplified Shear Design of Structural Concrete Members*, it was shown that the 1979 Interim revisions of the AASHTO Standard Specifications did not provide reliable results for predicting shear capacity. Conversely, the 2002 AASHTO Standard Specifications provided reasonable predictions of shear capacity with a low coefficient of variation in the test to predicted shear capacity ratios, and thus was found to be a useful tool for predicting the shear capacity of prestressed concrete members.

An initial objective of this project was to develop a screening method to determine the bridge girders most at risk for being undercapacity for shear. A previous companion project, Mn/DOT Report 2007-47 by Runzel et al., (2007), had determined that some girders in the Mn/DOT inventory designed by the 1979 Interim had stirrup spacings larger than those required by the 1979 Interim. To determine how widespread this problem was, a total of 54 bridges from the Mn/DOT database, known to have shear inventory rating factors less than unity, were selected to have their designs checked using the design code indicated on the bridge plans. It was concluded that if this problem was widespread, it would not be possible to implement an easy screening method to determine girders most at risk.

A check of the bridges revealed that there were a number of girders with stirrup spacings larger than those required by the specifications in use at the time of the girder design. This was attributed to possible errors in the design tools used by Mn/DOT. However, a check of the design tools showed that it was not possible to trace the sources of error in the vertical shear designs. The versions of the design software used by Mn/DOT changed over the years, and it was not possible to locate the particular source code associated with the individual bridge girder designs.

Even though girders were found to have larger stirrup spacings than those required by the design specifications in effect at the time of the girder designs, the screening tool described in Mn/DOT Report 2007-47 was applied to the selection of 54 bridges in the Mn/DOT inventory thought to have shear inventory ratings near or less than unity. For most of the girders investigated in this study, the screening tool indicated the right trend (i.e., girders with small length-to-spacing ratios tended to have lower capacity-to-demand ratios based on the 2002 AASHTO standard). However, there were a number of girders with small length-to-spacing ratios identified by the screening tool that had large capacity-to-demand ratios, and two girders with larger length-to-spacing ratios that had shear inventory ratings below 0.9. The use of the screening tool was found to be not applicable to determine the girders most at risk of being understrength for shear at design levels.

The 54 girders selected for study were subsequently evaluated per the 2002 AASHTO Standard Specifications and rated to determine the adequacy of the designs based on the LFR inventory and operating rating methods. According to the Manual for Condition Evaluation of Bridges (1994), the inventory rating level corresponds to the HS-20 design load for LFR and indicates a live load level that can safely utilize an existing structure for an indefinite period of time. The operating rating level describes the maximum permissible live loads to which the structure may be subjected. The operating rating level is used by Mn/DOT to restrict legal or permit overloads on bridges. Of the 54 bridges selected for study from the Mn/DOT database that had shear inventory rating factors below unity, none of the bridges were found to have shear operating rating factors less than unity. The smallest value for the shear operating rating factor was 1.05.

To determine potential reserve shear capacity of prestressed concrete girders, possible parameters that could contribute additional shear capacity not generally recognized by the specifications or rating tools were identified using existing test data and available literature.

Potential parameters identified included the contribution from end blocks at the beam ends due to the thickened cross section, differences between the nominal and measured 28-day concrete strengths, increase in concrete strength with time, and effect of short shear spans (or arching action). Apart from investigating sources of conservatism in determining the shear capacity, sources of conservatism were also sought with respect to determination of shear demand. These included potential conservatism in live load distribution factors and the effect of end diaphragms on load distributions.

End blocks present at the beam ends were found to be associated with deeper sections that already had inventory rating factors generally above unity. As a consequence, even though end blocks were ignored by the rating tools, considering their effect did not have a significant impact on the results; end blocks were not found to be present in the shallower girders, which were identified as the girders with the largest risk of having inventory rating factors less than unity. Two parameters related to concrete material properties were investigated. Differences between nominal and measured 28-day concrete strengths were found to be similar in Minnesota as elsewhere with this difference already accounted for in the reliability of the AASHTO design equations. The strength of concrete was found to increase by 20% over time, which resulted in an average increase in shear rating factor on the order of 6%. Although arching action associated with short-shear spans has the potential to add significant shear capacity not accounted for in the 2002 AASHTO Standard Specifications, arching action is only appropriate when the load is applied near the support. An investigation of the effect of load position on the inventory rating factors at the critical section revealed that in many cases the critical section continued to have inventory rating factors less than unity, even when the live load was placed away from the support, thus it was determined that arching action would not result in higher inventory rating factors at the critical section.

Apart from the potential conservatism in the shear capacity determination, possible inherent conservatism in the estimation of the live load demand as associated with shear live load distribution factors, for example, was also investigated. The 2002 AASHTO Standard Specifications were found to yield less conservative distribution factors compared to the other methods reviewed. The effect of existing end diaphragms in the bridges on shear live load distribution was also investigated through the findings from literature. The literature showed conflicting results on the effect of end diaphragms on shear live load distribution factors with respect to the degree of effectiveness.

For the 54 girders studied, shear rating at the critical section (i.e., $h/2$ from the face of support) was found to be a good indicator for shear rating throughout the girder, thus, if low shear inventory rating factors (below unity) are obtained at the critical section for a girder, the rating of the girder should be checked at other points of interest throughout the span. If the girder rates adequately for shear at the critical section, it is likely that it will rate adequately throughout. Special attention should be given to sections found to have stirrup spacings in excess of $h/2$. In these cases, the transverse reinforcement contribution to shear resistance should be discounted because the potential shear crack may not be intercepted by a stirrup.

Mn/DOT should perform visual inspections to look for evidence of diagonal web shear cracking in bridges with lower shear inventory and operating ratings that also have high Heavy Commercial Average Daily Traffic (HCADT) counts. Evidence of web shear cracking would indicate that the bridge had experienced a severe shear load. It should be noted, however, that diagonal cracking that may have developed under the presence of overload, may not be visible in the absence of the overload.

Chapter 1. Introduction

1.1 Status of Bridges

As of the year 2007, 25.4 percent of bridges in the United States were considered deficient. The Federal Highway Administration (FHWA) defines a deficient bridge as one that is functionally obsolete or structurally deficient (FHWA, 2007). A functionally obsolete bridge is no longer adequate for its task even though it may be structurally sufficient. A structurally deficient bridge is one showing signs of deterioration, but still providing safe passage over the structure.

A recent study by Friedland and Small (2003) indicated viaducts within the U.S. highway infrastructure are on average 40 years old with a theoretical design life of 50 years. The average age of the over 500 bridges that have experienced collapses in the United States, has been 52.5 years, with the most common cause for collapse being flood and scour, which accounted for almost 53 percent of the incidents. Other factors were bridge overload; collisions with trucks, trains, ships or barges; or deficiencies in design, material, construction or maintenance (Wardhana and Fabian, 2003).

Bridge rating and evaluation are vital to ensure adequate performance of the nation's transportation infrastructure for public safety.

1.2 Load Rating

Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load rating is defined as the determination of the live load carrying capacity of an existing bridge using existing bridge plans supplemented by information gathered from field inspection (MCE, 1994). Rating may be conducted for each of the bridge components including the deck, and individual elements of the superstructure and substructure. Typically, only the main elements of the superstructure and not the deck or substructure are rated. The individual element with the lowest rating factor controls the rating of the whole structure. This research was focused on the load rating of the superstructure for shear.

The following is a summary of two types of load rating levels (MCE, 1994);

- **Inventory rating level:** "generally corresponds to the customary design level of stresses but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, results in a live load which can safely utilize an existing structure for an indefinite period of time" (MCE, 1994). The Inventory rating level is based upon the HS-20 design load per Load Factor Rating (LFR) and the HL-93 design load per Load and Resistance Factor Rating (LRFR).
- **Operating rating level:** "generally describes the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at Operating level may shorten the life of the bridge" (MCE, 1994). As the Operating rating level reflects the absolute maximum permissible load that can be safely carried by the bridge (Chen and Duan, 1999), it can be used to provide information necessary for posting loads or rehabilitation of the structure based on the American

Association of State Highway and Transportation Officials (AASHTO) legal loads. This rating level can also be used to allow issuance of overload permits for loads different than the AASHTO legal loads, i.e., state defined permit vehicle loads. In this report, the operating rating level is based on an HS-20 design load.

From 1931 until 1994, customary bridge design procedures were in accordance with the Allowable Stress Design (ASD) or the Load Factor Design (LFD) method of the AASHTO Standard Specifications. In 1994, the AASHTO Load Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 1994) were adopted and could be used instead of the AASHTO Standard Specifications (AASHTO, 1989). By 2007, all states started utilizing the AASHTO LRFD Bridge Design Specifications for the design of new bridges.

The practice of load rating of bridges began shortly after the publication of the *AASHO Manual for Maintenance Inspection of Bridges* in 1970. As the design methodology for bridges changed over the years, the rating methodology has also needed to change. Around 1974, Mn/DOT bridge design changed from allowable stress methods to ultimate strength methods; however, Mn/DOT continued rating bridges using allowable stress methods until approximately 1990. Between the early 1990s and 2003, Mn/DOT bridge rating was based on Load Factor Rating (LFR) as outlined in the *1994 AASHTO Manual for Condition Evaluation of Bridges* (MCE). In 2003, FHWA adopted the AASHTO Guide Manual for Load and Resistance Factor Rating of Highway Bridges (LRFR), which was developed to be consistent with the AASHTO LRFD Bridge Design Specifications. In 2005, AASHTO approved a resolution to update the LRFR Manual and adopt it as the new *Manual for Bridge Evaluation* to replace the 1994 *MCE*. Although the manual emphasizes the LRFR method, it provides rating procedures for ASD, LFR and LRFR to allow states the option of rating their existing inventory with any of these methods.

At the start of this project, Mn/DOT was primarily using LFR methods for the evaluation and rating of existing bridges in their inventory, although they were beginning the switch to LRFD methods; hence, this research focuses on LFR ratings. Additionally, ratings may be performed by either experimental or analytical means. This study covers analytical ratings only.

1.2.1 Load Factor Rating (LFR)

In the load rating of bridge members, two methods for checking the capacity of the members are provided in the *Manual for Condition Evaluation of Bridges (MCE)*, 1994, Second Edition, the Allowable Stress method and Load Factor Design (LFD) method.

The Load Factor Rating (LFR) method, used by the Minnesota Department of Transportation (Mn/DOT) at the time of this investigation, specifies two levels of capacity ratings as defined in the previous section: inventory and operating. The rating factor for Load Factor Design (LFD) is as follows:

$$RF = \frac{\phi R_n - \gamma_D D}{\gamma_L LL(1+I)} \tag{1.1}$$

where R_n is the nominal capacity, according to the AASHTO Standard Specifications 1996, with interims through 2002, ϕ is the strength capacity reduction factor, γ_D and γ_L are the load factors for dead and live loads, respectively, D and LL are the dead and live load effects, respectively; and I is the impact factor for live load.

The load factors given in Table 1.1 differ for the live load between the inventory and operating ratings. The dead load effects are computed in accordance with the conditions existing at the time of analysis. When the inventory rating load factor is used in Eqn. (1.1), the shear rating factor (RF) is exactly equal to unity when the design shear capacity of the element is equal to the factored shear load in the member according to the 2002 *AASHTO Standard Specification* (i.e. Inventory $RF = 1$ when $\phi V_{n,STD2002}=V_u$). Hence, inventory rating factors of one and above are an indication that the component meets the *2002 AASHTO Standard Specifications*.

The live load to be used in the basic rating equation (1.1) should be the HS20 truck or lane loading as defined in the AASHTO Design Specifications and shown in Figure 1.1.

1.2.2 Rating Aids for LFR

The rating aids listed below provide guidance and support for LFR:

- *AASHTO Standard Bridge Design Specifications* (1996) with interims through 2002 and the *AASHTO Manual for Condition Evaluation of Bridges* (1994)

- Bridge Rating and Analysis of Structural Systems (Brass) (2007)

- AASHTO *Virtis* Version 5.6 (2007)

The *Manual for Condition Evaluation of Bridges* based on the *AASHTO Standard Bridge Design Specifications* is the customary reference used to rate bridges in LFR. BRASS (Wyoming Department of Transportation, 2007) is a program that assists in the rating of highway bridge girders according to the AASHTO Specifications, with the necessary bridge information (i.e., material properties, bridge geometry) provided by the *Virtis* (2007) database. Detailed information on BRASS and *Virtis* is given in Chapter 3.

1.3 Problem Statement

As of the year 2007, 14.4% of prestressed concrete bridges nationwide were considered either structurally deficient or functionally obsolete. Minnesota alone contributed 158 out of the 16,000 nationwide deficient prestressed concrete bridges (i.e., 1%) to these statistics. Of the 158 deficient prestressed concrete bridges in the State of Minnesota, roughly half are considered to be structurally deficient, and the other half are considered functionally obsolete (FHWA, 2007).

The Mn/DOT prestressed concrete bridge inventory, summarized in Chapter 4, indicates that there were 1244 bridges built in the State of Minnesota between 1929 and 2005. Of the 1244 built between 1929 and 2005, 59% of them were likely to have been designed using the pre-1983 AASHTO Standard Specifications. Potential shear design flaws identified in the pre-1983 AASHTO Standard Specifications included designing for shear at the quarter points (rather than at the critical section near the support) and continuing that stirrup spacing to the supports. In addition, the AASHTO 1979 Interim Specifications did not place a maximum limit on the amount of transverse reinforcement that could be used to resist shear. Both issues could lead to unconservative designs. The first issue because the shear demand at the quarter point is likely smaller than the shear demand at the current day critical section ($h/2$ away from the face of the support), and the second issue because there was no check for concrete diagonal compression failures which can occur when there is no limit on transverse reinforcement. It was further shown by the literature that the AASHTO 1979 Interim Specifications provided a lower reliability for shear design as discussed in Chapter 2.

Although there have been no visible signs of shear distress observed, many prestressed concrete girders in the Mn/DOT bridge inventory have shear inventory rating factors less than unity. Potential reasons for this discrepancy have been attributed to possible flaws in the rating tools used by Mn/DOT (i.e., *Virtis* software), additional shear capacity neglected in the capacity calculations using the rating tools (e.g., presence of end blocks, concrete strength increase with time) and an absence of loads on the structure in excess of those required to produce cracking.

The shear inventory rating predicted by the *Virtis* rating software used by Mn/DOT is sometimes less than unity, especially for bridges designed between 1961 and 1992. As a result, shear controls the rating of some bridges, often providing a much lower rating factor than the moment rating factor. In some cases, even the operating ratings are so low that they present problems when attempting to route overweight permit trucks over the bridges. Other states have reported similar problems (Colorado DOT, 1995).

The primary objective of this study was to resolve the discrepancies between the shear inventory rating and design methods. The apparent conservatism of the shear rating methods (i.e., LFR) results in indications of potential shear problems in many girders which show no visible signs of distress. Consequently, inventory rating methods make it difficult to discern the cases for which insufficient shear capacity may be a real concern. In this study, the shear design and inventory rating methods were compared using past testing and research. Possible parameters that contribute additional shear capacity in girders were identified using existing test data and available literature. Shear operating ratings were also investigated.

Recommendations were developed to give guidance on the appropriate rating tools to evaluate the shear capacity. In addition, recommendations were sought to provide screening and evaluation tools to be applied by Mn/DOT personnel with minimal effort to discern those cases for which the shear capacity rating would warrant further investigation.

1.4 Organization of the Document

Chapter 2 summarizes the methods and equations by which the investigated bridge girders were designed and their shear capacities were calculated. The shear models described include the AASHTO 1979 Interim Specifications, 2002 AASHTO Standard Specification, and the Strut-and-Tie Method of 2004 AASHTO LRFD.

Chapter 3 describes the *Virtis*-BRASS software used by Mn/DOT in rating prestressed concrete bridge girders and includes verification of the software with provided bridge examples from the Mn/DOT inventory and the PCI Bridge Design Manual (PCI, 2003). Errors identified in the software are summarized.

Chapter 4 presents the selection of a subset of Mn/DOT prestressed bridge girders used for this work. Each girder selected was redesigned for shear in accordance with the code believed to be in effect at time of design. The calculated spacings of shear reinforcement were compared to the provided spacings and originally undercapacity girders in the selected subset were identified.

In Chapter 5, the calculated ratio of the shear capacity to design shear demand and the shear inventory and operating rating factors for each girder in the subset are presented. The calculations were made in accordance with the 2002 AASHTO Standard Specifications. The girders with low design shear capacity-to-demand ratios and shear inventory ratings are identified. The results of a study relating the bridge geometry to the shear ratings (and ratios of shear capacity to demand) are summarized.

Chapter 6 summarizes an investigation of possible sources of additional shear capacity neglected by the 2002 AASHTO Standard Specifications. Using available data from the literature, additional sources of capacity investigated included effects of end blocks, concrete strength gain due to differences between nominal and measured strengths and aging, and arching action near the end regions of girders. Differences between shear live load distribution factors given by different provisions and research studies and the effect of end diaphragms on distribution factors from available data in the literature are also presented in this chapter.

Chapter 7 presents a summary of the study and the primary recommendations.

Four appendices augment the information in the main report. Appendix A provides sample calculations regarding shear design and capacity in accordance with the AASHTO Standard Specifications (1961, 1965-1969, 1973-1977-1979 Interim, 1983 and 2002) presented in Chapters 4 and 5. Appendix B presents the results of a field inspection of six of the bridges with shear inventory rating factors below unity that was performed by Mn/DOT personnel. Appendix C presents the concrete core test data from the literature used in Chapter 6, and Appendix E contains sample calculations for the shear live load distribution factors using the simplified methods presented in Chapter 6.

Chapter 2. Models for Shear Capacity and AASHTO Shear Provisions

2.1 Introduction

Classical beam theory, in which plane sections are assumed to remain plane, provides an accurate, simple, and effective model for designing a member to resist flexure, which is usually a primary consideration in the design of long-span prestressed concrete members. The ability of a section to resist shear or the combination of shear and flexure cannot be predicted with corresponding accuracy. The shear failure of prestressed concrete beams is distinctly different from flexural failure. In the case of shear, beams may fail abruptly without sufficient advance warning. Due to the difficulties of predicting shear behavior, it has been a major area of research in reinforced and prestressed concrete structures for decades.

This chapter presents a brief description of the models that provide the basis for the AASHTO shear design provisions discussed in this report. The models range from empirical sectional models to conceptual models. The two empirical sectional models discussed in this chapter are associated with the AASHTO Standard Specifications (2002) and the 1979 Interim Specifications (AASHTO, 1979). The conceptual model discussed here is the Strut-and-Tie Model (STM) of the 2004 AASHTO LRFD which is based on a truss analogy that can be applied to disturbed regions in which it cannot be assumed that plane sections remain plane.

Additionally, the shear provisions from the AASHTO Specifications in effect between 1961 and 2002 are compared to each other.

2.2 Shear Transfer Mechanisms in Prestressed Concrete Beams

Mechanisms for shear transfer in cracked, prestressed concrete beams consist of shear transfer in the compression zone, friction on the crack surfaces from aggregate interlock, dowel action of the reinforcing steel, shear transfer from the transverse steel, and the vertical component of the force in the draped prestressing strands (ASCE, 1973). Figure 2.1 shows the basic mechanisms of shear transfer, and a description of these mechanisms follows.

In a cracked concrete member subjected to flexure, the uncracked compression zone above the neutral axis contributes to shear resistance. The depth of the compression zone limits the magnitude of that shear resistance.

At crack locations, in-plane shear transfer is accomplished through the local roughness of the aggregates located along the crack surfaces which inhibit slip. This resistance mechanism is also known as aggregate interlock. The contribution of interface shear transfer to shear strength is dependent on the crack width and aggregate size. As the crack width decreases and the aggregate size increases, the magnitude of the resistance increases.

As the longitudinal reinforcing bars intersect the crack planes, dowel action of the reinforcement provides shear resistance. The contribution of dowel action to shear resistance is dependent upon the concrete cover beneath the longitudinal reinforcement and the ability of transverse reinforcement to restrain the vertical displacements of the longitudinal reinforcement at the inclined cracks.

After diagonal cracking occurs, tensile stresses develop in the shear reinforcement providing a path for stress transfer across the cracked surface. The shear reinforcement also restrains the growth of inclined cracks which improves the aggregate interlock and stress transfer across the cracked surface. The presence of shear reinforcement changes the relative contributions of the different shear resisting mechanisms. The shear resistance provided by transverse reinforcement is a function of the cross-sectional area, yield strength, and distribution of the steel.

In beams with small shear-span to depth ratios, arching action is a dominant shear transfer mechanism. This mechanism is not discussed in this chapter, because the sectional methods in the codes are not based on arching action.

2.3 AASHTO Standard Shear Provisions

The first edition of the AASHTO Standard Specifications was published in 1931, and was followed by revised editions in 1935, 1941, 1944, 1949, 1953, 1957, 1961, 1965, 1969, 1973, 1977, 1983, 1989, 1992, 1996, and 2002. The FHWA adopted the AASHTO LRFD specifications for the design of all new bridges after 2007; however, the 2002 Standard Specifications are still applicable for the evaluation, rating, maintenance and rehabilitation of existing structures.

Shear provisions in the AASHTO Standard Specifications, with the exception of the Strut-and-Tie model of the AASHTO LRFD, basically superimpose the shear carried by the concrete after cracking, V_c (which is incidentally the same value for shear assumed to initiate diagonal cracking in the concrete), with the shear taken by the transverse shear reinforcement at yielding, V_s, and the shear taken by the vertical component of the force in the draped prestressing strands, V_p, to determine the shear capacity of the section:

$$V_n = V_c + V_s + V_p \tag{2.1}$$

In the AASHTO Standard Specifications shear provisions, V_c is generally based on an empirical equation. Figure 2.2 shows the stirrup contribution to shear capacity

$$V_s = \frac{A_v f_{sy}\, jd \cot\theta}{s} \tag{2.2}$$

where the angle of the inclined shear cracks, θ, is implicitly conservatively defined as 45° in the AASHTO Standard Specifications shear provisions.

The procedures for determining the concrete and shear reinforcement contributions to shear resistance, and the method of handling the vertical component of the prestressing force, V_p, have changed over the years in the AASHTO Specifications. For simplicity, those specifications with similar shear capacity equations are identified herein with the most recent date of the specification. The AASHTO shear provisions between 1961 and the 1979 Interim, which used similar equations for shear capacity calculations, are presented as the 1979 Interim Specifications and, similarly, shear provisions between the 1980 Interim and 2002 Standard Specifications are presented as the 2002 AASHTO Standard Specifications as given in the following sections.

The stirrups used to carry the vertical shear are also used for horizontal shear resistance. Editions of the Standard Specifications prior to 1996 required that all vertical shear reinforcement in a beam extend into the cast-in-place slab to be used as reinforcement to resist horizontal shear.

Generally, the AASHTO Specifications require that ϕ times the factored horizontal shear capacity, ϕV_{nh} or ϕv_{nh} in terms of stress, (where ϕ is the shear strength reduction factor as given in Table 2.1), must exceed the factored vertical shear demand, V_u or v_u in terms of stress, respectively.

2.3.1 AASHTO 1979 Interim Specifications

The "Tentative Recommendations for Prestressed Concrete" published in 1958 by ACI-ASCE Joint Committee 323, serve as the basis for the shear design provisions found in the 1979 Interim (PCI, 2003).

The 1979 Interim is based on a truss model, with an additional concrete contribution term for shear resistance. In the 1979 Interim, the following equation is given for computing the concrete contribution to shear strength:

$$V_c = 0.06 f_c^{'} b_w jd \leq 180 b_w jd \tag{2.3}$$

where j is the ratio of distance between the centroids of the compression force and tension steel to the effective depth, d, at ultimate flexural capacity. According to the Precast/Prestressed Concrete Institute (PCI) Bridge Design Manual, a value of 0.9 can be conservatively used to estimate j for typical sections (PCI, 2003).

As shown in Eqn. (2.3), concrete strengths above 3000 psi do not increase the value of V_c due to the maximum limit.

The required area of web reinforcement is calculated using the expression given in Article 1.6.12 of the 1979 Interim:

$$A_v = \frac{\left(\frac{V_u}{\phi} - V_c\right) s}{2 f_{sy} jd} \tag{2.4}$$

where V_u is the factored shear demand at the section and ϕ is the shear strength reduction factor for shear, given as 0.9.

Using Eqns. (2.1) and (2.4) and ignoring the contribution from the vertical component of the prestressing force, the shear contribution of web reinforcement can be expressed as

$$V_s = \frac{2 A_v f_{sy} jd}{s} \tag{2.5}$$

where A_v is the area of the web reinforcement at the cross section; f_{sy} is the yield strength of the web reinforcement, which may not exceed 60 ksi, and s is the center-to-center longitudinal spacing of the web reinforcement in the vicinity of the cross section.

The factor of 2 in the shear reinforcement term represents the assumed benefit of prestressed concrete (PCI, 2003). This factor of 2 reflects an angle of 26.6° for inclined shear cracks with a horizontal projection equal to twice the effective shear depth, *jd*, thus more stirrups are assumed to cross a given crack compared to the 45° truss model of the 2002 AASHTO Standard Specifications which is discussed in Section 2.5.

The required minimum area of web reinforcement in the 1979 Interim is twice that required by the 2002 AASHTO Standard Specifications,

$$A_v \geq \frac{100 b_w s}{f_{sy}} \tag{2.6}$$

The spacing of web reinforcement must not be greater than 0.75*h*, where *h* is the total height of the section.

The 1961 - 1979 Interim AASHTO Standard Specifications specify the use of the classical strength of materials approach to calculate the horizontal shear stress at the interface

$$v_u = \frac{V_u Q_c}{I_c b_v} \tag{2.7}$$

where V_u is the factored vertical shear force at the section, b_v is the section width at the fiber being considered, Q_c is the first moment of area above the location being considered, and I_c is the moment of inertia of the entire composite cross section.

In the 1961 – 1979 Interim and 1983 AASHTO Standard Specifications, the stirrup spacing was limited to no more than four times the average thickness of the slab or 24 in. The minimum total area of the vertical ties was specified as the area of two No. 3 bars spaced at 12 in. which corresponds to 21.8 in. spacing for two No. 4 bars.

There is no maximum limit placed on V_s in the 1979 Interim. The ability to assume an unlimited amount of transverse reinforcement to resist shear can result in unconservative designs using the 1979 Interim because the concrete may undergo diagonal crushing before the capacity of the transverse steel is realized. In the 1979 Interim, there is no check to prevent the concrete diagonals from crushing.

There is no specified critical section for shear in the 1979 Interim, however the specification in Article 1.6.12 allows for the use of the same stirrup spacing that is calculated at the quarter points of the span to be extended to supports. The part emphasizing the critical sections for simply-supported bridges in Article 1.6.12 of the 1979 Interim is given below:

"*The critical sections for shear in simply supported beams will usually not be near the ends of the spans where the shear is maximum, but at some point away from the ends in a region of high moment.*

For the design of web reinforcement in simply supported members carrying moving loads, it is recommended that shear be investigated only in the middle half of the span length. The web reinforcement required at the quarter points should be used throughout the outer quarters of the span."

For long span members, this suggestion could result in particularly unconservative designs.

2.3.2 2002 AASHTO Standard Specification

Similar to the 1979 Interim, the shear provisions in the 2002 Standard Specifications are also based on a 45° truss model, with an additional concrete contribution term for shear resistance. However, the concrete term in the 2002 Standard Specification is more rational than the one found in the 1979 Interim. In the 2002 AASHTO Standard Specifications Section 9.20.2, the shear strength provided by the concrete is a function of the mode of shear failure that controls: flexure-shear failure or web-shear failure. As shown in Figure 2.3, flexure-shear cracks develop where moment is large and shear exists, and web-shear cracks occur in regions of high shear where the principal tensile stress reaches the tensile strength of the concrete. Web-shear cracks typically develop in thin-webbed members (i.e., I- or T-shaped sections).

A footnote to Article 9.20, the 2002 AASHTO Standard Specifications permit web reinforcement to be designed using the method presented in the 1979 Interim.

2.3.3 Concrete Contribution – Flexure-Shear Case

Flexure-shear cracking starts as a flexure crack on the tension face of a beam. As it extends up into the web, it develops into a diagonal shear crack. This can occur at a much lower principal tensile stress than that causing a web shear crack, because of the tensile stress concentration at the tip of the crack (PCA, 2002). Shear capacity controlled by flexure-shear cracking is the sum of the shear due to the load required to initiate flexural cracking plus an increment which transforms the flexural crack into an inclined crack. The flexure-shear cracking capacity (in pounds) is given in the 2002 Standard Specifications (Eqn. 9-27) as

$$V_{ci} = 0.6\sqrt{f_c^{'}}b_w d + V_d + \frac{V_i\, M_{cr}}{M_{\max}} \geq 1.7\sqrt{f_c^{'}}b^{'}d \tag{2.8}$$

where $f_c^{'}$ is the specified 28-day concrete strength (psi), b_w is the web width, d is the distance from the extreme compression fiber to the centroid of the prestressing force or to the centroid of the negative moment reinforcement for precast girder bridges made continuous, but d need not be taken less than $0.8h$ (where h is the total height of the section), and M_{max} is the maximum factored moment at the section due to externally applied loads which can be defined as $M_{max} = M_u - M_d$ where M_u is the factored bending moment at the section and M_d is the bending moment at the section due to unfactored dead load, V_i is the factored shear at the section due to the externally applied loads occurring simultaneously with M_{max} and can be defined as $V_i = V_{mu}-V_d$

where V_{mu} is the factored shear force occurring simultaneously with M_u and V_d is the shear due to unfactored dead load (sum of unfactored selfweight and unfactored superimposed dead load for composite sections) at the section under consideration, M_{cr} is the moment due to externally applied loads (after dead load) required to cause flexural cracking of the section and is given in the code as

$$M_{cr} = \frac{I_c}{y_{bc}}(6\sqrt{f_c^{'}} + f_{pe} - f_d) \tag{2.9}$$

where I_c is the gross moment of inertia of the composite section; y_{bc} is the distance from the centroid of the gross composite section to the extreme tension fiber of the precast beam; f_{pe} (psi) is the compressive stress in concrete due to effective pretension forces only (after allowance for all prestress losses) at the extreme fiber of the section where tensile stress is caused by externally applied loads; and f_d (psi) is the tensile stress due to unfactored (selfweight and superimposed) dead load, at the extreme fiber of the section where tensile stress is caused by externally applied loads.

2.3.4 Concrete Contribution - Web-Shear Case

The web-shear capacity is reached when the principal tensile stress reaches the tensile strength of the concrete and cracking occurs. The resistance to web-shear cracking is due to the tensile strength of the concrete in relation to the principal tension which is affected by the compressive forces in the section due to the prestressing force and the applied loads. If draped strands are used, the vertical component of the prestressing force, V_p, will also resist shear. The expression for web-shear strength usually governs near the supports for heavily prestressed beams with thin webs, especially when the beam is subject to large concentrated loads near the supports.

The equation for concrete resistance to web-shear is given by the 2002 AASHTO Standard Specifications (Eqn. 9-29) as

$$V_{cw} = (3.5\sqrt{f_c^{'}} + 0.3 f_{pc})b_w d + V_p \tag{2.10}$$

where f_{pc} (psi) is the compressive stress in the concrete (after allowance for all prestress losses) at the centroid of the cross section resisting externally applied loads or at the junction of web and flange when the centroid lies within the flange. In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section, or at the junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by the precast member acting alone.

While the vertical component of the prestressing, V_p, adds to the shear strength for web-shear cracking, V_{cw}, there is no effect of the same vertical component on the shear strength for flexure-shear cracking, V_{ci}. Thus, draped strands increase the concrete resistance to web-shear but can actually decrease the flexure-shear cracking load by decreasing the effective depth, d. Because web-shear generally controls near the support, the V_p contribution is important. However, away from the support, where flexure-shear generally controls, there are likely no draped strands (and hence no V_p).

2.3.5 Web Reinforcement Contribution

In Section 9.20.3 of the 2002 AASHTO Standard Specifications, the web reinforcement contribution is based on the conservative assumption of a 45° crack orientation; therefore, the horizontal projection of the crack is taken as *d*. The resulting equation is conservative for prestressed members, because the effect of prestressing causes the diagonal cracking to form at a shallower angle, thus intercepting more stirrups than predicted by the 45° model. With the assumption that the stirrups yield at failure, the 2002 AASHTO Standard Specifications (Eqn. 9-30) gives the shear resisted by the stirrups as

$$V_s = \frac{A_v f_{sy} d}{s} \tag{2.11}$$

where A_v is the area of the web reinforcement at the cross section; s is the center-to-center longitudinal spacing of the web reinforcement near the cross section, and f_{sy} is the yield strength of the nonprestressed web reinforcement. The design yield strength of web reinforcement is limited to 60 ksi.

The 2002 AASHTO Standard Specifications limits Eqn. (2.11) as follows

$$V_s \leq 8\sqrt{f_c^{'}} b_w d \tag{2.12}$$

This limit is imposed to avoid crushing of the concrete and to guard against excessive crack widths.

A minimum amount of web reinforcement is prescribed to provide some ductility except for the case where V_u is less than half ϕV_c where shear reinforcement may be omitted. The minimum amount of transverse reinforcement is specified by the 2002 AASHTO Standard Specifications (Eqn 9-31) as

$$A_v \geq \frac{50 b_w s}{f_{sy}} \tag{2.13}$$

Where f_{sy} is the yield strength of the vertical reinforcement in psi.

To ensure that each crack is intercepted by a vertical stirrup, maximum stirrup spacing, s_{max}, is required in the 2002 AASHTO Standard Specifications as the smaller of 0.75h or 24 in., and if $V_s > 4\sqrt{f_c^{'}} b_w d$, the maximum spacing, s_{max}, is to be reduced by half.

2.3.6 Horizontal Shear

In the 1983 - 2002 Standard Specifications, a simplified model that assumes constant shear through the effective depth of the section was utilized for calculating the horizontal shear demand. The simplified beam method is summarized as

$$v_u = \frac{V_u}{b_v d_v} \tag{2.14}$$

where d_v is the distance from the resultant compression force for the composite section to the centroid of longitudinal tension reinforcement, but need not be taken less than $0.8h$ for prestressed concrete members, where h is the height of the composite section.

According to the 1989 - 2002 AASHTO Standard Specifications, the tie area shall not be less than $50b_v s/f_y$, and tie spacing, s, shall not exceed four times the least web width of the support element, nor 24 in. for horizontal shear.

2.3.7 Critical Section

The location of the critical section for vertical shear for a prestressed member is taken at a distance $h/2$ from the face of the support. If the cross section of interest is within the transfer length region, a reduction in effective prestressing force must be considered when computing V_{cw}. The code assumes the prestressing force varies linearly from zero at the end of the tendon to the effective prestressing force at the end of the transfer length, which is given as 50 strand diameters.

The AASHTO Standard Specifications do not identify the location of the critical section for horizontal shear. According to the PCI Bridge Design Manual (2003), the critical section for horizontal shear may be taken as the same location as the critical section for vertical shear. Generally, tenth-point intervals along the span are also used to design for horizontal shear (PCI, 2003). This may be necessary to ensure that adequate reinforcement is provided for horizontal shear because the spacing or the area of the web reinforcement for vertical shear, which is extended into the deck and used for horizontal shear reinforcement, may vary along the span (PCI, 2003).

2.4 Summary of Differences in the Shear Provisions of the AASHTO Standard Specifications

Differences related to the demand and shear capacity of prestressed concrete bridges designed between 1961 and 2002 are presented in this section. Specifically, the comparison was made within the following published AASHTO Standard Specifications by year: 1961, 1965, 1969, 1973, 1977, 1979 Interim, 1980 Interim, 1983, 1989, 1992, 1996, and 2002. The main parameters that affect the design for shear in the AASHTO provisions are listed below, and Table 2.1 lists the variables and equations associated with the specific editions of the code.

- Load Factors (Live and Dead Load)
- Shear Strength Reduction Factors,
- Live Load Distribution Factors (LLDF)
- Effective Flange Width, b_{eff}
- Prestress Losses for Pretensioned Members
- Computation of j
- Limits on Effective Depth, d
- Shear Strength Provided by Concrete, V_c
- Shear Strength Provided by Web Reinforcement, V_s

- Limits for Web-Crushing Strength, $V_{s,max}$
- Minimum Area of Web Reinforcement, $A_{v,min}$
- Maximum Spacing of Web Reinforcement for Vertical Shear,
- Calculation of V_p
- Shear Strength Provided by Vertical Component of the Force in the Draped Prestressing Strands

Some of the parameters listed in Table 2.1 were assumed or inferred from the definitions or descriptions given by the AASHTO shear provisions. Between 1961 and 1969, the shear strength reduction factor was not specified since the ASD method was utilized. In this study it was assumed to be unity for those editions because the associated load factors were relatively high in comparison to the AASHTO shear provisions after 1969. Additionally, the AASHTO shear provisions between 1961 and 1973 defined V_u as "shear due to ultimate load and effect of prestressing." Thus, for the bridge girders designed by the shear provisions between 1961 and 1973 (inclusive), the shear demand was inferred to be reduced by the vertical component of the prestressing strand (i.e., V_u-V_p). This definition changed in the 1977 shear provisions to "the total design shear force at [the] section," where no deduction of V_p from V_u was assumed for the 1977 and 1979 Interim based on this definition.

The horizontal shear capacity, v_{nh}, as defined in the 1961-2002 AASHTO Standard Specifications depends on the interface conditions and amount of transverse reinforcement provided across the joint. A comparison of AASHTO horizontal shear provisions is shown in Table 2.2.

Over the years, AASHTO Standard Specifications have documented the acceptability of using alternate design methodologies. As an example, starting with the 1973 AASHTO Standard Specifications, a footnote was provided in the shear design section for prestressed concrete members that accepted the ACI 318-71 method for the design of web reinforcement as an alternate; this is the same method specified in the 2002 AASHTO Standard Specifications to calculate the shear capacity for prestressed concrete members. Similarly, starting with the 1983 AASHTO Standard Specifications, web reinforcement could be designed using the method presented in the 1979 Interim.

2.5 Strut-and-Tie Method

As described in the 2004 AASHTO LRFD Specification, the Strut-and-Tie Method (STM) is used principally in regions of concentrated forces, near supports and geometric discontinuities to determine concrete proportions and reinforcement quantities and patterns based on compression struts provided by the concrete, (tension ties provided by reinforcement, and the geometry of nodes at their points of intersection. This method is best-suited for regions of the member where plane sections can not be assumed to remain plane (i.e., D- or disturbed regions). Figure 2.4 shows an example of the distribution of D regions in a frame. In other regions (i.e., B-, Bernoulli regions), the strain distributions can be assumed to vary linearly through the section depth, and the response of the concrete member will be principally through beam action.

The STM provides insight regarding the flow of forces in disturbed regions. All stresses are condensed into compression and tension members joined by nodes shown in Figure 2.5. Article 5.6.3.1 of the AASHTO LRFD states that the STM should be considered when the distance between the centers of the applied load and the supporting reaction is less than approximately twice the member depth.

Per 2004 AASHTO LRFD Article 5.6.3.2, the factored resistance, P_r, of struts and ties shall be taken as that of axially loaded components:

$$P_r = \phi P_n \tag{2.15}$$

where P_n is the nominal resistance of strut or tie (kips), and where ϕ is the resistance factor for tension and compression specified in Article 5.5.4.2.

2.5.1 Strength of Ties

The strength of the ties depends directly on the type and strength of reinforcement used. The tie strength is given by the 2004 AASHTO LRFD 5.6.3.4.1-1 as

$$P_n = f_y A_{st} + A_{ps}\left(f_{pe} + f_y\right) \tag{2.16}$$

where f_y is the yield strength of mild steel longitudinal reinforcement; A_{st} is the total area of mild steel reinforcement in the tie, A_{ps} is the area of prestressing steel in the tie, and f_{pe} is the effective prestress. According to the 2004 AASHTO LRFD, the second term in Eqn. (2.16) is intended to ensure that the prestressing steel does not yield, thus a measure of control over unlimited cracking is maintained. However, it acknowledges that the stress in the prestressing elements will be increased due to the strain that will cause the concrete to crack. The increase in stress corresponding to this action is arbitrarily limited to the same increase in stress that the mild steel would undergo. In the absence of mild steel, f_y may be taken as 60 ksi for the second term of the equation. Additionally, the ties must be anchored in accordance with 2004 AASHTO LRFD Article 5.11 to ensure satisfactory transfer of the tension force to the node regions.

2.5.2 Strength of Struts

The nominal axial resistance of an unreinforced strut is given by the 2004 AASHTO LRFD as

$$P_n = f_{cu} A_{cs} \tag{2.17}$$

where f_{cu} is the limiting concrete compressive stress and A_{cs} is the effective cross-sectional area of the strut determined from consideration of the available concrete area and the anchoring or bearing conditions at the ends of the strut.

The limiting compressive stress in struts is given as

$$f_{cu} = \frac{f_c^{'}}{0.8 + 170\varepsilon_1} \leq 0.85 f_c^{'} \tag{2.18}$$

where ε_1 is the principal tensile strain in the cracked concrete, and is taken as

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s \tag{2.19}$$

where α_s is the smallest angle between the strut and adjoining ties, and ε_s is the tensile strain in the concrete in the direction of the tensile tie. According to 2004 AASHTO LRFD Section C5.6.3.3.3, if the concrete is not subjected to principal tensile strains, ε_1, greater than about 0.002, the concrete in the strut can resist a compressive stress of $0.85 f_c^{'}$, i.e., the limit for the regions of the strut not crossed by or joined to ties. The reinforcing bars of a tie are bonded to the surrounding concrete. If the reinforcing bars are to yield in tension, there should be significant tensile strains imposed on the concrete. As these tensile strains increase, f_{cu} decreases. The expression for ε_1 is based on the assumption that the principal compressive strain, ε_2, in the direction of the strut is equal to 0.002 and that the tensile strain in the direction of the tension tie equals ε_s.

As shown in Eqns. (2.18) and (2.19), as ε_s increases, ε_1 increases, and f_{cu} decreases. Likewise, as α_s decreases, $\cot^2 \alpha_s$ and ε_1 increase, and therefore f_{cu} decreases. In the limit, no compressive stresses would be permitted in a strut that is superimposed on a tension tie, i.e., α_s=0, a situation that violates compatibility. If the member is prestressed, ε_s can be taken as zero until the concrete precompression is overcome.

2.5.3 Strength of Nodes

In the absence of effective confining reinforcement, the concrete compressive stresses should not exceed $0.85 f_c^{'}$ in nodal regions bounded by struts and bearing areas, $0.75 f_c^{'}$ in nodal regions anchoring only one tension tie, and $0.65 f_c^{'}$ in nodal regions anchoring ties in more than one direction. The reduced stress limits on nodes anchoring ties are based on the detrimental effect of the tensile straining caused by these ties.

The STM is strictly an equilibrium model and is based on the lower bound theorem of plasticity. In other words, there may exist other load paths which could carry a greater load provided that geometry and strength requirements are satisfied. When the STM is used for design, the components of the model should be proportioned so that the ties will fail prior to the struts to provide ductility at failure.

2.6 Evaluation of AASHTO Shear Provisions, NCHRP Report 549

The objective of the National Cooperative Highway Research Program (NCHRP) Project 12-61 *Simplified Shear Design of Structural Concrete Members* was to evaluate the various shear design methods in existence and to develop proposed simplified shear design provisions that could ultimately replace the current AASHTO LRFD shear provisions. As a part of Project 12-61, a comprehensive database of shear tests on both regular reinforced and prestressed concrete beams was complied. This database was published in the appendices of NCHRP Report 549 (Hawkins et al., 2005).

The results from 743 shear tests on prestressed concrete members were contained in the database. A smaller subset of the database consisting of 85 prestressed members was used to evaluate various shear design provisions. This subset consisted of members with shear reinforcement that were considered to have properties similar to members used in practice.

Figure 2.6 shows the distribution of member properties of the subset of data. The report stated that all members in the database were selected specifically to avoid short shear spans, shallow depths, and heavily reinforced members. Furthermore, members were removed so that the final database consisted of a relatively evenly weighted set considering section shape, depth, concrete strength, and strength of shear reinforcement. Any members that possibly failed in flexure were also excluded from this database. However, further investigation revealed that the compiled database included specimens that had short shear span to depth ratios, i.e., $a/d < 2.5$. Hawkins et al. (2005) mentioned this and stated that 16 prestressed concrete members having a/d ratios of less than 2.4 were not excluded from the compiled database in order to include data for large high strength concrete girders tested very recently. As can be seen from Figure 2.6, out of 85 prestressed members there were 11 specimens that had a/d ratios between 1.5 and 2.0, and 13 specimens with a/d ratios between 2.0 and 2.5. This indicates that arching action may have contributed to the shear capacity in those members which in turn might have provided excessive shear capacity resulting in overly conservative test-to-predicted shear strength ratios.

In the appendices of Report 549, the results compiled from the select database were compared to the predictions of several design codes, and shear design methods available in the literature. Among those shear design approaches, the comparisons to the 2002 AASHTO Standard and 1979 AASHTO Interim Specifications are summarized in this section. The STM was not discussed in the appendices of Report 549, so there is no discussion of the STM in this section.

The test-to-predicted shear strength ratios based on the 2002 AASHTO and 1979 AASHTO Interim Standard Specifications are compared in Table 2.3. The mean was higher and the coefficient of variation was lower for the 2002 Standard Specifications in comparison with the 1979 Interim, suggesting that the 2002 Standard Specifications provided conservative results relative to the 1979 Interim Specifications.

For the 1979 Interim, the test-to-predicted shear strength ratio mean was 1.09 with a coefficient of variation of 0.383 thus a significant number of members had a V_{test}/V_{pred} ratio below unity. Figures 2.7 and 2.8, obtained from the parametric study published in the appendices of Report 549, help to explain the unconservative issues associated with the 1979 Interim which was most prevalent for lower f_c' and higher stirrup reinforcement ratios, $\rho_v f_{sy}$. The latter resulted because the 1979 Interim code did not place a limit on the maximum amount of shear reinforcement that could be assumed in design to carry shear, even though the concrete crushing strength of the diagonals physically limits the amount of transverse reinforcement that can be developed in the transmission of vertical shear. If this limit is exceeded, the concrete crushing causes a brittle failure at a load smaller than that anticipated. Members with a low concrete compressive strength were particularly susceptible to this type of failure. One of the reasons members with higher concrete compressive strengths were less susceptible was that the 1979 Interim ignored any associated increase in V_c with concrete compressive strengths above 3000 psi. Consequently, members with higher concrete compressive strengths may not have required the transverse reinforcement to yield and were better able to provide resistance to diagonal compression failure.

All of the members with a test-to-predicted ratio below 0.5 in Figures 2.7 and 2.8 were those with $f_c^{'}$ below 7 ksi and $\rho_v f_{sy}$ greater than 1,800 psi. Unlike the 1979 Interim, the 2002 Standard Specifications had a limit on the shear strength contribution from shear reinforcement, V_s, as summarized in Section 2.3.5.

2.7 Mn/DOT Report 2007-47 (University of Minnesota Tests)

Two shear capacity tests were performed at the University of Minnesota using the two ends of an 88 ft. long bridge girder removed from Mn/DOT Bridge No. 73023 to experimentally determine whether this bridge girder, which was designed using the 1979 Interim provisions, provided sufficient shear capacity.

The tested bridge girder was 54 in. deep, had a nominal concrete compressive strength of 6 ksi, and came from a bridge with 10 ft. girder spacing. To investigate the effect of the deck on shear capacity, the specimens were tested, one with and one without a deck. The effective shear area was increased by approximately 17% because of the associated increase in the effective depth when the deck was added. The specimen with a bridge deck failed at an applied shear approximately 19% greater than that of the specimen without a deck. Although the authors predicted that the specimen without a bridge deck should have failed by flexure-shear cracking, both specimens failed by web-shear cracking. The authors concluded that the change in the angle of principal compression due to the presence of the deck increased the concrete contribution to shear capacity, and thus increased the shear capacity for the specimen with the bridge deck. The authors also concluded that adding the bridge deck simply increased the shear capacity in proportion to the increased shear area.

To investigate how the specifications differ, the shear provisions of the 2004 AASHTO LRFD, 2002 Standard, and 1979 Interim Specifications were used to predict the shear capacity of the two bridge girder ends. All of the codes underpredicted the shear capacity of both specimens, but on average, the predictions from the 2002 Standard Specifications were found to be the closest to the measured capacity; V_{test}/V_{pred} was 1.24 for the specimen with bridge deck and 1.38 for the specimen without the bridge deck. The authors indicated that the presence of the lift hooks (three prestressing strands embedded in the beams) could be a possible reason for the conservative predictions by all Specifications. Because their exact location was unknown, the lift hooks were not accounted for in the shear capacity calculations.

Runzel et al., (2007) further stated that if the term for stirrup contribution V_s was calculated with the measured angle of principal compression instead of the implicitly assumed 45°, the predicted shear capacity from the 2002 Standard Specifications became nearly identical to the measured capacity, i.e., V_{test}, of both specimens. It was also observed that the angle of principal compression was predicted well by the 2004 AASHTO LRFD Specifications which is the key variable in the V_s contribution to shear capacity. Based on the results of the two shear tests, the 2002 Standard Specifications was further suggested to be the most reliable method for predicting shear capacity if the V_s term is calculated with an appropriate angle.

2.8 Summary of the Findings

From the results of the NCHRP 549 report, it was apparent that the 1979 Interim code provided unreliable shear capacity predictions. Conversely, the 2002 Standard Specifications provided reliable predictions of shear capacity, and thus was found to be a useful tool for predicting the shear capacity of prestressed concrete members. Additionally, the findings from the University of Minnesota single girder tests found the shear capacity predictions from the 2002 Standard Specifications closest to the measured capacity.

Chapter 3. Description and Verification of *Virtis* – BRASS Software

3.1 Introduction

Some of the prestressed concrete girders in the Mn/DOT inventory have inventory ratings below unity when rated using the tool *Virtis*-BRASS; however, these girders have not shown any signs of shear distress in the field according to bridge inspection reports. Possible explanations for this discrepancy, explored in this study, were attributed to potential inaccuracies in the *Virtis*-BRASS rating tool. To identify potential errors in *Virtis*-BRASS, five sample bridges were selected to compare inventory ratings obtained from the software to hand computations. The results presented in this chapter cover three of the five sample bridges. Because those three bridges are sufficient to provide representative examples of the range of errors found within *Virtis*-BRASS, the results of the other two bridges studied are not shown.

3.2 Software Description

Virtis is a widely used bridge management product developed under the close direction of AASHTO. Approximately two thirds of the states use *Virtis*. *O*ne component of *Virtis* is a database that stores material properties, cross-sectional properties, span lengths, and other pertinent bridge description information for the bridges in the inventory. Mn/DOT first began using *Virtis* in 2002. *Virtis* is used in conjunction with third party calculation engines for bridge rating. The only third party calculation engine that is capable of rating prestress beams is BRASS, which is currently being used by Mn/DOT. The load rating of the superstructure elements, particularly bridge girders, is done in accordance with Load Factor Design (LFD). The software is current with the 1994 Manual for Condition Evaluation of Bridges (MCE), with interim revisions through 2003. This manual refers back to the 16th Edition of the AASHTO Standard Specifications of Highway Bridges (1996), with interims through 2002. The shear provisions from the 2002 Standard Specifications were presented in detail in Chapter 2. From 1974-2002, Mn/DOT used BARS as a rating tool. However, it did not have the capability to rate prestressed girders for shear.

BRASS is a program that assists in the rating of highway bridge decks and girders in accordance with the AASHTO Specifications. The program performs the calculations using classical methods of structural analysis (e.g., using influence lines for live load effects). It calculates section properties, dead load effects, maximum live load with impact effects, member stresses and strengths, and finally rates the section by criteria set forth in the 1994 MCE with revisions through 2003. Ratings of the maximum load carrying capacity may be determined in one run for inventory and operating rating levels.

3.3 Verification Procedure with Load Factor Rating (LFR)

Virtis-BRASS was evaluated with five sample bridges to identify potential errors. Results of three representative examples of the five bridges are shown. The design example in Section 9.3 of the PCI Bridge Design Manual (2003) was selected as one of the samples because of its familiarity to many designers. In addition to the PCI bridge example, two bridge girders from the Mn/DOT inventory were selected for evaluation and comparison of results obtained by hand computations to the values determined by *Virtis*-BRASS.

Inventory and operating ratings (Load Factor Rating-LFR) for shear were determined in accordance with the 2002 Standard Specifications and the 1994 MCE. Besides comparing the rating factors, the components of the rating factor equation (Eqn (1.1)) for LFR were evaluated separately. Those components included the dead and live load effects and the shear capacity of the member.

3.3.1 PCI Bridge Example

The PCI Bridge Design Manual (2003) example (Section 9.3 of the manual) illustrates the design of a typical AASHTO-PCI 72 in. deep interior bulb-tee girder of a 120-ft single-span bridge. The example covers the design for flexure, shear and deflection at the critical sections. In this study, only shear was investigated. Figure 3.1 shows the bridge cross section. The superstructure consisted of six girders spaced at 9 ft on center. The girders were designed to act compositely with an 8 in. thick cast-in-place (CIP) concrete slab to resist all superimposed dead loads, live loads and impact. The design live load was an AASHTO HS20-44.

Discrepancies between the results regarding the dead load effects, live load effects, shear capacities (i.e., V_c for concrete contribution, V_s for shear reinforcement contribution and ϕV_n for design shear), and shear rating factors are shown in Tables 3.1 through 3.4, respectively, for sections at tenth points across the span. The percent differences are shown relative to the results obtained from *Virtis*-BRASS, i.e., *Diff* = (*VIRTIS-HAND CALC)**100/*VIRTIS*.

As can be seen from Table 3.1, no differences between results obtained from the *Virtis*-BRASS and hand computations due to the dead load effects were found. Tables 3.2 and 3.3 show that there were slight differences in the computations of live load effects and shear capacity between the *Virtis*-BRASS and the hand calculations.

The reason for the slight differences between the results for the live load effects was due to the Wheel Advancement Denominator (*WAD*) in BRASS, which was used to generate the live load actions. BRASS moved a unit load and created influence lines across the deck spans in incremental distances of 1/*WAD* of the length of span under consideration. Truck wheel loads and uniform loads were placed on these influence lines and the sum of the ordinates affected was used to calculate the actions. If a wheel load fell between two ordinates, the program used the closest ordinate. No interpolation was done. The *WAD* value defaulted to 100. To obtain more accurate results, this value was increased. When a *WAD* of 1000 was used, there were no differences between the hand and *Virtis*-BRASS calculations as shown in Table 3.5.

The small differences in shear capacity calculations shown in Table 3.3 were attributed to the following two factors: an error in *Virtis*-BRASS in the harping slope computation and the exclusion of the haunch area during the calculation of the composite section properties.

The harping slope computation in *Virtis*-BRASS was incorrect because the beam overhang past the support was not taken into account. *Virtis* assumed a shorter distance from the harp point to the end face. This assumption directly caused the center of gravity of all strands to the bottom fiber of the beam, i.e., "y_{bs}" to be overestimated and the eccentricity of the strands in the non-composite section, i.e., "$e_{non\text{-}composite}$" to be underestimated, as shown in Table 3.6. This difference caused d to be underpredicted in the harped region and thus, the shear capacity which is linearly dependent on d, when d was not taken as $0.8h$, to be underpredicted. On the other hand, the underestimation of $e_{non\text{-}composite}$ caused an overestimation of f_{pc} and M_{cr}, thus the shear capacity was estimated higher than it should have been. In general, the error in *Virtis*-BRASS for harping slope computations affected the parameters in such a way that the error was compensated due to those opposite canceling affects mentioned above, thus affecting the shear rating calculations insignificantly.

The discrepancies in shear rating factors were found to be negligible for this example, as shown in Table 3.4. The discrepancies in the live load computations (for *WAD*=100) and the error found in the calculation of the location of the harped strands in the girders were the main reasons for the differences in rating factors. Furthermore, the exclusion of the haunch area during the calculation of the composite section properties (found in *Virtis*) also contributed to the difference at every section along the span length.

3.3.2 Mn/DOT Bridge Examples

As opposed to the slight differences found in comparing the results of *Virtis*-BRASS to hand calculations using the PCI bridge example, investigation of other bridge examples from the Mn/DOT database revealed a significant error due to misinterpretation of the 2002 Standard Specifications in *Virtis*-BRASS. The misinterpretation was related to the computation of the shear capacity near end regions and hence affected the shear rating factor of those sections. Two bridges, 27068 and 83022 are shown as examples herein because they yielded some of the largest differences between the hand calculations and the *Virtis*-BRASS computations.

The error found in *Virtis*-BRASS was in the computation of the “f_{pc}” term used in the calculation of the concrete resistance to web-shear denoted as “V_{cw}” in Equation (9-29) of Article 9.20.2.3 in AASHTO Standard Specifications for Highway Bridges (1996 & 2002). The specification is given as

$$V_{cw} = (3.5\sqrt{f_c^{'}} + 0.3 f_{pc}) b_w d + V_p \tag{3.1}$$

In Eqn. (3.1), V_{cw} is the nominal concrete shear strength associated with web-shear (i.e., when diagonal cracking results from excessive principal tensile stress in the web). In a composite member, f_{pc} is the resultant compressive stress due to both prestress and moments resisted by the precast member acting alone at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange. In other words,

$$f_{pc} = \frac{P_{eff}}{A} - \frac{P_{eff} \times e \times (y_{bc} - y_b)}{I} + \frac{M_{DL-non\_comp} \times (y_{bc} - y_b)}{I} \tag{3.2}$$

when the centroid lies in the web, or

$$f_{pc} = \frac{P_{eff}}{A} - \frac{P_{eff} \times e \times (h_{web+b_f} - y_b)}{I} + \frac{M_{DL-non\_comp} \times (h_{web+b_f} - y_b)}{I} \tag{3.3}$$

when the centroid lies within the flange, where, P_{eff} is the effective prestress force, A is the cross-sectional area of the precast beam, e is the strand eccentricity for the non-composite precast beam, I is the moment of inertia about the centroid of the non-composite precast beam, y_{bc} is the distance from the centroid of the composite section to the extreme bottom fiber of the precast beam, y_b is the distance from the centroid of the non-composite precast beam to the extreme bottom fiber of the precast beam, h_{web+b_f} is the total height of the web and bottom flange thickness, and $M_{DL\text{-}non\_comp}$ is the moment due to the dead loads acting on the non-composite girder alone such as girder weight, composite deck weight, weight of haunches and diaphragms.

The analysis engine, *Virtis*-BRASS, did not include the latter definition of f_{pc} (i.e., Eqn. (3.3)) in the cases where the centroid was within the flange and used (y_{bc}-y_b) instead of $(h_{web+b_f} - y_b)$. This error caused the shear rating factors for the five sample bridges studied to be underestimated by up to 25 percent at the critical section, up to 35 percent at the end of the transfer length and up to 15 percent at sections away from the critical section (i.e., at 0.1L) where web-shear governs.

The effect of this error is illustrated by examining the interior girders of Bridge No. 27068_2 and Bridge No. 83022_1-3, where the numbers after the underscore indicate the span numbers of the bridge. The design geometrical and material properties of the girders are given in Table 3.7. For illustration purposes, the cross section of span 1-3 of Bridge No. 83022 is shown in Figure 3.2 with the composite cross section for one of the interior girders given in Figure 3.3. Tables 3.8 and 3.9 show the results obtained from the comparison between the hand computations and *Virtis*-BRASS for the shear capacities and rating factors, respectively. Sample calculations illustrating the comparison between the hand computations (based on the 2002 AASHTO Standard Specifications) and the *Virtis*-BRASS software results are given in Appendix A.2 for the critical section (i.e., *h*/2 from face of support) of an interior girder in Bridge No. 83022 _1-3.

This error was not detected in the investigation of the PCI bridge example, because in that case, the centroid of the composite section was located within the web, thus, the term (y_{bc}-y_b) was used in the hand calculations, as used in *Virtis*-BRASS. As the section gets deeper (e.g., 72 in. deep girder in PCI bridge example) it is more likely that the centroid of the composite section would be located within the web due to the large web height. This error has a larger effect on the shallower sections. As described later in Section 4.2, 54 girders which rated low in shear were selected from the Mn/DOT inventory for study. The girders investigated had depths that ranged from 36 to 72in. deep. In the investigation, the error in the definition of f_{pc} associated with *Virtis*-BRASS was found to primarily affect the rating of the shallower girder depths (i.e., 36, 40 and 45 in.). The centroid was always in the web for the 72 in. girders investigated. Only one 63 in. deep girder had the centroid above the flange. In this case, *Virtis*-BRASS underestimated f_{pc} by 6%. The only 60 in. deep girder investigated also had the centroid above the web; the underestimation of f_{pc} for this girder was 16%. Seven of the ten 54 in. deep girders had the centroid above the web. For these girders, the underprediction of f_{pc} varied between 1% and 12 %, with an average underprediction of 4% for the seven 54 in. deep girders. All of the girders with depths between 36 and 45 in. had the centroid above the web. The underprediction of f_{pc} for these girders varied between 0 and 40% with an average of 19%.

Another error found in the analysis engine of *Virtis*-BRASS was in the calculation of effective strand stress in the transfer length region. For the calculation of effective prestress when the section lies within the transfer length, Article 9.20.2.4 of 2002 AASHTO Standard Specifications states:

"For a pretensioned member in which the section at a distance h/2 from the face of support is closer to the end of the member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V_{cw}. The prestress force may be assumed to vary linearly from zero at the end of the tendon to a maximum at a distance from the end of the tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire."

This article is similar to Article 11.4.4 of ACI 318-05. In addition, ACI Section 12.9 (Development of Prestressing Strand) and its commentary provide Figure R12.9 (reproduced here as Figure 3.4) which shows the relationship between strand stress and the distance over which the strand is bonded to the concrete. The first linear portion of the curve represents the transfer length of the strand (i.e., $(f_{se}/3000)d_b$ where f_{se} is in psi), that is, the distance over which the strand should be bonded to the concrete to develop the effective strand stress, f_{se}. As depicted in the figure, the strand stress varies linearly from zero at the face of the girder to f_{se} at the end of the transfer length.

Virtis-BRASS calculated the effective strand stress within the transfer length improperly. *Virtis*-BRASS assumed that the prestress before the allowance of all losses, varies linearly from zero at the end of the girder to a maximum at a distance from the end of the transfer length. Then, the total prestress losses were subtracted from the aforementioned calculated prestress at every point. Thus, the total losses, which are a function of the applied prestress force, did not vary with the changing applied prestress force in the transfer length. This does not seem to be a reasonable approach compared to the ACI 318-05 Code approach. Moreover, the formula yields unacceptable results (i.e., negative values for effective strand stress) close to the centerline of the bearing. In the case of ACI 318-05, it is assumed that the stress at the end of the transfer length is reduced by the losses. This stress is then varied linearly to zero at the end of the strand. The following equations demonstrate the calculation of effective strand stress in the transfer zone for both cases:

ACI 318-05 & 2002 AASHTO Standard Specifications:

$$f_{se} = (f_{pjack} - TL)\frac{x}{l_t} \text{, if } x \le l_t \text{ and } f_{se} = (f_{pjack} - TL) \text{, if } x > l_t \tag{3.4}$$

Virtis-BRASS:

$$f_{se} = \left[f_{pjack}\left(\frac{x}{l_t}\right) \right] - TL \text{, if } x \le l_t \text{ and } f_{se} = (f_{pjack} - TL) \text{, if } x > l_t \tag{3.5}$$

where f_{se} is the effective stress after all losses, f_{pjack} is the jacking stress, x is the distance from the end of the beam, l_t is the transfer length and TL is the total prestress loss calculated at midspan of the beam.

This error did not impact the calculation of shear rating factors for the bridges evaluated in this study because the critical section for shear per the 2002 Standard Specifications (i.e., $h/2$ away from the face of the support) was out of the transfer zone.

3.4 Summary of the Errors Found in *Virtis*-BRASS

By comparing the results of *Virtis*-BRASS to hand computations for the sample bridges, the following errors were found:

1. Missing length of beam beyond support which causes wrong calculation of slope of the harped strands
2. Exclusion of haunch height and area in the calculation of d and composite section properties (I, A), respectively
3. Incorrect calculation of concrete resistance to web-shear (due to the incorrect value for f_{pc}) when the centroid is in the flange
4. Calculation of effective strand stress in transfer zone with an improper approach

The identified errors were communicated to the developers of the rating tool. There may be other errors in the rating tool, but the ones summarized above are those that were found by the comparison of the *Virtis*-BRASS results to the five sample bridges. Among those errors, the third item should be emphasized as the most important error because it significantly affects the results by underestimating the capacity near end regions where the shear rating is most likely to be less than unity.

Due to the identified errors found in *Virtis*-BRASS, spreadsheets were created to perform the capacity and rating calculations for the bridges investigated in the remainder of the project.

Chapter 4. Selection of Bridges to Investigate for Shear

4.1 Introduction

An initial objective of this project was to develop a screening method to determine the bridge girders most at risk for being undercapacity in shear. A previous project, Mn/DOT Report 2007-47 (Runzel et al., 2007), had determined that some girders in the Mn/DOT inventory designed by the 1979 Interim did not have stirrup spacing that met the 1979 Interim requirements. To determine how widespread this problem was, a number of Mn/DOT bridges known to have shear inventory rating factors less than unity, were selected to check their designs against the design code indicated on the bridge plans. If the reason for the discrepancy could not be identified, it would not be possible to implement an effective screening method to determine girders most at risk due to the apparent random nature of the problem.

4.2 Properties of Selected Bridge Examples from Mn/DOT Inventory

There are 1244 prestressed concrete I-girder bridges in Minnesota that were built between 1929 and 2005. Investigation by Mn/DOT engineers indicated that the bridges built after 1992 were designed to meet the shear requirements of the AASHTO Standard Specifications and had no shear rating problems.

The distribution of the number of bridges by construction year is shown in Figure 4.1. Fifty-nine percent of the bridges in the Mn/DOT bridge inventory were built between 1961 and 1983, and were likely to have been designed using a pre-1983 specification. As explained in Chapter 2, the pre-1983 AASHTO Specifications provided a lower reliability for shear design.

Bridges were selected for additional study from a subset of the bridge inventory provided by Mn/DOT. This subset of the inventory contained bridges known to rate low in shear, either at the inventory level, or at the operating level for standard permit trucks. In the selection of the bridges for further study, priority was given to those with shear inventory rating factors less than unity based on a *Virtis*-BRASS rating performed by Mn/DOT prior to the beginning of this project. As a secondary criterion, bridges that had relatively high Average Daily Traffic - ADT and Heavy Commercial Average Daily Traffic - HCADT counts were considered. A few bridges with inventory ratings slightly higher than unity, but with deeper girders were also selected so that the subset of bridges studied would include a variety depths, in addition to the variety in ages, and concrete strengths already included using the other criterion. Bridges with sidewalks were not included because of the added complexity in the analysis. In total, fifty-four bridge spans were selected for further study. Typical girder geometrical properties of these spans are given in Table 4.1. Table 4.2 lists the selected bridge spans and properties related to shear demand grouped by the year of the design specification indicated on the plans.

4.3 Redesign of the Selected Girders for Shear

The girders in the bridges selected for further study were redesigned for shear according to the Specifications marked on the plan sheet (the Specifications in effect at the time of design). Shear was investigated at every tenth point of the span, and at the critical sections for shear (quarter point for pre-1983 AASHTO Specifications, and $h/2$ from the face of the support for 1983 and later AASHTO Standard Specifications).

For simplicity, the bridges designed with similar Specifications are grouped together as follows:

1) Group 1: Bridges designed by 1961 AASHTO Standard Specifications
2) Group 2: Bridges designed by 1965-1969 AASHTO Standard Specifications
3) Group 3: Bridges designed by 1973-1977-1979 Interim AASHTO Standard Specifications
4) Group 4: Bridges designed by 1983 AASHTO Standard Specifications
5) Group 4*: Bridges designed by 1973-77-1979 Interim AASHTO Standard Specifications but built after 1983.

Group 4* is made of the same bridge girders as Group 4. However, it may be possible that these Group 4* girders were designed for shear in accordance with the 1979 Interim (or 1977 Standard) specifications, which was allowed by a footnote in the 1983 through 2002 Standard Specifications.

"The method for design of web reinforcement presented in the 1979 Interim AASHTO Standard Specifications for Highway Bridges is an acceptable alternate."

Sample design calculations for each design group are given in Appendix A, Section A.2. Stirrup spacing required to meet the horizontal shear provisions of the Specifications in effect at the time of design were also calculated and checked. Tables 4.3 through 4.7 show the provided and required stirrup spacings for Groups 1 through 4* at the critical sections, and tenth points. The numbers in parentheses in these tables are the stirrup spacings required for vertical/horizontal shear. The required horizontal shear spacing is shown in bold when it is the controlling spacing. The required stirrup spacings for horizontal shear are not shown if the controlling case is vertical shear. When the provided stirrup spacing was larger than the required stirrup spacing for vertical shear, the boxes are shaded blue. When the provided stirrup spacing was larger than the required stirrup spacing for horizontal shear, but sufficient for vertical shear, the boxes are hashed.

The difference between the provided and required stirrup spacings are given in Tables 4.8 through 4.11. The distribution of the magnitudes of the differences between the provided and calculated stirrup spacings required for vertical shear at the critical sections (i.e., $0.25L$ for Groups 1, 2, 3 and 4*, and $h/2$ from the face of the support for Group 4) are given in Table 4.12 and Figure 4.2. Additionally, Table 4.13 shows the differences between the provided and calculated stirrup spacings required for vertical shear at the critical sections as a percentage of the beam depth. Table 4.14 and Figure 4.3 show the distribution of the differences between the provided and required stirrup spacings at the critical sections with horizontal shear design requirements also considered. The four Group 1 bridges are not included in Tables 4.12, 4.13, 4.14 and Figures 4.2, 4.3, due to the conservative stirrup spacings provided in the girders of that group when compared to required spacings for vertical shear.

4.4 Observations on Design of Girders for Shear

Investigation of the differences between provided stirrup spacing and required stirrup spacing per the AASHTO Specifications in effect at the time of design reveal the following observations:

1) All Group 1 girders were found to satisfy the vertical shear provisions of the 1961 code. All but one bridge in Group 1 had conservative stirrup spacings for horizontal shear. One location on Bridge 27978-2 did not satisfy the required spacing for horizontal shear, but even in that case, the difference between the provided and calculated spacing was small (i.e., 1.1 in.).

2) In Group 2, all girders, except two, were found to have larger stirrup spacings between the end of the beam and the quarter point than required by the 1965-1969 AASHTO Standard Specifications. More than 50% of the girders were discovered to be undercapacity for vertical shear according to the 1965-1969 AASHTO Standard Specifications, mainly between a distance $h/2$ from the support to $0.3L$. At the critical section, $0.25L$, only 20% of the girders had smaller stirrup spacings than required by the Specifications in use at the time of design. Table 4.12 and Figure 4.3 demonstrate that 9 out of 11 Group 2 bridges were undercapacity for shear according to the 1965-1969 AASHTO Standard Specifications. Of the girders that did not meet the Specifications, approximately 55% had stirrups spaced between 6 to 9 in. over the required spacing for vertical shear.

3) For Group 3, nine of 28 bridges were undercapacity at the critical section ($0.25L$) according to the 1973-1977 AASHTO Standard Specifications. Among the nine, only two of them had provided stirrups spaced more than 3 in. larger than the required spacing.

4) For Groups 4 and 4* approximately 55 and 73% of the girders, respectively, were found to be undercapacity at the appropriate critical section, (i.e., $h/2$ and $0.25L$). Group 4* girders had less differences between the provided and required stirrup spacings for vertical shear compared to the Group 4 girders as shown in Figure 4.2. Similar results were also observed when the differences between the provided and required stirrup spacings for horizontal shear were compared (Figure 4.3). In general, the results from both groups were close and thus did not indicate whether the girders in Group 4 were likely designed with the 1979 Interim or 1983 Standard Specifications.

The design check of the girders revealed that, with the exception of Group 1 girders, 48% of all 50 girders within Groups 2, 3 and 4, and 52% of all 50 girders within Groups 2, 3 and 4* did not satisfy the vertical shear provisions of the specifications in effect at the time of design. Group 2 (1965-1969) had the highest number of initially undercapacity girders among all groups (nine of eleven).

