Bridge Analysis and Evaluation of Effects Under Overload Vehicles (Phase 2)

CFIRE 02-03
September 2012

National Center for Freight & Infrastructure Research & Education
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College of Engineering
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Technical Report Documentation Page

<table>
<tr><td colspan="2">1. Report No.
CFIRE 02-03</td><td colspan="2">2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.
CFDA 20.701</td></tr>
<tr><td colspan="4" rowspan="2">4. Title and Subtitle
Bridge Analysis and Evaluation of Effects Under Overload Vehicles (Phase 2)</td><td colspan="2">5. Report Date September 2012</td></tr>
<tr><td colspan="2">6. Performing Organization Code</td></tr>
<tr><td colspan="4">7. Author/s Dr. Han Ug-Bae, and Professor Michael G. Oliva</td><td colspan="2">8. Performing Organization Report No.
CFIRE 02-03</td></tr>
<tr><td colspan="4" rowspan="2">9. Performing Organization Name and Address
National Center for Freight and Infrastructure Research and Education (CFIRE)
University of Wisconsin-Madison
1415 Engineering Drive, 2205 EH
Madison, WI 53706</td><td colspan="2">10. Work Unit No. (TRAIS)</td></tr>
<tr><td colspan="2">11. Contract or Grant No.
DTRT06-G-0020</td></tr>
<tr><td colspan="4" rowspan="2">12. Sponsoring Organization Name and Address
Research and Innovative Technology Administration
United States Department of Transportation
1200 New Jersey Ave, SE
Washington, D.C. 20590</td><td colspan="2">13. Type of Report and Period Covered
Final Report [9/01/08 - 6/30/2012]</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code</td></tr>
<tr><td colspan="6">15. Supplementary Notes
Project completed for the USDOT by CFIRE.</td></tr>
<tr><td colspan="6">16. Abstract

The use of special purpose highway vehicles, over the legal limit in size and in weight, is increasing as industry grows and large items must be shipped over highways. Overload vehicle crossing of a bridge, even if it is a single crossing, may affect not only the short term behavior of the bridge but also the long term performance and life cycle cost of the bridge. There may be minor cracking or deterioration in the components of the bridge which are not critical in the short term period but can result in special maintenance, rehabilitation or reduced life span in the long term. It may be reasonable for the permit applicant to be responsible for the reduced life of the bridge. The work completed in this project aims to help agencies in evaluating the long term impact of the vehicles on bridges and in assigning the resulting cost to the permit applicants as an extension of 1st phase of the project. Long term behavior of concrete decks and steel girder bridges was investigated and a means to assign cost to the overloads was developed. Miner's damage accumulation rule and life cycle cost analysis of bridges were used to develop the means. Examples of assigning cost per crossing bridges to overloads were provided for practical application of the developed means. First set of example was performed for two pilot concrete decks and second set of example was performed for two pilot steel girder bridges.</td></tr>
<tr><td colspan="2">17. Key Words

Overload vehicle, bridge, long term behavior, fatigue, bridge life cycle cost, concrete deck, steel girder bridge, cost assignment</td><td colspan="4">18. Distribution Statement
No restrictions. This report is available through the Transportation Research Information Services of the National Transportation Library.</td></tr>
<tr><td colspan="2">19. Security Classification (of this report)
Unclassified</td><td colspan="2">20. Security Classification (of this page)
Unclassified</td><td>21. No. Of Pages
72</td><td>22. Price
-0-</td></tr>
</table>

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BRIDGE ANALYSIS AND EVALUATION OF EFFECTS UNDER OVERLOAD VEHICLES (PHASE 2)

EXECUTIVE SUMMARY

An overload vehicle travelling across a bridge, even if it is a single crossing, may affect not only the short term behavior of the bridge but also the long term performance and life cycle cost of the bridge. Generally, special permits are issued to overload vehicles without considering their cumulative effect on bridge components, but only considering the bridge strength capacity. The cumulative damage to the bridge may reduce the life of the bridge or induce unexpected fatigue failure of the bridge. Therefore, it is reasonable to examine the long term performance of bridges when issuing permits in addition to the short term effect during the crossing.

Long term performance of concrete decks and steel girder bridges subjected to overloads was investigated in comparison to the effects when subjected to an AASHTO standard vehicle which is used to design bridges. Overloads that can safely cross a bridge in the short term may cause long term problems such as fatigue failure or reduction of bridge service life that are not immediately evident.

It may be reasonable for the permit applicant to be responsible for the cost of repair, additional maintenance or reduced life of the bridges caused by passage of an overload vehicle. The user cost should be related to the total invested cost to maintain the service life of the bridge. The concept of life cycle cost is required to assess the assigned cost to the overload vehicles. A procedure to calculate bridge life cycle cost is outlined in this report for concrete bridge decks and steel girder bridges. This procedure could provide part of the estimate of fair cost assessment for use of bridges by overload vehicles.

Damage of the bridge components due to an overload is calculated using stress and cycles (S-N) relations and Miner's damage accumulation rule. Assigned cost is calculated using the life cycle cost of the bridge component and the damage accumulated to the bridge component.

The design concept used with prestressed concrete girder bridges is that cracking in the girders should be prohibited under short term loading as well as long term loading. Permits for overloads that would induce cracks in the girders should not be issued as a result of the process of

checking allowable tensile stress in the girders. Effects of damage to girders in bridges with prestressed concrete girders were, therefore, excluded in this research.

Examples of assigning cost per crossing for overload vehicles are provided for practical application of the proposed methods. A first example is provided for two concrete decks and a second example looks at two steel girder bridges.

CONTENTS

1. INTRODUCTION

1.1 Background

The use of special purpose highway vehicles, over the legal limit in size and in weight, is increasing as industry grows and large items must be shipped over highways. Those vehicles carry pressure vessels and transformers used in power plants, huge boilers, military hardware, wind turbine components and beams and barges that are becoming wider, longer and heavier. The vehicles frequently weigh 5 to 6 times the normal legal truck weight. Transportation agencies are asked to provide special permits for the vehicles along a specified pathway. Because of the unusual configuration of the vehicles it is difficult for those agencies to evaluate the effect of the vehicles on highway bridges. A simplified analysis method to predict the short term effects of overload vehicles on a bridge system – including deck, girders, and diaphragms has been performed during the 1st phase of this project (Bae 2009). Damage due to overloads on long term behavior, including fatigue problems, and cost assignment to the vehicles per bridge crossing are investigated in this report.

Overload vehicle travelling across a bridge, even if it is a single crossing, may affect not only the short term behavior of the bridge but also the long term performance and life cycle cost of the bridge. Generally, special permits are issued to overload vehicles without considering their cumulative effect on bridge components, but considering only the strength capacity of bridges (Mohammadi and Polepeddi 2000). Fatigue problems could result if the bridge is subjected to unexpected overloads in the future. The consequences of these occasional overloads with permits may be more critical than previously assumed when designing the bridge. They add to the cumulative damage in the bridge. Therefore, the long term behavior of bridges should be considered when issuing permits, in addition to the short term behavior.

There may be minor cracking or deterioration in the components of the bridge which are not critical in the short term period but can result in special maintenance, rehabilitation or reduced life span in the long term. Therefore, an evaluation of the long term effects caused by initial damage is important. The effects could be alleviated by the repair or maintenance of the bridge. The cost for repair, maintenance and reduction of life span of the bridge needs to be considered during the process of issuing the permit. It may be reasonable for the permit applicant to be responsible for the cost of repair, additional maintenance or reduced life of the bridges. The cost should be relevant to the total invested cost to maintain the service life of the bridge and the concept of the life cycle cost of bridge is required to assess the assigned cost to the overloads. The life cycle cost of a bridge is defined as sum of initial cost, expected life cycle maintenance cost and expected life cycle rehabilitation costs including repair/replacement costs, loss of contents or fatality and injury losses, road user costs, and indirect socioeconomic losses.

Research related to long term behavior of bridges subjected to overloads has been studied by several researchers. Brunea and Dicleli (1994) and Dicleli and Bruneau (1995) studied cumulative impacts of heavy permit trucks on steel bridges and developed a fatigue-based method to assess the reduction in service life due to the trucks. Miner's well known cumulative fatigue damage due to cycles of a variable amplitude loading was applied to develop the method. Mohammadi and Polepeddi (2000) investigated fatigue damage of five bridges from overloads in the range of 80 ~ 120 kips and found that the fatigue damage from the overloads can reduce about 3.5% of service life of the bridges. They developed a method for rating bridges under application of overloads using Miner's cumulative fatigue damage rule. Li et al. (2001) studied effects of load sequence and interaction, and overloading effect on the fatigue damage of bridges on the basis of a non-linear fatigue damage model. The model is derived from the theory of continuum damage mechanics for high-cycle fatigue and residual life can also be calculated by using the model. Sadeghi and Fathali (2007) performed deterioration analysis of concrete bridge decks under overloads from fatigue point of view. A method for determining damage effects of overloads on concrete decks considering fatigue effects was outlined. The relationship of the passing overloads and the number of allowable load cycles can be determined using the method. Goodman diagram based on working stress design method was used in the method.

Aforementioned studies provided methods to evaluate bridge damages induced by overloads and to predict reduction of the service life of bridges. The study to develop rational method to assign cost to the overloads per crossing bridges based on the resulting fatigue damage has not been performed yet. The work performed in this project aims to help agencies in evaluating the long term impact of the overload vehicles on bridges and in assigning the resulting cost to the permit applicants as an extension of 1st phase of the project. Life cycle cost analysis of bridges was used to determine assigned cost to the overloads.

1.2 Research Objectives / Task List

The objectives for the project include evaluation of possible long term effects of overload vehicles on bridges, assessment of life cycle cost, and establishment of a means of assigning cost per overload vehicle based on damage and resulting reduced service life. The approach to studying the effects and task list are outlined below.

1) Evaluation of long term effects of overload vehicles:

The effects of overload vehicles on bridges are not restricted to the short term behavior of the bridges. There may be a reduction of the fatigue life or initialization of cracks because of the excessive stress in the components of the bridges, particularly at connections or changes of cross section, caused by the heavier loads. The frequent occurrences of stress over the material fatigue endurance limit may significantly affect the fatigue life of the components. These could subsequently worsen due to repeated normal vehicle loads or freeze and thaw cycle inducing deterioration. The evaluation of the long term effects of the overload vehicles was performed in this report.

2) Assessment of bridge life cycle cost

The time between a bridge's construction and its replacement or removal from service is its service life. The sequence of actions and events and their outcomes–e.g., construction, usage, aging, damage, repair, renewal–that lead to the end of the service life and the condition of the bridge during its life compose the life cycle. The bridge life cycle cost is defined as the total cost of the bridge during its life cycle. Bridges are unique structures in transportation systems, and they require frequent and substantial maintenance and rehabilitation. A procedure to find bridge life cycle cost was outlined and it was used in development of a means to assign cost per crossing overload vehicles.

3) Development of a means to assign cost per overload

Overload vehicles may cause damage or cracks in bridge components. This might be permitted when the damage or cracks are repairable without a loss in structural load carrying capacity. It may be reasonable, however, for the permit applicant to be responsible for the cost of repair, additional maintenance or reduced life of the bridge. Assigning a standard cost may be difficult without structural evaluation. A method of assigning the cost as a function of the impact of the gross weight and configuration of the vehicles on the bridge was proposed.

The means of assigning the cost was developed for concrete decks and steel girder bridges. The design concept of prestressed concrete girders does not allow cracks under service loads. Permit for the overloads inducing cracks in the girders may not be issued in the process of checking allowable tensile stress of the girders. The bridges with prestressed concrete girders were, therefore, excluded in this research.

4) Cost assignment examples

Examples of assigning cost per crossing bridges to overloads were provided for practical application of the developed means to assign cost per overload. First set of examples was performed for two pilot concrete decks and second set of examples was performed for two pilot steel girder bridges.

2. LONG TERM BEHAVIOR OF BRIDGES UNDER OVERLOADS

2.1 Load Combinations Used to Investigate Long Term Behavior

It is required to select appropriate load combinations to investigate the long term behavior of the bridge components under overloads. There are four major load combinations prescribed in the AASHTO LRFD bridge design manual (2009) which can be used to investigate and compare short term and long term behavior under overloads or AASHTO standard vehicle. The load combinations are listed as follows:

1) STRENGTH I - Basic load combination relating to the normal vehicular use of the bridge without wind.
2) STRENGTH II - Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
3) SERVICE I - Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.
4) FATIGUE - Fatigue and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck having the axle spacing specified in Article 3.6.1.4.1. of AASHTO LRFD bridge design manual (2009).

The strength load combinations are used to evaluate whether the bridge is safe under ultimate loading condition. Generally permits for overload vehicles are issued when the bridge components are safe under the Strength load combination without considering the long term effect. This may result in long term problems such as fatigue failure or reduction of bridge service life. The Strength I load combination is applicable to AASHTO standard truck and the Strength II load combination is applicable to overloads. The service load combination can be used to check whether the bridge exhibits excessive deflection or cracking. It is applicable to both AASHTO standard truck and overloads. The fatigue load combination can be used to check if the bridge components have any possibility of fatigue failure during the service life of the structure.

The strength and service load combinations are used to design or to evaluate bridge components under short term loading while the fatigue load combination is used to evaluate bridge components under long term loading. Effects of overload vehicles using the load combinations were compared with those of AASHTO standard truck to investigate if overload vehicles have potential to damage bridges more than AASHTO standard truck which is used to design bridges focusing on concrete decks.

2.2 Comparison of Effects of Overload with AASHTO Standard Truck on Concrete Deck

Effects of two types of overload vehicles (single lane and dual lane overload vehicle) vs. AASHTO standard vehicle, HL 93 on the concrete deck were analyzed by finding maximum moment per unit width in the pilot concrete decks subjected to each vehicle. Five concrete decks with various spacing of girders (5 ft, 7 ft, 9 ft, 11 ft and 13 ft) were selected for the analysis. The decks are assumed to be supported by five girders. Aforementioned load combinations prescribed in the AASHTO LRFD bridge design manual, i.e. service load, fatigue load and strength I and II load, were used. Cracking of the concrete deck and yielding of the steel reinforcement are considered as permanent damage and can be investigated using the service load. Long term behavior including reduction of the service life of the deck can be investigated using the fatigue load. Short term instant failure of the deck can be investigated by the Strength I or II load.

The maximum moments per unit width were calculated by dividing moment in the deck subjected to single axle by the smaller of the AASHTO effective strip width and longitudinal axle spacing. The longitudinal axle spacing of the AASHTO standard vehicle is generally wider than the AASHTO effective strip width while the longitudinal axle spacing of the overload vehicles may be less than the AASHTO effective strip width.

The configurations of the axle load for each type of the vehicle are listed in Table 1. The selected overload vehicles are the most severe cases which got permits from WisDOT in the last 10 years. Effects of dynamic allowance were not considered in the load combination for the overload vehicles since the vehicles move slow enough (less than 5 mph) on the bridge to ignore the effect. Load factors, dynamic allowance and multi-presence factor applied to the analysis are listed in Table 2.

Table 1. Configurations of the axle loads used to investigate effects on concrete decks.

Type of Vehicle	Number of sets of wheels per axle	Weight per sets of wheels (non-factored)	Lateral wheel spacing	Minimum longitudinal axle spacing	Number of lanes loaded
AASHTO standard truck, HL 93	2	16.000 k	6 ft	14 ft	1~3
Single lane overload	2	18.800 k	8 ft	3.5 ft	1
Dual lane overload	4	13.125 k	4 ft + 6 ft + 4 ft	3.5 ft	1

Table 2. Load factors applied to the analysis.

		Load factor	Dynamic load allowance	Multi-presence factor
Service load combination	AASHTO standard truck	1.00	33%	Applied
	Single lane overload	1.00	-	Not applied
	Dual lane overload	1.00	-	Not applied
Fatigue load combination	AASHTO standard truck	0.75	15%	Not Applicable
	Single lane overload	1.00	-	Not applied
	Dual lane overload	1.00	-	Not applied
Strength load combination	AASHTO standard truck	1.75	33%	Applied
	Single lane overload	1.35	-	Not applied
	Dual lane overload	1.35	-	Not applied

1) Service load combination

The results from the analyses using the service load combination are listed in Table 3 and shown in Figure 1. The service moments under the overload vehicles in the deck are greater than those under the AASHTO standard truck in most of the cases indicating that the overload vehicles affect more than the AASHTO standard truck on cracking of the deck or yielding of the steel reinforcements. It was also found that effects of the severe dual lane overload on decks using service load combination is less than those of the severe single lane overload.

Table 3. Moment per unit width using service load combination.

Spacing of girders (ft)	Moment per unit width (kip-ft / ft)		
	AASHTO Standard truck	Single lane overload	Dual lane overload
5	5.41	5.64	4.04
7	6.14	7.77	5.82
9	8.32	9.96	8.27
11	9.45	12.15	11.05
13	10.51	14.34	14.05

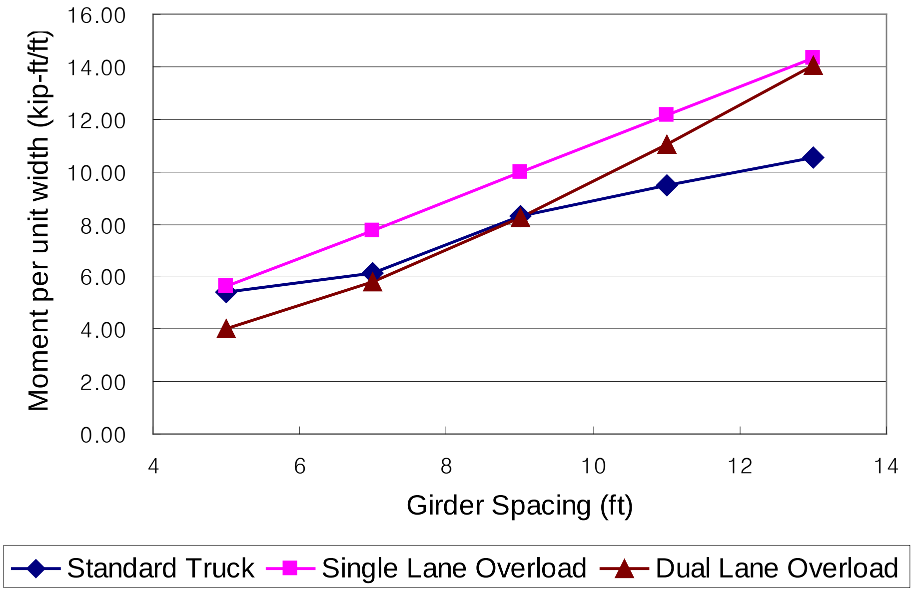


Figure 1. Moment per unit width in the deck vs. girder spacing using the service load combination.

The analysis results indicated that the severe overloads have more possibility to induce instant crack and/or yielding of steel reinforcement compared to AASHTO standard truck. The comparison of the cracking moment and the yielding moment of typical concrete deck with service moment under the AASHTO standard truck and the severe overloads were performed to investigate the possibility. An analysis of the typical concrete deck with 9 inch depth and 7 foot girder spacing reinforced by #4 transversal (perpendicular to the girder direction) steel reinforcements at 6 foot spacing designed according to AASHTO specification was performed. The 28 day compressive strength of the deck was assumed to be 4000 psi and yielding stress of the steel reinforcement was assumed to be 60 ksi. The analysis results are listed in Table 4 and shown in Figure 2. The results indicate that the severe overload vehicles are not like to induce the yielding of the reinforcement while they may cause the cracking of the deck. It is recommended to compare service moment in the deck induced by overload with cracking moment of the deck while issuing permits to prevent cracking problems in the concrete deck.

Table 4. Moments per unit deck width using service load combination in comparison with cracking and yielding moments.

Type of Moment	Moment per unit width (kip-ft/ft)
Cracking Moment	6.04
AASHTO Standard Truck	6.14
Single lane overload	7.77
Dual lane overload	5.82
Yielding Moment	13.47

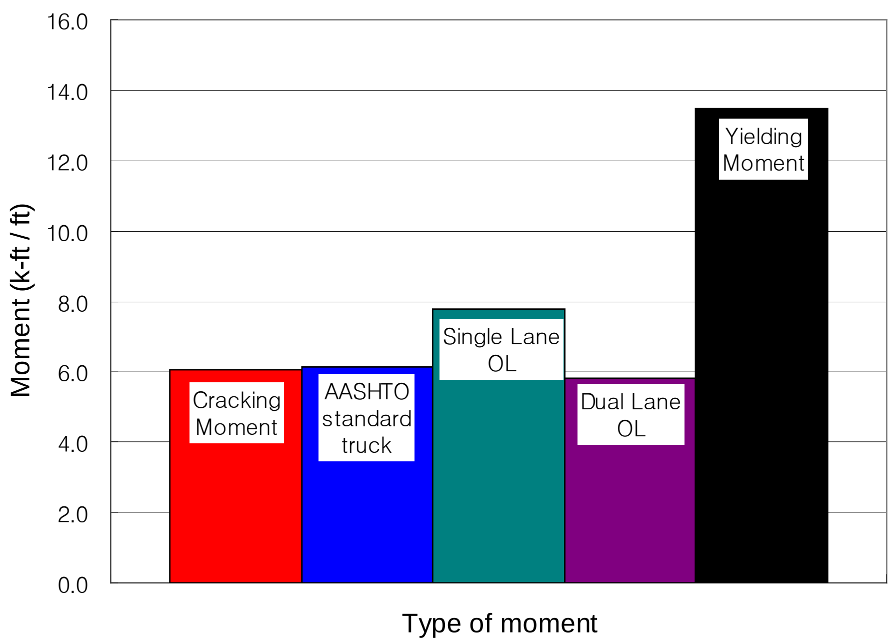


Figure 2. Comparison of the moments per unit deck width under service load and critical moments.

