Innovative Design and Construction Methods for

Off-System Steel Bridges

by

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Off-System Steel Bridges

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CHAPTER 1
Introduction

The purpose of this report is to give insight into the development of bridges for off-system bridge replacement that provide rapid, cost-effective, and functional solutions. The report is based on a project funded by the Texas Department of Transportation (TxDOT). TxDOT proposed the project because many off-system bridges in Texas are in need of rehabilitation or replacement.

1.1 BRIDGE INVENTORY

TxDOT presented a report that provided information to assist with this bridge design project. In this report, information from the National Bridge Inspection (NBI) database was included to characterize the critical population of off-system bridges. According to the NBI, in Texas there are 2,360 bridge structures classified as structurally deficient and 6,900 structures classified as structurally deficient or functionally obsolete. A structurally deficient bridge is one in which the load carrying capacity of the bridge is insufficient to carry current loads or has been measurably reduced. Functionally obsolete bridges are those in which the geometry in terms or width, span, clearance, etc. does not meet current standards. These functionally obsolete bridges reduce traffic flow or hydraulic flow in the stream. Structurally deficient bridges were presented as the most critical group, followed in importance by the functionally obsolete bridges.

The report presented by TxDOT also provided other useful bridge statistics collected from the NBI.

- Over 95% of the off-system bridges studied cross a waterway.

- Seventy percent of all structurally deficient off-system bridges have a length less than 60 ft, and 50% a length less than 45 ft.
- Seventy percent of all structurally deficient or functionally obsolete off-system bridges have a length less than 85 ft, and 50% a length less than 55 ft.
- Ninety-five percent of all structurally deficient bridges and 77% of all structurally deficient or functionally obsolete bridges have an approach roadway less than 24 ft wide.
- At least 52% of all structurally deficient off-system bridges used pile foundations, compared to 3% using drilled shafts.

These statistics gave important insight into the types of bridges that needed to be designed.

1.2 TXDOT'S RECOMMENDATIONS FOR OFF-SYSTEM BRIDGE SOLUTIONS

Based on their investigations, TxDOT provided a list of recommendations to be applied to the bridge solution. The list included, but was not limited to the following:

- The structure to be considered should be a waterway crossing with occasional overtopping by flood events.
- Span-to-depth ratio should be minimized.
- Overall roadway width of 26 ft including rails should be supported.
- Rail types and means of attachment need not be considered and should not limit the type of structural system developed.
- HS-20 is current design practice.
- The solution should support prestressed concrete piling, steel H-Piles, and drilled shafts.
- Road closure should be limited to one to two weeks.

- Solution should consider remoteness and difficult access.
- Phased construction and temporary bridges will not be possible.

1.3 Initial Ideas Based on NBI Data and TxDOT Recommendations

Most of the bridges were water crossings. When a bridge crosses a river or stream, it is important that it has sufficient hydraulic openings. Also, many streams in Texas flood in the rainy season. A bridge built over a waterway must allow room for water and debris to flow smoothly during a flood. If debris gets caught on a bridge during a flood it can block the stream flow and cause severe hydraulic forces to act on the bridge. A single span bridge is the best design to avoid debris blockage. A multiple span bridge would have piers that may be under water during flooding. The piers would disrupt stream flow and could lead to a blocked passage in severe weather conditions.

Another important fact from the statistics above is the range of span lengths of structurally deficient or functionally obsolete bridges. Most of the bridges are less than 85 ft in total length. By definition, the minimum span of a bridge is 20 ft. Therefore, the bridge spans that were focused on in this project were between 20 ft and 100 ft.

All recommendations by TxDOT were taken into account and are discussed in the following chapters.

1.4 Project Goal

The goal of this project was to design a bridge that could be rapidly constructed but was also economically reasonable. The goal construction duration was one week. The cost of the bridge should be comparable to current TxDOT bridge designs.

1.5 FOCUS OF REPORT

Although concrete bridge designs were considered for this project, this report focuses on steel bridge design only. The concrete designs alternatives, precast concrete I-girders and precast concrete double-T's, will not be discussed in this report.

Pre-manufactured bridges played an important role in this project. Pre-manufactured bridges are bridges that are pre-constructed, delivered to a construction site, and assembled at the sight. They can be assembled in a couple of weeks, and are currently a leading solution for rapid bridge replacement. Most pre-manufactured bridges, however, can only span up to 60 ft. Pre-manufactured bridges provide a pre-packaged method to rapidly replace a bridge. A major objective of this project was to design a bridge that is more economical and has a shorter construction time than pre-manufactured bridges. Cost data were collected from a pre-manufactured bridge company and are compared to cost data computed for the bridge designed in this project. This comparison is presented in Chapter 9.

1.6 ABUTMENTS AND FOUNDATION

1.6.1 Abutment Design

The abutments for this bridge project were researched and designed by another party, but will be briefly discussed here.

In order to reduce the amount of time required to construct the abutments in the field, precast concrete will be used instead of cast-in-place concrete. For convenience reasons abutments are usually made of cast-in-place concrete. However, constructing cast-in-place abutments requires many days of placing formwork and reinforcing steel, casting concrete, allowing the concrete to cure, and removing the formwork. The time required for these steps would greatly increase the total construction duration for this project.

Precast concrete abutments will require less than a day's time to construct. Once they arrive at the excavated construction sight, they will have to be lifted by a crane and placed on the excavated soil at the proper elevation. Since they will be placed before the piles, they will act as a template for driving the piles. They will be cast with openings large enough to accommodate the piles and pile-driving equipment. After the piles are driven, small closure pours will be necessary to seal the opening left between the abutment and the piles. The details of this design are not part of this thesis.

1.6.2 Foundation Design

The bridge foundation shall consist of three piles at each abutment. In order to facilitate the construction methods discussed in Chapter 8, the piles will be spaced between the girders. If necessary, drilled shafts may be used instead of piles. The details of the foundation design are not part of this thesis

CHAPTER 2
Background

2.1 Survey of Current Bridge Technology

The first step taken in this project after reviewing the information provided by TxDOT was to survey current bridge technology. This research included looking into both innovative and conventional building materials. Steel bridge systems investigated include trusses, box girders, and basic I-girders. Concrete materials researched include prestressed concrete I-beams, prestressed concrete slab-beams, and prestressed concrete double-T's. Cable-stay and suspension systems were also looked at for spans over 150 ft.

Deck materials that were examined include fiber-reinforced polymer decks, steel grid decks, corrugated metal decks, fully precast concrete decks, precast concrete deck panels with cast-in-place concrete topping, and full depth cast-in-place concrete decks.

This literature search was done using the Internet, printed journals, catalogues, and books. The materials were evaluated based on construction speed, cost, weight, and ease of maintenance. More information about the different materials is discussed in Sections 2.5 thru 2.8.

2.2 Possible Bridge Replacement Locations

While surveying current technology, site evaluation began. A list of Texas bridges in need of replacement was obtained from TxDOT. Eighteen sites from the list were visited. Ten bridge sites were in Austin, Tx and eight bridge sites were in Caldwell County, Tx. Detailed lists of the bridge sites are given in Tables 2.1 & 2.2.

Table 2. 1 Bridge sites visited in Austin

Austin Bridge Sites	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 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 perp. 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 2. 2 Bridge sites visited in Caldwell County

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*	12*	3	Bridge 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 rapid stream flow

*Approximate Length

The lengths of the bridges examined ranged from 20 ft to 900 ft. The bridges were made of many different materials. In Austin, there were four concrete box culverts. Three were rolled steel I-girder bridges, two were concrete arch bridges, and one bridge in Austin was a prestressed concrete beam bridge. In Caldwell County there were two timber bridges, three steel girder bridges, one metal culvert bridge, one steel railroad car bridge, and one bridge that was completely washed out and unidentifiable.

Most of the sights in Austin were in urban locations, which is convenient for construction, because the bridges are accessible. In Caldwell County, however, many bridges were located in remote locations that can only be accessed by one-lane gravel roads. This presents a few problems.

The first problem is that in some situations the nearest alternative stream crossing is miles away. Construction must be rapid so residents of the area will not be inconvenienced for a long period of time.

The second problem is one of construction accessibility. These bridge locations, along with many others in Texas may be located well over 100 miles from the nearest prestressing plant, steel fabricator, or construction supplier. The bridge components must be hauled a long distance, which will cause costs to increase. It would be difficult for trucks to transport large construction equipment and bridge members to bridges in which the access roads are very narrow. The trucks and equipment may also have to cross similar older posted bridges that may not be capable or supporting their loads. Therefore, equipment and member size had to be considered when designing the bridges.

2.3 SURVEY OF EXISTING BRIDGES IN NEED OF REPLACEMENT

2.3.1 Observations

A prominent problem with the bridges in Austin and Caldwell Counties is one of strength. The strength of some bridges are obviously unsatisfactory because the load ratings of the bridges are smaller than they should be. In a few cases in Caldwell County, insufficient strength and corrosion were displayed by cracking. For example, the bridge at County Road 108 at Boggy Creek, which has an allowable axle or tandem load of only 10 kips, has a crack of approximately one-inch thickness through the center of the abutment (Figure 2.1).

Figure 2. 1 CR 108 Abutment

An example of corrosion was found in the bridge at County Road 230 at Boggy Creek. The bridge had several areas where the deck was corroded through, the largest being about four inches long (Figure 2.2). The deck was made out of corrugated metal. Water had collected in the trough depressions causing the corrosion. This bridge had an allowable axle or tandem load of only 5 kips. It was apparent that some truck drivers used a small dirt path around the bridge instead of crossing over it. A typical large truck crossing over the bridge may have an axle weighing 20 kips and would exceed the load rating of this bridge by a factor of four.

Figure 2. 2 CR 230 Deck

2.3.2 Geometry and Safety Problems

A problem found mostly in Caldwell County was a lack of efficient guardrails. Some bridges had guardrails that would not have been able to resist the loads caused by automobiles driving into them. At CR 222 at Cowpen Creek for instance, a culvert had been recently constructed, but the guardrails were made of thin metal tubing that stood approximately one foot above the ground. The same type of guardrail was seen at CR 223 at Elm Creek. If hit by a large vehicle traveling the speed limit, this type of guardrail would likely fail. The guardrail at CR 176 at Cedar Creek was made of timber and was broken in several places.

