Spring 5-2013

Precast/Prestressed Concrete Truss-Girder for Roof Applications

Peter S. Samir
University of Nebraska-Lincoln, pscross@unomaha.edu

PRECAST PRESTRESSED CONCRETE TRUSS-GIRDER

FOR ROOF APPLICATIONS

By

Peter Samy Samir

A THESIS

Presented to the Faculty of

The Graduate College at the University of Nebraska

In Partial Fulfillment of Requirements

For the Degree of Master of Science

Major: Construction

Under the Supervision of Professor George Morcous and Professor Maher K. Tadros

Lincoln, Nebraska

May, 2013

PRECAST PRESTRESSED CONCRETE TRUSS-GIRDER

FOR ROOF APPLICATIONS

Peter Samy Samir, M.S.

University of Nebraska, 2013

Adviser: George Morcous and Maher K. Tadros

Steel trusses are the most popular system for supporting long span roofs in commercial buildings, such as warehouses and aircraft hangars. There are several advantages of steel trusses, such as lightweight, ease of handling and erection, and geometric flexibility. However, they have some drawbacks, such as high material and maintenance cost, and low fire resistance. In this paper, a precast concrete truss is proposed as an alternative to steel trusses for spans up to 160 ft. without intermediate supports. The proposed design is easy to produce and has lower construction and maintenance costs than steel trusses. The proposed design is an evolution of the system that was developed by e.Construct USA, LLC and was used in the construction of a cement storage facility at United Arab of Emirates. The proposed truss design consists of two segments that are formed using standard bridge girder forms (PCI and AASHTO girders) with block-outs in the web that result in having diagonals and vertical sections. The two segments are then connected using a wet joint and post-tensioned longitudinally. The proposed design optimizes the truss-girder member locations, cross-sections, and material use. A Finite Element Analysis for the truss-girder system is conducted to investigate stresses at truss connections and the wet joint. A 30-foot long truss specimen is constructed at the structural laboratory of UNL to investigate the constructability of the truss and the structural capacity of the diagonals, verticals, and connections. Testing results indicate the production and structural efficiency of the developed system.

To...

My parents, sister, and my future wife Marina

My friends

My professors and colleagues at Ain Shams University in Cairo, Egypt

My professors and colleagues at Peter Kiewit Institute

AUTHOR ACKNOWLEDGMENTS

The following advisors deserve credit for their guidance during the course of this research: Dr. George Morcous and Dr. Maher Tadros. Additionally, the members Precast/Prestressed Concrete Institute advisory committee are recognized and the financial support for the Daniel P. Jenny Fellowship is appreciated. Assistance was provided by Musa Alawneh (PhD Candidate) as a scholarship partner. Support was also given by University of Nebraska students and staff: Kelvin Lein, Afshin Hatami, Shaddi Assad, Mohamed El-Kady, and Micheal Asaad.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

Structural steel is typically and widely used to design roofs for warehouses and airplanes hangars. Examples of each system are provided in Figure 1.1 through Figure 1.4. Design considerations for roof support must take into account cost-effectiveness, speed of construction, structural capacity, aesthetic appearance, fire resistance, and structural integrity during construction and after completion. Designing using steel has been the only practice when it comes to long-spanned roofs. The ease of handling and erection, geometric flexibility, and lightweight of the structural steel components are all advantages of steel structures. Concrete has not been introduced into the field of roof supports for warehouses and airplanes hangars because of the difficulty of the construction and design of concrete components without being extremely heavyweight and expensive. e.Construct, USA, LLC has offered a state-of-art concrete-truss girder system that has been built and designed for a cement storage facility in Sharjah, UAE.

Figure 1.1 Interior of steel truss warehouse roof

Figure 1.2 Interior of cast-in-place parking structure

Figure 1.3 Steel trusses used for roof support

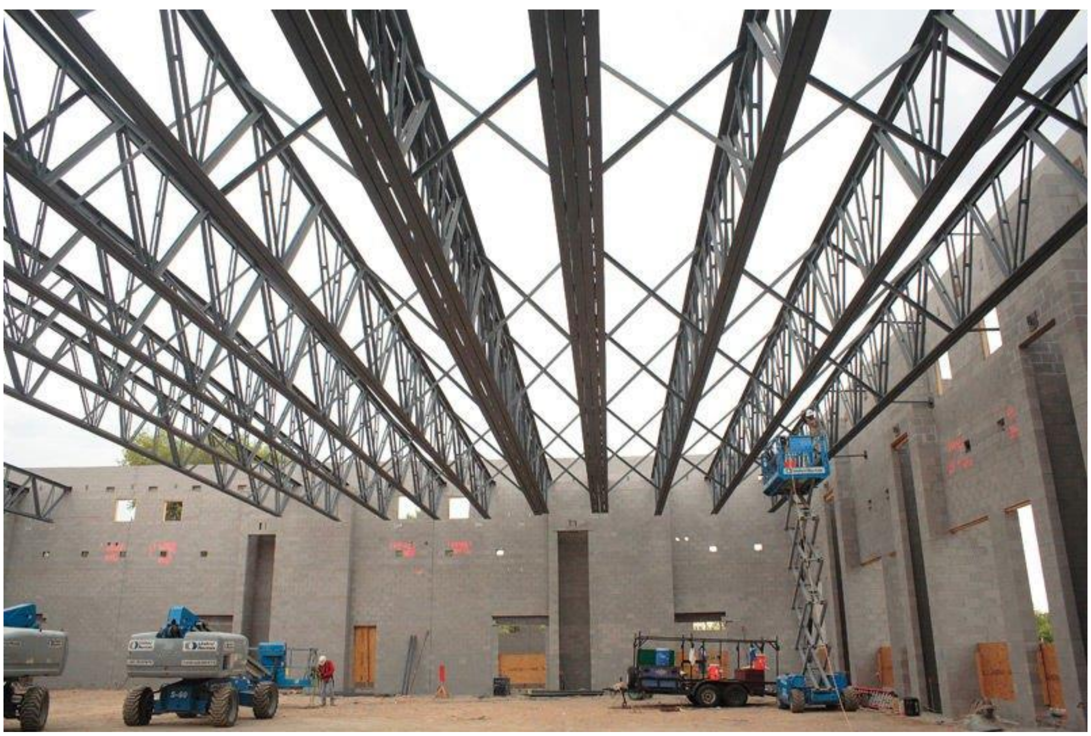

Figure 1.4 Interior of closely spaced steel trusses used for roof support

1.1 PROBLEM STATEMENT

Despite the aforementioned steel sections advantages, steel construction faces some challenges that tend to cause problems that are most often expensive to solve. These disadvantages include:

- Low fire-resistance
- Corrosion
- High maintenance cost
- Long periods of wait for steel production
- Increasing prices for steel (see Figure 1.5)

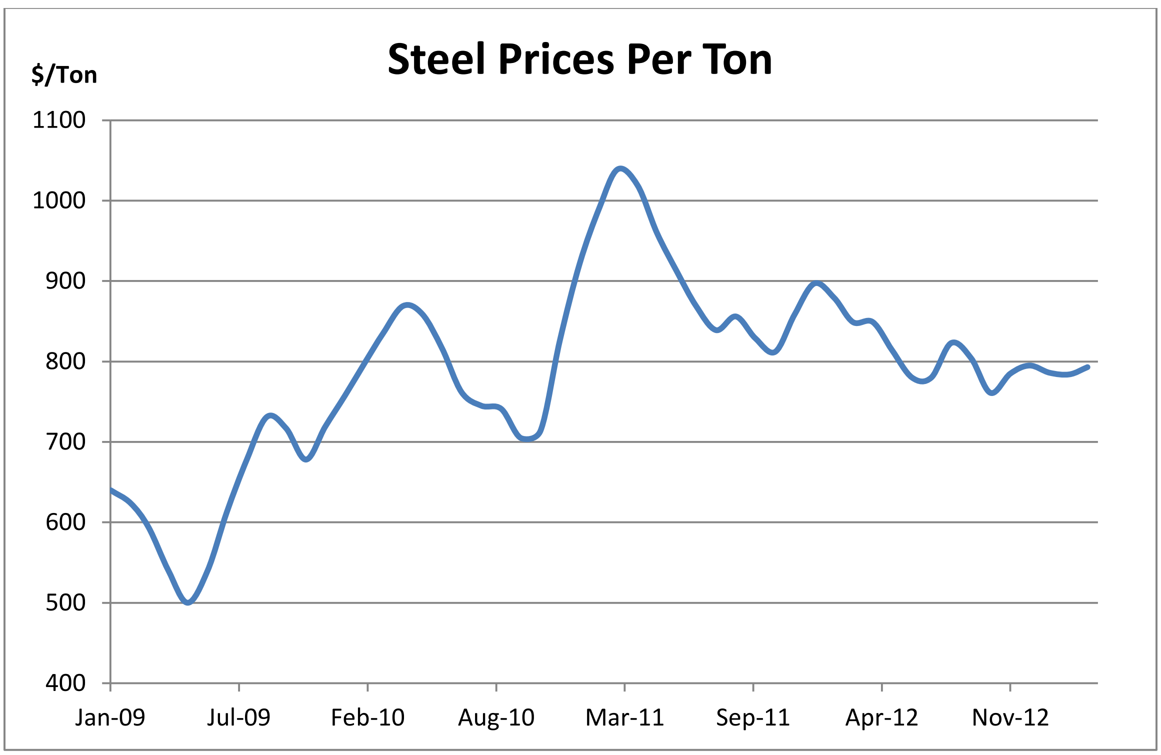

Figure 1.5 Chart showing increasing steel prices for the last two years (New York State Department of Transportation 2013)

All these disadvantages are addressed when comparing steel sections versus the concrete ones. While concrete can stand against steel in many structural designs, the construction of the concrete members has been the drawback that made concrete unable to stand up for the competition. Concrete has very low to no maintenance cost, high fire-resistance, and low material cost. However, the disadvantages of concrete that this research is directed to are:

- Heavy members weight for long spans
- Aesthetics

The fact that concrete does not work efficiently in resisting tensile stress makes it very difficult to design a truss made of solely reinforced concrete.

1.2 OBJECTIVES

The objective is to develop a precast/prestressed concrete truss for long span roofs. A set of objectives were developed in order to summarize exactly which corners need to be researched in order to develop the concrete truss. The spans targeted in this research range from 110 ft till up to 160 ft. Construction time and cost is one of the major research corners. It should be noted that these objectives are to be achieved by development of an optimized design that addresses the aforementioned disadvantages of concrete to be able to compete against steel. The main objective is developing a concrete truss system that is:

- Light in weight
- Fabricated using existing forms and production techniques. (economical to fabricate)
- Has capacity for snow loads at the Midwest
- Aesthetically pleasing

2.0 LITERATURE REVIEW

2.1 LOW SPANT-TO-DEPTH TRUSSES

The use of concrete trusses was first introduced in 1978 by W. Carroll, F. Beaufait, and R. Bryan in an ACI Journal article titled "Prestressed Concrete Trusses". Precast concrete trusses were first used in 1962 for the U.S. Science Pavilion (now Pavilion Science Center) in Seattle, WA; however these trusses were non-structural trusses (PCI 2004). In 1976, Rock Island Parking Structure was built using Vierendeel trusses (PCI 2004). Vierendeel trusses are trusses made out of rigid joints and no diagonals. The trusses used were almost 12 ft deep and had a clear span of 32 ft and 16 in. x 22 in. cross-section dimensions for the top, bottom, and vertical members (Shah and May 1977). The vertical members that are resisting tension forces were post-tensioned. The Pavilion Science Center arches and the Rock Island Parking façade are shown in Figure 2.1 and Figure 2.2, respectively.

Figure 2.1 Concrete trusses Pavilion Science Center in Seattle, WA

Figure 2.2 Vierendeel trusses used on the façade for the Rock Island Parking Structure

The ACI journal paper was published in 1978 by William Carroll about Prestressed Concrete Trusses (Carroll and al. 1978). The paper discussed two prototypes for the trusses. A prototype that had a clear span of 20 ft 4 in. and a depth of 2 ft, for a span to depth ratio of 10. The other prototype had 60 ft 10 in. and 8 ft 6 in. for span and depth, respectively, yielding a span to depth ratio of 7. The smaller trusses that were suggested in the paper consisted of diagonal members only without any vertical ones; however the bigger ones had only two verticals near the center of the trusses. Figure 2.3 and Figure 2.4 show the two prototypes of the trusses researched.

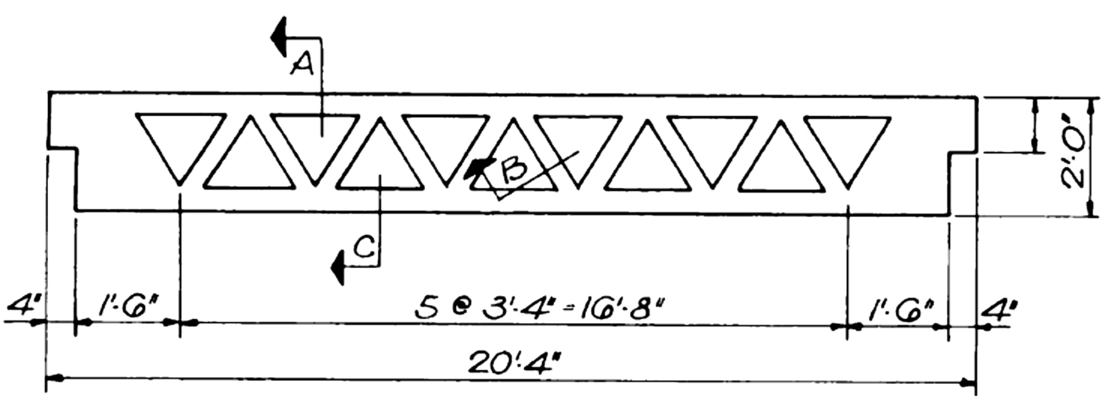

Figure 2.3 Concrete truss prototype-I (Carroll and al. 1978)

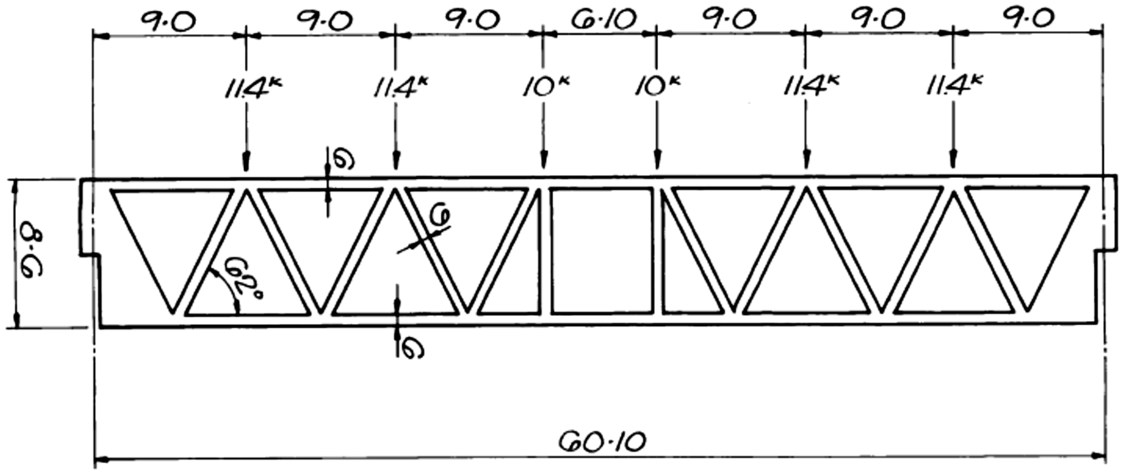

Figure 2.4 Concrete truss prototype-II (Carroll and al. 1978)

The paper did not discuss any details about the construction of the truss formwork as it only talked about the methods and locations for placing the strands and the hold-down devices. All the top, bottom, and diagonal members were prestressed. However, the prestressing in the diagonals was only 35% of the initial prestressing due to the stresses lost in friction at the hold down devices, see fig, (Carroll and al. 1978). The paper stated that the members cracked at an early stage of loading, which, according to the authors, was due to "the fact that the diagonals were not fully prestressed" (Carroll and al. 1978). The research stated that using concrete trusses would lower the price to almost half what it would cost if steel was to be used for the system.

In 2007 a new idea for concrete trusses evolved. A multi-level condominium building was built in Minneapolis, MN using what is called "ER-Post". The ER-Post is system invented by M. DeSutter of Erickson Roed & Associates. The purpose of the system was to provide a column-free space for the condominiums, Figure 2.5, (Trygestad and DeSutter 2007). DeSutter was able to merge Vierendeel trusses with pretensioning to design the precast/prestressed trusses used (DeSutter 2007). With a depth of 13.5 ft, the trusses had a span of 67.33 ft, see Figure 2.6 for details. The span to depth ratio was 5.

Figure 2.5 Erection of ER-POST trusses (Trygestad and DeSutter 2007)

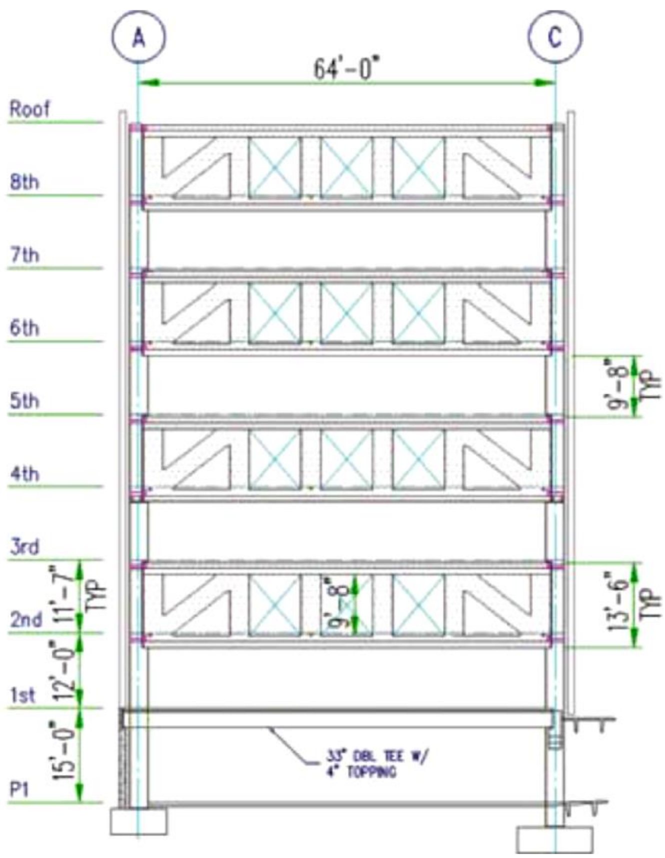

Figure 2.6 Elevation drawing of the ER-POST system (Trygestad and DeSutter 2007)

2.2 HIGH SPAN-TO-DEPTH TRUSSES

Even with the progress concrete designers has achieved, none were able to provide a system that can compete and eventually replace the steel trusses that are used for long spans as in warehouses, airplane hangars, and such. In 2010, a precast concrete truss-girder was used to support the roof of a coal storage facility in Sharjah, United Arab Emirates (UAE). Designed by e.Construct USA, LLC, the 1.5-meter (5-foot) deep truss had a span of 50 m (165 ft) without intermediate supports. Figure 2.7 shows the truss as fabricated while in the precast plant.

Figure 2.7 The truss designed by e.Construct USA, LLC (the view is center to end)

A full truss consists of two halves, each 25 m (82.5 ft) long. The two pieces are post-tensioned together forming one full 160 ft span. The trusses are arranged to be 10 m (30 ft) apart. Figure 2.8 shows a 3D view of the project and trusses arrangement. Each two halves of the truss rested on a temporary support (as shown in Figure 2.9) at the center of the span till post-tensioning was applied. Figure 2.10 is the elevation and cross-sectional dimensions of the top chord, bottom chord, verticals, diagonals, and end blocks as designed by e.Construct USA, LLC. All pictures are courtesy of e.Construct USA, LLC.

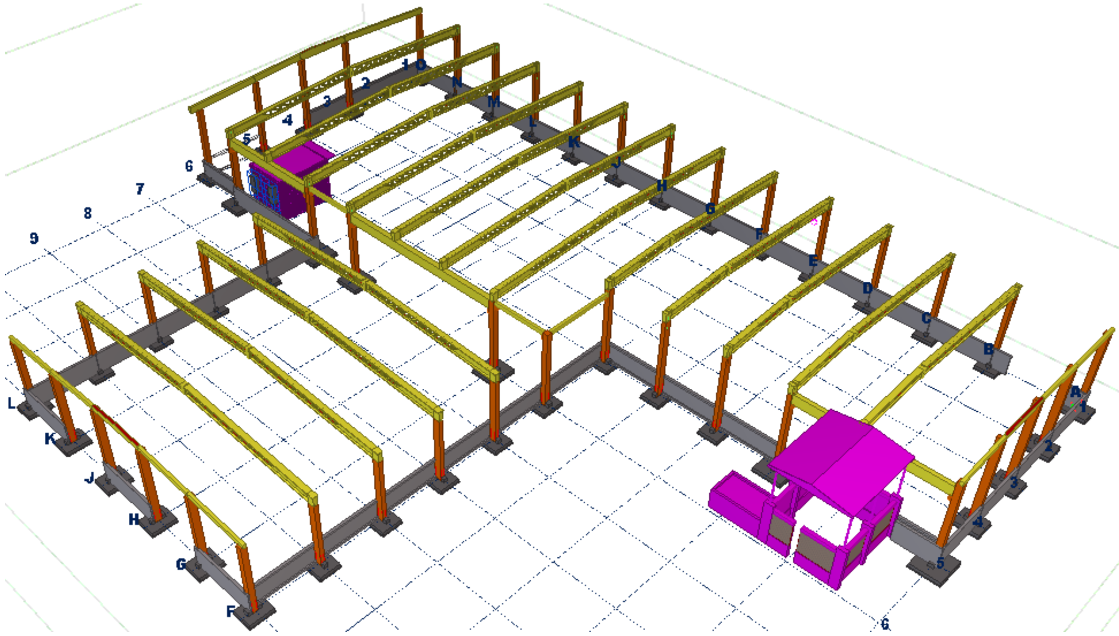

Figure 2.8 A 3D view of the trusses arrangement for the coal storage facility

Figure 2.9 The trusses resting on the temporary supports before post-tensioning

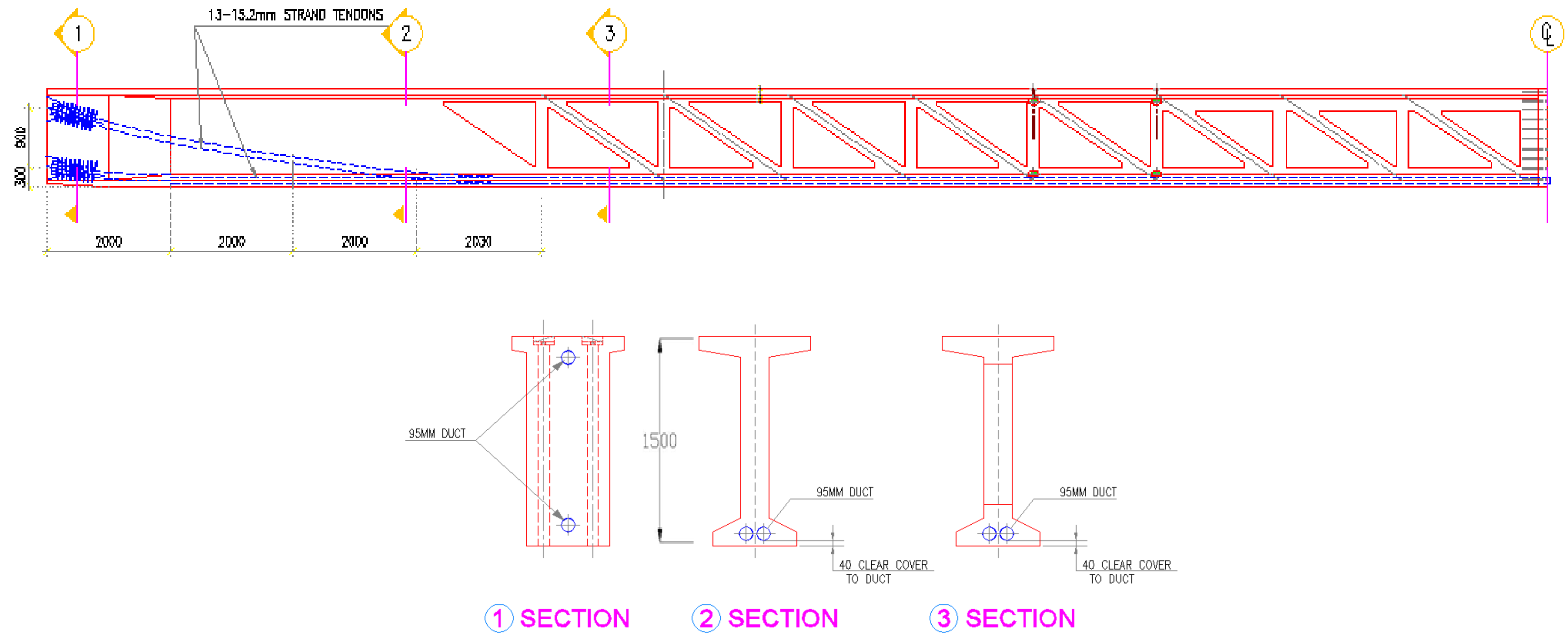

Figure 2.10 Elevation and cross-sectional dimensions of the top chord, bottom chord, verticals, diagonals, and end blocks

As claimed by e.Construct USA, LLC, the use of this system, along with Z-shape steel purlins and metal decking resulted in about 25% saving in the building cost. Despite the success of this project and the significant savings achieved by using concrete over the steel frame alternative, several enhancements could be researched to optimize the cost effectiveness of the truss-girder system for long span roof applications. This includes optimization of precast plant production, minimization of self-weight and simplification the construction sequence. This was the starting step towards the research that has been done.