2) Fatigue load combination

The results from the analyses using the fatigue load combination are listed in Table 5 and shown in Figure 3. The fatigue moments under the severe overload vehicles in the deck are greater than those under the AASHTO standard truck indicating that the overload vehicles affect more than the AASHTO standard truck on the fatigue life of the bridge. The results indicate that the passage of the severe overloads is likely to reduce service life of the bridge deck.

Table 5. Moment per unit width using fatigue load combination.

Spacing of girders (ft)	Moment per unit width (kip-ft / ft)		
	AASHTO Standard truck	Single lane overload	Dual lane overload
5	3.02	5.64	4.04
7	3.72	7.77	5.82
9	5.40	9.96	8.27
11	6.13	12.15	11.05
13	6.82	14.34	14.05

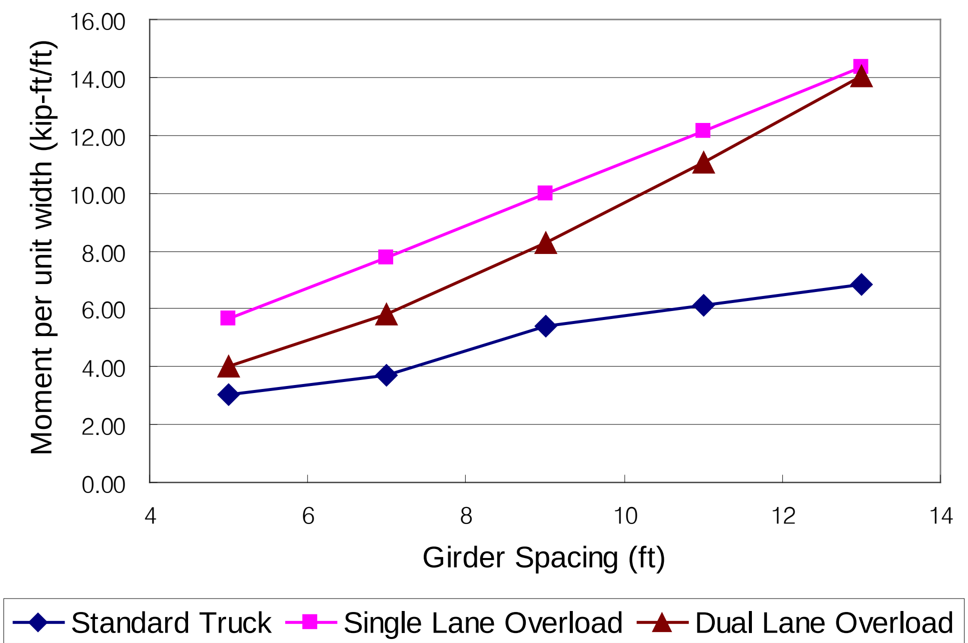


Figure 3. Moment per unit width in the deck vs. girder spacing using fatigue load combination.

3) Strength load combination

The results from the analyses using the strength I load combination are listed in Table 6 and shown in Figure 4. The strength moments under the severe overload vehicles in the deck are less than those under the AASHTO standard truck indicating that the chances of failure due to the overload vehicles are minimal. The result shows that permits could be issued to the severe overloads when the analysis of the bridge was done only checking ultimate stress using strength load combination. However, the passage of the severe overload possibly cause cracking problem and fatigue related issues. Therefore service load combination and fatigue load combination should be checked while issuing permits.

Table 6. Moment per unit width using strength load combination.

Spacing of girders (ft)	Moment per unit width (kip-ft / ft)		
	AASHTO Standard truck	Single lane overload	Dual lane overload
5	9.47	7.61	5.45
7	10.75	10.49	7.85
9	14.56	13.44	11.16
11	16.54	16.40	14.91
13	18.40	19.36	18.97

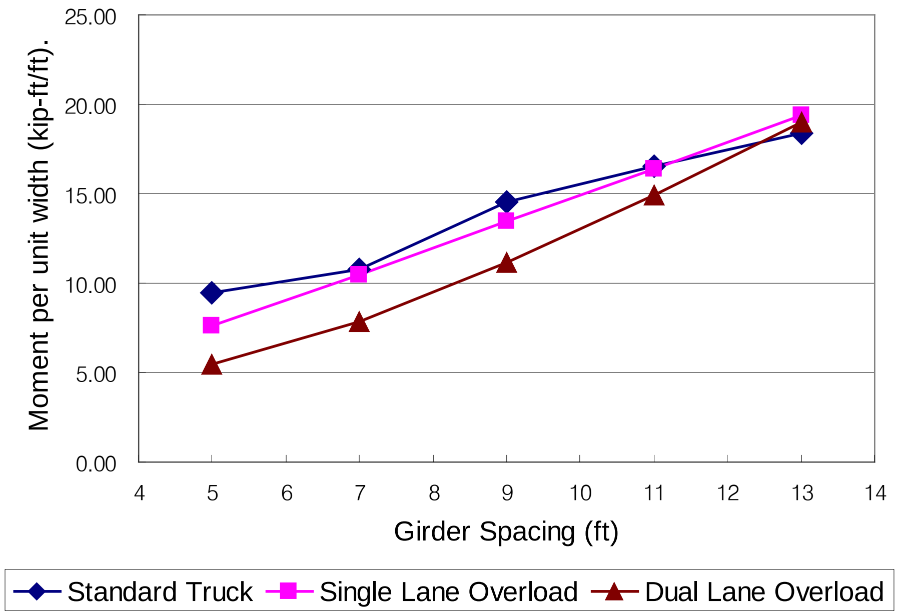
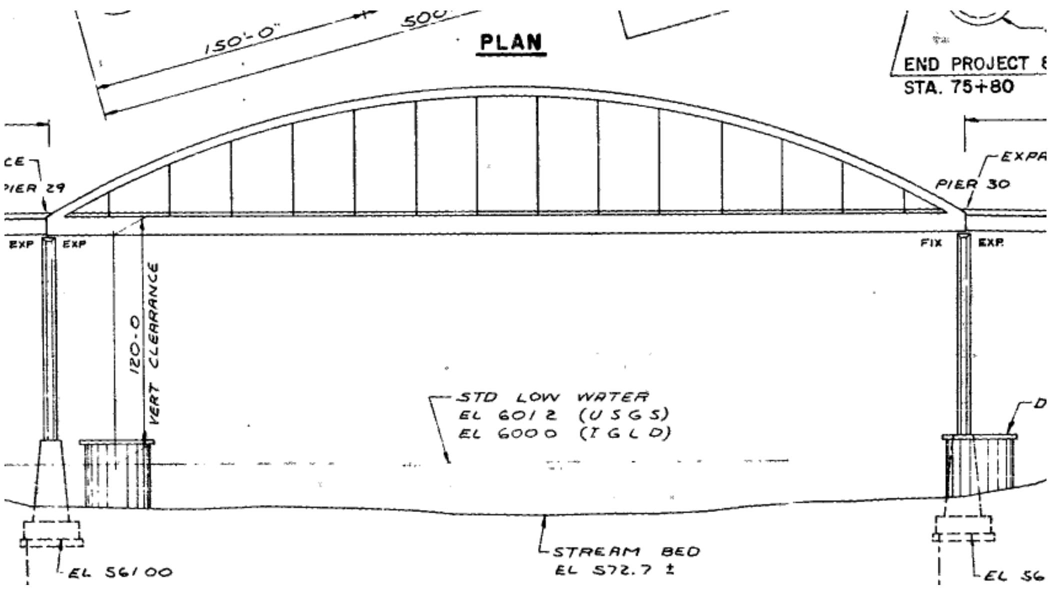


Figure 4. Moment per unit width in the deck vs. girder spacing using strength load combination.

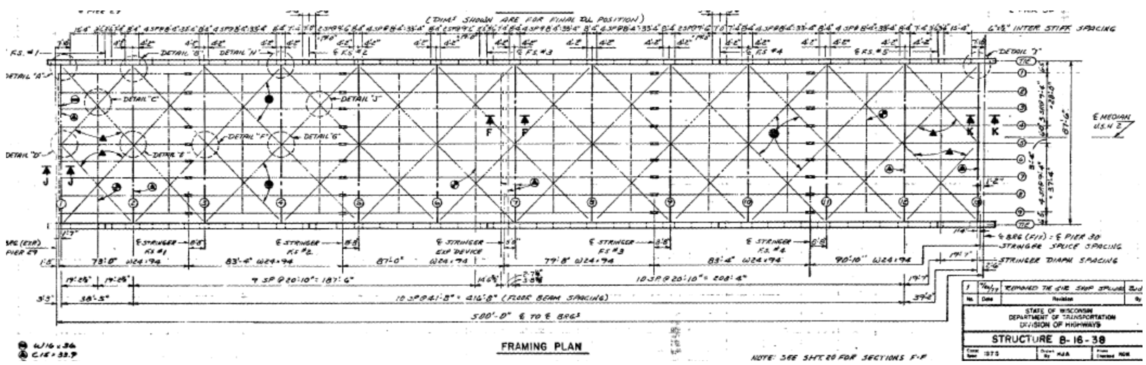
2.3 Comparison of Effects of Overload with AASHTO Standard Truck on Steel Girder Complex Bridge

Effects of two types of overload vehicles (single lane and dual lane overload vehicle) vs. AASHTO standard vehicle on a steel girder complex bridge were analyzed by loading each vehicle with strength and fatigue load combination on the Bong Bridge built in 1984 in Wisconsin (Figure 5). The purpose of the analysis is to investigate short term and long term behavior of the steel girder bridge. The bridge was analyzed for the 1st phase of the project as an example of analysis of complex bridge under overloads considering only strength load combination. The structural type of the bridge is a tied steel arch bridge with non-composite concrete deck. The total span of the bridge is 500 ft. and there are two main steel girders and two steel arch members. The girders and arches are rigidly connected to each other at the joint where they meet. At other points the girders are tied by cables to the arches. The width of the deck is 82 ft and four vehicle lanes are provided. There are nine stringers as longitudinal structural components in addition to the two main girders and two arches in the superstructure. There are thirteen transverse floor beams in the superstructure. The plans for the bridge are shown in Figure 6.

Figure 5. Bong Bridge (Wisconsin, Tied arch bridge, Span = 500 ft).



(a) Elevation



(b) Framing plan

Figure 6. Plans for Bong Bridge.

Three types of vehicular loads, i.e. the AASHTO LRFD standard truck and two types of overload vehicles were considered for the analysis. The configurations of the vehicles are shown in Table 7.

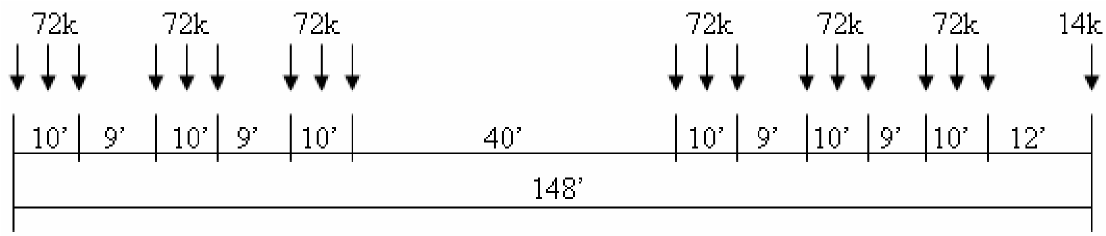
Table 7. Vehicle loads for Bong Bridge.

Type of the vehicle	Features
AASHTO LRFD standard truck *	- Negative moment truck train was included - 1 ~ 3 lane loading
Single lane overload*	- Gross Weight = 446 kips
Dual lane overload*	- Gross Weight = 670 kips - Transverse wheel spacing: 4’ + 4’ + 4’

* All the possible transverse and longitudinal live load locations were considered using the moving load option in SAP2000.

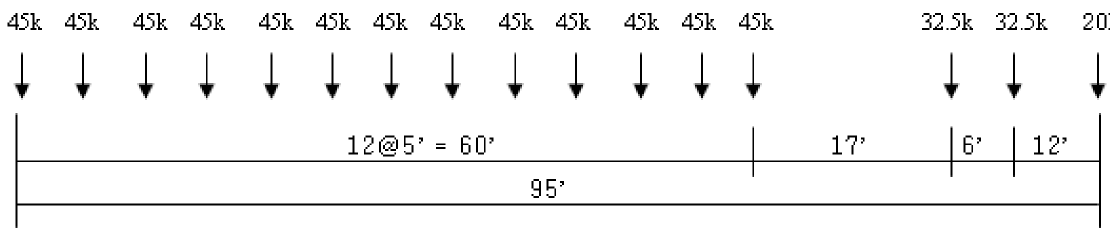
Selected overload vehicles for the analysis were the single lane overload vehicle and the dual lane overload vehicle shown in Figure 7. They are the heaviest vehicles in gross weight seen in the last ten years in Wisconsin. The transverse wheel spacing of the single lane overload vehicle was 8 ft. The exterior transverse wheel spacing of the dual lane overload vehicle was 4 ft and the interior transverse wheel spacing of the dual lane overload vehicle was 4 ft. Load combinations, load factor, dynamic allowance and multi-presence factors used in the analysis are identical to the configuration shown in Table 2. The vehicles are modeled using the moving load option in SAP2000 and all the possible transverse and longitudinal live load locations were considered.

Modeling of the bridge is shown in Figure 8. The frame element was used to model the main girders, the arches, the transverse arch bracing, the stringers and the floor beams. A truss element was used to model the bracings for the floor beams and the diaphragms for the stringers. A cable element was used to model the cables. The shell element was used to model the concrete deck. A special link defined to transfer only vertical force was used to model the connection of the deck and main girder to model a non-composite connection.



(a) Selected single lane overload vehicle for the analysis of Bong Bridge

(72k loads are sum of 3 axles, total gross weight = 446 kips)



(b) Selected single lane overload vehicle for the analysis of Bong Bridge

(Gross weight = 670 kips)

Figure 7. Selected overload vehicles for the analysis of Bong Bridge.

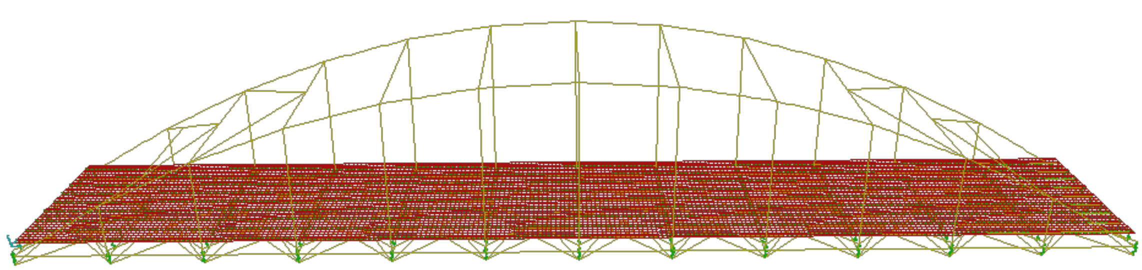


Figure 8. Three dimensional finite element modeling of Bong Bridge.

Analysis results are shown for one of the two steel main girders, namely the tension ties for the arch in Figures 9 and 10 using a strength load combination and fatigue load combination (with the AASHTO HL93 truck, not the fatigue vehicle). The results show moment envelopes under each type of live load. No other loads, except live load, were considered in the analysis. The moment in the tension tie for the arch shows some change at the location where the girder is supported by the cables as shown in Figures 9 and 10. The location of the vertical grids in the figures were selected as the same location as the location of the cables.

The live load moments in the main arch tie girder subjected to the single lane overload vehicle using strength load combination (Figure 9) were less than those subjected to the AASHTO HL93 truck load, while the live load moments of the main girder subjected to the dual lane overload vehicle were comparable to those subjected to the AASHTO HL93. These results in Figure 9 indicate that the main girders are stressed less when the single lane overload vehicle is present than a case where AASHTO HL 93 truck is present. When the dual lane overload vehicle passes, they are stressed comparably to a case where AASHTO HL93 truck passes.

The moments in the main girder using a fatigue load combination (Figure 10) show different results compared to the moments using the strength load combination (Figure 9). The moments in the main girder subjected to the single lane overload vehicle, using a fatigue load combination, were comparable to those subjected to the AASHTO truck. The main girder moments from the dual lane overload vehicle were higher than those subjected to the AASHTO truck. Though the dual lane vehicle creates higher moments, the fatigue effect may not be critical with the low number of cycles from overload vehicles. In some cases the fatigue limit state might still need to be considered in issuing permits because the higher stress condition at fewer cycles and still contribute to fatigue failure more than the lower AASHTO truck induced stresses at higher cycles.

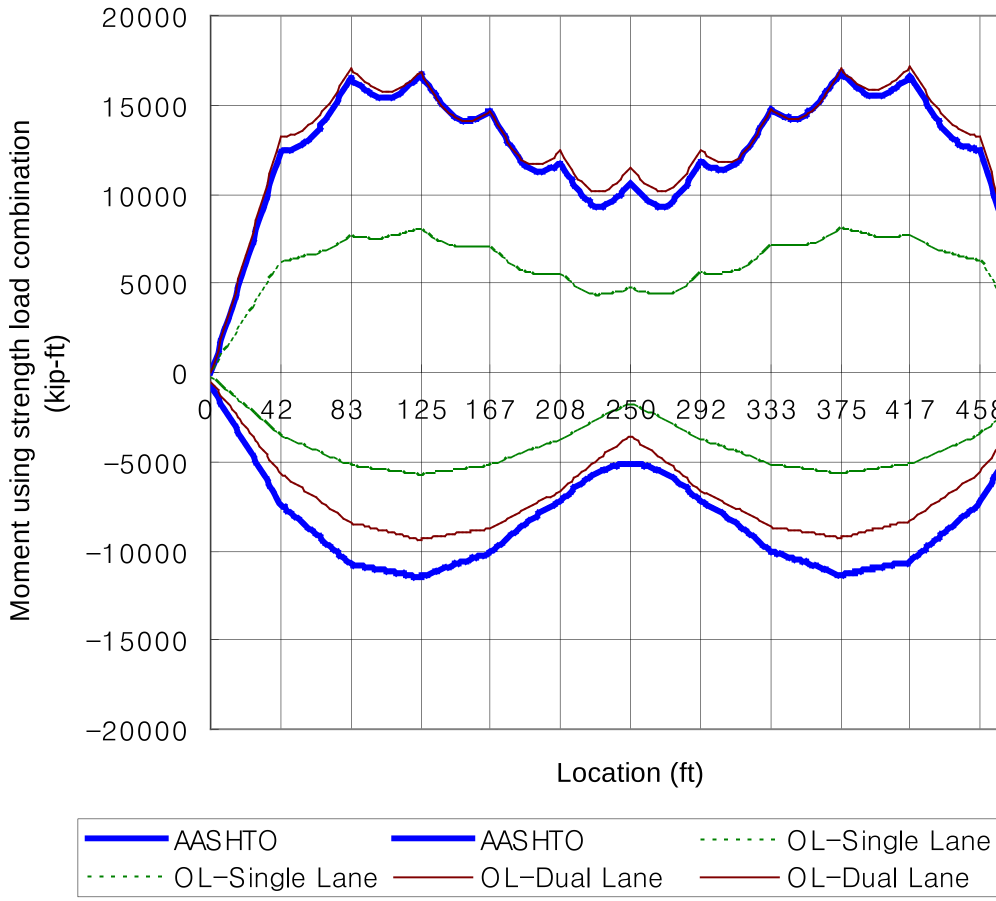


Figure 9. Moment envelope for Bong bridge main girder using strength load combination.