To try to identify potential sources of error in the design methodology used for these bridges, the Prestress Beam Program (PBP) that had been used as a design aid by Mn/DOT was investigated to determine whether there was a systematic error in stirrup design. A version of PBP obtained from the bridge office was run for one of the bridges in the inventory (i.e., Bridge 8011) for which a hard copy output from the program was also available for the bridge from the Mn/DOT records office. The version of the program that had been given to the University did not produce the same outputs as were found on the hard copy. Both the hard copy output and the results of the program run by the University were unconservative. Additionally, the stirrup spacings used in the bridge were different from both the hard copy output and the results of the program run by the University. As a consequence, it was not possible to determine how the shear design decisions had been made in the 1970's. Mn/DOT engineers informed the University that the program had been changed several times over the years and archive copies of the older versions of the code were not available. As a consequence, it was not possible to trace the sources for error in the vertical shear designs.

Without an understanding of the sources of error in the pre-1983 shear designs, it is difficult to give any guidance on a screening tool to identify the girders most at risk of being undercapacity according to the 2002 AASHTO Standard Specifications.

Chapter 5. Shear Capacity Evaluation and Operating and Inventory Rating of Bridges In Accordance with the 2002 AASHTO Standard Specifications

5.1 Introduction

The shear capacity and inventory and operating ratings of the selected bridges evaluated per the 2002 AASHTO Standard Specifications are presented in this chapter. The primary purpose of the evaluation of the shear capacity based on the specifications in use at the time of design, as presented in Chapter 4, was to determine if a screening tool could be developed based on bridge geometry and material properties to determine the bridges most at risk. Even with the large number of girders that did not have sufficient stirrup spacing to meet the specifications in effect at the time of design, a parametric study similar to that summarized in Mn/DOT Report 2007-47 (Runzel et al., 2007) was conducted, to determine whether the same philosophy had any practicality as a screening tool.

5.2 Shear Design per 2002 AASHTO Standard Specifications

The selected girders were evaluated according to the 2002 AASHTO Standard Specifications to determine the adequacy of the shear designs based on current rating methods. The shear design of the selected bridges were carried out by using the nominal material properties at the critical section per 2002 AASHTO Standard Specifications shear provisions, (i.e., a distance of $h/2$ away from the face of support), at every tenth point and the quarter points of the span. Only interior girders were evaluated because they carry more of the live load per the 2002 AASHTO Standard Specifications.

The spacing required by the horizontal shear requirements of 2002 AASHTO Standard Specifications was also investigated. Tables 5.1 to 5.3 show the provided and required stirrup spacing based on the 2002 AASHTO Standard Specifications for the groups investigated in Chapter 4. In these tables, numbers out of parentheses show the provided stirrup spacing, whereas, numbers in parentheses are the required stirrup spacing for vertical/horizontal shear per the 2002 AASHTO Standard Specifications. Where the stirrup spacing required for horizontal shear is shown in bold, the stirrup spacing was controlled by the horizontal shear requirements. In the tables, boxes shaded blue indicate that the provided stirrup spacing did not satisfy the vertical shear requirements. Boxes shaded with diagonal lines indicate that the provided stirrup spacing did not satisfy the horizontal shear requirements, but did satisfy the vertical shear requirements. The distribution of the magnitudes of the differences between the provided and required stirrup spacings for vertical shear at the critical section are given in Figure 5.1. Similarly, Figure 5.2 shows the distribution of the magnitudes of the differences between provided and required stirrup spacings based on both vertical and horizontal shear at the critical section. Sample calculations demonstrating the shear design per the 2002 Standard are given in Appendix A, Section A.3.

5.2.1 Effect of Horizontal Shear on Design of Girders per 2002 Standard Specifications

The AASHTO provisions for the spacing of transverse reinforcement to transfer horizontal shear forces also affect the stirrup spacings for vertical shear in bridge girders. As previously shown in Table 4.14, in the 2002 Standard Specifications, a maximum horizontal shear stress of 350 psi is allowed at the contact surface when minimum ties are provided and the contact surface is intentionally roughened. ACI 318-05 allows up to 500 psi for the identical case; hence it is generally believed that the AASHTO horizontal shear provisions are conservative.

When the required area of ties exceeds the required minimum area per Eqn. (5.2) or the 2002 Standard Specifications (i.e. provides at least 50 psi of shear stress capacity), shear strength can be increased by $(160f_y/40{,}000)b_v d$, for each percent of tie reinforcement crossing the contact surface in excess of the minimum as given in 2002 AASHTO Standard Specifications. This definition results in the following equation

$$V_{nh} = \left[330 + \left(\frac{160 f_y}{40000} 100 \frac{A_v}{b_v s}\right)\right] b_v d = 330 b_v d + 0.4 \frac{A_v f_y d}{s} \tag{5.1}$$

As can be seen from Tables 5.1 through 5.3, when implemented, this equation required small stirrup spacings near the end regions of the MN-36 deep girders (i.e., critical section, 0.1*L* and 0.2*L*). These girders have a narrow top flange width (i.e., 12 in., see Table 4.1) and relatively shallow depth, reducing the interface friction contribution. Starting with the 1989 AASTHO Standard Specifications, the stirrup spacings had to be no larger than four times the web width and in no case taken greater than 24 in. For this study, the 24 in. spacing limit governed all cases. The required minimum area of horizontal shear ties is given in the 2002 Standard Specifications as

$$A_{v,\min} = \frac{50 b_v s}{f_y} \tag{5.2}$$

which can be rewritten as

$$s_{\max} = \frac{A_v f_y}{50 b_v} \tag{5.3}$$

where A_v is taken as the area of the 2 No.4 bars which were used as stirrups (information obtained from bridge design plans) in all investigated girders.

When imposed, Eqn. (5.3) was found to control stirrup spacings for f_y/b_v ratios lower than 3,000 lbs/in^3 as demonstrated in Figure 5.3 and thus, horizontal shear controlled over vertical shear in several girders, especially at sections away from the critical section (Tables 5.2 and 5.3).

5.3 Observations on Shear Design of Girders per 2002 Standard

Similar to the discussion provided in Section 4.4, investigation of the differences between provided stirrup spacing and required stirrup spacing per the 2002 AASHTO Standard Specifications revealed the following observations:

1) Similar to the findings in Section 4.4, almost all Group 1 (1961) girders were found to have acceptable shear reinforcement spacings when compared to 2002 AASHTO Standard Specifications requirements. Two girders that did not satisfy the 2002 Standard

Specifications requirements for vertical shear at the critical section were the Bridge 27978-1 and Bridge 27978-2. The differences between the provided and required spacings for vertical shear were small (0.2 and 1.2 in., respectively).

2) Most locations along the girders in Group 2 (Table 5.1) did not meet the vertical shear stirrup spacing required by the 2002 AASHTO Standard Specifications. Differences between the required and provided stirrup spacings were found to be mainly between 3 to 9 in. as shown in Figure 5.2.

3) Table 5.2 shows that the Group 3 girders (1973-77) were mainly under capacity for vertical shear (per the 2002 AASHTO Standard Specifications) at the critical section and $0.1L$. Approximately 60% of the girders had less conservative stirrup spacings than the required per the 2002 AASHTO Standard Specification at the critical section. Out of the 16 girders found to be under capacity, 12 had stirrup spacings at least 6 in. larger than required and four of those 12 had stirrup spacings at least 9 in. larger than required at the critical section (h/2 away from the face of the support). As shown in Table 5.2, for relatively deep girders (i.e., depths of 54, 63 and 72 in.), the stirrup spacing for horizontal shear calculated from Eqn. (5.3) controlled over the spacing required for vertical shear through the span length except at the critical section.

4) Group 4 had the least number of girders that did not meet the 2002 AASHTO Standard Specifications for vertical shear among all groups (Table 5.3). Generally, the provided stirrup spacings were larger than required near the end regions. Table 5.3 shows that five out of 11 were found to be under capacity at the critical section for vertical shear per the 2002 AASHTO Standard Specifications. As given in Table 5.3 only two bridges, i.e., Bridge No. 8011 and 17007 did not meet the 2002 AASHTO Standard Specifications vertical shear provisions away from the end region (i.e. at $0.4L$). When horizontal shear was checked, Eqn (5.3) controlled the stirrup spacing for the 54 and 63 in. deep girders at sections away from the girder ends due to the reasons explained for similar girders in Group 3. Eqn (5.3) also controlled the stirrup spacing for 45 in. deep girders at sections near midspan due to the beam type, i.e., 45M, with wide top flange width (Table 4.1). The wider top flange width provided a larger interface for horizontal shear transfer by friction, hence fewer stirrups were needed to provide sufficient horizontal shear strength, and the stirrup spacing was controlled by Eqn. (5.3).

5) All relatively deep girders (63 and 72 in. high) had 9 in. stirrup spacings at the critical section, which yielded conservative designs at that location. By coincidence, all of the 63 and 72 in. deep girders with end blocks had 9 in. stirrup spacing at the critical section.

6) Four of the girders investigated had stirrup spacings greater than h/2 near midspan (24831-2, 31019, 27068-1, and 27068-2). In order to activate the stirrup contribution (V_s) to the shear capacity, the stirrup must cross the crack and have sufficient development length between its ends and the crack. Toward the center of a prestressed concrete beam, the cracks are more likely to form at roughly 45 degrees to the horizontal rather than at shallower angles. For a stirrup to provide shear resistance, the stirrup spacing should be no greater than h/2 to ensure that it engages a crack. When stirrup spacings are greater than h/2, near midspan, the V_s term should be discounted when calculating V_n. The lack

of stirrups crossing the potential crack also turns the mode of failure from one with some forewarning, to a brittle failure. Mn/DOT should be aware to look for girders that are relying on a contribution from the stirrups for shear rating at a particular location, but have stirrups at that location spaced further apart than $h/2$.

5.4 Shear Capacities and Ratings per 2002 AASHTO Standard Specifications

The shear capacities and ratings of the selected bridges were calculated by using the nominal material properties, stirrup spacings provided in the bridge plans and the 2002 (or 1996) AASHTO Standard Specifications shear provisions at the critical section, (i.e., a distance of $h/2$ away from the face of support), at every tenth point and the quarter points of the span. Only interior girders were analyzed because they carry more of the live load per the 2002 AASHTO Standard Specifications.

As given in the Manual for Condition Evaluation of Bridges (MCE, 1994 and interims) the dead load effects were computed in accordance with the conditions existing in the girders (i.e., dead load accounting for road widening and addition of new wearing course). An AASHTO HS20-44 loading was utilized for live load as mandated by the MCE. The shear demand (V_u) was taken as the sum of the factored dead and factored live loads occurring at the section under investigation. The ratio of the shear capacity multiplied by the strength reduction factor (ϕ of 0.9) to factored shear demand, (i.e., $\phi V_n/V_u$), and the corresponding shear inventory and operating rating factors are tabulated in Tables 5.4 through 5.9. The ratios and ratings presented in the tables were computed using the vertical shear design articles of the 2002 Standard Specifications (Articles 9.20.1, 9.20.2 and 9.20.3). The horizontal shear requirements were not evaluated. The tables are sorted by order of design year. The shaded cells show ratios that are below unity. Sample shear capacity and inventory rating calculations are presented in Appendix A, Section A.4.

5.5 Observations on Shear Capacity and Rating of Girders per 2002 AASHTO Standard Specifications

As shown in Table 5.5, Group 1 bridges (assumed designed per the 1961 AASHTO Standard Specifications) had inventory rating near or more than unity at all sections investigated and operating ratings greater than unity at all sections. This indicates that the shear capacity for these girders is adequate because as discussed in Section 2.7, the 2002 AASHTO Standard Specifications shear provisions were reliable for predicting shear capacity, V_n. Higher rating factors and capacity-to-demand ratios were expected because the provided stirrup spacings were significantly smaller than the spacings required by the 1961 AASHTO Standard Specifications. The smallest inventory rating factors occurred near the end regions of the girders for the first and second span of Bridge 27978, as 0.98 and 0.92, respectively. Sections near end regions of girders (i.e., $h/2$ away from the face of the support and $0.1L$), with a $\phi V_{n,STD2002}/V_u$ ratio of at least 0.90 were within one standard deviation of the mean of test results by Hawkins et al. (2005) as shown in Table 2.3. Although they may not have the same reliability as girders with higher ratings, they are not expected to fail in shear due to the conservative nature of the specifications. The value of the test-to-predicted ratio one standard deviation below the mean was equal to 1.11, which is still greater than one. Additionally, there are shear capacity sources (discussed in Chapter 6) ignored by the 2002 Standard Specifications which also decrease the probability of failure.

Group 2 bridges (1965-69) were found to have low design capacity-to-demand ratios and inventory ratings; however all the operating ratings were greater than unity. As shown in Table 5.4, the critical section for shear and sections including 0.1*L*, quarter point, 0.3*L* and 0.4*L* were identified as the sections having design capacity-to-demand ratios as low as 0.77. Similar to the capacity versus demand ratios, low shear inventory rating factors were obtained at the same sections. The lowest shear inventory rating factor was 0.68 at the critical section for the first span of Bridge No. 24825. This bridge also had the lowest shear operating rating factor (1.08) for this group.

Group 3 (1973-1977) had the bridges with the lowest design capacity-to-demand ratios and shear inventory ratings (0.74 for Bridge 48010 and 0.64 for Bridge 31019) at the critical section. The section at 0.1*L* also had design capacity-to-demand ratios as low as 0.81, indicating they were under capacity for shear. In all other sections, for all bridges except Bridge 31019, the design capacity-to-demand ratios and shear inventory ratings were least 0.90 (Table 5.6). All of the bridges in Group 3 had shear operating rating factors greater than unity, with the smallest being 1.12.

Five of the eleven bridges in Group 4 (1983 or 1977) had sufficient shear capacity throughout the girders. Eight of the eleven had design capacity-to-demand ratios greater than 0.9. The lowest design capacity-to-demand ratio was 0.85.

When low design capacity-to-demand ratios or inventory rating factors were obtained at locations other than the critical section, then, in almost all cases (except some girders in the 1983 Standard design group, Tables 5.8 and 5.9), low ratios or inventory rating factors were also found at the critical section. Thus, absence of low design capacity-to-demand ratios and inventory rating factors at the critical section is likely an indicator of girders that will be sufficient for shear at both the inventory and operating levels.

The $\phi V_{n,STD2002}/V_u$ ratios and rating factors at the critical section were examined to determine whether there was a relation between girder depth and these values. In general, deeper girders have larger shear capacities because of the increased area resisting shear. Thus, Figure 5.4 shows that girders with relatively deep sections (such as 63 and 72 in.), generally had shear inventory rating factors greater than unity (except one 63 in. deep girder) at the critical section. Figure 5.5 shows that all the girders had shear operating rating factors greater than unity, with the larger shear operating rating factors belonging to the deepest girders.

Seven spans had very low shear inventory rating factors (i.e., inventory rating factors < 0.85) in sections away from the end (at 0.25*L*, 0.3*L*, 0.4*L* and midspan): Bridge 27942, Bridge 24825_5, Bridge 62860, the three spans of Bridge 24831 and Bridge 31019. The reason that those bridges had very low shear inventory rating factors were likely due design errors (i.e., for Bridge 31019, the provided stirrup spacing is 33 in. at sections 0.4*L* and 0.5*L* which seems extremely unconservative even compared to maximum allowed stirrup spacing, i.e., 24 in., per 2002 AASHTO Standard Specifications per Article 9.20.3.2).

During the course of this study, the Mn/DOT Bridge Office performed visual inspections on six of the bridges located in the metro area that were noted in this report to have shear inventory ratings below unity. The Bridge Office personnel inspected the bridges to determine whether diagonal web cracking was present. The presence of this kind of cracking would indicate that the bridge had seen a severe shear load; however, it should be noted that diagonal cracking that may have developed under the presence of a heavy load, may not be visible in the absence of heavy load. The six bridges visited all had $\phi V_{n,2002}/V_u$ values at the critical section between 0.74 and 0.85, shear inventory ratings between 0.65 and 0.82, and shear operating ratings between 1.05 and 1.19.

Results of the visual inspections of the six bridges investigated in the metro area are contained in Appendix B. Cracking was only observed on Bridge 19033 but it appeared to be associated with end restraint rather than related to shear. Conclusions from the inspection were that the cracks in Bridge 19033 should be monitored over time.

It is recommended that Mn/DOT use a combination of shear inventory ratings, shear operating ratings, and the likelihood of the bridge experiencing heavy truck traffic loads, in selecting additional bridges out of its full inventory for additional visual inspection. The AASHTO live load factors account for multipresent heavy vehicles, so bridges that have high Heavy Commecial Daily Average Traffice (HCADT) counts are ones more likely to have seen large shear loads and hence, more likely to be the ones to exhibit shear cracking. Additional guidance on selecting bridges for visual inspection is given in Chapter 7 after a discussion of the potential sources of additional shear strength in Chapter 6.

5.6 Mn/DOT Report 2007-47

Runzel et al. (2007) conducted a parametric study on girders that exactly met the 1979 AASTHO Interim Specifications to investigate whether or not bridge girders with different characteristics were likely to be under capacity. Ten existing bridges from the Mn/DOT inventory that covered a wide range of girder depths, span lengths, concrete compressive strengths and girder spacings were selected for the study. Stirrup spacings were recalculated to exactly meet the 1979 Interim Specifications. Using the stirrup spacings calculated per the 1979 Interim, the shear capacities were calculated by the 2002 AASHTO Standard Specifications. The authors found that the ratio of span length to girder spacing, L/S_g, was a good indicator of $\phi V_{n,STD2002}/V_u$, as explained below.

Runzel et al. (2007) found that at the critical section, the contribution from the stirrups, V_s, to shear capacity was approximately 30% for most of the girders. Because the stirrup contribution was a consistent fraction of the capacity, the concrete resistance to web-shear, V_{cw} and shear demand, V_u, were the main variables in determining the adequacy of the girders. The concrete resistance to web-shear was found to be the controlling case for the concrete contribution near end regions (i.e., at critical section and 0.1L).

The parameters in the 2002 AASHTO Standard Specifications V_{cw} equation (Eqn. (2.10)) were investigated in depth in the Mn/DOT Report 2007-47. The value of f_{pc}, given by Eqn. (3.2) and (3.3), is the resultant compressive stress at the centroid of the composite section, or at the junction of web and flange when the centroid lies within the flange, due to both prestressing force and moments resisted by the precast member acting alone. Near the critical section, self-weight can be ignored, and Eqn (3.3) was rewritten as:

$$f_{pc} \approx \frac{P}{A}\left(1-\frac{e(y_{bc}-y_b)}{r^2}\right) \quad (5.4)$$

where P is the effective prestressing force, e is the eccentricity of the strands, A is the area of the precast beam, r is the radius of gyration, and the value of y_{bc}-y_b is taken as either the distance between the centroids of the composite and noncomposite section if the centroid of the composite section lies in the web, or the distance between the web and flange junction and the centroid of the noncomposite section if the centroid of the composite section lies above the web. Runzel et al. noted that the values of $e(y_{bc}-y_b)/r^2$ for each girder given in the report varied little for girders with the same depth. For each depth-based subset of girders, the effect of eccentricity was similar, so any relative increase in P would result in an increase in f_{pc}, and thus V_{cw}.

Runzel et al. (2007) also showed that the girder length, L, was related to the number of prestressing strands in the girders. Because the quantity of prestressing strands in the girders was directly related to P, girder length, L, was also found to correlate well with P and thus L was found to be well correlated to V_{cw} at the critical section.

The ultimate shear demand, V_u, was found to be linearly related to girder spacing, S_g, but had little dependence on L at the critical section. The authors suggested that, L/S_g would be a good indicator of $\phi V_{n,STD2002}/V_u$, and based on their study, suggested that girders with an L/S_g of 10 or greater were not unlikely to fail in shear (Runzel et al., 2007).

5.6.1 Application of Mn/DOT Report 2007-47

Even though many of the girders investigated in the present study did not meet the specifications in use at the time of their design, the use of L/S_g as an indicator for adequacy of shear design was investigated. To check the validity of the assumptions made by Runzel et al. (2007) on the current list of bridge spans, parameters (e.g., V_{cw} and V_s) were checked to see if they complied with the findings (i.e. that V_{cw} must control to use the screening tool and V_s needs to provide approximately 30% of the total shear capacity). Similar to the results from Mn/DOT Report 2007-47, the web shear capacity was found to govern the concrete shear contribution at the critical section, except for one girder, Bridge 9200 as shown in Table 5.10.

In the Mn/DOT Report 2007-47, the stirrup contribution, V_s, was found to be approximately 30% of the total shear capacity, and hence less important than V_{cw} or V_u on the low inventory rating factors calculated at the critical section. Table 5.11 shows the parameters as studied by Runzel et al. (2007), including the ratio of the V_s/V_u for the 54 bridges in the current study. Additionally, to apply the results from Mn/DOT Report 2007-47, the values of $e(y_{bc}-y_b)/r^2$ should not vary significantly for girders with the same depth. The corresponding values were also calculated and are shown in Table 5.11 for the 54 bridges.

In Table 5.11, the values shown in bold indicate significant deviation from Mn/DOT Report 2007-47 findings for the corresponding parameter. In general, 36 and 45 in. deep girders had V_s/V_u percentages around 30%, similar to that found by Runzel et al. (2007), however, the deeper sections such as; 54, 63 and 72 in. deep girders generally had a range of V_s/V_u between 12 and 52%. Unlike the girders studied by Runzel et al. (2007), the $e(y_{bc}-y_b)/r^2$ also varied even while keeping the girder depth constant. Even though the bridges studied did not have a constant percentage of V_n coming from V_s, and did not have near constant $e(y_{bc} - y_b)/r^2$ for constant girder depth, the L/S_g screening tool was investigated.

Figures 5.6 and 5.7 show $\phi V_{n,STD2002}/V_u$ and shear inventory rating factors at the critical section with respect to L/S_g for the girders from the Mn/DOT inventory studied. As shown in Figure 5.6, although the trend had significantly more scatter than that found by Runzel et. al. (2007), there was a correlation between L/S_g and $\phi V_{n,STD2002}/V_u$. All girders that had an L/S_g value of at least 10 had a $\phi V_{n,STD2002}/V_u$ of at least 0.9. The group of girders with the lowest values of $\phi V_{n,STD2002}/V_u$ (0.76 to 0.79) had the lowest L/S_g values (2.59 to 4.56). Considering Figure 5.7, only two bridges (45 and 54 in. deep girders) with L/S_g greater than 10 were shown to have inventory rating factors below 0.9.

Although the correlation between $\phi V_{n,STD2002}/V_u$ and L/S_g for the bridge girders from the subset of the Mn/DOT inventory was not as strong as that found in Runzel et al. for girders designed to exactly meet the vertical shear provisions of the 1979 AASHTO Interim Specifications, L/S_g was still found to be an relatively effective preliminary screening tool. Because many of the pre-1983 girders in the Mn/DOT inventory did not meet the vertical shear provisions of the specification in use at the time of design, no screening tool can be developed that can account for the unknown deviations in provided capacity from those required by the design. As a consequence, it is recommended that all of the girders be investigated individually. However, the L/S_g screening tool can be used to prioritize the order in which the girders should be evaluated.

Chapter 6. Investigation of Additional Shear Capacity

6.1 Introduction

As shown in Chapter 5, there are a number of bridges in the Minnesota inventory that do not have adequate capacity according to the 2002 AASHTO Standard Specification. Even though the inventory rating factors evaluated by the Load Factor Rating method were found to be less than unity, the Mn/DOT inspection reports indicated that the girders had no sign of shear distress. One possible reason may be additional shear strength mechanisms not considered in the 2002 AASHTO Standard Specification. Additionally, the shear live load demand may be overestimated using the shear live load distribution factors in the 2002 AASHTO Standard Specification. This chapter summarizes an investigation of these issues. Section 6.2 summarizes the investigation of the parameters investigated as potential sources of additional shear strength and Section 6.3 summarize the investigation of the potential conservatism in shear demand.

6.2 Investigated Parameters for Additional Shear Strength

The parameters investigated include:

- Contribution from end blocks at the beam ends
- Differences between nominal and measured 28-day concrete strengths
- Effect of increase in concrete strength with time
- Effect of short shear spans (arching action)

6.2.1 Contribution from End Blocks at the Beam Ends

In prestressed concrete girders, a large concentration of longitudinal compressive stress occurs at the bottom of the girder end and tensile stresses develop at the top of the girder end due to the large tendon prestressing forces and reduction in self-weight moments at the beam ends. Thus, it is sometimes necessary to increase the area of the cross section towards the support by means of "end blocks" to reduce the compressive and tensile stresses in the concrete caused by the prestress.

End blocks increase the area resisting shear and thus increase the shear capacity near the girder ends. In practice, the 2002 AASHTO Standard Specification is applied without considering this additional shear strength factor, (i.e., the increase in web width is ignored) and hence could result in very conservative estimations for $\phi V_{n,STD2002}/V_u$ and shear inventory rating factors for girders with end blocks.

In the Minnesota bridge inventory, end blocks were mainly used in deeper sections (such as 54, 63 and 72 in. deep girders). Thirteen of the 54 girders investigated in this study had end blocks. The geometry and dimensions of the end blocks for these girders are shown in Figure 6.1. As can be seen from Figure 6.1, there are two regions throughout the length of the end blocks; the constant width and tapered width regions.

Table 6.1 lists the 13 bridges examined that had end blocks, along with the design year, geometry, $\phi V_{n,STD2002}/V_u$, and shear inventory rating factors ignoring the presence of the end blocks. As was mentioned earlier, in general, the deeper beams had fewer problems with shear rating. Because most of the girders that had end blocks were deep beams, few of these bridges (i.e., only two) had shear inventory ratings less than unity. The two bridges that had shear inventory ratings less than unity before taking the increased width due to the end blocks into consideration were Bridge 22805-1 and Bridge 19813-3. In all but two of the 13 bridges (i.e., Bridge 19813-1 and 19813-3), the end block terminated prior to point along the beam where bridges had shear inventory ratings less than unity (i.e., 0.1*L*). Because the end block on Bridge 22805-1 terminated prior to 0.1*L*, and this bridge had a shear inventory rating factor lower than unity at 0.1*L*, the end blocks could not provide the increase in shear capacity to provide a shear inventory rating factor greater than one. For Bridge 19813-3, the end block was present at 0.1*L*, however this location was very close to the end of the end block (0.18 ft from the end), so the increase in web width at this location due to the tapered end block was insignificant. The presence of the end block brought the shear inventory rating factor for Bridge No. 19813-3 at 0.1*L* up to 0.96 from 0.90, not quite providing enough additional capacity to produce an inventory rating larger than unity.

Because only the deeper girders which tend not to have problems with shear inventory rating tend to have end blocks, and because the end blocks taper and tend to terminate prior to 0.1*L*, there is little likelihood that detailed calculations for the end block contribution near girder ends will provide sufficient additional capacity to increase the shear inventory ratings of underrated bridges up to unity and is likely not worth the effort to consider them.

6.2.2 Effect of Concrete Strength on Shear Strength

As concrete strength increases, the shear strength also increases. The concrete contribution to shear in the end regions of girders, in ACI 318-08 and the 2002 AASHTO Standard Specification for example, is regarded as being that due to web shear cracking (diagonal cracking), and therefore dependent on the tensile strength of the concrete. In the 2002 AASHTO Standard Specification, the shear strength of a member, when V_{cw} controls, is taken as directly proportional to $\sqrt{f_c'}$ which indicates that the concrete tensile strength is being used as the governing parameter.

In the following sections, the differences in nominal and measured 28-day concrete strengths and the effect of age on concrete strength are investigated as potential factors that might yield increased shear capacity predictions.

6.2.2.1 Nominal vs. Measured 28-day Concrete Strength

The strength of concrete for design is traditionally characterized by the 28-day value. Precast concrete components are required to achieve a minimum concrete strength at release. This often results in a concrete that has a 28-day compressive strength in excess of the specified 28-day strength (PCI, 2003). Thus, as a potential additional strength parameter, the difference between the measured and design 28-day strength of concrete cylinders obtained from a local precasting plant in Minnesota and from the literature (Nowak and Szerszen, 2003) were investigated.

A large portion of the precast prestressed concrete girders in Minnesota have been cast at the Cretex precasting plant in Elk River, Minnesota. Historical data on nominal, release and 28-day concrete strength from Elk River were obtained for the following time periods: 1974-1983, 1986-1993, 1995 and 1996. During those time periods, Type III Portland cement was used, water reducers began to be used around 1979 or 1980, and the Sure-Cure® system was used for cylinders since the early 1990s. Before that time cylinders were kept with the girder overnight, and then went into a lime bath.

The mean, bias λ (the ratio of measured concrete strength to specified design strength), and coefficients of variation, COV, for the measured $f_c^{'}$ from Elk River cylinders is shown grouped by nominal strengths in Table 6.2. The samples are grouped according to their design strengths at 500 psi intervals from 4750 to 7250 psi strengths. The mean nominal strengths for girders within each range were calculated (e.g., for the range 4750 psi < f_c'≤ 5250 psi, the mean nominal strength was 5015 psi, and the specified design strength was 5000 psi). The specified design strengths were shown for every 500 psi intervals between 5000 and 7000 psi, in order to have a convenient comparison with the data obtained from the literature. As can be seen from Table 6.2, the measured concrete strengths were significantly underestimated by the corresponding mean nominal strengths. The percent increase shown in Table 6.2 is obtained by taking the ratio of the difference between the mean measured strength and the mean nominal strength to the mean nominal strength.

The statistical parameters from the Elk River data were compared to the data from Nowak and Szerszen (2003). In this study, the data used to calibrate the strength reduction (i.e., resistance) factors for ACI 318-05. The primary focus of this study was the analysis of material properties based on material test data obtained from industry. As part of this study, Nowak obtained data from precasting plants throughout the U.S. The statistical parameters for ordinary plant-cast concrete strengths from Nowak's study were compared to the data from Elk River. Table 6.3 shows the statistical parameters (mean values, λ, and COV) for measured 28-day concrete strengths of ordinary plant-cast concretes listed by nominal strengths.

Comparison of the statistical parameters given in Tables 6.2 and 6.3 shows that the Elk River concretes had similar statistical values to those of the ordinary plant-cast concrete in Nowak's study. It could be argued that the typical overstrength of the concrete at 28 days is an inherent part of the overstrength assumed in design. Although the 2002 AASHTO Standard Specification is not a calibrated load and resistance factor design (LRFD) specification, it does use ACI 318 as a basis, which is a calibrated LRFD specification. Calibration of the resistance factors in ACI 318-05 (and previous versions) was in part based on the realized plant-cast 28-day concrete strengths found in Nowak and Szerszen (2003). Because the Elk River 28-day strengths showed similar statistical parameters as the Nowak and Szerszen data, the increase in realized 28-day strength over specified 28-day strengths from the Elk River concrete should not be used as reserved strength.

It should also be mentioned that the findings from this study may not be extended to all existing girders in Minnesota, because the results obtained from the Elk River plant do not necessarily represent concretes batched at other precasting plants used to cast Minnesota bridge girders.

6.2.2.2 Aging of Concrete

Concrete is usually specified with a 28-day compressive strength which is used in the design calculations. Due to continued hydration, the concrete strength continues to increase with time (ACI Committee 209, 1992). The type of cement used and the type of early curing used affect how much strength the concrete will eventually attain. For the same mix design, moist-cured (MC) concrete gains more strength over time than steam-cured (SC), and concrete made with Type I cement gains more strength over time than that made with Type III cement (Olson, 1991). This increase in strength over time above the 28-day strength was not accounted for in the calibration of ACI 318-05.

The girders in the Mn/DOT inventory that had low shear ratings were at least 15 years old. Thus, data from the literature (Riessauw and Taerwe (1980), Rabbat (1984), Scanlon and Mikhailovsky (1986), Olson (1991), Halsy and Miller (1996), Pessiki et.al. (1996), Saiidi et.al. (2000) and Runzel et. al. (2007)) documenting the effect of age on concrete strength were investigated to determine the impact of this potential source of increase in shear capacity on the shear inventory ratings. The results are summarized below.

6.2.2.3 Wood (1991)

The results of laboratory investigations of the variation of concrete strength and stiffness with age are summarized in the report by Wood to serve as a benchmark for interpreting the properties of in-situ concrete. Data were compiled from tests of approximately 5000 concrete prisms and 1500 concrete cylinders, representing nearly 300 combinations of cement type, mix proportions, and curing conditions. Specimens were tested at ages ranging from 1 day to 34 years.

The data were compiled from the results of four investigations initiated by the Portland Cement Association (PCA) between 1940 and 1956.

Series 308 (1940):

- A testing program was developed to define strength characteristics of concrete made from five types of Portland Cement (PC) following the adoption of American Society for Testing and Materials (ASTM) Tentative Specification C 150-40T, *Standard Specifications for Portland Cement*.
- The specimens were tested at ages ranging from 1 day to 5 years.

Series 356 (1947):

- Tests were carried out to evaluate the influence of curing conditions on concrete pavements made from Type I and III PC.
- Depending on curing conditions, the specimens were tested either 5 or 20 years after casting.

Series 374 (1950):

- Data on the strength of concretes made from five types of Portland cements (Type I, II, III, IV and V PCs) and air-entraining agents are reported through ages of 34 years.

Series 436 (1956):

- The strength of concrete made using Portland blast-furnace slag cements were compared to concrete made from Type I PC at ages ranging from 1 day to 27 years.

In general, the cements and aggregates used in the long-term studies satisfied the ASTM specifications in effect at the time the study was conducted. All the Type I and III PCs used satisfied the current ASTM requirements for chemical and compound composition.

Tested concrete specimens were stored in five different environments.

- Moist Curing – Continuous storage in a moist room at a temperature of 73 °F and 100% relative humidity.

- Air Curing – Cured 7 days in moist room, then stored indoors at temperatures between 70 and 75 ºF, with 50% relative humidity.

- Air Curing + pretest soaking – Cured 7 days in moist room, then stored indoors. Specimens were soaked in water at 75 ºF for 48 hours prior to testing.

- Outdoor Exposure at Skokie, Illinois – cured 7 days in moist room then stored outdoors on a clay loam.

- Outdoor Exposure at Dallas, Texas – cured 7 days in moist room then stored outdoors on a sandy soil.

The specimens with the outdoor exposure were stored side by side on the ground with soil packed around the sides. Only the top surface of the specimens was exposed to the atmosphere.

Two types of specimens were tested to determine the variation of concrete compressive strength with age: 6 in. modified cubes (compressive strength corresponding to the mean strength of six specimens) and 6 x 12 in. cylinders (compressive strength corresponding to the mean strength of three specimens). Six inch modified cubes were tested at all ages in all four investigations, however, only 6 x 12 in. cylinders in Series 308 were tested at all ages.

The compressive strength of cylinders and modified cubes cannot be compared directly because of differences in the aspect ratio of the specimens (Murdock et al., 1957). Generally, the compressive strengths of modified cubes are greater (4% for moist curing and 11% air curing) than those of cylinders.

The development of compressive strength with age was illustrated in Wood's report using data from Series 356 for concrete because the trends identified in the Series 356 results were found to be representative of the strength variations with time for all concrete mixes tested. Normalized compressive strength (compressive strength at time *t* divided by the measured 28-day strength) was used to quantitatively describe variations in compressive strength with time. Means and standard deviations of the normalized compressive strength data for both Types I and

III PC are shown in Tables 6.4 and 6.5 for moist-cured specimens and specimens with outdoor exposure, respectively.

The following conclusions were made by Wood (1991):

- The mean compressive strength was observed to increase with time for specimens stored in a moist environment.

- Differences between the compressive strength development of specimens cured in a moist room and specimens stored outdoors were small.

- After 20 years, the mean compressive strength of concrete specimens made from Types I and III PC were 30 to 40 percent higher than the 28-day strengths.

6.2.2.4 Core Test Data Available in Literature

Compressive strength tests on concrete cores taken from existing bridge structures are often used as a tool in evaluating the strength of the existing structure. It is well known that in situ concrete strengths may vary greatly from the strengths obtained from test cylinders. This can be attributed primarily to differences in curing and placing (McIntyre and Scanlon, 1990). In addition, concrete will typically continue to gain strength over time and the strength of mature concrete will be significantly higher than its specified 28-day strength.

The available core test data in the literature was investigated and summarized in Table 6.6. All of the cores were from prestressed concrete girders except one group, i.e., Scanlon and Mikhailovsky (1986), where the cores were obtained from a concrete bridge. Ratios obtained by dividing the long term concrete compressive strengths by design strengths show that the long-term strength was at least 154% of the 28-day design strength. As shown in the last column of this table, only four investigators provided data for the measured 28-day and long-term concrete compressive strengths. Girders tested at the University of Minnesota by Olson (1991) and Runzel et. al (2007) were made from Type III PCs. Other investigators did not provide the type of cement used in the tested cores, however, they were likely to be made from Type I or III Portland cement because those cement types represent approximately 90% of the cement used in concrete construction in the U.S (Wood, 1991). More detailed information on the compressive strength tests is given in Appendix C.

6.2.2.5 Comparison of Data from Wood (1991) with Available Core Test Data

The trends identified in Series 356 by Wood (1991) were found to be representative of the strength variations with time for all concrete mixes, thus the data from Series 356 were selected for comparison purposes.

Concrete used in the 54 girders investigated from the Mn/DOT inventory and historical cylinders from Elk River were all made from Type III PC. This indicates that all girders cast in Minnesota were likely made from Type III PC during the range of interest for the study. Also, all existing girders in service have been exposed to climate conditions which were similar to the cylinders stored outdoors in Skokie Illinois, reported by Wood (1991). However, starting from the early 1990s, cylinders obtained from Elk River, Minnesota (and probably the girders) have been heat-cured. Thus, the available core test data was compared to Series 356 specimens made from Type III PC for each curing condition, but the data from the concrete cores was more likely and conservatively comparable to the data from specimens exposed to outdoors.

Figure 6.2 shows the concrete compressive strength with age, for the specimen from Wood's report that were cured outside as well as the core tests from the literature. The discrete data points show the compressive strengths of concretes with three different water-cement ratios provided by Wood (1991) and the data points connected with lines correspond to the core test data from literature. The figure shows that the core test data shows trends that are similar to those of the specimens stored outdoors.

Table 6.7 summarizes the data obtained from Wood (1991) and core tests from the literature and compares the long term to measured 28-day concrete strength ratios. The data show that the ratios of long term concrete strength to measured 28-day strength from the core tests conducted at the University of Minnesota by Olson (1991) and Runzel et al. (2007) were similar to the ratios provided by Wood (1991). The ratio of 1.22 from Olson (1991), (i.e., a 22% increase in concrete strength with time), was the lowest increase with time found from the core data in the literature. Thus, a lower bound of 20% increase in concrete compressive strength over 20 years is conservatively recommended.

6.2.2.6 Comparison of Results Obtained from Literature with ACI 209

ACI Committee 209 (1992) recommends the following expression for predicting compressive strength at any time

$$(f_c^{'})_t = \frac{t}{a+\beta t}(f_c^{'})_{28} \tag{6.1}$$

where, $(f_c^{'})_t$ is the compressive strength of concrete at time *t* in days, $(f_c^{'})_{28}$ is the 28-day strength of concrete, *a* is a factor depending on type of cement and curing conditions (4.00 for moist-cured Type I cement, 2.30 for moist-cured Type III cement, 1.00 for steam-cured Type I cement, 0.70 for steam-cured Type III cement) and β is a factor to account for cement and curing conditions (0.85, 0.92, 0.95, and 0.98, for moist-cured Type I cement, moist-cured Type III cement, steam-cured Type I cement and steam-cured Type III cement, respectively).

Values of *a* and β for moist-cured Type III PC concretes (2.3 and 0.92, respectively) were used for comparison purposes. For a typical moist-cured Type III PC concrete, Eqn. (6.1) gives a value of 1.09 for the ratio of 20-year concrete strength to 28-day concrete strength. Thus, with a 9% predicted increase compared to the recommended lower bound of 20% increase in concrete strength over 20 years, ACI 209 was found to underestimate the concrete strength gain over time. The underestimation becomes higher for a typical steam-cured Type III PC concrete, i.e., 2% predicted increase compared to 9% increase estimated for moist-cured, because of the higher β factor, i.e., 0.98, used for steam-cured Type III PC concrete in Eqn. (6.1).

6.2.2.7 Shear Capacity Based on Recommended 20% Increase in 28-day Design $f_c^{'}$

The shear capacities predicted by the 2002 AASHTO Standard Specifications were recalculated for a number of investigated girders from Chapter 5 using the recommended 20%

increase in 28-day design $f_c^{'}$. Girders with relatively low $\phi V_{n,STD2002}/V_u$ ratios and shear inventory rating factors, i.e., values lower than unity, were selected. The influence of increased $f_c^{'}$ was investigated at sections near the beam ends and sections close to midspan.

Tables 6.8 through 6.10 show the results for the selected girders including the recalculated strengths compared to the original values at different sections (i.e., critical section, 0.1L, 0.3L, and 0.4L, respectively) along the span length. The 20% increase in concrete strength resulted in 3.4% and 5.8% increases on average in $\phi V_{n,STD2002}/V_u$ and shear inventory rating factors at the critical section as shown in Table 6.8.

Girders undercapacity according to 2002 AASHTO Standard Specifications had relatively low amounts of shear reinforcement compared to girders designed to exactly meet the specification. In these cases, an increase in the concrete contribution due to increased concrete strength, V_c, was more effective. As shown in Table 6.8, Bridge No. 27942 was found to have the highest increase in $\phi V_{n,STD2002}/V_u$ (i.e., 5.1%) and in shear inventory rating factors (i.e., 11.1%). This was due to the high contribution of V_c to V_n (88%), which was essentially due to the low amount of web reinforcement provided at the section.

Similar results were observed at 0.1*L* as shown in Table 6.9. This was expected because at both sections the web shear cracking term, V_{cw}, governed the concrete contribution. However, this was not the case for sections away from the end regions. In these cases, flexure-shear cracking, V_{ci}, was the governing term. Runzel et al. (2007) showed that an increase in $f_c^{'}$ had a larger impact on V_{cw} than V_{ci} The results shown in Table 6.10 are consistent with the findings from Runzel et al (2007); relatively small increases were obtained in shear strengths away from the end regions compared to those near end regions.

In general, the use of increased $f_c^{'}$ resulted in a small increase in the shear capacities predicted by the AASTHO 2002 Standard Specification for the selected girders. The reason for this can be attributed to the nature of the 2002 AASHTO Standard Specification equation *for* V_{cw} which directly proportions the shear strength of a member to the square root of $f_c^{'}$, so a 20% in $f_c^{'}$ will result in a 10% maximum increase in the concrete contribution to the member shear strength.

In conclusion, increases in concrete compressive strength were not found to significantly contribute to unaccounted for increases in shear capacity.

6.2.3 Arching Action

In this section, possible reserve shear strength is investigated near the end regions of the girders due to arching action. Consideration was given to the end regions of the girders because the shear inventory ratings at the critical section for shear according to the 2002 AASHTO Standard Specifications, i.e., *h*/2 away from face of the support, gave the lowest inventory ratings (i.e., generally well below unity) compared to other sections for most of the girders (Section 5.5).

As discussed in Section 2.5, in B-regions, beam behavior is expected, i.e., plane sections remain plane. However, in D-regions, complex load paths result from concentrated loads and discontinuities. Near the supports, the flow of forces is directed from the loads to the supports through arching action, as opposed to beam action. In the case of shear, the difference in behavior of the two types of regions (i.e., B and D) can be explained as follows (MacGregor, 1997).

The relationship between shear and moment can be written as:

$$V = \frac{dM}{dx} = \frac{d}{dx}(T\,jd) \tag{6.2}$$

which can be expanded as:

$$V = \frac{d(T)}{dx}\,jd + \frac{d(jd)}{dx}T \tag{6.3}$$

In B-regions, the lever arm, jd , remains relatively constant and the tension force adjusts to provide internal moment equilibrium. This can be expressed as:

$$\frac{d(jd)}{dx} = 0 \quad \text{and} \quad V = jd\,\frac{d(T)}{dx} \tag{6.4}$$

where dT/dx is the shear flow across any horizontal plane between the reinforcement and the compression zone. For beam action to exist, this shear flow must exist.

In D-regions, the tension force remains constant and the lever arm adjusts to provide the internal moment equilibrium, as illustrated by:

$$\frac{d(T)}{dx} = 0 \quad \text{and} \quad V = T\,\frac{d(jd)}{dx} \tag{6.5}$$

This occurs, for example, if the shear flow cannot be transmitted due to the steel being unbonded, or if the transfer of shear flow is prevented by an inclined crack extending from the load to the reactions. In such a case the shear is transferred by arching action rather than beam action, as illustrated in Figure 6.3. In this member the compression force, C, in the inclined strut and the tension force, T, in the reinforcement are constant over the length of the shear span.

Generally, beams with high shear span to depth ratios, a/d, (i.e.,, a/d higher than 2.5) exhibit beam action, however, deep beams (i.e., a/d less than 2.5) exhibit arching action, where the assumption of linear distribution of strains over the depth of the section is not appropriate.

Therefore, the behaviors of deep (or short) beams and slender beams are different and accounting for arching action where it exists may increase the predicted capacities.

6.2.3.1 Behavior of Deep Beams

ACI defines members that have concentrated loads within twice the member depth from the support as deep beams (ACI, 2005). The behavior of deep members is governed by different mechanisms of failure than those influencing the behavior of more slender members. Deep members can sustain loads far in excess of those leading to first diagonal cracking (Alshegeir and Ramirez, 1992).

For deep members, the inclined crack at failure will span between the point load and the support reaction. In a deep beam, the tied-arch action, as discussed in the previous section, is very significant and can carry a much higher load than the diagonal cracking load. Prestressing can significantly increase the capacity of the deep beams against shear failure, and the formation of both flexural and diagonal cracks is delayed until loads of about twice the corresponding cracking loads for non-prestressed beams are reached (Teng et al, 1998). It is, therefore, important that the strands be properly anchored to develop the required prestress force and any additional tensile force due to the applied loading (Alshegeir and Ramirez, 1992). Particular attention should be placed at points where the prestressing steel conditions are changed, such as debonding or draping points, and at simply supported ends to properly develop the arching mechanism (Alshegeir and Ramirez, 1992).

Additionally, for beams with stirrups, as the shear span to depth ratios for beams decrease, stirrups contribute to the shear strength of deep members through aggregate interlock by controlling the width of the main diagonal cracks (Alshegeir and Ramirez, 1992).

6.2.3.2 Shear Test Results from Literature for Deep Beams

A limited number of shear tests on beams with short shear spans was found in the literature. In NCHRP Report 549 (Hawkins et al., 2005), as previously discussed in Section 2.6, the results of shear tests on prestressed concrete deep beams conducted by different researchers were included in their report. The tested beams summarized varied in concrete strength, beam depth, shear span to depth ratios, a/d, (at most 2.52), type of loading and amount of shear reinforcement. Table 6.11 shows the parameters for the tested beams in addition to the comparison of measured shear capacity to the capacity predicted by the 2002 AASHTO Standard Specifications.

As shown in Table 6.11, three primary types of failures were observed; failure due to crushing of concrete in the web (web crushing), web-shear cracking (shear-tension) failures, and failure due to loss of anchorage (strand slip). In addition to these three failure modes, the failure of two beams were identified as "interface failure," when the horizontal shear capacity of the web-bottom flange interface was exceeded. However, according to Russell et al. (2003), one of the interface failures was combined with concrete spalling in the webs, and the other one with web crushing at the lower end of the diagonal strut. Also, two beams did not fail before the capacity of the test equipment was reached during the tests.

The capacities of the beams were calculated according to the 2002 AASHTO Standard Specifications, and the ratio of measured to predicted capacities are summarized in Table 6.11. For comparison purposes the capacity reduction factor was set to unity, i.e., ϕ=1.0.

Figure 6.4 summarizes *the* $V_{test}/V_{n,STD2002}$ ratios with corresponding a/d ratios for different types of failures. As shown in the figure, the beams that failed due to strand slip or lack of anchorage near the end regions generally had ratios lower than or close to unity.

For three beams; I-3, I-4 and II-1, tested by Kaufman and Ramirez (1988), it was observed that once the shear cracking load of the web was exceeded, the crack extended down toward the support and crossed the tension steel. Slip was recorded in all strands and any attempt to increase the load was followed by continuing slippage of the strand until crushing of the compression block under the load-bearing plate (Kaufman and Ramirez, 1988). The authors stated that the web-shear crack destroyed the transfer length bond between the concrete and strand. Thus, anchorage of the lower tension chord of the truss was destroyed. This mode of failure was identified as shear-tension by Kaufman and Ramirez (1988). Thus, according to the definition of the failure mode by Kaufman and Ramirez (1988), those tested beams were included with the beams that failed due to strand slip. However, Beam I-2, also tested by Kaufman and Ramirez (1988), had some length of beam beyond the support to provide sufficient anchorage. This beam had a $V_{test}/V_{n,STD2002}$ ratio higher than unity.

Beams tested by Rangan (1991) and Ma et al. (2000) failed in the web of the beam by crushing of the struts, yielding test-to-predicted ratios above unity. Beams tested by both authors had end blocks. Rangan (1991) stated that the end blocks were provided by increasing the web width to the full flange width over 4 in. length at the positions of two-point loads as well as the supports. Beams were also shown to have 5 in. of overhang past the support. However, other means of anchorage of strands at the end of the beams were not indicated by Rangan (1991). In the study by Ma et al. (2000), the end blocks were cast at both ends of the specimens where strands were well anchored (i.e., bent into the end block) to avoid any premature failure due to strand slip.

Strands need to have sufficient anchorage to develop a tied-arch mechanism. Thus, attention should be placed on checking that the strand can develop the required tie (strand) strength near the support when modeling the girders with the 2004 AASHTO LRFD STM specifications.

6.2.3.3 Applicability of Arching Action

As discussed in the previous sections, arching action could account for higher shear capacity than predicted by the 2002 AASTO Standard Specification near the end regions of bridge girders provided that the applied load (i.e., the rear tandem wheel load of an HS-20 truck) is within 2.5 girder depths of the support (i.e., a/d less than 2.5, which is the generally accepted bound associated with deep beam behavior) and the strands have sufficient anchorage.

The $\frac{\phi V_{n,\,STD\,2002}}{V_u}$ ratios and shear rating factors at the critical section of the girders shown in Tables 5.4-5.9 were the lowest when the rear tandem load was applied at the critical section. Although this ratio was minimized when the load was located at the critical section, $\frac{\phi V_{n,\,STD\,2002}}{V_u}$ may still be less than unity for cases where the load is placed further than $2.5h_c$ from the support centerline, for which case arching action may not apply. The maximum distance of the application of the rear tandem load from the centerline of the support yielding $\frac{\phi V_{n,\,STD\,2002}}{V_u}$ less than unity at the critical section (sample calculations shown in Appendix C) is shown in Table 6.12 along with the distance $2.5h_c$ for a number bridges in the inventory with shear inventory rating factors less than 0.85. Most of the bridge girders had $\frac{\phi V_{n,\,STD\,2002}}{V_u}$ less than unity even when the rear tandem was placed further than $2.5h_c$ from the support. Therefore, although arching action is expected to provide unaccounted for shear capacity when the load is near the support, it will not increase the shear inventory rating factors above unity for most of the girders listed in Table 6.12.

6.3 Investigated Parameters for Reduced Shear Demand

Rating depends on both capacity and demand. The previous sections have concentrated on finding additional sources of shear capacity. This section is focused on finding better estimates for the demand through a review of the literature. The parameters investigated include:

- Live load distribution factors
- Effect of end diaphragms.

6.3.1 Live Load Distribution Factors for Shear

The effect of live load on the main longitudinal members of a bridge is a function of the magnitude and location of truck wheel loads on the bridge deck surface and of the response of the bridge to these loads. The concept of live load distribution factors permits design engineers to predict bridge response by uncoupling the longitudinal and transverse effects of wheel loads from each other (Huo et al. 2005).

The AASHTO Standard Specifications for Highway Bridges and the AASHTO LRFD Bridge Design Specifications contain the most common methods in use for determining live load distribution factors. Results from these methods have been compared to analytical methods and field investigations found in the literature. For most cases, the design-specified methods overestimate the demand on the individual girders, producing conservative results (Puckett et al., 2005).

Because this study focuses on the rating of existing bridges with prestressed concrete I-girders based on the shear provisions of the 2002 AASHTO Standard Specification, the conservatism of the live load distribution factors in the 2002 AASHTO Standard Specification was of interest. Extensive experimental and analytical research has been conducted on I-girder bridges to determine the live load distribution factors for moment (Barr and Amin, 2006). However, very limited research on live load distribution for shear was found, despite some agencies finding that shear controlled the load rating of their bridges (Al-Mahaidi et al., 2000).

Because the results of recently developed simplified methods would be expected to yield less conservative distribution factors, the findings from those methods were compared to the results obtained using the AASHTO Standard method. A number of investigated bridges presented in Chapter 5 and two example bridges from common practice were utilized for comparison purposes as discussed in Section 6.3.1.6.

6.3.1.1 AASHTO Standard Method

According to the 2002 AASHTO Standard Specification, lateral shear distribution for interior girders is determined using the expression

$$\frac{S}{D} \tag{6.6}$$

where S is the girder spacing in feet, up to a maximum of 14 ft and D is a factor based on bridge type. In the specifications, the values of D are given for a single line of wheels. For prestressed concrete bridges designed for one lane of traffic and for two or more traffic lanes, the values of D for interior beams are 7 and 5.5, respectively. For exterior beams, the distribution factors are obtained by using the lever rule (AASHTO, 2002).

For cases where the maximum member stresses are generated by loading a number of traffic lanes simultaneously, the 2002 AASHTO Standard Specification takes into account the improbability of coincident maximum loading, and thus allows for the use of the following percentages of live loads:

One or two lanes loaded........... 100%
Three lanes loaded.................... 90%
Four or more lanes loaded........ 75%

Although the formulas presented in the AASHTO Standard Specifications are simple, some researchers have suggested that they can result in highly unconservative shear distribution factors (40% lower when compared to a finite element analysis) in some cases and may result in conservative values (50% higher when compared to a finite element analysis) in other cases (Zokaie and Imbsen, 1993). NCHRP Project 12-26 *Distribution of Wheel Loads on Highway Bridges* (Zokaie et al., 1991) found some inconsistencies in the AASHTO Standard Specification live load distribution factors. These inconsistencies include inconsistent reduction in load intensity for multiple lane loading, inconsistent changes in distribution factors for changes in design lane width, and inconsistencies in determination of wheel load distribution factors for different bridge types (Huo et al., 2003). The AASHTO Standard Specification simplified formulas were developed for non-skewed simply-supported bridges. Although these specifications state that they can be applied to the design of normal highway bridges, there are no additional guidelines regarding their applicability.

6.3.1.2 AASHTO LRFD Method

The AASHTO LRFD equations for live load distribution factors were developed under NCHRP Project 12-26 *Distribution of Wheel Loads on Highway Bridges*. The equations for live load distribution factors contained in the AASHTO LRFD Specifications are significantly different from those in the AASHTO Standard Specifications.

Equations 6.7 and 6.8 define the distribution factors for shear in I-girder bridges for one lane loaded and two or more lanes loaded, respectively, when the girder spacing is between 3.5 and 16ft, the span length is between 20 and 240 ft, the slab thickness is between 4.5 and 12 in., and the stiffness parameter given by $K_g = n\left(I + Ae_g^2\right)$ is between 10,000 and 7,000,000 in^4, where n is the modular ratio between the beam and deck concrete; I is the moment of inertia of the beam in^4; A is the area of girder, in^2; and e_g is the distance between the centers of gravity of the beam and deck, in.

$$g_{i\_1} = 0.36 + \frac{S}{25.0} \tag{6.7}$$

$$g_{i\_2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0} \tag{6.8}$$

where g_{i_1} is the shear live load distribution factor for an interior girder for one design lane loaded; g_{i_1} is the shear live load distribution factor for an interior girder for two or more design lanes loaded; S is the girder spacing in ft. When the number of girders is less than four, the AASHTO LRFD Specification states that the lever rule should be used.

The shear live load distribution factor for exterior girders without rigid midspan diaphragms for two or more traffic lanes is given as

$$g_{e\_2} = \left(0.6 + \frac{d_e}{10}\right) g_{i\_2} \tag{6.9}$$

where g_{e_2} is the shear live load distribution factor for exterior girder for two or more lanes loaded; d_e is the distance from the exterior web of the exterior beam to the interior edge of the curb or traffic barrier, ft, applicable for $-1.0 \le d_e \le 5.5$ ft. Distribution factors for shear in exterior girders where one design lane is loaded should be determined by the lever rule. The following multiple presence factors are to be included when using the lever rule:

One lane loaded..................... 100%
Two lanes loaded.................... 90%
Three lanes loaded.................. 75%
Four or more lanes loaded...... 65%

Other than for this case, multiple presence factors are incorporated into the live load distribution factor equations given in the AASHTO LRFD Specification for single- and multiple-lanes loaded.

The AASHTO LRFD Specification state that for bridge superstructures with diaphragms and cross frames, the distribution factor for the exterior beam shall not be less than that obtained by assuming that the cross section deflects and rotates as a rigid cross section.

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2} \tag{6.10}$$

where R is the reaction on the exterior beam in terms of lanes; N_L is the number of loaded lanes; e is the eccentricity of a design truck or a design lane load from the center of gravity of the pattern of girders; x is the horizontal distance from the center of gravity of the pattern of girders to the exterior girder; X_{ext} is the horizontal distance from the center of gravity of the pattern of girders to the exterior beams and N_b is the number of girders.

This pile analogy method implies some degree of transverse bending stiffness. At the same time the transverse and torsional superstructure stiffnesses associated with plate bending theory are ignored which may lead to the overconservative nature of this method (Huo et al., 2003). Also, according to a study by Tobias et al. (2004), this pile analogy analysis technique is not recommended for use in Illinois until further research is conducted due to the overconservative nature of the method. For these reasons, the live load distribution factors calculated by the pile analogy analysis were not investigated further in the present study.

The AASHTO LRFD Specifications state that the shear in the exterior beam at the obtuse corner of the bridge shall be adjusted when the line of support is skewed. Thus, the AAHTO LRFD method provides a skew increase factor for shear live load in both interior and exterior girders. The skew correction factor is given as

$$SCF_s = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3} \tan\theta \tag{6.11}$$

where SCF_s is the skew correction factor for shear; and θ is the skew angle. Equation (6.11) is only applicable to bridges with $0° \leq \theta \leq 60°$ and the same range of values of, S , L and N_b as given for Eqns. (6.7) and (6.8).

According to a study by Zokaie et al. (1993) it was found that the AASHTO LRFD formulas generally produced results that were within 5% of the results of a finite element analysis. The formulas presented in AASHTO LRFD for calculating distribution of live load shear are believed to be more complex and more accurate than the AASHTO Standard method in that they include the effects of several parameters (Huo et al., 2003).

The distribution factor formulas in the AASHTO LRFD include limited ranges of applicability. However, the equations become less accurate when the ranges of applicability are exceeded. It is mandated by the AASHTO LRFD specifications that a refined analysis such as finite element analysis or grillage analysis, be used to determine the distribution factors when these ranges are exceeded.

6.3.1.3 Henry's Equal Distribution Factor Method

Henry's equal distribution factor (EDF) method (Tennessee, 1996) is by far the simplest of all methods investigated. A former engineer of the Structures Division, Tennessee Department of Transportation (TDOT), Henry Derthick, developed this simplified method for calculating live load moment and shear distribution factors. Henry's method assumes equal distribution of live load effects to all beams, including interior and exterior beams. Because Henry's method requires only the width of the roadway, W , number of traffic lanes, N_L , number of beam lines, N_g , and the multiple presence factor, m , of the bridge, it can be applied without difficulty to different types of superstructures and beam arrangements. For most bridges, the distribution factors obtained from Henry's method are smaller than those obtained from the AASHTO Standard Specifications (Huo et al., 2003). Tennessee DOT specifications state that the designer should use the smaller value of lateral distribution factor of live load determined from the AASHTO Standard Specifications Article 3.23 or Henry's method in the design of primary beams (Huo et al., 2003). Thus, the majority of Tennessee bridges have been designed using Henry's EDF method for nearly four decades. The procedure of the equal distribution factor method for prestressed I-beams is as follows (Huo et al., 2003):

Step 1: Basic Equal Distribution Factor

a) Divide roadway width by 10 ft to determine the fractional number of traffic lanes.

b) Reduce the value from (a) by a factor obtained from a linear interpolation of multiple presence factors to determine the total number of traffic lanes considered for carrying live load on bridge. For multiple presence factors, AASHTO Standard specifications are utilized.

c) Divide the total number of lanes by the number of beams to determine the number of lanes of live load per beam, or the distribution factor of lane load per beam.

Step 2: Shear Factor Modification - Shear Distribution Factors

d) Multiply the value from (c) by a ratio of 6/5.5 to determine the distribution factor of wheel load per beam.

The multiplier 6/5.5 in Step 2 is used to amplify the distribution factor for steel and prestressed I-beams because the live load distribution factor to those types of beams is expected to be higher than the value obtained in Step 1.

6.3.1.4 Modified Henry's Method

The accuracy of Henry's method was reexamined and modification factors were developed through a comparison and evaluation study conducted by Huo et al. (2003). Twenty-four Tennessee bridges with six different types of superstructures, labeled Database 1 bridges, were selected for the comparison study. The results from Henry's method were compared to finite element analysis (FEA) results and other AASHTO equation results. Similar to the findings from other superstructure types, Henry's method was found to be very unconservative for shear distribution for precast concrete I-beam and bulb-tee bridges compared to FEA results.

The modification factors to Henry's method were initially developed based on a comparison between distribution factors from Henry's method and finite element analysis for Database 1. This database contained three precast concrete I-beam and four precast concrete bulb-tee girder bridges. Tables 6.13 and 6.14 show the values of shear distribution factors for exterior and interior girders, respectively, in precast concrete I- and bulb-tee beam bridges determined by FEA and simplified methods. As shown in Table 6.13, for exterior girders, the AASHTO LRFD Specification had conservative values compared to FEA and modified Henry's method gave less conservative values compared to the AASHTO LRFD but all were higher than the FEA results. The AASHTO Standard Specification yielded comparable results to FEA except for two bulb-tee girders, which had very unconservative distribution factors. For interior girders, as shown in Table 6.14, the modified Henry's method gave the most unconservative results compared to FEA for all girders. The AASHTO Standard had similar results compared to modified Henry's method. The AASHTO LRFD had unconservative values for bulb-tee girders compared to FEA; however, gave conservative results for I-beams.

The preliminary modification factors were calibrated according to the comparison between Henry's method and the AASHTO LRFD method for 419 real bridges that were analyzed in the NCHRP Project 12-26 (Zokaie and Imbsen, 1993) named Database 2. This database contained 30 precast concrete I-beam bridges and 36 precast concrete bulb-tee beam bridges. Investigation of these precast concrete I- and bulb-tee beams showed that the AASHTO LRFD method gave slightly conservative results compared to the modified Henry's method (The ratio of AASHTO LRFD to Henry's method had a mean value of 1.04 and a standard deviation of 0.2).

Two sets of modification factors for shear distribution were recommended. The first set included a single shear factor applicable to all structure types. The second set of modification factors included separate sets of factors for moment and shear. The effects of skew and span length were included in the second set of modification factors.

Henry's method for precast prestressed I-beams was modified based on the results of calibration with FEA analysis performed by Huo et al (2003) as follows.

Step 1: Basic Equal Distribution Factor

a) Proceed with the parts (a) through (c) of Step 1 shown previously for unmodified Henry's method.

Step 2: Superstructure Type Modification for shear

d) Multiply the value from (c) by 1.20, the structure modification factor for precast concrete sections, to obtain the shear distribution factor.

Step 3: Skew Angle Modification

e) Multiply the value from (d) by the skew modification factor; $(1.0 + 0.2 \tan\theta)$, where θ is the skew angle in degrees, for skewed bridges to get the final shear distribution factor.

For shear distribution, the unconservative Henry's method has been brought closer to the accurate finite element analysis through the use of these modification factors (Huo et al., 2003).

The modified Henry's method offers obvious advantages over the AASHTO Standard and AASHTO LRFD methods (Huo et al., 2003).

6.3.1.5 NCHRP Project 12-62

The goal of NCHRP Project 12-62 *Simplified Live Load Distribution Factor Equations* was to determine simpler, and possibly more accurate, methods to estimate transverse live load distribution in bridges. In NCHRP 12-62, literature and design specifications were reviewed and summarized in NCHRP Report 592 (Puckett et al., 2007).

Bridge data from four independent sources were used, NCHRP Project 12-26 (Zokaie and Imbsen, 1993) which is the basis for the current AASHTO LRFD distribution factors (809 bridges), a report on 24 bridges from Tennessee Tech, which were the same bridges used in the study by Huo et al. (2003), bridges entered into AASHTO *Virtis* and obtained from several departments of transportation (653 bridges); and a set of bridges to push the limits of reasonable application (74 bridges).

The following simplified methods for live load distribution available in the literature were reviewed in detail and compared to grillage analyses in Appendices D and N of Report 592, respectively:

1) AASHTO Standard Specifications (S over D formulas)
2) AASHTO LRFD Specifications
3) Lever Rule
4) Uniform Distribution Factor Method (Number of Lanes / Number of Girders)
5) Modified Henry's Method Distribution Factor
6) Work presented by Bakht and Jaeger (1992), which is basis for the current relatively simple Canadian Highway Bridge Design Code (CHBDC) method (CHBDC, 2000)
7) Work presented by Sanders and Elleby (1970) in NCHRP Report 83, which used orthotropic plate theory

As an example, Figures 6.5 through 6.7 show plots of the lever rule, AASHTO Standard method, and AASHTO LRFD method, respectively, compared to results of grillage analyses for shear live load distribution in an interior girder. It is important to note that no multiple presence or analysis factors (discussed later) have been included in the calculations of simplified distribution factors for comparison. As a measure of comparison among the methods, the correlation coefficient (i.e., R^2, given in top left corner of the figures) was extensively used by NCHRP Project 12-62. Based on the value of the correlation coefficient (R^2) between each simplified method and the grillage analysis, a calibrated version of the lever rule was proposed as a simplified method for shear live load distribution factors for slab on concrete I-girder bridges.

Figure 6.8 shows a plot of the calibrated lever rule for shear in an interior girder compared to grillage analysis. Tables 6.15 and 6.16 show the shear distribution factors calculated using the simplified methods compared to the grillage analysis results for exterior and interior girders, respectively, for slab on concrete I-girder bridges. The Canadian Highway Bridge Design Code method and method presented by Sanders and Elleby were excluded from the comparisons due to high scatter in the results. Except for the case of the interior girder with multiple lanes, the AASHTO Standard method was found to be the most conservative method and very high standard deviations were obtained for exterior girders indicating scattered results for this case. The AASHTO LRFD method and modified Henry's method performed well in comparison to the grillage analyses for shear distribution in the interior girders; however, poor results were obtained for exterior girders. As mentioned, no multiple presence or analysis factors, γ_a, were included in the comparison of the simplified methods and calibrated lever rule to grillage analyses.

Because the recommended simplified method for shear was based on the lever rule, the lever rule is described here. The lever rule is an approximate distribution factor method that assumes no transverse deck moment continuity at the interior beams, which renders the transverse deck cross section statically determinate. Direct equilibrium is used to determine the load distribution to the beam of interest. Lever rule formulas and example derivations of two lever rule equations are provided to facilitate lever rule computations in NCHRP Report 592. Equations were derived assuming constant 4-foot spacing between multiple vehicles.

For one lane and multiple lanes loaded, the proposed lever rule equation for shear live load distribution is

$$mg_v = m\gamma_a\left[a_v\left(g_{lever\_rule}\right) + b_v\right] \geq m\left[\frac{N_{lanes}}{N_g}\right] \tag{6.12}$$

where mg_v is the shear distribution factor with multiple presence; m is the multiple presence factor as specified in 2004 AASHTO LRFD Article 3.6.1.1.2 (3); γ_a is the analysis factor, also defined as the distribution simplification factor (DSF); a_v and b_v are calibration constants for shear and reactions, respectively; g_{lever_rule} is the distribution factor computed by the lever rule; N_{lanes} is the number of design lanes considered in the lever rule analysis; and N_g is the number of girders.

The multiple presence factors are applied explicitly after the distribution factor has been computed. The calibration constants for shear, a_v and b_v, are given in Table 6.17 for precast concrete beams. The last term in Eqn. (6.12) was included to represent the theoretical lower bound of a uniform distribution of live load.

For the calibrated lever rule, only one and two lanes loaded were considered. A study, presented in Appendix O of the Report 592, determined that when the multiple presence factors were included in the distribution factor values, the two-lane loaded case typically controlled. The difference between the two- and three-lanes loaded was found to be small when the three-lanes-loaded case controlled. The computation of the distribution factor was significantly simplified because the three-or-more-lanes loaded cases need not be considered. Thus, only one- and two-lanes loaded were the multiple presence factors used.

The shear skew adjustment factor is given as

$$SCF = 1.0 + 0.09\tan\theta \tag{6.13}$$

where SCF is the skew correction factor and θ is the skew angle in degrees.

Although the recommended method worked well for most cases, the analysis factor, γ_a, as shown in Table 6.18 was applied to the distribution factors due to significant variability observed. The analysis factors were calibrated so that the mean of the differences between the distribution factors and the results of the grillage analyses was zero.

Some limitations were indicated in the research by the authors of Report 592. For instance, the use of the proposed specifications was not recommended for direct application to the evaluation of existing structures because they were developed for design use. Puckett et al. (2007) further emphasized that simplifications that were inherent in this study might not be appropriate for decisions associated with a bridge closure, retrofit/maintenance, or permit vehicle assessment. Also, a study on the implementation within Load Resistance and Factor Rating (LRFR) was advised, specifically determining the appropriate analysis factors (or distribution simplification factors) consistent with other aspects of reliability calibration with the LRFR.

6.3.1.6 Comparison of the Simplified Methods with Selected Bridges

The AASHTO LRFD method, the calibrated lever rule (referred to as NCHRP method herein) and the modified Henry's method were compared to the AASHTO Standard method for a set of precast concrete I-girder bridges. The set included ten bridges from the Mn/DOT inventory, one example bridge from the PCI Bridge Design Manual (2003) and one from the Florida DOT LRFD design manual. Information on these bridges is given in Table 6.19. Appendix E contains sample calculations for the shear live load distribution factors using the simplified methods.

Figures 6.9 and 6.10 show comparisons of the live load distribution factors (LLDF) calculated using the simplified methods to those calculated using the AASHTO Standard method for exterior and interior girders, respectively.

Figure 6.9 clearly shows that, almost all simplified methods estimate higher shear distribution factors than the AASHTO Standard for exterior girders. The AASTHO LRFD method was found to give the highest values compared to the other two simplified methods. This finding agrees well with the results shown in Table 6.13 from the study by Huo et al. (2003). For exterior girders in all bridges, it was also observed that one-lane loaded was the controlling case for the AASHTO LRFD method and two-lanes loaded was the controlling case for the AASHTO Standard method. Compared to the AASHTO Standard method, slightly higher distribution factors were obtained both for the modified Henry's and NCHRP methods for the exterior girder. Huo et al. (2003) had also found more conservative results obtained from the modified Henry's method than the AASHTO Standard method when both compared to FEA results (Table 6.13), however, Puckett et al. (2007) had shown that the AASHTO Standard method was far more conservative than the NCHRP method when compared to a grillage analysis (Table 6.15).

For interior girders, the two-lanes-loaded case controlled for all bridges for all simplified methods used. Figure 6.10 shows that only the modified Henry's method, in general, yielded distribution factors smaller than those from that obtained from AASHTO Standard method. This agrees with the results shown in Table 6.14 by Huo et al. (2003). The last column in Table 6.14 for interior girders shows that the modified Henry's method generally gave unconservative results compared to the FEA results. The NCHRP method yielded the largest distribution factors as shown in Figure 6.10, which could be attributed to the use of an analysis factor for the two-lanes-loaded case as shown in Table 6.18.

In conclusion, any change from the AASHTO Standard method to one of the simplified methods discussed above would likely result in higher shear live load distribution factors in the exterior girder. For the interior girder, higher distribution factors would likely be obtained if the AASHTO LRFD or NCHRP methods were utilized. Although the modified Henry's method provided distribution factors less than those obtained from the AASHTO Standard method, it was concluded to be unreliable to use the modified Henry's method because of the unconservative results obtained by Huo et al. (2003) compared to FEA.