3.0 SYSTEM DEVELOPMENT

Table 3-1 Comparing the existing truss-girder system in Sharjah to the proposed truss system

POC	Sharjah Truss-Girder	Proposed System
Total Weight w/o Openings	112,400 lb.	151,800 lb.
Total Truss Weight	87,800	111,200 lb.
% Weight Saved by Openings	22%	27%
Span	165 ft	160 ft
Depth	5 ft	6 ft
Span/Depth ratio	33	26.7
Block-out Panels	Varies	8 ft
Solid Continuous Web	13.2% of total span length on each side	11.6% of total span length on each side
End Block	Rectangular Block	Same cross-section with thicker bottom flange
Post-tensioning Profile	Starts in the web then goes down to the bottom flange	Straight
Vertical Members	Reinforced concrete (Compression)	Steel Threaded Rods (Tension)
Diagonal Members	Reinforced concrete (Tension)	Reinforced concrete (Compression)
Girder Cross-section	Special made form	Commonly used bridge girder

The trusses designed by e.Construct had diagonals and verticals in tension and compression, respectively. Span to depth ratio was 33, which means the truss was able to resist great forces in the top and bottom flange with very little moment arm (depth), compared to span. The original truss depth was shallow enough for the application and the optimized one was required to be kept at a close value. For ease of fabrication, the truss was suggested to be made out of a girder form that already exists while using block-outs within the form to make the opening between the diagonals and verticals.

The truss-girder developed had a different structural system than the aforementioned one, despite the similarities. The differences and similarities are further discussed in Table 3-1. The chapter later on discusses the reasons behind the system selection. The analysis, design details, and construction sequence are all part of the system details. Despite the iterations the system had been through, only the main highlights are discussed.

3.1 BUILDING LAYOUT

A building layout similar to the aforementioned coal storage facility was suggested. The span for the trusses is set at 160 ft with a 5% slope (crowning). The building was proposed to be 300 ft longitudinally with 10 spaces between truss-girder lines, resulting in 30 ft spacing between every other truss-girder. Figure 3.2 is a plan view of the building layout representation. Figure 3.3 is an elevation view of the truss resting on columns.

Proposed detail for the truss-column connection is shown in Figure 3.4. The drawing shows 2-1” diameter headed high strength steel bars going through the bottom flanges and inserted in metal sleeves that are embedded in the columns. The sleeves are then filled with grout, grease, or both. A bearing pad is also shown in the drawing between the truss and the column.

The two truss pieces are connected together by a wet joint then post tensioning. The construction sequence, along with the post tensioning duct profile, is further explained in section 3.3. Each truss piece will have reinforcing bars that extend outside the concrete towards the connection with the other piece. The bars are then bent and concrete is casted in place to form the joint. The two trusses are then post-tensioned together for a total span length of 160 ft. The wet joint detail is shown in Figure 3.1.

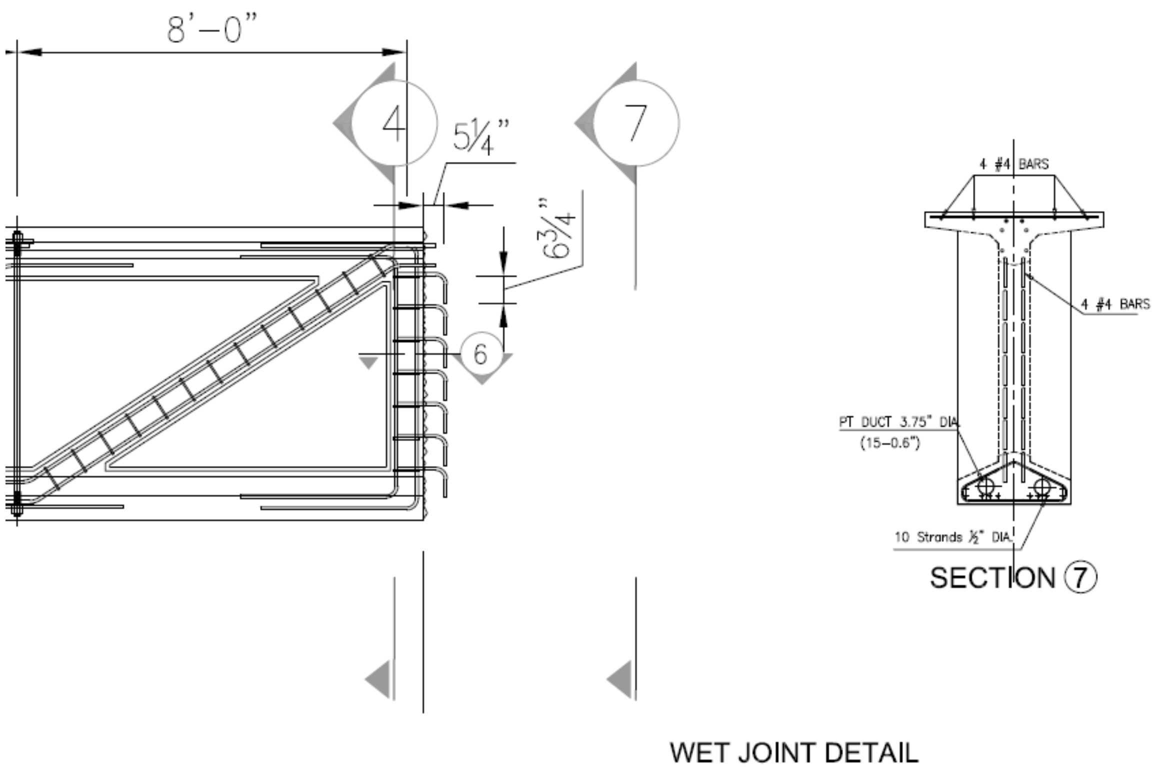

Figure 3.1 Proposed wet joint details for the truss

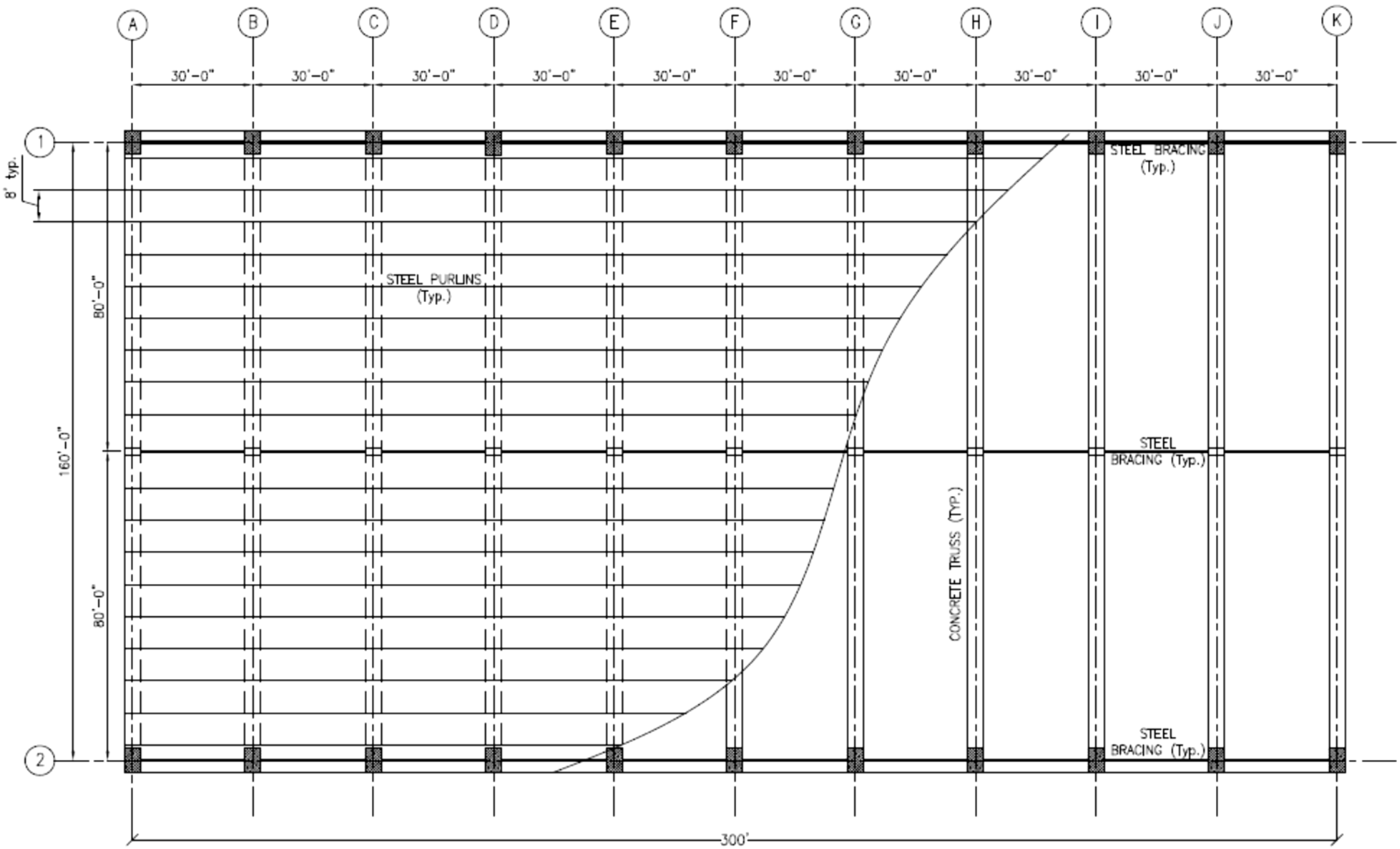

Figure 3.2 Plan view of the proposed building layout

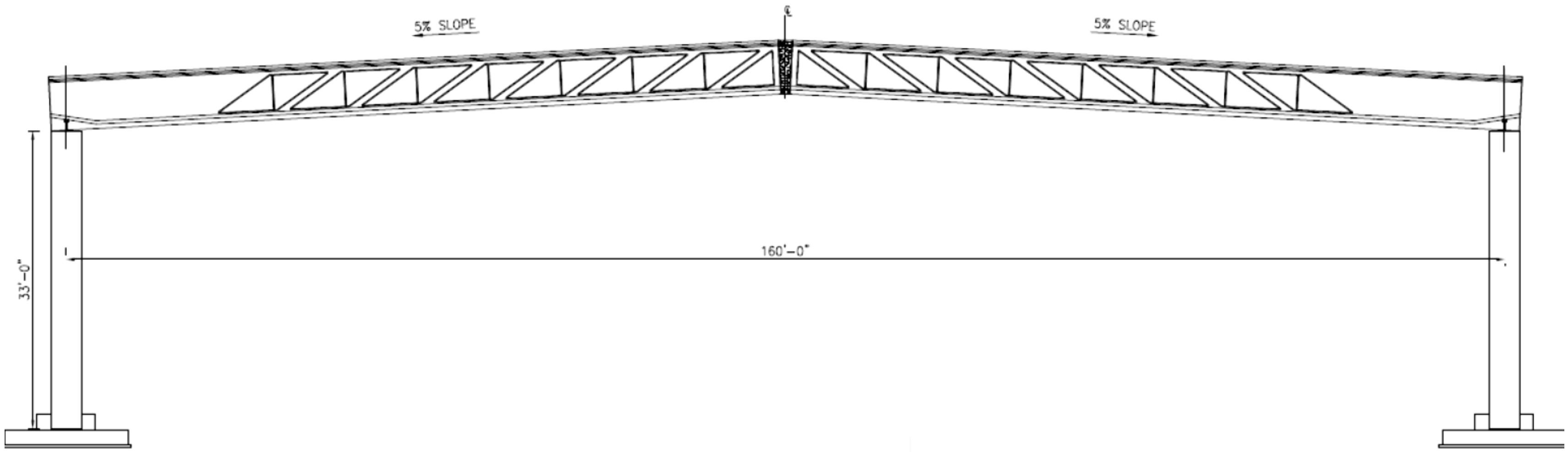

Figure 3.3 Elevation view of the proposed building layout

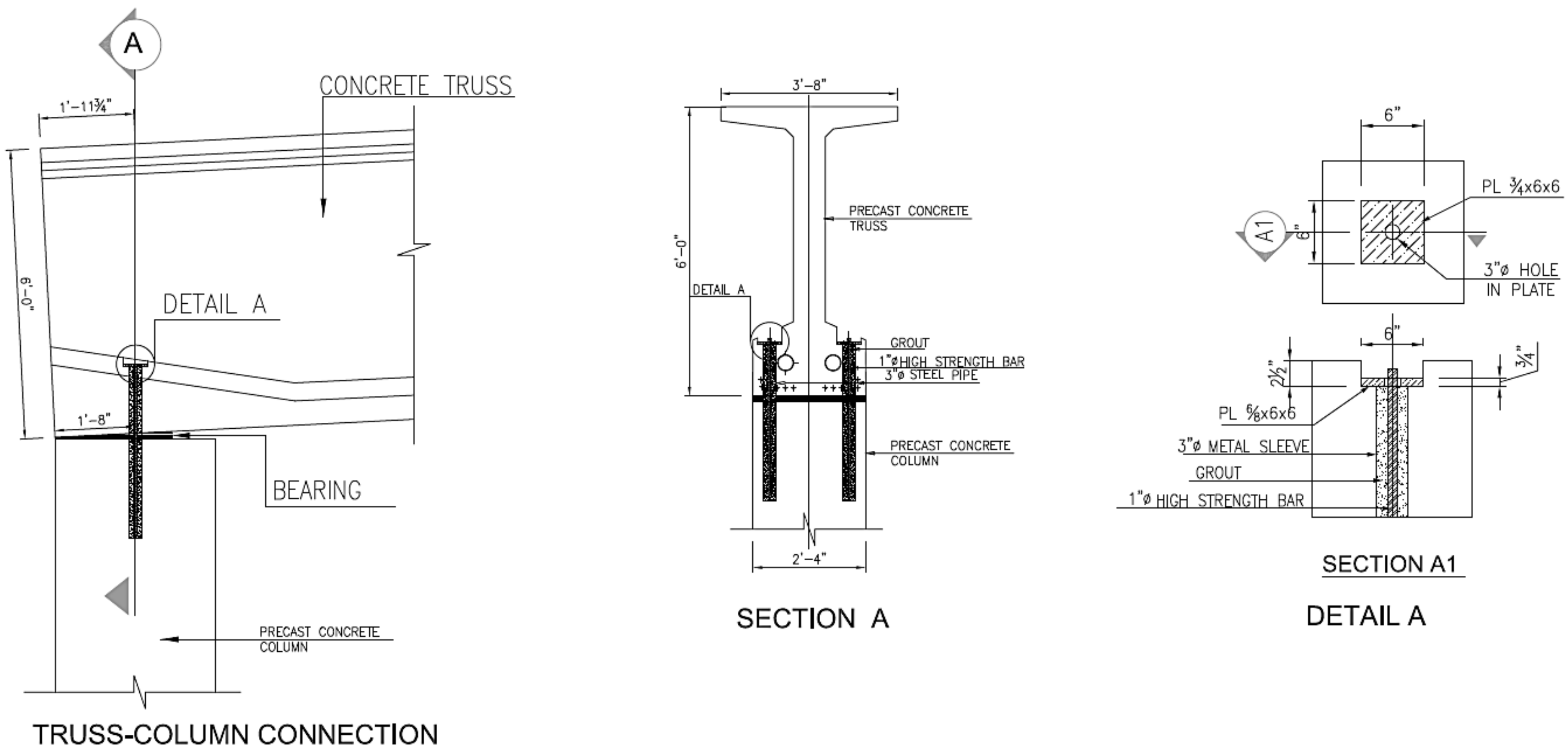

Figure 3.4 Truss-column connection

3.2 TRUSS DESIGN

The optimization of the truss was the main goal of the research. The objective of the following work was to provide the most efficient way to analyze, design, and build the truss.

3.2.1 Loads

The use of the truss is to support the roof for warehouses or hangars, in other words, there is no expected floor above the trusses. To adequately design for the loads expected in accordance to the code, the Minimum Design Loads for Buildings and Other Structures, or as known as ASCE 7-10, manual was used (ASCE/SEI 7 2010). On one hand, vertical loads that are expected to be applied on the roof were dead load (D), [roof] live load (L_r), and snow load (S). On the other hand, horizontal loads, such as wind and earthquakes, are considered to be resisted by shear walls or column bracings and were out of the scope of the study. As a result, it has been determined by the researchers that only vertical loads are to be resisted by the truss, that is dead, live, and snow loads.

According to ASCE 7-10, a live load (L_o) of 20 psf is specified for Ordinary flat, pitched, and curved roofs as required by Table 4-1. However, in accordance with Section 4.8.1, this load can be reduced according to the equation:

$$L_r = L_o R_1 R_2 \qquad \text{(4.8-1) (ASCE/SEI 7 2010)}$$

Where R_1= 0.6 (for tributary area greater than 600 sq. ft), and R_2=1.0 (for a slope of 5% only). This results in a reduced roof live load of 12 psf.

For snow loads, and since the roof slope is only 5%, flat roof snow load, p_f, was calculated according to the equation:

$$p_f = 0.7 C_e C_t I_s p_g \qquad \text{(7.3-1) (ASCE/SEI 7 2010)}$$

Where C_e= 0.9 (Table 7-2, exposure category B assumed), C_t= 1 (Table 7-3), I_s= 0.8 (Table 1.5-2, importance factor I). p_g, ground snow load, was determined through the advice by the Daniel P. Jenny Fellowship committee and advisors to be 60 psf, so as to include regions within the states that get substantial amounts of snow, resulting in design load of 30 psf.

Dead load is not specified by ASCE 7-10, it has been, however, recommended by the advising committee to assume a load of 15 psf for mechanical, electrical, and pluming (MEP) loads. Since the load combinations in Section 2.3.2 specifies either the roof live load or snow load to be used with other load cases, the snow load of 30 psf will be used for design in load combinations instead of the roof live load of 12 psf. Load combinations 1 (1.4D) and 3 (1.2D + 1.6S) will be compared after the weight of the truss-girder is suggested according to the preliminary design.

Roof loads are transferred to the trusses as point loads from light steel gage purlins resting on top of the truss at the location of the vertical members. As a preliminary design, and using the coal storage facility as a baseline, the vertical members were suggested to be 8 ft apart. As mentioned earlier, the spacing between the trusses was 30 ft, resulting in a tributary loading area for each point load of 80 sq. ft. Hence, the design was based on a dead load (D) of 3600 lb (3.6 kips) and a snow load of 7200 lb (7.2 kips).

3.2.2 Analysis

As a preliminary design, and to make the truss easy to fabricate, the truss was suggested to be constructed as a bridge girder with block-outs within the web to form the diagonal and vertical members. NU-1350 girders were chosen for the preliminary design. NU-1350 girders are 53.1in. (1350 mm) deep, that resulted in a span to depth ratio of 36. Figure 3.5 below is the overall dimensions for NU girders. Table 3-2 is the dimensions for various types of the NU girders.

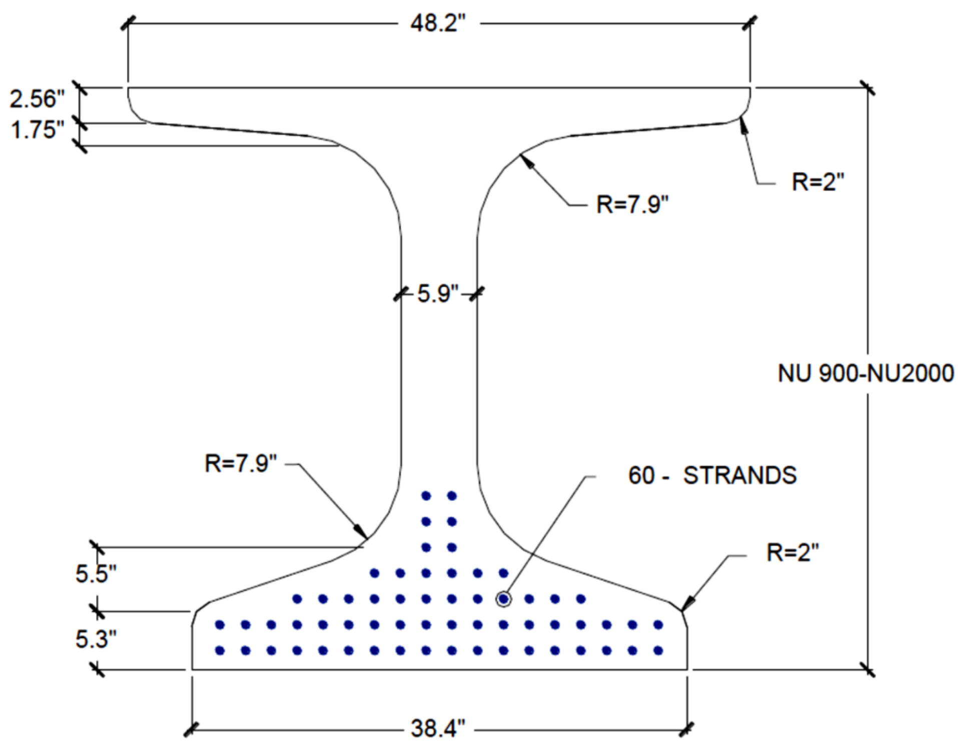

Figure 3.5 Prestressed Nebraska University (NU) girder dimensions

Table 3-2 Dimension details for various types of NU girders

Section	Height in (mm)	Web Width in (mm)	Top Flange Width in (mm)	Bottom Flange Width in (mm)	A in^2 (mm^2)	Y_b in (mm)	I in^2 ($mm^4 * 10^6$)	W_t Kips/ft KN/m
NU 900	35.4 (900)	5.9 (150)	48.2 (1225)	38.4 (975)	648.1 (418,111)	16.1 (410)	110,262 (45,895)	0.680 (9.85)
NU 1100	43.3 (1100)	5.9 (150)	48.2 (1225)	38.4 (975)	694.6 (448,111)	19.6 (497)	182,279 (75,870)	0.724 (10.56)
NU 1350	53.1 (1350)	5.9 (150)	48.2 (1225)	38.4 (975)	752.7 (485,610)	24.0 (608)	302,334 (126,841)	0.785 (11.44)
NU 1600	63.0 (1600)	5.9 (150)	48.2 (1225)	38.4 (975)	810.8 (523,111)	28.4 (722)	458,482 (190,835)	0.840 (12.33)
NU 1800	70.9 (1800)	5.9 (150)	48.2 (1225)	38.4 (975)	857.3 (553,111)	32.0 (814)	611,328 (254,454)	0.894 (13.03)
NU 2000	78.7 (2000)	5.9 (150)	48.2 (1225)	38.4 (975)	903.8 (583,111)	35.7 (906)	790,592 (329,069)	0.942 (13.74)

Structural analysis was performed using SAP2000 and Midas Civil structural analysis programs. The truss frames are shown in Figure 3.6 as it was modeled in SAP2000. The connections were all modeled as fixed. The web was widened to 8 in instead of 5.9 in so as to fit the steel reinforcements inside the verticals and diagonals. At the ends of the truss, the web remained continuous in order to adequately resist shear forces. The continuous web was 23% of the whole length of the truss. In other words, the first vertical location was 24 ft 10 in. from each end.

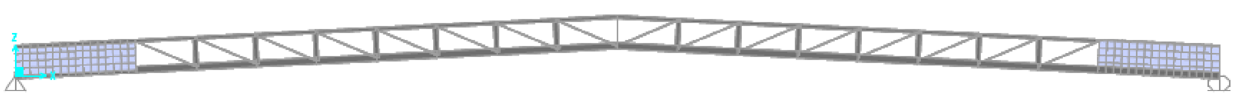

Figure 3.6 Cocnrete truss-girder analysis model

The loads, however, were extremely high compared to the section sizes. As a result, the tension members were chosen to be threaded rods embedded in the concrete. The axial compression force in the top flange reached almost 1800 kips, which the top flange was very thin to handle. To better handle the moment at the midspan, the truss depth needed to be increased deeper than 5 ft. The orientation of the diagonal members was also changed so they would be in compression instead of tension and the vertical members became tension resisting members. This change in the design facilitated the anchorage of the threaded rod inside the concrete truss. To decrease the dead load on the truss due to self-weight, it was found that the bottom flange did not need to have the full width on the NU girder. The bottom flange width was decreased by 9 in. from each end, resulting in a width of 22.5 in.

With all the modifications done to the cross-section, the stresses in the top flange were still very high. Increasing the thickness of the top flange was an option; however, all these modifications were only translated to construction cost. To avoid all these costs and in the same time find an optimized section, the PCI bulb tee girders were chosen as the bridge girder that is widely spread in most states and still matches the dimension of the cross-section needed. To satisfy the depth needs, PCI BT-72 was the girder to be used in design. The depth of the girder is 72 in. (1.83 m). The dimensions of the regular BT-72 is in Figure 3.8; however, the modified one for the truss-girder is 2 inches wider.

