Iditarod National Historic Trail Bridge Design Project

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April 22nd, 2024, CE A686

In Partial Fulfillment MSCE Requirements at the University of Alaska, Anchorage

Abstract

The Iditarod National Historic Trail is a recreational trail system that follows antiquated transport routes across the coast and through the interior of Alaska. This trail is unimproved in certain areas and crosses many waterways, including rivers and creeks. Since this trail is commonly used for hiking and backpacking, efforts have been undertaken to make water crossings safer and easier, particularly in the section that crosses the Kenai peninsula.

This study examines the feasibility of constructing a bridge over the Bertha Creek Iditarod Trail Crossing in Turnagain Pass. In this report, we prepare a 65% design of a pony truss bridge and a 10% design of a through truss bridge, including supporting calculations and construction drawings. Various types of structural modelling and analysis software are used to analyze the bridge alternatives. We compare and contrast these designs on multiple different metrics – primarily constructability, cost, and aesthetics.

The results indicate that, although a pony truss bridge would be more expensive to construct, it would be much easier to construct, could be almost entirely shop fabricated, and would better suit the intended usage of the trail system. Both bridge designs would require a helicopter, although the pony truss would take less time to construct, due to less welding and shoring activities being required on site. Considering the benefits of each alternative leads to the conclusion that a pony truss bridge is preferred over the construction of a through truss bridge at this location.

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Introduction

This project presents the design of a bridge that spans between the north and south banks of Bertha Creek, approximately 10 miles southeast of Portage, Alaska. This bridge will provide hikers and Forest Service personnel with easy access across the creek for the purpose of recreation and trail maintenance. The Forest Service is in the process of designing a bridge at this location, and this graduate project is being conducted concurrently with the professional design project. This project will explore two alternate bridge designs.

The conceptual design that is the focus of this project is a hot-rolled steel pony truss bridge. The trusses are composed of both rectangular and circular hollow structural sections (HSS), and the bridge deck is composed of steel bar grate bearing on wide flange structural sections.

This project also considers a hot-rolled steel through truss bridge as an alternative design. This bridge utilizes circular HSS members for the top chords, truss bracing, framing, and subfloor crossties. The bottom chords are wide flange sections.

Each design is in accordance with relevant portions of AISC 360-16. Provisions specific to the AASHTO Bridge Design Specification (9^{th} Ed.) are not considered in the design basis. The loads used are determined from applicable portions of ASCE 7-16 with project-specific modifications, and include dead, live, snow, and seismic.

The pony truss bridge design has been prepared at the 65% design level and includes a rigorous analysis in RISA-3D and MASTAN2, a 65% construction drawing set, and a partial foundation design. The through truss bridge has only been prepared at the concept level, and has been included in order to compare the pros and cons of each truss type.

Supporting Information

Information is provided below that includes a holistic view of the project and includes relevant details to the results of the design.

Background

The Iditarod National Historic Trail (INHT) is an expansive trail system that traverses the Alaskan wilderness from Seward, Alaska, to Nome, Alaska. The main trail is over 1000 miles and ties into approximately 1400 miles of additional connectors and offshoots. Due to the trail system being identified and/or constructed in finite increments at different times, the date of creation is not clear; however, the Bureau of Land Management claims that, although it was used over most of the last two centuries, it was established as a historic trail by Congress in 1978 (BLM, n.d.).

The portion of this trail that runs between Seward and Portage is frequently used by nature hikers, backpackers, and tourists throughout the summer season in southcentral Alaska. It boasts scenic views and overlooks several water features along the duration of the trail. Efforts have been undertaken to make this trail more accessible, which have spurred several projects like this one recently. The trail crosses several rivers and creeks that pose hazards for casual hikers and children. Additionally, some of the larger rivers and creeks have carved large gorges into the rock that complicate the crossings further.

Site Information

Approximately 10 miles southeast of Portage, Alaska, Bertha Creek is one of multiple INHT creek crossings in the region that the Forest Service has identified as benefitting from the construction of a river crossover bridge. The river runs through a small canyon with relatively flat plateaus on either side. The total elevation change from the river surface to the top of the canyon is approximately 40’, and the bridge will need to be at least 50’ long to pass over the river and span to the plateaus on the canyon edges.