Four bridges that were visited in Caldwell County had no guardrails at all. Some of the bridges that had no guardrails were CR 108 at Boggy Creek, CR 230 at Boggy Creek, CR 240 at San Marcos River, and CR 262 at San Marcos River.

Another serviceability problem was bridge width. In the rapidly growing city of Austin, many of the bridges are too narrow to handle the current amount of traffic. Barton Creek Rd. at Barton Springs is one example of this. This undivided four-lane bridge is a landmark bridge at the entrance to Zilker Park in Austin. The roadway leading to the bridge is currently being widened so that it will be a divided roadway. With the widening of the roadway, the approaches and bridge will need to be rehabilitated to accommodate the traffic flow.

2.3.3 Span Lengths

The bridge lengths considered for this project range from 20 ft to 100 ft. Three bridges that we visited in Austin were considerably longer than 100 ft, and are therefore not within the scope of this project. The bridge at E 7th Street and Tillery Street, for example, was approximately 900 ft long. The bridge at Barton Springs Road over Barton Creek (255 ft) and the bridge at Red Bud Trail over the Colorado River (152 ft) were also too long for the scope of this project. Other bridges visited in Austin ranged from 20 ft to just over 100 ft, and were within the scope of this project. The bridges in Caldwell County ranged from 30 ft to 70 ft in length and were therefore also acceptable for the scope this project. For more detailed information about the bridge lengths see Tables 2.1 & 2.2.

2.4 TECHNOLOGIES ELIMINATED FROM CONSIDERATION

After surveying typical bridge sights, evaluating construction methods, and examining cost studies, some building materials were eliminated from further consideration. These technologies were eliminated based on cost, construction, and durability considerations.

2.4.1 Deck Systems

2.4.1.1 FRP Decks

A Fiber Reinforced Polymer Deck system was one of the first deck types considered. FRP decks are strong, lightweight, and easy to install. However, after investigating FRP decks further, they were found an unfit deck material for this project. The estimated price per square foot of an FRP deck was $45. When compared to a concrete deck, which is only approximately $10 per square foot, FRP decks are extremely expensive. Maintenance is also considered a negative factor for FRP decks because FRP is a relatively new material. Most of the bridges in need of replacement are in remote locations where skills needed to repair these decks are not available. FRP decks might be reconsidered in the future if the market increases and the price decreases.

2.4.1.2 Corrugated Metal Decks

Another type of deck that was investigated for this bridge design was a corrugated metal deck. These decks are lightweight, inexpensive, and easy to construct. Corrugated metal decks were removed from further consideration, however, due to the impact of corrosion upon them. The bridge on CR230 at Boggy Creek had a corrugated metal deck that was completely rusted through in some places. The corrosion was caused by water that collected on the corrugated metal in places where asphalt had eroded.

2.4.1.3 Full Depth Cast-in-Place Concrete Decks

The most prominently used deck in bridge design is the full depth cast-in-place concrete deck. At $10 per square ft., this is a very cost efficient deck design. It is also the most well known in the field, meaning it would be easy to find laborers in rural Texas with experience in constructing cast-in-place decks. However, this deck is inefficient it terms of construction speed. When constructing CIP decks, formwork is required. Placement and removal of formwork as well as curing time for the deck add to construction time, significantly increasing the total duration of the project.

2.4.1.4 Fully Precast Concrete Decks

Fully precast concrete decks were initially considered because no forms are required for their construction. However, they were found to be very difficult to construct. When constructing a precast deck, problems often arise in post tensioning, grouting, shimming and connecting the panels to the girders. Therefore, full depth precast panels were not further considered as a deck replacement alternative.

2.4.2 Bridge Systems

Steel truss bridges were considered in the initial stages of this project. The construction of a truss bridge can be rapid if the truss is assembled in a yard prior to bridge construction. However, the size of a steel truss presents a problem. A truss may be too large to be transported by truck to these remote locations. Construction of a truss would also require large lifting equipment, which may also not be attainable at a sight in a remote location.

Steel box girders were also initially considered, but are also heavy, and therefore have the same construction problems as steel trusses.

2.5 Steel Bridges Selected for Detailed Evaluation

The girder types selected for bridge replacements are rolled steel wide flanged beams and welded steel plate girders. The deck types we chose for further consideration were steel grid decks (SGD) and precast stay-in-place (SIP) pretensioned concrete panel forms with cast-in-place (CIP) concrete topping.

2.5.1 Steel I-Shaped Girders

Although many types of steel bridges were looked at, the chosen bridge design is an I-girder bridge. I-girders are smaller than trusses or box girders, and will be easier to install because of the rural location of most of the bridges in need of replacement. Smaller, more lightweight members will be easier to transport to remote locations, and will also be easier to maneuver once they are there.

Besides being lightweight, there are a few other advantages of using steel I-shaped girders. Rolled shapes are readily available because they are rolled in many locations across the United States and Texas. Steel I-girders are relatively inexpensive on a per pound basis. Steel girders are also easy to maintain when weathering steel is used.

A disadvantage of steel girders is that longer span steel girders require large sections, and in turn become expensive. Steel girder design is discussed in Chapter 4.

2.5.2 Steel Grid Decks

The steel grid deck is advantageous because of its ease of placement. After the beams are placed, the grid deck must simply be laid on top of the beams and connected. Another positive factor of the steel grid is that it is open so that in case of flood conditions it would allow water and some debris to flow through. The disadvantage of the steel grid deck is that the price is approximately $27 per square foot. Steel grid decks are further discussed in Chapter 3.

2.5.3 Precast Concrete Deck Panels with CIP Topping

The other deck alternative is a precast SIP concrete panel deck with a CIP concrete topping. Precast concrete panels approximately 8 ft by 6.5 ft by 4 in thick are transported to the construction sight and placed on the girders. A CIP concrete topping is then placed on top of the deck. This deck type is similar to a full depth CIP deck, but the precast panels serve as the formwork. This type of deck was chosen over a full depth CIP deck for the concrete deck alternative because less construction time is needed since formwork is not required. Precast concrete panel decks are discussed further in Chapter 3.

2.6 Basic Bridge Configuration

During the design process, eight span lengths were considered. Bridge lengths studied included 20, 30, 40, 60, 70, 80, 90, and 100 ft. These lengths represent a range of bridge spans found in the field during site evaluation and were used as design examples. The width of the bridges was 26 ft as specified by TxDOT.

All bridge designs considered for rapid and economic replacement in this project will be single span, simply supported structures. A single span structure with supports on each bank is the best bridge design for a waterway crossing. If flooding occurs, a single span structure will allow the clearest passage through the streambed, disrupting the least amount of water and debris.

A multiple span bridge would require an extra pier and foundation in the streambed, disrupting stream flow and also increasing the construction time of the bridge. Although they are not being discussed in this project, multiple span designs could be used for bridges spanning over 100 ft in length, using pier construction methods similar to those for the abutments.

As mentioned in Chapter 1, TxDOT recommended guardrail design not be included in this project.

2.7 STEEL BRIDGE OPTIMIZATION

Many aspects of the bridge configuration were examined to determine the most efficient design possible. These aspects include number of girders, girder spacing, using rolled wide flanged shapes versus using welded plate girders, and using a composite versus a non-composite deck

Chapters 5, 6, and 7 discuss these aspects and the steps taken to optimize the bridge design.

CHAPTER 3

Deck Design

3.1 DECK ALTERNATIVES

There were two different deck alternatives chosen for the bridge. The first is a steel grid deck. A steel grid is the most rapidly constructible alternative, but also the most expensive. The second alternative, precast concrete panels with cast-in-place topping, is less expensive than a steel grid deck. However, it will take longer to construct.

3.2 STEEL GRID DECK

3.2.1 Steel Grid Deck Background

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 in which the duration of bridge or 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. These two types of grid decks are discussed in Section 3.2.2.

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.

3.2.2 Types of Steel Grid Decks

There are two main types of steel grid decks. They are grid reinforced concrete decks (Figure 3.1) and open grid decks (Figure 3.2). 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 3.1 and 3.2) 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 3.3) 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 3.4) resemble the four-way grid deck system, but have riveted connections instead of welded connections.

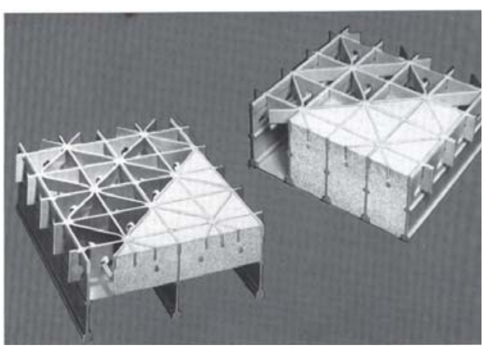


Figure 3. 1 Four-Way Filled Grid

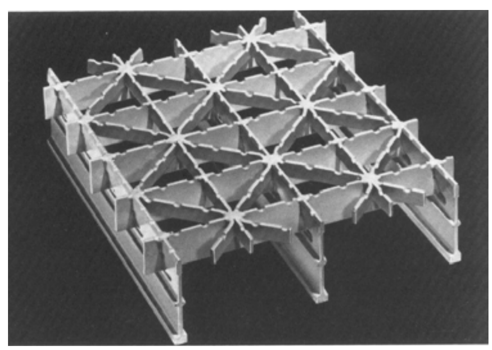


Figure 3. 2 Four-Way Open Grid

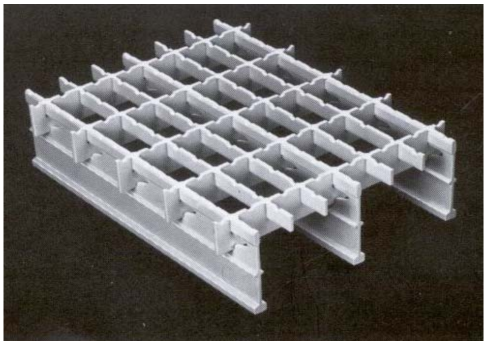


Figure 3. 3 Two-Way Open Grid

Figure 3. 4 Riveted Deck

3.2.3 Steel Grid Deck Selection

3.2.3.1 Construction Speed

An open grid deck was chosen instead of a concrete filled grid deck because it will take less time to construct. An open deck is simply placed on the girders and attached with a simple connection. A filled concrete deck system increases the total construction time for the bridge since additional time is needed for the concrete to be poured and to cure.