6.3.2 Effect of End Diaphragms on Shear LLDF

According to Puckett et al. (2005), consideration of secondary elements, such as diaphragms and barriers, has been shown to make a significant difference in lateral load distribution in some cases. Among those secondary elements, the presence of end diaphragms would be expected to affect the shear live load distribution near end regions of girders; likely to provide more uniform and reduced shear live load distribution factors (LLDF).

As presented in Chapter 5, most of the investigated girders with low shear inventory ratings had their worst ratings near the ends. The shear LLDF's calculated at the critical section directly affect the shear demand and, thus, the capacity-to-demand ratio and shear inventory rating factor. Because the shear demand would be affected by the presence of end diaphragms as mentioned above, the results obtained from literature on the effect of end diaphragms on shear LLDF's are presented and discussed in this section.

Effects of end diaphragms for most of the bridges were studied by Huo et al. (2003). The pier and abutment supports were modeled with and without support diaphragms using the finite element analysis program, ANSYS. According to Huo et al. (2003), the distribution factors from the analysis without diaphragms were normally larger than those with diaphragms.

End diaphragms were modeled as beam elements. In Huo's study, the AASHTO Standard HS20-44 truck loading or HL-93 truck loading in the AASHTO LRFD Specifications were considered. As many trucks as possible were placed on a bridge in the transverse direction depending on the width of the bridge. Shears were determined after the addition of each truck until the maximum values were obtained. The AASHTO Standard intensity reduction factors were used for three and four truckload results (0.9 and 0.75, respectively). Trucks were moved in both the longitudinal and lateral directions on each bridge and shear on the beams was calculated.

For skewed bridges, the first truck was moved until its location of maximum influence for the beam under investigation was found. The trucks were placed at locations near to the supports because the maximum shear usually occurs very near to the abutments or piers. The bridge was loaded with one, two or three trucks and the position of maximum shear was found by moving these trucks independently as well as together.

In the case of non-skewed bridges it was found that the second and the third trucks should be placed alongside the first truck to produce maximum shear. For skewed bridges, the second truck had to be placed away from the first truck longitudinally and both trucks were moved dependently to obtain the maximum shear. Figure 6.11 shows the sample loading patterns for live load shear on non-skewed and skewed precast concrete I-girder bridges.

Table 6.20 shows the effect of end diaphragms on precast concrete I- and bulb-tee beams both in exterior and interior girders from Huo's study (2003). As shown, the end diaphragms generally reduced the distribution factors, however, the effectiveness ranged from a negligible amount, i.e., 0.1 %, to a noticeable amount, i.e., approximately, 17%.

Effects of end diaphragms were also studied by Puckett et al. (2007) as presented in Appendix L of Report 592. The bridge used for comparison was Bridge No. 24, which was used by Huo et al. (2003). The plan views of Bridge No. 24 with and without considering skew are shown in Figures 6.12 and 6.13, respectively. The bridge was modeled for all loading patterns with and without end diaphragms using the analysis program, SAP2000. The bridge was remodeled with the finite element analysis program ANSYS for a sample loading pattern to verify the results obtained from SAP2000. For Puckett's study, only fatigue loading was considered. This loading consisted of a single design truck that had the same axle weights as an AASHTO Standard HS20-44 truck loading, but instead of a variable spacing from 14.0 to 30.0 ft. between the 32 kip axles, it had a constant spacing of 30.0 ft.

Each model was analyzed with and without support diaphragms. Skew angles of 0, 30, and 60 degrees were used. The single fatigue loading truck was moved in the transverse direction at 1 ft increments away from the curb as shown in Figure 6.14. The distribution factors for shear were obtained at the girder support as described by Puckett et al. (2007). The location reference was based on 10^{th} points of the span. For example, location 100 indicated a point 0 percent along the length of the span, or, at the left end of the span which was used as the critical location for calculating the maximum shear distribution factors. The FEA results for shear LLDFs were also obtained at the end of the span in the study by Huo et al. (2003).

Tables 6.21 and 6.22 show the effects of end diaphragms on shear distribution factors of Bridge No. 24 for different skew angles with different loading patterns for exterior and interior girders, respectively. Puckett found that the support diaphragms caused an increase in the shear distribution factor in direct conflict with Huo's study. The increase in the shear distribution factor due to support diaphragms was found to be small, in general. For skewed bridges, the effect of support diaphragms on shear distribution at the obtuse corner of the bridge was negligible. It was not shown by the authors, but they indicated that support diaphragms caused a decrease in shear values at the acute corner; however the values were not critical since the obtuse corner controlled.

The literature showed conflicting results on the effect of end diaphragms on shear live load distribution factors with respect to the degree of effectiveness. Although the researchers analyzed the same bridge, Bridge No. 24 in this case, in one study decreases of up to near 17% and in the other study small increases were obtained for the shear live load distribution factors when the end diaphragms were included. Hence, no conclusion can be made on the effect of end diaphragms on live load distribution factors for shear and a recommendation is made to determine the effect of end diaphragms on shear live load distribution factors experimentally.

Chapter 7. Summary, Conclusions and Recommendations

7.1 Summary and Conclusions

Many prestressed concrete girders in the Mn/DOT bridge inventory have rated low for shear. Although some girders have had shear inventory ratings below unity, no visible signs of shear distress have been noted in any of the girders. To determine adequate safety for prestressed concrete bridges, many DOTs rely most heavily on the operating rating, and in this study, all of the girders had shear operating rating factors based on HS-20 above unity. Although there has not been concern with regard to the shear capacity of prestressed concrete bridge girders, there was interest in better understanding the reasons for the inconsistency in the low shear rating results relative to the observed good performance.

One of the reasons for the discrepancy between the good performance and low shear inventory ratings was suspected to be conservatism in the rating methods (i.e., LFR). Other suspected reasons included potential flaws in the rating tools used by Mn/DOT (i.e., *Virtis*-BRASS software) including neglecting possible additional shear capacity parameters (e.g., end blocks). These issues have made it difficult to discern cases for which shear capacity may be a concern.

To identify potential errors in the *Virtis*-BRASS rating tool, five sample bridges were selected to compare load ratings obtained from the software to hand computations. The errors found in *Virtis*-BRASS software were summarized. An error related to the incorrect calculation of concrete resistance to web-shear was discovered to cause the shear rating factors to be underestimated by up to 25 percent at the critical section for shear, up to 35 percent at the end of the transfer length and up to 15 percent at sections away from the critical section provided that web-shear cracking governed for shear.

From the results of NCHRP Project 12-61, *Simplified Shear Design of Structural Concrete Members*, it was shown that the 2002 AASHTO Standard Specifications provided reasonable predictions of shear capacity with a low coefficient of variation in the test to predicted shear capacity ratios, and thus was found to be a useful tool for predicting the shear capacity of prestressed concrete members. Conversely, the 1979 Interim revisions of the AASHTO Standard Specifications did not provide reliable results for predicting shear capacity.

Another objective of this project was to investigate the applicability of a screening method to determine the bridge girders most at risk for being undercapacity for shear. When investigating the application of the screening tool developed in the study by Runzel et al. (2007), it was determined that some girders in the Mn/DOT inventory designed by the 1979 Interim did not have stirrup spacings that met the 1979 Interim design requirements. As a result, it was not possible to effectively implement a screening method to determine girders most at risk; however, priority should be given to older girders with span/spacing (L/S_g) less than 10, as recommended by Mn/DOT Report 2007-47.

In reviewing the capacity of the prestressed bridge girders, particular attention should be paid to the stirrup spacings exceeding $h/2$ within the span. In these cases, the calculated shear resistance provided by the transverse reinforcement, V_s, should be discounted due to the likelihood of a shear crack not being intercepted by a stirrup.

Potential sources of additional shear capacity not considered by the design specifications and the bridge rating tool used by Mn/DOT (*Virtis*/BRASS) were investigated including the contribution from end blocks, the differences between nominal and measured 28-day concrete strengths, the effect of increase in concrete strength with time, and the effect of short shear spans (or arching action). Additionally, potential sources of conservatism in shear demand were also investigated including methods for determining live load distribution factors and the effect of end diaphragms on live load distribution factors.

Investigation of the girders in the Mn/DOT inventory revealed that end blocks were used in deeper sections, whereas, no end blocks were found in the shallower sections. In general, the deeper girders had higher shear capacities and higher shear inventory ratings even without considering the contributions from end blocks. Thus, detailed calculations for the end block contribution will not likely affect the shear inventory ratings.

Differences between the design and measured 28-day concrete cylinder strengths obtained from a local precasting plant in Minnesota and from the literature were investigated, and the companion statistical parameters were compared. It was found that the bias and coefficient of variation of the measured to specified strength from the local precasting plant concrete was no different from other precast plants and should not be used as reserved strength, as the increase in measured 28-day strength over the specified design strength is already accounted for in the 2002 AASHTO Standard Specifications.

Data from literature demonstrating the aging effect on concrete strength was investigated. All of the bridges from the Mn/DOT inventory studied had ages in excess of 20 years. A lower bound of 20% increase in concrete compressive strength over 20 years was conservatively recommended. In general, considering the lower bound of 20% increase in $f_c^{'}$ resulted in a small increase in the shear capacities (about 2-5%) predicted by the 2002 AASTHO Standard Specification for the Mn/DOT girders investigated.

Possible reserve shear strength was investigated near the end regions of the girders due to arching action. However, it was found that the critical section had inventory rating factors less than unity even when the load was placed away from the support, so inclusion of the effects of arching action would not improve those rating factors.

The AASHTO Standard method for computing the shear live load distribution factors was investigated by comparing its predictions with recently developed simplified methods and finite element analysis results available in the literature. It was found that any change from the AASHTO Standard method to one of the simplified methods would likely result in higher shear live load distribution factors in the exterior girder, where shear inventory ratings are currently not controlling. However, this was found to be less likely to happen when the NCHRP recommended lever-rule method (as discussed in Section 6.3.1.5) was used instead of the AASHTO Standard method. For the interior girder, higher distribution factors would likely be obtained when the AASTHO LRFD or the NCHRP recommended lever-rule method was utilized. No experimental verification of live load distribution factors for shear was found in the literature. The effect of existing end diaphragms on shear live load distribution was also investigated through the findings from literature. The literature showed conflicting results on the effect of end diaphragms on shear live load distribution factors, with one numerical study showing a decrease in LLDFs in the presence of end diaphragms, and a different numerical study showing an increase in LLDFs in the presence of end diaphragms for the same investigated bridge.

Based on the summarized findings recommendations are given in the next section.

7.2 Recommendations

Virtis-BRASS should be used for shear rating of bridges in the Mn/DOT inventory. The errors found in *Virtis*-BRASS have been reported to the vendor and the code has been revised.

Regarding potential sources of increased shear capacity, detailed calculations for the end block contribution are not recommended as they were not found to have much of an impact on the shear ratings. Concrete strength gain with time was found to provide a slight increase in shear capacity. Concrete compressive strengths of girders that are at least 20 years old can be assumed to increase by 20% from the nominal 28-day concrete compressive design strengths, which will likely produce a 2 to5% increase in shear capacity. Consequently, it is reasonable to increase the *Virtis* calculated shear inventory and operating ratings by approximately 6% to account for this effect.

The accuracy of the shear live load distribution factors needs to be further assessed by conducting field studies. These tests could also be used to resolve the conflicting results of the numerical studies found in the literature regarding the effect of end diaphragms on shear live load distribution factors.

The screening tool recommended by the previous Mn/DOT sponsored companion study (Mn/DOT Report 2007-47 by Runzel et al., 2007) is not applicable for the girders in the Mn/DOT inventory because of the large number of girders that did not meet the requirements of the specification in effect at the time of design. All bridges should be checked and rated individually; however, priority should be given to evaluating girders with small span-to-spacing ratios (i.e., $L/S_g < 10$). In addition, particular attention should be paid to girders which have stirrup spacings in excess of $h/2$ within the span. In these cases, the calculated shear resistance provided by the transverse reinforcement, V_s, should be discounted due to the likelihood of a shear crack not being intercepted by a stirrup.

Shear rating at the critical section (i.e., $h/2$ from the face of support) is a good indicator for shear rating throughout the girder, thus, if low shear rating factors (below unity) are obtained at the critical section for a girder, the rating of the girder should be checked at other points of interest throughout the span. If the girder rates acceptably for shear at the critical section, it is likely that it will rate throughout, unless the stirrups located within the span are not likely to intercept a potential shear crack due to spacings in excess of $h/2$.

When selecting bridges for additional visual inspection, Mn/DOT should consider not only the shear inventory and operating ratings, but also the Heavy Commercial Average Daily Traffic (HCADT) counts on the bridge. Bridges with higher HCADT are more likely to see higher live loads than bridges with small HCADT due to the increased probability of multipresence truck loading.

When performing the inspections, it should be noted that diagonal cracking that may have developed under the presence of overload, may not be visible in the absence of the overload due to the effect of the prestress. If possible, heavy sand trucks should be placed on the bridge, or run over the bridge when the inspections are taking place.

References

AASHO, *Standard Specifications for Highway Bridges*, 8th edition, Washington, D.C., 1961.

AASHO, *Standard Specifications for Highway Bridges*, 9th edition, Washington, D.C., 1965.

AASHO, *Standard Specifications for Highway Bridges*, 10th edition, Washington, D.C., 1969.

AASHO, *Standard Specifications for Highway Bridges*, 11th edition, Washington, D.C., 1973.

AASHTO, *Standard Specifications for Highway Bridges*, 12th edition, Washington, D.C., 1977.

AASHTO, *Interim Specifications: Standard Specifications for Highway Bridges*, Washington, D.C., 1979.

AASHTO, *Standard Specifications for Highway Bridges*, 13th edition, Washington, D.C., 1983.

AASHTO, *Standard Specifications for Highway Bridges*, 14th edition, Washington, D.C., 1989.

AASHTO, *AASHTO LRFD Bridge Design Specifications*, 1st edition, Washington, D.C., 1994.

AASHTO, *Manual for Condition Evaluation of Bridges* (MCE), 2nd Edition, 1994.

AASHTO, *Standard Specifications for Highway Bridges*, 16th edition, Washington, D.C., 1996.

AASHTO, *AASHTO LRFD Bridge Design Specifications*, 2nd edition, Washington, D.C. 1998.

AASHTO, *Standard Specifications for Highway Bridges*, 17th edition, Washington, D.C., 2002.

AASHTO, *AASHTO LRFD Bridge Design Specifications*, 3rd edition, Washington, D.C., 2004.

AASHTO, *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges,* 1st Edition and Interim, Washington, D.C., 2005.

ACI Committee 318. B*uilding Code Requirements for Structural Concrete* (ACI 318-05) and *Commentary* (ACI 318R-05). American Concrete Institute, Farmington Hills, MI., 2005.

ACI Committee 209, *Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures* (ACI 209R-92), Farmington Hills, MI., 1992.

Al-Mahaidi, R., Taplin, G., and Giufre, A., "Load Distribution and Shear Strength Evaluation of an Old Concrete T-Beam Bridge," *Transportation Research Record* No. 1696, 2000, 52-62.

Alshegeir A., and Ramirez, J., "Strut-tie Approach in Pretensioned Deep Beams," *ACI Structural Journal*, V. 89, N. 3, May-June 1992, 296-304.

ASCE-ACI Committee 426, "The Shear Strength of Reinforced Concrete Members." *Journal of the Structural Division*, ASCE, V. 99, N. ST6, 1973, 1091-1187.

Bakht, B., and Jaeger, L.G., "Simplified Methods of Bridge Analysis for the Third Edition of OHBDC," *Can. J. Civ. Eng.*, V. 19, N. 4, 1992, 551–559.

Barr, P.J., and Amin, N., "Shear Live-Load Distribution Factors for I-Girder Bridges," *Journal of Bridge Engineering*, V. 11, N. 2, March/April 2006, 197-204.

Bruce, R.N.; Russell, H.G.; and Roller, J.J., *Fatigue and Shear Behavior of HPC Bulb-Tee Girders*" Interim Report - Louisiana Transportation Research Center, Research Report No. FHWA/LA. 03/382, Baton Rouge, LA, 2003.

Chen, W.F., and Duan, L., "Mainntenance, Inspection and Rating," In *Bridge Engineering Handbook*, CRC Press LLC, Boca Raton, FL, 1999, pp. 49-1, 49-30.

Colorado Department of Transportation, *Bridge Rating Manual*, Denver, CO, 1995.

FHWA National Bridge Inventory Data. 2007, "Deficient Bridges by State and Highway System," http://www.fhwa.dot.gov/bridge/deficient.htm, Accessed March 28, 2008.

Friedland, I.M., and Small, E.P., "FHWA Bridge Research and Technology Deployment Initiatives," P*roc., 2003 Mid-Continent Transportation Research Symposium,* Iowa State University, Ames, IA, 2003.

Halsey, J.T., and Miller, R.A., "Destructive Testing of Two Forty-Year-Old Prestressed Concrete Bridge Beams," *PCI Journal*, V. 41, N. 5, September-October 1996, 84-93.

Hawkins, N.M., Kuchma, D.A., Mast, R.F., Marsh, M.L., and Reineck, K.H., *NCHRP Report 549 Simplified Shear Design of Structural Concrete Members*. Transportation Research Board, 2005.

Huo, X.S., Conner, S.O., and Iqbal, R., *Re-examination of the Simplified Method (Henry's Method) of Distribution Factors for Live Load Moment and Shear*, Tennessee DOT Project No. TNSPR-RES 1218, Tennessee Technological University, Cookeville, TN, 2003.

Kaufman, M.K., and Ramirez, J.A., "Re-Evaluation of the Ultimate Shear Behavior of High-Strength Concrete Prestressed I-Beams," *ACI Structural Journal*, V. 85, N. 3, May-June 1988, 295-303.

Ma, Z., Tadros, M.K., and Baishya, M., "Shear Behavior of Pretensioned High Strength Concrete Bridge I-Girders," *ACI Structural Journal*, V. 97, N. 1, Jan. – Feb 2000, 185-192.

MacGregor, J.G., *Reinforced Concrete Mechanics and Design*, Prentice-Hall Inc., Englewood Cliffs, NJ, 1997.

McIntyre, M., and Scanlon, A., "Interpretation and Application of Core Test Data in Strength Evaluation of Existing Concrete Bridge Structures," *Canadian Journal of Civil Engineering*, V.17, 1990, 471-480.

Murdock, J.W., and Kesler, C.E., "Effect of Length to Diameter Ratio of Specimen on the Apparent Compressive Strength of Concrete," *Bulletin No. 221*, American Society for Testing and Materials, Philadelphia, PA, April 1957, 68-73.

Nowak, A.S., and Szerszen, M.M., "Calibration of Design Code for Buildings (ACI 318): Part 1—Statistical Models for Resistance," *ACI Structural Journal*, V. 100, N. 3, May-June 2003, 377-382.

Olson, S.A., "Impact Damage and Repair of AASHTO Type III Girders", PhD Thesis, Department of Civil Engineering, University of Minnesota, Minneapolis, MN, April 1991.

PCA, *Notes on ACI 318-02 Building Code Requirements for Structural Concrete*, Portland Cement Association (PCA), Skokie, IL, 2002.

PCI, *Precast Prestressed Concrete Bridge Design Manual*, 2nd edition, Chicago, IL, 2003.

Pessiki, S., Kaczinski, M., and Wescott, H.H., "Evaluation of Effective Prestress Force in 28-Year-Old Prestressed Concrete Bridge Beams," *PCI Journal*, V. 41, N. 6, November-December 1996, 78-89.

Puckett, J.A., Huo, X.S., Patrick, M.D., Jablin, M.C., Mertz, D., and Peavy, M.D, "Simplified Equations for Live-Load Distribution in Highway Bridges," *Proceedings of the Transportation Research Board 6th International Bridge Engineering Conference: Reliability, Security, and Sustainability in Bridge Engineering*, Boston, MA, July 2005, 67-78.

Puckett, J.A., and Mertz, D., *NCHRP Report 592; Simplified Live Load Distribution Factor Equations,* Transportation Research Board, 2007.

Rabbat B.G., "25-Year-Old Prestressed Concrete Bridge Girders Tested," *PCI Journal*, V. 29, N. 1, January-February 1984, 177-179.

Rangan, B.V., "Web Crushing Strength of Reinforced and Prestressed Concrete Beams," *ACI Structural Journal*, V. 88, N. 1, January-February 1991, 12-16.

Riessauw, F.G., and Taerwe, L., "Tests on Two 30-Year-Old Prestressed Concrete Beams," *PCI Journal*, V. 25, N. 6, November-December 1980, 70-72.

Runzel, B., Shield, C.K., and French, C.W., *Mn/DOT Report 2007-47; Shear Capacity of Prestressed Concrete Beams,* Minnesota Departement of Transportation, 2007.

Saiidi, M., Labia Y, and Douglas, B., "Full Scale Testing and Analysis of 20-Year-Old Pretensioned Concrete Box Girders," *PCI Journal*, V.45, N. 2, March-April 2000, 96-105.

Sanders, W.W., Jr., and Elleby, H.A., *NCHRP Report No. 83 Distribution of Wheel Loads on Highway Bridges*, Transportation Research Board, Washington, D.C., 1970.

Scanlon, A., and Mikhailovsky, L., "Strength Evaluation of an Existing Concrete Bridge Based on Core and Nondestructive Test Data," *Canadian Journal of Civil Engineering*, V. 14, N.2, April 1987, 145-154.

Schlaich, J., Schafer, I., and Jennewin, M., "Towards a consistent design of structural concrete," *J. Prestressed Concrete Inst*., V. 32 No.3, 1987, 74-150.

Shahawy, M.A., and Batchelor, B., "Shear Behavior of Full Scale Prestressed Concrete Girders: Comparison Between AASHTO Specifications and LRFD Code," *PCI Journal*, V. 41, N. 3, May-June 1996, 48-53.

"Structures Manual." Florida State Department of Transportation. http://www.dot.state.fl.us/structures/StructuresManual/CurrentRelease/PrecastBeamExample.pdf. Date Accessed May 2008.

Teng, S., Kong, F.K., and Poh, S.P., "Shear Strength of Reinforced and Prestressed Concrete Deep Beams. Part I: Current Design Methods and a Proposed Equation," *Proceedings of the Internation of Civil Engineers: Structures and Buildings*, V. 128, N. 2, 1998, 112-123

Tennessee Structures Memorandum 043, "Lateral Distribution of Structural Loads," Tennessee Department of Transportation, Nashville, TN, 1996.

The Canadian Standards Association (CSA), *Canadian Highway Bridge Design Code, 2000 Edition,* Toronto, Ontario, Canada, 2000.

Tobias, D.H., Anderson, R.E., Khayyat, S.Y., Uzman, Z.B., and Riechers, K.L., "Simplified AASHTO Load and Resistance Factor Design Girder Live Load Distribution in Illinois," *Journal of Bridge Engineering*, V. 9, N 6, November/December, 2004, 606-613.

Wardhana, K., and Fabian, C.H., ‘‘Analysis of Recent Bridge Failures in the United States,’’ *J. Perform. Constr. Facil*, V. 17, N.3, 2003, 144-150.

Wood, S.L., “Evaluation of the Long-Term Properties of Concrete,” *Research and Development Bulletin RD102*, Portland Cement Association, Skokie, IL, 1991.

Wyoming Department of Transportation, *Bridge Rating and Analysis of Structural Systems (BRASS)*, Version 7, Cheyenne, WY, 2007.

Zokaie, T., Osterkamp, T.A., and Imbsen, R.A., *Distribution of Wheel Loads on Highway Bridges,* NCHRP 12-26 Project Final Report, Vol. 1, Transportation Research Board, National Research Council, Washington, D.C. 1991.

Zokaie, T., and Imbsen, R.A., *Distribution of Wheel Loads on Highway Bridges*, NCHRP 12-26 Project Report, Vol. 2, Transportation Research Board, National Research Council, Washington, D.C., 1993.

Zokaie, T., Mish, K.D., and Imbsen, R.A., *Distribution of Wheel Loads on Highway Bridges*” NCHRP 12-26 Project Final Report, Vol. 3, Transportation Research Board, National Research Council, Washington, D.C., 1993.

TABLES

Table 1.1 Load Factors for Rating Levels

	Inventory	**Operating**
γ_D	1.30	1.30
γ_L	2.17	1.30

Table 2.1 Comparison of AASHTO Shear Provisions

Parameters	Load Factors		Shear Strength Reduction Factors, ϕ	(LLDF)[1]		Effective Flange Width, b_{eff}	Prestress Losses for Pretensioned Members	Computation of Flexural Arm, j	Limits on Effective Depth, d
AASHTO SPEC.YEAR	Live Load	Dead Load		One Lane Loaded	Two or More Lanes Loaded				
1961	2.5	1.5	Not Specified, Assumed as 1.0	S / 7.0	S / 5.5	$\min\begin{pmatrix} \frac{L}{4} \\ s \\ 12*t_s+t_w \end{pmatrix}$	35,000 psi	$1-0.6\left(\frac{p\,f_{su}}{f'_c}\right)$ or 7/8 (0.875)	None
1965	″	″	″	″	″	″	″	$1-0.6\left(\frac{p\,f_{su}}{f'_c}\right)$	″
1969	″	″	″	″	″	″	″	″	″
1973	2.17	1.3	0.9	″	″	″	Detailed Method or 45,000 psi	″	″
1977 & 1979-Interim	″	″	″	″	″	″	Detailed or 45,000 psi (Not Needed[2])	″	″
1980-Interim, 1983 & 1989	″	″	″	″	″	″	″	Not Needed for Shear Design	0.8h
1992, 1996 & 2002	″	″	″	″	″	$\min\begin{pmatrix} \frac{L}{4} \\ s \\ 12*t_s+b_e \end{pmatrix}$	″	″	″

[1] LLDF: Live Load Distribution Factors

[2] Specified parameter is not used (or included) in the design and capacity calculations for shear in 1977 AASHTO Standard Specifications & 1979 Interim

Table 2.1 (Continued) Comparison of AASHTO Shear Provisions

Parameters / AASHTO SPEC.YEAR	Location of Critical Section	Shear Strength Provided by Concrete, V_c	Shear Strength Provided by Web Reinforcement, V_s	Limits for Web-Crushing Strength, $V_{s,max}$	Minimum Area of Web Reinforcement, $A_{v,min}$	Maximum Spacing of Web Reinforcement for Vertical Shear, $s_{max\_vertical}$	Calculation of V_p
1961	Quartpoint	$0.03 f'_c b_w jd \le 90b' jd$	$2A_v f_{sy} j \frac{d}{s}$	None	$0.0025 b_w s$	$0.75h$	Included in V_u [1]
1965	″	$0.06 f'_c b_w jd \le 180b' jd$	″	″	″	″	″
1969	″	″	″	″	″	″	″
1973	″	″	″	″	$\frac{100 b_w s}{f_{sy}}$	″	″
1977 & 1979-Interim	″	″	″	″	″	″	Not Included[1]
1980-Interim, 1983 & 1989	h/2 from face of support	V_{ci} or V_{cw}	$A_v f_{sy} \frac{d}{s}$	$8\sqrt{f'_c} b_w d$	$\frac{50 b_w s}{f_{sy}}$	$\min(0.75h, 24")$ if $V_s > 4\sqrt{f'_c} b_w d$ then $\min(0.375h, 12")$	Included in V_{cw}
1992, 1996 & 2002	″	″	″	″	″	″	″

[1]Inferred from the definition of V_u in the specification

Note: The definition of some of the notations shown in Tables 2.1 and 2.2 are as follows:
b_e: Effective web width of the beam (Per Article 9.8.3.1 of 2002 AASHTO Standard Specifications)
b', b_w : Web width of the beam
b_v: Width of cross section at the contact surface being investigated for horizontal shear
t_s: Slab thickness

Table 2.2 Comparison of AASHTO Horizontal Shear Provisions

Parameters	Horizontal Shear Demand, v_u	Horizontal Shear Capacity, v_{nh}		Maximum Horizontal Shear Spacing, $s_{vh,\max}$
AASHTO SPEC. YEAR		Condition	v_{nh} (psi)	
1961	$\frac{V_u Q_c}{I_c b_v}$	- Minimum (min.) reinforcement provided: - Min. reinforcement provided and contact surface is artificially roughened: - When provided reinforcement is in excess of min. reinforcement and contact surface is artificially roughened:	75 psi 150 psi 225 psi	$\min\begin{pmatrix} 4*t_{flange} \\ 24in. \\ A_v/(A_{v,\#3}/ft) \end{pmatrix}$
1965	″	″	″	″
1969	″	″	″	″
1973	″	- Min. reinforcement provided: - Min. reinforcement provided and contact surface is artificially roughened: - For each % of stirrup crossing the joint in excess of min. reinforcement:	75 psi 300 psi 150 psi	″
1977 & 1979-Interim	″	″	″	″
1980-Interim, 1983	$\frac{V_u}{b_v d_v}$	- Min. reinforcement provided and the contact surface is clean but not roughened: - Min. reinforcement provided and contact surface is artificially roughened: - For each % of stirrup crossing the joint in excess of min. reinforcement:	80 psi 350 psi $160f_y/40000$ psi	$\min\begin{pmatrix} 4*t_{least} \\ 24in. \\ A_v/(A_{v,\#3}/ft) \end{pmatrix}$
1989	″	″	″	$\min\begin{pmatrix} 4*t_{least} \\ 24in. \\ A_v f_y/(50b_v) \end{pmatrix}$
1992, 1996 & 2002	″	″	″	″

Table 2.3 Comparison of V_{test}/V_{pred} for AASHTO Shear Provisions

	2002 Standard $\frac{V_{test}}{V_{pred}}$	1979 Interim $\frac{V_{test}}{V_{pred}}$
Number of Beams	85	85
Mean	1.318	1.09
COV	0.156	0.383
Probability of $\frac{V_{test}}{V_{pred}} < 1$	6.2 %	41.3%

Table 3.1 Dead Load Effects

	DEAD LOAD SHEARS AND MOMENTS					
	Virtis-**BRASS LFD**		**HAND COMPUTATIONS**		**% Difference[3]**	
POI[1]	Moment (ft.kips)	Shear (kips)	Moment (ft.kips)	Shear (kips)	**Moment**	**Shear**
Critical Section[2]	450	114	450	114	0.0	0.0
0.1*L*	1310	97	1310	97	0.0	0.0
0.2*L*	2330	73	2330	73	0.0	0.0
0.25*L*	2730	61	2730	61	0.0	0.0
0.3*L*	3060	49	3060	49	0.0	0.0
0.4*L*	3500	24	3500	24	0.0	0.0
0.5*L*	3640	0	3640	0	0.0	0.0

[1]POI: Point of Interest

[2]Critical section: The critical section for shear is located at a distance *h*/2 from the face of the support, according to the 1996 Standard Specifications Article 9.20.1.4

[3]The percent differences are shown relative to the results obtained from *Virtis*-BRASS, i.e., *Diff* = (*VIRTIS-HAND CALC)**100/*VIRTIS*.

Table 3.2 Live Load Effects with WAD[1]=100 (default)

	LIVE LOAD SHEARS AND MOMENTS					
	Virtis-**BRASS LFD**		**HAND COMPUTATIONS**		**% Difference**	
POI	Moment (ft.k)	Shear (k)	Moment (ft.k)	Shear (k)	**Moment**	**Shear**
Critical Section	240.7	63.0	242.1	63.3	-0.6	-0.6
0.1*L*	702.5	59.1	699.9	58.8	0.4	0.4
0.2*L*	1234.9	52.4	1229.5	52.2	0.4	0.4
0.25*L*	1437.2	49.0	1430.5	48.8	0.5	0.5
0.3*L*	1597.0	45.7	1588.9	45.4	0.5	0.5
0.4*L*	1798.4	38.8	1800.1	38.6	-0.1	0.6
0.5*L*	1862.7	31.8	1852.1	31.6	0.6	0.8

[1]WAD=Wheel Advancement Denominator

Table 3.3 Comparison of Shear Capacities for PCI Bridge Example

	SHEAR CAPACITY COMPUTATIONS								
	Virtis-**BRASS LFD**			**HAND COMPUTATIONS**			**% Difference**		
POI	V_c (k)	V_s (k)	ϕV_n (k)	V_c (k)	V_s (k)	ϕV_n (k)	V_c	V_s	ϕV_n
Critical Section	225.7	128.0	318.3	224.3	128.0	317.1	0.6	0.0	0.4
0.1*L*	254.4	130.0	345.9	253.7	130.1	345.5	0.3	-0.1	0.1
0.2*L*	207.2	136.1	309.0	208.1	136.2	309.9	-0.4	-0.1	-0.3
0.25*L*	160.5	139.2	269.7	161.1	139.3	270.3	-0.4	-0.1	-0.2
0.3*L*	128.0	142.2	243.2	128.4	142.3	243.6	-0.3	0.0	-0.2
0.4*L*	78.6	148.4	204.3	78.7	148.4	204.3	-0.1	0.0	0.0
0.5*L*	61.0	148.4	188.4	61.0	148.4	188.4	0.0	0.0	0.0

Table 3.4 Comparison of Shear Rating Factors with WAD = 100

INVENTORY & OPERATING RATING FACTORS FOR SHEAR						
	Virtis-**BRASS LFD**		**HAND COMPUTATIONS**		**% Differences**	
POI	Inventory RF	Operating RF	Inventory RF	Operating RF	**Inv. RF % Diff.**	**Oper. RF % Diff.**
Critical Section	1.25	2.09	1.23	2.06	1.20	1.26
0.1*L*	1.71	2.48	1.72	2.49	-0.26	-0.50
0.2*L*	1.89	2.82	1.90	2.83	-0.88	-0.51
0.25*L*	1.79	2.95	1.81	2.98	-0.84	-0.78
0.3*L*	1.82	2.99	1.83	3.01	-0.78	-0.73
0.4*L*	2.05	3.38	2.06	3.40	-0.66	-0.64
0.5*L*	2.73	4.55	2.75	4.59	-0.82	-0.76

Table 3.5 Live Load Effects with WAD = 1000

	LIVE LOAD SHEARS AND MOMENTS					
	Virtis-**BRASS LFD**		**HAND COMPUTATIONS**		**% Difference**	
POI	Moment (ft.kips)	Shear (kips)	Moment (ft.kips)	Shear (kips)	**Moment**	**Shear**
Critical Section	240	63	242	63	0.0	0.0
0.1*L*	700	59	700	59	0.0	0.0
0.2*L*	1230	52	1230	52	0.0	0.0
0.25*L*	1430	49	1430	49	0.0	0.0
0.3*L*	1590	45	1590	45	0.0	0.0
0.4*L*	1800	39	1800	39	0.1	0.1
0.5*L*	1850	32	1850	32	0.0	0.1

Table 3.6 Discrepancies in Calculation of Strand Locations

	Virtis-**BRASS LFD**		**HAND COMPUTATIONS**		**% Differences**	
POI	y_{bs} (in)	$e_{non\text{-}composite}$ (in)	y_{bs} (in)	$e_{non\text{-}composite}$ (in)	**% Diff. y_{bs}**	**% Diff. e**
Critical Section	17.1	19.5	17.0	19.6	0.68	-0.58
0.1*L*	15.0	21.6	14.9	21.7	0.63	-0.42
0.2*L*	12.0	24.7	11.9	24.7	0.53	-0.24
0.25*L*	10.4	26.2	10.4	26.2	0.45	-0.17
0.3*L*	8.9	27.7	8.8	27.8	0.35	-0.10
0.4*L*	5.8	30.8	5.8	30.8	0.00	0.01
0.5*L*	5.8	30.8	5.8	30.8	0.00	0.01

Table 3.7 Girder Properties of the Bridge No. 27068_2 and Bridge No. 83022_1-3

Bridge No	Year Built	Year of Design Spec.	Girder Depth (in)	Web Width (in)	Span Length (ft)	Girder Spacing (ft)	Girder f'_c (psi)	e^1 at End of Girder (in)	*e* at End of Harping Distance (in)	# of Strands[2]	Type of Strand[3] (ksi)
27068_2	1981	1977	36	6	56.8	7.2	6000	6.8	12.0	16 (4)	270 (LR)
83022_1-3	1975	1973	45	7	56.8	10.8	5000	8.8	16.5	18 (6)	270 (SR)

[1] *e*: Eccentricity for the non-composite section
[2] Number in parenthesis is the number of draped strands
[3] LR: Low-relaxation, SR: Stress-relieved

Table 3.8 Comparison of Shear Capacities for Bridges 27068_2 and 83022_1-3

BRIDGE NO. 27068_2											
	Virtis-BRASS GIRDER LFD				HAND COMPUTATIONS				% Relative Difference[3]		
POI	V_c (k)	Controlling V_c (k)	V_s (k)	ϕV_n (k) [2]	V_c (k)	Controlling V_c (k)	V_s (k)	ϕV_n (k)	V_c (k)	V_s (k)	Capacity ϕV_n (k)
End of TL[1]	87.2	V_{cw}	89.8	159.3	105.8	V_{cw}	89.8	176.1	21.3	0.0	10.5
Critical Section	93.6	V_{cw}	43.0	123.0	110.5	V_{cw}	43.0	138.2	18.1	0.0	12.4
0.1L	110.1	V_{cw} *	43.8	138.5	122.6	V_{cw}	43.8	149.8	11.4	0.0	8.2
BRIDGE NO. 83022_1-3											
	Virtis-BRASS GIRDER LFD				HAND COMPUTATIONS				% Relative Difference		
POI	V_c (k)	Controlling V_c (k)	V_s (k)	ϕV_n (k) [2]	V_c (k)	Controlling V_c (k)	V_s (k)	ϕV_n (k)	V_c (k)	V_s (k)	Capacity ϕV_n (k)
End of TL	116.8	V_{cw}	61.9	160.8	143.0	V_{cw}	61.9	184.4	22.4	0.0	14.7
Critical Section	129	V_{cw}	62.5	172.3	151.1	V_{cw}	62.5	192.2	17.1	0.0	11.5
0.1L	148.2	V_{cw}	63.6	190.6	163.9	V_{cw}	63.6	204.8	10.6	0.0	7.4

[1] TL: Transfer Length

[2] ϕ: Shear strength reduction factor, $\phi = 0.9$

[3] The percent differences are shown relative to the results obtained from *Virtis*-BRASS, i.e., *Diff* = (*HAND CALC-VIRTIS)**100/*VIRTIS*.

* Values shown in the table are only for Inventory Rating for brevity. The controlling V_c value at 0.1L for Bridge No. 27068_2 is V_{ci} for the case of Operating Rating calculations. All other controlling V_c are V_{cw} for both Inventory and Operating Rating calculations for both bridges.

Table 3.9 Comparison of Shear Rating Factors for Bridges 27068_2 and 83022_1-3

BRIDGE NO. 27068_2						
	Virtis-**BRASS LFD**	**Hand Computations**	**% Relative Difference**	*Virtis*-**BRASS LFD**	**Hand Computations**	**% Relative Difference[1]**
POI	Inventory RF	Inventory RF	Inv. RF % Diff.	Operating RF	Operating RF	Oper. RF % Diff.
End of TL	1.03	1.20	**15.6**	1.73	1.99	15.5
Critical Section	0.73	0.87	**20.3**	1.21	1.46	20.3
0.1*L*	1.00	1.12	**12.3**	1.66	1.67	0.5*
BRIDGE NO. 83022_1-3						
	Virtis-**BRASS LFD**	**Hand Computations**	**% Relative Difference**	*Virtis*-**BRASS LFD**	**Hand Computations**	**% Relative Difference[1]**
POI	Inventory RF	Inventory RF	Inv. RF % Diff.	Operating RF	Operating RF	Oper. RF % Diff.
End of TL	0.47	0.62	**32.0**	0.78	1.09	30.9
Critical Section	0.61	0.74	**21.2**	1.02	1.24	20.1
0.1*L*	0.83	0.93	**11.8**	1.39	1.55	11.9

[1]The percent differences are shown relative to the results obtained from *Virtis*-BRASS, i.e., *Diff* = (*HAND CALC-VIRTIS)**100/*VIRTIS*.

*This small percentage is expected when compared to other large percentages because *Virtis*-BRASS calculates V_{ci} accurately opposed to V_{cw} calculations.

Table 4.1 Properties of the Girder Sections of Selected Mn/DOT Bridges

Girder Type	**Girder Depth (in)**	**Top Flange Width (in)**	**Bottom Flange Width (in)**	**Web Thickness (in)**	**Area of Precast Girder (in^2)**
MN 36 (AASHTO Type II)*	36	12	18	6	369
MN 40	40	12	22	6	485
MN 45 (AASHTO Type III)	45	16	22	7	560
45 M	45	30	26	7	624
MN 54 (AASHTO Type IV)	54	20	26	8	789
54 M	54	30	26	6	678
63 M	63	30	26	6	732
72 M	72	30	26	6	786

*MN 36, MN 45 and MN 54 correspond to AASHTO Girder Types II, III and IV, respectively.

Table 4.2 Girder Properties of the Selected Mn/DOT Bridges Grouped by Year

Bridge No	Year Built	Year of Design Spec.	Girder Depth (in)	Web Thickness (in)	Span Length (ft)	Girder Spacing (ft)	Girder f'_c (psi)	A_{ps} (in^2)	Type of Strand (ksi)
27978-1	1965	1961	40	6	43	11.2	5000	3.1	270 (LR)
27978-2	1965	1961	40	6	52	11.2	5000	4.3	270 (LR)
27978-3	1965	1961	40	6	37	11.2	5000	2.8	270 (LR)
9200	1963	1961	54	8	93	7.0	5000	7.3	250 (SR)
9603-1_3	1968	1965	40	6	35	13.6	5000	2.3	250 (SR)
9603-2	1968	1965	40	6	61	8.5	5000	3.7	250 (SR)
62825-1_3	1969	1965	45	7	38	12.5	5000	2.3	250 (SR)
62825-2	1969	1965	45	7	81	7.5	5410	5.2	250 (SR)
24825_1	1970	1965	45	7	51	11.8	5000	3.4	270 (SR)
24825_5	1970	1965	45	7	67	10.9	5810	4.9	270 (SR)
62860	1970	1965	60	8	101	10.8	6000	8.6	270 (SR)
24831-1	1970	1969	36	6	46	12.7	5816	3.7	270 (SR)
24831-2	1970	1969	36	6	55	9.5	5838	3.7	270 (SR)
24831-3	1970	1969	36	6	40	12.7	5000	2.8	270 (SR)
27942	1973	1969	54	8	97	7.5	5840	6.6	270 (SR)
19033	1978	1973	36	6	51	9.5	6000	3.4	270 (SR)
36006-1_3	1976	1973	36	6	42	11.0	5000	2.8	270 (LR)
83030	1975	1973	36	6	54	9.5	6000	4.0	270 (SR)
31019	1976	1973	45	7	59	13.1	6000	5.2	270 (SR)
49016_1-3	1974	1973	45	7	47	12.8	5000	3.4	270 (SR)
49016_2	1974	1973	45	7	76	7.7	5135	4.9	270 (SR)
83022_1-3	1975	1973	45	7	57	10.8	5000	3.7	270 (SR)
83022_2	1975	1973	45	7	64	10.8	5697	4.9	270 (SR)
73852-1_4	1976	1973	54	8	63	11.0	5000	4.0	270 (SR)
73852-2_3	1976	1973	54	8	88	8.3	5000	6.0	270 (SR)
73872_1-4	1976	1973	54	8	58	14.7	5000	4.4	270 (SR)
73872_2-3	1976	1973	54	8	79	11.0	5000	6.7	270 (SR)
73865	1976	1973	63	6	98	9.3	5600	6.4	270 (SR)
22805-1	1976	1973	63	6	83	11.1	5000	5.5	270 (SR)
22805-2_3	1976	1973	63	6	102	8.3	6000	6.7	270 (SR)
22805-4	1976	1973	63	6	96	8.3	5000	6.1	270 (SR)
36005	1976	1973	63	6	105	8.8	6000	7.3	270 (SR)
73860	1976	1973	63	6	105	8.8	5900	7.0	270 (SR)
27068-1_3	1981	1977	36	6	43	10.8	6000	2.8	270 (LR)
27068-2	1981	1977	36	6	57	7.2	6000	3.1	270 (LR)
55031	1985	1977	40	6	56	12.3	6000	4.9	270 (SR)
48010	1979	1977	45	7	43	12.5	5000	2.4	270 (SR)
46004	1981	1977	45	7	76	8.9	5986	5.8	270 (SR)
25013-1_3	1982	1977	54	8	77	13.5	5900	7.3	270 (SR)
61001	1981	1977	54	8	95	7.3	6000	6.6	270 (SR)
19813-1	1979	1977	72	6	54	13.0	6000	2.4	270 (LR)
19813-2	1979	1977	72	6	119	6.5	6000	6.4	270 (LR)
19813-3	1979	1977	72	6	72	13.0	6000	4.0	270 (LR)
14006-1_3	1988	1983	36	6	49	9.8	5027	2.8	270 (LR)
14006-2	1988	1983	36	6	64	7.9	7000	4.3	270 (LR)
8011	1988	1983	45	6	76	11.0	6500	6.1	270 (LR)
9011	1990	1983	45	6	78	9.8	6861	6.1	270 (LR)
17007	1987	1983	45	6	78	12.0	7000	7.0	270 (LR)
43011-1_2	1989	1983	45	6	82	8.0	6496	5.5	270 (LR)
33003	1989	1983	54	6	95	8.0	5900	6.1	270 (LR)
27749-2_3	1989	1983	63	6	104	7.3	4500	5.5	270 (LR)
27749-4_8	1989	1983	63	6	104	8.9	5500	6.7	270 (LR)
27749-10_15	1989	1983	63	6	104	9.5	5500	6.7	270 (LR)
2552	1989	1983	54	6	85	10.3	6000	6.3	270 (SR)

Table 4.3 Provided and Required Stirrup Spacings for Group 1 Bridges (Designed by 1961 AASHTO Standards)

Girder Properties			Provided and Required[1] Stirrup Spacings (in)						
Bridge No	**Design Spec.**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
27978-1	1961	40	7.0 (11.3 / **9.4**[2])	7.0 (11.3)[3]	7.0 (11.3 / **9.7**)	7.0 (11.3)	7.0 (13.7)	18.0 (22.9 / **21.8**)	18.0 (26.7 / **21.8**)
27978-2	1961	40	8.0 (9.3 / **6.9**)	8.0 (9.3 / **8.6**)	8.0 (9.3)	8.0 (9.3)	8.0 (11.2)	18.0 (18.3)	18.0 (25.4 / **21.8**)
27978-3	1961	40	5.0 (12.9)	5.0 (12.9)	6.0 (12.9)	6.0 (12.9)	6.0 (15.8)	18.0 (26.7 / **21.8**)	18.0 (26.7 / **21.8**)
9200	1961	54	12.0 (20.0)	12.0 (20.0)	12.0 (20.0)	12.0 (20.0)	18.0 (20.0)	18.0 (20.0)	18.0 (20.0)

[1]Numbers shown in parentheses are the required stirrup spacing (vertical shear / horizontal shear)
[2]Numbers in bold indicate that the stirrup spacing was governed by the horizontal shear reinforcement spacing limit.
[3]The required stirrup spacing for horizontal shear are not shown in the boxes if the controlling case is vertical shear, i.e., required stirrup spacing for vertical shear is less than that required for horizontal shear.
Note: Boxes shaded with color indicate that the provided stirrup spacings were larger than the required vertical shear stirrup spacings, but may or may not be greater than that required by horizontal shear. Boxes shaded with diagonal lines show that the provided stirrup spacings were greater than the required horizontal shear stirrup spacing but less than the required vertical shear stirrup spacing.

Table 4.4 Provided and Required Stirrup Spacings for Group 2 Bridges (Designed by 1965-1969 AASHTO Standards)

Girder Properties			Provided and Required Stirrup Spacings (in)						
Bridge No	**Design Spec.**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
9603-1_3	1965	40	9.0 (11.5 / **6.5**)	9.0 (11.5 / **7.2**)	12.0 (11.5)	12.0 (11.5 / **9.0**)	12.0 (14.7 / **13.0**)	18.0 (26.7 / **21.8**)	18.0 (26.7 / **21.8**)
9603-2	1965	40	12.0 (14.2 / **9.5**)	12.0 (14.2 / **13.0**)	12.0 (14.2 / **10.0**)	12.0 (14.2 / **13.6**)	12.0 (17.7)	18.0 (26.7 / **21.8**)	18.0 (26.7 / **21.8**)
62825-1_3	1965	45	9.0 (15.7 / **10.0**)	12.0 (15.7 / **12.2**)	12.0 (15.7 / **10.0**)	18.0 (15.7 / **14.1**)	18.0 (21.1)	18.0 (22.9 / **21.8**)	18.0 (22.9 / **21.8**)
62825-2	1965	45	9.0 (17.3 / **15.1**)	9.0 (17.3 / **9.0**)	12.0 (17.3 / **15.2**)	12.0 (17.3)	18.0 (22.9 / **21.8**)	18.0 (22.9 / **21.8**)	18.0 (22.9 / **21.8**)
24825_1	1965	45	14.0 (13.2 / **7.0**)	14.0 (13.2 / **8.5**)	14.0 (13.2)	20.7 (13.2 / **9.9**)	20.7 (16.9 / **14.2**)	20.7 (22.9 / **21.8**)	20.7 (22.9 / **21.8**)
24825_5	1965	45	13.0 (12.1 / **6.0**)	13.0 (12.1 / **7.6**)	13.0 (12.1)	17.0 (12.1 / **8.8**)	17.0 (15.1 / **12.1**)	21.0 (22.9 / **21.8**)	21.0 (22.9 / **21.8**)
62860	1965	60	18.0 (12.7 / **7.4**)	18.0 (12.7 / **10.5**)	18.0 (12.7)	20.0 (12.7)	20.0 (17.7)	20.0 (20.0)	20.0 (20.0)
24831-1	1969	36	9.0 (9.1 / **3.4**)	9.0 (9.1 / **3.9**)	9.0 (9.1 / **5.5**)	12.0 (9.1 / **6.9**)	12.0 (11.1 / **9.4**)	18.5 (18.9 / **11.5**)	18.5 (26.7 / **21.8**)
24831-2	1969	36	12.0 (11.6 / **4.8**)	12.0 (11.6 / **5.7**)	12.0 (11.6 / **8.5**)	18.0 (11.6 / **11.5**)	18.0 (14.4)	24.0 (26.7 / **21.8**)	24.0 (26.7 / **21.8**)
24831-3	1969	36	11.0 (10.2 / **4.2**)	11.0 (10.2 / **4.6**)	11.0 (10.2 / **6.9**)	18.5 (10.2 / **9.1**)	18.5 (12.8)	18.5 (21.8 / **15.9**)	18.5 (21.8 / **21.8**)
27942	1969	54	29.0 (20.0 / **11.1**)	29.0 (20.0 / **16.9**)	29.0 (20.0)	29.0 (20.0)	29.0 (20.0)	29.0 (20.0)	29.0 (20.0)

Table 4.5 Provided and Required Stirrup Spacings for Group 3 Bridges (Designed by 1973-1977 AASHTO Standards)

Girder Properties			**Provided and Required Stirrup Spacings (in)**						
Bridge No	**Design Spec.**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
19033	1973	36	20.0 (20.2 / **2.8**)	20.0 (20.2 / **3.6**)	20.0 (20.2 / **7.2**)	22.0 (20.2 / **14.3**)	22.0 (25.4 / **21.8**)	22.0 (27.0 / **21.8**)	22.0 (27.0 / **21.8**)
36006-1_3	1973	36	20.0 (19.4 / **2.7**)	20.0 (19.4 / **3.3**)	21.0 (19.4 / **6.3**)	21.0 (19.4 / **12.0**)	21.0 (24.7 / **21.8**)	21.0 (27.0 / **21.8**)	21.0 (27.0 / **21.8**)
83030	1973	36	12.0 (19.1 / **2.7**)	12.0 (19.1 / **3.3**)	12.0 (19.1 / **6.1**)	12.0 (19.1 / **10.8**)	18.0 (23.9 / **21.8**)	18.0 (27.0 / **21.8**)	18.0 (27.0 / **21.8**)
31019	1973	45	15.0 (13.5 / **2.8**)	15.0 (13.5 / **3.6**)	15.0 (13.5 / **8.7**)	15.0 (13.5)	21.0 (16.7)	33.0 (28.6 / **21.8**)	33.0 (28.6 / **21.8**)
49016_1-3	1973	45	18.0 (20.3 / **5.3**)	18.0 (20.3 / **7.5**)	18.0 (20.3)	21.0 (20.3)	21.0 (26.0 / **21.8**)	21.0 (33.8 / **21.8**)	21.0 (33.8 / **21.8**)
49016_2	1973	45	21.0 (28.5 / **21.8**)	21.0 (28.5 / **21.8**)	21.0 (28.5 / **21.8**)	21.0 (28.5 / **21.8**)	21.0 (33.8 / **21.8**)	21.0 (33.8 / **21.8**)	21.0 (33.8 / **21.8**)
83022_1-3	1973	45	18.0 (23.1 / **2.9**)	18.0 (23.1 / **3.7**)	18.0 (23.1 / **9.9**)	18.0 (23.1 / **21.8**)	18.0 (29.6 / **21.8**)	21.6 (33.8 / **21.8**)	21.6 (33.8 / **21.8**)
83022_2	1973	45	15.0 (21.1 / **2.5**)	15.0 (21.1 / **3.2**)	15.0 (21.1 / **8.0**)	15.0 (21.1)	15.0 (26.9 / **21.8**)	21.7 (33.8 / **21.8**)	21.7 (33.8 / **21.8**)
73852-1_4	1973	54	21.0 (28.4 / **21.8**)	21.0 (28.4 / **21.8**)	21.0 (28.4 / **21.8**)	21.0 (28.4 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)
73852-2_3	1973	54	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)
73872_1-4	1973	54	18.0 (19.0 / **17.1**)	18.0 (19.0)	18.0 (19.0)	18.0 (19.0)	18.0 (25.0 / **21.8**)	18.0 (30.0 / **21.8**)	18.0 (30.0 / **21.8**)
73872_2-3	1973	54	21.0 (23.4 / **21.8**)	21.0 (23.4 / **21.8**)	21.0 (23.4 / **21.8**)	21.0 (23.4 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)	21.0 (30.0 / **21.8**)
73865	1973	63	9.0 (26.1 / **21.8**)	22.0 (26.1 / **21.8**)	22.0 (26.1 / **21.8**)	22.0 (26.1 / **21.8**)	22.0 (34.1 / **21.8**)	22.0 (40.0 / **21.8**)	22.0 (40.0 / **21.8**)
22805-1	1973	63	9.0 (19.3)	22.0 (19.3)	22.0 (19.3)	22.0 (19.3)	22.0 (24.7 / **21.8**)	22.0 (33.3 / **21.8**)	22.0 (33.3 / **21.8**)
22805-2_3	1973	63	9.0 (23.6 / **21.8**)	22.0 (23.6 / **21.8**)	22.0 (23.6 / **21.8**)	22.0 (23.6 / **21.8**)	22.0 (31.3 / **21.8**)	22.0 (33.3 / **21.8**)	22.0 (33.3 / **21.8**)
22805-4	1973	63	9.0 (25.5 / **21.8**)	22.0 (25.5 / **21.8**)	22.0 (25.5 / **21.8**)	22.0 (25.5 / **21.8**)	22.0 (33.3 / **21.8**)	22.0 (33.3 / **21.8**)	22.0 (33.3 / **21.8**)
36005	1973	63	9.0 (26.0 / **21.8**)	21.0 (26.0 / **21.8**)	21.0 (26.0 / **21.8**)	21.0 (26.0 / **21.8**)	21.0 (34.8 / **21.8**)	21.0 (40.0 / **21.8**)	21.0 (40.0 / **21.8**)
73860	1973	63	9.0 (27.1 / **21.8**)	21.0 (27.1 / **21.8**)	21.0 (27.1 / **21.8**)	21.0 (27.1 / **21.8**)	21.0 (35.9 / **21.8**)	21.0 (40.0 / **21.8**)	21.0 (40.0 / **21.8**)
27068-1_3	1977	36	19.0 (16.8 / **2.3**)	19.0 (16.8 / **2.7**)	19.0 (16.8 / **4.5**)	21.0 (16.8 / **6.7**)	21.0 (20.6 / **13.0**)	21.0 (27.0 / **21.8**)	34.0 (27.0 / **21.8**)
27068-2	1977	36	21.0 (25.3 / **4.4**)	21.0 (25.3 / **6.0**)	21.0 (25.3 / **17.5**)	21.0 (25.3 / **21.8**)	21.0 (27.0 / **21.8**)	21.0 (27.0 / **21.8**)	31.0 (27.0 / **21.8**)
55031	1977	40	14.0 (13.5 / **2.5**)	14.0 (13.5 / **3.1**)	14.0 (13.5 / **6.2**)	20.0 (13.5 / **12.6**)	20.0 (16.1)	20.0 (25.5 / **21.8**)	20.0 (30.0 / **21.8**)
48010	1977	45	21.0 (20.1 / **4.9**)	21.0 (20.1 / **6.4**)	21.0 (20.1)	21.0 (20.1)	21.0 (25.2 / **21.8**)	21.0 (33.8 / **21.8**)	21.0 (33.8 / **21.8**)
46004	1977	45	15.0 (20.2 / **6.7**)	15.0 (20.2 / **13.7**)	15.0 (20.2)	15.0 (20.2)	15.0 (21.8 / **21.8**)	15.0 (33.8 / **21.8**)	15.0 (33.8 / **21.8**)
25013-1_3	1977	54	17.0 (14.3 / **5.2**)	17.0 (14.3 / **9.7**)	17.0 (14.3)	17.0 (14.3)	22.0 (17.6)	22.0 (33.8 / **21.8**)	22.0 (33.8 / **21.8**)
61001	1977	54	22.0 (29.2 / **21.8**)	22.0 (29.2 / **21.8**)	22.0 (29.2 / **21.8**)	22.0 (29.2 / **21.8**)	22.0 (30.0 / **21.8**)	22.0 (30.0 / **21.8**)	22.0 (30.0 / **21.8**)
19813-1	1977	72	9.0 (26.2 / **21.8**)	9.0 (26.2 / **21.8**)	15.0 (26.2 / **21.8**)	15.0 (26.2 / **21.8**)	15.0 (33.2 / **21.8**)	15.0 (40.0 / **21.8**)	15.0 (40.0 / **21.8**)
19813-2	1977	72	9.0 (36.7 / **21.8**)	15.0 (36.7 / **21.8**)	15.0 (36.7 / **21.8**)	15.0 (36.7 / **21.8**)	18.0 (40.0 / **21.8**)	18.0 (40.0 / **21.8**)	18.0 (40.0 / **21.8**)
19813-3	1977	72	9.0 (21.9 / **21.8**)	16.0 (21.9 / **21.8**)	16.0 (21.9 / **21.8**)	16.0 (21.9 / **21.8**)	16.0 (27.2 / **21.8**)	16.0 (40.0 / **21.8**)	16.0 (40.0 / **21.8**)

Table 4.6 Provided and Required Stirrup Spacings for Group 4 Bridges (Designed by 1983 AASHTO Standards)

Properties			**Provided and Required Stirrup Spacings (in)**						
Bridge No	**Design Spec.**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
14006-1_3	1983	36	15.0 (10.1 / **5.4**)	15.0 (14.5 / **7.7**)	15.0 (24.0 / **21.8**)	18.0 (22.0 / **21.8**)	18.0 (21.0)	18.0 (24.0 / **21.8**)	18.0 (24.0 / **21.8**)
14006-2	1983	36	15.0 (24.0 / **6.8**)	15.0 (24.0 / **10.5**)	15.0 (24.0 / **21.8**)	18.0 (23.8 / **21.8**)	18.0 (23.0 / **21.8**)	18.0 (24.0 / **21.8**)	18.0 (24.0 / **21.8**)
8011	1983	45	12.0 (8.4)	12.0 (12.9)	12.0 (24.0 / **21.8**)	16.0 (17.9)	16.0 (16.9)	24.0 (19.4)	24.0 (24.0 / **21.8**)
9011	1983	45	15.0 (7.0)	15.0 (10.4)	15.0 (15.1)	18.0 (13.2)	18.0 (13.2)	18.0 (15.2)	18.0 (22.5 / **21.8**)
17007	1983	45	6.0 (7.6)	12.0 (11.8)	12.0 (19.5)	12.0 (14.9)	12.0 (14.3)	18.0 (16.1)	18.0 (22.6 / **21.8**)
43011-1_2	1983	45	15.0 (8.8)	15.0 (14.7)	15.0 (20.0)	18.0 (16.9)	18.0 (16.6)	18.0 (19.0)	18.0 (24.0 / **21.8**)
33003	1983	54	21.0 (12.0)	21.0 (24.0 / **21.8**)	21.0 (19.1)	21.0 (17.1)	21.0 (17.4)	21.0 (20.8)	21.0 (24.0 / **21.8**)
27749-2_3	1983	63	9.0 (11.6)	20.9 (21.8)	20.9 (24.0 / **21.8**)	20.9 (24.0 / **21.8**)	20.9 (24.0 / **21.8**)	20.9 (24.0 / **21.8**)	20.9 (24.0 / **21.8**)
27749-4_8	1983	63	9.0 (9.8)	21.5 (17.9)	21.5 (24.0 / **21.8**)	21.5 (20.5)	21.5 (19.9)	21.5 (21.8 / **21.8**)	21.5 (24.0 / **21.8**)
27749-10_15	1983	63	9.0 (8.8)	21.5 (15.1)	21.5 (21.1)	21.5 (17.7)	21.5 (17.5)	21.5 (20.1)	21.5 (24.0 / **21.8**)
2552	1983	54	9.0 (10.4)	17.8 (16.5)	17.8 (24.0 / **21.8**)	17.8 (21.4)	17.8 (20.8)	17.8 (24.0 / **21.8**)	17.8 (24.0 / **21.8**)

Table 4.7 Provided and Required Stirrup Spacings for Group 4* Bridges (Designed by 1977-1979 Interim AASHTO Standards)

Properties			**Provided and Required Stirrup Spacings (in)**						
Bridge No	**Design Spec.**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
14006-1_3	1983-77	36	15.0 (19.0 / **2.5**)	15.0 (19.0 / **3.0**)	15.0 (19.0 / **5.1**)	18.0 (19.0 / **7.8**)	18.0 (27.0 / **17.1**)	18.0 (27.0 / **21.8**)	18.0 (27.0 / **21.8**)
14006-2	1983-77	36	15.0 (19.6 / **3.2**)	15.0 (19.6 / **4.2**)	15.0 (19.6 / **8.4**)	18.0 (19.6 / **17.1**)	18.0 (24.1 / **21.8**)	18.0 (27.0 / **21.8**)	18.0 (27.0 / **21.8**)
8011	1983-77	45	12.0 (13.6)	12.0 (13.6)	12.0 (13.6)	16.0 (13.6)	16.0 (16.3)	24.0 (27.8 / **21.8**)	24.0 (33.8 / **21.8**)
9011	1983-77	45	15.0 (13.1)	15.0 (13.1)	15.0 (13.1)	18.0 (13.1)	18.0 (15.6)	18.0 (24.9 / **21.8**)	18.0 (33.8 / **21.8**)
17007	1983-77	45	6.0 (12.0)	12.0 (12.0)	12.0 (12.0)	12.0 (12.0)	12.0 (14.3)	18.0 (23.3 / **21.8**)	18.0 (33.8 / **21.8**)
43011-1_2	1983-77	45	15.0 (16.4)	15.0 (16.4)	15.0 (16.4)	18.0 (16.4)	18.0 (19.8)	18.0 (32.7 / **21.8**)	18.0 (33.8 / **21.8**)
33003	1983-77	54	21.0 (17.5)	21.0 (17.5)	21.0 (17.5)	21.0 (17.5)	21.0 (21.6)	21.0 (38.6 / **21.8**)	21.0 (40.0 / **21.8**)
27749-2_3	1983-77	63	9.0 (24.8 / **21.8**)	20.9 (24.8 / **21.8**)	20.9 (24.8 / **21.8**)	20.9 (24.8 / **21.8**)	20.9 (31.2 / **21.8**)	20.9 (40.0 / **21.8**)	20.9 (40.0 / **21.8**)
27749-4_8	1983-77	63	9.0 (18.9)	21.5 (18.9)	21.5 (18.9)	21.5 (18.9)	21.5 (23.3 / **21.8**)	21.5 (40.0 / **21.8**)	21.5 (40.0 / **21.8**)
27749-10_15	1983-77	63	9.0 (17.6)	21.5 (17.6)	21.5 (17.6)	21.5 (17.6)	21.5 (21.5)	21.5 (37.5 / **21.8**)	21.5 (40.0 / **21.8**)
2552	1983-77	54	9.0 (17.0)	17.8 (17.0)	17.8 (17.0)	17.8 (17.0)	17.8 (21.1)	17.8 (38.2 / **21.8**)	17.8 (40.0 / **21.8**)

Table 4.8 Differences between Provided and Required Stirrup Spacings for Groups 1 & 2

Properties				**Difference (in) = Provided - Required**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
27978-1	1961	HS20	40	-4.3[1] (**-2.4**)[2]	-4.3[3]	-4.3 (**-2.7**)	-4.3	-6.7	-4.9 (**-3.8**)	-8.7 (**-3.8**)
27978-2	1961	HS20	40	-1.3 (**1.1**)	-1.3 (**-0.6**)	-1.3	-1.3	-3.2	-0.3	-7.4 (**-3.8**)
27978-3	1961	HS20	40	-7.9	-7.9	-6.9	-6.9	-9.8	-8.7 (**-3.8**)	-8.7 (**-3.8**)
9200	1961	HS20	54	-8.0	-8.0	-8.0	-8.0	-2.0	-2.0	-2.0
9603-1_3	1965	HS20	40	-2.5 (**2.5**)	-2.5 (**1.8**)	0.5	0.5 (**3.0**)	-2.7 (**1.0**)	-8.7 (**-3.8**)	-8.7 (**-3.8**)
9603-2	1965	HS20	40	-2.2 (**2.5**)	-2.2 (**-1.0**)	-2.2 (**2.0**)	-2.2 (**-1.6**)	-5.7	-8.7 (**-3.8**)	-8.7 (**-3.8**)
62825-1_3	1965	HS20	45	-6.7 (**-1.0**)	-3.7 (**-0.2**)	-3.7 (**2.0**)	2.3 (**3.9**)	-3.1	-4.9 (**-3.8**)	-4.9 (**-3.8**)
62825-2	1965	HS20	45	-8.3 (**-6.1**)	-8.3 (**0.0**)	-5.3 (**-3.2**)	-5.3	-4.9 (**-3.8**)	-4.9 (**-3.8**)	-4.9 (**-3.8**)
24825_1	1965	HS20	45	0.8 (**7.0**)	0.8 (**5.5**)	0.8	7.4 (**10.8**)	3.8 (**6.5**)	-2.2 (**-1.2**)	-2.2 (**-1.2**)
24825_5	1965	HS20	45	0.9 (**7.0**)	0.9 (**5.4**)	0.9	4.9 (**8.2**)	1.9 (**4.9**)	-1.9 (**-0.9**)	-1.9 (**-0.9**)
62860	1965	HS20	60	5.3 (**10.6**)	5.3 (**7.5**)	5.3	7.3	2.3	0.0	0.0
24831-1	1969	HS20	36	-0.1 (**5.6**)	-0.1 (**5.1**)	-0.1 (**3.5**)	2.9 (**5.1**)	0.9 (**2.6**)	0.4 (**7.0**)	-8.2 (**-3.3**)
24831-2	1969	HS20	36	0.4 (**7.2**)	0.4 (**6.3**)	0.4 (**3.5**)	6.4 (**6.5**)	3.6	-2.7 (**2.2**)	-2.7 (**2.2**)
24831-3	1969	HS20	36	0.8 (**6.8**)	0.8 (**6.4**)	0.8 (**4.1**)	8.3 (**9.4**)	5.7	-5.2 (**2.6**)	-8.2 (**-3.3**)
27942	1969	HS20	54	9.0 (**17.9**)	9.0 (**12.1**)	9.0	9.0	9.0	9.0	9.0

[1]Numbers shown out of parentheses are the differences between the provided and required stirrup spacings for vertical shear

[2]Numbers in parentheses (in bold) are the differences between the provided and required stirrup spacings for horizontal shear

[3]If the required stirrup spacing for vertical shear is less than that required for horizontal shear, the difference between the provided stirrup spacing and that required by horizontal shear is not shown in the boxes, i.e., no numbers within parentheses are shown.

Note: Boxes shaded with color indicate that the provided stirrup spacings were larger than the required vertical shear stirrup spacings, but may or may not be greater than that required by horizontal shear. Boxes shaded with diagonal lines show that the provided stirrup spacings were greater than the required horizontal shear stirrup spacing but less than the required vertical shear stirrup spacing.

