PRECAST PRESTRESSED CONCRETE TRUSS-GIRDER FOR ROOF APPLICATIONS

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ABSTRACT

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 truss design consists of two segments that are formed using standard bridge girder forms with block-outs in the web to form diagonals and verticals. The two segments are then connected using a wet joint and post-tensioned longitudinally. The truss was analyzed and designed using the strut and tie model. A 30-foot long truss specimen was constructed at the structural laboratory of University of Nebraska-Lincoln to investigate the constructability and the structural performance of the proposed system. Testing results indicated the production and structural efficiency of the system.

Keywords: Truss, Warehouse, Hangars, Roof, Post-tension

INTRODUCTION

Design criteria for long-span roofs include structural integrity during erection and at service, cost-effectiveness, speed of construction, aesthetic appearance, and fire resistance. Structural steel is widely used for long-span roof applications, such as warehouses and airplanes hangars. The ease of handling and erection, geometric flexibility, and lightweight of the structural steel are the main advantages. However, structural steel has some disadvantages, such as high initial and maintenance cost and low fire resistance. On the other hand, precast concrete has low initial and maintenance cost and high fire-resistance in addition to speed of construction. However, existing precast concrete sections for roof applications cannot span much over 100 ft in addition to being heavy in weight. For example, the longest span for a precast double tee (8DT32) is 102 ft for 30 psf superimposed service load. The weight of such section is approximately 74 psf [1].

The objective of this research is to develop a precast/prestressed concrete truss for long span roofs (ranging from 100 ft to 160 ft). The main characteristics of the developed system are:

- Light in weight
- Fabricated using existing forms, materials, and production techniques. (economical to fabricate)
- Aesthetically pleasing
- Economical and easy to erect.

LITERATURE REVIEW

In 1976, Rock Island Parking structure was built using Vierendeel trusses that consisted of rigid joints and no diagonals [2]. The trusses were almost 12 ft deep and had a clear span of 32 ft. In 1978, W. Carroll, F. Beaufait, and R. Bryan published an ACI Journal article titled “Prestressed Concrete Trusses”. Two prototypes for the trusses were discussed, one with a clear span of 20 ft 4 in. and a depth of 2 ft, for a span-to-depth ratio of 10, and the other had a clear span of 60 ft 10 in. and a depth of 8 ft 6 in. for a span-to-depth ratio of 7 [3]. Fig. 1 and Fig. 2 show the two prototypes of the two prototypes.

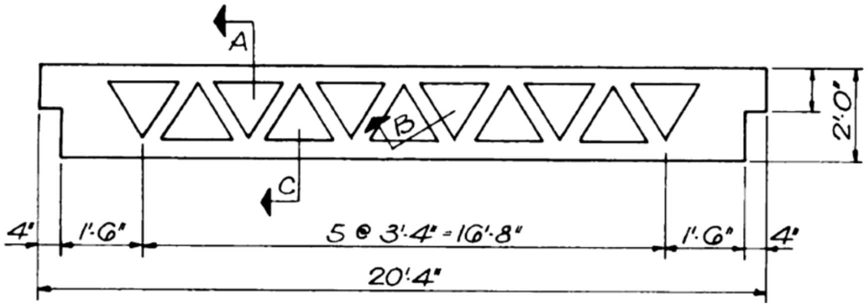

Fig. 1 Concrete truss prototype-I [3]

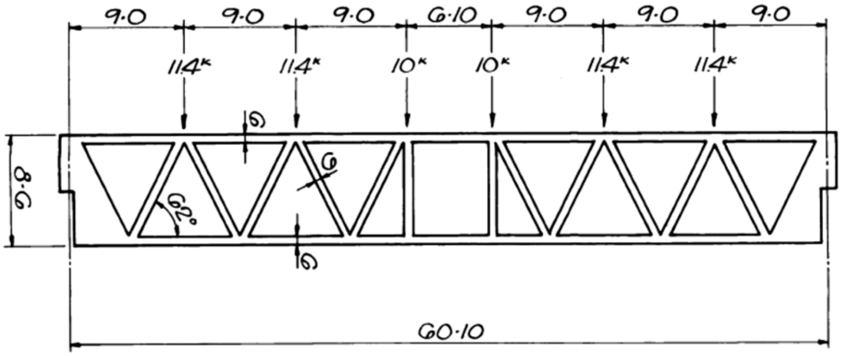

Fig. 2 Concrete truss prototype-II [3]

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" [4]. The ER-Post is a system developed by M. DeSutter of Erickson Roed & Associates and built by Kerkstra Precast. The purpose of the system was to provide a column-free space for the condominiums (Fig. 3). DeSutter was able to merge Vierendeel trusses with pretensioning to design the precast/prestressed trusses used [5]. With a depth of 13.5 ft, the trusses could span 67.33 ft yields a span-to-depth ratio of 5.

Fig. 3 Erection of ER-POST trusses [6]

In 2010, a precast concrete truss-girder was used to support the roof of a storage facility in Sharjah, United Arab Emirates (UAE). Designed by e.Construct USA, LLC, the 5-foot deep truss had a 165-ft span without intermediate supports. Fig. 4 shows the trusses during erection. A full truss consists of two halves, 82.5 ft long each. The two pieces are post-tensioned together forming one full 165-ft span. The trusses are arranged to be 30 ft apart. The trusses had diagonals and verticals in tension and compression, respectively. Span to depth ratio was 33.