3.2.3.2 Cost

The cost of the steel grid deck also had to be considered when choosing which one was most suitable. 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 Chapter 6, 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 3.5.

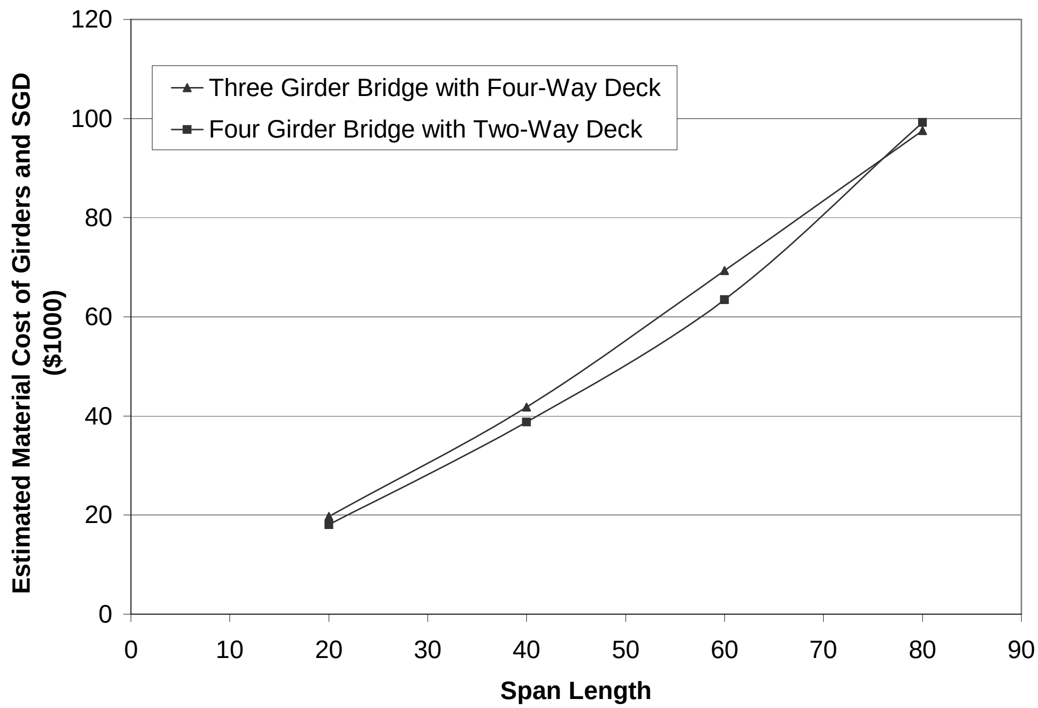


Figure 3. 5 Cost Comparisons for Three-Girder Bridge with Four-Way Open Grid Deck and Four-Girder Bridge with Two-Way Open Grid Deck

Figure 3.5 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 estimation of this price is discussed in Section 7.2.2. The price data for the grid decks are discussed in the previous paragraph. 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.

3.2.3.3 Weights

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.

3.2.4 Grid Deck Chosen and Used in Design Calculations

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 3.3, and is available in grade A36 steel (F_y = 36 ksi) or grade 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 is the one in which main bars were spaced every 6 in. This deck had a clear span capacity of 6.85 ft. The deck that could have supports spaced at 7.75 ft was chosen because it could facilitate the range of spacings being studied at that time.

3.3 PRECAST CONCRETE PANELS WITH CAST-IN-PLACE CONCRETE TOPPING

Most bridges in the United States have cast-in-place (CIP) concrete deck systems. However, many other concrete deck systems have been developed for construction of new bridges and replacement of deteriorated bridge decks. One of these systems is the precast SIP prestressed concrete deck panel system.

This deck consists of panels of 3 to 4 in. in depth that function 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 of the hinge 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.

3.3.1 Overhangs

With the configuration of the design bridge, 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.

3.3.2 SIP Panel Deck Design

The design concrete deck was 8.5 in. thick. The panels used should be 4 in. thick. 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 3.6 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 to the precast panels. 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.

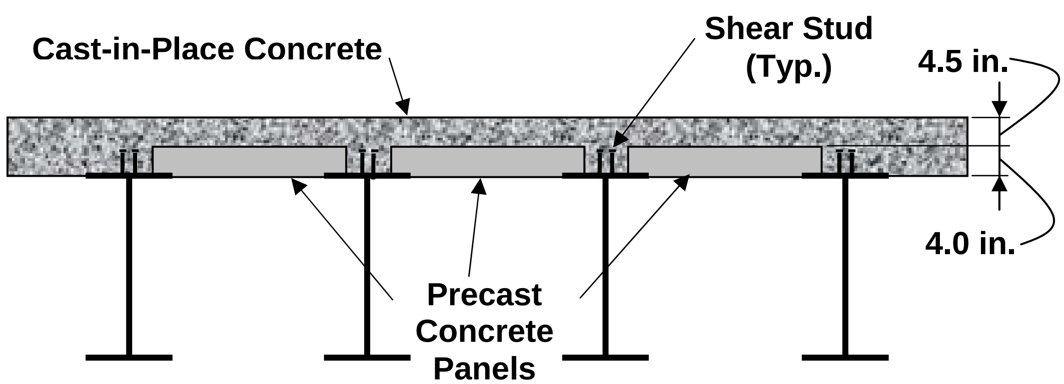


Figure 3. 6 Partial Precast Deck and Shear Studs

3.4 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 panel and removable forms, pour the concrete, allow for curing of the concrete, and removal of the forms. In a typical construction project, four days is only a fraction of the total construction time. However, since the goal construction duration for this project was approximately one week, four days of construction makes a significant difference.

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 4
Steel Bridge Design

4.1 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.

4.1.1 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.

4.1.2 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.

4.1.3 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 (Section 7.2.1). 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.

4.2 BRIDGE LOADING

4.2.1 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 4.1 was developed to determine for which span lengths the tandem load controlled, and for which span lengths the HS20 truck live load controlled.

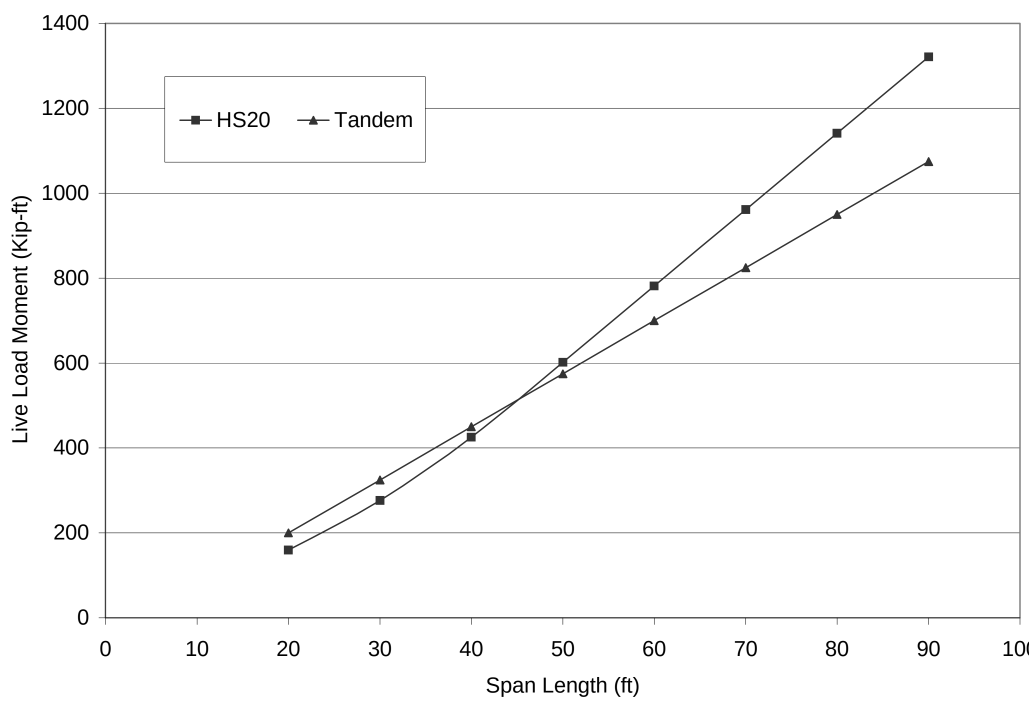


Figure 4. 1 Comparing Live Load Moments

As seen in Figure 4.1, 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.

4.2.2 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.

4.3 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 4.2). 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.

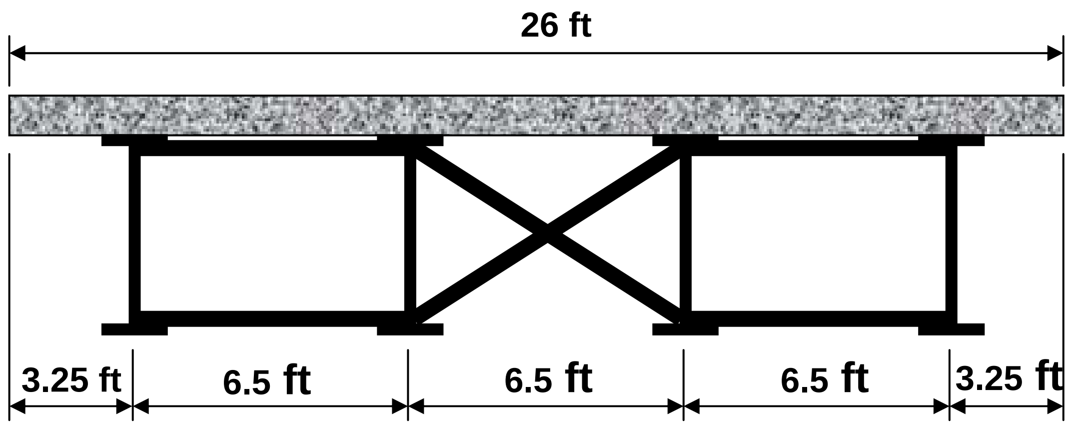


Figure 4. 2 Typical Bridge Cross Bracing Layout

4.3.1 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.

4.3.2 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.

4.3.3 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.