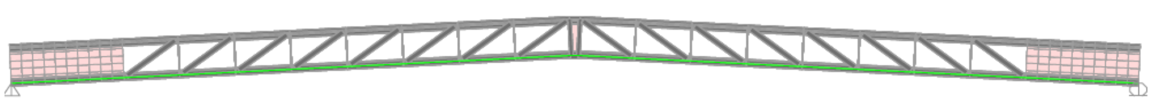

Figure 3.7 SAP2000 model for final truss-girder general design

Final analysis was done on the model shown in Figure 3.7. Service loads were self-weight, superimposed dead loads, and snow load, totaling 10.8 kips every 8 ft in addition to the self-weight, see Figure 3.9 for dead loads and Figure 3.10 for live loads modeling. The ultimate loading was determined to be 1.2D + 1.6S (ASCE/SEI 7 2010). Threaded

24

rods (vertical members) were modeled as axial load bearing members with moment releases at the ends. Prestressing consists of 10-0.6" 270 ksi low relaxation strands pretensioned at 0.75 the ultimate stress of the strands. Post-tensioning was designed preliminarily as 2 ducts each with 14-0.6" strand at jacking stress of 0.75 ultimate stress.

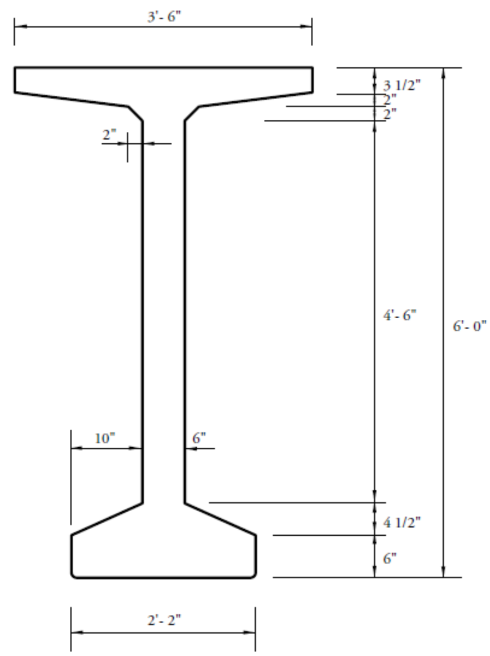

Figure 3.8 AASHTO-PCI BT-72 girder cross-section (PCI 2003)

Maximum axial loads on the top flange, bottom flange, vertical, and diagonal members were 1683 kips (compression), 1698 kips (tension), 136 kips (tension), and 268 kips (compression), respectively. The loads while handling was also analyzed. It was found that the loads induced in the top and bottom flanges will not lead to having special considerations will designing. However, tensile forces, with a maximum of 61 kips, acted on diagonal members and compression forces, with a maximum of 29 kips, was resisted by vertical members. Figure 3.11 is the member labeling for analysis results reporting.

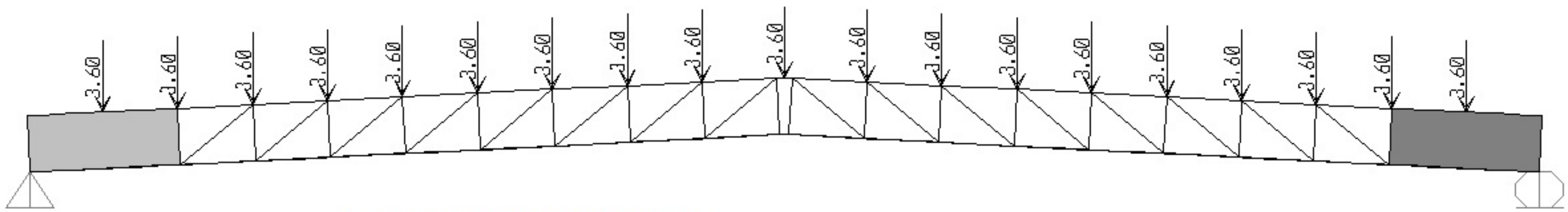

Figure 3.9 Super-imposed dead loads on the truss-girder as modeled (kips)

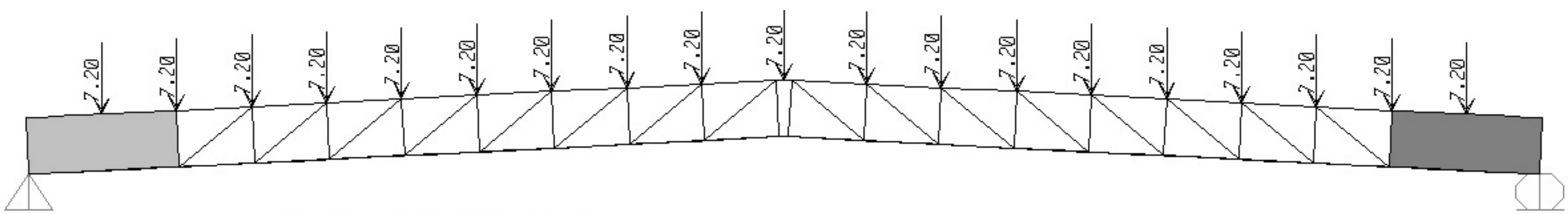

Figure 3.10 Snow loads on the truss-girder as modeled (kips)

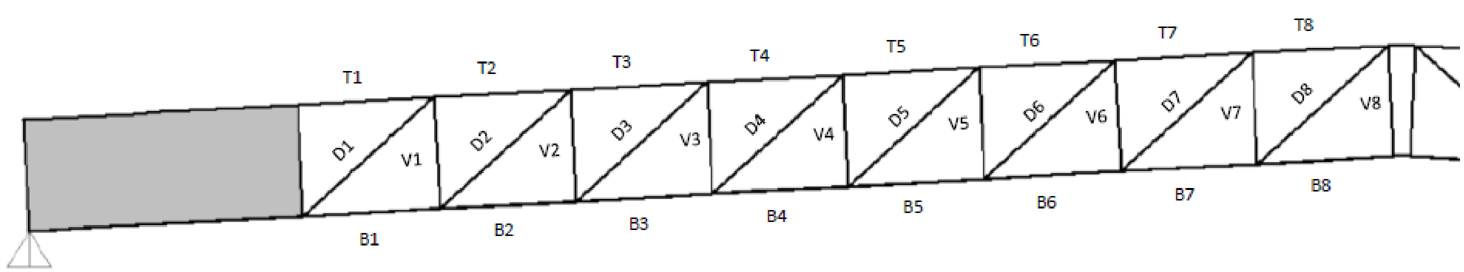

Figure 3.11 Members labeling for analysis reporting

Moment diagrams for construction and ultimate loads phases are shown in Figure 3.12 and Figure 3.13, respectively. Results of frame analysis are reported in Table 3-3 and Table 3-4 are for ultimate and construction loads, respectively. Forces due to service loads are reported in Table 3-5.

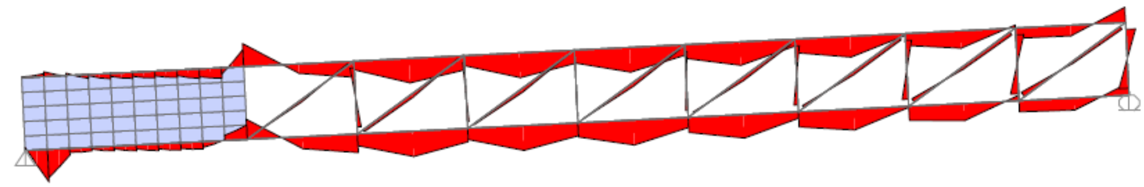

Figure 3.12 Moment diagrams due to construction loads

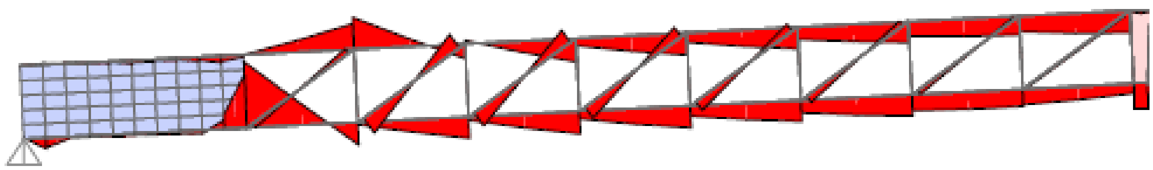

Figure 3.13 Moment Diagrams due to ultimate loads

Table 3-3 Analysis results for ultimate loads

Ultimate Loads			
Table: Element Forces - Frames			
Frame	P (Kips)	V (Kips)	M (Kip-ft)
B1	866.8	-9.3	44.3
B2	1088.9	-0.7	38.4
B3	1273.7	-1.6	43.8
B4	1425.8	-1.0	46.5
B5	1544.3	-0.6	48.2
B6	1629.2	-0.4	50.4
B7	1681.7	1.1	45.5
B8	1697.6	-1.5	58.2
D1	-267.9	0.7	1.8
D2	-258.6	1.5	8.2
D3	-215.6	1.2	7.1
D4	-177.1	1.0	7.0
D5	-137.9	0.9	6.6
D6	-98.6	0.7	6.2
D7	-60.7	0.5	5.4
D8	-17.5	0.5	5.8
T1	-650.9	-8.4	36.2
T2	-879.1	-0.6	36.4
T3	-1099.0	-0.9	40.7
T4	-1282.1	-0.2	42.8
T5	-1432.4	0.2	44.5
T6	-1549.1	0.8	45.2
T7	-1632.2	1.0	47.4
T8	-1683.1	2.0	47.5
V1	135.9	-5.0	10.7
V2	122.0	-3.9	8.5
V3	99.6	-3.3	7.2
V4	78.3	-2.7	5.8
V5	57.0	-2.0	4.4
V6	35.2	-1.3	3.0
V7	15.4	-0.8	1.7
V8	83.9	-0.1	0.3

Table 3-4 Analysis result for loads during construction

Construction Stage			
Table: Element Forces - Frames			
Frame	P (Kips)	V (Kips)	M (Kip-ft)
B1	127.3	0.4	4.5
B2	143.7	1.5	6.1
B3	148.3	1.5	6.0
B4	141.8	1.7	6.0
B5	124.0	1.8	5.7
B6	94.9	2.0	5.1
B7	54.5	2.1	4.1
B8	3.0	2.3	3.0
D1	-28.1	0.4	0.5
D2	-18.5	0.4	0.8
D3	-4.9	0.3	0.8
D4	8.2	0.3	0.8
D5	21.4	0.2	0.8
D6	34.6	0.2	0.7
D7	47.9	0.1	0.8
D8	60.7	0.1	0.8
T1	-104.3	0.4	3.9
T2	-128.3	1.4	5.9
T3	-144.1	1.5	6.0
T4	-148.1	1.6	6.0
T5	-141.0	1.8	5.7
T6	-122.5	1.9	5.0
T7	-92.9	2.0	4.2
T8	-51.8	2.3	2.6
V1	13.0	-0.5	1.0
V2	6.7	-0.2	0.4
V3	-0.7	0.0	0.1
V4	-7.9	0.3	0.5
V5	-15.1	0.5	1.0
V6	-22.3	0.7	1.5
V7	-29.3	0.9	2.0
V8	-35.9	1.1	2.4

Table 3-5 Analysis results for service loads

Service Loads			
Table: Element Forces - Frames			
Frame	P (Kips)	V (Kips)	M (Kip-ft)
B1	642.1	-6.9	32.8
B2	806.6	-0.5	28.5
B3	943.5	-1.2	32.5
B4	1056.1	-0.7	34.4
B5	1143.9	-0.4	35.7
B6	1206.8	-0.3	37.3
B7	1245.7	0.8	33.7
B8	1257.5	-1.1	43.1
D1	-198.4	0.5	1.4
D2	-191.5	1.1	6.1
D3	-159.7	0.9	5.3
D4	-131.2	0.8	5.2
D5	-102.1	0.6	4.9
D6	-73.0	0.5	4.6
D7	-45.0	0.3	4.0
D8	-13.0	0.4	4.3
T1	-482.1	-6.2	26.8
T2	-651.2	-0.4	27.0
T3	-814.1	-0.6	30.2
T4	-949.7	-0.1	31.7
T5	-1061.0	0.2	33.0
T6	-1147.5	0.6	33.5
T7	-1209.1	0.8	35.1
T8	-1246.8	1.5	35.2
V1	100.7	0.0	0.0
V2	90.4	0.0	0.0
V3	73.8	0.0	0.0
V4	58.0	0.0	0.0
V5	42.2	0.0	0.0
V6	26.1	0.0	0.0
V7	11.4	0.0	0.0
V8	29.6	-0.1	0.2

Maximum deflection due to super imposed service load was found to be 6.5 in. at midspan (equaling L/304 deflection), which satisfies l/240 requirement (ACI 318 2011).

Table 3-6 shows deflections due to different loading conditions and the net deflections at these stages.

Table 3-6 Deflections at different stages of loading

Type of load	Deflection
Post-tensioning	- 7.8 in
Dead Load	4.8 in
Erection Deflection (using PCI deflection equations)	- 5.2 in
SID	3.2 in
Net Deflection (using PCI deflection equations)	3.45 in
SIL	6.3 in

+ve for downward and -ve for upward deflections

3.2.3 Detailing

The truss was design according to the strut and tie method explained in Appendix A in the ACI 318-11 code. The diagonal members are designed as reinforced concrete sections, while the vertical ones are designed as steel threaded rods with no concrete casing. The threaded rod are made out of B7 105 ksi steel, F_y= 105 ksi and F_u=125 ksi. The design of the threaded rods was based on limit state requirement $\varphi F_y \geq F_u$. Considering handling loads, compression in threaded rods was designed according to the AISC Steel Construction Manual for compression members (AISC 2012).

For the tensile strength of the threaded rods, a strength reduction factor, φ, of 0.9 was used (AISC 2012). The minimum area required was 1.44 in^2 for F_y= 105 ksi. Threaded rod diameter is reported according to the full diameter of the rod including the threads. As a result, a 1½ in. diameter rod has only 1.375 in. pitch diameter for an area of 1.49 in^2, see Table 3-7 for different sizes specifications. Despite the low tensile force carried by the vertical members near the midspan, the rods were checked for compression during handling and a minimum size of 1½ in diameter rods was needed to avoid buckling according to Chapter E (AISC 2012). The anchoring of the rods to the girder flanges is explained later in the laboratory specimen construction section.

The truss system was designed to be constructed with a compressive strength of 8000 psi. The diagonal members were design as the struts in Appendix A (ACI 318 2011). A strength reduction factor, φ, of 0.75 was used, as $\varphi F_{ns} \geq F_u$, where F_{ns} is the strength of the area of the concrete and the steel reinforcements added together i.e.:

$$F_{ns} = f_{ce}A_{cs} + A_s' f_s' \qquad \text{(A-5) (ACI 318 2011)}$$

The diagonal members were designed as 8 in. x 8 in. members, to account for concrete flowability during construction. According to equation (A-5), the area of concrete was sufficient enough to resist the compression forces; however, Section 10.9.1 specifies a minimum area of steel reinforcements in compression members of 0.01 the area of concrete. The minimum area of steel rebar required for the diagonal members were found accordingly to be 0.64 in^2, i.e. 4-#4 60,000-psi steel bars.

Table 3-7 Threaded rod sizes and specifications (TSA Manufacturing 2012)

ROUND BAR STOCK WEIGHT

PITCH DIAMETER FOR ROLLED THREADS			FULL DIAMETER FOR CUT THREAD		
UNC THREAD DIAMETER	PITCH DIAMETER	WEIGHT PER FOOT	UNC THREAD DIAMETER	FULL DIAMETER	WEIGHT PER FOOT
¼ - 20	0.215	0.124	¼ - 20	0.250	0.167
5/16 - 18	0.274	0.201	5/16 - 18	0.313	0.262
⅜ - 16	0.331	0.293	⅜ - 16	0.375	0.376
7/16 - 14	0.387	0.400	7/16 - 14	0.438	0.513
½ - 13	0.445	0.529	½ - 13	0.500	0.668
⅝ - 11	0.560	0.838	⅝ - 11	0.625	1.044
¾ - 10	0.680	1.236	¾ - 10	0.750	1.504
⅞ - 9	0.795	1.689	⅞ - 9	0.875	2.046
1 - 8	0.910	2.213	1 - 8	1.000	2.673
1-⅛ - 7	1.025	2.808	1-⅛ - 7	1.125	3.383
1-¼ - 7	1.145	3.504	1-¼ - 7	1.250	4.176
1-⅜ - 6	1.264	4.271	1-⅜ - 6	1.375	5.054
1-½ - 6	1.375	5.054	1-½ - 6	1.500	6.014

Even though calculations showed that only 4-#4 bars are required, it has been found through analysis that construction loads will result in tension forces in the concrete diagonal members. To design for crack control while handling, only 30,000 psi strength was considered for the steel rebar to resist the tensile forces (PCI 2010). The area of steel that was found to be needed for crack control during handling was 1.5 in^2, i.e. 4-#6 steel bars. #3 ties where used at 8 in. spacing in accordance to 7.10.5 (ACI 318 2011).

The truss was designed as strut and tie as well for the bottom flange (as ties) and the top flange (as strut), with a strength reduction factor of 0.9 (ACI 318 2011). The cross-section of the truss that the design was based on is shown in Figure 3.14. The beam was designed to resist the forces on the deferent components.

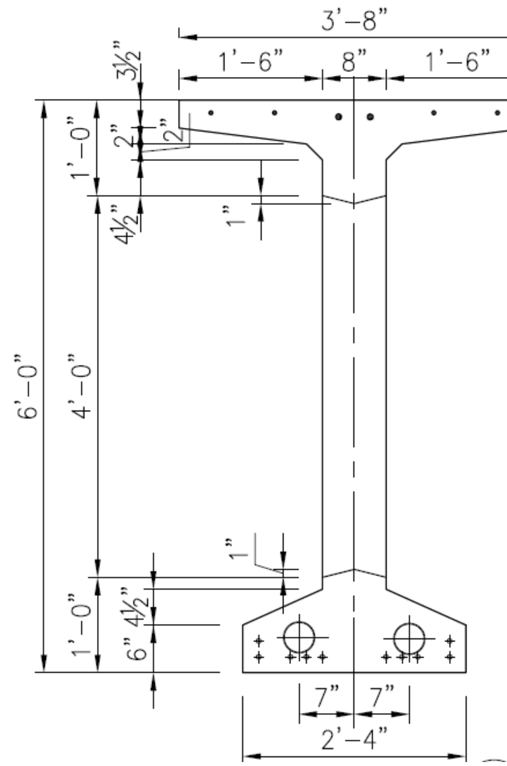

Figure 3.14 Corss-section of the BT-72 girder showing top and bottom chords for the truss-girder system

The compression block (top flange) had a gross area of 270 in^2. The factored strength of the top flange was calculated to be 1690 kips, including 4-#4 60,000-psi steel bars. For prestressing, two post-tensioning ducts were used each with 12-0.6" grade 270 low relaxation strands. 10-0.6" grade 270 low relaxation strands are designed for prestressing to protect the truss against cracking during the construction phase. The maximum ultimate tension the bottom flange was designed for was 1800 kips.

Cracking load was calculated to be 1830 kips, where cracking stress for concrete is $7.5\sqrt{0.85f_c'}$ and initial prestressing of 0.75 the ultimate strands stress (PCI 2010). Total prestressing losses were assumed to be 20%.

3.2.4 Calculations

The following figures and tables are details and design tables and calculations for the full size truss-girder system.

Table 3-8 Member loads comparison

Table: Element Forces - Frames												
Frame	Ultimate			Construction			Controling					
	P (Kips)	V (Kips)	M (Kip-ft)	P (Kips)	V (Kips)	M (Kip-ft)	Pc (Kips)	Pt (Kips)	V (Kips)	M (Kip-ft)		
B1	866.8	-9.3	44.3	127.3	0.4	4.5	NA	866.8	-9.3	44.3		
B2	1088.9	-0.7	38.4	143.7	1.5	6.1	NA	1088.9	1.5	38.4		
B3	1273.7	-1.6	43.8	148.3	1.5	6.0	NA	1273.7	1.5	43.8		
B4	1425.8	-1.0	46.5	141.8	1.7	6.0	NA	1425.8	1.7	46.5		
B5	1544.3	-0.6	48.2	124.0	1.8	5.7	NA	1544.3	1.8	48.2		
B6	1629.2	-0.4	50.4	94.9	2.0	5.1	NA	1629.2	2.0	50.4		
B7	1681.7	1.1	45.5	54.5	2.1	4.1	NA	1681.7	2.1	45.5		
B8	1697.6	-1.5	58.2	3.0	2.3	3.0	NA	1697.6	2.3	58.2		
D1	-267.9	0.7	1.8	-28.1	0.4	0.5	-267.9	NA	0.4	1.8		
D2	-258.6	1.5	8.2	-18.5	0.4	0.8	-258.6	NA	0.4	8.2		
D3	-215.6	1.2	7.1	-4.9	0.3	0.8	-215.6	NA	0.3	7.1		
D4	-177.1	1.0	7.0	8.2	0.3	0.8	-177.1	8.2	0.3	7.0		
D5	-137.9	0.9	6.6	21.4	0.2	0.8	-137.9	21.4	0.2	6.6		
D6	-98.6	0.7	6.2	34.6	0.2	0.7	-98.6	34.6	0.2	6.2		
D7	-60.7	0.5	5.4	47.9	0.1	0.8	-60.7	47.9	0.1	5.4		
D8	-17.5	0.5	5.8	60.7	0.1	0.8	-17.5	60.7	0.1	5.8		
T1	-650.9	-8.4	36.2	-104.3	0.4	3.9	-650.9	NA	-8.4	36.2		
T2	-879.1	-0.6	36.4	-128.3	1.4	5.9	-879.1	NA	1.4	36.4		
T3	-1099.0	-0.9	40.7	-144.1	1.5	6.0	-1099.0	NA	1.5	40.7		
T4	-1282.1	-0.2	42.8	-148.1	1.6	6.0	-1282.1	NA	1.6	42.8		
T5	-1432.4	0.2	44.5	-141.0	1.8	5.7	-1432.4	NA	1.8	44.5		
T6	-1549.1	0.8	45.2	-122.5	1.9	5.0	-1549.1	NA	1.9	45.2		
T7	-1632.2	1.0	47.4	-92.9	2.0	4.2	-1632.2	NA	2.0	47.4		
T8	-1683.1	2.0	47.5	-51.8	2.3	2.6	-1683.1	NA	2.3	47.5		
V1	135.9	0.0	0.0	13.0	0.0	0.0	NA	135.9	0.0	0.0		
V2	122.0	0.0	0.0	6.7	0.0	0.0	NA	122.0	0.0	0.0		
V3	99.6	0.0	0.0	-0.7	0.0	0.0	-0.7	99.6	0.0	0.0		
V4	78.3	0.0	0.0	-7.9	0.0	0.0	-7.9	78.3	0.0	0.0		
V5	57.0	0.0	0.0	-15.1	0.0	0.0	-15.1	57.0	0.0	0.0		
V6	35.2	0.0	0.0	-22.3	0.7	0.0	-22.3	35.2	0.0	0.0		
V7	15.4	0.0	0.0	-29.3	0.0	0.0	-29.3	15.4	0.0	0.0		
V8	40.0	-0.1	0.3	-35.9	1.1	2.4	-35.9	40.0	1.1	2.4		

Table 3-9 Tables developed for vertical threaded rods design

Table: Element Forces - Frames										
Frame	Ultimate			Construction			Controling			
	P (Kips)	V (Kips)	M (Kip-ft)	P (Kips)	V (Kips)	M (Kip-ft)	Pc (Kips)	Pt (Kips)	V (Kips)	M (Kip-ft)
V1	135.9	0.0	0.0	13.0	0.0	0.0	NA	135.9	0.0	0.0
V2	122.0	0.0	0.0	6.7	0.0	0.0	NA	122.0	0.0	0.0
V3	99.6	0.0	0.0	-0.7	0.0	0.0	-0.7	99.6	0.0	0.0
V4	78.3	0.0	0.0	-7.9	0.0	0.0	-7.9	78.3	0.0	0.0
V5	57.0	0.0	0.0	-15.1	0.0	0.0	-15.1	57.0	0.0	0.0
V6	35.2	0.0	0.0	-22.3	0.0	0.0	-22.3	35.2	0.0	0.0
V7	15.4	0.0	0.0	-29.3	0.0	0.0	-29.3	15.4	0.0	0.0
V8	40.0	-0.1	0.3	-35.9	1.1	2.4	-35.9	40.0	1.1	2.4

Design as Tension Memebers

Not a moment resisting element

Steel Properties:

Fy= 105 ksi **A139 B7 Grade steel**

ROUND BAR STOCK WEIGHT						
PITCH DIAMETER FOR ROLLED THREADS				ULL DIAMETER FOR CUT THREA		
UNC THREAD DIAMETER	PITCH DIAMETER	Area	WEIGHT PER FOOT	UNC THREAD DIAMETER	FULL DIAMETER	WEIGHT PER FOOT
¼ - 20	0.215	0.036	0.124	¼ - 20	0.25	0.167
5/16 - 18	0.274	0.059	0.201	5/16 - 18	0.313	0.262
⅜ - 16	0.331	0.086	0.293	⅜ - 16	0.375	0.376
7/16 - 14	0.387	0.118	0.4	7/16 - 14	0.438	0.513
½ - 13	0.445	0.156	0.529	½ - 13	0.5	0.668
⅝ - 11	0.56	0.246	0.838	⅝ - 11	0.625	1.044
¾ - 10	0.68	0.363	1.236	¾ - 10	0.75	1.504
⅞ - 9	0.795	0.496	1.689	⅞ - 9	0.875	2.046
1 - 8	0.91	0.65	2.213	8-Jan	1	2.673
1-⅛ - 7	1.025	0.825	2.808	1-⅛ - 7	1.125	3.383
1-¼ - 7	1.145	1.03	3.504	1-¼ - 7	1.25	4.176
1-⅜ - 6	1.264	1.255	4.271	1-⅜ - 6	1.375	5.054
1-½ - 6	1.375	1.485	5.054	1-½ - 6	1.5	6.014

