Swarthmore College Timber Bridge:

A King (Post) Among Bridges

Peter Brennan
Jaime Cardenas-Navia
Jay Charles
Edward Goldstein

Advisor: Professor Faruq M. A. Siddiqui
Swarthmore College

E90: Senior Design Project
May, 2007

Table of Contents

Abstract

For our senior design project we designed, constructed and loaded a bridge to enter the 2007 ASCE/FPS National Timber Bridge Design Competition. In doing this we intended to gain experience with realistic design constraints, using wood as an engineering material, project management, and to do well in the competition. Our 3.8 x 1.4 meter bridge consisted of a simple frame, made up of three longitudinal girders and five transverse diaphragms, supported by three pre-tensioned steel cables running under a post structure at mid-span. The deck consisted of transverse 2 x 12 planks connected by two steel rods running the entire length of the bridge and screwed into the frame. After one hour at the full load of 20 kN our bridge had deflected 8.66 millimeters at mid-span, 91% of the maximum allowable deflection, and the deck had deflected 1.54 millimeters, 28% of its maximum allowable deflection. Out of twelve teams competing in this year's competition, our team was awarded the 2nd most innovative design, the 3rd best overall design, and took 4th in monetary winnings. Now that the competition is over, the bridge has been removed from Hicks basement, and will be used to span Dicks Run Creek in Professor Everbach's backyard.

Acknowledgements

We would like to thank our advisor, Professor Faruq Siddiqui, for his gentle guidance throughout the course of the project; Grant "Smitty" Smith, for the extensive shop work and construction advice provided; Professor Fred Orthlieb, for helping us through our toughest times with his extensive practical expertise.; Professors Everbach, Macken for their patience in kindly sharing the cramped basement space; Ed Jaoudi for his AutoCAD assistance; Swarthmore College Facilities Department and all the students who helped us on loading day, for their time and effort, without which we could not have performed the loading. We would also like to thank our sponsors, VSL International, McFeely's Square Drive Screws, and Simpson Strong-Tie, for their generous material donations.

List of Tables

List of Figures

1 Introduction

In the National Timber Bridge Design Competition, open to student chapters of the American Society of Civil Engineers (ASCE) and the Forest Products Society (FPS), each team is required to design, build, and test a bridge constructed from wood structural members. The competition has three main objectives: first, to promote interest in the use of wood as a competitive bridge construction material; second, to generate innovative and cost-effective timber bridge design techniques; and third, to develop an appreciation of the engineering capabilities of wood. This competition was originally developed with rural America in mind, where there are thousands of deficient bridges that limit the movement of cattle and/or grain from fields to mills or barns. These same bridges also limit those in rural America from easy access to goods and services. The U.S. Department of Agriculture (USDA) sees modern timber bridges as a partial solution to these problems. In addition to being economical, aesthetic, and easy to install, the USDA believes that using timber bridges for short spans can boost a local economy since the timber and labor come from the community. Beyond this, wood is a renewable natural resource that is safe for use in agriculture applications.

We chose this competition as the foundation for our senior design project because we felt it would be an ideal exercise to gain experience with realistic design constraints. By seeing our bridge through, from design to completion, we gained invaluable experience. While it is an oft-quoted engineering maxim that unexpected and unconsidered problems will arise, simply stating this truth never carries the full weight of its implications on to the students. By witnessing and coping with such problems during the design and construction process, we are better equipped to be successful and practical engineers than ever before.

To begin planning for this competition, even before considering our bridge design, we considered how we would meet the most basic functionality needs of the project: construction, weighing and loading. The only viable space to accomplish these necessities was the cage in the basement of Hicks, not only for the space itself, but for the translating mechanical assist pulley system which would allow us to weigh the bridge, and assist in construction. In addition to selecting the space, we also gave consideration early on to the material and method of loading the bridge. We planned to achieve the full 20 kN load with palettes of solid concrete blocks loaned from Fizzano Brothers Concrete, loaded onto the bridge with a forklift from the Swarthmore College Facilities Department.

2 Codified Competition Design Criteria

Below we present a codified version of the 2007 competition rules, referred to in the design section. The rules can be found in their original version on the competition website at http://www.msrcd.org/b-rules07.htm. Figure 2.1 shows the bridge and test dimensions specified by the competition.

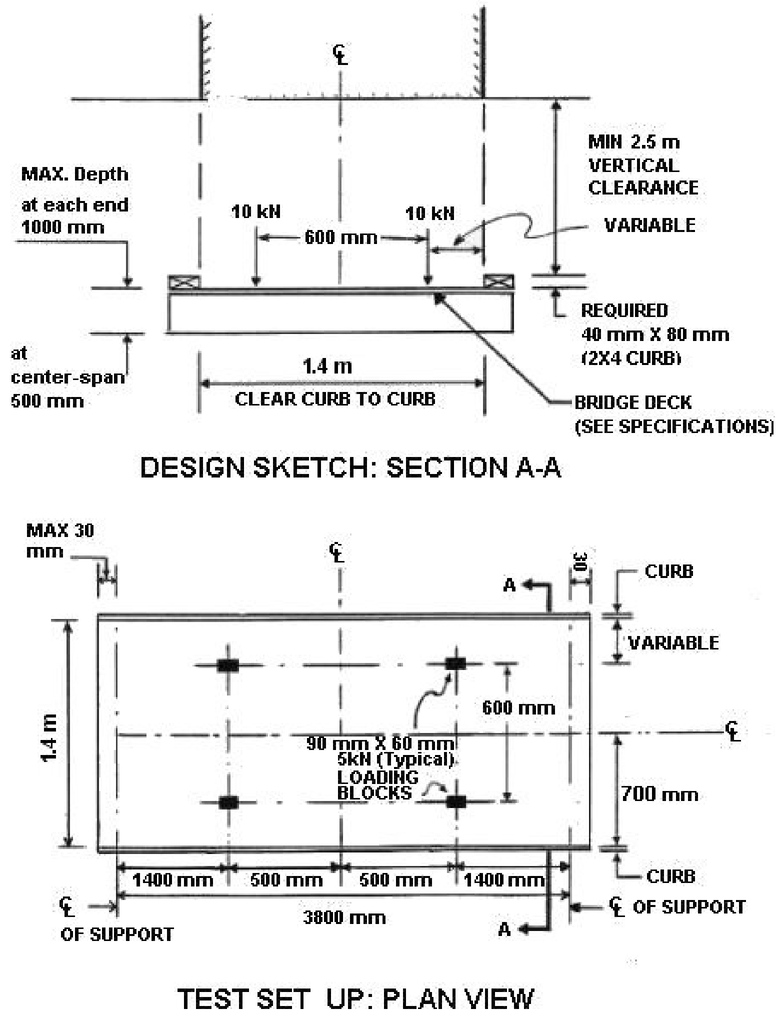

Figure 2.1: Competition Dimensions and Loading Setup

I. Treatment, Pre-stressing and Weight

a. All wood members must be treated to appropriate AWPA standards for their intended use.

b. Pre-stressing of any bridge component is allowed, but must be completed at least 48 hours prior to testing.

c. The bridge must be weighed before testing, and the weight of all non-wood materials must not exceed 25% of total bridge weight.

II. Bridge Materials and Geometries

a. Wood from a commercially available species must be used for all structural members (able to resist compression).

b. No individual piece of wood shall exceed a length of 2.1 meters;

i. This applies to pieces used in built-up members, including deck panels, but does not apply to the length of the built-up member itself

ii. This does not apply to any non-wood component, like cables, ties, traps or rods.

c. The bridge deck need not be wood, but it must effectively transfer loads, at all locations on its surface, to the supports structure, be treated to withstand weather, repetitive traffic, and all other real-life applications.

i. The deck must be uniform in thickness and material throughout, and may not be preferentially strengthened under expected loading points.

d. The span length from center line to center line of supports shall be 3.8 meters.

i. The bearing area of supports shall extend longitudinally for a maximum of 60 mm.

e. The lateral clearance shall be 1.4 meters between the insides of each curb.

i. Curbs must be connected to the bridge, and may be part of arch or truss superstructure.

ii. Curb material must meet the same requirements as deck material, except for load bearing.

f. The vertical or overhead clearance shall be 2.5 meters from the deck surface.

g. The depth of any understructure shall not exceed 500 mm at center-span and 1000 mm at the support, as measured from the deck surface to the lowest point of the understructure.

III. Bridge Loading and Deflection Monitoring

a. A test load of 20 kN shall be applied over designated loading points, in four equal increments of 5 kN, and remain for one hour.

i. The full load of 20 kN must be achieved in not less than 5 minutes, and not more than 20 minutes.

ii. Deflection measurements will be recorded after each load increment, and at four 15 minute intervals after achieving full load.

b. The load will be seated on four 60x90 mm loading blocks situated in a 0.6x1 meter rectangular pattern, as shown in figure ___, whose position will be chosen according to the deflection being tested.

c. Maximum vertical bridge deflection shall not exceed 9.5 mm, as measured at mid-span of the longitudinal beam bearing the greatest load.

i. Maximum vertical bridge deflection shall be measured with the load rectangle described in B directly over both longitudinal and lateral centerlines.

ii. Subtraction from deflection due to the compression and deflection of supports is not allowed.

d. Maximum Vertical Net Deck Deflection shall not exceed the deck span divided by 100.

i. Deck span is defined as the shortest side of the largest "deck panel," where a deck panel is defined as any area of clear-span deck bordered by the 2 nearest longitudinal, and 2 nearest transverse, supporting members.

ii. Gross deck deflection shall be measured under the centroid of the loading block placed over the center of the largest deck panel (this point should also correspond to the weakest point on the deck, locating the four-point loading setup to cause maximum deck deflection).

iii. Net deck deflection shall be determined by subtracting the average deflection of the two structural members forming the longer side of the deck panel from the gross deck deflection.

3 Design

3.1 Design Concepts and Constraints

Since we designed our bridge for a national competition, the principal constraints were already determined for us. From these constraints, codified in section 2, we felt that article III.c, limiting maximum vertical bridge deflection to 9.5 mm under the full 20 kN load, would be our greatest challenge. Therefore, we set about designing a bridge with a high stiffness-to-weight ratio. The next constraint to divert our design's path was article II.f, requiring a 2.5 meter clearance above the deck surface. This meant that to stiffen any truss superstructure at its top with lateral bracing, necessitated a square-like 2.5 x 3.8 meters truss. Since our design team found this aspect ratio unappealing, we began exploring a pony-truss design, with the deck, itself, providing lateral bracing at the truss' mid-height. This design was modeled in ANSYS, but performed poorly, allowing deflections a full order of magnitude higher than our previous trussed designs, and bringing us back to the drawing board. At this point our progress on a final design began to stagnate; simultaneously, we began to consider our first realistic design constraint: how to load the bridge, in Hicks basement, with 20 kN of concrete blocks in twenty minutes. Having been told of a Bobcat front-loader available from the Facilities Department, we decided to remove the back door of Hicks and load the bridge directly with the Bobcat. This method of loading, we quickly realized, would necessitate having no superstructure above the deck, allowing the front-loader's forks to swing over the bridge. Thus without one member modeled, the shape of our design was chosen for us by a realistic design constraint: our bridge would be supported by an understructure.

Other realistic constraints also played a role in the design process. We strove for an economically efficient design both to stay within the E90 budget, and to approximate real construction projects, where contracts may be won and lost on price. As an example, this concern for cost led us to design a more complex understructure than we had envisioned to avoid using an expensive hardwood, like Mahogany. To stay under budget we had to seek sponsorship, usually in the form of free products. Therefore we had to adjust our design to utilize the products of whichever companies we could convince to sponsor us. Another concern was the environmental impact of our bridge. As specified by the rules, the bridge had to be capable of outdoor use, and must therefore be treated against weather and rot. In an effort to minimize any negative effect on the bridge or construction environment, we selected wood treated with Alkaline Copper Quaternary, a non-toxic chemical. Finally, with no woodworking experience, only four workers, and two months for construction, we had a vested interest in a highly manufacturable design. The manufacturability of our design is evidenced by the standard quality of its components: all nominally-sized sections with perpendicular angles, requiring no angled cuts and minimal modifications or handling from store-shelf to bridge-pier.

3.1.1 Refining the Design

We found that side trusses did an inefficient job of limiting deflection in the center of the bridge. We therefore investigated the possibility of directly supporting the center of the deck from below so that the load did not have to first be transferred to the edges of the bridge before to be taken up by the truss. When this design was modeled using cables for the understructure, we found that it did a much more efficient job of limiting deflection than any trussed design. The use of an understructure rather than a superstructure also kept the deck accessible from all sides, which was desirable for ease of loading. ANSYS drawings of some of these early designs are shown below, in figures 3.1 through 3.3. The final design is visible in figure 3.4. The change from an inverted queen post to an inverted king post (from a trapezoidal shape to a triangular shape) was done because a king post would be easier to construct, require less material, and have almost no affect on the total deflection.

