DEPARTMENT OF TRANSPORTATION

Implementation of a Refined Shear Rating Methodology for Prestressed Concrete Girder Bridges

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Lower than desirable shear ratings at the ends of prestressed concrete beams have been the topic of ongoing research between MnDOT and the University of Minnesota. A recent study by the University of Minnesota entitled *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* (French et al., 2016) sought to increase the shear rating of a prestressed concrete beam via a refined live load distribution factor, considering the location-based load distribution of each axle along the span. The study also presented a screening tool to determine the likelihood that the refined methodology would improve the shear rating of a given prestressed concrete beam bridge. The research presented within this report implemented the refined shear rating methodology presented by French et al. (2016). MnDOT selected 522 bridges from its inventory noted as having shear rating deficiencies for potential re-evaluation as part of this research. After employing the screening tool, the initial selection was reduced to 127 applicable bridges, of which 50 were selected for re-evaluation. For the 50 bridges evaluated, the refined rating methodology was found to improve the shear ratings by an average of 16 percent. The screening tool also proved to be an effective means of determining the candidacy of a given bridge for re-evaluation.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
A_{ps}	Area of prestressing steel
A_s	Area of nonprestressed tension reinforcing steel
A_v	Area of shear reinforcement within a distance s
b_v	Concrete girder web width
BrR	Bridge Rating (formerly Virtis) software package by AASHTOware (Version 6.7.1)
C	Capacity (or strength) of a bridge element under consideration
CFRP	Carbon fiber reinforced polymer
d_v	Effective shear depth of a composite concrete beam and deck
DC	Dead load of components and non-structural attachments
DW	Dead load of wearing surfaces and utilities
E_p	Modulus of elasticity of prestressing tendons
E_s	Modulus of elasticity of reinforcing steel
f'_c	Specified compressive strength of concrete for use in design
F_{po}	A parameter taken as the modulus of elasticity of prestressing tendons multiplied by the locked-in difference in strain between the prestressing tendons and the surrounding concrete
F_y	Specified minimum yield strength of reinforcing bars
h	overall depth of a concrete beam excluding haunch and deck
HL-93	A notional design load consisting of an HS20 design vehicle plus a 640 plf superimposed lane load
HS20	A notional design vehicle consisting of an 8,000 lb axle and two 32,000 lb axles (axles spaced at 14'-0")
HS20-44	A notional design load consisting of an HS20 design vehicle or a 640 plf lane load with a 18,000 lb concentrated load for moment effects or a 26,000 lb concentrated load for shear effects
K_1	Aggregate source correction factor (taken as 1.0)

L	Span length of a prestressed concrete beam (bearing-to-bearing)
LL+IM	Vehicular live load plus vehicular dynamic load allowance
LLDF	Live Load Distribution Factor
LRFD	Load and Resistance Factor Design
LRFR	Load and Resistance Factor Rating
MBE	AASHTO Manual for Bridge Evaluation
MPF	Multiple Presence Factor
M_u	Factored moment in a prestressed concrete beam at the location under consideration
NBI	National Bridge Inventory
N_u	Applied axial force in a prestressed concrete beam at the location under consideration (taken as positive if tensile)
s	Center-to-center spacing of shear reinforcement steel
s_{xe}	Crack spacing parameter allowing for the influence of aggregate size
S	Girder spacing measured perpendicular to a concrete beam
STIP	State Transportation and Improvement Plan
V_c	Nominal shear resistance provided by tensile stresses in concrete
V_p	Component in the direction of the applied shear of the effective prestressing force (taken as positive if resisting the applied shear)
V_s	Shear resistance provided by shear reinforcing steel
V_u	Factored shear force in a prestressed concrete beam at the location under consideration
w_c	Unit weight of cured concrete
β	Factor relating the effect of longitudinal strain on the shear capacity of concrete
ε_s	Longitudinal tensile strain in concrete at the centroid of the tension reinforcement
λ	Statistical bias factor to account for the actual versus specified concrete strength
γ_{DC}	Load factor for DC loads
γ_{DW}	Load factor for DW loads

γ_{LL}	Load factor for LL loads
θ	Skew angle of the bearing lines at the end of a given span, measured from a line taken perpendicular to the span centerline; angle of inclination of diagonal compressive stresses

EXECUTIVE SUMMARY

Lower than desirable shear ratings at the ends of prestressed concrete beams, particularly for bridges designed in accordance with the 1979 and earlier provisions of the American Association of State Highway and Transportation Officials' Specifications for Highway Bridges, have been the topic of ongoing research between MnDOT and the University of Minnesota. An improved understanding of this topic and a computational method to increase the load rating of prestressed concrete bridges governed by shear can help MnDOT's Bridge Office better plan and prioritize bridge repair projects. Additionally, increased load ratings would have a positive impact on freight mobility by reducing unnecessary overweight truck rerouting, detouring, and permit denials. Several studies have been commissioned by MnDOT to better understand the apparent disconnect between the insufficient load rating result, based on more recent AASHTO provisions, and the lack of any observable shear deficiency noted during in-service field inspections.

The most recent of these studies was a report by the University of Minnesota entitled *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* (French et al., 2016). The study sought to increase the shear rating of a prestressed concrete beam via a refined live load distribution factor, considering the location-based load distribution of each axle along the span. The study also presented a screening tool that could be used to determine the likelihood that the shear rating of a given prestressed concrete beam bridge could be improved based on the refined methodology.

The primary objective of the research presented within this report was to implement the refined shear rating methodology presented by French et al. (2016) in the University of Minnesota report. MnDOT selected 522 bridges from its inventory noted as having shear rating deficiencies for potential re-evaluation as part of this research. After employing the screening tool, the initial selection was reduced to 127 applicable bridges, of which 50 were selected for re-evaluation using the refined rating methodology.

For the 50 bridges evaluated, the refined rating methodology was found to improve the shear ratings by an average of 16 percent. The screening tool also proved to be an effective means of determining the candidacy of a given bridge for re-evaluation. A general correlation was shown between the results of the screening tool and the likelihood for shear rating improvement. However, the correlation was not strong enough to permit an accurate prediction of the magnitude of rating improvement.

Finally, a quantified benefit analysis of the refined rating methodology was performed. The analysis compared the implementation of the refined rating methodology to a physical repair method, namely the use of a carbon fiber reinforced polymer (CFRP) wrap, to improve the shear rating for a given prestressed beam bridge. The analysis revealed an average cost savings of approximately $68,128, or 66 percent, per bridge by employing the refined rating methodology as presented.

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Given the relatively large inventory of prestressed girder bridges in Minnesota, the topic of shear capacity and its effect on load ratings has been of acute significance to MnDOT. Several studies have been commissioned by MnDOT to better understand the shear behavior at the ends of prestressed beams and to explore methods for shear load rating refinement. Increasing the load rating of a bridge governed by shear can help the Bridge Office better plan and prioritize bridge repair projects and have a positive impact on freight mobility by reducing unnecessary overweight truck rerouting, detouring, and permit denials.

Below is the rating factor equation, per the Load and Resistance Factor Rating (LRFR) procedures of the *AASHTO 2010 Manual for Bridge Evaluation (MBE) 6A.4.2.1.* As shown, the rating factor for a given bridge can be improved by either improving the capacity, C, such that the numerator of the rating equation is increased or by refining the live load demand, LL+IM, in an effort to decrease the denominator. Most of the previous shear rating research conducted by MnDOT focused on the former, attempting to improve the predicted shear capacity of a prestressed beam section.

$$RF = \frac{C - \gamma_{DC}(DC) - \gamma_{DW}(DW)}{\gamma_{LL}(LL + IM)}$$

However, a recent study, detailed in a report titled *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges*, by French et al. (2016), focused on the applied demand rather than the capacity. Built on the findings of previous studies that focused on the accuracy of the shear capacity predicted by AASHTO equations, this study focused on improving the shear rating of a prestressed beam primarily by refining the calculation for the amount of live load distributed to each girder (live load distribution factor). Since it was found that not all bridges could be improved via the refined rating distribution factor methodology, a screening tool was presented to better determine the applicability of the methodology. The report also provided several, relatively smaller modelling assumptions that can be used to improve the overall accuracy of a bridge rating.

1.2 OBJECTIVES AND SCOPE

The primary objective of this research was to implement the findings and recommendations of French et al. (2016) regarding the use of a refined live load distribution factor for shear at the end of a prestressed beam. A list of 522 prestressed girder bridges, noted as having end of beam shear deficiencies, was provided by MnDOT. From this initial list, 50 bridges were selected based on the screening tool presented by French et al. (2016) for reanalysis using the refined shear rating methodology. The results of these analyses were then used to better establish the validity of the refined methodology to improve the bridge shear rating, as presented by French et al. (2016). Finally, a quantified benefit analysis was performed, comparing the refined rating methodology to a physical repair as a means of improving the shear rating for a typical bridge.

1.3 ORGANIZATION

Chapter 2 summarizes the findings of previous studies regarding the shear capacity and live load distribution factors of prestressed concrete beams. The previous studies explored as part of this chapter are the direct predecessors to the research discussed herein.

Chapter 3 discusses the selection criteria used to determine which bridges were eligible for re-evaluation as part of this study. The details of the refined rating methodology, including the computation of a refined live load distribution factor, improved effective member stiffness, and modifications to the existing Bridge Rating files, are also discussed in this chapter.

Chapter 4 presents the results of the implementation of the refined rating methodology. The overall effectiveness of the refined methodology, as well as the selection criteria, are discussed in detail.

Chapter 5 estimates the quantified benefits of the full deployment of the refined rating methodology for the statewide bridge system. The cost to implement the refined rating methodology is estimated and compared against the estimated cost to execute a typical prestressed concrete beam shear repair.

Chapter 6 summarizes the key findings and provides final recommendations regarding the use of the refined rating methodology to improve the shear ratings of prestressed concrete girder bridges.

Appendices, used to supplement the preceding chapters, are attached at the end of this report.

CHAPTER 2: FINDINGS OF PREVIOUS STUDIES

Shear at the ends of prestressed concrete beams, and the possibility of lower than desirable load ratings for bridges designed according to the AASHTO bridge design provisions of 1979 and earlier, has been the topic of ongoing research between MnDOT and the University of Minnesota. Recently, three studies were completed that built upon one another to better understand the intricacies of the current and historical shear design specifications and their effect on the shear performance of in-service bridges. The research also sought methods to either validate the performance of these older bridges, despite their low load ratings, or refine the rating methods to more accurately reflect their behavior. A summary of these studies, as applicable to this report, is presented in the following sections.

2.1 SHEAR CAPACITY OF PRESTRESSED CONCRETE BEAMS

The report *Shear Capacity of Prestressed Concrete Beams* (Runzel et al. 2007) was the first in the series of reports conducted for MnDOT by the University of Minnesota regarding the shear capacity of prestressed beams. At the time of the report, the current AASHTO design provisions, both 2002 Standard and 2004 LRFD, indicated that girders designed per the 1979 interim provisions may not meet the shear strength required for HL-93 or HS20-44 loading between the support and 10 percent of the span length away from the support (0.1L). The goal of the research was to investigate the differences between the shear strengths calculated using the 1979, 2002, and 2004 bridge specifications. The predicted shear strengths were compared against experimental data to determine whether in-service bridges designed using the 1979 provisions were at risk of having low shear capacities.

As the level of understanding of the shear behavior of prestressed concrete bridges increased, the shear provisions found in the 1979 specifications were revised in both the 1983 and 1994 specifications. These changes resulted in an increase in the amount of shear reinforcement required near the ends of the beams. The primary reason for the increase in required reinforcement was that the location of the critical section for shear was moved towards the end of the beam, where the demand is greater. The 1979 provisions specified a critical shear location of one quarter of the overall span length. The amount of shear reinforcement required at the critical shear location was then permitted to be applied between the quarter point and the end of the beam. Given that the demand is less at one quarter of the span than at a distance closer to the support, this resulted in significantly less shear reinforcement required between the critical location and the end of the beam than would be required by the current provisions.

In addition to the location of the critical section, the shear capacity predicted by the current provisions was refined in order to better predict the capacity provided by the concrete, prestressing, and shear reinforcement. The 1979 specifications simplified the computation of the shear capacity provided by prestressing by specifying that the shear capacity due to shear reinforcement be doubled to account for any prestressing effects. Furthermore, no defined limit was placed on the amount of shear reinforcement that could be provided in the section, under the assumption that the shear capacity could be increased indefinitely with an increasing amount of reinforcement. This provision was particularly unconservative as it could lead to a brittle shear failure due to crushing of internal concrete shear struts. One aspect of the 1979 specifications that was conservative, however, was the upper limit that was placed on the predicted shear capacity attributed to the concrete. This provision often underestimated the concrete shear capacity, particularly for concrete strengths greater than 3 ksi. Even with this conservative assumption, the 1979 specifications were generally unconservative and provided a less reliable prediction of the shear capacity when compared to more recent specifications.

In order to better understand the consequences of the 1979 shear provisions and the reliability of the 1979, 2002, and 2004 provisions, laboratory testing was performed on a girder obtained from an out-of-service MnDOT bridge that was likely designed using the 1979 provisions. A single concentrated load was placed on the test girder at a shear span-to-depth ratio of approximately 2.7 to prevent a possible increase in shear capacity due to arching action. The 1979 provisions were found to provide a more conservative prediction of total shear strength when compared to the 2002 and 2004 provisions. However, the predictions of the 2002 and 2004 provisions were found to be significantly more reliable, while still conservative. Between the 2002 and 2004 provisions, the 2002 provisions provided the closest prediction of the total shear strength. Additionally, modifying the 2002 provisions to account for the measured angle of principal compression, instead of the assumed angle of 45 degrees, when computing the shear resistance provided by the shear reinforcement produced a predicted total shear capacity that was nearly identical to the results of the laboratory testing. Though the laboratory testing demonstrated that the shear capacity of the girder was adequate for current loading requirements, the results could not be used to broadly state that all girders designed using the 1979 provisions would provide desirable load ratings.

A parametric study of typical MnDOT bridge girders designed using the 1979 provisions was performed in order to predict the likelihood that a given girder would have an inadequate shear strength for current loading and design requirements. The parametric study indicated that bridges with short span lengths and a large spacing between girders were more likely to be under-designed for shear. This was due to a direct correlation between the span length, required amount of prestressing, and overall shear strength. Additionally, the shear demand was directly related to the girder spacing. Therefore, bridges with a low ratio of span length to girder spacing were likely to have a greater discrepancy between the provided shear capacity and demand. Bridges detailed per the 1979 provisions and with a span length to girder spacing ratio greater than 10 were shown to have adequate shear strength, while a ratio less than 8.5 indicated a susceptibility to inadequate shear strength that would require further analysis.

2.2 DISCREPANCIES IN SHEAR STRENGTH OF PRESTRESSED BEAMS WITH DIFFERENT SPECIFICATIONS

The report *Discrepancies in Shear Strength of Prestressed Beams with Different Specifications* (Dereli et al. 2007) was the companion piece to the report written by Runzel et al. (2007). Picking up where Runzel left off, the report investigated possible discrepancies related to the rating methods and rating software (Virtis) that resulted in inadequate design level shear ratings despite no sign of shear distress noted on the corresponding inspection reports. The researchers selected 54 bridges, that had been identified within MnDOT's inventory as being at risk for inadequate shear capacity, for further evaluation and rating per the 2002 AASHTO Standard Specifications. The analysis of these bridges was also used to ascertain the validity of the screening tool presented by Runzel et al. (2007). Finally, additional means of reserve shear capacity and live load distribution factor refinement were discussed.

The primary objective of the research was to validate the screening tool, defined by Runzel et al. (2007), against the rating results of 54 bridges that were identified to be at risk for inadequate shear capacity. The bridges analyzed within this study were checked against the design code listed on their plans to verify that the shear reinforcement was correctly detailed. As noted by Runzel et al. (2007), some of the girders designed according to the 1979 provisions had shear reinforcement that was detailed at a larger spacing than what was required by design. In fact, roughly 50 percent of the bridges that were designed per the provisions ranging from 1965 to 1979 were found to not meet the shear design requirements of the time and, thus, had an inadequate shear capacity. The source of these errors, however, could not be traced. Therefore, using the span length to girder spacing ratio, as defined by Runzel et al. (2007) for properly designed girders, as a screening tool to assess the adequacy of a girder for shear was found to be ineffective. Additionally, the shear analysis of the 54 bridges revealed that some of the bridges with low span length to girder spacing ratios had a high capacity to demand ratio and vice versa, which violated the predictions of the screening tool put forth by Runzel et al. (2007). However, in general, the predicted correlation between span length to girder spacing ratio and the capacity to demand ratio, though not as strong as anticipated, was shown. As a result, the report recommended that the span length to girder spacing ratio be used only as a preliminary screening tool to prioritize bridges for further analysis, but that an in-depth investigation of the shear capacity of each bridge be performed.

Additional means of reserve shear capacity, not accounted for within the Virtis rating analysis or 2002 Standard Specifications, that were investigated included the contribution of the web end block, nominal vs. 28-day vs. actual concrete strength gained over time, and arching action for shorter shear spans. Methods for refining the shear demand used for rating were also investigated and included refining the live load distribution factors and incorporating the effect that the end diaphragms have on the force distribution at the end of the span. A summary of the findings for each of these topics is as follows.

The web end blocks were not found to significantly alter the shear rating results. Typically, web end blocks were only found on deeper sections that already had an adequate shear inventory rating. Conversely, shallower girders, which were more susceptible to having an inadequate shear inventory rating, typically did not have web end blocks. Additionally, the end blocks typically were either tapered or had already terminated at the critical shear location of h/2. Therefore, the impact that they had on the shear capacity of the section was often minimal.

Since prestressed beams are required to obtain a specific concrete compressive strength at the time of strand release, the 28-day strength is often greater than the specified design value, as the mix is controlled by achieving the initial strength as quickly as possible. Statistical bias factors, presented by Nowak and Szerszen (2003), to account for this variation were developed based on mix design strength data that was obtained nationally from precast concrete plants. The national data was compared against historical strength data obtained from the Cretex precast plant in Elk River, Minnesota, as that was where the majority of the in-service prestressed concrete beams were cast. The comparison indicated that the statistical parameters of the Elk River mixes agreed with those determined by Nowak and Szerszen (2003) on a national level. However, this variation in actual concrete strength is already accounted for in the reliability of the AASHTO LRFD equations as part of the resistance factor calibrations. Since the 28-day strength data for bridges within Minnesota were shown to agree with the data used for calibration, an additional amount of statistical variation should not be accounted for as a source of reserve strength.

Previous studies indicated that the concrete strength does continue to increase over time due to the ongoing hydration process. The amount of strength gain is a function of several variables, including the curing process and cement type. Unlike the statistical variation of the mix design, this strength gain was not accounted for within the development of the AASHTO LRFD equations. Therefore, based on previous research, a lower-bound increase in concrete compressive strength of 20 percent was recommended for concrete at least 20 years of age. A reanalysis of girders with relatively low rating factors showed that the incorporation of this 20 percent increase in concrete strength resulted in an average increase in shear rating at the critical section of approximately 6 percent.

Arching action was only appropriate when the load was applied within 2.5 girder depths from the support. Typically, the critical section for shear, h/2, had an inadequate rating factor even as the live load was located further than 2.5 girder depths away from the support. Therefore, arching action generally did not prove to be an effective means of increasing the shear ratings.

Various simplified methods for decreasing the estimated live load demand were also considered as a means of improving the shear ratings. An overview of the live load distribution factors of the 2002 Standard and 2004 LRFD provisions, Henry's equal distribution factor method, modified Henry's method, lever rule, and a calibrated lever rule (NCHRP method) were presented. A comparison of each of the methods determined that the live load distribution factors of the 2002 provisions generated the minimum live load demand. Additionally, it was found that the two-lane-loaded case governed for all the bridges and methods considered. Thus, attempts to refine the live load distribution factors via a simplified method were unsuccessful.