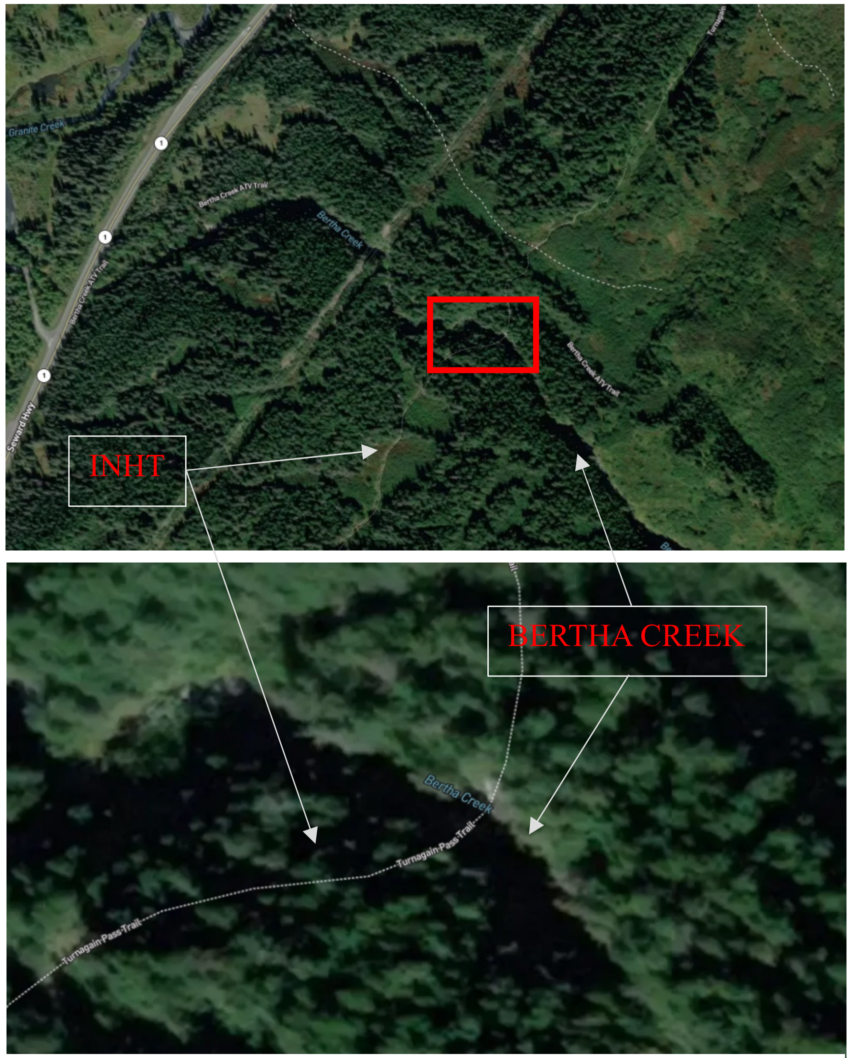


Figure 1: Project Location, Shown at Approximate Scale of 1”=1000’ (a) and 1”=200’ (b)

The location where the INHT crosses Bertha Creek is only approximately 1000 feet east of Seward Highway and is accessible from the highway via existing ATV trails that run through the area. Additionally, there is a powerline easement that passes within 600 ft of the site that may be able to provide access for larger vehicles. The area is heavily wooded; extensive clearing and grubbing will need to be carried out for any areas that are unimproved from their natural condition.

The Forest Service has conducted a preliminary soil investigation and determined that the area has a shallow soil overburden (less than 2 ft in most areas) and has a solid bedrock layer underneath, making the location ideal for installing a foundation for the bridge. Bore holes were not taken to determine the thickness of the bedrock layer, but it is assumed that the bedrock will be at least thick enough to install a concrete cold joint with doweled rebar.

The project site is in the Chugach National Forest and experiences heavy precipitation throughout the year. This leads to the Turnagain Arm region having one of the heaviest design snow loads in the state. Additionally, it causes Bertha Creek to experience heavy seasonal flooding, which further increases the need for an elevated crossing.

Project Conception

This bridge design project was initially commissioned as a 35% alternatives analysis investigating a suspended arch bridge and a traditional pony truss bridge. However, due to project management constraints, it was decided that the pony truss bridge design would be elevated to a 65% design level and the suspended arch bridge design would be replaced by a 10% through truss alternative design. It was determined early on that this 65% design should contain a rigorous analysis of the structure and structural members but should stop short of fully designing and detailing the connections, which would be more applicable to a 95% design.