(AASHTO = HL-93 load, OL = Overload)

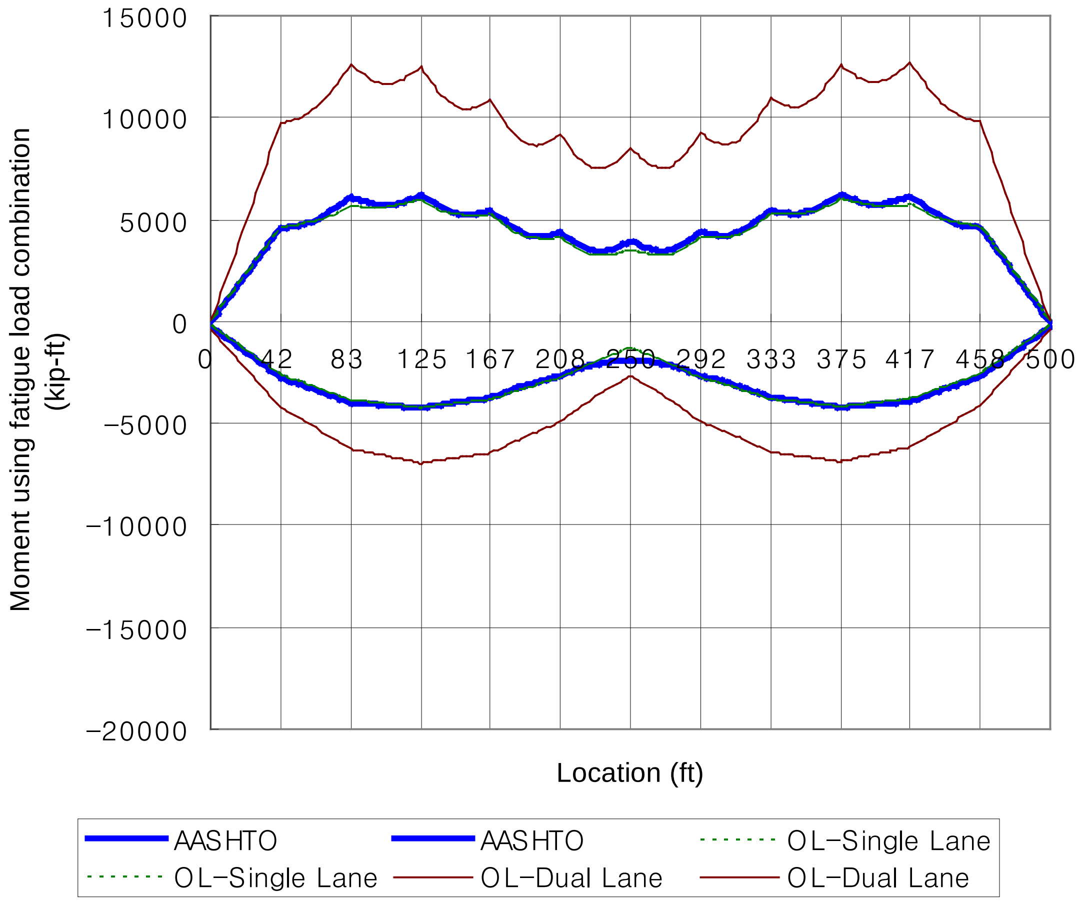


Figure 10. Moment envelope for Bong bridge main girder using fatigue load combination.
(AASHTO = HL-93 load, OL = Overload)

3. BRIDGE LIFE CYCLE COST

There may be damage, including minor cracking or deterioration in the components of a bridge, which are not critical in the short term period after passage of overloads but they can reduce service life span of the bridge in the long term. Therefore, an evaluation of the long term effects of initial damage is important. The cost for the reduction of service life of bridges may need to be considered during the process of issuing the permit. It may be reasonable for the permit applicant to be responsible for the cost of reduced life of the bridge. The cost should be relevant to the total invested cost to build the bridge and to maintain the service life of the bridge. The concept of the life cycle cost of a bridge is, therefore, required to assess the assigned cost to the overloads. The bridge life cycle cost was studied as a step to develop a means to assign cost to overloads.

3.1 Concept and Background

The time between a bridge's construction and its replacement or removal from service is its service life. The sequence of actions and events and their outcomes–e.g., construction, usage, aging, damage, repair – that lead to the end of the service life and the condition of the bridge during its life compose the life cycle. The bridge life cycle cost is defined as the total cost of the bridge during its life cycle. The concept of the bridge life cycle cost has been used to choose the most cost effective alternative for the construction and maintenance of bridges and communicate the value of those choices to public (Al-Wazeer et al. 2005). A tool to calculate the bridge life cycle cost was developed by Hawk (2003). The method was developed as a part of NCHRP (National Cooperative Highway Research Program) project 12-43 to serve as a tool that can be applied to the decision-making process for the repair or selection of cost-effective alternatives for the preservation of bridge assets for short-term and long-term planning horizons.

Many researchers have been using the life-cycle concept in various applications such as design of steel bridges (Lee et al. 2006), evaluation of existing prestressed concrete bridges (Liang et al. 2007), examination of engineered cementitious composite link slab (Kendall et al 2008), and development of a service life prediction model (Cheung et al. 2008)

Bridges are unique structures in transportation systems, and they require frequent and substantial maintenance, rehabilitation, and replacement. Consequently, maintenance and rehabilitation costs are a significant part of the total costs in bridge life cycle cost. A bridge life-cycle cost is a sum of the following:

- Design cost
- Construction cost
- Maintenance cost
- Rehabilitation cost
- User cost
- Salvage value

The initial portion of the bridge life cycle cost includes the design cost and the construction cost. Examples showing comparison of the initial cost to the life cycle cost of bridges are shown in Figure 11. The major portion of the difference between the initial cost and the life cycle cost is the maintenance cost and the rehabilitation cost. The sum of the maintenance and rehabilitation costs was approximately 5 ~ 20 % of the life cycle cost (Lee et al. 2006).

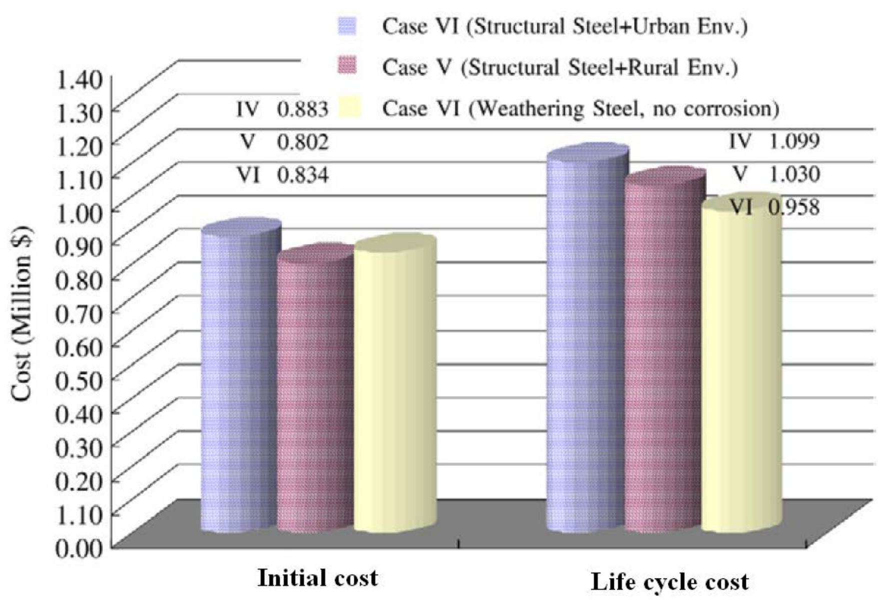


Figure 11. Comparison of the initial cost to the life cycle cost of bridges. (Lee et al. 2006)

3.2 Cash Flow, Discount Rate and Net Present Value

Concepts of cash flow, discount rate and net present value are required to calculate bridge life cycle cost and they are briefly described here. General concepts related to finance are described first and then the concepts related to the bridge life cycle cost are described.

1) Cash Flow

Cash Flow is the movement of cash into or out of a business, project, or financial product. It is usually measured during a specified, finite period of time. Measurement of cash flow can be used for calculating other parameters that give information on the companies' value and situation. Cash flow can be used for calculating parameters such as:

- to determine a project's rate of return or value. The time of cash flows into and out of projects are used as inputs in financial models such as internal rate of return, and net present value.
- cash flow can be used to evaluate the 'quality' of income generated by accrual accounting. When Net Income is composed of large non-cash items it is considered low quality.
- to evaluate the risks within a financial product, e.g. matching cash requirements, evaluating default risk, re-investment requirements, etc.

Cash flow is a generic term used differently depending on the context. It may be defined by users for their own purposes. It can refer to actual past flows, or to projected future flows. It can refer to the total of all the flows involved or to only a subset of those flows. Subset terms include 'net cash flow', operating cash flow and free cash flow.

The cash flow used to calculate the bridge life cycle cost is related only to the design cost, construction cost, maintenance cost and rehabilitation cost which are the investments during a specified, finite period of time since generally there is no income from the bridge.

2) Discount rate

The discount rate is defined as the interest rate charged to commercial banks and other depository institutions on loans they receive from their regional Federal Reserve Bank's lending facility--the discount window. The discount rate can mean

- an interest rate a central bank charges depository institutions that borrow reserves from it, for example for the use of the Federal Reserve's discount window.

- the same as interest rate; the term "discount" does not refer to the common meaning of the word, but to the meaning in computations of present value, e.g. net present value or discounted cash flow
- the annual effective discount rate, which is the annual interest divided by the capital including that interest; this rate is lower than the interest rate; it corresponds to using the value after a year as the nominal value, and seeing the initial value as the nominal value minus a discount; it is used for Treasury Bills and similar financial instruments

The discount rate is a critical factor to estimate life cycle cost of the bridge in net present value (NPV) since the value of the bridge changes with time. Possible values for the discount rate were found from references as shown in Table 8. The value proposed by the Office of Management and Budget in USA is judged to be the most reasonable value and it will be used in this research.

Table 8. Discount rates from references.

References	Discount rate	Analysis Year
Hawk, H., (2003) "Bridge Life-cycle Cost Analysis", National Cooperative Highway Research Program, Transportation Research Board, Washington, DC, NCHRP Report 483.	5.8 %	2002
Al-Wazeer, A., Harris, B., and Nutakor, C., (2005), "Applying Life Cycle Cost Analysis to Bridges", Public Roads, FHWA, Vol. 69, No. 3.	4.2 %	2005
Lee, K. M., Cho, H. N., and Cha, C. J., (2006), "Life-cycle Cost-effective Optimum Design of Steel Bridges Considering Environmental Stressors", Engineering Structures, Vol. 28, No. 9, pp. 1252-1265.	4.0 %	2006
2008 Kendall, A., Keoleian, G. A., and Helfand, G. E., (2008) "Intergated Life-cycle Assessment and Life-cycle Cost Analysis Model for Concrete Bridge Deck Applications", Journal of Infrastructure Systems, ASCE, Vol. 14, No. 3, pp. 214-222.	4.0 %	2005
Office of Management and Budget, USA	5.1 % 5.5 % 5.2 % 5.2 % 5.1 % 4.9 % 4.5 % **4.5 %**	2003 2004 2005 2006 2007 2008 2009 2010

3) Net present value

The net present value (NPV) of a time series of cash flows, both incoming and outgoing, is defined as the sum of the present values (PVs) of the individual cash flows. In the case when all future cash flows are incoming (such as coupons and principal of a bond) and the only outflow of cash is the purchase price, the NPV is simply the PV of future cash flows minus the purchase price (which is its own PV). NPV is a central tool in discounted cash flow analysis, and is a standard method for using the time value of money to appraise long-term projects. Used for capital budgeting, and widely throughout economics, finance, and accounting, it measures the excess or shortfall of cash flows, in present value terms, once financing charges are met.

The NPV of a sequence of cash flows takes as input the cash flows and a discount rate or discount curve and outputting a price; the converse process in discounted cash flow analysis - taking a sequence of cash flows and a price as input and inferring as output a discount rate (the discount rate which would yield the given price as NPV) - is called the yield, and is more widely used in bond trading.

Each cash inflow/outflow is discounted back to its present value (PV). Then they are summed. Therefore NPV is the sum of all terms,

$$\frac{R_t}{(1+i)^t} \tag{1}$$

where, *t* - the time of the cash flow

i - the discount rate (the rate of return that could be earned on an investment in the financial markets with similar risk.)

R_t - the net cash flow (the amount of cash, inflow minus outflow) at time t. For educational purposes, R_0 is commonly placed to the left of the sum to emphasize its role as (minus) the investment.

The result of this formula if multiplied with the Annual Net cash in-flows and reduced by Initial Cash outlay will be the present value but in case where the cash flows are not equal in amount then the previous formula will be used to determine the present value of each cash flow separately. Any cash flow within 12 months will not be discounted for NPV purpose.

The NPV at the time of a certain overload crossing a bridge is the bridge life cycle cost used to assign cost to the overload. The assigned cost to the overload would be a portion of the bridge life cycle cost which is evaluated by the degree of damage the overload induces.

3.3 Calculation of Bridge Life Cycle Cost

The bridge life cycle cost is usually calculated for alternatives of the bridge project plan at the time of a decision making process prior to building the bridge and is used to evaluate the alternatives and select the alternative with the best economy. Assume that there is an alternative to build a bridge with initial cost of $ 5,000,000 and maintenance cost of $800,000 per 10 years. The life cycle cost of the bridge at the base year (2010) can be calculated as shown in Table 9. A discount rate of 4.5% in Table 8 recommended by the Office of Management and Budget, was used. Each present value in the table was calculated by Eq. (1). The net present value of the bridge life cycle cost for the alternative is $ 6,286,567 which is the sum of the present values.

Table 9. Calculation of bridge life cycle cost prior to build the bridge.

Year	2010	2020	2030	2040	2050	2060	Sum
Cash flow ($)	5,000,000	800,000	800,000	800,000	800,000	800,000	9,000,000
t (years)	0	10	20	30	40	50	
$1/(1+i)^t$	1.000	0.644	0.415	0.267	0.172	0.111	
PV ($)	5,000,000	515,142	331,714	213,600	137,543	88,568	**6,286,567 (NPV)**

where, *t* is difference of the time from the time when the NPV is calculated, *i* is discount rate, PV is present value and NPV is net present value.

The calculation of the bridge life cycle cost, for assigning a cost from overloads crossing a bridge, is different from the calculation of the bridge life cycle cost for choosing alternatives since the time of interest is different. The NPV for the calculation of the bridge life cycle cost for assigning cost to overload needs to be calculated at the time the overloads cross the bridge. Assume that a certain overload crosses a bridge in the year 2030. Use the identical bridge used above and shown in Table 9. The NPV which is the life cycle cost of the bridge at the time of crossing the bridge can be calculated as shown in Table 10. Difference of the time from the time when the NPV is calculated (*t*) is changed and the NPV is calculated to be $ 15,161,402 which is the bridge life cycle cost for assigning cost to the overload crossing the bridge.

Table 10. Calculation of bridge life cycle cost in 2030, used to assign cost to overloads.

Year	2010	2020	2030	2040	2050	2060	Sum
Cash flow ($)	5,000,000	800,000	800,000	800,000	800,000	800,000	9,000,000
t (years)	-20	-10	0	10	20	30	
$1/(1+i)^t$	2.412	1.553	1.000	0.644	0.415	0.267	
PV ($)	12,058,570	1,242,376	800,000	515,142	331,714	213,600	**15,161,402 (NPV)**

where, t is difference of the time from the time when the NPV is calculated, i is discount rate, PV is present value and NPV is net present value.

4. DEVELOPMENT OF A MEANS TO ASSIGN COST PER OVERLOAD CROSSING OF A BRIDGE

Overload vehicles causing damage or cracks in bridge components may be permitted when the damage and cracks are repairable without losing capacity and safety is not in jeopardy. It may be reasonable, however, for the permit applicant to be responsible for the reduced life of the bridge. A means to assign cost to the permit applicant for concrete decks and steel girder bridges is developed and described in this chapter.

Miner's damage rule and S-N relations for structural members which are used to calculate degree of cumulative fatigue damage are described. The degree of cumulative fatigue damage is used to calculate reduction of service life of bridges and the assigned cost is calculated by multiplying the bridge life cycle cost by the percentage of reduction of the service life (which is the same as the degree of the cumulative fatigue damage).

4.1 Miner's Rule

Methods to predict damage accumulation and/or bridge life consumption from overload vehicles are provided in references (Bruneau and Dicleli 1994, Dicleli and Bruneau 1995, Mohammadi and Polepeddi 2000, Li et al. 2001, Sadeghi and Fathali 2007, Cheung et al. 2008 and Wang et al. 2009). The most common method is to apply Miner's rule in calculating damage accumulation and to utilize the "stress range - number of cycles" to failure (S-N) relation to compute the number cycles to failure for bridge components. (Bruneau and Dicleli 1994, Dicleli and Bruneau 1995, Mohammadi and Polepeddi 2000 and Wang et al. 2009)

The S-N relation is defined as

$$N = \frac{C}{S^m} \tag{2}$$

where N = number of cycles to failure for the stress range S (ksi); and C and m = constants. C and m are given by AASHTO LRFD for various structural details as listed in Section 4.2.

The Miner's damage equation is

$$D = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \dots\dots\dots + \frac{n_k}{N_k} \tag{3}$$

where D = total damage accumulated in the structural component, n = the number of load cycles causing a particular stress level, and N = the number of cycles to failure at the same stress level.

To compute D, the stress population (or various levels of stress that occur) for a given bridge is divided into k ranges. For each range "i", the stress range value S_i is used in Eq. (2) to compute the corresponding number of cycles N_i that will cause failure at the stress S_i. The actual recorded number of cycles in a bridge, n_i, for the S_i stress range is then divided by N_i to compute the portion of the total damage caused by S_i. The damage associated with all k stress ranges is then computed using Eq. (3). The damage (D) from a single passage of an overload vehicle can be calculated by finding N associated with the maximum stress induced by the overload from Eq. 2 and using it alone in Eq. (3) with the number of times (n) that peak stress is developed as the overload vehicle crosses. The Eq. (3) for this case can be simplified to Eq. (4) if the overload only induces a single cycle (n=1) of the maximum stress.

$$D = \frac{1}{N} \tag{4}$$

The total life reduction due to the overload vehicles can be calculated by multiplying the resulting damage (D) by the total life of the bridge.

4.2 S-N Relations

S-N relations for structural members are required to calculate damage (D) of the structural members using Miner's damage accumulation rule as described in Section 4.1. The relations for reinforced concrete deck and steel members are described here.

1) Reinforced concrete deck:

Two types of fatigue related failures are commonly identified in concrete decks. The first type is fatigue failure of the steel reinforcement and the second type is debonding of the concrete surrounding the steel reinforcement resulting in loss of the capacity of deck. Two S-N relations, from reference literature, related to the two types of fatigue failure are shown in Figures 12 and 13.

Figure 12 shows a typical S-N relation for deformed steel reinforcements (Structural concrete: textbook on behaviour, design and performance: updated knowledge of the CEB/FIP Model code 1990”). The constants *C* and *m* in Eq. (2) for the relation were found by curve fitting to test data and they are taken respectively as 6 x 10^{17} and 8.5 (when *S* is in ksi units). Stress less than 20 ksi is not applicable to this relation since the number of cycles without failure, N, corresponding to a stress less than 20 ksi is infinity. This relation can be used to calculated damage in the deck that could lead to fatigue failure of the steel reinforcement.

Figure 13 shows moving wheel load - N relations of bridges with concrete decks under moving constant wheel load. The relation is given in terms of the ratio of applied load to static ultimate strength versus log N_{pf}. N_{pf} is number of cycles of the moving wheel load crossing the bridge specimens to failure. The relation was found from moving load tests (Petrou et al. 1994). The moving wheel load-N relationship can be used as an alternative to an S-N relation for decks failing by debonding of the concrete surrounding the steel reinforcement. The Figure illustrates that a bridge would fail with relatively few cycles with a heavier wheel load even though the amount of wheel loading is less than the ultimate static capacity of the bridge. This effect may be due to the movement of the load and progressive spread of damage along the path of the wheel, instead of damage just in the near vicinity of a static load.

This relation for deck damage was found for decks with isotropic and orthotropic steel reinforcement patterns. The spacing of the girders supporting the deck was 7 ft or 10 ft. This relation can only be applied to decks with certain girder spacings, due to limited test data, and the relation is dependant on the amount of steel reinforcement. It appears that additional data needs to be collected to use this relation for more general cases. In the meantime the S-N relation shown in Figure 12 can be used to calculate damage to steel reinforced concrete decks leading to steel fatigue failure with C = 6 x 10^{17} and m = 8.5 for Eq. (2). Figure 13 should only be used in cases matching the test situation.

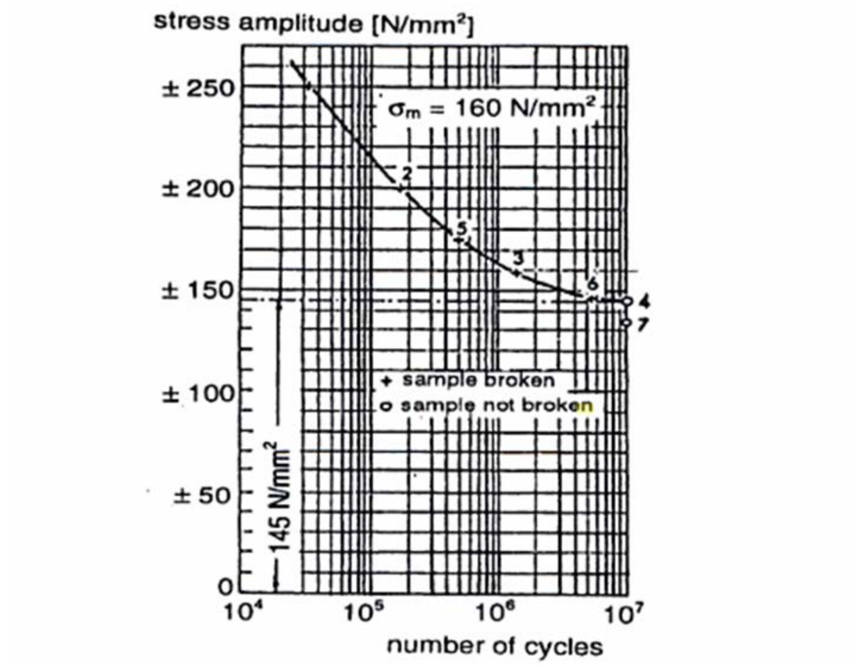


Figure 12. Typical fatigue strength curve of a deformed reinforcing bar.

