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16. Abstract

The report documents an economic design study to determine the best alternatives for rapid and economical replacement of bridges in rural locations. Two lane simple span bridges with spans up 100 feet were considered. A steel bridge using rolled W sections with either metal grid deck or a composite cast in place concrete deck was found to provide an economical solution. If speed on construction was important, the metal grid deck provides significant construction time savings. The cast in place concrete deck reduces the bridge cost, but may double or triple construction time. The study included evaluation of prestressed concrete girders, double T beams, and solid slab bridges. The prestressed girder with a cast in place slab provides the most economical solutions for longer spans. For shorter spans, less than 60 feet, the rolled beam steel bridge was the most economical. The concrete bridges require larger cranes for erection and trucks for transportation which makes them less suitable for rural bridges built by small local contractors. Details of the steel girder bridge design are included including a novel cross frame design and a detailed LRFD example.

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Economical and Rapid Construction Solutions for Replacement of Off System Bridges

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Products

This report contains P1 and P2, Draft Specifications and Presentation Material.

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CHAPTER 1 Introduction

1.1 Scope of Study

The purpose of this study was to evaluate effective design solutions for off-system bridge replacement done in a rapid, cost-effective, and functional manner. Off system bridges, bridges not on major highways are the bulk of the deficient bridges in the state's inventory. Most of these bridges are on small two lane roads which may not be paved. Many are located in the flood plane and therefore are likely to be completely submerged during their service life. Often when the bridge is closed for replacement, the resulting detour can be very long or in some cases there is not alternate route. Due to the long detours, speed of construction was considered to be a major goal in determining the best solution for these bridges in order to reduce the impact of construction upon trip time. The span ranges considered in this study were from 20 to 100 foot with a 26 foot bridge with specified by the sponsor. The sponsor asked that the guard rail and its mounting not be included in the design or in the determination of costs.

Simplicity of construction was also considered as a primary goal of a successful design. These bridges are typical put out for bid one at a time. The small size of the contracts and the remote construction sites precludes competitive bids from large contractors who may have access to large cranes and able to perform sophisticated field construction. If the contracts for the bridges could be bundled so that a series of similar bridges could be let under on contract, larger more efficient contractors maybe attracted to the project. However, the funding for each of these rural bridges comes from separate county funds which generally cannot be combined. Therefore the premise of the design study was that smaller less sophisticated contractors would generally be building the bridges one at a time.

1.2 Survey of Deficient Bridges

A list of Texas bridges near Austin in need of repair or replacement was provided by TxDOT. Eighteen sights from the list were visited. Ten bridge sites were in Austin and eight bridge sites were in Caldwell County, Texas. A detailed list of the bridge surveyed is given in see Tables 1 & 2. The bridges in Austin were all on paved roads and generally out of the flood plane. The bridges in Caldwell County were generally on two lane rural roads and most were in the flood plane of the river or stream they crossed. All of these bridges were slated for replacement either due the load rating or geometry. The Austin bridges were in relatively good shape and in most cases had geometric deficiencies. The Caldwell Bridges were all structurally deficient as well as being very narrow. One bridge was washed away from a flood.

The lengths of the bridges ranged from 20 ft to 900 ft. The bridges were made of different materials. In Austin, there were four bridges that were concrete box culverts. Three had rolled steel W- shapes, two were concrete arches and one bridge in Austin had prestressed concrete beams. In Caldwell County there were two timber bridges, three steel stringer bridges, one metal culvert bridge, one steel railroad car bridge, and one bridge that was completely washed out could not be identified.

Table 1.1   Table 1.1 Austin Bridges Surveyed

Bridge Location	Type	Length (ft)	Width (ft)	Spans	Comments
5th St at Shoal Creek	Arch Shaped Concrete Girders	109	63	3	
51st St at Tannehill Branch	Concrete Slab Bridge Concrete Slab Piers	41	52	4	
Barton Springs Rd at Barton Creek	Concrete Arch Concrete Deck Concrete Piers	255	60	3	4 Arches at Zilker Park
E 7th at Tillery St. and ANW RR	Steel Plate Girder Concrete Deck Rectangular Concrete Piers	900	60	17	
Lamar Blvd at Shoal Creek	Prestressed Concrete Girders Concrete Deck	117	60	3	Skew bridge, outer girders parallel to skew, inner girders perpendicular to piers
Manor Rd at Boggy Creek	Concrete Slab Bridge Concrete Slab Pier	24	50	2	
Mt Bonnell Rd at Dry Creek	Concrete Slab Bridge Concrete Pier	30	26	2	
Old Manor Rd at Tannehill Branch	Steel I-Girders Concrete Deck Concrete Abutments	53	25	1	9 Girders
Red Bud Trail at Colorado River	Steel I-Girders Concrete Deck Concrete Slab Piers	152	29	3	5 Girders
S 1st St at Boulding Creek	Concrete Slab Bridge Concrete Abutments	20	60	1	Abutments Built into rock

Table 1.2 Table 1.2 Caldwell County Bridge Sites

Caldwell County Bridge Sites	Type	Length (ft)	Width (ft)	Spans	Comments
CR 108 at Boggy Creek	Steel I-Girder Bridge Timber Deck Concrete Abutments	41	16	1	10-W12 Shape Girders
CR 176 at Cedar Creek	Timber Girder Bridge Timber Deck Timber Piers	70	20	4	
CR 222 at Cowpen Creek	Culvert 5 Steel Pipes		14		
CR 223 at Elm Creek	Rail Car Bridge Steel Plate Deck Steel Piers	33	15	2	
CR 230 at Boggy Creek	Steel Girder Bridge Corrugated Metal Deck Concrete Abutments	37	18	1	Ex. Girder: Channel Steel, Int. Girder: I-beam
CR 240 at San Marcos River	Timber Girder Bridge Timber Deck with paving, Timber Piers	73	14	1	Bridge only crosses small portion of the flood plain
CR 247 at San Marcos River N	Hollow Steel Tube Piers Concrete Abutments Bridge Type unknown	60 (Approx.)	12 (Approx.)	3	Bridge has been washed away by river.
CR 262 at San Marcos River	Steel I Girder Bridge Timber Deck Concrete/Masonry Abutments	31	11.5	1	Waterway width shortened under bridge causing rapids

The bridges in Caldwell Country were felt to be more typical of the off system bridges requiring replacement. Hence, these county bridges were used to focus upon the type of sites, geometries, and substructures that would be encountered in other rural counties. The conditions observed during our visit to the Caldwell County bridge sites is summarize in the following sections.

1.3 Bridge Problems

The bridges had several levels of replacement urgency. Some bridges have no obvious damage, or small cracking only, leading to a suspicion that the main problem is serviceability. Others have serious damage such as holes through decks and cracked abutments. The bridge washed out in a flood presented the most extreme urgency for replacement. Almost all of the bridges were posted with a low load rating. These low rating can cause considerable economic hardship since commerce must be routed to other roads.

In Caldwell County, serviceability was not the major problem with the bridges. The bridge at County Road 108 at Boggy Creek had an allowable axle or tandem load of 10 kips, this low rating was probably due to a crack of approximately one-inch width through the center of the abutment as shown in Figure 1.1. The 40 foot span bridge has a one lane timber deck supported on 10 W12 steel stringers. The bridge had no guard rail and no available detour. The timber deck appeared to be in good condition as did the steel stringers.

Figure1.1 Bridge at County Road 108 and Boggy Creek

The bridge at County Road 230 at Boggy Creek has an allowable axle or tandem load of only 5 kips. The bridge is shown in Figure 1.2. The bridge had steel W shape interior stringers and exterior C shape stringers. The deck consisted of corrugated steel topped with asphalt. It had several holes through its deck, the largest was about four inches in diameter. Figure 1.3 shows the underside of the bridge deck revealing a line of corrosion along the length of the deck. The 4 inch hole in the deck was along this line. The cause for this localized corrosion was not evident. It may have occurred from leakage of a corrosive product from a vehicle passing over the bridge. This type of hidden corrosion under the asphalt wearing surface is an inherent weakness of the type of deck system. At this site, some drivers used a small dirt path around the bridge instead of traveling over it. A large truck with an axle weighing 22 kips would exceed this load rating by more than a factor of four.

Figure 1.2 Bridge at County Road 230 and Boggy Creek

Figure 1.3 Corrosion in Bridge Deck

Another factor evident at many of the bridge sites is the location of the bridges within the flood plane. Many of the bridges in Caldwell County that were surveyed were under water during a recent flood. The wide flood planes would have required bridges lengths of the order of 500 feet to reach an elevation above the flood plane. Figures 1.4 and 1.5 are photos of two bridges that were underwater in the recent flood. The height of flood water determined by the debris in the adjacent trees is indicated by an arrow. Due to the wide flood planes where these bridges are located, it would be prudent to design the bridges to survive when water rises above them.

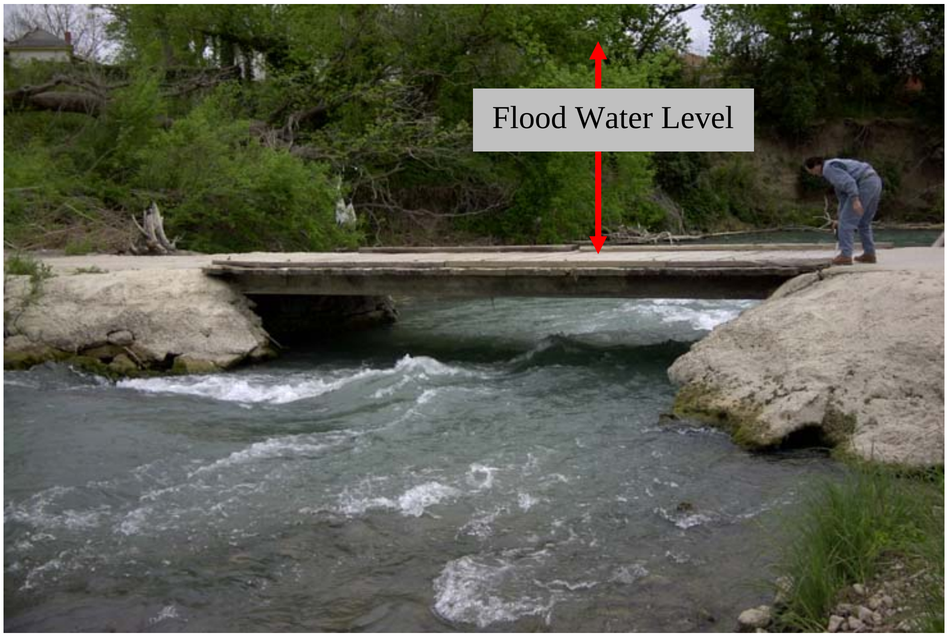


Figure 1.4 Flood Water Level at Bridge Site

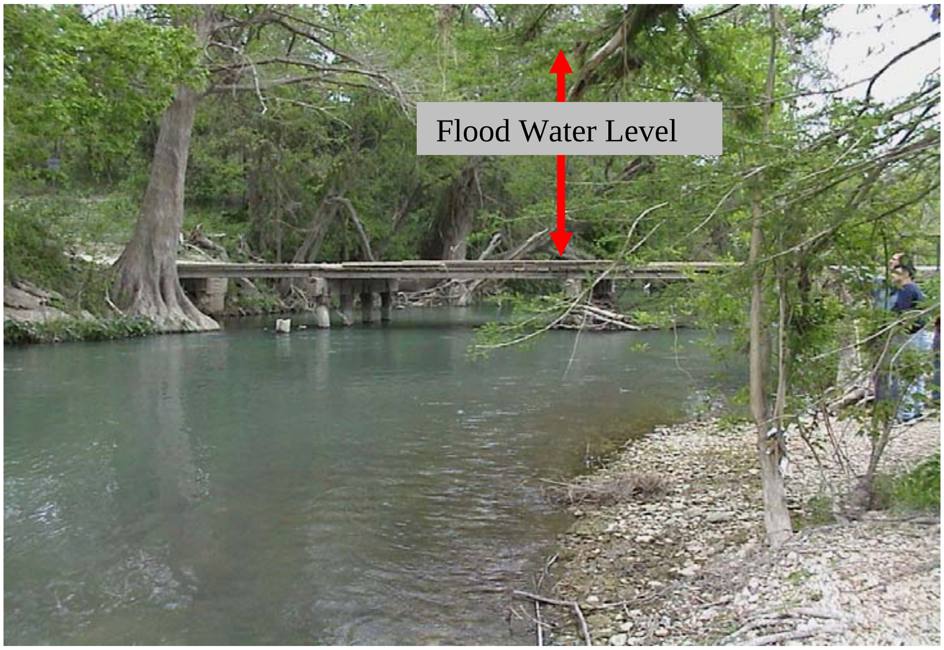


Figure 1.5 Flood Water Level at Bridge Site

Another serviceability problem found mostly in Caldwell County was a lack of effective guardrails. Some bridges had non-load bearing, decorative, guardrails while others had no guardrails. At CR 222 at Cowpen Creek in Figure 1.6 for instance, a culvert had been recently constructed, but the guardrails were made of thin metal tubing that stood approximately one foot

Figure 1.6 New Guard Rail Installed at Culvert

above the ground. The same type of guardrail was seen at CR 223 at Elm Creek. This type of guardrail would not be able to contain a vehicle on the roadway if hit by a vehicle traveling the speed limit,. The guardrail at CR 176 at Cedar Creek shown in Figure 1.7 was made of timber and had broken off in several places. Some of the bridges that have no guardrails at all are CR 108 at Boggy Creek, CR 230 at Boggy Creek, CR 240 at San Marcos River, and CR 262 at San Marcos River.

Figure 1.7 Broken Wooden Guard Rail

CHAPTER 2 Selection of Bridge Decks

A wide ranging literature survey and discussions among bridge owners was undertaken to determine possible bridge types and deck systems. Based upon the survey of bridges and viewing the level of construction and maintenance evident, high tech solutions such as FRP decks were eliminated from consideration. They very expensive at an estimated $45 per square foot and are not available in the small quantities required for these short and relatively narrow bridges. They are also a product that typical county maintenance personnel would be familiar with.

Corrugated metal decks were also removed from further consideration due to the corrosion found in these decks. The bridge on CR230 at Boggy Creek had a corrugated metal deck that was completely rusted through in some places. Water can collect on the corrugated metal in places where asphalt is eroded due to improper maintenance leading to hidden corrosion under the wearing surface. Bridge engineers from other states expressed corrosion concerns and observations the poor service performance of this deck system. This type of deck appeared at first to be a good choice due to its low cost and simple paving with asphalt. However, due to the potential short life service life from development of hidden corrosion it was not considered further.

Full width full depth precast concrete decks were carefully considered. The have the advantage of rapid construction since they eliminate the forming and curing stages associated with a cast-in-place deck. However, the small quantities involved in these bridges would make it uneconomical for a producer to setup a casting and prestressing bed. The sophistication required to produce decks made it unlikely that smaller rural plants could be setup economically. If a large number of bridges with the same deck width could be put together into one bid package, these types of decks might be viable. One other concern raised was the ability to fit these precast elements to stringers with varying cambers. For the one off bridges that are typical for counties, precast deck panels were not considered to provide a economical or realistic solution.

Cast-in-place conventional concrete decks provide one of the cheapest and durable deck systems. Their drawback is the time required for forming, casting, and curing the deck. These operations can add a week or two to the construction schedule. A cast-in-place deck was left in the matrix of possible decks due to its low cost and simplicity of construction. The use of precast concrete forms between the girders speeds up the construction, reduces the amount of concrete to be placed, and was the method of forming considered in the trial designs for cast-in-place decks.

Timber or wooden decks were not considered at the direction of the sponsor. These are considered by the public as inferior and have poor skid resistance. They are simple fabricate and install. The timber decks that we observed in our survey were in good shape and appeared to be holding up well. The cost of the lumber is increasing consequently they may not be an economical alternative. However, their simplicity and durability make still make them suitable for rural bridges on very low volume roads.

Metal grid decks were investigated and found to have some desirable characteristics. They require no forming or curing time. They are relatively light and therefore they do not require large lifting equipment. They have an open section which allows air to escape from between the girders in a flood. The air trapped between the girders with a solid deck increases the buoyancy of the bridge and increases the chances of the bridge being washed away in a flood. The down side to these decks is their high cost and reduced skid resistance. They were left in the matrix of decks to be considered due to their ability to be rapidly constructed. Detailed discussion of grid decks is given in the next section.

2.1 Steel Grid Deck

The first steel grid deck was constructed in the 1930's on the Oakland Bay Bridge. This grid deck was a grid reinforced concrete bridge deck. Its purpose was to provide a strong, lightweight deck compared to other alternatives of that time. Both grid reinforced concrete decks and open steel grid decks have been used on many bridges in the Eastern United States. They are usually used for situations where rapid deck replacement is an important factor. They are also commonly used when a lightweight deck is necessary, such as in bascule bridges or other moveable bridges. Many companies produce steel grid decks. Two particular grid deck companies, L.B. Foster and American Grid, provided the grid deck data used in this report. These companies contributed information such as load capacities, support spacing requirements, deck dimensions, and deck weight.

2.1.1 Types of Steel Grid Decks

There are two main types of steel grid decks. They are grid reinforced concrete decks (Figure 2.1) and open grid decks (Figure 2.3). Grid reinforced decks consist of a steel grid filled with concrete at either half of its depth or full depth. An open grid deck is a steel grid with no concrete fill. Steel grid decks come in three different structural configurations, four-way, two-way, and riveted. Four-way (Figures 2.1 and 2.3) decks consist of rolled main beams and smaller secondary members at 45, 90, and 135 degrees with respect to the main bars. Two-way decks (Figure 2.2) have main longitudinal members with secondary bars in the perpendicular direction only. In both four-way and two-way decks the members have welded connections. Riveted steel decks (Figure 2.4) resemble the four-way grid deck system, but have riveted connections instead of welded connections.

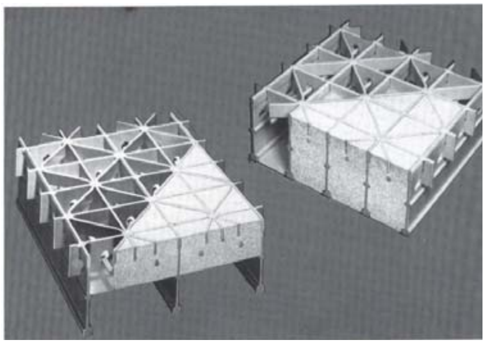


Figure 2.1 Four-Way Filled Grid

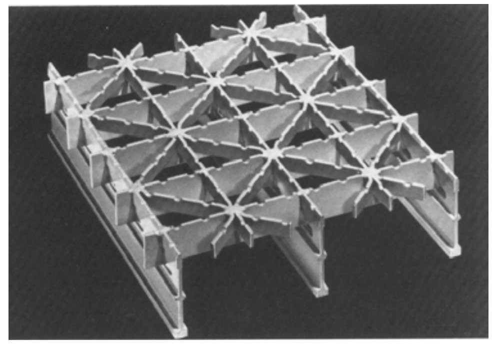


Figure 2.3 Four-Way Open Grid

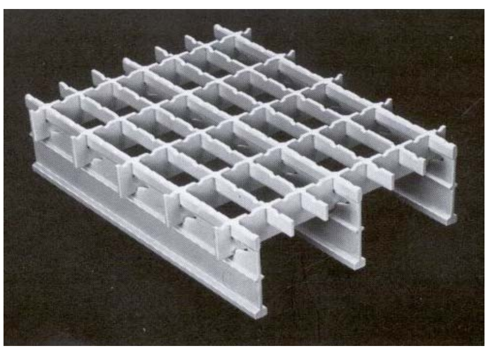


Figure 2.2 Two-Way Open Grid

Figure 2. 4 Riveted Deck

2.1.2 Cost of Grid Decks

The cost of the various steel grid deck options was evaluated. The cost of an open two-way deck was quoted as approximately $27 per square ft, while the cost of an open four-way system was approximately $32 per square ft. The open two-way system can facilitate girder spacing up to 7.85 ft and the open four-way system can facilitate a girder spacing of 9.61 ft. As seen in 4, a four-girder bridge would require a girder spacing of 6.5 ft, while a three-girder bridge would require a girder spacing of 8.7 ft. The two-way grid system could be used with a four-girder bridge, but the three-girder bridge would require a four-way grid. An estimated cost was developed to compare the prices of a four-girder bridge with a two-way open grid deck and a three-girder bridge with a four-way open grid deck. The results are shown in Figure 2.5.