4.3.3.1 Plate Girder Properties

$t_{tf} = 2\,\text{in}$	$t_{web} = 0.5\,\text{in}$	$t_{bf} = 1.6875\,\text{in}$
$b_{tf} = 12\,\text{in}$	$d_{web} = 50\,\text{in}$	$b_{bf} = 12\,\text{in}$

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

$$F_y = 50\,\text{ksi}$$

$$E = 29000\,\text{ksi}$$

The total depth of each girder is

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

The cross-sectional area of the girder is

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

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.

4.3.3.2 Moment Calculations

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

$$M_{cr} = 3.14 \cdot E \cdot \left(\frac{I_{yc}}{L_b}\right) \cdot \left[0.772 \cdot \left(\frac{J}{I_{yc}}\right) + 9.87 \cdot \left(\frac{d}{L_b}\right)^2\right]^{\frac{1}{2}} \qquad \text{(Eqn. 4.1)}$$

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 4.1,

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

From AISI's Bridge Design Program,

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

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.

4.3.3.3 Bracing Design

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

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

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

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

$$M_{br} = \frac{0.005 \cdot L_b \cdot span \cdot (M_{max})^2}{h \cdot n \cdot E \cdot I_{eff} \cdot {C_{bb}}^2} \qquad \text{(Eqn. 4.2)}$$

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 4.2,

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

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}.

$$F_{cr} = Fy_{br} \cdot \left[1 - \frac{Fy_{br} \cdot \left(\frac{L_{br}}{rx_{br}} \right)^2}{\left(4 \cdot \pi^2 \cdot E\right)} \right] = 18.25\,\text{ksi} \qquad \text{(Eqn. 4.3)}$$

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.

4.3.3.4 Stiffness

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

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_g} \qquad \text{(Eqn. 4.4)}$$

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.

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

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

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.

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

Referring back to Equation 4.4,

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

$$\beta_{T,\text{req'd.}} = \frac{2.4 \cdot \text{span} \cdot {M_{max}}^2}{n \cdot E \cdot I_{eff} \cdot {C_{bb}}^2} = 81980\,\frac{\text{in} \cdot \text{kip}}{\text{rad}} \qquad \text{(Eqn. 4.5)}$$

$\beta_b > \beta_t$, 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. For the bracing configuration used here, however, a force of $4M_{br}/S$ results. This force will be designated as F_B and will act on the interior girders at each bracing point (see Figure 4.3). This force will act upward on one girder and downward on the other.

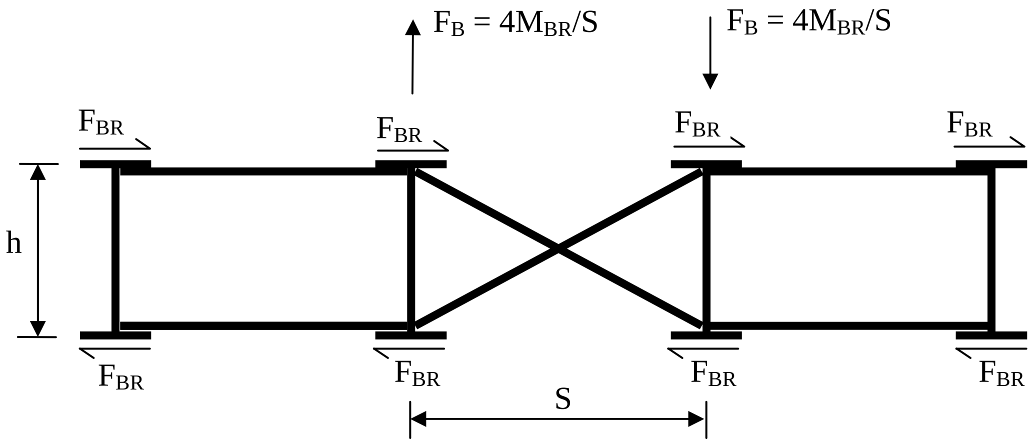


Figure 4. 3 Diagram of Brace Forces Acting on Girders

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 $F_B L/2$.

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

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

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.

CHAPTER 5
Composite Vs. Non-Composite Bridge Design

5.1 Background

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 chapter 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 5.6.

Feasibility of a composite deck also had to be considered. Whether or not the decks chosen in the project could be composite with the steel girders had to be determined. 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 in Sections 5.2 and 5.3, respectively.

5.2 Composite Feasibility for 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.

5.3 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.

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. Shear studs are discussed in Section 5.4.1.

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.

5.4 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 5.1. The estimated costs of the girders, based on "per pound" prices discussed in Section 7.2, are compared in Figure 5.2. Information for both rolled beams and plate girders are shown in these figures.

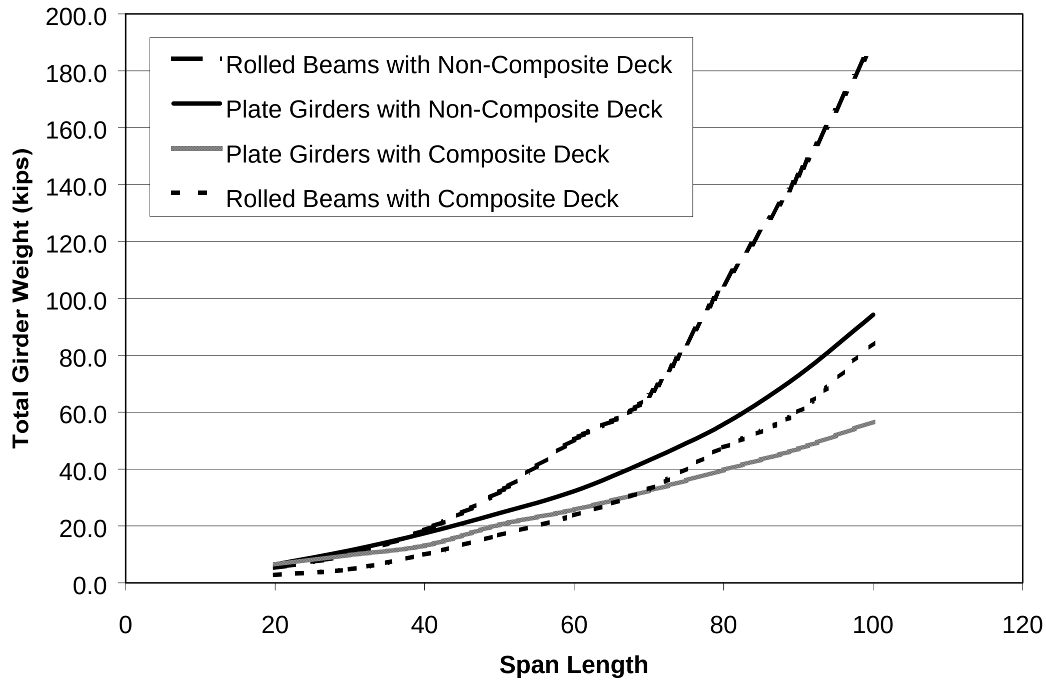


Figure 5. 1 Comparison of Composite Vs. Non-Composite Girder Weight

Figure 5.1 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 5.2 shows the cost comparisons. These costs are based on unit costs of $0.35 or $0.50 per pound for rolled beams (see Section 7.2.1.1), $0.62 per pound for plate girders, and $1.00 per shear stud.

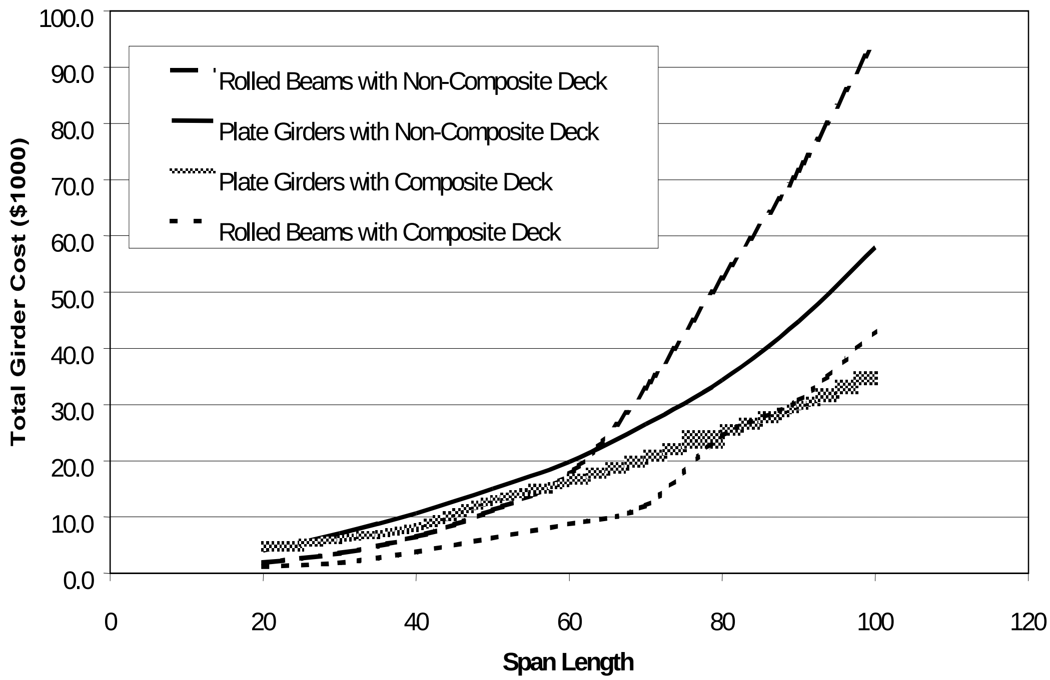


Figure 5. 2 Comparison of Composite Vs. Non-Composite Girder Cost

When comparing girder costs, shear stud costs were included for composite designs. As seen in Figure 5.2 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 on the information discussed in Section 5.2 and the data shown in Figure 5.2 Composite bridges were used for concrete deck bridges and non-composite designs were used for bridges with steel grid decks.

CHAPTER 6
Number and Spacing of Girders

6.1 OPTIMIZING NUMBER OF GIRDERS

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. Many girder number and spacing combinations were designed and are discussed in this section.

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. The girder sizes are shown in Table 6.1 and the weights are plotted in Figures 6.1, 6.2, 6.3, and 6.4.