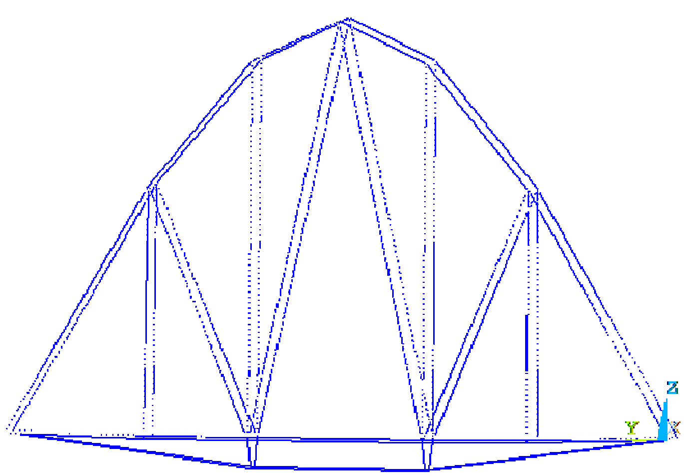

Figure 3.1: Side Truss Design, Laterally Braced at the Top While Maintaining Minimum Clearance According to Article II.f

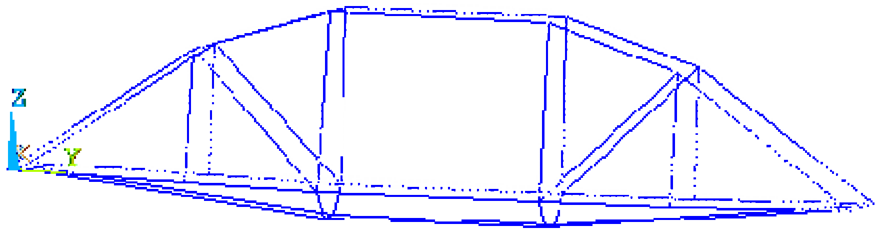

Figure 3.2: Shallow Pony Truss Design

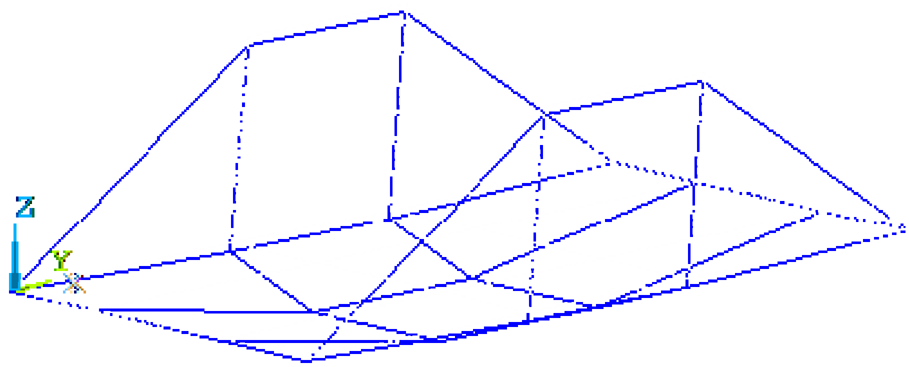

Figure 3.3: Queen Post Design

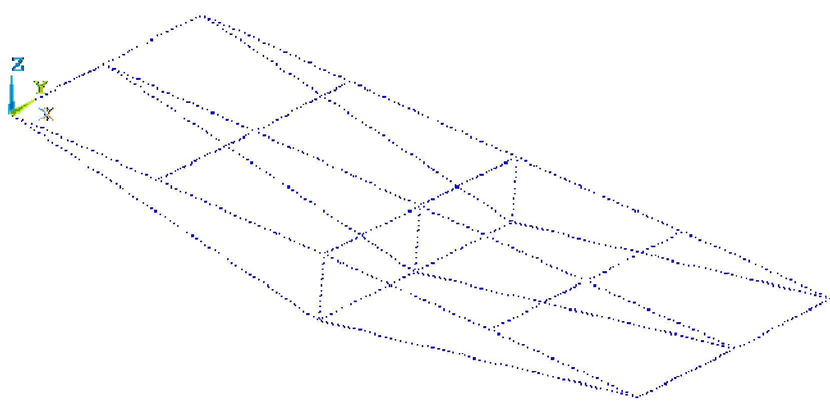

Figure 3.4: Our Inverted King Post Design

3.1.2 Girders

Three equally spaced main girders ran the length of the bridge. Since no piece of wood could be longer than 2.1 meters according to article II.b, each girder had to be built up from smaller pieces of timber. Originally, our design called for each of these girders to be made of sections of 2 x 10s placed next to each other (making the cross-section of each beam 3" by 9.25") with staggered breaks to avoid weak points. The design was later changed to make the two outside girders out of three 2 x 10s placed next to each other so as to increase the stiffness of the frame. The arrangement of the individual pieces of timber in the side and center beams can be seen in figure 3.5. The beams were fabricated first by gluing and clamping, and then by driving screws through their entire width. The glue should have been strong enough to resist any shear forces that might have occurred between the timber sections; the screws were added in case the glue was not sufficient, though the number of screws was later increased when it came to our attention that the chemicals used in pressure treating the wood could have a negative affect on the strength of the glue.

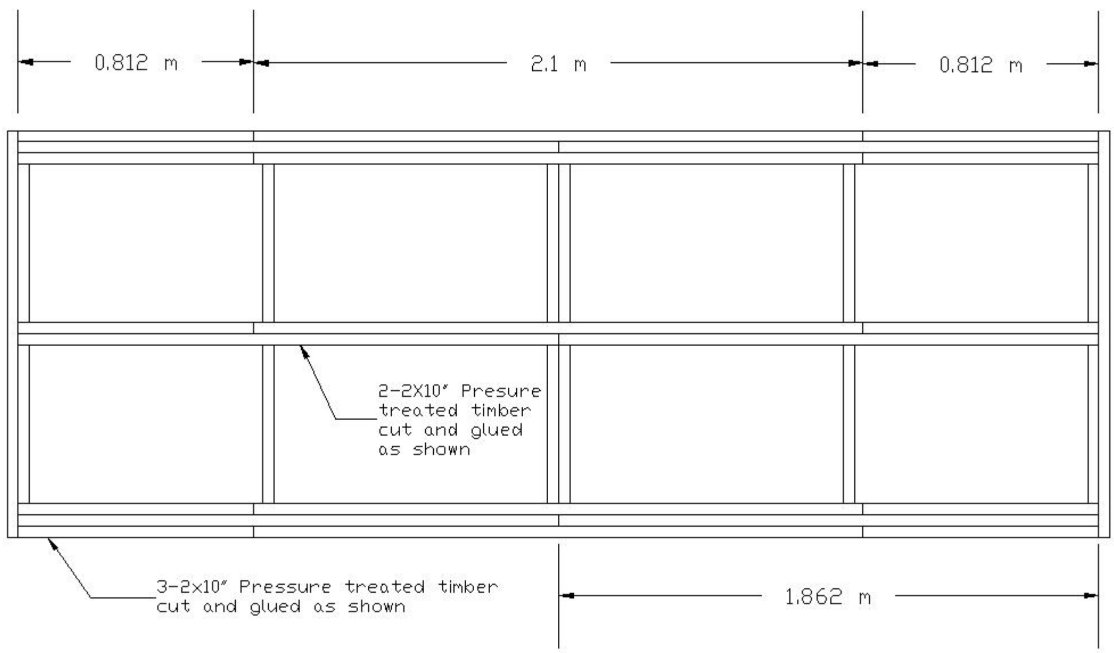

Figure 3.5: Plan View of Frame Structure

3.1.3 Diaphragms

The diaphragms, transverse pieces of timber running between the girders at five points along the length of the bridge, can also be seen above in figure 3.5. They purposes they served were twofold: stiffening the deck to prevent excessive deck deflection, and providing lateral support to the girders to avoid buckling from the compression induced by pre-tensioning and loading. All outer diaphragms were made from 2 x 6s, while the two center diaphragms were built up using the same method employed for the girders.

3.1.4 Understructure

The understructure provided support for the main beams, and consisted of cables running under a king post frame from each end of the bridge. Any bridge deflection would cause the cables to lengthen and push up on the king post, thereby acting to limit bridge deflection. The effectiveness of the cables in resisting deflection was limited by criterion II.g, limiting the max depth of the understructure to 0.5 meters. This would limit the depth of the angle of the cables, and therefore how much vertical resistance they would supply for a given axial load.

3.1.4.1 Understructure Framing

Originally the understructure frame was to consist of posts made from 4 x 4's extending down from the main beams and connected to each other for lateral support. K-bracing was also to be used, both to support the deck and to lend stability to the understructure frame. When analyzing the structure in ANSYS, this seemed like a logical approach, but once we drew the design to scale, it became clear that the space was too crowded. Because of this, the design was changed to have a single length of 4 x 6 running transversely below the center diaphragms, as seen in figure 3.6. This change not only saved construction time and slightly increased the strength, it also barely increased the weight to fill the rest of the crowded space.

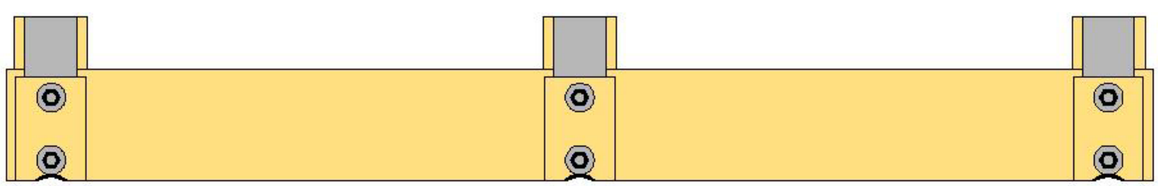

Figure 3.6: Elevation of King Post Understructure

3.1.4.2 Cable Seatings and Anchorages

On the bottom of the king post, at each point where the wire was to rest, something had to be done to prevent the cable from crushing the wood, and make sure that it did not curve beyond the allowable bending radius. Going past the allowable bending radius had to be avoided to prevent kinking, which would lower cable strength and raise the possibility of catastrophic failure. At either end of the bridge, where the cables came through the main girders, they would be seated in T-5 Anchorages donated by VSL International. These anchorages would, in turn, be resting on appropriately angled timber anchor blocks, shown in detail in figure 3.7. Without this timber anchor block, the cables might kink when pre-tensioned. Since these blocks would be transferring the cable stress to the bridge in compression, they had to be checked for a bearing failure. While the block would be strongest in compression if its grain was parallel to resultant force, it was chosen not to be for ease of manufacturability, as this would require two angled cuts per block instead of one. Therefore, the allowable compressive strength was determined for the designated grain angle using Hankinson's formula, given in calculation 8, and bearing failure was reevaluated.

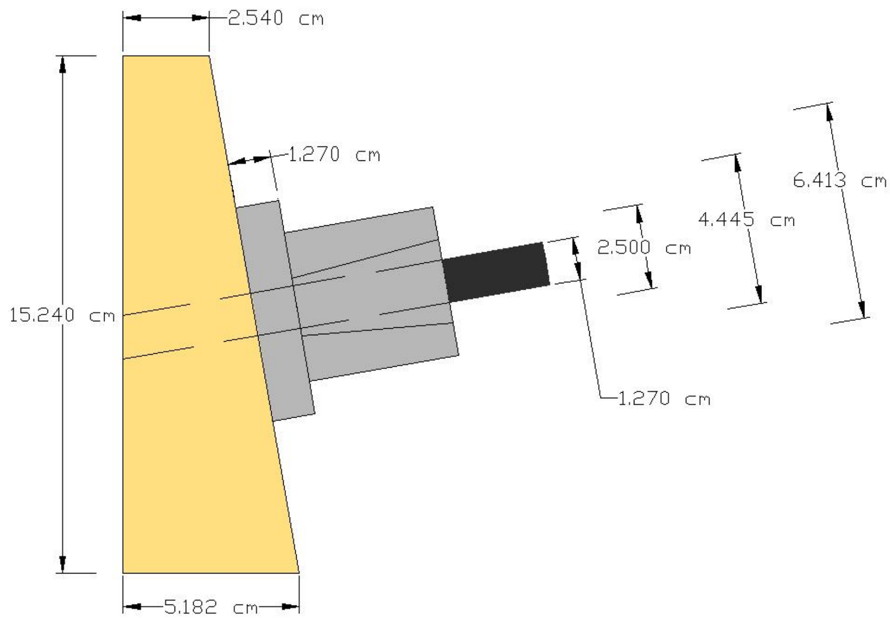

Figure 3.7: Detail of Cable Anchorages, showing T-5 Anchor and Timber Anchor Block

3.1.5 Connections

To attach the diaphragms and understructure to the main beams, we used both commercially available joist hangers and custom designed connectors from Simpson Strong-Tie that acted as both joist hangers and post caps. Before selecting the connectors from the available models, we had to determine the maximum load that the connections could possibly have to resist. This maximum load is 5000N (1124lbs), and occurs when one of the four loading points sits directly above the edge of one of the diaphragms. The custom combination connectors used to connect the center diaphragms and posts were fabricated from low gage metal, and though they were not load rated, it was clear from their bulk that they would be more than sufficient for any forces that might occur at those joints. For the diaphragms at the quarter points we selected the HUS26 joist hanger from Simpson Strong-Tie, which, with a load rating of 2565 lbs, was more than sufficient for the purpose. For the diaphragms adjacent to the end plates, LUC26Z hangers were selected. These connectors were only load rated for 710lbs, but because these diaphragms were attached to the bridge's end plate, this strength was sufficient. In addition, the concealed flanges allowed us to place it directly on the end of the girder.