Finally, a literature review indicated conflicting results on the effect that end diaphragms have on live load distribution. Therefore, due to this uncertainty, the use of the end diaphragms to decrease the live load shear demand near the end of the beams was not considered. The report recommended that the influence of the end diaphragms on the live load distribution factors be validated experimentally to gain a better understanding of their behavior.

2.3 INVESTIGATION OF SHEAR DISTRIBUTION FACTORS IN PRESTRESSED CONCRETE GIRDER BRIDGE

The report *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* (French et al. 2016) built upon the research performed by Runzel et al. (2007) and Dereli et al. (2010) and was the primary source of information for this study. The report presented a method for improving shear ratings at the ends of prestressed concrete beams by way of refining the live load distribution factors used for the rating analysis. The results of laboratory testing and detailed finite element modeling were used to validate the methodology and present a simplified approach to determine the refined live load distribution factors. Interior girders were the focus of this research, as they were assumed to govern the shear rating analysis.

Previous AASHTO approximate live load distribution factor equations for shear in prestressed concrete beams with concrete decks were based on empirical information and were developed based on a single HS20 truck. Generally, the AASHTO approximate distribution factors were only a function of the beam spacing and did not consider the longitudinal positioning of the individual axle loads. Additionally, other parameters that affect shear distribution, namely the longitudinal and transverse bridge stiffness, girder dimensions and spacing, skew, etc. were not considered. As a result, the true distribution behavior of the bridge was not often captured as accurately as possible. Furthermore, the same distribution factor was applied to every axle, creating an averaging effect rather than considering the effect that its location has on its transverse distribution.

The report proposed the derivation of a refined live load distribution factor using an axle based approach. Rather than applying a single distribution to every axle, this approach sought to account for the positioning of each axle and the reduced live load demand that resulted. As the individual axle loads were placed further from the support, it was found that more of the bridge cross section was engaged, thereby increasing the transverse force distribution among girders and decreasing the shear demand at the end of a given girder. Skew was accounted for by applying a simplified version of the AASHTO LRFD correction factor for skews greater than 30 degrees.

In order to determine which bridges would benefit from the axle based distribution method, a screening tool was developed. A dimensionless ratio of the longitudinal to transverse superstructure stiffness was derived and used to screen the candidacy of each bridge. It was determined that bridges with a screening tool ratio of less than 1.5 were likely to benefit from the axle based distribution method. The typical AASHTO methods of live load distribution were recommended for bridges not passing this initial screening tool criteria.

The results of the laboratory testing and detailed finite element modelling confirmed the results of the refined rating methodology and screening criteria and were used to generate a simplified method for computing the axle-based distribution factors. The final recommendation consisted of a two-dimensional finite element grillage analysis of the superstructure with HS20 axle loads positioned per the lever rule to maximize the live load demand on the first interior girder. Though the inelastic laboratory testing indicated that shear redistribution occurred as the girder stiffness decreased, a linear elastic analysis, using gross section properties was conservatively recommended. A more detailed discussion of the derivation of the refined live load distribution factors and associated finite element grillage modelling is presented within this report.

The researchers also investigated the effect that secondary elements, such as traffic barriers and end diaphragms had on the shear distribution across the section. Additionally, the effect that torsion of the section had on the shear demand for a given girder was investigated. Traffic barriers were found to carry shear load when the axles were spaced directly over the exterior girder line or as close to the barrier as possible. However, the effectiveness of the barrier to transmit shear was decreased as the distance from the point of load application to the barrier increased, to the extent that the barrier was typically ineffective when the loads were placed over the first interior girder line. The end diaphragms were found to increase the shear demand at the ends of the girders by approximately 4 to 6 percent. Torsional effects were also found to increase the total shear demand, particularly when the load was placed between girder lines, thereby increasing the torsion effect for the adjacent girders. However, since the live load was typically placed directly over the first interior beam in order to maximize the vertical shear demand, the increased torsional demand due to shear effects was relatively minimal. Intermediate diaphragms, if properly connected, were also found to influence shear distribution. It was found that well-connected intermediate diaphragms acted to reduce torsion effects within the section, thereby decreasing the shear demand for a given beam. However, it was concluded that additional study was required to better understand this behavior. Therefore, in order to simplify the refined live load distribution analysis, it was determined that these secondary effects could reasonably be ignored.

CHAPTER 3: METHODOLOGY

3.1 SELECTION OF BRIDGES FOR STUDY

Historically, if a prestressed concrete bridge rating was governed by shear, but the field inspection did not indicate any signs of shear distress, MnDOT directed the rating engineer to ignore shear. This rationale was consistent with MBE 6A.5.8, which states that, "In-service concrete bridges that show no visible signs of shear distress need not be checked for shear when rating for the design load or legal loads." Therefore, the BrR analysis settings were typically modified such that shear was not considered or, "turned off," during the rating analysis, resulting in a bridge rating governed by flexure.

Due to this practice, MnDOT has an inventory of prestressed concrete rating files that do not consider shear as part of the rating analysis. 522 of these bridges were selected by MnDOT as candidates for re-evaluation as part of this study. Of the 522 bridges, 50 were selected for a refined analysis based on a combination of the results of the screening tool developed by French et al. (2016), their priority as indicated by the State Transportation Improvement Program (STIP) list provided by MnDOT, the superstructure NBI condition rating, and general consistency with the bridges studied as part of the report *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* (French et al. 2016). Further explanation of each of these selection criteria is provided in the following sections.

Example calculations for the required refined methodology input, as well as the computation of the refined live load distribution factor, are shown in APPENDIX B
Sample Calculations (BR 53005). Additionally, an example input file for the creation of the STAAD two-dimensional finite element grillage model is provided in APPENDIX C
Sample STAAD Input (BR 53005).

3.1.1 Criteria for Bridge Selection

3.1.1.1 Implementation of Screening Tool

As mentioned in section 2.3 , a screening tool was developed as part of the University of Minnesota report *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* (French et al. 2016). The screening tool was used as the primary selection criterion, as it determined whether the methodology used to refine the live load distribution factor would result in a decreased shear demand. Bridges that did not pass this initial screening process were not considered for further analysis, as the refined methodology would not likely result in an improved shear rating.

The screening tool required the computation of a dimensionless ratio of the longitudinal flexural stiffness to the transverse flexural stiffness, simply referred to as the, "Stiffness Ratio." Bridge information was collected from the existing BrR files and existing bridge plans and used to compute the following ratio:

$$\text{Stiffness Ratio} = \frac{I_{long}}{L^3}\frac{S^3}{I_{trans}}$$

The longitudinal stiffness, I_{long}, was computed for a typical interior beam and was based on the gross composite stiffness of the prestressed beam and tributary deck section. The average stool thickness, minimum deck thickness, and the plan specified concrete compressive strengths for both the beam and deck were used to compute the stiffness of the transformed composite section. The transverse stiffness, I_{trans}, was computed for a 12" wide section of the bridge deck, oriented perpendicular to the longitudinal bridge axis. The minimum deck thickness was used to compute the gross transverse stiffness. The remaining parameters consisted of the span length in feet, *L*, and the girder spacing in feet, *S*.

The stiffness ratio was computed for all 522 bridges, which were then sorted in order of ascending stiffness ratio. The parametric study performed as part of the University of Minnesota report indicated that the refined distribution methodology benefitted bridges with a stiffness ratio less than 1.5 (French et al. 2016). Therefore, of the 522 bridges, only those that had a stiffness ratio less than 1.5 were considered for further analysis.

While most of the bridges within this study consisted of a single span, it should be noted that additional consideration was given to multiple span bridges. Several cases were encountered in which some of the spans produced a stiffness ratio less than 1.5 and the other spans produced a ratio greater than 1.5. In this scenario, it was necessary to evaluate the existing shear rating, without any refinement, for each span. If the span that governed the overall bridge rating had a stiffness ratio less than 1.5, then the bridge was considered for further analysis. However, if the governing span had a stiffness ratio greater than 1.5, the bridge was classified as not passing the screening test, as the refined distribution methodology would only benefit the non-governing spans, while the rating for the governing span would remain unchanged.

3.1.1.2 Presence on STIP List

The State Transportation Improvement Program (STIP) list provided by MnDOT indicated bridges that were scheduled for funding within the state fiscal year, as part of a four-year transportation improvement program. Of the bridges with a stiffness ratio less than 1.5, MnDOT placed a rating priority on any bridge that was already on the STIP list. The intent of the priority was that the outcome of the rating could potentially affect the proposed scope of work for a given bridge. Additionally, any issues related to the shear rating could potentially be mitigated as part of the proposed scope. It should be noted that none of the bridges that passed the screening tool were found on the STIP list.

3.1.1.3 NBI Rating

The remaining bridges were further sorted by the NBI superstructure condition rating, as listed on the current bridge inventory report. Priority was given to bridges with lower superstructure NBI condition ratings. Similar to the STIP list, this prioritization was done with the assumption that bridges in poorer condition were more likely to be scheduled for funding and repair work than a similar bridge in better condition. As much as possible, bridges were selected for further analysis in ascending order based on their NBI condition rating.

3.1.1.4 Consistency with University of Minnesota Report Assumptions

Finally, the bridges were evaluated on a case-by-case basis for re-evaluation based on their similarity to the assumptions and conditions outlined in the University of Minnesota report by French et al. (2016). The bridges investigated in the University of Minnesota report were relatively simple structures with uniform cross sections. Therefore, to be consistent with the report, bridges with splayed beams, widened decks, a combination of beam sizes, beams with differing concrete strengths, etc. were generally avoided as part of this study.

3.2 REFINED FINITE ELEMENT MODELING

Based on the aforementioned criteria, 50 bridges were selected for re-evaluation using the refined distribution methodology outlined in the University of Minnesota report. As mentioned in section 2.3 the refined live load distribution methodology accounted for the placement of each axle to determine a more accurate live load distribution factor, compared to the averaging effect employed by the AASHTO approximate equations. The first step of the refined method involved creating a two-dimensional finite element grillage model, which was then used to compute a more accurate live load distribution factor. The following is a detailed description of each of the components of the finite element analysis.

3.2.1 Material Properties

Since gross section properties were used for both the longitudinal and transverse elements of the grillage model, the concrete strengths for the deck and prestressed beams were the only relevant material properties at this stage of the analysis. The concrete compressive strength of the deck and the final concrete compressive strength for the prestressed beams, f'_c, were obtained from the existing bridge plans. Both values were modified for a minimum assumed strength gain of 20 percent and were multiplied by a statistical bias factor, λ, used to account for mix design variability between the actual strength and the specified design strength (French et al. 2016). Per MnDOT direction, the assumed 20 percent gain over time was only applied to bridges that were 20 years and older.

As mentioned in French et al. (2016) and discussed in greater detail in Nowak and Szerszen (2003), there is often variability between the plan specified concrete strength and the actual as-built strength. This variability can typically be attributed to the quality of materials, workmanship, curing procedures, conservative mix design, etc. The statistical bias factors determined by Nowak and Szerszen (2003) assumed an average level of material quality and workmanship and were based on samples of ready mix and precast concrete. It was found that concrete mixes were often designed for a particular strength at a point in time less than the typical 28-day plan value. Thus, the actual concrete strength at 28-days was often greater than the nominal design specified strength. This difference in strength was found to be particularly true for older bridges. The statistical bias factors assumed for the prestressed beam and the deck, per Nowak and Szerszen (2003) and based on precast and ready mix concrete respectively, are presented in APPENDIX A
Statistical Bias Factors. Linear interpolation was used to determine the statistical bias factor for concrete strengths not explicitly listed in the tables. Additionally, since the statistical bias factor becomes asymptotic for higher strength ordinary plant-cast concrete, the factor for precast concrete strengths greater than 6,500 psi, but less than 7,000 psi was taken as 1.14.

The modified concrete strengths were ultimately used to transform the longitudinal deck and beam element into a single composite section. This was accomplished by computing the modulus of elasticity for each of the elements independently, based on the modified strength values, and using the modulus of the beam as the basis for the transformed composite section. The equation used to compute the modulus of elasticity for each element is provided below, per 2014 AASHTO LRFD 5.4.2.4 for normal weight concrete.

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f'_c}$$

3.2.2 Element Stiffness Calculations

In order to obtain an accurate distribution of the shear forces across the section, the composite stiffness of each longitudinal element was required for the two-dimensional grillage model. Similar to the computations required for the screening tool, the average stool thickness, minimum structural deck thickness, and effective flange width (per 2014 AASHTO LRFD 4.6.2.6) for each longitudinal beam element were used to compute the gross composite stiffness of a given section. However, unlike the stiffness computed for the screening tool, the modified concrete strengths were used to transform the stool and effective deck width to that of the longitudinal beam element to compute the stiffness of the composite section.

Most of the 50 re-evaluated bridges were of uniform cross section. Therefore, the stiffness of the exterior beam and a typical interior beam were usually the only computations required for the two-dimensional grillage analysis. There were a few instances in which the beam spacing was not uniform between longitudinal elements. These bridges required additional longitudinal stiffness computations for each unique composite section in order to better reflect the overall stiffness of the system.

3.2.3 Two-Dimensional Grillage Model

A finite element model of the superstructure was used to more accurately determine the shear demand at the end of a given beam using an axle approach, compared to the averaging approach used by the approximate methods of 2014 AASHTO LRFD Chapter 4. Per the final recommendations of the University of Minnesota report, a simple two-dimensional grillage model of the superstructure was shown to be effectively as accurate as a more refined, and more complicated, three-dimensional form of analysis. Therefore, the finite element software STAAD.Pro V8i was used to create a two-dimensional finite element model.

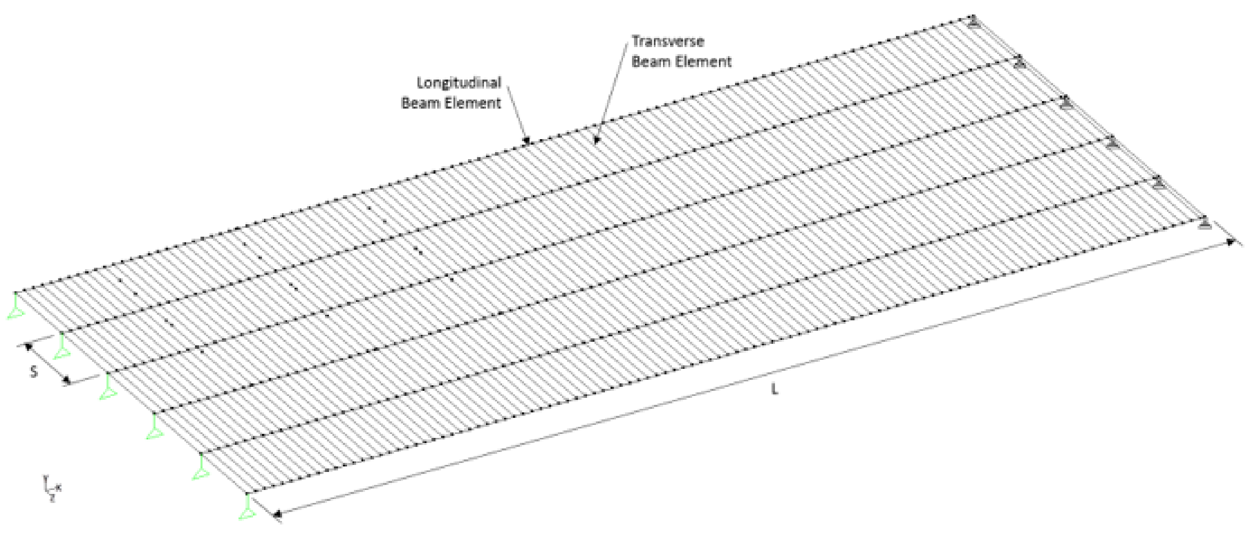


Figure 3.1 Two-Dimensional Grillage Model

The two-dimensional finite element grillage model consisted of a network of longitudinal and transverse beam elements, as shown in Figure 3.1. The longitudinal elements represented the composite beam section with a modulus of elasticity equal to that of the modified beam concrete and a composite stiffness as described above. The transverse elements were placed at 12 inch increments along the longitudinal elements and represented the transverse stiffness of the deck. The transverse beam elements were modeled using a 12-inch rectangular section with a height equal to the structural thickness of the deck and a modulus of elasticity equal to that of the modified deck concrete.

Simple supports were provided at the end of each beam while the deck elements were continuous over each beam. This network of beam elements modeled the ability of the deck to distribute load to adjacent beams and to take into consideration the position of each axle when computing the live load distribution factor for shear at the end of the beam.

3.2.4 Effect of Skew

Per the University of Minnesota report, the bridge skew, intermediate and end diaphragms, and stiffening effect of the barriers were all ignored within the two-dimensional grillage model. The primary reason for this was to simplify the model. Depending on the amount of skew, an adjustment was made to the distribution factor to account for skew effects, as detailed in subsequent sections.

3.3 CALCULATION OF REVISED LIVE LOAD DISTRIBUTION FACTORS

Once the two-dimensional finite element grillage model was created, axle loads were placed on the superstructure to maximize the shear demand for a given beam. From there, the live load shear demand was used to compute a refined live load distribution factor, which was used for all trucks to be rated. A detailed discussion of each phase of this process is presented herein.

3.3.1 Live Load Configuration

The only truck that was considered as part of this analysis was the HL-93 design truck, which is equivalent to the HS20 truck used in the 2002 Standard Specifications. For simplicity, the live load distribution factor obtained from the design truck analysis was conservatively applied to the lane load portion of the HL-93 loading as well as all routine and special permit trucks as required to complete MnDOT’s bridge rating and load posting reports. The live load distribution factor could be refined for each specific truck, however that level of detail was beyond the scope of this project. Additionally, this distribution factor was also applied to the lane load portion of the HL-93 loading used for the load and resistance factor rating method.

The wheel loads, spaced six feet transversely per 2014 AASHTO LRFD 3.6.1.2.2, were positioned such that the demand was maximized transversely based on the lever rule. This configuration varied based on the beam spacing and whether one or two lanes of live load were considered. Governing wheel positions for each configuration, as presented by French et al. (2016), are shown in Figure 3.2. It should be noted that, though the two-lane configuration was shown to govern the shear rating for an interior beam by French et al. (2016), the single lane configuration was necessary to compute the live load distribution factor required for special permit trucks. Additionally, a study by Puckett et al. (2007) found that the two-lane configuration, when including the effects of multiple presence, typically governs over the configurations involving three or more lanes. Furthermore, three-lane configurations that did happen to govern over the two-lane configuration typically only produced a demand that was approximately 10 percent greater than the two-lane configuration.

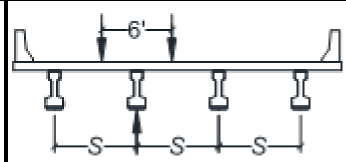
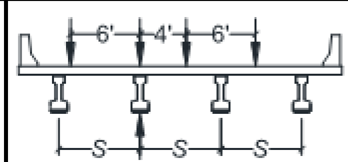
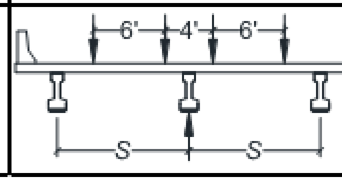
Number of Lanes Loaded	Resultant Load on Interior Girder at Cross Section where Loads are Applied (indicated by arrow under girder in diagram)	Application Range	Loading Diagram
1	$1 - 3/S$	$S > 6$ ft	
2	$3/2 - 5/S$	$6 < S \leq 10$ ft	
2	$2 - 10/S$	$10 < S \leq 16$ ft	

Figure 3.2 Axle Positions to Maximize Shear in Interior Beams According to the Lever Rule (French et al. 2016)

Longitudinally, the axle loads were positioned to maximize the shear demand near the support. While there has historically been much discussion as to the proper definition of "near the support", French et al. (2016) recommended a critical location of one-tenth the span length for consistency. This location approximated the length of the disturbed region near the end of the beam, equal to the depth of the composite section. Therefore, the heaviest, rear axle was placed at 0.1L when computing the maximum live load shear demand for a given beam, as shown in Figure 3.3.