Fig. 4 Concrete trusses resting on the temporary supports before post-tensioning (Courtesy of e.Construct, USA, LLC)

SYSTEM DEVELOPMENT

The precast concrete truss system proposed in this study is an evolution of the UAE system presented earlier. The main differences between the two systems include: 1) using steel threaded rods for all the verticals (tension members) and reinforced concrete diagonals (compression members); 2) using forms of typical precast/prestressed bridge girders, such as AASHTO and bulb tee girders, with block-outs to create the truss openings, which reduce the truss weight by at least 27%; 3) using self-consolidating concrete (SCC) to ensure the quality and economy of truss fabrication; and 4) place post-tensioning ducts in the bottom flange only, which eliminates the need for thicker web at the girder ends. Fig. 5 shows the layout of an example building for which the proposed system is designed.

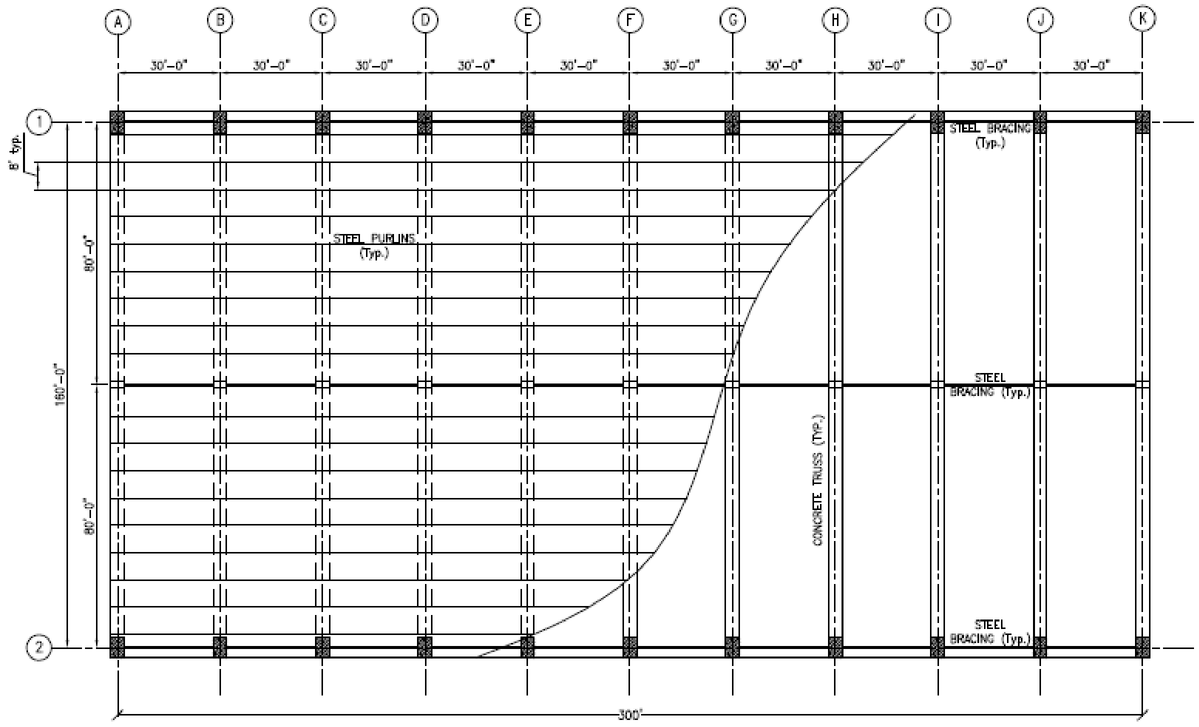

Fig. 5 Plan view of the proposed building layout

According to ASCE 7-10, a snow load of 30 psf and a load of 15 psf is assumed for mechanical, electrical, and pluming (MEP) utilities, roof purlins, and corrugated metal sheet are used in roof design. Load combinations of 1.4D and 1.2D + 1.6S are considered in the analysis during construction and at service respectively [7]. Figure 6 shows the shape and dimensions of one half of the truss. Structural analysis is performed using SAP2000 and Midas Civil structural analysis software. Solid web is provided at the truss end to adequately resist shearing forces. Figure 7 shows the cross section of the bulb tee girder used for designing the truss. The depth of the truss is 72 in., which yields a span-to-depth ratio of 27. The truss is prestressed using 10-0.6 in. grade 270 low relaxation strands and post-tensioned using 2 ducts, each with 12-0.6 in. strands.