Table 4.9 Differences between Provided and Required Stirrup Spacings for Group 3

Properties				**Difference (in) = Provided - Required**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
19033	1973	HS20	36	-0.2 (**17.2**)	-0.2 (**16.4**)	-0.2 (**12.8**)	1.8 (**7.7**)	-3.4 (**0.2**)	-5.0 (**0.2**)	-5.0 (**0.2**)
36006-1_3	1973	HS20	36	0.6 (**17.3**)	1.6 (**17.7**)	1.6 (**14.7**)	1.6 (**9.0**)	-3.7 (**-0.8**)	-6.0 (**-0.8**)	-6.0 (**-0.8**)
83030	1973	HS20	36	-7.1 (**9.3**)	-7.1 (**8.7**)	-7.1 (**5.9**)	-7.1 (**1.2**)	-5.9 (**-3.8**)	-9.0 (**-3.8**)	-9.0 (**-3.8**)
31019	1973	HS20	45	1.5 (**12.2**)	1.5 (**11.4**)	1.5 (**6.3**)	1.5	4.3	4.4 (**11.2**)	4.4 (**11.2**)
49016_1-3	1973	HS20	45	-2.3 (**12.7**)	-2.3 (**10.5**)	-2.3	0.7	-5.0 (**-0.8**)	-12.8 (**-0.8**)	-12.8 (**-0.8**)
49016_2	1973	HS20	45	-7.5 (**-0.8**)	-7.5 (**-0.8**)	-7.5 (**-0.8**)	-7.5 (**-0.8**)	-12.8 (**-0.8**)	-12.8 (**-0.8**)	-12.8 (**-0.8**)
83022_1-3	1973	HS20	45	-5.1 (**15.1**)	-5.1 (**14.3**)	-5.1 (**8.1**)	-5.1 (**-3.8**)	-11.6 (**-3.8**)	-12.2 (**-0.2**)	-12.2 (**-0.2**)
83022_2	1973	HS20	45	-6.1 (**12.5**)	-6.1 (**11.8**)	-6.1 (**7.0**)	-6.1	-11.9 (**-6.8**)	-12.1 (**-0.2**)	-12.1 (**-0.2**)
73852-1_4	1973	HS20	54	-7.4 (**-0.8**)	-7.4 (**-0.8**)	-7.4 (**-0.8**)	-7.4 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)
73852-2_3	1973	HS20	54	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)
73872_1-4	1973	HS20	54	-1.0 (**0.9**)	-1.0	-1.0	-1.0	-7.0 (**-3.8**)	-12.0 (**-3.8**)	-12.0 (**-3.8**)
73872_2-3	1973	HS20	54	-2.4 (**-0.8**)	-2.4 (**-0.8**)	-2.4 (**-0.8**)	-2.4 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)	-9.0 (**-0.8**)
73865	1973	HS20	63	-17.1 (**-12.8**)	-4.1 (**0.2**)	-4.1 (**0.2**)	-4.1 (**0.2**)	-12.1 (**0.2**)	-18.0 (**0.2**)	-18.0 (**0.2**)
22805-1	1973	HS20	63	-10.3	2.7	2.7	2.7	-2.7 (**0.2**)	-11.3 (**0.2**)	-11.3 (**0.2**)
22805-2_3	1973	HS20	63	-14.6 (**-12.8**)	-1.6 (**0.2**)	-1.6 (**0.2**)	-1.6 (**0.2**)	-9.3 (**0.2**)	-11.3 (**0.2**)	-11.3 (**0.2**)
22805-4	1973	HS20	63	-16.5 (**-12.8**)	-3.5 (**0.2**)	-3.5 (**0.2**)	-3.5 (**0.2**)	-11.3 (**0.2**)	-11.3 (**0.2**)	-11.3 (**0.2**)
36005	1973	HS20	63	-17.0 (**-12.8**)	-5.0 (**-0.8**)	-5.0 (**-0.8**)	-5.0 (**-0.8**)	-13.8 (**-0.8**)	-19.0 (**-0.8**)	-19.0 (**-0.8**)
73860	1973	HS20	63	-18.1 (**-12.8**)	-6.1 (**-0.8**)	-6.1 (**-0.8**)	-6.1 (**-0.8**)	-14.9 (**-0.8**)	-19.0 (**-0.8**)	-19.0 (**-0.8**)
27068-1_3	1977	HS20	36	2.2 (**16.7**)	2.2 (**16.3**)	2.2 (**14.5**)	4.2 (**14.3**)	0.4 (**8.0**)	-6.0 (**-0.8**)	-7.0 (**12.2**)
27068-2	1977	HS20	36	-4.3 (**16.6**)	-4.3 (**15.0**)	-4.3 (**3.5**)	-4.3 (**-0.8**)	-6.0 (**-0.8**)	-6.0 (**-0.8**)	4.0 (**9.2**)
55031	1977	HS20	40	0.5 (**11.5**)	0.5 (**10.9**)	0.5 (**7.8**)	6.5 (**7.4**)	3.9	-5.5 (**-1.8**)	-10.0 (**-1.8**)
48010	1977	HS20	45	0.9 (**16.1**)	0.9 (**14.6**)	0.9	0.9	-4.2 (**-0.8**)	-12.8 (**-0.8**)	-12.8 (**-0.8**)
46004	1977	HS20	45	-5.2 (**8.3**)	-5.2 (**1.3**)	-5.2	-5.2	-10.1 (**-6.8**)	-15.8 (**-3.8**)	-15.8 (**-3.8**)
25013-1_3	1977	HS20	54	2.7 (**11.8**)	2.7 (**7.3**)	2.7	2.7	4.4	-8.0 (**0.2**)	-8.0 (**0.2**)
61001	1977	HS20	54	-7.2 (**0.2**)	-7.2 (**0.2**)	-7.2 (**0.2**)	-7.2 (**0.2**)	-8.0 (**0.2**)	-8.0 (**0.2**)	-8.0 (**0.2**)
19813-1	1977	HS20	72	-17.2 (**-12.8**)	-17.2 (**-12.8**)	-11.2 (**-6.8**)	-11.2 (**-6.8**)	-18.2 (**-6.8**)	-25.0 (**-6.8**)	-25.0 (**-6.8**)
19813-2	1977	HS20	72	-27.7 (**-12.8**)	-21.7 (**-6.8**)	-21.7 (**-6.8**)	-21.7 (**-6.8**)	-22.0 (**-3.8**)	-22.0 (**-3.8**)	-22.0 (**-3.8**)
19813-3	1977	HS20	72	-12.9 (**-12.8**)	-5.9 (**-5.8**)	-5.9 (**-5.8**)	-5.9 (**-5.8**)	-11.2 (**-5.8**)	-24.0 (**-5.8**)	-24.0 (**-5.8**)

Table 4.10 Differences between Provided and Required Stirrup Spacings for Group 4

Properties				**Difference (in) = Provided - Required**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
14006-1_3	1983	HS20	36	4.9 (**9.6**)	0.5 (**7.3**)	-9.0 (**-6.8**)	-4.0 (**-3.8**)	-3.0	-6.0 (**-3.8**)	-6.0 (**-3.8**)
14006-2	1983	HS20	36	-9.0 (**8.2**)	-9.0 (**4.5**)	-9.0 (**-6.8**)	-5.8 (**-3.8**)	-5.0 (**-3.8**)	-6.0 (**-3.8**)	-6.0 (**-3.8**)
8011	1983	HS20	45	3.6	-0.9	-12.0 (**-9.8**)	-1.9	-0.9	4.6	0.0 (**2.2**)
9011	1983	HS25	45	8.0	4.6	-0.1	4.8	4.8	2.8	-3.8
17007	1983	HS20	45	-1.6	0.2	-7.5	-2.9	-2.3	1.9	-3.8
43011-1_2	1983	HS25	45	6.2	0.3	-5.0	1.1	1.4	-1.0	-3.8
33003	1983	HS25	54	9.0	-3.0 (**-0.8**)	1.9	3.9	3.6	0.2	-0.8
27749-2_3	1983	HS25	63	-2.6	-0.9	-3.1 (**-0.9**)	-3.1 (**-0.9**)	-3.1 (**-0.9**)	-3.1 (**-0.9**)	-3.1 (**-0.9**)
27749-4_8	1983	HS25	63	-0.8	3.6	-2.5 (**-0.3**)	1.0	1.6	-1.2 (**-0.3**)	-2.5 (**-0.3**)
27749-10_15	1983	HS25	63	0.2	6.4	0.4	3.8	4.0	1.4	-2.5 (**-0.3**)
2552	1983	HS20	54	-1.4	1.3	-6.2 (**-4.0**)	-3.6	-3.0	-6.2 (**-4.0**)	-6.2 (**-4.0**)

Table 4.11 Differences between Provided and Required Stirrup Spacings for Group 4*

Properties				**Difference (in) = Provided – Required**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
14006-1_3	1983-77	HS20	36	-4.0 (**12.5**)	-4.0 (**12.0**)	-4.0 (**9.9**)	-1.0 (**10.2**)	-5.1 (**0.9**)	-9.0 (**-3.8**)	-9.0 (**-3.8**)
14006-2	1983-77	HS20	36	-4.6 (**11.8**)	-4.6 (**10.8**)	-4.6 (**6.6**)	-1.6 (**0.9**)	-6.1 (**-3.8**)	-9.0 (**-3.8**)	-9.0 (**-3.8**)
8011	1983-77	HS20	45	-1.6	-1.6	-1.6	2.4	-0.3	-3.8 (**2.2**)	-9.8 (**2.2**)
9011	1983-77	HS25	45	1.9	1.9	1.9	4.9	2.4	-6.9 (**-3.8**)	-15.8 (**-3.8**)
17007	1983-77	HS20	45	-6.0	0.0	0.0	0.0	-2.3	-5.3 (**-3.8**)	-15.8 (**-3.8**)
43011-1_2	1983-77	HS25	45	-1.4	-1.4	-1.4	1.6	-1.8	-14.7 (**-3.8**)	-15.8 (**-3.8**)
33003	1983-77	HS25	54	3.5	3.5	3.5	3.5	-0.6	-17.6 (**-0.8**)	-19.0 (**-0.8**)
27749-2_3	1983-77	HS25	63	-15.8 (**-12.8**)	-3.9 (**-0.9**)	-3.9 (**-0.9**)	-3.9 (**-0.9**)	-10.3 (**-0.9**)	-19.1 (**-0.9**)	-19.1 (**-0.9**)
27749-4_8	1983-77	HS25	63	-9.9	2.6	2.6	2.6	-1.8 (**-0.3**)	-18.5 (**-0.3**)	-18.5 (**-0.3**)
27749-10_15	1983-77	HS25	63	-8.6	3.9	3.9	3.9	0.0	-16.0 (**-0.3**)	-18.5 (**-0.3**)
2552	1983-77	HS20	54	-8.0	0.8	0.8	0.8	-3.3	-20.4 (**-4.0**)	-22.2 (**-4.0**)

Table 4.12 Distribution of Differences between Provided and Required Stirrup Spacings for Vertical Shear

Difference (in) = Provided - Required for Vertical Shear at the Critical Sections							
Group #	**Total**	**Under-capacity**	**0 - 1† in**	**1 - 2 in**	**2 - 3 in**	**3 - 6 in**	**6 - 9 in**
2	11	9	1	0	2	1	5
3	28	9	2	3	2	1	1
4	11	6	1	0	0	2	3
4*	11	8	2	1	2	3	0
% of Undercapacity Girders for Vertical Shear at the Critical Sections							
Group #	**% of Total**	**% Under-capacity**	**0 - 1 in**	**1 - 2 in**	**2 - 3 in**	**3 - 6 in**	**6 - 9 in**
2	22	81.8	11.1	0.0	22.2	11.1	55.6
3	56	32.1	22.2	33.3	22.2	11.1	11.1
4	22	54.5	11.1	0.0	0.0	22.2	33.3
4*	22	72.7	22.2	11.1	22.2	33.3	0.0

† For the ranges shown in the table, the upper limit value is included and the lower limit value is excluded, i.e., for range 1 - 2 in, the values with 2 in are included, however the values with 1 in are excluded in the range. (This is true for all ranges except 0 – 1 in, where both limits are included in the range)

Table 4.13 Distribution of Differences between Provided and Required Stirrup Spacings for Vertical Shear as a Percentage of Beam Depth

Difference (in) = Provided - Required for Vertical Shear at the Critical Sections							
Group #	**Total**	**Under-capacity**	**0% - 2% of *d***	**2% - 5% of *d***	**5% - 10% of *d***	**10% - 15% of *d***	**> 15% of *d***
2	11	9	1	0	2	2	4
3	28	14	1	5	1	1	1
4	11	7	1	0	1	2	2
4*	11	10	2	2	3	1	0
% of Undercapacity Girders for Vertical Shear at the Critical Sections							
Group #	**% of Total**	**% Under-capacity**	**0% - 2% of *d***	**2% - 5% of *d***	**5% - 10% of *d***	**10% - 15% of *d***	**> 15% of *d***
2	22	81.8	11.1	0.0	22.2	22.2	44.4
3	56	50.0	11.1	55.6	11.1	11.1	11.1
4	22	63.6	16.7	0.0	16.7	33.3	33.3
4*	22	90.9	25.0	25.0	37.5	12.5	0.0

† For the ranges shown in the table, the upper limit value is included and the lower limit value is excluded, i.e., for range 2% - 5% of *d* the values with 5% of *d* are included, however the values with 2% *d* are excluded in the range. (This is true for all ranges except 0% – 2% of *d* where both limits are included in the range)

Table 4.14 Distribution of Differences between Provided and Required Stirrup Spacings, Horizontal Shear Requirements Included

Difference (in) = Provided - Required for Critical Sections, Horizontal Shear Limit Included								
Group #	**Total**	**Under-capacity**	**0 - 1† in**	**1 - 2 in**	**2 - 3 in**	**3 - 6 in**	**6 - 9 in**	**> 9 in**
2	11	9	0	0	1	2	4	2
3	28	14	6	2	2	0	2	2
4	11	7	1	0	0	1	4	1
4*	11	10	3	1	2	3	0	1
% of Undercapacity Girders for Critical Sections, Horizontal Shear Included								
Group #	**% of Total**	**% Under-capacity**	**0 - 1 in**	**1 - 2 in**	**2 - 3 in**	**3 - 6 in**	**6 - 9 in**	**> 9 in**
2	22	81.8	0.0	0.0	11.1	22.2	44.4	22.2
3	56	50.0	42.9	14.3	14.3	0.0	14.3	14.3
4	22	63.6	14.3	0.0	0.0	14.3	57.1	14.3
4*	22	90.9	30.0	10.0	20.0	30.0	0.0	10.0

† For the ranges shown in the table, the upper limit value is included and the lower limit value is excluded, i.e., for range 1 - 2 in, the values with 2 in are included, however the values with 1 in are excluded in the range. (This is true for all ranges except 0 – 1 in, where both limits are included in the range)

Table 5.1 Provided and Calculated Stirrup Spacings per 2002 Standard (1961-65-69)

Properties				**Provided and Required[1] Stirrup Spacings according to 2002 Standard (in)**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
27978-1	1961	HS20	40	7.0 (6.8[2])	7.0 (8.1)	7.0 (18.3)	7.0 (24.0/**20.0**)	7.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)
27978-2	1961	HS20	40	8.0 (6.8/**5.4**[3])	8.0 (8.9)	8.0 (23.2/**20.0**)	8.0 (20.6/**20.0**)	8.0 (18.2)	18.0 (20.5/**20.0**)	18.0 (24.0/**20.0**)
27978-3	1961	HS20	40	5.0 (8.9)	5.0 (10.0)	6.0 (23.8/**20.0**)	6.0 (24.0/**20.0**)	6.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)
9200	1961	HS20	54	12.0 (24.0/**16.0**)	12.0 (24.0/**16.0**)	12.0 (24.0/**16.0**)	12.0 (24.0/**16.0**)	18.0 (22.3/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)
9603-1_3	1965	HS20	40	9.0 (5.1)	9.0 (5.5)	12.0 (9.2)	12.0 (13.9)	12.0 (15.4)	18.0 (22.7/**20.0**)	18.0 (24.0/**20.0**)
9603-2	1965	HS20	40	12.0 (6.8)	12.0 (10.0)	12.0 (18.2)	12.0 (14.1)	12.0 (13.6)	18.0 (16.1)	18.0 (24.0/**20.0**)
62825-1_3	1965	HS20	45	9.0 (7.2)	12.0 (8.3)	12.0 (17.9)	18.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)	18.0 (24.0/**20.0**)
62825-2	1965	HS20	45	9.0 (11.7)	9.0 (24.0/**20.0**)	12.0 (21.2/**20.0**)	12.0 (15.8)	18.0 (15.3)	18.0 (17.5)	18.0 (24.0/**20.0**)
24825_1	1965	HS20	45	14.0 (6.5)	14.0 (8.3)	14.0 (24.0/**20.0**)	20.7 (16.6)	20.7 (15.6)	20.7 (18.4)	20.7 (24.0/**20.0**)
24825_5	1965	HS20	45	13.0 (6.8)	13.0 (11.6)	13.0 (17.1)	17.0 (12.6)	17.0 (11.9)	21.0 (14.0)	21.0 (24.0/**20.0**)
62860	1965	HS20	60	18.0 (10.9)	18.0 (24.0/**13.3**)	18.0 (13.8/**13.3**)	20.0 (11.2)	20.0 (11.2)	20.0 (13.8/**13.3**)	20.0 (24.0/**13.3**)
24831-1	1969	HS20	36	9.0 (5.3/**2.2**)	9.0 (6.7 / **2.6**)	9.0 (10.5/**4.9**)	12.0 (9.0)	12.0 (9.0)	18.5 (11.2)	18.5 (17.8)
24831-2	1969	HS20	36	12.0 (8.4/**3.5**)	12.0 (13.6/**4.6**)	12.0 (11.5)	18.0 (10.1)	18.0 (10.3)	24.0 (13.1)	24.0 (20.2)
24831-3	1969	HS20	36	11.0 (6.6/**2.7**)	11.0 (8.0/**3.2**)	11.0 (9.1/**6.4**)	18.5 (8.6)	18.5 (9.1)	18.5 (12.6)	18.5 (18.8)
27942	1969	HS20	54	29.0 (18.6/**16.0**)	29.0 (24.0/**16.0**)	29.0 (24.0/**16.0**)	29.0 (23.7/**16.0**)	29.0 (20.9/**16.0**)	29.0 (23.4/**16.0**)	29.0 (24.0/**16.0**)

[1] Numbers shown in parentheses are the required stirrup spacing (vertical shear/horizontal shear) per 2002 Standard.

[2] The required stirrup spacings for horizontal shear are not shown in the boxes if the controlling case is vertical shear, i.e., required stirrup spacing for vertical shear is less than that required for horizontal shear.

[3] Numbers in bold show that the stirrup spacing is governed by the horizontal shear reinforcement spacing limit.

Note: Boxes shaded with color indicate that the provided stirrup spacings were larger than the required vertical shear stirrup spacings, but may or may not be greater than that required by horizontal shear. Boxes shaded with diagonal lines show that the provided stirrup spacings were greater than the required horizontal shear stirrup spacing but less than the required vertical shear stirrup spacing.

Table 5.2 Provided and Calculated Stirrup Spacings per 2002 Standard (1973-77)

Properties				**Provided and Required Stirrup Spacings according to 2002 Standard (in)**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
19033	1973	HS20	36	20.0 (11.8/**5.8**)	20.0 (18.3/**7.9**)	20.0 (23.6)	22.0 (19.2)	22.0 (19.0)	22.0 (23.1)	22.0 (24.0)
36006-1_3	1973	HS20	36	20.0 (10.9/**5.2**)	21.0 (12.0/**6.5**)	21.0 (24.0/**19.7**)	21.0 (22.8)	21.0 (22.7)	21.0 (24.0)	21.0 (24.0)
83030	1973	HS20	36	12.0 (13.0/**5.5**)	12.0 (21.4/**7.4**)	12.0 (24.0)	12.0 (19.2)	18.0 (18.8)	18.0 (22.5)	18.0 (24.0)
31019	1973	HS20	45	15.0 (6.7/**5.0**)	15.0 (9.7/**8.1**)	15.0 (22.8)	15.0 (15.5)	21.0 (14.4)	33.0 (16.1)	33.0 (24.0)
49016_1-3	1973	HS20	45	18.0 (8.6)	18.0 (10.7)	18.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)
49016_2	1973	HS20	45	21.0 (18.6)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)
83022_1-3	1973	HS20	45	18.0 (10.6)	18.0 (15.3)	18.0 (24.0)	18.0 (24.0)	18.0 (24.0)	21.6 (24.0)	21.6 (24.0)
83022_2	1973	HS20	45	15.0 (11.1)	15.0 (18.9)	15.0 (24.0)	15.0 (24.0)	15.0 (22.2)	21.7 (24.0)	21.7 (24.0)
73852-1_4	1973	HS20	54	21.0 (12.5)	21.0 (19.5/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)
73852-2_3	1973	HS20	54	21.0 (22.9/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)
73872_1-4	1973	HS20	54	18.0 (8.7)	18.0 (11.1)	18.0 (24.0)	18.0 (21.6)	18.0 (20.7)	18.0 (24.0)	18.0 (24.0)
73872_2-3	1973	HS20	54	21.0 (14.4)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)
73865	1973	HS20	63	9.0 (11.8)	22.0 (21.9/**16.0**)	22.0 (24.0/**16.0**)	22.0 (24.0/**16.0**)	22.0 (24.0/**16.0**)	22.0 (24.0/**16.0**)	22.0 (24.0/**16.0**)
22805-1	1973	HS20	63	9.0 (7.4)	22.0 (10.9)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (22.4/**16.0**)	22.0 (23.6/**13.3**)	22.0 (24.0/**13.3**)
22805-2_3	1973	HS20	63	9.0 (11.3)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)
22805-4	1973	HS20	63	9.0 (12.5)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)	22.0 (24.0/**13.3**)
36005	1973	HS20	63	9.0 (14.6)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)
73860	1973	HS20	63	9.0 (15.4)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)
27068-1_3	1977	HS20	36	19.0 (10.6/**5.4**)	19.0 (12.0/**6.8**)	19.0 (24.0/**23.2**)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	34.0 (24.0)
27068-2	1977	HS20	36	21.0 (15.7)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	31.0 (24.0)
55031	1977	HS20	40	14.0 (7.2/**6.5**)	14.0 (9.8)	14.0 (24.0)	20.0 (24.0)	20.0 (16.1)	20.0 (17.9)	20.0 (24.0)
48010	1977	HS20	45	21.0 (9.4)	21.0 (11.3)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)	21.0 (24.0)
46004	1977	HS20	45	15.0 (15.5)	15.0 (24.0)	15.0 (24.0)	15.0 (24.0)	15.0 (24.0)	18.0 (24.0)	18.0 (24.0)
25013-1_3	1977	HS20	54	17.0 (9.8)	17.0 (16.5)	17.0 (24.0)	17.0 (19.3)	22.0 (17.8)	22.0 (19.7)	22.0 (24.0)
61001	1977	HS20	54	22.0 (24.0)	22.0 (24.0)	22.0 (24.0)	22.0 (24.0)	22.0 (24.0)	22.0 (24.0)	22.0 (24.0)
19813-1	1977	HS20	72	9.0 (13.0)	9.0 (15.0)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)
19813-2	1977	HS20	72	9.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)
19813-3	1977	HS20	72	9.0 (10.3)	16.0 (13.5)	16.0 (24.0/**16.0**)	16.0 (24.0/**16.0**)	16.0 (24.0/**16.0**)	16.0 (24.0/**16.0**)	16.0 (24.0/**16.0**)

Table 5.3 Provided and Calculated Stirrup Spacings per 2002 Standard (1983 or 1977)

Properties				**Provided and Required Stirrup Spacings according to 2002 Standard (in)**						
Bridge No	**Design Spec.**	**Design Load**	**Girder Depth (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
14006-1_3	1983	HS20	36	15.0 (10.1/**6.7**)	15.0 (14.5/**9.9**)	15.0 (24.0)	18.0 (22.0)	18.0 (21.0)	18.0 (24.0)	18.0 (24.0)
14006-2	1983	HS20	36	15.0 (24.0 / **8.6**)	15.0 (24.0/**14.3**)	15.0 (24.0)	18.0 (23.8)	18.0 (23.0)	18.0 (24.0)	18.0 (24.0)
8011	1983	HS20	45	12.0 (8.4)	12.0 (12.9)	12.0 (24.0/**16.0**)	16.0 (17.9/**16.0**)	16.0 (16.9/**16.0**)	24.0 (19.4/**16.0**)	24.0 (24.0/**16.0**)
9011	1983	HS25	45	15.0 (9.5)	15.0 (16.4/**16.0**)	15.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)
17007	1983	HS20	45	6.0 (7.6)	12.0 (11.8)	12.0 (19.5/**16.0**)	12.0 (14.9)	12.0 (14.3)	18.0 (16.1/**16.0**)	18.0 (22.6/**16.0**)
43011-1_2	1983	HS25	45	15.0 (12.0)	15.0 (24.0/**16.0**)	15.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)	18.0 (24.0/**16.0**)
33003	1983	HS25	54	21.0 (19.6/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)	21.0 (24.0/**16.0**)
27749-2_3	1983	HS25	63	9.0 (15.3)	20.9 (24.0/**16.0**)	20.9 (24.0/**16.0**)	20.9 (24.0/**16.0**)	20.9 (24.0/**16.0**)	20.9 (24.0/**16.0**)	20.9 (24.0/**16.0**)
27749-4_8	1983	HS25	63	9.0 (12.0)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)
27749-10_15	1983	HS25	63	9.0 (11.7)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)	21.5 (24.0/**16.0**)
2552	1983	HS20	54	9.0 (10.4)	17.8 (16.5/**16.0**)	17.8 (24.0/**16.0**)	17.8 (21.4/**16.0**)	17.8 (20.8/16.0)	17.8 (24.0/**16.0**)	17.8 (24.0/**16.0**)

Table 5.4 $\frac{\phi V_{n,\ STD\,2002}}{V_u}$ at Point of Interests (1961-1969)

Properties				$\phi V_{n,2002\text{-}STD} / V_u$						
Bridge No	**Year Built**	**Design Spec.**	**h (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
27978-1	1965	1961	40	0.99[1]	1.06	1.35	1.53	1.67	1.30	1.60
27978-2	1965	1961	40	0.94	1.03	1.28	1.29	1.31	1.04	1.23
27978-3	1965	1961	40	1.28	1.33	1.52	1.72	1.90	1.44	1.71
9200	1963	1961	54	1.25	1.30	1.28	1.22	1.06	1.18	1.77
9603-1_3	1968	1965	40	0.77	0.81	0.91	1.05	1.08	1.08	1.33
9603-2	1968	1965	40	0.80	0.94	1.12	1.06	1.05	0.95	1.21
62825-1_3	1969	1965	45	0.92	0.88	1.11	1.08	1.09	1.22	1.57
62825-2	1969	1965	45	1.08	1.33	1.17	1.10	0.94	0.99	1.50
24825_1	1970	1965	45	0.77	0.86	1.11	0.95	0.92	0.96	1.19
24825_5	1970	1965	45	0.82	0.97	1.06	0.92	0.88	0.84	1.06
62860	1970	1965	60	0.91	1.07	0.94	0.85	0.82	0.85	1.31
24831-1	1970	1969	36	0.82	0.91	1.05	0.90	0.89	0.81	0.98
24831-2	1970	1969	36	0.90	1.03	0.99	0.82	0.81	0.77	0.92
24831-3	1970	1969	36	0.85	0.91	0.94	0.77	0.77	0.85	1.01
27942	1973	1969	54	0.94	1.10	1.06	0.96	0.92	0.93	1.58

[1]The shaded boxes indicate that $\phi V_{n,2002\text{-}STD} / V_u$ is less than unity at the section.

Table 5.5 Inventory and Operating Ratings at Point of Interests (1961-1969)

Properties				Shear Rating Factors													
Bridge No	Year Built	Design Spec.	h (in)	At h_c / 2		***0.1L***		***0.2L***		***0.25L***		***0.3L***		***0.4L***		***0.5L***	
				Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op
27978-1	1965	1961	40	0.98[1]	1.42	1.08	1.52	1.44	1.81	1.67	1.94	1.82	2.12	1.34	1.68	1.60	2.26
27978-2	1965	1961	40	0.92	1.32	1.05	1.46	1.37	1.58	1.38	1.68	1.39	1.81	1.05	1.54	1.23	1.98
27978-3	1965	1961	40	1.37	1.86	1.43	1.94	1.66	2.03	1.89	2.20	2.10	2.40	1.51	1.74	1.72	2.28
9200	1963	1961	54	1.50	1.34	1.57	1.41	1.49	1.52	1.37	1.57	1.10	1.39	1.24	1.65	1.77	2.95
9603-1_3	1968	1965	40	0.71	1.12	0.75	1.17	0.89	1.34	1.06	1.46	1.10	1.57	1.09	1.63	1.33	2.14
9603-2	1968	1965	40	0.70	1.09	0.92	1.29	1.17	1.48	1.08	1.48	1.07	1.49	0.94	1.41	1.21	2.01
62825-1_3	1969	1965	45	0.89	1.32	0.85	1.27	1.14	1.47	1.10	1.38	1.11	1.47	1.26	1.75	1.55	2.51
62825-2	1969	1965	45	1.14	1.42	1.54	1.53	1.26	1.53	1.15	1.49	0.91	1.29	0.98	1.43	1.49	2.55
24825_1	1970	1965	45	0.68	1.08	0.80	1.20	1.15	1.37	0.93	1.30	0.90	1.33	0.95	1.42	1.18	2.02
24825_5	1970	1965	45	0.72	1.10	0.96	1.28	1.10	1.38	0.88	1.27	0.84	1.24	0.81	1.25	1.06	1.77
62860	1970	1965	60	0.82	1.14	1.13	1.17	0.90	1.20	0.74	1.09	0.71	1.08	0.79	1.18	1.31	2.18
24831-1	1970	1969	36	0.75	1.16	0.88	1.30	1.06	1.47	0.88	1.31	0.87	1.31	0.77	1.21	0.98	1.67
24831-2	1970	1969	36	0.86	1.26	1.04	1.32	0.98	1.39	0.77	1.17	0.75	1.16	0.73	1.16	0.92	1.53
24831-3	1970	1969	36	0.80	1.22	0.88	1.31	0.92	1.36	0.72	1.13	0.72	1.13	0.83	1.29	1.01	1.68
27942	1973	1969	54	0.88	1.12	1.21	1.16	1.11	1.23	0.93	1.23	0.87	1.20	0.90	1.28	1.55	2.71

[1]The shaded boxes indicate that the rating factor is less than unity at the section.

Table 5.6 $\phi V_{n,STD2002}/V_u$ at Point of Interests (1973-1977)

Properties				$\phi V_{n,2002\text{-}STD}$ / V_u						
Bridge No	**Year Built**	**Design Spec.**	**h (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
19033	1978	1973	36	0.85	0.98	1.04	0.96	0.95	1.02	1.28
36006-1_3	1976	1973	36	0.83	0.89	1.08	1.02	1.03	1.15	1.45
83030	1975	1973	36	1.03	1.17	1.24	1.20	1.02	1.11	1.43
31019	1976	1973	45	0.76	0.88	1.09	1.01	0.89	0.77	0.88
49016_1-3	1974	1973	45	0.76	0.84	1.09	1.09	1.08	1.16	1.51
49016_2	1974	1973	45	0.97	1.16	1.21	1.13	1.12	1.21	1.64
83022_1-3	1975	1973	45	0.83	0.95	1.21	1.13	1.12	1.10	1.40
83022_2	1975	1973	45	0.91	1.06	1.24	1.17	1.17	1.07	1.36
73852-1_4	1976	1973	54	0.85	0.98	1.25	1.17	1.16	1.24	1.61
73852-2_3	1976	1973	54	1.02	1.23	1.25	1.18	1.17	1.27	1.85
73872_1-4	1976	1973	54	0.79	0.87	1.11	1.05	1.05	1.13	1.44
73872_2-3	1976	1973	54	0.91	1.07	1.19	1.11	1.09	1.15	1.48
73865	1976	1973	63	1.12	1.00	1.19	1.11	1.10	1.18	1.65
22805-1	1976	1973	63	0.91	0.81	1.12	1.04	1.01	1.03	1.29
22805-2_3	1976	1973	63	1.09	1.05	1.19	1.11	1.08	1.14	1.71
22805-4	1976	1973	63	1.13	1.03	1.18	1.10	1.08	1.16	1.66
36005	1976	1973	63	1.20	1.10	1.18	1.11	1.10	1.21	1.81
73860	1976	1973	63	1.22	1.10	1.22	1.15	1.14	1.24	1.76
27068-1_3	1981	1977	36	0.83	0.90	1.13	1.05	1.05	1.17	1.19
27068-2	1981	1977	36	0.91	1.09	1.29	1.21	1.20	1.29	1.33
55031	1985	1977	40	0.76	0.88	1.15	1.19	0.93	0.95	1.17
48010	1979	1977	45	0.74	0.81	1.07	1.08	1.08	1.19	1.55
46004	1981	1977	45	1.01	1.24	1.29	1.22	1.21	1.19	1.62
25013-1_3	1982	1977	54	0.85	0.99	1.11	1.04	0.94	0.96	1.23
61001	1981	1977	54	1.10	1.24	1.28	1.21	1.19	1.29	2.14
19813-1	1979	1977	72	1.19	1.26	1.28	1.47	1.62	1.84	2.39
19813-2	1979	1977	72	1.52	1.47	1.62	1.68	1.58	1.83	2.99
19813-3	1979	1977	72	1.07	0.94	1.25	1.37	1.37	1.49	1.95

Note: Bridge numbers shown in red font were also investigated by Runzel et.al (2007) in Mn/DOT Report 2007-47

Table 5.7 Shear Inventory and Operating Ratings at Point of Interests (1973-1977)

Properties				Shear Rating Factors													
Bridge No	**Year Built**	**Design Spec.**	**h (in)**	**At h_c / 2**		***0.1L***		***0.2L***		***0.25L***		***0.3L***		***0.4L***		***0.5L***	
				Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op
19033	1978	1973	36	0.79	1.19	0.97	1.31	1.06	1.41	0.94	1.37	0.93	1.38	1.03	1.54	1.28	2.11
36006-1_3	1976	1973	36	0.77	1.19	0.86	1.27	1.10	1.36	1.03	1.43	1.03	1.51	1.17	1.75	1.45	2.36
83030	1975	1973	36	1.04	1.42	1.24	1.50	1.32	1.65	1.26	1.70	1.02	1.46	1.13	1.66	1.42	2.40
31019	1976	1973	45	0.64	1.04	0.89	1.21	1.13	1.36	1.01	1.41	0.85	1.26	0.72	1.13	0.88	1.43
49016_1-3	1974	1973	45	0.67	1.07	0.78	1.18	1.12	1.42	1.12	1.43	1.10	1.52	1.19	1.73	1.49	2.58
49016_2	1974	1973	45	0.96	1.29	1.25	1.34	1.31	1.46	1.20	1.54	1.17	1.55	1.26	1.78	1.64	2.74
83022_1-3	1975	1973	45	0.74	1.13	0.93	1.30	1.29	1.47	1.18	1.55	1.16	1.56	1.13	1.60	1.41	2.32
83022_2	1975	1973	45	0.85	1.21	1.10	1.38	1.35	1.52	1.25	1.60	1.23	1.62	1.08	1.54	1.36	2.26
73852-1_4	1976	1973	54	0.78	1.16	0.97	1.32	1.35	1.43	1.24	1.51	1.22	1.61	1.29	1.82	1.59	2.72
73852-2_3	1976	1973	54	1.03	1.28	1.41	1.33	1.42	1.45	1.29	1.53	1.25	1.56	1.35	1.81	1.85	3.08
73872_1-4	1976	1973	54	0.69	1.08	0.80	1.19	1.15	1.36	1.07	1.43	1.06	1.48	1.16	1.67	1.42	2.38
73872_2-3	1976	1973	54	0.85	1.19	1.12	1.26	1.29	1.36	1.16	1.43	1.13	1.50	1.18	1.65	1.46	2.50
73865	1976	1973	63	1.23	1.42	1.00	1.28	1.33	1.39	1.19	1.46	1.15	1.45	1.24	1.67	1.65	2.75
22805-1	1976	1973	63	0.85	1.18	0.67	1.05	1.19	1.30	1.06	1.36	1.01	1.35	1.04	1.46	1.29	2.14
22805-2_3	1976	1973	63	1.19	1.35	1.09	1.25	1.36	1.35	1.19	1.41	1.13	1.39	1.19	1.57	1.71	2.84
22805-4	1976	1973	63	1.27	1.41	1.07	1.25	1.32	1.35	1.17	1.41	1.13	1.41	1.22	1.61	1.66	2.77
36005	1976	1973	63	1.41	1.49	1.20	1.29	1.33	1.40	1.19	1.42	1.16	1.43	1.28	1.69	1.81	3.01
73860	1976	1973	63	1.43	1.54	1.19	1.31	1.39	1.42	1.25	1.50	1.22	1.51	1.32	1.76	1.76	2.93
27068-1_3	1981	1977	36	0.77	1.18	0.87	1.29	1.17	1.41	1.07	1.44	1.07	1.53	1.20	1.78	1.19	1.93
27068-2	1981	1977	36	0.87	1.26	1.12	1.39	1.40	1.51	1.29	1.60	1.26	1.70	1.34	1.91	1.33	2.16
55031	1985	1977	40	0.65	1.05	0.83	1.23	1.20	1.46	1.25	1.67	0.91	1.33	0.94	1.42	1.17	1.95
48010	1979	1977	45	0.65	1.05	0.74	1.15	1.10	1.40	1.11	1.48	1.10	1.56	1.23	1.79	1.54	2.59
46004	1981	1977	45	1.02	1.34	1.38	1.43	1.44	1.57	1.32	1.67	1.30	1.69	1.24	1.74	1.62	2.70
25013-1_3	1982	1977	54	0.76	1.13	0.99	1.27	1.17	1.36	1.05	1.41	0.91	1.29	0.94	1.38	1.22	2.08
61001	1981	1977	54	1.21	1.28	1.47	1.33	1.51	1.46	1.35	1.56	1.31	1.57	1.40	1.81	2.11	3.62
19813-1	1979	1977	72	1.28	1.64	1.38	1.74	1.40	1.70	1.64	1.82	1.82	1.98	2.00	2.50	2.41	3.61
19813-2	1979	1977	72	2.23	1.82	2.04	1.59	2.27	1.79	2.32	1.94	2.05	1.96	2.20	2.48	2.99	4.98
19813-3	1979	1977	72	1.11	1.42	0.90	1.26	1.37	1.57	1.53	1.67	1.51	1.79	1.60	2.15	1.94	3.20

Table 5.8 $\dfrac{\phi V_{n,\ STD\,2002}}{V_u}$ at Point of Interests (1983)

Properties				$\phi V_{n,2002\text{-}STD} / V_u$						
Bridge No	**Year Built**	**Design Spec.**	**h (in)**	**At h_c / 2**	***0.1L***	***0.2L***	***0.25L***	***0.3L***	***0.4L***	***0.5L***
14006-1_3	1988	1983	36	0.86	0.99	1.20	1.07	1.06	1.16	1.49
14006-2	1988	1983	36	1.21	1.31	1.22	1.10	1.10	1.21	1.64
8011	1988	1983	45	0.87	1.02	1.20	1.03	1.02	0.91	1.08
9011	1990	1983	45	0.85	1.03	1.27	1.13	1.11	1.16	1.44
17007	1987	1983	45	1.12	1.00	1.14	1.08	1.08	0.95	1.14
43011-1_2	1989	1983	45	0.93	1.14	1.37	1.22	1.21	1.29	1.71
33003	1989	1983	54	0.98	1.19	1.21	1.14	1.14	1.24	1.72
27749-2_3	1989	1983	63	1.26	1.14	1.38	1.32	1.31	1.43	2.10
27749-4_8	1989	1983	63	1.17	1.08	1.30	1.21	1.19	1.28	1.73
27749-10_15	1989	1983	63	1.12	1.03	1.24	1.16	1.13	1.20	1.61
2552	1989	1983	54	1.06	0.98	1.12	1.06	1.06	1.16	1.53

Table 5.9 Shear Inventory and Operating Ratings at Point of Interests (1983)

Properties				**Shear Rating Factors**													
Bridge No	**Year Built**	**Design Spec.**	**h (in)**	**At h_c / 2**		***0.1L***		***0.2L***		***0.25L***		***0.3L***		***0.4L***		***0.5L***	
				Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op	Inv	Op
14006-1_3	1988	1983	36	0.80	1.21	0.99	1.40	1.26	1.57	1.09	1.54	1.08	1.55	1.19	1.76	1.48	2.47
14006-2	1988	1983	36	1.32	1.41	1.45	1.47	1.31	1.60	1.13	1.54	1.13	1.56	1.26	1.80	1.62	2.76
8011	1988	1983	45	0.79	1.16	1.04	1.37	1.31	1.53	1.05	1.42	1.03	1.42	0.89	1.33	1.08	1.76
9011	1990	1983	45	0.75	1.12	1.04	1.36	1.42	1.49	1.19	1.48	1.15	1.53	1.20	1.69	1.45	2.37
17007	1987	1983	45	1.19	1.48	0.99	1.33	1.21	1.48	1.12	1.48	1.10	1.49	0.94	1.39	1.14	1.89
43011-1_2	1989	1983	45	0.88	1.21	1.24	1.43	1.59	1.58	1.34	1.57	1.30	1.64	1.37	1.84	1.71	2.84
33003	1989	1983	54	0.97	1.25	1.34	1.31	1.35	1.41	1.23	1.50	1.21	1.52	1.31	1.75	1.71	2.87
27749-2_3	1989	1983	63	1.55	1.56	1.28	1.37	1.70	1.51	1.57	1.61	1.51	1.70	1.60	1.98	2.10	3.50
27749-4_8	1989	1983	63	1.34	1.48	1.15	1.30	1.51	1.42	1.35	1.50	1.31	1.58	1.37	1.79	1.73	2.88
27749-10_15	1989	1983	63	1.22	1.42	1.06	1.29	1.40	1.40	1.25	1.48	1.21	1.51	1.27	1.70	1.61	2.69
2552	1989	1983	54	1.11	1.37	0.97	1.28	1.19	1.40	1.09	1.42	1.09	1.44	1.20	1.67	1.53	2.56

Table 5.10 Controlling V_c for 2002 Standard Equation at Point of Interests

Bridge No	*h* (in)	Design Spec.	At $h_c / 2$	*0.1L*	*0.2L*	*0.25L*	*0.3L*	*0.4L*	*0.5L*
24831-1	36	1969	V_{cw}*	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
24831-2	36	1969	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
24831-3	36	1969	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
19033	36	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
36006-1_3	36	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
83030	36	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27068-1_3	36	1977	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27068-2	36	1977	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
14006-1_3	36	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
14006-2	36	1983	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27978-1	40	1961	V_{cw}	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}
27978-2	40	1961	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27978-3	40	1961	V_{cw}	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}
9603-1_3	40	1965	V_{cw}	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}
9603-2	40	1965	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
55031	40	1977	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
62825-1_3	45	1965	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
62825-2	45	1965	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
24825_1	45	1965	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
24825_5	45	1965	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
31019	45	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
49016_1-3	45	1973	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
49016_2	45	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
83022_1-3	45	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
83022_2	45	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
48010	45	1977	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
46004	45	1977	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
8011	45	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
9011	45	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
17007	45	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
43011-1_2	45	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}

*The shaded cells indicate that web-shear cracking load governs over flexure-shear cracking load.

Table 5.10 (Continued) Controlling V_c for 2002 Standard Equation at Point of Interests

Bridge No	*h* (in)	Design Spec.	At $h_c / 2$	*0.1L*	*0.2L*	*0.25L*	*0.3L*	*0.4L*	*0.5L*
9200	54	1961	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27942	54	1969	V_{cw}*	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
73852-1_4	54	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
73852-2_3	54	1973	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
73872_1-4	54	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
73872_2-3	54	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
25013-1_3	54	1977	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
61001	54	1977	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
33003	54	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
2552	54	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
62860	60	1965	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
73865	63	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
22805-1	63	1973	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
22805-2_3	63	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
22805-4	63	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
36005	63	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
73860	63	1973	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27749-2_3	63	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27749-4_8	63	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
27749-10_15	63	1983	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
19813-1	72	1977	V_{cw}	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}
19813-2	72	1977	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}	V_{ci}
19813-3	72	1977	V_{cw}	V_{cw}	V_{cw}	V_{ci}	V_{ci}	V_{ci}	V_{ci}

*The shaded cells indicate that web-shear cracking load governs over flexure-shear cracking load.

Table 5.11 Parameters for All Girders at Critical Section

Bridge No	h (in)	$f_c^{'}$ (psi)	L (ft)	S_g (ft)	$\frac{L}{S_g}$	$\frac{V_s}{V_n}$%	Radius of Gyration, r (in)	$\frac{(y_{bc}-y_b)}{r}$ *	$\frac{e}{r}$	$\frac{e(y_{bc}-y_b)}{r^2}$	$\frac{\phi V_{n,\ STD\ 2002}}{V_u}$	Shear RF	
												Inv	Op
14006-1_3	36	5027	49.3	9.8	5.01	35	11.75	0.95	0.63	0.60	0.86	0.80	1.21
14006-2	36	7000	63.6	7.9	8.08	26	11.75	0.95	0.37	0.35	1.21	1.32	1.41
19033	36	6000	50.6	9.5	5.32	25	11.75	0.95	0.48	0.45	0.85	0.79	1.19
27068-1_3	36	6000	42.8	10.8	3.98	27	11.75	0.95	0.41	0.39	0.83	0.77	1.18
27068-2	36	6000	56.8	7.2	7.92	28	11.75	0.95	0.64	0.61	0.91	0.87	1.26
36006-1_3	36	5000	41.8	11.0	3.80	25	11.75	0.95	0.40	0.38	0.83	0.77	1.19
24831-1	36	5816	45.6	12.7	3.60	32	11.75	0.95	0.42	0.40	0.82	0.75	1.16
24831-2	36	5838	54.6	9.5	5.75	25	11.75	0.95	0.38	0.36	0.90	0.86	1.26
24831-3	36	5000	39.5	12.7	3.12	27	11.75	0.95	0.20	0.19	0.85	0.80	1.22
83030	36	6000	54.3	9.5	5.71	34	11.75	0.95	0.44	0.41	1.03	1.04	1.42
9603-1_3	40	5000	35.2	13.6	2.59	38	13.44	0.83	0.43	0.36	0.77	0.71	1.12
9603-2	40	5000	61.1	8.5	7.19	32	13.44	0.83	0.62	0.51	0.80	0.70	1.09
55031	40	6000	56.3	12.3	4.56	33	13.44	0.83	0.65	0.54	0.76	0.65	1.05
27978-1	40	5000	42.8	11.2	3.83	42	13.44	0.83	0.54	0.45	0.99	0.98	1.42
27978-2	40	5000	52.3	11.2	4.69	35	13.44	0.83	0.43	0.35	0.94	0.92	1.32
27978-3	40	5000	36.5	11.2	3.27	50	13.44	0.83	0.51	0.42	1.28	1.37	1.86
8011	45	6500	76.0	11.0	6.91	34	16.36	0.80	0.58	0.46	0.87	0.79	1.16
9011	45	6861	77.6	9.8	7.89	31	16.36	0.80	0.68	0.55	0.85	0.75	1.12
17007	45	7000	77.6	12.0	6.47	49	16.36	0.80	0.56	0.45	1.12	1.19	1.48
62825-1_3	45	5000	38.0	12.5	3.04	36	14.96	0.88	0.45	0.40	0.92	0.89	1.32
62825-2	45	5410	81.2	7.5	10.82	33	14.96	0.88	0.61	0.54	1.08	1.14	1.42
43011-1_2	45	6496	82.1	8.0	10.27	33	16.36	0.80	0.72	0.58	0.93	0.88	1.21
24825_1	45	5000	51.5	11.8	4.35	25	14.96	0.88	0.56	0.50	0.77	0.68	1.08
24825_5	45	5810	66.7	10.9	6.11	24	14.96	0.88	0.60	0.53	0.82	0.72	1.10
48010	45	5000	43.2	12.5	3.45	28	14.96	0.88	0.66	0.59	0.74	0.65	1.05
31019	45	6000	59.4	13.1	4.52	25	14.96	0.88	0.62	0.55	0.76	0.64	1.04
46004	45	5986	75.5	8.9	8.47	29	14.96	0.88	0.63	0.55	1.01	1.02	1.34
49016_1-3	45	5000	47.3	12.8	3.70	29	14.96	0.88	0.61	0.54	0.76	0.67	1.07
49016_2	45	5135	75.7	7.7	9.87	24	14.96	0.88	0.59	0.52	0.97	0.96	1.29
83022_1-3	45	5000	56.8	10.8	5.28	29	14.96	0.88	0.66	0.58	0.83	0.74	1.13
83022_2	45	5697	63.6	10.8	5.91	30	14.96	0.88	0.60	0.53	0.91	0.85	1.21

Table 5.11(Continued) Parameters for All Girders at Critical Section

Bridge No	h (in)	$f_c^{'}$ (psi)	L (ft)	S_g (ft)	$\frac{L}{S_g}$	$\frac{V_s}{V_n}$%	Radius of Gyration, r (in)	$\frac{(y_{bc}-y_b)}{r}$*	$\frac{e}{r}$	$\frac{e(y_{bc}-y_b)}{r^2}$	$\frac{\phi V_{n,\ STD\,2002}}{V_u}$	Shear RF	
												Inv	Op
27942	54	5840	97.3	7.5	12.98	12	18.18	0.84	0.63	0.53	0.94	0.88	1.12
33003	54	5900	95.5	8.0	11.93	23	19.80	0.83	0.53	0.43	0.98	0.97	1.25
2552	54	6000	85.3	10.3	8.32	43	19.80	0.90	0.53	0.48	1.06	1.11	1.37
9200	54	5000	92.5	7.0	13.21	23	V_{ci} controls				1.25	1.50	1.34
25013-1_3	54	5900	77.3	13.5	5.71	23	18.18	0.84	0.56	0.47	0.85	0.76	1.13
73852-1_4	54	5000	63.3	11.0	5.75	25	18.18	0.84	0.68	0.57	0.85	0.78	1.16
73852-2_3	54	5000	87.6	8.3	10.62	22	18.18	0.84	0.61	0.52	1.02	1.03	1.28
61001	54	6000	95.2	7.3	12.98	20	18.18	0.84	0.63	0.53	1.10	1.21	1.28
73872_1-4	54	5000	57.9	14.7	3.95	25	18.18	0.84	0.53	0.45	0.79	0.69	1.08
73872_2-3	54	5000	78.8	11.0	7.16	21	18.18	0.84	0.54	0.45	0.91	0.85	1.19
62860	60	6000	100.7	10.8	9.30	15	20.31	0.47	0.57	0.26	0.91	0.82	1.14
27749-2_3	63	4500	104.1	7.3	14.36	51	23.14	0.81	0.97	0.78	1.26	1.55	1.56
27749-4_8	63	5500	104.1	8.9	11.71	45	23.14	0.85	0.88	0.75	1.17	1.34	1.48
27749-10_15	63	5500	104.1	9.5	10.93	46	23.14	0.88	0.88	0.77	1.12	1.22	1.42
73865	63	5600	98.0	9.3	10.50	47	23.14	0.86	0.88	0.76	1.12	1.23	1.42
22805-1	63	5000	82.8	11.1	7.47	45	23.14	0.96	0.91	0.88	0.91	0.85	1.18
22805-2_3	63	6000	102.3	8.3	12.31	41	23.14	0.87	0.88	0.76	1.09	1.19	1.35
22805-4	63	5000	95.8	8.3	11.52	41	23.14	0.91	0.81	0.74	1.13	1.27	1.41
36005	63	6000	105.2	8.8	12.02	43	23.14	0.88	0.80	0.70	1.20	1.41	1.49
73860	63	5900	105.0	8.8	12.00	44	23.14	0.77	0.84	0.65	1.22	1.43	1.54
19813-1	72	6000	53.5	13.0	4.12	52	26.40	0.89	0.92	0.82	1.19	1.28	1.64
19813-2	72	6000	119.3	6.5	18.36	46	26.40	0.67	0.94	0.63	1.52	2.23	1.82
19813-3	72	6000	71.9	13.0	5.53	50	26.40	0.89	0.93	0.83	1.07	1.11	1.42

The shaded cells indicate the shear capacity-to-demand ratios and rating factors which are less than unity.

Table 6.1 $\frac{\phi V_{n,\ STD\ 2002}}{V_u}$ and Shear Inventory Rating Factors (RFs) for Girders without Considering End Blocks

Properties						**$\phi V_{n,2002STD}/V_u$ (Shear Inventory RF)**		
Bridge No	**Design Spec.**	***h* (in)**	***L* (ft)**	**0.1*L* (ft)**	**Critical section from end (ft)**	**At $h_c/2$**	**0.1*L***	**Presence of End Blocks at $h_c/2$**
9200	1961	54	92.5	9.88	3.25	1.25 (1.50)	1.30 (1.57)	YES (C.W.*)
73865	1973	63	98	10.43	4.29	1.12 (1.23)	1.00 (1.00)	YES (T.W.)
22805-1	1973	63	82.8	8.91	4.34	**0.91 (0.85)**	**0.81 (0.67)**	YES (T.W.)
22805-2_3	1973	63	102.3	10.86	4.34	1.09 (1.19)	1.05 (1.09)	YES (T.W.)
22805-4	1973	63	95.8	10.20	4.34	1.13 (1.27)	1.03 (1.07)	YES (T.W.)
36005	1973	63	105.2	11.14	4.32	1.20 (1.41)	1.10 (1.20)	YES (T.W.)
73860	1973	63	105	11.13	4.25	1.22 (1.43)	1.10 (1.19)	YES (T.W.)
27749-2_3	1983	63	104.1	11.04	4.21	1.26 (1.55)	1.14 (1.28)	YES (T.W.)
27749-4_8	1983	63	104.1	11.04	4.21	1.17 (1.34)	1.08 (1.15)	YES (T.W.)
27749-10_15	1983	63	104.1	11.04	4.21	1.12 (1.22)	1.03 (1.06)	YES (T.W.)
19813-1	1977	72	53.5	5.98	4.68	1.19 (1.28)	1.26 (1.38)	YES (T.W.)
19813-2	1977	72	119.3	12.56	4.68	1.52 (2.23)	1.47 (2.04)	YES (T.W.)
19813-3	1977	72	71.9	7.82	4.68	1.07 (1.11)	**0.94 (0.90)**	YES (T.W.)

*C.W. = In the constant width region of end block, T.W. = In the tapered width region of end block

Table 6.2 Statistical Parameters for 28-day Concrete Compressive Strength for Elk River Cylinders

Bridge Cylinders from Elk River						
Design f_c' (psi)	# of samples	Mean Nominal f'_c, (psi)	Mean Measured f_c', (psi)	% increase[1]	λ^2	COV
4750 - 5250	1419	5015	6917	37.9	1.38	0.118
5250 - 5750	333	5549	7179	29.4	1.29	**0.130**
5750 - 6250	1226	5982	7318	22.3	1.22	**0.144**
6250 - 6750	276	6471	8157	26.1	1.26	**0.117**
6750 - 7250	719	6956	8457	21.6	1.22	0.112

[1] % increase = 100*[(Mean Measured *f'c* – Mean Nominal *f'c*) / Mean Nominal *f'c*]
[2] λ = Bias Factor: Ratio of mean strength to nominal value

Table 6.3 Statistical Parameters for 28-day Concrete Compressive Strength from Nowak and Szerszen (2003)

Nowak - Statistical Parameters for Ordinary Plant-cast Concrete					
Design f_c' (psi)	# of samples	Mean Measured f_c', (psi)	% increase	λ	COV
4750 - 5250	330	6905	38.1	1.38	0.120
5250 - 5750	26	6565	19.4	1.19	0.101
5750 - 6250	493	6945	15.8	1.16	0.090
6250 - 6750	325	7415	14.1	1.14	0.081

Table 6.4 Variation in f_c' Normalized with Age to $f'_{c,28\_day}$ for Moist-Cured Specimens

Moist-Cured Specimens from Wood's Report						
Age At Test	Type of Cement					
	Type I			**Type III**		
	# of Specimens	Mean	COV	# of Specimens	Mean	COV
1 day	50	0.17	0.39	10	0.30	0.34
3 days	46	0.46	0.22	24	0.62	0.17
7 days	72	0.70	0.13	28	0.81	0.10
28 days	68	1.00	0.00	28	1.00	0.00
3 months	59	1.15	0.06	19	1.08	0.04
1 year	68	1.23	0.08	28	1.10	0.06
3 years	35	1.32	0.11	13	1.18	0.05
5 years	42	1.33	0.07	24	1.15	0.09
10 years	44	1.36	0.11	22	**1.24**	0.06
20 + years†	39	1.48	0.11	22	**1.32**	0.09

† Includes 20-year data from Series 356, 27-year data from Series 436, and 34- year data from Series 374.

Table 6.5 Variation in $f_c^{'}$ Normalized with Age to $f'_{c,28\_day}$ for Specimens Stored Outdoors

Outdoor Exposure – Specimens from Wood's Report						
Age At Test	Type of Cement					
	Type I			**Type III**		
	No.	Mean	COV	No.	Mean	COV
1 day	0	-	-	0	-	-
3 days	21	0.53	0.19	21	0.69	0.22
7 days	21	0.78	0.11	21	0.88	0.14
28 days	21	1.00	0.00	21	1.00	0.00
3 months	18	1.09	0.06	18	1.06	0.06
1 year	18	1.19	0.06	18	1.13	0.05
3 years	18	1.23	0.08	18	1.13	0.09
5 years	21	1.37	0.08	21	1.27	0.07
10 years	12	1.39	0.11	12	**1.25**	0.05
20 + years†	12	1.42	0.09	12	**1.33**	0.06

Table 6.6 Core Test Data from Literature

Investigators	Number of Cores Tested	$f'_{c\,\text{Design\_28-day}}$ (psi)	Mean $f'_{c\,\text{Measured\_28-day}}$ (psi)	Mean Measured $f'_{c\,\text{Long\_Term}}$ (psi)	Age at Time of Long-Term Test (years)	$\frac{f'_{c\,\text{Long\_Term}}}{f'_{c\,\text{Design\_28-day}}}$	$\frac{f'_{c\,\text{Long\_Term}}}{f'_{c\,\text{Measured\_28-day}}}$
Riessauw and Taerwe (1980)	N/A	N/A	7800	13800	30	-	1.77
Rabbat (1984)	N/A	5000	N/A	10100	25	2.02	-
Scanlon and Mikhailovsky (1986)	31	3000	N/A	5335	34	1.78	-
Olson (2 in. cores) (1991)*	N/A	5000	6700	8615	20	1.72	1.29
Olson (4 in. cores) (1991)*	N/A	5000	6700	8147	20	1.63	1.22
Halsy and Miller (1996)	3	N/A	6028	11790	40	-	1.96
Pessiki et.al. (1996) (1)	5	5100	N/A	8760	28	1.72	-
Pessiki et.al. (1996) (2)	5	5100	N/A	8180	28	1.60	-
Saiidi et.al. (2000)	8	5500	N/A	8450	20	1.54	-
Runzel et. Al. (2007)*	8	5900	7953	10130	20	1.72	1.27

"N/A" = The specified parameters were not provided by the corresponding investigators.

* Girders tested at University of Minnesota.

Table 6.7 Comparison of Data from Wood (1991) and Core Test Data

Investigators	Number of Cores Tested	Mean $f'_{c\,\text{Measured\_28-day}}$ (psi)	Mean Measured $f'_{c\,\text{Long\_Term}}$ f'_c (psi)	Age at Time of Test (years)	$\dfrac{f'_{c\,\text{Long\_Term}}}{f'_{c\,\text{Measured\_28-day}}}$
Riessauw and Taerwe (1980)	N/A	7800	13800	30	1.77
Olson (2 in. cores) (1991)	N/A	6700	8615	20	1.29
Olson (4 in. cores) (1991)	N/A	6700	8147	20	**1.22**
Halsy and Miller (1996)	3	6028	11790	40	1.96
Runzel et. Al. (2007)	8	7953	10130	20	1.27
[1]Wood (1991) Outdoor Exposure*	72	6588	8792	20	**1.33**

[1] Only the specimens from Series 356 were included.

*****Only the specimens stored outdoors in Skokie, Illinois were tested at 20 years. Thus, specimens stored outdoors in Dallas, Texas were not included.

Table 6.8 Calculated $\frac{\phi V_{n,\ STD\ 2002}}{V_u}$ and Shear Rating Factors at Critical Section Based on 20% Increase in $f_c^{'}$

Bridge No	h (in)	Old $f_c^{'}$ (psi)	New $f_c^{'}$ (psi)	Old V_{cw} (k)	V_s (k)	Old ϕV_n (k)	% of V_{cw} in V_n	V_u (k)	New V_{cw} (k)	New ϕV_n (k)	Old $\frac{\phi V_n}{V_u}$	New $\frac{\phi V_n}{V_u}$	% increase in $\frac{\phi V_n}{V_u}$	Old Inv. RF	New Inv. RF	% increase in RF
24831-1	36	5816	6979	143.7	66.5	189.2	68	231.9	149.5	194.3	0.82	0.84	2.7	0.75	0.78	4.0
27068-1_3	36	6000	7200	128.0	46.5	157.1	73	190.1	133.2	161.8	0.83	0.85	3.0	0.77	0.80	4.3
36006-1_3	36	5000	6000	132.3	44.2	158.8	75	191.9	137.5	163.5	0.83	0.85	3.0	0.77	0.80	4.2
55031	40	6000	7200	141.0	71.0	190.7	67	251.7	147.4	196.5	0.76	0.78	3.0	0.65	0.69	5.0
9603-1_3	40	5000	6000	115.7	71.8	168.8	62	218.4	121.4	173.9	0.77	0.80	3.1	0.71	0.74	4.3
9603-2	40	5000	6000	113.6	52.9	149.8	68	186.3	119.2	154.9	0.80	0.83	3.4	0.70	0.75	5.8
27978-1	40	5000	6000	123.8	90.5	192.9	**58**	195.3	129.4	197.9	0.99	1.01	**2.6**	0.98	1.02	**3.5**
48010	45	5000	6000	133.7	51.9	167.0	72	225.1	141.2	173.8	0.74	0.77	4.0	0.65	0.69	6.3
31019	45	6000	7200	177.4	60.7	214.3	75	282.2	185.7	221.7	0.76	0.79	3.5	0.64	0.68	6.2
49016_1-3	45	5000	6000	144.3	58.7	182.6	71	240.2	151.5	189.2	0.76	0.79	3.6	0.67	0.71	5.6
24825_1	45	5000	6000	149.3	51.0	180.3	75	233.5	156.7	186.9	0.77	0.80	3.7	0.68	0.72	6.0
24825_5	45	5810	6972	170.2	54.9	202.6	76	248.1	178.2	209.8	0.82	0.85	3.5	0.72	0.76	6.2
83022_1-3	45	5000	6000	151.1	62.5	192.2	71	231.4	158.8	199.2	0.83	0.86	3.6	0.74	0.79	6.1
73872_1-4	54	5000	6000	202.1	68.5	243.5	75	308.9	211.8	252.2	0.79	0.82	3.6	0.69	0.73	6.1
27942	54	5840	7008	213.2	28.1	217.2	**88**	231.3	225.4	228.2	0.94	0.99	**5.1**	0.88	0.97	**11.1**
62860	60	6000	7200	284.7	49.8	301.0	85	330.0	296.3	311.5	0.91	0.94	3.5	0.82	0.89	7.7
												MAX	5.1		MAX	11.1
												MIN	2.6		MIN	3.5
												AVG.	3.4		AVG.	5.8

Table 6.9 Calculated $\frac{\phi V_{n,\ STD\,2002}}{V_u}$ and Shear Rating Factors at 0.1L Based on 20% Increase in $f_c^{'}$

Bridge No	h (in)	Old $f_c^{'}$ (psi)	New $f_c^{'}$ (psi)	Old V_{cw} (kips)	V_s (kips)	Old ϕV_n (kips)	% of V_{cw} in V_n	V_u (kips)	New V_{cw} (kips)	New ϕV_n (kips)	Old $\frac{\phi V_n}{V_u}$	New $\frac{\phi V_n}{V_u}$	% increase in $\frac{\phi V_n}{V_u}$	Old Inv. RF	New Inv. RF	% increase in RF
36006-1_3	36	5000	6000	136.6	42.1	178.7	76	179.9	141.8	165.5	0.89	0.92	2.9	0.86	0.89	4.0
27068-1_3	36	5000	6000	131.3	46.5	177.8	74	177.8	136.5	164.7	0.90	0.93	2.9	0.87	0.90	4.0
9603-1_3	40	5000	6000	116.5	71.8	188.3	62	210.2	122.2	174.6	0.81	0.83	3.0	0.75	0.78	4.2
55031	40	6000	7200	155.7	72.0	227.7	68	232.4	162.2	210.8	0.88	0.91	2.9	0.83	0.87	4.3
48010	45	5000	6000	139.7	52.6	192.4	73	213.5	147.4	180.0	0.81	0.84	4.0	0.74	0.79	5.9
49016_1-3	45	5000	6000	150.7	59.5	210.2	72	226.1	158.1	195.8	0.84	0.87	3.5	0.78	0.82	5.2
24825_1	45	5000	6000	156.3	51.2	207.5	75	218.3	163.7	193.4	0.86	0.89	3.6	0.80	0.84	5.4
62825-1_3	45	5000	6000	138.1	58.9	197.1	70	200.6	145.5	183.9	0.88	0.92	3.7	0.85	0.89	5.1
31019	45	6000	7200	195.0	61.8	256.7	76	261.3	203.4	238.6	0.88	0.91	3.3	0.83	0.87	5.1
73872_1-4	54	5000	6000	209.2	68.5	277.7	75	288.5	218.9	258.6	0.87	0.90	3.5	0.80	0.85	5.5
22805-1	63	5000	6000	183.1	56.0	239.1	77	266.6	191.9	223.1	0.81	0.84	3.7	0.67	0.72	7.6
												MIN	2.9		MIN	4.0
												MAX	4.0		MAX	7.6
												AVG.	3.4		AVG.	5.1

Table 6.10 Calculated $\frac{\phi V_{n,\ STD\,2002}}{V_u}$ and Shear Rating Factors at 0.3L and 0.4L Based on 20% Increase in $f_c^{'}$

Bridge No	h (in)	Old $f_c^{'}$ (psi)	New $f_c^{'}$ (psi)	Old V_{ci} (kips)	V_s (kips)	Old ϕV_n (kips)	% of V_{ci} in V_n	V_u (kips)	New V_{ci} (kips)	New ϕV_n (kips)	Old $\frac{\phi V_n}{V_u}$	New $\frac{\phi V_n}{V_u}$	% increase in $\frac{\phi V_n}{V_u}$	Old Inv. RF	New Inv. RF	% increase in RF
At Section 0.3L																
24831-3	36	5000	6000	81.1	33.2	114.3	71	133.7	83.4	105.0	0.77	0.79	2.1	0.72	0.74	2.7
24831-2	36	5838	7006	72.7	34.3	107.0	68	119.3	74.7	98.2	0.81	0.82	1.9	0.75	0.77	2.6
24825_5	45	5810	6972	104.6	46.1	150.8	69	153.6	107.6	138.3	0.88	0.90	1.9	0.84	0.87	2.8
31019	45	6000	7200	123.9	47.2	171.1	72	173.5	127.1	156.9	0.89	0.90	1.9	0.85	0.87	2.6
62860	60	6000	7200	124.0	47.3	171.3	72	187.7	127.7	157.5	0.82	0.84	2.2	0.71	0.74	4.0
At Section 0.4L																
24831-2	36	5838	7006	47.2	26.9	74.0	64	86.7	48.9	68.2	0.77	0.79	2.3	0.73	0.75	2.8
24831-1	36	5816	6979	61.3	35.2	96.5	63	107.7	63.2	88.6	0.81	0.82	2.0	0.77	0.79	2.4
31019	45	6000	7200	80.4	31.1	111.4	72	129.8	83.0	102.7	0.77	0.79	2.4	0.72	0.75	3.1
24825_5	45	5810	6972	65.0	38.8	103.7	63	111.0	67.4	95.6	0.84	0.86	2.4	0.81	0.84	3.0
62860	60	6000	7200	78.8	49.8	128.6	61	136.0	82.0	118.7	0.85	0.87	2.5	0.79	0.82	3.7
												MIN	1.9		MIN	2.4
												MAX	2.5		MAX	4.0
												AVG.	2.2		AVG.	3.0

Table 6.11 Shear Test Results of Deep Pretensioned I-Girders Compared to the 2002 AASHTO Standard Specifications

Reference	Beam Name	f'_c (psi)	d (in)	b_w (in)	a/d	Loading*	$\rho_v f_y$ (psi)	V_{test} (k)	$V_{nSTD2002}$ (k)	V_{test} / $V_{nSTD2002}$	Means of Anchorage	Failure Mode
Kaufman Ramirez (1988)	I-2	8340	25.5	6.00	2.35	SS-2PL	117	145.0	104.9	1.38	Beam Overhang	Web-crushing
	I-3	8370	25.5	6.00	2.35	SS-2PL	139	100.0	110.4	**0.91**	None	Strand Slip
	I-4	8370	25.5	6.00	2.35	SS-2PL	117	110.0	108.2	1.02	None	Strand Slip
	II-1	9090	33.3	6.00	2.52	SS-2PL	164	140.0	156.2	**0.90**	None	Strand Slip
Rangan (1991)	II-1	6525	22.2	2.52	2.48	SS-2PL	1327	103.6	60.0	1.73	-	Web-crushing
	II-2	4568	22.2	2.48	2.48	SS-2PL	2244	85.2	50.6	1.68	-	Web-crushing
	II-3	6467	22.2	2.87	2.48	SS-2PL	1164	110.0	67.2	1.64	-	Web-crushing
	II-4	6235	22.2	2.91	2.48	SS-2PL	1910	107.8	66.9	1.61	-	Web-crushing
	III-1	5800	22.1	2.60	2.50	SS-2PL	1287	82.7	58.8	1.41	-	Web-crushing
	III-2	5365	22.1	2.60	2.50	SS-2PL	2142	87.8	57.7	1.52	-	Web-crushing
	III-3	5655	22.1	3.03	2.50	SS-2PL	1103	89.1	67.9	1.31	-	Web-crushing
	III-4	5365	22.1	2.87	2.50	SS-2PL	1936	101.8	63.4	1.60	-	Web-crushing
Russell Bruce Roller (2003)	BT6Live	11780	77.0	6.00	1.56	SS-3PL	417	630	395.0	1.59	-	Strand Slip
	BT6Dead	11590	77.0	6.00	1.56	SS-3PL	472	596	422.0	1.41	-	Strand Slip
	BT7Live	12400	77.0	6.00	1.56	SS-3PL	641	654	499.0	1.31	-	Did not fail
	BT7Dead	12730	77.0	6.00	1.56	SS-3PL	282	645	523.0	1.23	-	Interface failure
	BT8Live	11850	77.0	6.00	1.56	SS-3PL	708	639	327.0	1.95	-	Did not fail
	BT8Dead	11310	77.0	6.00	1.56	SS-3PL	315	600	338.0	1.78	-	Interface failure
Shahawy Batchelor (1996)	A0-00-R_N	8480	39.3	6.00	2.17	SS-1PL	669	313.0	245.6	1.27	-	Strand Slip
	A0-00-R_S	8480	39.3	6.00	2.17	SS-1PL	669	276.0	245.6	1.12	-	Shear-Tension
	A0-00-RD_N	7300	39.3	6.00	1.89	SS-1PL	669	230.0	241.1	**0.95**	No Confine. bars	Strand Slip
	A0-00-RD_S	7300	39.3	6.00	2.17	SS-1PL	669	228.0	245.6	**0.93**	No Confine. bars	Strand Slip
	A3-00-RB_S	7100	39.3	6.00	2.17	SS-1PL	669	275.0	245.6	1.12	-	Strand Slip
	B1-00-R_N	7450	39.3	6.00	1.53	SS-1PL	669	245.0	235.6	1.04	-	Strand Slip
	B1-00-R_S	7450	39.3	6.00	1.38	SS-1PL	669	232.0	233.3	**0.99**	-	Strand Slip
	C0-00-RD_N	7113	39.3	6.00	1.53	SS-1PL	669	189.0	236.7	**0.80**	No Confine. bars	Strand Slip
Ma et al. (2000)	AVW14408X	8100	48.0	5.9	1.13	SS-3PL	71	593.0	330	1.80	Bent Strands	Web-crushing
	BVW20408X	10780	47.6	5.9	1.13	SS-3PL	136	589.8	510	1.16	Bent Strands	Web-crushing

*SS-1 PL: Simple span - 1 Point Loading

Table 6.12 Comparison of Maximum Distance to the Rear Tandem from the Support for $\phi V_{n,\ STD\,2002}/V_u < 1$ with $2.5h_c$ to Check the Applicability of Arching Action

Bridge No.	Shear Inventory RF	Maximum distance to the rear tandem from the support for $\frac{\phi V_{n,\ STD\,2002}}{V_u} < 1$ (ft)	$2.5h_c$ (ft)
31019	0.64	20.0	11.8
55031	0.65	18.1	10.5
48010	0.65	13.8	11.6
49016_1-3	0.67	14.5	11.5
24825_1	0.68	15.6	11.6
73872_1-4	0.69	17.5	13.4
9603-2	0.70	17.2	10.3
9603-1_3	0.71	9.2	10.5
24825_5	0.72	18.3	11.6
83022_1-3	0.74	14.4	11.9
24831-1	0.75	10.9	9.7
9011	0.75	19.1	11.5
25013-1_3	0.76	19.0	13.4
36006-1_3	0.77	9.3	9.6
27068-1_3	0.77	9.7	9.6
73852-1_4	0.78	14.5	13.1
19033	0.79	10.5	9.6
8011	0.79	10.8	11.4
24831-3	0.80	8.3	9.7
14006-1_3	0.80	10.0	9.8
62860	0.82	19.1	14.6

Table 6.13 Shear Live Load Distribution Factors for Exterior Girders (Huo et al. 2003)

Bridge No.	Beam Spacing (ft)	Skew Angle (deg)	Span Length (ft)	Beam	FEA	AASHTO Standard	AASHTO LRFD	Mod. Henry's Method	FEA / Mod. Henry's Method
6*	9.0	21.3	67.6	Exterior	0.677	0.750	0.960	0.786	0.86
7*	9.0	33.5	76.0	Exterior	0.700	0.694	0.934	0.826	0.85
24*	10.6	0.0	74.3	Exterior	0.841	0.850	0.945	0.860	0.98
5	8.8	15.0	124.3	Exterior	0.730	0.743	0.926	0.768	0.95
8	10.3	0.0	115.5	Exterior	0.784	0.810	0.920	0.869	0.90
22	8.3	26.7	159.0	Exterior	0.756	0.610	0.785	0.861	0.88
23	8.3	17.5	151.3	Exterior	0.727	0.610	0.765	0.832	0.87
								AVG	0.90
								COV	0.05

* Precast Concrete I-Beams

Table 6.14 Shear Live Load Distribution Factors for Interior Girders (based on Huo et al. 2003)