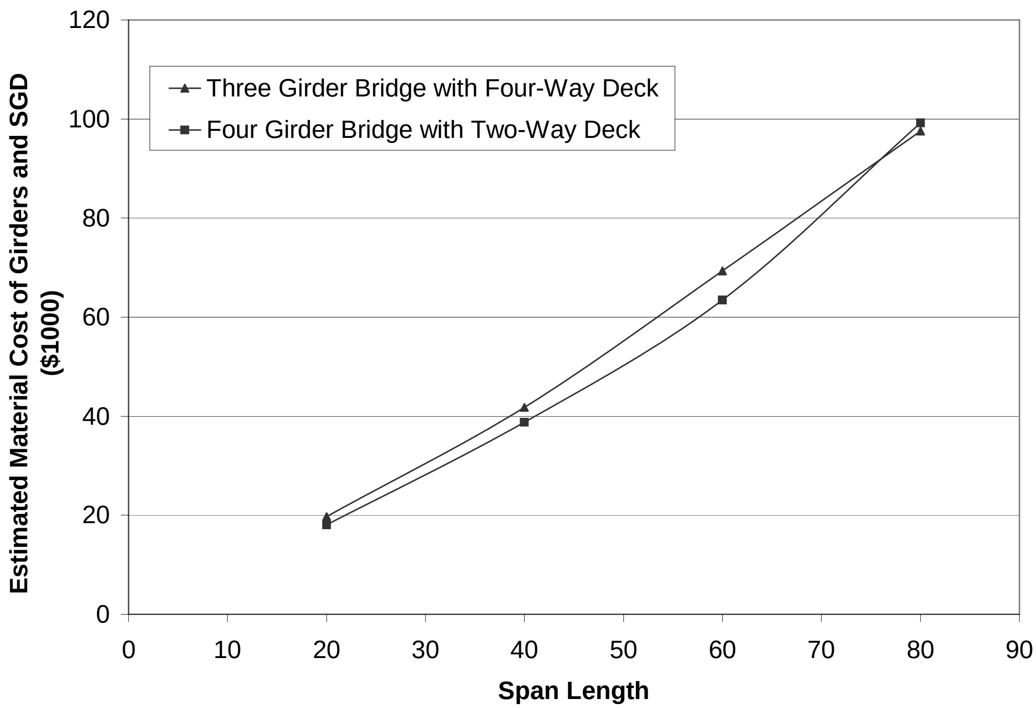


Figure 2.5 Cost Comparisons for Three-Girder Bridge with Four-Way Open Grid Deck and Four-Girder Bridge with Two-Way Open Grid Deck

Figure 2.5 is based on plate girder bridges designed for an HS-20 truck live load using the AISI bridge design program. The plate girder price used in this estimation was $0.62 per lb. The chart shows that for the majority of the bridge lengths, the four-girder bridge system with a two-way deck is less expensive than a three-girder system with a four-way deck. Therefore, the two-way system was chosen as the design steel grid deck.

The weights of these grid decks did not play an import role in deciding which type is more suitable. The weight of the two-way open grid deck system that could facilitate the design bridge was 23.4 psf, while the weight of the four-way system was 25.75 psf. The weight difference was only 2.35 psf and considered negligible.

2.1.3 Grid Deck Selection

The steel grid deck chosen for this project was the 5 in. RB open deck manufactured by the L.B. Foster Company. This deck is shown in Figure 2.2, and is available in A36 steel (F_y = 36 ksi) or A588 steel (F_y = 50 ksi). The main rolled beam depth is equal to the deck depth, which is 5 in. The main 5 in. deep members are placed transverse to traffic so that the secondary members are in the longitudinal direction. The main rolled beams can be spaced at 3, 4, 6, or 8 inches with secondary members at every 2 inches. The transverse members are spaced at 4 inches. With girders spaced at 6.5 ft, the deck chosen for this project was the 5 inch RB with main bars spaced at 4 inches and the steel is grade A588. Grade A588 steel was chosen because it is stronger than A36 steel and because it is weathering steel and require less maintenance. The section moduli for this bridge are 5.124 in^3/ft for the top steel and 5.993 in^3/ft for the bottom steel. This type of deck can have a clearance between supports of up to 7.75 ft. The next strongest 5 in. RB deck has the main bars spaced every 6 in. This deck had a clear span capacity of 6.85 ft. The deck with 4 inch bar spacing was used in the designs because it could facilitate the range of spacings being initially evaluated. The slightly lighter deck with 6 in. spacing could have been used with the final 4 girder designs.

2.2 Precast Concrete Panels with Cast-in-Place Concrete Topping

Most bridges in the United States have cast-in-place (CIP) concrete deck systems. One of the most popular systems uses precast SIP (Stay In Place) prestressed concrete deck panels for the formwork. This deck consists of panels of 3 to 4 in. in depth that functions as forms for the CIP concrete topping. The precast panels also house the positive moment reinforcing steel. The panels are butted against each other in the longitudinal direction with no continuity between them. This system is advantageous because it has a higher construction speed than a full depth CIP concrete deck. This is because of the elimination of field forming between the girders and the reduction in the amount of concrete placed in the field. The price of this deck system is also attractive as it is approximately $8 per square ft. The panel cost is $3 per square ft., and the CIP concrete cost is $5 per square ft. The bridge weight is approximately 106 psf.

2.2.1 Forming Deck Overhang

The configuration of the stinger bridge design results in a portion of the deck beyond the fascia stringers. Formwork will be required for the overhangs. The SIP panels can only be used between the girders because they must be supported at each end. The panels cannot develop any moment because they simply lay on top of the girders. Since SIP panels cannot be used in the overhangs, CIP concrete must be used. Standard removable forms must be used to form the CIP concrete. The need for standard forms is detrimental. Standard forms will require additional costs and construction time. The additional construction time will be needed to place and remove the forms.

2.2.2 Deck Design

The design concrete deck selected was 8.5 in. thick. Standard 4 inch panels where chosen. The CIP portion of the deck should be 4.5 in. thick except in the overhangs and on top of the girders where it will be 8.5 in. thick. Figure 2.6 is a sketch of a transverse section of this deck system. Space is left open on top of the girders so that the CIP concrete can bond to the precast panels and provide space for shear connectors. It is important for the CIP concrete to bond to the precast panels on all sides to unify the deck system. The CIP concrete also forms a bond with the shear studs in the area above the girders.

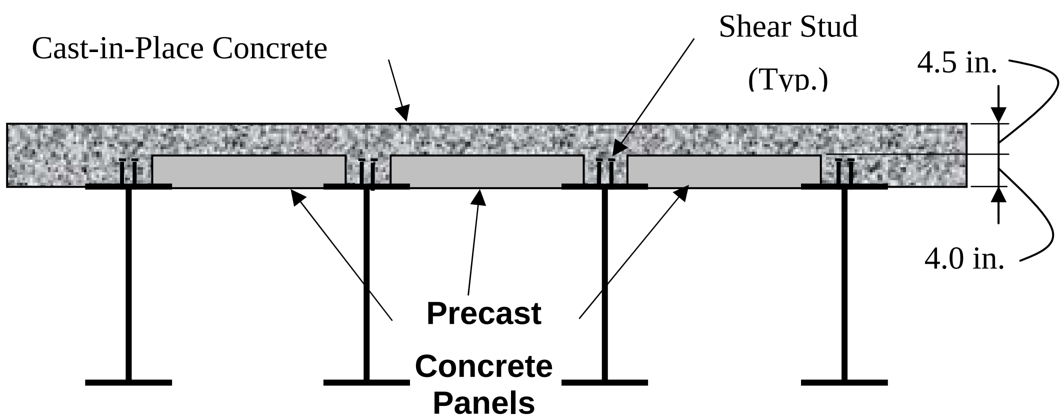


Figure 2.6 Partial Precast Deck and Shear Studs

2.3 Advantages and disadvantages of the deck systems

The most important advantages and disadvantages of the deck systems are related to construction speed and cost. The construction speed is an advantage for the steel grid deck because it will only take one day to place it. The SIP panel form system will require approximately four days to place the panels and removable forms, pour the concrete, cure the concrete, and remove the forms. In a typical construction project, four days is only a fraction of the total construction time. However, since the goal for the construction duration for this project was approximately one week, four days of construction makes is significant.

The price to speed up the construction process is high. At $27 per square ft., the steel grid costs over three times what the deck constructed with SIP forms cost. When the bridge length is 80 ft, the steel grid deck costs approximately $40,000 more than the concrete deck with precast panels. The owner must decide if the short construction time of the steel grid deck is worth this cost difference.

Chapter 3 Bridge Structural Geometry

Based upon our inspection of the bridge sites and discussions with the sponsor we restricted our bridge design to conventional stringer bridge designs. This decision was made to meet the goal of a bridge system that was easy and rapid to construct using equipment that a small contractor would have. The basic stringer bridge is shown below in Figure 3.1. Note steel stringer are shown but the same configuration was used for prestressed concrete stringers.

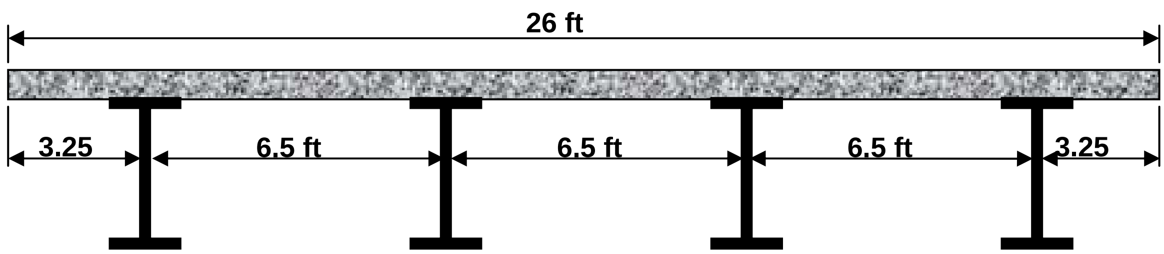


Figure 3.1 Girder Layout

3.1 Girder Layout Investigation

The four girder layout came from preliminary designs which considered 3, 4, and 5 stringers. The 26 ft deck width specified by TxDOT was to accommodate two traffic lanes. The number of girders for this two-lane bridge was optimized to develop a least cost solution. This influence of girder spacing upon bridge cost and weight was done using the AISI Bridge Design Software. The AISI software produces an optimum design using the Load Factor Design method. Three, four, and five stringer bridges using grade 50 steel and both rolled and built up plate girders were designed for spans from 20 to 80 foot. Fewer girders can save cost by allowing a deeper section and higher section modulus per girder, using less total steel. When the projected construction time is a matter of days, reducing the number of girders that need to be placed can also save a great amount of time.

However, there are a few problems with using a small number of girders. A sufficient number of girders must be used to provide adequate redundancy. Using only two girders would create a problem because if one of them failed the whole bridge would immediately collapse. Also, using fewer girders requires greater spacing between the girders. If girders are set at too wide of spacing, deck strengths and thickness must be increased. The last problem is that if the girders are too deep they may be heavy and require more expensive equipment to be lifted into place during construction.

Girder weight is important for material cost reasons as well. Steel girders are sold at a per pound price, so a lighter bridge is usually a less expensive bridge. To optimize the bridge economy, six different girder configurations were compared. They were rolled beam and plate girder bridges each using three, four and five girders.

Figures 3.2 and 3.3 compare total girder weight, the sum of all the girder weights, of three, four, and five- girder bridges versus span length for both rolled beams and plate girders, respectively. Both graphs show that the total weight of the three-girder bridge alternative is the lightest. The difference due to girder spacing is quite small for rolled beams and much larger for the plate girder sections which can be tailored to provide a very efficient design. However, a three-girder system supporting a 26 ft wide deck would require a girder spacing of over 8 ft with an overhang of 5 ft. This girder spacing would require a more expensive and heavier deck than the four-girder bridge.

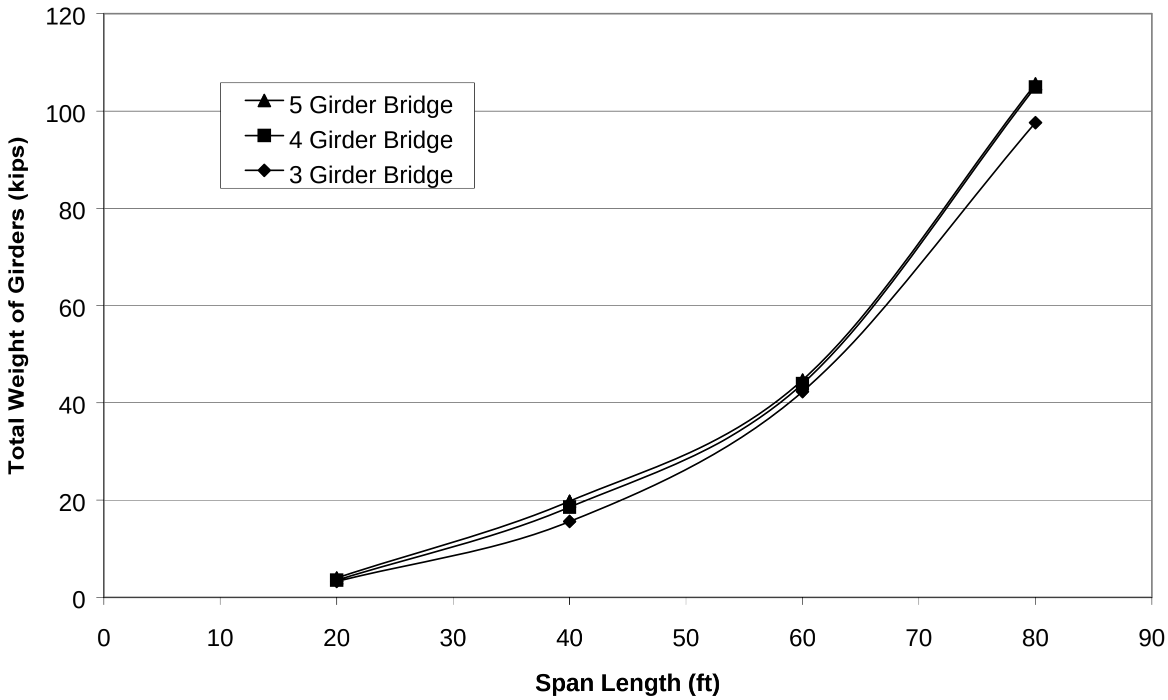


Figure 3.2 Total Girder Weight versus Span Length for Rolled Beams

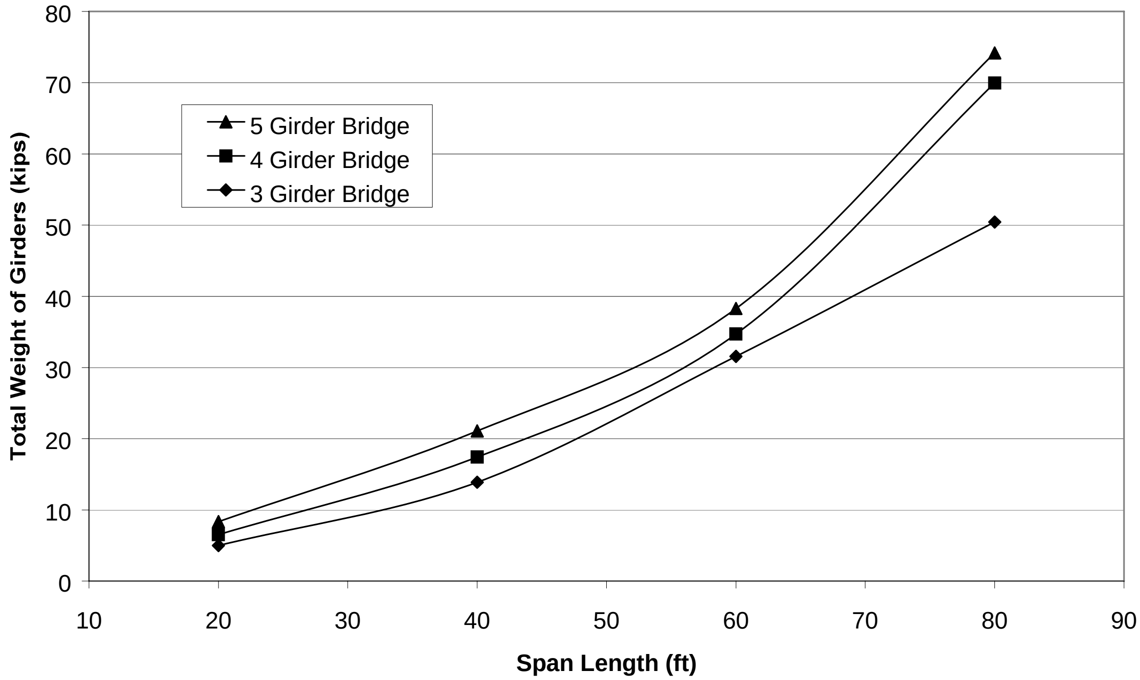


Figure 3.3 Total Girder Weight versus Span Length for Plate Girders

Although it is not the lightest, the most appropriate number of girders to use for rapid bridge replacement is four. A four-girder bridge with a 26 ft wide deck requires a girder spacing of approximately 6.5 ft. A four-girder bridge is the lightest design within the range of the spacing requirements.

Three different girder spacings were designed to find the one that would optimize girder size for the four-girder bridge. The spacings were 6 ft, 6-½ ft, and 7 ft with overhangs of 4 ft, 3-¼ ft, and 2-½ ft respectively. The girder spacings between six and seven feet do not affect the weight of the girders. However, even though all three spacings were found equally efficient, the girder spacing that was chosen was 6-½ ft with a 3-¼ ft overhang. With this particular spacing, all four girders have a tributary width of 6-½ ft. This spacing simplifies construction because each of the four girders carries approximately the same dead load and can be the same sized for the same loading ignoring the weight of a guardrail.

3.2 Precast Double T Beam Layout

Precast double T beams were selected for evaluation for shorter spans. These have been used successfully in parts of Texas near precast plants setup to produce these sections in sizes suitable for bridges. The advantage of these sections is the top flange of the T serves as the form work for the cast-in-place topping slab. The elimination of extensive formwork and the reduction in the amount concrete to be placed .should reduce cost and speed construction. The time required for curing however would be comparable to that of the cast-in-place deck on a stringer bridge.

The T beam layout developed of the two lane road is shown below in Figure 3.4. The available sizes of double T's limit the span of these bridges to 60 feet. The disadvantage for this type of bridge is the weight of the sections. They require large trucks and cranes to transport and place the girders.

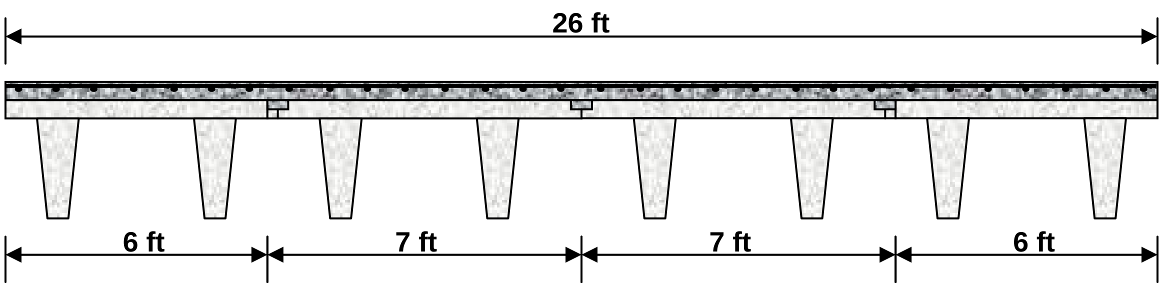


Figure 3.4 Double T Beam Bridge Layout

Chapter 4 Preliminary Cost Estimates

Preliminary designs were made to compare the relative costs of alternatives. The span range was from 20 to 90 feet. The costs estimates included the material costs and an estimated addition for placement in the field. Only the superstructure cost was included in this study. The unit costs used to estimate bridge cost are shown in the Table 4.1.