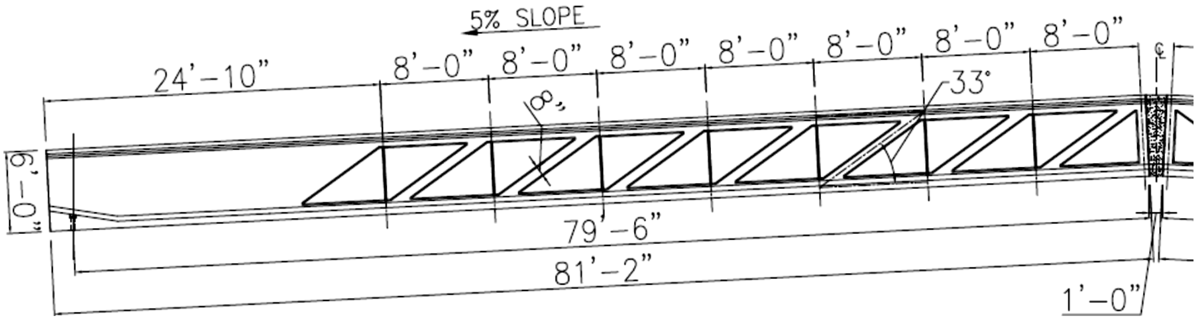

Fig. 6 Elevation of half of the truss

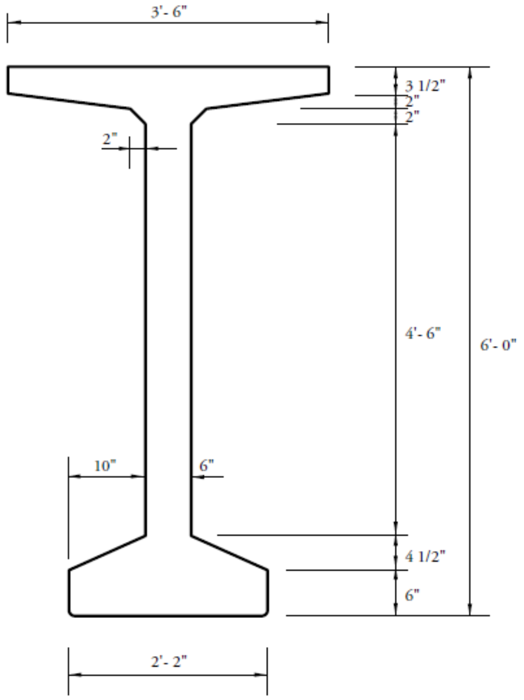

Fig. 7 AASHTO-PCI BT-72 girder cross-section [8]

Table 1 summarizes the analysis results of the truss at different loading cases. Table 2 shows deflections due to different loads and the net deflection. The deflection due to superimposed service load is 8.3 in., which satisfies the L/240 limit [9].

Table 1 Analysis results of the example truss

	Construction	**Service**	**Ultimate**
Top (kips)	-148	-1247	-1683
Bottom (kips)	148	1258	1698
Vertical (kips)	13, -36	101	136
Diagonal (kips)	-28, 61	-199	-268
Tensile forces are positive, compressive forces are negative			

Table 2 Deflections at different stages of loading

Type of load	**Deflection (in)***
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

The proposed concrete truss is designed using the strut and tie model presented in Appendix A in the ACI 318-11 code. Diagonals are designed as reinforced concrete struts, while the verticals are designed as 1½ in. diameter threaded rod ties made of B7 105 ksi steel, F_y= 105 ksi and F_u=125 ksi. Some rods are subjected to compression during construction and are checked against buckling under construction loads [11]. Diagonals are designed as 8 in. square struts made of 8,000 psi concrete and reinforced with 4#6 grade 60 steel bars. Even though only 4#4 bars are required near midspan, the 4#6 bars are used to control cracking during construction.

The proposed method of fabrication is using block-outs to make the openings in the web. The block-outs can be made using steel, wood, or foam that is glued to the steel forms. The post-tensioning ducts are designed to be 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 at these ends). Fig. 8 shows the duct profile and the end block is shown in Fig. .

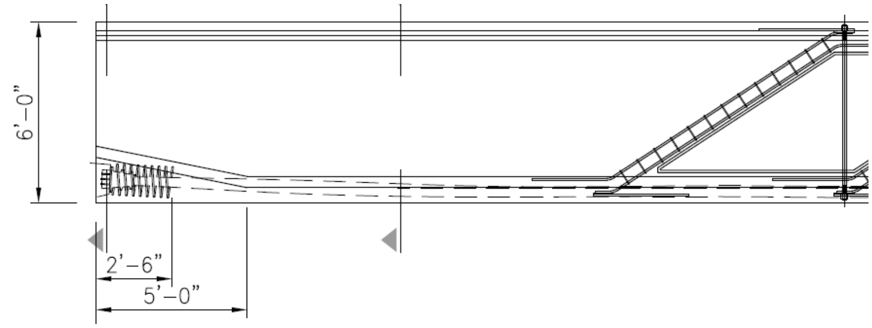

Fig. 8 Profile of the post-tensioning ducts

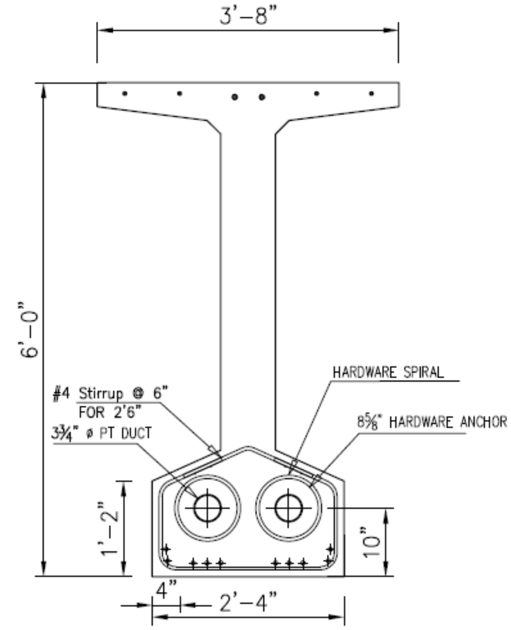

Fig. 9 The suggested end block for the post tensioning ducts

The production of the truss is greatly facilitated using pre-assembled reinforcing steel cage welded to the plates used as washers to anchor the threaded rods as shown in Figure 10. These plates have welded anchoring bars to prevent their pullout from concrete. The construction sequence is suggested to be as follows:

- After fabricating the halves of each truss at the plant, they are transported to the site.
- Trusses are erected and supported on columns from one side and on temporary supports at midspan.
- The wet joint is then formed, reinforced, cast in place.
- Post-tensioning is fully applied to each truss.
- Temporary supports are removed and the bracings are added.
- Light gage purlins used to support the roofing material are installed.
- Mechanical, electrical, and pluming components are installed as well as the corrugated metal roof.
- Threaded rods are sprayed for rust resistance and fire-proofing if needed.

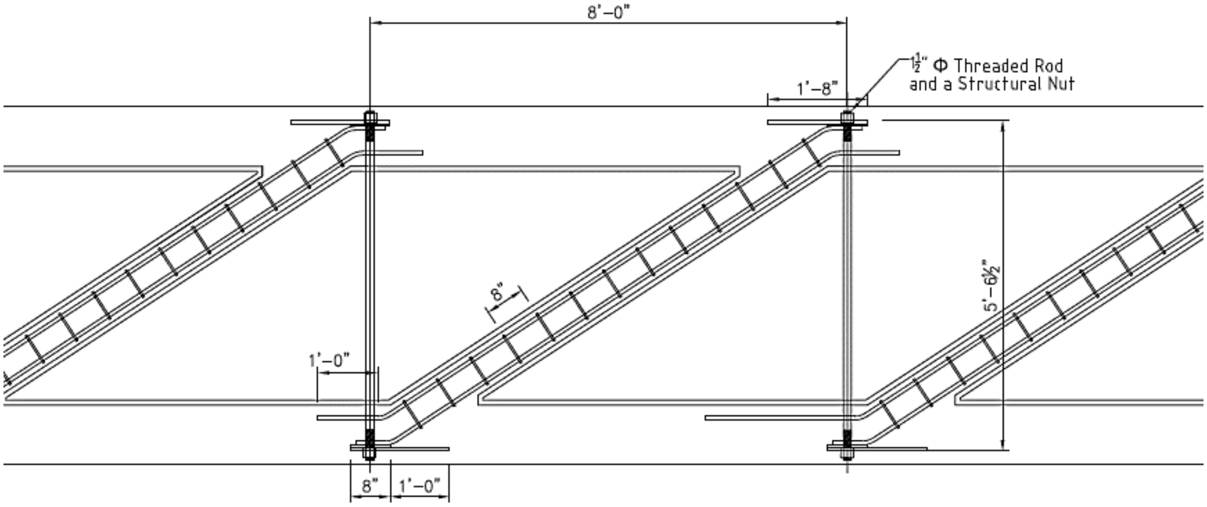

Fig. 80 Diagonal and vertical reinforcements' assembly

EXPERIMENTAL INVESTIGATION

The purpose of the experimental work is to evaluate the constructability and structural adequacy of the proposed truss. The specimen was formed using a 30 ft long Iowa type D bridge girder form, shown in Figure 11, because of its availability to a local precast producer. The form was shipped to the structural laboratory in Omaha, NE to fabricate the specimen. To reduce the weight of the specimen, a 4 in block-out was made at the bottom flange to have a total depth of 4 ft 4 in. Fig. 10 shows the specimen elevation and different sections.