(Structural concrete: textbook on behaviour, design and performance: updated knowledge of the CEB/FIP Model code 1990")

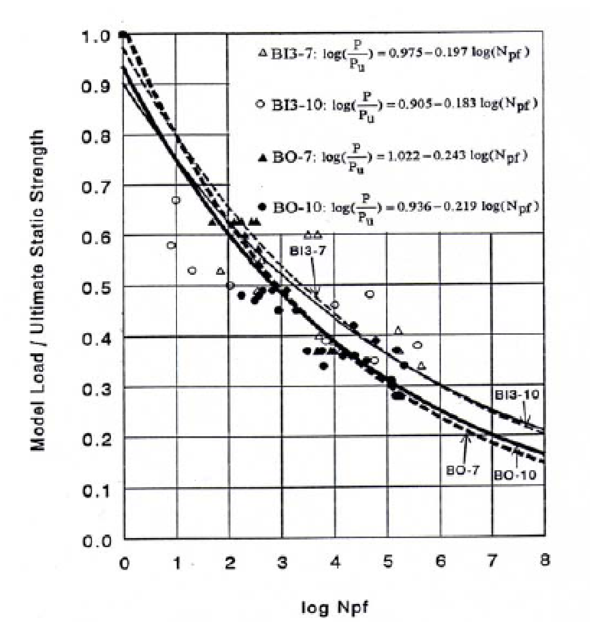


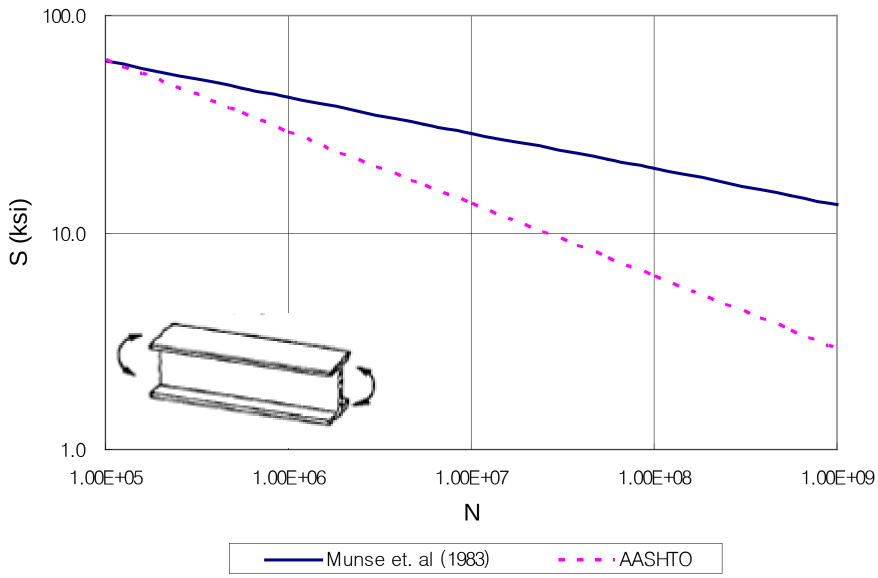
Figure 13. S-N fatigue curves under moving constant wheel load in terms of the ratio applied load to static ultimate strength versus log N_{pf}. (Petrou 1994)

(where BI3-7: Deck with isotropic reinforcement with 7ft girder spacing,

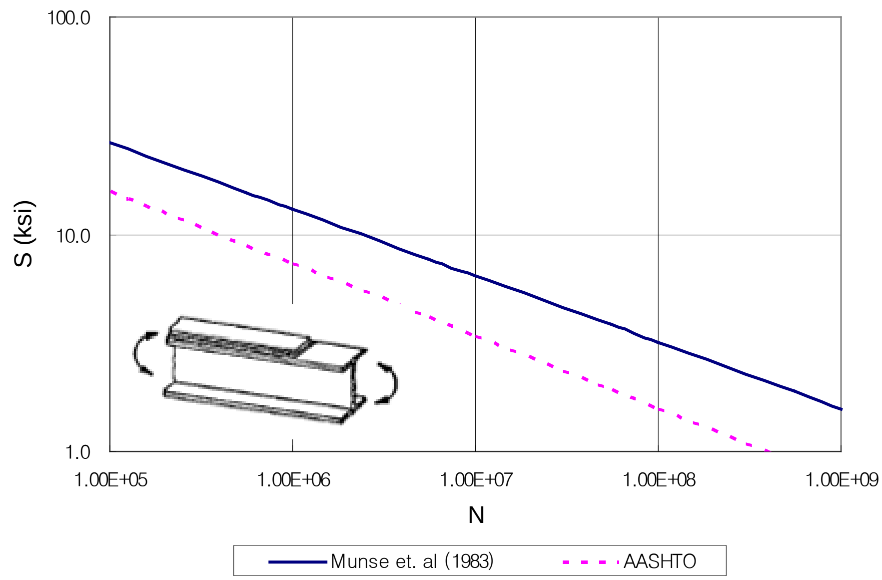
BI3-10: Deck with isotropic reinforcement with 10ft girder spacing,
BO-7: Deck with orthotropic reinforcement with 7ft girder spacing and
BO-10: Deck with orthotropic reinforcement with 10ft girder spacing)

2) Steel members:

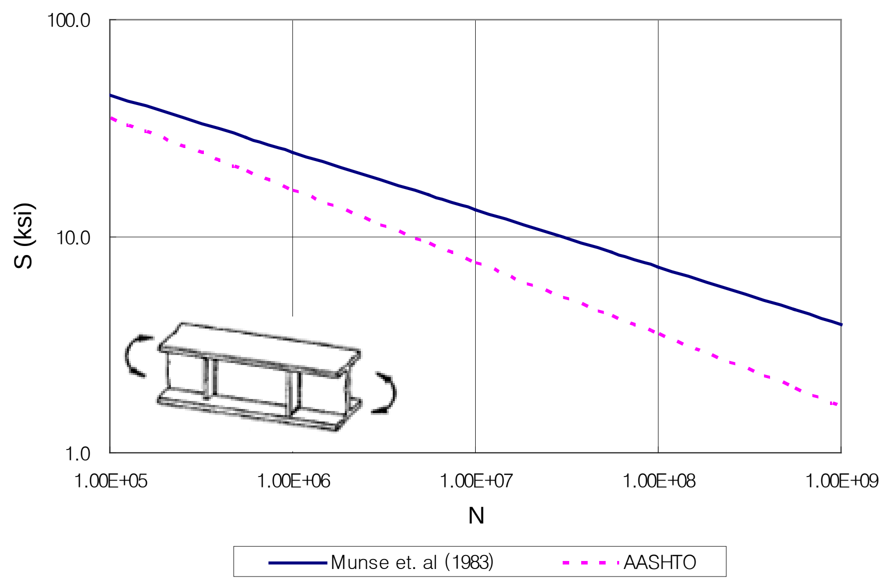
The S-N relations for steel members are included in provisions of the AASHTO LRFD and by Munse et al. (1983) for different structural details. A comparison of suggested relations is shown in Figure 14a-14d and Tables 11 & 12 for specific structural details.



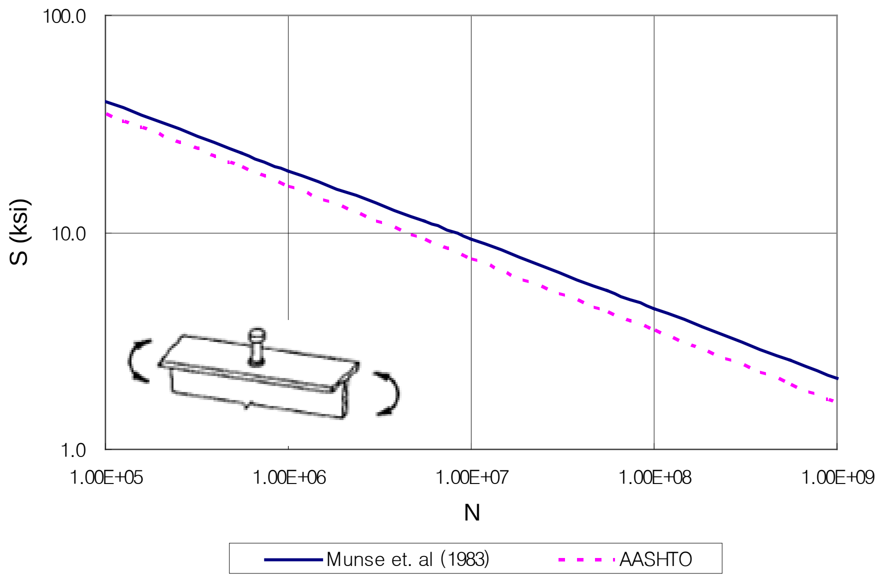
14a) Steel beam without welding, stiffener or studs,



14b) Steel beam with welded cover plate on flange,



14c) Beam with welded web stiffeners,



14d) Beam with welded studs,

Figure 14. Comparison of S-N relation for steel beams (in log scale).

Table 10. S-N relations for steel beams given by Munse et al. (1983).

Detail	*C**	*m**
2	1.12E+06	6.048
4	4.60E+05	5.663
5	1.90E+03	3.278
7	5.90E+03	3.771

** C and m are used in Eq. (2) for S-N relation*

Table 11. S-N relations for steel beams given by AASHTO LRFD.

Detail	*C**	*m**	Threshold (ksi)
2	2.50E+10	3	24
4	1.20E+10	3	16
5	3.90E+08	3	2.6
7	4.40E+09	3	12

** C and m is used in Eq. (2) for S-N relation*

It appears that the S-N relations given by AASHTO LRFD are more conservative than the values from a study by Munse et. al (1983). The AASHTO lines (Fig. 14) predict fewer cycles to failure at a certain stress range. A higher accumulation of damage will, therefore, be predicted and a higher cost will be assigned to an overload vehicle when the S-N relations from AASHTO LRFD is used. Since AASHTO is the common reference standard for design, the AASHTO relations for S-N are recommended for calculation of accumulated damage to bridges from overload vehicle crossings.

4.3 Procedure for calculation of assigned permit cost to overload vehicles

A procedure to calculate assigned costs to overload vehicles, as part of the permit fees when crossing a bridge, is described here step by step for concrete decks and for girders in steel girder bridges.

1) Concrete Deck

- Step 1: Calculate the maximum bending moment per unit width in the deck due to the overload vehicle axle weight using the AASHTO strip method. Calculate the corresponding maximum stress in the transverse (perpendicular to the girder direction) steel reinforcement. The maximum moment per unit width needs to be calculated by dividing the moment in the deck subjected to a single overload vehicle axle by the smaller of the width of the AASHTO effective strip width or the longitudinal axle spacing of the overload vehicle. Use 1.00 for the fatigue load factor for overloads and do not apply a multi-presence factor. The dynamic load allowance can also be ignored because the overload vehicle should be required to cross the bridge slower than 15 MPH.

- Step 2: Use the S-N relation given in Eq. (2) with stress in ksi units to find N with C = 6×10^{17} and m = 8.5 for constants.

- Step 3: Examine the number of heavily loaded axles on the overload vehicle to estimate the number of cycles (n) of high stress as the vehicle crosses. Then calculate damage (D) using Eq. (3) or (4).

- Step 4: Find life cycle cost of the deck using the procedure described in Section 3.3.

- Step 5: Calculate assigned cost to the overload using the following equation:
  Assigned cost = *D* x Life cycle cost of the deck.

2) Steel girder bridges.

- Step 1: Calculate the maximum moment range and corresponding stresses in critical steel girders. Use 1.00 for the fatigue load factor for overloads and do not apply a multi-presence factor. The dynamic load allowance can also be ignored because the overload vehicle should be required to cross the bridge slower than 15 MPH. It is recommended that the distribution equations developed from the phase I of this project be used to estimate the portion of the overload vehicle weight that is carried by an individual girder before calculating the moment range.

- Step 2: Use the S-N relation given in Eq. (2) to find N. Use appropriate C and m values from Table 11 to calculate N depending on the detailing of the girder.

- Step 3: Examine whether a single crossing of the vehicle will create multiple cycles (n) of moment range. Calculate damage (D) using Eq. (3) or (4).

- Step 4: Find life cycle cost of the deck using the procedure described in Section 3.3.

- Step 5: Calculate assigned cost to the overload using the following equation.
  Assigned cost = D x Life cycle cost of the deck

4.4 Approximate method for steel girder bridge – future work

As a first step in estimating cost impacts of overloads, the maximum alternating moment in the steel girder needs to be calculated to find an assigned cost as described in Section 4.3. Structural analysis, or a substitute approximate method, is required to find the stress for each bridge. It is recommended as a future study to develop a quick method which can be used to find an assigned cost without performing structural analysis by using the following steps or a similar method.

a) Define typical designs for bridges subjected to the standard vehicle with different bridge configurations (span length and number of span),
b) Find load induced bending moments for different configurations of overload vehicles (length and gross weight) and the different bridge configurations (span length and number of span),
c) Find maximum service stress due to overload vehicles using the information from steps a and b,
d) Find life reduction due to the maximum service stress from an S-N curve,
e) Find average life cycle cost per span of the bridge,
f) Calculate an assigned cost as: (average life cycle cost per span) x (span length) x (reduced life / estimated life) = Assigned costs,
g) Develop a single equation to estimate assigned costs (variables = number of span, span length, vehicle length and gross weight).

5. COST ASSIGNMENT EXAMPLES

Examples of assigning cost per crossing of bridges to overload vehicles are provided to illustrate the practical application of the method described in the previous sections. A first example looks at two pilot concrete decks and a second example looks at two pilot steel girder bridges.

5.1 Cost Assignment Examples for Concrete Decks

Two examples of assigning permit cost to three types of vehicles crossing a bridge are shown for concrete decks. The analyses were completed for two types of overload vehicles, i.e. a single lane overload and a dual lane overload, and also the AASHTO LRFD HL-93 truck for comparison. The selected overload vehicles affecting the bridge deck most severely were based on the Wisconsin State overload vehicle permit history. The configurations of the vehicles were summarized in Table 1.

1) Example 1 for concrete deck:

The configuration of the selected concrete deck for example 1 is as follows,

- Depth of the deck = 9 in
- Spacing of the girders = 7 ft
- Number of girders = 5
- Width of the deck = 32 ft
- Length of the bridge = 250 ft
- Length of the overhang is 2 ft from the center of the exterior girder
- Area of the deck = 8000 ft^2
- Lateral steel reinforcement of the deck = #4 bars with 6 inch spacing at top and bottom

The following assumptions are made for the calculation:

- The bridge was built in 1980
- Overload vehicle crosses the bridge once in 2010
- Service life of the bridge is 45 years ignoring the overload effect
- Discount rate is 4.5 %

- Construction and design cost of the deck is 13 $/ft^2$ (\$104,000)
- Maintenance cost including deck overly is 7.3 $/ft^2$ per 15 year (\$58,400 per 15 year)

Step 1: Calculate maximum transverse moment per unit width in the deck using the AASHTO strip method (AASHTO LRFD 4.6.2.1.3) with the strip spanning between girders. Then calculate the corresponding maximum stress in the lateral (perpendicular to the girder direction) steel reinforcement.

Calculated maximum moments per unit width and corresponding maximum stresses in the steel reinforcements are listed in Table 12.

Table 12. Maximum fatigue load moments in the deck and stresses in the reinforcement for deck example 1.

	AASHTO HL93 truck	Single lane overload	Dual lane overload
Maximum moment per unit width (kip-in/ft)	44.596	93.257	69.840
Maximum stress in steel reinforcement (ksi)	16.130	33.731	25.261

Step 2: Use S-N relation given in Eq. (2) to find N. Use C = 6×10^{17} and m = 8.5 for constants.

$$N = \frac{C}{S^m} \tag{2}$$

The result is listed in the Table 13.

Table 13. Calculated *N* for concrete deck example 1.

	AASHTO Standard truck	Single lane overload	Dual lane overload
N	Infinity*	61,646	719,975

* *N* for AASHTO standard truck is infinity since the stress in the reinforcement is less than 20ksi.

Step 3: Calculate damage (D) using Eq. (4).

$$D = \frac{1}{N} \tag{4}$$

The result is listed in the Table 14.

Table 14. Calculated *D* for concrete deck example 1.

	AASHTO Standard truck	Single lane overload	Dual lane overload
D	0	1.622E-05	1.389E-06

Step 4: Find life cycle cost of the deck using the procedure described in Section 3.3. The result is listed in the Table 15.

Table 15. Life cycle cost of deck example 1 in the year of 2010.

year	1970	1985	2000	2015	Total
Cash flow ($)	104,000	58,400	58,400	-	220,800
t	-40	-25	-10	5	
$1/(1+i)^t$	5.816	3.005	1.553	0.802	
PV ($)	604,864	175,492	90,695	-	**871,051 (NPV)**

Step 5: Calculate assigned cost to the overload using the following equation.

Assigned cost = *D* x Life cycle cost of the deck

The result is listed in the Table 16.

Table 16. Assigned cost to vehicles for concrete deck example 1.

	AASHTO Standard truck	Single lane overload	Dual lane overload
Assigned cost ($/crossing)	$0	$14.13	$1.21

2) Example 2 for concrete deck:

The configuration of the selected concrete deck for example 2 is as follows,

- Depth of the deck = 9 in
- Spacing of the girders = 7 ft
- Number of girders = 5
- Width of the deck = 40 ft
- Length of the bridge = 500 ft
- Length of the overhang is 2 ft from the center of the exterior girder
- Area of the deck = 20,000 ft^2
- Lateral steel reinforcement of the deck = #4 bars with 6 in spacing top and bottom

The following assumptions are made for the calculation:

- The bridge was built in 1970
- Overload vehicle crosses the bridge once in 2010
- Service life of the bridge is 45 years ignoring the overload effect
- Discount rate is 4.5 %
- Construction and design cost of the deck is 13 $\$/ft^2$ (\$ 260,000)
- Maintenance cost including deck overly is 7.3 $\$/ft^2$ per 15 year (\$146,000 per 15year).

Step 1: Calculate maximum transverse moment per unit width in the deck using the AASHTO strip method and calculate the corresponding maximum stress in the lateral (perpendicular to the girder direction) steel reinforcement.

Calculated maximum moment per unit width and corresponding maximum stress in the steel reinforcement are listed in Table 17.

Table 17. Maximum moment in the deck and stress in the reinforcement for deck example 2.

	AASHTO Standard truck	Single lane overload	Dual lane overload
Maximum moment per unit width (kip-in/ft)	44.596	93.257	69.840
Maximum stress in steel reinforcement (ksi)	16.130	33.731	25.261

Step 2: Use S-N relation given in Eq. (2) to find N. Use C = 6×10^{17} and m = 8.5 for constants.

The result is listed in the Table 18.

Table 18. Calculated *N* for concrete deck example 2.

	AASHTO Standard truck	Single lane overload	Dual lane overload
N	Infinity*	61,646	719,975

* *N* for AASHTO standard truck is infinity since the stress in the reinforcement is less than 20ksi.

<u>Step 3:</u> Calculate damage (D) using Eq. (4).

The result is listed in the Table 19.

Table 19. Calculated *D* for concrete deck example 2.

	AASHTO Standard truck	Single lane overload	Dual lane overload
D	0	1.622E-05	1.389E-06

<u>Step 4:</u> Find life cycle cost of the deck using the procedure described in chapter 3.3.

The result is listed in the Table 20.

Table 20. Life cycle cost of deck example 2 in the year of 2010.

Year	1980	1995	2010	2025	Total
Cash flow ($)	104,000	58,400	58,400	-	220,800
t	-30	-15	0	15	
$1/(1+i)^t$	3.745	1.935	1.000	0.517	
PV ($)	389,513	113,020	58,400	-	**560,934 (NPV)**

<u>Step 5:</u> Calculate assigned cost to the overload using the following equation.

Assigned cost = *D* x Life cycle cost of the deck

The result is listed in the Table 21.

Table 21. Assigned cost to vehicles for concrete deck example 2.

	AASHTO Standard truck	Single lane overload	Dual lane overload
Assigned cost ($/crossing)	$0	$9.10	$0.78

* N for AASHTO standard truck is ∞ since the stress in the reinforcement is less than 20ksi.

5.2 Cost Assignment Examples for Steel Girder Bridges

Two steel plate girder bridges were selected for cost assignment examples and the configurations of the bridges are listed in Table 22. The axle load and spacing configurations of the selected single lane overload vehicle crossing the selected bridge are shown in Figure 14. It is assumed that the overload vehicle crosses each bridge once in 2030, that life of each bridge, ignoring effect of overload vehicles, is 75 years and that the average discount ratio (i) is 4.5% from 2006 to 2081. Exterior girders were chosen for the calculation of the damage of the steel girder since the maximum moment occurs in the exterior girders. Girder distribution factors were calculated using the lever rule since the number of girders are less than 5.

Costs for the concrete deck including design cost, construction cost and maintenance cost is excluded in the calculations since the assigned cost to overloads for the concrete deck can be calculated separately using the method described in this report. The plans for the bridges are attached in the Appendix.

Table 22. Configurations of the bridges used in the example analyses for cost assignment.