Project Deliverables

This project included the following project deliverables for the pony truss bridge at a 65% design level:

1. Basis of Design
2. Production Drawings
3. Design Models
4. Supporting Calculations

The basis of design was included in this report and discusses all pertinent information and decisions concerning the design of the bridge. There were geometric, load-related, and safety concerns that have influenced the design, and these were explained in detail in the basis section herein.

The production drawings included a rough site layout overlayed on an existing survey, construction notes, and structural drawings prepared at the 65% design level. They did not include other engineering disciplines, as the focus of this design project was structural. Certain architectural components were included in the structural sheets that have been deemed vital to the functionality of the bridge, such as the bar grate used for the road deck and the chain link fencing used to adhere to applicable provisions of 29 CFR 1910.

Three dimensional models were included as separate files alongside this report. Although portions of the 3D files were included as figures, the base files hold additional supporting information that may be necessary to review the design. These 3D models were prepared in RISA-3D (RISA Tech Inc., n.d.) and MASTAN2 (Ziemian et al., n.d.).

The structural calculations included everything necessary to bring the design to a 65% completion level. Weld sizes were not calculated and were instead sized using AISC 360-16 maximum size provisions in most situations.

Pony Truss Bridge - Basis of Design

Primary Design Criteria

This project used ASCE 7-16 for load calculations, and considered loads due to gravity, live (intended usage), snow, and seismic. Seismic loads were approximated using the full dead load of the bridge applied laterally as a seismic load in conjunction with the self-weight. This was done in part to avoid utilizing the AASHTO Bridge Design Specification, which was not available for this project. Wind calculations were not conducted as the resulting loads would be rather small compared to seismic loads, since the cross-sectional area of bridge subject to wind loads is limited to the structural members, and no sheathing is present.

Instead of applying a uniform live load outlined in ASCE 7-16 Ch. 4, it was determined that since the bridge is not intended to function as a high occupancy bridge and instead will transport at most a single offroad vehicle at one time, applying a moving point load along the bridge length was more appropriate. When loaded with snow, the bridge would not be able to accommodate crossing by vehicles or a large number of pedestrians, and with the large magnitude of snow load that the bridge will experience, a uniform live load would not govern. A 3000 lb design vehicle with a wheelbase of 4 ft was adopted to approximate the side-by-side all-terrain vehicles used by the forestry service. No unapproved vehicles will have access. Since uniform occupancy-based live loads are not being considered in this design, bollards and signage will need to be posted that limit the usage to what was considered in this design process.

Load Resistance Factor Design (LRFD) load combinations were generated in RISA using the nominal loads calculated from ASCE 7. Since the bar grate road decking spans transversely across the full width of the bridge between the truss bottom chords, the uniform area loads were converted to line loads and applied to the bottom of the trusses using the tributary width of each bottom chord.

Since the bridge was designed using structural steel, AISC 360-16 was used to evaluate the structural performance of the bridge elements. These calculations were embedded in the RISA 3D analysis, and output unity/utilization ratios for each member in accordance with Chapter H of the AISC Structural Steel Specification. The unity ratios are representative of the combined axial, shear, and flexural ratios of the applied loads to the strength of the member. A unity ratio of 1.0 or greater is indicative of failure. For the purposes of this project, the objective was for the maximum unity ratio for all members of the bridge to not exceed 0.8.

The anchor embedments used in the foundation were checked against all applicable provisions of ACI 318-14 Ch. 17. The loads considered in the anchor and foundation design were generated from the worst-case boundary reactions from the RISA model. Since it was assumed that the bridge would have pin and roller connections for the RISA model, long-slot connections were used for all the anchors. These calculations were conducted in the report generated by HILTI Profis, based on the three-dimensional foundation model created in the software. Since the controlling conditions were not created from a load combination involving seismic loads, overstrength factors in accordance with ACI 318-14 Sec. 17.2.3.5.3 were not required to be considered and were neglected.

Secondary Design Criteria

Buckling Resistance

Since the compression chord of the pony truss had an unbraced length that exceeds 50 feet, it was beneficial to conduct an elastic buckling analysis of the bridge in MASTAN to examine the buckling mechanics of the bridge. The pony truss bridge was remodeled in MASTAN and the worst-case vertical design loading was applied to the new model. Lateral loads were not considered in the MASTAN model because the seismic load combinations had relatively small vertical loads compared to the strictly vertical load combinations. Compression in the top chord and consequent top chord buckling resulted from maximizing the vertical load, and it was decided that modelling noncontrolling load combinations in MASTAN would complicate the design without adding any value to the project.