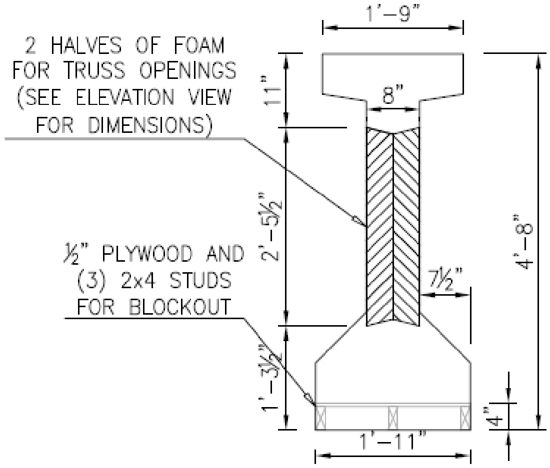

Fig. 91 Cross-section of the truss specimen and foam block-out

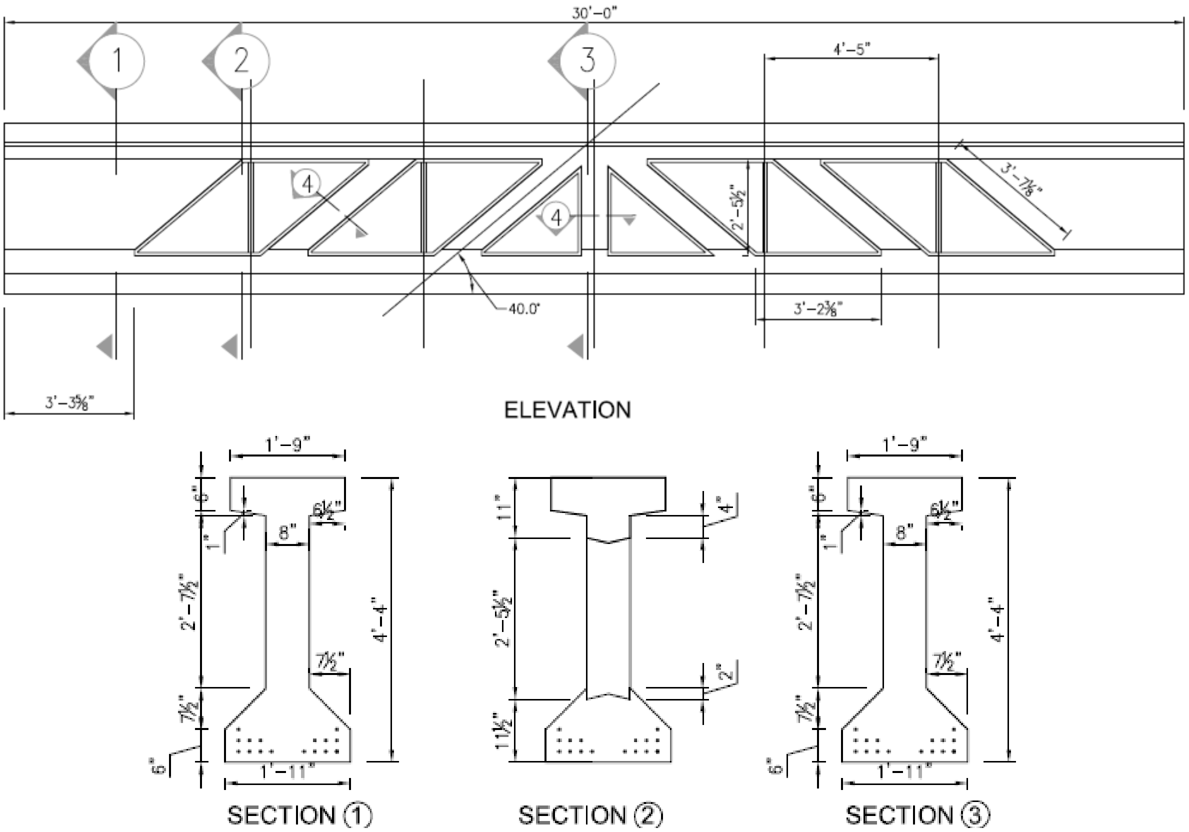

Fig. 102 Elevation and sections of the specimen

Frame analysis and Finite Element Analysis (FEA) are performed to predict the capacity of the specimen. In the frame model, concrete webs are modeled as shell members, while verticals, diagonals, top chord and bottom chord members are modeled as frame elements. In the FE model, all members are modeled as solids to investigate the stresses at the connections between the diagonals, verticals, top flange and bottom flange members. The

FE and frame models are shown in Fig. 13113 and Fig. 124, respectively. A comparison between the analysis results of the frame and FE model is shown below in Table 3. Also, FE analysis results show high stress concentrations at the acute angles in the connections between the diagonals and top and bottom flanges.