Table from the TSA Technical Specs website

1. Design for Tension

Axial Design

Frame	Pt (Kips)	As (in2)	TR
V1	135.9	1.44	1 1/2
V2	122.0	1.29	1 1/2
V3	99.6	1.05	1 1/4
V4	78.3	0.83	1 1/4
V5	57.0	0.60	1
V6	35.2	0.37	1
V7	15.4	0.16	1
V8	40.0	1.48	4 #6 bars

2. Design for Compression

Axial Design (Buckling) AISC 13th Chapter E

Frame	Pt (Kips)	TR
V1	NA	1 1/2
V2	NA	1 1/2
V3	0.7	1 1/4
V4	7.9	1 1/4
V5	15.1	1 1/2
V6	22.3	1 1/2
V7	29.3	1 1/2
V8	35.9	4 #6 bars

Value shown is the minimum between tension and compression

Crack Control (fy=30 ksi)

Frame	Pt (Kips)	As (in2)	Rebar
V8	40.0	1.33	4 #6 bars

Table 3-10 Tables developed to design the diagonal concrete members

Table: Element Forces - Frames										
Frame	Ultimate			Construction			Controling			
	P (Kips)	V (Kips)	M (Kip-ft)	P (Kips)	V (Kips)	M (Kip-ft)	Pc (Kips)	Pt (Kips)	V (Kips)	M (Kip-ft)
D1	-267.9	0.7	1.8	-28.1	0.4	0.5	-267.9	NA	0.4	1.8
D2	-258.6	1.5	8.2	-18.5	0.4	0.8	-258.6	NA	0.4	8.2
D3	-215.6	1.2	7.1	-4.9	0.3	0.8	-215.6	NA	0.3	7.1
D4	-177.1	1.0	7.0	8.2	0.3	0.8	-177.1	8.2	0.3	7.0
D5	-137.9	0.9	6.6	21.4	0.2	0.8	-137.9	21.4	0.2	6.6
D6	-98.6	0.7	6.2	34.6	0.2	0.7	-98.6	34.6	0.2	6.2
D7	-60.7	0.5	5.4	47.9	0.1	0.8	-60.7	47.9	0.1	5.4
D8	-17.5	0.5	5.8	60.7	0.1	0.8	-17.5	60.7	0.1	5.8

Steel Properties:

Fy= 60 ksi

4 #4 bars	0.78 in^2	4 #6 bars	1.77 in^2	4 #8 bars	3.14 in^2
4 #5 bars	1.23 in^2	4 #7 bars	2.41 in^2	4 #9 bars	4.00 in^2

1. Design for Tension

Axial Design

Frame	Pt (Kips)	As (in2)	Rebar
D1	NA	NA	NA
D2	NA	NA	NA
D3	NA	NA	NA
D4	8.2	0.15	4 #4 bars
D5	21.4	0.40	4 #4 bars
D6	34.6	0.64	4 #4 bars
D7	47.9	0.89	4 #5 bars
D8	60.7	1.12	4 #5 bars

Crack Control (fy=30 ksi)

Frame	Pt (Kips)	As (in2)	Rebar
D1	NA	NA	NA
D2	NA	NA	NA
D3	NA	NA	NA
D4	5.6	0.19	4 #4 bars
D5	14.9	0.50	4 #4 bars
D6	24.4	0.81	4 #5 bars
D7	33.6	1.12	4 #5 bars
D8	44.5	1.48	4 #6 bars

2. Design for Compression Design as struts (ACI 318-11 Appendix A)

$A_{st} = 0.01A_g$ ACI 10.9.1

Ast= 0.64 in^2

Axial Design

Frame	Pc (Kips)	As (in2)	Rebar
D1	-267.9	1.50	4-#6 Bars
D2	-258.6	1.20	4-#6 Bars
D3	-215.6	0.64	4 #4 bars
D4	-177.1	0.64	4 #4 bars
D5	-137.9	0.64	4 #4 bars
D6	-98.6	0.64	4 #4 bars
D7	-60.7	0.64	4 #4 bars
D8	-17.5	0.64	4 #4 bars

Final Minimum Reinforcement

Frame	As (min)	Rebar	Stirrups
D1	1.50	4-#6 Bars	#3 ties @ 8 in spacing*
D2	1.20	4-#6 Bars	
D3	0.64	4 #4 bars	
D4	0.64	4 #4 bars	
D5	0.64	4 #4 bars	
D6	0.81	4 #5 bars	
D7	1.12	4 #5 bars	
D8	1.48	4 #6 bars	

*Ties are needed for confinment according to ACI 318 7.10.5

Table 3-11 Design table for top flange

Table: Element Forces - Frames										
Frame	Ultimate			Construction			Controling			
	P (Kips)	V (Kips)	M (Kip-ft)	P (Kips)	V (Kips)	M (Kip-ft)	Pc (Kips)	Pt (Kips)	V (Kips)	M (Kip-ft)
T1	-650.9	-8.4	36.2	-104.3	0.4	3.9	-650.9	NA	-8.4	36.2
T2	-879.1	-0.6	36.4	-128.3	1.4	5.9	-879.1	NA	1.4	36.4
T3	-1099.0	-0.9	40.7	-144.1	1.5	6.0	-1099.0	NA	1.5	40.7
T4	-1282.1	-0.2	42.8	-148.1	1.6	6.0	-1282.1	NA	1.6	42.8
T5	-1432.4	0.2	44.5	-141.0	1.8	5.7	-1432.4	NA	1.8	44.5
T6	-1549.1	0.8	45.2	-122.5	1.9	5.0	-1549.1	NA	1.9	45.2
T7	-1632.2	1.0	47.4	-92.9	2.0	4.2	-1632.2	NA	2.0	47.4
T8	-1683.1	2.0	47.5	-51.8	2.3	2.6	-1683.1	NA	2.3	47.5

Design as the compression block of a beam

Ag= 270.0 in2

Max. Compression=0.9(0.85*f'c*270)= 1652.4 kips

Add steel reinforcement: (fy=60ksi)

Use 4 #4 rebar= 0.785398 in2

Compression in steel=0.9*fy*As= 42.4115 kips

Max. Compression=0.9(0.85*f'c*270)= 1647.593 kips

Total Compression force= 1690.005 kips

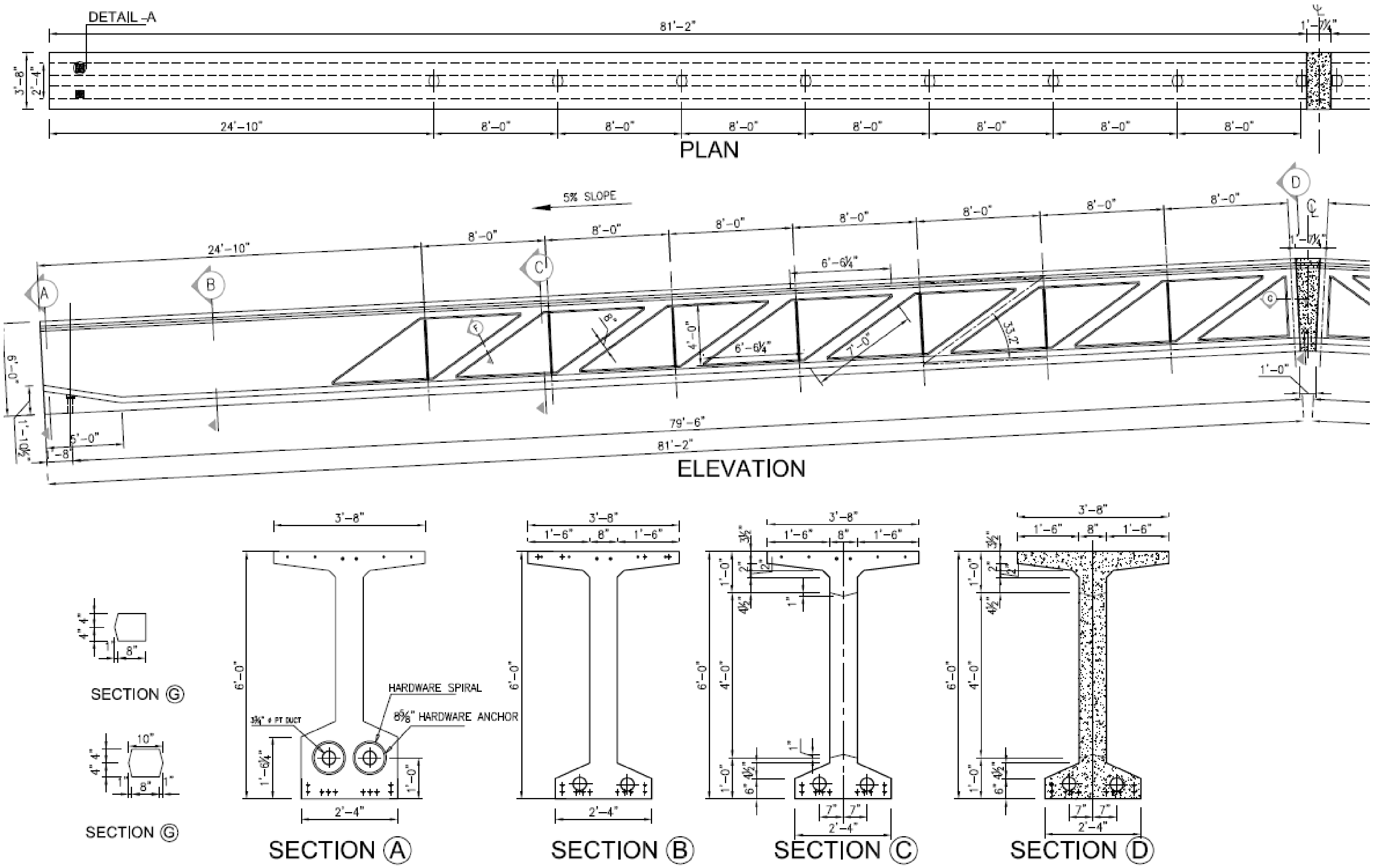

Figure 3.15 Concrete truss dimensions shop ticket

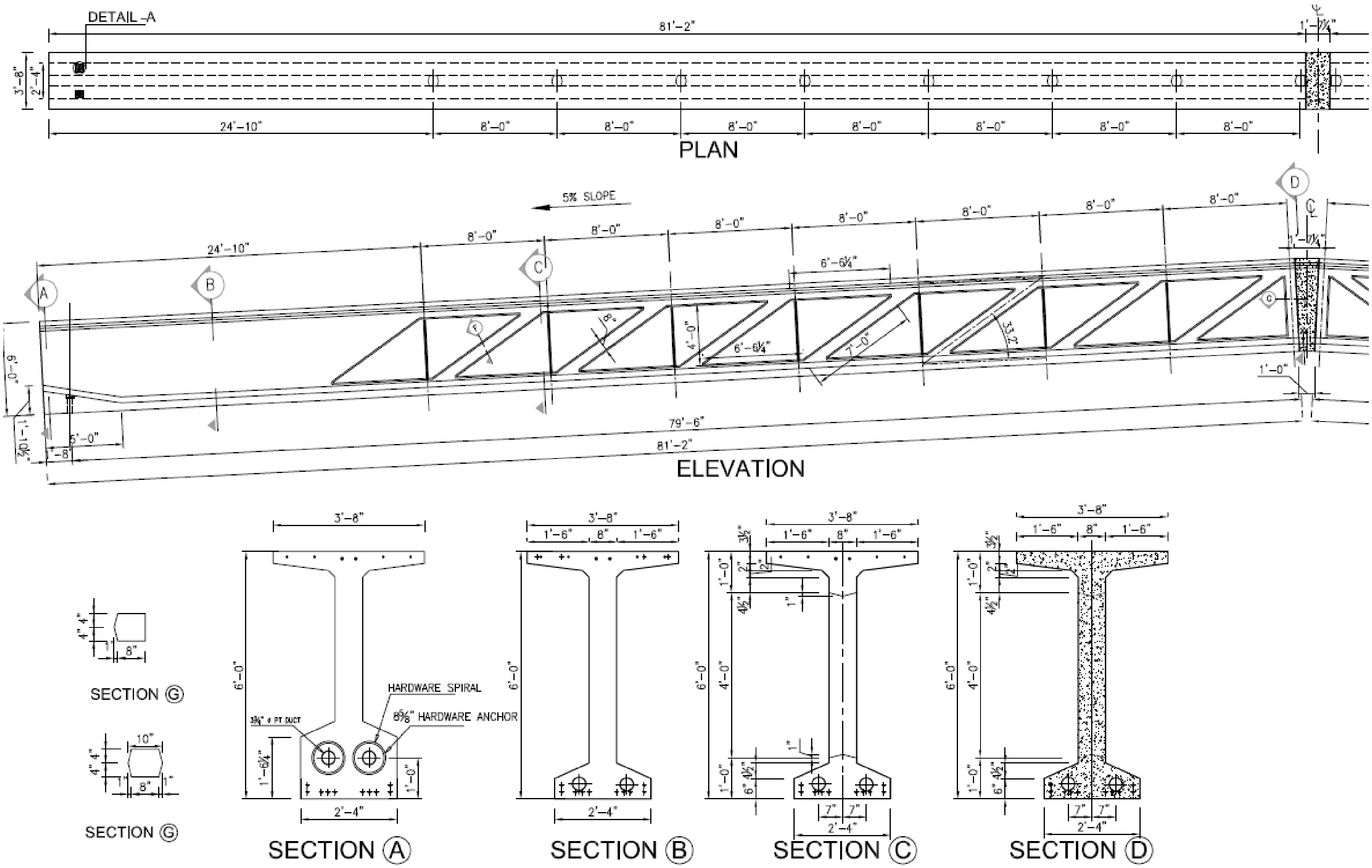

Figure 3.16 Elevation and cross-sections of one half of the truss

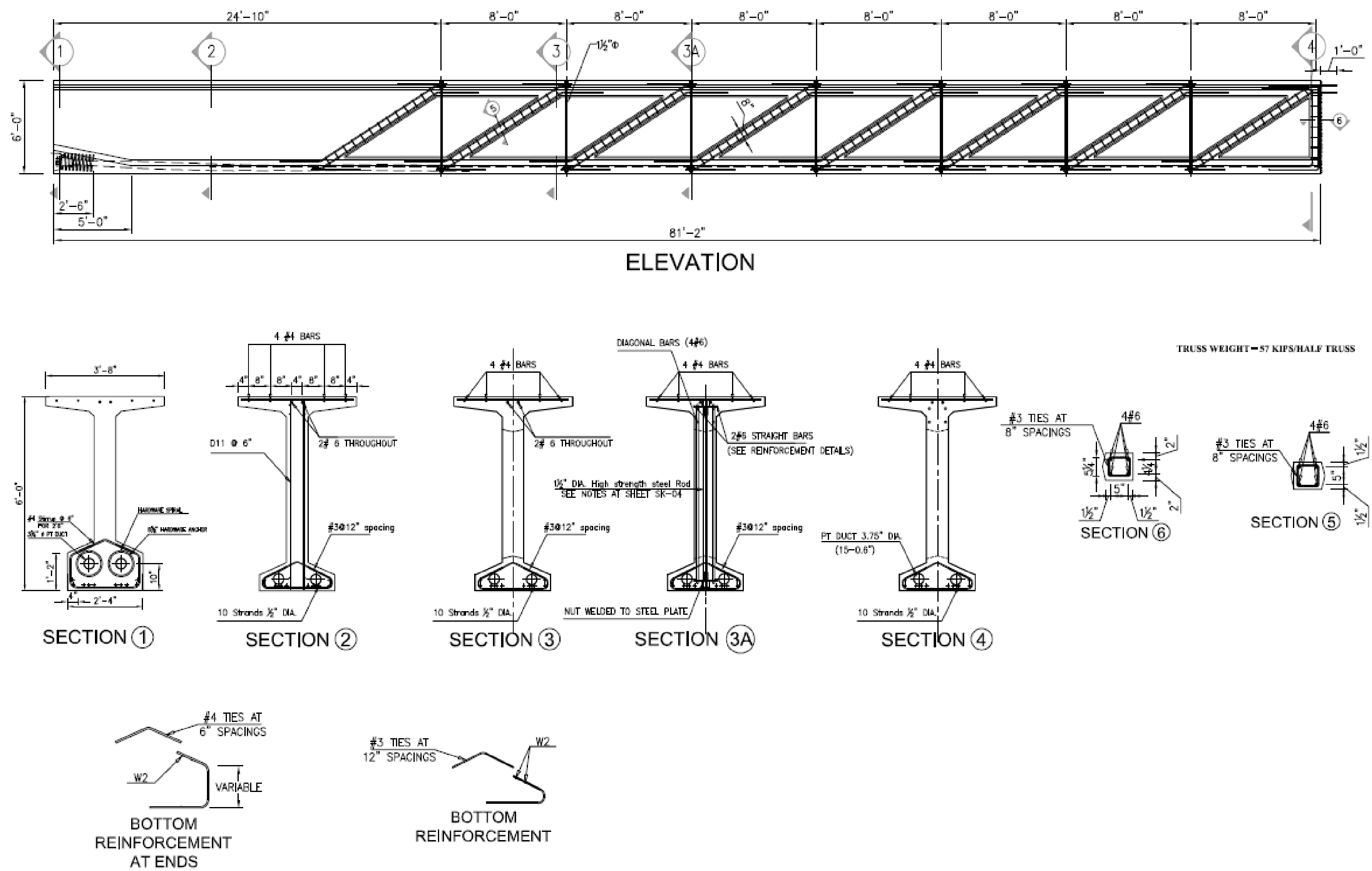

Figure 3.17 Reinforcements dimensions shop ticket

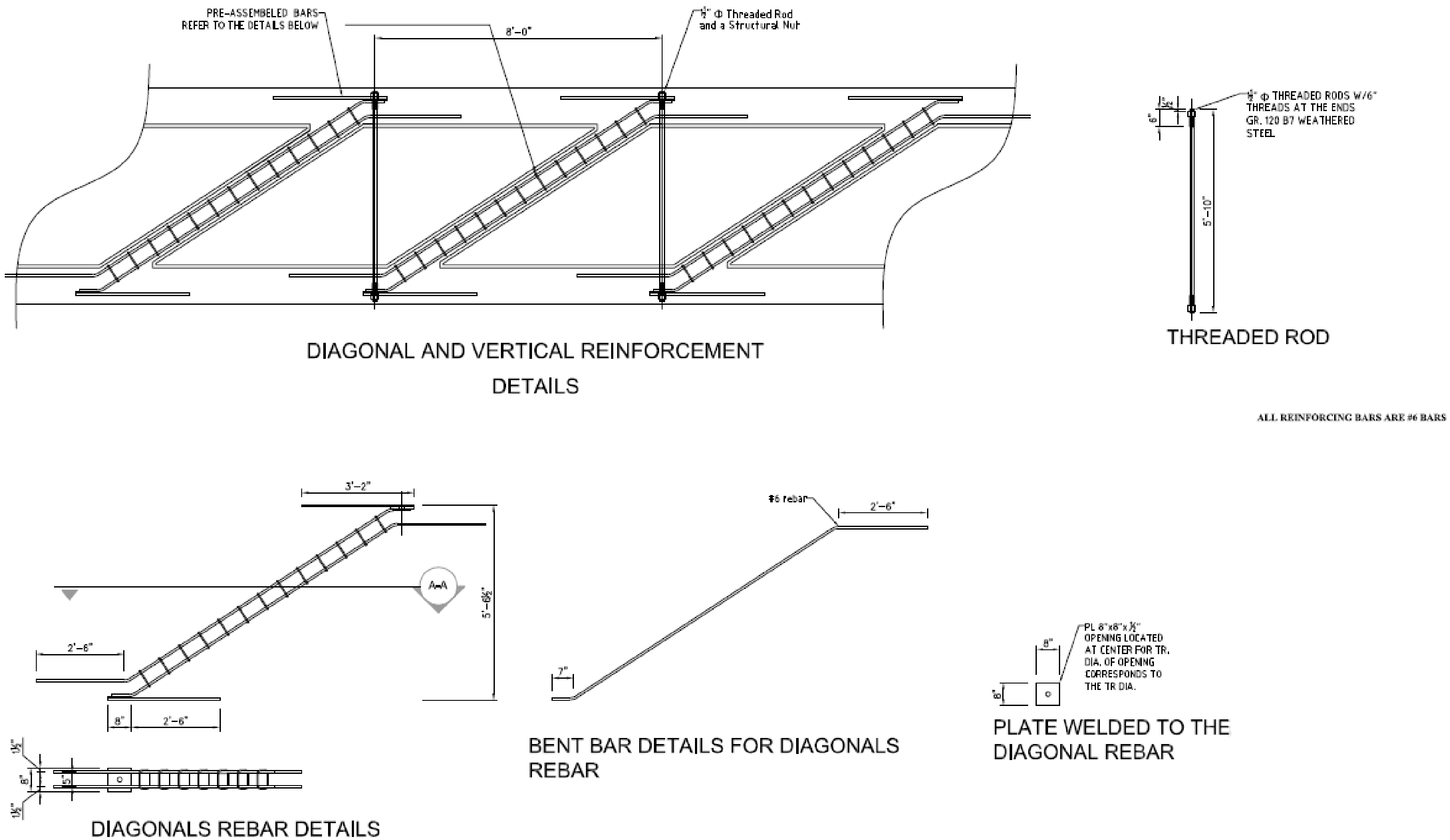

Figure 3.18 Pre-assembled reinforcing bars details

3.3 CONSTRUCTION

The construction of the truss is one of the main research purposes. If easy fabrication can be achieved, time, effort, and money will be saved. The proposed method of fabrication is using Styrofoam block-outs to make the openings in the web and have the threaded rod pass through it. The foam can be glued to the steel form so it acts as one piece together. To facilitate stripping, it was suggested to have the edges slopped out so the pieces can slide out with the steel form easily.

Post-tensioning design was designed to go straight through the truss except at the ends. The two post tensioning ducts were suggested to go up at the ends and go on top of each other in the web. Such practice will require a bigger end block to be able to take the post-tensioning forces. The suggested design is shown in Figure 3.19 and Figure 3.20. In addition to this, the advising committee questioned the effect of the twist occurring in the post-tensioning ducts in order to have them on top of each.

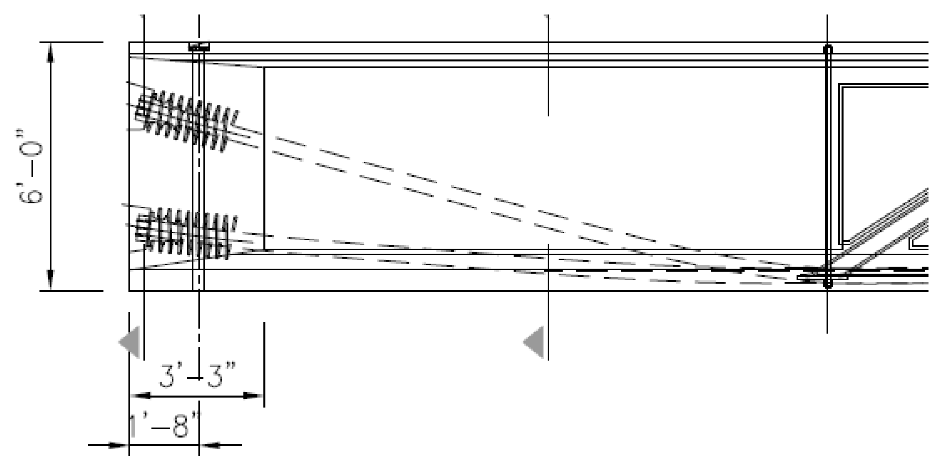

Figure 3.19 Preliminary suggested profile for the post-tensioning ducts

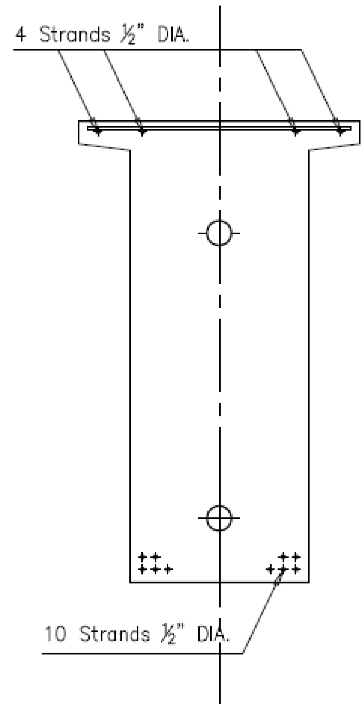

Figure 3.20 Preliminary Suggested end block for the truss post-tensioning system

As an alternative and to optimize the truss construction, the post-tensioning ducts were suggested to be left straight throughout the span of the truss and slightly elevated at the ends while remaining in the bottom flange (and having a slightly thicker bottom flange). Figure 3.21 shows the new design profile and the end block is shown in Figure 3.22.