The load applied to the bridge model in MASTAN was incrementally increased until the bridge experienced its first buckling mode. This analysis method only considered the elastic range of each member and did not take into account additional strength that would be gained by pushing individual members into their inelastic deformation range. This failure load was returned as a ratio to the applied load – the Capacity-to-Demand Ratio (CDR). A CDR of 1.0 would theoretically indicate that the bridge would experience buckling at the design load level; however, due to factory tolerances and imperfections in the geometry of the finished bridge that would potentially lower the buckling resistance, a minimum CDR of 4.5 was sought for this project.

Geotechnical Considerations

Although a completed geotechnical report for this site was not available, verbal correspondence with members of the geotechnical investigation team revealed that the surrounding area is almost entirely composed of a shallow (~2 ft deep) layer of loose soil overtop of solid bedrock of indeterminate thickness. This is convenient for the foundation design, as piers can be erected and bonded to the bedrock layer, which means that footings are not required.

Duty to Have Fall Protection

29 CFR 1910.28 "requires employers to provide protection for each employee exposed to fall and falling object hazards." (OSHA, 1970). Since the Forestry Service is not exempt from this requirement, it was decided to specify that chain-link fencing be installed inboard from both trusses to prevent falls. Toe boards were considered as well, but it was determined that, since the creek below is not intended as a working area, falling object risks are not substantial enough to warrant toe boards.

Through Truss Bridge – Basis of Design

Design Criteria

The loads and load combinations used in the through truss bridge design were taken from the pony truss bridge; only the self-weight of the bridge differed. The design criteria were generally the same; however, since the top chords are braced together at each crosstie, a detailed elastic buckling analysis was not prepared for this alternative. Portal frames were included on either end of the bridge to further brace the top chord against buckling. Additionally, for a UTV to be able to pass underneath the portal frames and truss crossties, the truss needs to be much taller than the pony truss. A minimum of 8 ft was chosen as the required clearance for this bridge. For the RISA model, 0.8 was chosen as the maximum unity ratio for any individual member of the bridge.

Pony Truss Bridge - Design Results

Bridge Geometry

An in-depth description of the proposed Bertha Creek bridge has been included below. Although some items are left for the 95% design, this is what was used to create the structural models and analyze the structural stability of the bridge. Figure 2 shows the 3D isometric view of the proposed design.