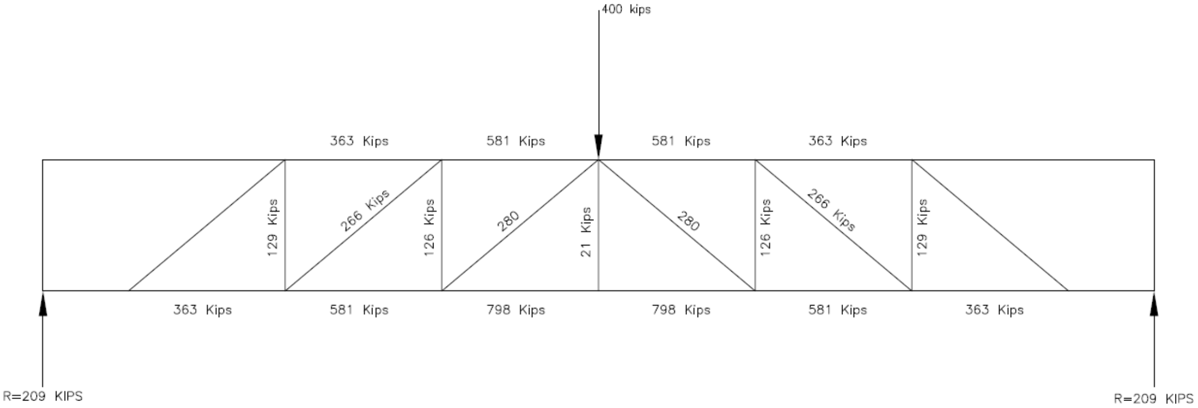

Fig. 124 The frame model used to analyze the truss specimen

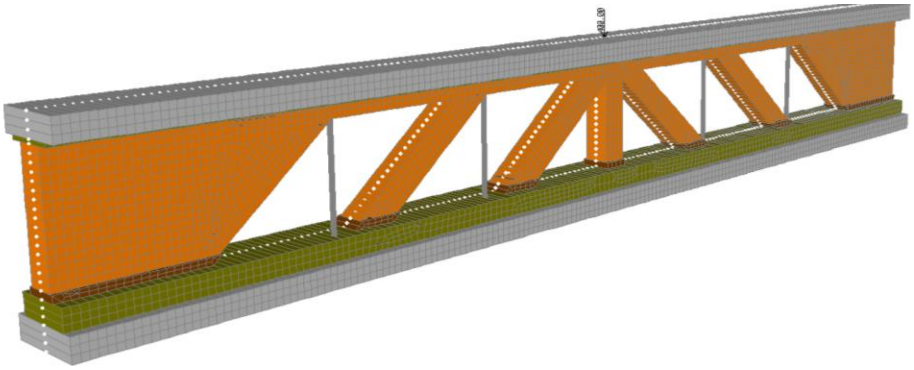

Fig. 1311 The FE model used to analyze the truss specimen

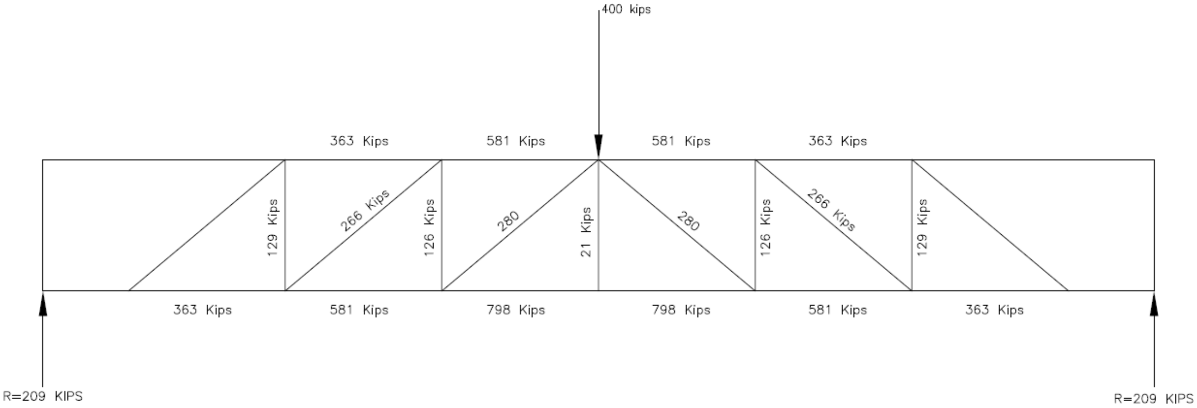

Fig. 124 The frame model used to analyze the truss specimen

Table 3 Comparison between the 2-D frame analysis and the finite element analysis outputs

	Frame Analysis	**FE Analysis**

Diagonals (kips, compression)	2	253
Verticals (kips, tension)	129	120
Camber (in.)	0.2	0.20
Deflection (in.)	0.8	0.65
Total deflection (in.)	1	0.85
Cracking Load (kips)	330	300

The specimen diagonal and vertical members have the same design presented earlier for the 160-ft span truss. For designing the top and bottom flanges of the specimen, a midspan point load of 400 kips is used to estimate the top and bottom flange reinforcement. A total of 12-0.7 in. diameter strands were used in the bottom flange and 2#8 bars were used as the top flange reinforcements.

Construction sequence had 5 phases. First, 12 0.7-in. strands were tensioned for a total jacking stress of 202 ksi. Bottom flange confinement reinforcement were tied to the strands and bearing plates were placed. Second, block-outs were made to resemble the full size specimen to evaluate its constructability. The 8-in. thick block-outs consisted of two 4-in. thick pieces. 0.75 in. x 0.75 in. grooves were made in the Styrofoam pieces to place the rods in. To ease the foam removal from the concrete web, plastic sheets were wrapped around the edges of the foam. Third, rebar cages was assembled and attached to the form. Threaded rods were anchored in the top and bottom flange using ½ in. thick 8 in. x 8 in. Gr 50 steel plates and structural nuts. The diagonal reinforcements and 2#6 anchors bars were welded the plates. Top flange reinforcements were tied together with the stirrups as a cage and placed after the form was closed. Figure 15 shows the reinforcement of diagonal and vertical members before closing the form.

Fig. 15 Form block-out and vertical and diagonal reinforcements

Forth, the truss was cast using ready-mix self-consolidating concrete (SCC). The mix was made with 3/8 in. maximum nominal size aggregate and had a 28 in. spread. Pouring the concrete started at the middle vertical. Two snake cameras were attached at the bottom of the truss, one at each end to observe the flow of concrete in the bottom flange. SCC flowability was adequate to fill the entire form without any vibration. One problem was reported during fabrication is the uplift of some foam block-outs. The glue attaching the block-outs to the steel form was not strong enough to hold them in place. Wood spacers were used between the foam and reinforcement to prevent them from floating.

Finally, the form was stripped and strands were released after 3 days (release strength was 7,800 psi). Removing the Styrofoam block-outs was a challenging task as it needed saw cutting and hammering to break the foam into smaller pieces and remove without damaging the concrete. The movement of the Styrofoam have resulted in a slightly different dimensions for the verticals and diagonals as the concrete cover thickness was changed.