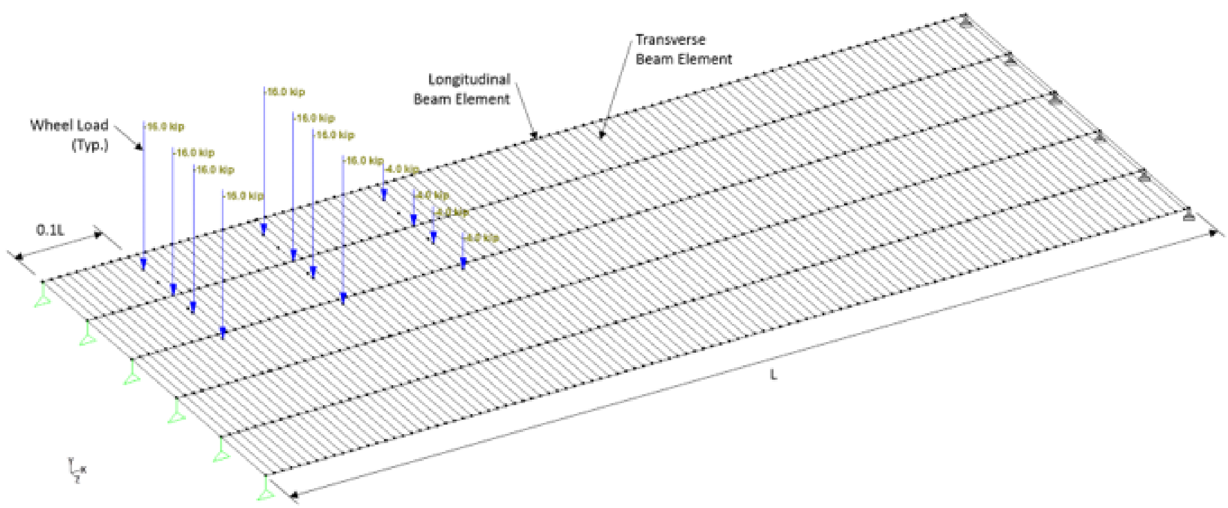


Figure 3.3 Typical Grillage Model Loading Configuration for Two Lanes

3.3.2 Interior Girder Live Load Distribution Factor

Consistent with the assumptions and focus of the University of Minnesota report, the shear demand of the first interior beam typically governed the rating. Therefore, the live load reactions at the first interior beam were used to compute the refined one and two-lane live load distribution factors. For simplicity, the live load distribution factors for the first interior beam were conservatively applied to all other interior beams if the bridge was of uniform cross section.

The live load distribution factor was found by computing the ratio of the live load reaction obtained from the two-dimensional grillage model to that of an identically loaded simply supported beam, assumed to carry the entire live load. For simplicity, the live load at the reaction, instead of the demand at the critical section of 0.1L, was used to compute the live load distribution factor. Reaction values from the two-dimensional grillage analysis, $R_{predicted}$, were recorded for both the single lane and two lane live load configurations. The reaction for a simply supported beam of equivalent span length and with design truck axle loads also placed at 0.1L was then computed by hand via linear analysis techniques. This reaction represented the upper bound live load demand for a single beam element, R_{max}. Once both values were obtained, the following equation was used to compute the live load distribution factor, LLDF. In order to account for the likelihood of additional loading in the design lanes, the live load distribution was also multiplied by the appropriate multiple presence factor, MPF, per 2014 AASHTO LRFD 3.6.1.1.2.

$$LLDF = \frac{R_{predicted}}{R_{max}} * MPF$$

Bridges with non-uniform cross sections required an iterative approach to determine the governing live load distribution factor. An initial rating analysis of the bridge was performed using unrefined live load distribution factors to determine which beam produced the minimum, governing shear rating. The two-dimensional grillage analysis was then performed with the live load placed such that the demand on the governing beam was maximized and the refined live load distribution factors were computed. The bridge was re-evaluated using the refined live load distribution factors for the governing beam, while the rest of the beams maintained their initial, unrefined values. If the beam that initially governed the rating analysis still governed, then the analysis was complete and no additional distribution factors were computed. However, if the reduction in live load resulting from the refined distribution factor caused a different beam to govern the rating, the process was repeated for the newly governing beam. This cycle was repeated until the use of the refined distribution factors converged on a single, governing beam.

3.3.3 Exterior Girder Live Load Distribution Factor

Since refined live load distribution methods were not addressed for the exterior girders as part of the University of Minnesota report, refined distribution factors were only computed for the interior girders. Generally, the exterior girders, even with unrefined distribution factors, did not govern the rating analysis, save for a few instances involving special permit trucks. Therefore, for simplicity and consistency with the University of Minnesota report, the exterior girder live load distribution factors were unchanged from the initial AASHTO approximate values.

3.3.4 Skew Correction

Skew was ignored during the creation of the two-dimensional grillage model to simplify its construction. This was necessary as the transverse beam elements are typically required to be placed perpendicular to the longitudinal beam elements in order to correctly capture the distribution of shear forces across the section (French et al. 2016). Orienting the beam elements to capture the structure skew would result in an undesirable amount of modelling complexity near the supports.

However, the effects of skew could not be completely ignored. 2014 AASHTO LRFD 4.6.2.2.3c specifies that the distribution factor for shear in exterior and interior beams should be increased by a correction factor at the obtuse corner and decreased linearly to a factor of unity at mid-span. AASHTO also states that if the beams are well connected and behave as a unit, only the exterior and first interior beams need to be adjusted. Furthermore, AASHTO specifies that this correction factor should not be applied in addition to modeling skewed supports. This increase in shear demand is due to a shortened load path to the support at the obtuse corner. However, the effect that skew has on the shear demand for an interior girder is more complicated, as the increase in shear due to skew is reduced as the distance from the obtuse corner is increased.

In an effort to simplify this issue, it was recommended that the bridge be modeled as a straight bridge with an equivalent span length and beam spacing and that the skew correction factor be conservatively applied to the computed interior beam live load distribution factor. The University of Minnesota report presented a version of the 2014 AASHTO LRFD skew correction factor equation for concrete decks on concrete beams that was simplified by Puckett et al. (2007). Though the AASHTO equation specifies a range of applicability for skews between 0 and 60 degrees, studies have shown that skews less than 30 degrees have little, if any, effect on the shear distribution for interior beams (French et al. 2016). Therefore, the skew correction factor presented by Puckett et al. (2007), and provided below, was only recommended for bridges with a skew greater than 30 degrees. This approach to correcting for skew in interior beams was shown to be conservative by French et al. (2016) when compared to the results of finite element analyses of skewed bridges. For simplicity, the skew correction factor was applied uniformly over the entire beam, instead of linearly varying the factor as prescribed by AASHTO.

$$Skew\ Correction = 1.0 + 0.09 * tan\theta$$

3.4 BRIDGE RATING SOFTWARE REFINED INPUT

The final phase of the re-evaluation process involved updating the existing BrR models supplied by MnDOT. The existing models were first verified for accuracy based on the existing plans and then modified to implement the refined rating parameters. Once updated, the refined models were used to compute new load and resistance factor ratings and complete the MnDOT bridge rating and load posting reports.

3.4.1 Verification of Existing Bridge Rating Model

Before any modifications were made to the existing BrR models, shear was turned on and an initial load and resistance factor rating (LRFR) was performed for each bridge to establish a baseline rating. It was not uncommon that the initial was governed by shear at the end of the beam, in which case points of interest were added to the analysis at h/2 and tenth points (Minnesota Department of Transportation (MnDOT) 2006). These, “as-is,” ratings were recorded and used as a point of comparison between the ratings obtained from the updated models and the refined models.

The existing BrR models were then compared against the existing plans and updated to make the model as accurate as possible, prior to implementing refined distribution factors and modified concrete strengths. Upon the completion of any updates, an updated LRFR rating analysis was performed and recorded. Common items that required verification or modification included:

- The control options for each beam were modified so that the general procedure for shear capacity per AASHTO was consistently used for all models.
- Verifying and correcting concrete and steel reinforcement material properties to match the existing plan values. Prestressed concrete stress limits and ranges were also defined as necessary.
- Verifying that the correct beam shape was used and modifying the beam shape as necessary.
- Revising the concrete deck thickness to reflect changes in wearing course. For consistency between models, the deck thicknesses were revised such that the first deck pour, used to define the non-composite loads and composite section, included all but the top ½" of any concrete wearing course. The top ½" of concrete wearing course was applied as a composite wearing course dead load. Any changes to the structural thickness also required that the deck profiles be recomputed to reflect any changes to the effective flange widths for load factor ratings (LFR) that may be performed in the future.
- Revising the shear reinforcement layout to match the existing plans.
- Older versions of the BrR software did not allow for the definition of web end blocks. Thus, bridges that implemented beams with end blocks were rarely defined as such. Therefore, web end blocks were defined as necessary to increase the shear capacity at the ends of the beams.
- Traffic values for average daily truck traffic (ADTT) were input as part of the general bridge information required to compute the correct load factors used for routine permit rating factors per MBE Table 6A.4.5.4.2a-1.
- Verification was made that the system factors, per MBE 6A.4.2.4, were defined correctly. This was done by ensuring that, "All Other Girder/Slab Bridges," was selected from the system factor drop-down within the member alternatives, factor tab.
- All the live load distribution factors were recomputed within BrR to verify that any changes in stiffness resulting from the above modifications were considered during the moment distribution factor computations.

Finally, the refined shear live load distribution factors were used to override the computed LRFD factors and the concrete strengths were revised to reflect a 20 percent strength gain. It should be noted that the modified concrete compressive strength used for the deck and final compressive strength for the prestressed beams only included the 20 percent strength gain specified for bridges 20 years and older. Unlike the properties used for the computation of the composite stiffness, the statistical bias factor was not included in the modified concrete strengths for the rating analysis, as its effect was accounted for in the calibration of the LRFD resistance factors (Dereli et al. 2007).

CHAPTER 4: RESULTS

4.1 SCREENING TOOL

The initial analysis consisted of the implementation of a screening tool used to select the 50 bridges for re-evaluation. The screening tool required the computation of a ratio of the longitudinal flexural stiffness to the transverse flexural stiffness, referred to as the stiffness ratio. Bridges that had a governing span with a stiffness ratio less than 1.5 were considered for re-evaluation.

Of the 522 bridges that were submitted by MnDOT for re-evaluation, 127 (approximately 24 percent) had a stiffness ratio that was less than 1.5 for the governing span. While the number of bridges containing a span with a stiffness ratio less than 1.5 was 267 (approximately 51 percent), it was important to consider whether the span that governed the rating had a stiffness ratio less than 1.5, as that span would ultimately determine the overall bridge rating. The governing span was determined based on the initial shear rating of the existing, as-is BrR model without any significant modifications. Though the use of the un-verified models did create the potential for errors in the determination of the governing span, this was necessary from an efficiency standpoint as it was prohibitive to verify all the models during this phase of the process, and any modifications were unlikely to alter the outcome regarding the governing span. A summary of the results of the screening tool implementation is presented in APPENDIX D
Stiffness Ratio Results Summary.

In general, the screening tool proved to be an effective means of selecting bridges for re-evaluation. Depending on the availability of the existing plans, the information required to implement the screening tool, and the computational effort, was relatively minimal. More importantly, all 50 bridges that were re-evaluated were improved by the refined methods. Therefore, the use of the stiffness ratio and the limit of 1.5 were both satisfactory. Further discussion regarding the stiffness ratio is provided in the following section.

4.2 OVERALL REFINED RATING RESULTS

While the refined rating methodology did yield improvements to the shear ratings, the increase in rating factors was relatively modest. On average, the refined rating methodology increased the shear ratings by approximately 16 percent. As stated previously, all 50 of the bridges that were re-evaluated using the refined methodology resulted in an increase in shear rating. The maximum observed increase was approximately 29 percent. Figure 4.1 provides a before and after breakdown of the rating factor distribution among the 53 spans that comprised the 50 re-evaluated bridges.

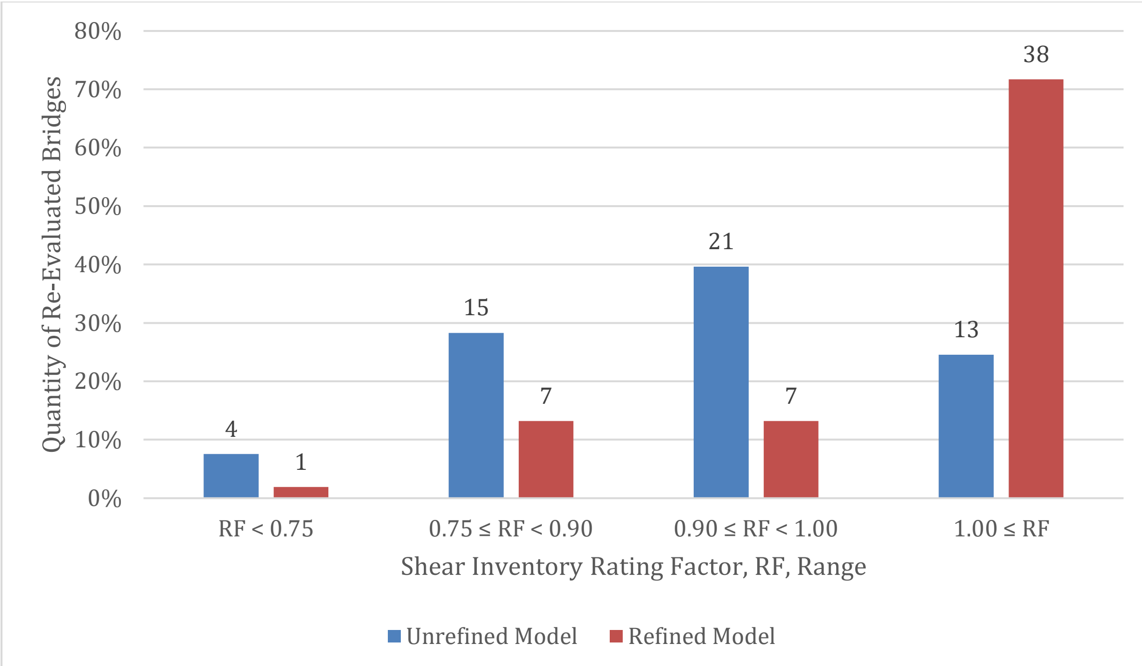


Figure 4.1 Shear Inventory Rating Factor Distribution of Re-Evaluated Bridges

An inventory level rating greater than 1.0 indicates a superstructure that has a capacity meeting the requirements of AASHTO. Per MBE 6A.4.3.1, "Bridges that pass HL-93 screening at the Inventory level will have adequate capacity for all AASHTO legal loads and State legal loads that fall within the exclusion limits described in the AASHTO LRFD Bridge Design Specifications." Initially, only 25 percent of the inventory level shear rating factors obtained from the unrefined BrR models were greater than 1.0. As shown in Figure 4.1, the refined rating methodology increased this amount to 72 percent. From a design adequacy perspective, this is a significant improvement. However, this finding should be viewed with a degree of caution, as it may also be noted that a large portion of that improvement was from bridges that initially had an inventory level shear rating between 0.9 and 1.0.

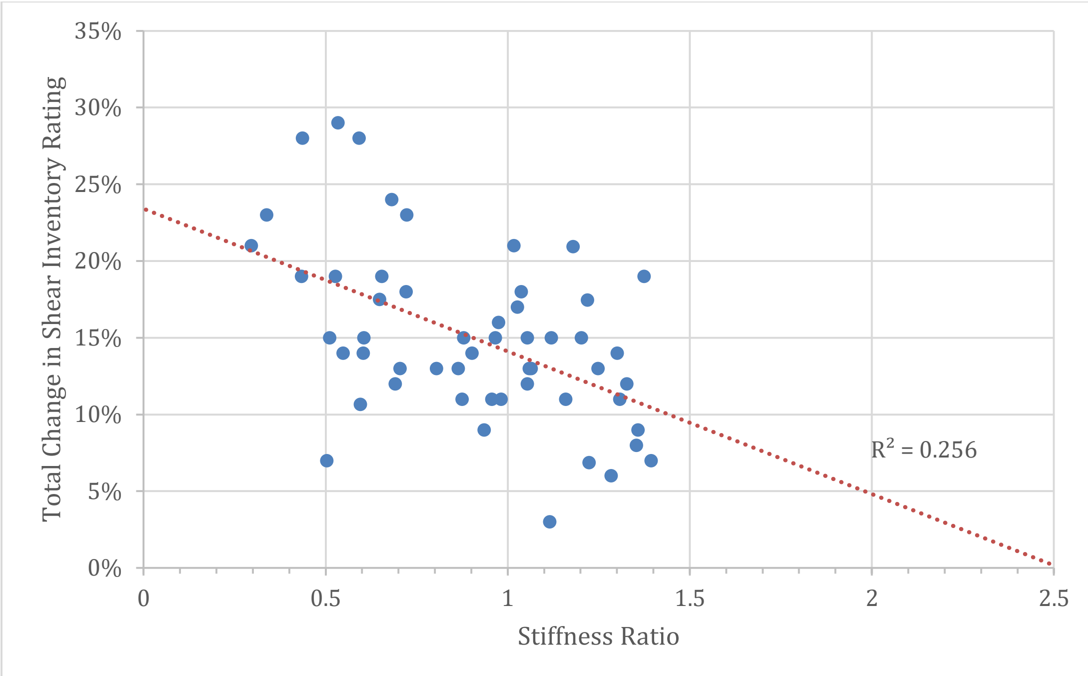


Figure 4.2 Correlation between Stiffness Ratio and Refined Rating Improvement

Figure 4.2 is a plot of the percentage change in rating versus the corresponding stiffness ratio due to the refined rating methodology for each of the 50 bridges that were re-evaluated. As shown by the trendline, there is an indirect correlation between the stiffness ratio and expected amount of improvement from the refined rating methodology. However, the spread of the data points around the trendline indicate that there is not a strong correlation between the stiffness ratio and the magnitude of the expected change in rating, as indicated by the coefficient of determination, R^2, of approximately 0.26. The projected trendline predicts a cessation of rating improvement at an approximate stiffness ratio of 2.5. This correlation is consistent with the findings and recommended stiffness ratio limit of 1.5 presented by French et al. (2016).

A complete set of rating results for each of the 50 re-evaluated bridges can be found in APPENDIX E Rating Results Summary.

4.3 REFINED LIVE LOAD DISTRIBUTION FACTORS

To better understand each component of the refined rating methodology, the change in inventory rating attributed solely to the refined live load distribution factor was plotted against the stiffness ratio, as shown in Figure 4.3. Since the live load is the only term in the denominator of the rating equation, as shown in section 1.1 there exists an indirect relationship between the magnitude of the live load distribution factor and the overall shear rating. Therefore, the approximate change in rating factor can be obtained by scaling the initial rating by the change in live load distribution factor. The reason for the slight discrepancy between the change in magnitude of the rating factor and the live load distribution factor was due to the shear capacity also being a function of the applied load. However, the effect that the change in live load had on the capacity, and thus the overall rating, was found to be negligible.