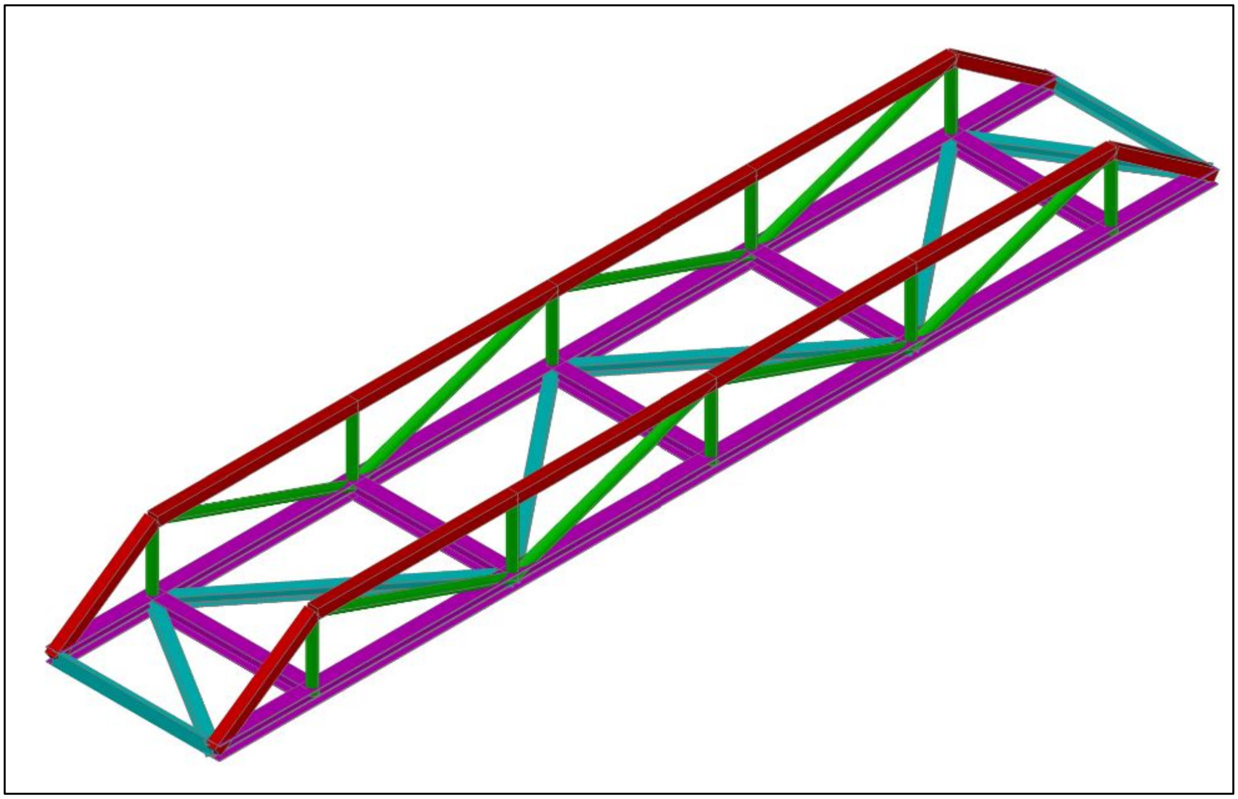


Figure 2: Pony Truss - Bridge Overview

This bridge is a pony-truss bridge; the top chords are unbraced over the full 50 ft length. The bottom chord of the truss is made up of W8x31 structural steel members that span the full length of the canyon. They are spaced 8 ft on center, making the clear width of the bridge approximately 7’-4”, not including the mesh fall protection devices installed inboard from the trusses. These beams bear on 3’-0” long by 1’-6” wide reinforced concrete piers that are bonded to the underlying bedrock layer below. Each beam-to-pier connection utilizes (4) 5/8” diameter cast-in-place anchor bolts embedded a minimum of 8” into the pier. These anchors tie into the bottom of the bridge stringers with long-slot bolt holes to allow some rotation and thermal expansion/contraction of the road deck. Pier reinforcing has been detailed in accordance with ACI 318-14 provisions and includes (14) #4 longitudinal bars and #3 stirrups required for anchor embedments.

The bottom chords have been provided with bracing under the road deck. This bracing is composed of W6x15 members, and spans diagonally across the (4) 10 ft long bays created by the longitudinal beams and the transverse floor beams. The bracing beams are fully welded and help make the road deck more rigid in the transverse direction.

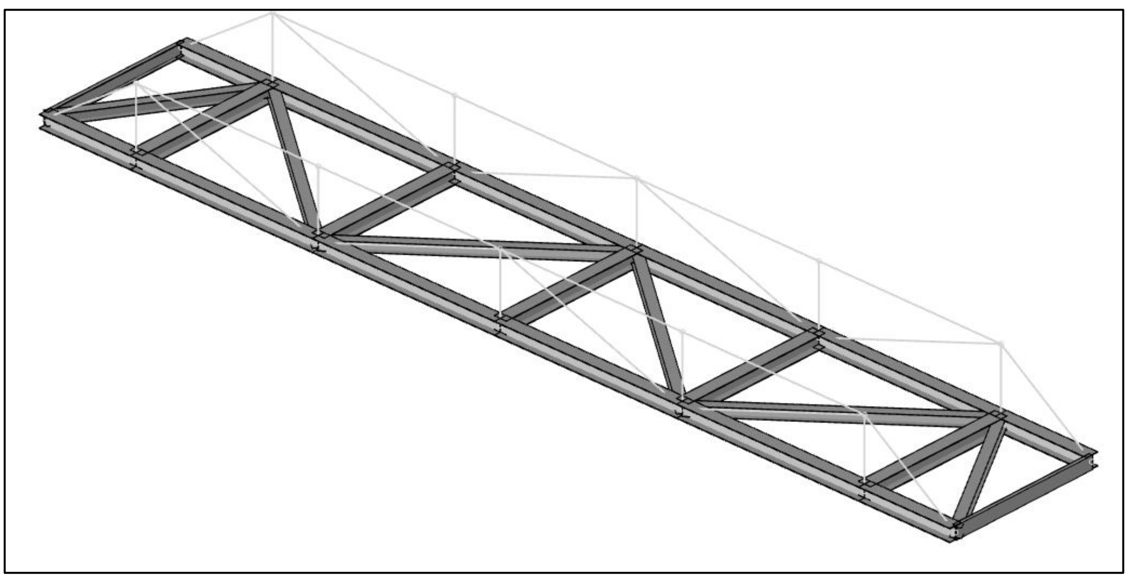


Figure 3: Pony Truss - Road Deck Framing

The truss (shown in Figure 4) is composed of HSS, both rectangular and circular. The centerline of the top chord of the truss is 3’-6” above the centerline of the road deck beam, making the total height of the truss approximately 40” above the surface of the road deck. The top chords are composed of HSS6x6x5/16. The truss verticals and diagonals are composed of HSS3.000x0.250 and are connected to the larger flange members via full perimeter fillet welds. These trusses are cladded with chain link mesh mounted with saddle clips.