Table 6. 1 Girder Number Optimization Data

Designation	Span Length (ft)	Deck Thickness (in)	# of girders	Interior Girder Type			Exterior Girder Type			Weight per interior girder (kips)	Weight per exterior girder (kips)	Total Girder Weight (kips)
				tf	web	bf	tf	web	bf			
P20-8.5-3	20	8.5	3	12x3/4	13x1/2	12x3/4	12x3/4	13x1/2	12x3/4	1.67	1.67	5.00
P20-8.5-4	20	8.5	4	12x3/4	12x1/2	12x3/4	12x3/4	12x1/2	12x3/4	1.63	1.63	6.53
P20-8.5-5	20	8.5	5	12x3/4	12x1/2	12x3/4	12x3/4	12x1/2	12x3/4	1.67	1.67	8.34
R20-8.5-3	20	8.5	3		W24x55			W24x55		1.10	1.10	3.30
R20-8.5-4	20	8.5	4		W21x44			W21x44		0.88	0.88	3.52
R20-8.5-5	20	8.5	5		W18x40			W18x40		0.80	0.80	4.00
				tf	web	bf	tf	web	bf			
P40-8.5-3	40	8.5	3	12x3/4	32x1/2	12x3/4	12x3/4	32x1/2	12x3/4	4.63	4.63	13.88
P40-8.5-4	40	8.5	4	12x3/4	28x1/2	12x3/4	12x3/4	28x1/2	12x3/4	4.36	4.36	17.42
P40-8.5-5	40	8.5	5	12x3/4	26x1/2	12x3/4	12x3/4	26x1/2	12x3/4	4.22	4.22	21.10
R40-8.5-3	40	8.5	3		W33x130			W33x130		5.20	5.20	15.60
R40-8.5-4	40	8.5	4		W30x116			W30x116		4.64	4.64	18.56
R40-8.5-5	40	8.5	5		W30x99			W30x99		3.96	3.96	19.80
				tf	web	bf	tf	web	bf			
P60-8.5-3	60	8.5	3	12x1-9/16	40x1/2	12x1-1/16	12x1-9/16	40x1/2	12x1-1/16	10.51	10.51	31.54
P60-8.5-4	60	8.5	4	12x1-1/8	40x1/2	12x3/4	12x1-1/8	40x1/2	12x3/4	8.68	8.68	34.71
P60-8.5-5	60	8.5	5	12x3/4	39x1/2	12x3/4	12x3/4	39x1/2	12x3/4	7.66	7.66	38.28
R60-8.5-3	60	8.5	3		W40x235			W40x235		14.10	14.10	42.30
R60-8.5-4	60	8.5	4		W40x183			W40x183		10.98	10.98	43.92
R60-8.5-5	60	8.5	5		W40x149			W40x149		8.94	8.94	44.70
				tf	web	bf	tf	web	bf			
P80-8.5-3	80	8.5	3	14x1-1/2	50x1/2	14x1-1/8	14x1-1/2	50x1/2	14x1-1/8	16.81	16.81	50.43
P80-8.5-4	80	8.5	4	12x2	40x1/2	12x1-11/16	12x2	40x1/2	12x1-11/16	17.49	17.49	69.96
P80-8.5-5	80	8.5	4	12x1-7/16	40x1/2	12x1-7/16	12x1-7/16	40x1/2	12x1-7/16	14.84	14.84	74.18
R80-8.5-3	80	8.5	3		W36x359*			W36x359*		32.53	32.53	97.59
R80-8.5-4	80	8.5	4		W36x328			W36x328		26.24	26.24	104.96
R80-8.5-5	80	8.5	5		W40x264			W40x264		21.12	21.12	105.60