3.1.6 Deck

Our deck represented a variation on a standard stress laminated deck, which usually consists of longitudinal members placed next to each other and then compressed by bars that run transversely across the deck. Due to the small scale of our project and the standard sizes in which timber is available, a standard stress laminated deck did not make sense. Instead, the design called for 2 x 12's to be laid transversely on the frame. Each 2 x 12 would have two holes drilled through it to accommodate 1/2" steel rods that would run longitudinally the entire length of the deck. These rods were threaded on each end so that once the deck planks were in place, the whole deck could be compressed. Between the friction generated by compression, and the strength of the steel rod itself, each deck plank could effectively transfer applied loads to adjacent deck planks. A schematic of a standard stress-laminated deck is shown in figure 3.8, while our deck can be seen in figure 3.9.

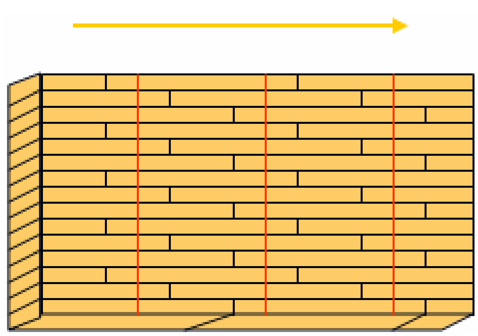

Figure 3.8: A Typical Stress-Laminated Deck with Tensioning Rods Shown in Red

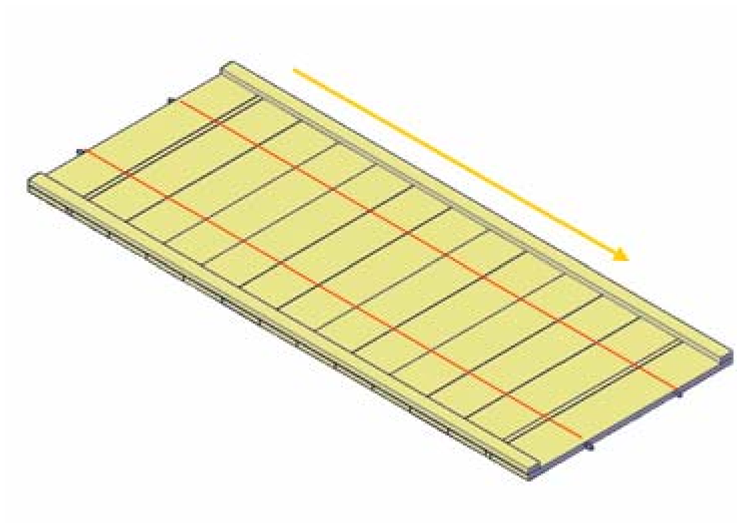

Figure 3.9: Our Version of a Stress-Laminated Deck

3.1.7 Cables and Pre-tensioning

The bridge was supported using seven-strand 170ksi steel cable originally intended for use in the post-tensioned concrete industry. We chose to use post-tensioning cable instead of wire rope for both performance related and aesthetic reasons. The cables used by the post-tensioning industry are both significantly stronger and stiffer than equivalently sized wire rope. This meant that we could provide more support for the bridge with lighter cables than would be possible with wire ropes. The cables were also superior to the wire rope because the tensioning process could be carried out without the use of turnbuckles, which would have been both heavy and unattractive. Furthermore, by pre-tensioning the cables with a calibrated hydraulic ram instead of turnbuckles, we were able to see exactly what the force in the cable was.

3.1.8 Substructure: Unistrut Loading Supports

While the concrete block supports we were using for construction were adequate for the task, they would be inadequate for supporting the bridge under the full 20 kN load. According to article III.c.ii, and deflection or compression of the supports may not be subtracted from the overall bridge deflection, and we therefore wanted to minimize any support deflection. To accomplish this, we envisioned a simple, laterally stabilized, portal frame with cross bracing. The lateral stability, accomplished with the splayed legs of the frame, was of particular importance to prevent any movement or shifting during the loading process. The cross-bracing of the frame also had to be scaled down due to the difficult of crossing Unistrut. A variety of frames were tested in ANSYS using real constants from a Unistrut design handbook. Each model was checked for safety by comparing the ANSYS predicted load values with tabulated design loads in the Unistrut handbook. The final model had to be scaled down again due to availability problems, and is shown schematically in figures 3.10 and 3.11. Subjected to a slightly overestimated concentrated load, the ANSYS model showed deflections of less than .2 mm, which was deemed acceptable. Once this geometry was finalized, the frame's orientation and connectors were chosen to fit the space.

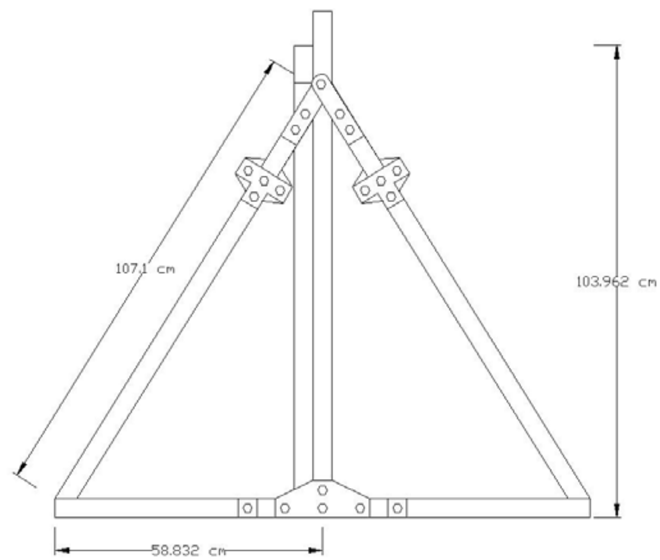

Figure 3.10: Unistrut Substructure Side Elevation

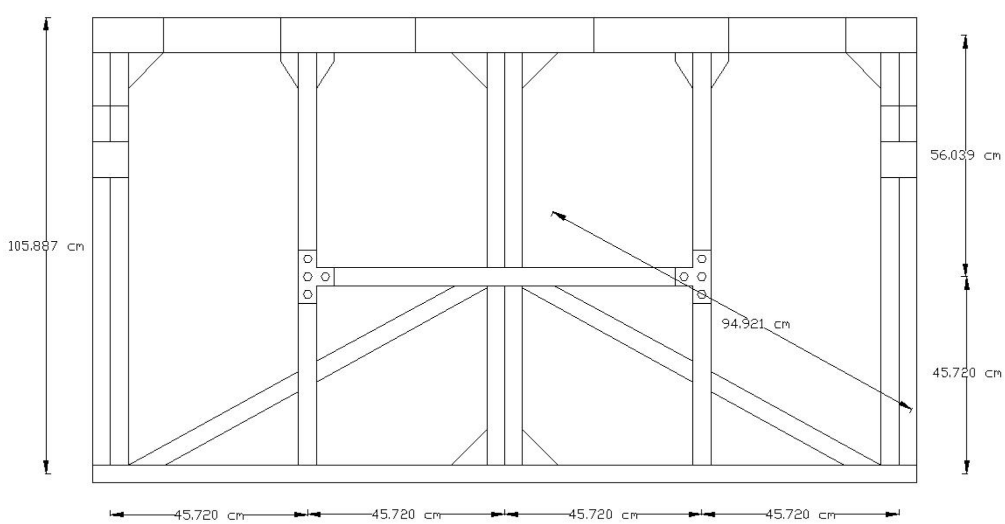

Figure 3.11: Unistrut Substructure End Elevation

3.2 Calculations & Analysis (sealed design)

Table 3.1 below shows all important calculations and checks that had to be made to ensure the bridge was safe and met specifications.

Table 3.1: Important Design Calculation Results

Criterion	Allowable Value	Calculated Value	Source†
Beams			
Flexural Stress	7.24 MPa	2.84 MPa	ANSYS 1 and Calculation 1
Shear Stress	1.21 MPa	418 kPa	Calculation 2
Buckling*	P_{cr} = 193.5 kN	P = 9.39 kN	Calculation 3 and ANSYS 2
Deflection	9.50 mm	4.50 mm	ANSYS 1
Diaphragms			
Flexural Stress	8.62 MPa	5.53MPa	Calculation 4
Shear Stress	1.21 MPa	131.2 kPa	Calculation 5
Deck			
Flexural Stress	10.34 MPa	7.90 MPa	ANSYS 3
Shear on Metal Rod		59.2 MPa	Calculation 6
Deflection	5.48 mm	1.42 mm	ANSYS 3
Anchor Blocks			
Crush Strength	10.42 MPa	3.00 MPa	ANSYS 2 and Calculation 7
Sliding	No Slip	No Slip	Calculation 8
Cables and Pre-stressing			
Axial Force	69.6 kN	11.51 kN	Calculation 9 and ANSYS 2
Camber**	15 mm	1.38 mm	ANSYS 4

†Calculations are shown following this table. ANSYS input and output files are attached in appendix ___
*P_{cr} indicates the value of axial force in the center beam at which buckling will occur. P indicates the axial force that the center beam was expected to see.
**Allowable value set by rules

Calculation 1 – Flexural stress in main beams

The flexural stress in the main beams (in this case the center beam, since it was the lightest) was determined using values for moment obtained from the ANSYS analysis, which ignored the effects of pre-stressing. This was done because the flexural stresses in the main beams are greater if pre-stressing is taken into account. Thus, by ignoring the effects of pre-stressing, we ensure that our calculated value for flexural stress is conservative.

From ANSYS 1, we know the maximum moment (M) in the center beam is 1.988 kN-m.

$$\sigma_{flexural} = \frac{My}{I_x}$$

Where I_x is the second moment of area about the horizontal axis.

$$I_x = \frac{b \cdot h^3}{12}; \; b = .0762m; \; h = .235m$$
$$I_x = .0000824m^4$$

And where y is the distance from the neutral axis to the farthest fiber.

$$y = .1178m$$

Thus the flexural stress is

$$\sigma_{flexural} = \frac{(1.988kN \cdot m)(.1178m)}{.0000824m^4} = 2.84MPa$$

Since this is smaller than the allowable stress of 7.24 MPa, the design is sufficient.

Calculation 2 – Shear stress in main beams

Start by assuming that the greatest shear (V) that the center beam will see is 10 kN. (This is a conservative assumption since there are two other beams and the total load is only 20 kN) The equation for shear stress (τ) is:

$$\tau = \frac{VQ}{It}$$

Where I is the second moment of area about the axis perpendicular to the direction of the shear force, and t is the width of the cross-section along the axis perpendicular to the direction of the applied load. Q is the first moment of area of the upper half of the beam, since we are interested in the maximum shear stress which occurs at the neutral axis. We have:

$$Q = \frac{b \cdot h^2}{8} = .000526 m^3$$

Making the shear stress

$$\tau = \frac{(5kN)(.000526m^3)}{(.0000824m^4)(.0762m)} = 418kPa$$

Since this is smaller than the allowable stress of 1.21 MPa, the design is sufficient

Calculation 3 – Buckling in main beams

Because of the geometry of the bridge and the fact that we pre-stressed the cables, the main beams are always in compression. As with any other compression member, a check had to be made to ensure that the main beams would not buckle. As with calculations 1 and 2, this calculation will be done using the dimensions of the center beam, since it is the lightest. The beams will not buckle in the weak plane because they are continuously supported in that direction by the bridge deck, which is screwed into the main beams. Thus we must only check buckling in the strong axis. Buckling resistance is dependant on the slenderness ratio which is defined as the length of the member divided by the radius of gyration (*r*), which is defined as

$$r = \sqrt{\frac{I}{A}}$$

$$A = 27.75 in^2$$

$$I = 197.9 in^4$$

Thus the radius of gyration, *r* is equal to:

$$r = \sqrt{\frac{197.9in^4}{27.75in^2}} = 2.67in$$

The length, *l* of the beam is:

$$l = 3.72m = 146.6in$$

Thus the slenderness ratio is:

$$Slenderness\,ratio = \frac{l}{r} = \frac{146.6in}{2.67in} = 54.9$$

Given the slenderness ratio, we can use the graph in figure 3.12 to find the ratio of the critical buckling stress, f_{cr}, to the maximum compressive stress F_c.