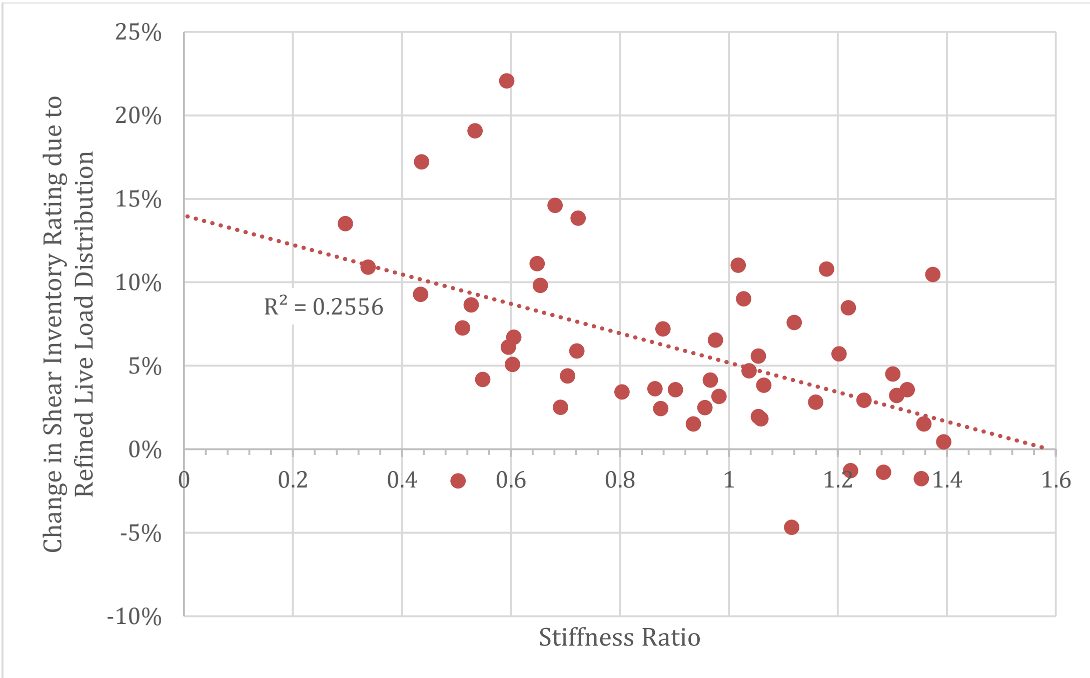


Figure 4.3 Correlation between Stiffness Ratio and Change in Governing Live Load Distribution Factor

An analysis of the live load distribution factors yielded the same indirect correlation between the stiffness ratio and predicted reduction in live load distribution factor that was observed for the total change in shear inventory rating, shown in Figure 4.2. On average, the refined rating methodology decreased the live load distribution factor, and therefore increased the shear inventory rating, by 7 percent. As before, the correlation between the stiffness ratio and the magnitude of expected change in rating was not very strong, given the R^2 value of approximately 0.26. Furthermore, of the 50 bridges that were re-evaluated, the refined rating methodology actually increased the live load distribution factors for five of the bridges, resulting in a negative change in shear inventory rating. This observation was counter to the results predicted by French et al. (2016). Therefore, as directed by MnDOT, the approximate 2014 AASHTO LRFD live load distribution factors were used to complete the final rating form these bridges, in lieu of the refined live load distribution factors, to maximize the bridge rating. However, for the sake of comparison, all of the rating results presented herein are those obtained via the refined live load distribution factors.

Figure 4.3 is a plot of the change in shear inventory rating due to the governing live load distribution factor versus the stiffness ratio. It was observed on several bridges that the governing shear rating location along the interior beam did change as the live load distribution factors were altered. This was generally true for skewed bridges, as a constant reduction factor was applied for simplicity instead of a linear variation. Additionally, the change in live load demand altered the capacity at a given section, and hence the rating, as discussed in greater detail below. Therefore, Figure 4.3 is not necessarily a plot of the change in shear rating, or live load distribution factor, at a constant location.

4.4 INCREASED CONCRETE STRENGTHS

The effect that the 20 percent increase in concrete compressive strength, to account for time-dependent strength gain, had on the shear ratings was also analyzed. Initially, it was assumed that the concrete strength would have a relatively minor impact compared to the refined live load distribution factors. This was due to the observation that the concrete compressive strength is typically a relatively minor component of the overall shear resistance of the prestressed beam section, particularly near the end of the beam where the concentration of shear reinforcement is greatest.

The nominal shear strength of the section is primarily equal to the sum of the shear resistances provided by the concrete and steel reinforcement. The concrete shear resistance, V_c, is directly affected by the compressive strength of the concrete, f'_c, as shown in 2014 AASHTO LRFD equation 5.8.3.3-3, presented below.

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v$$

Since the square root of the concrete compressive strength is used to determine the concrete shear resistance, a 20 percent increase in strength only yields an increase in strength of approximately 9.5 percent. As discussed by Runzel et al. (2007), the concrete compressive strength can also have a direct impact on the effective shear depth, d_v, if the dimension is governed by the distance between the resultant compressive and tensile forces within the section. Therefore, in addition to directly impacting the concrete shear resistance, a change in the effective shear depth also affects the concrete shear resistance via the longitudinal tensile strain, ε_s, and the ability of the diagonally cracked concrete to transmit tension, β, as presented in the following equations per 2014 AASHTO LRFD 5.8.3.4.2. Per Runzel et al. (2007), an increase in concrete compressive strength was found to decrease β. However, the percentage change in concrete shear resistance due to a change in effective shear depth was primarily governed by the square root of the concrete compressive strength.

$$\varepsilon_s = \frac{\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}}{E_s A_s + E_p A_{ps}}$$

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)}\frac{51}{(39 + s_{xe})}$$

Furthermore, the concrete shear resistance is typically only 25 to 50 percent of the overall shear capacity near the end of the beam, depending on the shear reinforcement layout and presence of web end blocks. The nominal shear resistance due to the steel shear reinforcement, V_s, is presented below per 2014 AASHTO LRFD equation 5.8.3.3-4.

$$V_s = \frac{A_v f_y d_v cot\theta}{s}$$

Similar to the computation of the concrete shear resistance, the compressive strength also has an indirect effect on the shear reinforcement resistance via the angle of principal compression, θ, presented below per 2014 AASHTO LRFD 5.8.3.4.2. Runzel et. al (2007) found that an increase in concrete compressive strength increased the angle of principal compression, which corresponds to an increase in the amount of shear reinforcement that is intercepted by a given shear crack.

$$\theta = 29 + 3500\varepsilon_s$$

A graphical representation of the shear resistance components is presented in Figure 4.4 for a sample of 25 bridges.

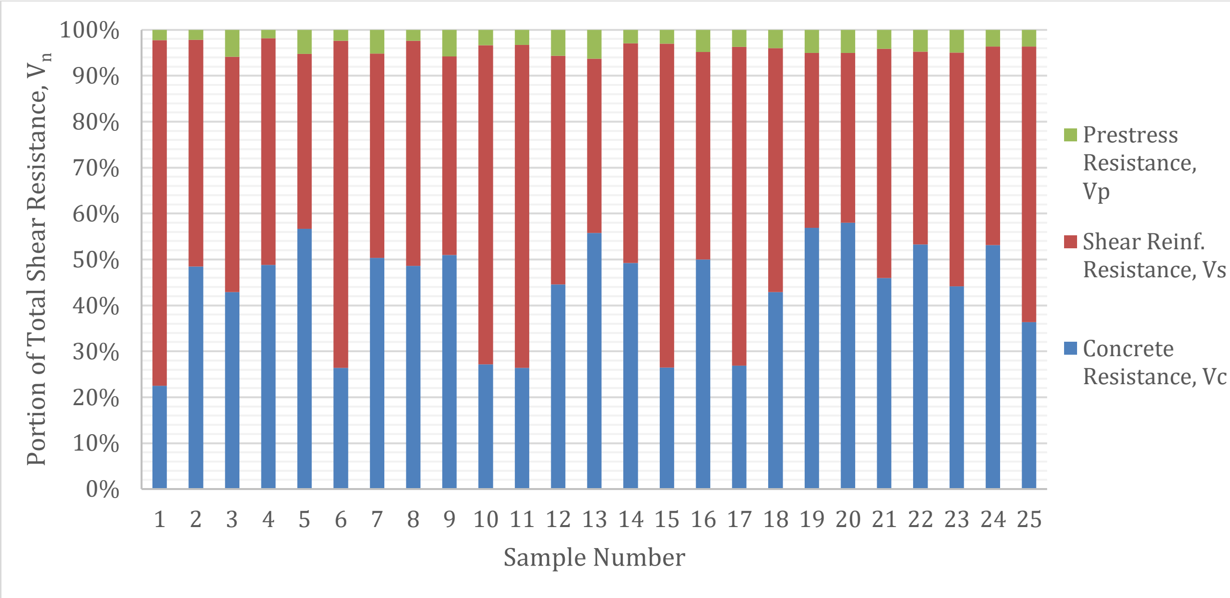


Figure 4.4 Total Shear Resistance Composition per 25 Samples

Therefore, the 9.5 percent increase in concrete shear capacity, applied to approximately 25 to 50 percent of the total shear capacity, generally only yielded an increase in shear resistance of approximately 2 to 5 percent. This increase in shear capacity was consistent was the findings of the parametric study conducted by Dereli et. al (2010) for the 2002 Standard Specifications.

However, given that the difference between capacity and the dead load is in the numerator of the rating equation, the effect that this increase in capacity had on the rating was dependent on the magnitude of the capacity relative to the dead load demand, and could not be directly inferred. This concept is represented graphically in Figure 4.5. As the ratio of capacity to dead load decreased, while holding the live load constant, the greater the increase that a given percentage change in capacity had on the increase in rating.

A plot of the change in shear inventory rating due to the stiffness ratio, shown in Figure 4.6, indicated that the relatively modest increase in shear capacity generally resulted in an average increase in shear rating of approximately 9 percent. This result was also consistent with the findings presented by Dereli et. al (2010), which found an average increase in shear rating of approximately 6 percent due to a 20 percent increase in concrete strength. Therefore, the increase in concrete strength resulted in a similar amount of improvement in shear inventory rating that was found for the refined live load distribution factor.

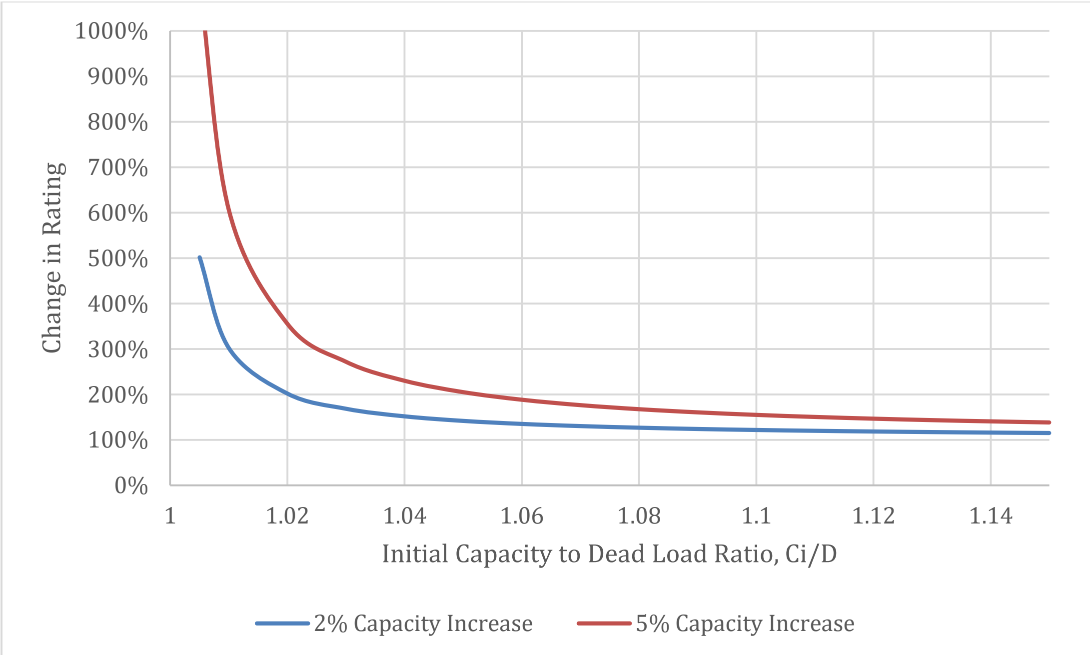


Figure 4.5 Change in Rating due to a 20 Percent Change in Initial Capacity to Dead Load Ratio, C_i/D

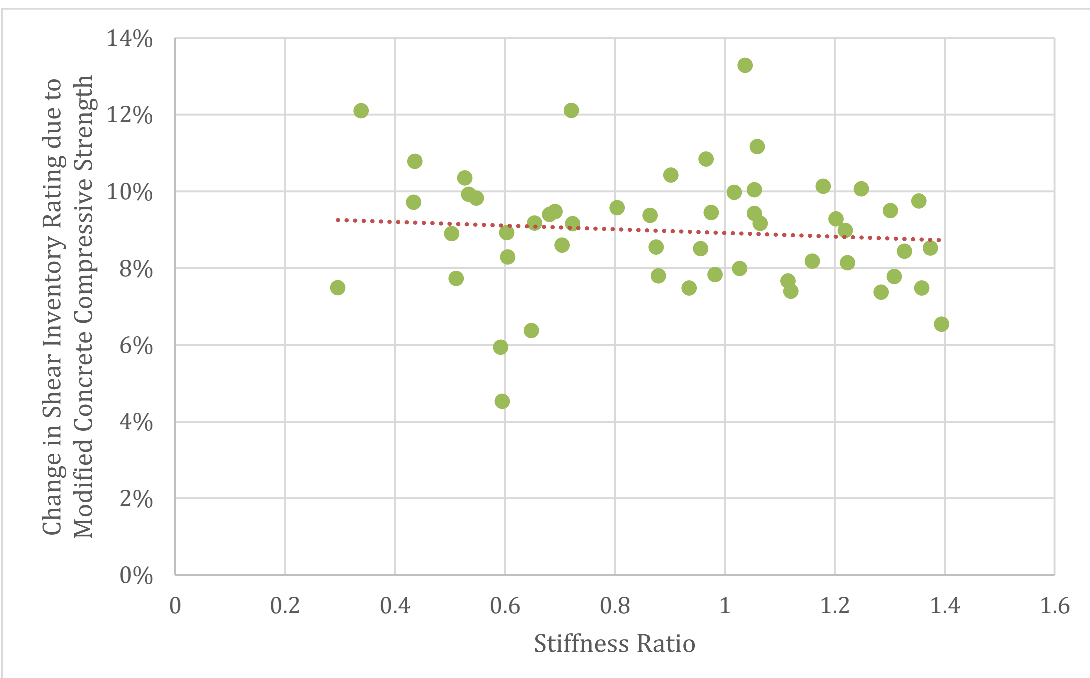


Figure 4.6 Increase in Shear Rating due to Modified Concrete Compressive Strength, f'_c

CHAPTER 5: QUANTIFIED BENEFITS OF FULL DEPLOYMENT OF REFINED RATING METHODOLOGY

Implementation of the refined rating methodology proved to be a relatively effective means of improving the shear rating for prestressed concrete bridges. To determine a quantified (financial) benefit for full deployment of the refined rating methodology, a cost-benefit analysis was conducted for the utilization of the refined rating methodology versus undertaking bridge repairs without consideration of any potential shear rating improvements.

5.1 LOAD CAPACITY TARGETS FOR BRIDGES ON THE STATE SYSTEM

Chapter 2 of MnDOT's *Fiscal Year 2016 through 2020 Bridge Preservation and Improvement Guidelines* (BPIG) established a load carrying capacity target for bridges on the state system with signs posting load limits below legal weight of 0 percent. Additionally, chapter 6 of the BPIG requires that all bridge rehabilitations must include any strengthening or modifications required to produce a structure that has a minimum superstructure LRFR inventory rating of 0.9 or greater. Therefore, for the purposes of establishing the financial benefit for implementation of the refined rating methodology, it was assumed that bridges with HL-93 inventory rating factors greater than or equal to 0.9 would not require repairs or re-evaluation to meet load capacity targets. Only bridges with rating factors less than 0.9 were assumed to be eligible for any benefits associated with refined shear ratings.

The 50 bridges that were part of this study required the evaluation of a total of 53 spans. As shown in Table 5.1, of these 53 spans, 19 had an initial shear inventory rating factor less than 0.9. After re-evaluation using the refined rating methodology, 12 of these 19 spans (63 percent) saw rating factor improvement such that no repairs would be required to meet load capacity targets.

Table 5.1 Inventory Rating Factor Distribution of Re-Evaluated Bridges

Shear Inventory Rating Factor (RF) Range	Distribution of Re-Evaluated Spans (53 Total)	
	Unrefined Rating	Refined Rating
1.00 ≤ RF	13	38
0.90 ≤ RF < 1.00	21	8
0.75 ≤ RF < 0.9	14	6
RF < 0.75	5	1

5.2 METHODS AND COSTS FOR REPAIRS TO INCREASE SHEAR CAPACITY

Though only recently implemented as a means of concrete repair by MnDOT, the use of carbon fiber reinforced polymer (CFRP) has been widely used as a method for increasing the capacity and service life of substructure elements, most notably pier columns. Research and use in other states has shown that CFRP can also be used as an effective method for increasing both the flexural and shear capacity of superstructure elements (Wipf et al. 2004; Simpson II et al. 2006; Rizkalla et al. 2007; Becher 2013).

Figure 5.1 shows the typical configuration of a CFRP shear repair for a prestressed concrete beam. CFRP strips are bonded to the webs of the beams to increase shear capacity above the desired level. The CFRP strips are anchored by wrapping around the bottom flange, covering approximately two-thirds of the bottom flange width. Since shear capacity deficiencies generally occur within 10-15 percent of the span length, it was assumed that 20 percent of the span length from each end of each beam would be wrapped with CFRP.

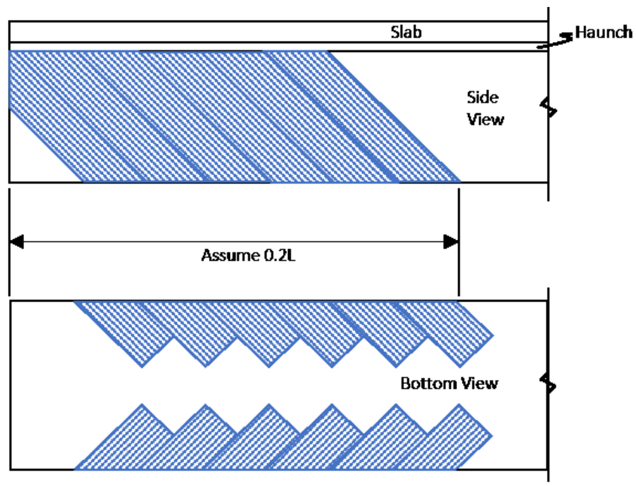


Figure 5.1 Typical CFRP Shear Repair Configuration (Reproduced per Simpson II et al. 2006)

The cost associated with utilizing CFRP to increase the shear capacity of prestressed concrete beams was estimated to be on the order of $50 per square foot (Hooley 2017). For the purpose of developing an average repair cost, CFRP quantities were estimated for the 19 spans with shear inventory ratings less than 0.9. It should be noted that the estimated CFRP cost is approximate in nature and represents the average cost for a typical repair of this type. The actual repair cost for a given bridge could be greater or less, depending on the requisite amount of additional shear capacity, number of CFRP layers required to produce an adequate shear rating, site access, etc. An estimated CFRP repair cost for each span is provided in

Table 5.2. Given these estimated costs, the average CFRP repair cost was calculated to be approximately $109,123 per span.