* Cover Plate Dimensions 80 ft x 14 in x 1 in

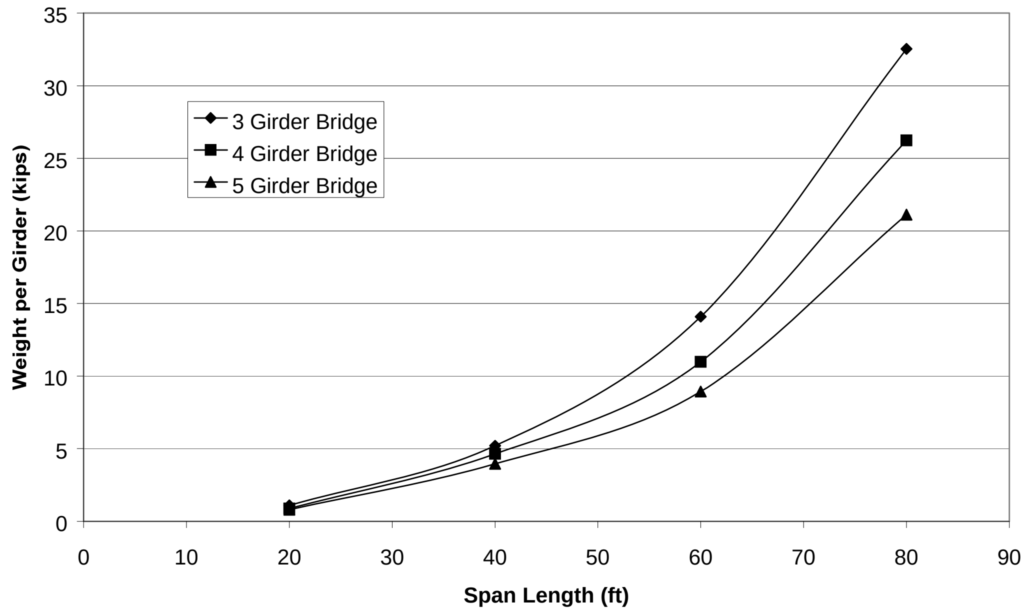


Figure 6. 1 Weight per Girder versus Span Length for Rolled Beams

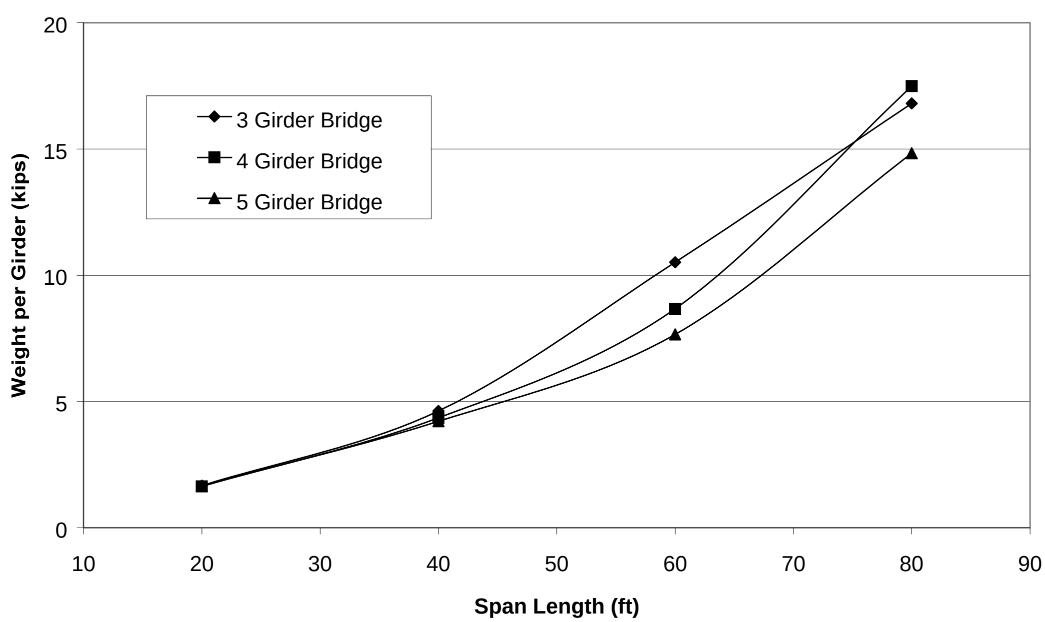


Figure 6. 2 Weight per Girder versus Span Length for Plate Girders

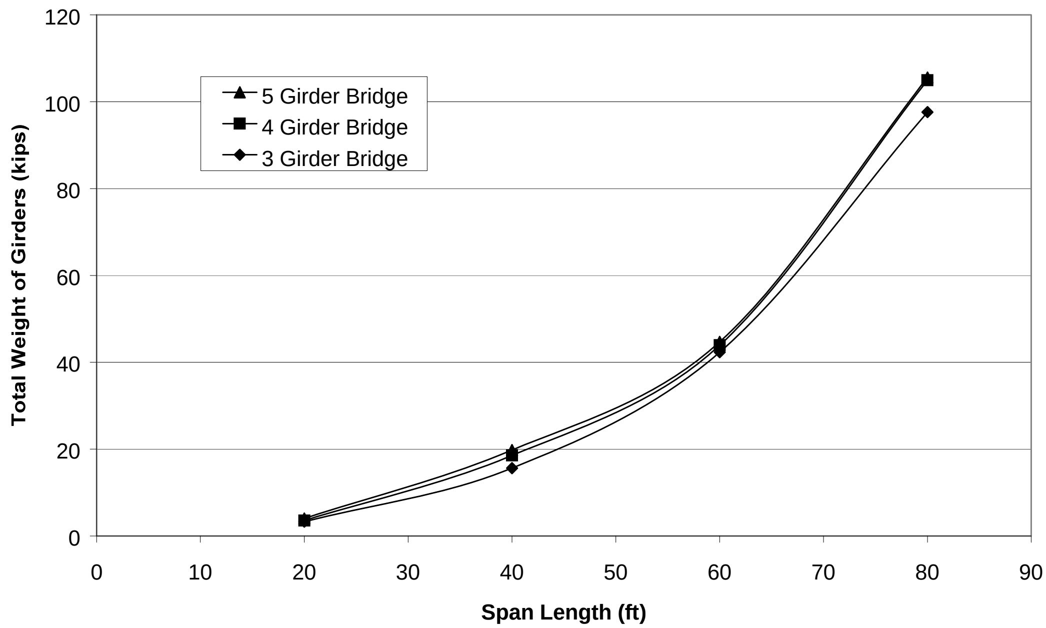


Figure 6. 3 Total Girder Weight versus Span Length for Rolled Beams

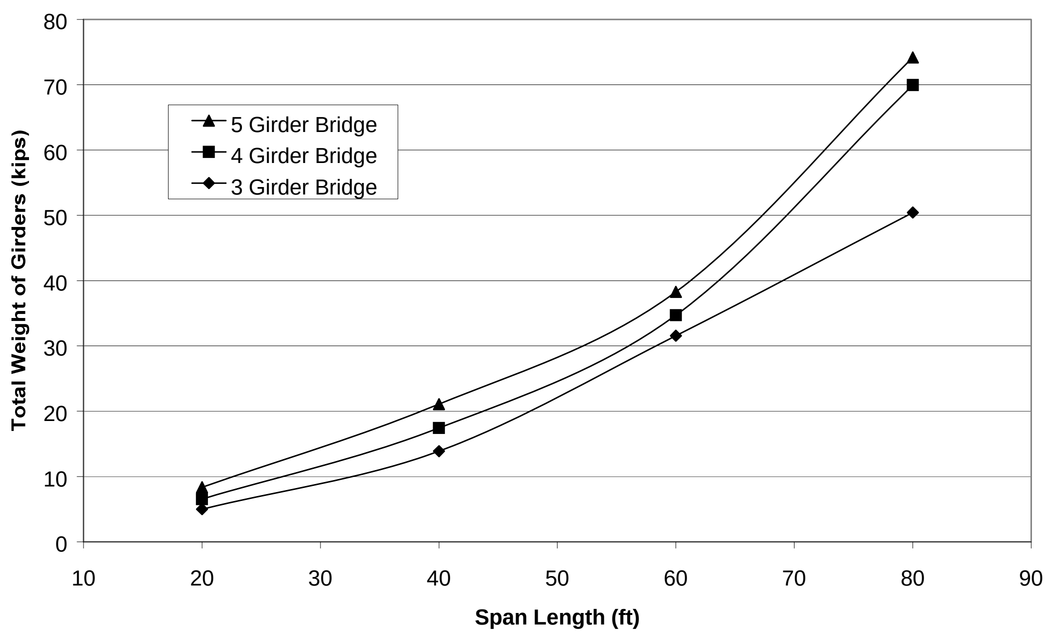


Figure 6. 4 Total Girder Weight versus Span Length for Plate Girders

In Table 6.1 and Figures 6.2, 6.2, 6.3, and 6.4, an HS-20 truck load was used as the bridge live load to calculate moments in the girders. Table 6.1 gives the sizes of the girders including all the plates used in the plate girder designs. Individual girder weight for the six different girder configurations is shown in Figures 6.1 and 6.2. The three-girder bridge alternative has significantly greater individual girder weights than the four or five-girder bridge for spans using rolled shapes.

For the plate girder bridges, the girders of the three-girder bridge weigh more than the other two alternatives for most spans. However, as seen in Figure 6.2, the three-girder bridge alternative is lighter than the four-girder bridge alternative for the 80 ft span plate girder bridge. This is because 50 in. deep girders were used for the 80 ft span three-girder bridge. Forty in. girders were originally tested for the three-girder 80 ft span, but the design program would not allow girders less than 50 in. deep for this configuration. Fifty in. deep girders are more efficient than 40 in. girders because they have greater moment capacity with less weight.

As seen in Table 6.1, other 80 ft span girders are 40 in. deep, and heavier than the 50 in. deep girder. Although a 50 in. deep girder weighs less, it may cause a price increase for the total project construction. A 10 in. increase in girder depth will require ten additional inches of soil to be excavated or additional fill to raise the approach grade, increasing costs.

Figures 6.3 and 6.4 compare total girder weight of three, four, and five-girder bridges versus span length for rolled beams and plate girders, respectively. Both graphs show that the total weight of the three-girder bridge alternative is the lightest. However, a three-girder system supporting a 26 ft wide deck would require a girder spacing of over 8 ft with an overhang of over 4 ft. This girder spacing would require a more expensive and heavier deck than the four-girder bridge.

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. The maximum girder spacing for the deck design discussed in Section 3.2.3 is 7.85 ft. A four-girder bridge is the lightest design within the range of the spacing requirements.

6.2 GIRDER SPACING

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. Girder spacings ranging 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 simplifies construction because each of the four girders carries approximately the same dead load and can be the same size, instead of having different exterior and interior girders.

CHAPTER 7

Rolled Beam and Welded Plate Girder Comparison

7.1 INTRODUCTION

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.

7.2 STEEL COSTS

7.2.1 Steel Costs for Rolled Beams

Figure 7.1 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.

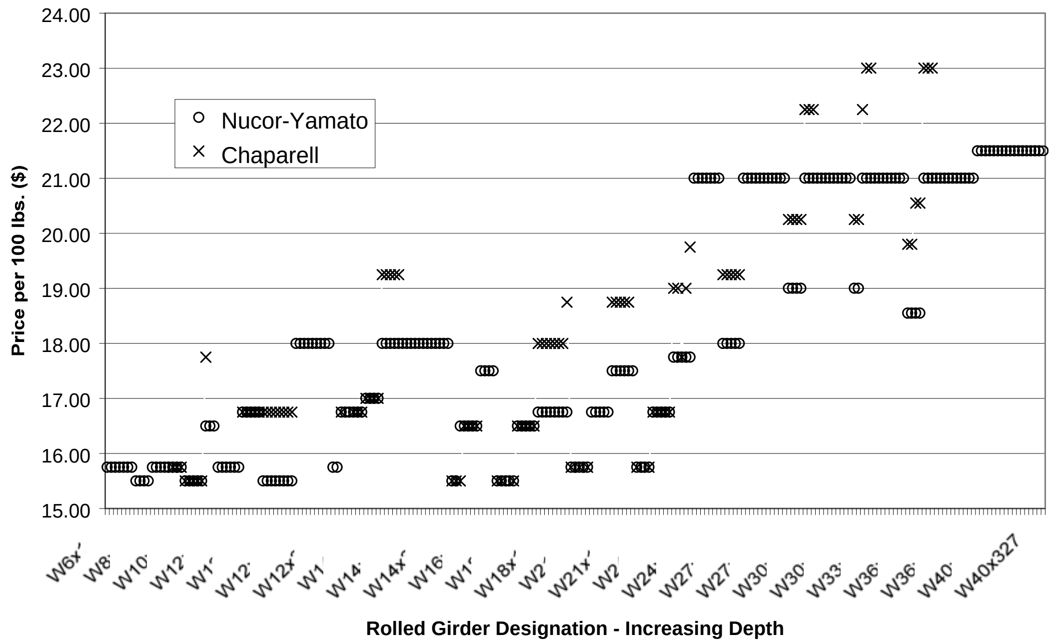


Figure 7. 1 Comparison of Steel Costs per Pound with Girder Designation

As seen in Figure 7.1, 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. 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.

7.2.1.1 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.

7.2.1.2 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.

7.2.2 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.

7.3 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 Figs. 7.2 and 7.3, respectively. The quantities compared are for non-composite bridges with 8.5 in. concrete decks and four girders spaced at 6.5 ft.

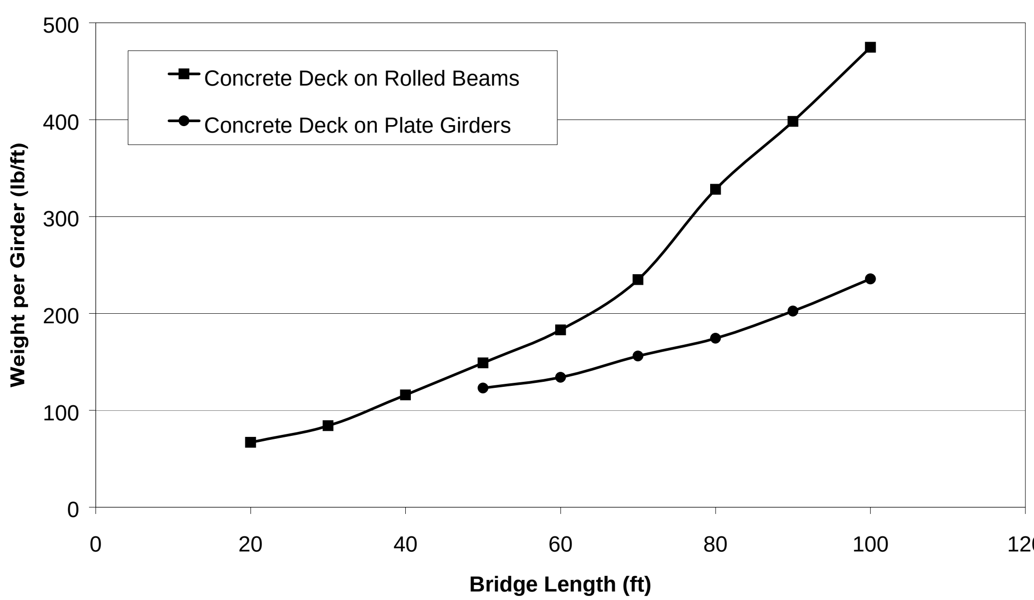


Figure 7. 2 Comparison of Girder Weights for Bridges with Concrete Decks

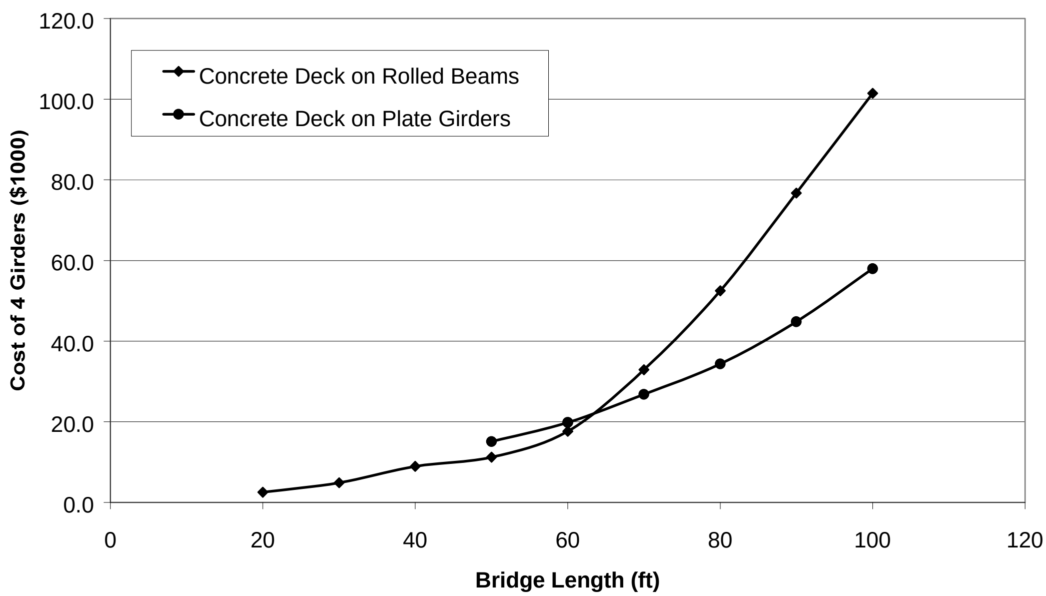


Figure 7. 3 Comparison of Girder Costs for Bridges with Concrete Decks

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 7.3. 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 7.4 and 7.5, respectively.