Table 1.3 Table 4.1 Unit Costs

Item	Price
CIP Concrete Deck	$10 per sf
Prestressed Concrete Deck Panels	$3 per sf
CIP Concrete Topping (for Precast Panels)	$5 per sf
Prestressed Concrete Beam (TY A, B, C, IV)	$60 per lf
Prestressed Concrete Slab Beam	$85 per lf
Prestressed Concrete T-Beam	$125 per lf
CIP T-Beam Topping	$5 per sf
Steel Grid Deck	$27 per sf
Rolled Steel Beam (Sections Smaller Than 30 in.)	$0.35 per lb
Rolled Steel Beam (Sections 30 in. or Larger)	$0.50 per lb
Steel Plate Girders	$0.70 per lb

The steel girder designs were done using the AISI short span program. Both rolled and welded built up steel plate girder sections were designed for each span length. A cost survey of rolled sections was undertaken. At the time of this study only two steel mills in the U.S. were producing W shapes. The cost per pound for the shapes appeared to depend upon the competition. If only one mill was producing a particular shape the cost was higher. Multiple mills produced the smaller shapes which reduced their unit costs. These cost differences are reflected in the table. The break in cost occurred at the 30 in. beam depth. The cost should for the steel stringers include fabrication costs. The steel designs with a steel grid deck are non-composite and composite when a cast-in-place deck is used. Composite bridges were designed and the cost compared with non-composite. The composite design required smaller and lighter girders. The cost added cost of the shear studs and additional bracing required during construction for the smaller composite girders somewhat reduces their economic advantage. In order to have a fair comparison with the prestressed girder bridges which are composite, the steel bridges used in the cost comparison with a concrete deck are also designed as composite. The prestressed girder and double T designs were done using TxDOT design aids and are composite designs.

In order to provide a basis for comparison, the cost for a truss bridge from U.S. Bridge for 40 and 60 foot spans with metal grid deck and a concrete deck were included for comparison. The U.S. Bridge costs include the engineering cost but do not include shipping or erection costs.

The Figure 4.1 shows a plot of estimated superstructure cost versus span length for the various alternatives. The lowest cost bridge for spans above 60 foot is the prestressed girder with the cast-in-place deck. The low cost of this design is not unexpected. It mirrors the reality of current bridge construction in the state. The lower cost of the steel bridge with a cast-in-place concrete deck for the shorter span was a surprise. The high cost of the steel grid deck increases the cost of the steel bridge as well as the bridges from U.S. Bridge. The double T beams are not competitive with the alternatives. Prestressed slab bridges provide cost comparable with prestressed girders for spans up to 40 feet.

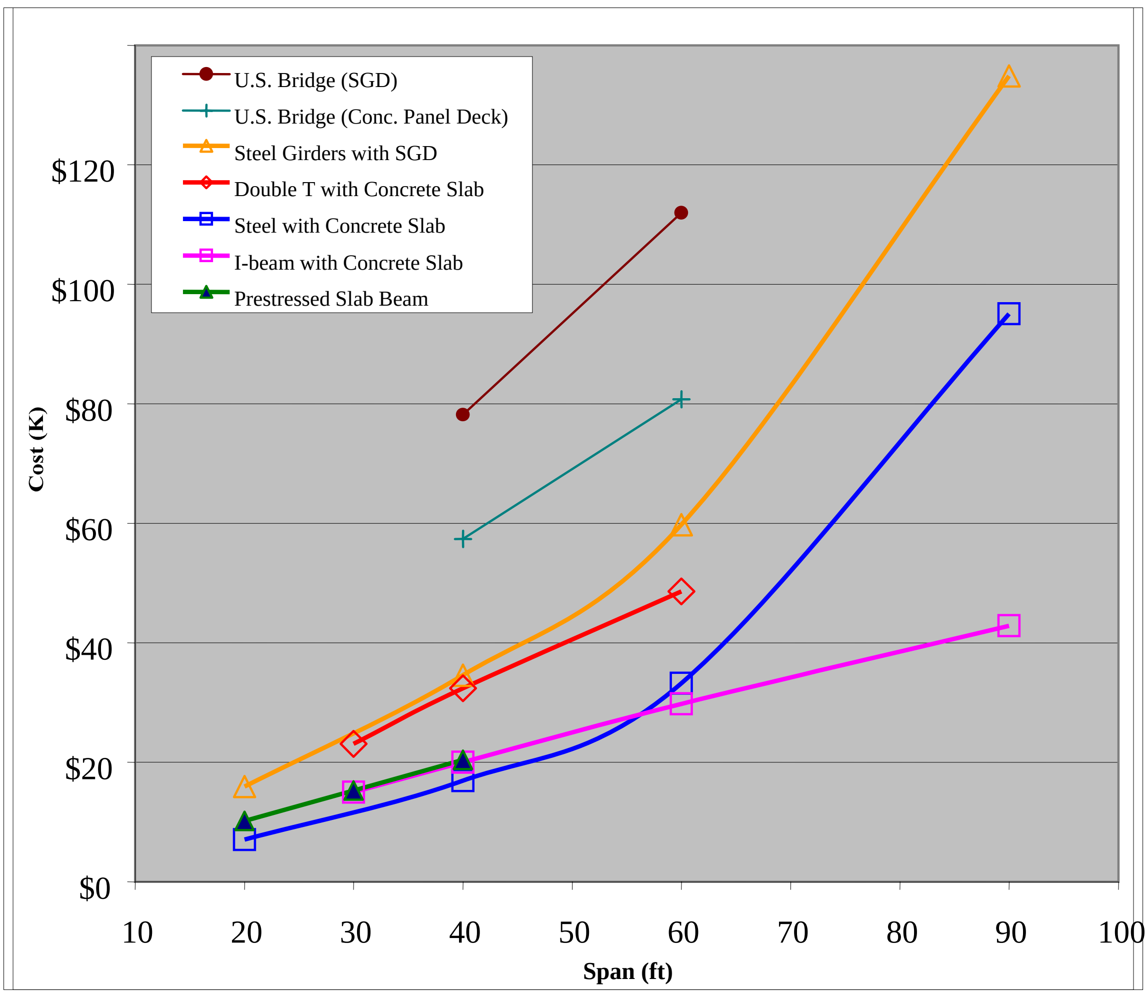


Figure 4.1 Total Estimated Bridge Costs

Based upon this economic study and the input from the sponsor, the design of the steel girder bridge with a open grid deck was studied in more detail to develop refined designs. The prestressed girder bridge with a cast-in-place deck is a standard TxDOT design and does not require further study. It is the least cost alternative for longer spans is the best choice when construction time is not critical and large cranes and trucks are available.

Chapter 5 Steel Bridge Design

5.1 AISI Bridge Design Program

The design of the steel girders was done using the AISI Short Span Steel Bridge Software. The program allowed many different bridge configurations to be analyzed in a short period of time. The designs presented in this chapter are based upon the AASHTO Load Factor Design. A design using LRFD specification is given in Appendix A.

5.2 Background of the Design Program

The American Iron and Steel Institute created this bridge design program in 1995. It is based on the Strength Design Method (Load Factor Design) of the AASHTO Standard Specifications for Highway Bridges. The software has two modes, design and rating. In the design mode the software finds the minimum weight solution by iterating between a range of minimum and maximum cross section dimensions specified by the user. In the rating mode, the user can input exact cross section properties and the software will solve for both an inventory and operating rating. For this project the design mode was used.

5.3 Capabilities of the Design Program

The bridges that the program designs are simply supported rolled wide-flanged shapes or welded plate girders. Data such as span length, deck width, girder type, design load, number of lanes, and number of girders had to be input into the program. Other data such as spacing of cross bracing, distribution factors, and impact factor can be input by the user or chosen by the program. The program chooses the lightest girder and calculates design loads, maximum allowable loads, shear forces, moments, and deflections throughout the bridge.

5.4 Rolled Beam Pricing

In some cases the program chose a rolled beam that was the lightest but not the least cost alternative. For example, when designing the 50 ft span non-composite bridge with a concrete deck, the section chosen by the AISI Program was a W40x149 with a section modulus equal to 512 in^3. Since this shape was only offered by one steel mill in the United States, the price per pound was estimated at $0.50. If it had been offered by more than one steel mill in the US, the price would have been estimated at $0.35 per pound. In cases like this, alternative shapes with equal or greater section moduli were chosen if they lessened the total price of the girder. In this particular case, a W36x160 with a section modulus of 542 in^3 was used because it is offered by Nucor-Yamato and TXI Chaparral. The price of a W36x160 is only $56.00 per ft. since the per-pound cost is $0.35. At $0.50 per pound, a W40x149 is $74.50 per linear ft. For a 50 ft span with four girders, using the W36x160 saves approximately $3,700 per bridge over the W40x149.

5.5 Bridge Loading

5.5.1 Live Load

Although many off-system bridges are in remote locations, most of them have large trucks and different types of heavy farm equipment traveling over them on a regular basis. An HS20 load was used in the AISI program input for the truck live load. The software defaults to an HS20 truck load as the design live load and fatigue vehicle. If the user does not want to use an HS configured load, he/she must provide a live load configuration. Section 3.6.1.3.1 of AASHTO LRFD Bridge Design Specifications (1998) states that the extreme force effect for the design vehicular live load shall be taken as the larger of the effect of the design tandem load combined with the design lane load or the design truck load combined with the design lane load. A design tandem consists of two 25 kip axles spaced at 4 ft (AASHTO 3.6.1.2.3).

Since the program defaults to an HS20 truck load as the design live load, it does not determine if the tandem load controls over the truck live load. However, the user can check the tandem load by entering a tandem configuration into the vehicular information screen in the AISI software. The chart in Figure 5.1 was developed to determine for which span lengths the tandem load controlled, and for which span lengths the HS20 truck live load controlled.

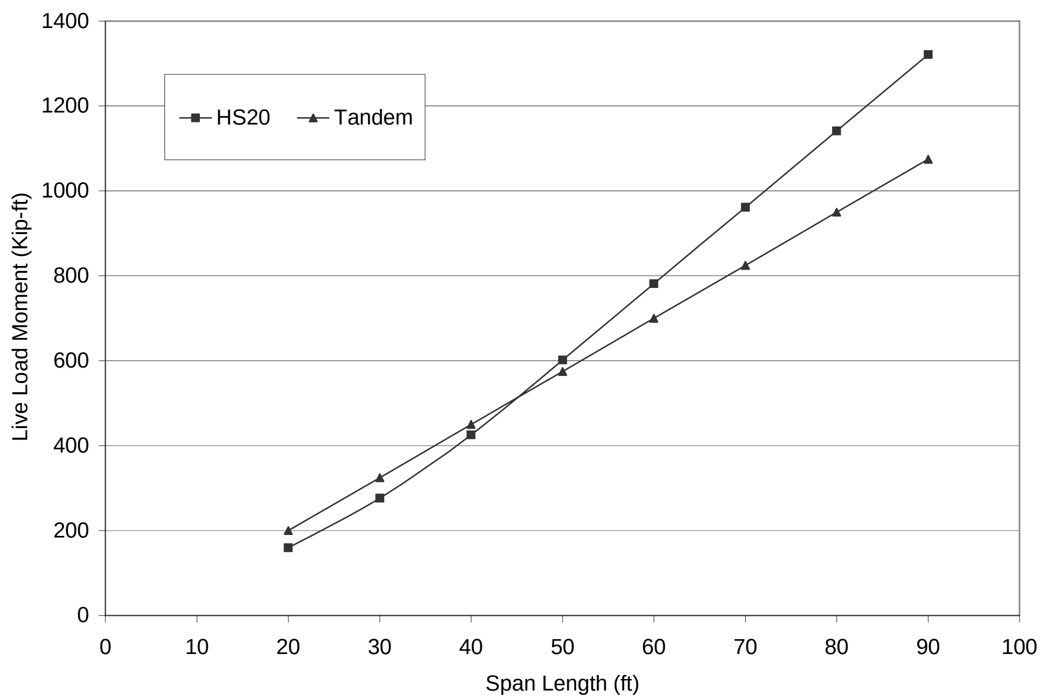


Figure 5. 1 Comparing Live Load Moments

As seen in Figure 5.1, the design tandem load controlled for spans ranging from 20 ft to approximately 45 ft. The HS20 truck load controlled for spans greater than 45 ft. Therefore, tandem loading was used to test spans 40 ft or less, while HS20 truck loading was used for spans greater than 40 ft.

5.5.2 Dead Load

The AISI design program automatically takes the dead load of the girder into account. The deck dead load however, must be input by the user. Two different deck dead loads were used. The steel grid design deck was a 5-inch RB weighing 23.4 psf. The concrete design deck was 8.5 in. thick sand weight concrete weighing 106 psf. Both composite and non-composite bridges were designed with a concrete deck.

5.6 Non-Conventional Cross Frames

A non-conventional bracing arrangement was chosen for this bridge design to speed up the construction process. This type of system has been used in England on much larger bridges. Instead of traditional bridge beam bracing with cross frames between each girder, this bracing style has only one cross frame between the two center girders. Struts will connect each interior girder to the adjacent exterior girder (see Figure 5.2). The struts connect the bracing between the center girders and the exterior girders. This system is similar to single bay bracing used in most buildings.

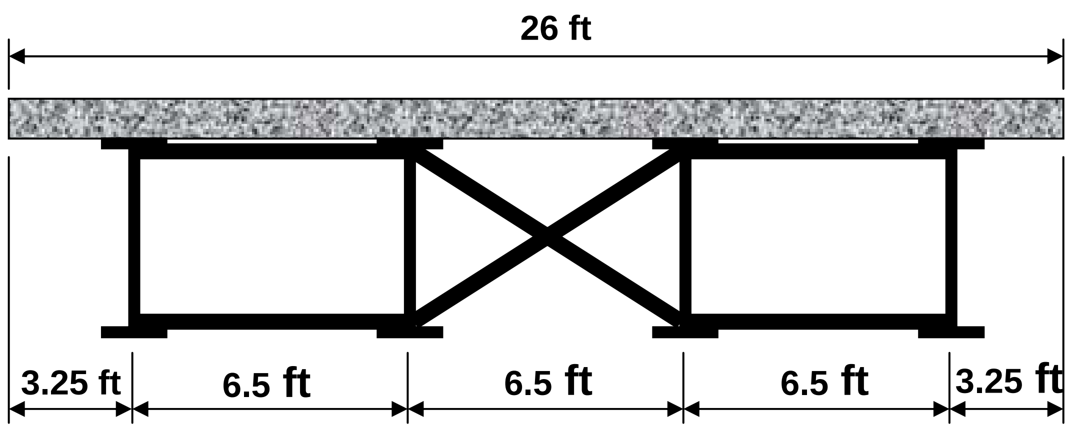


Figure 5. 2 Typical Bridge Cross Bracing Layout

5.7 Advantages of Cross Frame Configuration

In typical cross bracing configurations with cross frames between each girder it is usually difficult to fit the cross bracing into place between the girders. This is because there is often a slight difference in the camber of the girders. The girders in these cases are vertically adjusted by a crane to allow the cross frames to be fitted into place.

This bracing configuration can greatly reduce time during construction. Since only one cross brace is used at each brace location, only one girder must be vertically adjusted in order to make the cross braces fit. Once the two interior girders are placed, the exterior girders must be simply connected with struts. The camber of the girders will not affect the difficulty of placing struts. If adjustments are necessary to connect the struts to the girders, the girders may be moved with ease in the horizontal direction.

5.7.1 Spacing of Cross Frames

For rolled beams three cross braces may be spaced so that there is one set of bracing on each end of the bridge and one in the middle. Plate girders have deeper webs and narrower flange widths than rolled beams, causing them to have less torsional stiffness. Therefore, they are susceptible to greater torsional forces than rolled beams. Five or more braces are required for spans over approximately 80 ft, and four are required for spans ranging from 50 to 80 ft.

5.7.2 Cross Frame Calculations

The cross frame calculations for the 100 ft plate girder bridge are shown on the following pages. *The Fundamentals of Beam Bracing* (Yura, 1993) was used as an aid for the bracing calculations.

Plate Girder Properties

$t_{tf} = 2\,in$ $\quad$ $t_{web} = 0.5\,in$ $\quad$ $t_{bf} = 1.6875\,in$

$b_{tf} = 12\,in$ $\quad$ $d_{web} = 50\,in$ $\quad$ $b_{bf} = 12\,in$

The steel used by the AISI bridge design program was M 270 Grade 50.

$$F_y = 50\,ksi$$

$$E = 29000\,ksi$$

The total depth of each girder is

$$d = t_{tf} + d_{web} + t_{bf} = 53.7\,in.$$

The cross-sectional area of the girder is

$$A = t_{tf} \cdot b_{tf} + t_{web} \cdot d_{web} + t_{bf} \cdot b_{bf} = 69.25\,in.^2$$

The total span length, L, is equal to 100 ft. The girder spacing, S, is equal to 6.5 ft. There are cross frames at five locations along the bridge (n = 5), so the unbraced length, L_b, is 25 ft.

Moment Calculations

The elastic lateral buckling capacity of a girder, M_{cr}, is

$$M_{cr} = 3.14 \cdot E \cdot \left(\frac{I_{yc}}{L_b}\right) \cdot \left[0.772 \cdot \left(\frac{J}{I_{yc}}\right) + 9.87 \cdot \left(\frac{d}{L_b}\right)^2\right]^{\frac{1}{2}} \qquad \text{(Eqn. 5.1)}$$

In order to calculate M_{cr}, St. Venant's torsional constant (J) and the moment of inertia about the y-axis of the compression flange (I_{yc}) had to be determined first.

$$J = \left[\frac{\left(d_{web} \cdot t_{web}{}^3\right)}{3}\right] + \left[\frac{\left(b_{tf} \cdot t_{tf}{}^3\right)}{3}\right] + \left[\frac{\left(b_{bf} \cdot t_{bf}{}^3\right)}{3}\right] = 53.31 \text{in.}^4$$

$$I_{yc} = \frac{\left(t_{tf} \cdot b_{tf}{}^3\right)^4}{12} = 288.0 \text{in.}^4$$

From Equation 5.1,

$$M_{cr} = 4935 \text{kip} \cdot \text{ft}$$

From AISI's Bridge Design Program,

$$M_{max} = 4065 \text{kip} \cdot \text{ft}$$

M_{max} is the total maximum factored load moment calculated by the AISI Bridge Design Program. This moment was that of an exterior plate girder with a concrete deck. This girder was used because it had a greater M_{max} value when compared to interior girders or bridges with steel grid decks. The maximum factored moment for the plate girder bridge with a steel grid deck was 3189 kip-ft. It is less than the maximum factored load moment for the concrete deck bridge because the steel grid deck weighs less (23.4 psf vs. 106 psf). Since M_{cr} is greater than M_{max}, and this is the greatest M_{max} for the 100 ft. span plate girder bridge, the unbraced girder length of 25 ft is satisfactory for all the non-composite 100 ft. span plate girders tested.