Table 5.2 Estimated CFRP Repair Costs

Structure Number	Span Length, L (feet)	Beam Type	Number of Beams	Beam Perimeter (inches)	CFRP Length per Beam (feet)	CFRP Quantity (ft^2)	CFRP Cost ($25/$ft^2$)
27193	77	Narrow Top Flange 54M	10	137.8	30.8	3536.2	$176,809
27042	77.83	AASHTO Type III (45”)	5	114.6	31.1	1486.6	$74,332
23006	71.67	AASHTO Type III (45”)	6	114.6	28.7	1642.8	$82,139
32807	96.67	Narrow Top Flange 54M	5	137.8	38.7	2219.8	$110,988
27738	104.29	54M	8	158.4	41.7	4404.5	$220,223

24837	55.33	AASHTO Type II (36”)	6	91.5	22.1	1012.1	$50,605
42011	104	63M	5	176.4	41.6	3057.1	$152,857
27744	91	45M	10	140.4	36.4	4258.0	$212,900
62070	83.33	AASHTO Type III (45”)	14	114.6	33.3	4456.7	$222,837
9049	79.92	AASHTO Type III (45”)	7	114.6	32.0	2137.2	$106,859
69081	54.75	AASHTO Type II (36”)	6	91.5	21.9	1001.5	$50,074
9148	75.29	AASHTO Type III (45”)	9	114.6	30.1	2588.6	$129,431
46816	64.06	1996 Narrow Top Flange 40”	5	105.4	25.6	1125.6	$56,280
12003 (Span 2)	61.63	1996 Narrow Top Flange 40”	6	105.4	24.7	1299.5	$64,974
12003 (Span 1&3)	59.96	1996 Narrow Top Flange 40”	6	105.4	24.0	1264.3	$63,214
69074	75.48	AASHTO Type III (45”)	5	114.6	30.2	1441.8	$72,088
22004	67.33	1996 Narrow Top Flange 40”	6	105.4	26.9	1419.7	$70,984
17004	66.25	1996 Narrow Top Flange 40”	6	105.4	26.5	1396.9	$69,845
55030	56.35	AASHTO Type II (36”)	10	91.5	22.5	1717.9	$85,896
Average CFRP Repair Cost per Span							$109,123

5.3 COSTS FOR IMPLEMENTATION OF REFINED RATING METHODOLOGY

The refined rating methodology required the completion of two primary tasks: The computation of the refined live load distribution factor and updating the BrR model for re-evaluation. Additionally, each of these tasks received a quality control (QC) check by a qualified person who did not participate in the initial rating. The total cost to complete each of these tasks for the 53 spans evaluated as part of this study is provided in

Table 5.3. These costs were based on hourly payroll rates comparable with industry averages and included benefits and overhead. On average, the cost to perform a refined rating of a single bridge span was approximately $792 and took 9 hours of labor to complete.

Table 5.3 Refined Rating Methodology Cost Analysis (53 Bridge Spans)

Task	Cost	Hours
Analysis of Bridge Distribution Factor	$15,000	175
Analysis of Bridge Distribution Factor (QC Check)	$5,000	58
Bridge Database Update	$7,400	78
Bridge Database Update (QC Check)	$14,500	153
Total Cost	$41,950	464
Average Rating Cost per Span	$791.50	9

5.4 BENEFIT OF REFINED RATING METHODOLOGY

As mentioned previously, approximately 63 percent of the 19 spans with HL-93 inventory ratings less than 0.9 were improved using the refined rating methodology alone. The remaining 37 percent would require a CFRP repair to sufficiently improve the governing shear inventory rating above minimum established target. Since a strong correlation was not found between the stiffness ratio and expected amount of shear rating improvement, as reiterated in Figure 5.2 for the 19 spans, it was recommended that the refined rating methodology first be implemented to determine the need for a physical repair. Assuming that these percentages can be applied to the remainder of prestressed concrete girder bridges on the state inventory with a stiffness ratio below the 1.5 threshold, the average cost to improve a bridge span with an inventory rating below 0.9 was found to be \$40,995 based on the following computation:

$$Avg.\,Rating\ Improvement\ Cost = Avg.\,Rating\ Cost + 37\% * Avg.\,CFRP\ Cost$$

The average cost savings, or benefit, of the full deployment of the refined rating methodology for bridges with HL-93 inventory rating factors less than 0.9 was considered to be the difference between the average cost to simply repair a span using CFRP (\$109,123) and the average cost to first implement the refined rating methodology to improve the ratings and better determine which spans require a physical repair (\$40,995). Therefore, using the refined rating methodology to first improve the bridge rating yielded an average cost savings of approximately \$68,128 per span. This savings may also be presented in the form of a benefit-cost ratio of approximately 1.66, meaning that first implementing the refined rating methodology yielded an average cost saving benefit of 66 percent compared to preemptively undertaking a typical CFRP repair.

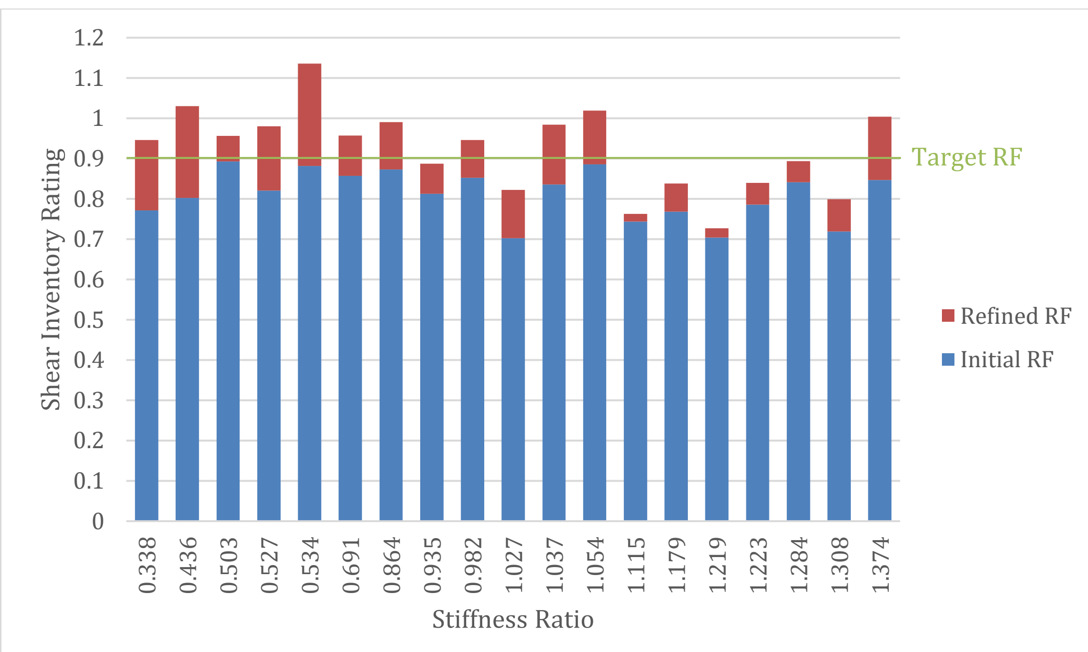


Figure 5.2 Correlation between Stiffness Ratio and Shear Rating Improvement due to Refined Methodology

CHAPTER 6: SUMMARY AND CONCLUSIONS

The topic of lower than desirable shear ratings at the ends of prestressed beams, despite any observable signs of distress, has been of continued interest to MnDOT. This is particularly true given the relatively large inventory of prestressed concrete bridges throughout Minnesota. Per the findings and recommendations of the University of Minnesota report *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges* (French et al. 2016), a refined load rating methodology was implemented to more accurately assess the shear ratings of prestressed concrete girder bridges. The methodology employed a two-dimensional grillage finite element model to more accurately predict the live load distribution for shear at the ends of a beam, by considering the individual placement of each axle within the span. Additionally, a 20 percent increase in concrete strength was recommended for bridges 20 years or older to account for time-dependent strength gain.

MnDOT selected 522 bridges for re-evaluation as part of this study. The applicability of the refined rating methodology was first established for each bridge via the screening tool recommended by French et al. (2016). The screening tool reduced the total number of applicable bridges to 127, of which 50 were selected for analysis based on their NBI condition rating and consistency with the University of Minnesota report.

In general, the refined rating methodology was found to improve the shear ratings for all 50 of the bridges that were re-evaluated by an average of 16 percent. Thus, the screening tool proposed by French et al. (2016) was effective in determining the candidacy of a given bridge for re-evaluation. However, the correlation between the stiffness ratio and the amount of shear rating improvement was not strong enough to preclude a full implementation of the refined rating methodology. Therefore, it is not recommended that a refined shear load rating be determined by simply scaling the existing shear rating by the computed stiffness ratio. On average, the refined live load distribution factor and the increased concrete strengths were found to contribute relatively equally to the amount of shear rating improvement.

Implementation of the refined rating methodology was more cost effective when compared to physical methods of improving the shear rating for a given prestressed beam, namely via the use of a carbon fiber reinforced polymer (CFRP) wrap. A quantified benefit analysis revealed an average cost savings of approximately $68,128, or 66 percent, by employing the refined rating methodology versus a typical CFRP repair for a given prestressed beam bridge to improve its shear rating.

It is reasonable to presume that the remaining 77 bridges that were not re-evaluated as part of this study, but to which the refined rating methodology was applicable, would benefit from the refined rating methodology. However, the remaining bridges, approximately 76 percent of the initial 522 submitted by MnDOT, that were not deemed candidates for re-evaluation would likely require physical repairs to meet target load rating goals. Therefore, research regarding alternate methods for shear rating improvement of prestressed concrete girder bridges has continued merit.

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APPENDIX A

STATISTICAL BIAS FACTORS

The following statistical bias factors for ready mix (deck) concrete and plant-cast (prestressed beam) concrete are presented per Nowak and Szerszen (2003).

Table A.1 Statistical Bias Factors

Ordinary Ready Mix Concrete	
f'_c (psi)	Statistical Bias Factor, λ
3,000	1.35
3,500	1.21
4,000	1.235
4,500	1.14
5,000	1.15
6,000	1.12
Ordinary Plant-Cast Concrete	
f'_c (psi)	Statistical Bias Factor, λ
5,000	1.38
5,500	1.19
6,000	1.16
6,500	1.14
High Strength Concrete	
f'_c (psi)	Statistical Bias Factor, λ
7,000	1.19
8,000	1.09
9,000	1.16
10,000	1.13
12,000	1.04

APPENDIX B

SAMPLE CALCULATIONS (BR 53005)

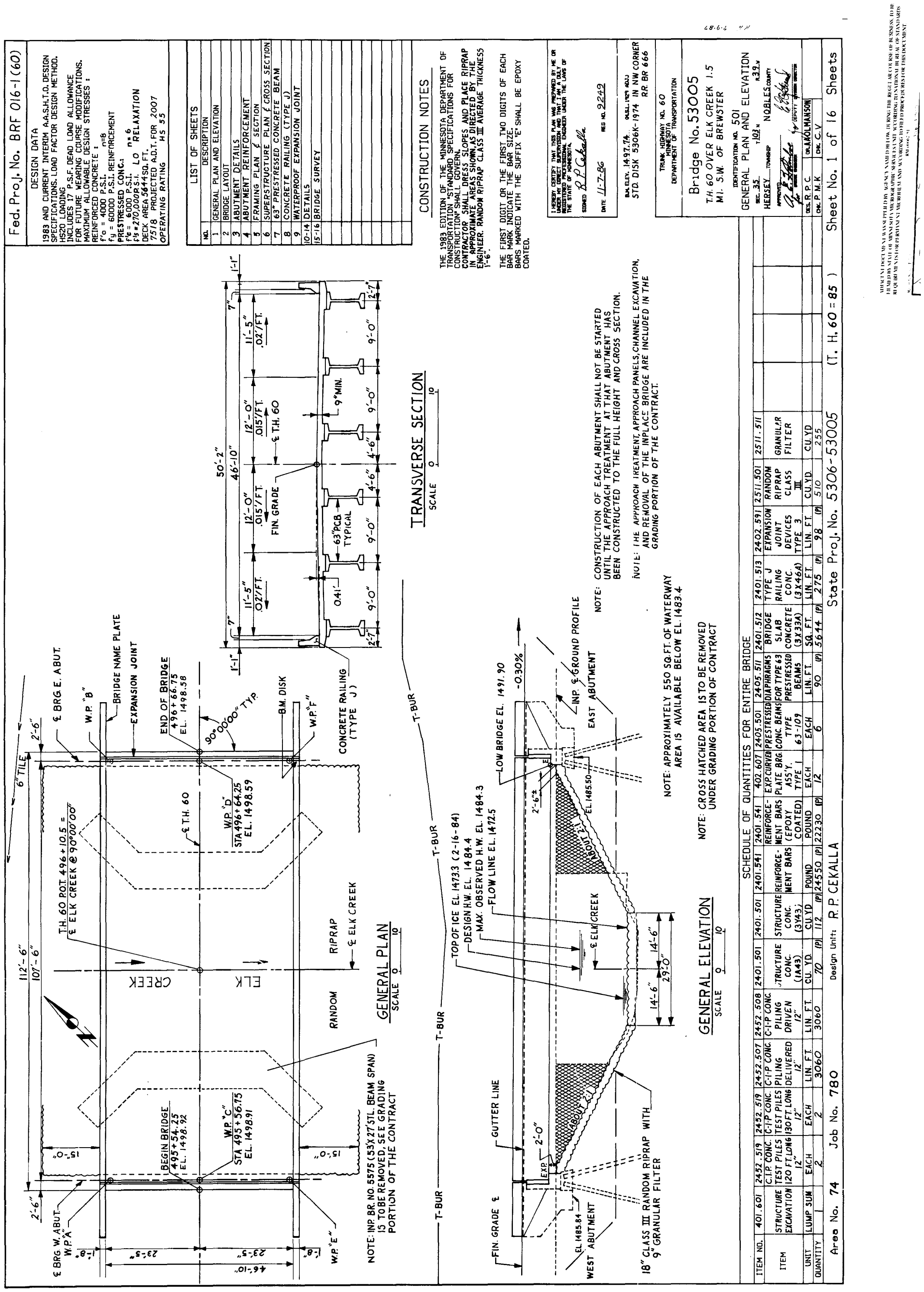

Figure B.1 BR 53005 General Plan and Elevation

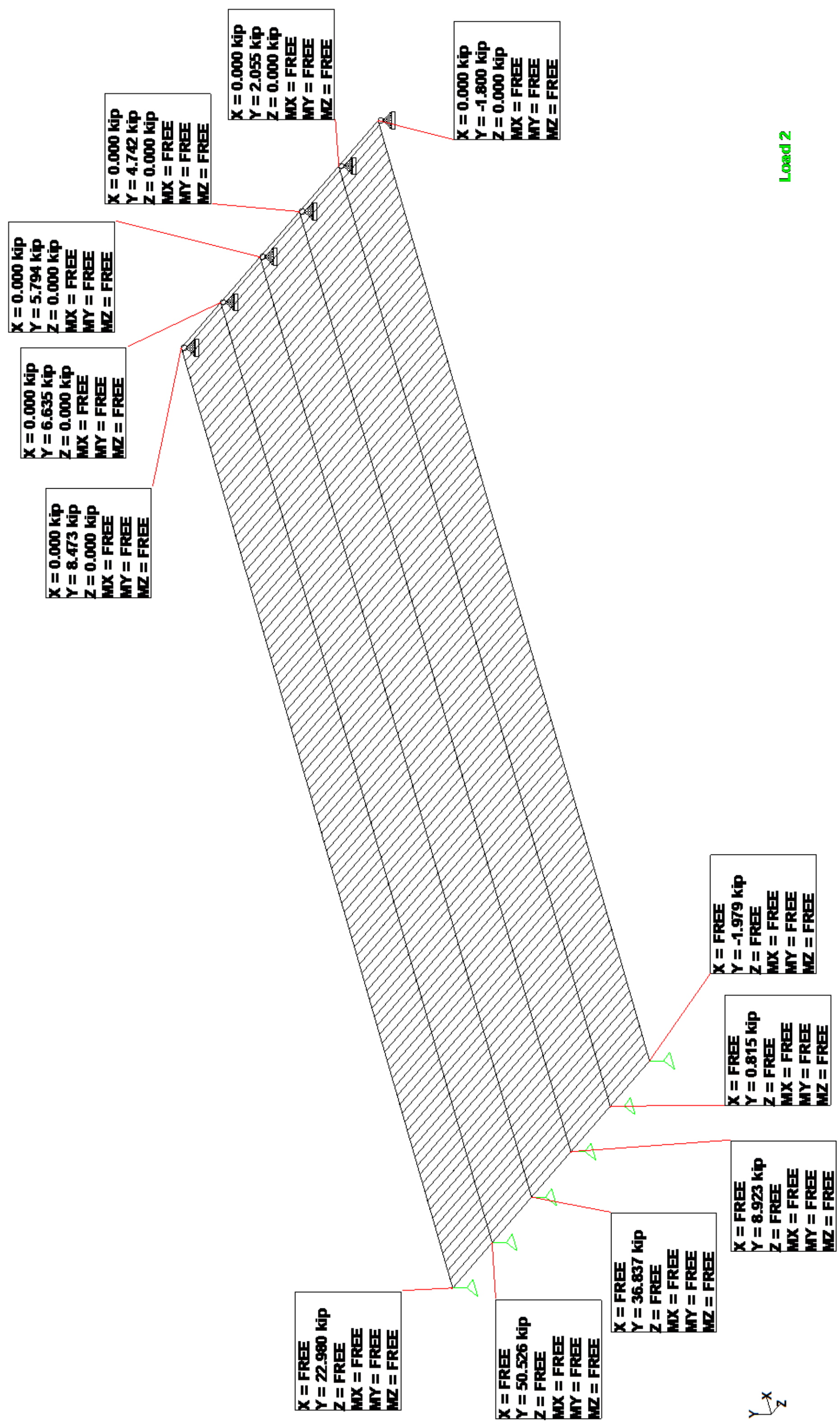

Figure B.2 BR 53005 STAAD Model/Output

Project Name: Prestressed Concrete Beam Shear Rating
Client: MnDOT
Comm. No.: 9348 Date: 7/27/17 By: NTS Chkd. By: RMS
Filename: H:\Projects\09000\9348\BR\Doc\Report Figures\[9348_BR 53005 Input and LLDF_Reduced.xlsx]Sheet 1

Bridge No.: 53005

STIFFNESS RATIO

Component	Nominal f'_c (ksi)	E (ksi)
Beam	5.75	4369.18
Deck	4	3644.15

Non-Composite Section Properties:

Beam Spacing (ft)	Deck Thickness (in)	Stool Thickness (in)	Stool Width (in)	Non-Composite I_{beam} (in^4)	Non-Composite A_{beam}	h_{beam} (in)	Non-Composite
9.00	8.5	2	30	392056	732	63	31.1

n (E_{beam}/E_{deck})	1.20

Transformed Section Properties:

Effective Deck Width (in)	Stool Width (in)
90.08	25.02

Composite Section Properties:

A (in^2)	CG from bottom (in)	I_{long} (in^4)
1547.71	51.04	950031.45

Transverse Stiffness:

I_{trans} (in^4)
614.13

Span Length:

L (ft)
107.50

Stiffness Ratio:

0.91

STAAD INPUT

CONCRETE MODULUS OF ELASTICITY

Increase nominal concrete compressive strengths by a factor of 1.2*λ, per French (2016) *Investigation of Shear Distribution Factors in Prestressed Concrete Girder Bridges*.