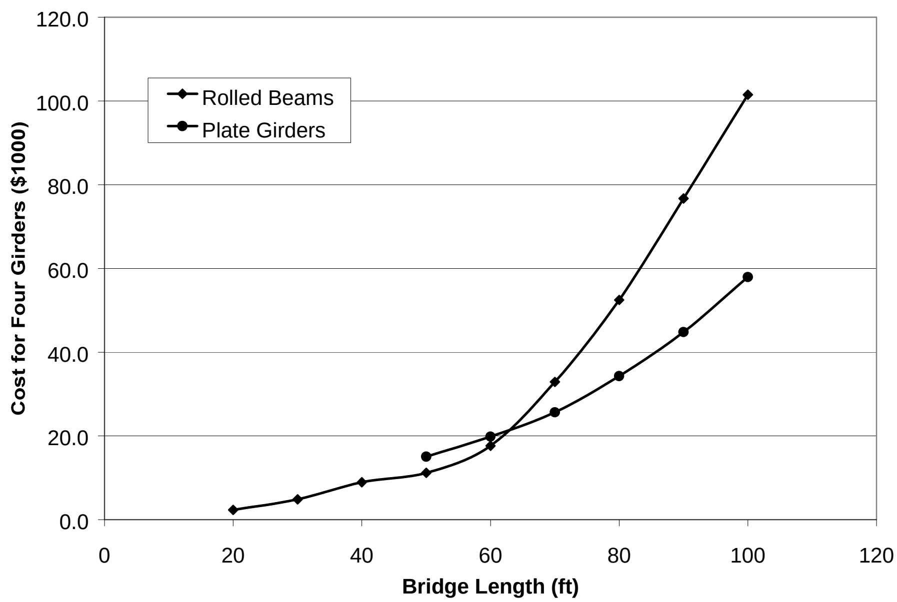


Figure 7. 4 Comparison of Girder Costs for Bridges with Steel Grid Decks

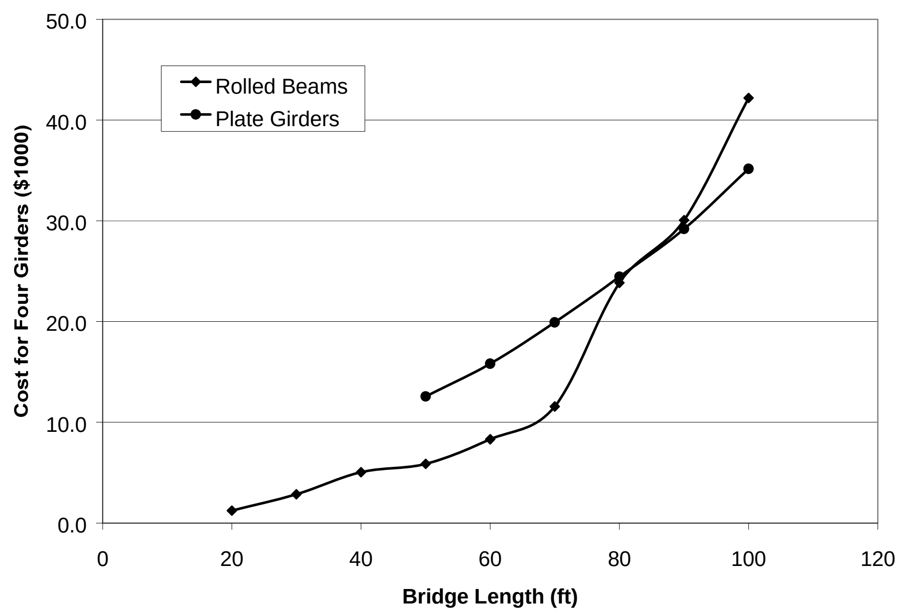


Figure 7. 5 Comparison of Girder Costs for Bridges with Composite Concrete

Figure 7.4 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. This data is shown in Figure 7.5.

CHAPTER 8
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.

8.1 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.

8.2 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 8.1. 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.

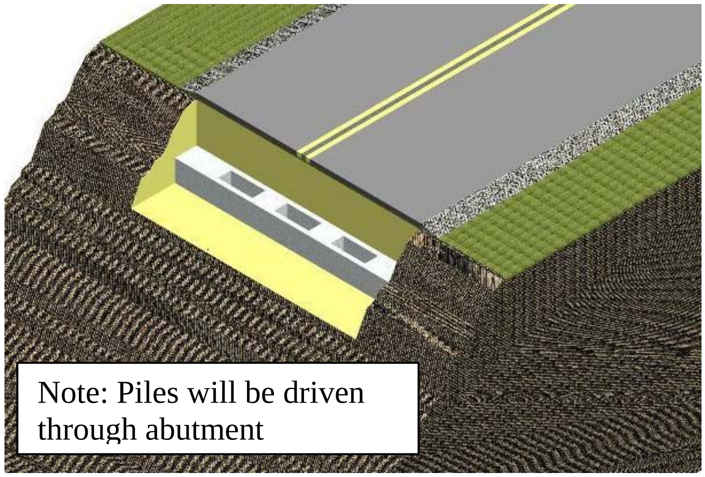
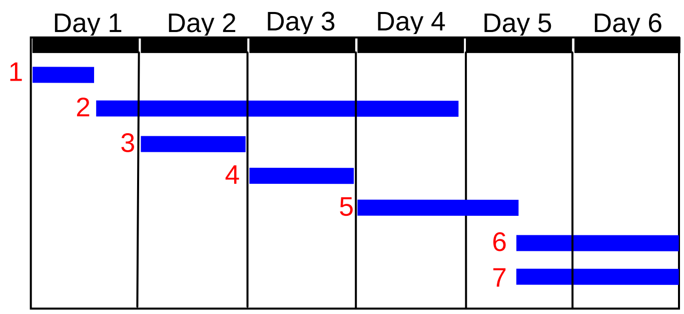


Figure 8. 1 Precast Abutment

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 8.2 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.



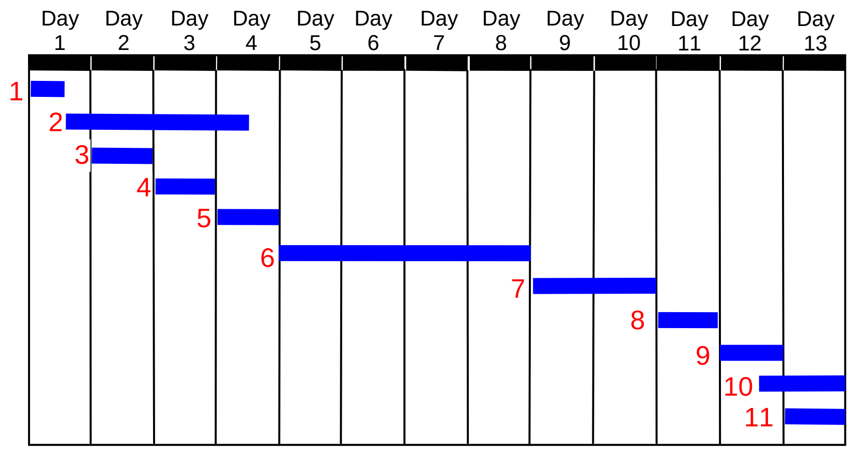
Constructions Steps

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 8. 2 Construction process for steel bridge built with a steel grid deck

As seen in Figure 8.2, 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 8.3 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.



<u>Constructions Steps</u>

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 8. 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 9
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 4.3. The foundations shall be driven piles unless the soil is unfit for piles. In these cases the foundations shall be drilled shafts.

9.1 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 Solution A bridge. 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.

9.2 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 7.2.

9.3 COMPARING FINAL DESIGNS WITH CURRENT INDUSTRY

The approximate costs of Solutions A and B are compared in Figure 9.1. 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.

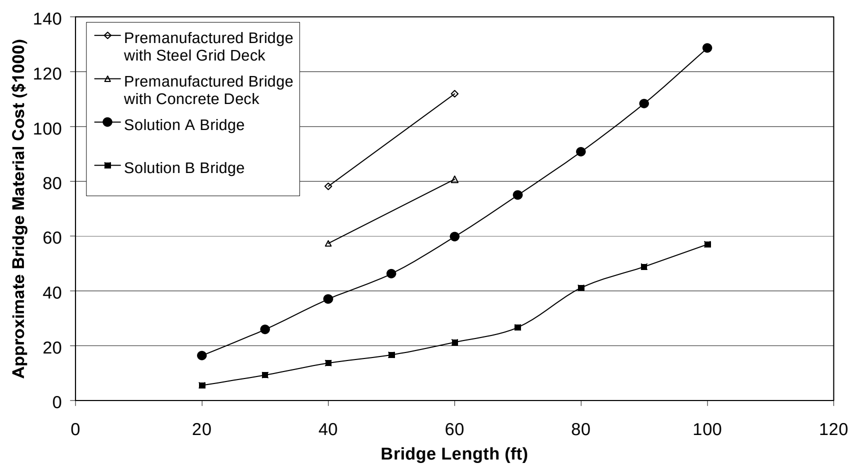


Figure 9. 1 Comparing Final Designs with Current Industry Solutions

Figure 9.1 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 - AISC's Steel Shape Availability Survey*