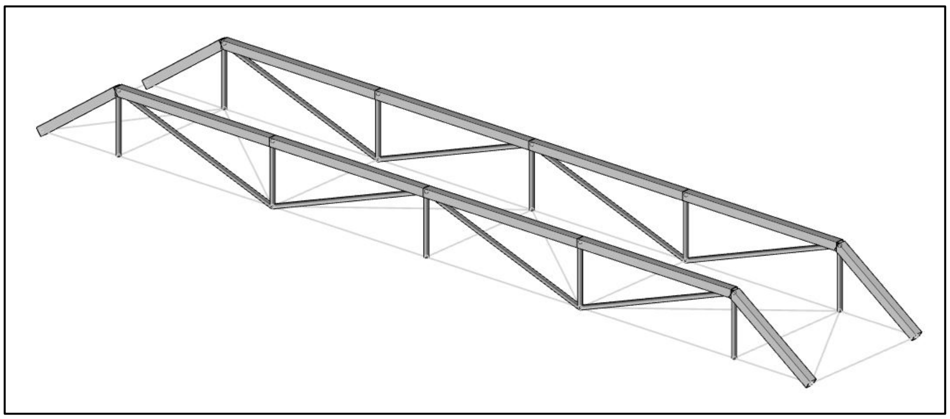


Figure 4: Pony Truss – Truss Framing

The road deck is composed of serrated steel bar grating with 2” x ¼” bearing bars. This bar grating is fastened to the W8x31 beams with saddle clips. This was designed prescriptively with the McNichols bar grate catalog and will adequately resist the combined and factored uniform loads described above.

Analysis

This design bridge was analyzed using a combination of RISA 3D, MASTAN2, and HILTI Profis. The software results are attached as a 65% calculations package in Appendix A of this report. The RISA Model showed satisfactory structural performance under the loads described in Appendix A. The highest unity ratio (0.74) was experienced by the diagonal bracing. Although this was relatively high, it was below the threshold of 0.8, which was an acceptable margin of safety. The top chord had a maximum unity ratio of 0.64, which is well below the specified threshold.

ASCE 7-16 LRFD load combinations were compiled using the internal processes of RISA 3D. This showed that, by far, the controlling combination was ASCE 7-16 combination 3. This combination includes dead, live, and snow loads, and resulted in a 1.017 klf line load applied to each stringer concurrently with the 3000 lb moving point load. This was the load that was exported to MASTAN2 for the next portion of the analysis. Since MASTAN2 uses consistent units, lbs and inches were chosen for this design. The load was therefore converted to an 84.5 lb/in distributed load on each beam for use in the the MASTAN2 buckling analysis.

The MASTAN model showed a higher margin of safety than the RISA analysis. The results showed a CDR of approximately 5.5 for the first buckling mode, which means that the top chord would not buckle until 5.5 times the design load was placed on the bridge. In reality, failure would occur before this point due to imperfections in the bridge materials and construction. This model was used to optimize the trusses, which resulted in the HSS6x6x5/16 members. Once the buckling compression load was determined from the MASTAN model, it was used to calculate the effective unbraced length of the top chord. This process resulted in an effective length of 12.3 ft. This effective length, approximated as 12, was put into RISA with an effective length factor of

1.0 to maintain accuracy of the analytical model. Since the resultant CDR from MASTAN was indicative of a much higher margin of safety than the RISA results under a design loading, it showed that this bridge will not experience loads high enough to cause buckling at the design level, and that top chord out-of-plane buckling will not be the ultimate failure mode for the bridge. Instead, the high unity ratio in RISA-3D indicates that the outer diagonal truss members will fail first due to the combined effects of tension and flexural yielding.