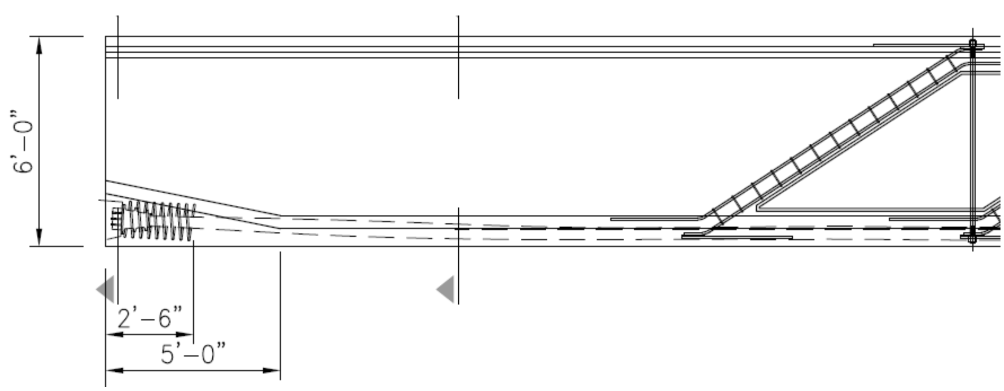

Figure 3.21 Profile of the alternative design of the post-tensioning ducts

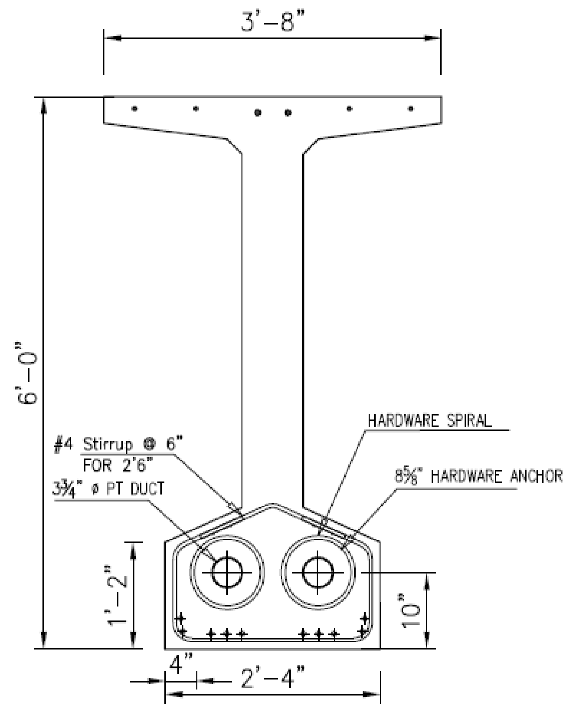

Figure 3.22 The suggested alternative for the end block for the post tensioning ducts

The production of the trusses can be greatly facilitated using pre-assembled reinforcing steel for the diagonals with a plate welded at the ends to anchor the threaded rods to it. The plate can then have anchoring bars embedded in the concrete and welded to it. Further details are explained in the truss shop tickets in the appendix. The laboratory specimen construction section discusses the system and alteration that can be done.

The construction sequence is suggested to be as follows:

- After the manufacturing process at the plant the truss-girders are transported to the project site.
- The truss-girders are erected and supported on columns from one side and on temporary supports at the midspan.
- The wet joint is then cast in place.

- After casting the wet joint, the post-tensioning is then applied.
- After the application of post-tensioning the temporary supports are removed and the bracings are added.
- Light gage purlins are then used to connect the roofing material with the truss supporting system. The purlins have to be resting on the diagonal/vertical connection with the top flange to avoid any unnecessary moments.
- Mechanical, electrical, and pluming components are installed at the ends.

The threaded rods are recommended to be sprayed for rust resistance and fire-proofing, according to the fire-proofing codes.

4.0 EXPERIMENTAL INVESTIGATION

4.1 SPECIMEN ANALYSIS

The purpose of the experimental work was not only to test the truss structural adequacy, but also to provide the best construction sequence and recommendations for the truss. The truss was formed using Iowa type D steel forms provided through Coreslab Structures. The form was 30 ft long. In order to avoid any extra weight, a 4 in block-out was made at the bottom of the form to have a total depth of 4 ft 4 in. The cross-section of the specimen is shown in Figure 4.1.

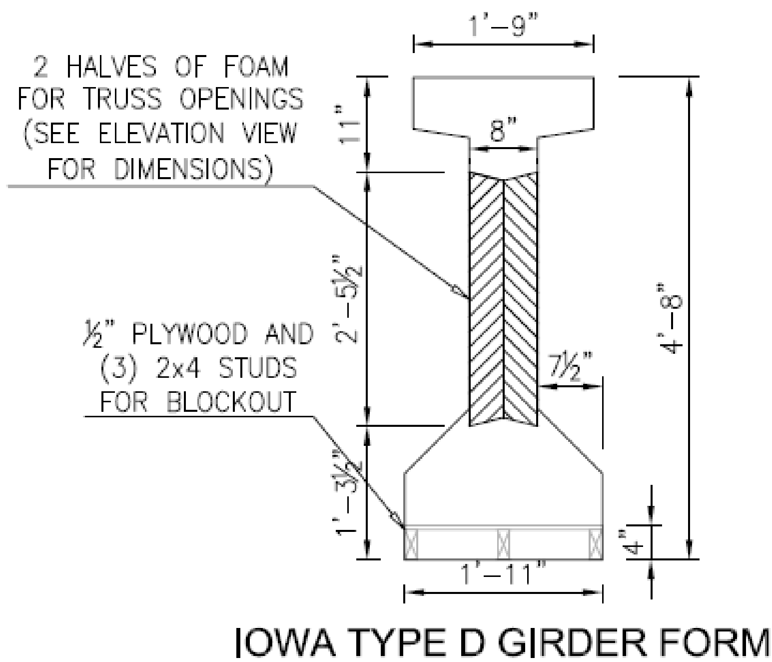

Figure 4.1 Cross-section of the truss specimen and foam block-out

To keep the forces resulted in the diagonal and vertical members similar to and representable enough to the proposed system, the diagonals were designed to be at a 40° angle from the bottom flange, see Figure 4.2 and Figure 4.3 for more details. Frame analysis and Finite Element Analysis (FEA) were done and results are reported in this section. A comparison between the outputs of two types of analysis is shown in Table 4-1.

Table 4-1 Comprison between the 2-D fram analysis and the finite element analysis output

POC	2-D Frame Analysis	Finite Element Analysis
Diagonal Members	279 kips (Compression)	253 kips (Compression)
Vertical Members	129 kips (tension)	120 kips (tension)
Camber	0.2 in.	0.20 in.
Deflection	0.8 in.	0.65 in.
Total deflection	1 in.	0.85 in.
Cracking Load	330 kips	300 kips

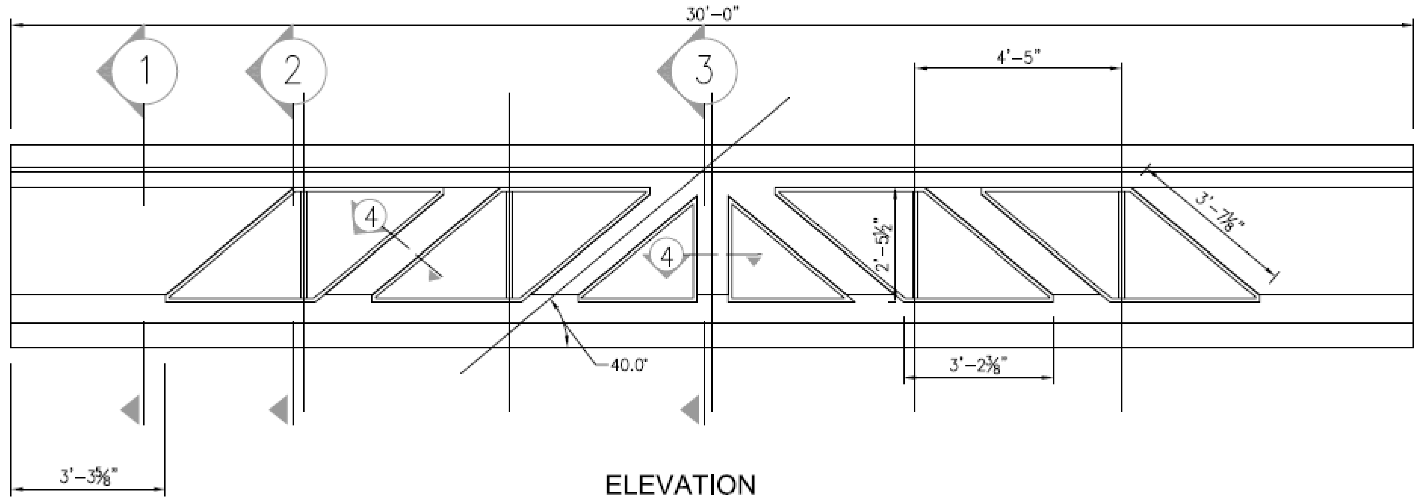

Figure 4.2 Elevation of the truss specimen

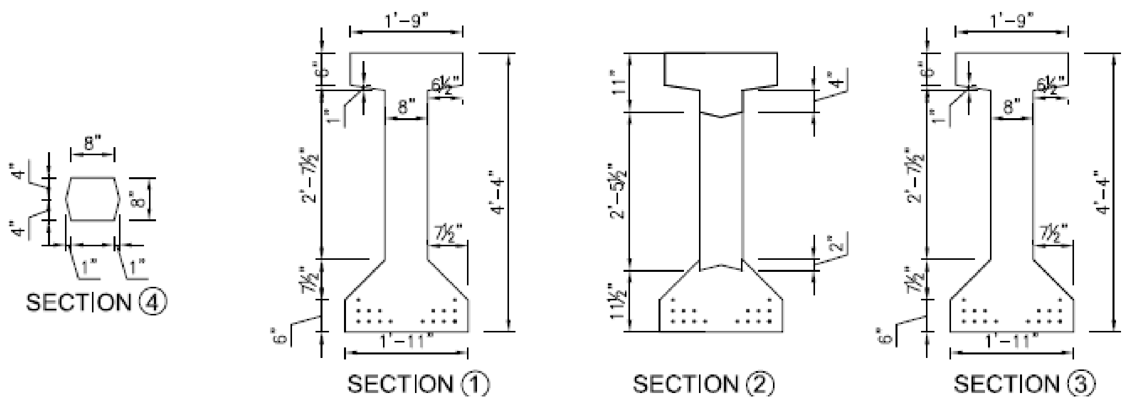

Figure 4.3 Sections of the specimen

4.1.1 2-D Analysis

The analysis was done exactly as the proposed full size truss analysis. The concrete webs were modeled as shell members and the rest were frame elements. Only the threaded rods had moment releases at the ends. After different trials, it was found that a point load of 400 kips will result in the forces needed to test the structural adequacy of the truss, specifically the diagonal and vertical members.

The preliminary prestressing design included 16 0.6-in low relaxation strands. However, due to shortage of time and materials, the strands were substituted with 12 0.7-in strands. According to the bottom flange stresses, it was found that cracking is expected at a load of 330 kips. The calculations were made according to the $7.5\sqrt{f_c'}$ stress limit for cracking (PCI 2010). The strands jacking stress was $0.75f_u$ and total prestress losses of 20% of the jacking stress. Total effective prestressing was assumed to be 160 ksi.

The frame analysis showed that under dead load, a deflection of 0.03 in. occurs. Under the design load of 400 kips, the truss is supposed to deflect 1.20 in. at midspan. Since the analysis is done on an elastic model with no cracking taken in account, deflection is recognized as 0.16 in every 50 kips of load till cracking. According to the predicted cracking load of 330 kips, the deflection at midspan under elastic behavior is estimated to be about 1 in.

Appling the 400-kip load, the maximum resulting compression force in the diagonal members is 280 kips at the diagonals close to the middle, compared to 269 kips in the full size truss. The maximum tension force in the vertical members is 129 kips at the outside verticals, compared to 136 kips in the full size truss. All the forces are shown on the truss in Figure 4.4.

Despite the forces information provided, the analysis results do not provide a complete picture of the stresses (shell forces) at the connections between the diagonal, vertical and horizontal members of the truss. To better predict the behavior of the truss, as well as compare analysis results to test results, a finite element analysis was performed on the concrete truss specimen. The details of the analysis along with the results are further explained in the following subsection.

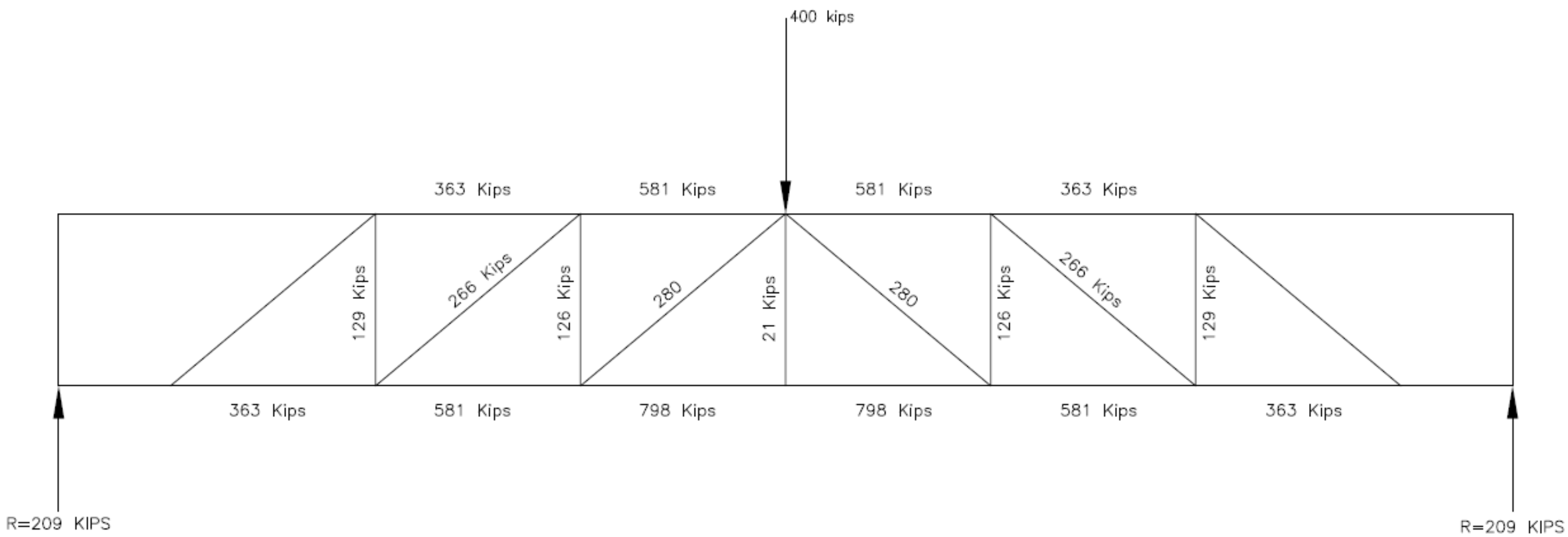

Figure 4.4 Axial loads on specimen at testing

4.1.2 Finite Element Analysis

A finite element model was prepared to investigate the stresses at the connections between the diagonal and vertical members and the top and bottom flanges. The model is shown in Figure 4.6 and the approximated section for the shell members in Figure 4.5. The analysis results show high tensile forces at the acute angles in the connections between the diagonals and top and bottom flanges.

Prestressing consisted of 2 layers of 12 0.7 in. diameter strands. The bottom layer at 2 in. for the bottom had 8 strands, and the top layer at 4 in. from the bottom had 4 strands. The strands jacking stress was $0.75f_u$ and total prestress losses of 20% of the jacking stress. Total effective prestressing was assumed to be 160 ksi. According to the bottom flange stresses, it was found that cracking is expected at a load of 300 kips. The calculations were made according to the $7.5\sqrt{f_c'}$ stress limit for cracking (PCI 2010). At 300 kips, the bottom flange stresses reached a maximum of 850 psi, while the PCI limit is 750 psi.

The connection between the diagonal end of the web and the bottom flange shows very high tensile forces as can be seen in Figure 4.7. As a matter of fact, cracking was found in the model to occur under a load of 170 kips, at which the tensile stress is higher than the stress limit for the non-cracked section. These tensile stresses are predicted to cause cracking at those location and could as well be the reason for failure. The forces were similar to the frame analysis previously done with only slight differences.

The differences are a due to the approximated section used for the shell elements, while the frame model was done using the exact section. The maximum forces in the diagonal members was 250 kips (versus 280 kips in the frame analysis). The maximum force in the vertical members was 120 kips (compared to 136 in the frame analysis). The maximum deflection was at the midspan and was found to be 0.8 in. (versus 1.2 in. in the frame analysis). The camber due to prestressing was 0.2 in. As a result, the total change in deflection that could be predicted for and compared with the testing results is 1 in. (the 0.8 in. deflection in addition to the 0.2 camber).

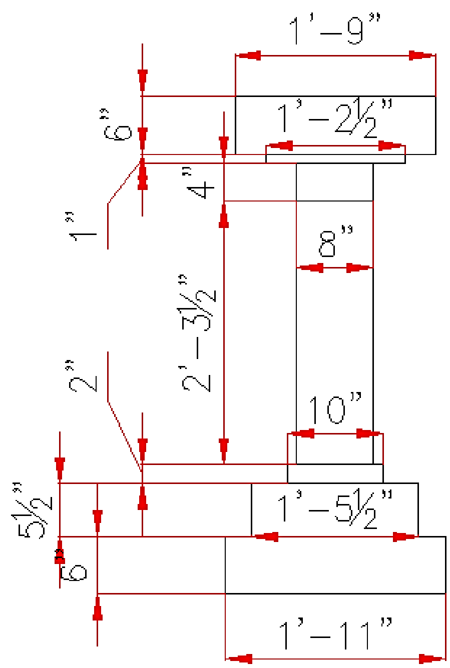

Figure 4.5 Approximated section used for analysis

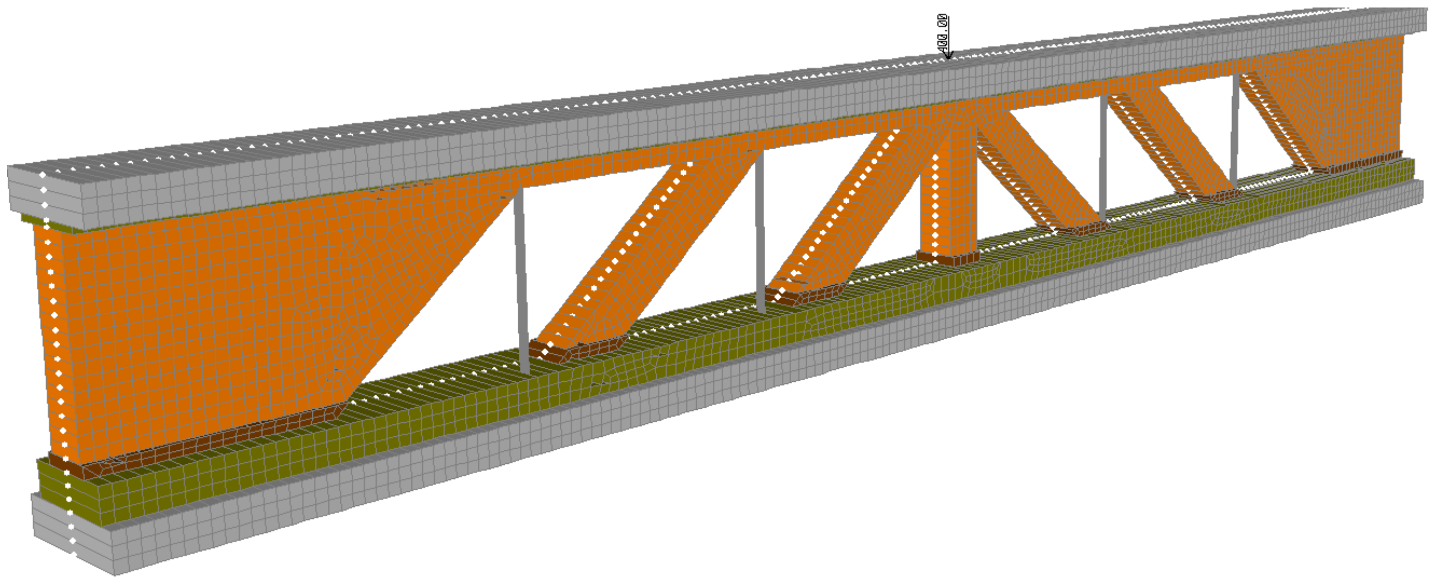

Figure 4.6 The FE model used to analyze the truss

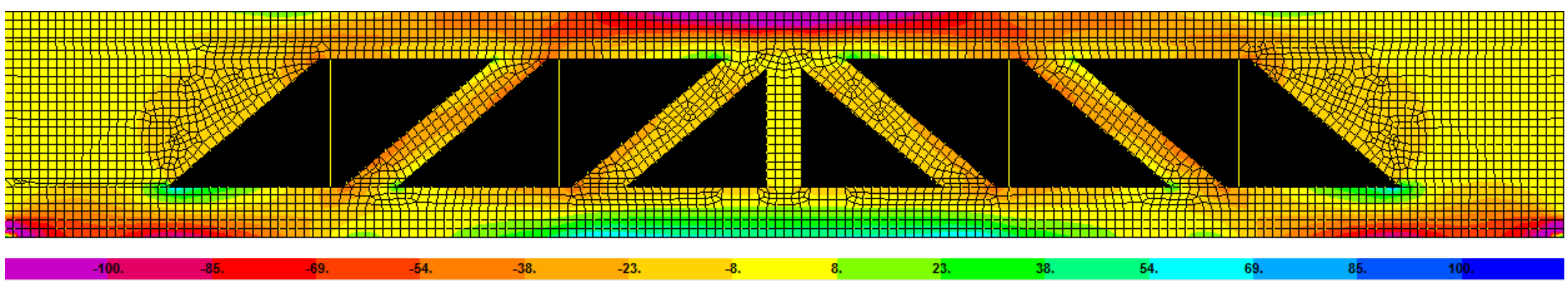

Figure 4.7 The stresses in the truss specimen at 400 kips

4.2 SPECIMEN DESIGN

The specimen was design the same way the 160-ft span truss was designed. The diagonal and vertical members were the same as the previous design, essentially because the goal of the testing is to have the same forces in them as before. Detailing of the rebar was the same, only just a shorter diagonal. Calculations showed that total prestressing losses was 13% of the jacking stress.

For the flexure design, the truss was dealt with as a beam when designing the top and bottom (prestressing) reinforcement. The flexure strain compatibility program designed by the Nebraska University was used for the flexure design. The moment due to the point load of 400 kips was equal to PL/4 = 400 kips x 30 ft/4 = 3000 ft-kip. 12 0.7-in strands were used with 4 strands in the top layer and 8 in the bottom layer of strands. 2 #8 bars were used for the top flange reinforcements. The design flexure strength was equal to 3224 ft-kip. Refer to **Error! Reference source not found.** for a snapshot of the program.

The span to depth ratio is different in the 160-ft span proposed system than the specimen we tested. As a result, the applied load needed to result in having the service load equivalent member forces was different than the one needed to result in the same service load stresses in the full span system at the bottom flange (for prestressing). That made it irrelevant for the specimen to keep track of the cracking load, except for the intent of comparing the actual deflection to the analysis model deflection. Specimen shop tickets are shown in Figure 4.8 to Figure 4.9.