Structure I.D.	Built year	Span (ft)	Width (ft)	# of girders	Girder Spacing (ft)	Deck depth (in)
B180176 (Example 1)	2006	142 + 155	27	3	10	9
B180167 (Example 2)	2006	220 + 270 + 270 + 230	60	4	12.5	10

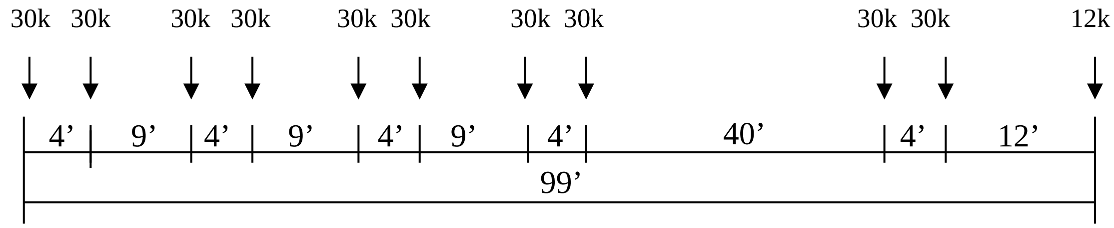


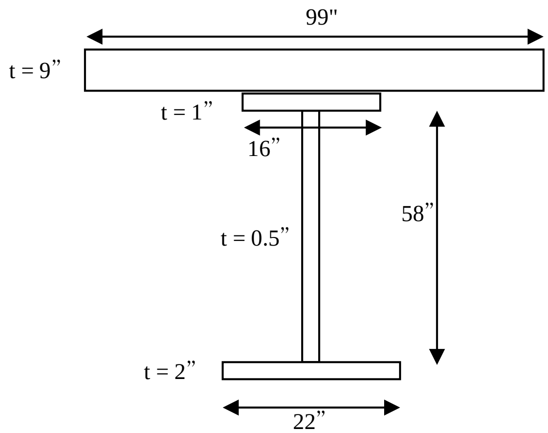
Figure 14. Axle load and spacing configurations of the selected overload vehicle.

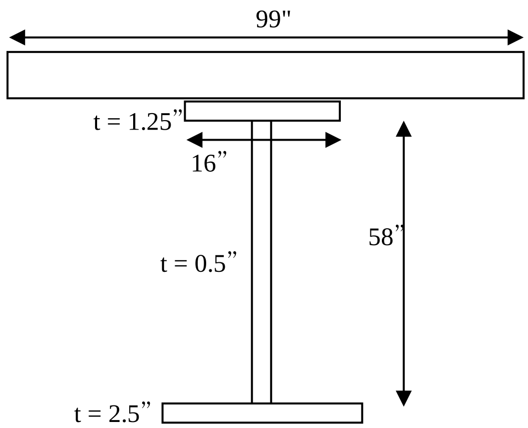
(lateral spacing of the wheels is 8ft)

1) Example 1 for steel girder bridge.

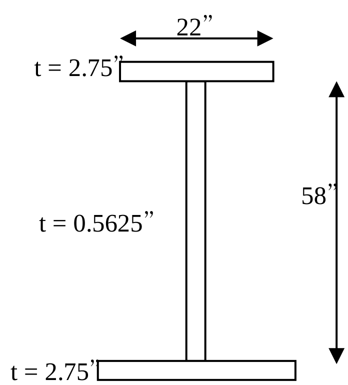
<u>Step 1:</u> Calculate maximum alternating moments and corresponding stresses in the steel sections for the selected critical sections.

Critical sections and sectional properties for the steel girder in example 1 are shown in Figure 15 and Table 23 to calculate maximum stresses. Calculated maximum alternating moments and corresponding stresses are listed in Table 24.





a) Positive moment region of the span 1 b) Positive moment region of the span 2



c) Negative moment region over the pier

Figure 15. Critical sections for steel girder example 1.

Table 23. Sectional properties of critical sections for steel girder example 1.

	Positive moment region of the span 1	Positive moment region of the span 2	Negative moment region over the pier
Moment of inertia (in^4)	150,008	174,528	120,862
Distance of the center of gravity from the top of the section (in)	24.11	26.31	31.75
Structural detail	Steel beam with studs	Steel beam with studs	Steel beam with stiffeners

Table 24. Maximum alternating moments and maximum stress at the critical sections for steel girder example 1.

	Positive moment region of the span 1	Positive moment region of the span 2	Negative moment region over the pier
Maximum positive moment* (kip-ft)	4178.79	4649.89	0
Maximum negative moment* (kip-ft)	-1211.63	-969.22	-2867.55
Maximum alternating moment* (kip-ft)	5390.42	5619.11	2867.55
Maximum stress (ksi)	19.79	17.16	9.04

* Load distribution factor was calculated (0.700) and applied to the results

<u>Step 2:</u> Use S-N relation given in Eq. (2) to find N. Use C and m given in Table 11 to calculate N. Structural details used to select C and m are steel beam with studs for positive moment regions and steel beam with stiffeners for negative moment region. The results are listed in Table 25.

Table 25. Calculated *N* for steel girder example 1.

	Positive moment region of the span 1	Positive moment region of the span 2	Negative moment region over the pier
N	567,695	870,765	5,955,900

<u>Step 3:</u> Calculate damage (D) using Eq. (4).

The result is listed in the Table 26. The largest damage governs the cost assigned to the overload and it will be used for the rest of the calculation.

Table 26. Calculated *D* for steel girder example 1.

	Positive moment region of the span 1	Positive moment region of the span 2	Negative moment region over the pier
D	1.762E-06	1.148E-06	0.168E-06

<u>Step 4:</u> Find life cycle cost of the steel girder using the procedure described in chapter 3.3.

The result is listed in the Table 27. Construction and design cost excluding cost for the deck are included in the cash flow for the base year (2006) and maintenance cost for painting girders in a 15 year cycle are included in the cash flow for the rest of the years.

Table 27. Life cycle cost for steel girder example 1 in the year of 2030.

year	2006	2021	2036	2051	2066	2081	Total
Cash flow ($)	697,557	65,042	65,042	65,042	65,042	65,042	1,022,769
t	-24	-9	6	21	36	51	
$1/(1+i)^t$	2.876	1.486	0.768	0.397	0.205	0.106	
PV ($)	2,006,184	96,659	49,946	25,808	13,336	6,891	**2,198,823 (NPV)**

<u>Step 5:</u> Calculate assigned cost to the overload using the following equation.

Assigned cost = *D* x Life cycle cost of the deck

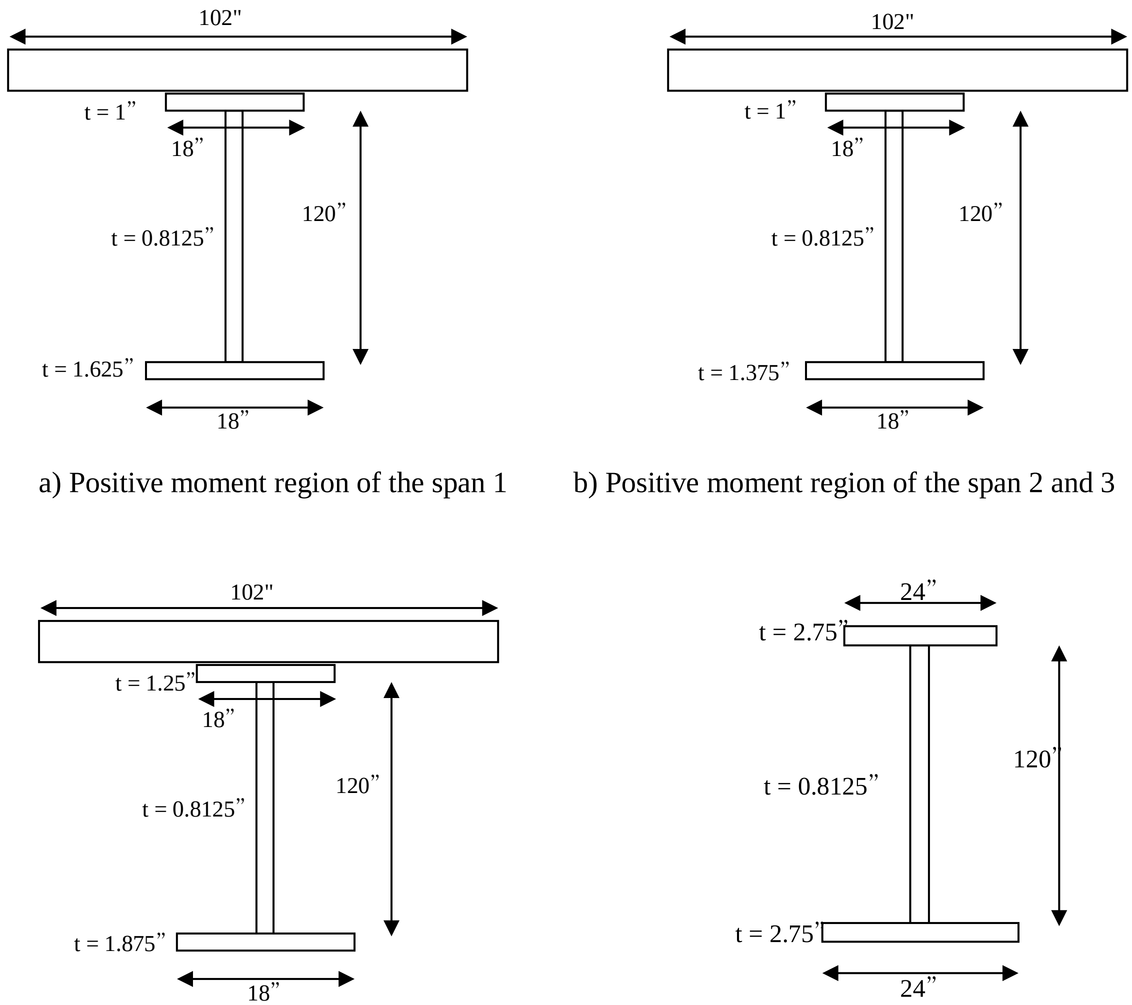
Assigned cost per crossing the bridge is

1.762E-06 (D) x $ 2,198,823 (NPV) = **<u>$3.87</u>**

2) Example 2 for steel girder bridge:

<u>Step 1:</u> Calculate maximum alternating moments and corresponding stresses at the steel sections for the selected critical sections.

Critical sections and sectional properties for the steel girders of example 2 are shown in Figure 16 and Table 28 to calculate maximum stresses. Calculated maximum alternating moments and corresponding stresses are listed in Table 29.



a) Positive moment region of the span 1

b) Positive moment region of the span 2 and 3

c) Positive moment region of the span 4

d) Negative moment region over the pier

Figure 16. Critical sections for steel girder example 2.

Table 28. Sectional properties of critical sections for steel girder example 2.

	Positive moment region of the span 1	Positive moment region of the span 2 and 3	Positive moment region of the span 4	Negative moment region over the pier
Moment of inertia (in^4)	627,309	590,427	670,855	614,313
Distance of the center of gravity from the top of the section (in)	42.72	41.21	43.78	62.75
Structural detail	Steel beam with studs	Steel beam with studs	Steel beam with studs	Steel beam with stiffeners

Table 29. Maximum alternating moments and maximum stress at the critical sections for steel girder example 2.

	Positive moment region of the span 1	Positive moment region of the span 2 and 3	Positive moment region of the span 4	Negative moment region over the pier
Maximum positive moment* (kip-ft)	8260.136	8351.792	8702.456	1407.064
Maximum negative moment* (kip-ft)	-2223.456	-1923.86	-2169.8	-5259.96
Maximum alternating moment* (kip-ft)	10483.592	10275.66	10872.26	6667.024
Maximum stress (ksi)	18.03	19.04	17.38	8.17

* Load distribution factor was calculated (0.760) and applied to the results

Step 2: Use S-N relation given in Eq. (2) to find N. Use C and m given in Table 11 to calculate N. Structural details used to select C and m are steel beam with studs for positive moment regions and steel beam with stiffeners for negative moment region. The result is listed in the Table30.

Table 30. Calculated *N* for steel girder example 2.

	Positive moment region of the span 1	Positive moment region of the span 2 and 3	Positive moment region of the span 4	Negative moment region over the pier
N	750,698	637,458	838,115	8,068,383

Step 3: Calculate damage (D) using Eq. (4).

The result is listed in the Table 31. The largest damage governs the cost assigned to the overload and it will be used for the rest of the calculation.

Table 31. Calculated *D* for steel girder example 2.

	Positive moment region of the span 1	Positive moment region of the span 2 and 3	Positive moment region of the span 4	Negative moment region over the pier
D	1.332E-06	**1.569E-06**	1.193E-06	0.124E-06

Step 4: Find life cycle cost of the steel girder using the procedure described in chapter 3.3.

The result is listed in the Table 32. Construction and design cost excluding cost for the deck are included in the cash flow for the base year (2006) and maintenance cost for painting girders in 15 year cycle are included in the cash flow for the rest of the year

Table 32. Life cycle cost for steel girder example 2 in the year of 2030.

year	2006	2021	2036	2051	2066	2081	Total
Cash flow ($)	4,949,758	437,306	437,306	437,306	437,306	437,306	7,136,288
t	-24	-9	6	21	36	51	
$1/(1+i)^t$	2.876	1.486	0.768	0.397	0.205	0.106	
PV ($)	14,235,572	649,878	335,805	173,518	89,660	46,329	15,530,763

Step 5: Calculate assigned cost to the overload using the following equation.

Assigned cost = *D* x Life cycle cost of the deck

Assigned cost per crossing the bridge is

1.569E-06 (D) x $ 15,530,763 (NPV) = **$24.36**

This cost should be added to the cost for deck usage calculated in the first examples.

6. SUMMARY

Overload vehicle travelling across a bridge, even if it is a single crossing, may affect not only the short term behavior of the bridge but also the long term performance and life cycle cost of the bridge. Generally, special permits are issued to overload vehicles without considering their cumulative effect on bridge components but considering only the ultimate capacity of bridges. The cumulative damage occurring in the bridge may reduce the life of the bridge and induce unexpected fatigue failure of the bridge. Therefore, it is suggested that the investigation of long term behavior of bridges might be considered when issuing permits in addition to the short term effects.

Long term behavior of concrete decks and steel girder bridges subjected to overloads was investigated with a comparison to those subjected to the AASHTO HL93 truck which is used in design. Overloads which are safe considering short term strength can cause long term problems such as fatigue failure or reduction of bridge service life.

It may be reasonable for the permit applicant to be responsible for the cost of repair, additional maintenance or reduced life of the bridges. The cost should be related to the total invested cost in the bridge including that to maintain the service life of the bridge. The concept of a life cycle cost is required to assess the assigned cost from the overload vehicle effects. A procedure to calculate bridge life cycle cost is outlined for concrete decks and steel girder bridges to be used as a part of steps used to calculate assigned cost to the overloads.

The means of assigning the cost was developed for concrete decks and steel girder bridges. Damage of the bridge components due to the overload was calculated using S-N relations and Miner's damage accumulation rule. The assigned cost was calculated using the life cycle cost of the bridge component and the damage accumulated in the bridge component.

The design concept for prestressed concrete girders is that cracking in the girders is prohibited under short term loading as well as long term loading. Permits for the overload vehicles should not be issued for those inducing cracks in the girders. A process of checking allowable tensile stress of the girders should be employed. Bridges with prestressed concrete girders were, therefore, excluded in this research.

Examples of assigning cost per crossing for bridges with overload vehicles were provided to illustrate practical application of the developed means to assign cost per overload. A first set of examples was performed for two pilot concrete decks and a second set of example was performed for two pilot steel girder bridges.

The actual costs due to damage, from the overload vehicles crossing the selected example bridges, was minimal. The total cost due to damage of a concrete deck and the steel supporting girders, based on reduced fatigue life, can be expected to be less than $75 in most cases. In general use of the process for assigning the cost of damage described here will usually not be practical unless significant damage, and damage costs greater than $75, is expected. This is more likely to be the case in bridges designed for loads considerably less than the present AASHTO LRFD HL93 truck loading.

7. REFERENCES

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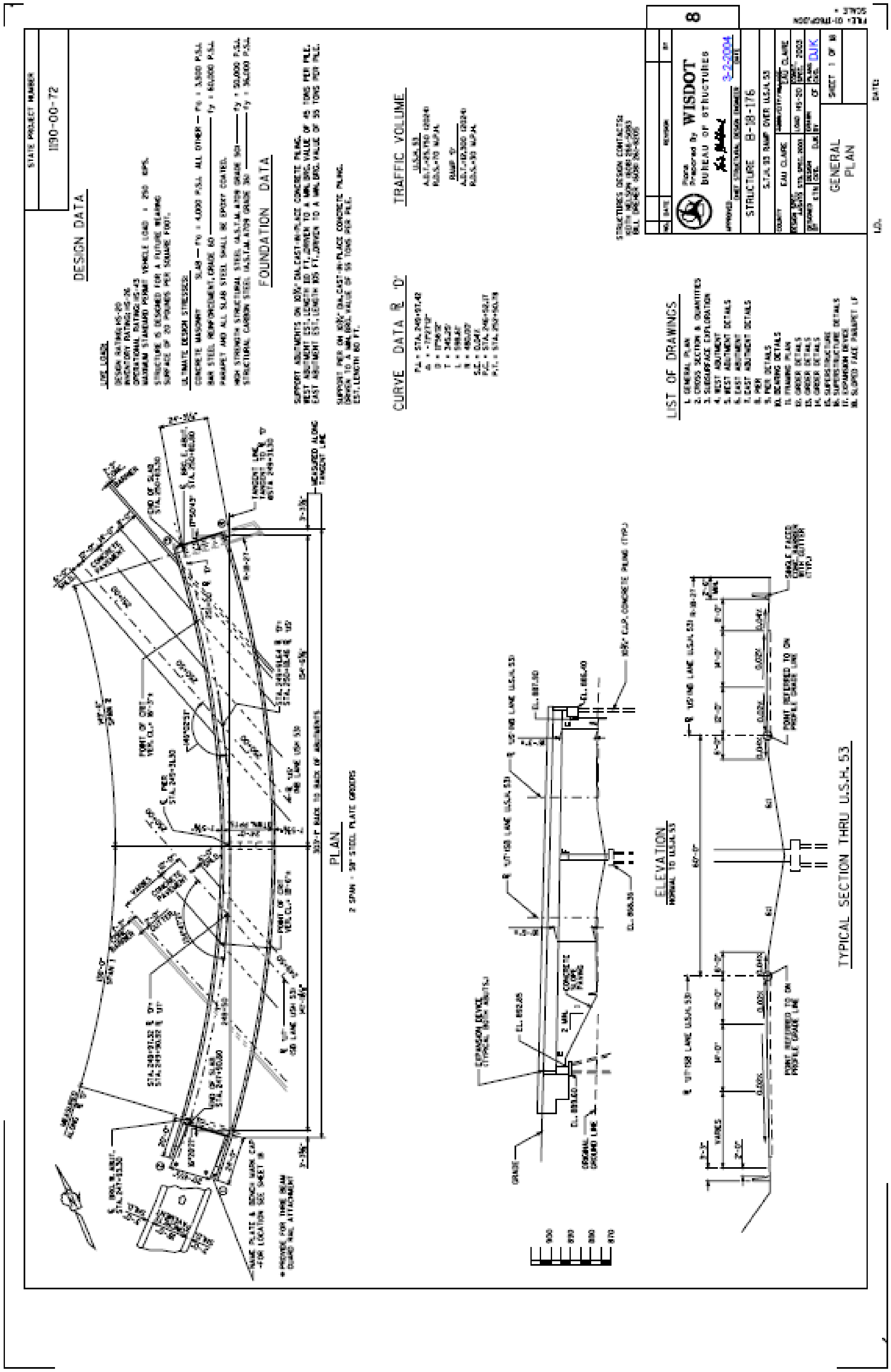
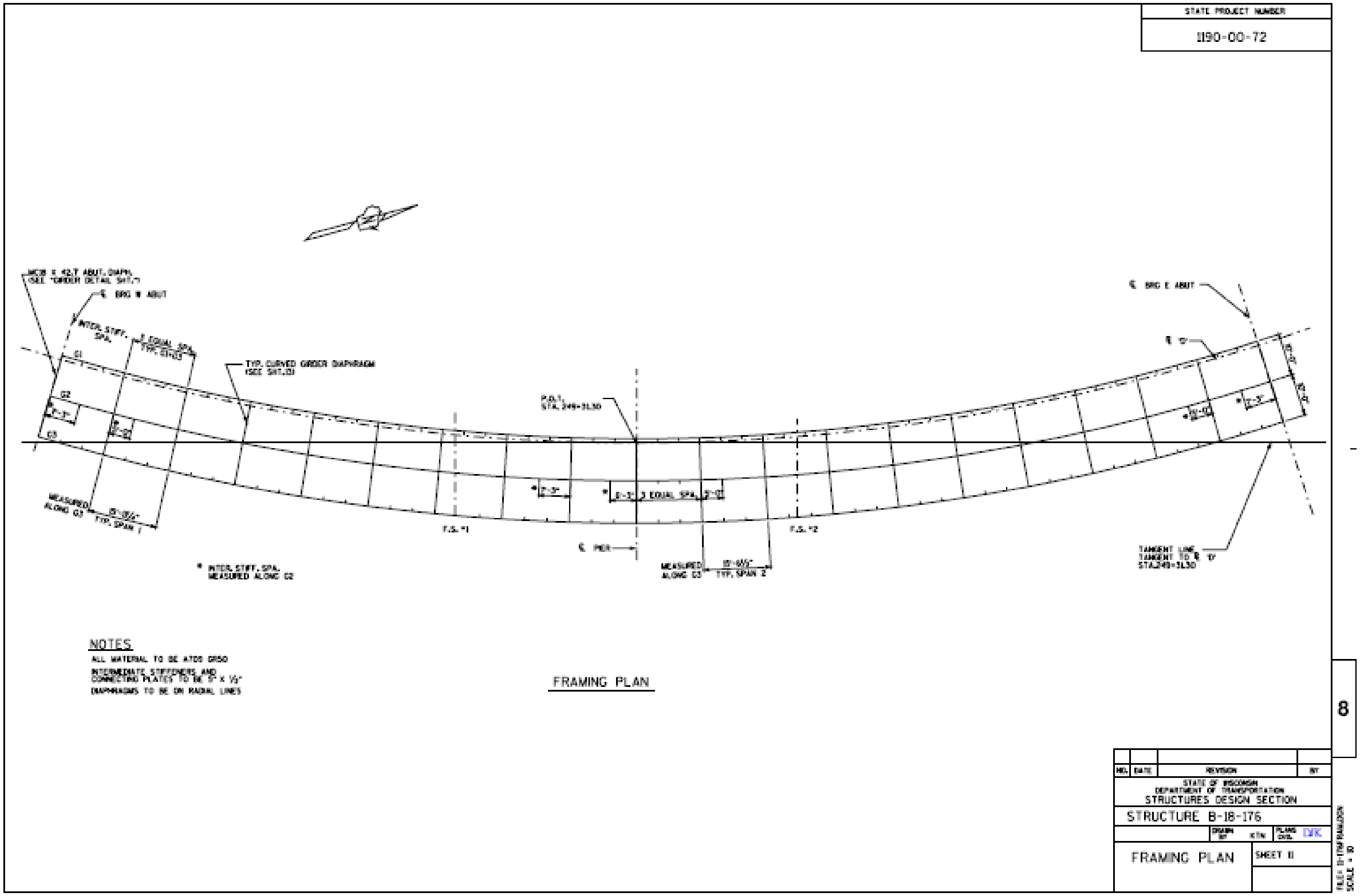
Appendix