Bracing Design

Try L2-1/2x2-1/2x3/8 as cross frames. The brace properties are

$$Fy_{br} = 36\,\text{ksi} \qquad Ix_{br} = 0.984\,\text{in.}^3$$

$$A_{br} = 1.73\,\text{in.}^2 \qquad rx_{br} = 0.753\ \text{in.}$$

Calculate the torsional brace strength requirement, M_{br}.

$$M_{br} = \frac{0.005 \cdot L_b \cdot span \cdot (M_{max})^2}{h \cdot n \cdot E \cdot I_{eff} \cdot {C_{bb}}^2} \qquad \text{(Eqn. 5.2)}$$

C_{bb} is a modification factor corresponding to effectively braced beams; Lbr is the length of the cross brace; h is the distance between flange centroids.

$$L_{br} = \left[d^2 + (S - t_{web})^2\right]^{\frac{1}{2}} = 94.3\text{in.}$$

$$h = d_{web} + \frac{t_{tf}}{2} + \frac{t_{bf}}{2} = 51.84\,\text{in.}$$

For doubly symmetric sections I_{eff} is two times I_{yc}. However, this girder was not doubly symmetric. The thickness of the top flange was 2 in., while the thickness of the bottom flange was 1.6875 in. The following equation was used to calculate $I_{eff.}$

$$I_{eff} = I_{yc} + \frac{t}{c} I_{yt}$$

The depth of the girder's compression and tension zones are c and t, respectively; I_{yt} is the moment of inertia about the y-axis of the tension flange.

$$c = \frac{\left[t_{tf} \cdot b_{tf} \cdot \left(\frac{t_{tf}}{2} \right) + t_{web} \cdot d_{web} \cdot \left(t_{tf} + \frac{d_{web}}{2} \right) + t_{bf} \cdot b_{bf} \cdot \left(t_{tf} + d_{web} + \frac{t_{bf}}{2} \right) \right]}{A}$$

$$c = 25.5\,\text{in.}$$

$$t = d - c = 28.14\,\text{in.}$$

$$I_{yt} = \frac{\left(t_{bf} \cdot b_{bf}^{\ 3} \right)}{12} = 243.0\,\text{in.}^4$$

$$I_{eff} = 556.2\,\text{in.}^4$$

Applying these figures to Equation 5.2,

$$M_{br} = 1025\,\text{kip} \cdot \text{in.}$$

The horizontal brace force, F_{br}, is the torsional brace strength requirement divided by the moment arm, h, between the top and bottom flanges.

$$F_{br} = \frac{M_{br}}{h} = 19.78\,\text{kips}$$

The maximum brace force, F_{max}, is the diagonal brace force.

$$F_{max} = F_{br} \cdot \frac{L_{br}}{S} = 23.91\,\text{kips}$$

The critical stress in the cross brace is F_{cr}.

$$F_{cr} = Fy_{br} \cdot \left[1 - \frac{Fy_{br} \cdot \left(\frac{L_{br}}{rx_{br}} \right)^2}{\left(4 \cdot \pi^2 \cdot E \right)} \right] = 18.25\,\text{ksi} \qquad \text{(Eqn. 5.3)}$$

P_u is the allowable force in the cross brace.

$$P_u = 0.85 \cdot A_{br} \cdot F_{cr} = 26.84\,\text{kips}$$

Since the allowable force is greater than the maximum brace force, this brace design is suitable.

Stiffness

The required brace stiffness of the cross frame system is β_T.

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_g} \qquad \text{(Eqn. 5.4)}$$

The stiffness of the stiffener, β_{sec}, is usually considered. However, this factor can be assumed to be infinity in this case. This is because the stiffener is the full depth of the girder, and will therefore evenly distribute forces transferred from the cross frames to the girders. Warping will not be allowed at the girder section where the stiffener is connected.

The brace stiffness and girder stiffness are β_b and β_g, respectively.

$$\beta_b = \frac{A_{br} \cdot E \cdot S^2 \cdot h^2}{L_{br}{}^3} = 979000 \frac{in \cdot kip}{rad}$$

$$\beta_g = \frac{12S^2EI_x}{L^3}$$

However, in multi-girder systems, the factor 12 can conservatively be changed to $24(n-1)^2/n$. For a four-girder bridge, this is equal to 54.

$$\beta_g = \frac{54S^2EI_x}{L^3} = 192000 \frac{in \cdot kip}{rad}$$

Referring back to Equation 4.4,

$$\beta_T = 160500 \, in \cdot \frac{kip}{rad}$$

$$\beta_{T,req'd.} = \frac{2.4 \cdot span \cdot M_{max}{}^2}{n \cdot E \cdot I_{eff} \cdot C_{bb}{}^2} = 81980 \frac{in \cdot kip}{rad} \qquad \text{(Eqn. 5.5)}$$

$\beta_b > \beta_t$, O.K.

When traditional beam bracing is used and there are cross braces between each set of girders, the vertical force caused by the cross frames acting on the exterior girder is $4M_{br}/3S$ at each brace point. The bracing configuration used here, however, produces a force of $4M_{br}/S$. This force will be designated as F_B and will act on the interior girders at each bracing point (see Figure 5.3). This force will act upward on one girder and downward on the other.

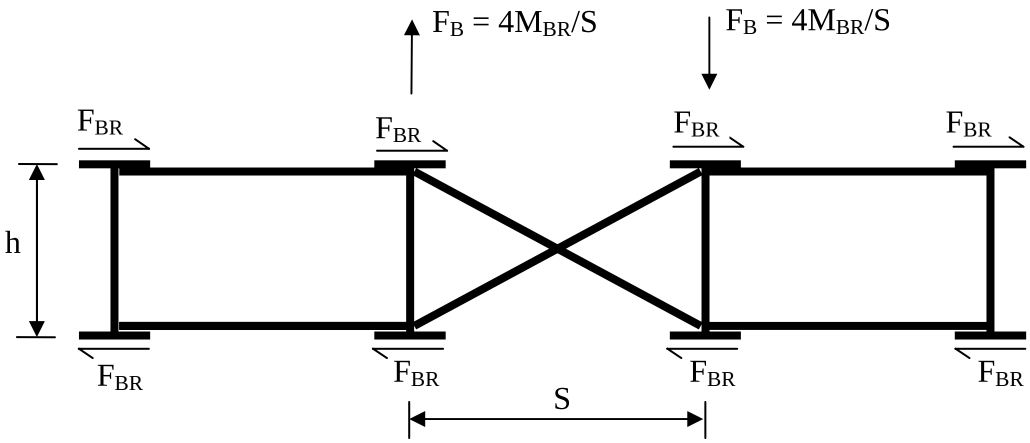


Figure 5.3 Diagram of Brace Forces Acting on Girders

The additional moment caused by the downward force from the braces should be checked and considered as point loads acting at each bracing point on the interior girders. The upward force does not need to be checked as it is in the upward direction and will only lessen the moments caused by bridge loading. The number of braces, n, equals five. Braces at the ends may be ignored, as their load will be directly transferred to beam supports. The maximum moment caused by three equal evenly spaced point loads acting on a beam is PL/2, or in this case $F_BL/2$.

$$F_B = \frac{4M_{br}}{S} = 52.58\,\text{kips}$$

$$M_B = 0.5 \cdot F_B \cdot L = 2629\,\text{kip} \cdot \text{ft}$$

Now, the total moment acting on the girders is M_{tot}.

$$M_{tot} = M_{max} + M_B = 6599\,\text{kip} \cdot \text{ft}$$

With the brace force included in the moment calculation, M_{tot} was greater than M_{cr} (4939 K-ft). The design had to be re-evaluated. Either another brace had to be added or the girder section had to be increased. Adding another brace so that n was six caused the elastic lateral buckling capacity, M_{cr}, to be 7267 K-ft. This is a sufficient capacity for the factored load moments and the moments caused by the force in the braces.

5.8 Comparison of Composite and non Composite Designs

While searching for the most economical type of bridge, both composite and non-composite designs were considered. Composite bridges are usually preferred over non-composite bridges because they utilize their decks to provide an increased section modulus. This allows the bridge to support a greater moment without increasing the girder size.

The purpose of this study is to determine which bridge type, composite or non-composite, is less expensive and most suitable for a rapidly constructed short span bridge design. Composite bridges were originally thought to be the less expensive alternative because their total girder weight is potentially less. Girders used in composite bridges require more labor than non-composite bridge girders, however. The additional labor could raise the cost of the girders used for a composite bridge design. The costs are compared and discussed in Section 5.12.

Feasibility of a composite deck also had to be considered. The two different deck types, steel grid decks and precast stay-in-place (SIP) prestressed concrete deck panel systems, are discussed and evaluated for composite design.

5.9 Feasibility of Composite Steel Grid Decks

Steel bridges are usually made composite using shear studs to connect the girders to the cast-in-place concrete. Some steel grid decks are constructed with concrete fill. However, pouring the concrete into the deck requires much more time than simply placing the deck. Therefore, the steel grid deck used in this design is an open deck and has no concrete fill. Without concrete, there is no simple way to create a connection between the steel girders and the steel grid deck to render the bridge composite. A composite design was not feasible for the steel grid deck alternative.

5.10 Composite Precast Panel Decks

The other deck alternative, an SIP panel deck, has a cast-in-place concrete topping. Although the bottom of the deck consists mainly of the precast panels, open strips are left above the girders where shear studs are located. When the cast-in-place concrete is placed, it forms a bond with the shear studs. Therefore, this type of deck can be made composite with the steel girders. A sketch of this deck system is shown in Figure 3.1.

5.11 Shear Stud Design and Costs

Although a composite bridge was a feasible design when a precast SIP deck was used, it was not necessarily the less expensive alternative. The main cost difference between a composite bridge and an equal size non-composite bridge was in the shear studs.

The shear studs used in composite steel bridges are welded to the tops of the girders. The studs used on the girders in this project were 3/4 in. diameter. Shear studs are often field welded to the girders. However, to reduce construction time, the shear studs should be welded to the girders prior to transport to the construction site.

The studs must be welded to the girders by the steel fabricator. They are welded by a machine that uses the studs themselves as the electrodes. The machine requires a great amount of power to melt steel with diameters as large as 0.75 in. The cost for the fabricator to weld the shear studs is $1.00 per stud. This is based upon the cost quoted by a local fabricator.

For composite bridge design there must be sufficient face area of shear studs in the concrete to transfer the moment and shear forces from the steel girders to the concrete. Typically, two to four shear studs are placed per row in the transverse direction. The rows of studs must be spaced at 24 in. or less in the longitudinal direction. Two studs were placed in each transverse row. The AISI Bridge Design Program determined the longitudinal spacing of the shear studs. The spacing ranged from 11 in. to 16 in.

5.12 Cost Comparison of Composite and Non-Composite Bridges

Costs comparisons were made based on bridge weights calculated by the AISI Bridge Design Program. These comparisons are based on girders designed for HS-20 truck live loads. After composite and non-composite bridges were designed by the program, the girder weights were compared. This is shown in Figure 5.4. Information for both rolled beams and plate girders are shown in these figures.

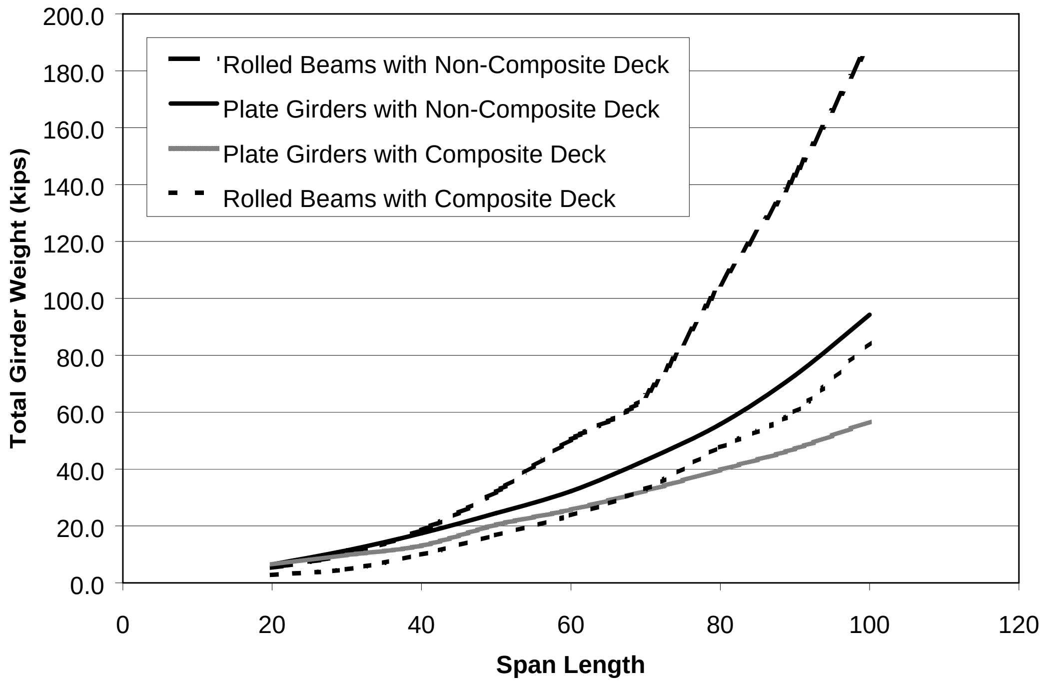


Figure 5.4. Comparison of Composite Vs. Non-Composite Girder Weight

Figure 5.4 shows that the non-composite bridge designs were heavier than the composite bridge designs. This is because they required deeper steel sections since the deck does not assist the girders in moment resistance.

Figure 5.5 shows the cost comparisons. These costs are based on unit costs of \$0.35 or \$0.50 per pound for rolled beams, \$0.62 per pound for plate girders, and \$1.00 per shear stud.

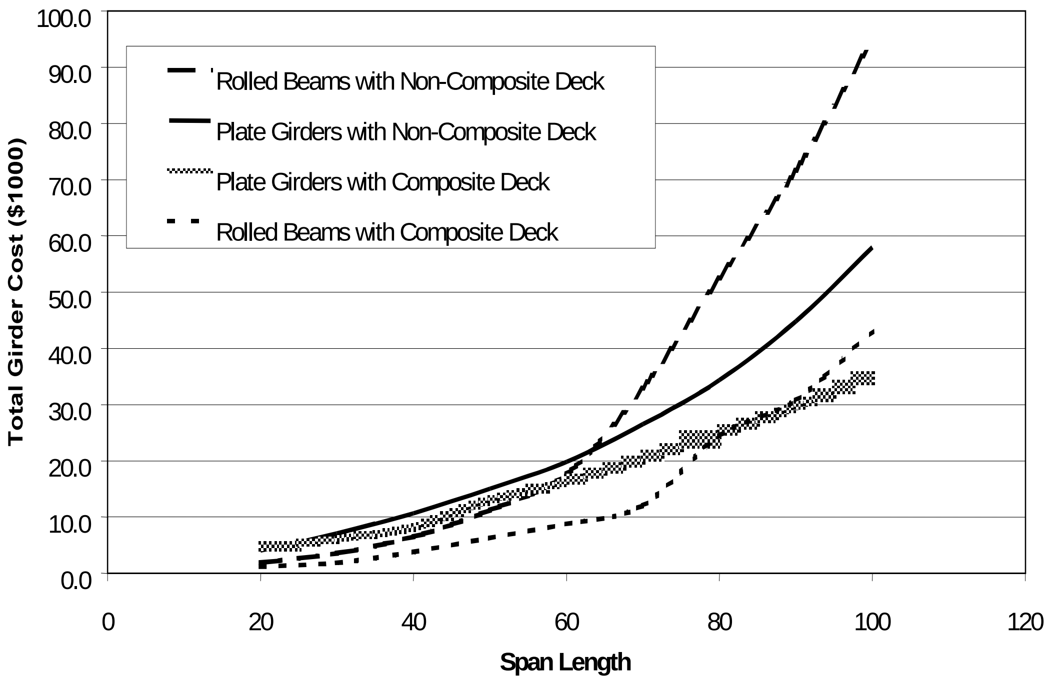


Figure 5.5 Comparison of Composite Vs. Non-Composite Girder Cost

When comparing girder costs, shear stud costs were included for composite designs. As seen in Figure 5.5 rolled beams with non-composite decks cost less than plate girders with composite decks for spans approximately 45 ft or less. However, a composite design, whether using rolled beams or plate girders, was the least expensive for every span in this study. For spans less than approximately 80 ft, rolled beam bridges with composite decks are less expensive. Plate girders are less expensive for spans ranging from 80 ft to 100 ft.

Based upon the costs and differences in construction speed both composite bridges with cast-in-place concrete deck bridges and non-composite designs with steel grid decks were selected as candidate configurations for these off system bridges.

5.13 Detailed Cost Study of Rolled versus Welded Girders

Two types of steel I-shaped girders were evaluated. The first alternative, used in bridge designs ranging from 20 ft to 100 ft, was rolled beams or W- Shapes. Rolled beams are convenient because they are manufactured in steel mills in accordance with the applicable ASTM specifications and require very little additional fabrication work. They are also the least expensive alternative for I-girders because they are rolled in mass quantities. Rolled W Shapes come in depths ranging from 4 in. to 44 in.

The second alternative was to use welded plate girders. Plate girders were designed for spans ranging from 50 ft to 100 ft. Plate girders are more expensive on a per pound basis than rolled beams because the hot rolled plate must be cut and welded to form the "I" cross section. Plate girders are not produced in mass quantities; the desired dimensions are set by the design requirements.

Plate girders are generally lighter than rolled beams because less material is used is sections of similar depth. The web and flange size of plate girders can be optimized to produce a lighter section than a rolled beam.

5.13.1 Steel Costs for Rolled Beams

Figure 5.6 shows the mill prices for rolled beams from two American steel mills, Nucor-Yamato Steel Co. in Blytheville, Arkansas, and TXI Chaparral Steel in Petersburg, Virginia. The prices are as of March 21, 2003. This data was acquired from the company websites.

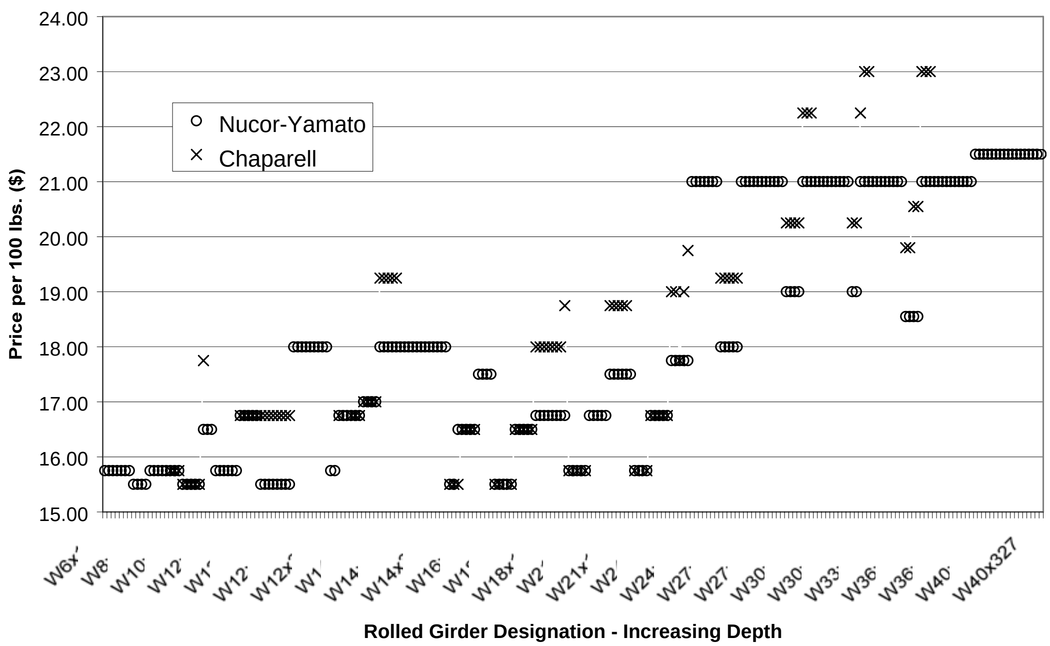


Figure 5.6. Comparison of Steel Costs per Pound with Girder Designation

As seen in Figure 5.6, the price per pound of rolled steel girders increases as the girder section increases. Most wide flange shapes ranging from 6 to 12 in. deep cost between \$15 and \$17 per 100 lbs. Section depths ranging from 12 to 27 in. cost between \$15 and \$20 per 100 lbs. The cost for sections between 27 and 40 in. deep range from about \$20 to \$23 per 100 lbs.