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
4x13x4	X	X		X		X				X		X		X	X
5x16x5	X									X					
5x19x5	X									X					
6x8.5x4		X		X		X		X		X		X		X	X
6x9x4	X	X		X		X		X		X		X		X	X
6x12x4	X	X		X		X		X		X		X		X	X
6x15x6	X		X	X		X		X		X	X				X
6x16x4	X	X		X		X		X		X		X			X
6x20x6	X		X	X		X		X		X	X				X
6x25x6	X		X	X		X		X		X	X				X
8x10x4	X			X		X		X		X		X		X	X
8x13x4	X			X		X		X		X		X		X	X
8x14x5.25	X														
8x15x4	X			X		X		X		X		X			X
8x18x5.25	X	X	X	X		X		X		X	X				X
8x21x5.25	X	X	X	X		X		X		X	X				X
8x24x6.5	X			X						X	X				X
8x28x6.5	X			X						X	X				X
8x31x8	X	X	X							X	X				X
8x35x8	X	X	X							X	X				X
8x40x8	X	X	X							X	X				X
8x48x8	X	X	X							X	X				X
8x58x8	X	X	X								X				X
8x67x8	X	X	X								X				X
10x12x4	X			X		X		X		X					X
10x15x4	X	X	X	X		X		X		X					X
10x16x5.75	X														
10x17x4	X	X	X	X		X		X		X					X
10x19x4	X	X	X	X		X		X		X					X

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
10x22x5.75	X	X	X	X				X		X	X				X
10x26x5.75	X	X	X	X				X		X	X				X
10x30x5.75	X	X	X	X				X		X	X				X
10x33x8	X									X	X				X
10x39x8	X									X	X				X
10x45x8	X									X	X				X
10x49x10	X		X								X				X
10x54x10	X		X								X				X
10x60x10	X		X								X				X
10x68x10	X		X								X				X
10x77x10	X		X								X				X
10x88x10	X		X								X				X
10x100x10	X		X								X				
10x112x10	X		X								X				
12x14x4	X			X				X		X					X
12x16x4	X		X	X				X		X	X				X
12x19x4	X		X	X				X		X	X				X
12x21x6.5	X														
12x22x4	X		X	X				X		X	X				X
12x26x6.5	X			X						X	X				X
12x30x6.5	X									X	X				X
12x35x6.5	X									X	X				X
12x40x8	X									X	X				X
12x45x8	X									X	X				X
12x50x8	X									X	X				X
12x53x10	X										X				X
12x58x10	X										X				X
12x65x12	X		X								X				X
12x72x12	X		X								X				X
12x79x12	X		X								X				X
12x87x12	X		X								X				X

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
12x96x12	X		X								X				X
12x106x12	X		X								X				X
12x120x12	X		X								X				X
12x136x12	X		X								X				X
12x152x12	X		X								X				
12x170x12	X		X								X				
12x190x12	X		X								X				
12x210x12	X		X								X				
12x230x12	X		X								X				
12x252x12			X								X				
12x279x12			X								X				
12x305x12			X								X				
12x336x12			X								X				
14x22x5	X		X							X	X				X
14x26x5	X		X							X	X				X
14x30x6.75	X		X							X	X				X
14x34x6.75	X		X							X	X				X
14x38x6.75	X		X							X	X				X
14x43x8	X										X				X
14x48x8	X										X				X
14x53x8	X										X				X
14x61x10	X										X				X
14x68x10	X										X				X
14x74x10	X										X				X
14x82x10	X										X				X
14x90x14.5	X		X								X				X
14x99x14.5	X		X								X				X
14x109x14.5	X		X								X				X
14x120x14.5	X		X								X				X
14x132x14.5	X		X								X				X
14x145x16	X		X								X				

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
14x159x16	X		X								X				
14x176x16	X		X								X				
14x193x16	X		X								X				
14x211x16	X		X								X				
14x219x16															
14x233x16	X		X								X				
14x257x16	X		X								X				
14x283x16	X		X								X				
14x311x16	X		X								X				
14x342x16	X		X								X				
14x370x16	X		X								X				
14x398x16	X		X								X				
14x426x16	X		X								X				
14x455x16	X		X												
14x500x16	X		X												
14x550x16	X		X												
14x605x16	X		X												
14x665x16	X		X												
14x730x16	X		X												
16x26x5.5	X		X								X				X
16x31x5.5	X		X								X				X
16x36x7	X		X								X				X
16x40x7	X		X								X				X
16x45x7	X		X								X				X
16x50x7	X		X								X				X
16x57x7	X		X								X				X
16x67x10.25	X										X				X
16x77x10.25	X										X				X
16x89x10.25	X										X				
16x100x10.25	X										X				
18x35x6	X		X								X				X

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
18x40x6	X		X								X				X
18x41x7.5	X										X				X
18x45x7.5	X										X				X
18x46x6	X		X								X				X
18x50x7.5	X		X								X				X
18x55x7.5	X		X								X				X
18x60x7.5	X		X								X				X
18x65x7.5	X		X								X				X
18x71x7.5	X		X								X				X
18x76x11	X										X				X
18x86x11	X										X				X
18x97x11	X										X				X
18x106x11	X										X				X
18x119x11	X										X				X
18x130x11											X				X
18x143x11											X				
18x158x11											X				
18x175x11											X				
18x192x11											X				
18x211x11											X				
18x234x11											X				
18x258x11											X				
18x283x11											X				
18x311x11											X				
21x44x6.5	X		X								X				X
21x48x8.25	X										X				X
21x50x6.5	X		X								X				X
21x55x8.25	X										X				X
21x57x6.5	X		X								X				X
21x62x8.25	X		X								X				X
21x68x8.25	X		X								X				X

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
21x73x8.25	X		X								X				X
21x83x8.25	X		X								X				X
21x93x8.25	X		X								X				X
21x101x12.25			X								X				X
21x111x12.25			X								X				X
21x122x12.25			X								X				X
21x132x12.25			X								X				X
21x147x12.25			X								X				X
21x166x12.25			X								X				X
21x182x12.25			X								X				
21x201x12.25			X								X				
21x223x12.25			X								X				
21x248x12.25			X								X				
21x275x12.25			X								X				
24x55x7	X		X								X				X
24x56x9											X				
24x61x9											X				
24x62x7	X		X								X				X
24x68x9	X		X								X				X
24x76x9	X		X								X				X
24x84x9	X		X								X				X
24x94x9	X		X								X				X
24x103x9			X								X				X
24x104x12.75	X		X								X				X
24x114x9			X												
24x117x12.75	X		X								X				X
24x128x9			X												
24x131x12.75	X		X								X				X
24x146x9			X												
24x146x12.75	X		X								X				X
24x162x12.75	X		X								X				X

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
24x163x9			X												
24x176x12.75	X		X								X				
24x192x12.75	X		X								X				
24x198x9			X												
24x207x12.75			X								X				
24x229x12.75	X		X								X				
24x250x12.75			X								X				
24x279x12.75	X		X								X				
24x306x12.75	X		X								X				
24x335x12.75	X										X				
24x370x12.75	X										X				
27x84x10	X		X								X				X
27x94x10	X		X								X				X
27x102x10	X		X								X				X
27x114x10	X		X								X				X
27x129x10	X		X								X				X
27x143x10															
27x146x14			X								X				
27x159x10			X												
27x161x14			X								X				
27x178x14			X								X				
27x182x10			X												
27x194x14			X								X				
27x201x10			X												
27x217x14			X								X				
27x221x10			X												
27x235x14			X								X				
27x258x14			X								X				
27x281x14			X								X				
27x307x14			X								X				
27x336x14											X				

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
27x368x14											X				
30x90x10.5			X								X				X
30x99x10.5	X		X								X				X
30x108x10.5	X		X								X				X
30x116x10.5	X		X								X				X
30x124x10.5	X		X								X				X
30x132x10.5	X		X								X				X
30x148x10.5	X		X								X				X
30x173x15			X								X				
30x191x15			X								X				
30x211x15			X								X				
30x235x15			X								X				
30x261x15			X								X				
30x292x15			X								X				
30x326x15			X								X				
30x357x15											X				
30x391x15											X				
33x118x11.5	X		X								X				X
33x130x11.5	X		X								X				X
33x141x11.5	X		X								X				X
33x152x11.5	X		X								X				X
33x169x11.5	X		X								X				X
33x187x11.5			X												
33x201x15.75			X								X				
33x204x11.5			X												
33x219x11.5			X												
33x221x15.75			X								X				
33x241x15.75			X								X				
33x263x15.75			X								X				
33x291x15.75			X								X				
33x301x11.5															

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
33x318x15.75			X								X				
33x354x15.75			X								X				
33x387x15.75			X								X				
36x135x12	X		X								X				X
36x150x12	X		X								X				X
36x160x12	X		X								X				X
36x170x12	X		X								X				X
36x182x12	X		X								X				X
36x194x12	X		X								X				X
36x210x12	X		X								X				X
36x230x16.5	X		X								X				
36x232x12			X								X				
36x245x16.5	X		X								X				
36x256x12			X								X				
36x260x16.5	X		X								X				
36x280x16.5	X		X								X				
36x286x12			X												
36x300x16.5	X		X								X				
36x318x12			X												
36x328x16.5	X		X								X				
36x350x12			X												
36x359x16.5	X		X								X				
36x387x12			X												
36x393x16.5	X										X				
36x439x16.5	X														
36x527x16.5	X														
36x650x16.5	X														
40x149x12	X		X								X				
40x167x12	X		X								X				
40x183x12	X		X								X				
40x199x16	X										X				

W Shapes	Arcelor International America	Bayou Steel	Corus	Gerdau Ameristeel	North Star	Nucor Bar Mills	Nucor Jackson	Nucor Kankakee	Nucor Seattle	Nucor-Berkeley	Nucor-Yamato	Roanoke	Sigosa Steel Company	SMI	TXI Chaparral
40x211x12	X		X								X				
40x215x16	X										X				
40x235x12	X		X								X				
40x249x16	X										X				
40x264x12	X		X								X				
40x277x16	X										X				
40x278x12	X														
40x294x12			X								X				
40x297x16	X										X				
40x324x16	X										X				
40x327x12			X								X				
40x331x12	X														
40x362x16	X										X				
40x372x16											X				
40x392x12	X														
40x397x16	X										X				
40x431x16	X										X				
40x503x16	X														
40x593x16	X														
44x230x16	X														
44x262x16	X														
44x290x16	X														
44x335x16	X														

*Availability as of May 1, 2003

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