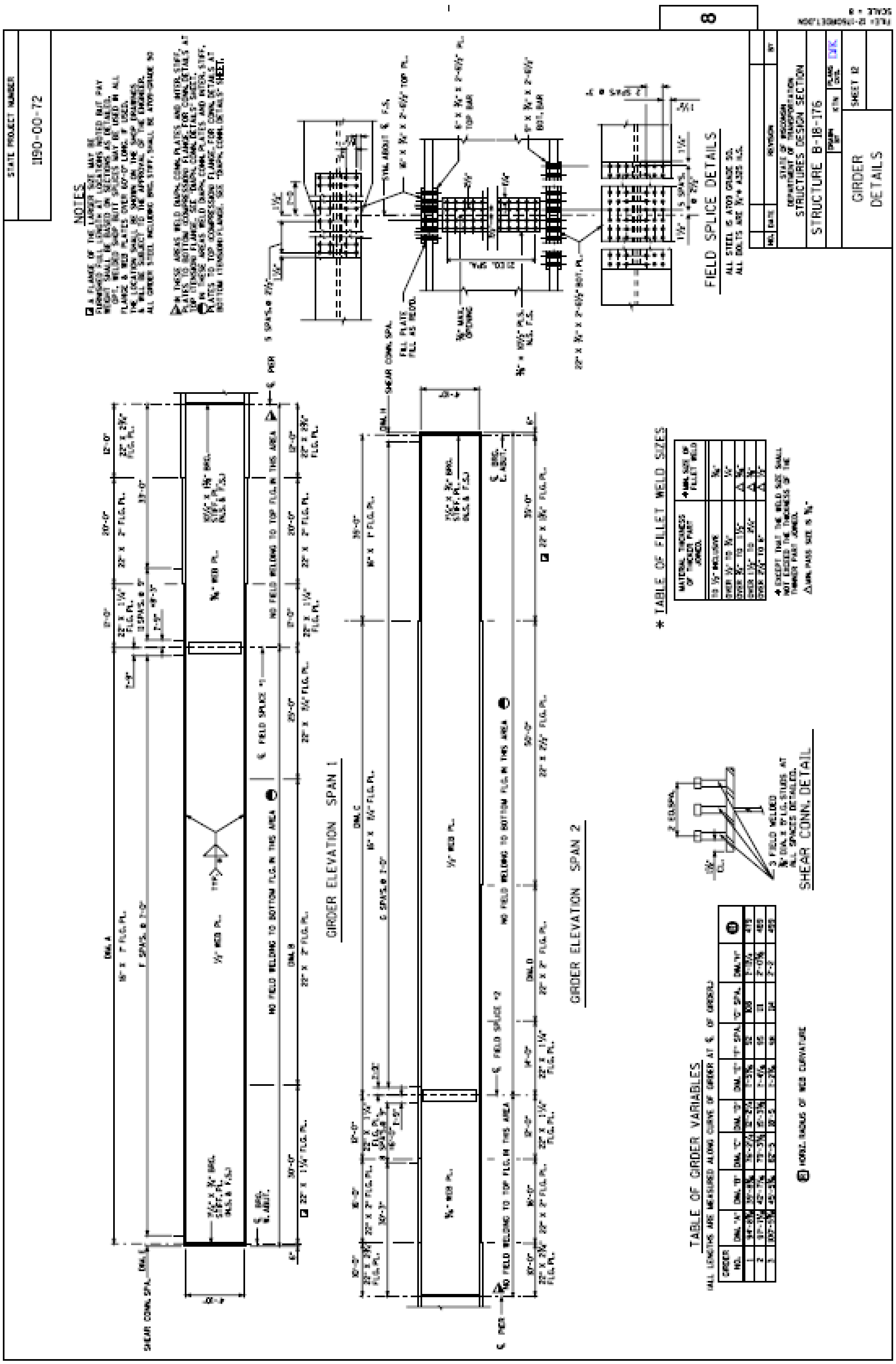
1. Plans for B180176 used for steel girder example 1

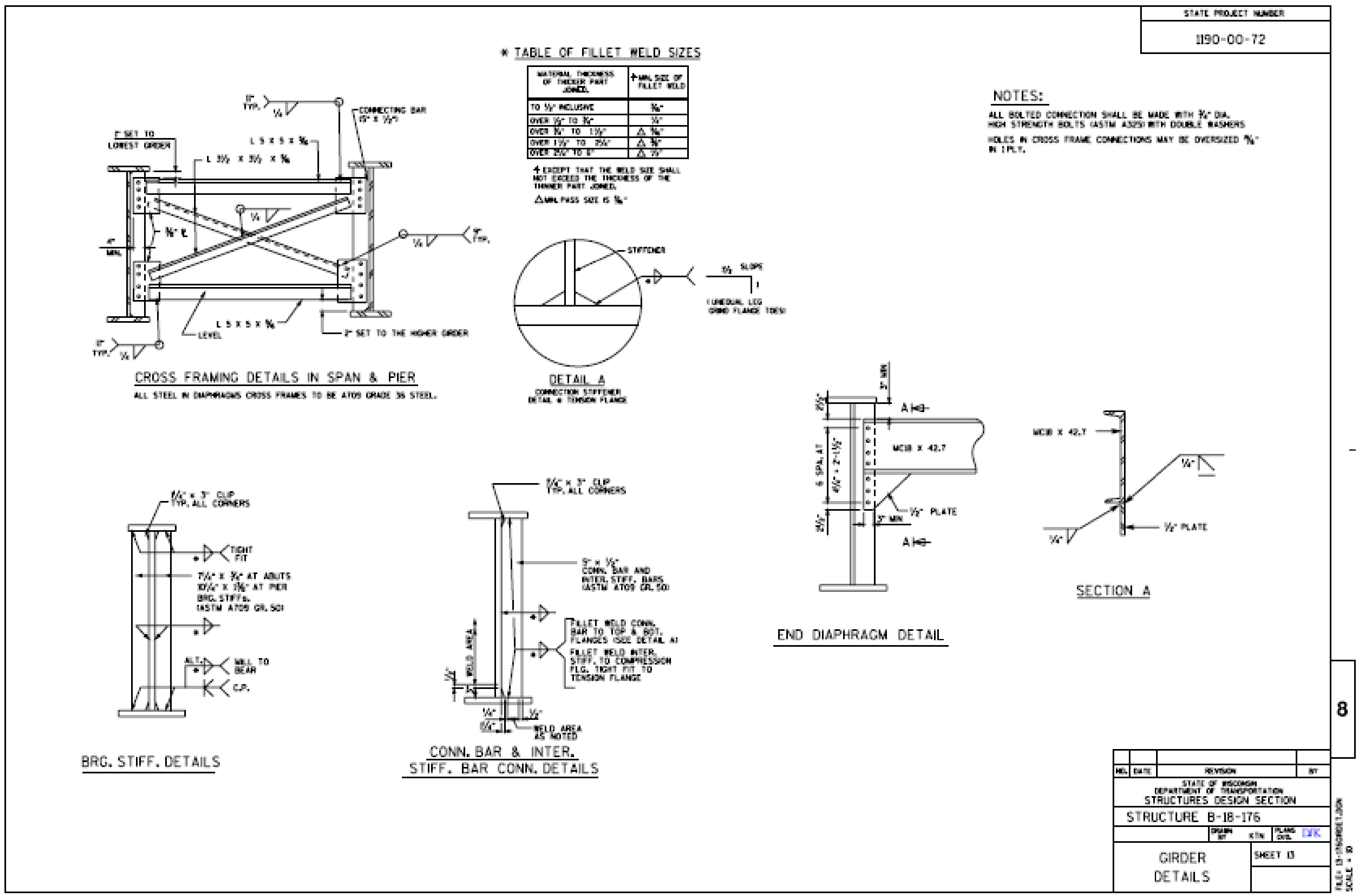


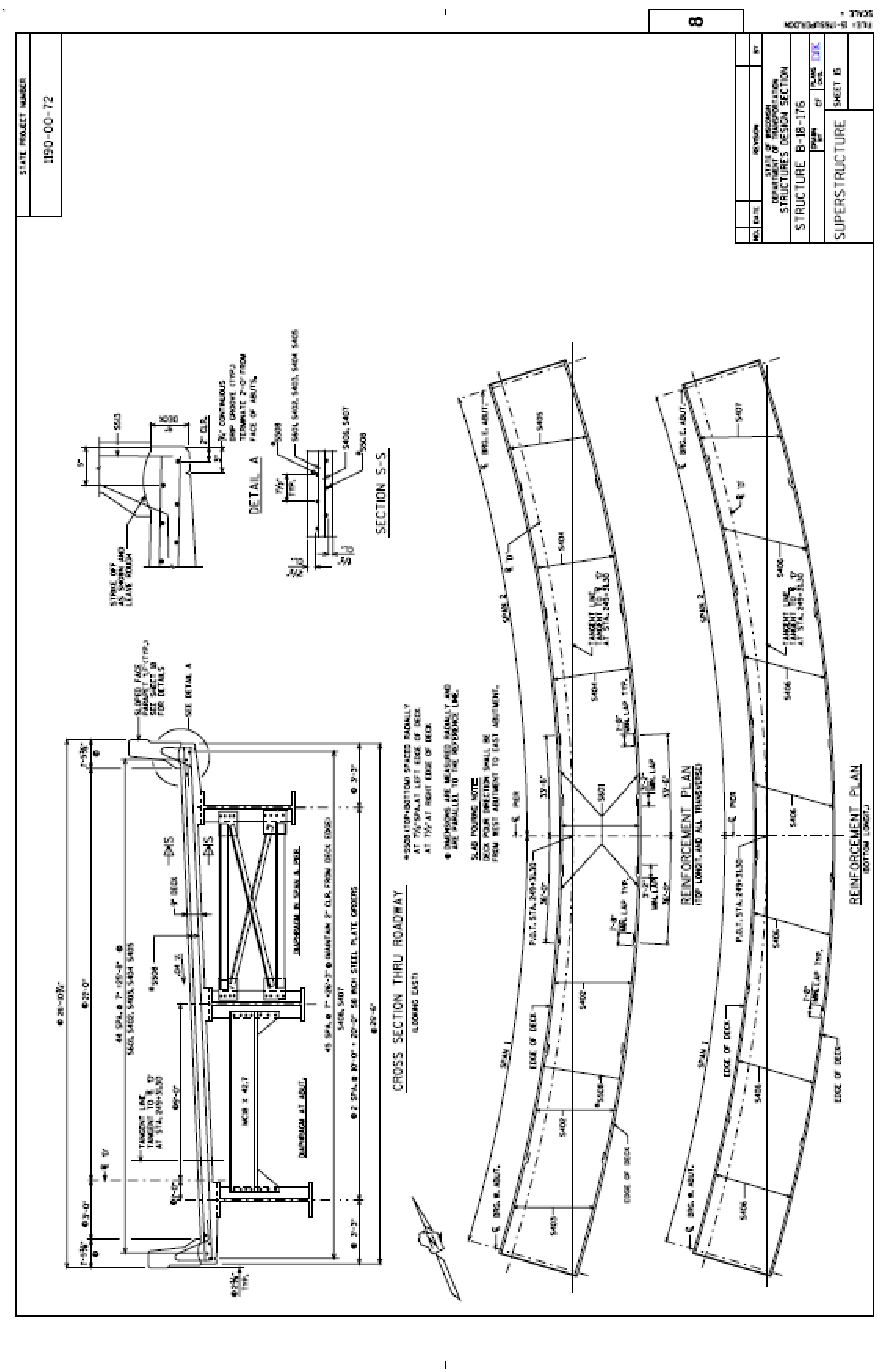
1190-00-72
DESIGN DATA
FOUNDATION DATA
TRAFFIC VOLUME
CURVE DATA
LIST OF DRAWINGS
WISDOT
STRUCTURE B-18-176
GENERAL PLAN
PLAN
ELEVATION
TYPICAL SECTION THRU U.S.H. 53



STATE PROJECT NUMBER
1190-00-72
℄ BRG W ABUT
INTER. STIFF. SPA.
TYP. CURVED GIRDER DIAPHRAGM (SEE SHT.10)
P.O.T. STA. 249+31.30
℄ BRG E ABUT
F.S. #1
F.S. #2
℄ PIER
MEASURED ALONG G3
TYP. SPAN 1
TYP. SPAN 2
TANGENT LINE TANGENT TO ℄ 'D' STA.249+31.30
* INTER. STIFF. SPA. MEASURED ALONG G2
NOTES
ALL MATERIAL TO BE A709 GR50
INTERMEDIATE STIFFENERS AND CONNECTING PLATES TO BE 5" x ½"
DIAPHRAGMS TO BE ON RADIAL LINES
FRAMING PLAN
8
NO. DATE REVISION BY
STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION
STRUCTURE B-18-176
FRAMING PLAN
SHEET 11

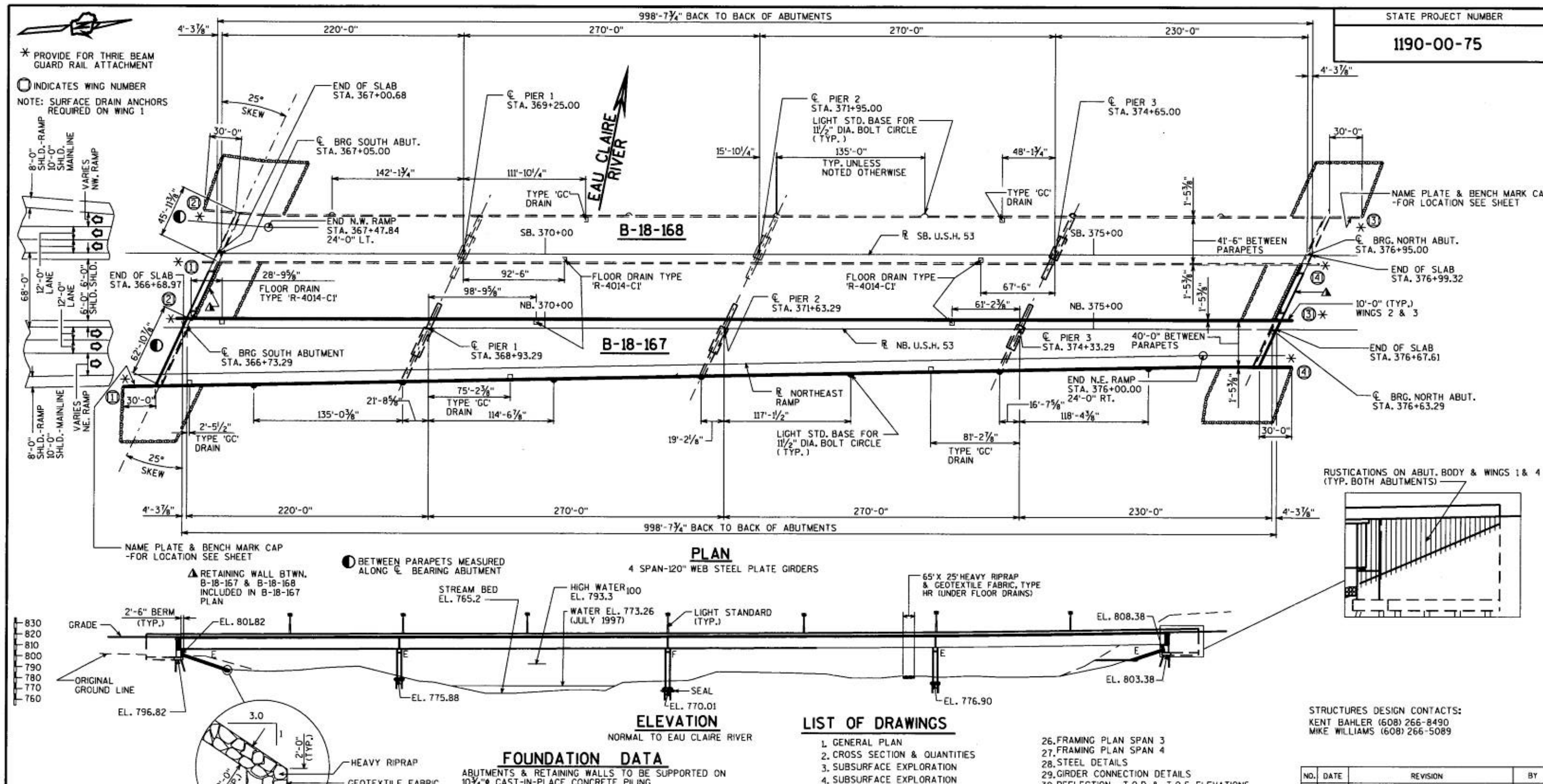

STATE PROJECT NUMBER
1190-00-72
NOTES
GIRDER ELEVATION SPAN 1
GIRDER ELEVATION SPAN 2
FIELD SPLICE DETAILS
ALL STEEL IS A709 GRADE 50.
TABLE OF FILLET WELD SIZES
SHEAR CONN. DETAIL
TABLE OF GIRDER VARIABLES
STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION
STRUCTURE B-18-176
GIRDER DETAILS
SHEET 12
8


STATE PROJECT NUMBER
1190-00-72
NOTES:
ALL BOLTED CONNECTION SHALL BE MADE WITH ¾" DIA. HIGH STRENGTH BOLTS (ASTM A325) WITH DOUBLE WASHERS
HOLES IN CROSS FRAME CONNECTIONS MAY BE OVERSIZED IN 1 PLY.
* TABLE OF FILLET WELD SIZES
MATERIAL THICKNESS OF THICKER PART JOINED.
MIN. SIZE OF FILLET WELD
EXCEPT THAT THE WELD SIZE SHALL NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED.
MIN. PASS SIZE IS
CONNECTING BAR
2" SET TO LOWEST GIRDER
2" SET TO THE HIGHER GIRDER
LEVEL
CROSS FRAMING DETAILS IN SPAN & PIER
ALL STEEL IN DIAPHRAGMS CROSS FRAMES TO BE A709 GRADE 36 STEEL.
STIFFENER
SLOPE
(UNEQUAL LEG GRIND FLANGE TOES)
DETAIL A
CONNECTION STIFFENER DETAIL @ TENSION FLANGE
MC18 X 42.7
PLATE
END DIAPHRAGM DETAIL
SECTION A
TYP. ALL CORNERS
TIGHT FIT
BRG. STIFFs. (ASTM A709 GR. 50)
MILL TO BEAR
C.P.
BRG. STIFF. DETAILS
CONN. BAR AND INTER. STIFF. BARS (ASTM A709 GR. 50)
FILLET WELD CONN. BAR TO TOP & BOT. FLANGES (SEE DETAIL A)
FILLET WELD INTER. STIFF. TO COMPRESSION FLG. TIGHT FIT TO TENSION FLANGE
WELD AREA AS NOTED
CONN. BAR & INTER. STIFF. BAR CONN. DETAILS
STATE OF WISCONSIN DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION
STRUCTURE B-18-176
GIRDER DETAILS
SHEET 13
8


STATE PROJECT NUMBER
1190-00-72
8
STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION
STRUCTURE B-18-176
SUPERSTRUCTURE
SHEET 15
CROSS SECTION THRU ROADWAY
(LOOKING EAST)
DETAIL A
SECTION S-S
REINFORCEMENT PLAN
(TOP LONGIT. AND ALL TRANSVERSE)
REINFORCEMENT PLAN
(BOTTOM LONGIT.)
DIAPHRAGM AT ABUT.
DIAPHRAGM IN SPAN & PIER
SPAN 1
SPAN 2
EDGE OF DECK
SLAB POURING NOTE

Appendix

2. Plans for B180167 used for steel girder example 2



* PROVIDE FOR THRIE BEAM GUARD RAIL ATTACHMENT

◯ INDICATES WING NUMBER

NOTE: SURFACE DRAIN ANCHORS REQUIRED ON WING 1

DESIGN DATA

LIVE LOAD:

DESIGN RATING: HS-20
INVENTORY RATING: HS-29
OPERATIONAL RATING: HS-49
MAXIMUM STANDARD PERMIT VEHICLE LOAD = 250 KIPS.