The as-fabricated price is not the same as the mill price, however. The fabricator's price is increased due to work that must be done to the girders such as cutting to length, drilling bolt holes, and welding stiffeners to the girders.

The approximate price of the rolled steel beams was determined by contacting a local Austin steel fabricator. The price quoted was \$0.35 per pound for most shapes that are less than 30 in. deep. Some shapes that are 30 in. deep or greater cost approximately \$0.50 per pound, while others may be as expensive as \$1.00 per pound depending on the availability of the steel.

5.13.2 Influence of Steel Availability upon Cost

Availability of rolled steel beams can be determined from AISC's website. A chart of all W-shape availability is included in Appendix A. The two steel mills that supply this region of the United States are Nucor-Yamato Steel Co. and TXI Chaparral Steel. The price for a W-shape that can be provided by both of these manufacturers is less than the price if only one supplier is available because of industry competition. For example, the price quoted from an Austin distributor for a W30x148 was \$0.50 per lb, while the price quoted from the same distributor for a W 40x183 was \$1.00 per lb. Both Nucor-Yamato and TXI Chaparral roll the W 30x148, while the W 40x183 is rolled by only Nucor-Yamato.

Both mills in this region provide most W-shapes less than 30 in. deep. Some heavy W-shapes that are 30 in deep or greater are provided only by Nucor-Yamato. As seen in Appendix A, many W-shapes are not provided by either mill. A W30x187 for example is manufactured only by Corus, which is located in Europe. To estimate steel costs, \$0.35 per pound was assumed the cost for beams rolled by both Nucor-Yamato and TXI Chaparral, while \$0.50 per pound was used for rolled shapes produced by only one of these companies.

5.13.3 Cover Plates

Rolled beams designed for spans of 90 and 100 ft required cover plates. Rolled beams without cover plates did not have enough moment capacity for these spans. The cover plates should be welded to the bottom flange of each girder. This will increases the moment of inertia of the girders, increasing the strength of the bridge.

5.13.4 Plate Girders Costs

The price of a welded plate girder was assumed to be \$0.62 per pound. The cost of rolled plate steel is approximately \$0.27 per lb. An additional \$0.35 per lb is assumed for cutting and welding the plates and fabrication of the plate girders. Plate girders may be the more economic choice in many situations, however. A deeper section can be used increasing the moment of inertia for a given steel area, better utilizing the material.

5.14 Cost Comparison

Bridges were designed using both rolled sections and plate girder sections, and girder weight and cost were compared. The weight and cost of the rolled beam and plate girder bridges versus span length are shown in Figures. 5.7 and 5.8, respectively. The quantities compared are for non-composite bridges with 8.5 in. concrete decks and four girders spaced at 6.5 ft.

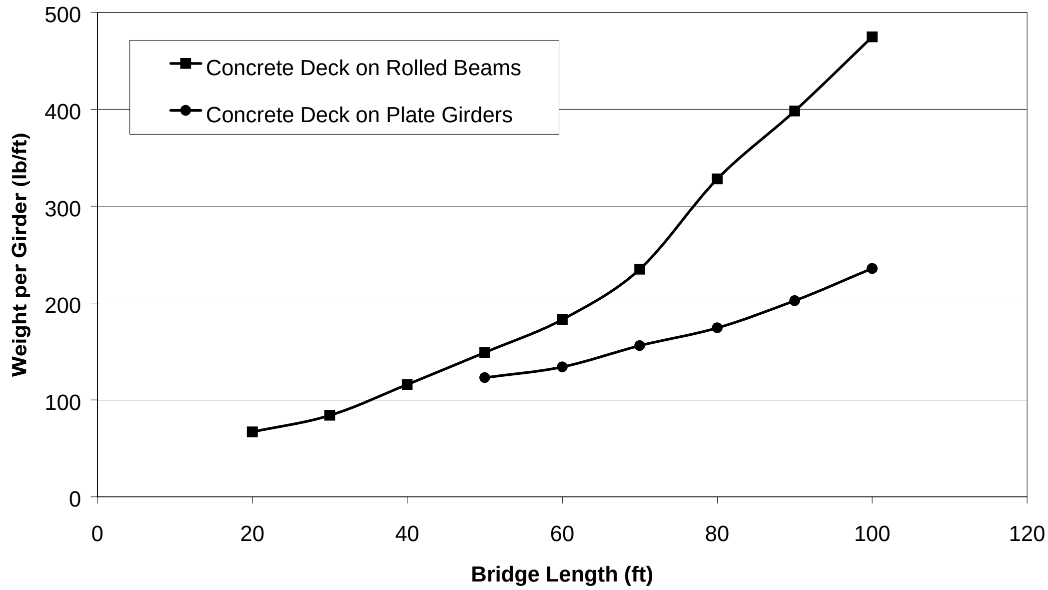


Figure 5.7. Comparison of Girder Weights for Bridges with Concrete Decks

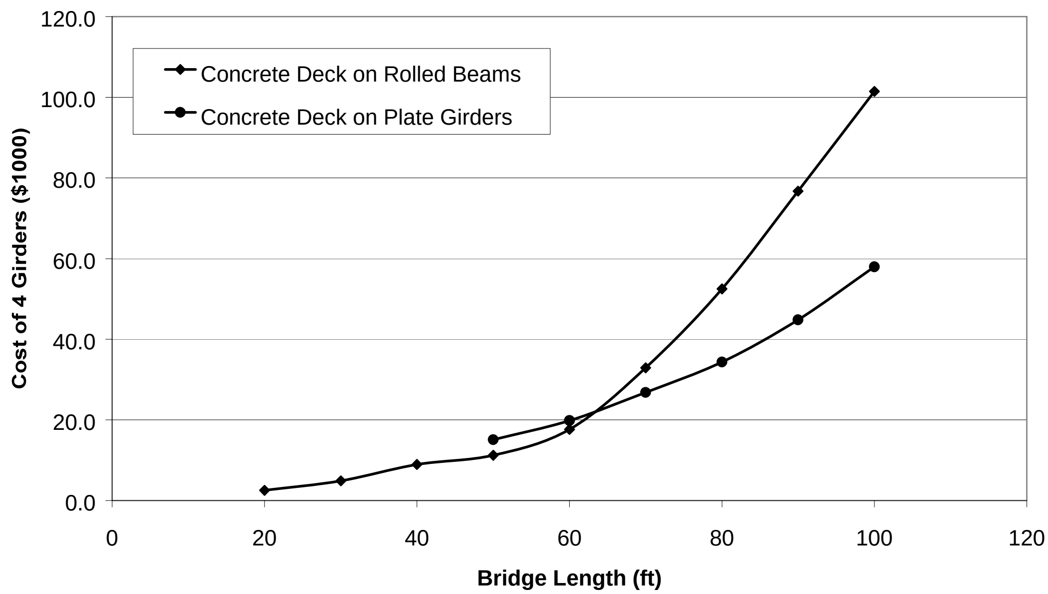


Figure 5.8. Comparison of Girder Costs for Bridges with Concrete Decks

Even though bridges designed using plate girders are lighter for spans ranging from 50 to 100 ft, they are not the least expensive. This is shown in Figure 5.8. The cost data show that for non-composite spans approximately 65 ft or greater plate girders are the more economic alternative, while rolled shapes are the more economic choice for non-composite spans 65 ft or less.

Similar comparisons were made for bridges with steel grid decks and bridges with 8.5 in. composite concrete decks. Comparisons showing costs of rolled beams vs.

costs of plate girders with steel grid decks and composite concrete decks are shown in Figures 5.9 and 5.10, respectively.

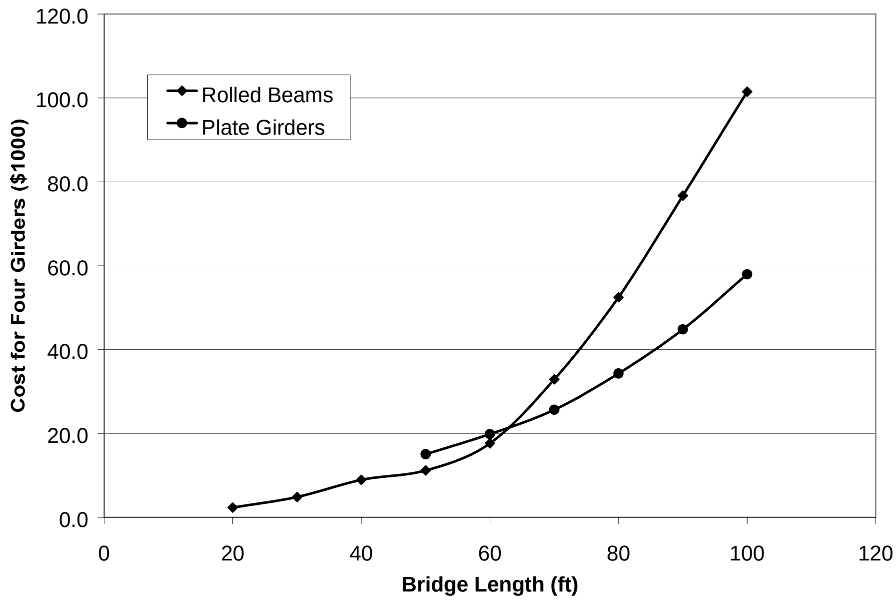


Figure 5.9 Comparison of Girder Costs for Bridges with Steel Grid Decks

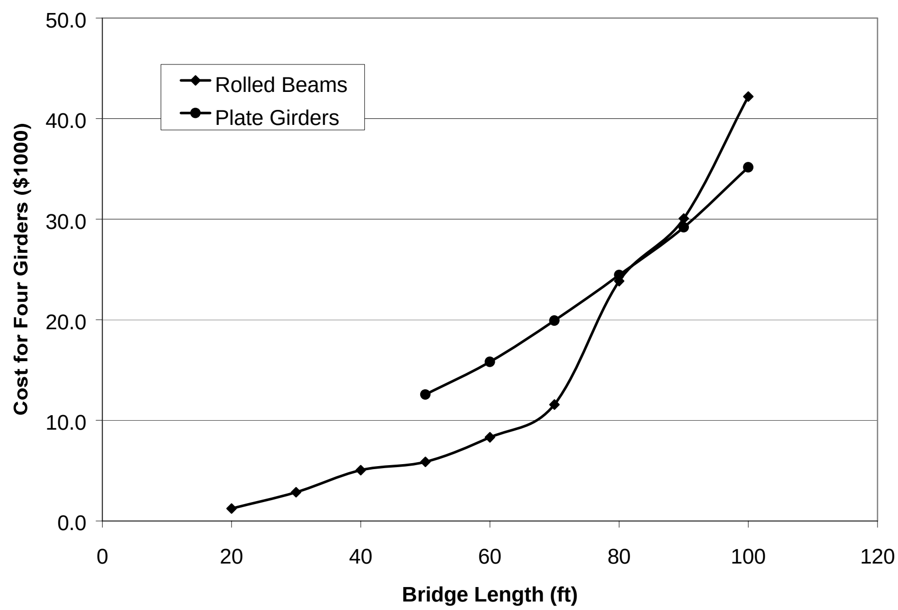


Figure 5.10 Comparison of Girder Costs for Bridges with Composite Concrete

Figure 5.9 shows that for steel grid decks, rolled beams are less expensive for spans less than 65 ft, while plate girders are less expensive for spans greater than approximately 65 ft. This is very similar to the cost comparison for bridges with concrete decks. When designing the bridges, the dead load difference between the concrete deck and the steel grid deck did not contribute greatly to the total load acting on the bridge. Therefore, similar girder sizes were used for both cases.

The composite bridge, however, allows for much lighter girders. For this type of bridge, the rolled beams cost less than plate girder for spans approximately 80 ft or less. Plate girders should be used for bridges spanning greater than 80 ft.

CHAPTER 6 Construction

Novel construction methods will be used in this project in order to quicken the construction process. These methods of construction and scheduling of the construction tasks are discussed in this chapter.

6.1 Critical Path

In a typical construction project there is a critical path. A critical path is the continuous chain of activities from project-start to project-finish, whose durations cannot be exceeded if the project is to be completed on the project-finish date. The critical path controls the duration of the bridge's construction. A typical critical path for a single span bridge would usually have an order in which the site is excavated first; piles are constructed next, followed by the construction of abutments, placement of the girders, and casting the deck. Each one of these phases would be dependent on the phase preceding it. When a problem or delay arises in one phase of a critical path, the whole project is delayed.

6.2 Construction Order

In this project the critical path of construction was modified to decrease the total construction time. The construction order for this bridge will go as follows. The construction sight will first be excavated. After the sight is excavated, the precast abutments will be placed. A rendering of this type of abutment can be seen in Figure 6.1. The piles and girders will be placed in the next phase. After the piles and girders are placed, the deck will be placed. In the case of the bridge with the steel grid deck, the deck can be placed before the pile driving is complete, but the deck cannot be finished until the piles are in place.

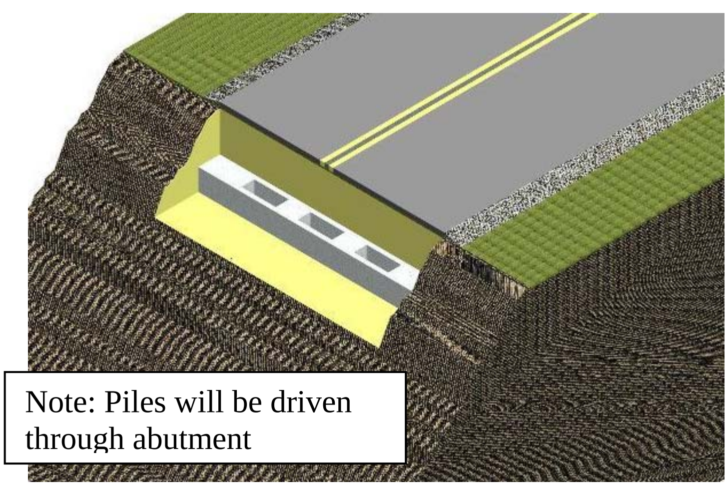
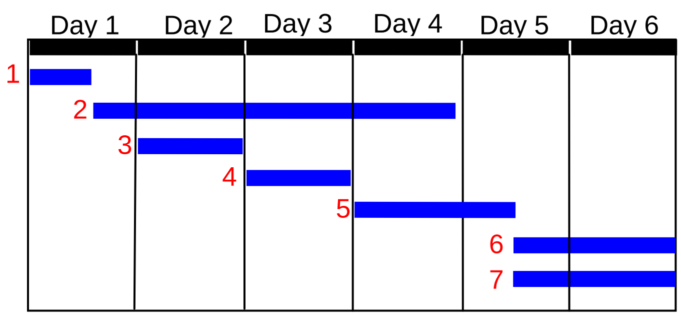


Figure 6. 1 Precast Abutment

Constructing a bridge in this method will create many advantages. The main advantage lies in the precast abutments. These abutments will allow simultaneous construction of the girders and the piles. This eliminates time because two major parts of the construction process will be completed simultaneously. Prior to the bridge's construction, the consulting construction crews will have to determine at precisely which times they will be working. This is in order to avoid any conflicts between the pile driving crew and the crew that will be placing the girders.

Another advantage is that driving the piles will be removed from most of the critical path. Figure 6.2 shows the critical path of construction of a steel girder bridge with a steel grid deck after the sight is excavated. It can be seen in this Figure that when building a bridge with a steel grid deck and precast abutments, the girders and part of the deck may be placed before the piles are driven. Piles are not required to place and brace the girders as they are in conventional construction. If a steel grid deck is used, it may be placed before the piles in all areas of the bridge except where it would obstruct pile driving.



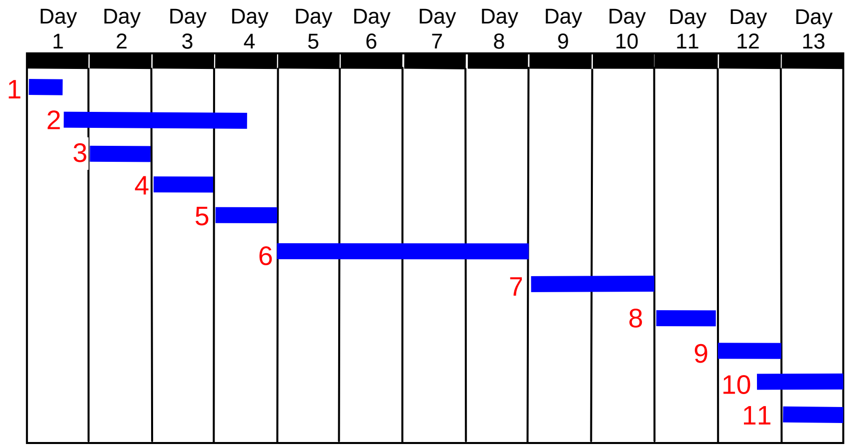
Constructions Steps

1. Install abutments
2. Drive piles
3. Install interior girders and cross frames
4. Install exterior girders and struts
5. Install Deck
6. Finish approach
7. Install railings

Figure 6. 2 Construction process for steel bridge built with a steel grid deck

As seen in Figure 6.2, after the existing bridge is removed, the construction of a steel girder bridge with precast abutments and a steel grid deck can be completed in six days.

Figure 6.3 shows the critical path of construction for as steel bridge with a precast panel deck with CIP concrete topping. This diagram also considers construction after the site is excavated.



Constructions Steps

1. Install abutments
2. Drive piles
3. Install interior girders and cross frames
4. Install exterior girders and struts
5. Place SIP concrete panels
6. Place removable forms (2 days per side)
7. Place reinforcing steel
8. Pour concrete
9. Allow concrete to cure
10. Finish approach
11. Install Guardrail

Figure 6. 3 Construction process for steel bridge built with a concrete deck

Constructing a precast SIP concrete panel deck with CIP topping would take a much greater amount of time than a similar bridge with a steel grid deck. Steps five thru nine in the construction process for the concrete deck bridge are related to deck construction. Using a concrete deck takes about seven days more in a construction project than using a steel grid deck.

Chapter 7 Conclusions

The research performed in this project resulted in two bridge replacement solutions, Solutions A and B. Solution A has an extremely rapid construction speed, but is also expensive. Solution B is inexpensive and has a slower construction speed than Solution A.

Both design solutions will be simply supported I-girder bridges sitting on precast abutments. There will be four girders in each bridge design. The girders will be spaced at 6.5 ft with 3.25 ft overhangs to provide a 26 ft wide roadway. The girders will be either rolled wide-flange shapes or welded plate girders depending on the length of the bridge. They will be braced by the cross frame system discussed in Section 5.7. The foundations shall be driven piles unless the soil is unfit for piles. In these cases the foundations shall be drilled shafts.

7.1 Solution A

Solution A is a steel girder bridge with a 5 in. RB steel grid deck (4 in. main bar spacing). This non-composite bridge will be constructed rapidly. All steel elements of the bridge will be shop welded and all concrete elements of the bridge will be precast in most cases. The only instances in which all elements are not precast are when drilled shafts must be used because the soil at the bridge location impedes pile driving. The construction of this bridge should take approximately eight days. The cost of the bridge material is expensive relative to the material in Solution B. However, other costs will be reduced with the Bridge Solution A. Less labor will be required since the construction time is reduced. Exact labor costs are difficult to estimate in this phase of design. One way to get a rough estimate of savings from reduced labor is to base labor solely on estimated construction time. When estimating labor costs based on construction time, labor costs for the Solution A bridge will be approximately half of what they will be for Solution B. This is because construction of the Solution A bridge will take about half the time that Solution B's construction will take. A relation between savings in labor costs and increased material costs is unknown.

If Solution A is chosen, rolled beams should be used for spans under 65 ft, while plate girders should be used for spans greater than 65 ft. Plate girders are the less expensive alternative for non-composite spans greater than 65 ft, while rolled beams are less expensive for non-composite spans under 65 ft.