The analysis conducted with HILTI Profis revealed that the anchors holding the bridge to the piers perform adequately under the design loads. The loads used in the analysis were taken from the reaction forces output from RISA. The Profis model showed that under those conditions, the controlling failure mode for this connection would be shear failure of the concrete anchors; however, the design loads only placed the utilization ratio for this failure mode at 0.35, meaning that there was still a 65% safety margin under design conditions.

Through Truss Bridge - Design Results

Bridge Geometry

General Information and design results of the 10% through truss design have been included below. Although this alternate design has not progressed as far as the primary pony truss design, it has been designed thoroughly enough to compare and contrast the two alternatives. Figure 5 shows the 3D isometric view of the proposed through truss.

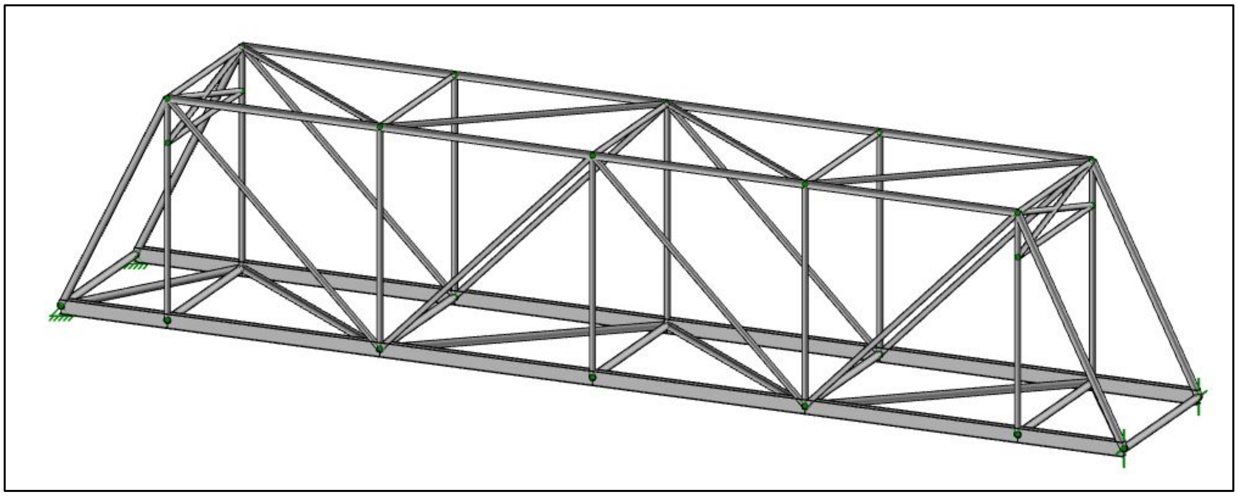


Figure 5: Through Truss - Bridge Overview

This bridge is a through truss bridge; the top chord is braced at each 10 ft node, and there is a portal frame on either end to resist racking of the structure. The bottom chords are 50 ft long W8x10 sections. These stringers are spaced 8 ft on center, making the clear width of the bridge approximately 7'-9". The top chord (shown in Figure 7) is composed of HSS4.000x0.250 and the diagonal and vertical truss members are composed of HSS3.000x0.125. The top chord has a height of 10 ft from the centerline of the bridge to the centerline of the top chord with a clearance of 7'-4" from the road deck to the bottom of the portal frames. The lateral truss braces, including the top chord cross ties and portal frames that span above the road deck, are HSS3.000x0.125.

The stringers have been provided with bracing under the road deck. This bracing, including the diagonals and the crossties, is composed of HSS4.000x0.250, and spans across the (4) 10 ft long bays created by the longitudinal beams and the crossties. The road deck matches the construction material used on the pony truss design and will be subject to the same loads.