Component	Nominal f'_c (ksi)	Statistical Bias Factor, λ	Effective f'_c (ksi)	E (ksi)
Beam	5.75	1.175	8.1075	5188.11
Deck	4	1.235	5.928	4436.29

EXTERIOR BEAM - COMPOSITE MOMENT OF INERTIA (LONGITUDINAL)

Non-Composite Section Properties:

Beam Spacing (ft)	Deck Thickness (in)	Stool Thickness (in)	Stool Width (in)	Non-Composite I_{beam} (in^4)	Non-Composite A_{beam} (in^2)	h_{beam} (in)	Non-Composite CG_{beam} from bottom (in)
7.08	8.5	2	30	392056	732	63	31.1

n (E_{beam}/E_{deck})	1.17

Transformed Section Properties:

Effective Deck Width (in)	Stool Width (in)
72.68	25.65

Composite Section Properties:

A (in^2)	CG from bottom (in)	I (in^4)
1401.10	49.13	895189.91

INTERIOR BEAM - COMPOSITE MOMENT OF INERTIA (LONGITUDINAL)

Non-Composite Section Properties:

Beam Spacing (ft)	Deck Thickness (in)	Stool Thickness (in)	Stool Width (in)	Non-Composite I_{beam} (in^4)	Non-Composite A_{beam} (in^2)	h_{beam} (in)	Non-Composite CG_{beam} from bottom (in)
9.00	8.5	2	30	392056	732	63	31.1

n (E_{beam}/E_{deck})	1.17

Transformed Section Properties:

Effective Deck Width (in)	Stool Width (in)
92.35	25.65

Composite Section Properties:

A (in^2)	CG from bottom (in)	I (in^4)
1568.28	51.27	956677.75

HL-93 TRUCK LIVE LOAD DISTRIBUTION FACTOR

Span Length (ft)	Skew Angle, Θ (deg)
107.5	0

Compute distribution factor for rear truck axle placed at 0.1*Span Length.

Number of Lanes	STAAD Reaction (k)	Skew Correction*	MPF	LLDF (lanes)
1	34.131	1.00	1.2	0.700
2	50.526	1.00	1	0.863

*Skew correction factor computed per Puckett et. al. (2007) for skew angles greater than 30 degrees.

APPENDIX C

SAMPLE STAAD INPUT (BR 53005)

C.1 STAAD Input

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE 02-Nov-16
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
1 0 0 0; 2 0 0 8.99967; 3 0 0 18.0003; 4 0 0 27.001; 5 0 0 36; 6 0 0 45.002;
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1156 599 609; 1157 609 619; 1158 619 629; 1159 580 590; 1160 590 600;
1161 600 610; 1162 610 620; 1163 620 630; 1164 581 591; 1165 591 601;
1166 601 611; 1167 611 621; 1168 621 631; 1169 582 592; 1170 592 602;
1171 602 612; 1172 612 622; 1173 622 632; 1174 583 593; 1175 593 603;
1176 603 613; 1177 613 623; 1178 623 633; 1179 634 641; 1180 641 648;
1181 648 655; 1182 655 662; 1183 662 669; 1184 635 642; 1185 642 649;
1186 649 656; 1187 656 663; 1188 663 670; 1189 636 643; 1190 643 650;
1191 650 657; 1192 657 664; 1193 664 671; 1194 637 644; 1195 644 651;
1196 651 658; 1197 658 665; 1198 665 672; 1199 638 645; 1200 645 652;
1201 652 659; 1202 659 666; 1203 666 673; 1204 639 646; 1205 646 653;
1206 653 660; 1207 660 667; 1208 667 674; 1209 640 647; 1210 647 654;
1211 654 661; 1212 661 668; 1213 668 675; 1215 677 103; 1217 706 183;
1220 679 263; 1236 698 37; 1237 699 117; 1238 700 197; 1239 701 277;
1240 702 51; 1241 703 131; 1242 704 211; 1243 705 291; 1252 706 684;
1259 99 695; 1260 113 696; 1261 20 695; 1262 695 180; 1263 180 260;
1264 34 696; 1265 696 194; 1266 194 274; 1267 48 697; 1268 697 208;
1269 208 288; 1270 127 697; 1271 695 101; 1272 696 115; 1273 697 129;
1277 685 686; 1279 687 688; 1290 22 23; 1293 101 677; 1295 684 262;
1296 181 706; 1297 677 682; 1298 682 683; 1299 683 706; 1300 22 680;
1301 680 681; 1302 681 677; 1304 686 699; 1306 688 700; 1307 689 276;
1308 690 691; 1309 691 703; 1310 692 693; 1311 693 704; 1312 694 290;
DEFINE MATERIAL START
ISOTROPIC CONCRETEBEAM
E 747088
POISSON 0.17
DENSITY 0.150336
ALPHA 5e-006
DAMP 0.05
G 193846
TYPE CONCRETE

```
STRENGTH FCU 576
ISOTROPIC CONCRETEDECK
E 638826
POISSON 0.17
DENSITY 0.150336
ALPHA 5e-006
DAMP 0.05
G 193846
TYPE CONCRETE
STRENGTH FCU 576
ISOTROPIC EXTCONCRETEBEAM
E 747088
POISSON 0.17
DENSITY 0.150336
ALPHA 5e-006
DAMP 0.05
G 193846
TYPE CONCRETE
STRENGTH FCU 576
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
82 TO 88 93 TO 102 105 107 TO 116 119 121 TO 171 174 TO 186 188 TO 200 202 -
203 TO 405 927 TO 966 987 TO 1026 1044 TO 1071 1215 1217 1220 1237 TO 1239 -
1241 TO 1243 1259 1260 1270 TO 1273 1293 -
1296 PRIS AX 10.8908 IX 1 IY 1 IZ 46.1361
MEMBER PROPERTY AMERICAN
487 TO 526 530 TO 536 540 TO 596 600 TO 666 670 TO 891 912 TO 916 -
1079 TO 1213 1252 1261 TO 1269 1277 1279 1295 1297 TO 1302 1304 1306 TO 1311 -
1312 PRIS YD 0.708333 ZD 1
MEMBER PROPERTY AMERICAN
1 TO 10 12 TO 38 40 TO 81 406 TO 486 917 TO 926 967 TO 986 1027 TO 1043 1072 -
1073 TO 1078 1236 1240 1290 PRIS AX 9.72986 IX 1 IY 1 IZ 43.1708
CONSTANTS
MATERIAL CONCRETEBEAM MEMB 1 TO 10 12 TO 38 40 TO 88 93 TO 102 105 -
107 TO 116 119 121 TO 171 174 TO 186 188 TO 200 202 TO 486 917 TO 1078 1215 -
1217 1220 1236 TO 1243 1259 1260 1270 TO 1273 1290 1293 1296
MATERIAL CONCRETEDECK MEMB 487 TO 526 530 TO 536 540 TO 596 600 TO 666 670
-
671 TO 891 912 TO 916 1079 TO 1213 1252 1261 TO 1269 1277 1279 1295 -
1297 TO 1302 1304 1306 TO 1312
SUPPORTS
7 TO 12 PINNED
1 TO 6 FIXED BUT FX FZ MX MY MZ
LOAD 1 LOADTYPE None  TITLE 1 LANE
JOINT LOAD
681 682 686 687 FY -16
```

```
691 692 FY -4
LOAD 2 LOADTYPE None  TITLE 2 LANES
JOINT LOAD
677 680 683 TO 685 688 689 699 FY -16
690 693 694 703 FY -4
PERFORM ANALYSIS

FINISH
```

C.2 STAAD Input Modifications

The following is a checklist of items that require modification when creating a new STAAD model from a copy of an old model for a given bridge.

- Adjust the beam node locations in order to match the beam spacings shown on the plans.
- Add additional beam lines as necessary.
- Modify the longitudinal beam element properties to match the interior and exterior beam element definitions determined as part of the initial input calculations.
- Adjust the locations of the bearing nodes to achieve the correct span length. Add or delete nodes along the length of the beam, as necessary, to maintain nodes at one foot increments.
- Add or delete transverse beam elements corresponding to the node modifications of the previous bullet item. Assign the transverse beam properties to any new transverse beam elements.
- Modify the longitudinal and transverse beam properties, as well as their respective concrete moduli of elasticity, to match the moment of inertia values obtained from the input calculations.
- Reposition the truck loads so that the rear axle is located at the node, at one foot increments, closest to 0.1L. Add nodes to the transverse beam elements at each axle location, as necessary, in order to apply the wheel loads between longitudinal beam elements. It should be noted that the truck locations may need to be modified if it is determined that a beam other than the first interior governs the shear rating. In this case, the transverse position of the truck should be modified to maximize the shear demand for the governing beam and transverse beam element nodes be added as described above.

APPENDIX D

STIFFNESS RATIO RESULTS SUMMARY

Table D.1 Stiffness Ratio Results Summary

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
1	27792	1	0.251	0.052	6
1		2	1.293	0.052	6
1		3,10	0.407	0.052	6
1		4,6	1.007	0.052	6
1		5	0.507	0.052	6
1		7	0.052	0.052	6
1		8	0.411	0.052	6
1		9	0.086	0.052	6
2	27736	1	0.060	0.060	7
2		2	1.171	0.060	7
3	13801	1,4	19.107	0.095	7
3		1,4	1.510	0.095	8
3		2,3	0.698	0.095	8
3		2,3	0.095	0.095	6
4	82022	1	12.144	0.102	7
4		2	0.102	0.102	7
4		3	26.652	0.102	7
5	6869	1,2,3	0.164	0.164	7
5		1,2,3	0.537	0.164	7
6	69022	1	0.173	0.173	7
7	55057	1	0.201	0.201	7
7		2	0.374	0.201	6
8	04002	1,3 - fascia	0.347	0.275	6
8		2 - fascia	0.336	0.275	6
8		1,3 - 50"	0.285	0.275	7
8		2 - 50"	0.275	0.275	7
8		1,3 - 72"	0.617	0.275	7
8		2 - 72"	0.597	0.275	7
9	27737	1,2	0.288	0.288	7
10	27550	1,2	0.289	0.289	7
11	6805	1	3.213	0.290	7
11		2	1.645	0.290	7
11		3	4.945	0.290	7
11		4	1.404	0.290	6
11		5	1.219	0.290	6
11		7	1.494	0.290	6
11		9	2.204	0.290	6
11		10	0.290	0.290	6
11		11	3.135	0.290	6
12	34029	1,2	0.296	0.296	7
13	55049	1	0.303	0.303	9
14	27076	1,2,3	0.321	0.321	9

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
15	27744	1	0.338	0.338	8
16	62875	1,2 - B2	0.330	0.356	7
16		3,4 - B8	1.565	0.356	7
16		3,4 - B9	0.991	0.356	7
16		5,6,7 - B21	1.836	0.356	7
16		5,6,7 - B24	0.356	0.356	8
17	02550	1	0.399	0.399	8
18	69575	1	0.419	0.419	7
19	86005	1	1.405	0.429	7
19		2,3,4	0.118	0.429	8
19		5	0.429	0.429	8
20	62035	1,2	0.434	0.434	8
21	62070	1,2	0.436	0.436	8
22	62062	1,6	0.484	0.455	7
22		2,3,4,5	0.455	0.455	7
23	27545	1,2	0.460	0.460	7
24	70004	1	1.278	0.476	7
24		2	0.476	0.476	8
25	27749	1	0.500	0.500	8
26	27193	1	0.503	0.503	5
27	02813	1	0.511	0.511	5
28	27738	1,2	0.527	0.527	5
29	85025	1	0.534	0.534	5
30	9049	1,2,3	0.534	0.534	5
31	55003	1,2	0.539	0.539	7
32	79020	1	0.543	0.543	7
33	84003	1	0.548	0.548	7
34	27942	1	0.562	0.562	7
35	32812	1,4	1.477	0.582	7
35		2,3	0.582	0.582	7
36	21002	1	0.582	0.582	8
37	27087	1	1.491	0.589	8
37		2	0.589	0.589	7
37		3	0.628	0.589	7
37		4	1.129	0.589	7
38	19826	1,2	0.592	0.592	8
39	36005	1	0.595	0.595	8
40	33003	1,2	0.603	0.603	8
41	25016	1	0.605	0.605	7
42	43011	1,2	0.648	0.648	7
43	19013	1	0.654	0.654	7
44	02811	1	0.654	0.654	7

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
45	46834	1	0.656	0.656	7
46	82021	1,2,3	0.670	0.670	7
47	28003	1	0.681	0.681	7
48	27042	2,3,4	0.691	0.691	7
49	37006	1,2,3,4	0.704	0.704	8
50	27899	1	1.100	0.707	7
50		2	0.707	0.707	7
50		3	0.859	0.707	8
51	55018	1	0.902	0.721	6
51		2	0.721	0.721	6
52	19014	1,2	0.723	0.723	6
53	9570	1,4	14.078	0.736	6
53		2,3	0.736	0.736	6
54	5955	1	0.919	0.747	6
54		2	0.747	0.747	7
55	27714	1	1.442	0.804	8
55		2	0.804	0.804	8
56	87010	1,2,3	0.804	0.804	6
57	9148	1,2	0.864	0.864	6
58	9472	1	2.277	0.867	7
58		2	0.867	0.867	7
59	53005	1	0.875	0.875	7
60	07041	1	0.879	0.879	7
61	12002	1,2	0.903	0.903	7
62	9881	1,3	3.978	0.904	8
62		2	0.711	0.904	7
62		1A,3A	2.993	0.904	7
62		2A	0.904	0.904	7
63	20001	1,3	24.998	0.905	7
63		2	0.905	0.905	7
64	69087	1	0.908	0.908	7
65	69081	1,2,3	0.935	0.935	7
66	17008	1,2,3	0.939	0.939	7
67	85829	1	14.488	0.944	7
67		2	0.944	0.944	7
67		3	12.220	0.944	7
68	76005	1	1.302	0.952	7
68		2	0.952	0.952	7
68		3	1.451	0.952	7
69	09005	1	0.956	0.956	No BIR/SIR
70	9894	1,2	0.966	0.966	No BIR/SIR
71	79022	1,2,3	0.975	0.975	No BIR/SIR

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
72	55060	1	0.982	0.982	7
73	46816	1	0.982	0.982	7
74	27707	1	1.075	0.983	7
74		2	0.983	0.983	7
75	39006	1	1.017	1.017	7
76	27551	1,2	1.026	1.026	7
77	12003	2	1.027	1.027	7
77		3	1.115	1.027	7
78	27523	1,2	1.027	1.027	7
79	68004	1	1.031	1.031	7
80	32807	1	1.037	1.037	7
81	42011	1	1.054	1.054	7
82	22815	1	1.054	1.054	7
83	82020	1	4.154	1.059	7
83		2	1.059	1.059	7
83		3	4.269	1.059	7
84	9432	1,2	1.059	1.059	7
85	83017	1,2,3	1.064	1.064	7
86	69103	1	1.065	1.065	7
87	14013	1	1.850	1.067	7
87		2	1.067	1.067	7
88	07040	1,4	1.092	1.092	7
88		2,3	1.107	1.092	7
89	9869	1,4	11.073	1.097	7
89		2,3	1.097	1.097	7
90	46004	1,2	1.120	1.120	5
91	27733	1	1.145	1.145	7
91		2	1.145	1.145	7
92	19076	1	1.153	1.153	7
92		3,4	2.242	1.153	7
93	27630	1	1.159	1.159	7
93		2	1.394	1.159	7
94	27R07	1	1.173	1.173	6
95	24865	1	1.179	1.179	6
96	69074	1	1.179	1.179	6
97	73860	1,2	1.202	1.202	6
98	24837	1,2,3	1.219	1.219	7
99	38009	1,2	1.222	1.222	5
100	22004	1	1.223	1.223	5
101	9567	1,2,4	1.588	1.225	5
101		3	1.225	1.225	5
102	27V16	1	1.240	1.240	5