7.2 Solution B

The Solution B design is a steel girder bridge with a composite concrete deck. The deck will be comprised of 4 in. precast SIP panel forms with a 4.5 in. CIP concrete topping slab. The material costs for this alternative are very inexpensive. At $8 per square ft, this concrete deck is less than one third the cost of the steel grid deck. The negative aspect of this bridge is it's slow construction speed. Although the deck is formed with SIP panels, removable forms will be required along the edges of the bridge. A great amount of time is involved with placing the forms, placing rebar, pouring concrete, and removing the forms. Because of this it will take approximately 15 days or three working weeks to construct the Solution B bridge. This is approximately twice as long as it takes to construct the Solution A bridge.

If Solution B is the chosen bridge design, rolled beams shall be used as the girders for spans 80 ft or less. Welded plate girders should be used if the bridge span is between 80 ft and 100 ft. This is because for composite designs, rolled beams are less expensive for spans less than 80 ft while plate girders are less expensive for spans over 80 ft. The cost data can be seen in Figure 5.10.

7.3 Comparing Final Designs with Current Industry

The approximate costs of Solutions A and B are compared in Figure 7.1. Also compared in the figure are costs of a two pre-manufactured bridges, one with a steel grid deck, and the other with a concrete deck. It should be noted that construction costs are not included for Solutions A and B, while assembly costs are included for the pre-manufactured bridges.

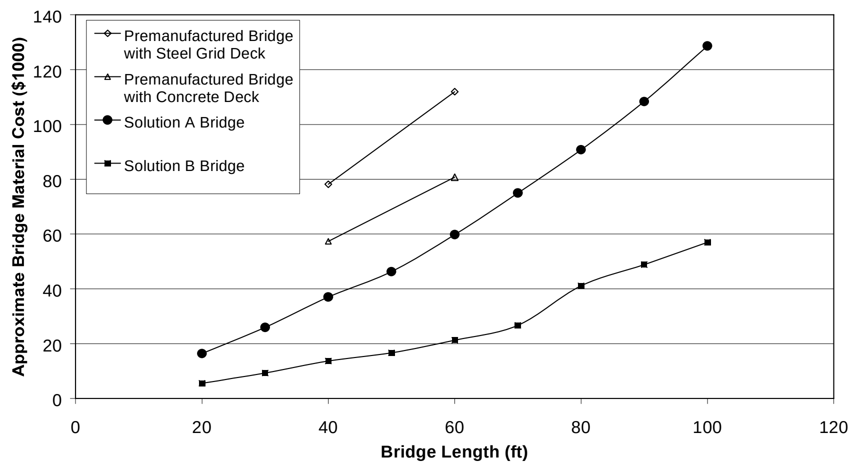


Figure 7. 1 Comparing Final Designs with Current Industry Solutions

Figure 7.1 shows that the costs of the bridges vary greatly with their lengths. Although Solution A appears to be much more expensive than Solution B, the difference in cost may be inaccurate. If labor costs were included the gap in cost between Solutions A and B would be less. This is because Solution B requires twice as many hours of labor and equipment rental.

The owner of the bridge should evaluate the bridge site to determine which bridge design is better suited for the area. If there are no nearby alternate routes and people are greatly inconvenienced by a road closure, the construction speed is of great importance and Solution A should be used. If construction speed is of less importance, Solution B should be used.

Appendix A
LRFD Design of 60ft Span Bridge

Design Parameters

The following is a list of parameters upon which this design is based:

1. Single Span, which is specified to be 60ft.
2. Total bridge width is 26ft, and the clear roadway width is 24ft.
3. Open steel grid deck with overhang, non-composite.
4. The deck should allow pedestrian to walk on.
5. Rolled steel beam, Grade 50.
6. Fatigue design is not performed for this bridge due to the small amount of the traffic.
7. 4 cross-frames are used, which spaced at 20ft.
8. T-6 standard railing system.
9. Flood load design is required.

Summary of Design Steps

The following is a summary of the major design steps included in this project:

Design Step 1 - General Information
Design Step 2 - Open Steel Grid Deck Design
Design Step 3 - Steel Girder Design
Design Step 4 - Miscellaneous Steel Design
(i.e., shear connectors, bearing stiffeners, cross frames, deck-girder connectors)
Design Step 5 - Abutment Design

Software

ANSYS7.0 is used for the finite element analysis in this design procedure.

Design Step 1 - General Information

Design Criteria

Governing specifications: *AASHTO LRFD Bridge Design Specifications* (Second Edition, 1998). In the following parts, it is named as AASHTO for brief.

Design methodology: Load and Resistance Factor Design (LRFD)

Live load requirements: HL-93

Deck width: 26 ft

Clear roadway width: 24ft
Span length: 60 ft

Skew angle: 0 degree

Structural steel yield stress: $F_y = 50 Ksi$

Structural steel tensile stress: $F_u = 65 Ksi$

Steel density: $W_s = 0.490 Kcf$

Concrete density: $W_c = 0.150 Kcf$

Future wearing surface: none

<u>Design Factors from *AASHTO LRFD Bridge Design Specifications*</u>

A single, combined η is required for every structure. When a maximum load factor from AASHTO *Table 3.4.1-2* is used, the factored load is multiplied by η, and when a minimum load factor is used, the factored load is divided by η. All other loads, factored in accordance with AASHTO *Table 3.4.1-1*, are multiplied by η if a maximum force effect is desired and are divided by η if a minimum force effect is desired. In this design, it is assumed that all η factors are equal to 1.0.

$$\eta_D = 1.0 \qquad \eta_R = 1.0 \qquad \eta_I = 1.0$$

For loads for which the maximum value of γ_i is appropriate:

$$\eta = \eta_D \cdot \eta_R \cdot \eta_I \text{ and } \eta \geq 0.95$$

For loads for which the minimum value of γ_i is appropriate:

$$\eta = \frac{1}{\eta_D \cdot \eta_R \cdot \eta_I} \text{ and } \eta \leq 1.00$$

Therefore, in this design the load modifier factor η is:

$$\eta = 1.00$$

The following is a summary of other design factors from the *AASHTO LRFD Bridge Design Specifications.* See additional information in the Specifications, section 3.

Load Factors: (AASHTO, Table 3.4.1-1 and Table 3.4.1-2)

Load combinations and Load Factors					
Limit States	Load Factors				
	DC		LL	IM	WA
	Min.	Max.			
Strength I	0.90	1.25	1.75	1.75	1.00
Service II	1.00	1.00	1.30	1.30	1.00

Resistance Factors: (AASHTO, 6.5.4.2)

Resistance Factors		
Material	Type Of Resistance	Resistance Factors, ϕ
Structural Steel	For Flexure	$\phi_f = 1.00$
	For Shear	$\phi_v = 1.00$
	For Axial Compression	$\phi_c = 0.90$
	For Bearing	$\phi_b = 1.00$

Multiple Presence Factors: (AASHTO, Table 3.6.1.4.2-1)

Multiple Presence Factors	
Number of Lanes loaded	Multiple Presence Factors, m
1	1.20
2	1.00
3	0.85
>3	0.65

Dynamic Load Allowance: (AASHTO, Table 3.6.2.1-1)

Dynamic Load Allowance	
Limit State	Dynamic Load Allowance, IM
Fatigue and Fracture Limit State	15%
All Other Limit State	33%

Span Arrangement:

For this design, the span arrangement is presented in Figure 1.

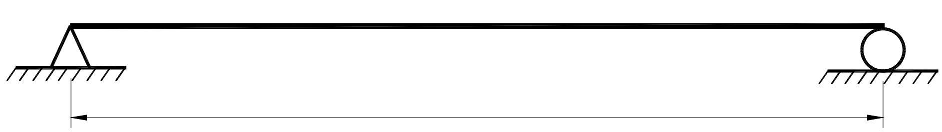


Figure1. Bridge span arrangement

Bridge Cross Section:

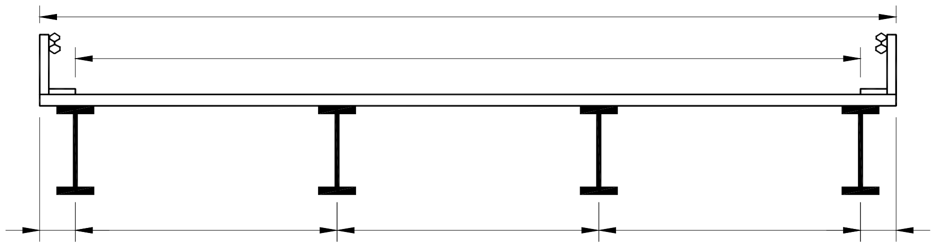


Figure2. Bridge section arrangement

Design Step 2 - Open Steel Grid Deck Design

Design Criteria

Governing specifications: *AASHTO LRFD Bridge Design Specifications* (Second Edition, 1998). In the following parts, it is named as AASHTO for brief.

Design methodology: Load and Resistance Factor Design (LRFD)

Steel Grid Deck

All the following design in this step is based on the information that provided by *American Bridge*.

Table 1. 5 inch open I-beam-Truss-Lok Type steel gird deck

HS20 Load Table (In accordance with AASHTO allowable stress design method)							
5 inch open I-beam Truss-Lok Type	M' Bar Weight (#/ft)	M' Bar Spacing (in)	Sectional Properties (in^3/ft)		Max. Continuous Clear Span (ft)		Approximate Weight (lb/sf)
			Top Steel	Bottom Steel	A 36	A 588	
LS	5.52	3-3/4	6.467	7.326	7.09	9.59	25.9

Note: The design clear spans in the table are based on the AASHTO allowable stress method. The maximum continuous clear span will be checked with LRFD method in the following part.

Determine the Width of Equivalent Interior Strips

The deck spans primarily in the direction perpendicular to the traffic direction, and the equivalent interior strip width is subject to AASHTO, Table 4.6.2.1.3-1.

$$W_E = 1.25P + 4.0S_b$$

Where W_E is the equivalent interior strip width in inch, P is axle load in Kip and S_b is spacing of grid bars in inch.

$$W_E = 1.25 \times 32 + 4.0 \times 3.75 = 55in$$

Compute Live Load Effects

Basic parameters:

1. The minimum distance from the center of design vehicle wheel to the inside faces of parapet is1 foot.
2. The minimum distance between the wheels of two adjacent design vehicles is 2 feet.
3. Dynamic load allowance, IM, $IM = 0.33$
4. Load factor for live load, strength I, $\gamma_{LL} = 1.75$
5. AASHTO, HL-93 truck load is used in design.

In finite element analysis for determining the live load effects, unit load are used to stand for the wheel loads. Based on AASHTO standard truck load, HS-20, the wheel load will be two16kips, which are 6ft apart. Both single truck load and two trucks load cases are considered in finite element analysis. The results are given in Table 1.

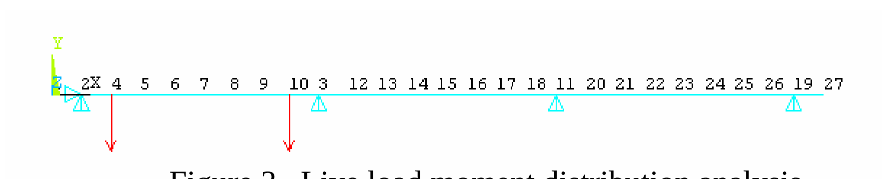


Figure 3. Live load moment distribution analysis

Compute Dead Load Effects

Since the dead load is distributed evenly along the whole span, 1ft wide strip is chosen to calculate its effects. The dead load includes the weight of steel grid deck and the weight of railings. The Finite Element Analysis model is shown in Figure.

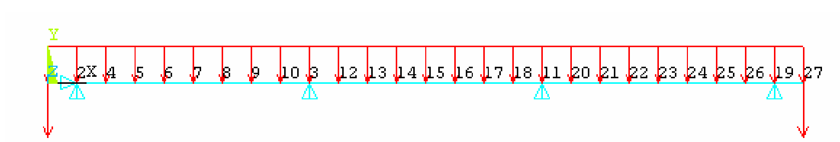


Figure4. Dead load moment distribution analysis

Dead Load:

Steel grid deck weight: 25.9lb/lft
Railing: 23lb/lft, concentrated line load at edge of deck.

The moments at each node are given in Table 2.

After the dead load moments are computed for the deck and railing, the correct load factors must be identified. The load factors for dead loads are:

$\gamma_{DC,\max} = 1.25 \qquad \gamma_{DC,\min} = 0.90$

Table 2A Unit Live Load Moments (K-in)

Single truck load									
Nodes	**1**	**2**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Case1	0	0	10.962	9.924	8.885	7.847	6.809	5.771	4.733
Case2	0	0	8.258	16.517	12.775	9.034	5.292	1.551	-2.191
Case3	0	0	5.926	11.853	17.779	11.706	5.632	-0.441	-6.515
Case4	0	0	3.956	7.912	11.868	15.825	7.781	-0.263	-8.307
Case5	0	0	2.339	4.677	7.016	9.354	11.693	2.032	-7.630
Case6	0	0	1.065	2.129	3.194	4.259	5.324	6.388	-4.547
Case7	0	0	0.125	0.251	0.376	0.502	0.627	0.752	0.878
Case8	0	0	-0.488	-0.977	-1.465	-1.953	-2.442	-2.930	-3.418
Case9	0	0	-0.785	-1.570	-2.355	-3.140	-3.925	-4.710	-5.495

Single truck load (continued)							
3	**12**	**13**	**14**	**15**	**16**	**17**	**18**
-8.306	-7.014	-5.723	-4.431	-3.140	-1.848	-0.557	0.735
-5.933	-5.010	-4.088	-3.165	-2.243	-1.320	-0.398	0.525
-12.589	-0.497	-0.406	-0.315	-0.223	-0.132	-0.041	0.050
-16.351	-5.506	5.339	4.184	3.029	1.874	0.719	-0.436
-17.291	-8.062	1.168	10.398	7.628	4.857	2.087	-0.684
-15.482	-8.191	-0.899	6.392	13.684	8.975	4.267	-0.442
-10.997	-5.920	-0.844	4.233	9.310	14.386	7.463	0.539
-3.907	-1.276	1.356	3.987	6.618	9.249	11.880	2.511
-6.280	5.720	5.720	5.720	5.720	5.720	5.720	5.720

Single truck load (continued)									
11	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**19**	**27**
2.026	1.773	1.520	1.266	1.013	0.760	0.507	0.253	0	0
1.447	1.266	1.085	0.905	0.724	0.543	0.362	0.181	0	0
0.142	0.124	0.106	0.089	0.071	0.053	0.035	0.018	0	0
-1.591	-1.392	-1.193	-0.994	-0.795	-0.597	-0.398	-0.199	0	0
-3.454	-3.022	-2.590	-2.159	-1.727	-1.295	-0.863	-0.432	0	0
-5.151	-4.507	-3.863	-3.219	-2.575	-1.931	-1.288	-0.644	0	0
-6.384	-5.586	-4.788	-3.990	-3.192	-2.394	-1.596	-0.798	0	0
-6.859	-6.001	-5.144	-4.287	-3.429	-2.572	-1.715	-0.857	0	0
-6.280	-5.495	-4.710	-3.925	-3.140	-2.355	-1.570	-0.785	0	0

Table 2B Unit Live Load Moments (K-in)

Double truck load									
Nodes	1	2	4	5	6	7	8	9	10
Case10	0	0	10.177	8.354	6.531	4.707	2.884	1.061	-0.762
Case11	0	0	10.104	8.209	6.313	4.418	2.522	0.627	-1.269
Case12	0	0	10.164	8.328	6.491	4.655	2.819	0.983	-0.854
Case13	0	0	10.318	8.636	6.954	5.272	3.590	1.908	0.226
Case14	0	0	10.530	9.060	7.590	6.120	4.650	3.180	1.710
Case15	0	0	10.763	9.526	8.289	7.052	5.815	4.578	3.341
Case16	0	0	10.979	9.959	8.939	7.918	6.898	5.877	4.857
Case17	0	0	11.143	10.285	9.428	8.571	7.714	6.856	5.999
Case18	0	0	11.215	10.430	9.645	8.860	8.075	7.290	6.505
Case19	0	0	7.401	14.802	10.203	5.605	1.006	-3.593	-8.192
Case20	0	0	7.460	14.921	10.381	5.842	1.302	-3.238	-7.777
Csae21	0	0	7.615	15.229	10.844	6.458	2.073	-2.312	-6.698
Case22	0	0	7.827	15.653	11.480	7.307	3.133	-1.040	-5.213
Case23	0	0	8.060	16.119	12.179	8.238	4.298	0.357	-3.583
Case24	0	0	8.276	16.552	12.828	9.105	5.381	1.657	-2.067
Case25	0	0	8.439	16.879	13.318	9.757	6.197	2.636	-0.925
Case26	0	0	5.128	10.257	15.385	8.514	1.642	-5.230	-12.101
Case27	0	0	5.283	10.565	15.848	9.130	2.413	-4.304	-11.022
Case28	0	0	5.495	10.989	16.484	9.979	3.473	-3.032	-9.537
Case29	0	0	5.728	11.455	17.183	10.910	4.638	-1.635	-7.907
Case30	0	0	5.944	11.888	17.832	11.777	5.721	-0.335	-6.391
Case31	0	0	3.312	6.625	9.937	13.249	4.562	-4.126	-12.814
Case32	0	0	3.524	7.049	10.573	14.098	5.622	-2.853	-11.329
Case33	0	0	3.757	7.515	11.272	15.029	6.787	-1.456	-9.699
Case34	0	0	1.907	3.814	5.721	7.628	9.534	-0.559	-10.652

Double truck load (continued)							
3	12	13	14	15	16	17	18
-14.585	-1.294	-0.002	1.289	2.581	3.872	5.164	6.455
-15.164	-4.504	6.157	4.817	3.478	2.138	0.799	-0.541
-14.690	-6.475	1.740	9.955	6.170	2.385	-1.400	-5.186
-13.456	-7.456	-1.456	4.544	10.544	4.544	-1.456	-7.456
-11.760	-7.698	-3.636	0.426	4.488	8.550	0.611	-7.327
-9.897	-7.450	-5.004	-2.557	-0.111	2.336	4.782	-4.771
-8.164	-6.964	-5.764	-4.564	-3.363	-2.163	-0.963	0.237
-6.859	-6.490	-6.121	-5.752	-5.383	-5.014	-4.645	-4.276
-6.280	-6.280	-6.280	-6.280	-6.280	-6.280	-6.280	-6.280
-12.791	-2.500	7.792	6.083	4.375	2.666	0.958	-0.751
-12.317	-4.471	3.375	11.221	7.067	2.913	-1.241	-5.395
-11.083	-5.452	0.179	5.810	11.441	5.072	-1.297	-7.666
-9.387	-5.694	-2.001	1.692	5.385	9.078	0.770	-7.537
-7.524	-5.446	-3.369	-1.291	0.787	2.864	4.942	-4.981
-5.791	-4.960	-4.129	-3.297	-2.466	-1.635	-0.804	0.027
-4.485	-4.485	-4.485	-4.485	-4.485	-4.485	-4.485	-4.485
-18.973	0.042	7.057	14.071	9.086	4.101	-0.884	-5.870
-17.739	-0.939	3.861	8.660	13.460	6.260	-0.940	-8.141
-16.042	-1.181	1.681	4.542	7.404	10.266	1.127	-8.011
-14.180	-0.933	0.313	1.559	2.806	4.052	5.298	-5.455
-12.447	-0.447	-0.447	-0.447	-0.447	-0.447	-0.447	-0.447
-21.501	-5.948	9.606	13.159	16.713	8.266	-0.180	-8.627
-19.805	-6.189	7.426	9.041	10.657	12.272	1.887	-8.497
-17.942	-5.942	6.058	6.058	6.058	6.058	6.058	-5.942
-20.745	-8.745	3.255	15.255	15.255	15.255	3.255	-8.745

Table 2B Unit Live Load Moments (K-in) (continued)