Bridge No.	Beam Spacing (ft)	Skew Angle (deg)	Span Length (ft)	Beam	FEA	AASHTO Standard	AASHTO LRFD	Mod. Henry's Method	FEA / Mod. Henry's Method
6*	9.0	21.3	67.6	Interior	0.917	0.818	0.943	0.786	1.17
7*	9.0	33.5	76.0	Interior	0.770	0.818	0.991	0.826	0.93
24*	10.6	0.0	74.3	Interior	0.940	0.962	0.990	0.860	1.09
5	8.8	15.0	124.3	Interior	0.931	0.795	0.900	0.768	1.21
8	10.3	0.0	115.5	Interior	0.960	0.935	0.971	0.869	1.10
22	8.3	26.7	159.0	Interior	0.933	0.757	0.898	0.861	1.08
23	8.3	17.5	151.3	Interior	0.932	0.757	0.875	0.832	1.12
								AVG	1.10
								COV	0.08

* Precast Concrete I-Beams

Table 6.15 Ratio of Shear Live Load Distribution Factors from Simplified Methods to Those from Grillage Analysis for Exterior Girders (Puckett et al. 2007)

Shear, One Lane, Exterior, Slab-on-Concrete I-girder					Shear, Multiple Lanes, Exterior, Slab-on-Concrete I-girder				
Parameter	AASHTO Standard	AASHTO LRFD	Modified Henry's Method	Calibrated Lever Rule	Parameter	AASHTO Standard	AASHTO LRFD	Modified Henry's Method	Calibrated Lever Rule
Average	1.455	1.305	0.959	0.988	Average	1.378	1.307	0.999	0.998
STD	0.414	0.172	0.394	0.033	STD	0.403	0.312	0.293	0.043
COV	0.285	0.132	0.411	0.033	COV	0.292	0.239	0.293	0.043
Count	67	69	69	69	Count	67	69	15	69

Table 6.16 Ratio of Shear Live Load Distribution Factors from Simplified Methods to Those from Grillage Analysis for Interior Girders (Puckett et al. 2007)

Shear, One Lane, Interior, Slab-on-Concrete I-girder					Shear, Multiple Lanes, Interior, Slab-on-Concrete I-girder				
Parameter	AASHTO Standard	AASHTO LRFD	Modified Henry's Method	Calibrated Lever Rule	Parameter	AASHTO Standard	AASHTO LRFD	Modified Henry's Method	Calibrated Lever Rule
Average	1.314	1.296	0.999	1.003	Average	0.968	1.134	1.004	1.011
STD	0.135	0.094	0.085	0.048	STD	0.111	0.154	0.092	0.094
COV	0.103	0.072	0.085	0.048	COV	0.115	0.136	0.092	0.093
Count	73	73	73	73	Count	73	73	15	73

Table 6.17 Live Load Shear Calibration Factors (Puckett et al. 2007)

Structure Type	AASHTO LRFD Cross Section Type[1]	Shear							
		Exterior				Interior			
		One Loaded Lane		Two or More Lanes		One Loaded Lane		Two or More Lanes	
		a_v	b_v	a_v	b_v	a_v	b_v	a_v	b_v
		Lever Rule							
Precast Concrete Beams[1]	h, i, j, k	0.83	0.07	0.92	0.06	1.08	-0.13	0.94	0.03

[1]Corresponding cross section types are shown in Table 4.6.2.2.1-1 per 2004 AASHTO LRFD Article 4.6.2.2.1. Cross section types include Precast Concrete; I-Beam, Bulb-Tee Beam, Tee Section with Shear Keys with or without Transverse Post-Tensioning, Double Tee with Shear Keys with or without Transverse Post-Tensioning, Channel with Shear Keys

Table 6.18 Analysis Factors, γa, for Shear Based on One-Half STD[1] (Puckett et al. 2007)

Structure Type	AASHTO LRFD Cross Section Type	Action	Exterior		Interior	
			One Loaded Lane	Two or More Lanes	One Loaded Lane	Two or More Lanes
Precast Concrete Beams	h, i ,j, k	Shear	1.00	1.00	1.00	1.05

[1] STD: Standard Deviation

Table 6.19 Parameters of Selected Bridges for Shear LLDF Comparison

Bridge No	Year Built	h (in)	L (ft)	S_g (ft)	Number of Girders	Clear Roadway Width (ft)	Beam $f_c^{'}$ (psi)	Slab $f_c^{'}$ (psi)	Slab Thickness (in)	Skew Angle (degree)	Overhang (ft)
27978-2	1965	40	52.3	11.2	4	38.5	5000	4000	8.00	20.0	3.67
27978-3	1965	40	36.5	11.2	4	38.5	5000	4000	8.00	20.0	3.67
25013-1_3	1982	54	77.3	13.5	4	45.4	5900	4000	9.50	0.0	4.08
62860	1970	60	100.7	10.8	6	56.6	6000	4000	9.00	45.7	3.75
PCI Example	-	72	120.0	12.0	4	42.0	7000	4000	7.50	0.0	4.25
Example FDOT	-	54	90.0	8.0	5	42.0	6500	4500	8.00	30.0	4.54
19813-2	1979	72	119.3	6.5	7	41.8	6000	4000	8.50	36.5	3.75
24831-3	1970	36	39.5	12.7	4	40.5	5000	4000	9.75	13.3	4.00
14006-2	1988	36	63.6	7.9	6	42.8	7000	4000	8.50	18.9	3.42
9200	1963	54	92.5	7.0	5	30.0	5000	4000	7.00	0.0	4.75
61001	1981	54	95.2	7.3	7	46.8	6000	4000	8.50	0.0	3.75
17007	1987	45	77.6	12.0	4	40.8	7000	4000	8.50	52.8	3.92

Table 6.20 Diaphragm Effect on Finite Element Analysis Results for Precast Concrete Beams (Huo et al 2003)

Bridge No.	Structure Type*	Finite Element Analysis – Shear LLDF[1]					
		Interior Beam			Exterior Beam[2]		
		With Support Diaphragm	Without Support Diaphragm	% Difference[3]	With Support Diaphragm	Without Support Diaphragm	% Diff
5	Precast Concrete BT Beam	0.931	1.000	6.9	0.730	0.850	14.1
8	Precast Concrete BT Beam	0.960	1.060	9.4	0.784	0.853	8.1
22	Precast Concrete BT Beam	0.933	1.016	8.2	0.756	0.761	0.7
23	Precast Concrete BT Beam	0.932	1.010	7.7	0.727	0.736	1.2
6	Precast Concrete I-Beam	0.917	0.918	0.1	0.677	0.694	2.4
7	Precast Concrete I-Beam	0.770	0.835	7.8	0.700	0.712	1.7
24	Precast Concrete I-Beam	0.940	1.130	16.8	0.841	0.931	9.7

*BT: Bulb-tee

[1] LLDF: Live Load Distribution Factors

[2] Values are shown for obtuse corner of the exterior girder, if the bridge is skewed

[3] % Difference = 100*(Without Support Diaphragm – With Support Diaphragm) / (Without Support Diaphragm)

Table 6.21 Diaphragm Effect on Grillage Analysis Results at Obtuse Corner, Beam 1, Bridge No. 24 (Puckett et al., 2007)

Exterior Girder		With Rigid Support Diaphragms			Without Rigid Support Diaphragms					
Skew Angle (deg)		0	30	60	0	30	60	0	30	60
Distance from Barrier (ft)	Load	Shear Distribution Factors from Grillage Analysis						% Diff in DF		
2	MOVE1	0.761	0.788	0.791	0.753	0.783	0.792	-1.1	-0.6	0.1
3	MOVE2	0.681	0.710	0.720	0.670	0.703	0.722	-1.6	-1.0	0.3
4	MOVE3	0.599	0.630	0.649	0.587	0.621	0.651	-2.0	-1.4	0.3
5	MOVE4	0.516	0.548	0.577	0.504	0.539	0.581	-2.4	-1.7	0.7
6	MOVE5	0.434	0.468	0.507	0.423	0.459	0.511	-2.6	-2.0	0.8
7	MOVE6	0.363	0.400	0.449	0.359	0.400	0.455	-1.1	0.0	1.3
8	MOVE7	0.321	0.357	0.404	0.318	0.357	0.410	-0.9	0.0	1.5
9	MOVE8	0.288	0.320	0.364	0.285	0.320	0.371	-1.1	0.0	1.9

Table 6.22 Diaphragm Effect on Grillage Analysis Results at Obtuse Corner, Beam 8, Bridge No. 24 (Puckett et al., 2007)

Interior Girder		With Rigid Support Diaphragms			Without Rigid Support Diaphragms					
Skew Angle (deg)		0	30	60	0	30	60	0	30	60
Distance from Barrier (ft)	Load	Shear Distribution Factors from Grillage Analysis						% Diff in DF		
2	MOVE1	0.758	0.788	0.835	0.750	0.783	0.823	-1.1	-0.6	-1.5
3	MOVE2	0.678	0.711	0.764	0.669	0.703	0.747	-1.3	-1.1	-2.3
4	MOVE3	0.596	0.631	0.689	0.586	0.621	0.669	-1.7	-1.6	-3.0
5	MOVE4	0.513	0.549	0.612	0.504	0.539	0.591	-1.8	-1.9	-3.6
6	MOVE5	0.432	0.468	0.534	0.424	0.458	0.518	-1.9	-2.2	-3.1
7	MOVE6	0.366	0.404	0.464	0.364	0.403	0.466	-0.5	-0.2	0.4
8	MOVE7	0.326	0.361	0.419	0.323	0.361	0.422	-0.9	0.0	0.7
9	MOVE8	0.293	0.326	0.380	0.291	0.325	0.382	-0.7	-0.3	0.5

FIGURES

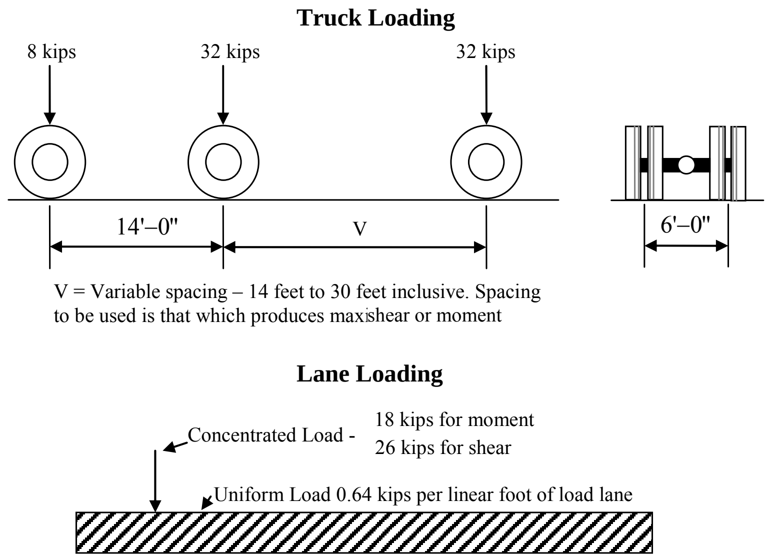
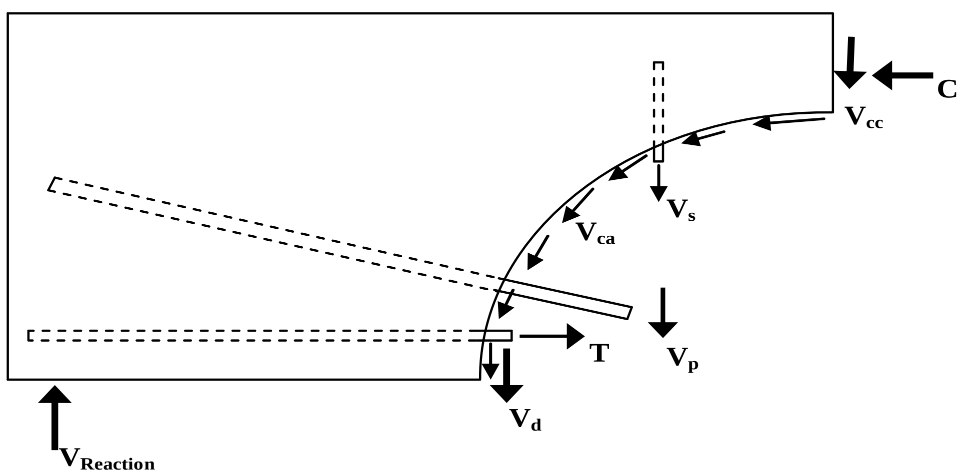


Figure 1.1 AASHTO HS-20 Loading



V_{cc}: Shear in Compression Zone
V_d: Dowel Action
V_{ca}: Aggregate Interlock
V_s: Stirrup Contribution
V_p: Vertical Component of Prestressing Force

Figure 2.1 Shear Transfer/Actions Contributing to Shear Resistance (Hawkins et al. 2005)

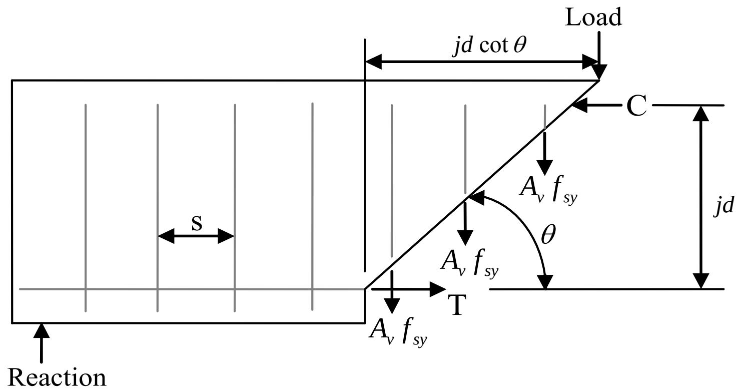


Figure 2.2 Stirrup Contribution

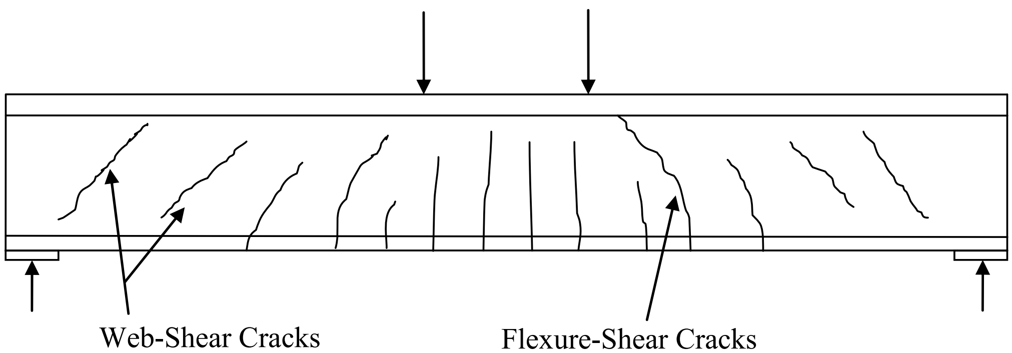


Figure 2.3 Types of Shear Cracks in Prestressed Concrete Beams

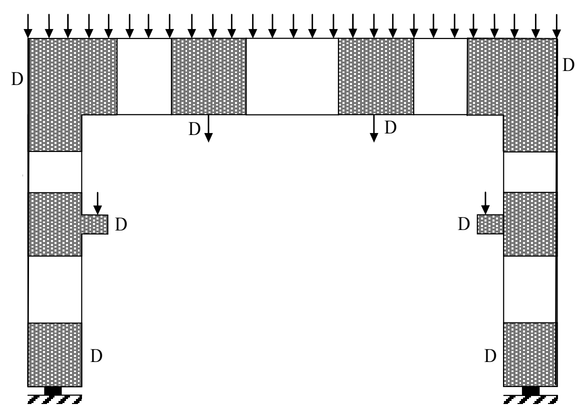


Figure 2.4 D-Regions in a Frame (Schlaich et al., 1987)

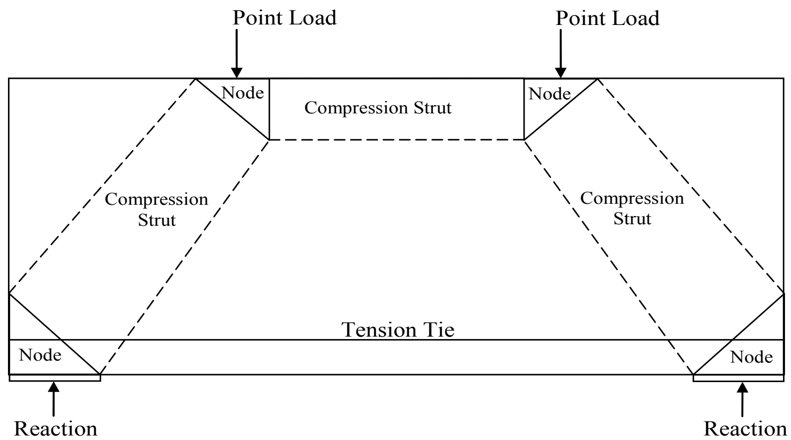


Figure 2.5 Strut-and-Tie Model for a Simple Deep Beam Figure

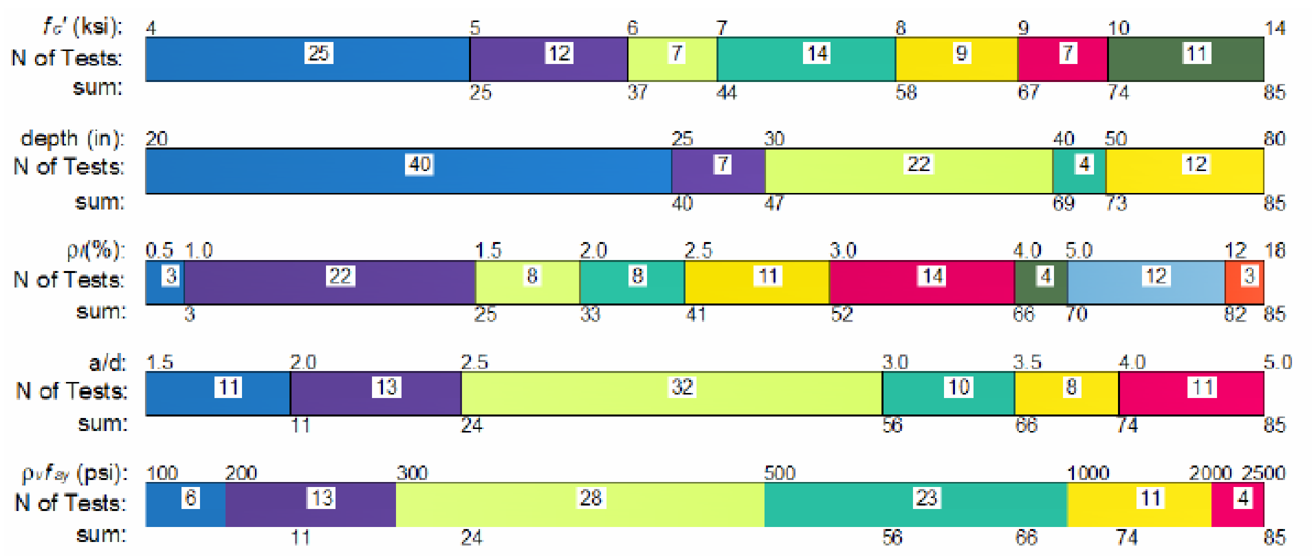


Figure 2.6 Range in Parameters for Prestressed Concrete Members (Hawkins et al., 2005)

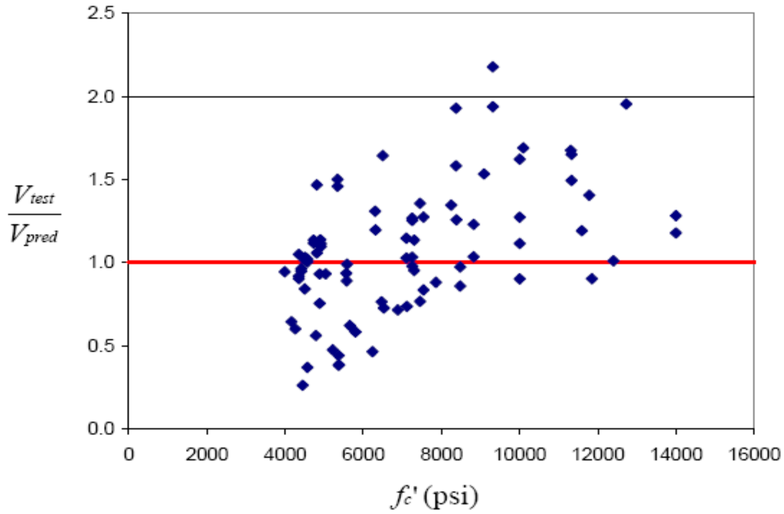


Figure 2.7 1979 Interim $\frac{V_{test}}{V_{pred}}$ vs. f_c' for 85 Prestressed Members (Hawkins et al., 2005)

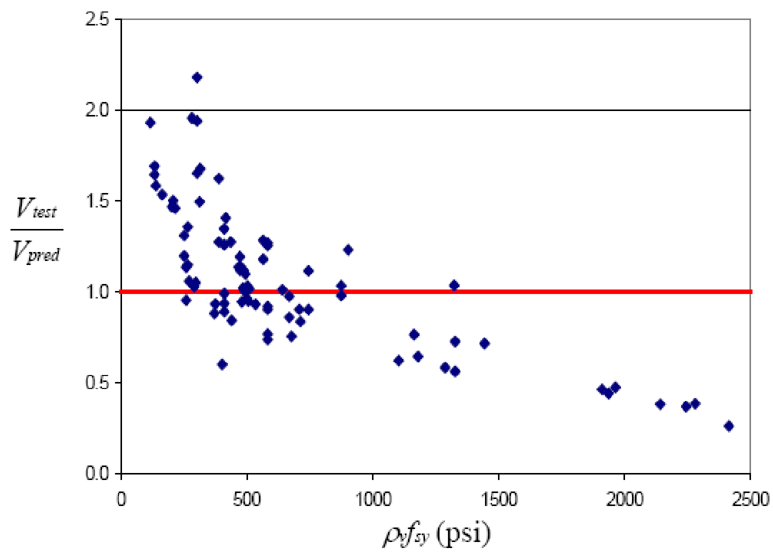


Figure 2.8 1979 Interim $\frac{V_{test}}{V_{pred}}$ vs. Stirrup Reinforcement Ratio for 85 Prestressed Members (Hawkins et al., 2005)

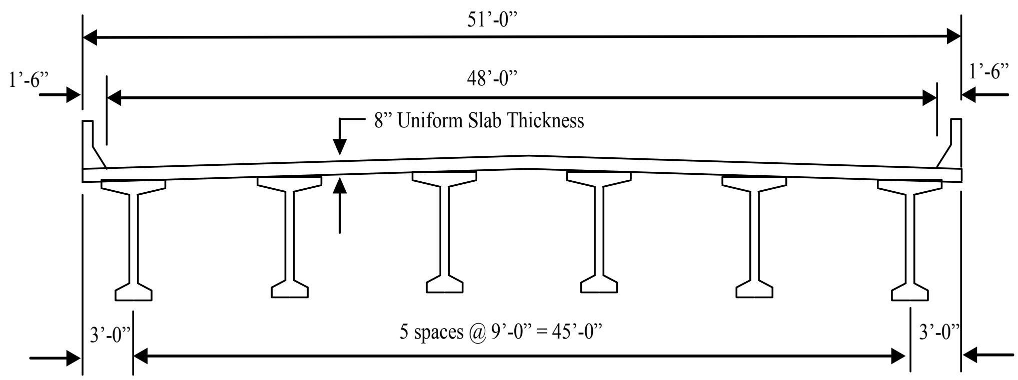


Figure 3.1 PCI Bridge Example – Bridge Cross Section

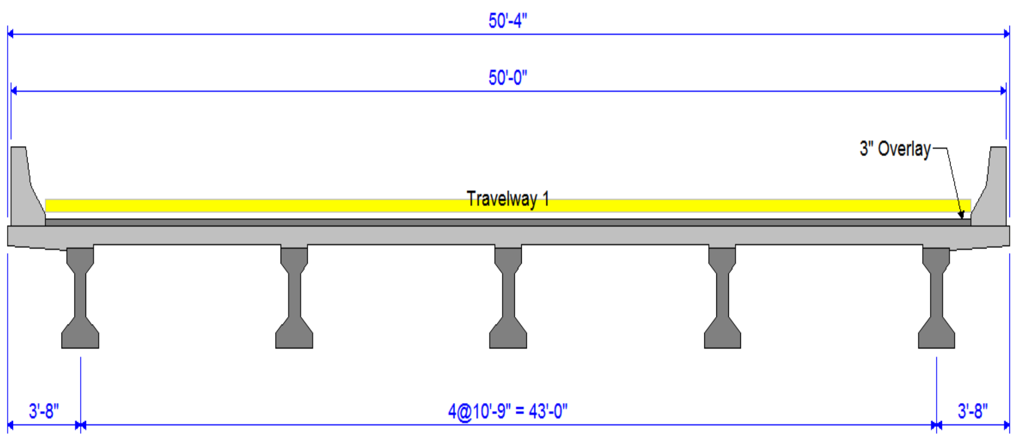


Figure 3.2 Bridge No. 83022_1-3 Cross Section

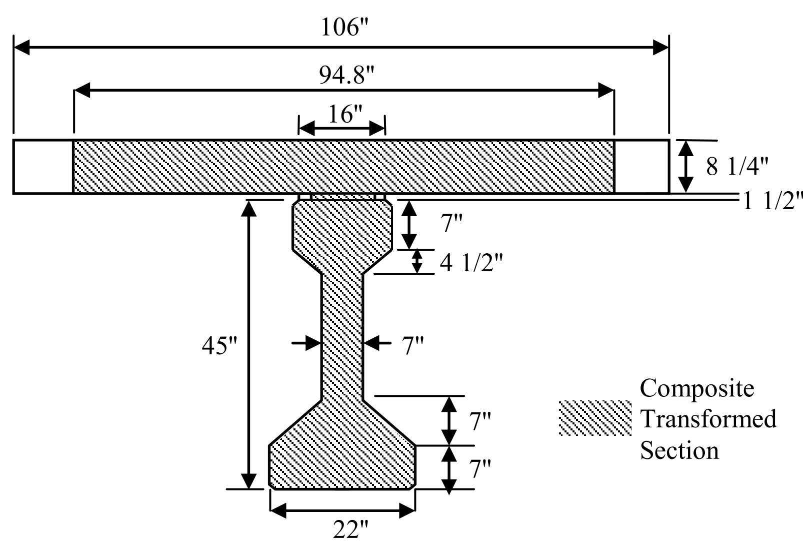


Figure 3.3 Interior Girder Composite Cross Section for Bridge No. 83022_1-3

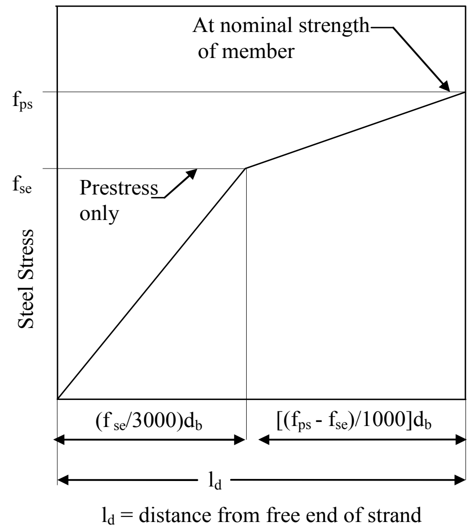


Figure 3.4 Idealized Bilinear Relationship Between Steel Stress and Distance from the Free End of Strand (ACI 318-05)

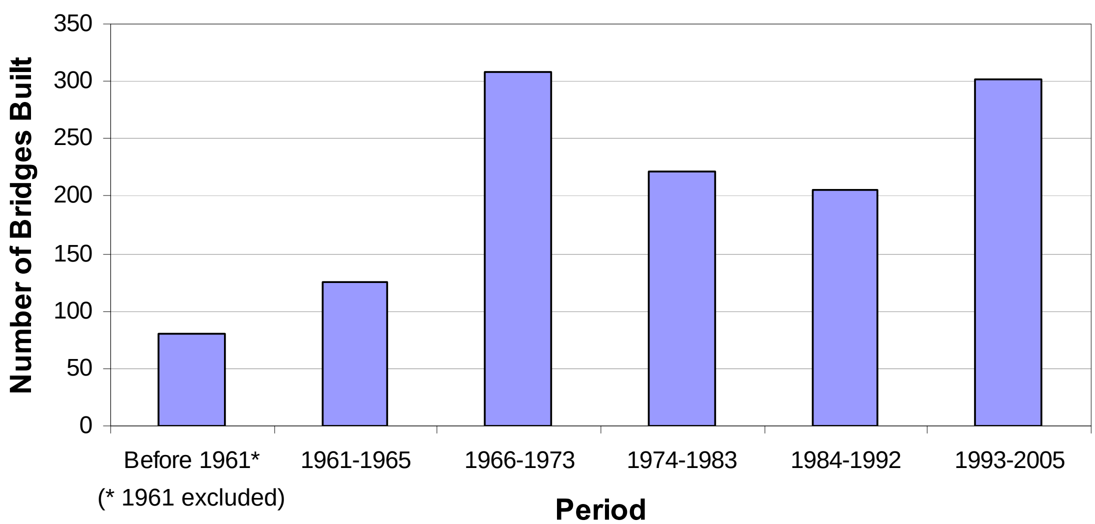


Figure 4.1 Number of Bridges Built with Respect to Time

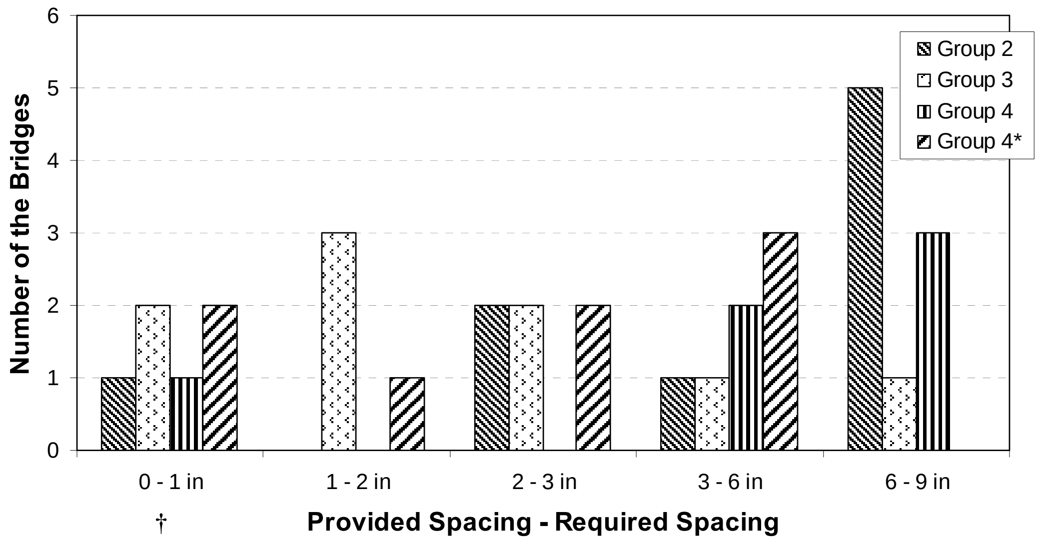


Figure 4.2 Distribution of Inadequate Stirrup Spacing for Vertical Shear (Provided – Required)

† For the ranges shown in the figure, the upper limit value is included and the lower limit value is excluded, i.e., for range 1 - 2 in, the values with 2 in are included, however the values with 1 in are excluded in the range. (This is true for all ranges except 0 – 1 in, where both limits are included in the range). The same applies to Figures 4.2 through 5.2.

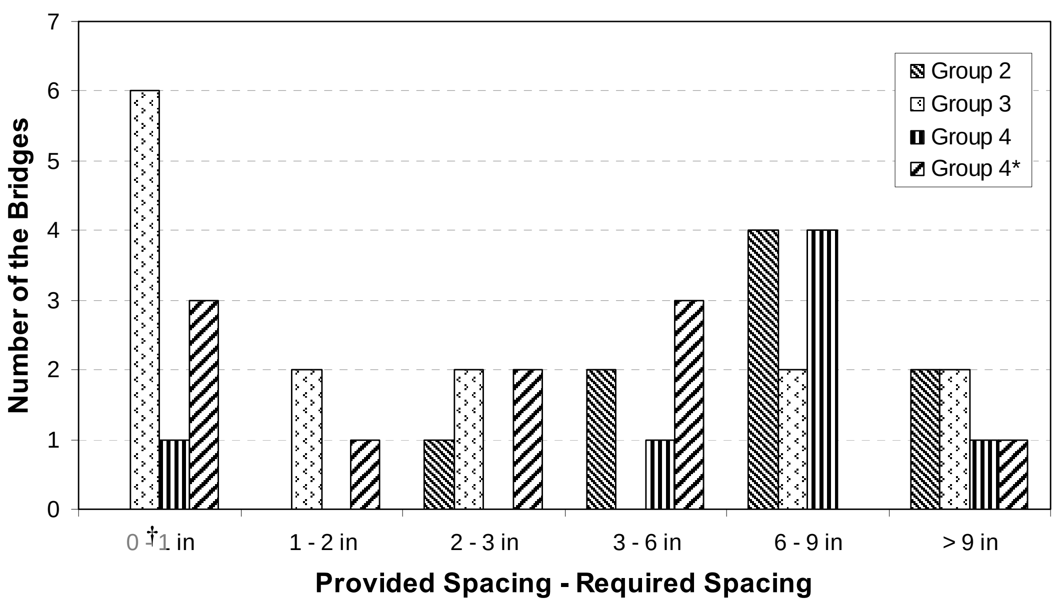


Figure 4.3 Distribution of Inadequate Stirrup Spacing (Provided – Required) (Horizontal Shear Included)

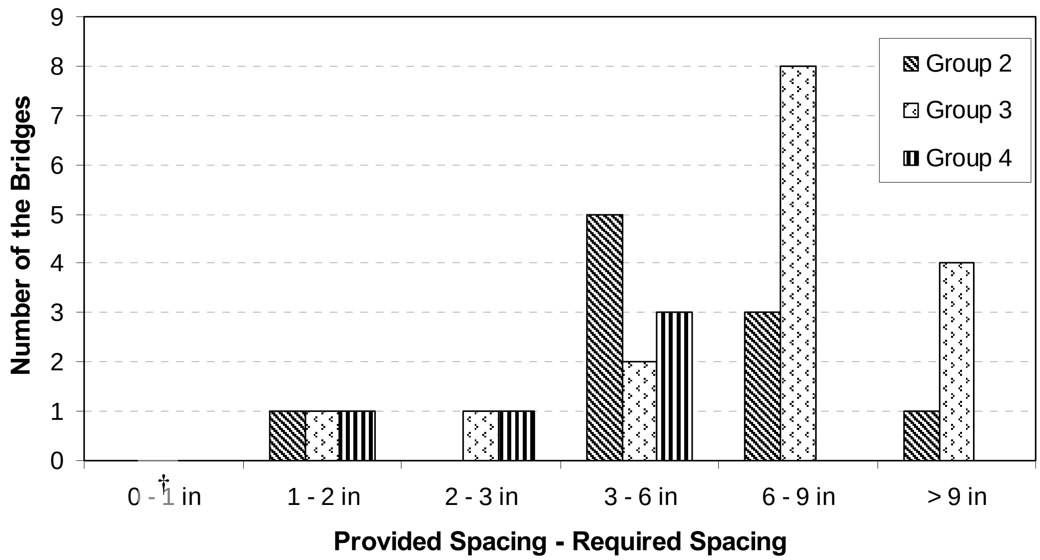


Figure 5.1 Distribution of Bridges in Design Groups (designed according to 2002 AASHTO Standard Specifications) with Stirrup Spacing Differences
(Horizontal Shear Excluded)

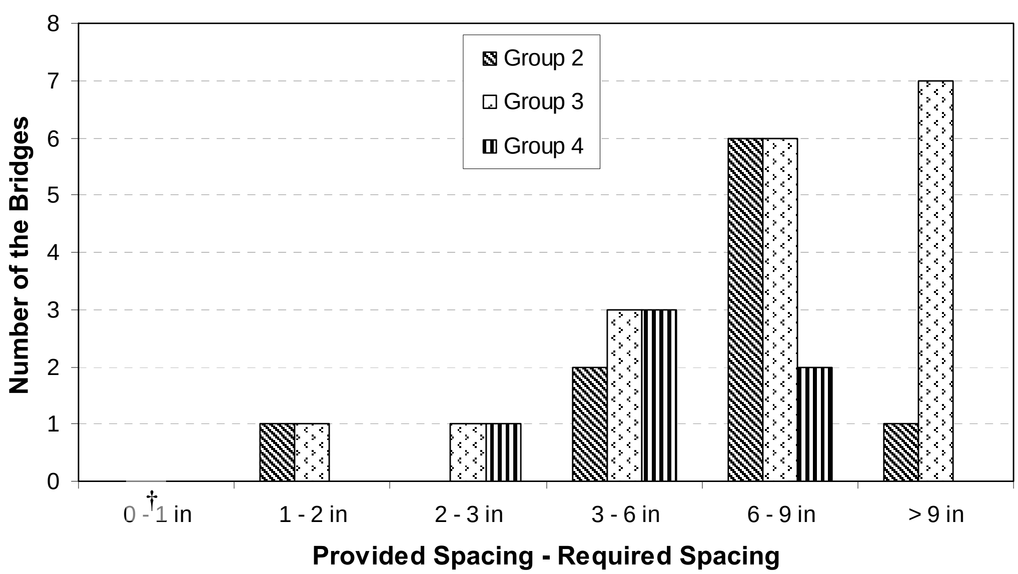


Figure 5.2 Distribution of Bridges in Design Groups with Stirrup Spacing Differences
(Horizontal Shear Included)

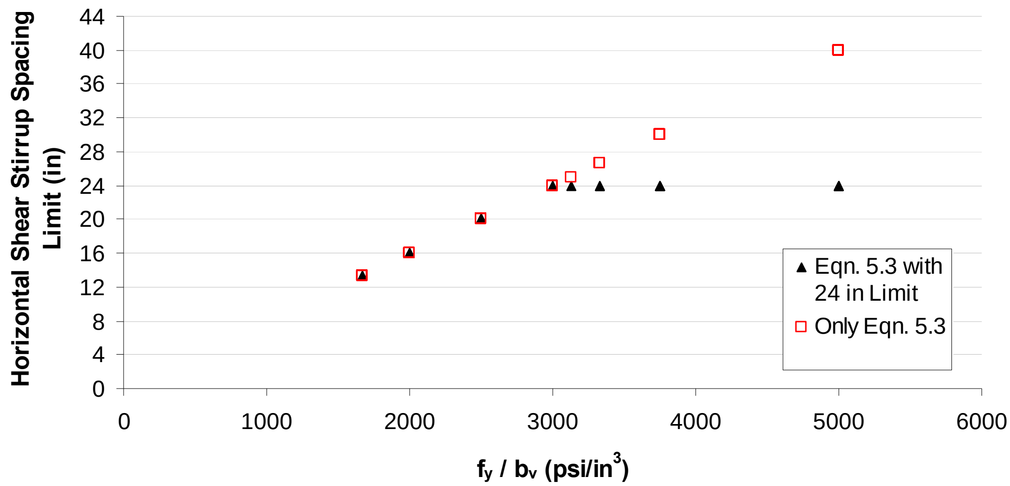


Figure 5.3 Required Spacing per Eqn. (5.3) vs. $\frac{f_y}{b_v}$ with and without 24 in. Spacing Limit

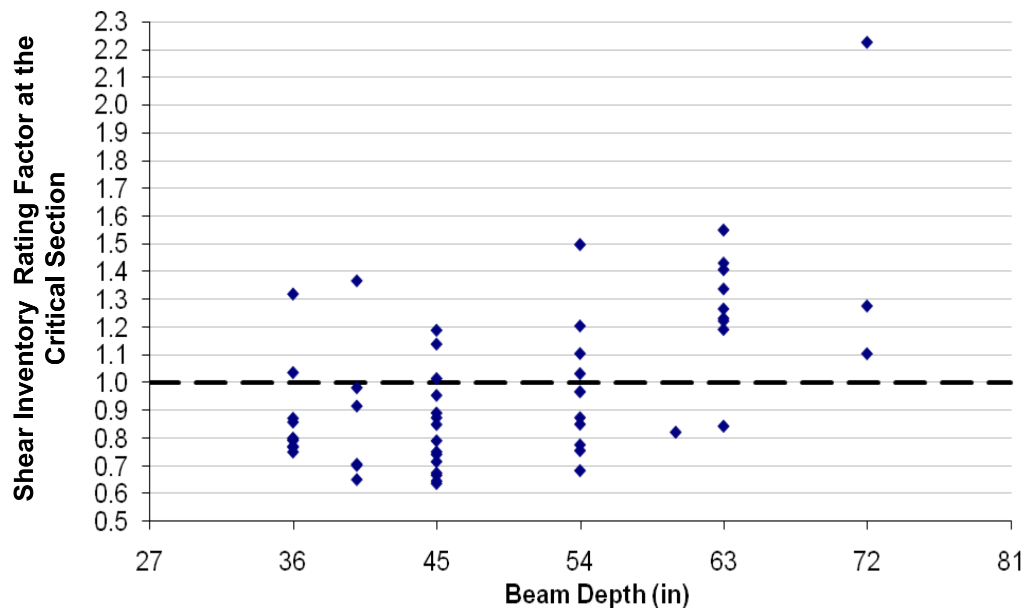


Figure 5.4 Shear Inventory RF at the Critical Section vs. Beam Depth

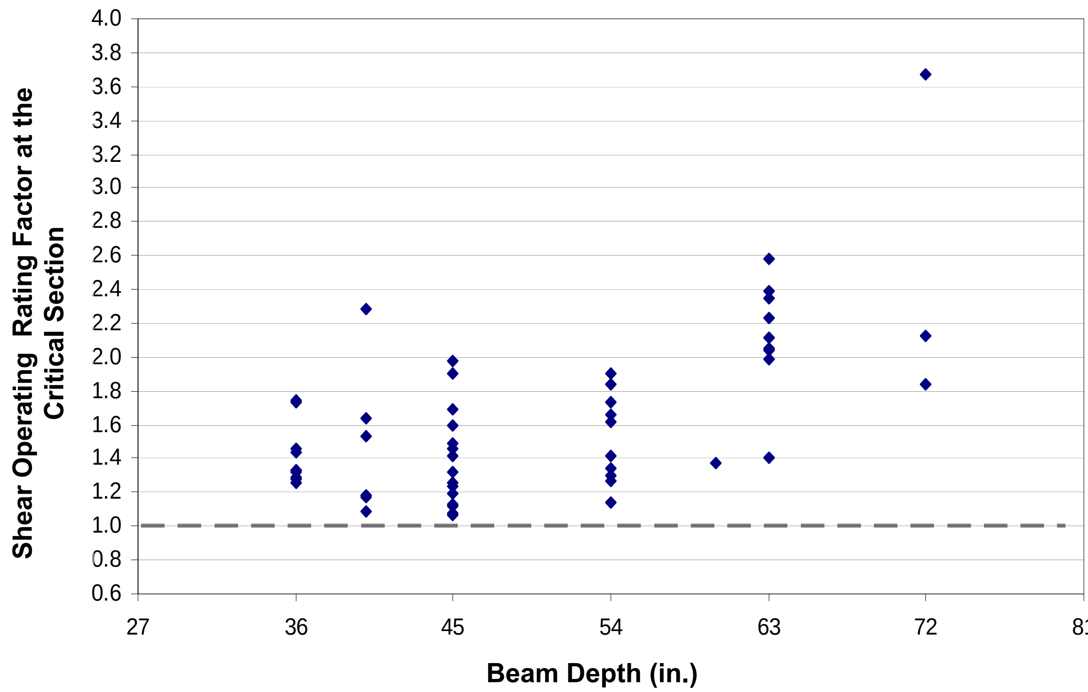


Figure 5.5 Shear Operating RF at the Critical Section vs. Beam Depth

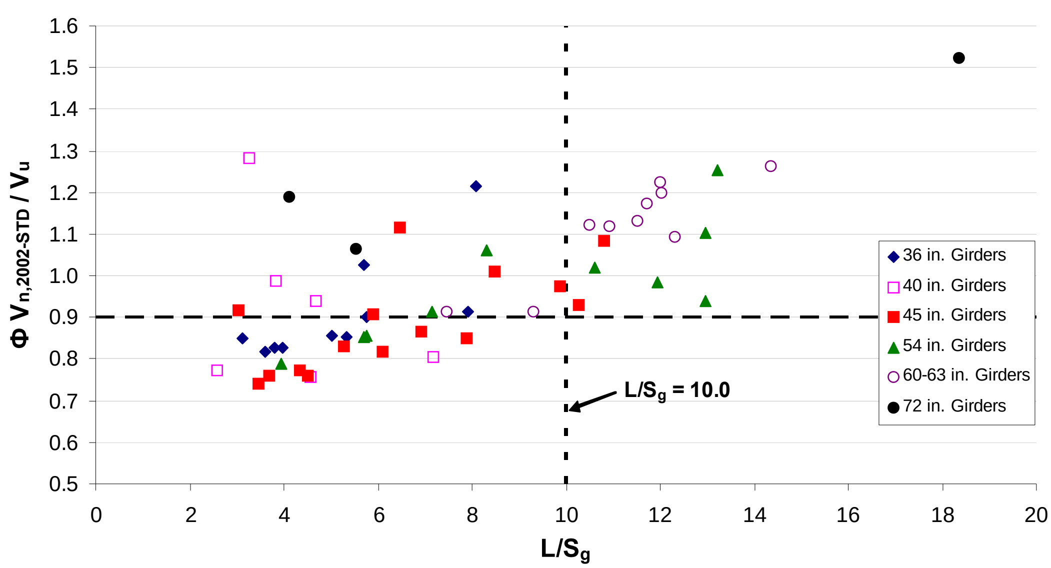


Figure 5.6 $\frac{\phi V_{n,\ STD\ 2002}}{V_u}$ at the Critical Section vs. $\frac{L}{S_g}$ for All Girders

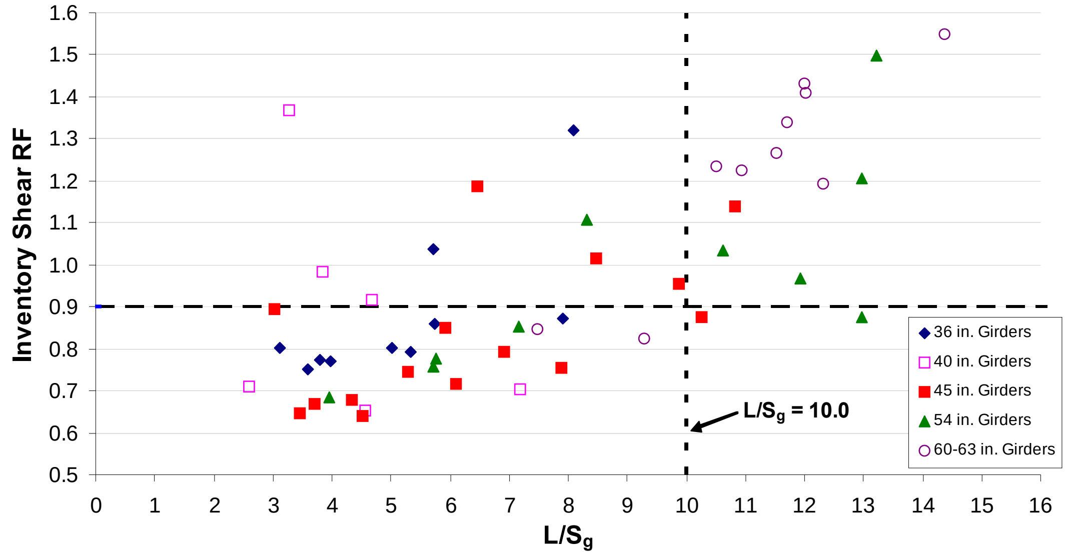
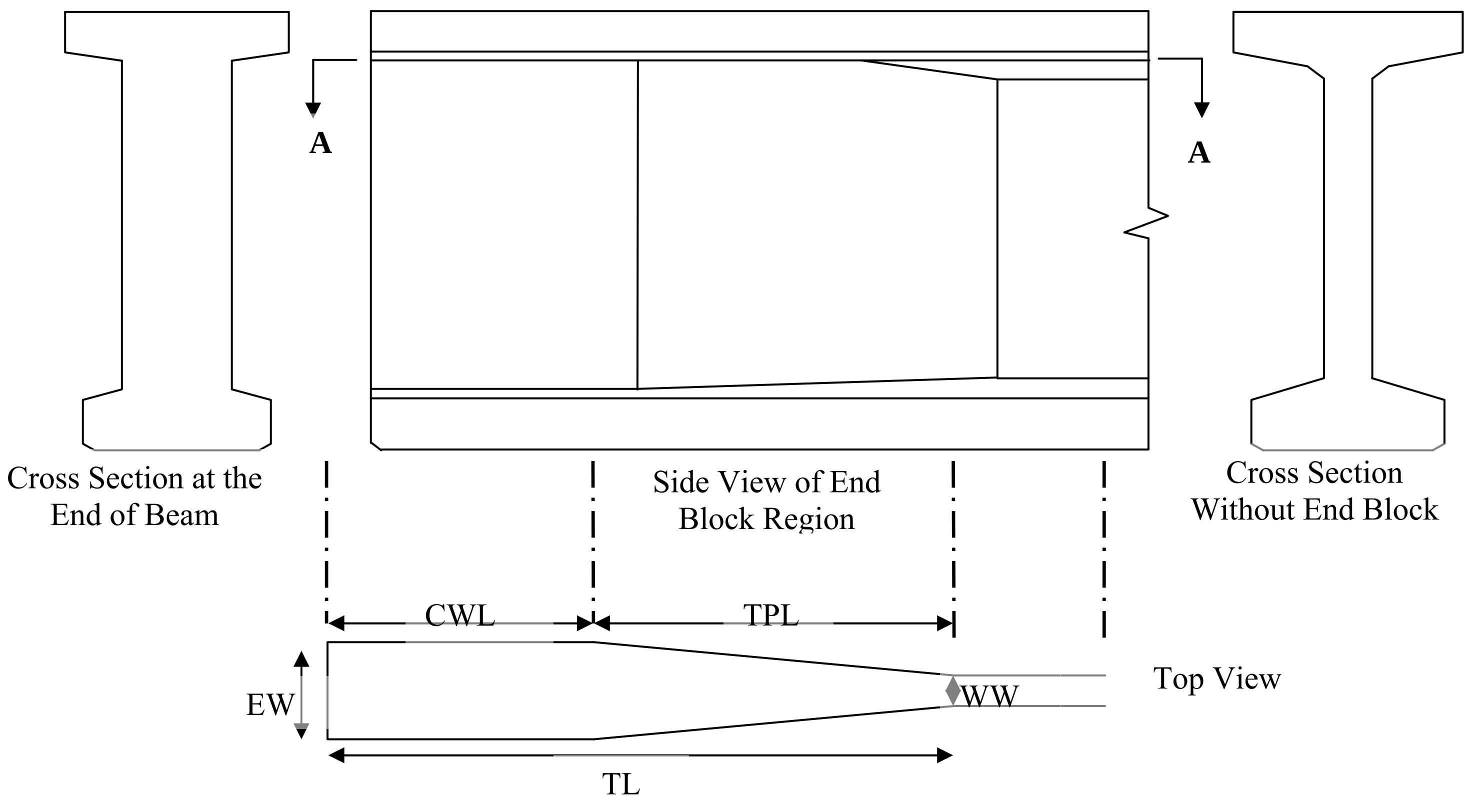


Figure 5.7 Inventory Rating Factor at the Critical Section vs. $\dfrac{L}{S_g}$ for All Girders



Section A-A	MN 54	63 M	72 M
End Width - **EW** (in.)	20	16	16
Web Width - **WW** (in.)	8	6	6
Constant Width Length - **CWL** (ft)	4.5	3	4
Tapered Length - **TPL** (ft)	1	4	4
Total Length - **TL** (ft)	5.5	7	8

Figure 6.1 Geometry and Dimensions of End Blocks for 54, 63 and 72 in. Deep Girders

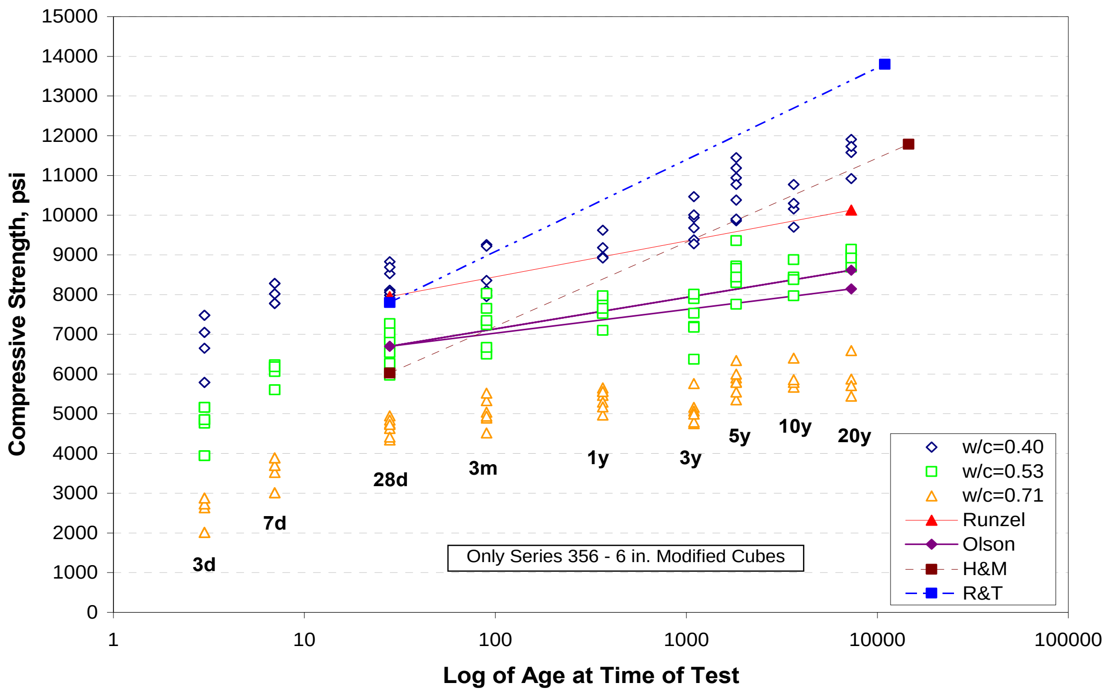


Figure 6.2 PC Type III - Outdoor Exposure from Wood (1991) and Core Test Data from the Literature

Note: H&M stands for the data from Halsey and Miller (1996) and R&T stands for the data from Riessauw and Taerwe (1980).

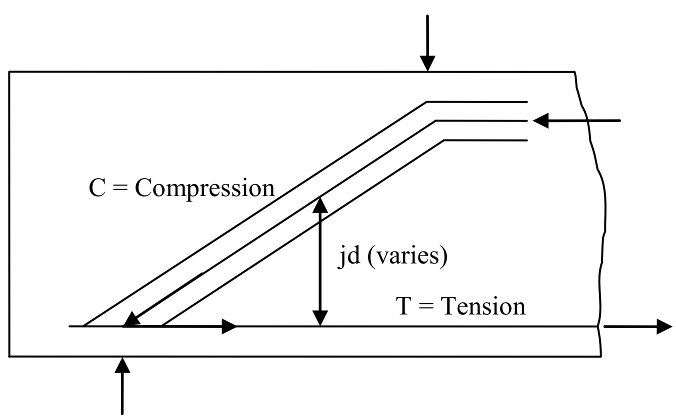


Figure 6.3 Arching Action in a Beam (MacGregor, 1997)

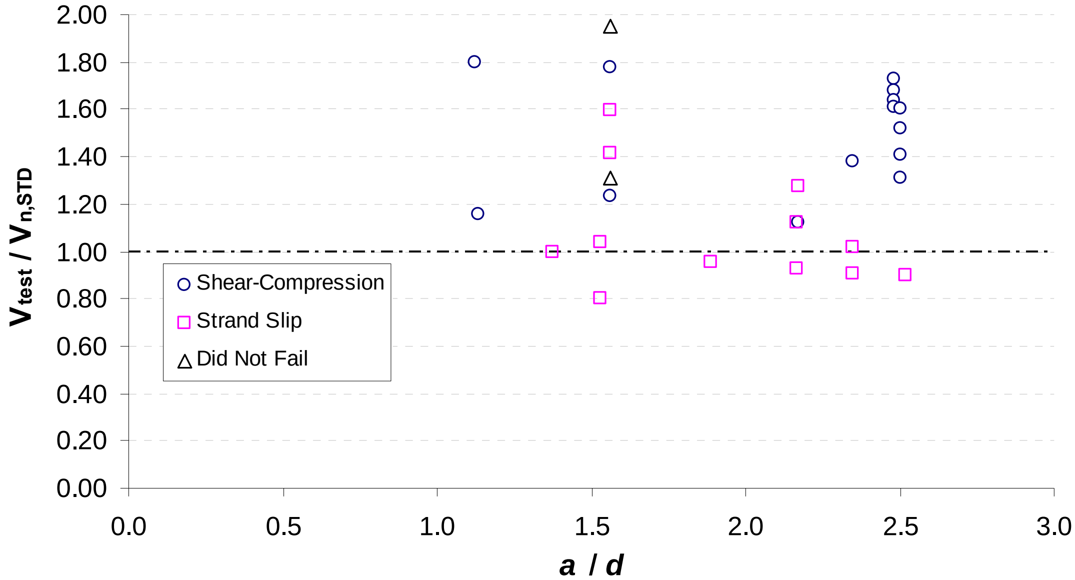


Figure 6.4 $\dfrac{V_{test}}{V_{n,\ STD\,2002}}$ vs. a/*d* for Different Types of Failure

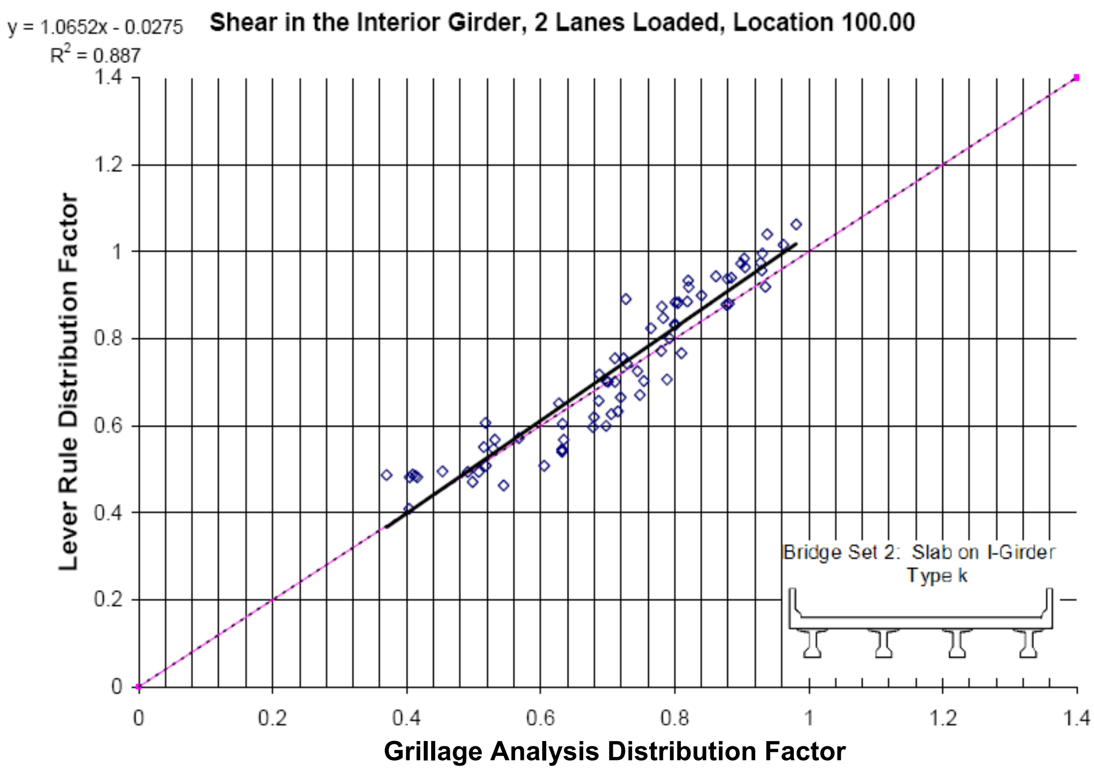


Figure 6.5 Lever Rule LLDFs vs. Grillage Analysis LLDFs for Interior Girders, for Two-Lanes Loaded, at the End of Span (Puckett et al. 2007)

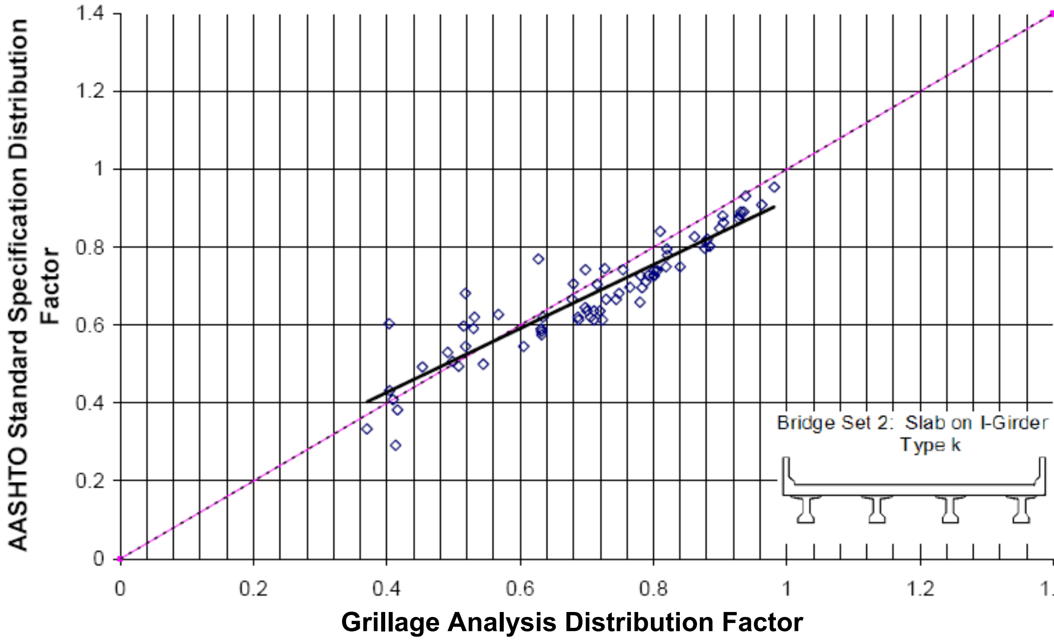


Figure 6.6 Comparison of AASHTO Standard LLDF with Grillage Analysis LLDF for Interior Girders, for Two-Lanes Loaded, at the End of Span (Puckett et al. 2007)

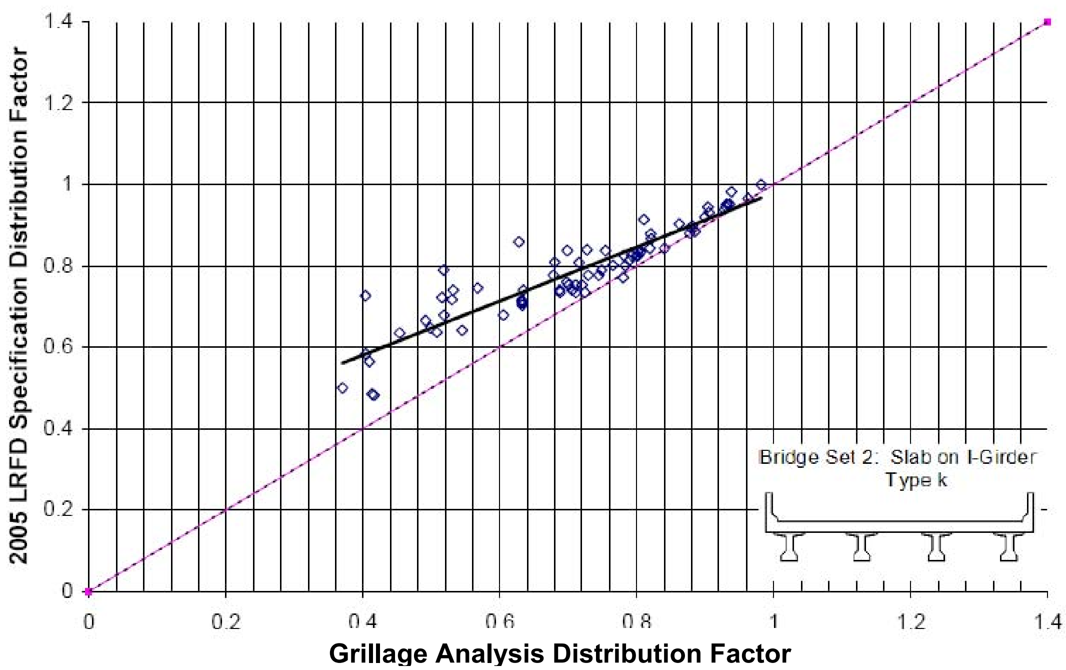


Figure 6.7 AASHTO LRFD LLDF vs. Grillage Analysis LLDF for Interior Girders, for Two-Lanes Loaded, at the End of Span (Puckett et al. 2007)

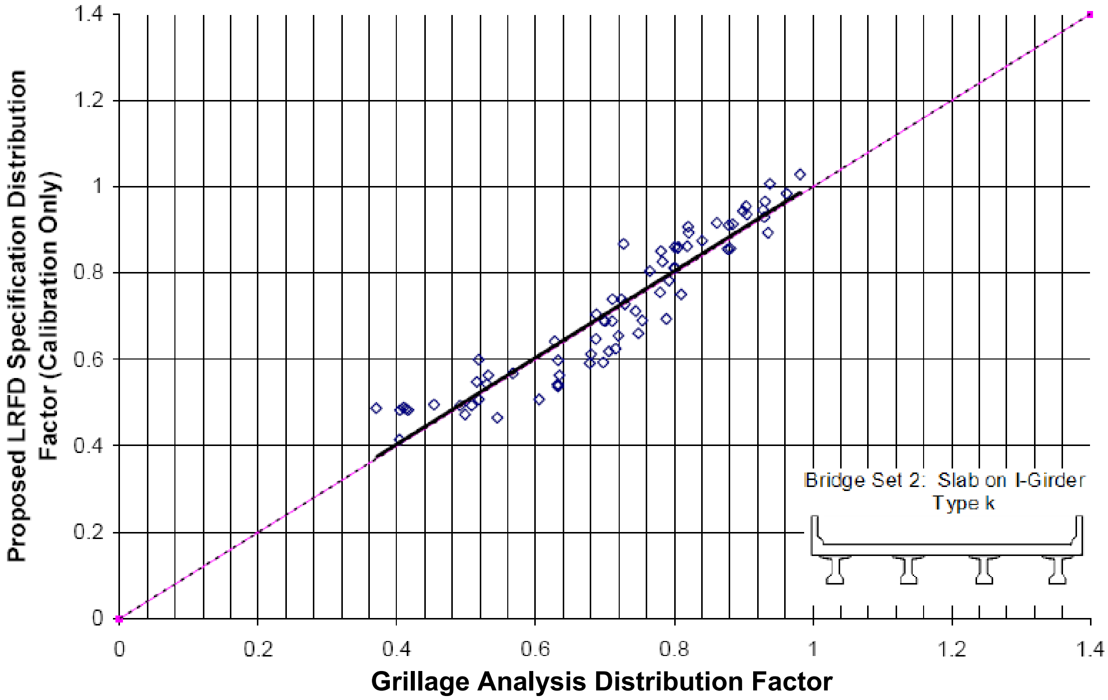


Figure 6.8 Calibrated Lever Rule LLDF vs. Grillage Analysis LLDF for Interior Girders, for Two-Lanes Loaded, at the End of Span (Puckett et al. 2007)

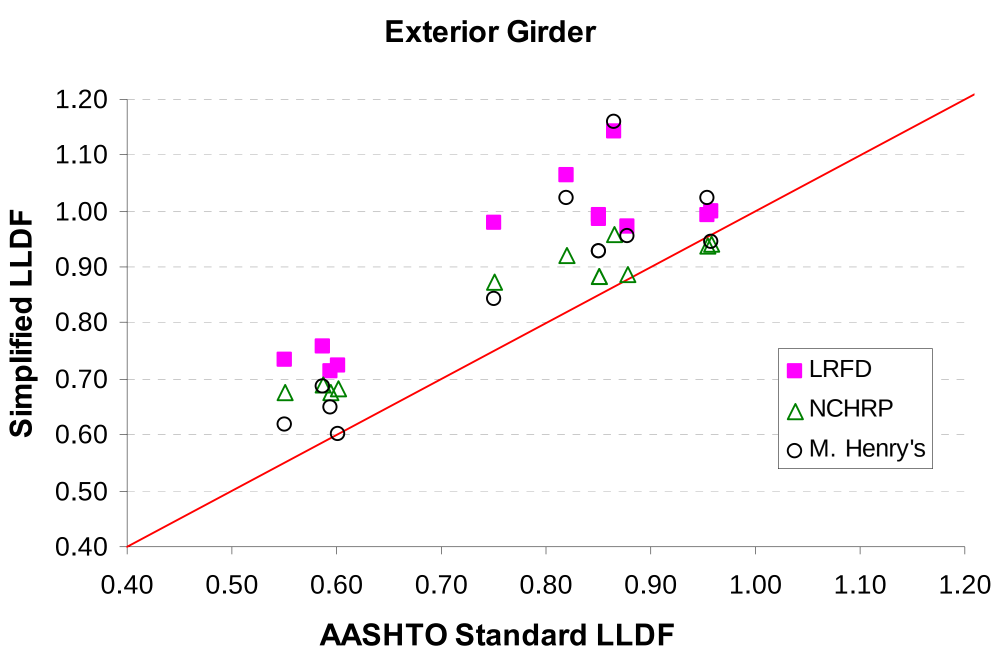


Figure 6.9 Simplified Method LLDFs vs. AASHTO Standard LLDF for Exterior Girders

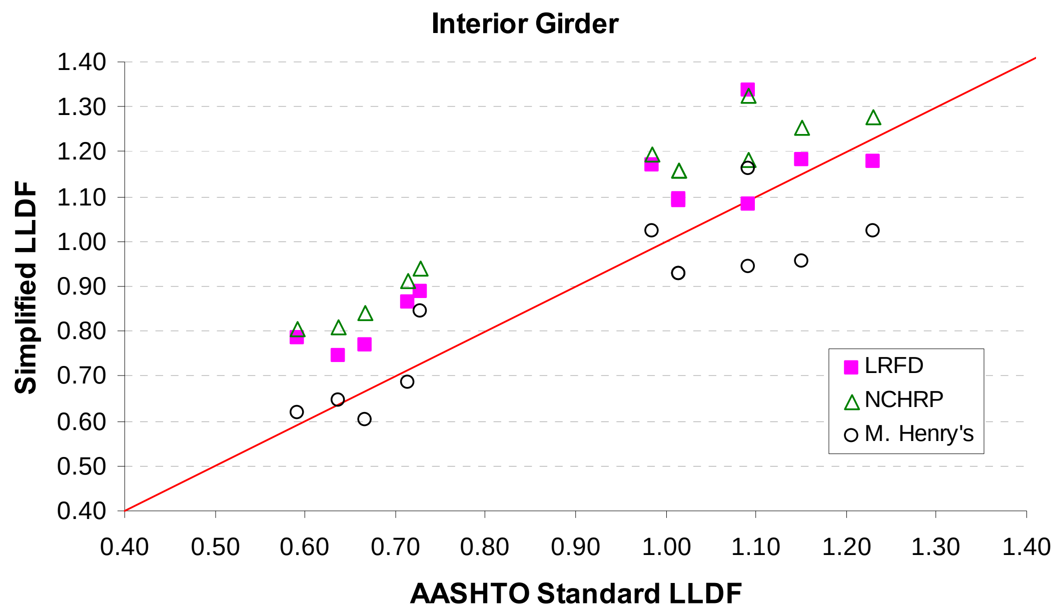


Figure 6.10 Simplified LLDFs vs. AASHTO Standard LLDF for Interior Girders

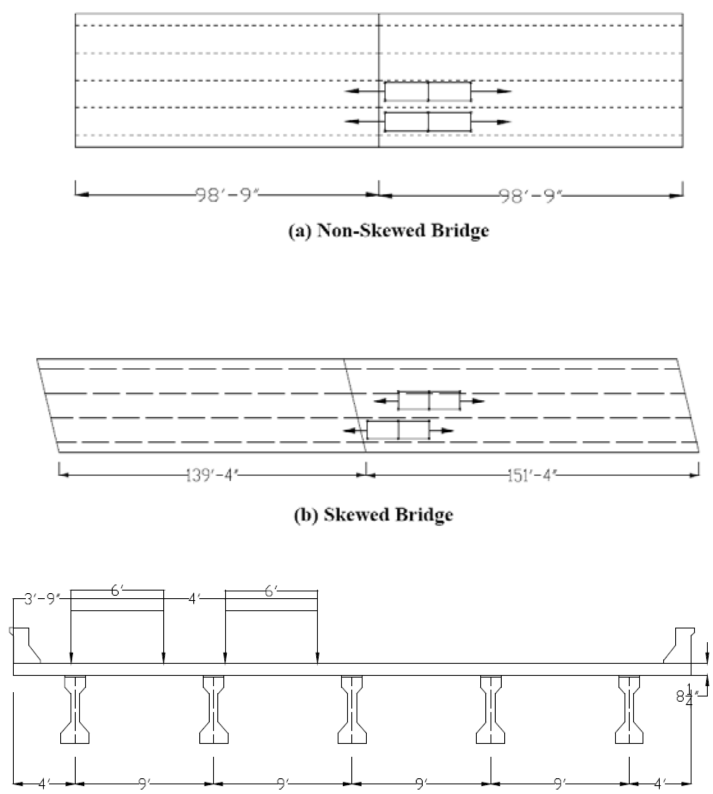


Figure 6.11 Sample Loading Patterns for Live Load Shear for Precast Concrete Beams (Huo et al., 2003)

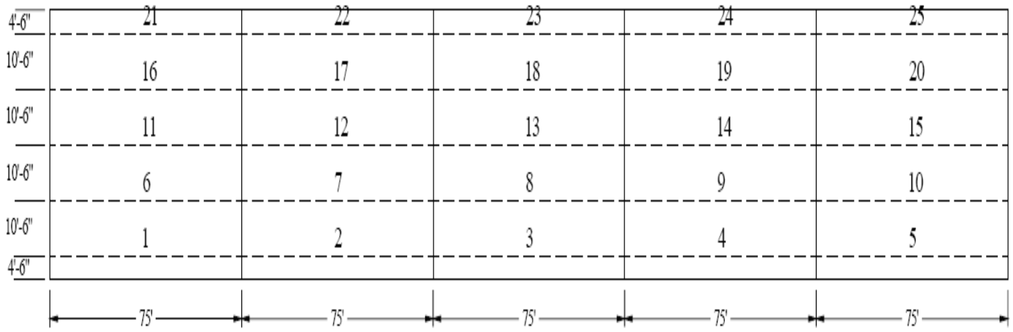


Figure 6.12 Plan View of Bridge No. 24 Without Skew (Puckett et al., 2007)

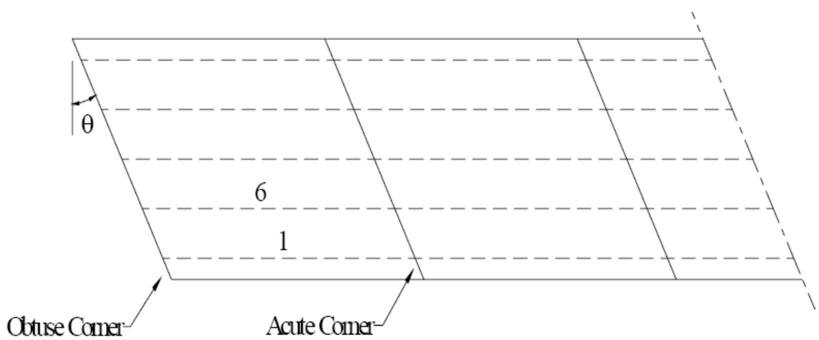


Figure 6.13 Plan View of Bridge No. 24 with Skew (Puckett et al., 2007)

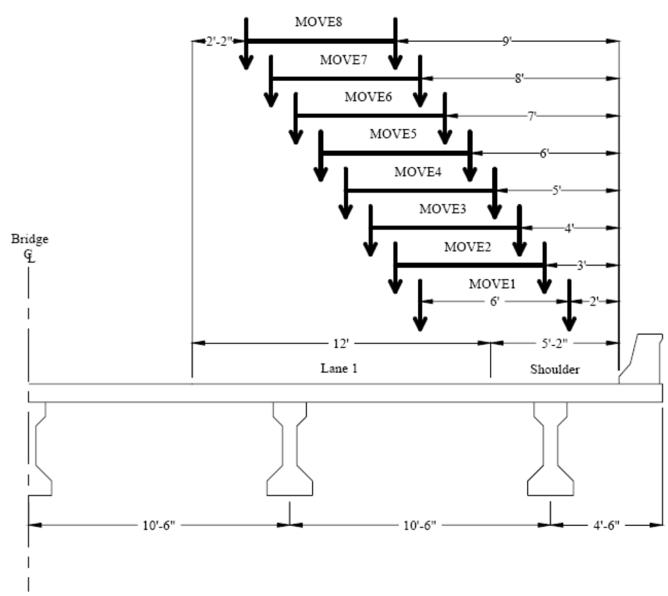


Figure 6.14 Loading Locations (Puckett et al., 2007)

Appendix A

Sample Shear Calculations

A.1 Introduction

This appendix contains sample shear calculations associated with the material presented in the main body of this document. Sample calculations showing the error in the calculation of web-shear capacity in *Virtis*-BRASS are given in Section A.2. Samples of the design and capacity calculations discussed in Section 2.3 for the AASHTO Standard shear provisions are given in Section A.3. The design calculations were performed for four bridges that were selected from the 54 investigated girders (Chapters 4 and 5). The sample calculations presented in this section are from the critical sections as defined by each of the codes in effect at the time of design of the girders.