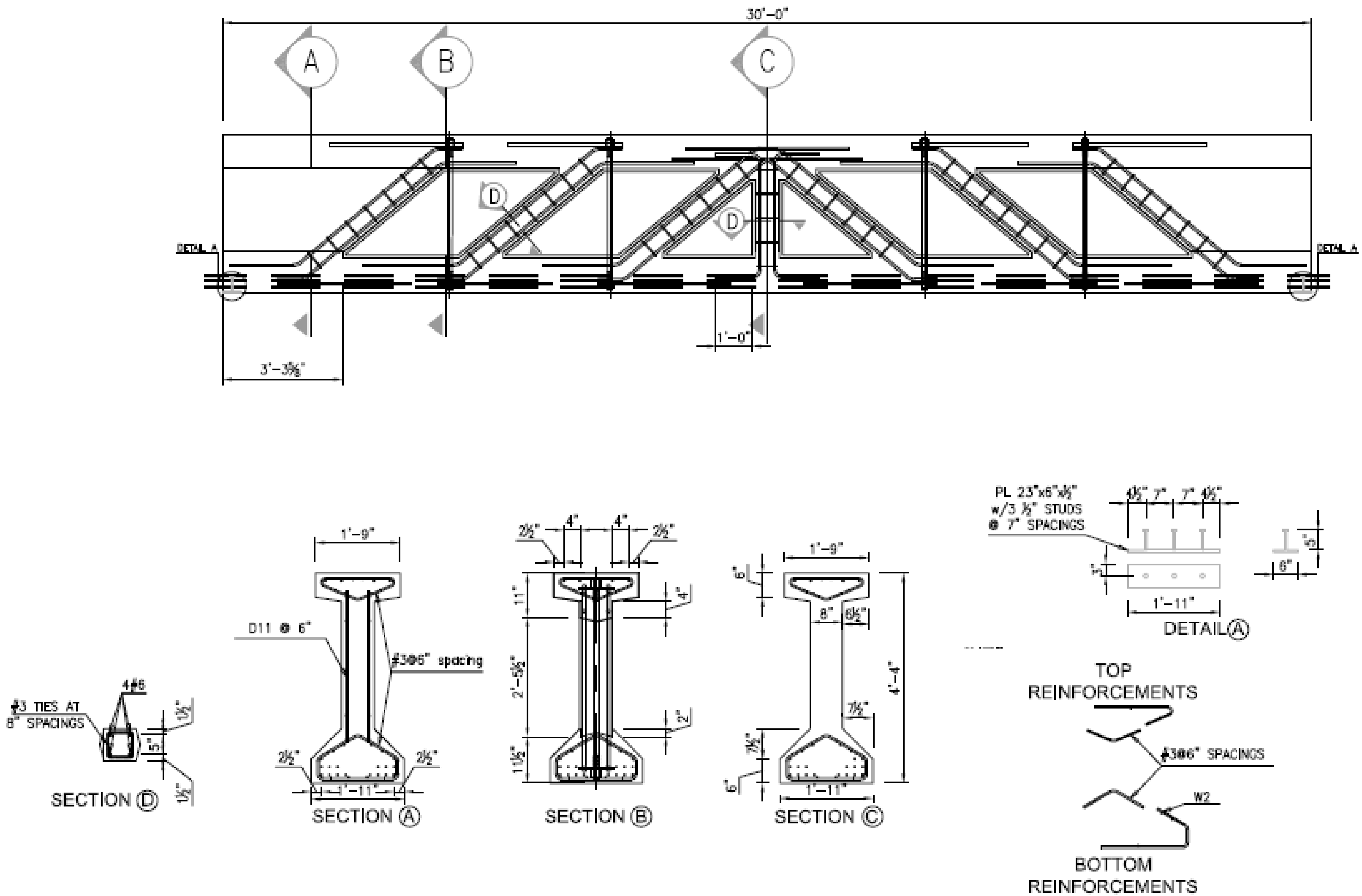

Figure 4.8 Elevation and cross-sections of reinforcements

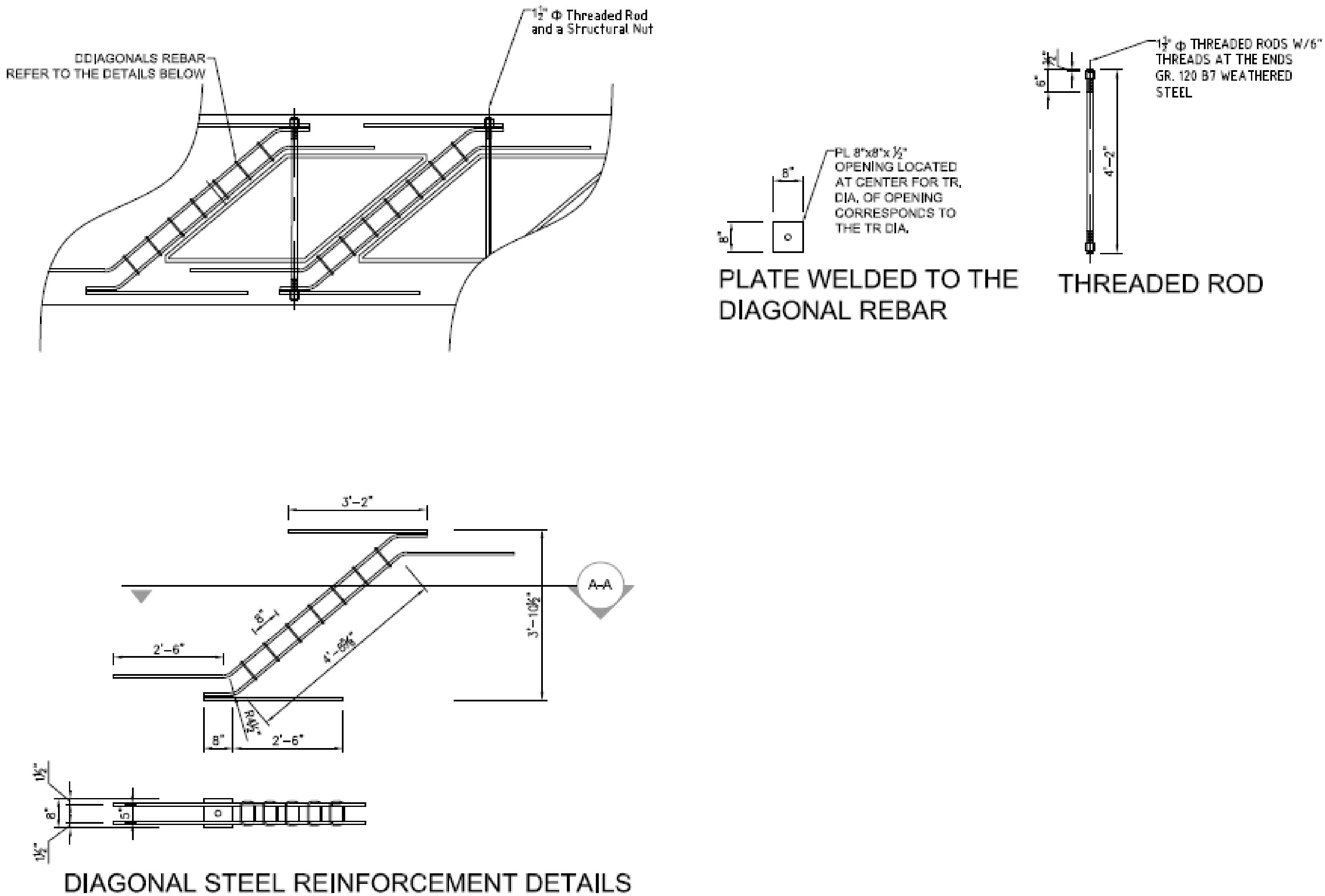

Figure 4.9 Reinforcements details

4.3 SPECIMEN FABRICATION

Construction of the 30-ft specimen was somehow challenging, especially at the beginning of the construction. Construction sequence had 5 phases, laying the strands in the prestressing bed, cutting the foam block-outs and gluing them on the steel forms, assembling the steel diagonals and fixing them with the form, casting the concrete in the forms, and finally stripping the forms. Before putting the forms in place, the forms needed to be raised 3.5 in. to have the center of gravity of the strands match with the center of the jack. 4.1 in block-out made consisting of ¾" plywood (0.625 in. thick) screwed on three 2x4s longitudinally.

4.3.1 Fabrication Sequence

4.3.1.1 Phase 1: Strands for prestressing

The prestressing bed located at the structural lab of the Peter Kiewit Institute is 60 ft long and 10 ft wide. The ends of the bed have steel plates anchored in the 12-in. wide side walls of the bed as shown in Figure 4.10.

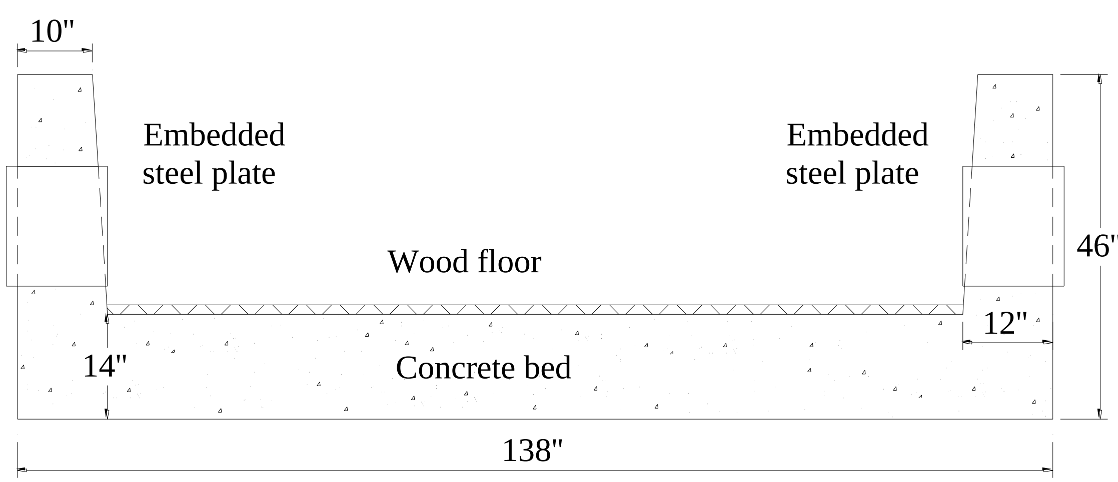

Figure 4.10 The ends of the prestressing bed

At the north end of the bed, steel plates are stacked on top of each other with spacers located were the strands are intended to be, then an anchoring steel plate is placed where the chucks will be resting against. At the other end, the same assembly exists; however, prestressing jacks are attached to the plates already anchored in the bed side walls. These jacks are used for releasing the strands. The stacked plates rest against these jacks. Figure 4.11 and Figure 4.12 show longitudinal profiles of the north and south ends. The capacity of the prestressing bed is 1,000,000 lb with only 750,000 lb allowable maximum prestressing force.

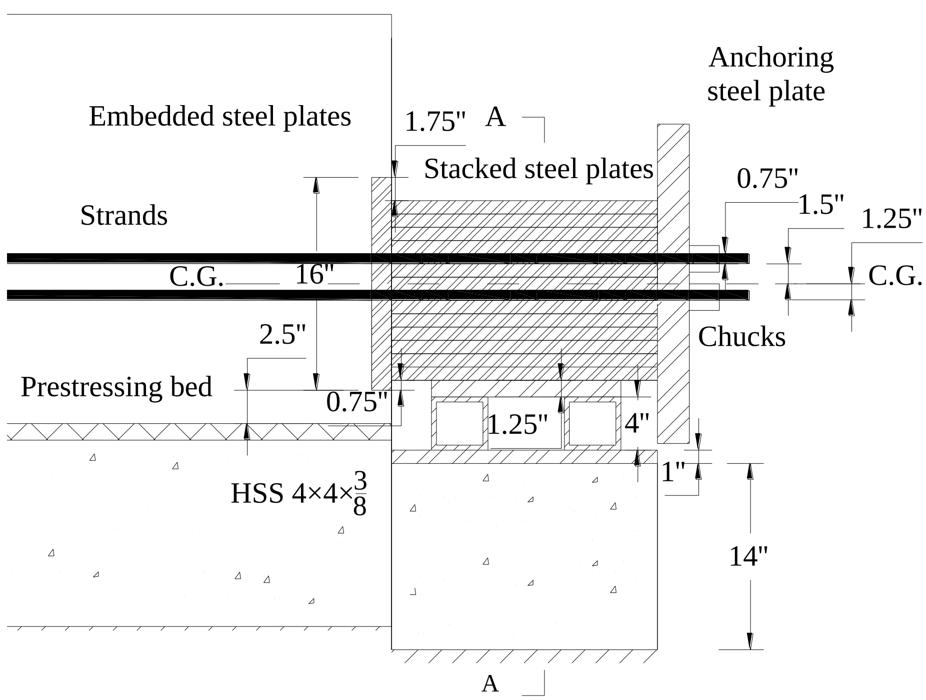

Figure 4.11 Longitudinal profile of the north abutment

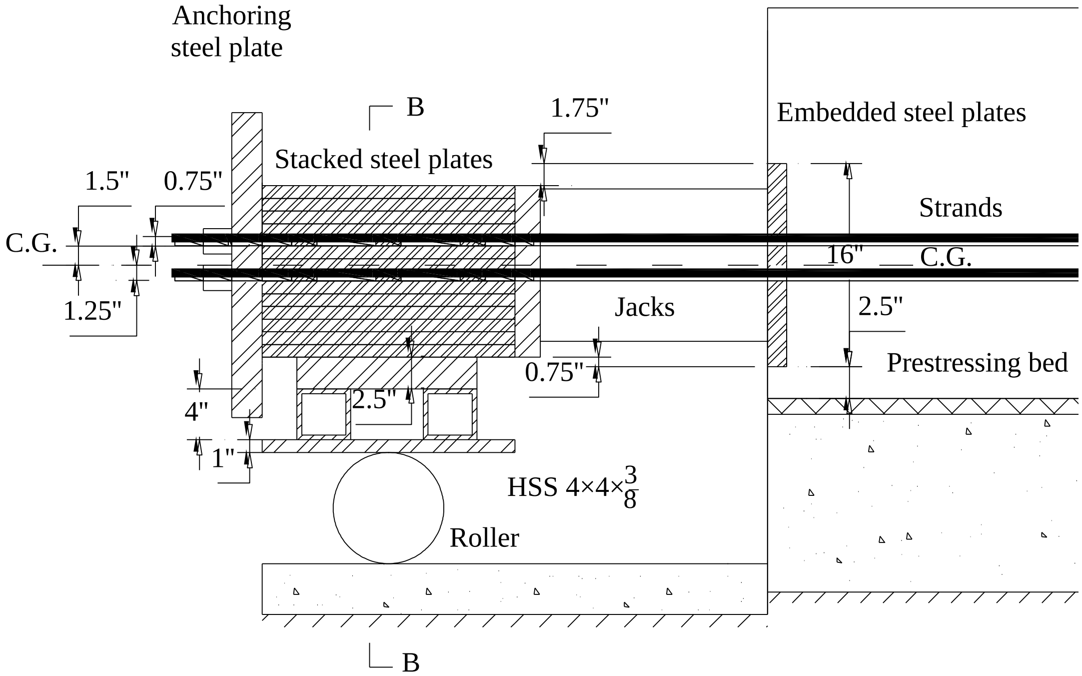

Figure 4.12 Longitudinal profile of the south abutment

12 0.7-in. diameter low relaxation strands were used for prestressing for a total jacking force of 728 kips (0.75 of the ultimate stress of 270 ksi). The strands were anchored against steel plates as mentioned before. The steel plates with the strands locations are shown in Figure 4.13.

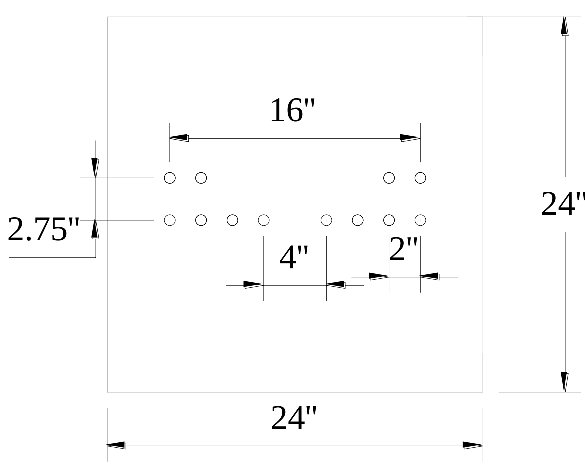

Figure 4.13 Anchoring steel plates with the strands locations shown

The strands were laid out on the bed on top of the plywood, as shown in Figure 4.14. They were tensioned just enough to get them straight. The plywood was then centered with the strands and one side of the steel form was put and fixed against the plywood block-out. 2X4 timber was used to hold the steel forms in place at the bottom. Figure 4.15 show the side of the form in place with the timber behind it to support it in place. The strands were then tensioned to 100 ksi, half of the jacking stress, in order to start putting the rebar together, were some would be resting on the strands. Bottom flange stirrups were then tied on to the strands (Figure 4.16) and bearing plates were also put in place before the end bulkheads were attached.

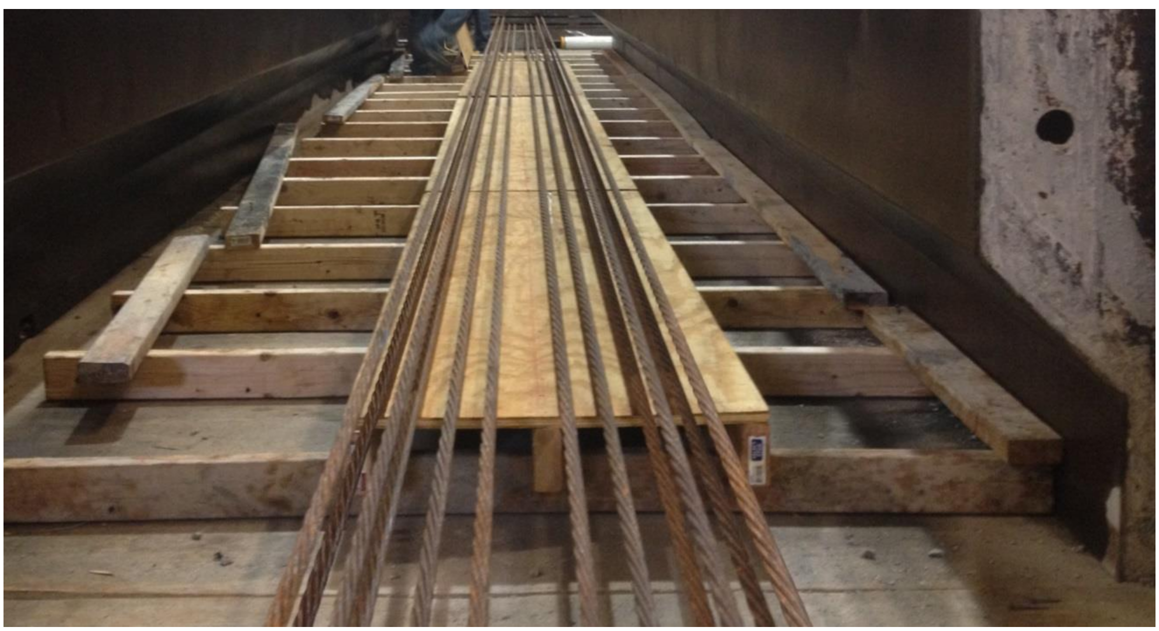

Figure 4.14 Strands laid out on the grid and the block-out platform

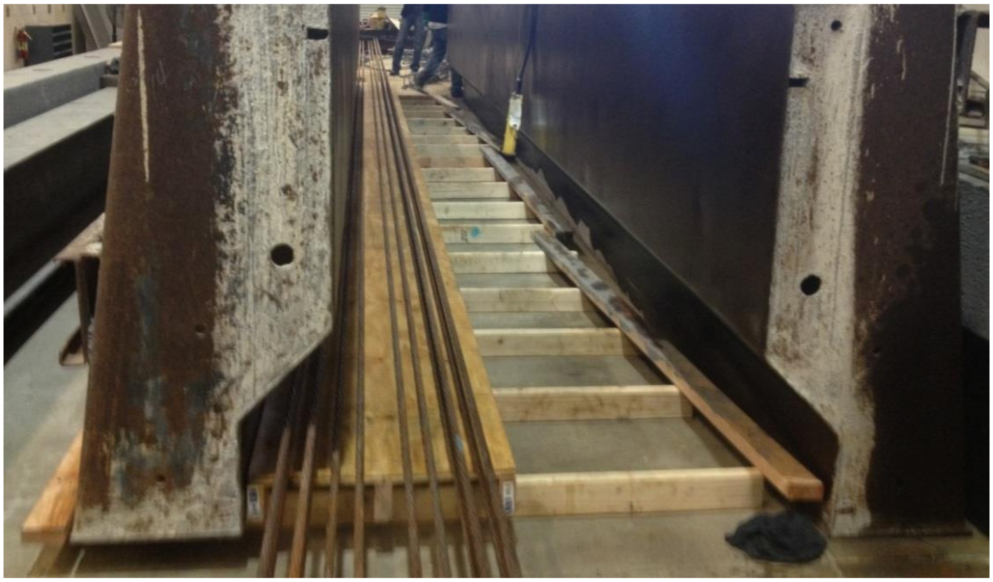

Figure 4.15 Fixing one side of the form in place

Figure 4.16 The strands after tieing the stirrups and putting the end bulkheads

4.3.1.2 Phase 2: Foam block-outs

Foam block-outs were used in order to test the constructability of the truss. The foam pieces for the specimen were not as deep as the ones for the full size truss, as the specimen is shallower than the full size. The block-outs were made to resemble the full size specimen in order to adequately test the feasibility of its construction. Figure 4.17 represents the block-out made out of Styrofoam. The dimensions and shape in the 3D drawing is for the block-outs as cut and used. This shape differs than the full-size one by not having the sloped edges or the chamfered edges. This difference occurred due to the shortage of time and proper equipment as the block-outs were cut and glued together in the structural lab.

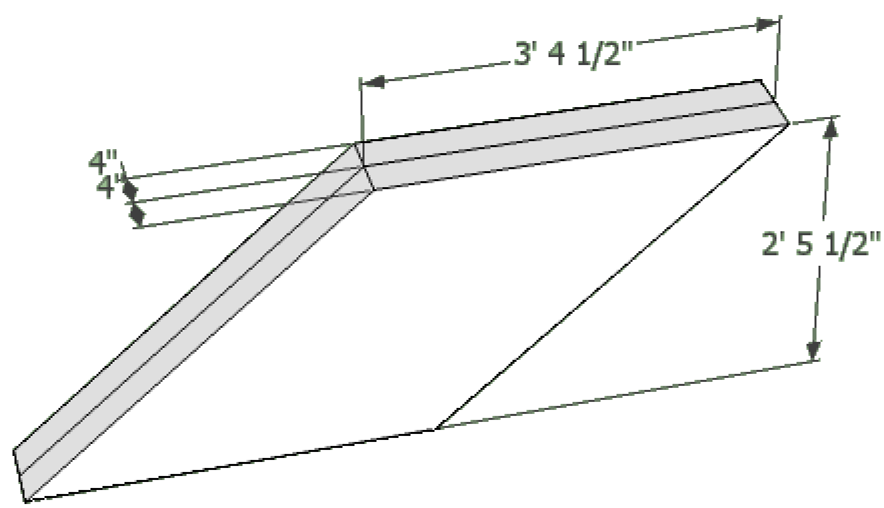

Figure 4.17 3D graphic and dimensions of the foam block-out

The 8-in. thickness of the block-outs was divided into two 4 in. thick pieces. All the pieces shown in the pictures are only 4 in. thick. 4'x8' boards of foam were cut into the required shape and size. Rectangular pieces were first cut then diagonally cut again as shown in Figure 4.18.

Figure 4.18 The Styrofoam rectangular pieces with the line representing the diagonal cut

Figure 4.19 Two pieces of foam put together after being cut diagonally

0.75 in. x 0.75 in. grooves were removed from the edges of the Styrofoam pieces shown in Figure 4.19. To ease the foam removal from the concrete web, plastic sheets were wrapped around the edges of the foam. Figure 4.20 clearly shows the grooves and the plastic sheets in the foam pieces. Assistive lines were drawn on the steel forms prior to gluing the foam so as to facilitate the measuring process for locating the foam block-outs. All the foam pieces glued on the steel form are represented in Figure 4.21. The other half of the block-outs were planned to be glued on the other side of the form. To avoid errors in the location of the foam pieces due to slight differences in measurements between the two form sides, the other half of the form was decided to be glued on the already existing foam pieces after attaching the rebar.

Figure 4.20 The groove and the plastics sheet on the block-outs

Figure 4.21 The Styrofoam block-outs after being glued on the steel form

4.3.1.3 Phase 3: Reinforcements layout

To properly test the ease of fabrication of the truss, the same amounts of rebar was to be used. Attaching the rebar was the challenging and most time and effort consuming phase in the construction. The diagonal members had 4 #6 bars and #3 ties spaced at 8 in. along the member. The vertical members were 1½ in. diameter threaded rod as mentioned before in the designs section. The rod was anchored in the top and bottom flange by means of a ½ in. thick 8 in. x 8 in. Gr 50 steel plate and a structural nut tacked on to the plate. The first plan was to have the reinforcements for each diagonal preassembled and then connected together in the form by the mean of the threaded rod.

The diagonal reinforcements were tied together and the plates were welded in place with the nut for the threaded rod tacked only on the plates that were to be located at the bottom flange. The diagonals assembly had plates welded on the short ends. 2 #6 bars were also welded 5 in. center to center on the other side of the plate. To avoid out-of-plan bending, 2 4-in. #6 bars pieces were welded on the steel plate perpendicular to and between the outside welded bars.

The main problem that this plan caused was due to irregularity in the bars dimensions, as the bars received from the supplier were 3 in. shorter along the diagonal part than the aforementioned figure. One other disadvantage of this system was that it was very heavy; hence, very hard to deal with and move to account for some tolerances. Two actions were taken to deal with these problems. The first thing was to straighten the bars at the longer ends, and then cut the bars to make them short. The long bars welded to the steel plates were also shortened from 2.5 ft to 8 in. long past the edge of the plate. This fixed the dimension problem as well as made the reinforcements assembly more lightweight. To account for the errors in dimension that could have occurred due to changing the rebar geometry, the reinforcing ties were untied to make the bars slide freely against each other.

The changes that were done made the fabrication process very easy. The assembly can be seen in Figure 4.22.. These reinforcing bars were the most challenging part of the fabrication due to the interference between the bars from the verticals and those supporting the diagonals. Four 4 in. x 4 in W2 meshes were put at each end of the truss as shear reinforcements. Figure 4.23 through Figure 4.24 are different views of the final stage in the reinforcement fabrication for the diagonal and vertical members.