Double truck load (continued)									
11	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**19**	**27**
-4.253	-3.722	-3.190	-2.658	-2.127	-1.595	-1.063	-0.532	0	0
-1.880	-1.645	-1.410	-1.175	-0.940	-0.705	-0.470	-0.235	0	0
-8.971	2.651	2.272	1.893	1.515	1.136	0.757	0.379	0	0
-13.456	-2.774	7.908	6.590	5.272	3.954	2.636	1.318	0	0
-15.265	-5.857	3.551	12.959	10.368	7.776	5.184	2.592	0	0
-14.325	-6.534	1.257	9.047	16.838	12.628	8.419	4.209	0	0
-10.563	-4.742	1.078	6.898	12.719	18.539	12.359	6.180	0	0
-3.907	-0.418	3.070	6.558	10.047	13.535	17.023	8.512	0	0
-6.280	6.505	7.290	8.075	8.860	9.645	10.430	11.215	0	0
-2.459	-2.152	-1.844	-1.537	-1.230	-0.922	-0.615	-0.307	0	0
-9.549	2.144	1.838	1.532	1.225	0.919	0.613	0.306	0	0
-14.035	-3.281	7.474	6.228	4.983	3.737	2.491	1.246	0	0
-15.844	-6.363	3.117	12.598	10.078	7.559	5.039	2.520	0	0
-14.903	-7.041	0.822	8.685	16.548	12.411	8.274	4.137	0	0
-11.141	-5.249	0.644	6.537	12.429	18.322	12.215	6.107	0	0
-4.485	-0.925	2.636	6.197	9.757	13.318	16.879	8.439	0	0
-10.855	1.002	0.859	0.716	0.572	0.429	0.286	0.143	0	0
-15.341	-4.423	6.495	5.412	4.330	3.247	2.165	1.082	0	0
-17.149	-7.506	2.138	11.782	9.425	7.069	4.713	2.356	0	0
-16.209	-8.183	-0.157	7.869	15.895	11.922	7.948	3.974	0	0
-12.447	-6.391	-0.335	5.721	11.777	17.832	11.888	5.944	0	0
-17.073	-5.939	5.195	4.239	3.463	2.598	1.732	0.866	0	0
-18.882	-9.022	0.838	10.699	8.559	6.419	4.280	2.140	0	0
-17.942	-9.699	-1.456	6.787	15.029	11.272	7.515	3.757	0	0
-20.745	-10.652	-0.559	9.534	7.628	5.721	3.814	1.907	0	0

Table 2 Un-factored Dead Load Moments (K-in)

Deck Weight and Parapet Weight									
Nodes	**1**	**2**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Case1	0	-0.431	0.473	1.067	1.350	1.322	0.983	0.333	-0.627

Deck Weight and Parapet Weight (continued)							
3	**12**	**13**	**14**	**15**	**16**	**17**	**18**
-1.898	-0.810	-0.033	0.433	0.588	0.433	-0.033	-0.810

Deck Weight and Parapet Weight (continued)									
11	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**19**	**27**
-1.898	-0.627	0.333	0.983	1.322	1.350	1.067	0.473	-0.431	0

Note: The yellow cells stand for the supports position, and the red cells stand for wheel load positions.

<u>Computer Factored Positive and Negative Moments</u>

Multiple presence factor, m:

With one lane loaded, m = 1.20
With two lanes loaded, m = 1.00
With three lanes loaded, m = 0.85

The maximum moments are calculated by the following formula:

$$M_{\max} = m\gamma_{LL}(1+IM)M_{LL} + \gamma_{DC}M_{DC}$$

Maximum Positive Moment:

Strength I:
One lane loaded:

$$M_{\max} = 1.20 \times 1.75 \times (1+0.33) \times 17.779 kip.in \times 16 kip + 1.25 \times 1.350 kip.in \times \frac{55 in}{12 in/ft} = 66.85 kip.ft$$

Two lanes loaded:

$$M_{\max} = 1.00 \times 1.75 \times (1+0.33) \times 18.539 kip.in \times 16 kip + 1.25 \times 1.350 kip.in \times \frac{55 in}{12 in/ft} = 58.18 kip.ft$$

Service II:
One lane loaded:

$$M_{\max} = 1.20 \times 1.30 \times (1+0.33) \times 17.779 kip.in \times 16 kip + 1.00 \times 1.350 kip.in \times \frac{55 in}{12 in/ft} = 49.70 kip.ft$$

Two lanes loaded:

$$M_{\max} = 1.00 \times 1.30 \times (1+0.33) \times 18.539 kip.in \times 16 kip + 1.00 \times 1.350 kip.in \times \frac{55 in}{12 in/ft} = 43.25 kip.ft$$

Maximum Negative Moment:

Strength I:
One lane loaded:

$$M_{\max} = -(1.20 \times 1.75 \times (1+0.33) \times 17.2911 kip.in \times 16 kip + 1.25 \times 1.898 kip.in \times \frac{55 in}{12 in/ft})$$

$$= -65.30 kip.ft$$

Two lanes loaded:

$$M_{\max} = -(1.00 \times 1.75 \times (1+0.33) \times 21.501 kip.in \times 16 kip + 1.25 \times 1.898 kip.in \times \frac{55 in}{12 in/ft})$$

$$= -67.63 kip.ft$$

Service II:
One lane loaded:

$$M_{\max} = -(1.20 \times 1.30 \times (1+0.33) \times 17.291 kip.in \times 16 kip + 1.00 \times 1.898 kip.in \times \frac{55in}{12in/ft})$$
$$= -48.56 kip.ft$$

Two lanes loaded:

$$M_{\max} = -(1.00 \times 1.30 \times (1+0.33) \times 21.501 kip.in \times 16 kip + 1.00 \times 1.898 kip.in \times \frac{55in}{12in/ft})$$
$$= -50.29 kip.ft$$

Therefore the design moments for the deck are:

Table 3 Summary of control moment in deck

	Maximum positive moment (kip-ft)	**Maximum negative moment (kip-ft)**
Strength I	66.85	-67.63
Service II	49.70	-50.29

<u>Design for Positive Flexure in Deck</u>

Check the maximum stresses in deck against the yielding stress of steel.

$$\sigma_{\max} \le \phi_f \cdot F_y \qquad \phi_f = 1.0$$

As provided by American Grid, the sectional properties are:

$$W_{top} = 6.467 in^3/ft \qquad W_{bottom} = 7.326 in^3/ft$$

Maximum stress from strength I is:

$$\sigma_{\max} = \frac{M_{\max}}{W_{top}} = \frac{66.85 kip.ft \times 12in/ft}{6.467 in^3/ft \times 55in/(12in/ft)} = 27.1 ksi \le 1.0 \times 50 ksi \qquad \text{OK}$$

Maximum stress from service II is:

$$\sigma_{\max} = \frac{M_{\max}}{W_{top}} = \frac{49.70 kip.ft \times 12in/ft}{6.467 in^3/ft \times 55in/(12in/ft)} = 20.1 ksi \le 1.0 \times 50 ksi \qquad \text{OK}$$

<u>Design for Negative Flexure in Deck</u>

Check the maximum stresses in deck against the yielding stress of steel.

$$\sigma_{\max} \le \phi_f \cdot F_y \qquad \phi_f = 1.0$$

As provided by American Grid, the sectional properties are:

$$W_{top} = 6.467 in^3/ft \qquad W_{bottom} = 7.326 in^3/ft$$

Maximum stress from strength I is:

$$\sigma_{\max} = \frac{M_{\max}}{W_{top}} = \frac{67.63kip.ft \times 12in/ft}{6.467in^3/ft \times 55in/(12in/ft)} = 27.4ksi \le 1.0 \times 50ksi \qquad \text{OK}$$

Maximum stress from service II is:

$$\sigma_{\max} = \frac{M_{\max}}{W_{top}} = \frac{50.29kip.ft \times 12in/ft}{6.467in^3/ft \times 55in/(12in/ft)} = 20.4ksi \le 1.0 \times 50ksi \qquad \text{OK}$$

<u>Check for Live Load Deformation in Deck</u>

This maximum live load deflection is computed based on the following:

1. All design lanes are loaded.
2. The number and position of loaded lanes is selected to provide the worst effect.
3. The live load portion of Service II Limit State is used.
4. Dynamic load allowance is included.

Finite element analysis is performed in determining the maximum live load deflection in deck. The equivalent strip of deck is modeled with beam element. Based on the equivalent interior strip model and the sectional properties that are provided by American Grid, the properties of the beam element are:

Equivalent strip width: $W_E = 55in$

Sectional properties of the steel grid deck:

$$W_{top} = 6.467in^3/ft \qquad W_{bottom} = 7.326in^3/ft$$

$$h = 5 - 3/16\,in$$

Assume that the moment of inertia is $I \;\; in^4$ and the distance from top fiber to the neutral axis is $x \;\; in^3$, then:

$$W_{top} = \frac{I}{x} \qquad W_{bottom} = \frac{I}{h-x}$$

Therefore, solve the equation group, we have:

$$x = 2.7553in \quad I = 17.8185in^4/ft$$

For the equivalent strip,

$$I_E = I \cdot W_E = 17.8185in^4/ft \times \frac{55in}{12in/ft} = 81.6681in^4$$

Service II limit state:

$$\gamma_{LL} = 1.00 \qquad \gamma_{IM} = 1.00 \qquad IM = 0.33$$

Design truck HS-20:

Wheel load: $16kips$

Load on equivalent strip:

$$1.00 \times (1 + 0.33) \times 16kips = 21.28kips$$

Finite element analysis is performed based on the above information and the results of maximum upwards and downwards deformations are given in Table.

Table 4 Live Load Deformation of Deck

	$\Delta_{\max,up}$ (in)	$\Delta_{\max,down}$ (in)		$\Delta_{\max,up}$ (in)	$\Delta_{\max,down}$ (in)		$\Delta_{\max,up}$ (in)	$\Delta_{\max,down}$ (in)
Case1	0.03482	0.07815	**Case13**	0.03105	0.05382	**Case25**	0.04655	0.09320
Case2	0.02487	0.08652	**Case14**	0.03403	0.07200	**Case26**	0.03015	0.06066
Case3	0.00444	0.08902	**Case15**	0.03726	0.09108	**Case27**	0.03235	0.06634
Case4	0.00788	0.08022	**Case16**	0.04477	0.09836	**Case28**	0.03532	0.07401
Case5	0.01774	0.06114	**Case17**	0.05589	0.09587	**Case29**	0.03855	0.08235
Case6	0.02681	0.05848	**Case18**	0.06521	0.08902	**Case30**	0.04153	0.09002
Case7	0.03351	0.06285	**Case19**	0.03208	0.05584	**Case31**	0.02484	0.07918
Case8	0.03627	0.05960	**Case20**	0.03299	0.05817	**Case32**	0.02782	0.06437
Case9	0.03312	0.05642	**Case21**	0.03519	0.06385	**Case33**	0.03105	0.07245
Case10	0.02208	0.04848	**Case22**	0.03817	0.07152	**Case34**	0.01747	0.08734
Case11	0.00946	0.04581	**Case23**	0.04140	0.08798	-	-	-
Case12	0.02885	0.04815	**Case24**	0.04438	0.09569	-	-	-

Therefore, maximum downwards deflection is 0.09836in, and maximum upwards deflection is 0.06521in.

$$Span = 8\,ft = 96in$$

$$\Delta_{allowable} = \frac{Span}{800} = \frac{96in}{800} = 0.1200in \quad \text{OK}$$

Design Step 3 - Steel Girder Design

Design Criteria

Governing specifications: *AASHTO LRFD Bridge Design Specifications* (Second Edition, 1998). In the following parts, it is named as AASHTO for brief.

Design methodology: Load and Resistance Factor Design (LRFD)

Basic Design Information

Span length: 60feet
Span number: 1
Skew angle: 0 degree
Number of girders: 4
Girder spacing: 8feet
Deck overhang: 1foot

Cross frame spacing: 20feet
Yielding strength (beam): $F_y = 50ksi$
Deck thickness: 5in
Parapet weight: 23lb/lft
Future wearing surface: none
Deck width: 26feet
Clear road way width: 24feet

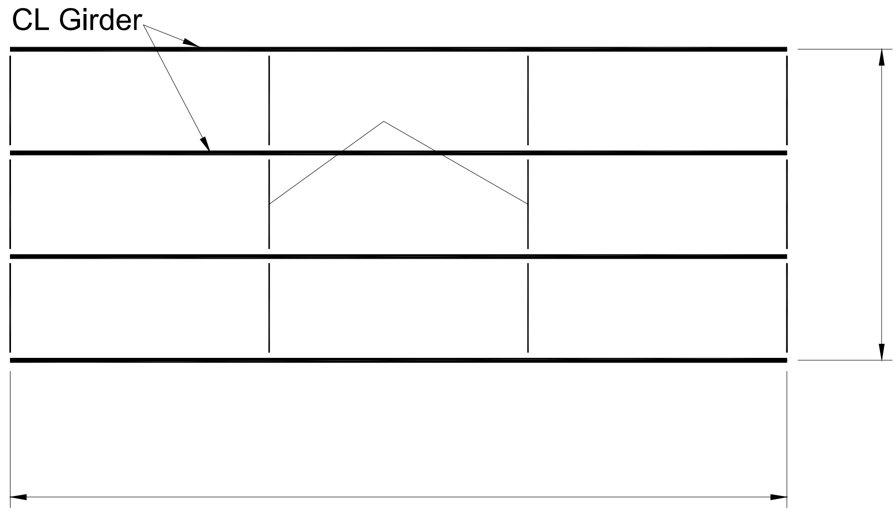


Figure5. Framing plan

Preliminary design

American iron and steel institute shot span steel bridge design program, AISCBEAM, is used to do the preliminary design. The program is based on Load Factor Design. The beam is chosen by the program on the criteria of minimum weight, while in some cases, it is not the least cost alternative.

For this bridge design, the AISCBEAM choose W40×183 as the optimized design section. While based on AISC rolled beam availability survey, this section has limited availability and therefore higher price. Therefore other widely made sections are trailed in AISCBEAM. W36×230 passed the program checking. Therefore, the following parts will check the W36×230 with Load Resistance Factor Design.

W36×230 sectional properties:

W36×230								
Area (in^2)	Depth (in)	Web		Flange		Nominal weight (lb/ft)	Elastic Properties	
		Thickness (in)	Height (in)	Width (in)	Thickness (in)		Axis X-X (in^4)	Axis Y-Y (in^4)
67.6	35.90	0.760	31.125	16.470	1.26	230	15000	940

Determine Dead Load effects

Dead Loads:

Girder self weight: 230lb/ft
Deck weight: 25.9lb/sf
Parapet weight: 23lb/lf

Load Combinations:

It is assumed that all the dead loads, including parapet weight, are distributed evenly among all the girders.

Total dead load on all the girders:

$$W_{DC,total} = (W_G + W_{DE} + W_P) = (0.230 \times 4 + 0.0259 \times 26 + 0.023 \times 2) = 1.6394 kip/ft$$

Dead load on each girder:

$$W_{DC} = \frac{W_{DC,total}}{4} = 0.4099 kip/ft$$

Analysis model:

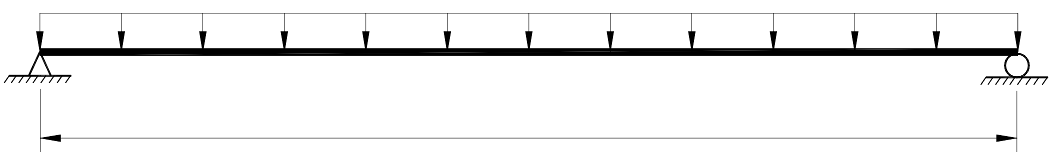


Figure6. Dead load analysis model

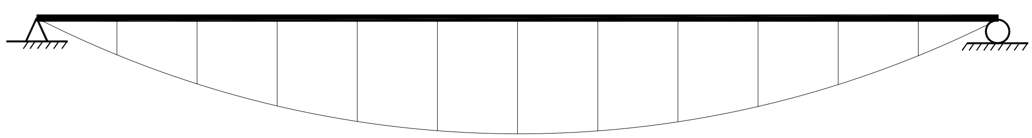


Figure7. Dead load moment distribution

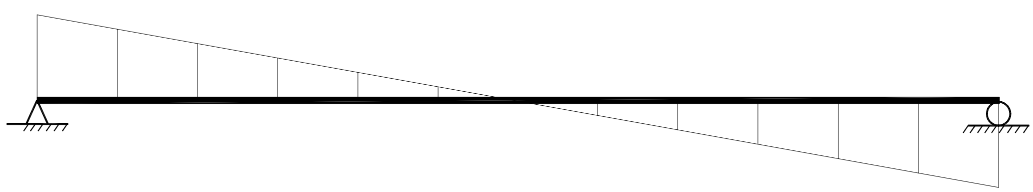


Figure8. Dead load shear distribution

$$M_{\max} = \frac{1}{8} W_{DC} L^2 = \frac{1}{8} \times 0.4099 kip/ft \times 60^2 = 184.43 kip.ft$$

The maximum moment occurs at the mid span.

$$V_{\max} = \frac{1}{2} W_{DC} L = \frac{1}{2} \times 0.4099 kip/ft \times 60 ft = 12.30 kip$$

The maximum shear force occurs at the ends of the girders.

Determine Live Load effects

The design vehicular live load shall consist of a combination of the: (AASHTO, 3.6.1.2)

- Design truck or design tandem, and
- Design lane load

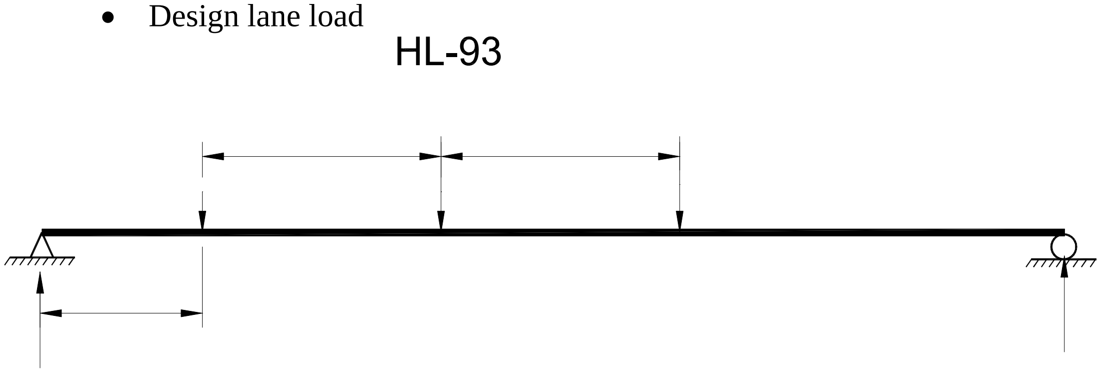


Figure9. HL-93 truck

Computer the maximum moment and shear that created by AASHTO HL-93 truck:

$$R_B = \frac{1}{L}\left[T1X + T2(X+14) + T3(X+14+14)\right]$$
$$= \frac{1}{60}\left[8X + 32(X+14) + 32(X+14+14)\right]$$
$$= 1.2X + 22.4$$

- Maximum moment:

Assume that the maximum moment is at T2:

$$M_{T2} = R_B(60 - X - 14) - 32 \times 14$$

$$\frac{dM_{T2}}{dX} = 1.2(46 - X) - (1.2X + 22.4)$$

Let $\frac{dM_{T2}}{dX} = 0$, then solve for X:

$X = 13.667 ft$

Then substitute X into M_{T2}, we get:

$M_{T2,\max} = 806.53 kip.ft$

Assume that the maximum moment is at T3:

$$M_{T3} = R_B(60 - X - 14 - 14)$$

$$\frac{dM_{T2}}{dX} = 1.2(32 - X) - (1.2X + 22.4)$$

Let $\frac{dM_{T2}}{dX} = 0$, then solve for X:

$$X = 6.667\,ft$$

Then substitute X into M_{T2}, we get:

$$M_{T3,\max} = 770.13 kip.ft$$

Therefore the maximum HL-93 truck load moment is:

$$M_{T2,\max} = 806.53 kip.ft$$

It occurs at 27.667ft from one end support.