This section contains samples of the shear design calculations from the 1961, 1965-1969, 1973-1977-1979 Interim, and 1983 AASHTO Standards Specification and contains one sample of the shear capacity calculations from the 2002 AASHTO Standard Specification.

A.2 Calculations Showing Error Related to Web-Shear Capacity in *Virtis*-BRASS

This section contains sample calculations to illustrate the most significant error found in *Virtis*-BRASS on shear rating calculations. The error was related to the calculation of the "f_{pc}" term of the concrete resistance to web-shear "V_{cw}" of Equation (9-29) in the 2002 AASHTO Standard Specifications (Article 9.20.2.3). As discussed in Section 3.3.2, Bridge 83022-1_3 was one of the sample bridges utilized for the comparison of the hand computations to the *Virtis*-BRASS output. The girder properties of the Bridge 83022-1_3 are given in Table A.1.

Calculations were made at the critical section as defined in the 2002 AASHTO Standard Specification, which was *h*/2 away from the face of the support, or 36.1 in.

<u>Shear Forces at the Critical Section</u>

- Unfactored shear force due to live load, $V_{LL} = 70.5\text{ kips}$,

  <u>*Virtis*-BRASS</u>: $V_{LL,Virtis-Brass} = 70.4\text{ kips}$ (with WAD=1000)

- Unfactored shear force due to total dead load, $V_d = 60.5\text{ kips}$

  <u>*Virtis*-BRASS</u>: $V_{DL,Virtis-Brass} = 60.5\text{ kips}$

- Total factored shear force, $V_u = 1.3(V_d + 1.67V_{LL}) = 231.4\text{ kips}$, load factors were obtained per the 2002 Standard Table 3.22.1A.

<u>Web-shear Cracking Strength</u>, V_{cw}

Per the 2002 Standard Eqn. (9-29):

$$V_{cw} = (3.5\sqrt{f_c^{'}} + 0.3 f_{pc})b_w d + V_p$$

Compressive stress in the concrete (after allowance for all prestress losses at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, f_{pc}

Note: In a composite member, f_{pc} is the resultant compressive stress at the centroid of the composite section or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.

Figure A.1 shows the composite section properties of the interior girder of Bridge 83022-1_3 calculated according to the 2002 AASHTO Standard Specification. The section properties given by *Virtis*-BRASS were the same as those shown in Figure A.1.

$$f_{pc} = \frac{P_{se}}{A} - \frac{P_{se} e \Delta y}{I} + \frac{(M_{dg} + M_{ds})\Delta y}{I}$$

where

P_{se} is the effective prestress force, $P_{se} = 515.8\ \text{kips}$

A is the cross-sectional area of the precast beam, $A = 560\ \text{in}^2$

e is the strand eccentricity for the non-composite precast beam, $e = 9.9\ \text{in}$

I is the moment of inertia about the centroid of the non-composite precast beam,

$I = 125{,}390\ \text{in}^4$

$\Delta y = (y_{bc} - y_b)$ if centroid is within the web,

$\Delta y = (h_{web+b_f} - y_b)$ if centroid is within the flange.

y_{bc} is the distance from the centroid of the composite section to the extreme bottom fiber of the precast beam, $y_{bc} = 40.74\ \text{in}$

y_b is the distance from the centroid to the extreme bottom fiber of the non-composite precast beam $y_b = 20.27\ \text{in}$

h_{web+b_f} is the total height of the web and bottom flange thickness,

$h_{web+b_f} = 33.5\ \text{in}$

The centroid is in the flange, so $\Delta y = (h_{web+b_f} - y_b) = (33.5 - 20.27) = 13.23\ \text{in}$

M_{dg} is the unfactored moment due to self weight of the girder, $M_{dg} = 48.8$ ft.kips

M_{ds} is the unfactored moment due to self weight of the slab and the diaphragms, $M_{ds} = 103.4$ ft.kips (*Virtis*-BRASS yielded the same values for the dead load moments)

$$f_{pc} = \frac{(515.8\text{kips})}{(560\text{in}^2)} - \frac{(515.8\text{kips})(9.9\text{in})(13.23\text{in})}{(125{,}390\text{in}^4)}..$$
$$+ \frac{(48.8 + 103.4\text{ ft.kips})(13.23\text{in})}{(125{,}390\text{in}^4)}\left(\frac{12\text{in}}{\text{ft}}\right)$$

$$= 0.921\text{ksi} - 0.539\text{ksi} + 0.193\text{ksi} = 0.575\text{ ksi}$$

<u>*Virtis*-BRASS</u>: $f_{pc,Virtis-Brass} = 0.351\text{ ksi}$, due to the use of the wrong definition for Δy (i.e., Δy taken as $(y_{bc} - y_b)$=20.47in:

$$f_{pc} = \frac{(515.8\text{kips})}{(560\text{in}^2)} - \frac{(515.8\text{kips})(9.9\text{in})(20.47\text{in})}{(125{,}390\text{in}^4)}..$$

$$+ \frac{(48.8 + 103.4\text{ ft.kips})(20.47\text{in})}{(125{,}390\text{in}^4)}\left(\frac{12\text{in}}{\text{ft}}\right)$$

$$= 0.921\text{ksi} - 0.833\text{ksi} + 0.298\text{ksi} = 0.386\text{ ksi}$$

<u>Vertical component of the prestressing force for harped strands</u>, V_p

$$V_p = f_{se} A_{ps-harped} \sin\psi = (140.5\text{ksi})(3.672\text{in}^2)(\sin(5.9°)) = 13.24\text{ kips}$$

<u>*Virtis*-BRASS</u>: $V_p = 13.58\text{ kips}$, due to the differences in harping slope as mentioned in Section 3.3.1

$$V_{cw} = \left(3.5\frac{\sqrt{5000\text{ psi}}}{1000} + 0.3(0.575\text{ksi})\right)(7\text{in})(46.88\text{in}) + (13.24\text{kips}) = 151.1\text{ kips}$$

<u>*Virtis*-BRASS</u>: $V_{cw,Virtis-Brass} = 129.3\text{ kips}$

$$V_{cw} = \left(3.5\frac{\sqrt{5000\,psi}}{1000} + 0.3(0.351\text{ksi})\right)(7\text{in})(46.88\text{in}) + (13.58\text{kips}) = 129.3\text{ kips}$$

Difference between *Virtis*-BRASS and hand calculations for V_{cw}

$$Diff. = \frac{abs(VIRTIS - HAND\ CALC)}{VIRTIS} * 100 = \frac{abs(V_{cw,Virtis\_Brass} - V_{cw})}{V_{cw,Virtis\_Brass}} * 100$$

$$= \frac{abs(129.3 - 151.1)}{129.3} * 100 = 17\%$$

<u>Stirrup Contribution to Shear Capacity</u>, V_s

Per the 2002 Standard Eqn. (9-30):

$$V_s = \frac{A_v f_{sy} d}{s} \quad \text{where } V_s \le 8\sqrt{f_c^{'}} b_w d$$

Note: Provided stirrup spacing at the design section was 18 in. (taken from the bridge plan),

$$V_s = \frac{0.4\text{in}^2 (60\text{ksi})(46.88\text{in})}{(18\text{in})} = 62.5\text{ kips}$$

$$V_s = 62.5\text{ kips} < 8\frac{\sqrt{5{,}000\text{psi}}}{1{,}000}(7\text{in})(46.88\text{in}) = 185.6\text{ kips, O.K.}$$

Thus, $V_s = 62.5$ kips

<u>*Virtis*-BRASS</u>: $V_{s,Virtis-Brass} = 62.5$ kips

<u>Shear Capacity</u>, V_n

$$V_n = V_c + V_s = 151.1 + 62.5 = 213.6\text{ kips}$$

<u>*Virtis*-BRASS</u>: $V_{n,Virtis-Brass} = V_{c,Virtis-Brass} + V_{s,Virtis-Brass} = 129.3 + 62.5 = 191.8$ kips

<u>Shear Inventory Rating Factor</u>, $Inv.RF = \frac{(\phi V_{n,STD2002}) - (1.3)(V_d)}{(1.3)(1.67)(V_{LL})}$

$$Inv.RF = \frac{(0.9)(213.6\text{kips}) - (1.3)(60.5\text{kips})}{(1.3)(1.67)(70.5\text{kips})} = 0.74$$

<u>*Virtis*-BRASS</u>: Inventory rating factor is given as 0.61 in the output file.
$Inv.RF_{Virtis-Brass} = \frac{(0.9)(191.8\text{kips}) - (1.3)(60.5\text{kips})}{(1.3)(1.67)(70.4\text{kips})} = 0.61$, thus the source of the error (i.e., in "f_{pc}" term) is verified.

Difference between *Virtis*-BRASS and hand calculations for rating factor:

$$Diff. = \frac{abs(Inv.RF_{Virtis-Brass} - Inv.RF)}{Inv.RF_{Virtis-Brass}} * 100 = \frac{abs(0.61 - 0.74)}{0.61} * 100 = 21\%$$

A.3 Shear Design Calculations according to the AASHTO Standard Specifications

This section contains samples of shear design calculations for the AASHTO Standard Specifications used to redesign the investigated bridges (Section 4.3). The presented design specifications are given for four different groups as 1961, 1965-1969, 1973-1977-1979 Interim, and 1983 AASHTO Standards. Four bridges were selected from the investigated bridges and stirrup design calculations are shown corresponding to the specification that was in effect at the time of their design. The selected bridges were known to be undercapacity (except the bridge designed according to the 1961 Standard) and had low shear capacity-to-demand ratios at the corresponding critical section of the specification in effect at the time of the design. The material and sectional properties of the bridges (needed for design calculations) are given in Table A.1

A.3.1 AASHTO 1961 Standard Stirrup Design at the Critical Section

The 1961 AASHTO Standard Specification stirrup design calculations are illustrated by Bridge 27978-2. According to the 1961 AASHTO Standard Specification, the critical section was located at a quarter of the length of the girder from the support.

<u>Shear force at the critical section</u>

The factored ultimate shear included the dead load from: girder, slab, diaphragms, barrier, and wearing course; and live load from AASHTO HS20-44. The shear force due to dead and live load was computed using *Virtis*-BRASS. A value of WAD=1000 (as discussed in Section 3.3.1) was used to provide sufficient accuracy in the live load shear acting at the section.

- Unfactored shear force due to total dead load, $V_{DL} = 27.3$ kips

- Unfactored shear force due to live load, $V_{LL} = 54.3$ kips

- Factored ultimate shear force per the 1961 Standard Article 13.6, $V_u = 1.5V_{DL} + 2.5V_{LL}$

  $V_u = 1.5(27.3) + 2.5(54.3) = 176.7$ kips

However, V_u is defined as *"the shear due to ultimate load and effect of prestressing"* in Article 13.2, thus the vertical component of the prestressing force for harped strands, V_p, is directly subtracted from V_u :

Note: V_p is calculated by using a value of 35 ksi for estimating the total prestress losses according to 1961 Standard Article 13.8 (B):

$V_p = 20.6$ kips

$V_u = 176.7 - 20.6 = 156.1$ kips

Concrete contribution to shear capacity, V_c

Per the 1961 Standard Article 7.7:

$$V_c = \min\left(0.03 f'_c b_w jd,\ 90 b_w jd\right)$$

Ratio of the distance between the centroid of compression and centroid of tension to the depth, j

For rectangular or flanged sections in which the neutral axis lies within the flange, the ultimate flexural strength shall be assumed per the 1961 Standard Article 13.10 (A) as:

$$M_u = A_{ps} f_{pu} d\left(1 - 0.6\frac{p f_{su}}{f'_c}\right)$$

From this equation, it can be inferred that:

$$j = 1 - 0.6\left(\frac{p f_{su}}{f'_c}\right) \text{where } f'_c = f'_{c,slab} \text{ in this case}$$

- Average stress in the prestressing strand at ultimate load, f_{su}

$$f_{su} = f_{pu}\left(1 - 0.5\frac{\rho f_{pu}}{f'_c}\right) \text{where } f'_c = f'_{c,slab} \text{ in this case}$$

- Prestressing steel reinforcement ratio, ρ

$$\rho = \frac{A_{ps}}{b_{eff}\, d}$$

- Effective flange width, b_{eff} per the 1961 Standard Article 7.4 (A):

$$b_{eff} = \min\begin{pmatrix} L/4 \\ S_g \\ 12t_s + b_w \end{pmatrix} = \min\begin{pmatrix} \frac{(52.3\text{ft})(12\text{in/ft})}{4} \\ (11.2\text{ft})(12\text{in/ft}) \\ 12(8\text{in}) + 6\text{in} \end{pmatrix} = \min\begin{pmatrix} 157\text{ in} \\ 134\text{ in} \\ 102\text{ in} \end{pmatrix} = 102\text{ in}$$

$$\rho = \frac{4.284\text{in}^2}{(102\text{in})(40.96\text{in})} = 0.001$$

Per the 1961 Standard Article 13.10 (C):

$$f_{su} = (270\text{ksi})\left(1 - 0.5\frac{(0.001)(270\text{ksi})}{(4\text{ksi})}\right) = 260.7\text{ ksi}$$

$$j = 1 - 0.6\left(\frac{0.001(260.7\text{ksi})}{(4\text{ksi})}\right) = 0.96$$

$$V_c = \min\left(0.03(5\text{ksi})(6\text{in})(0.96)(40.96\text{in}),\ \frac{(90\text{psi})}{(1{,}000)}(6\text{in})(0.96)(40.96\text{in})\right)$$

$$= \min(35.4\text{kips}, 21.2\text{kips}) = 21.2\text{ kips}$$

<u>Required stirrup spacing for double leg #4 reinforcing bars</u>

Per the 1961 Standard Article 13.13:

$$s_{req'd} = \frac{2A_v f_{sy} jd}{\left(\frac{V_u}{\phi} - V_c\right)} = \frac{2(2)(0.2\text{in}^2)(40\text{ksi})(0.96)(40.96\text{in})}{\left(\frac{156.1\text{kips}}{1.0} - 21.2\text{kips}\right)} = 9.3\text{ in}$$

Note that shear strength reduction factor, ϕ, is taken as 1.0, there is no information on the value of ϕ in the 1961 Standard.

<u>Check maximum stirrup spacing</u>

Per the 1961 Standard Article 13.13:

$$s_{\max} = \min\left(\frac{A_v}{0.0025b_w}, 0.75h_c\right) = \left(\frac{(0.4\text{in}^2)}{0.0025(6\text{in})}, 0.75(49.5\text{in})\right) = \min(26.7\text{in}, 37.1\text{ in}) = 26.7\text{ in}$$

$s_{req'd} < s_{\max}$ O.K.

<u>Check horizontal shear design at the quarter point</u>

The horizontal shear design at the quarter point of the span is carried out according to Article 13.14 of the 1961 Standard. The ultimate horizontal shear demand (in stress) is given by Article 13.14 (B) as

$$v_u = \frac{V_u Q_c}{I_c b_v}$$

where V_u is the ultimate factored vertical shear force at the section, Q_c is the first moment of area above the fiber being considered, and, I_c is the moment of inertia of the entire composite cross-sectional area, and b_v is the section width at the fiber being considered.

For this case, $Q_c = 8{,}101\,\text{in}^3$, $I_c = 314{,}689\,\text{in}^4$, $b_v = 16\,\text{in}$.

$$v_u = \frac{(156.1\text{kips})(8{,}101\text{in}^3)}{(314.689\text{in}^4)(16\text{in})} = 251.1\,\text{psi}$$

<u>Check stirrup spacing requirement for horizontal shear,</u>$s_{vh,max}$

$$s_{vh,\max} = \min\begin{pmatrix} 4(t_{flange}) \\ 24in \\ A_v/(A_{v,\#3}/ft) \end{pmatrix}$$

where $A_v/(A_{v,\#3}/ft)$ represents Article 13.14 (D) of the 1961 Standard. According to the article, the minimum area of transverse reinforcement shall not be less than the area of two #3 bars spaced at 12 in. Thus, the maximum spacing for double leg #4 stirrups was:

$$\frac{\text{area of two \#4 bars}}{\text{s}_{\max}} = \frac{\text{area of two \#3 bars}}{12\text{in}}$$

$$s_{\max} = \frac{(2)(0.2\text{in}^2)(12\text{in})}{(2)(0.11\text{in}^2)} = 21.8\,\text{in}$$

$$s_{vh,\max} = \min\begin{pmatrix} 4(6\text{in}) \\ 24\text{in} \\ 21.8\text{in} \end{pmatrix} = 21.8\,\text{in}$$

Then $s_{req'd} = 9.3\,\text{in} < s_{vh,\max} = 21.8\,\text{in}$ O.K.

According to Article 13.14 (C) of the 1961 Standard, when the provided transverse reinforcement is in excess of the requirements of Article 13.14 (D), as illustrated above, and the contact surface of the precast element is artificially roughened, then the horizontal shear capacity at the interface can be taken as 225 psi.

$$s_{vh,req'd} = \frac{A_v f_{sy}}{(v_u - \phi v_{nh})(b_v)} = \frac{(2)(0.2\text{in}^2)(40{,}000\text{psi})}{\left(251.1\text{psi} - (1.0)(225\text{psi})\right)\left(16\text{in}\right)} = 38.3\,\text{in}$$

$$s_{req'd} = 9.3\,\text{in} < s_{vh,req'd} = 38.3\,\text{in}$$

Therefore, at the critical section, the design spacing for #4 stirrups was 9.3 in.

A.3.2 AASHTO 1965-1969 Standard Stirrup Design at the Critical Section

The 1965-1969 AASHTO Standards stirrup design calculations are illustrated using Bridge 24831-3. According to the 1965-1969 Standards, the critical section was located at the quarter point of the span.

Shear force at the critical section

For the computation of the shear forces and moments, the same procedure presented in Section A.2.1 was carried out.

- Unfactored shear force due to total dead load, $V_{DL} = 23.4$ kips

- Unfactored shear force due to live load, $V_{LL} = 55.1$ kips

- Factored ultimate shear force per the 1961 Standard Article 6.6, $V_u = 1.5V_{DL} + 2.5V_{LL}$

$$V_u = 1.5(23.4) + 2.5(55.1) = 172.8 \text{ kips}$$

However, V_u is defined as *"the shear due to ultimate load and effect of prestressing"* in Article 6.2 of the 1965-1969 Standards, thus the vertical component of the prestressing force for harped strands, V_p, is directly subtracted from V_u :

Note: V_p is calculated by using a value of 35 ksi for estimating the total prestress losses

according to the 1965-69 Standards Article 6.8 (B):

$$V_p = 19.8 \text{ kips}$$

$$V_u = 172.8 - 19.8 = 153.0 \text{ kips}$$

Concrete contribution to shear capacity, V_c

Per the 1965-69 Standards Article 6.13:

$$V_c = \min\left(0.06 f'_c b_w jd,\ 180 b_w jd\right)$$

Ratio of the distance between the centroid of compression and centroid of tension to the depth, j

For rectangular or flanged sections in which the neutral axis lies within the flange, the ultimate flexural strength shall be assumed per the 1965-69 Standards Article 6.10 (A) as

$$M_u = A_{ps} f_{pu} d\left(1 - 0.6\frac{p f_{su}}{f'_c}\right)$$

From this equation, it can be inferred that:

$$j = 1 - 0.6\left(\frac{p\, f_{su}}{f'_c}\right) \text{ where } f'_c = f'_{c,slab} \text{ in this case}$$

- Average stress in the prestressing strand at ultimate load, f_{su}

$$f_{su} = f_{pu}\left(1 - 0.5\frac{\rho f_{pu}}{f_c^{'}}\right) \text{ where } f'_c = f'_{c,slab} \text{ in this case}$$

- Prestressing steel reinforcement ratio, ρ

$$\rho = \frac{A_{ps}}{b_{eff}\, d}$$

- Effective flange width, b_{eff}

Per the 1965 and 1969 Standards Article 7.4 (A):

$$b_{eff} = \min\begin{pmatrix} L/4 \\ S_g \\ 12t_s + b_w \end{pmatrix} = \min\begin{pmatrix} \frac{(39.5\text{ft})(12\text{in/ft})}{4} \\ (12.7\text{ft})(12\text{in/ft}) \\ 12(9.75\text{in}) + 6\text{in} \end{pmatrix} = \min\begin{pmatrix} 119\text{ in} \\ 152\text{ in} \\ 123\text{ in} \end{pmatrix} = 119\text{ in}$$

$$\rho = \frac{2.754\text{in}^2}{(119\text{in})(37.33\text{in})} = 0.0006$$

Per the 1965-69 Standards Article 6.10 (C):

$$f_{su} = (270\text{ksi})\left(1 - 0.5\frac{(0.0006)(270\text{ksi})}{(4\text{ksi})}\right) = 264.3\text{ ksi}$$

$$j = 1 - 0.6\left(\frac{0.0006(264.3\text{ksi})}{(4\text{ksi})}\right) = 0.98$$

$$V_c = \min\left(0.06(5\text{ksi})(6\text{in})(0.98)(37.33\text{in}),\ \frac{(180\text{psi})}{(1{,}000)}(6\text{in})(0.98)(37.33\text{in})\right)$$

$$= \min(54.0\text{kips}, 39.3\text{kips}) = 39.3\text{ kips}$$

<u>Required stirrup spacing for double leg #4 reinforcing bars</u>

Per the 1965-69 Standards Article 6.13:

$$s_{req'd} = \frac{2A_v f_{sy} jd}{\left(\frac{V_u}{\phi} - V_c\right)} = \frac{2(2)(0.2\text{in}^2)(40\text{ksi})(0.98)(37.33\text{in})}{\left(\frac{153.0\text{kips}}{1.0} - 39.3\text{kips}\right)} = 10.2\text{ in}$$

Note that shear strength reduction factor, ϕ, is taken as 1.0, there is no information on the value of ϕ in the 1965-69 Standards.

Check maximum stirrup spacing

Per the 1965-69 Standards Article 6.13:

$$s_{\max} = \min\left(\frac{A_v}{0.0025b_w}, 0.75h_c\right) = \left(\frac{(0.4\text{in}^2)}{0.0025(6\text{in})}, 0.75(46.8\text{in})\right) = \min(26.7\text{in}, 35.1\text{ in}) = 26.7\text{ in}$$

$s_{req'd} < s_{\max}$ O.K.

Check horizontal shear design at the quarter point

The horizontal shear design at the section is carried out according to Article 6.14 of the 1965-69 Standards. Similar to the 1961 Standard (Section A.2.1), the ultimate horizontal shear demand (in stress) is given by Article 6.14 (B) as

$$v_u = \frac{V_u Q_c}{I_c b_v}$$

For this case, $Q_c = 7{,}086\text{ in}^3$, $I_c = 243{,}647\text{ in}^4$, $b_v = 12\text{ in}$.

$$v_u = \frac{(153.0\text{kips})(7{,}086\text{in}^3)}{(243{,}647\text{in}^4)(12\text{in})} = 370.7\text{ psi}$$

Check stirrup spacing requirement for horizontal shear, $s_{vh,\max}$

$$s_{vh,\max} = \min\begin{pmatrix} 4(t_{flange}) \\ 24in \\ A_v/(A_{v,\#3}/ft) \end{pmatrix}$$

where $A_v/(A_{v,\#3}/ft)$ represents the Article 6.14 (D) of the 1965-69 Standards.

Similar to that shown in Section A.2.1, $A_v/(A_{v,\#3}/ft)$ yields 21.8 in. stirrup spacing.

$$s_{vh,\max} = \min\begin{pmatrix} 4(6\text{in}) \\ 24\text{in} \\ 21.8\text{in} \end{pmatrix} = 21.8\text{ in}$$

Then $s_{req'd} = 9.3\text{ in} < s_{vh,\max} = 21.8\text{ in O.K.}$

According to Article 6.14 (C) of the 1965-69 Standards, when the provided transverse reinforcement is in excess of the requirements of Article 6.14 (D), as illustrated above, and the contact surface of the precast element is artificially roughened, then the horizontal shear capacity at the interface can be taken as 225 psi.

$$s_{vh,req'd} = \frac{A_v f_{sy}}{(v_u - \phi v_{nh})(b_v)} = \frac{(2)(0.2\text{in}^2)(40{,}000\text{psi})}{(370.7\text{psi} - (1.0)(225\text{psi}))(12\text{in})} = 9.1\text{ in}$$

$$s_{req'd} = 10.2\text{ in} > s_{vh,req'd} = 9.1\text{ in}$$

Therefore, at the critical section, for #4 stirrups, horizontal shear design for stirrup spacing, 9.1 in., controlled over the vertical shear design spacing, 10.2 in.

A.3.3 AASHTO 1973-1977-1979 Interim Standard Stirrup Design at the Critical Section

The 1973-1977-1979 Interim AASHTO Standards (will be referred as the 1977 Standard in the text) stirrup design calculations are illustrated using Bridge 48010. According to the 1977 Standard, the critical section was located at a quarter of the length of the girder from the support.

<u>Shear force at the critical section</u>

For the computation of the shear forces and moments, the same procedure presented in Section A.2.1 was carried out.

- Unfactored shear force due to total dead load, $V_{DL} = 27.4\text{ kips}$

- Unfactored shear force due to live load, $V_{LL} = 56.8\text{ kips}$

- Factored ultimate shear force per the 1977 Standard Article 1.2.22,
  $V_u = 1.3(1.0(V_{DL}) + 1.67(V_{LL})) = 1.3(1.0(56.8) + 1.67(27.4)) = 158.7\text{ kips}$

Contrary to the 1961 and 1965-69 Standards, V_u is defined as *"the total applied design shear force at section"* in Article 6.2 of the 1977 Standard, thus the vertical component of the prestressing force for harped strands, V_p, was not subtracted from V_u (Except in the case of the 1973 Standard, where the definition of V_u is exactly the same definition given in the 1961 and 1965-69 Standards).

Concrete contribution to shear capacity, V_c

Per the 1977 Standard Article 6.13:

$$V_c = \min\left(0.06 f'_c b_w jd,\ 180 b_w jd\right)$$

Ratio of the distance between the centroid of compression and centroid of tension to the depth, j

For rectangular or flanged sections in which the neutral axis lies within the flange, the ultimate flexural strength shall be assumed per the 1977 Standard Article 6.9 (A) as

$$M_u = A_{ps} f_{pu} d\left(1 - 0.6\frac{\rho f_{su}}{f'_c}\right)$$

From this equation, it can be inferred that:

$$j = 1 - 0.6\left(\frac{\rho f_{su}}{f'_c}\right) \text{ where } f'_c = f'_{c,slab} \text{ in this case}$$

- Average stress in the prestressing strand at ultimate load, f_{su}

Per the 1977 Standard Article 6.9 (C):

$$f_{su} = f_{pu}\left(1 - 0.5\frac{\rho f_{pu}}{f'_c}\right) \text{ where } f'_c = f'_{c,slab} \text{ in this case}$$

- Prestressing steel reinforcement ratio, ρ

$$\rho = \frac{A_{ps}}{b_{eff} d}$$

- Effective flange width, b_{eff}

Per the 1973 and 1977 Standards Article 6.23 (A):

$$b_{eff} = \min\begin{pmatrix} L/4 \\ S_g \\ 12t_s + b_w \end{pmatrix} = \min\begin{pmatrix} \frac{(43.2\text{ft})(12\text{in/ft})}{4} \\ (12.5\text{ft})(12\text{in/ft}) \\ 12(9.25\text{in}) + 7\text{in} \end{pmatrix} = \min\begin{pmatrix} 130\text{ in} \\ 150\text{ in} \\ 118\text{ in} \end{pmatrix} = 118\text{ in}$$

$$\rho = \frac{2.448\text{in}^2}{(118\text{in})(49.1\text{in})} = 0.0004$$

$$f_{su} = (270\text{ksi})\left(1 - 0.5\frac{(0.0004)(270\text{ksi})}{(4\text{ksi})}\right) = 266.1\,\text{ksi}$$

$$j = 1 - 0.6\left(\frac{0.0004(266.1\text{ksi})}{(4\text{ksi})}\right) = 0.98$$

$$V_c = \min\left(0.06(5\text{ksi})(7\text{in})(0.98)(49.1\text{in}),\ \frac{(180\text{psi})}{(1{,}000)}(7\text{in})(0.98)(49.1\text{in})\right)$$

$$= \min(81.1\text{kips}, 60.8\text{kips}) = 60.8\,\text{kips}$$

<u>Required stirrup spacing for double leg #4 reinforcing bars</u>

Per the 1977 Standard Article 6.13:

$$s_{req'd} = \frac{2A_v f_{sy} jd}{\left(\frac{V_u}{\phi} - V_c\right)} = \frac{2(2)(0.2\text{in}^2)(60\text{ksi})(0.98)(49.1\text{in})}{\left(\frac{158.7\text{kips}}{0.9} - 60.8\text{kips}\right)} = 20.1\,\text{in}$$

The shear strength reduction factor, ϕ, is 0.9, in the Article 6.5 of the 1977 Standard.

<u>Check maximum stirrup spacing</u>

Per the 1977 Standard Article 6.13:

$$s_{\max} = \min\left(\frac{A_v f_{sy}}{100b_w}, 0.75h_c\right) = \left(\frac{(0.4\text{in}^2)(60{,}000\text{psi})}{(100\text{psi})(6\text{in})}, 0.75(55.8\text{in})\right) = \min(34.3\text{in}, 41.8\,\text{in}) = 34.3\,\text{in}$$

$s_{req'd} < s_{\max}$ O.K.

<u>Check horizontal shear design at the quarter point</u>

The horizontal shear design at the section is carried out according to Article 6.14 of the 1977 Standard. Similar to the 1961 Standard (Section A.2.1), the ultimate horizontal shear demand (in stress) is given by Article 6.14 (C) as

$$v_u = \frac{V_u Q_c}{I_c b_v}$$

For this case, $Q_c = 11{,}026\,\text{in}^3, I_c = 471{,}884\,\text{in}^4, b_v = 16\,\text{in}$.

$$v_u = \frac{(158.7\text{kips})(11{,}026\text{in}^3)}{(471{,}884\text{in}^4)(16\text{in})} = 231.7\text{ psi}$$

Check stirrup spacing requirement for horizontal shear, $s_{vh,max}$

$$s_{vh,\max} = \min\begin{pmatrix} 4(t_{flange}) \\ 24in \\ A_v/(A_{v,\#3}/ft) \end{pmatrix}$$

where $A_v/(A_{v,\#3}/ft)$ represents the Article 6.14 (D) of the 1977 Standard.

Similar to that shown in Section A.2.1, $A_v/(A_{v,\#3}/ft)$ yields 21.8 in. stirrup spacing.

$$s_{vh,\max} = \min\begin{pmatrix} 4(7\text{in}) \\ 24\text{in} \\ 21.8\text{in} \end{pmatrix} = 21.8\text{ in}$$

Then $s_{req'd} = 20.1\text{ in} < s_{vh,\max} = 21.8\text{ in}$ O.K.

When the minimum requirements of Article 6.14 (D) are met and the contact surface of the precast element is clean and intentionally roughened, then the horizontal shear capacity at the interface can be taken as 300 psi.

Check $v_u - \phi(v_{nh}) = 231.7\text{psi} - 0.9(300\text{psi}) = -38.3\text{ psi}$

Because $\phi(v_{nh}) > v_u$, there is no need for additional stirrups for horizontal shear, provided that the minimum requirements of Article 6.14 (D) are met.

$s_{req'd} = 20.1\text{ in} > s_{vh,\max} = 21.8\text{ in}$

Therefore, at the critical section, the design spacing for #4 stirrups was 20.1 in.

A.3.4 AASHTO 1983 Standard Stirrup Design at the Critical Section

The 1983 AASHTO Standard stirrup design calculations are illustrated using Bridge 9011. According to the 1983 Standard, the critical section was *h*/2 away from the face of the support.

Shear forces and moments at the critical section

The factored ultimate shear included the dead load from: girder, slab, diaphragms, barrier, and wearing course; and live load from AASHTO HS25-44. The shear force was computed using *Virtis*-BRASS. A value of WAD=1000 (as discussed in Section 3.3.1) was used to achieve the required accuracy in estimating the live load shear and moment acting at the section.

- Unfactored shear force due to self weight of the girder, $V_{dg} = 24.1$ kips

- Unfactored shear force due to self weight of the slab, $V_{ds} = 39.7$ kips

- Unfactored shear force due to self weight of the diaphragms, $V_{dd} = 3.2$ kips

- Unfactored shear force due to self weight of barriers and wearing course,

$V_{dw} = 6.54$ kips

- Unfactored shear force due to total dead load, $V_d = V_{dg} + \mathrm{V}_{\mathrm{ds}} + V_{dd} + \mathrm{V}_{\mathrm{dw}} = 73.5$ kips

- Unfactored shear force due to live load, $V_{LL} = 84.7$ kips

- Total factored shear force, $V_u = 1.3(V_d + 1.67V_{LL}) = 279.1$ kips, load factors were

obtained per the 1983 Standard Table 3.22.1A.

- Unfactored moment due to self weight of the girder, $M_{dg} = 73.1$ ft.kips

- Unfactored moment due to self weight of the slab, $M_{ds} = 120.4$ ft.kips

- Unfactored moment due to self weight of the diaphragms, $M_{dd} = 18.2$ ft.kips

- Unfactored moment due to self weight of barriers and wearing course,

$M_{dw} = 19.9$ ft.kips

- Unfactored moment due to total dead load,

$M_d = M_{dg} + M_{\mathrm{ds}} + M_{dd} + M_{\mathrm{dw}} = 231.6$ ft.kips

- Unfactored moment force due to live load, $M_{LL} = 246.4$ ft.kips

- Total factored moment, $M_u = 1.3(M_d + 1.67M_{LL}) = 836.0$ ft.kips

- Maximum factored moment at section due to externally applied loads,

$M_{\max} = M_u - M_d = 836.0 - 231.6 = 604.4$ ft.kips

- Factored shear force at section due to externally applied loads occurring simultaneously with $M_{\max} : V_i = 168.5\,\text{kips}$

Composite and noncomposite section properties

The calculations for the transformed composite section properties are provided below and the results are summarized in Table A.2.

Effective flange width, b_{eff}

Per the 1983 Standard Article 9.8.1:

$$b_{eff} = \min\begin{pmatrix} L/4 \\ S_g \\ 12t_s + b_w \end{pmatrix},\; b_{eff} = \min\begin{pmatrix} \dfrac{(77.6\text{ft})(12\text{in/ft})}{4} \\ (9.8\text{ft})(12\text{in/ft}) \\ 12(8.5\text{in}) + 6\text{in} \end{pmatrix} = \min\begin{pmatrix} 233\,\text{in} \\ 132\,\text{in} \\ 118\,\text{in} \end{pmatrix} = 118\,\text{in}$$

Modular ratio between slab and girder, n

$$n = \frac{E_{girder}}{E_{slab}}$$

Per the 1983 Standard Eqn. (9-8): $E = 33w_c^{1.5}\sqrt{f_c'}$

$$\text{E}_{\text{girder}} = 33(145\text{lb/ft}^3)^{1.5}\left(\frac{\sqrt{6{,}861\text{psi}}}{1{,}000}\right) = 4{,}773\,\text{ksi}$$

$$\text{E}_{\text{slab}} = 33(145\text{lb/ft}^3)^{1.5}\left(\frac{\sqrt{4{,}000\text{psi}}}{1{,}000}\right) = 3{,}644\,\text{ksi}$$

$$n = \frac{4{,}773\text{ksi}}{3{,}644\text{ksi}} = 1.31$$

- Transformed slab width = $\dfrac{b_{eff}}{n} = \dfrac{118in}{1.31} = 90.1\,\text{in}$

- Transformed slab area = (Transformed slab width) (t_s) = (90.1in) (8.5in) = 765.8 in^2

- Transformed haunch width = $\dfrac{t_{top-flange}}{n} = \dfrac{30\text{in}}{1.31} = 22.9\,\text{in}$

- Transformed haunch area = (Transformed haunch width) (t_{haunch})

$= (22.9\text{in})\ (1.5\text{in}) = 34.4\text{in}^2$

Using the information summarized in Table A.2,

$$y_{bc} = \frac{\sum A(y_b)}{A} = \frac{54{,}381\text{in}^3}{1{,}424\text{in}^2} = 38.18\text{ in}$$

$$I_c = \sum A(y_{bc} - y_b)^2 + \sum I = 279{,}445\text{in}^4 + 171{,}668\text{in}^4 = 451{,}113\text{ in}^4$$

<u>Total prestress losses,</u> Δf_s

Per the 1983 Standard Eqn. (9-3):

$$\Delta f_s = SH + ES + CR_c + CR_s$$

<u>Loss due to concrete shrinkage</u>, *SH*

Per the 1983 Standard Eqn. (9-4):

$SH = 17{,}000 - 150RH$ (in psi), for pretensioned members

$$SH = \frac{[17{,}000 - 150(72)]\text{psi}}{1{,}000} = 6.2\text{ ksi}$$

<u>Loss due to elastic shortening</u>, *ES*

Per the 1983 Standard Eqn. (9-6):

$ES = \dfrac{E_{ps}}{E_{ci}} f_{cir}$, for pretensioned members

<u>Modulus of elasticity of concrete at transfer,</u> E_{ci}

Per the 1983 Standard Eqn. (9-8):

$$E_{ci} = 33w_c^{1.5}\sqrt{f_{ci}^{'}} = 33(145\text{lb/ft}^3)^{1.5}\left(\frac{\sqrt{6{,}140\text{psi}}}{1{,}000}\right) = 4515\text{ ksi}$$

<u>Concrete stress at the center of gravity of the strands due to prestressing force and dead load of girder immediately after transfer</u>, f_{cir}

Per the 1983 Standard (Article 9.16.2.1.2), f_{cir} shall be computed at the section or sections of maximum moment (i.e., midspan for simply-supported spans).

- Unfactored moment due to self weight of the girder at midspan, $M_{dg,0.5L} = 505.4$ ft.kips

- Unfactored moment due to self weight of the slab and diaphragms at midspan,

$M_{ds,0.5L} = 919.1\,\text{ft.kips}$

- Unfactored moment due to self weight of barriers and wearing course at midspan,

$M_{dw,0.5L} = 137.0\,\text{ft.kips}$

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}\, e_{mid}^{\;2}}{I} - \frac{M_{dg,0.5L}\, e_{mid}}{I}$$

P_{si} is the pretension force after allowing for the initial losses. The 1983 Standard Article 9.16.2.1.2 allows the pretension force after allowance for initial losses to be estimated as $0.69f_{pu}$ for low-relaxation strands.

$$P_{si} = 0.69\, f_{pu} A_{ps} = 0.69(270\text{ksi})(6.12\text{in}^2) = 1{,}140.2\,\text{kips}$$

$$f_{cir} = \frac{1140.2\text{kips}}{624\text{in}^2} + \frac{(1140.2\text{kips})\,(17.59\text{in})^2}{167{,}048\text{in}^4} - \frac{(505.4)(17.59\text{in})}{167{,}048\text{in}^4}\left(\frac{12\text{in}}{\text{ft}}\right)$$

$$= 1.827 + 2.113 - 0.639 = 3.301\,\text{ksi}$$

$$ES = \frac{28{,}500\text{ksi}}{4515\text{ksi}}(3.301\text{ksi}) = 20.84\,\text{ksi}$$

<u>Loss due to creep of concrete</u>, CR_c

Per the 1983 Standard Eqn. (9-9):

$$CR_c = 12\, f_{cir} - 7\, f_{cds}$$

<u>Concrete stress at the center of gravity of the strands due to all dead loads except the dead load present at the time the prestressing was applied</u>, f_{cds}

$$f_{cds} = \frac{M_{ds,0.5L}\, e_{mid}}{I} + \frac{M_{dw,0.5L}\, e_{mid-c}}{I_c}$$

$$= \frac{(919.1\text{ft.kips})(17.59\text{in})}{167{,}048\text{in}^4} + \frac{(137\text{ft.kips})(33.43\text{in})}{451{,}113\text{in}^4} = 1.284\,\text{ksi}$$

$$CR_c = 12(3.301\text{ksi}) - 7(1.284\text{ksi}) = 30.63\,\text{ksi}$$

<u>Loss due to relaxation of prestressing steel, CR_s</u>

Per the 1983 Standard Eqn. (9-10):

$CR_s = 20{,}000 - 0.4ES - 0.2(SH + CR_c)$, (in psi), for stress-relieved and 250 to 270 ksi strand

$$CR_s = 20 - 0.4(20.84\text{ksi}) - 0.2(6.2\text{ksi} + 30.63\text{ksi}) = 1.07\text{ ksi}$$

<u>Effective stress in the prestressing strands after all losses</u>, f_{se}

$$\Delta f_s = 6.2 + 20.84 + 30.63 + 1.07 = 58.8\text{ ksi}$$

$$f_{se} = f_{pjack} - \Delta f_s = 202.5 - 58.8 = 143.7\text{ ksi}$$

<u>Distance from the extreme compression fiber to the centroid of the prestressing strand, d</u>

$$d = \max(d_p, 0.8h) = \max(43.75\text{ in}, 44.0\text{ in}) = 44.0\text{ in}$$

<u>Flexure-shear Cracking Strength</u>, V_{ci}

Per the 1983 Standard Eqn. (9-27):

$$V_{ci} = 0.6\sqrt{f_c^{'}}b_w d + V_d + \frac{V_i M_{cr}}{M_{\max}} \geq 1.7\sqrt{f_c^{'}}b_w d$$

<u>Minimum V_{ci}</u>

$$V_{ci,\min} = 1.7\sqrt{f_c^{'}}b_w d = 1.7\frac{\sqrt{6{,}861\text{psi}}}{(1{,}000)}(6\text{in})(44.0\text{in}) = 37.2\text{ kips}$$

<u>Cracking moment,</u> M_{cr}

Per the 1983 Standard Eqn. (9-28):

$$M_{cr} = \frac{I_c}{y_{bc}}(6\sqrt{f_c^{'}} + f_{pe} - f_d)$$

<u>Compressive stress in the concrete due to effective prestressing force at extreme tension fiber, f_{pe}</u>

$$P_{se} = f_{se}A_{ps} = (143.7\text{ksi})(6.12\text{in}^2) = 879.8\text{ kips}$$

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}ey_b}{I} = \frac{(879.8\text{kips})}{(624\text{in}^2)} + \frac{(879.8\text{kips})(11.09\text{in})(22.34\text{in})}{(167{,}049\text{in}^4)}$$

$$= 2.715\text{ ksi}$$

Stress due to unfactored dead load at extreme tension fiber, f_d

$$f_d = \frac{(M_{dg} + M_{ds} + M_{dd})y_b}{I} + \frac{M_{dw.}\, y_{bc}}{I_c}$$

$$= \frac{(73.1 + 120.4 + 18.2\text{ft.kips})(22.34\text{in})}{(167{,}049\text{in}^4)} + \frac{(19.9\text{ ft.kips})(38.18\text{in})}{(451{,}113\text{in}^4)} = 0.360\text{ ksi}$$

$$M_{cr} = \frac{I_c}{y_{bc}}(6\sqrt{f_c^{'}} + f_{pe} - f_d)$$

$$= \frac{(451{,}113\text{in}^4)}{(38.18\text{in})}\left(\frac{6\sqrt{6{,}861\text{psi}}}{1{,}000} + 2.715\text{ksi} - 0.360\text{ksi}\right)\left(\frac{1\text{ ft}}{12\text{in}}\right) = 2{,}807.9\text{ ft.kips}$$

$$V_{ci} = 0.6\frac{\sqrt{6{,}861\text{psi}}}{1{,}000}(6\text{in})(44.0\text{in}) + (73.5\text{kips}) + \frac{(168.5\text{kips})(2{,}807.9\text{ft.kips})}{(604.4\text{ft.kips})} = 255.1\text{ kips} > 37.2\text{ kips}$$

Web-shear strength, V_{cw}

Per the 1983 Standard Eqn. (9-29):

$$V_{cw} = (3.5\sqrt{f_c^{'}} + 0.3 f_{pc})b_w d + V_p$$

Compressive stress in the concrete (after allowance for all prestress losses at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, f_{pc}

Note: In a composite member, f_{pc} is the resultant compressive stress at centroid of composite section or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.

$$f_{pc} = \frac{P_{se}}{A} - \frac{P_{se} e\,\Delta y}{I} + \frac{(M_{dg} + M_{ds} + M_{dd})\Delta y}{I}$$

$\Delta y = (y_{bc} - y_b)$ if centroid is within the web,

$\Delta y = (h_{web+b_f} - y_b)$ if centroid is within the flange.

The centroid lies within the flange for this case, thus,

$$\Delta y = (h_{web+b_f} - y_b) = (35.50\text{in} - 22.34\text{in}) = 13.16\text{ in}$$

$$f_{pc} = \frac{(879.8\text{kips})}{(624\text{in}^2)} - \frac{(879.8\text{kips})(11.09\text{in})(13.16\text{in})}{(167{,}048\text{in}^4)} ..$$

$$+ \frac{(73.1 + 120.4 + 18.2\text{ ft.kips})(13.16\text{in})}{(167{,}048\text{in}^4)}\left(\frac{12\text{in}}{\text{ft}}\right)$$

$$= 1.410\text{ksi} - 0.768\text{ksi} + 0.200\text{ksi} = 0.842\text{ ksi}$$

Vertical component of the prestressing force for harped strands, V_p

$$V_p = f_{se} A_{ps-harped} \sin\psi = (143.7\text{ksi})(1.53\text{in}^2)(\sin(4.02°)) = 15.4\text{ kips}$$

$$V_{cw} = \left(3.5\frac{\sqrt{6{,}861\text{psi}}}{1{,}000} + 0.3(0.842\text{ksi})\right)(6\text{in})(44.0\text{in}) + (15.4\text{kips}) = 158.6\text{ kips}$$

Concrete contribution to shear capacity, V_c

$V_c = \min(V_{ci}, V_{cw}) = \min(255.1\text{kips}, 158.6\text{kips}) = 158.6$ kips, V_{cw} controls.

Required stirrup spacing for double leg #4 reinforcing bars

Per the 1983 Standard Eqn. (9-30):

$$s_{req'd} = \frac{A_v f_{sy} d}{\left(\frac{V_u}{\phi} - V_c\right)} = \frac{2(0.2\text{in}^2)(60\text{ksi})(44.0\text{in})}{\left(\frac{279.1}{0.9} - 158.6\right)} = 7.0\text{ in}$$

Check limits on stirrup spacing

- Per the 1983 Standard Article 9.20.3.2:

$s_{\max,1} = \min(0.75h, 24\text{in})$ if $V_s > 4\sqrt{f_c^{'}} b_w d$, $s_{\max,1} = \min(0.375h, 12\text{in})$

Check $V_s > 4\sqrt{f_c^{'}} b_w d$,

$$V_s = \frac{V_u}{\phi} - V_c = \frac{279.1}{0.9} - 158.6 = 151.5\text{ kips},\quad 4\sqrt{f_c^{'}} b_w d = 4\frac{\sqrt{6{,}861\text{psi}}}{1{,}000}(6\text{in})(44.0\text{in}) = 87.5\text{ kips}$$

$V_s = 151.5\text{ kips} > 87.5\text{ kips}$, thus, $s_{\max,1} = \min(0.375h, 12\text{in})$

$$s_{\max,1} = \min\big(0.375(55.0\text{in}), 12\text{in}\big) = 12\text{ in}$$

- From the 1983 Standard Eqn. (9-31)

$$s_{\max,2} = \frac{A_v f_{sy}}{50 b_w} = \frac{(0.4\text{in}^2)(60{,}000\text{psi})}{50(6\text{in})} = 80.0\text{ in}$$

$$s_{\max} = \min\left(s_{\max,1}, s_{\max,2}\right) = \min(80\text{ in}, 12\text{ in}) = 12\text{ in}$$

$s_{\max} > s_{req'd}$ O.K.

<u>Check horizontal shear design at the critical section</u>

The horizontal shear design at the critical section is carried out in accordance with Article 9.20.4 of the 1983 Standard. Similar to the 1961 Standard (Section A.2.1), the ultimate horizontal shear demand (in stress) is given by the 1983 Standard Article 9.20.4.3 as

$$v_u = \frac{V_u Q_c}{I_c b_v}$$

For this case, $Q_c = 9{,}068\text{ in}^3, I_c = 451{,}113\text{ in}^4, b_v = 30\text{ in}$.

$$v_u = \frac{(279.1\text{kips})(9{,}068\text{in}^3)}{(451{,}113\text{in}^4)(30\text{in})} = 187.0\text{ psi}$$

<u>Check stirrup spacing requirement for horizontal shear</u>, $s_{vh,max}$

$$s_{vh,\max} = \min\begin{pmatrix} 4(t_{least}) \\ 24in \\ A_v/(A_{v,\#3}/ft) \end{pmatrix}$$

where $A_v/(A_{v,\#3}/ft)$ represents the Article 9.20.4.4 of the 1983 Standard.

Similar to that shown in Section A.2.1, $A_v/(A_{v,\#3}/ft)$ yields 21.8 in. stirrup spacing.

$$s_{vh,\max} = \min\begin{pmatrix} 4(6\text{in}) \\ 24\text{in} \\ 21.8\text{in} \end{pmatrix} = 21.8\text{ in}$$

Then $s_{req'd} = 7.0\text{ in} < s_{vh,\max} = 21.8\text{ in}$ O.K.

When the minimum requirements of the 1983 Standard Article 9.20.4.4 are met and the contact surface of the precast element is clean and intentionally roughened, then the horizontal shear capacity at the interface can be taken as 300 psi.

Check $v_u - \phi(v_{nh}) = 187.0\text{psi} - 0.9(300\text{psi}) = -83.0\text{ psi}$

Because $\phi(v_{nh}) > v_u$, there is no need for additional stirrups for horizontal shear, provided that the minimum requirements of Article 9.20.4.4 are met.

Therefore, at the critical section, the design spacing for #4 stirrups was 7.0 in.

A.4 Shear Capacity and Rating Calculations According to 2002 AASHTO Standard Specifications

As indicated in Section 5.4, sample shear capacity and rating calculations were determined at the critical section according to the 2002 Standard in this section. Bridge 48010 (interior girder) was selected for illustration because it had the lowest $\phi V_{n,STD2002}/V_u$ among the investigated girders.

According to the 2002 Standard, the critical section was h/2 from the face of the support, i.e., ~28 in. (or ~43 in. from the end of the girder) for this case.

Material and sectional properties

All material and sectional properties are given in Table A.1.

Composite section properties

The calculations for the composite section are provided below and the results are summarized in Table A.3. The variables are also illustrated in Figure A.2.

Effective flange width, b_{eff}

Per the 2002 Standard Article 9.8.3.2:

$$b_{eff} = \min\begin{pmatrix} L/4 \\ S_g \\ 12t_s + b_w \end{pmatrix}$$, for narrow-type top flanges (i.e., AASHTO Type III)

$$b_{eff} = \min\begin{pmatrix} \dfrac{(43.2\text{ft})(12\text{in/ft})}{4} \\ (12.5\text{ft})(12\text{in/ft}) \\ 12(9.25\text{in}) + 7\text{in} \end{pmatrix} = \min\begin{pmatrix} 130\text{ in} \\ 150\text{ in} \\ 118\text{ in} \end{pmatrix} = 118\text{ in}$$

Modular ratio between slab and girder, n

$$n = \frac{E_{girder}}{E_{slab}}$$

Per the 2002 Standard Eqn. (9-8): $E = 33w_c^{1.5}\sqrt{f_c^{'}}$

$$E_{girder} = 33(145\text{lb/ft}^3)^{1.5}\left(\frac{\sqrt{5{,}000\text{psi}}}{1{,}000}\right) = 4074\text{ ksi}$$

$$E_{slab} = 33(145\text{lb/ft}^3)^{1.5}\left(\frac{\sqrt{4{,}000\text{psi}}}{1{,}000}\right) = 3644\text{ ksi}$$

$$n = \frac{4074\text{ksi}}{3644\text{ksi}} = 1.12$$

- Transformed slab width = $\frac{b_{eff}}{n} = \frac{118in}{1.12} = 105.5\text{ in}$

- Transformed slab area= (Transformed slab width) (t_s) = (105.5in)(9.25in) = 976.3 in^2

- Transformed haunch width = $\frac{t_{top-flange}}{n} = \frac{16\text{in}}{1.12} = 14.3\text{ in}$

- Transformed haunch area (Transformed haunch width) (t_s) = (14.3in)(1.5in) = 21.5 in^2

Using the information summarized in Table A.3,

$$y_{bc} = \frac{\sum A(y_b)}{A} = \frac{62245\text{in}^3}{1557.7\text{in}^2} = 39.96\text{ in}$$

$$I_c = \sum A(y_{bc} - y_b)^2 + \sum I = 339{,}528\text{in}^4 + 132{,}356\text{in}^4 = 471{,}884\text{ in}^4$$

<u>Shear forces and moments at the critical section</u>

The factored ultimate shear included the dead load from: girder, slab, diaphragms, barrier, and wearing course; and live load from AASHTO HS20-44. The shear forces and moments were computed using *Virtis*-BRASS. A value of WAD=1000 (as discussed in Section 3.3.1) was used to achieve the required accuracy in estimating the live load shear and moment acting at the section.

- Unfactored shear force due to self weight of the girder, $V_{dg} = 11.2\text{ kips}$

- Unfactored shear force due to self weight of the slab and diaphragms, $V_{ds} = 30.3\text{ kips}$

- Unfactored shear force due to self weight of barriers and wearing course, $V_{dw} = 4.8\text{ kips}$

- Unfactored shear force due to total dead load, $V_d = V_{dg} + \text{V}_{\text{ds}} + \text{V}_{\text{dw}} = 46.3\text{ kips}$

- Unfactored shear force due to live load, $V_{LL} = 76.1\text{ kips}$

- Total factored shear force, $V_u = 1.3(V_d + 1.67V_{LL}) = 225.1\text{ kips}$, load factors were obtained per the 2002 Standard Table 3.22.1A.

- Unfactored moment due to self weight of the girder, $M_{dg} = 35.7\text{ ft.kips}$

- Unfactored moment due to self weight of the slab and diaphragms, $M_{ds} = 96.1\text{ ft.kips}$

- Unfactored moment due to self weight of barriers and wearing course, $M_{dw} = 15.2\text{ ft.kips}$

- Unfactored moment due to total dead load, $M_d = M_{dg} + \mathrm{M_{ds}} + \mathrm{M_{dw}} = 147.0\text{ ft.kips}$

- Unfactored moment force due to live load, $M_{LL} = 223.9\text{ ft.kips}$

- Total factored moment, $M_u = 1.3(M_d + 1.67M_{LL}) = 677.1\text{ ft.kips}$

- Maximum factored moment at section due to externally applied loads, $M_{\max} = M_u - M_d = 677.1 - 147.0 = 530.1\text{ ft.kips}$

- Factored shear force at section due to externally applied loads occurring simultaneously with $M_{\max}$: $V_i = 177.8\text{ kips}$

<u>Total prestress losses, Δf_s</u>

Per the 2002 Standard Eqn. (9-3):

$$\Delta f_s = SH + ES + CR_c + CR_s$$

<u>Loss due to concrete shrinkage</u>, *SH*

Per the 2002 Standard Eqn. (9-4):

$SH = 17{,}000 - 150RH$ (in psi), for pretensioned members

$$SH = \frac{[17{,}000 - 150(70)]\text{psi}}{1{,}000} = 6.5\text{ ksi}$$

<u>Loss due to elastic shortening</u>, *ES*

Per the 2002 Standard Eqn. (9-6):

$$ES = \frac{E_{ps}}{E_{ci}} f_{cir}\text{, for pretensioned members}$$

<u>Modulus of elasticity of concrete at transfer, E_{ci}</u>

Per the 2002 Standard Eqn. (9-8):

$$E_{ci} = 33 w_c^{1.5} \sqrt{f_{ci}^{'}} = 33(145\text{lb/ft}^3)^{1.5}\left(\frac{\sqrt{4{,}500\text{psi}}}{1{,}000}\right) = 3865\text{ ksi}$$

<u>Concrete stress at the center of gravity of the strands due to prestressing force and dead load of girder immediately after transfer</u>, f_{cir}

Per the 2002 Standard (Article 9.16.2.1.2), f_{cir} shall be computed at the section or sections of maximum moment (i.e., midspan for simply-supported spans).

- Unfactored moment due to self weight of the girder at midspan, $M_{dg,0.5L} = 140.3$ ft.kips

- Unfactored moment due to self weight of the slab and diaphragms at midspan, $M_{ds,0.5L} = 392.2$ ft.kips

- Unfactored moment due to self weight of barriers and wearing course at midspan, $M_{dw,0.5L} = 59.9$ ft.kips

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}\, e_{mid}^{\;2}}{I} - \frac{M_{dg,0.5L}\, e_{mid}}{I}$$

P_{si} is the pretension force after allowing for the initial losses. The 2002 Standard Article 9.16.2.1.2 allows the pretension force after allowance for initial losses to be estimated as $0.63f_{pu}$ for stress-relieved strands (abbreviated as SR, in Table A.3).

$$P_{si} = 0.63 f_{pu} A_{ps} = 0.63(270\text{ksi})(2.448\text{in}^2) = 416.4\text{ kips}$$

$$f_{cir} = \frac{416.4\text{kips}}{560\text{in}^2} + \frac{(416.4\text{kips})(17.27\text{in})^2}{125{,}390\text{in}^4} - \frac{(140.3\text{ft.kips})(17.27\text{in})}{125{,}390\text{in}^4}\left(\frac{12\text{in}}{\text{ft}}\right)$$

$$= 0.744 + 0.990 - 0.232 = 1.502\text{ ksi}$$

$$ES = \frac{28{,}500\text{ksi}}{3865\text{ksi}}(1.502\text{ksi}) = 11.08\text{ ksi}$$

Loss due to creep of concrete, CR_c

Per the 2002 Standard Eqn. (9-9):

$$CR_c = 12 f_{cir} - 7 f_{cds}$$

Concrete stress at the center of gravity of the strands due to all dead loads except the dead load present at the time the prestressing was applied, f_{cds}

$$f_{cds} = \frac{M_{ds,0.5L}\, e_{mid}}{I} + \frac{M_{dw,0.5L}\, e_{mid-c}}{I_c}$$

$$= \frac{(392.2\text{ft.kips})(17.27\text{in})}{125{,}390\text{in}^4} + \frac{(59.9\text{ft.kips})(36.96\text{in})}{471{,}884\text{in}^4} = 0.705\,\text{ksi}$$

$$CR_c = 12(1.502\text{ksi}) - 7(0.705\text{ksi}) = 13.09\,\text{ksi}$$

Loss due to relaxation of prestressing steel, CR_s

Per the 2002 Standard Eqn. (9-10):

$CR_s = 20{,}000 - 0.4ES - 0.2(SH + CR_c)$, (in psi), for stress-relieved and 250 to 270 ksi strand

$$CR_s = 20 - 0.4(11.08\text{ksi}) - 0.2(6.5\text{ksi} + 13.09\text{ksi}) = 11.65\,\text{ksi}$$

Effective stress in the prestressing strands after all losses, f_{se}

$$\Delta f_s = 6.5 + 11.08 + 13.09 + 11.65 = 42.3\,\text{ksi}$$

$$f_{se} = f_{pjack} - \Delta f_s = 189.0 - 42.3 = 146.7\,\text{ksi}$$

Distance from the extreme compression fiber to the centroid of the prestressing strand, d

$$d = \max(d_p, 0.8h) = \max(45.42\,\text{in}, 44.60\,\text{in}) = 45.42\,\text{in}$$

Flexure-shear strength, V_{ci}

Per the 2002 Standard Eqn. (9-27):

$$V_{ci} = 0.6\sqrt{f_c'}\,b_w d + V_d + \frac{V_i M_{cr}}{M_{\max}} \geq 1.7\sqrt{f_c'}\,b_w d$$

Minimum V_{ci}

$$V_{ci,\min} = 1.7\sqrt{f_c^{'}}b_w d = 1.7\frac{\sqrt{5{,}000\text{psi}}}{(1{,}000)}(7\text{in})(45.42\text{in}) = 38.2\text{ kips}$$

Cracking moment, M_{cr}

Per the 2002 Standard Eqn. (9-28):

$$M_{cr} = \frac{I_c}{y_{bc}}(6\sqrt{f_c^{'}} + f_{pe} - f_d)$$

Compressive stress in the concrete due to effective prestressing force at extreme tension fiber, f_{pe}

$$P_{se} = f_{se}A_{ps} = (146.7\text{ksi})(2.448\text{in}^2) = 359.1\text{ kips}$$

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}ey_b}{I} = \frac{(359.1\text{kips})}{(560\text{in}^2)} + \frac{(359.1\text{kips})(9.94\text{in})(20.27\text{in})}{(125{,}390\text{in}^4)}$$

$$= 1.218\text{ ksi}$$

Stress due to unfactored dead load at extreme tension fiber, f_d

$$f_d = \frac{(M_{dg} + M_{ds})y_b}{I} + \frac{M_{dw.}y_{bc}}{I_c}$$

$$= \frac{(35.7 + 96.1\text{ ft.kips})(20.27\text{in})}{(125{,}390\text{in}^4)} + \frac{(15.2\text{ ft.kips})(39.96\text{in})}{(471{,}884\text{in}^4)} = 0.271\text{ ksi}$$

$$M_{cr} = \frac{I_c}{y_{bc}}(6\sqrt{f_c^{'}} + f_{pe} - f_d)$$

$$= \frac{(471{,}884\text{in}^4)}{(39.96\text{in})}\left(\frac{6\sqrt{5{,}000\text{psi}}}{1{,}000} + 1.218\text{ksi} - 0.271\text{ksi}\right)\left(\frac{1\text{ ft}}{12\text{in}}\right) = 1{,}349.8\text{ ft.kips}$$

$$V_{ci} = 0.6\frac{\sqrt{5{,}000\text{psi}}}{1{,}000}(7\text{in})(45.42\text{in}) + (46.3\text{kips}) + \frac{(177.8\text{kips})(1{,}349.8\text{ft.kips})}{(530.1\text{ft.kips})} = 237.6\text{ kips} > 38.2\text{ kips}$$

Web-shear strength, V_{cw}

Per the 2002 Standard Eqn. (9-29):

$$V_{cw} = (3.5\sqrt{f_c^{'}} + 0.3 f_{pc})b_w d + V_p$$

Compressive stress in the concrete (after allowance for all prestress losses at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, f_{pc}:

Note: In a composite member, f_{pc} is the resultant compressive stress at centroid of composite section or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone.

$$f_{pc} = \frac{P_{se}}{A} - \frac{P_{se} e \Delta y}{I} + \frac{(M_{dg} + M_{ds})\Delta y}{I}$$

$\Delta y = (y_{bc} - y_b)$ if centroid is within the web,

$\Delta y = (h_{web+b_f} - y_b)$ if centroid is within the flange.

As shown in Figure A.2, the centroid lies within the flange, thus,

$$\Delta y = (h_{web+b_f} - y_b) = (33.5\text{in} - 20.27\text{in}) = 13.23\text{ in}$$

$$f_{pc} = \frac{(359.1\text{kips})}{(560\text{in}^2)} - \frac{(359.1\text{kips})(9.94\text{in})(13.23\text{in})}{(125{,}390\text{in}^4)} ..$$
$$+ \frac{(35.7 + 96.1\text{ ft.kips})(13.23\text{in})}{(125{,}390\text{in}^4)}\left(\frac{12\text{in}}{\text{ft}}\right)$$

$$= 0.641\text{ksi} - 0.377\text{ksi} + 0.167\text{ksi} = 0.431\text{ ksi}$$

Vertical component of the prestressing force for harped strands, V_p

$$V_p = f_{se} A_{ps-harped} \sin\psi = (146.7\text{ksi})(0.612\text{in}^2)(\sin(8.88°)) = 13.9\text{ kips}$$

$$V_{cw} = \left(3.5\frac{\sqrt{5{,}000\text{psi}}}{1{,}000} + 0.3(0.431\text{ksi})\right)(7\text{in})(45.42\text{in}) + (13.9\text{kips}) = 133.7\text{ kips}$$

Concrete contribution to shear capacity, V_c

$V_c = \min(V_{ci}, V_{cw}) = \min(237.6\text{kips}, 133.7\text{kips}) = 133.7$ kips, V_{cw} controls.