$$F_c = 1600\ psi$$

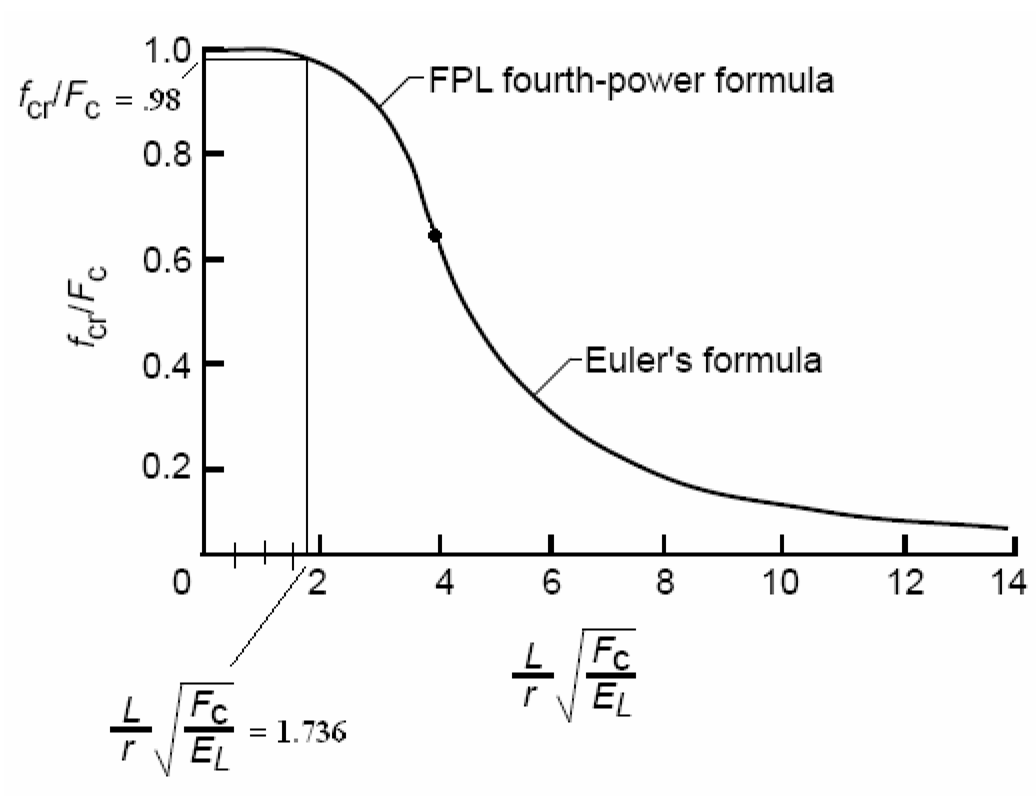

Figure 3.12: Column Design Aid [USDA, 1999]

As shown above,

$$\frac{l}{r} \cdot \sqrt{\frac{F_c}{f_{cr}}} = 54.9 \cdot \sqrt{\frac{1600\,psi}{1600000\,psi}} = 1.736$$

Thus we have

$$f_{cr} = 1600\,psi \cdot .98 = 1568\,psi$$
$$P_{cr} = f_{cr} \cdot A = 1568\,psi \cdot 27.75 in^2$$
$$P_{cr} = 43.5 kips$$

$$43.5\ kips = 193.5\ kN$$

According to the ANSYS analysis, the greatest compressive force on the center beam is 9.39 kN, which is well below the critical load. Thus the design is sufficient.

Calculation 4 – Flexural stress in diaphragms

The highest stress that a diaphragm could experience occurred when one of the four loads was directly over the center of a diaphragm.

$$\sigma_{flexural} = \frac{My}{I_x}$$
$$M = \frac{PL}{4} = 685 N \cdot m$$
$$I_x = .00000865 m^4$$
$$\sigma_{flexural} = \frac{(685 N \cdot m)(.0699 m)}{(.00000865 m^4)} = 5.53 MPa$$
$$\sigma_{allowable} = 8.62 MPa$$

Since the flexural stress is less than the allowable stress, the design is sufficient.

Calculation 5 – Shear stress in diaphragms

The greatest shear stress in a diaphragm happens when one of the four loading points is directly above the edge of the diaphragm. When this happens, the shear in the member is just under 5 kN.

$$\tau = \frac{VQ}{I_x t}$$

$$Q = \frac{b \cdot h^2}{8} = .0000929m^3$$

$$I_x = .00000865m^4$$

$$\tau = \frac{(5kN)\left(.0000929m^3\right)}{\left(.00000865m^4\right)\left(.0381\right)} = 131.2kPa$$

$$\tau_{allowable} = 605kPa$$

Since the shear stress is less than the allowable stress, the design is sufficient.

Calculation 6 – Shear in steel deck rod

The steel rod in the deck transfers load from plank to plank, and in doing so can experience significant shear stress. This stress had to be checked against allowable values for our particular type of steel.

$$\tau_{\max} = \frac{3V_{\max}}{2A} \quad \textit{For circular cross section}$$

$$V_{\max} = 5kN$$

$$A = \left(.25in^2\right)\pi = .1963in^2 = .0001267m^2$$

$$\tau_{\max} = \frac{3 \cdot 5kN}{2 \cdot .0001267m^2} = 59.2MPa$$

$$\tau_{allowable} = \frac{\sigma_{yield}}{2 \cdot (F.S.)}$$

$$\sigma_{yield} = 878MPa; \quad F.S. = 2.0$$

$$\tau_{allowable} = \frac{(878MPa)}{2 \cdot 2} = 220MPa$$

Since the shear stress is less than the allowable stress, the rod is sufficient. The value for yield stress was provided by the manufacturer.

Calculation 7 – Crush strength of bearing blocks

Crushing in the bearing blocks would prevent the cables from effectively supporting the bridge. Thus we had to check that the area of the metal bearing block was sufficient to distribute the compression of the cables without crushing. If it were not sufficient, we would have to make larger metal bearing plates.

Area of the bearing plate, A: 5.94in^2

Since our bearing blocks were not exactly parallel to the grain Hankensen's formula had to be used to determine the allowable stress at an angle of 10.24 degrees. The formula is as follows:

$$F_n = \frac{F_g F_p}{F_g \sin^2(\theta) + F_p \cos^2(\theta)}$$

Where

F_n = allowable stress at angle θ

F_g = allowable stress parallel to grain = 1600 psi

F_p = allowable stress perpendicular to grain = 565 psi

This results in

$$F_n = 1512 \text{ psi} = 10.42 \text{ MPa}$$

From ANSYS, the greatest force in the cables is 8946 N, which equals 2011 lbs.

The stress on the bearing block is

$$\sigma_{compressive} = \frac{P}{A} = \frac{2588 lbs}{5.94 in^2} = 435 psi = 3.00 MPa =$$

Since this is well below the allowable stress, the area of the base-plate is sufficient.

Calculation 8 – Bearing block sliding

It was important to know if the bearing blocks would slide downwards when the cables were tensioned. If they would be sliding, we would need to know the downward force to determine how many screws to attach the blocks with. If we call the tension in the cable T and the coefficient of friction μ_{static} , then to avoid slipping the following must be true:

$$T\,\mu_{static}\cos(\theta) \geq T\sin(\theta)$$
$$\theta = 10.24^{\circ}\,;\,\mu_{static} = .25$$
$$T(.25)(.984) \geq T(.1778)$$
$$(.246) \geq (.1778)$$

Thus we did not have to worry about the blocks slipping downwards.

Calculation 9 – Cable tension

Though our cables had a high ultimate strength, we still had to check that they would not suffer tensile failure.

$$A_{cable} = .184in^2$$
$$\sigma_{crit} = 170ksi\ ;\ F.S. = 2.00$$
$$T_{max} = \frac{A_{cable}\cdot\sigma_{crit}}{F.S.} = \frac{.184in^2\cdot 170ksi}{2.00}$$
$$T_{max} = 15.64kips$$

$$15.64\ kips = 69.6\ kN$$

From ANSYS 2, we know that the maximum force in the cable is 11.51 kN, far less than 69.6 kN, so the cables are adequate.

4 Sponsorship

Although the majority of our building materials, including all timber components, were purchased by the department, we were able to offset some of the costs associated with the project by gaining sponsorship from three companies. Simpson Strong-Tie donated the 16 joist hangers and the three custom made combination post cap joist hanger. McFeely's Square Drive Screws donated the Auger Point stainless steel screws that were used throughout the bridge. VSL International donated the cables that were used to support the understructure of the bridge. In addition to the cables, they also provided us with the cable anchorages and the ram that was used to pre-tension the cables. Overall, our sponsors saved us over $1400 in material costs. Without their support we would not have had the opportunity to use such high quality materials and tools in the construction of the bridge.

5 Construction

The details of the construction of the bridge are presented in this section. A summary of the weights of all screws, nails, and bolts used in construction is provided in Table 5.1.

Table 5.1: Bridge and Nonwood Weights

Material	Mass (kg)
Glue	2.11
Nails	2.40
Screws	6.30
Nuts, Bolts, & Washers	4.61
Hangers	33.02
Cables/Misc Metals	23.90
Subtotal	**72.34**
Total Bridge Weight	447.00
% Nonwood Weight	**16.18**

5.1 Frame

5.1.1 Beams

The three girders and the two center diaphragms were built-up members, constructed using the following procedure. For the diaphragms, this was necessary to fit the width of the Simpson Strong-Tie custom connections, while for the girders, this was necessary to span the 3.8 meter gap while complying with article II.b, and having no individual member longer than 2.1 meters. The highest quality timbers from our available supply, those members with the fewest visible defects, were selected and cut to the proper dimensions for gluing and screwing. Before assembling each beam, its components were dry-fit and examined to ensure optimal grain direction, dimensional accuracy, and flushness. The components were then laid flat, side-by-side, and wiped down with a damp cloth. A layer of Gorilla Glue was applied as quickly as possible to one side of the components being glued. The components were then placed together, nailed together to prevent any sliding while being handled, and tightly clamped, as shown in figure 5.1.

Figure 5.1: Fabrication of Built-Up Girders: Clamped after Gluing

There were approximately two clamps per foot of beam being glued. The beams were then left clamped overnight for the Gorilla Glue to cure under pressure. The clamps were removed the following day, but the girders were not screwed together for several days because the screws had not yet arrived. As a result of the Gorilla Glue expanding and inherent flaws in the wood, some of the individual members in each girder warped away from one another. This was partially corrected by inserting wood glue into the crevices once the screws had been installed. Once all the screws were in place, the original positioning nails were removed. The screws we used were stainless steel auger point screws, provided by McFeely's Square Drive Screws, as shown in figure 5.2.

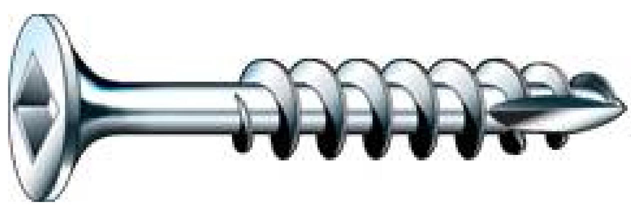

Figure 5.2: A McFeely's 305 Stainless Auger Point Screw.

We selected these screws for their treatment and the auger point, which tends to drill and remove some wood rather than push and split the wood, like conventional screws. These screws performed satisfactorily, but were occasionally stripped just before being fully driven in because stainless steel is not as hard as tool steel. However, this caused few problems, as we were able fully drive them in, or remove them, as necessary with a vise grip.

5.1.2 Joints/Connectors

The girders and diaphragms were connected using 19 Simpson Strong-Tie timber connectors, 3 of which were custom-built to our specified dimensions. These custom-built connectors, shown in figure 5.3, were a combination of joist hanger and post cap and employed ½" bolts.

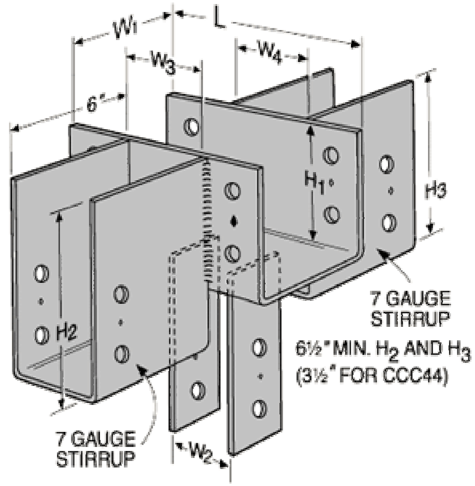

Figure 5.3: CCC and CCT Custom Connector Dimensions

The other 16 connectors, shown in figures 5.4 and 5.5, were simple joist hangers requiring 10d and 16d nails.

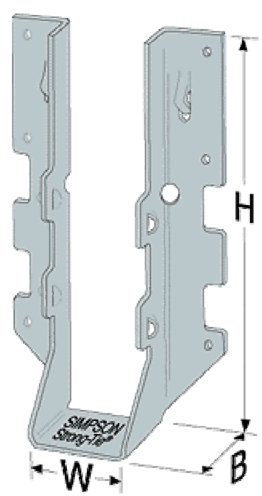

Figure 5.4: HUS26 Joist Hanger used at Quarter Points

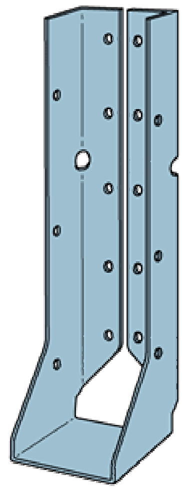

Figure 5.5: LUC26Z Joist Hanger with Concealed Flanges used at Bridge Ends

All connectors were dry-fitted before the final connection, whereby we discovered that custom connections dry-fit in place and level, bolt holes were marked and drilled through the girders. The other sixteen connectors were first nailed to the girders, then the diaphragms were placed into the connectors, leveled, and nailed in place. For the diaphragms at the 1/4 and 3/4 span of the bridge, the connectors were staggered by ½ a nail diameter such that nails within the girders would not interfere with one another. The staggering was 1/8" away from the center of the bridge on one side and 1/8" towards the center of the bridge on the other. All diaphragms were leveled by placing strips of timber in the connectors, with the diaphragms resting on top of these strips. The tops of the girders were leveled by planing sections of the girder that were too tall, and affixing strips of timber to the sections that were to short.