STRUCTURE IS DESIGNED FOR A FUTURE WEARING SURFACE OF 20 POUNDS PER SQUARE FOOT.

ULTIMATE DESIGN STRESSES:

CONCRETE MASONRY SLAB — f'c = 4,000 P.S.I. ALL OTHER — f'c = 3,500 P.S.I.
BAR STEEL REINFORCEMENT, GRADE 60 — fy = 60,000 P.S.I.
HIGH STRENGTH STRUCTURAL STEEL (ASTM A709, GRADE 50W) — fy = 50,000 P.S.I.

TRAFFIC VOLUME

NB U.S.H. 53
A.D.T.=14,125 (2024)
R.D.S.=70 M.P.H.

FOUNDATION DATA

ABUTMENTS & RETAINING WALLS TO BE SUPPORTED ON 10¾"Ø CAST-IN-PLACE CONCRETE PILING DRIVEN TO A MINIMUM BEARING VALUE OF 55 TONS PER PILE.
ESTIMATED 75'-0" LONG @ SOUTH ABUTMENT
ESTIMATED 80'-0" LONG @ NORTH ABUTMENT
PIER TO BE SUPPORTED ON HP 14 X 73 STEEL PILING DRIVEN TO A MINIMUM BEARING VALUE OF 95 TONS PER PILE.
ESTIMATED 50'-0" LONG @ PIER 1
ESTIMATED 60'-0" LONG @ PIER 2
ESTIMATED 80'-0" LONG @ PIER 3

HYDRAULIC DATA

100 YEAR FREQUENCY
Q_{100} =33000 C.F.S.
VEL. = 2.9 F.P.S.
HW.= EL. 793.3
WATERWAY AREA= 11500 SQ. FT.
DRAINAGE AREA= 815.5 SQ. MI.
ROAD OVERTOPPING = NA
SCOUR CRITICAL CODE = 5
2 YEAR FREQUENCY
Q_2 =8600 C.F.S.
HW_2 = EL. 781.3

LIST OF DRAWINGS

STRUCTURES DESIGN CONTACTS:
KENT BAHLER (608) 266-8490
MIKE WILLIAMS (608) 266-5089

NO.	DATE	REVISION	BY

Plans Prepared By WISDOT BUREAU OF STRUCTURES
APPROVED Finn Hubbard SDR 8-2-04
CHIEF STRUCTURAL DESIGN ENGINEER DATE

STRUCTURE B-18-167

N.B. U.S.H. 53 OVER EAU CLAIRE RIVER

COUNTY: EAU CLAIRE		TOWN/CITY: TOWN: SEYMOUR CITY: ALTOONA	
DESIGN SPEC. AASHTO STD. SPEC. 2003		LOAD HS-20	CONST. SPEC. 2003
DESIGNED BY MGW	DESIGN CK'D. SDR	DRAWN BY CRJ	PLANS CK'D. MGW

GENERAL PLAN

SHEET 1 OF 45

28

FILE= 01-167GEN.DGN
SCALE = 45

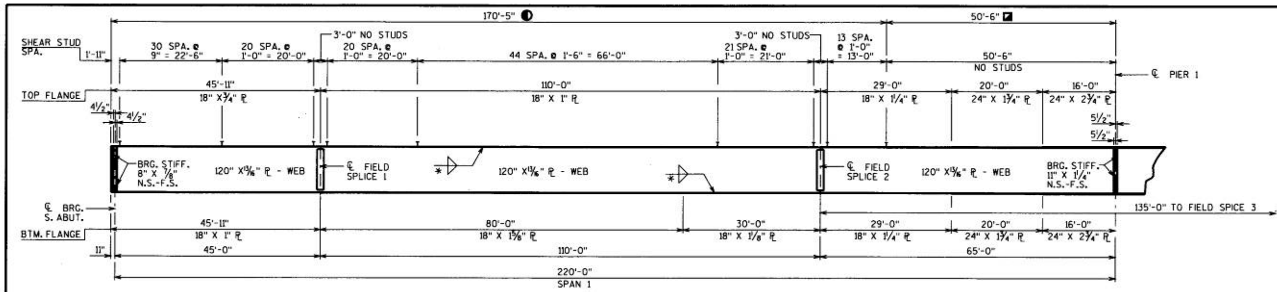
I.D. 1190-00-05C

DATE: JULY'04

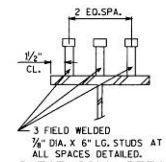
8

8

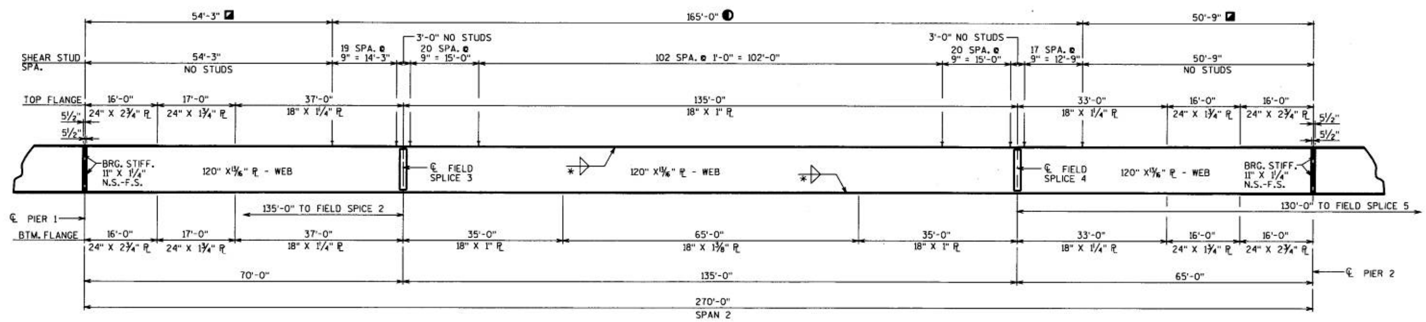
STATE PROJECT NUMBER
1190-00-75



GIRDER 1-3 ELEVATION SPAN 1



SHEAR CONN. DETAIL



GIRDER 1-3 ELEVATION SPAN 2

◐ NO FIELD WELDING TO BOTTOM FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO TOP (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT BOTTOM (TENSION) FLANGE.

◩ NO FIELD WELDING TO TOP FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO BOTTOM (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT TOP (TENSION) FLANGE.

NOTE: ALL STRUCTURAL STEEL USED IN SUPERSTRUCTURE IS TO BE ASTM A709, GRADE 50W, UNLESS NOTED OTHERWISE.

OPTIONAL WELDED SHOP SPLICES MAY BE USED FOR ALL FLANGE AND WEB PLATES OVER 60'-0" LONG. IF USED, THE LOCATION OF THE SPLICE SHALL BE SHOWN ON SHOP DRAWINGS AND WILL BE SUBJECT TO THE APPROVAL OF THE STRUCTURES DESIGN SECTION.

* TABLE OF FILLET WELD SIZES

MATERIAL THICKNESS OF THICKER PART JOINED.	† MIN. SIZE OF FILLET WELD
TO 1/2" INCLUSIVE	3/16"
OVER 1/2" TO 3/4"	1/4"
OVER 3/4" TO 1 1/2"	△ 5/16"
OVER 1 1/2" TO 2 1/4"	△ 3/8"
OVER 2 1/4" TO 6"	△ 1/2"

† EXCEPT THAT THE WELD SIZE SHALL NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED.

△ MIN. PASS SIZE IS 5/16"

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-18-167

DRAWN BY CRJ | PLANS CK'D. MGW

GIRDER DETAILS
GIRDERS 1,2, & 3
SPANS 1 & 2

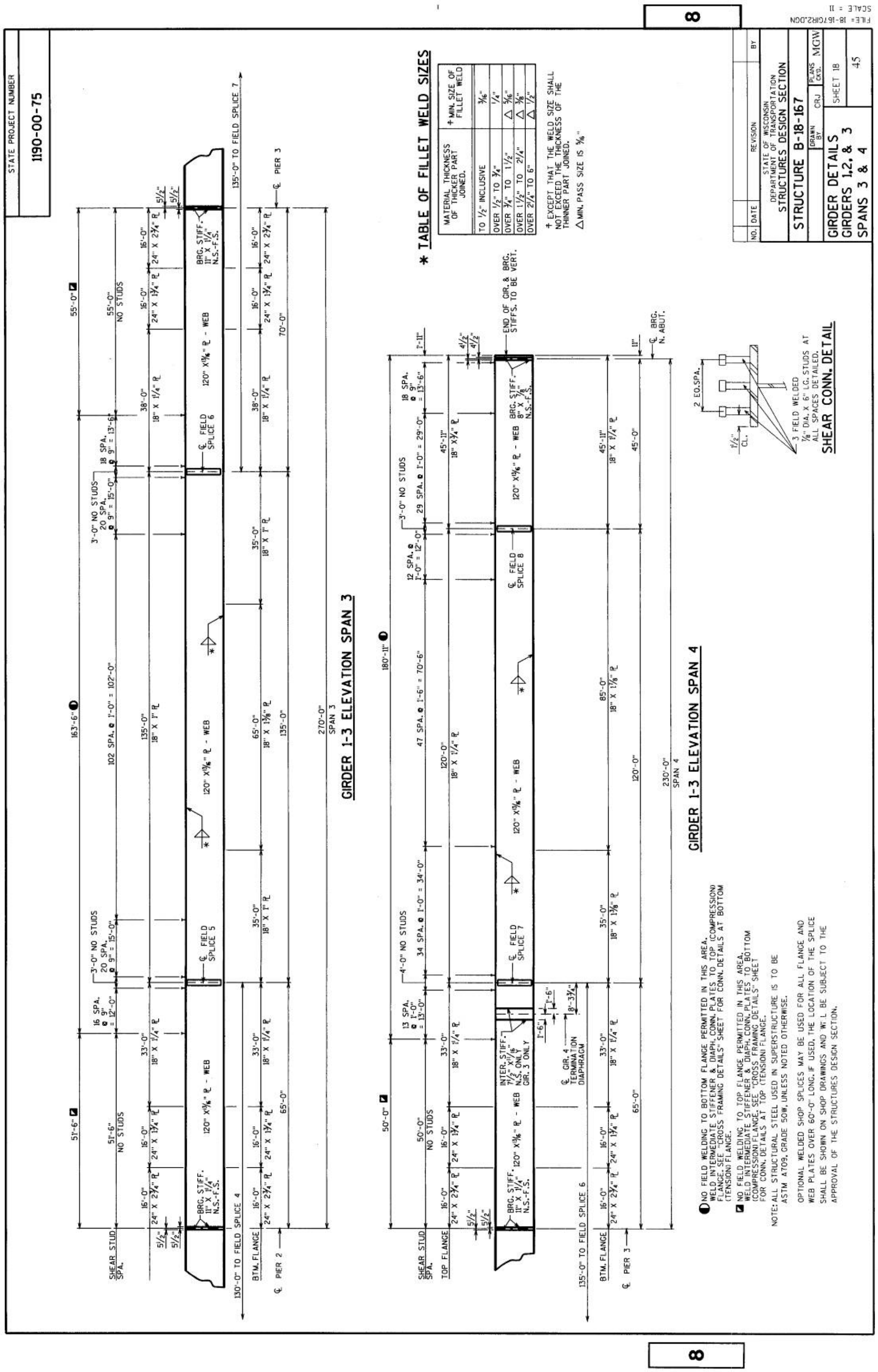
SHEET 17

44

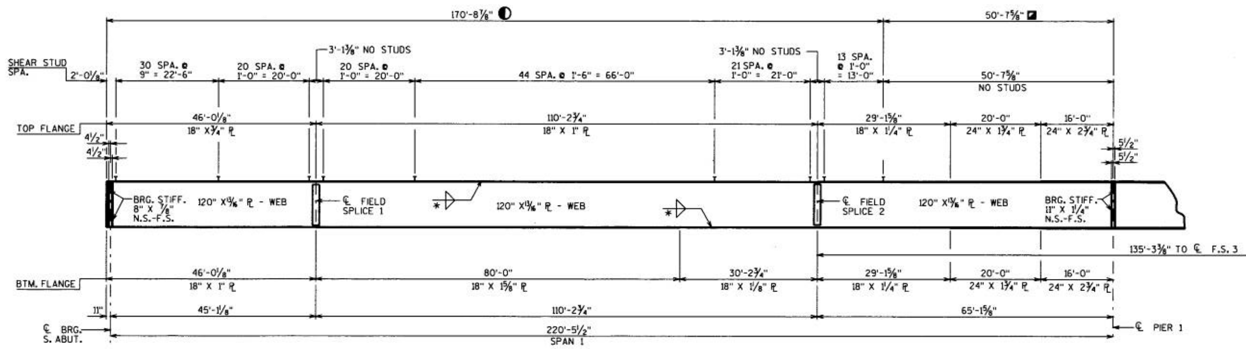
8

8

FILE: 17-167GIR1.DGN
SCALE : 11


STATE PROJECT NUMBER
1190-00-75
GIRDER 1-3 ELEVATION SPAN 3
GIRDER 1-3 ELEVATION SPAN 4
* TABLE OF FILLET WELD SIZES
SHEAR CONN. DETAIL
STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION
STRUCTURE B-18-167
GIRDER DETAILS
GIRDERS 1,2,& 3
SPANS 3 & 4
SHEET 18
45
8

STATE PROJECT NUMBER
1190-00-75



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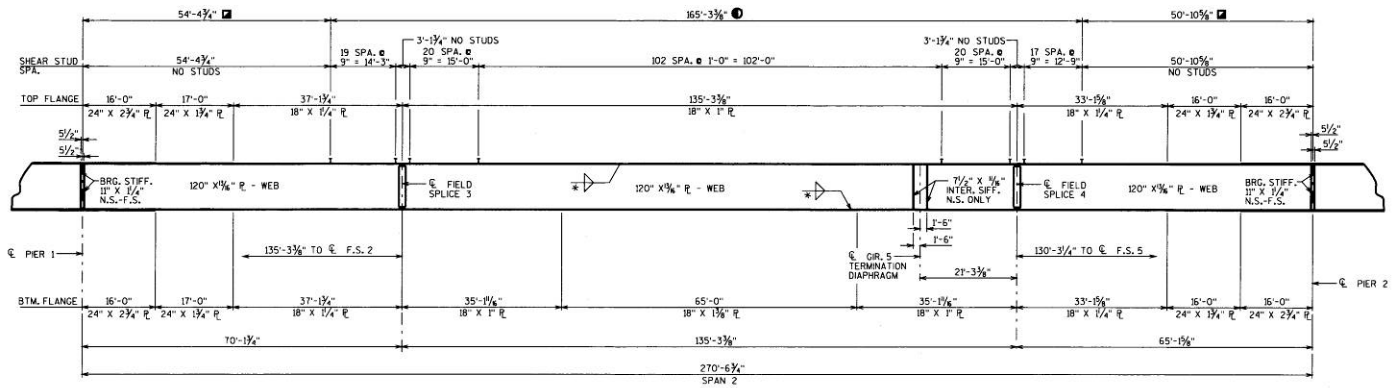

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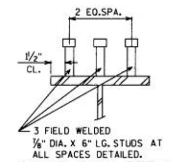


SHEAR CONN. DETAIL

◐ NO FIELD WELDING TO BOTTOM FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO TOP (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT BOTTOM (TENSION) FLANGE.

◩ NO FIELD WELDING TO TOP FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO BOTTOM (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT TOP (TENSION) FLANGE.

NOTE: ALL STRUCTURAL STEEL USED IN SUPERSTRUCTURE IS TO BE ASTM A709, GRADE 50W, UNLESS NOTED OTHERWISE.

OPTIONAL WELDED SHOP SPLICES MAY BE USED FOR ALL FLANGE AND WEB PLATES OVER 60'-0" LONG. IF USED, THE LOCATION OF THE SPLICE SHALL BE SHOWN ON SHOP DRAWINGS AND WILL BE SUBJECT TO THE APPROVAL OF THE STRUCTURES DESIGN SECTION.

* TABLE OF FILLET WELD SIZES

MATERIAL THICKNESS OF THICKER PART JOINED.	+ MIN. SIZE OF FILLET WELD
TO ½" INCLUSIVE	³⁄₁₆"
OVER ½" TO ¾"	¼"
OVER ¾" TO 1½"	△ ⁵⁄₁₆"
OVER 1½" TO 2¼"	△ ⅜"
OVER 2¼" TO 6"	△ ½"

+ EXCEPT THAT THE WELD SIZE SHALL NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED.

△ MIN. PASS SIZE IS ⁵⁄₁₆"

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-18-167

DRAWN BY CRJ | PLANS CK'D. MGW

GIR. 4 DETAILS
SPANS 1 & 2

SHEET 19

46

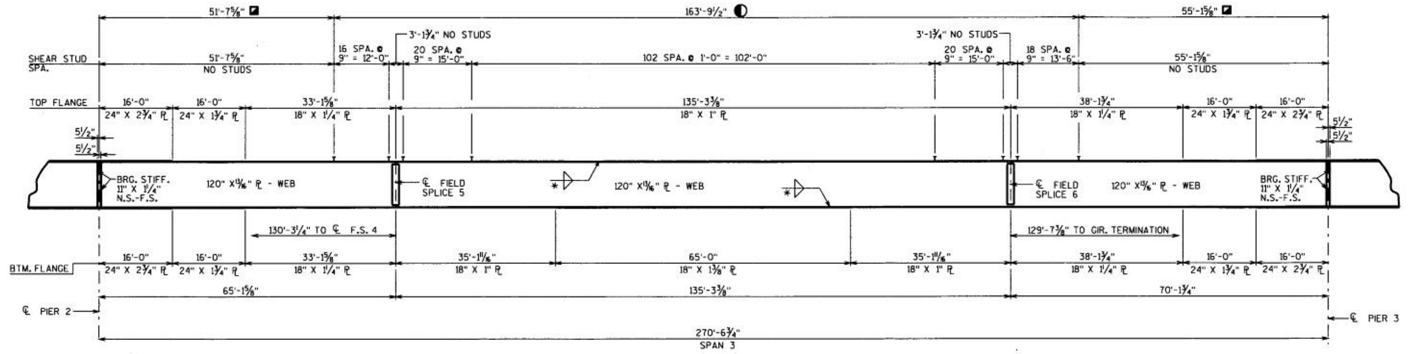
FILE: 19-167DR3.DGN
SCALE = 11

8

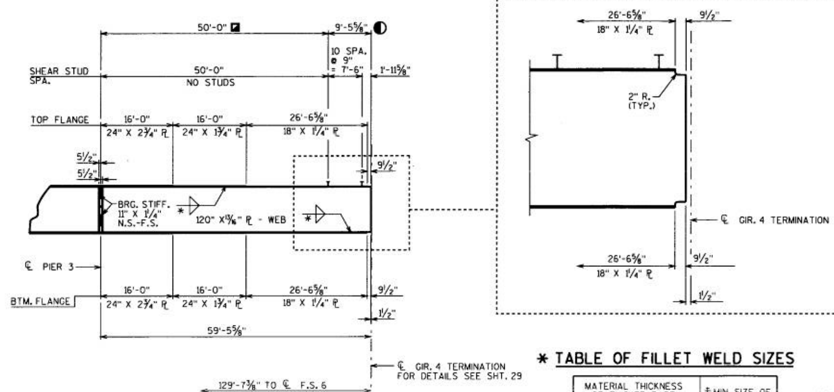
8

STATE PROJECT NUMBER

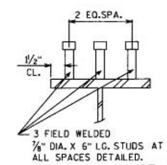
1190-00-75



GIRDER 4 ELEVATION SPAN 3



GIRDER 4 ELEVATION SPAN 4



SHEAR CONN. DETAIL

◐ NO FIELD WELDING TO BOTTOM FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO TOP (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT BOTTOM (TENSION) FLANGE.

◪ NO FIELD WELDING TO TOP FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO BOTTOM (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT TOP (TENSION) FLANGE.

NOTE: ALL STRUCTURAL STEEL USED IN SUPERSTRUCTURE IS TO BE ASTM A709, GRADE 50W, UNLESS NOTED OTHERWISE.

OPTIONAL WELDED SHOP SPLICES MAY BE USED FOR ALL FLANGE AND WEB PLATES OVER 60'-0" LONG. IF USED, THE LOCATION OF THE SPLICE SHALL BE SHOWN ON SHOP DRAWINGS AND WILL BE SUBJECT TO THE APPROVAL OF THE STRUCTURES DESIGN SECTION.

* TABLE OF FILLET WELD SIZES

MATERIAL THICKNESS OF THICKER PART JOINED.	ǂ MIN. SIZE OF FILLET WELD
TO ½" INCLUSIVE	3/16"
OVER ½" TO ¾"	¼"
OVER ¾" TO 1½"	△ 5/16"
OVER 1½" TO 2¼"	△ ⅜"
OVER 2¼" TO 6"	△ ½"

ǂ EXCEPT THAT THE WELD SIZE SHALL NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED.