Figure 4.22 The diagonal reinforcement assembly

Figure 4.23 All the reinforcements after being fixed in the form

Figure 4.24 Full span of the specimen before closing the form

The top flange was designed for 4 #4 bars for the full 30-foot span specimen. Due to the time contains, 2 #8 bars were used instead as they were already in the lab. The top flange reinforcements were done and tied together with the stirrups as in Figure 4.25. After the form was closed, the bars with the stirrups sat on 2 in. high chairs on the steel form. The hats for the stirrups were not added as the supplied ones had a smaller angel than the truss was designed for; therefore, the hats were not used for the stirrups as they did not fit in the form. After the form was closed, 2x4 timber ties were bolted to brace the steel forms against opening. The prestressing strands were then pretensioned to 202.5 ksi (0.75 ultimate stress of 270 ksi). Final elongation of the strands was 6 in. Figure 4.26 shows the truss formwork right before casting the truss.

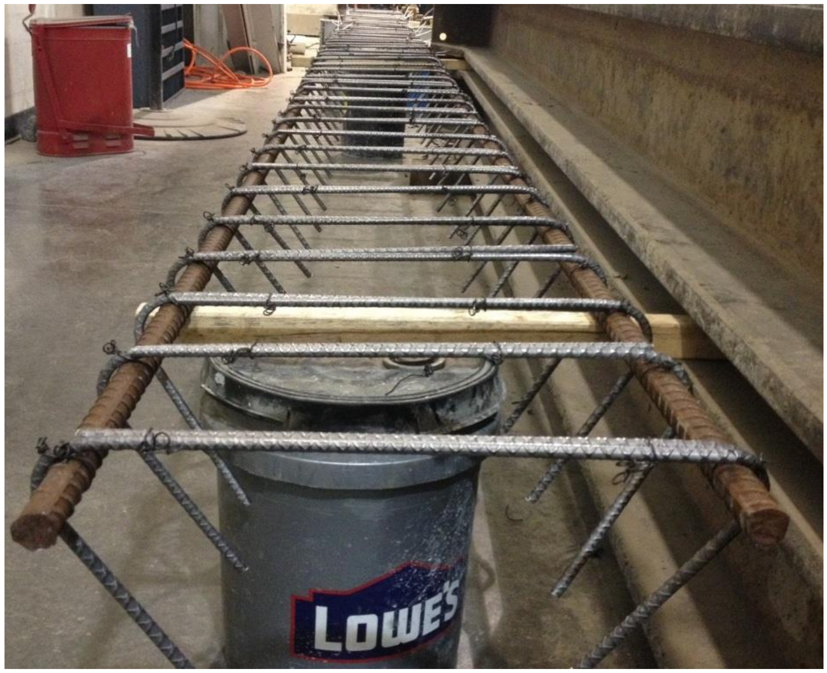

Figure 4.25 The top flange reinforcements after assembly

Figure 4.26 The form after adding the top flange reinforcements and adding ties at the top

4.3.1.4 Phase 4: Casting the truss

Self-consolidating concrete was delivered on March 11th, 2013 to the structural lab. The mix was made with 3/8 in. maximum nominal size aggregates. The concrete had a spread test of 28 in. (Figure 4.27). The concrete was patched and ordered for a 28-day strength of 8,000 psi. Cylinders were casted for strength testing at the time of stripping and testing.

Figure 4.27 Self-consolidating concrete spread test

Pouring the concrete started at the middle vertical as shown in Figure 4.28. Two snake cameras were attached at the bottom of the truss, one at each end. Another camera was recording the pouring process from the outside. The main purpose of the snake cameras was to test the flowability of concrete through the bottom flange at the locations of the vertical and diagonal embedment. The concrete mix passed successfully through the bottom flange. Figure 4.29 shows the concrete flowing from the location of the pour at the middle vertical to the location of the camera at the south bottom end of the specimen.

Figure 4.28 Pouring the concrete through the middle vertical member

Figure 4.29 Snapshot from the south end recording while pouring

No issues were faced till the bottom flange was completely filled. However, as soon as the concrete started to go up the diagonals and the web, the buoyant forces on the Styrofoam block-outs was extensively high. The glue attaching the block-outs to the form was not strong enough to hold them in place, and not very long after the concrete started to go above the lower bottom of the block-outs, the foam was detached from its place and floated. The buoyant force was very high that prying the foam down was not effective.

2x4 timber pieces were but on top of the foam as spacers between the foam and the steel plates that were welded to the diagonals, and between the foam and the top flange steel reinforcements. Figure 4.30 shows how the block-outs were held by means of the top flange reinforcements. The weight of the steel reinforcement was not enough to hold the foam in place. Timber spacers were slid between the reinforcements and the ties holding the steel form (as shown in Figure 4.31) to prevent them from floating. Foam has proved that it is not the efficient or suitable material for the block-outs and alternatives had to be suggested. Alternatives are discussed in the conclusion section.

Figure 4.30 Holding down the block-outs

Figure 4.31 Holding down the steel reinforcements

After the pouring was complete, the specimen was covered and left to gain its strength. The concrete cylinders were kept in the curing room to be tested later at the time of stripping and testing.

4.3.1.5 Phase 5: Stripping the formwork

The form was stripped and strands were released at a 3-day strength of 7,800 psi. The steel forms were easily removed away from the truss. The foam, however, was embedded in the concrete and did not come out as one piece as planned before construction. The pressure of the concrete on the form led to the form being widened by an average increase of 0.5 in. for a maximum widening of 1 in. at the northern end and no widening at the southern one. That increase in thickness caused the web to be wider than the Styrofoam at some locations, which increased the difficulty of removing the block-outs (See Figure 4.32).

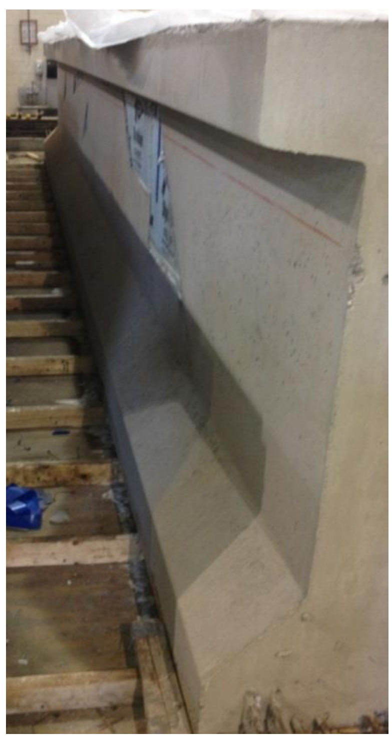

Figure 4.32 The side of the truss after moving the steel form (View is South to North)

Removing the Styrofoam block-outs was a more challenging task than what was planned. Concrete shrinkage enhanced their positioning inside the concrete. The web widening led to having some pieces covered from both side with concrete layers. The concrete layers needed to be chipped out first as in Figure 4.33.

Figure 4.33 Chipping out the concrete layers covering the block-outs

After extensive hammering and pushing on the block-outs to try to take them out of the concrete in one piece, the foam was cut using an electric saw shown in Figure 4.34 to smaller pieces and these pieces were then hammered out.

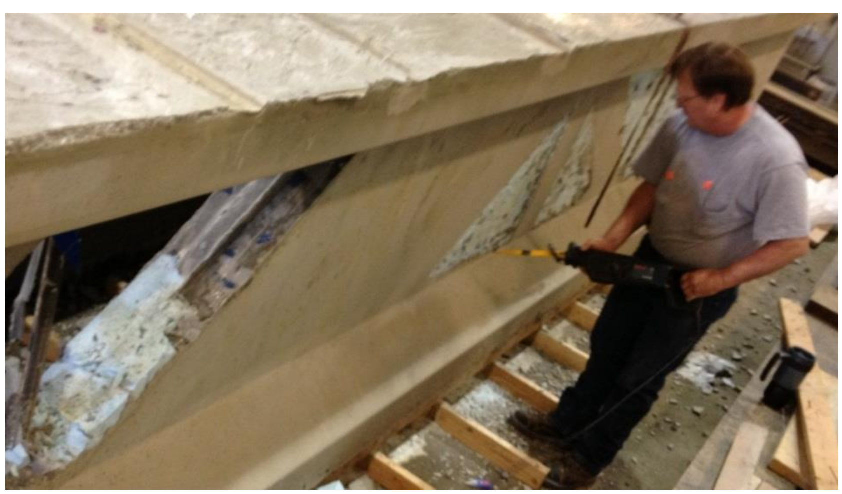

Figure 4.34 Cutting through the block-outs into smaller pieces to remove them

Even though the plastic covering on the edges of the block-outs was not as helpful as expected in the removal process, the triangular pieces at the middle were more challenging to remove. A small layer of the edges of block-outs in the middle could not be removed from the concrete. The difference in the finish between the middle block-outs and the rest of the truss can be clearly seen in Figure 4.35. A chipping hammer was used as in Figure 4.36 to remove as much as possible without risking damage the concrete

Figure 4.35 The middle of the truss after stripping the forms

Figure 4.36 Chipping out of the Styrofoam pieces that were bonded to the concrete

The flotation and movement of the block-outs during the casting process caused some irregularity in the shape of the truss. Two diagonal members were widened and the other two became thinner, along the length of the truss. The block-out locations were shifted as well with the movement of the Styrofoam. The two vertical members at each end were slightly rotated. A sample of the irregularity that happened during the fabrication process is shown in Figure 4.37.

Figure 4.37 Some irregularity that occurred in the truss diagonal and vertical members and locations of block-outs

The strands were released by means of the prestressing jack mentioned in section 4.3.1. End zone cracking occurred at both ends, as the end zone reinforcements needed to be closer to the ends. Cracking at the south side shown in Figure 4.38 was a little more extensive than the north side cracking shown in Figure 4.39. Very slight cracking occurred at the south end side of the web. No camber occurred due to prestressing.

Figure 4.38 End zone cracking at the south end

Figure 4.39 End zone cracking at the north end

Some shrinkage cracks occurred in the top flange, some of which could be barely seen, others were very extensive. Most of the big cracks occurred at the locations were the wood holding the Styrofoam block-outs was left. The shrinkage cracking were believed to not be crucial when testing the truss as they occurred only in the top flange, which is a compression member. One of the biggest cracks can be seen in Figure 4.40.

Figure 4.40 Shrinkage cracking where a 2x4 timber piece was left

4.3.2 Lessons Learned

The truss specimen fabrication was very challenging during the starting period. According to the reinforcing steel assembly experience, it was found that the assembly, as specified in the shop drawings, was very heavy to handle and control. The weight of the assembly was not the only issue; the length of the diagonal reinforcements was also very big and caused many movement restrictions.

Assembling the reinforcements beforehand and then attaching it to the whole form looked as one of many ways to ease fabrication. However, this research found that the assembly when put together and tied very hard made it hard to align the verticals together. Due to the difficulty to add any tolerances to the assembly after tying them together, it has been advised that the ties should only be tied to one side of the diagonals, if tied at all.

One other thing that was learned through the pouring process is that foam and concrete do not mix. As mentioned before the foam block-outs floated in the concrete and the glue was not strong enough to resist the buoyant forces on the block-outs, especially due to the size of the foam combined with the depth of the beam. Even if a pre-caster was able to find a way to glue the foam to the steel using a strong enough gluing material, stripping would not be easy, and they might as well risk losing the foam and breaking it.

The reason why stripping could be challenging when using foam is that the concrete shrinks on the foam when it is hardened. That shrinkage leads to having the concrete holding on to the foam pieces. The presence of foam will also make it hard to have chairs or supports for the reinforcements to keep cover as foam is compressible. When considering other materials to be used, wood would not be a suitable material. Despite the fact that wood would not be compressed like foam, wood will have the same buoyance problem the foam had. Solutions for these problems are discussed in the conclusion chapter.

4.4 SPECIMEN TESTING

4.4.1 Test Setup

After finishing the stripping step, the test setup was prepared. Two 3 ft high concrete blocks were placed 29.5 ft apart center to center. A steel frame with a loading jack mounted on it was placed midway between the two blocks. The truss was put through the frame resting on the concrete blocks. Steel rollers were placed between the truss and the concrete blocks. The rollers were centered on the 6 in. bearing plates that were imbedded in the truss at the ends. The loading jack (with a 400-kip capacity) was centered on the middle vertical, which in turn the midspan of the truss. The test setup is shown in Figure 4.41.

Figure 4.41 The truss testing setup

To properly interpret the test results, the western truss face was painted in white. The paint was very helpful in tracking any cracks that occurred during loading. Strain gages were also attached to the steel rods, the concrete members, and the top and bottom flanges (at the midspan). On one hand, the steel rods were grinded so as to remove the threads and make a smooth surface for the strain gage. On the other hand, the concrete strain gages were all placed on the non-painted side of the truss. In both cases, the strain gages were glued on the steel and concrete using Superglue.

After soldering wires to the strain gages, the wires were soldered to stranded wires that were connected on the other end to an Optim Megadac data acquisition system. To avoid the strain gages getting torn or pulled out of place, the wires were securely tapped to the members at the end that was soldered. Slippage monitors were placed on one center strand to measure for any slippage during testing. The slippage monitors were also connected to the Optim Megadac. The Megadac at the Peter Kiewit Institute has the ability to interpret data on 16 channels.

A deflection gage was also installed to measure any deflection that occurred during loading. An angle was embedded in the bottom of the truss at the midspan. A fishing line was then connected to the gage. For the loading jack, rubber pads were placed on top of the truss at the midspan to make a smooth surface for a uniform loading. On top of the pad, a 2-in steel plate was placed then a load cell centered on the plate. The loading cell is used to measure the amount of load applied by the jack. Another 3 plates were then placed between the loading jack and the load cell.

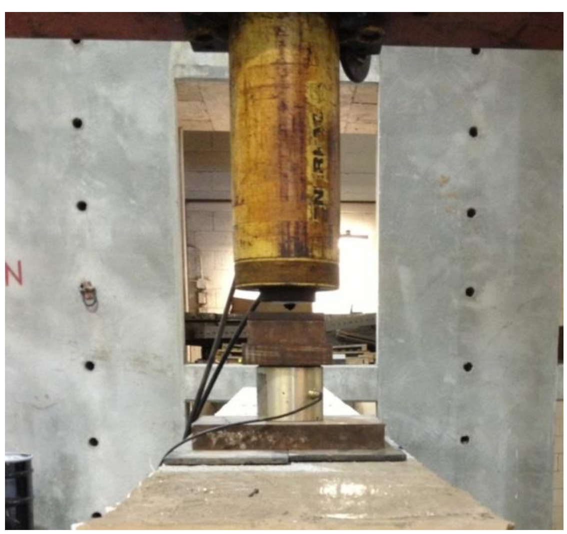

Figure 4.42 The plates' organization and the load cell at the point of loading

The gages were checked and balanced and the testing was ready to start. The specimen with the strain gages attached is shown in Figure 4.43. Cylinders were tested and the concrete strength was found to be 10,500 psi when the testing was done.

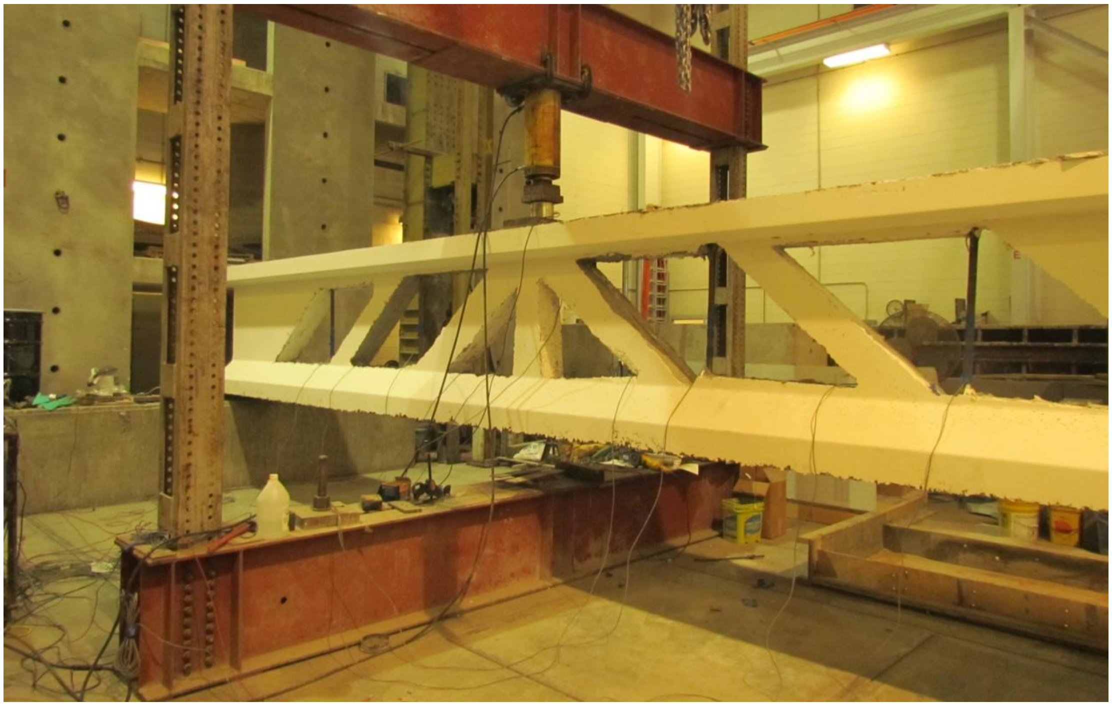

Figure 4.43 The truss specimen with the strain gages and cells attached

4.4.2 Loading

As testing started, every 50 kips of load applied the testing was paused and cracks (if any) were marked. After marking the loading continued, then after 50 more kips the same procedures were done till the 250-kip load mark. Then loading continued till 385 kips when the truss failed.

After the first 50 kips, the deflection had reached 0.19 in. There were no cracking at 50 kips loading point. The loading continued till 100 kips at 0.39 in deflection. At 100 kips, cracking started to show at the top of far northern diagonal connection with the top flange at the sharp angle side (Figure 4.44).

Figure 4.44 The only cracking that occurred at 100 kips of load.

At 150 kips, the deflection reached 0.57 in. Cracking at 150 kips started at the connection between the far north diagonal and the web (Figure 4.45). The middle vertical started to crack as well at the top and bottom from both sides (Figure 4.46). At 200 kips the cracking had affected all the sharp angle connections, especially at the web/diagonal interface (Figure 4.47). At 250 kips, most cracking severity did not increase than the 200-kip loading point except at the south diagonal/web joint where cracking increased. Despite the cracking, the deflection at 200 kips and 250 kips was 0.75 in and 0.93 in, respectively.

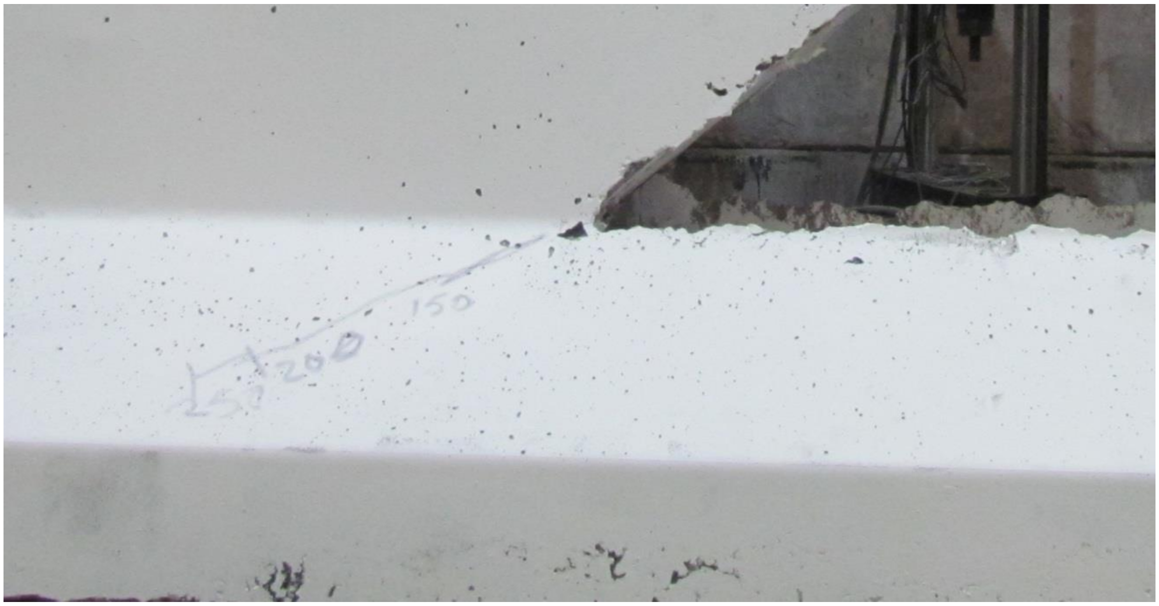

Figure 4.45 Cracking extents vs. loading points for the north diagonal to web conection

Figure 4.46 The middle vertical cracking at the 150 kip loading point

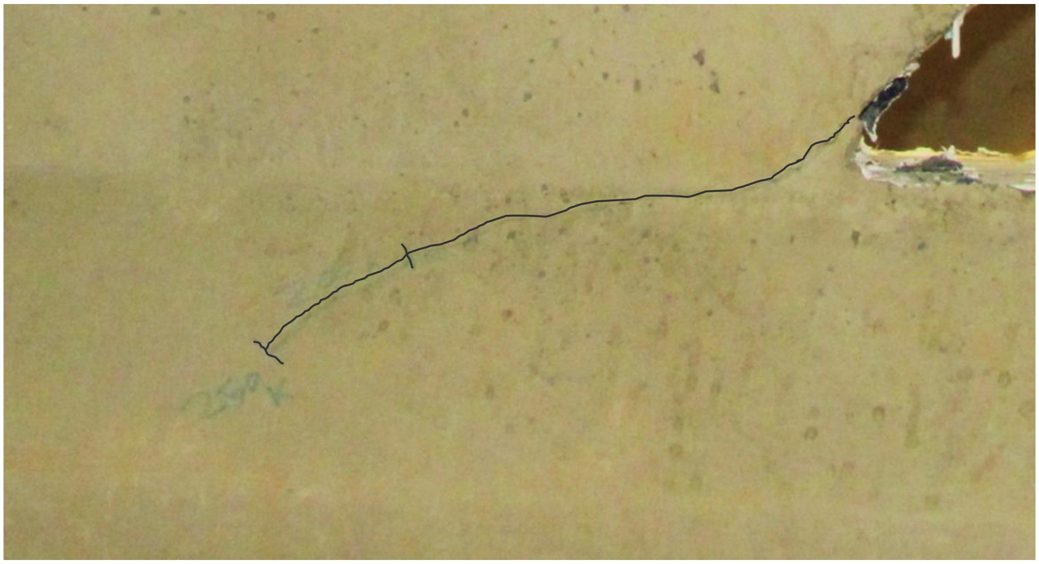

Figure 4.47 Cracking at the southern diagonal and web interface at the 200 kip and 250 kip loading points

After the 250-kip cracking investigation point, the load increased uniformly without interruptions with some excessive cracking at locations of the rods embedment in the bottom flange (Figure 4.48) till failure at 385 kips of loading. The failure was rather dramatic; however, it was not sudden as evident cracking was seen before the failure (Figure 4.49). The failure occurred at the far south steel rod where the embedded steel pulled out of the bottom flange and the rod snapped up causing the diagonal to snap with it and the top flange to buckle and fail (Figure 4.50). The failure of embedment occurred as one of the #6 short bars that were welded onto the plate sheared and the other one pulled-out (Figure 4.51).

Figure 4.48 Excessive cracking before failure

Figure 4.49 Excessive cracking right before and at location of failure

Figure 4.50 The moment of failure of the truss

Figure 4.51 The failure of the anchoring bars (one in shear while the other pullout)

The failure is believed to be associated with the fact that the stirrups hats were not placed and the shear reinforcing mesh was not extended all the way to the bottom of the flange to make up for the absence of the hats (Figure 4.52). The hats would have provided more confinement for the concrete, hence, more strength. Despite the fact that the loading did not achieve the minimum load 400 kips for design before failure, which means the members are not adequate to carry the axial loads for the full span truss-girder, the analysis of results had different findings.

Figure 4.52 Location of failure

4.4.3 Lessons Learned

Testing the structural capacity of the truss cleared a few potential problems that could occur. The block-outs were cut with sharp edges, unlike the detail in the shop tickets. These sharp edges caused the stress to be concentrated at these connections; hence, lead to extreme cracking and then failure. The other thing that accelerated the failure was the absence of the bottom flange stirrup hats.

4.5 ANALYSIS OF RESULTS

As the size of the concrete members changed, the forces expected in them changed. To get the correct force in the concrete members, the cross-sectional area needed to be known. Therefore, the dimensions of the members were taken and are shown in Figure 4.53. The top dimension is the depth of the member and the bottom one is the thickness. Also the sides are noted as south and north to facilitate the demonstration of the results.

The need for the 400-kip load was to assess the structural adequacy of the diagonal and vertical members and their embedment in the top and bottom flanges, as the analysis indicated that the same axial forces induced in the diagonal and vertical members of the full span truss-girder will be induced at such load.