5.2 Understructure

5.2.1 King Post

The king post was by far the most intricate component of the bridge's structure. The king post consisted of a 4x6 running the width of the bridge, with three 4x4 blocks screwed on top, and semi-circular grooves for the cables on the bottom. The three custom connectors fit over these wooden blocks and ran down the sides of the post. Blocks were then placed on either side of the 4x6 to extend the support given to the curved half-pipe controlling the 18" cable bending radius. These blocks had two holes in them corresponding to the two all-thread rods that ran through the post and were fastened to the connectors with washers and nuts. The very bottom of the connectors had to be milled off to meet the maximum depth of understructure constraint from article II.g and provide room for the cables. Underneath each connector, a semi-circular path was carved out using a Dremel rotary tool to make room for the 1" outside-diameter steel half-pipe. A picture taken during this process is shown in figure 5.6.

Figure 5.6: King Post Construction Showing Milled Grooves and Half-Pipes

This required cutting an appropriately curved path through the king post, the wooden blocks, the steel connector, and the washers. The outside edges of the half-pipes were welded to 2 2x4x1/8" thick steel plates, connected to the 4x6 with two screws. Epoxy was placed into the semi-circular groove before attaching and screwing the half-pipe-and-plate fixture; the epoxy ensured a more even load transfer from the cable to the king post. In order to achieve the correct curvature for the initially straight half-pipes, a mold was cut out of wood on the bandsaw; with the half-pipe inside, the mold was placed in the hydraulic press and loaded until the half-pipe matched the mold curvature. This process was repeated for all three half-pipes using same wooden mold to ensure the same curvature for all three. Once the post was bolted to all three connectors and the half-pipes were in place, the whole assembly was positioned and bolted to the girders.

5.2.2 Cables and Anchorages

In order to accommodate the 1/2" cables (5/8" thick with sheathing), 3/4" holes were drilled from the center of the ends of the girders downward at an angle of 10.24°. We found that drilling the 28" holes from both ends with a 15" drill bit was exceedingly difficult. An aluminum guide was built to ensure that the holes were at the correct angle and did not significantly veer off in any direction. The guide would be nailed into place, the whole would be drilled, and then the guide would be removed. While the guide helped, we still found that our two holes were not meeting in the middle of the girder. We were finally able to drill each hole using the guide, and a wood drill and drill extender purchased from Home Depot. A view of the wood drill extending through the entire hole can be seen in figure 5.7.

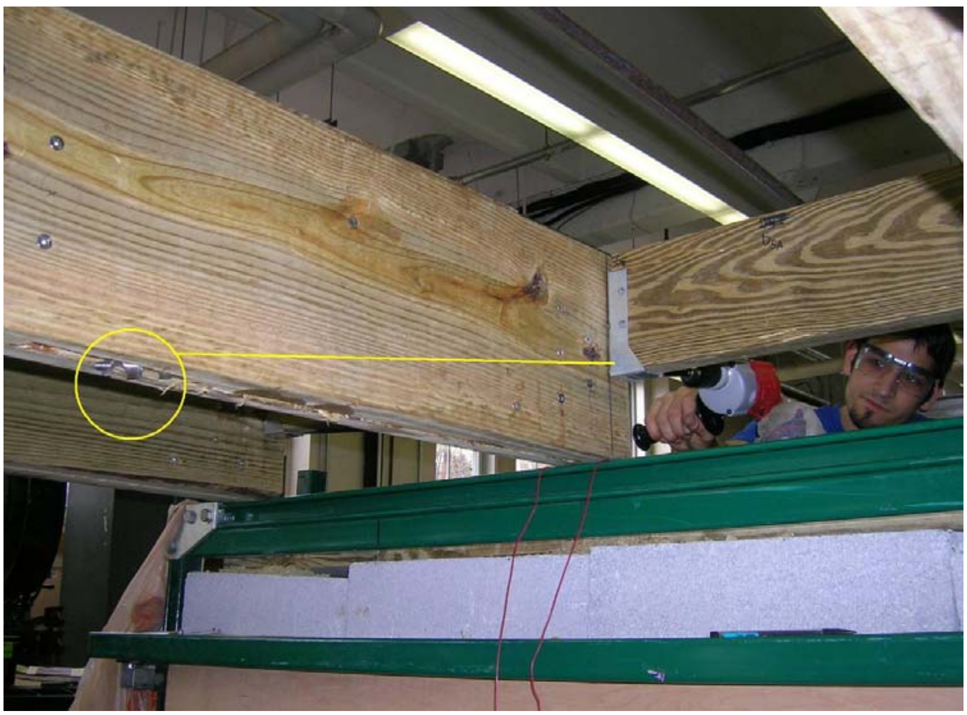

Figure 5.7: Wood Drill and Extender Going Through the entire 28" Hole

A 2x10 header plate was screwed into the frame on both ends of the bridge to help distribute the forces from the cables. Timber anchor blocks were then screwed into the header plate, and the 3/4" holes for the cables were extended through both header plate and anchorage at the same 10.24° angle. The cables were then inserted into the bridge, passing through the girders and under the half-pipes.

The cables were anchored on each end of the bridge with T-5 anchorages provided by VSL, a schematic of which is shown in figure 5.8. The header plate, anchor blocks, cables and anchorages are shown in-situ in figure 5.9.

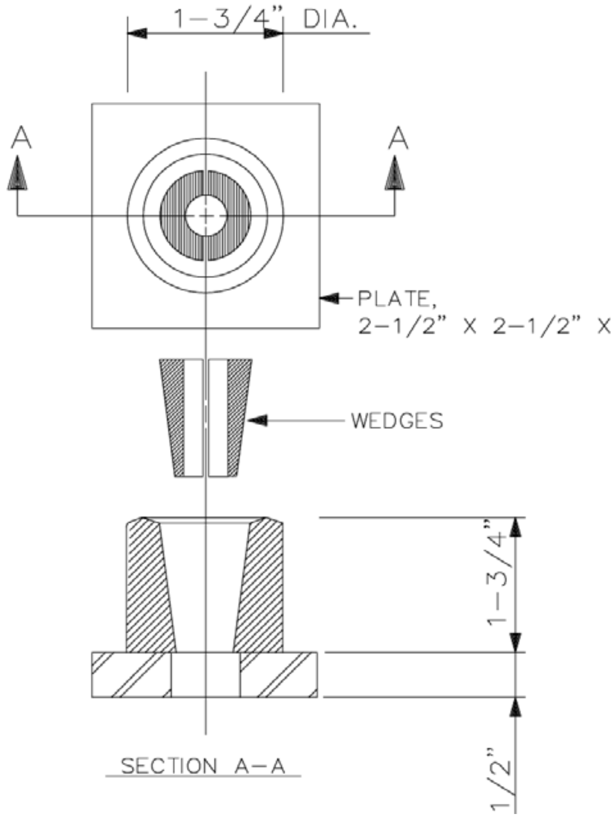

Figure 5.8: VSL T-5 Anchor Detail

Figure 5.9: Close-up of Header Plate, Anchor Block, Cable and Anchorage

We pre-tensioned the cables 48 hours prior to loading with a ram provided by VSL. The cables under the outer girders were pre-tensioned to 1500 lbs for both outside cables, while the central cable was pre-tensioned to 1000 lbs for the center cable. The ram calibration curve, along with additional data and schematics for VSL cables and tensioning systems is included in Appendix C. In order to meet VSL workplace safety requirements, we erected an eight foot square wall to stand directly behind the cables being pre-tensioned, thereby obstructing any cable that should slip and break away from the bridge under high load. The wall consisted of two layers of 3/4" thick plywood nailed together, and then to a 2x4 frame. The wall is shown below in figure 5.10.

Figure 5.10: 8' x 8' Safety Wall

5.3 Deck

The deck consisted of 13 2x12, and 2 2x2, planks resting transversely on top of the frame, with 2 1/2" steel rods passing longitudinally through all deck panels at 1/3 and 2/3 the width of the bridge. For each deck panel, we undertook the tedious process of precisely marking the holes, securing the panel perpendicular to the drill-press with clamps, and then drilling the holes from both ends. A 5/8" drill bit was the largest drill bit that passed entirely through each panel, but a 3/4" drill bit was used on the outside thirds of each panel to provide play where the planks came together. With this extra wiggling room, the panels were worked onto the rods by pressing a wooden block on the edge of the panel between the rods and lightly hammering. This process is illustrated in figure 5.11.

Figure 5.11: Deck Planks Being Pushed onto Deck Rods

The ends of the rods were threaded at the ends, and employed a nut over a washer and a 2x4x1/8" thick steel plate on either end used to tighten the deck into compression. Short 1.5" screws were inserted on both sides of the rod, to prevent any splitting of the wood due to the rod. The deck was attached to the frame using 4.5" screws, two screws in each non-center diaphragm, four screws in each center diaphragm, and six per deck plank into the girders. The curb, which consisted of 1.9m long sections of 2x4 laid flat, was connected to the deck using one 2.5" screw per deck plank.

6 Weighing and Loading

6.1 Weighing

One reason we chose the Hicks basement cage was for the translating mechanical assist pulley system, which would allow us to weigh the bridge. This system consists of two parallel beams that are spanned by a third beam on rollers. Attached to the third transverse beam is a pulley assembly also with rollers. As a result of this design, this system can be used anywhere in the basement cage area. It has been rated for a maximum load of one ton. Due to the load rating of the lift, we performed wood volume calculations to confirm that the bridge, when lifted for weighing, would not exceed the load rating. In order to weigh the bridge, we connected a calibrated strain gauge transducer to a hook attached to the pulley. We then wrapped chains around the mid section of the bridge, where they could be wrapped around the metal connectors and not dig into the wood, and hoisted the bridge into the air. The strain gauge transducer was attached to a signal conditioning box which listed a value of 5418 μstrain, equivalent to 985 lbs. This entire assembly is shown in figure 6.1, and with another angle of the suspended in bridge is shown in figure 6.2.

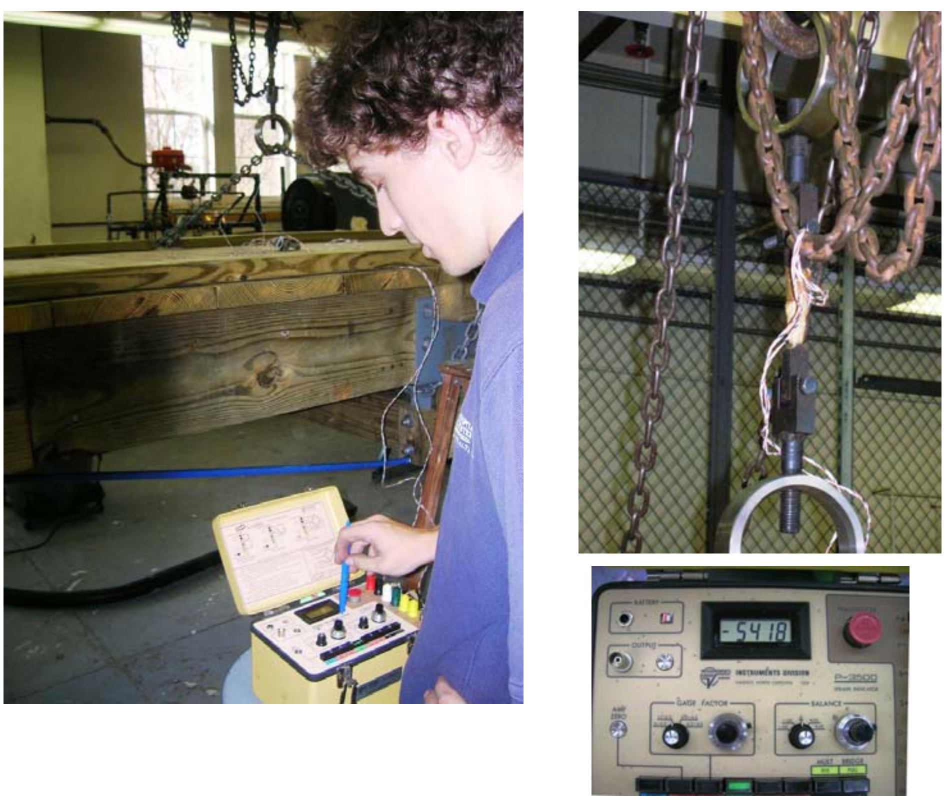

Figure 6.1: Strain Gauge Transducer Setup

Figure 6.2: Bridge Suspended About Mid-Span Connectors During Weighing

6.2 Unistrut Loading Substructure

While the bridge was hoisted into the air for the weighing, we had the opportunity to swap out the original construction supports for the Unistrut loading supports, as seen in figure 6.2. While the concrete block construction supports were certainly adequate to the task, we had designed our Unistrut substructure to undergo minimal deflection under the 20 kN load. This was done to minimize overall bridge deflection since, according to article III.c.ii of the codified rules, subtraction from overall deflection due to compression of supports was not allowed.