SPECIMEN TESTING

Two steel rollers were placed under the truss and were centered on the 6 in. wide bearing plates resulting in a span of 29.5 ft. Strain gages were attached to the threaded rods, concrete diagonals, and top and bottom flanges (at the midspan) as shown in Figure 16. LVDTs were used to measure strand slippage during testing and a deflection gage was installed to measure midspan deflection. Concrete strength was found to be 10,500 psi on the testing day. Specimen was visually inspected for cracking at 50 kip increments of loading up to failure. The failure occurred as shown in Figure 17 at a load of 385 kips due to the pullout of most south vertical threaded rod from the bottom flange along with the connected diagonal.

Fig. 16 Instrumented truss specimen during loading

Fig. 137 Failure of the truss specimen

ANALYSIS OF RESULTS

Figure 18 shows the labeling of truss verticals and diagonals as well as their actual dimensions. Figure 19 plots the load-deflection relationship of the tested specimen. This plot indicates that the cracking load was 355 kips, which is very close to the predicted cracking load of 330 kips. The measured strains in all steel threaded rods were used to calculate the stresses, and consequently, the forces in all verticals. Figure 20 plots these forces versus load. This plot indicates that all rods have reached their yield stress and the design load of 136 kips, which indicates the efficiency of their design. Similarly, the forces that acted on the diagonals were calculated using the measured strains as shown in Figure 21. This plot indicates that the diagonal forces exceeded the design load of 268 kips reaching an axial force of 325 kips.