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
103	83012	1	6.569	1.248	5
103		2,3	1.248	1.248	5
103		4	7.979	1.248	5
104	56013	1	1.260	1.260	7
105	17004	1,2,3	1.284	1.284	7
106	9469	1,4	7.336	1.285	7
106		2,3	1.285	1.285	7
107	36007	1,3	1.522	1.301	7
107		2	1.301	1.301	7
108	79009	1,2,3,4,5	1.303	1.303	7
109	55030	1	1.308	1.308	7
110	37007	1,2,3,4	1.314	1.314	6
111	73865	1	1.327	1.327	6
112	73866	1	1.327	1.327	6
113	27750A	1	1.331	1.331	6
114	27555	1,2	1.346	1.346	7
115	27903	1	11.873	1.353	7
115		2	1.353	1.353	7
116	27750	1	1.358	1.358	7
117	22819	1,4	4.231	1.363	7
117		2,3	1.363	1.363	8
118	23006	1,2	1.374	1.374	8
119	85017	1	1.375	1.375	7
120	85018	1	1.375	1.375	8
121	27895	1,2	1.384	1.384	8
122	27897	1,2	1.402	1.402	7
123	36020	1,2	1.435	1.435	7
124	55035	1,2	1.438	1.438	7
125	64003	1	1.440	1.440	7
126	9603	1,3	25.357	1.495	8
126		2	1.495	1.495	8
127	55024	1,2	1.499	1.499	8
128	85839	1	16.062	1.510	
128		2,3	1.510	1.510	
128		4	9.984	1.510	
129	73869	1,3	7.822	1.515	
129		2	1.515	1.515	
130	73870	1,3	7.822	1.515	
130		2	1.515	1.515	
131	66822	1	74.520	1.523	
131		2,3,4	5.144	1.523	
131		5	1.523	1.523	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
131		6	21.529	1.523	
132	25027	1	1.525	1.525	
132		2,3	0.306	1.525	
133	27770D	1G54-3	1.528	1.528	
133		1G54-5	1.474	1.528	
133		1G54-16	1.080	1.528	
133		1G54-17	4.993	1.528	
133		1G54-22	6.080	1.528	
133		6	0.718	1.528	
133		7,8,9	1.179	1.528	
134	27770A	1,2,3,4	1.793	1.534	
134		5	1.446	1.534	
134		6,7	1.534	1.534	
134		8	1.730	1.534	
135	53822	1	8.589	1.536	
135		2	2.159	1.536	
135		3	1.536	1.536	
135		4	6.087	1.536	
136	9011	1,2,3	1.537	1.537	
137	75000	1	1.537	1.537	
138	46003	1	1.541	1.541	
139	43004	1	10.164	1.550	
139		2,3	1.550	1.550	
139		4,5,6	1.643	1.550	
140	55033	1,2,3	1.595	1.595	
141	13004	1,3	2.949	1.595	
141		2	1.595	1.595	
142	66823	1	12.274	1.598	
142		2,3,4	4.187	1.598	
142		5	1.598	1.598	
143	7027	1	1.600	1.600	
144	37005	1,2,3,4	1.603	1.603	
145	73858	1	11.241	1.606	
145		2,3	1.606	1.606	
145		4	19.813	1.606	
146	07029	1SB	0.486	1.612	
146		1NB	1.612	1.612	
146		2SB	1.658	1.612	
146		2NB	3.814	1.612	
147	85842	1,3	5.144	1.616	
147		2	1.616	1.616	
148	55028	1,2,3	1.618	1.618	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
149	12001	1	3.938	1.621	
149		2,3	1.621	1.621	
149		4	1.771	1.621	
150	32806	1,4	6.306	1.635	
150		2,3	1.635	1.635	
151	85823	1	5.826	1.636	
151		2	5.448	1.636	
151		3	1.636	1.636	
151		4	1.646	1.636	
151		5	1.758	1.636	
152	2024	1	8.737	1.636	
152		2	1.636	1.636	
153	27075	1	1.649	1.649	
154	28020	1	1.664	1.664	
155	9070	1,3	8.268	1.667	
155		2	1.667	1.667	
156	27044	1,2,3,4	1.675	1.675	
157	73807	1,3	14.033	1.678	
157		2	1.678	1.678	
158	73808	1,3	14.033	1.678	
158		2	1.678	1.678	
159	69025	1,3	9.797	1.678	
159		2	1.678	1.678	
160	46002	1	1.679	1.679	
161	4005	1,4	16.261	1.679	
161		2,3	1.679	1.679	
162	86811	1,4	13.034	1.693	
162		2,3	1.693	1.693	
163	62860	1,2	1.710	1.710	
164	34023	1	1.720	1.720	
165	4013	1,2	1.736	1.736	
166	85822	1,4	15.820	1.738	
166		2,3	1.738	1.738	
167	9605	1	23.545	1.741	
167		2	2.085	1.741	
167		3	1.741	1.741	
167		4	14.747	1.741	
168	82001	1,4 WB	5.002	1.752	
168		2,3 WB	1.752	1.752	
168		1,4 EB	6.046	1.752	
168		2,3 EB	2.120	1.752	
169	37010	1	1.752	1.752	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
170	24005	1,2,3	1.753	1.753	
171	24831	1	5.869	1.753	
171		2	1.753	1.753	
171		3	9.162	1.753	
172	27799	1SB	4.888	1.760	
172		1NB	7.957	1.760	
172		3SB	2.786	1.760	
172		3NB	1.760	1.760	
172		5SB	4.285	1.760	
172		5NB	3.116	1.760	
172		9SB	4.353	1.760	
173	32810	1,4	10.621	1.761	
173		2,3	1.761	1.761	
174	42006	1,3	24.916	1.767	
174		2	1.767	1.767	
175	85845	1	8.023	1.771	
175		2	1.771	1.771	
175		3	8.070	1.771	
176	85846	1	8.023	1.771	
176		2	1.771	1.771	
176		3	8.070	1.771	
177	32002	1,2	1.777	1.777	
178	83015	1	15.500	1.785	
178		2	1.785	1.785	
178		3	23.186	1.785	
179	13810	1,4	25.447	1.794	
179		2,3	1.794	1.794	
180	27770B	1	0.067	1.806	
180		2,3,4	1.806	1.806	
181	27945	1	43.050	1.817	
181		2	1.817	1.817	
181		3	2.026	1.817	
181		4	2.186	1.817	
182	55815	1,3	14.880	1.836	
182		2	1.836	1.836	
183	62875A	1,2	0.330	1.836	
183		3,4 - B9	1.077	1.836	
183		3,4 - B12	1.565	1.836	
183		5,6,7 - B18	0.600	1.836	
183		5,6,7 - B29	0.487	1.836	
183		5,6,7 - B15	1.836	1.836	
184	31030	1,2,3	1.839	1.839	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
185	70519	1	0.324	1.842	
185		2	1.842	1.842	
186	82018	1	1.848	1.848	
187	53815	1,4	7.884	1.863	
187		2	1.863	1.863	
187		3	1.863	1.863	
188	69887B	1	1.866	1.866	
188		2	3.262	1.866	
189	14811	1,4	16.360	1.869	
189		2,3	1.869	1.869	
190	24825	1	7.597	1.871	
190		2	1.871	1.871	
190		3	2.087	1.871	
190		4	2.306	1.871	
190		5	2.635	1.871	
191	46818	1,2,3	1.879	1.879	
192	27770E	1	5.611	1.879	
192		2,3,4,5	4.332	1.879	
192		6	1.988	1.879	
192		7	1.252	1.879	
192		8	1.879	1.879	
193	86815	1	7.038	1.888	
193		2,3	1.888	1.888	
193		4	10.707	1.888	
194	49023	1	1.900	1.900	
195	83021	1	18.897	1.918	
195		2,3	1.918	1.918	
195		4	9.432	1.918	
196	27741	1	1.919	1.919	
197	84801	1,4	29.927	1.931	
197		2,3	1.931	1.931	
198	2552	1,2	1.938	1.938	
199	62827	1,2	1.952	1.952	
200	71001	1,4	18.212	1.958	
200		2	1.958	1.958	
200		3	2.424	1.958	
201	39011	1,2,3	1.961	1.961	
202	69106	1,2,3	1.972	1.972	
203	7039	1,4	1.976	1.976	
203		2,3	2.025	1.976	
204	24862	1,4	20.239	1.981	
204		2,3	1.981	1.981	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
205	69064	1,3	6.107	1.982	
205		2	1.982	1.982	
206	83022	1,3	2.854	2.000	
206		2	2.000	2.000	
207	12009	1,2,3,4,5,6	2.002	2.002	
208	19859	1	11.965	2.007	
208		2,3	2.007	2.007	
208		4,5	2.796	2.007	
208		6	19.052	2.007	
209	83005	1	7.002	2.057	
209		2	0.850	2.057	
209		3	0.414	2.057	
209		4	2.057	2.057	
210	25017	1,2,3	2.060	2.060	
211	70801	1,4	30.223	2.061	
211		2,3	2.061	2.061	
212	21802	1,3	2.068	2.068	
212		2	0.683	2.068	
213	13807	1,4	23.897	2.074	
213		2,3	2.074	2.074	
214	58006	1	3.503	2.081	
214		2	0.137	2.081	
214		3	2.081	2.081	
215	74813	1	23.940	2.087	
215		1 - widened	8.956	2.087	
215		2,3	2.087	2.087	
215		2,3 - widened	0.190	2.087	
215		4	26.944	2.087	
215		4 - widened	6.600	2.087	
216	32802	1,4	11.007	2.090	
216		2,3	2.090	2.090	
217	74806	1,4	61.028	2.091	
217		2,3	2.091	2.091	
218	25009	1,2,3	2.116	2.116	
219	22801	1,4	9.827	2.133	
219		2,3	2.133	2.133	
220	22811	1,4	9.827	2.135	
220		2,3	2.135	2.135	
221	5859	1,2,3	2.143	2.143	
222	22809	1,4	5.068	2.165	
222		2,3	2.165	2.165	
223	19016	1	22.110	2.165	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
223		2	2.165	2.165	
223		3	27.666	2.165	
224	86809	1,4	18.919	2.165	
224		2,3	2.165	2.165	
225	60006	1,4	15.833	2.165	
225		2,3	2.165	2.165	
226	64002	1	2.166	2.166	
227	9822	1,4	17.208	2.173	
227		2,3	2.173	2.173	
228	46814	1,4	6.863	2.175	
228		2,2	2.175	2.175	
229	86807	1,4	14.241	2.198	
229		2,3	2.198	2.198	
230	32006	1,2	2.215	2.215	
231	31010	1,2,3	2.219	2.219	
232	42007	1,2,3	2.224	2.224	
233	48013	1,4	25.812	2.253	
233		2,3	2.253	2.253	
234	46823	1	9.176	2.259	
234		2	2.259	2.259	
235	64004	1	2.281	2.281	
236	34013	1	2.285	2.285	
237	32815	1,4	9.538	2.286	
237		2,3	2.286	2.286	
238	49009	1	25.210	2.296	
238		2	14.425	2.296	
238		3	2.296	2.296	
238		4	13.377	2.296	
239	20007	1,3	2.302	2.302	
239		2	0.694	2.302	
240	82817	1	27.459	2.303	
240		2	2.303	2.303	
240		3	0.730	2.303	
240		4	36.260	2.303	
241	8011	1,2	2.317	2.317	
242	02028	1-N fascia	0.224	2.319	
242		1-SIB1	2.319	2.319	
242		1-S fascia	1.223	2.319	
242		2-N fascia	0.209	2.319	
242		2-SIB1	1.488	2.319	
242		2-S fascia	0.773	2.319	
243	60008	1	2.320	2.320	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
244	32804	1,4	10.324	2.338	
244		2,3	2.338	2.338	
245	19015	1	22.110	2.348	
245		2	2.348	2.348	
245		3	27.666	2.348	
246	36013	1,2,3,4,5	2.350	2.350	
247	69818B	3	3.140	2.351	
247		6	2.351	2.351	
248	27226	1	3.393	2.353	
248		2	2.353	2.353	
248		3	0.842	2.353	
249	58001	1,3	2.357	2.357	
249		2	2.382	2.357	
250	31009	1,2,3	2.379	2.379	
251	24860	1,4	6.463	2.387	
251		2,3	2.387	2.387	
252	70802	1,4	32.281	2.388	
252		2,3	2.388	2.388	
253	55022	1,2,3,4,5	2.388	2.388	
254	83026	1,4	20.443	2.393	
254		2,3	2.393	2.393	
255	62044	2	2.397	2.397	
255		3	7.285	2.397	
256	24806	1,4	35.384	2.412	
256		2,3	2.412	2.412	
257	56804	1	31.958	2.414	
257		2	2.414	2.414	
258	82816	1,4	32.674	2.417	
258		2,3	2.417	2.417	
259	9471	1	23.659	2.435	
259		2	1.954	2.435	
259		3	2.435	2.435	
259		4	0.506	2.435	
260	9528	1	21.073	2.439	
260		2	5.014	2.439	
260		3	4.609	2.439	
260		4	2.439	2.439	
261	55805	2	2.446	2.446	
261		3	8.293	2.446	
262	19022	1,4	15.125	2.472	
262		2,3	2.472	2.472	
263	22820	1,4	11.341	2.487	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
263		2,3	2.487	2.487	
264	74013	1	2.492	2.492	
265	85826	1,4	22.229	2.499	
265		2,3	2.499	2.499	
266	9463	1,3	2.501	2.501	
266		2	0.902	2.501	
267	69035	1 - NB	19.395	2.506	
267		2,3 - NB	2.506	2.506	
267		4 - NB,SB	49.239	2.506	
267		1 - SB	15.025	2.506	
267		2,3 - SB	3.747	2.506	
268	22812	1,4	13.155	2.559	
268		2,3	2.559	2.559	
269	62837	1	10.726	2.581	
269		2,3	2.581	2.581	
269		4	7.842	2.581	
270	7037	1,2,3	2.630	2.630	
271	22802	1,4	6.468	2.638	
271		2,3	2.638	2.638	
272	66811	1	4.946	2.640	
272		2,3	2.640	2.640	
272		4	2.836	2.640	
273	21803	1,3	13.121	2.645	
273		2	2.645	2.645	
274	82019	1	6.224	2.647	
274		3	2.647	2.647	
274		4	6.705	2.647	
275	9830	1,4	30.461	2.648	
275		2,3	2.648	2.648	
276	22810	1,4	5.603	2.672	
276		2,3	2.672	2.672	
277	66812	1	2.674	2.674	
277		2,3	1.297	2.674	
277		4	0.449	2.674	
278	46813	1,4	27.851	2.692	
278		2,3	2.692	2.692	
279	27536	1	2.701	2.701	
279		2	3.356	2.701	
280	83025	1	9.757	2.703	
280		2,3	2.703	2.703	
280		4	7.922	2.703	
281	85814	1,4	33.776	2.709	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
281		2,3	2.709	2.709	
282	22805	1	2.725	2.725	
282		2,3	0.552	2.725	
282		4	0.691	2.725	
283	9829	1,4	30.611	2.741	
283		2,3	2.741	2.741	
284	69088	1	2.783	2.783	
285	55803	1,3	14.575	2.784	
285		2	2.784	2.784	
286	56802	1,4	46.077	2.784	
286		2,3	2.784	2.784	
287	27963	1	8.846	2.801	
287		2	2.563	2.801	
287		3	0.566	2.801	
287		4	3.320	2.801	
287		5	2.839	2.801	
287		6	2.801	2.801	
287		7	17.885	2.801	
288	34524	2	2.808	2.808	
288		3 - B8	0.314	2.808	
288		3 - B11	0.775	2.808	
288		4	0.304	2.808	
288		5,6	0.115	2.808	
288		7	1.110	2.808	
289	32816	1,4	33.182	2.833	
289		2,3	2.833	2.833	
290	32818	1	17.708	2.833	
290		2	2.833	2.833	
291	9258	1,4	2.836	2.836	
291		2,3	0.570	2.836	
292	62079	1,2,3	2.842	2.842	
293	69082	1,2,3	2.851	2.851	
294	46824	1,4	15.649	2.852	
294		2,3	2.852	2.852	
295	22806	1,4	2.861	2.861	
295		2,3	0.753	2.861	
296	17007	1	2.891	2.891	
297	32805	1	13.663	2.891	
297		2	2.891	2.891	
298	32801	1	33.867	2.891	
298		2,3	2.891	2.891	
298		4	14.605	2.891	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
299	32814	1,4	15.757	2.891	
299		2,3	2.891	2.891	
300	3006	1,4	24.865	2.943	
300		2.3	2.943	2.943	
301	19055	2	0.294	2.986	
301		4	2.986	2.986	
302	27799R	1	1.423	3.023	
302		2	0.786	3.023	
302		3	0.621	3.023	
302		4	2.144	3.023	
302		5	0.435	3.023	
302		8	1.899	3.023	
302		9	3.023	3.023	
303	69887A	2	3.045	3.045	
303		4	1.374	3.045	
304	85847	1	21.575	3.117	
304		2,3	3.117	3.117	
304		4	7.753	3.117	
305	79024	1	3.159	3.159	
306	77014	1,2	3.171	3.171	
307	14006	1,3	3.175	3.175	
307		2	0.702	3.175	
308	56012	1,3	4.296	3.176	
308		2	3.176	3.176	
309	65002	1,2	3.178	3.178	
310	69818A	1,2,3.4	3.226	3.226	
311	53814	1,3	3.230	3.230	
311		2	0.431	3.230	
312	27840	1	119.656	3.256	
312		2	1.557	3.256	
312		3	0.765	3.256	
312		4	3.256	3.256	
313	27568	1,2….39	3.270	3.270	
314	02027	1 - NIB1	3.297	3.297	
314		1 - SIB1	2.978	3.297	
314		2 - NIB1	0.293	3.297	
314		2 - SIB3	1.353	3.297	
314		2 - SIB1	1.725	3.297	
314		int. beam orig.	1.899	3.297	
315	44002	1,2	3.357	3.357	
316	85815	1	9.614	3.362	
316		2	3.362	3.362	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
317	32003	1,2	3.413	3.413	
318	09820	1	3.423	3.423	
318		2,3	1.322	3.423	
318		4	27.377	3.423	
319	42005	1	3.488	3.488	
320	73812	1,4	38.324	3.492	
320		2,3	3.492	3.492	
321	46831	1,4	8.406	3.499	
321		2,3	3.499	3.499	
322	25007	1,2	3.524	3.524	
323	50007	1,2	3.598	3.598	
324	73872	1,4	15.757	3.601	
324		2,3	3.601	3.601	
325	46832	1,4	17.125	3.611	
325		2,3	3.611	3.611	
326	49019	1,4	25.796	3.627	
326		2.,3	3.627	3.627	
327	51001	1	3.640	3.640	
328	9716	1,4	49.795	3.641	
328		2,3	3.641	3.641	
329	27799L	1	9.080	3.648	
329		2,3	1.407	3.648	
329		4	3.648	3.648	
329		5,6,7,8	2.008	3.648	
329		9	1.933	3.648	
330	53001	1	3.651	3.651	
331	74010	1	1.108	3.674	
331		2	0.843	3.674	
331		3	3.172	3.674	
331		4	3.674	3.674	
332	19825	1	9.190	3.702	
332		2,3,4	3.702	3.702	
332		5	2.411	3.702	
332		6	0.989	3.702	
332		7	1.928	3.702	
333	27787	1	3.718	3.718	
334	9832	1,4	49.176	3.738	
334		2,3	3.738	3.738	
335	19041	1,4	9.029	3.773	
335		2,3	3.773	3.773	
336	79025	1	3.775	3.775	
337	46807	1	3.777	3.777	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
338	84805	1,4	28.974	3.785	
338		2,3	3.785	3.785	
339	4009	1	3.790	3.790	
340	82002	1	5.949	3.792	
340		2,3	3.792	3.792	
340		4	14.474	3.792	
341	27077	1	0.639	3.816	
341		9	3.816	3.816	
342	27740	1	3.817	3.817	
343	62851	1	23.271	3.827	
343		2,3	3.827	3.827	
343		4	60.782	3.827	
344	73815	1,4	21.378	3.839	
344		2,3	3.839	3.839	
345	62852	1	28.914	3.840	
345		2,3	3.840	3.840	
345		4	75.520	3.840	
346	27957	1,2	3.895	3.895	
347	5006	1,4	27.272	3.903	
347		2,3	3.903	3.903	
348	64005	1,3	6.587	3.910	
348		2	3.910	3.910	
349	46804	1,2,3	3.967	3.967	
350	48004	1,2	4.006	4.006	
351	69111	1,2,3	4.033	4.033	
352	9825	1,3	9.209	4.047	
352		2	4.047	4.047	
353	69067	1,3	6.730	4.051	
353		2	4.051	4.051	
354	55806	1	14.522	4.062	
354		2	4.062	4.062	
354		3	13.963	4.062	
355	27716	1,2	4.083	4.083	
355		3	0.102	4.083	
355		4	0.072	4.083	
355		5	0.250	4.083	
355		6	0.074	4.083	
356	9521	1	35.007	4.095	
356		2	4.095	4.095	
356		3	56.473	4.095	
357	9827	1,2,3	4.105	4.105	
358	74809	1,3	4.188	4.188	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
358		2	4.188	4.188	
359	24809	1,4	10.058	4.189	
359		2,3	4.189	4.189	
360	22001	1,2	4.197	4.197	
361	58809	1	56.022	4.199	
361		2	4.199	4.199	
362	50801	1,4	49.413	4.230	
362		2,3	4.230	4.230	
363	73877	1,2	4.240	4.240	
363		3	6.952	4.240	
364	73878	1,3	4.240	4.240	
364		2	6.952	4.240	
365	12011	1,2	4.259	4.259	
366	69046	1	4.263	4.263	
367	25023	1	12.665	4.273	
367		2,3	4.273	4.273	
368	74805	1,4	58.147	4.283	
368		2,3	4.283	4.283	
369	74821	1,4	58.147	4.283	
369		2,3	4.283	4.283	
370	7015	1,3	6.689	4.331	
370		2	4.331	4.331	
371	53801	1,4	58.147	4.336	
371		2,3	4.336	4.336	
372	67809	1,4	50.610	4.336	
372		2,3	4.336	4.336	
373	53809	1	50.610	4.340	
373		2	4.340	4.340	
374	46817	2	4.346	4.346	
375	53811	1	48.640	4.356	
375		2	4.356	4.356	
376	53824	1	48.640	4.356	
376		2	4.356	4.356	
377	67811	1,4	51.245	4.390	
377		2,3	4.390	4.390	
378	23007	1,2	4.450	4.450	
379	23007b	1,2	4.450	4.450	
380	9007	1,2	4.462	4.462	
381	9723	1,4	13.144	4.479	
381		2,3	4.479	4.479	
382	27770F	1,2,3,4	1.297	4.485	
382		5	4.485	4.485	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
383	55810	1	1.443	4.558	
383		2	1.766	4.558	
383		3	1.484	4.558	
383		4	4.588	4.558	
384	62876	1,2	0.359	4.586	
384		3,4	0.635	4.586	
384		5	4.586	4.586	
384		6	1.215	4.586	
385	42002	1	4.600	4.600	
386	5005	1	32.667	4.601	
386		2,3	4.601	4.601	
386		4	24.430	4.601	
387	25014	1,2,3	4.625	4.625	
388	86803	1,4 - 45	4.625	4.625	
388		2,3 - 45	0.261	4.625	
389	69061	1,2,3	4.661	4.661	
390	11006	1,2,3	4.679	4.679	
391	24817	1,4	36.764	4.721	
391		2,3	4.721	4.721	
392	31028	1	4.778	4.778	
393	85828	1,4	17.637	4.786	
393		2,3	4.786	4.786	
394	53007	1,3	4.801	4.801	
394		2	0.375	4.801	
395	69068	1,3	7.539	4.815	
395		2	4.815	4.815	
396	10022	1	4.830	4.830	
396		2	0.402	4.830	
396		3	1.607	4.830	
396		4	0.939	4.830	
396		5	5.743	4.830	
397	24813	1,4	9.906	4.905	
397		2,3	4.905	4.905	
398	27831A	1	5.067	5.067	
398		2,3	6.895	5.067	
398		4	6.596	5.067	
399	27082	1,4	26.981	5.120	
399		2,3	5.120	5.120	
400	83037	1,3	5.188	5.188	
400		2	0.547	5.188	
401	55031	1,2,3	5.208	5.208	
402	16002	1	5.321	5.321	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
403	79023	1	3.836	5.339	
403		2,3	1.135	5.339	
403		4	5.339	5.339	
404	62839	1	7.028	5.379	
404		2	5.379	5.379	
404		3	12.598	5.379	
405	25013	1,2,3	5.441	5.441	
406	24833	1	7.419	5.514	
406		2	0.854	5.514	
406		3	0.375	5.514	
406		4	5.514	5.514	
407	31022S	1	0.437	5.585	
407		2-G8	1.491	5.585	
407		2-G9	1.862	5.585	
407		2-G10	2.340	5.585	
407		3,4,5,6	0.592	5.585	
407		7	5.585	5.585	
407		8-G48	0.889	5.585	
407		8-G49	1.395	5.585	
407		8-G50	2.374	5.585	
407		9	0.566	5.585	
408	31019	1,4	5.585	5.585	
408		2	0.853	5.585	
408		3	8.748	5.585	
409	82812	1,3	5.646	5.646	
409		2	6.102	5.646	
410	87005	1,2,3	5.695	5.695	
411	34014	1	7.962	5.760	
411		2,3	0.998	5.760	
411		4	5.760	5.760	
412	62834	1	13.049	5.913	
412		2,3	5.913	5.913	
412		4	28.141	5.913	
413	32004	1	5.954	5.954	
414	49020	1	5.957	5.957	
415	46811	1,3	6.013	6.013	
415		2	0.157	6.013	
416	27831B	1	6.053	6.053	
416		2,3	6.682	6.053	
416		4	6.657	6.053	
416		5	6.617	6.053	
416		6	6.336	6.053	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
417	1007	1,2,3	6.064	6.064	
418	70027	1	6.064	6.064	
419	87009	1,2,3	6.116	6.116	
420	27068	1,3	6.139	6.139	
420		2	0.702	6.139	
421	27129	1	28.364	6.196	
421		2	1.371	6.196	
421		3	0.282	6.196	
421		4	6.196	6.196	
422	75002	1,2	6.299	6.299	
423	34007	1	17.257	6.306	
423		2,3	1.327	6.306	
423		4	6.306	6.306	
424	27852	1	40.201	6.340	
424		2	0.523	6.340	
424		3	6.340	6.340	
424		4	2.647	6.340	
424		5	0.848	6.340	
424		6	30.630	6.340	
425	27793	1	34.662	6.382	
425		2	8.299	6.382	
425		3	6.382	6.382	
425		4	4.720	6.382	
425		5	1.267	6.382	
425		6	29.714	6.382	
426	69085	1	0.310	6.403	
426		2	0.284	6.403	
426		3	6.403	6.403	
427	27978	1	11.777	6.442	
427		2	6.442	6.442	
427		3	18.912	6.442	
428	69076	1,3	6.535	6.535	
428		2	0.262	6.535	
429	24840	1	4.071	6.647	
429		2	0.617	6.647	
429		3	6.647	6.647	
430	27130	1	36.678	6.660	
430		2,3,4	0.634	6.660	
430		5	6.660	6.660	
431	07016	1,3	6.689	6.689	
431		2	1.037	6.689	
432	73813	1,4	46.647	6.728	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
432		2,3	6.728	6.728	
433	14810	1	6.746	6.746	
434	49014	1,4	6.754	6.754	
434		2	3.083	6.754	
434		3	0.339	6.754	
435	46822	1	6.903	6.903	
436	27089	1,4	6.973	6.973	
436		2,3	1.114	6.973	
437	36006	1	7.099	7.099	
438	50002	1	7.136	7.136	
439	24839	1	4.453	7.215	
439		2	0.670	7.215	
439		3	7.215	7.215	
440	62821	1	0.851	7.252	
440		2	0.352	7.252	
440		3	7.252	7.252	
441	27965	1	20.897	7.302	
441		2	7.302	7.302	
441		3	6.463	7.302	
441		4	5.707	7.302	
441		5	0.258	7.302	
442	69075	1,3	7.349	7.349	
442		2	0.564	7.349	
443	69086	1	5.507	7.411	
443		2	0.539	7.411	
443		3	7.411	7.411	
444	19056	1	7.463	7.463	
444		2	0.316	7.463	
444		4	8.525	7.463	
445	34008	1	17.257	7.519	
445		2,3	1.427	7.519	
445		4	7.519	7.519	
446	17003	1,2	7.599	7.599	
447	22814	1,4	7.636	7.636	
447		2,3	1.117	7.636	
448	31020	1,4	7.636	7.636	
448		2	1.258	7.636	
448		3	11.956	7.636	
449	01015	1,3	7.649	7.649	
449		2	0.837	7.649	
450	19053	1	7.762	7.762	
450		2,3	0.559	7.762	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
450		4	15.278	7.762	
451	9002	2	7.904	7.904	
452	13811	1,4	7.970	7.970	
452		2,3	0.638	7.970	
453	22825	1,4	8.089	8.089	
453		2,3	0.465	8.089	
454	73861	1,4	8.242	8.242	
454		2,3	1.091	8.242	
455	73857	1	8.383	8.383	
455		2	7.566	8.383	
455		3,4	0.676	8.383	
455		5	12.963	8.383	
456	69890	1,3	8.411	8.411	
456		2	0.246	8.411	
457	83027	1,4	8.478	8.478	
457		2,3	0.453	8.478	
458	55017	1	8.523	8.523	
458		2	0.570	8.523	
458		4	17.280	8.523	
459	9830	1,4	71.011	8.992	
459		2,3	8.992	8.992	
460	56018	1	9.254	9.254	
461	73852	1,4	9.529	9.529	
461		2,3	1.385	9.529	
462	27950	1	12.038	9.719	
462		2	0.419	9.719	
462		4	9.719	9.719	
463	4874	1	9.736	9.736	
464	86817	1,3	9.766	9.766	
464		2	1.125	9.766	
465	69814	1,3	9.820	9.820	
465		2	0.926	9.820	
466	83016	1	10.168	10.168	
466		2	1.120	10.168	
466		3	14.790	10.168	
467	73850	1	16.122	10.176	
467		2	1.203	10.176	
467		3	10.176	10.176	
468	18008	1,3	10.215	10.215	
468		2	0.258	10.215	
469	18007	1,3	10.217	10.217	
469		2	0.258	10.217	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
470	27853	1	1.226	10.533	
470		3	10.533	10.533	
470		4	3.841	10.533	
470		5	187.774	10.533	
471	49037	1,3	10.594	10.594	
471		2	1.002	10.594	
472	56001	1,4	10.723	10.723	
472		2,3	0.313	10.723	
473	73817	1,4	10.882	10.882	
473		2,3	0.824	10.882	
474	19018	1	1.272	11.202	
474		2,3	1.128	11.202	
474		4	11.202	11.202	
475	52007	1	11.913	11.913	
475		2,3	1.165	11.913	
475		4	9.632	11.913	
476	48020	1,3	11.925	11.925	
476		2	0.250	11.925	
477	42013	1	0.787	11.965	
477		2	0.558	11.965	
477		3	11.965	11.965	
478	73022	1,3	12.059	12.059	
478		2	0.167	12.059	
479	19813	1	29.645	12.203	
479		2	0.299	12.203	
479		3	12.203	12.203	
480	09821	1,4	12.241	12.241	
480		2,3	0.731	12.241	
481	48019	1,3	12.298	12.298	
481		2	0.258	12.298	
482	31022N	1-G5	0.792	12.485	
482		2-G13	12.485	12.485	
482		2-G12	8.205	12.485	
482		2-G14	20.406	12.485	
482		3,4,5,6	0.592	12.485	
482		8-G32	0.600	12.485	
482		8-G33	0.889	12.485	
482		8-G34	1.395	12.485	
482		8-G35	2.374	12.485	
482		9	0.566	12.485	
483	24801	1,4	12.624	12.624	
483		2,3	0.312	12.624	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
484	56007	1,4	12.772	12.772	
484		2,3	1.131	12.772	
485	69892	1,3	12.842	12.842	
485		2	0.999	12.842	
486	04017	1	21.294	13.141	
486		2	0.446	13.141	
486		3	0.543	13.141	
486		4	13.141	13.141	
487	04018	1	20.817	13.141	
487		2,3	0.446	13.141	
487		4	13.141	13.141	
488	19860	1	1.189	13.273	
488		2,3,4	1.471	13.273	
488		5	1.277	13.273	
488		6	13.273	13.273	
489	55808	1,4	13.841	13.841	
489		2,3	0.341	13.841	
490	02023	1	14.300	14.303	
490		1 - widening	1.753	14.303	
490		2	1.636	14.303	
490		2 - widening	0.701	14.303	
491	48009	1,3	15.388	15.388	
491		2	0.637	15.388	
492	48010	1,3	15.388	15.388	
492		2	0.637	15.388	
493	49016	1,3	15.439	15.439	
493		2	1.262	15.439	
494	69891	1,3	15.443	15.443	
494		2	1.344	15.443	
495	13803	1,4	15.573	15.573	
495		2,3	0.867	15.573	
496	27041	1,4	16.359	16.359	
496		2,3	0.503	16.359	
497	07019	1,3	16.555	16.555	
497		2	1.409	16.555	
498	62835	1,4	17.966	17.966	
498		2,3	1.038	17.966	
498		1,4 - 54	18.017	18.017	
498		2,3 - 54	1.971	18.017	
499	27540	1	18.401	18.401	
499		2	0.596	18.401	
499		3	27.724	18.401	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
500	86808	1,4	18.984	18.984	
500		2,3	0.933	18.984	
501	55814	1	32.721	19.602	
501		2,3	0.545	19.602	
501		4	19.602	19.602	
502	14807	1,4	19.610	19.610	
502		2,3	1.244	19.610	
503	13802	1,4	20.219	20.219	
503		2,3	1.071	20.219	
504	46806	1	16.760	20.546	
504		2	1.209	20.546	
504		3	20.546	20.546	
505	52006	1,4 SB	19.069	20.956	
505		2,3 SB	0.691	20.956	
505		1,4 NB	20.956	20.956	
505		2,3 NB	0.578	20.956	
506	30005	1	21.692	21.692	
506		2,3	0.268	21.692	
506		4	10.538	21.692	
507	19814	1	71.986	22.552	
507		2	0.303	22.552	
507		3	22.552	22.552	
508	19067	1	4.821	22.820	
508		2,3	0.650	22.820	
508		4	22.820	22.820	
509	55813	1,4	23.646	23.646	
509		2,3	0.543	23.646	
510	24835	1,4	24.020	24.020	
510		2,3	1.012	24.020	
511	24836	1,4	24.379	24.379	
511		2,3	1.001	24.379	
512	56816	1	24.835	24.835	
512		2	0.462	24.835	
513	62825	1,3	26.021	26.021	
513		2	0.722	26.021	
514	14808	1,4	27.521	27.521	
514		2,3	1.002	27.521	
515	62029	1	21.561	27.980	
515		2	1.281	27.980	
515		3	27.980	27.980	
516	27865	1	46.058	29.248	
516		3	0.712	29.248	