6.3 Loading Setup

The competition rules provided us with basic guidelines to follow regarding load application. According to the competition rules, the loading setup had to be implemented in such a way as to measure the bridge's maximum vertical deflection and its maximum net deck deflection. To accomplish this, we had to perform two distinct loadings, with slightly different loading setups. The rules specified that the load must be applied in four equal increments of 5 kN each, achieving the full 20 kN load in not less than five, and not more than twenty, minutes. Deflection measurements were then taken after each loading increment, and at four 15 minute intervals after achieving full load. We used dial gauges to monitor the deflection of all designated points of interest. Competition rules specified that for the maximum vertical deflection measurement, the dial gauge had to be located at the mid-span of the longitudinal beam "receiving the greatest loading." In the case of our bridge, this should have been at the mid-span of the center longitudinal beam. However, due to the understructure post located at this point in the beam, the gauge could not be placed directly on the beam. Therefore, we obtained permission from the competition coordinator to monitor the deflection of the lowest point of the understructure directly beneath the center girder, the bottom of the cable sheathing. In the case of the maximum net deck deflection, three dial gauges were used as specified by the competition rules. One was placed in the center of the largest deck panel, and the other two were placed under the beams directly surrounding that deck panel. The locations of all gauges and points of load application are shown in figure 6.3.

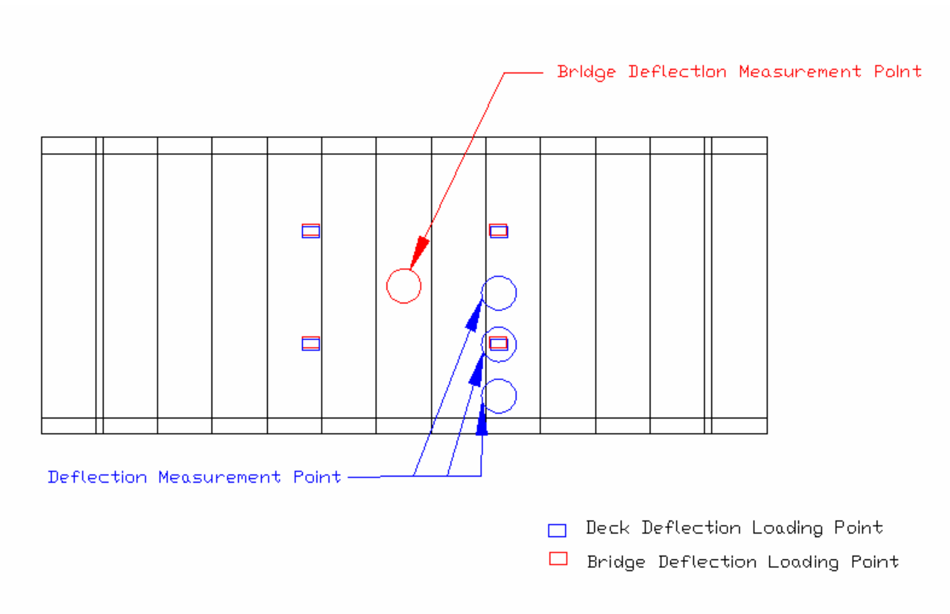

Figure 6.3: Plan View of Bridge and Deck Deflection Loading Setups

To load the bridge, we stacked 129 solid 3.5"x7.5"x15.5" concrete blocks on a sufficiently rigid half inch plate of aluminum seated on four 60mm x 90mm steel blocks over similarly-sized 70-durometer rubber pads. This configuration, shown in detail in figure 6.4, was used to ensure adequate support for the 20 kN load, and prevent a bearing failure of the wood in direct contact.

Figure 6.4: Close-up Detail of Loading Point Transfer Mechanism

The four loading increments were prearranged on palettes that were then brought to the bridge by a forklift. Since there was not enough space for the forklift to unload the palette and back up, each load of blocks was arranged symmetrically over the four loading points by students, with small calibration weights used to complete each full quarter increment. This process is illustrated in figure 6.5.

Figure 6.5: Manpowered Loading Process

Since it had been raining for hours before loading, both the blocks and the palettes had gotten heavier, and we made slight adjustments to each load increments. This loading process was carried out once to determine overall bridge deflection with the load placed directly in the center, and again for deck deflection with the centroid of one loading block placed directly over the centroid of the largest deck panel. The first loading took 18.5 minutes to complete while the second took only 10 minutes. A picture of the bridge supporting the full 20 kN load is shown in figure 6.6.

Figure 6.6: Trimetric view of Bridge Supporting Full Load

7 Data

7.1 Deflections

As the load came to its full value, we observed no discernable audible or visible signs of change, besides a significant tightening of the cables. In the hour following the application of the final load increment, the bridge deflection crept an additional 4.0%. The total bridge deflection of 8.66 millimeters was greater than the predicted deflection of 4.5 millimeters obtained from our ANSYS analysis, but since the structure was designed conservatively, the deflection was still within allowable limits. Explanations for our excessive bridge deflection will be presented in the discussion section. The deck performed as expected, with the longitudinal steel rods transferring the force of the load from one deck plank to the next, and limiting deflection to only 28% of the allowable value. Interestingly, 45 minutes after achieving full load, the deck had stiffened such that the net deck deflection stopped creeping, and only the girders crept more. Throughout the loading cycles, the center girder experienced slightly higher deflections than the outside girders due to its thinner cross-section and higher load. There were no apparent asymmetries in the girder deflections that might point to fabrication inconsistencies, uneven stresses in the cables, or anchorage eccentricities. The deflection results and calculations are summarized below in Table 7.1. Net deck deflection was calculated by subtracting the average deflection of the beams on either side of the deck area from deck deflection.

Table 7.1: Deflection Data in Millimeters

	Mid-Span of Bridge	Beam Left	Beam Right	Beam Average	Gross Deck	Net Deck
Loading Bridge						
1st Increment	1.88	1.66	1.30	1.48	1.80	0.32
2nd Increment	4.04	3.28	2.64	2.96	3.76	0.80
3rd Increment	6.16	4.80	4.05	4.43	5.56	1.14
4th Increment	8.33	6.43	5.35	5.89	7.34	1.45
Bridge Fully Loaded						
15 minutes	8.48	6.50	5.44	5.97	7.47	1.50
30 minutes	8.56	6.55	5.45	6.00	7.52	1.52
45 minutes	8.61	6.58	5.49	6.03	7.57	1.54
60 minutes	8.66	6.62	5.50	6.06	7.59	1.54
% Maximum Allowable	**91.17%**					**28.06%**

7.2 Competition Results

Out of 12 teams competing this year, our team was awarded the 2nd most innovative design, the 3rd best overall design, and placed fourth in monetary winnings with $450. We had hoped to achieve greater recognition for our stiff deck design, but were otherwise satisfied with the results. The official 2007 competition results table is included at the end of this report, before the appendices.

7.3 Cost Estimates

Based on the results of past competitions, we entered the project expecting to spend $500-$1500, most likely in the higher end due to Swarthmore College's lack of construction resources relative to other competing institutions. A summary of our budget, broken down into sponsored and non-sponsored expenditures, is shown below in Table 7.2.

Table 7.2: Project Budget

Materials Purchased	Cost
Timber	$506.40
Nails & Glue	$70.00
Threaded Rods	$80.00
Material Purchased Subtotal	$656.40
Prize Winnings	($450.00)
Net Cost to Department	$206.40
Sponsored Materials	
Screws (McFeely's Square Drive Screws)	$392.00
Steel Connectors (Simpson Strong Tie)	$806.90
Cables and Pre-tensioning Equipment (VSL)	$250.00
Sponsored Materials Subtotal	$1,448.90

As can be seen in table 7.2, our total budget exceeded our initial estimate. However, sponsored materials composed 69% of the total budget, such that our actual expenditures were in the lower end of the prediction. Timber was the largest purchase the Swarthmore Engineering Department paid for, composing 77%, by cost, of non-sponsored materials.

We were under the impression that the NTBDC sponsor, Weyerhauser, would be covering this cost. This was not the case. The NTBDC website suggested seeking sponsorship from the local ASCE chapter, the FPS, or a local lumber yard. All of these possibilities were pursued unsuccessfully. Nevertheless, after subtracting for our prize winnings, our ultimate expenditure of $206.40 is well below the $100 per student budget allotted for the senior design project.

Other materials were purchased along the course of the project, but were not included in the budget in Table 7.2. This was due to their undiminished value and therefore future usefulness to the department. These materials include many unistrut members and connectors, a shopvac, various drill bits, a FatMax hammer, and two large levels.

8 Discussion and Reanalysis

While our bridge did meet all of the minimum competition criteria, the total bridge deflection of 8.66 millimeters was significantly higher than the 4.50 millimeters we had anticipated. There are several possible reasons for this excessive deflection: the wood used was not intended for structural use, the ANSYS analysis did not take into account either the shifting or settling of joints in the frame or shear deflection, and the supports relied on bolted connections, not nearly as stiff as the rigid connections assumed to exist in the ANSYS analysis.

8.1 Non-structural grade wood used

Timber is rated according to its density and consistency. Dense select structural grade timber, the highest grade available, is cut from the densest and most consistent pieces of timber, resulting in the highest stiffness and ultimate strength; their consistency is ensured by more stringent inspection requirements, which preclude most defects from appearing in structural grade timber. Unfortunately, our bridge was constructed with #2 grade timber, which, of ten possible classifications for 2 x 10's, is the third weakest. This could have had a very significant impact on the total deflection since dense select structural grade timber has a modulus of elasticity about 35% greater than that of the #2 wood that we used. However, even when using this lower modulus of elasticity, our model showed lower deflections than we observed. It should be noted that while the higher density of structural grade timber would tend to lower deflections, the exclusion of defects, like knots and edge cracks or checks, plays an even more significant role in low deflections. This becomes clear when one considers that all timber design equations are based on a relatively homogeneous material with no defects. Therefore, we attribute a significant amount of our excess deflection to the presence of numerous defects in our poorly graded timber, as shown in figure 8.1.

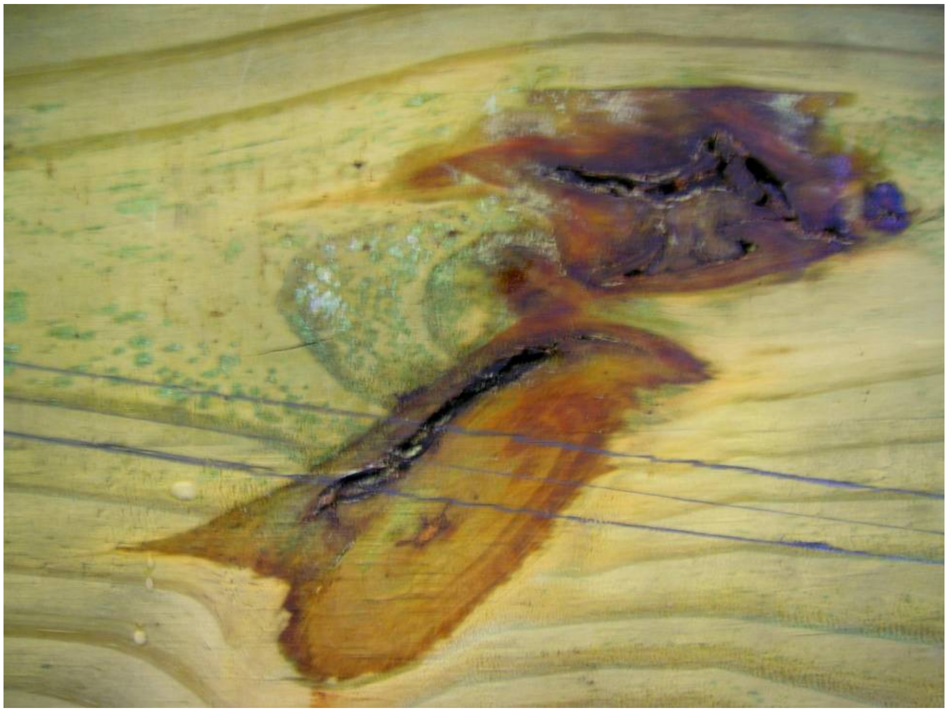

Figure 8.1: Less than Desirable Defects in our #2 Grade Timber

8.2 Settlement of joints and Shear Deflection

We did not take into account the possible settlement of joints largely because we were unaware that it might occur. There were certainly numerous locations on our bridge with bolted connections with slight play in the holes. In retrospect, it becomes obvious that these gaps would be the first to close under heavy load, contributing to deflection. These settlements would remain after the load was removed, showing a permanent deflection. However, since our bridge recovered all but 0.004 inches of deflection when the load was removed, we believe the effect of joint settlement was minimal.