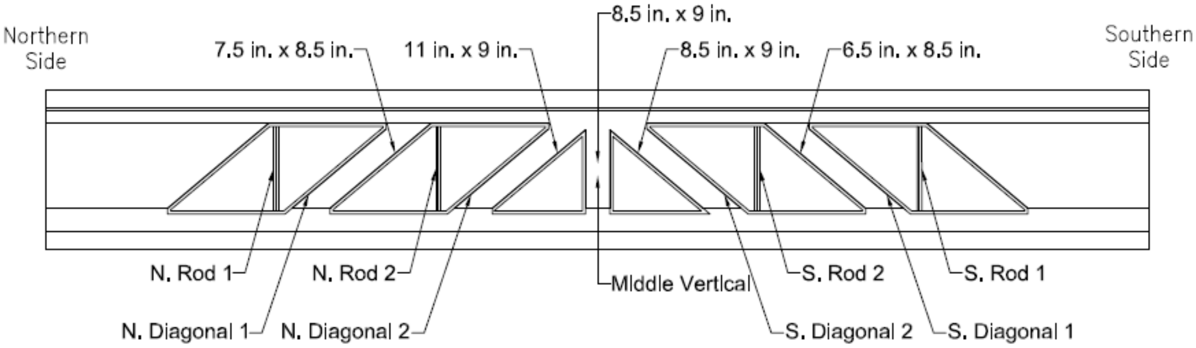

Fig. 148 Verticals and diagonals identification and dimensions after pouring

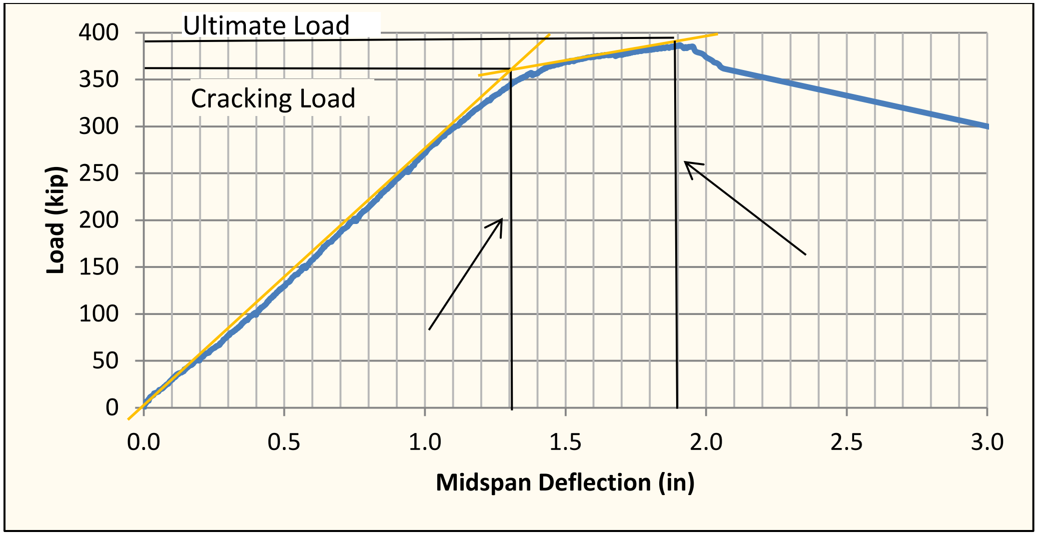

Fig. 19 Load vs. deflection curve

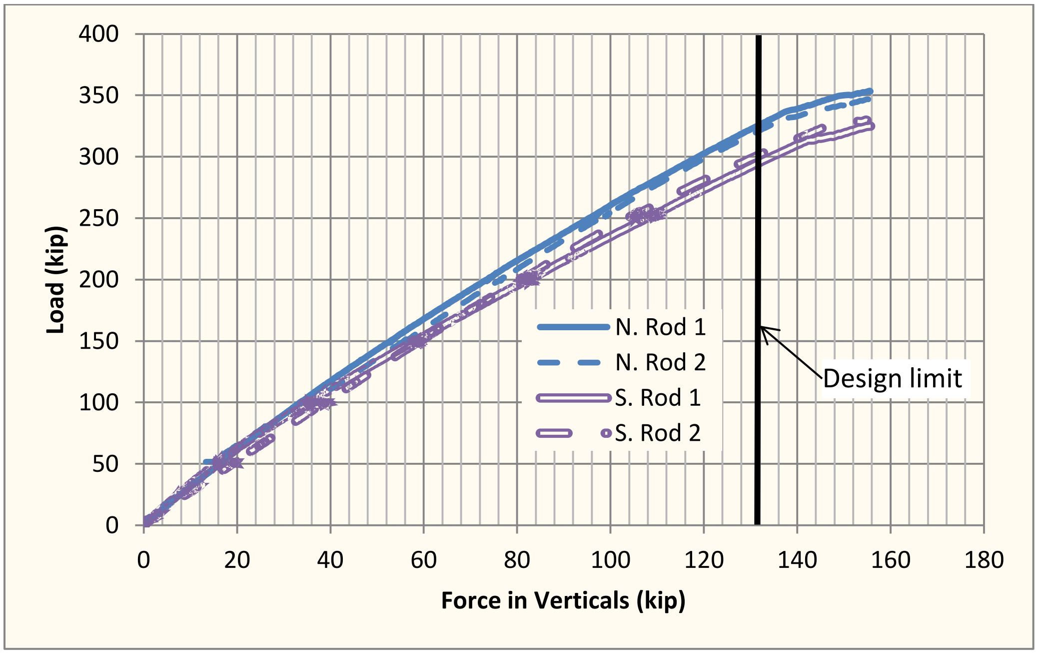

Fig. 150 Forces induced in Threaded Rods (TR) while loading (tension)

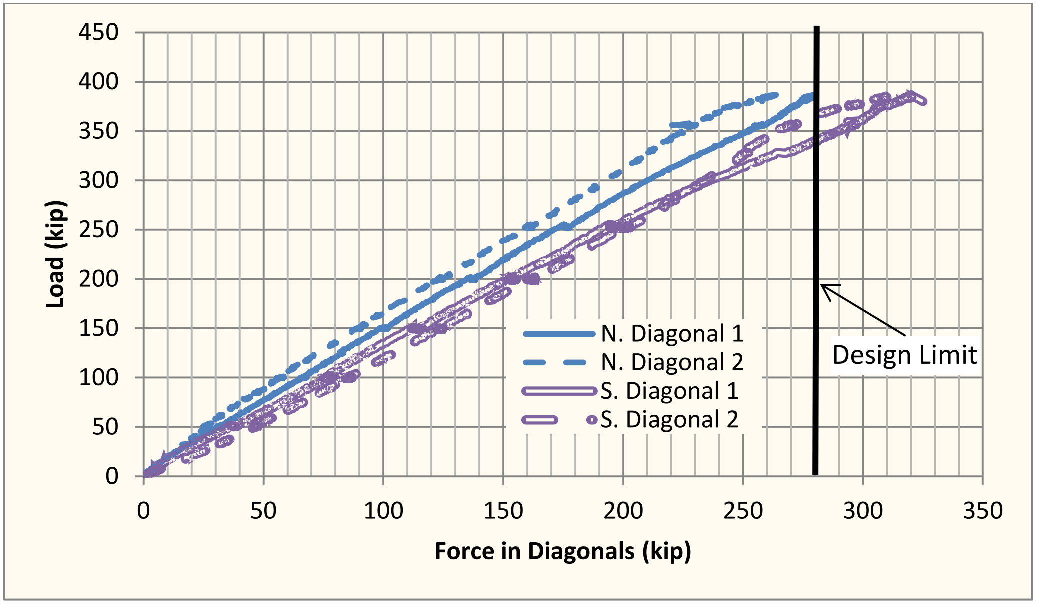

Fig. 161 Forces induced in diagonals while loading (compression)

CONCLUSIONS & RESOMMENDATIONS

CONCLUSIONS

The objective of this study is to develop a precast concrete truss for long-span roofs. The proposed system was analyzed, designed and tested, and the following conclusions were made. First, fabrication of the proposed truss is practical, economical, and efficient. The use of existing bridge girder forms and Self-Consolidating Concrete (SCC) significantly simplifies the fabrication process. Second, the proposed truss can span up to 160 ft., which is adequate for most long-span roof applications, such as warehouses and hangers. The truss is made of two 80 ft long segments that can be easily transported, erected, and post-tensioned using commercially available equipment and hardware. Finally, frame analysis and FEA can accurately predict the actual behavior of the proposed system as proved by testing

RECOMMENDATIONS

Based on the analytical and experimental investigation conducted in this project, the following recommendations are made:

- Chamfered or curved corners between diagonals and top and bottom chords are recommended to avoid stress concentrations and cracking.
- To avoid congested reinforcement, threaded rod anchorage plates should have short anchor bars (8 to 12 in. in length) welded to the plates.
- Avoid using Styrofoam for block-outs and using welded light gage steel pans instead to eliminate floating problems and facilitate form stripping.
- Assembling the rebar cage as one piece is cumbersome in handling and requires tight high tolerances. It is recommends to tie diagonals after placing in the forms to accommodate the production tolerances.

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