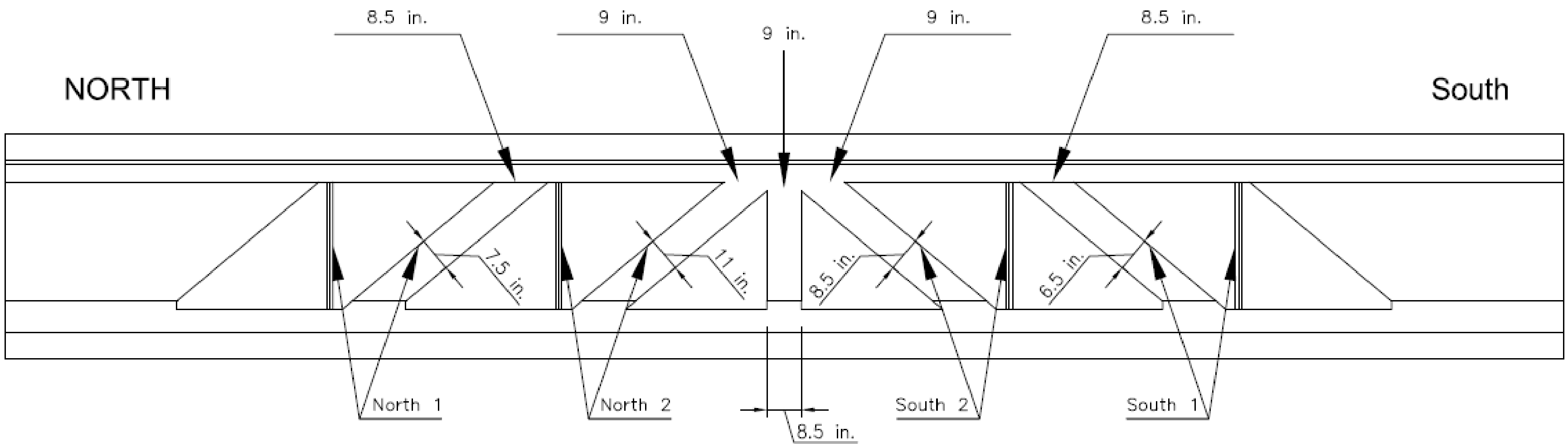

Figure 4.53 The concrete members dimensions after pouring

The design noted that cracking in the bottom flange would not occur till loading reaches 330 kips (300 kips in case of the FE approximated section model). Analysis of deflection curve in Figure 4.54 shows that cracking did not occur till the load reached about 355 kips. The fact that cracking did not occur as designed means that strand slippage did not occur, which is true according to the gages readings presented in Figure 4.55. The accuracy of the gage is to one thousandth, which means for any value less than 0.001, no change in strand location occurred.

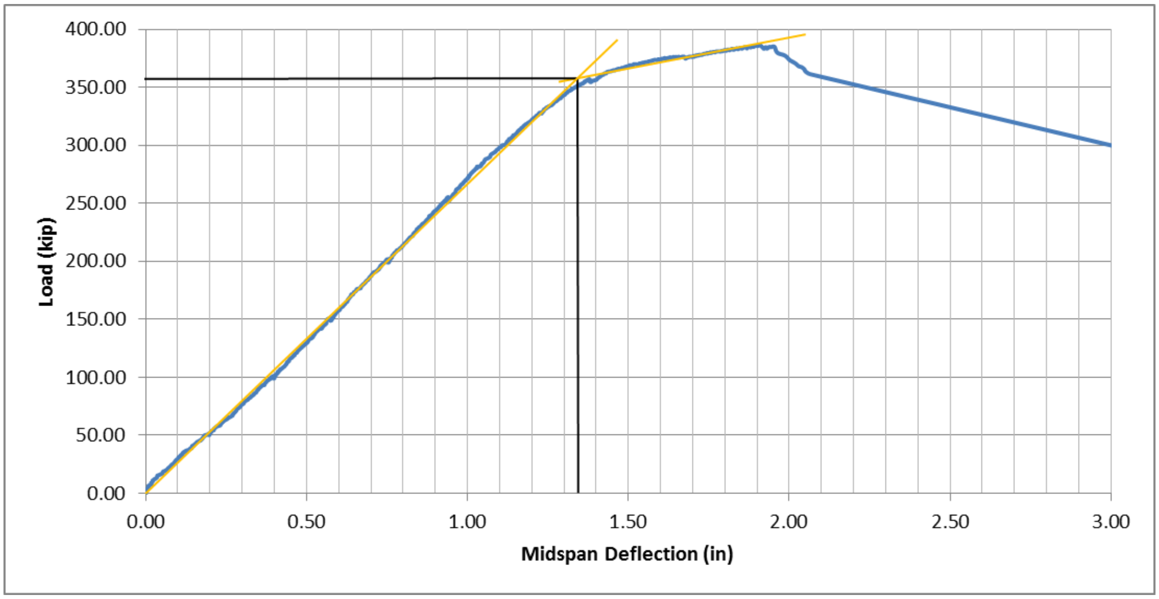

Figure 4.54 Load vs. deflection curve (black lines indicate end of elastic behavior)

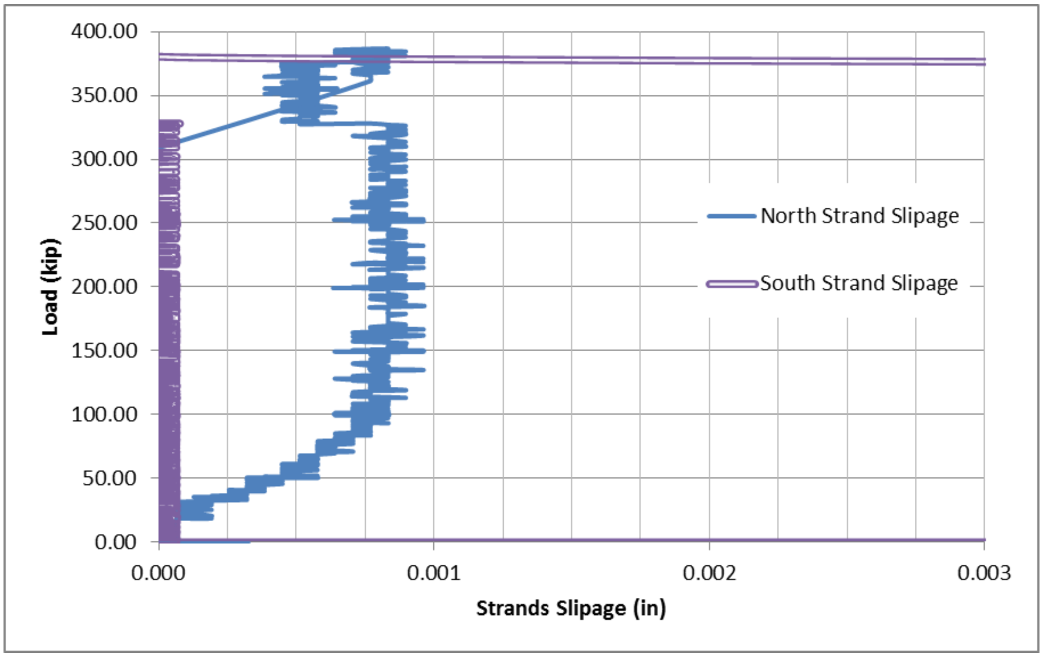

Figure 4.55 Strand Slippage

Due to the construction issues that occurred during pouring, the angles of the diagonals and verticals were slightly changed. This change resulted in different member axial forces than what the analysis projected. As a matter of fact, the members did not only meet, but also exceeded the design loads for the full span model. The strain and forces in the steel were very close, as they all were vertical with very slight differences. In Figure 4.56, the strain in the TR at the location of failure by far the highest, which explains why the failure occurred at that location.

All the rods reached their yield point with a force of 155 kips, which is more than the design force of 136 kips. The design load was achieved at a testing load of approximately 290 kips. The forces induced in the TR are presented in Figure 4.57. By analyzing the two plots, it can be notices that the south TRs yielded before the north ones, which indicates some shift in symmetry.

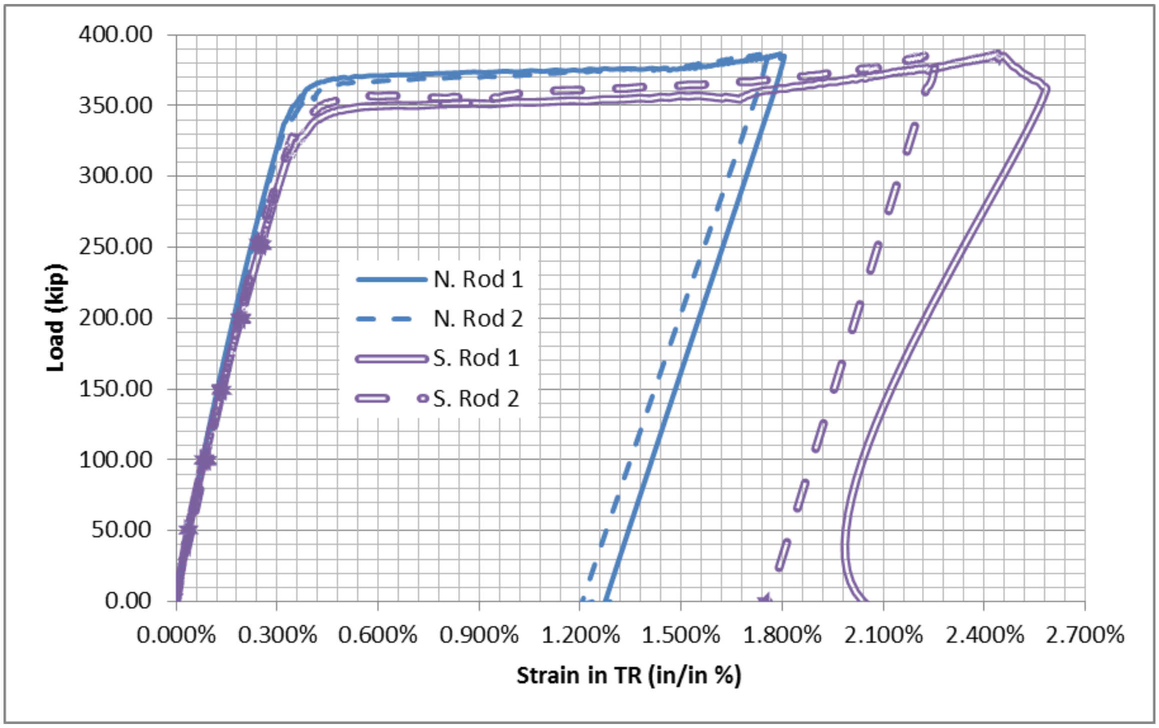

Figure 4.56 Strains in the Threaded Rods (TR) while loading (tension)

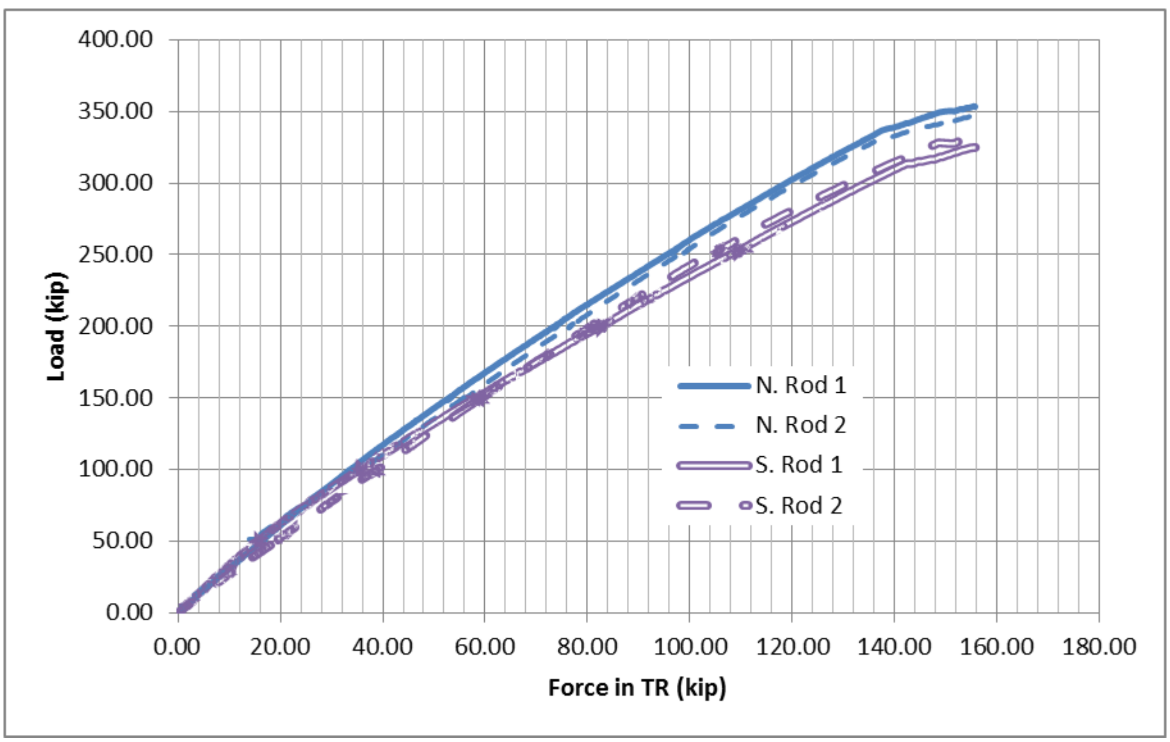

Figure 4.57 Forces induced in Threaded Rods (TR) while loading (tension)

Similarly, the forces that acted on the diagonals were much higher than the design loads, despite the lower specimen failure load. The strain plot (Figure 4.58) and forces induced (Figure 4.59) show that the diagonals exceeded the design load of 268 kips reaching an axial force of 325 kips. The diagonals, unlike the TRs, differed from each other in terms of strain and forces. The difference in the diagonal than the vertical members is that the angles varied from one to the other with the change in locations and rotation of the block-outs while pouring. This pouring problem did not only affect orientation of the diagonals, but also the size of the diagonals as mentioned earlier. The size change affected the relative stiffness, which in turn is believed to have caused the shift in symmetry.

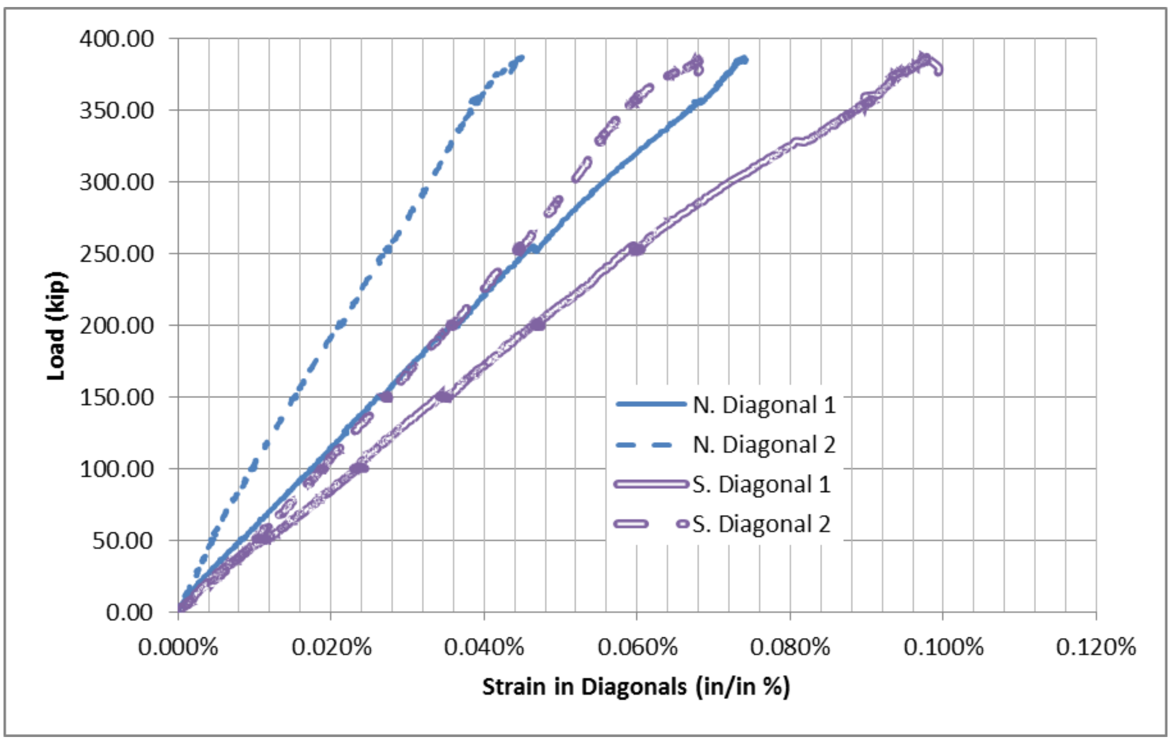

Figure 4.58 Strains in the diagonals while loading (compression)

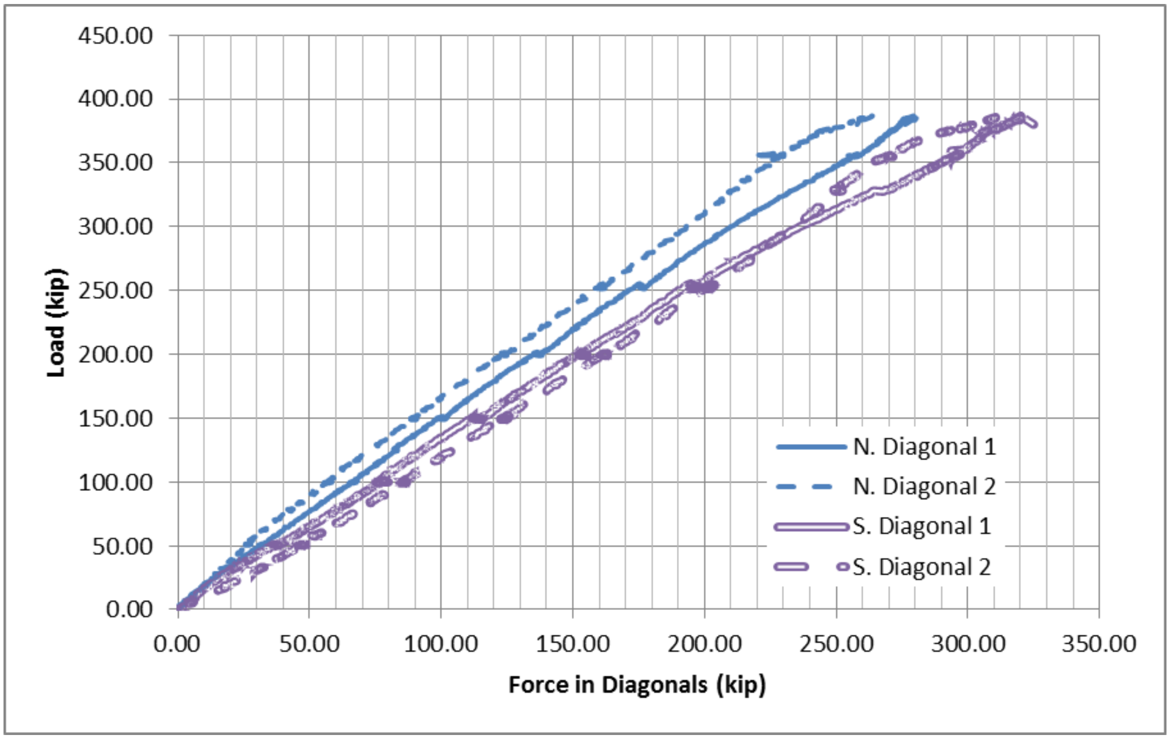

Figure 4.59 Forces induced in diagonals while loading (compression)

5.0 CONCLUSIONS & RESOMMENDATIONS

5.1 CONCLUSIONS

This research aimed at proposing a truss system that could be successfully built and analyzed to standard. There were three outcomes to that research. The outcomes were:

- The ability to fabricate a truss system made from reinforced/prestressed concrete members
- Developing the system to have a maximum span of 160 ft with a span-to-depth ratio of about 27
- Analyzing the truss system correctly and having the analysis result to match the actual behavior of the system.

First, fabrication of the truss was the main questionable aspect of a having it made from concrete. Through the research done, a fabrication method was introduced and developed to prove that a truss can be fabricated from concrete. The fabrication was made possible through the block-out system proposed and recommended later on in this section. One key material in the fabrication is the Self-Consolidating Concrete (SCC). Using SCC was successful in getting concrete to fill all the gaps in the truss.

Second, it can be concluded the truss-girder system can extend to a span of 160 ft. Despite the fact that one truss piece is only 80 ft long, the span of 160 ft could be achieved with the post-tensioning proposed in the design section. The materials used for designing the truss are commercially available nationwide and can be used with, whether it was the SCC, B7 steel rods, or the 0.6-in low relaxation strands.

Finally, the presence of an analytical model that matches actual behavior while testing was crucial to have. The analytical model developed, whether it is the frame or the shell model, was able to accurately predict and calculate the member forces in the truss-girder system under given loads. All in all, the research was able to provide a reliable system that can be built and developed with accordance to the building code, while achieving the economy, sustainability, and structural integrity of the system. The following section are some solutions to the issues faced by the research team.

5.2 RECOMMENDATIONS

It has been found through testing that cracking occurred at the sharp angles at the diagonals anchoring with the top and bottom flanges. As a result, having chamfered ends instead of the sharp ones, as described in the full size truss-girder construction section 3.3, is highly recommended to avoid stress concentrations and cracking. Another option is to have smooth curved corners instead of the sharp ones

Another recommendation for the design is to avoid unnecessary reinforcement to decrease the weight. It has been found that the TR anchoring mechanism provided was able to sustain the design load efficiently with only 8 in anchoring bars welded to the steel plate.

As the construction of the truss was a major part of the study, many practices are highly recommended. According to the research done, the use of Styrofoam block-outs can lead to destructive results. Even if gluing the foam to the steel was preformed successfully, the stripping step could waste time, effort, and money. This issue can be easily addressed by welding steel pans (which can be light gage one) to the forms. This practice will increase the efficiency of pouring, and allow multiple reuse of the same form, unlike the foam that could be easily broken while stripping.

The weight of the reinforcing steel is another important issue. Assembling the rebar completely and then attaching it as one piece to the form can be a tedious job. Even if the measurement were done very precisely, the handling of the system can waste time and effort. As an alternative, the diagonals can be tied together while in the form after they have been screwed in with the vertical threaded rods (as shown in Figure 5.1). This practice will allow the diagonals to slide against each other for some tolerances. It is important to coordinate with the designer to achieve the optimum and minimum amount of steel needed for development as this is the best way to achieve maximum efficiency.

Another suggestion for fabrication is to assemble all the reinforcements outside the form with precise measurement. After the assembly is tied together, the steel can be lifted by means of a crane and then placed in the form. Given that measurements are precise enough and materials provided has low dimensions tolerances, this method could be the easiest among other options.

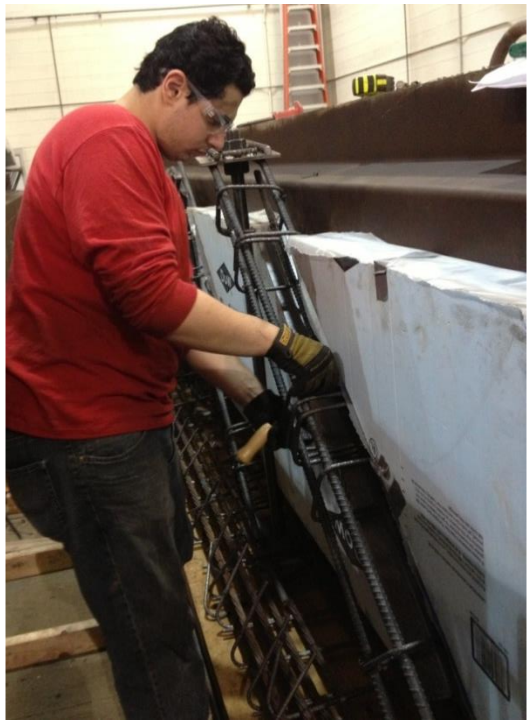

Figure 5.1 Tying the diagonal member reinforcements together after attching the vertical

6.0 REFERENCES

ACI 318. *Building Code Requirments for Structural Concrete (ACI 318-11) and Commentary.* Farmington Hills: American Concrete Institute, 2011.

AISC. *Steel Construction Manual.* 14th. Chicago: American Institute of Steel Construction, 2012.

ASCE/SEI 7. *Minimum Design Loads for Buildings and Other Structures.* Reston, VA: American Society of Civil Engineers, 2010.

Carroll, William T., et al. "Prestressed Concrete Trusses." *ACI Journal*, August 1978: 367-373.

DeSutter, Michael. United States of America Patent 7,275,348 B2. 2007.

New York State Department of Transportation. *Office of Construction: Fuel, Asphalt, & Steel Price Adjustments.* March 22, 2013. https://www.dot.ny.gov/main/business-center/contractors/construction-division/fuel-asphalt-steel-price-adjustments (accessed March 2013).

PCI. "The 50 Most Significant Precast Concrete Projects." *ASCENT Magazine*, Winter 2004: 10-31.

—. *PCI Design Hnadbook.* 7th. Chicago: Precast/Prestressed Concrete Institute, 2010.

—. *Precast Prestressed Concrete Bridge Design Manual.* 2nd. Chicago: PCI, 2003.

Shah, Pradeep H., and Howard R. May. "Precast Vierendeel Trusses Provide Unique Structural Facade for Parking Structure." *PCI Journal*, July-August 1977: 24-39.

Trygestad, Amy, and Mike DeSutter. "Bookmen Stacks, Cobalt Condos Use ER-POST for Column-Free Space." *PCI Journal*, May-June 2007: 58-71.

TSA Manufacturing. *TSA Technical Information.* September 21, 2012. www.tsamfgomaha.biz/index.php?page=TechInfo#rndBrStkWt.