△ MIN. PASS SIZE IS 3/16"

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-18-167

DRAWN BY CRJ | PLANS CK'D. MGW

GIR. 4 DETAILS
SPANS 3 & 4

SHEET 20

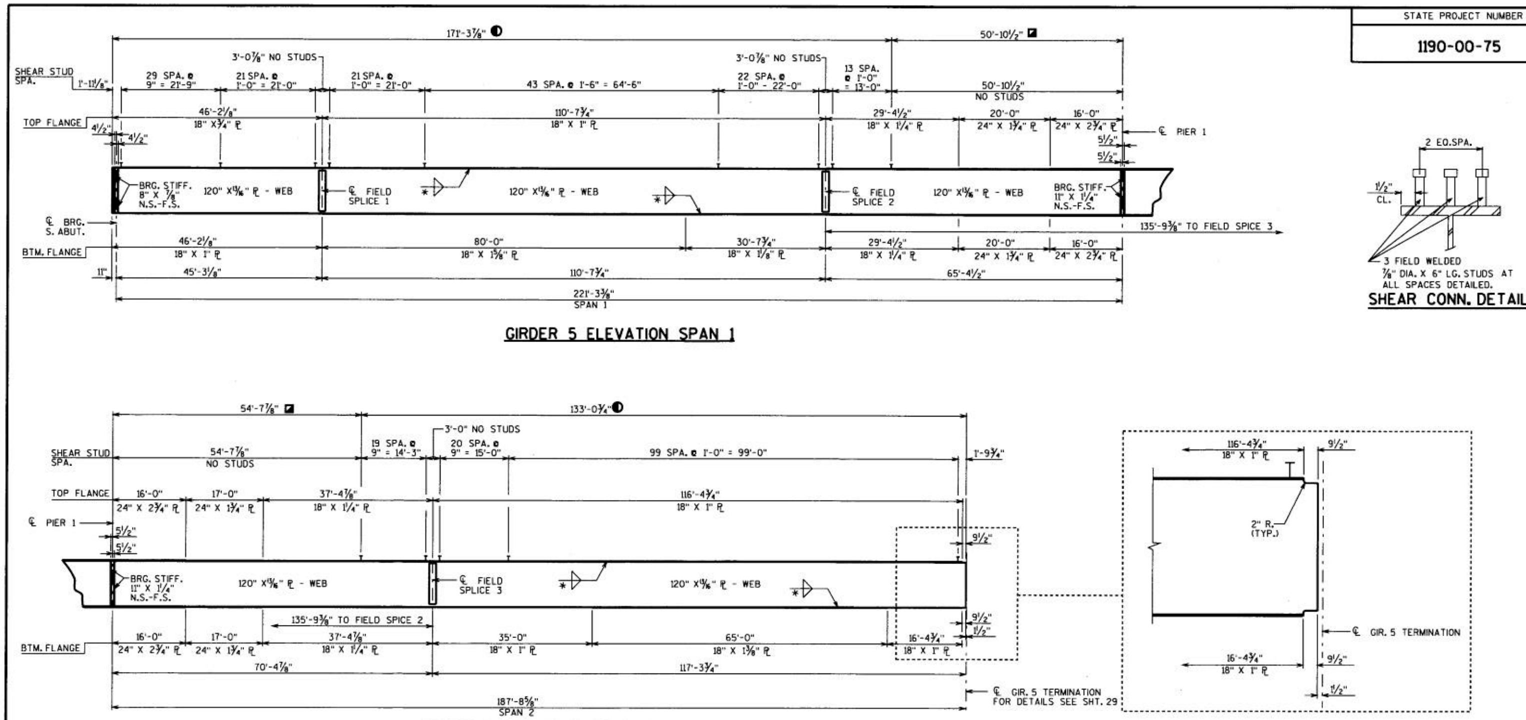
47

FILE= 20-167GIR4.DGN
SCALE = 11

8

STATE PROJECT NUMBER

1190-00-75



GIRDER 5 ELEVATION SPAN 1

GIRDER 5 ELEVATION SPAN 2

SHEAR CONN. DETAIL

◐ NO FIELD WELDING TO BOTTOM FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO TOP (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT BOTTOM (TENSION) FLANGE.

◪ NO FIELD WELDING TO TOP FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO BOTTOM (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT TOP (TENSION) FLANGE.

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OPTIONAL WELDED SHOP SPLICES MAY BE USED FOR ALL FLANGE AND WEB PLATES OVER 60'-0" LONG. IF USED, THE LOCATION OF THE SPLICE SHALL BE SHOWN ON SHOP DRAWINGS AND WILL BE SUBJECT TO THE APPROVAL OF THE STRUCTURES DESIGN SECTION.

* TABLE OF FILLET WELD SIZES

MATERIAL THICKNESS OF THICKER PART JOINED.	+ MIN. SIZE OF FILLET WELD
TO 1/2" INCLUSIVE	3/16"
OVER 1/2" TO 3/4"	1/4"
OVER 3/4" TO 1 1/2"	△ 5/16"
OVER 1 1/2" TO 2 1/4"	△ 3/8"
OVER 2 1/4" TO 6"	△ 1/2"

+ EXCEPT THAT THE WELD SIZE SHALL NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED.

△ MIN. PASS SIZE IS 5/16"

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-18-167

DRAWN BY CRJ PLANS CK'D. MGW

GIRDER 5 DETAILS
SPANS 1 & 2

SHEET 21

48

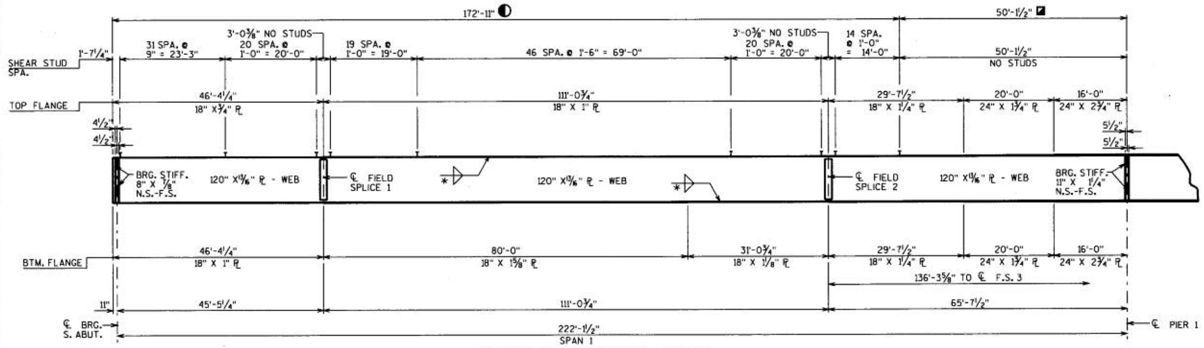
FILE= 21-167GIR5.DGN
SCALE = 11

8

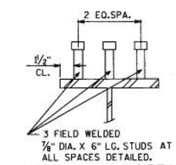
8

STATE PROJECT NUMBER

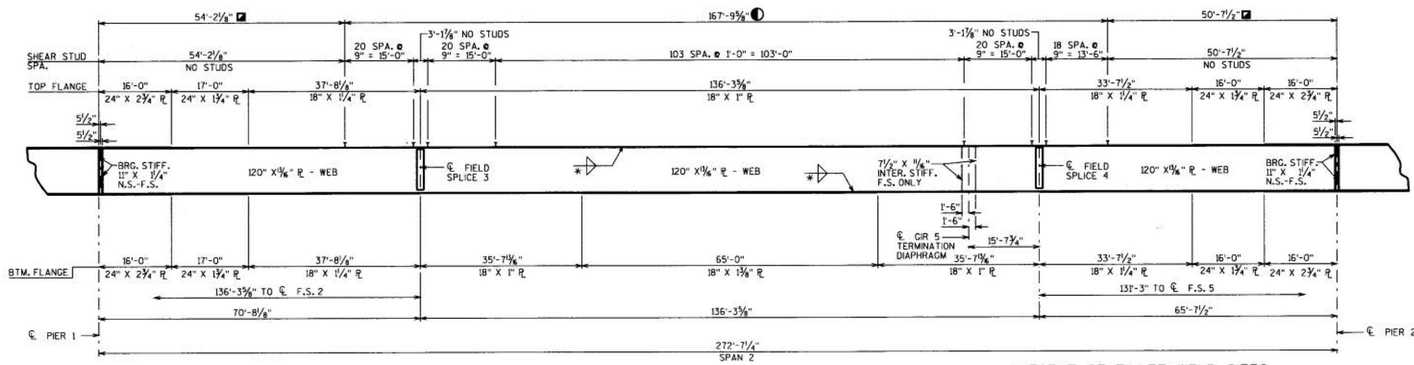
1190-00-75



GIRDER 6 ELEVATION SPAN 1



SHEAR CONN. DETAIL



GIRDER 6 ELEVATION SPAN 2

◐ NO FIELD WELDING TO BOTTOM FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO TOP (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT BOTTOM (TENSION) FLANGE.

◩ NO FIELD WELDING TO TOP FLANGE PERMITTED IN THIS AREA. WELD INTERMEDIATE STIFFENER & DIAPH. CONN. PLATES TO BOTTOM (COMPRESSION) FLANGE. SEE "CROSS FRAMING DETAILS" SHEET FOR CONN. DETAILS AT TOP (TENSION) FLANGE.

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* TABLE OF FILLET WELD SIZES

MATERIAL THICKNESS OF THICKER PART JOINED.	+ MIN. SIZE OF FILLET WELD
TO 1/2" INCLUSIVE	3/16"
OVER 1/2" TO 3/4"	1/4"
OVER 3/4" TO 1 1/2"	△ 5/16"
OVER 1 1/2" TO 2 1/4"	△ 3/8"
OVER 2 1/4" TO 6"	△ 1/2"

+ EXCEPT THAT THE WELD SIZE SHALL NOT EXCEED THE THICKNESS OF THE THINNER PART JOINED.

△ MIN. PASS SIZE IS 5/16"

NO.	DATE	REVISION	BY

STATE OF WISCONSIN
DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION

STRUCTURE B-18-167

DRAWN BY CRJ | PLANS CK'D. MGW

GIR. 6 DETAILS
SPANS 1 & 2

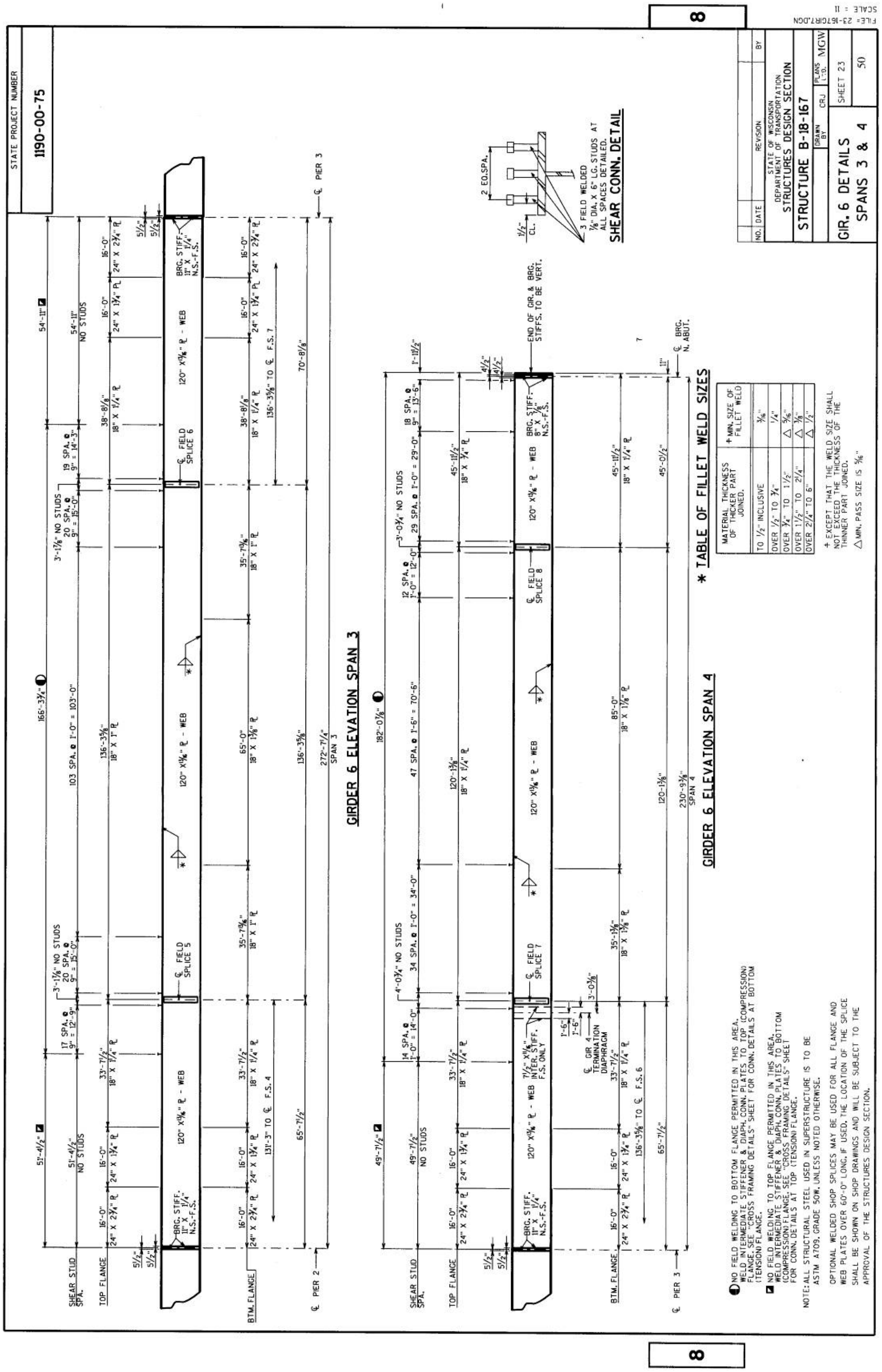
SHEET 22

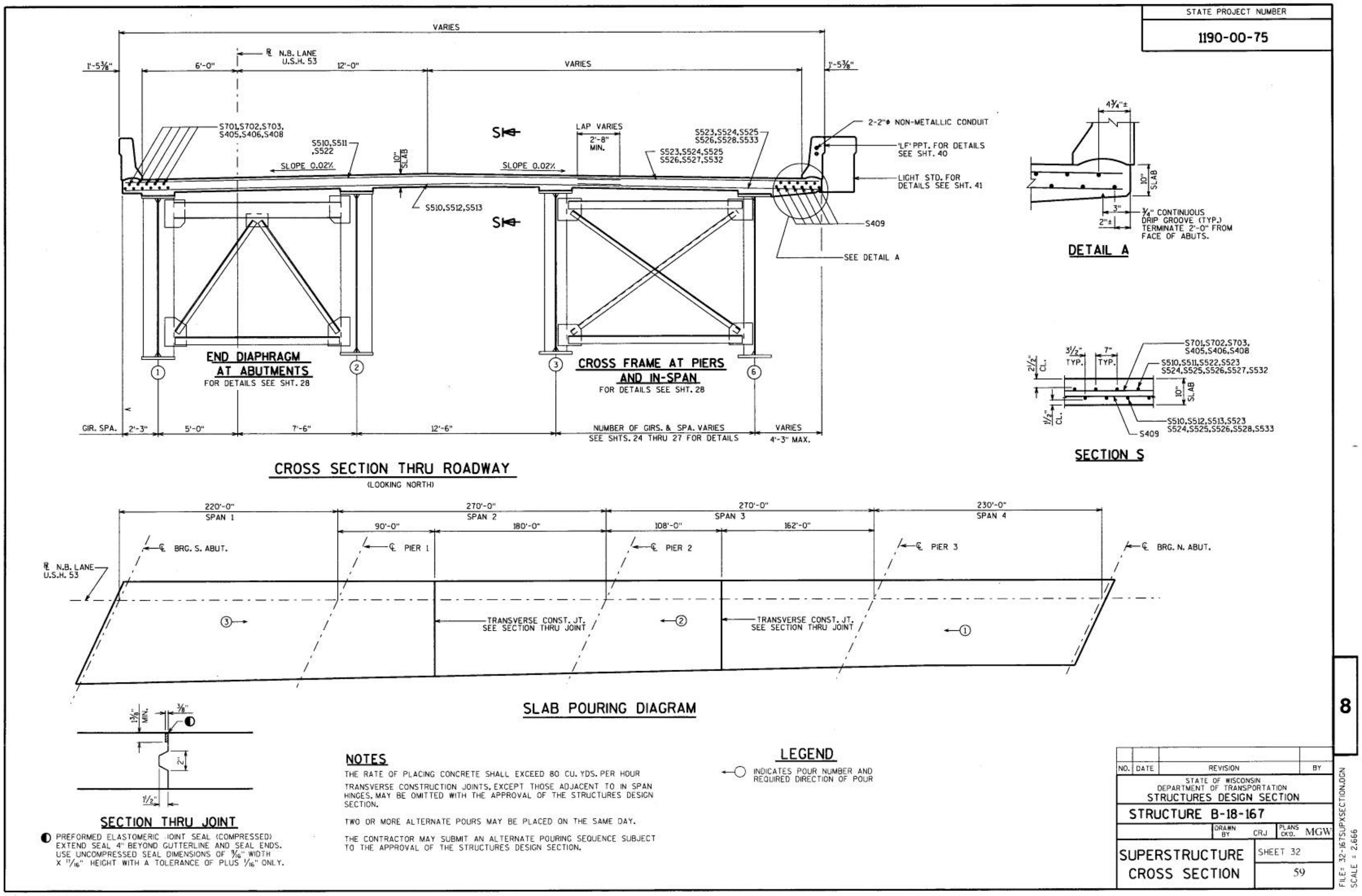
49

FILE: 22-167GIR6.DGN
SCALE = 11

8

8


STATE PROJECT NUMBER
1190-00-75
GIRDER 6 ELEVATION SPAN 3
GIRDER 6 ELEVATION SPAN 4
SHEAR CONN. DETAIL
* TABLE OF FILLET WELD SIZES
STRUCTURE B-18-167
GIR. 6 DETAILS
SPANS 3 & 4
SHEET 23
8


STATE PROJECT NUMBER
1190-00-75
CROSS SECTION THRU ROADWAY
(LOOKING NORTH)
VARIES
℄ N.B. LANE U.S.H. 53
1'-5⅜"
6'-0"
12'-0"
S701,S702,S703, S405,S406,S408
S510,S511 ,S522
SLOPE 0.02%
10" SLAB
S510,S512,S513
LAP VARIES 2'-8" MIN.
S523,S524,S525 S526,S528,S533
S523,S524,S525 S526,S527,S532
2-2"φ NON-METALLIC CONDUIT
'LF' PPT. FOR DETAILS SEE SHT. 40
LIGHT STD. FOR DETAILS SEE SHT. 41
S409
SEE DETAIL A
END DIAPHRAGM AT ABUTMENTS
FOR DETAILS SEE SHT. 28
CROSS FRAME AT PIERS AND IN-SPAN
FOR DETAILS SEE SHT. 28
GIR. SPA.
2'-3"
5'-0"
7'-6"
12'-6"
NUMBER OF GIRS. & SPA. VARIES SEE SHTS. 24 THRU 27 FOR DETAILS
VARIES 4'-3" MAX.
DETAIL A
4¾"±
3"
2"±
¾" CONTINUOUS DRIP GROOVE (TYP.) TERMINATE 2'-0" FROM FACE OF ABUTS.
SECTION S
3½" TYP.
7" TYP.
2½" CL.
1½" CL.
S701,S702,S703, S405,S406,S408
S510,S511,S522,S523 S524,S525,S526,S527,S532
S510,S512,S513,S523 S524,S525,S526,S528,S533
SLAB POURING DIAGRAM
220'-0" SPAN 1
270'-0" SPAN 2
270'-0" SPAN 3
230'-0" SPAN 4
90'-0"
180'-0"
108'-0"
162'-0"
℄ BRG. S. ABUT.
℄ PIER 1
℄ PIER 2
℄ PIER 3
℄ BRG. N. ABUT.
℄ N.B. LANE U.S.H. 53
TRANSVERSE CONST. JT. SEE SECTION THRU JOINT
SECTION THRU JOINT
1⅜" MIN.
⅜"
½"
PREFORMED ELASTOMERIC JOINT SEAL (COMPRESSED) EXTEND SEAL 4" BEYOND GUTTERLINE AND SEAL ENDS. USE UNCOMPRESSED SEAL DIMENSIONS OF ⅜" WIDTH X 11/16" HEIGHT WITH A TOLERANCE OF PLUS 1/16" ONLY.
NOTES
THE RATE OF PLACING CONCRETE SHALL EXCEED 80 CU. YDS. PER HOUR
TRANSVERSE CONSTRUCTION JOINTS, EXCEPT THOSE ADJACENT TO IN SPAN HINGES, MAY BE OMITTED WITH THE APPROVAL OF THE STRUCTURES DESIGN SECTION.
TWO OR MORE ALTERNATE POURS MAY BE PLACED ON THE SAME DAY.
THE CONTRACTOR MAY SUBMIT AN ALTERNATE POURING SEQUENCE SUBJECT TO THE APPROVAL OF THE STRUCTURES DESIGN SECTION.
LEGEND
INDICATES POUR NUMBER AND REQUIRED DIRECTION OF POUR
NO. DATE REVISION BY
STATE OF WISCONSIN DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN SECTION
STRUCTURE B-18-167
DRAWN BY CRJ
PLANS CK'D. MGW
SUPERSTRUCTURE CROSS SECTION
SHEET 32
59
FILE: 32-167SUPXSECTION.DGN
SCALE = 2.666
8

CFIRE

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Department of Civil and Environmental Engineering
1410 Engineering Drive, Room 270
Madison, WI 53706
Phone: 608-263-3175
Fax: 608-263-2512
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