Stirrup contribution to shear capacity, V_s

Per the 2002 Standard Eqn. (9-30):

$$V_s = \frac{A_v f_{sy} d}{s} \text{ where } V_s \leq 8\sqrt{f_c^{'}} b_w d$$

Note: Provided stirrup spacing at the design section was 21 in. (taken from the bridge plan),

$$V_s = \frac{0.4\text{in}^2(60\text{ksi})(45.42\text{in})}{(21\text{in})} = 51.9\text{ kips}$$

$$V_s = 51.8\text{ kips} < 8\frac{\sqrt{5{,}000\text{psi}}}{1{,}000}(7\text{in})(45.42\text{in}) = 180.0\text{ kips}, \text{ O.K.}$$

Thus, $V_s = 51.9\text{ kips}$

<u>Shear capacity</u>, V_n

$$V_n = V_c + V_s = 133.7 + 51.9 = 185.6\text{ kips}$$

<u>Shear capacity to shear demand ratio</u>, $\frac{\phi V_{n,\ STD\,2002}}{V_u}$

Per the 2002 Standard Article 9.14: $\phi = 0.9$ is the strength capacity reduction factor for shear

$$\frac{\phi V_{n,\ STD\,2002}}{V_u} = \frac{(0.9)(185.6\text{kips})}{(225.1\text{kips})} = 0.74$$

<u>Shear inventory rating factor</u>, $Inv.RF = \frac{(\phi V_{n,\,STD2002}) - (1.3)(V_d)}{(1.3)(1.67)(V_{LL})}$

$$Inv.RF = \frac{(0.9)(185.6\text{kips}) - (1.3)(46.3\text{kips})}{(1.3)(1.67)(76.1\text{kips})} = 0.65$$

Table A.1 Girder Properties of the Bridge No. 83022_1-3

Bridge No	Year Built	Year of Design Spec.	Girder Depth (in)	Web Width (in)	Span Length (ft)	Girder Spacing (ft)	Girder f'_c (psi)	e[1] at End of Girder (in)	e at End of Harping Distance (in)	# of Strands[2]	Type of Strand[3] (ksi)
83022_1-3	1975	1973	45	7	56.8	10.8	5000	8.8	16.5	18 (6)	270 (SR)

[1] e: Eccentricity for the non-composite section
[2] Number in parenthesis is the number of draped strands
[3] LR: Low-relaxation, SR: Stress-relieved

Table A.2 Information for Transformed Composite Section Properties for Bridge 9011

Element	Transformed Area, A (in^2)	y_b (in)	$A(y_b)$ (in^3)	$A(y_{bc}-y_b)^2$ (in^4)	I (in^4)	$I + A(y_{bc}-y_b)^2$ (in^4)
Girder	624	22.34	13943	156547	167048	323595
Slab	766	50.75	38866	120931	4611	125542
Haunch	34	45.75	1572	1967	8	1975
Sum, Σ	1424	-	54381	279445	171668	451113

Table A.3 Information for Transformed Composite Section Properties for Bridge 48010

Element	Transformed Area, A (in^2)	y_b (in)	$A(y_b)$ (in^3)	$A(y_{bc}-y_b)^2$ (in^4)	I (in^4)	$I + A(y_{bc}-y_b)^2$ (in^4)
Girder	560	20.27	11351	217080	125390.0	342470
Slab	976	51.125	49912	121728	6961.0	128689
Haunch	22	45.75	982	719.97	4.5	724
Sum, Σ	1558	-	62245	339528	132355.5	471884

Table A.4 Properties of the Bridges Designed by the AASHTO Standards (Section A.2)

Properties	Variable	Values			
Bridge No	-	**27978-2**	**24831-1_3**	**48010**	**9011**
Year Built	-	1965	1970	1979	1990
Year of Design Spec.	-	1961	1969	1977	1983
Overall Geometry	**Variable**	**Values**			
Type of the girder	-	MN-40	MN-36	MN-45	45-M
Span Length (ft)	L	52	40	43	78
Girder Spacing (ft)	S_g	11.2	12.7	12.5	9.8
Number of Girders	N_b	4	4	4	5
Roadway Width (ft)	W	38.5	41.7	40.8	42.8
Section Properties					
Noncomposite Section Properties	**Variable**	**Values**			
Area of the girder (in^2)	A	485	369	560	624
Height of the girder (in)	h	40	36	45	45
Moment of inertia about the centroid of the non-composite girder (in^4)	I	87654	50979	125390	167049
Distance from centroid to extreme bottom fiber of the non-composite section (in)	y_b	17.88	15.83	20.27	22.34
Web Width (in)	b_w	6	6	7	6
Top flange width (in)	b_v	16	12	16	30
Eccentricity at the critical section (in)	e	N.N.[1]	N.N.	9.94[2]	11.09
Eccentricity at the midspan (in)	e_{mid}	N.N.	N.N.	17.27[2]	17.59
Composite Section Properties	**Variable**	**Values**			
Height of the composite section (in)	h_c	49.50	46.75	55.75	55.00
Haunch thickness (in)	t_{haunch}	1.50	1.00	1.5	1.5
Slab thickness (in)	t_s	8.00	9.75	9.25	8.50
Distance from centroid to extreme bottom fiber of the composite section (in)	y_{bc}	34.58	35.03	39.96	38.18
Moment of inertia-composite section (in)	I_c	314689	243647	471884	451113
Eccentricity at the midspan (in)	$e_{mid\text{-}c}$	N.N.	N.N.	36.96[2]	33.43
Material Properties					
Concrete Properties	**Variable**	**Values**			
Concrete strength of girder at 28 days psi)	$f'_{cgirder}$	5000	5000	5000	6861
Concrete strength of deck at 28 days (psi)	f'_{cslab}	4000	4000	4000	4000
Prestressing Strands	**Variable**	**Values**			
Strand type	-	270(LR)	270(SR)	270(SR)	270(LR)
Number of straight strands	-	20	10	12	30
Number of harped strands	-	8	8	4	10
Harping slope (degrees)	Ψ	5.77	6.02	8.88	4.02
Area of all strands (in^2)	A_{ps}	4.284	2.754	2.448	6.120
Ultimate strength (ksi)	f_{pu}	270	270	270	270
Modulus of elasticity (ksi)	E_{ps}	28500	28500	28500	28500
Reinforcing Bars	**Variable**	**Values**			
Yield strength (ksi)	f_{sy}	40	40	60	60
Area of stirrups at a cross section (in^2)	A_v	0.4	0.4	0.4	0.4

[1] N.N.: Not Needed, those values are not needed for the corresponding design calculations.
[2] These values are not used in Section A.2.3, but in Section A.3

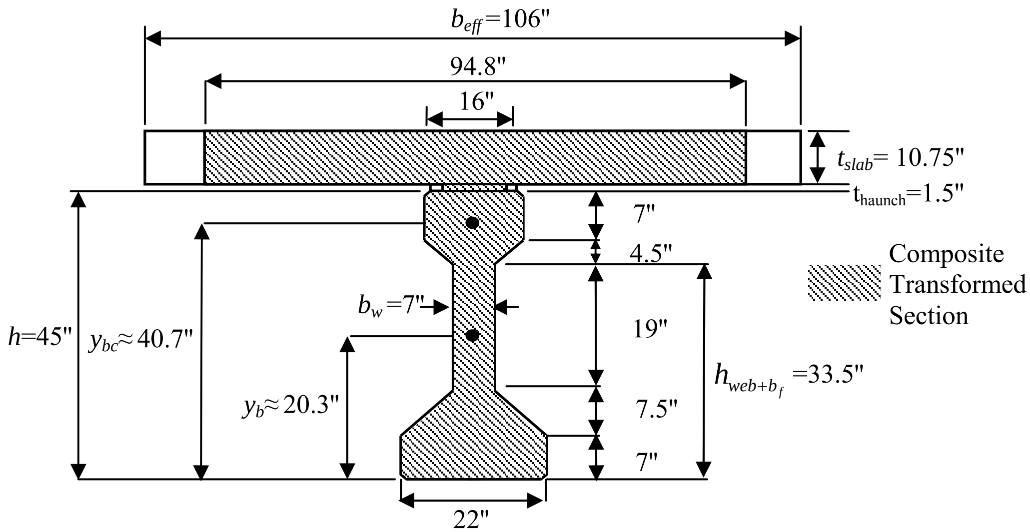


Figure A.1 Composite Cross Section for Interior Girder of Bridge 83022

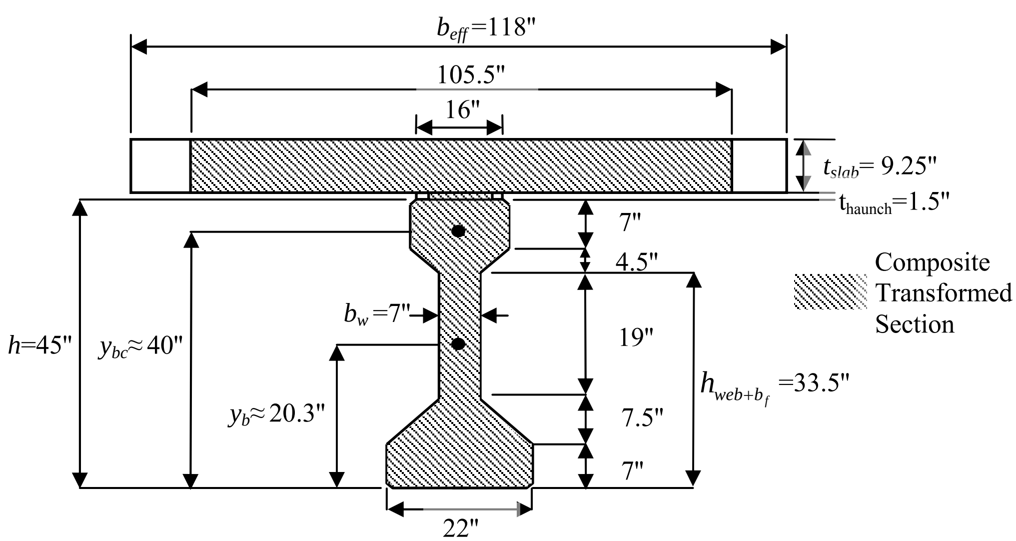


Figure A.2 Composite Cross Section for Interior Girder of Bridge 48010

Appendix B

Bridge Inspection Report

Shear Discrepancies Investigation

Mn/DOT Bridge Inspection Unit

April 2009

Background

University of Minnesota Study-

- Shear Capacity of Prestressed Concrete Bridge Girders
- Bridge girder shear design is governed by AASHTO specifications which have changed significantly with time.
- The 1979 Interim shear provisions typically require less shear reinforcement than 2004 LRFD and 2002 Standard.
- Many Mn/DOT bridges were designed according to the 1979 Interim.

Inventory Study Conducted by the U of MN

- Shear capacities of bridges were calculated by using the nominal material properties, stirrup spacing in the bridge plans and the 1996 (or 2002) AASHTO shear provisions.
- Mn/DOT Bridge Inspection Unit asked to perform visual inspection (within “hands –reach”) of six bridges within 60 mile radius of the Metro area.
- Objective of visual inspections is to determine the presence of diagonal shear cracks.

Selected Bridges

- Six Bridges Near the Metro Area selected for Visual Inspection.
- Selection Based on capacity/demand ration (c/d) calculated using nominal material properties, stirrup spacing in the bridge plans and the 1996 (or 2002) AASHTO shear provisions.
- Majority of (c/d) less than 1.0 at $h_c/2$ or 0.1*L- at or near supports (see Selected Bridge Details).
- Able to perform “Hands-on” (within two feet) Inspection at these locations by walking up abutments.

Selected Bridge Details

Bridge #	Location	County	Span	Year Built	Total Length (ft)	ϕ*Vn, 1996STD/Vu						
						Design Spec.	H (in)	At hc/2	0.1* L	0.2* L	0.3* L	0.4* L
9603	I-35W over Co Rd I	Ramsey	1,3	1968	136.8	1965	40	0.77	0.81	0.91	1.08	1.08
19033	US 52 SB over the Vermillion River	Dakota	1,3	1978	156.9	1973	36	0.85	0.98	1.04	0.96	1.02
27068	TH 7 over Recreational Trail	Hennepin	1,3	1981	150.6	1977	36	0.83	0.90	1.13	1.05	1.17
48010	US 52 NB over CSAH 31	Mille Lacs	1,3	1979	167.7	1977	45	0.74	0.81	1.07	1.08	1.19
73872	CR 159 over I-94	Stearns	1,4	1976	279.0	1973	54	0.79	0.87	1.11	1.05	1.13
62860	35W over NB Ramp & TH 280 SB	Ramsey	1,3	1970	210.0	1965	60	0.83	0.90	1.13	1.05	1.17

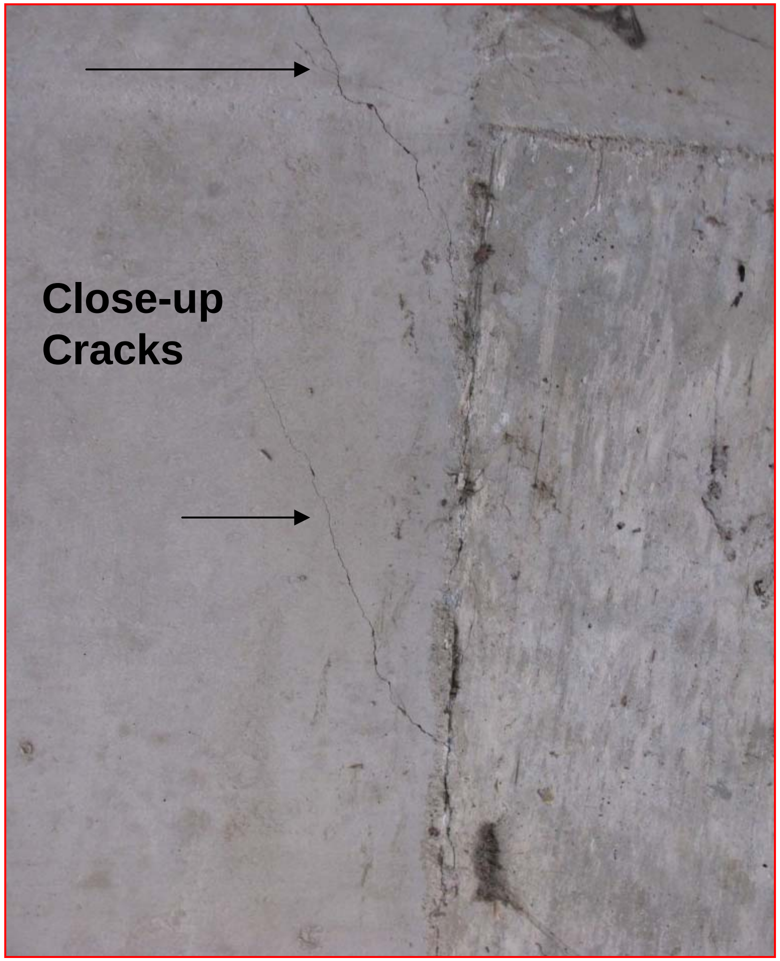
Bridge 9603

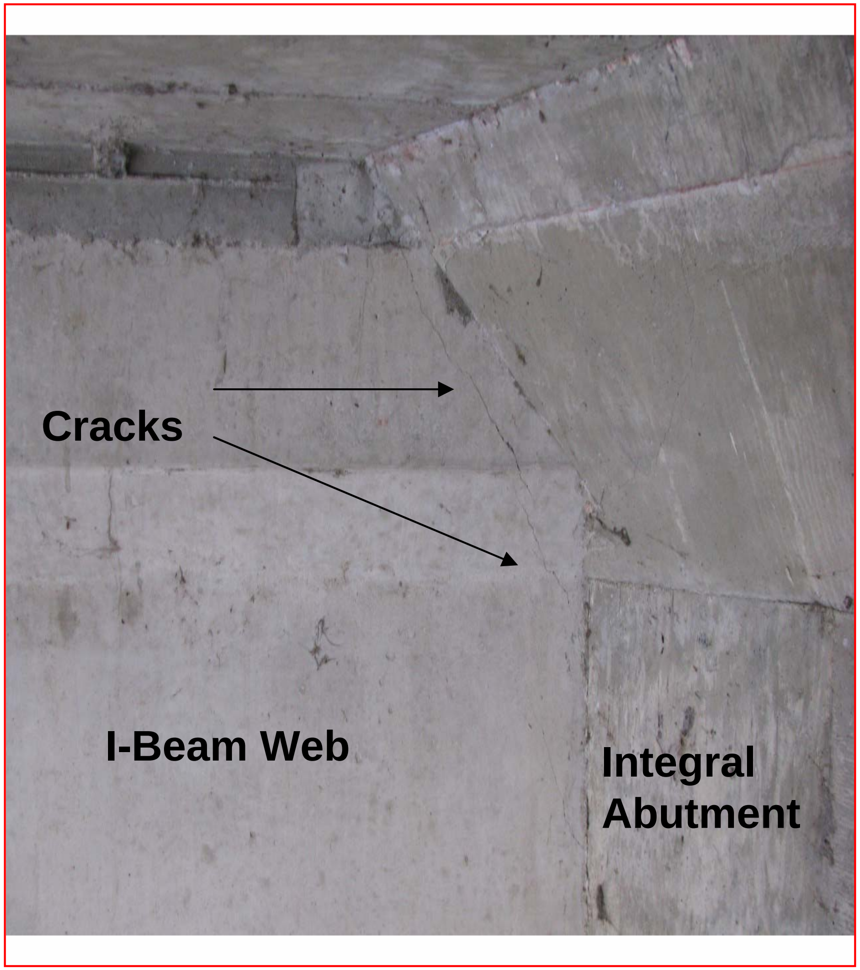
- 35W over Co Rd I
- Pile Bent Abutments
- Spans 1 & 3
- Visual Inspection performed by walking up the abutments.
- **No diagonal shear cracks found on girders at or near supports**.

Bridge 19033

- US 52 SB over the Vermillion River.
- Pile Bent Abutments
- Visual Inspection performed by walking up abutments.
- 2 linear feet of diagonal cracking found on the east side of girder 2 in span 1 (see photos & sketch)

Bridge 19033





Bridge 19033

- Photos of cracking reviewed by Mn/DOT Load Rating Engineer and University of Minnesota.
- Cracks only present in one location (one girder, only on the east side).
- **Recommendation to monitor cracks for changes or the presence of addition cracks**.

Bridge 27068

- TH 7 over Recreational Trail.
- Parapet Abutments
- Visual inspection performed by walking up abutments.
- **No diagonal shear cracks found on girders at or near supports**.

Bridge 48010

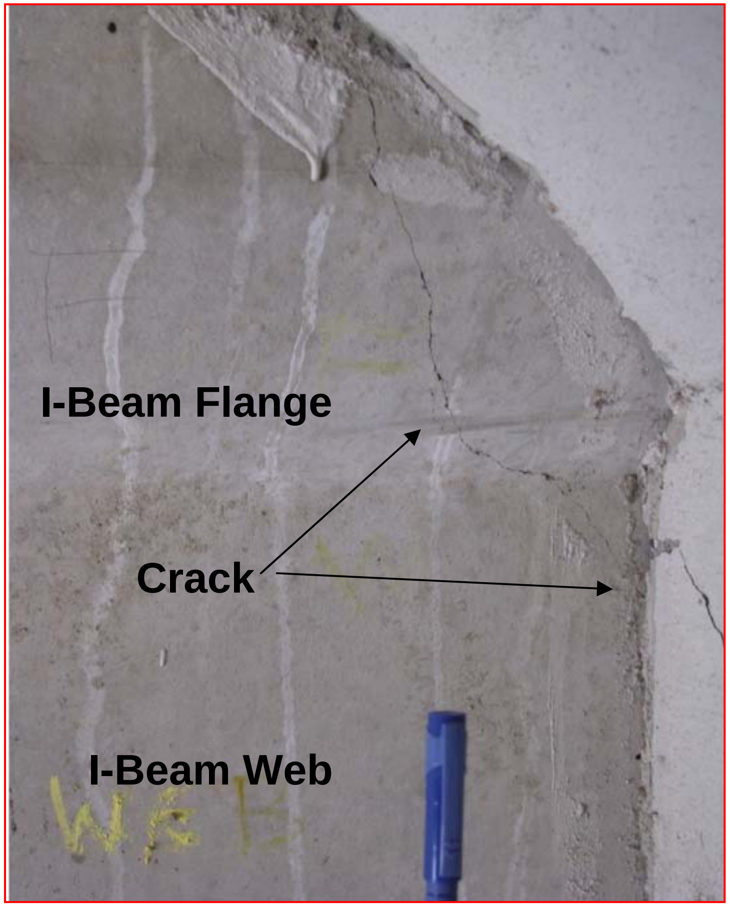
- US 169 NB over CSAH 31.
- Pile Bent Abutments.
- Twin bridge, 48009 – US 169 SB over CSAH 31 also inspected.
- “Hands-on” inspection performed by walking up abutments.

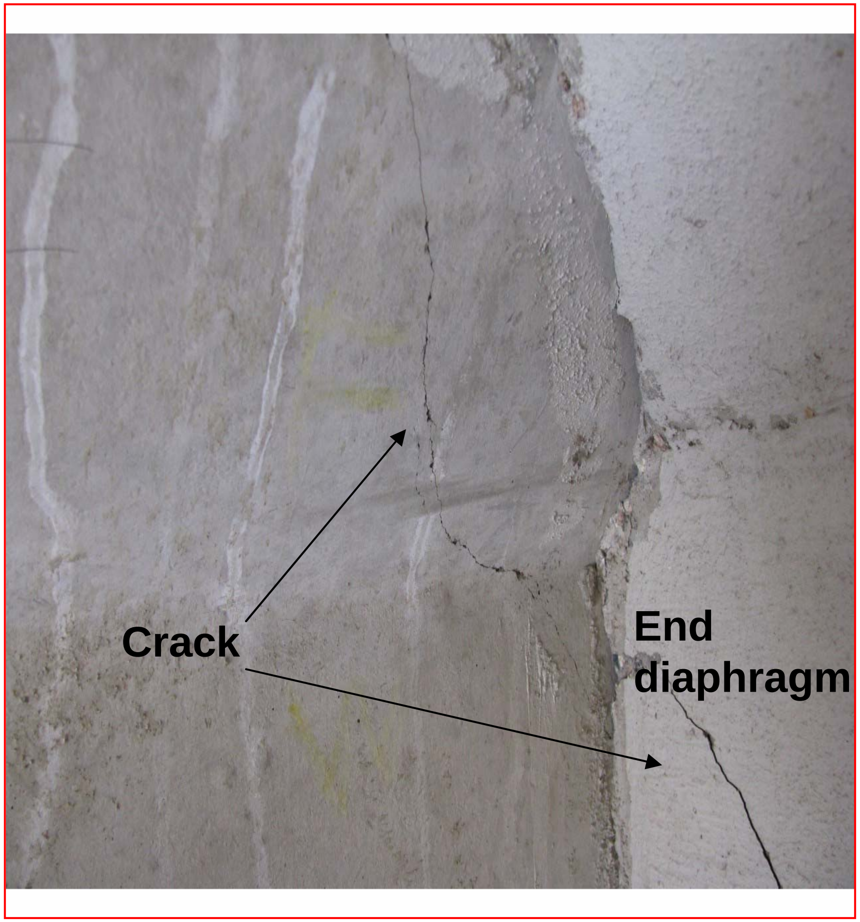
Looking East

Bridge 48009 & 48010

- Spans 1 & 3 of both bridges inspected.
- **Small cracks found at almost every girder (both sides of girder) within 6-inches of the end diaphragm (see photos).**
- Inspection Team was accompanied by Cathy French (U of MN) and Lowell Johnson (Mn/DOT Load Rating Engineer).

Bridge 48009 & 48010





Similar Cracks found on almost all girders at Supports

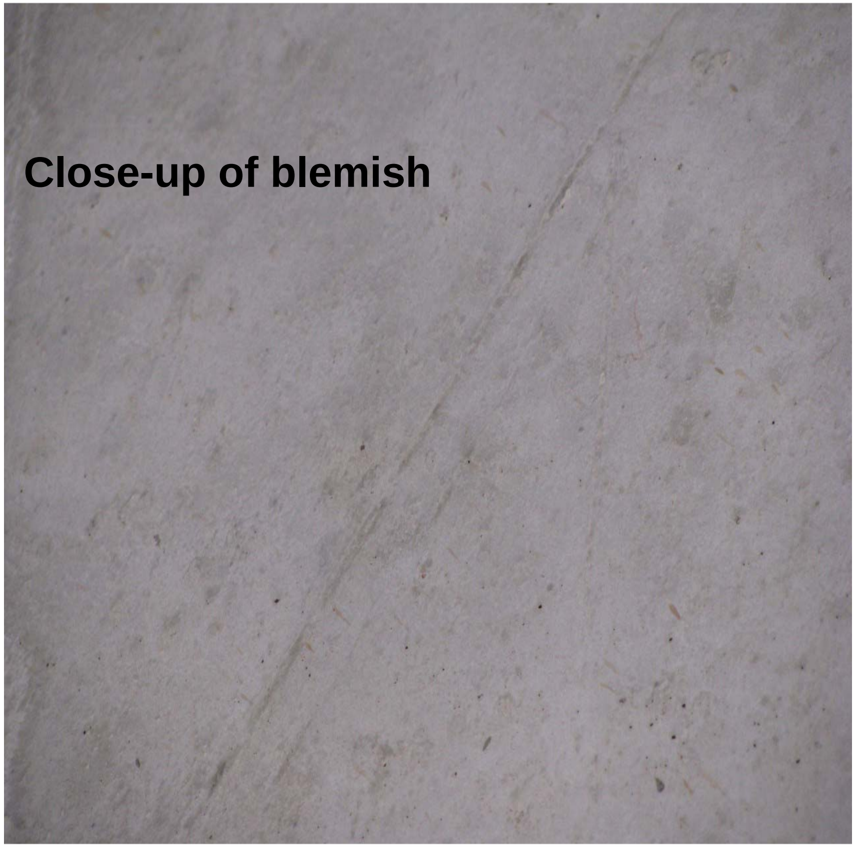
Bridge 48009 & 48010

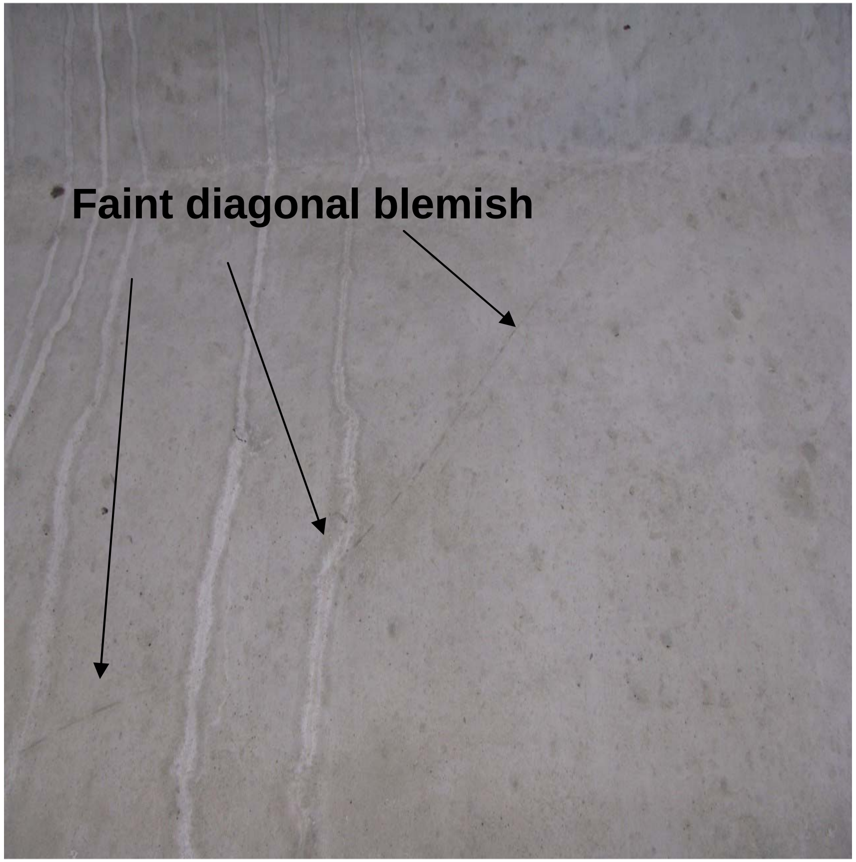
- Cracks near end diaphragms look very similar to cracks found on Bridge 19033.
- Both 48009 & 48010 have Pile Bent Abutments (similar to 19033).
- Recommendation to perform additional inspection with Boom Van to get a “hands-on” look at the opposite ends of the girders where cracks were found (at piers).

Additional Inspection

- With Boom Van Spans 1 and 3 of both bridges inspected at piers (opposite end of beams where cracks were found).
- Vertical blemishes were found on girders due to staining from swallow's nest (see photos).
- A diagonal blemish found on girder 3 (east side) of **49009** and **49010** near pier (see Photos).
- Blemish appears to be from flaw on form work.
- Blemish does not have the appearance of a crack.

Additional Inspection of Bridge 48010





Conclusions of Additional Inspection of 48009 & 48010

- **No cracks were found on girders of Spans 1 & 3 at piers**.
- Blemishes were from staining and most likely a flaw in the form work.

Bridge 73872

- CR 159 over 94.
- Pile Bent Abutments
- “Hands-on” visual inspection performed by walking up the abutments.
- **No diagonal shear cracks found on girders at or near supports**.

Bridge 62860

- 35W over NB Ramp & TH 280 SB.
- Parapet Abutments.
- Due to access issues (heavy traffic area) and full height abutments, unable to perform “hands-on” inspection.
- **Recommendation** – check other bridges if shear cracks are found look into inspecting bridge with snooper and provided traffic control.

Next Step…

- The visual inspection performed on five of the six selected bridges do not show evidence of shear cracks.
- Cracks found on 19033, 48009 and 49010 only at end diaphragms (no cracks were found at opposite ends of beams).
- Inspection team will perform further inspections as necessary if advised by the University of Minnesota.

Attachments

- Sketches of Bridge 19033, 48009, 48010
- Bridge Plans

STATE OF MINNESOTA

DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS

DESIGN OF BR 19033

SHEET NO. ______

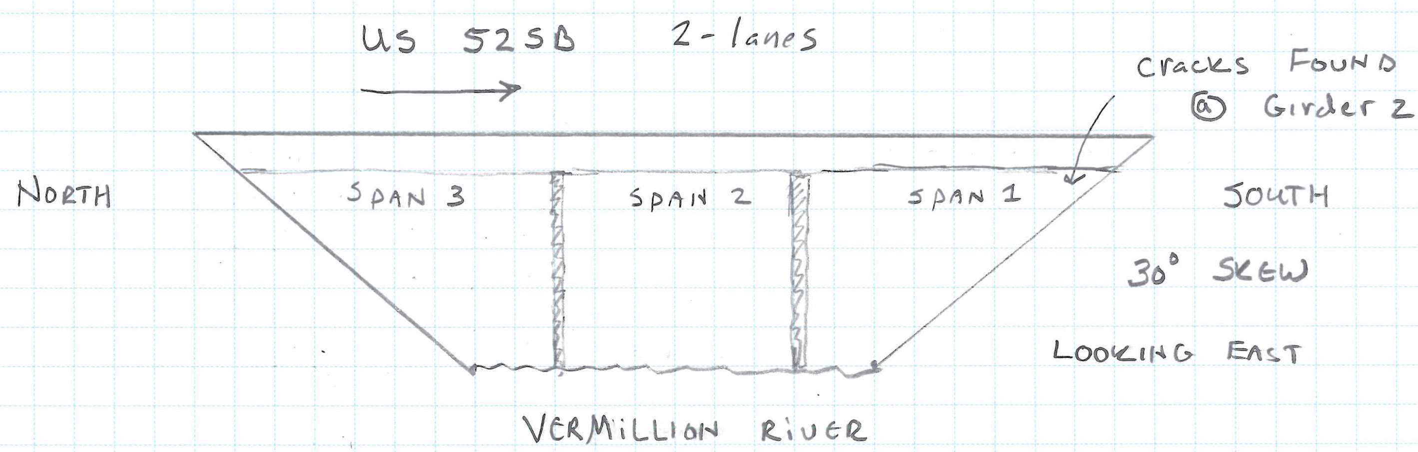
BRIDGE NO. ______

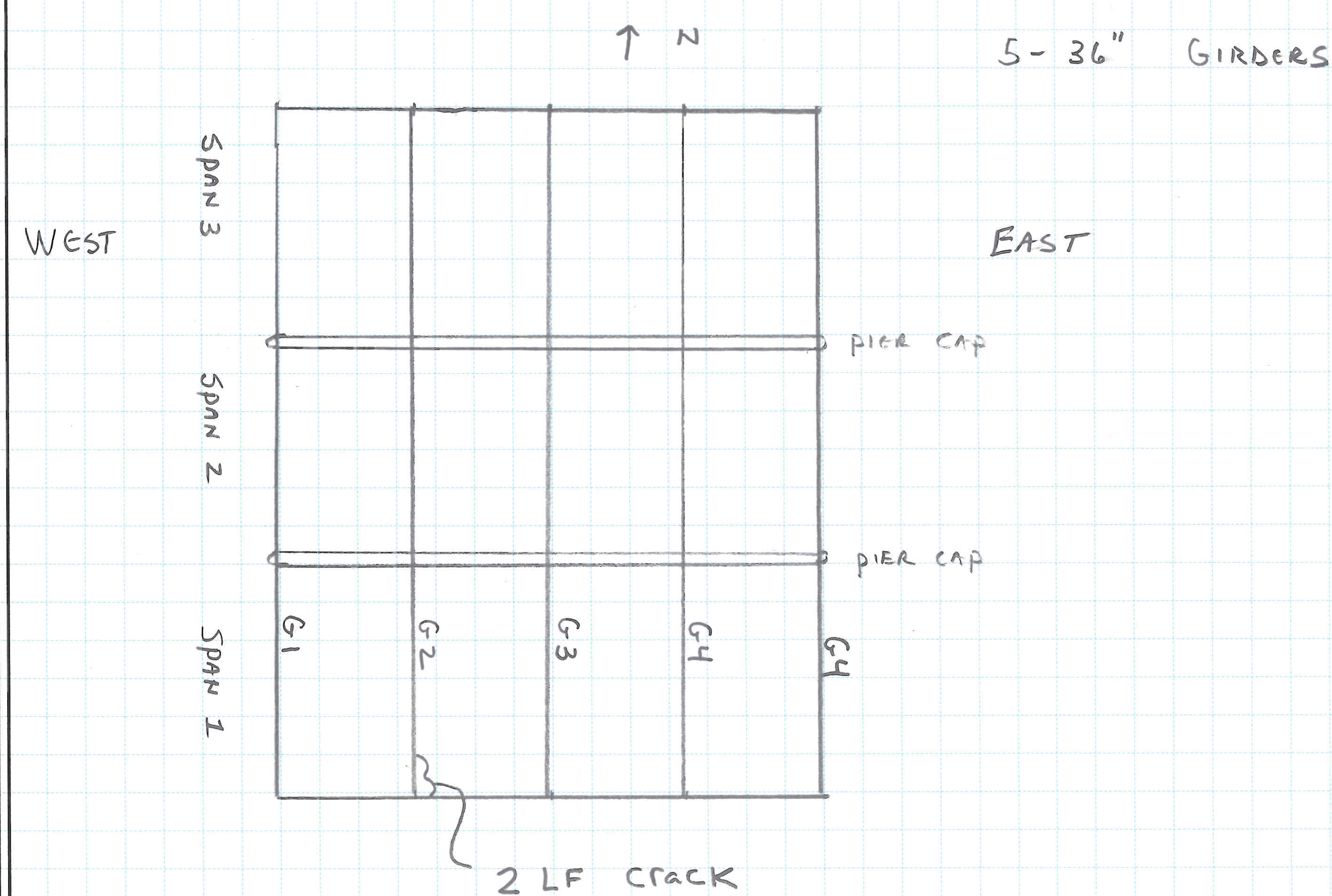
MADE BY. JJ

CHKG BY. ______

DATE. ______

INSPECTION DATE
4-6-09





STATE OF MINNESOTA

DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS

DESIGN OF BR 48010

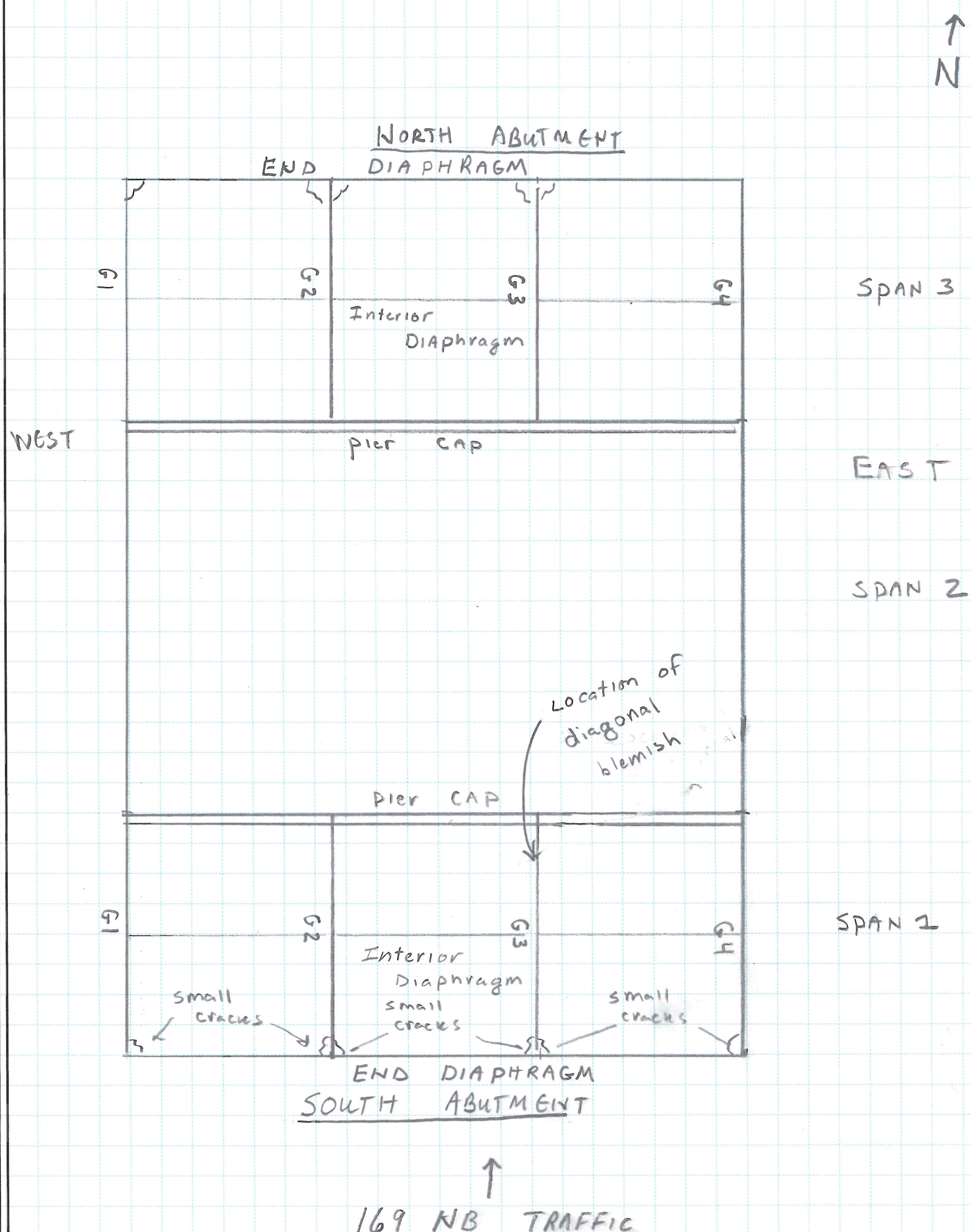
SHEET NO.

BRIDGE NO.

MADE BY.

CHKG BY.

DATE.



STATE OF MINNESOTA

DEPARTMENT OF TRANSPORTATION

DESIGN COMPUTATIONS

SHEET NO.

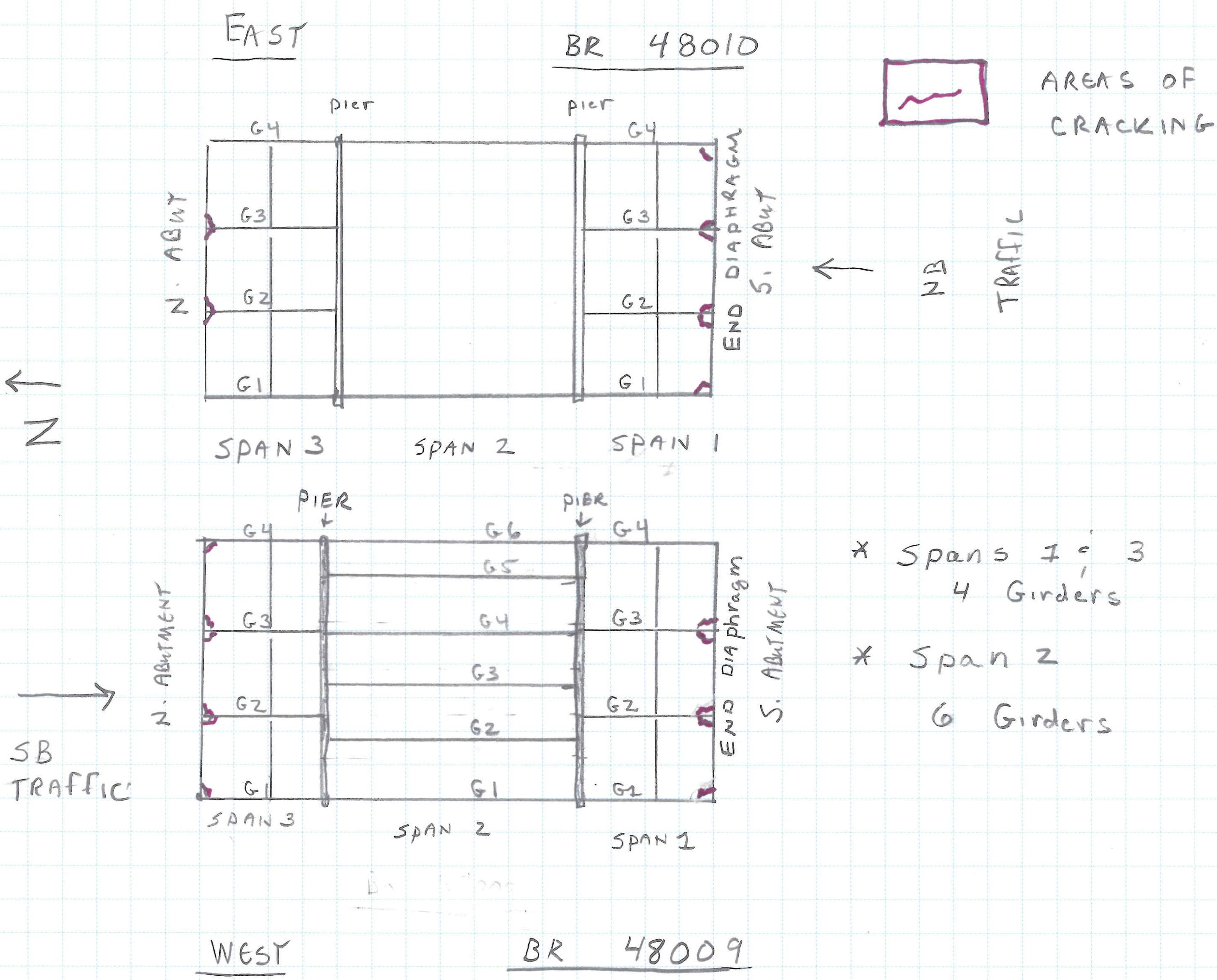
BRIDGE NO.

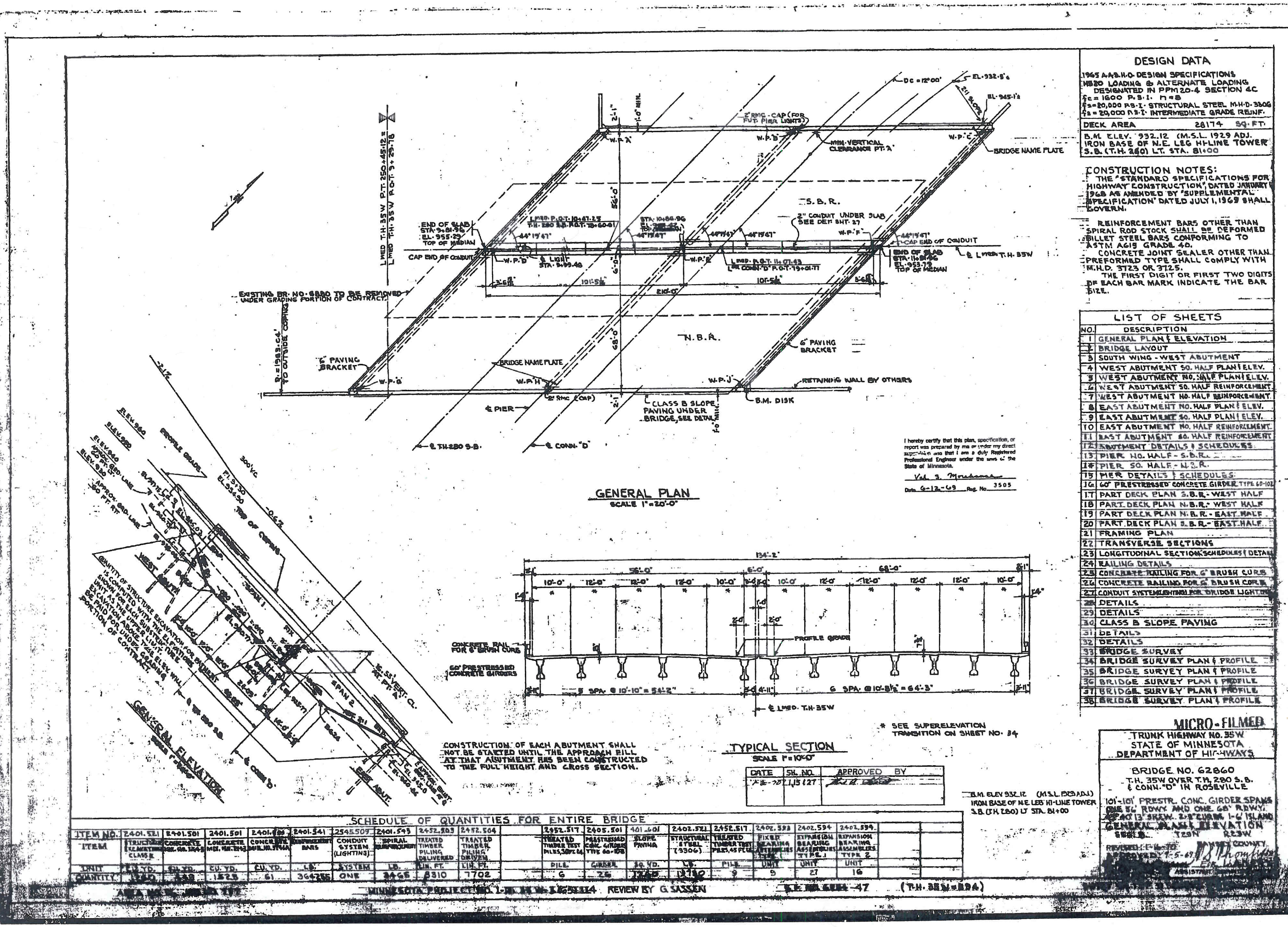
MADE BY.

CHKG BY.

DATE.

DESIGN OF BR 48010 & BR 48009




DESIGN DATA
1965 A.A.S.H.O. DESIGN SPECIFICATIONS
HS20 LOADING & ALTERNATE LOADING DESIGNATED IN PPM 20-4 SECTION 4C
f'c = 1600 P.S.I. n = 8
fs = 20,000 P.S.I. STRUCTURAL STEEL M.H.D. 3306
fs = 20,000 P.S.I. INTERMEDIATE GRADE REINF.
DECK AREA 28174 SQ. FT.
B.M. ELEV. 932.12 (M.S.L. 1929 ADJ. IRON BASE OF N.E. LEG HI-LINE TOWER S.B. (T.H. 280) LT. STA. 81+00
CONSTRUCTION NOTES:
THE "STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION", DATED JANUARY 1968 AS AMENDED BY "SUPPLEMENTAL SPECIFICATION" DATED JULY 1, 1969 SHALL GOVERN.
REINFORCEMENT BARS OTHER THAN SPIRAL ROD STOCK SHALL BE DEFORMED BILLET STEEL BARS CONFORMING TO ASTM A615 GRADE 40.
CONCRETE JOINT SEALER OTHER THAN PREFORMED TYPE SHALL COMPLY WITH M.H.D. 3723 OR 3725.
THE FIRST DIGIT OR FIRST TWO DIGITS OF EACH BAR MARK INDICATE THE BAR SIZE.
LIST OF SHEETS
NO. DESCRIPTION
1 GENERAL PLAN & ELEVATION
2 BRIDGE LAYOUT
3 SOUTH WING - WEST ABUTMENT
4 WEST ABUTMENT SO. HALF PLAN & ELEV.
5 WEST ABUTMENT NO. HALF PLAN & ELEV.
6 WEST ABUTMENT SO. HALF REINFORCEMENT
7 WEST ABUTMENT NO. HALF REINFORCEMENT
8 EAST ABUTMENT NO. HALF PLAN & ELEV.
9 EAST ABUTMENT SO. HALF PLAN & ELEV.
10 EAST ABUTMENT NO. HALF REINFORCEMENT
11 EAST ABUTMENT SO. HALF REINFORCEMENT
12 ABUTMENT DETAILS & SCHEDULES
13 PIER NO. HALF - S.B.R.
14 PIER SO. HALF - N.B.R.
15 PIER DETAILS & SCHEDULES
16 60" PRESTRESSED CONCRETE GIRDER TYPE 60-102
17 PART DECK PLAN S.B.R. - WEST HALF
18 PART DECK PLAN N.B.R. - WEST HALF
19 PART DECK PLAN N.B.R. - EAST HALF
20 PART DECK PLAN S.B.R. - EAST HALF
21 FRAMING PLAN
22 TRANSVERSE SECTIONS
23 LONGITUDINAL SECTION SCHEDULES & DETAILS
24 RAILING DETAILS
25 CONCRETE RAILING FOR 6" BRUSH CURB
26 CONCRETE RAILING FOR 6" BRUSH CURB
27 CONDUIT SYSTEM (LIGHTING) FOR BRIDGE LIGHTING
28 DETAILS
29 DETAILS
30 CLASS B SLOPE PAVING
31 DETAILS
32 DETAILS
33 BRIDGE SURVEY
34 BRIDGE SURVEY PLAN & PROFILE
35 BRIDGE SURVEY PLAN & PROFILE
36 BRIDGE SURVEY PLAN & PROFILE
37 BRIDGE SURVEY PLAN & PROFILE
38 BRIDGE SURVEY PLAN & PROFILE
MICRO-FILMED
TRUNK HIGHWAY NO. 35W
STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS
BRIDGE NO. 62860
T.H. 35W OVER T.H. 280 S.B. & CONN. "D" IN ROSEVILLE
101'-101' PRESTR. CONC. GIRDER SPANS
T29N R23W
GENERAL PLAN
SCALE 1" = 20'-0"
GENERAL ELEVATION
TYPICAL SECTION
SCALE 1" = 10'-0"
S.B.R.
N.B.R.
BRIDGE NAME PLATE
6" PAVING BRACKET
RETAINING WALL BY OTHERS
B.M. DISK
CLASS B SLOPE PAVING UNDER BRIDGE, SEE DETAIL
EXISTING BR. NO. 6880 TO BE REMOVED UNDER GRADING PORTION OF CONTRACT.
2" CONDUIT UNDER SLAB SEE DET. SHT. 27
MIN. VERTICAL CLEARANCE PT. "A"
CONCRETE RAIL FOR 6" BRUSH CURB
60" PRESTRESSED CONCRETE GIRDERS
PROFILE GRADE
5 SPA. @ 10'-10" = 54'-2"
6 SPA. @ 10'-8½" = 64'-3"
* SEE SUPERELEVATION TRANSITION ON SHEET NO. 34
CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED UNTIL THE APPROACH FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE FULL HEIGHT AND CROSS SECTION.
DATE | SH. NO. | APPROVED BY
1,15 & 27
B.M. ELEV 932.12 (M.S.L. 1929 ADJ.) IRON BASE OF N.E. LEG HI-LINE TOWER S.B. (T.H. 280) LT STA. 81+00
SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE
ITEM NO. | 2401.521 | 2401.501 | 2401.501 | 2401.541 | 2545.509 | 2401.543 | 2452.503 | 2452.504 | 2452.517 | 2405.501 | 401.601 | 2402.521 | 2452.517 | 2402.593 | 2402.594 | 2402.594
UNIT | CU. YD. | CU. YD. | CU. YD. | CU. YD. | LB. | SYSTEM | LB. | LIN. FT. | LIN. FT. | PILE | GIRDER | SQ. YD. | LB. | PILE | UNIT | UNIT | UNIT
QUANTITY | 1523 | 61 | 364255 | ONE | 3465 | 8310 | 7702 | 6 | 26 | 9 | 9 | 27 | 16
REVIEW BY G. SASSEN
I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Registered Professional Engineer under the laws of the State of Minnesota.
Date 6-12-69 Reg. No. 3505

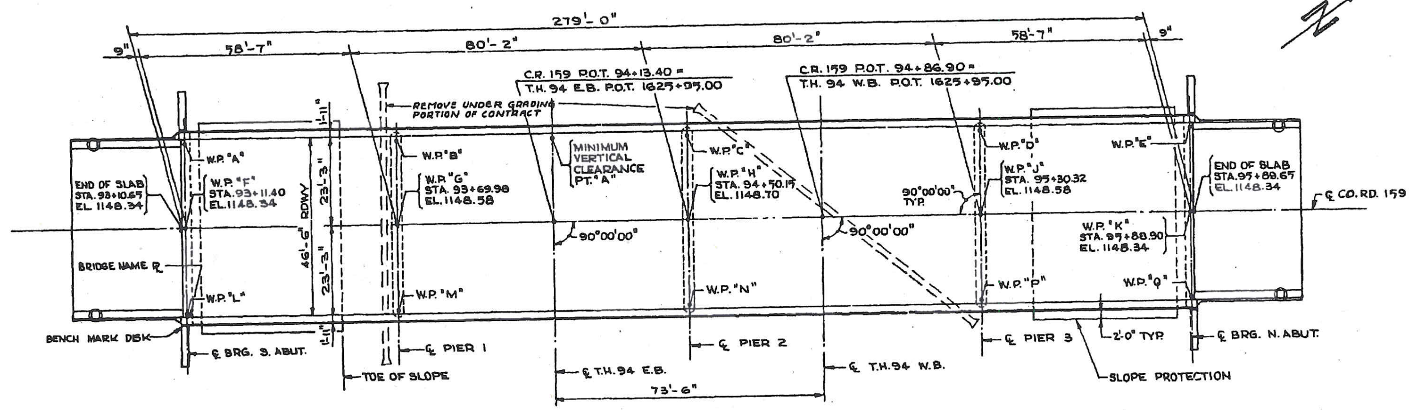
Fed. Proj. No. I 094-2(85)155

DESIGN DATA

1973 AASHTO DESIGN SPECIFICATIONS
LOAD FACTOR DESIGN METHOD
* HS 20 LOADING
MAXIMUM ALLOWABLE DESIGN STRESSES:
REINFORCED CONCRETE:
f'_c = 4000 p.s.i. n=8
f_y = 60000 p.s.i. REINFORCEMENT
PRESTRESSED CONCRETE:
f'_c = 6000 p.s.i. n=6
f'_s = 270000 p.s.i.
DECK AREA 14043 SQ.FT.
640 PROJECTED ADT FOR 1996
* INCLUDES 17 P.S.F. DEAD LOAD ALLOWANCE FOR FUTURE WEARING COURSE.

LIST OF SHEETS

NO	DESCRIPTION
1	GENERAL PLAN AND ELEVATION
2	BRIDGE LAYOUT
3	ABUTMENT DETAILS
4	ABUTMENT REINFORCEMENT
5	PIER 1 AND 3 DETAILS
6	PIER 1 AND 3 REINFORCEMENT
7	PIER 2 DETAILS AND REINFORCEMENT
8	54" PRESTRESSED CONCRETE BEAM TYPE 54-79
9	54" PRESTRESSED CONCRETE BEAM TYPE 54-80
10	FRAMING PLAN
11	SUPERSTRUCTURE DETAILS
12	CONCRETE RAILING (TYPE G)
13	BRIDGE APPROACH TREATMENT
14	BRIDGE APPROACH PANEL
15	STABILIZED AGGREGATE SLOPE PAVING
16	DETAILS
17	DETAILS
18	DETAILS
19	BRIDGE SURVEY
20	BRIDGE SURVEY PLAN AND PROFILE



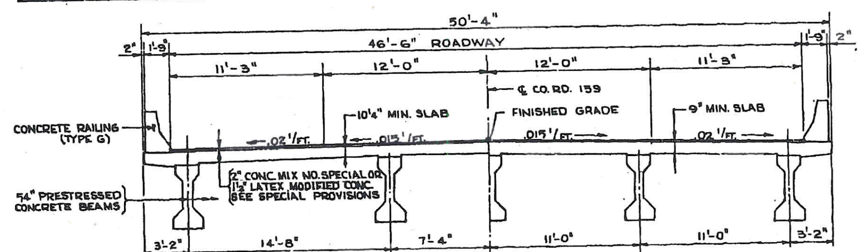
GENERAL PLAN

CONSTRUCTION NOTES

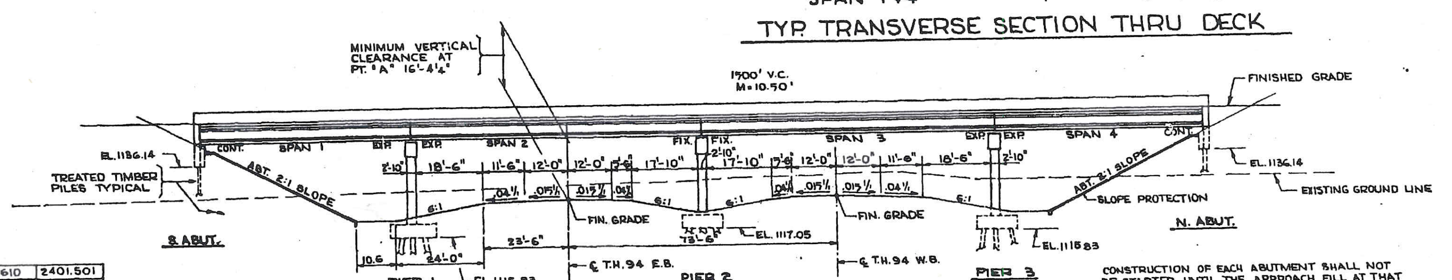
THE MINNESOTA HIGHWAY DEPARTMENT "STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION" DATED JANUARY 1, 1972 SHALL GOVERN AS AMENDED BY "SUPPLEMENTAL SPECIFICATIONS" DATED JANUARY 1, 1974.

BRIDGE SEAT REINFORCEMENT SHALL BE CAREFULLY PLACED TO AVOID INTERFERENCE WITH DRILLING HOLES FOR ANCHOR RODS. THE SUPERSTRUCTURE BEAMS SHALL BE ERECTED IN FINAL POSITION PRIOR TO DRILLING HOLES FOR AND PLACING ANCHOR RODS.

THE FIRST DIGIT OR THE FIRST TWO DIGITS OF EACH BAR MARK INDICATES THE BAR SIZE



TYP. TRANSVERSE SECTION THRU DECK



CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED UNTIL THE APPROACH FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE BOTTOM OF THE APPROACH FILL TREATMENT.

I HEREBY CERTIFY THAT THIS PLAN WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

SIGNED Gary E. Wellhausen
DATE 2-17-76 REG. NO. 8170

B.M. ELEV. 1103.92 (M.S.L. 1929 ADJ.)
STD DISK IN CONC. BOX CULV. 2.8 MI. N.W. ST. JOSEPH

401.610	2401.501
STRUCTURE EXCAVATION	CONCRETE, MIX NO. 3Y46 A.
LUMP SUM	CU. YD.
ONE	70 (P)

SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE

ITEM NO.	2401.501	2401.501	2401.501	2401.511	2401.541	401.621	2401.543	2402.583	2405.501	2405.501
ITEM	CONCRETE, MIX NO. 1A43	CONCRETE, MIX NO. 3Y43	CONCRETE, MIX NO. 3Y33	CONCRETE, MIX NO. SPECIAL	REINFORCEMENT BARS	REINFORCEMENT BAR COATING	SPIRAL REINFORCEMENT	FIXED BEARING ASSEMBLIES TYPE 2	PRESTRESSED CONC. GIRDERS TYPE 54-79	PRESTRESSED CONC. GIRDERS TYPE 54-80
UNIT	CU. YD.	CU. YD.	CU. YD.	SQ. FT.	POUND	LUMP SUM	POUND	EACH	EACH	EACH
QUANTITY	103 (P)	198 (P)	415 (P)	12973 (P)	134390 (P)	ONE	2868 (P)	10	8	10

ITEM NO.	2405.511	2405.511	2452.503	2452.504	2452.517	2452.517	2452.517	2514.503	105.614	301.603	402.601
ITEM	ELASTOMERIC BEARING PAD, TYPE 1	ELASTOMERIC BEARING PAD, TYPE 2	TREATED TIMBER PILING DELIVERED	TREATED TIMBER PILING DRIVEN	TREATED TIMBER TEST PILES, 30 FEET LONG	TREATED TIMBER TEST PILES, 40 FEET LONG	TREATED TIMBER TEST PILES, 50 FEET LONG	AGGREGATE SLOPE PAVING	BRIDGE APPROACH TREATMENT	BRIDGE APPROACH PANELS	EXPANSION ASSEMBLY, TYPE 2.0'
UNIT	EACH	EACH	LIN. FT.	LIN. FT.	EACH	EACH	EACH	SQ. YD.	LUMP SUM	LUMP SUM	LIN. FT.
QUANTITY	18	8	2470	2274	6	2	2	570 (P)	ONE	ONE	93

TRUNK HIGHWAY NO 94
STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS

Bridge No. 73872

T.H. 94 UNDER CO. RD. 159 7.9 MI. N.W. OF JCT. (S.W. OF ST. CLOUD) OF T.H. 23 & T.H. 94. 59'-80'-80'-59' PRESTRESSED CONC. BEAM SPANS 46'-6" ROADWAY (IDENT NO. 501) NO SKEW BARRIER RAILING.

GENERAL PLAN AND ELEVATION

SEC. 36 & 25 T. 125 N. R. 30 W.
AVON TOWNSHIP STEARNS COUNTY

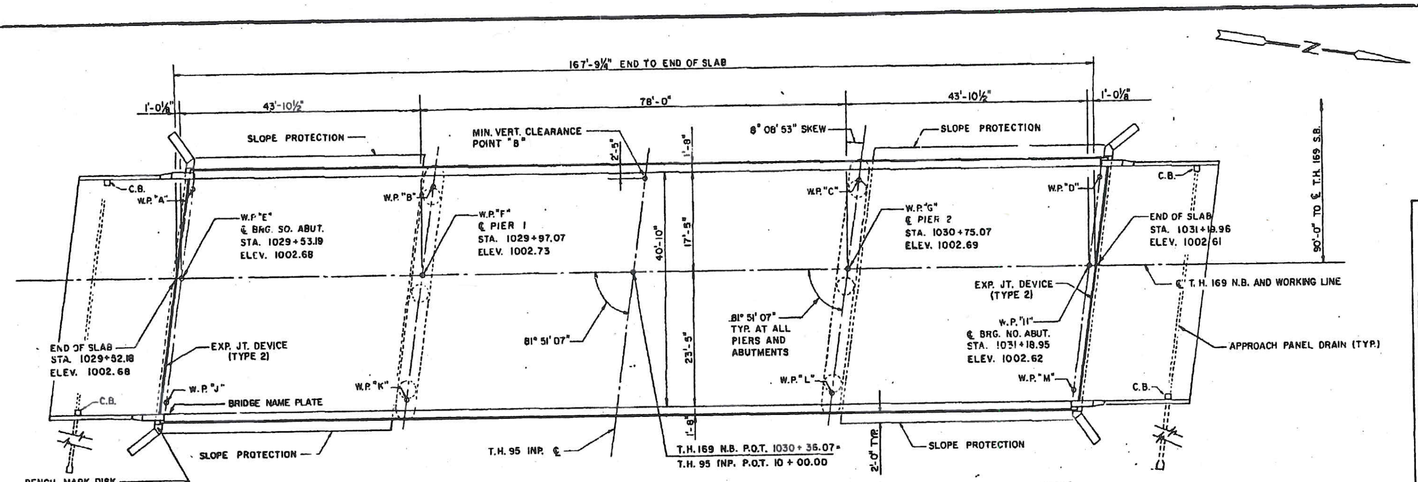
APPROVED 3-17-76
[signature] BRIDGE ENGINEER [signature] DIVISION DIRECTOR

DES. G E W	DR. G.R.F.	73872
CHK. D H	CHK. D.A.H.	

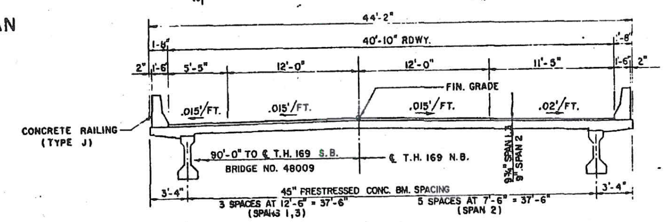
Area No. 35 **Job No. 027** Design Squad G.E. WELLHAUSEN

State Proj. No. 7380-87 (T.H. 94 = 392) **Sheet No. 1 of 20 Sheets**

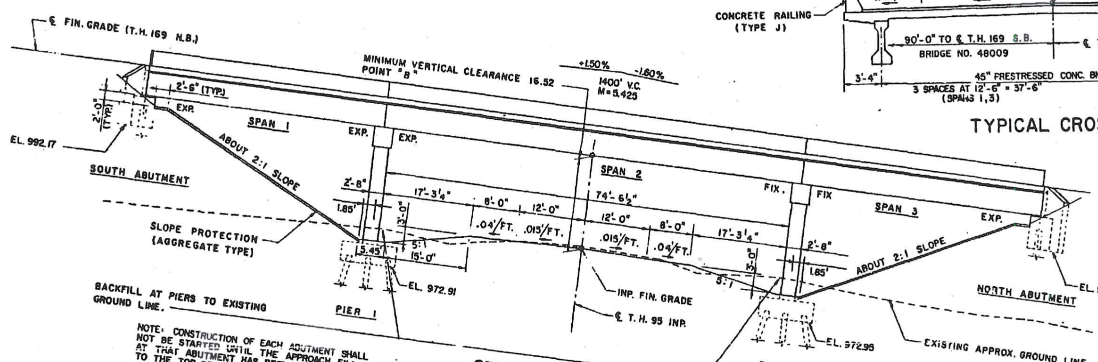
Fed. Proj. No. F 018-1(47)



GENERAL PLAN



TYPICAL CROSS SECTION



GENERAL ELEVATION

NOTE: CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED UNTIL THE APPROACH FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE TOP OF SUBGRADE.

A 30 DAY WAITING PERIOD SHALL BE REQUIRED BETWEEN THE COMPLETION OF THE APPROACH EMBANKMENT CONSTRUCTION AND THE START OF ABUTMENT FOUNDATION CONSTRUCTION.

DESIGN DATA

1977 A.A.S.H.T.O. DESIGN SPECIFICATIONS.
LOAD FACTOR DESIGN METHOD*
DESIGN LOADING HS 20

F'c = 4000 PSI n = 8	REINFORCED CONCRETE
F'c = 6000 PSI n = 6	PRESTRESSED CONCRETE
Fy = 60000 PSI	REINFORCEMENT BARS
F's = 270000 PSI	PRESTRESSING STRANDS

7310 PROJECTED A.D.T. FOR 1998
DECK AREA = 7410 SQ. FT.

* INCLUDES 17 PSF DEAD LOAD ALLOWANCE FOR FUTURE WEARING COURSE

LIST OF SHEETS

NO	DESCRIPTION
1	GENERAL PLAN AND ELEVATION
2	BRIDGE LAYOUT
3-6	NORTH & SOUTH ABUTMENT DETAILS & REINF.
7-8	PIERS 1 & 2
9-11	SUPERSTRUCTURE DETAILS
12-13	45" PRESTRESSED CONCRETE BEAM
14	TYPE J RAILING
15-22	DETAIL SHEETS
23-24	BRIDGE SURVEY

CONSTRUCTION NOTES

THE MINNESOTA DEPARTMENT OF TRANSPORTATION "STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION", 1978 EDITION, SHALL GOVERN.

BRIDGE SEAT REINFORCEMENT SHALL BE CAREFULLY PLACED TO AVOID INTERFERENCE WITH DRILLING HOLES FOR ANCHOR RODS. THE SUPERSTRUCTURE BEAMS SHALL BE ERECTED IN FINAL POSITION PRIOR TO DRILLING HOLES FOR AND PLACING ANCHOR RODS.

THE FIRST DIGIT OR THE FIRST TWO DIGITS OF EACH BAR MARK INDICATE THE BAR SIZE. BARS MARKED WITH THE SUFFIX "E" SHALL BE EPOXY COATED IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

I HEREBY CERTIFY THAT THIS PLAN WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA

SIGNED Max J. Dankenberger
DATE May 1, 1978 REG. NO. 9164

B.M. ELEV. 982.27 (M.S.L. 1975 ADJ.) SPK. IN 18" COTTONWOOD 50' SO. OF INP. T.H. 95 & 160' W. OF N.B. 169 X-ING.

B.M. ELEV 982.27 (M.S.L. 1975 ADJ.)

VAN DOREN-HAZARD-STALLINGS
ARCHITECTS-ENGINEERS-PLANNERS
MINNEAPOLIS, MN. TOPEKA, KS.

TRUNK HIGHWAY NO 169
THE MINNESOTA DEPARTMENT OF TRANSPORTATION

Bridge No. 48010

T.H. 169 N.B. OVER PRESENT T.H. 95 IN PRINCETON
43'-7 8'-43' PRESTRESSED CONCRETE BEAM SPANS 40'-10" RDWY. 8°-08'-53" SKEW TWO CONC. RAIL TYPE J
IDENTIFICATION NO. 501

GENERAL PLAN AND ELEVATION

SEC 32 TWP 36 R 26
PRINCETON TOWNSHIP MILLE LACS COUNTY

APPROVED 9-1-78
K.V. Benthin
for DIRECTOR ENGINEERING SERVICES

DES. M.J.D.	CH. D.H.P.		48010
DR. T.A.C.	CH. M.J.D.		

SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE

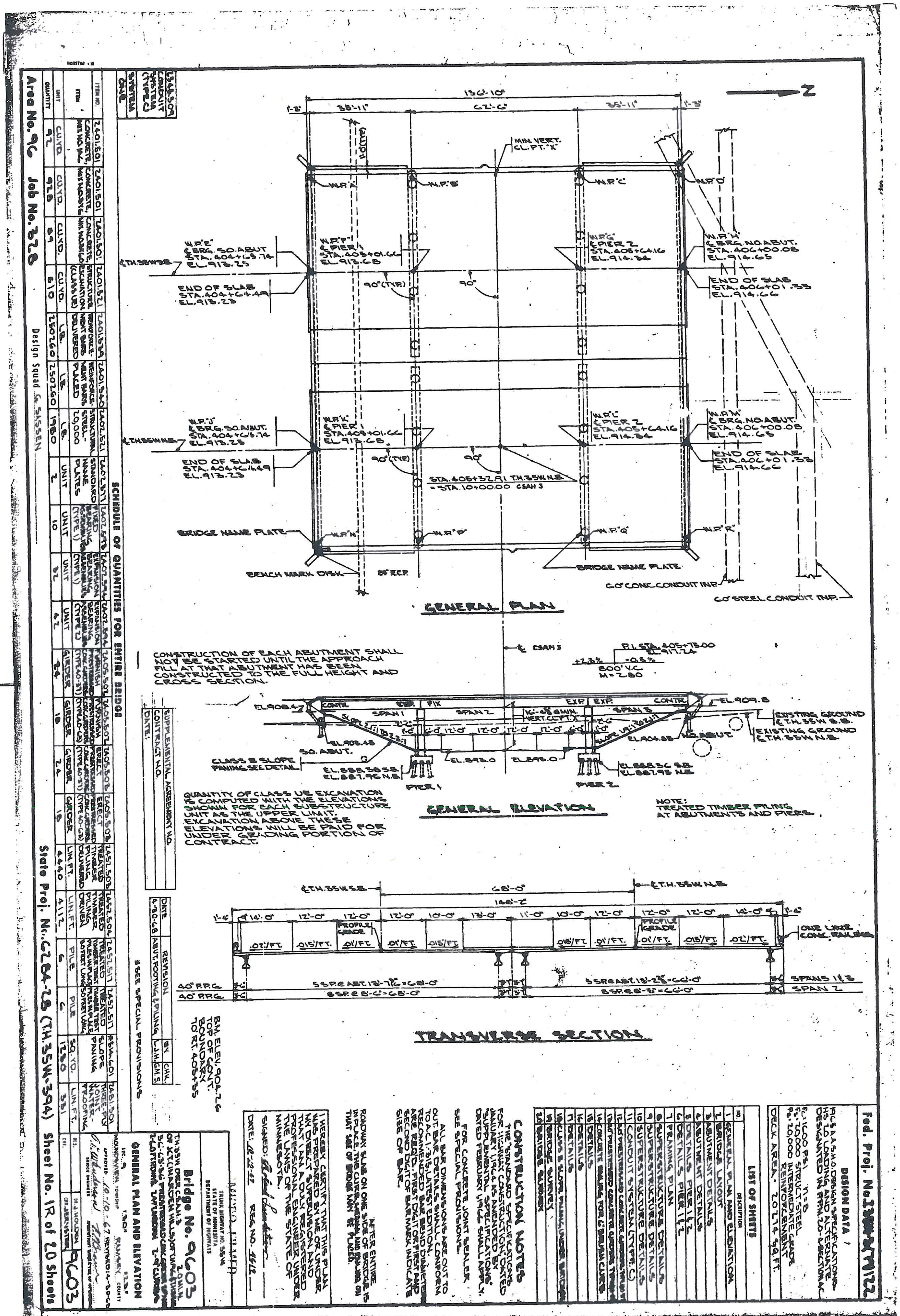
2401.501	2401.501	401.605	401.605	401.606	401.601	2401.541	2401.541	2401.543	2402.590	2402.590	2402.592	2452.503	2452.504	2452.517	2452.517	2452.517
CONCRETE MIX NO. 1A43	CONCRETE MIX NO. 3Y43	BRIDGE SLAB CONCRETE, MIX NO. 3X33	WEARING COURSE, CONC. MIX NO. SPECIAL	TYPE J RAILING CONCRETE, MIX NO. 3X46A	STRUCTURE EXCAVATION	REINFORCEMENT BARS	REINFORCEMENT BARS (EPOXY COATED)	SPIRAL REINFORCEMENT	ELASTOMERIC BEARING PAD TYPE 1	ELASTOMERIC BEARING PAD TYPE 2	ELASTOMERIC BRG. ASSEMBLY (TYPE 1)	TREAT. TIMBER PILING DELIVERED	TREAT. TIMBER PILING DRIVEN	TREAT. TIMBER TEST PILES 30 FT. LONG	TREAT. TIMBER TEST PILES 45 FT. LONG	TREAT. TIMBER TEST PILES 50 FT. LONG
CU. YD.	CU. YD.	SQ. FT.	SQ. FT.	LIN. FT.	LUMP SUM	POUND	POUND	POUND	EACH	EACH	EACH	LIN. FT.	LIN. FT.	EACH	EACH	EACH
66 (P)	168 (P)	7410(P)	6850 (P)	331 (P)	1	59930 (P)	30830 (P)	2496(P)	8	10	10	1324	1216	2	2	2

2452.517	2405.501	2402.591	2405.501	2405.511	105.608	301.601	2514.503
TREAT. TIMBER TEST PILES 25 FT. LONG	PRESTRESSED CONC. BEAMS TYPE 45-45	EXPANSION JOINT DEVICE TYPE 2	PRESTRESSED CONC. BEAMS TYPE 45-78	DIAPHRAGMS FOR TYPE 45 PRESTR. BEAMS	BRIDGE APPROACH TREATMENT	BRIDGE APPROACH PANELS	AGGREGATE SLOPE PAVING
EACH	EACH	LIN. FT.	EACH	LIN. FT.	LUMP SUM	LUMP SUM	SQ. YD.
2	8	87 (P)	6	242 (P)	1	1	480 (P)

Area No. 35 Job No. 172 Mn/DOT REVIEW BY R.J. MILLER DESIGN GROUP

State Proj. No. 4811-48009 (T.H. 169=18) Sheet No. 1 of 24 Sheets

9-5-78 AWS


GENERAL PLAN
GENERAL ELEVATION
TRANSVERSE SECTION
SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE
CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED UNTIL THE APPROACH FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE FULL HEIGHT AND CROSS SECTION.
NOTE: TREATED TIMBER PILING AT ABUTMENTS AND PIERS.
CONSTRUCTION NOTES
LIST OF SHEETS
DESIGN DATA
Bridge No. 9603
GENERAL PLAN AND ELEVATION
Area No. 96 Job No. 328
State Proj. No. 6284-28 (T.H. 35W-394) Sheet No. 1R of 20 Sheets

Minn. Proj. No. IX5003 (7)

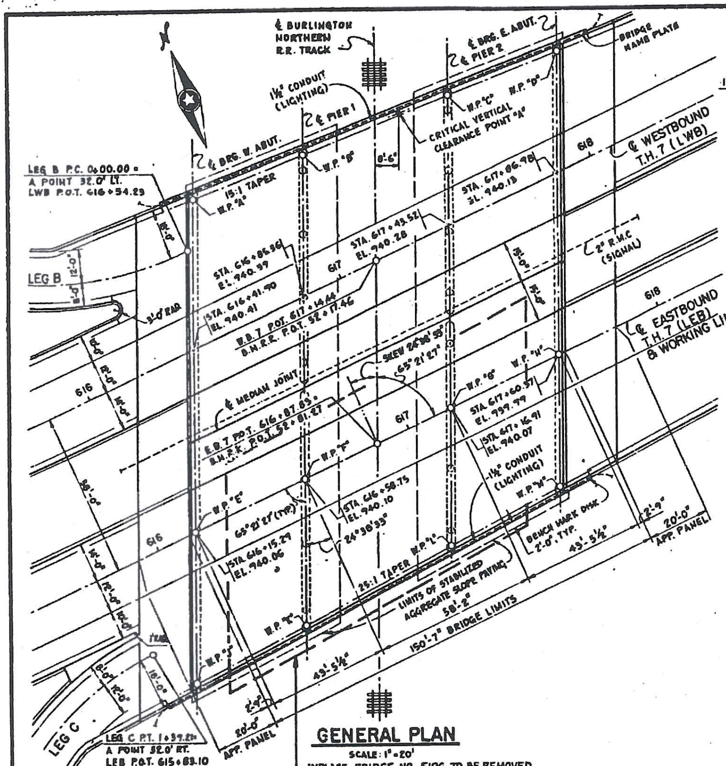
DESIGN DATA

1977 AND INTERIM A.A.S.H.T.O. DESIGN SPECIFICATIONS LOAD FACTOR DESIGN METHOD - HS 20 LOADING INCLUDES 17.5 PSF DEAD LOAD ALLOWANCE FOR FUTURE WEARING COURSE MODIFICATION.
MAXIMUM ALLOWABLE DESIGN STRESSES:
REINFORCED CONCRETE: f'_c = 4,000 P.S.I. (N-8); f_y = 60,000 P.S.I. (GR.60)
PRESTRESSED CONCRETE: f'_c = 6,000 P.S.I. (N-6); f'_s = 270,000 P.S.I.

DECK AREA = 22,935 SQ.FT.

LIST OF SHEETS

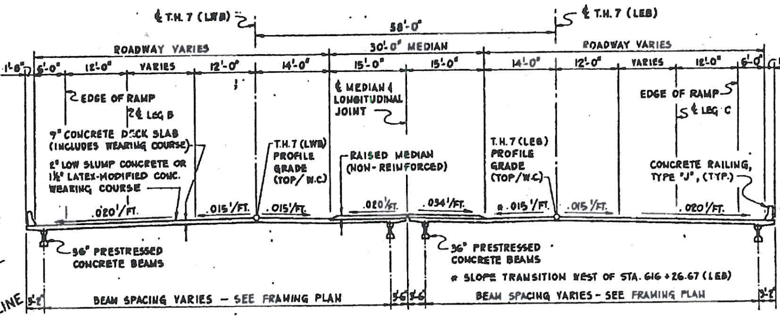
NO.	DESCRIPTION
1	GENERAL PLAN AND ELEVATION
2	BRIDGE LAYOUT
3	WEST ABUTMENT
4	WEST ABUTMENT
5	WEST ABUTMENT DETAILS
6	EAST ABUTMENT
7	EAST ABUTMENT
8	EAST ABUTMENT DETAILS
9	PIER 1N
10	PIER 1S
11	PIER 2N
12	PIER 2S
13	MISC. SUBSTRUCTURE DETAILS
14	36" PRESTRESSED CONCRETE BEAM, TYPE 36-45
15	36" PRESTRESSED CONCRETE BEAM, TYPE 36-60
16	FRAMING PLAN - NORTH
17	FRAMING PLAN - SOUTH
18	DECK PLAN OF NORTH SLAB
19	DECK PLAN OF SOUTH SLAB
20	RAILING ELEVATIONS, SUPERSTRUCTURE QUANTITIES
21	CONCRETE RAILING (TYPE "J")
22	WATERPROOF EXPANSION DEVICE
23	WATERPROOF EXPANSION DEVICE
24	BRIDGE APPROACH PANEL
25	STABILIZED AGGREGATE SLOPE PAVING
26	CONDUIT SYSTEM FOR LIGHTING
27	STD. DETAILS B101 AND B201
28	STD. DETAILS B306 AND B305
29	STD. DETAILS B303 AND B403
30	STD. DETAILS B802 AND B803
31	STD. DETAIL B801
32	BRIDGE SURVEY
33	BRIDGE SURVEY PLAN AND PROFILE
34	BRIDGE SURVEY PLAN AND PROFILE



GENERAL PLAN

SCALE: 1" = 20'

INPLACE BRIDGE NO. 5196 TO BE REMOVED



TYPICAL SECTION

SCALE: 3/16" = 1'-0"

GOVERNING SPECIFICATIONS:

THE 1978 EDITION OF THE MINNESOTA DEPARTMENT OF TRANSPORTATION "STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION" SHALL GOVERN.

REINFORCEMENT BARS:

REINFORCEMENT BARS SHALL BE DEFORMED BILLET STEEL BARS CONFORMING TO Mn/DOT 3301.

ALL REINFORCEMENT BARS ARE DESIGNATED ON THE PLANS BY BAR MARKS. THE LETTER INDICATES THE LOCATION AND THE FIRST DIGIT OF A THREE-DIGIT BAR MARK, OR THE FIRST TWO DIGITS OF A FOUR-DIGIT BAR MARK, INDICATE THE BAR SIZE.

BAR DETAILS SHALL CONFORM TO A.C.I.-315, LATEST EDITION. ALL BENT BAR DIMENSIONS ARE GIVEN OUT-TO-OUT.

ALL BARS OF A SERIES SHALL VARY BY A CONSTANT INCREMENT. THE SHORTEST AND THE LONGEST BAR LENGTHS OF A SERIES GROUP ARE TABULATED IN THE BAR LIST.

THE CLEAR DISTANCE BETWEEN REINFORCEMENT BARS AND FACE OF CONCRETE SHALL BE 3" IN FOOTINGS AND 2" ELSEWHERE, UNLESS OTHERWISE NOTED.

BARS MARKED WITH SUFFIX "E" SHALL BE EPOXY COATED IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

CONSTRUCTION NOTES:

BRIDGE SEAT REINFORCEMENT SHALL BE CAREFULLY PLACED TO AVOID INTERFERENCE WITH DRILLING HOLES FOR ANCHOR RODS. THE SUPERSTRUCTURE GIRDERS (CONC. BEAMS) SHALL BE ERECTED IN FINAL POSITION PRIOR TO DRILLING HOLES FOR AND PLACING ANCHOR RODS.

CONSTRUCTION OF EACH ABUTMENT SHALL NOT BE STARTED UNTIL 30 DAYS AFTER THE EMBANKMENT FILL AT THAT ABUTMENT HAS BEEN CONSTRUCTED TO THE BOTTOM OF THE APPROACH FILL TREATMENT.

ALL EXPOSED EDGES OF CONCRETE SHALL BE CHAMFERED ¾" UNLESS OTHERWISE NOTED.

TERMINATE AND CAP ALL CONDUIT WITH 18" COVER.

DRAINAGE AND EROSION CONTROL

RESTORE SIDE DITCHES AFTER PLACEMENT OF SLOPE PAVING TO PROVIDE DRAINAGE AS DIRECTED BY ENGINEER. RESTORATION COSTS SHALL BE INCLUDED IN PRICE BID FOR STRUCTURE EXCAVATION.

OTHER NOTES

PILE NOTES .... SHEET 13
SUPERSTRUCTURE .... SHEETS 16 & 19

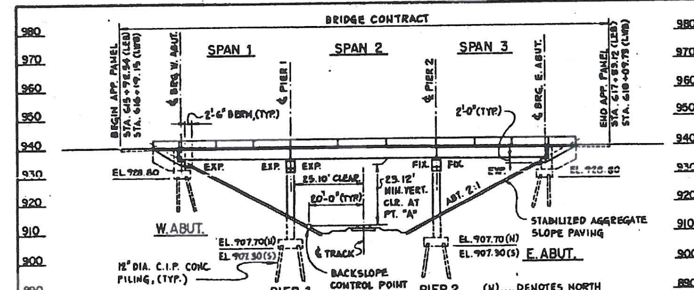
QUANTITY SUMMARIES

WEST ABUTMENT .... SHEET 5
EAST ABUTMENT .... SHEET 6
PIERS .... SHEET 19
SUPERSTRUCTURE .... SHEET 20

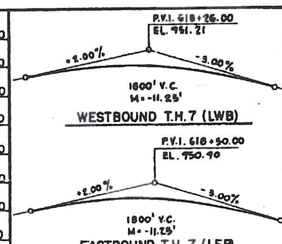
LEGEND:

(SPEC) ... DENOTES MINNESOTA DEPARTMENT OF TRANSPORTATION CONSTRUCTION SPECIFICATIONS REFERENCE.

(T) ..... TOP	(TYP.) .... TYPICAL	(N.F.) .... NEAR FACE
(E.F.) ... EACH FACE	(SPS.) .... SPACES	(SER.) .... SERIES
(F.F.) ... FAR FACE	(B) ...... BOTTOM	(W.P.) .... WORKING POINT



GENERAL ELEVATION



PROFILE GRADE

BENCH MARK: ELEVATION 907.34 (M.S.L. 1929 ADJ.) STD. M.H.D. DISK "2706-A" S.E. CORNER BR. NO. 5197 T.H. 7 OVER MINNEHAHA CREEK

SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE

ITEM NO.	2401.501	2401.501	2401.541	2401.541	2401.545	2545.509	2402.590	2402.590	2402.590	2402.591	2402.592	2405.501	2405.501	2405.511	2452.507	2452.508	2452.519	2452.519	2514.505	2545.509	501.601
ITEM	CONCRETE, MIX NO. 3Y43	CONCRETE, MIX NO. 1A43	REINFORCEMENT BARS	REINFORCEMENT BARS, (EPOXY-COATED)	SPIRAL REINFORCEMENT	CONDUIT SYSTEM (SIGNALS)	ELASTOMERIC BEARING PAD, TYPE 1	ELASTOMERIC BEARING PAD, TYPE 2	ELASTOMERIC BEARING PAD, TYPE 3	EXPANSION JOINT DEVICES, TYPE 3	ELASTOMERIC BEARING ASSEMBLY, TYPE 1	PRESTRESSED CONCRETE BEAMS, TYPE 36-45	PRESTRESSED CONCRETE BEAMS, TYPE 36-60	DIAPHRAGMS FOR TYPE 36 PRESTRESSED BEAMS	CAST-IN-PLACE CONCRETE PILING, DELIVERED	CAST-IN-PLACE CONCRETE PILING, DRIVEN	CAST-IN-PLACE CONC. TEST PILES, 50 FT. LONG	CAST-IN-PLACE CONC. TEST PILES, 55 FT. LONG	AGGREGATE SLOPE PAVING	CONDUIT SYSTEM (LIGHTING)	BRIDGE APPROACH PANELS
UNIT	CU.YD.	CU.YD.	POUND	POUND	POUND	LUMP SUM	EACH	EACH	EACH	LIN.FT.	EACH	EACH	EACH	LIN.FT.	LIN.FT.	LIN.FT.	EACH	EACH	SQ.YD.	LUMP SUM	LUMP SUM
QUANTITY	538 (P)	349 (P)	152,490 (P)	79,300 (P)	6800 (P)	ONE	16	38	90	330 (P)	24	32	22	974 (P)	5,700	5,700	17	5	2,160 (P)	ONE	ONE

QUANTITIES, CONT'D

401.604	401.601	401.605	401.606	401.605	401.606	2442.501	104.607
NEOPRENE WATERSTOP	STRUCTURE EXCAVATION	BRIDGE SLAB CONC., MIX NO. 3X33	WEARING COURSE CONC., MIX NO. SPECIAL	RAISED MEDIAN CONC., MIX NO. 3Y46	TYPE "J" RAILING CONC. MIX NO. 3X46	REMOVE OLD BRIDGE NO. 5196	HAUL SALVAGED MATERIAL
LIN.FT.	LUMP SUM	SQ.FT.	SQ.FT.	SQ.FT.	LIN.FT.	LUMP SUM	LUMP SUM
151 (P)	ONE	22,927 (P)	17,854 (P)	4,498 (P)	335 (P)	ONE	ONE

I HEREBY CERTIFY THAT THIS PLAN WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

SIGNED Roger O. Cole

DATE 6/30/80 REG. NO. 8715

BANISTER SHORT ELLIOTT HENDRICKSON BALLOU & McFARLIN, INC. CONSULTING ENGINEERS
7505 WEST HIGHWAY SEVEN
MINNEAPOLIS, MINNESOTA 55426

TRUNK HIGHWAY NO. 7
STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

Bridge No. 27068

T.H. 7 OVER BURLINGTON NORTHERN INC.
0.1 MI. E. OF JCT. OF C.S.A.H. 18 AND T.H. 7 IN HOPKINS. 44'-58'-44' PRESTRESSED CONC. BEAM SPANS; 30' RAISED MEDIAN.
SPAN I.D. NO. 501

GENERAL PLAN AND ELEVATION

SEC. 19 T. 117 N. R. 21 W. HENNEPIN COUNTY
TOWNSHIP

APPROVED 7-17-80

BRIDGE ENGINEER

DIRECTOR ENGINEERING SERVICES

DES. R.O.C.	DR. G.M.	27068
CHK. RLD	CHK. R.O.C.	

Area No. 57 Job No. 317

REVIEWED BY R. J. MILLER DESIGN GROUP

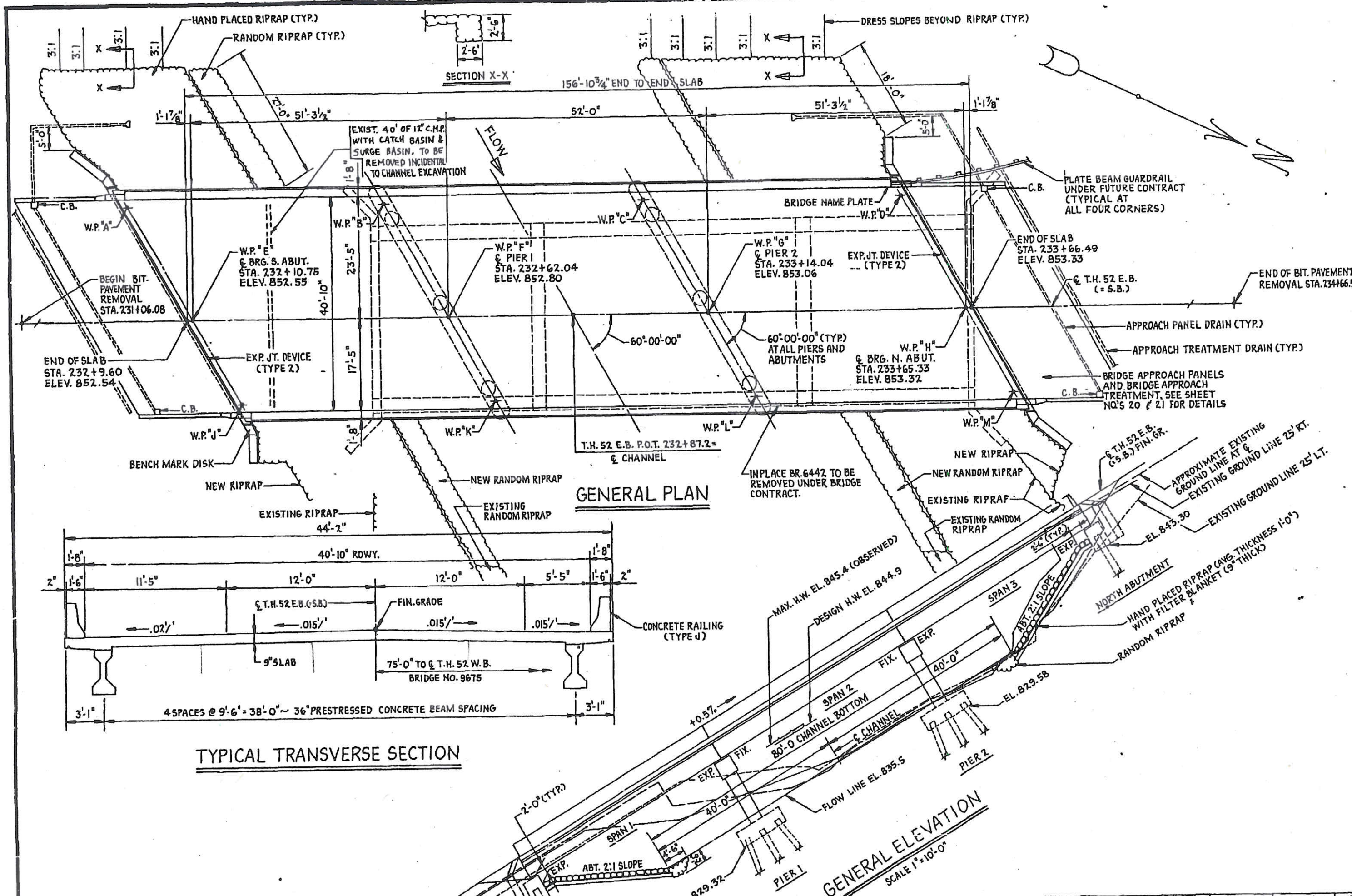
State Proj. No. 27-600-03

Sheet No. 1 of 34 Sheets

BKBM Job # 79314

Form 40-16

MnDOT Bridge Dorothy Hanson 7/4/ Scott Hei 8-5-85



Fed. Proj. No. STATE FUNDS CHAP 339

DESIGN DATA

1973 A.A.S.H.T.O. DESIGN SPECIFICATIONS
DESIGN LOADING HS20
LOAD FACTOR DESIGN METHOD
f'_c = 4000 P.S.I. n=8
f_y = 60000 P.S.I. REINFORCEMENT

6000 PROJECTED A.D.T. FOR 1997

DECK AREA = 6930 SQ. FT.

LIST OF SHEETS

NO.	DESCRIPTION
1	GENERAL PLAN AND ELEVATION
2	BRIDGE LAYOUT
3-6	NORTH AND SOUTH ABUTMENTS DETAILS & REINF.
7-8	PIERS 1 & 2
9-12	SUPERSTRUCTURE DETAILS
13	36" PRESTRESSED CONCRETE BEAM
14	TYPE J RAILING
15-21	DETAIL SHEETS
22-23	BRIDGE SURVEY

CONSTRUCTION NOTES

THE MINNESOTA HIGHWAY DEPARTMENT "STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION" DATED JANUARY 1, 1972 SHALL GOVERN AS AMENDED BY "SUPPLEMENTAL SPECIFICATIONS" DATED APRIL 1, 1976.

BRIDGE SEAT REINFORCEMENT SHALL BE CAREFULLY PLACED TO AVOID INTERFERENCE WITH DRILLING HOLES FOR ANCHOR RODS. THE SUPERSTRUCTURE BEAMS SHALL BE ERECTED IN FINAL POSITION PRIOR TO DRILLING HOLES FOR AND PLACING ANCHOR RODS.

THE FIRST DIGIT OR THE FIRST TWO DIGITS OF EACH BAR MARK INDICATE THE BAR SIZE. BARS MARKED WITH THE SUFFIX "E" SHALL BE EPOXY COATED IN ACCORDANCE WITH THE SPECIAL PROVISIONS.

I HEREBY CERTIFY THAT THIS PLAN WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

SIGNED Conrad S. Hovland
DATE 10-5-77 REG. NO. 6972

B.M. ELEV. 851.419 (M.S.L. 1929 ADJ.)
M.H.D. MON. NO. 1906E-1 S.E. COR. E.B. BR. NO. 6442 L.W.B. T.H. 52 232+51 57' LT.
B.M. ELEV. 851.012 (M.S.L. 1929 ADJ.) STD. M.H.D. BRASS PLUG S.E. COR. W.B. BR. 9675 ABUT. L.W.B. T.H. 52 232+72 28' RT.

TRUNK HIGHWAY NO. 52
MINNESOTA
DEPARTMENT OF TRANSPORTATION

Bridge No. 19033

T.H. 52 E.B. OVER VERMILLION RIVER 4.2 MILES NO. OF JUNCTION (IN HAMPTON) OF T.H. 50 & T.H. 52
51'-52'-51' PRESTRESSED CONCRETE BEAM SPANS 40'-10" RDWY. 30°00'00" SKEW TWO CONC. RAIL. (TYPE J)
IDENTIFICATION NO. 501

GENERAL PLAN AND ELEVATION

SEC. 20 T. 114 N. R. 18 W.
HAMPTON TOWNSHIP DAKOTA COUNTY

APPROVED 10-6-77
[signature] BRIDGE ENGINEER [signature] DIRECTOR, DESIGN SERVICES

DES. B. IWEN	DR. D. BENSON	19033
CHK. M. ALI	CHK. T. JORDET	

SCHEDULE OF QUANTITIES FOR ENTIRE BRIDGE

ITEM NO.	ITEM	UNIT	QUANTITY
2021.501	MOBILIZATION	LUMP SUM	1
2031.501	FIELD OFFICE TYPE D	EACH	1
2105.511	COMMON CHANNEL EXCAVATION	CU. YD.	2000
2104.521	SALVAGE PLATE BEAM GUARDRAIL	LIN. FT.	330 (P)
2104.505	REMOVE BITUMINOUS SURFACING	SQ. YD.	645
401.616	FOUNDATION PREPARATION PIER 2	LUMP SUM	1
2511.504	FILTER BLANKET, TYPE II	CU. YD.	160
401.616	FOUNDATION PREPARATION, PIER 1	LUMP SUM	1
402.601	EXPANSION JOINT DEVICE, TYPE 2	LIN. FT.	95 (P)
405.601	PRESTRESSED CONCRETE BM'S, TYPE 36-52	EACH	15
405.610	DIAPHRAGMS FOR TYPE 36 PRESTR. BEAMS	LIN. FT.	266 (P)
105.614	BRIDGE APPROACH TREATMENT	LUMP SUM	1
301.603	BRIDGE APPROACH PANELS	LUMP SUM	1
2401.501	CONCRETE, MIX. NO. 3X33A	CU. YD.	24 (P)
2401.501	CONCRETE, MIX. NO. 1A43	CU. YD.	66 (P)
2401.501	CONCRETE, MIX NO. 3Y43	CU. YD.	159 (P)
401.605	DECK SLAB CONCRETE, MIX. NO. 3X33A	SQ. FT.	6930 (P)
401.606	TYPE J RAILING CONCRETE, MIX NO. 3X46A	LIN. FT.	308 (P)
401.610	STRUCTURE EXCAVATION	LUMP SUM	1
2401.541	REINFORCEMENT BARS	POUND	49360 (P)
2401.541	REINFORCEMENT BARS (EPOXY COATED)	POUND	31130 (P)
2401.543	SPIRAL REINFORCEMENT	POUND	1518 (P)
2405.511	ELASTOMERIC BEARING PAD, TYPE 1	EACH	10
2405.511	ELASTOMERIC BEARING PAD, TYPE 2	EACH	10
402.625	ELASTOMERIC BRG. ASSEMBLIES, TYPE I	EACH	10
2442.501	REMOVE OLD BRIDGE	LUMP SUM	1
2452.503	TREATED TIMBER PILING DELIVERED	LIN. FT.	1000
2452.504	TREATED TIMBER PILING DRIVEN	LIN. FT.	960
2452.517	TREATED TIMBER TEST PILES, 60 FEET LONG	EACH	4
2452.503	UNTREATED TIMBER PILING DELIVERED	LIN. FT.	1520
2452.504	UNTREATED TIMBER PILING DRIVEN	LIN. FT.	1444
2452.517	UNTREATED TIMBER TEST PILES, 50 FEET LONG	EACH	4
2511.501	RANDOM RIPRAP, CLASS B	CU. YD.	125
2511.502	HAND-PLACED RIPRAP, CLASS B	CU. YD.	230

Area No. 95 **Job No.** 782 Design Squad C. HOVLAND **State Proj. No.** 1906-19033 (T.H. 52=116) **Sheet No.** 1 **of** 23 **Sheets**

Appendix C

Core Test Data from Literature

C.1 Introduction

This appendix presents the results of the concrete core tests found in the literature. The results were used to investigate the effect of aging on concrete strength.

C.2 Riessauw and Taerwe (1980)

After the roadway widening of the Desmet Bridge superstructure in Ghent (Belgium) in 1976, two of the prestressed concrete girders were tested to failure at Ghent University. The bridge had been in service since 1949.

The deck of the bridge, with a span length of 94.6 ft and a total width of 58.8 ft, had 35 post-tensioned I-girders. The flanges of each girder were 20 in. wide, the web was 7 in. thick and the total depth was 44 in.

The measured 28-day strength of concrete was 7800 psi. Testing of the cores obtained from the girders showed that the concrete compressive strength reached a value of 13,800 psi (equivalent cube strength). This reflects an increase of 77% of the 28-day compressive strength in 30 years. No information was given on the number and size of the tested cores.

C.3 Rabbat (1984)

Three precast prestressed concrete girders (I-girders with a composite deck) were removed from a bridge on the Illinois Tollway and were tested at the Construction Technology Laboratories in Skokie, Illinois, in 1984. The girders were 25 years old at the time of testing.

No detailed information was given on the sectional properties of the girders, except that the thickness of the composite deck was 7.5 in.

Four in. diameter cores were extracted from the girder web. The number of the tested cores was not given by the author. Concrete compressive strength was found to be 10,100 psi. According to Rabbat (1984), the design specification called for concrete compressive strength of 4000 psi at transfer and 5000 psi at 28 days. However, no detailed information was given on the measured 28-day concrete strength of the girders.

C.4 Scanlon and Mikhailovsky (1987)

The authors described an investigation carried out to assess the variability of concrete strength and member dimensions in a 34 year-old two lane concrete bridge structure located near Lethbridge, Alberta, Canada. The concrete strength variability was assessed based on a combination of compressive strength tests on cores, and two series of nondestructive tests.

The superstructure consisted of three continuous concrete spans with five T-girders. The deck was 7 in. thick, and the girders varied in depth from 57 in. over the piers to 27 in. over the abutments and at middle span. The stem (web) width of the girders was 17 in.

The minimum 28-day concrete compressive strength was specified as 3000 psi. A series of cores was taken from twenty-one randomly selected locations in the girder stems to provide direct measurement of concrete strength. Cores were drilled with a 4 in. diameter diamond drill. Cores taken from the stems were placed in two groups. The first group consisted of 10 single cores, each having a length of approximately half the girder width. The second group consisted of 11 pairs of cores, each pair obtained by drilling through the stem and breaking the core in two pieces, each approximately half the girder stem width. One core was discarded because it included a steel reinforcing bar.

The compressive strength for 31 cores taken from the girder stems are shown in Table C.1.

C.5 Olson (1991):

The results of four AASHTO Type III I-girder tests, which evaluated impact damage and repairs, were described in this study (at University of Minnesota). The girders, obtained from Bridge No. 27915 of the Mn/DOT bridge inventory, had been fabricated in 1967. They were removed from service in 1984 as a result of a road realignment project.

For the girder concrete, the cement was Universal Atlas Type III Portland Cement. The minimum girder concrete compressive strength was specified as 4500 psi and 5000 psi at transfer of prestress and 28 days, respectively. The average measured 28-day cylinder strength was 6770 psi. The measured concrete strengths at the time of testing from the non-destructive tests and core tests are given in Table C.2. The number of the tested cores was not provided by the author.

C.6 Halsey and Miller (1996)

Two specimens from a 40-year-old inverted T-girder prestressed concrete bridge were tested to destruction. These girders matched specimens tested in 1954 as the prototype for this bridge. The girders had a 27-ft design span. The girders were 12 in. deep, and had 3 in. thick webs. Detailed information on the girder sections are given by Halsey and Miller (1996).

Concrete samples were removed from the second specimen by core drilling. Three usable from the inverted T-girders were taken. Each core was 2.75 in. in diameter and was cut into 6 in. long samples (cores yielded two usable samples).

Because of the shape of the inverted T-girders, the cores had to be removed from the web to have sufficiently large specimens. When the cores were tested in compression, the measured cross-sectional area of the core was used.

According to the authors, because the inverted T-girders tested in 1994 were made under the same conditions as the S-1 and S-2 prototypes of 1954, a comparison of behaviors was reasonable.

Table C.3 shows the results of the compression tests conducted on the precast girders by the authors.

C.7 Pessiki et al. (1996)

Pessiki et al. (1996) summarized the results of an experimental study to determine the effective prestress force in two full-scale prestressed concrete I-girders that were removed from a bridge after a period of 28 years in service.

The two I-girders tested were 60 in. deep and 24 in. wide (both top and bottom flanges) with an 8 in. thick web and a span length of 89 ft.

After the ultimate load tests, several 4 in. diameter 8 in. long core samples were removed from each girder to determine concrete material properties. The cores were removed from each girder web adjacent to its intersection with the top flange in uncracked areas. For each girder, five cores were taken for compressive strength tests.

Compression tests conducted following ASTM C39 test procedures resulted in compressive strengths of 8760 psi for the first specimen (marked as Girder 3-J) and 8180 psi for the second specimen (marked as Girder 4-J). Shop drawings for the girders specified a 28-day design compressive strength of 5100 psi. The average compressive strength of 8440 psi from the cores was 65% greater than the original design strength.

Table C.4 summarizes the results obtained from the core tests. The measured 28-day concrete strengths were not provided by the authors.

C.8 Labia et al. (1997) and Saiidi et al. (2000)

These studies present an investigation regarding the behavior of two full-scale prestressed concrete girders that were in service for 20 years. The 28-day compressive strength of the concrete was specified at 5500 psi with steam curing.

The concrete material properties were obtained by testing the cores taken from the two specimens. Eight cores were extracted from the end blocks of box Girders 1 and 2. Seven cores were approximately 3.75 in. diameter and one core was 5.5 in. diameter. Five cores (three from Girder 1 and two from Girder 2) were capped and tested in compression. The average compressive strength of the capped cores for Girder 1 and 2 were 7864 psi and 8265 psi, respectively.

Three 3.75 in. diameter cores, extracted from the end block of Girder 2, were tested in order to obtain a complete stress/strain relationship for concrete. The average compressive strength of these cores was 9983 psi, which was higher than the measured values for the capped specimens. According to the authors, the higher compressive strength was due to the ends of these cores being ground as opposed to being capped. The results for the ground specimens were reduced by 15% to account for this effect as recommended by the testing agency (Labia et al., 1997). The average reduced strengths of the ground specimens and the measured values for the capped cores were 8450 psi.

The results of the core tests are summarized in Table C.5.

C.9 Runzel, et al. (2007)

Two shear capacity tests were performed at the University of Minnesota using the two ends of an 88 ft. long bridge girder removed from Mn/DOT Bridge No. 73023. The tested bridge girder was 54 in. deep with an 8 in. thick web, and had a nominal concrete compressive strength of 6 ksi, and came from a bridge with 10 ft. girder spacing.

Upon completion of the capacity tests, eight 4 in. by 8 in. cores were obtained according to ASTM C42 (ASTM, 99) for compressive strength tests from uncracked regions of the web in the non-tested end of one of the specimens. The specimens had a protective coating on one side of the girder, thus the protective coating was removed by cutting off the end of each cylinder.

The compression tests were conducted according to ASTM C39 (ASTM, 01). The concrete cores were capped with sulfur capping compound, and loaded at a rate of 450 lbs/s until they failed (Runzel et al. 2007). The test results are shown in Table C.6.

The 28-day measured and design concrete compressive strengths for some of the girders of the 20-year-old Bridge No. 73023 (the same bridge as one of the girders tested by Runzel et al., 2007) were obtained from the records of the Cretex precasting plant in Elk River, which were provided by Mn/DOT. Table C.7 shows the 28-day measured and design concrete strengths for four girders of Bridge 73023.

From the results in Tables C.6 and C.7, the measured average 28-day concrete strength was observed to increase by 27.4% over 20 years.

References

Halsey, J.T., and Miller, R.A., "Destructive Testing of Two Forty-Year-Old Prestressed Concrete Bridge Beams," *PCI Journal*, V. 41, No. 5, September-October 1996, pp. 84-93.

McIntyre, M., and Scanlon, A., "Interpretation and Application of Core Test Data in Strength Evaluation of Existing Concrete Bridge Structures," *Canadian Journal of Civil Engineering*, V.17 pp. 471-480, 1990.

Olson, S.A., "Impact Damage and Repair of AASHTO Type III Girders", PhD Thesis, Department of Civil Engineering, University of Minnesota, Minneapolis, MN, April 1991.

Pessiki, S., Kaczinski, M., and Wescott, H.H., "Evaluation of Effective Prestress Force in 28-Year-Old Prestressed Concrete Bridge Beams," *PCI Journal*, V. 41, No. 6, November-December 1996, pp. 78-89.

Rabbat, B.G., "25-Year-Old Prestressed Concrete Bridge Girders Tested," *PCI Journal*, V. 29, No. 1, January-February 1984, pp. 177-179.

Riessauw, F.G., and Taerwe, L., "Tests on Two 30-Year-Old Prestressed Concrete Beams," *PCI Journal*, V. 25, No. 6, November-December 1980, pp. 70-72.

Runzel, B., Shield, C.K., and French, C.W., *Mn/DOT Report 2007-47; Shear Capacity of Prestressed Concrete Beams*. Center for Transportation Studies, University of Minnesota, Twin Cities 2007.

Saiidi, M., Labia Y., and Douglas, B., “Full Scale Testing and Analysis of 20-Year-Old Pretensioned Concrete Box Girders,” *PCI Journal*, V.45, No. 2, March-April 2000, pp. 96-105.

Scanlon, A., and Mikhailovsky, L., “Strength Evaluation of an Existing Concrete Bridge Based on Core and Nondestructive Test Data,” *Canadian Journal of Civil Engineering*, V. 14, No.2, April 1987, pp. 145-154.

Wood, Sharon L., “Evaluation of the Long-Term Properties of Concrete,” *Research and Development Bulletin RD102*, Portland Cement Association, Skokie, 1991, 93 pp.

Table C.1 Concrete Strength of Cores Tested by Scanlon and Mikhailovsky (1987)

Core No.	**Compressive Strength* (psi)**	**Core No.**	**Compressive Strength* (psi)**
1	5,333	18	5,117
2	6,285	19	5,575
3	6,395	20	6,041
4	6,489	21	5,232
5	4,892	22	6,285
6	5,517	23	6,066
7	4,857	24	5,416
8	6,955	25	4,267
9	5,166	26	4,722
10	5,407	27	6,637
11	5,403	28	5,646
12	4,321	29	5,100
13	4,722	30	6,489
14	5,646	31	3,990
15	3,056	**Mean**	**5,335**
16	2,949	**STD**	**956**
17	5,411	**COV**	**0.18**

* Compressive strengths shown were corrected by the authors for L/D (L, height of the core, D, diameter of the core) ratio less than 2.

Table C.2 Concrete Strengths Obtained from Non-destructive Testing and Core Tests (Olson, 1991)

Test	**Average (psi)**	**COV**
Schmidt	7,960	0.042
Windsor Probe	6,500	0.102
Pulse Velocity	10,700	0.053
4 in. cores	8,615	0.058
2 in. cores	8,147	0.074

Table C.3 Concrete Strength of Cores Tested by Halsey and Miller (1996)

Specimen Name	# of Cores Tested in 1994	Average Measured f'_c (psi) (1994 tests)	28-day Measured f'_c (psi) (1954 tests)*	
1 - Girder	2	10,450	Girders in S-1	6,560
2 - Girder	2	11,360	Girders in S-2	5,495
3 - Girder	2	13,540	Average	6,028
	Average	11,790		

*The number of the tested cores was not provided.

Table C.4 Concrete Strength of Cores Tested by Pessiki et al. (1996)

Specimen Name	# of Cores Tested	28-day Design f'_c (psi)	Measured f'_c (psi)	% Increase
Girder 3-J	5	5,100	8,760	71.8
Girder 4-J	5	5,100	8,180	60.4
	Average	5,100	8,440	65.5

Table C.5 Strength of Cores Tested by Labia et al. (1997) and Saiidi et al. (2000)

Specimen Name	# of Cores Tested	Measured Mean f'_c (psi)
Capped-Girder 1	3	7,864
Capped-Girder 2	2	8,265
Ground	3	9,983
Corrected Ground by 15%	3	8,486
Total (or Average)	8	8,450
Design 28-day f'_c (psi)	Measured Mean f'_c for All Cores (psi)	% Increase
5,500	8,450	53.6

Table C.6 Concrete Strength of Cores Tested by Runzel et al. (2007)

Concrete Compressive Strength Test Results				
Cylinder Number	**Diameter (in)**	**Length (in)**	***L/D***	**f'_c (psi)**
1	3.85	8.12	2.11	10,430
2	3.85	8.09	2.10	10,590
3	3.85	8.15	2.12	10,080
4	3.85	8.22	2.14	10,650
5	3.85	8.13	2.11	9,880
6	3.85	8.13	2.11	9,000
7	3.85	8.05	2.09	11,630
8	3.85	8.27	2.15	8,780
			Average	10,130
			STD Dev	925
			COV	0.091

Table C.7 28-day Design and Measured Concrete Strengths of Bridge 73023

Date of Test	**Bridge No.**	**Girder No.**	**ID**	**28-day Design f'_c (psi)**	**28-day Measured f'_c (psi)**
5/19/1987	73023	3	K1	5,900	8,360
5/22/1987	73023	6	M	5,900	8,210
5/28/1987	73023	7	44A	5,900	7,670
6/2/1987	73023	8	47A	5,900	7,570
			Average	5,900	7,953
				STD	391
				COV	0.049

Appendix D

Procedure for Finding the Truck Loading Configuration to Check the Applicability of Arching Action

D.1 Introduction

As mentioned in Section 6.2.3.3, the procedure to find the maximum distance of the application of the rear tandem wheel load of an HS-20 truck from the centerline of the support which yields $\frac{\phi V_{n,\ STD\,2002}}{V_u}$ less than unity at the critical section is described in this appendix. A sample calculation is also shown for Bridge 48010, which is one of the bridges with a low shear inventory rating factor (i.e., 0.65).

D.2 Procedure to Find Distance and Sample Calculation for Bridge 48010

Find the truck load configuration as shown in Figure D.1 which yields $\phi V_n \le V_u$ at the critical section ($h_c/2$, where h_c is the height of the composite section).

The shear value due to the truck loading at distance $h_c/2$ ($V_{LL@h/2}$) will be the same as the reaction value, R at A. Find R by summing the moments at B:

$\sum M_B = 0$, $(32)(L - x) + (32)(L - x - 14) + (8)(L - x - 28) = (R)(L)$

Where L is the span length and x is distance from the center of the support to the rear tandem.

Then,

$$R = \frac{(72)(L) - (72)(x) - 672}{L}$$

And because $V_{LL@hc/2} = R$ for the same load configuration;

$$V_{LL@\frac{h_c}{2}} = \frac{(72)(L) - (72)(x) - 672}{L}$$

To find the furthest rear tandem position which yields $\phi V_n \le V_u$:

<u>Demand, V_u, at $h_c/2$</u>:

$$V_u = (\gamma_{LL})(1 + I)(LLDF)(V_{LL@\frac{h_c}{2}}) + (\gamma_{DL})(V_{DL@\frac{h_c}{2}})$$

where, γ_{LL}, is the live load factor (1.3*1.67 = 2.17), I, is impact factor for live load, $LLDF$, is the live distribution factor for shear, $V_{LL@hc/2}$ is as defined above, γ_{DL} is the dead load factor (1.3*1.0=1.3), and $V_{DL@h_c/2}$ is the shear at $h_c/2$ due to dead load.

<u>Capacity ϕV_n at $h_c/2$</u>:

$$\phi V_{n@\frac{h_c}{2}} = \phi(V_{c@\frac{h_c}{2}} + V_{s@\frac{h_c}{2}})$$

Where for almost all cases $V_{c@\frac{h_c}{2}}$ is controlled by V_{cw} at $h_c/2$:

$$V_{cw} = (3.5\sqrt{f_c^{'}} + 0.3 f_{pc}) b_w d + V_p$$

(Note: Both V_s and V_{cw} are independent of truck load configuration)

<u>From $V_u \geq \phi V_n$</u>:

$$(\gamma_{LL})(1+I)(LLFD)(V_{LL@\frac{h_c}{2}}) + (\gamma_{DL})(V_{DL@\frac{h_c}{2}}) \geq \phi V_{n@\frac{h_c}{2}}$$

Insert the equation for $V_{LL@\frac{h_c}{2}}$:

$$(\gamma_{LL})(1+I)(LLDF)\left(\frac{(72)(L)-(72)(x)-672}{L}\right) + (\gamma_{DL})(V_{DL@\frac{h_c}{2}}) \geq \phi V_{n@\frac{h_c}{2}}$$

$$\longrightarrow \quad x \quad \leq \quad (L) - \frac{\left[\frac{\left[\phi V_{n@\frac{h_c}{2}} - (\gamma_{DL})(V_{DL@\frac{h_c}{2}})\right](L)}{(\gamma_{LL})(1+I)(LLDF)} + 672\right]}{72}$$

If the value x calculated from the equation above is greater than (L – 28 ft), then the above equation and the truck loading configuration shown in Figure D.1 is no longer valid. Thus, for x > (L – 28 ft), the new valid configuration will be as shown in Figure D.2 and the following procedure should be used to find x:

$\sum M_B = 0$, $(32)(L - x) + (32)(L - x - 14) = (R)(L)$

Then,

$$R = V_{LL@\frac{h_c}{2}} = \frac{(64)(L)-(64)(x)-448}{L}$$

Similar to the rearrangement of the parameters shown previously,

$$x \leq (L) - \frac{\left[\frac{\left[\phi V_{n@\frac{h_c}{2}} - (\gamma_{DL})(V_{DL@\frac{h_c}{2}})\right](L)}{(\gamma_{LL})(1+I)(LLDF)} + 448\right]}{64}$$

Sample calculation of x for Bridge 48010 is shown as follows:

$$\phi V_{n@\frac{h_c}{2}} = 167.0\, kips\,,\ \phi V_{DL@\frac{h_c}{2}} = 46.3\, kips\,,$$

L = 43.17 ft, I = 0.3, $LLDF$ = 1.136

$$x \quad \leq \quad (43.17) - \frac{\left[\frac{[167-(1.3)(46.3)](43.17)}{(2.17)(1+0.3)(1.136)} + 672\right]}{72}$$

$$x \quad \leq \quad 13.86 \;\; ft \;\; \text{(from centerline of support)},$$

Also check (L - x) = (43.17 – 13.86) = 29.31 ft > 28 ft, O.K.

And h_c = 4.65 ft, thus 2.5 h_c = 11.61 ft.

Thus, this shows that, the shear capacity-to-demand ratio will be less than 1 at the critical section, i.e., h_c /2 at most a distance of x = 13.86 ft which is already greater than 2.5 h_c = 11.61 ft which is the maximum distance to consider an arching action contribution to shear capacity. Therefore, arching action may not add additional capacity.

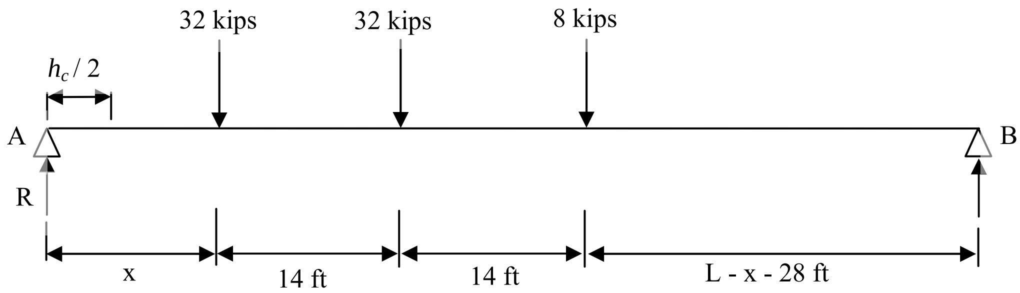


Figure D.1 HS-20 Truck Loading Configuration to Find x for $L - x \geq 28\,ft$

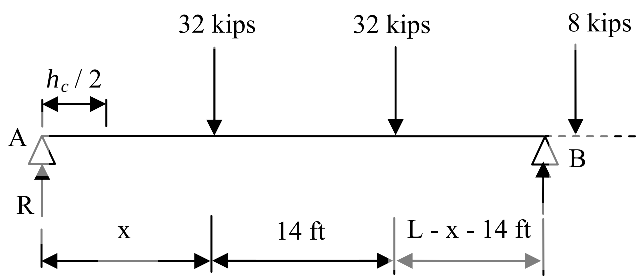


Figure D.2 HS-20 Truck Loading Configuration to Find x for $L - x < 28\,ft$

Appendix E

Sample Calculations of Shear Live Load Distribution Factors

E.1 Introduction

As discussed in Section 6.2.4.5, the shear live load distribution factors (LLDFs) obtained from the AASHTO LRFD Specification, the calibrated lever rule (NCHRP 12-62 method) and modified Henry's methods were compared to shear LLDFs calculated using the AASHTO Standard method.

In this appendix, sample calculations of shear live load distribution factors (LLDFs) from those four simplified methods are presented for Bridge No. 17007 (presented in the analyzed set of bridges in Section 6.2.4.5).

E.2 Shear LLDF Calculations for Bridge No. 17007

The sectional and material properties used for the shear LLDF calculations of Bridge 17007 are as follows:

Area of girder, A = 624 in^2

Moment of inertia of the precast girder, I = 167,048 in^4

Compressive strength of girder concrete, $f'_{c,girder}$ = 7000 psi

Compressive strength of slab concrete, $f'_{c,slab}$ = 4000 psi

Modulus of elasticity of girder concrete, $E'_{c,girder}$= 4,821 ksi

Modulus of elasticity of slab concrete, $E'_{c,slab}$ = 3,644 ksi

Number of girders, N_g = 4

Skew angle, θ = 52.75 degrees

Girder spacing, S = 12.0 ft

Girder span length, L = 77.6 ft

Edge-to-Edge width of bridge (clear roadway width), W = 40.84 ft

Thickness of the concrete slab, t_s = 8.5 in

E.2.1 2002 AASHTO Standard Specification Shear Live Load Distribution Factors

This section summarizes the AASHTO Standard method shear live load distribution factor calculations for Bridge 17007 (see Section 6.2.4.1).

Per the AASHTO 2002 Standard Article 3.6.2

Number of design lanes, $N_L = \frac{W}{12} = \frac{40.84}{12} \approx 3.4$

AASHTO 2002 Article 3.6.3 states that the fractional parts of design lanes shall not be used. Thus, $N_L = 3$

(a) Interior Girder:

Because $N_L = 3$, the distribution factor for two or more lanes is used.

Two or More Design Lanes Loaded

Per the AASHTO 2002 Standard Table 3.23.1

$$g_{interior} = \frac{1}{2}\left(\frac{S_g}{5.5}\right) = \frac{1}{2}\left(\frac{12}{5.5}\right) = 1.091$$

(b) Exterior Girder:

Per the AASHTO 2002 Standard Article 3.23.2.3.1.2, the lever rule method was used.

One Design Lane Loaded

Figure E.1 illustrates the application of lever rule method to exterior girders for one lane loaded case.

The lever rule assumes no transverse deck moment continuity at the interior beams, which renders the transverse deck cross section statically determinate. The direct equilibrium method is used to determine the load distribution to the beam of interest.

Summing the moments at point A:

$$(g_{lever-rule-1})(2R)(12\text{ft}) = (R)(6.25\text{ft}) + (R)(12.25\text{ft})$$

$$g_{lever-rule-1} = \frac{(1)(12\text{ft})}{(\frac{1}{2})(6.25\text{ft}) + (\frac{1}{2})(12.25\text{ft})} = 0.771$$

When the lever rule is used, the multiple presence factors should be applied to the distribution factors. The multiple presence factor for one lane loaded case is 1.0 (Article 3.12.1 of 2002 Standard).

$$g_{exterior-1} = (0.771)(1.0) = 0.771$$

Two or More Design Lanes Loaded

For two or more design lanes loaded, the manual placement of the load during the application of lever rule becomes cumbersome. As mentioned in Section 6.2.4.4, the lever rule equations are provided by Puckett et al. (2007), in the NCHRP Report 592, to facilitate lever rule computations. Figures E.2 and E.3 show those lever rule equations for exterior and interior girders, respectively. The equations were utilized for the application of the lever rule method hereafter.

To use Figure E.2, the parameter, d_e, which is the distance from the center of the exterior girder to the location of the centroid of the outermost wheel group (in feet), was calculated.

It should be noted that this "d_e" should not be confused with the parameter "d_e" defined by the AASHTO LRFD Code (Section .4.6.2.2.1) as "the distance from exterior web of exterior beam and the interior edge of curb or traffic barrier". To avoid this, the parameter "d_e" from the NCHRP Report 592 is referred as "d_{eNCHRP}" in this appendix,

From Figure E.1, $d_{e,NCHRP} = d_e - 2\text{ft} = 2.25\text{ft} - 2\text{ft} = 0.25\text{ ft}$ and $\text{S} = 12\text{ ft}$,

From Figure E.2, and for $(d_{e,NCHRP} + S) = 12.25\text{ ft}$,

$$g_{lever-rule-2} = \frac{3}{2} + \frac{3(d_{e,NCHRP})}{2S} - \frac{8}{S} = \frac{3}{2} + \frac{3(0.25)}{2(12)} - \frac{8}{12} = 0.865$$

The multiple presence factor for two-lanes loaded case is 1.0 (Article 3.12.1 of 2002 Standard).

$$g_{exterior-2} = (0.865)(1.0) = 0.865$$

Thus,

$$g_{exterior} = \max\left(g_{exterior-1}, g_{exterior-2}\right) = \max\left(0.771, 0.865\right) = 0.865$$

E.2.2 2004 AASHTO LRFD for Shear Live Load Distribution Factors

This section summarizes the shear live load distribution factor calculations for Bridge 17007 using the AASHTO LRFD method (see Section 6.2.4.2).

(a) Interior Girder:

Per the 2004 AASHTO LRFD Table 4.6.2.2.3a-1:

One Design Lane Loaded

$$g_{interior-1} = 0.36 + \frac{S}{25.0} = 0.36 + \frac{12}{25.0} = 0.840$$

Two or More Design Lanes Loaded

$$g_{interior-2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0} = 0.2 + \frac{12}{12} - \left(\frac{12}{35}\right)^{2.0} = 1.082$$

- Per 2004 AASHTO LRFD Eqn. (4.6.2.2.1-1) and Eqn. (4.6.2.2.1-2), respectively:

$$K_g = n\left(I + Ae_g^2\right) \text{ and } n = \frac{E_{c,girder}}{E_{c,slab}}$$

where K_g is the longitudinal stiffness parameter, in^4, n is the modular ratio between beam and deck, and e_g is the distance between the centers of gravity of the girder and slab, in.

$e_g = y_t + 0.5t_s$, where y_t is the distance from centroid to the extreme top fiber of the non-composite precast girder

y_t = 22.66 in. for Bridge 17007

$$e_g = 22.66\,\text{in} + 0.5(8.5\,\text{in}) = 26.91\,\text{in}$$

$$n = \frac{E_{c,girder}}{E_{c,slab}} = \frac{4{,}821\,\text{ksi}}{3{,}644\,\text{ksi}} = 1.323$$

$$K_g = 1.323\left(167{,}048\text{in}^4 + (624\text{in}^2)(26.91\text{in})^2\right) = 818{,}536\,\text{in}^4$$

$$\text{Range of Applicability:} \begin{cases} 3.5\text{ ft} \le \text{S} \le 16\text{ ft} \;\;, \text{S} = 12\text{ ft, OK} \\ 20\text{ ft} \le \text{L} \le 240\text{ ft, L} = 77.6\text{ ft, OK} \\ 4.5\text{ in} \le \text{t}_s \le 12\text{ in, t}_s = 8.5\text{ in, OK} \\ 10{,}000\text{ in}^4 \le \text{K}_g \le 7{,}000{,}000\text{ in}^4, \text{K}_g = 818{,}536\text{ in}^4\text{, OK} \\ \text{N}_\text{b} \ge 4, \text{N}_\text{b} = 4\text{, OK} \end{cases}$$

- <u>Skew Correction Factor</u>, *SCF*:

Per 2004 AASHTO LRFD Table 4.6.2.2.3c-1:

$$SCF = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3} \tan\theta$$

$$\text{Range of Applicability:}\begin{cases} 0 \le \theta \le 60°\text{, } \theta = 52.75°\text{, OK} \\ 3.5\text{ ft} \le S \le 16\text{ ft , } S = 12\text{ ft, OK} \\ 20\text{ ft} \le L \le 240\text{ ft, } L = 77.6\text{ ft, OK} \\ N_b \ge 4\text{, } N_b = 4\text{, OK} \end{cases}$$

$$SCF = 1.0 + 0.20\left(\frac{12.0(77.6\text{ft})(8.5\text{in})^3}{(818{,}536\text{in}^4)}\right)^{0.3} \tan(52.75°) = 1.236$$

$$g_{interior} = (SCF)\left(\max\left(g_{interior-1,}\, g_{interior-2}\right)\right) = (1.236)(1.082) = 1.338$$

(b) Exterior Girder

Per 2004 AASHTO LRFD Table 4.6.2.2.3b-1:

<u>One Design Lane Loaded</u>

Use lever rule method, see AASHTO Standard method for exterior girder in Section E.2.1 (b):

$$g_{exterior\_1} = 0.771$$

<u>Two or More Design Lanes Loaded</u>

$$g_{exterior\_2} = \left(0.6 + \frac{d_e}{10}\right) g_{interior}$$

where d_e is the distance from exterior web of exterior beam and the interior edge of curb or traffic barrier, ft (Article 4.3 from 2004 AASHTO LRFD). It is applicable only for, $-1.0 \le d_e \le 5.5\, ft$.

Figure E.1 illustrates the d_e value, which is 2.25 ft.

$$g_{exterior\_2} = \left(0.6 + \frac{2.25}{10}\right)(1.338) = 1.104$$

Thus,

$$g_{exterior} = \max\left(g_{exterior-1,}\, g_{exterior-2}\right) = \max\left(1.338,\, 1.104\right) = 1.338$$

E.2.3 Modified Henry's Method for Shear Live Load Distribution Factors

This section summarizes the shear live load distribution factor calculations for Bridge 17007 using the Modified Henry's method (see Section 6.2.4.3.1).

- Calculate the distribution factor using the (unmodified) Henry's method.

$$g_{Henry's} = \frac{1}{2}\left((N_L)(IF)\frac{2}{N_g}\right)$$

- Number of Lanes $= N_L = \dfrac{W}{10} = \dfrac{40.84\text{ ft}}{10\text{ ft}} = 4.084$

- Find the Intensity Factor (*IF*), (Huo et al., 2003), by interpolating the multiple presence factors of AASHTO Standard according to the value obtained for Number of Lanes (i.e., 4.084).

For 4 or more lanes, the multiple presence factor is 0.75 (Article 3.12.1 of 2002 Standard).

Thus, $IF = 0.75$

$$g_{Henry's} = \frac{1}{2}\left((N_L)(IF)\frac{2}{N_g}\right) = \frac{1}{2}\left((4.084)(0.75)\frac{2}{4}\right) = 0.766$$

- Apply the superstructure type modification and the skew correction factor for shear:

- Superstructure Type Modification Factor

The structure modification factor for precast concrete sections is 1.20 for shear.

- Skew Correction Factor, *SCF*:

$$SCF = (1.0 + 0.2\tan\theta) = (1.0 + 0.2\tan(52.75°)) = 1.263$$

$$g_{Mod.Henry's} = (g_{Henry's})(1.20)(SCF) = (0.766)(1.20)(1.263) = 1.161$$

Because the modified Henry's method assumes equal distribution of live load effects to all girders, $g_{Mod.Henry's} = 1.161$ is the shear live load distribution factor for both interior and exterior girders.

E.2.4 NCHRP 12-62 Recommended Method for Shear Live Load Distribution Factors

This section summarizes the shear live load distribution factor calculations for Bridge 17007 using the simplified method recommended by Puckett et al. (2007), (see Section 6.2.4.4), namely the calibrated rule.

For the calibrated lever rule, only one and two lanes loaded are considered. In Appendix O of the NCHRP Report 592, when the multiple presence factors are included in the distribution factor values, the two-lane loaded case typically controlled. The difference between the two- and three-lanes loaded cases was small when the three-lanes loaded case controlled. Therefore, only on one and two lanes loaded are the multiple presence factors used (Puckett et al. (2007)).

For one lane and multiple lanes loaded for shear the proposed equation is

$$g_v = m\gamma_a\left[a_v\left(g_{lever-rule}\right)+b_v\right] \geq m\left[\frac{N_{lanes}}{N_g}\right]$$

where g_v is the live load distribution factor (subscript v for shear), m is the multiple presence factor as specified in 2004 AASHTO LRFD Article 3.6.1.1.2 (3), γ_a is analysis factor, also defined as distribution simplification factor (DSF), a_v and b_v are the calibration constants for shear and reactions, $g_{lever\text{-}rule}$ is the distribution factor computed by the lever rule , N_{lanes} is the number of design lanes considered in the lever rule analysis, and N_g is the number of girders.

(a) Interior Girder:

One Design Lane Loaded

$N_{lanes} = 1$

$m = 1.2$ 2004 AASHTO LRFD Article 3.6.1.1.2 (3)

$a_v = 1.08$ and $b_v = -0.13$ Table 6.17 in Chapter 6

$\gamma_a = 1.00$ Table 6.18 in Chapter 6

From Figure E.3 and for $S = 12$ ft and $d_{e,NCHRP} = 0.25\text{ft} > 0$

$$g_{lever-rule-1} = 1 - \frac{3}{S} = 1 - \frac{3}{12} = 0.75$$

$$g_{interior-1} = (1.2)(1.00)\left[(1.08)(0.75) + (-0.13)\right] \geq 1.2\left[\frac{1}{4}\right]$$

$$g_{interior-1} = 0.816 > 0.3$$

Skew Correction Factor, *SCF*:

$$SCF = 1.0 + 0.09\tan\theta = 1.0 + 0.09\tan(52.75°) = 1.118$$

$$g_{interior-1} = (0.816)(1.118) = 0.913$$

Two or More Design Lanes Loaded

$N_{lanes} = 2$

$m = 1.0$ 2004 AASHTO LRFD Article 3.6.1.1.2 (3)

$a_v = 0.94$ and $b_v = 0.03$ Table 6.17 in Chapter 6

$\gamma_a = 1.05$ Table 6.18 in Chapter 6

From Figure E.3 and for $S = 12$ ft and $d_{e,NCHRP} = 0.25\text{ft} > 0$

$$g_{lever-rule-2} = 2 - \frac{10}{S} = 2 - \frac{10}{12} = 1.167$$

$$g_{interior-2} = (1.0)(1.05)\left[(0.94)(1.167) + (0.03)\right] \geq 1.0\left[\frac{2}{4}\right]$$

$$g_{interior-2} = 1.183 > 0.5$$

Skew Correction Factor, *SCF*:

$$SCF = 1.0 + 0.09\tan\theta = 1.0 + 0.09\tan(52.75°) = 1.118$$

$$g_{interior-2} = (1.183)(1.118) = 1.323$$

Thus,

$$g_{interior} = \max\left(g_{interior-1}, g_{interior-2}\right) = \max\left(0.913, 1.323\right) = 1.323$$

b) Exterior Girder

One Design Lane Loaded

$N_{lanes} = 1$

$m = 1.2$ 2004 AASHTO LRFD Article 3.6.1.1.2 (3)

$a_v = 0.83$ and $b_v = 0.07$ Table 6.17 in Chapter 6

$\gamma_a = 1.00$ Table 6.18 in Chapter 6

From Figure E.2 and for $S = 12$ ft and $d_{e,NCHRP} = 0.25\text{ft}$

$$g_{lever-rule-1} = 1 + \frac{d_{e,NCHRP}}{S} - \frac{3}{S} = 1 + \frac{0.25}{12} - \frac{3}{12} = 0.771 \text{ (Same value as in Section E.2.1 (b))}$$

$$g_{exterior-1} = (1.2)(1.00)\left[(0.83)\left(0.771\right) + (0.07)\right] \geq 1.2\left[\frac{1}{4}\right]$$

$$g_{exterior-1} = 0.852 > 0.3$$

Skew Correction Factor, *SCF*:

$$SCF = 1.0 + 0.09\tan\theta = 1.0 + 0.09\tan(52.75°) = 1.118$$

$$g_{exterior-1} = (0.852)(1.118) = 0.953$$

Two or More Design Lanes Loaded

$N_{lanes} = 2$

$m = 1.0$ 2004 AASHTO LRFD Article 3.6.1.1.2 (3)

$a_v = 0.92$ and $b_v = 0.06$ Table 6.17 in Chapter 6

$\gamma_a = 1.00$ Table 6.18 in Chapter 6

From Figure E.2 and for $S = 12$ ft and $d_{e,NCHRP} = 0.25$ft

$$g_{lever-rule-2} = \frac{3}{2} - \frac{3(d_{e,NCHRP})}{2S} - \frac{8}{S} = \frac{3}{2} - \frac{3(0.25)}{2(12)} - \frac{8}{(12)} = 0.865$$

(Same value as in Section E.2.1 (b))

$$g_{exterior-2} = (1.0)(1.00)\left[(0.92)\left(0.865\right) + (0.06)\right] \geq 1.0\left[\frac{2}{4}\right]$$

$$g_{exterior-2} = 0.855 > 0.5$$

Skew Correction Factor, *SCF*:

$$SCF = 1.0 + 0.09\tan\theta = 1.0 + 0.09\tan(52.75°) = 1.118$$

$$g_{exterior-2} = (0.855)(1.118) = 0.957$$

Thus,

$$g_{exterior} = \max\left(g_{exterior-1,}\, g_{exterior-2}\right) = \max\left(0.953,\ 0.957\right) = 0.957$$

The shear live load distribution factors calculated using the simplified methods shown in this appendix for Bridge 17007 are summarized in Table E.1.

References

AASHTO, *Standard Specifications for Highway Bridges*, 17th edition, Washington, D.C., 2002.

AASHTO, *AASHTO LRFD Bridge Design Specifications*, 3rd edition, Washington, D.C., 2004.

Huo, X.S., Conner, S.O., and Iqbal, R., *Re-examination of the Simplified Method (Henry's Method) of Distribution Factors for Live Load Moment and Shear*, Final Report, Tennessee DOT Project No. TNSPR-RES 1218, Tennessee Technological University, Cookeville, TN (June 2003).

Puckett, J.A., Huo, X.S., Patrick, M.D., Jablin, M.C., Mertz, D., Peavy, M.D, "Simplified Equations for Live-Load Distribution in Highway Bridges," *International Bridge Engineering Conference,* TRB: 6IBECS-069, July 2005.

Puckett, J.A., and Mertz, D., "NCHRP Report 592; Simplified Live Load Distribution Factor Equations." *Transportation Research Board,* 2007.

Table E.1 Shear LLDF's Calculated by Simplified Methods for Bridge No. 17007

Method	Interior Girder	Exterior Girder
AASHTO Standard	1.091	0.865
AASHTO LRFD	1.338	1.143
Modified Henry's	1.161	1.161
NCHRP 12-62	1.323	0.957

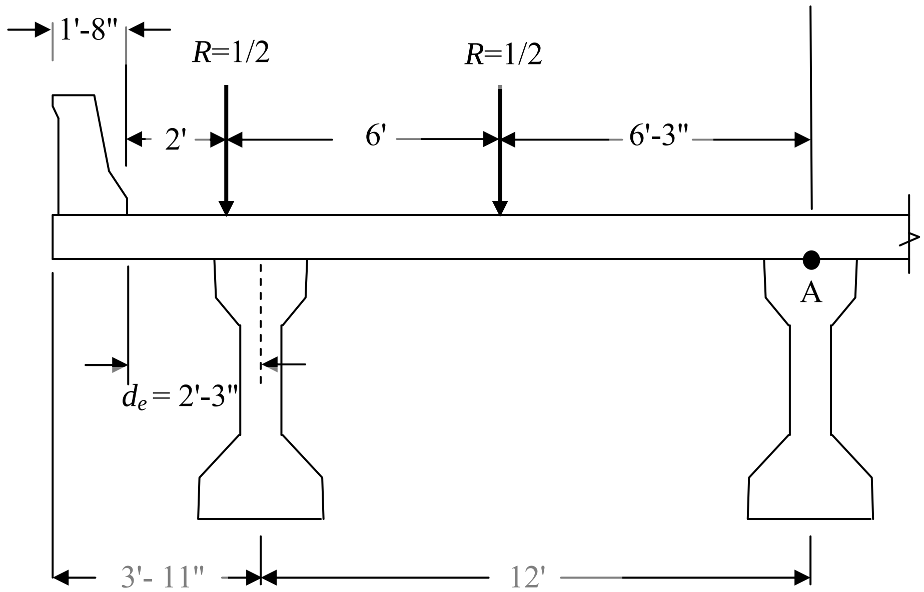


Figure E.1 Application of Lever Rule for Exterior Girder, One Lane Loaded

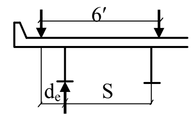
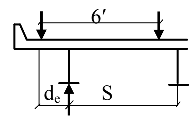
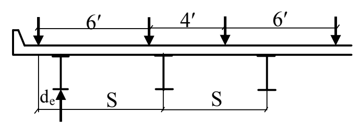
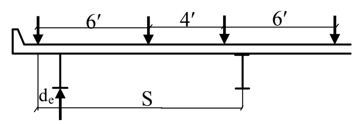
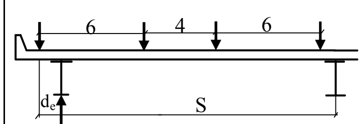
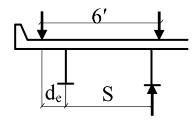
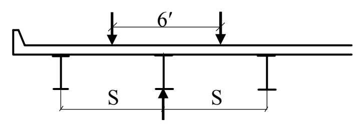
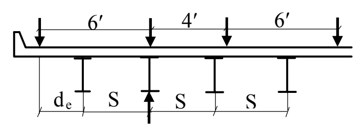
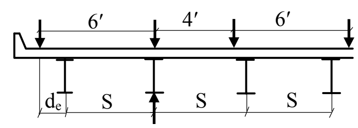
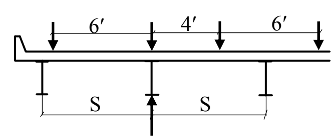
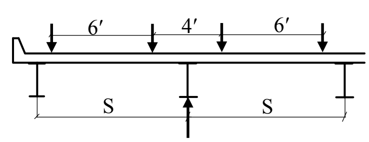
Number of Loaded Lanes	Distribution Factor	Range of Application	Loading Diagram	Number of Wheels to Beam
1	$\frac{1}{2}+\frac{d_e}{2S}$	$(d_e+S)\le 6\,ft$ $\lvert d_e\rvert < S$		1
	$\frac{1}{2}+\frac{d_e}{S}-\frac{3}{S}$	$(d_e+S)>6\,ft$		2
2 or more	$\frac{1}{2}+\frac{d_e}{S}-\frac{3}{S}$	$(d_e+S)\le 10\,ft$		2
	$\frac{3}{2}+\frac{3d_e}{2S}-\frac{8}{S}$	$10<(d_e+S)\le 16\,ft$		3
	$2+\frac{2d_e}{S}-\frac{16}{S}$	$16<(d_e+S)\le 20\,ft$		4

Figure E.2 Lever Rule Equations for Exterior Girders (Puckett et al. 2007)

Number of Loaded Lanes	Distribution Factor	Range of Application	Loading Diagram	Number of Wheels to Beam
1	$\frac{1}{2}$	$S \leq 6\,ft$ $d_e \geq 0$		1
	$\frac{1}{2} - \frac{3}{S}$	$S > 6\,ft$ $d_e \geq 0$		2
2 or more	$\frac{1}{2}$	$S \leq 4\,ft$ $d_e \geq 0$		1
	$1 - \frac{2}{S}$	$4 < S \leq 6\,ft$ $d_e \geq 0$		2
	$\frac{3}{2} - \frac{5}{S}$	$6 < S \leq 10\,ft$ $d_e \geq 0$		3
	$2 - \frac{10}{S}$	$10 < S \leq 16\,ft$ $d_e \geq 0$		4

Note: If d_e<0, use lever rule and manually place the vehicle for critical effect on the first interior beam

Figure E.3 Lever Rule Equations for Interior Girders (Puckett et al. 2007)