The bridge was not modeled to take into account the effects of shear deflection, as its aspect ratio was greater than five, and therefore shear deflection was not expected to play a large role. However, in reanalysis, we found that due to the low shear modulus of wood, our girders, with an aspect ratio of about 15, experienced an extra 10% deflection due to shear.

8.3 Unistrut Loading Substructure

The Unistrut supports deflected far more than they were designed for, for a variety of reasons. This deflection was so excessive and unexpected, that we attribute most of our unaccounted for bridge deflection to the supports. Due to time constraints, we unfortunately did not monitor support deflection during loading. We performed a a test with a team member's weight over the supports, and linearly extrapolated from that a support deflection of about 2 mm at full load. Since this was already over an order of magnitude higher than expected, there were clearly some incorrect assumptions in our model. Two differences between our actual frame and the ANSYS model frame which added significant deflections were the use of bolted connections, and the slightly out-of-plane orientation necessary to achieve crossing struts, as can be seen in figure 8.2.

Figure 8.2: Side Elevation of Unistrut Substructure Showing two Transverse Planes and Shimming

The bolted connections did not behave as rigidly as the rigid connections in ANSYS, undergoing much more rotation and translation to resist moments. Another factor to significantly affect support deflection was the necessary use of shims to level the frame on the uneven basement floor. Since the ANSYS analysis restricted vertical deflection of all load bearing points on the ground, it would provide a poor approximation of the less restrained shimmed frame. In fact, analyses performed with a less rigid floor, less vertical restraint, show five-fold increases in deflection. Not only would shims provide a non-ideal bearing surface to resist load, but each collection of shims had to compress a significant amount, about 1 or 2 mm, before becoming rigid. This was clear because we were still able to slightly move the shims before the load was placed, so some compression occurred to restrain their movement. Also, since the shims were formed by shearing, each shim had a slightly raised lip around the edge, making some compression necessary before the full bearing area of the shim could be utilized. All these factors significantly increased the deflection of the supports, which therefore increased the overall bridge deflection. The frame could also have been redesigned with columns directly under the concentrated loads, as it should have been designed originally, but was not due to time constraints.

9 Future Bridge Life

At the outset of the project, we had no planned location for the bridge to be placed and made serviceable. As such, preliminary investigations into the deconstruction and disposal of the bridge were undertaken. Fortunately, Professor Carr Everbach of the Swarthmore College Engineering Department found a home for the bridge after viewing the finished product.

Figure 9.1: Dicks Run Creek, Downstream View at Proposed Bridge Crossing

9.1 Bridge Location

The bridge will span Dicks Run Creek running through Professor Everbach's backyard. End views of the bridge location are shown below in Figure 9.2. Property on both sides of the creek is owned by Professor Everbach, making any spanning operation viable.

Figure 9.2: Proposed Bridge Location, End Views A and B

9.2 Bridge Transportation

The bridge needed to be transported from the basement of Hicks where it was resting on its Unistrut supports to as close as possible to its final location in Professor Everbach's backyard, situated a few miles away. The bridge was first raised using the lift in Hicks basement so that the Unistrut supports could be replaced with concrete blocks. This left the bridge resting much closer to the ground. The bridge was then flipped over and replaced onto the concrete substructure by attaching the hoist to one side, lifting, then translating the hoist across the bridge and lowering. With the bridge then in the correct orientation, it was once again hoisted so that the concrete substructure could be removed. A trailer with a cradle to hold the bridge was then rolled underneath it. Both the trailer and the cradle were property of Professor Fred Orthlieb. With the bridge resting in the cradle, the hoist was removed and the bridge was strapped to the trailer. The trailer was then pushed out of the cage, where it was attached to Professor Orthlieb's and transported to Professor Everbach's property with great haste.

Once the bridge had arrived at Professor Everbach's property, the bridge needed to be moved from the trailer on the driveway downhill to the edge of the creek. One end of the bridge was lifted from the trailer using a pry-bar and was guided down a track built of two 2x6 rails resting on concrete blocks. A tether attached to the back of the bridge and held with manpower was employed to prevent gravity from getting the best of the bridge. The bridge remains at the edge of the creek, resting on the rails and concrete blocks, awaiting the construction of its final resting place.

9.3 Site Specifications and Considerations

The most obvious complication with this site is how to make our 3.8 meter bridge span a 7 meter gap. This means that additional supports need to be constructed to span the remaining length. Initially, the design shown below in Figure 9.3 was considered as the simplest possible.

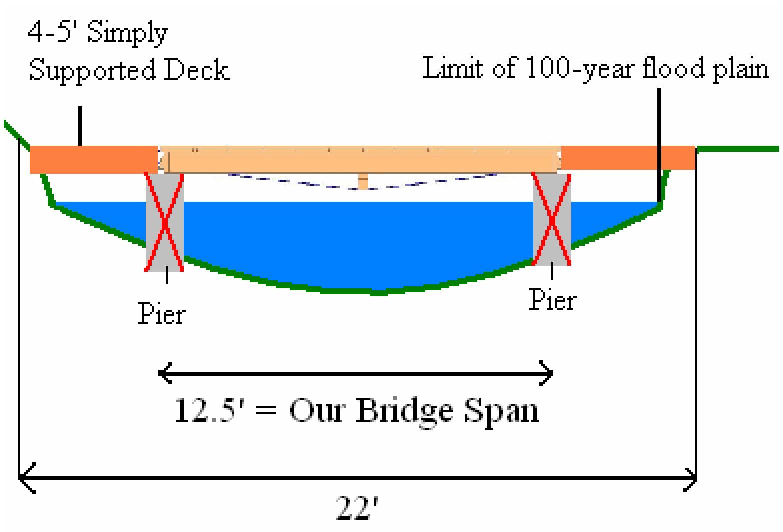

Figure 9.3: Simple Structural Alternative Requiring USACE Permission

While the design's simplicity is appealing, allowing us to support our bridge simply on two piers, and likewise have simply supported deck extensions, it is unadvisable. Dicks Run Creek is a 100 year floodway, designated as such by the United States Army Corps of Engineers (USACE). Therefore, no obstructions, such as piers, may be impede the creek flow without permission from the USACE. Therefore, we envision an alternate design, shown in figure 9.4, to be utilized by Professor Everbach this summer. The basic challenge for any alternative is how to transfer vertical loads from those points, where we cannot place piers, to the bank, thereby simply supporting both our bridge, and the deck extensions. Our primary alternative involves counter-weighting the deck extensions over a concrete fulcrum laid above the floodplain. This is desirable over a rigid cantilever connection because it would be hard to achieve the necessary moment resistance without a very heavy concrete pour. The counterweight could be achieved with poured concrete, creek stones, or even earth weight if the left bank is excavated. One secondary alternative we have considered would involve suspending the bridge ends with wire rope slung around nearby trees.

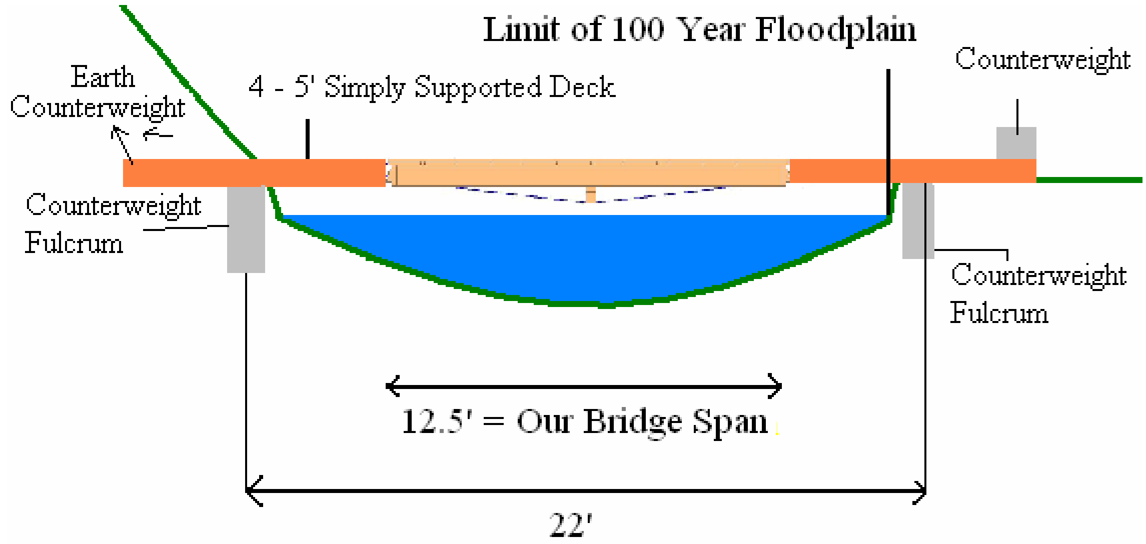

Figure 9.4: Primary Structural Alternative

The structure of the deck extensions will be a simplified extension of our bridge frame. Support will be maintained from two outer girders sistered to the outer girders of our bridge, or a large central girder running underneath the bridge deck. For such a short deck section so close to shore, no diaphragms will be necessary to help transfer loads.

Other considerations include effects to the environment, safety, and service-life. The steel rods running through the deck are not galvanized, but this is not an issue, as the bridge is more than capable of handling service loads without them, and so they can be removed or allowed to rust. The wood will be painted or coated for enhanced protection from the elements. As for safety, the bridge has been designed to hold much more than it will bear during service considering it will be spanning a small creek in a residential neighborhood. The additional supports will also be overdesigned by using large factors of safety. In the event that the creek floods, the bridge will be firmly attached to the deck extensions, but even if these connections fail, it will not be damaging to nearby property, as it will most likely float down the creek a short distance before becoming stuck. Manpower should be sufficient to move the bridge back to its proper place and reattach it. If the bridge or its supports experience some mode of failure, the repercussions are likely to be minimal, as the bridge is suspended no more than five feet from the ground. In any case, Professor Everbach has insurance on his property for damages up to $1 million, and is willing to assume any liability.

10 Conclusions and Lessons Learned

"Measure twice, cut once" is a quote our advisor, Professor Siddiqui, is fond of saying. It was originally from a master carpenter, a status we were ever so slightly closer to attaining at the completion of the project than at the outset. A master carpenter knows how difficult and costly it can be to fix a hastily made mistake and how important it is to take the time to fully think out the problem and do the job correctly the first time around. For this project, we faced a number of obstacles to getting the job done right the first time around, mainly inexperience and time constraints. This section highlights some of the many difficulties we encountered during this project, and the lessons we took away from them.

10.1 Construction

With no significant construction or wood-working experience, our team faced a steep learning curve, which we surmounted through much trial and error, and a little research. The major lessons learned are provided in this section in roughly the order they were discovered. Our girders were the first major components to be constructed, and they were not at all properly prepared or joined. All composite girder members should have been planed and leveled to identical dimensions before gluing and screwing. This would have saved the significant time spent on planing and leveling all girders, which required the most work but were too large to be brought back to the shop after assembly. The tops of the girders were leveled by cutting thin strips of timber in various sizes and gluing them on. Diaphragms were leveled from underneath by placing timber shims into the hangers. This can be seen in Figure 10.1.

Figure 10.1: Diaphragms Requiring Bottom Shims

A top view of the outside girders is shown below in Figure 10.2. It is clear that in the composite beam's design, the outside 2x10s coincide. This creates two cross-sections in which flexural rigidity is significantly lower. This was a mistake for which there was no fix, short of remaking the beams. In addition, after the beams had been constructed, they were milled down to their final length. This length was supposed to be 146.606", but due to an error in memory, the beams were cut to 146.06", instead. Fortunately, this did not affect the bridge's span because of the header plates, but the bearing surface was 0.3" shorter than allowed by the competition rules.

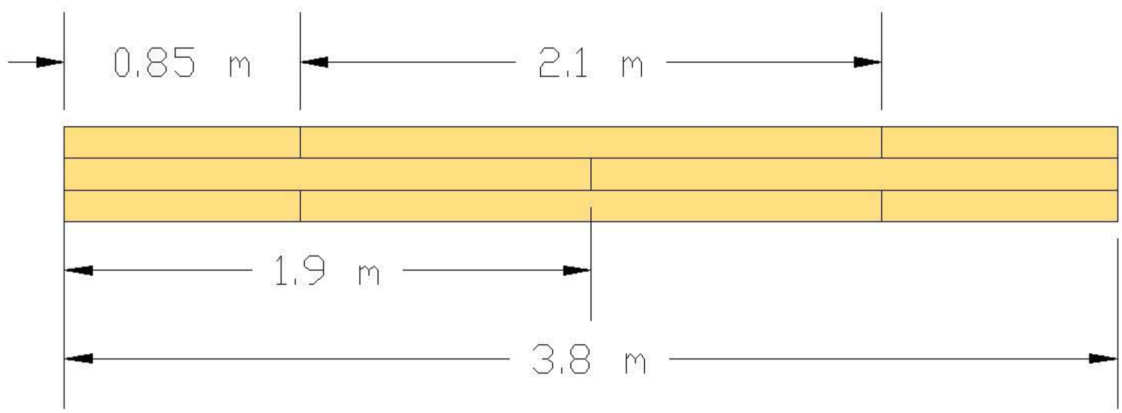

Figure 10.2: Lap Profile of Built-Up Girder

As mentioned in section 5.1.1, all composite beams underwent significant warping because they were glued and clamped together, but not screwed together until days later. This was a mistake that occurred due to the unavailability of screws at the time of glueing, but due to time constraints and a limited supply of clamps, our only other option would have been to place weights on the beams for days on end. This method was used with reasonable success for straightening some of the more warped deck panels. A photo of the results of this poor planning is shown in Figure 10.3.