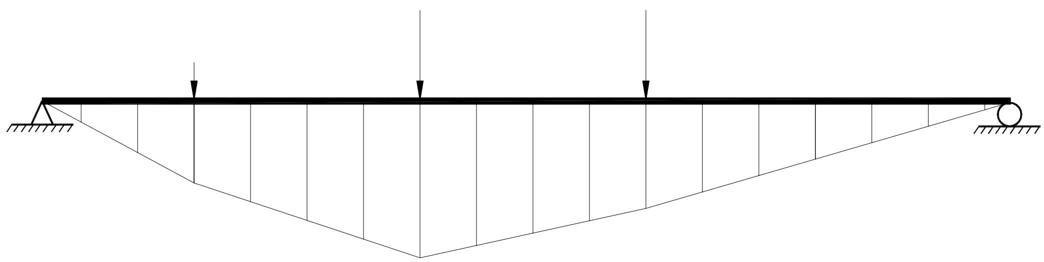


Figure10. HL-93 truck load moment distribution

- Maximum shear

  From $R_B = 1.2X + 22.4$, where $0 \le X \le 32\,ft$, then:

  $$V_{\max} = R_{B,\max} = 1.2 \times 32 + 22.4 = 60.8 kip$$

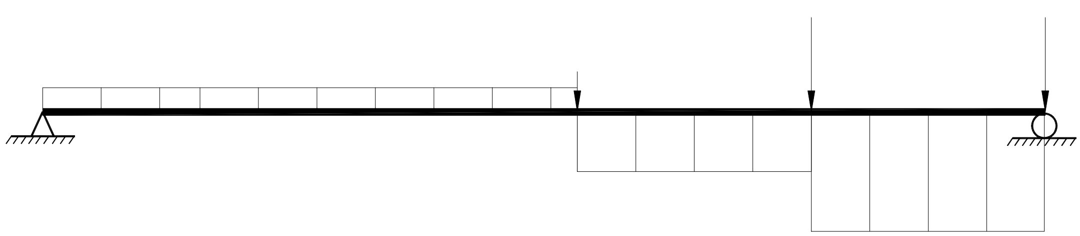


Figure11. HL-93 truck load shear distribution

Computer the maximum moment that created by AASHTO tandem:

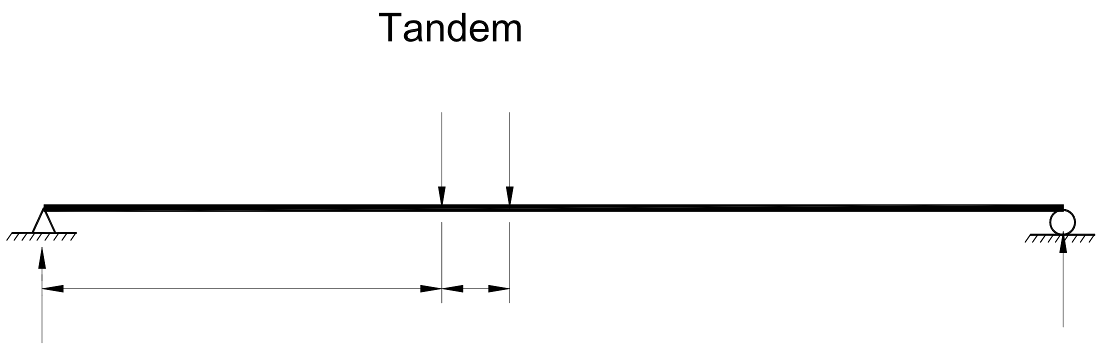


Figure13. Tandem load

- Maximum moment:

$$R_B = \frac{1}{L}\left[T1X + T2(X+4)\right]$$
$$= \frac{1}{60}\left[25X + 25(X+4)\right]$$
$$= \frac{5}{6}X + \frac{5}{3}$$

As known, the maximum moment shall occur under one of the concentrated load:

$$M_{T2} = R_B(60 - X - 4)$$
$$\frac{dM_{T2}}{dX} = \frac{5}{6}(56 - X) - \left(\frac{5}{6}X + \frac{5}{3}\right)$$

Let $\frac{dM_{T2}}{dX} = 0$, then solve for X:

$$X = 27.0\,ft$$

Then substitute X into M_{T2}, we get:

$$M_{T2,\max} = 700.83 kip.ft < 806.53 kip.ft$$

- Maximum shear:

From $R_B = \frac{5}{6}X + \frac{5}{3}$, where $0 \le X \le 56\,ft$, then:

$$V_{\max} = R_{B,\max} = 1.2 \times 56 + 22.4 = 48.33 kip < 60.8 kip$$

Therefore the HL-93 truck controls.

$$M_{truck} = 806.53 kip.ft$$

$$V_{truck} = 60.8 kip$$

Computer the maximum moment that created by AASHTO lane load:

The design lane load shall consist of a load of 0.64klf, uniformly distributed in the longitudinal direction. (AASHTO, 3.6.1.2.4)

$$W_{Lane} = 0.64kip/ft$$

$$M_{lane,\max} = \frac{1}{8}W_{Lane}L^2 = \frac{1}{8}\times 0.64\times 60^2 = 288kip.ft$$

$$V_{lane,\max} = \frac{1}{2}W_{Lane}L = \frac{1}{2}\times 0.64\times 60 = 19.2kip$$

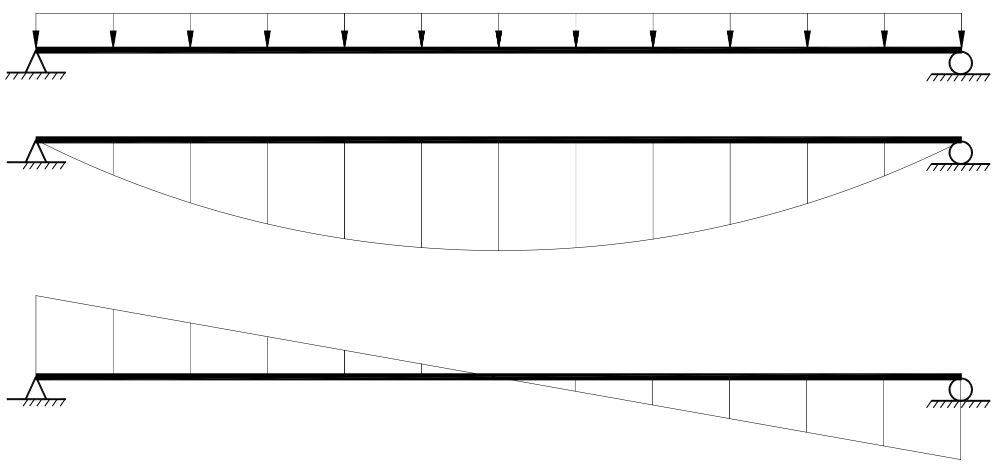


Figure14. Lane load moment and shear distribution

Determine Live Load Distribution Factors

Moment distribution factors:

- Interior girder:

According to AASHTO, Table 4.6.2.2.2b-1,

$S = 8ft \qquad t_g = 5in > 4in$

The moment distribution factor for interior beam is:

$S/10.0 = 8/10.0 = 0.80$

- Exterior girder:

According to AASHTO, Table 4.6.2.2.2d-1, lever rule shall be used to calculate moment distribution factor for exterior girder.

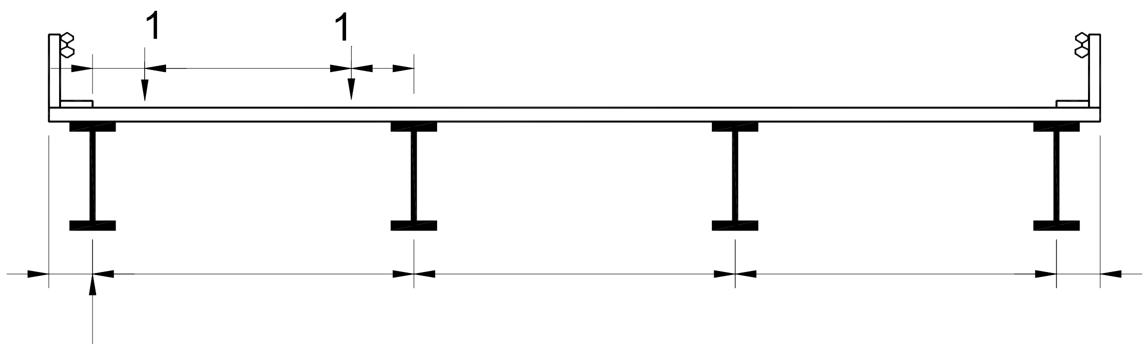


Figure15. Lever rule

One lane loaded:

$$\sum M_A = 0$$

$$R = \frac{1 \times 7 + 1 \times 1}{2 \times 8} = 0.5$$

Check Rigid Body Rule:

$$R = \frac{N_L}{N_b} + \frac{X_{ext} \sum^{N_L} e}{\sum^{N_b} x^2} \quad \text{(AASHTO, 4.6.2.2.2d)}$$

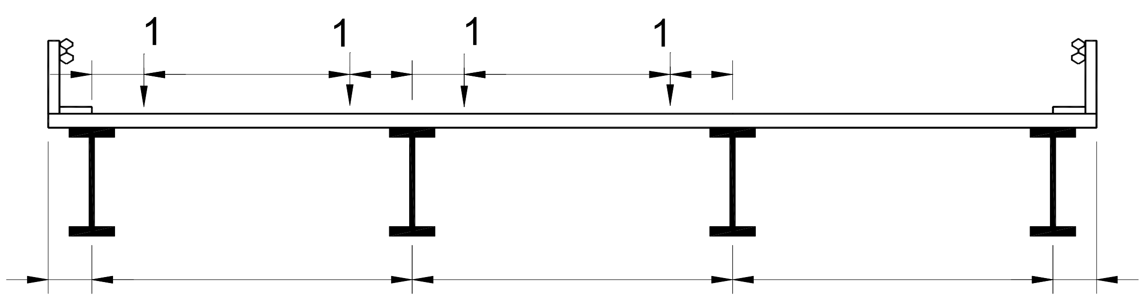


Figure16. Rigid body rule

Number of design lanes: $N_L = 2$

Number of girders: $N_b = 4$

Horizontal distance from the center of gravity of the pattern of girders to the exterior girder: $X_{ext} = 12\,ft$

Eccentricity of a lane load from the center of the pattern of girders:

$e_1 = 11\,ft \qquad e_2 = 5\,ft \qquad e_3 = 3\,ft \qquad e_4 = -3\,ft$

Horizontal distance from the center of gravity of the pattern of girders to each girder:

$x_1 = -x_4 = 12\,ft \qquad x_2 = -x_3 = 4\,ft$

Therefore,

$$R = \frac{2}{4} + \frac{12(11+5+3-3)}{12^2 \times 2 + 4^2 \times 2} = 1.1$$

The moment distribution factor for exterior beam is:

$$R = 1.1$$

Shear distribution factors:

- Interior girder:

According to AASHTO, Table 4.6.2.2.3a-1, the shear distribution factor for interior beam shall be calculated by lever rule.

One lane loaded:

$$\sum M_A = 0$$

$$R = \frac{1\times 7 + 1\times 1}{2\times 8} = 0.5$$

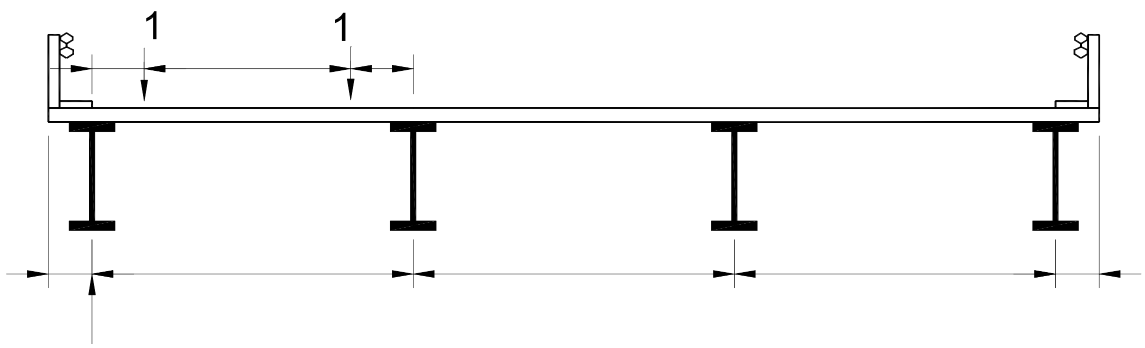


Figure17. Lever rule

- Exterior girder:

According to AASHTO, Table 4.6.2.2.3b-1, the shear distribution factor for interior beam shall be calculated by lever rule.

One lane loaded:

$$\sum M_A = 0$$

$$R = \frac{1\times 7 + 1\times 1}{2\times 8} = 0.5$$

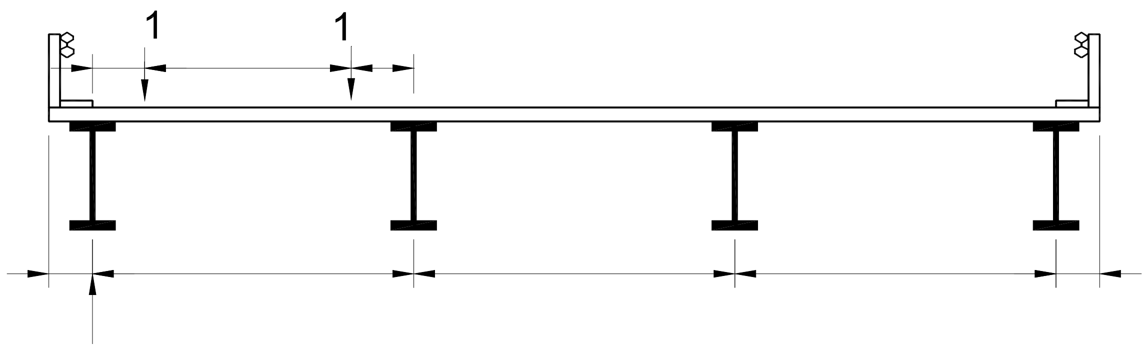


Figure18. Lever rule

AASHTO, 4.6.2.2.1 specifies that multiple presence factors shall not be used with the approximate load assignment methods other than statical moment or lever arm method because these factors are already incorporated in the distribution factors.

Multiple presence factor, m:

With one lane loaded, m = 1.20

With two lanes loaded, m = 1.00

Table 5 Summary of Live Load Distribution Factors

	Moment	**Shear**
Interior girder	0.8×1.0=0.8	0.5×1.2=0.6
Exterior girder	1.1×1.2=1.32	0.5×1.2=0.6

<u>Combine Load Effects</u>

Load Factors: (AASHTO, Table 3.4.1-1 and Table 3.4.1-2)

Load combinations and Load Factors					
Limit States	Load Factors				
	DC		LL	IM	WA
	Min.	Max.			
Strength I	0.90	1.25	1.75	1.75	1.00
Service II	1.00	1.00	1.30	1.30	1.00

Resistance Factors: (AASHTO, 6.5.4.2)

Resistance Factors		
Material	Type Of Resistance	Resistance Factors, ϕ
Structural Steel	For Flexure	$\phi_f = 1.00$
	For Shear	$\phi_v = 1.00$
	For Axial Compression	$\phi_c = 0.90$
	For Bearing	$\phi_b = 1.00$

- Only strength I will control in this design case.
- Adding the maximum moment from dead load and live load together, without considering the small position difference of their occurring sections.
- Design the interior and exterior girder with the same section.

Strength I:

Table6 Summary of Max. Moments and Max. Shear Forces

Load Case		Max. Moment (kip.ft)	Max. Shear (kip)
Dead Load		184.43	12.30
Live Load	Lane Load	288.00	19.20
	Truck Load	806.28	60.80

Maximum moment controlling the design is: (occurs in exterior girder)

$$M_{\max} = 1.25\times184.43 + 1.75\times1.32\times288 + 1.75\times(1+0.33)\times1.32\times806.28$$
$$= 3372.95 kip.ft$$

Maximum moment controlling the design is: (occurs in exterior girder)

$$V_{\max} = 1.25\times12.30 + 1.75\times0.6\times19.20 + 1.75\times0.6\times60.8 = 99.38 kip$$

Check the W36×230:

Flexure capacity:

$$M_p = \left(A_f h + \frac{A_w h}{4}\right)F_y = \left(20.75\times33.38 + \frac{23.66\times33.38}{4}\right)\times50$$
$$= 3708.66 kip.ft > 3372.95 kip.ft \qquad \text{OK}$$

Shear capacity:

$$\frac{D}{t_w} = \frac{31.125}{0.76} = 46.5 < 2.46\sqrt{\frac{E}{F_{yw}}} = 2.46\sqrt{\frac{29000}{50}} = 59.2$$

$$V_n = 0.58F_{yw}Dt_w = 0.58\times50\times31.125\times0.76 = 686.0 kip$$

$$V_r = \phi_v V_n = 1.0\times686.0 = 686.0 kip$$

$$V_{\max} = 99.38 kip < 686.0 kip \qquad \text{OK}$$

<u>Check for the deflection of girders under live loads</u>

According to AASHTO 2.5.2.6.2, criteria for deflection:

- All design lanes are loaded.
- All supporting components are assumed to deflect equally.
- When investigating the maximum relative displacements, the number and position of loaded lanes are selected to provide the worst differential effect.
- The live load portion of Load combination service I of AASHTO, Table 3.4.1-1 is used, including the dynamic load allowance, IM.

Bridge sectional properties

Because all the girders are assumed to have equal deflection, the 4 girders can be group as one. The sectional properties will be:

$$E = 29000ksi \qquad I = 4I_g = 4\times 15000in^4 = 60000in^4 \qquad L = 60\,ft$$

Lane load deflection

Lane load:

$W_{Lane} = 0.64kip/\,ft$

Number of loaded lane:

$n = 2$

Load factor:

$\gamma_{LL} = 1.00$

The maximum deflection occurs at the mid span, which is:

$$\Delta_{Lane,\max} = \frac{5W_{Lane}L^4}{384EI} = \frac{5\times 0.64\times 2\times 60^4}{384\times 29000\times 60000/12^2} = 0.0179in$$

Truck load deflection

Design truck:

HL-93

Number of loaded lane:

$n = 2$

Load factor:

$\gamma_{LL} = 1.00$

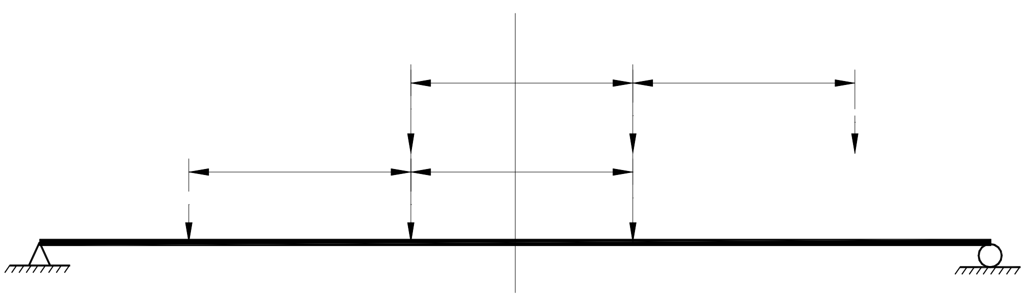


Figure18. Maximum deflection at mid span

The maximum deflection occurs at the mid span when the 2 HL-93 trucks are positioned as shown in Figure 18.

$$\Delta_{truck,\max} = 0.5602in$$

Therefore,

$$\Delta_{\max} = IM(\Delta_{Lane,\max} + \Delta_{truck,\max}) = 1.33 \times (0.0179 + 0.5602) = 0.7689in$$

$$\frac{L}{800} = \frac{60 \times 12}{800} = 0.9000in$$

$$\Delta_{\max} < \frac{L}{800} \qquad \text{OK}$$