BRIDGE DATA			SPAN STIFFNESS RATIO	MIN./GOV BRIDGE STIFFNESS RATIO	NBI RATING
BRIDGE COUNT	STRUCTURE NUMBER	SPAN			
516		4	29.248	29.248	
517	84803	1,4	34.293	34.293	
517		2,3	1.107	34.293	
518	67810	1,4	36.542	36.542	
518		2,3	1.311	36.542	
519	58810	1	36.542	36.542	
519		2	1.276	36.542	
520	24847	1,4	42.577	42.577	
520		2,3	0.500	42.577	
521	27843	1WB	16.912	43.271	
521		1EB	25.334	43.271	
521		2WB	0.541	43.271	
521		2EB	0.436	43.271	
521		3WB	0.708	43.271	
521		3EB	0.695	43.271	
521		4WB	34.132	43.271	
521		4EB	43.271	43.271	
522	46805	1,3	46.304	46.304	
522		2	0.724	46.304	

Selected for refined analysis

APPENDIX E

RATING RESULTS SUMMARY

Table E.1 Rating Results Summary

	HL-93 RATINGS			REFINED PERMIT RATINGS				
STRUCTURE NUMBER	Unrefined LRFR Inventory RF	Refined LRFR Inventory RF	Updated/Refined Improvement	STD. A	STD. B	STD. C	P411	P413
34029	1.32	1.60	21%	2.41	1.99	1.85	2.972 (2.813) (STR II flexure)	2.72
53005	1.07	1.19	11%	1.83	1.47	1.33	2.05	2.03
27193	0.89	0.98	9%	1.46	1.21	1.12	2.045 (2.030) (G7)	1.872 (1.858) (G7)
19013	1.07	1.28	19%	1.88	1.51	1.36	2.178 (2.087) (Fascia)	2.146 (2.056) (Fascia)
27042	0.83	0.96	16%	1.45	1.19	1.09	1.78	1.64
19014	0.91	1.13	23%	1.66	1.33	1.20	1.88	1.85
23006	0.85	1.00	19%	1.53	1.27	1.18	1.97	1.80
79022	0.94	1.09	16%	1.68	1.36	1.23	1.936 (1.922) (Fascia)	1.895 (1.881) (Fascia)
32807	0.84	0.98	18%	1.48	1.20	1.09	1.835 (1.608) (Fascia)	1.781 (1.561) (Fascia)
83017	0.99	1.11	13%	1.67	1.34	1.21	2.04	1.99
73860	0.98	1.13	15%	1.75	1.40	1.27	1.939 (1.908) (Fascia)	1.911 (1.881) (Fascia)
27738	0.82	0.98	19%	1.50	1.21	1.09	1.713 (1.551) (Fascia)	1.685 (1.525) (Fascia)
24837	0.62	0.73	17%	1.11	0.95	0.92	1.35	1.30
42011	0.89	1.02	15%	1.57	1.27	1.14	1.81	1.78
73866	1.06	1.19	12%	1.73	1.40	1.27	2.152 (2.057) (Fascia)	2.094 (2.011) (Fascia)
27744	0.77	0.95	23%	1.44	1.17	1.06	1.646 (1.472) (Fascia)	1.605 (1.435) (Fascia)
22815	0.90	1.01	12%	1.51	1.22	1.10	1.788 (1.636) (Fascia)	1.757 (1.608) (Fascia)
07041	0.92	1.06	15%	1.56	1.26	1.14	1.856 (1.598) (Fascia)	1.820 (1.566) (Fascia)
62035	0.92	1.12	22%	1.67	1.37	1.25	2.01	1.88
62070	0.80	1.03	28%	1.52	1.24	1.13	1.84	1.74

	HL-93 RATINGS			REFINED PERMIT RATINGS				
STRUCTURE NUMBER	Unrefined LRFR Inventory RF	Refined LRFR Inventory RF	Updated/Refined Improvement	STD. A	STD. B	STD. C	P411	P413
02813	1.00	1.14	15%	1.61	1.33	1.22	2.22	2.03
9049	0.88	1.14	29%	1.74	1.42	1.30	2.095 (2.022) (Fascia)	1.943 (1.876) (Fascia)
84003	0.98	1.11	13%	1.70	1.39	1.27	2.03	1.94
69081	0.76	0.89	17%	1.39	1.20	1.15	1.64	1.58
12002	1.16	1.49	28%	2.28	1.91	1.80	2.813 (2.585) (Fascia)	2.616 (2.404) (Fascia)
33003	0.97	1.11	14%	1.70	1.38	1.25	1.986 (1.975) (Fascia)	1.923 (1.913) (Fascia)
25016	0.97	1.11	15%	1.70	1.39	1.28	2.092 (1.763) (Fascia)	1.938 (1.633) (Fascia)
27903	0.92	1.02	10%	1.55	1.27	1.16	1.92	1.84
28003	1.25	1.55	24%	2.44	1.95	1.74	2.407 (2.269) (Fascia)	2.372 (2.235) (Fascia)
27750	1.22	1.32	9%	2.01	1.65	1.52	2.66	2.43
37006	0.98	1.11	13%	1.68	1.39	1.28	2.07	1.88
55018	0.95	1.09	14%	1.63	1.32	1.20	1.978 (1.918) (Fascia)	1.903 (1.845) (Fascia)
	0.96	1.12	18%	1.69	1.37	1.24	1.995 (1.862) (Fascia)	1.947 (1.818) (Fascia)
87010	0.92	1.04	13%	1.60	1.29	1.16	1.804 (1.750) (Fascia)	1.782 (1.729) (Fascia)
9148	0.87	0.99	13%	1.52	1.26	1.17	1.81	1.67
9894	1.02	1.17	15%	1.79	1.45	1.32	2.25	2.14
46816	0.85	0.95	11%	1.43	1.20	1.13	1.885 (1.760) (Fascia)	1.773 (1.663) (Fascia)
39006	0.99	1.20	21%	1.83	1.49	1.36	2.237 (2.003) (Fascia)	2.122 (1.900) (Fascia)
12003	0.68	0.82	21%	1.27	1.07	1.02	1.94	1.82
	0.63	0.76	21%	1.18	1.00	0.95	1.49	1.41
9432	0.93	1.05	13%	1.60	1.30	1.18	1.96	1.87

	HL-93 RATINGS			REFINED PERMIT RATINGS				
STRUCTURE NUMBER	Unrefined LRFR Inventory RF	Refined LRFR Inventory RF	Updated/Refined Improvement	STD. A	STD. B	STD. C	P411	P413
46004	1.03	1.17	15%	1.79	1.47	1.36	2.289 (2.280) (Fascia)	2.091 (2.084) (Fascia)
27630	1.15	1.27	11%	1.90	1.53	1.39	2.288 (1.930) (Fascia)	2.234 (1.885) (Fascia)
	1.35	1.45	7%	2.16	1.75	1.59	2.672 (2.333) (Fascia)	2.582 (2.255) (Fascia)
69074	0.69	0.84	21%	1.27	1.05	0.97	1.657 (1.633) (Fascia)	1.514 (1.492) (Fascia)
22004	0.79	0.85	8%	1.32	1.11	1.05	1.68	1.57
83012	1.00	1.12	13%	1.72	1.40	1.29	2.318 (2.253) (Fascia)	2.153 (2.093) (Fascia)
17004	0.84	0.91	8%	1.41	1.19	1.13	1.79	1.68
09005	1.11	1.24	11%	1.90	1.53	1.39	2.233 (2.165) (Fascia)	2.168 (2.102) (Fascia)
36007	0.96	N/A	N/A	1.48	1.21	1.12	1.91	1.75
	0.94	1.08	15%	1.64	1.34	1.23	2.06	1.92
55030	0.68	0.80	18%	1.24	1.06	1.02	1.53	1.46
36005	0.97	1.07	11%	1.65	1.32	1.19	1.889 (1.875) (Fascia)	1.862 (1.848) (Fascia)
43011	1.04	1.22	17%	1.86	1.52	1.38	2.272 (2.143) (Fascia)	2.133 (2.012) (Fascia)

Line girder model
AASHTO LLDF used in lieu of refined LLDF
Stiffness ratio > 1.5.