Figure 10.3: Gaps on the Bottom of the Central Girder due to Warping

When our connectors arrived, we discovered that the outside girders were too wide to fit into the hangers. We initially used a hand sander to remove 1/2" from the width of the girders along 11" of their length. In addition to being extremely time consuming, dirtying our workspace, and leaving the surface of the wood wavy because of differences in density.

Figure 10.4: Quarter Point Diaphragms Staggering

Figure 10.4, shown above, provides a close-up view of where the two quarter-span diaphragms connect to the center girder. If you look closely, you can see that the diaphragms do not meet up at exactly the same location. They were shifted off-center in opposite directions by half a nail diameter so that the nails would not collide with one another.

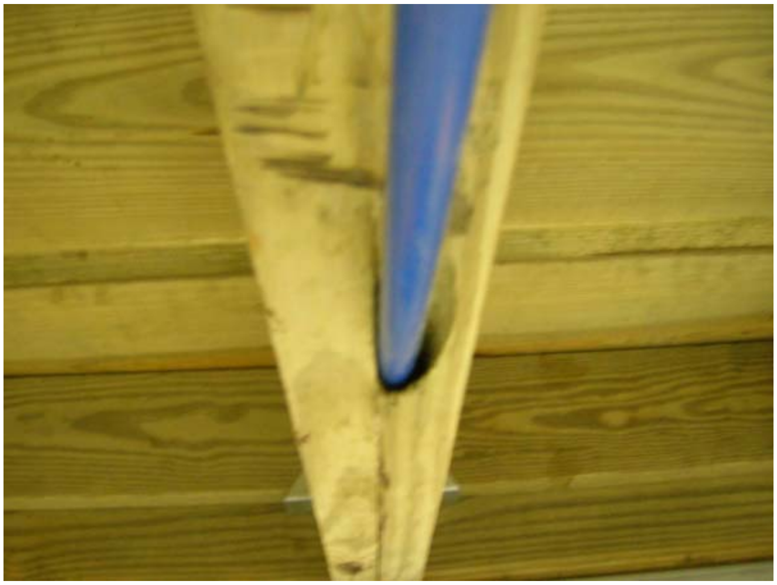

Figure 10.5: Cable Exiting Central Girder Slightly Off-Center

When drilling through the girders, imprecise methods involving strings and visual sightings were initially used. The results of this are shown above in Figure 10.5. As can be seen, the cable does not exit the girder at its exact center. This problem was solved by machining an aluminum guide set to the correct angle and easily attached to the end of any girder. The improvement was obvious.

When designing the understructure king post, we considered how to transfer the considerable force being applied to the posts from the cables. A harder material than our #2 grade timber would be required to seat the cable without crushing. The quick, and ultimately incorrect, decision was made to build the member out of mahogany. The only available mahogany was much larger than required and cost $200. This would have increased the cost of our project by 30%. Ultimately, this idea was scrapped in favor of our actual design, involving steel half-pipes to transfer the cable load to the wood, and thereby prevent crushing. Fortunately, the piece of mahogany was returned.

Over half the length of members we had selected when designing our Unistrut Loading Substructure were 1001C sections, whose geometry is shown in figure 10.6.

Figure 10.6: Geometry of P1001C Section

We placed our Unistrut order late, only four weeks before we needed it, and when the order arrived, it contained no 1001C Sections. Extra connectors were used to create alternative ways of connecting single channel members into 1001C-like members, until these, too, ran out. Finally, we had to change our design slightly to accommodate the lack of 1001C sections. The two bulky and imprecise solutions we used are shown below in Figures 10.7 and 10.8.

Figure 10.7: Unistrut 1001C Connector Solution 1

Figure 10.8: Unistrut 1001C connector Solution 2

The materials we had were barely sufficient to create the necessary 1001C sections. Our design also needed to be reanalyzed, because the improvised double sections did not run the full length originally planned.

10.2 Loading

While we clearly wish we had received the 1001C sections, we also wish we had understood the weaknesses of bolted connections. Had we known their weakness, we would have designed with more columns directly under the points of load application to minimize the necessary moment resistance at each bolted joint. This would have been beneficial, since bolted joints must deflect and twist more than rigid or cantilever connections to achieve the same moment resistance. As we mentioned in the discussion section, we experienced excessive deflection from our Unistrut support structures for a variety of reasons. Besides what was mentioned in the discussion, we should have redesigned the support structure to optimally resist the concentrated loads from the girders. The frame was initially designed using a geometry that assumed a distributed load, and therefore had columns evenly spaced along the quarter points. If we had placed the 1/4 and 3/4 columns directly under the outside girders instead, this would have reduced the ANSYS predicted deflection by a factor of three.

We had measured the clearances of the cage door beforehand to ensure that the Facilities Department forklift could fit. Unfortunately, we did not determine beforehand whether or not the forklift could actually drop palettes on the bridge and maneuver away. Since it could not, this necessitated some last minute scrambling for manpower, and some sore arms and backs the next day. A picture showing the tightness of the space used by the forklift is shown below in figure 10.9.

Figure 10.9: Forklift Backing out Carefully

There were 0.004" of deflection after the first loading that was never recovered. The bridge should have been loaded prior to our actual test load to remove such settlements from our results. The cables should have been pre-tensioned several times, as much of the tensioning was lost by the time of testing due to creep in the wood frame. The cables also probably should have been tensioned with to a higher load, as we were nowhere near the maximum camber restriction, or the buckling load of the girders.

10.3 Project Management: Planning and Working Effectively as a Group

It would have been helpful for our project to have finalized the design before beginning construction. Unfortunately, this was not possible, partly because we had to begin construction relatively early on, and partly because our lack of wood-working experience meant we had to continuously make changes to impossible or unwieldy designs. One thing we learned about conducting a project with four people was that without proper planning and coordination, we would never see efficient contributions from all four group members. A lack of "office time," a time when all team members were guaranteed to be present, contributed to much frustration, wasted time, and miscommunication. Were we to conduct this project again, we would certainly make an effort to make our schedules more compatible, especially when selecting classes in the previous semester. During the final two weeks of construction, when time pressure forced us to drop most other work and spend 10-12 hour days in Hicks, the communal office time allowed us to massively boost our efficiency. Without this office time, and without fully detailed schedules and plans, there arose an issue of accountability. Without a rigid schedule and recording system, it was difficult to hold any given group member accountable for work left undone. Unfortunately it took us the better part of the semester to come up with methods to effectively plan and accomplish work, but we finally got it working for the final presentation and report.

10.4 Working with Wood

Many of the biggest unforseen problems encountered during design and construction arose from our unfamiliarity with wood as a building material. The first problem was that the Southern Pine wood that we purchased to construct the bridge was not rated for structural uses. Had we been more familiar with working with, or acquiring, wood, we may have been better buyers.

We eventually realized that the dimensioned lumber we had purchased was by no means uniform, often with a variation in height of up to ¾" between successive 2x10s. Therefore, after gluing 2x10s together to construct our girders, we realized that we were left with far from plane surfaces, which would require many long hours of shimming and planing before it could be used. Had we understood such variables before beginning, we would have run each component through the joiner to achieve plane surfaces of similar heights before gluing.

Figure 10.10: Shrinking of Green Deck Planks Exposing Gaps

Another issue associated with the wood we purchased was its greenness, or high initial moisture content. This caused two main problems. One was encountered during the drilling of the holes for the cables. The wood was so moist, that the wood was very difficult to drill and in some cases we actually saw steam or water bubbling out of the wood. An even larger issue that we've observed since the loading is the wood amount of moisture the wood has lost. This is evident in that the decking has shrunk significantly leaving large gaps now between the decking boards, as shown in figure 10.10. While this shrinking may be less than desirable aesthetically, and puts some stress on the deck screws, it also guarantees adequate drainage off the bridge deck, and the screws are in no danger of failing.

Figure 10.11: Opposing Grain Orientations on Deck Could Lead to Cupping and Ponding

Another problem with our deck is shown in figure 10.11, with the opposing semi-circular grain angles of adjacent planks. All planks should have been laid in the orientation of the plank on the left of figure 10.11. Since wood tends to warp in the direction of the grain, a process known as cupping, the left plank would warp into a concave up shape, which would allow water to drain away to either side. The plank on the right, on the other hand, will warp into a concave down shape, trapping water in an effect known as ponding. The dangers of ponding will be minimal here due to the small deck area and gaps between planks, but it should still have been avoided.

Had we understood the qualities and existence of wood drills, we could have saved much time, frustration and money on buying long metal drills, which were inadequate to the task anyway. The metal drills not only required us to drill the cable holes from both ends, which proved very difficult, but also could only be used for ½" at a time. With no central bore, the metal drills did not remove material like the wood drill, and would soon begin to compress wood pulp at its tip rather than continue drilling. We were finally saved from this frustration by John Charles' father, who suggested using a wood drill with extender for a much cheaper and more effective solution.

The wood was also never perfectly straight, and was often bowed or curved. We encountered many problems when we tried to attach the bowed 2x10s together to form the built up longitudinal beams. As a result of using the bowed wood, the individual members of the girders separated from one another after the glue dried and we removed the clamps. The bowing that we observed may have been a result of the low wood grade and would therefore have been minimal with structural grade timber. Had we planned adequately to receive the screws prior to gluing the girders, this might have been avoided as it would have forced the wood to overcome the strength of the screws and the glue to bow back.

11 References

Forest Products Laboratory. 1999. Wood handbook—"Wood as an engineering material." Gen. Tech. Rep. FPL–GTR–113. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory. 463 p.

Post Tensioning Manual 1976 Post-Tensioning Institute, 2nd ed. Glenview Illinois PG:120

Ritter, Michael A. "Timber Bridges: Design, Construction, Inspection, and Maintenance." Washington, DC: USDA Forest Service, FPL, 1992.

Southern Pine Council. (2007). "Southern Pine Design Values, Table 1: Dimension Lumber" <http://southernpine.com/designvalues1.shtml> (Mar. 28, 2007).

Southwest Mississippi Resource Conservation and Development (RC&D) "Bridge Design and Test Setup." National Timber Bridge Design Competition., Inc. 5 Dec 2006, <http://www.msrcd.org/bridge.htm>.

2007 NATIONAL TIMBER BRIDGE DESIGN COMPETITION Summary of Results For rules of completed 2007 competition click here											
BRIDGE PERFORMANCE AWARDS – TOP 3 RECEIVE PRIZE MONEY											
School Team	Net Bridge Deflection[1] mm	Net Deck Deflection[2] (percent of allowable)	Total Bridge Weight kg	Percent Non-Wood %[3]	Best Overall Design	Best Support Structure	Best Deck	Most Practical Design	Most Aesthetic Design	Most Innovative Design	Total Awards $
B OR SU ASCE/FPS	3.95	71.2	384	7.13	6	5	11	9	**2**	6	200
506 OR SU ASCE/FPS	3.31	49.8	419	2.11	**2**	**2**	5	**2**	7	10	900
Clarkson Univ ASCE	15.93	68.7	635	10.7	DQ	DQ	12	12	5	**3**	100
Ohio SU ASCE	1.78	0.30	593	1.15	7	4	**2**	10	8	9	300
Swarthmore College ASCE	8.67	28.1	447	16.2	**3**	9	8	7	9	**2**	450
U of TN @ Martin ASCE	11.08	30.0	358	9.7	DQ	DQ	6	6	**3**	5	100
U of AR @ LR FPS	4.42	11.9	611	3.7	5	6	**3**	4	4	8	200
Team A - San Fran State ASCE	8.64	66.0	320	13.4	9	8	10	8	**1**	7	300
Team B- San Fran State ASCE	80.3	47.8	378	8.27	DQ	DQ	7	5	9	7	0
US Military Academy ASCE	2.92	10.8	425	1.40	4	**1**	**1**	**1**	9	4	1300
U of MO @ Columbia ASCE	5.42	71.6	297	7.34	8	7	9	**3**	6	10	100
OK State Univ ASCE	3.20	0.81	498	4.60	**1**	**3**	4	11	9	**1**	1050

[1] Maximum allowed = 9.5 mm
[2] Maximum allowable = deck span divided by 100.
A - Monitor and/or load points to determine deck deflection were not properly chosen to measure.
B – Deck deflection exceeded maximum allowable.
[3] Maximum allowed = 25%
DQ = Not eligible for performance award or Best Overall Design. Deflection (bridge and/or deck) exceeded allowable.
ASCE = American Society of Civil Engineers
FPS = Forest Products Society