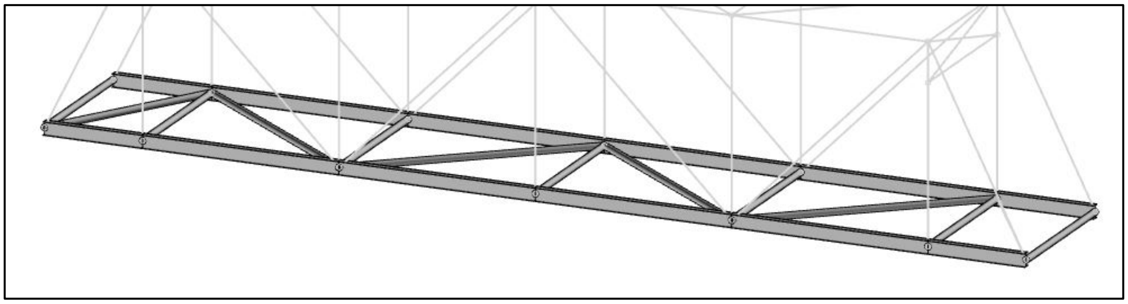


Figure 6: Through Truss - Road Deck Framing

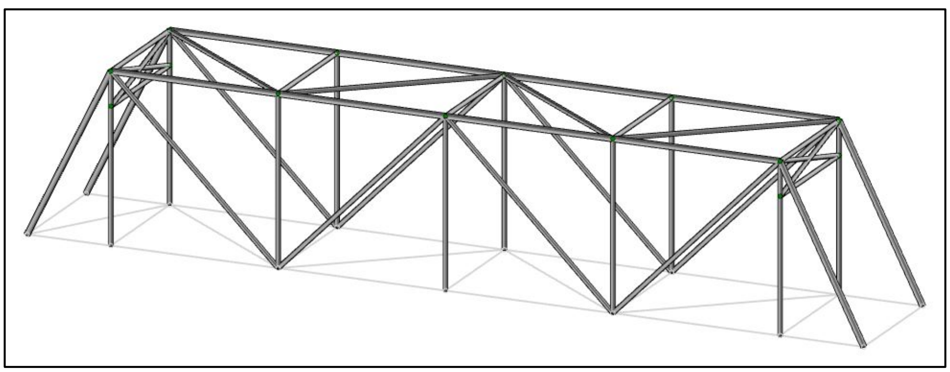


Figure 7: Through Truss - Truss Framing

Analysis

This design bridge was analyzed using RISA 3D. The loads experienced by the foundation and the loads experienced by the bridge itself were the same as the pony truss bridge, and it was determined that MASTAN would not be required for this design. The RISA printouts and models have been attached as a 10% calculations package in Appendix B of this report. The RISA Model showed satisfactory structural performance under the loads described in Appendix B. The highest unity ratio (0.77) was experienced by the diagonal truss members. Although this was essentially at the maximum unity ratio established earlier in the project, since this was only a 10% design, this was acceptable for comparison and contrast purposes. The top chord had a maximum unity ratio of 0.53, which is well below the project constraints.

Alternatives Analysis

Main Considerations

As part of this project, the pony truss bridge and through truss bridge were compared. The main metrics considered were constructability, i.e. feasibility of construction, cost, and aesthetics.

Constructability

The main concern driving the constructability of each bridge design was the feasibility of transporting construction materials and personnel to the site. The Bertha Creek INHT crossing is approximately 1,000 ft from the Seward Highway, and creating a haul road would permanently impact the natural landscape of the area.

The pony truss was more compact in overall geometry, but with much larger steel sections. This lead to the pony truss being much heavier, with a total weight of 8,936 lbs, not including the bar grating. In contrast with the weight, the bridge would take up less volume post-construction. With how compact this design turned out; it was decided that the bridge should be almost entirely shop fabricated, with the bridge being split into approximately 3 equal weight pieces (approximately 16' long) with shop splices in the truss and floor bracing. The bridge is only 8 ft wide and would fit on a flat-bed semitruck, which could transport the bridge sections from the port of Anchorage to the staging zone on the Seward Highway. Although it would likely be possible to use two larger pieces and still utilize trucks for transport, using three sections would make the bridge easier to move with helicopters.

This would leave minimal work to be done on site, although cargo helicopters would still need to be utilized to pick the bridge pieces off the trailers and place them on the foundations. Research was conducted to determine if helicopter transport and placement was feasible for this design, and it was determined that multiple companies in the region claim to have helicopters capable of carrying a cargo load of up to 6,000 lbs, which exceeds the weight requirements any of the three bridge pieces. This alternative is more convenient for the bridge construction itself and would require much less time than significant field fabrication activities.

The through truss had much smaller members, weighed only 4,870 lbs, and had an overall larger geometry when one considered the height of the truss. Because this bridge weighed less and was much larger than the pony truss once assembled, it was decided that this bridge should be field fabricated so that it could be transported in bundles. Additionally, the welds would be smaller and could likely be done single pass, which would mean that the welding construction would be faster, and therefore less likely to be impacted by environmental factors. The construction materials could be staged at the location where Bertha Creek crosses the Seward Highway and could be picked and transported to the project site in 1,000 lb bundles. All personnel and smaller construction tools could be transported to site on the existing trails via UTV, with larger tools, shoring, and rigging equipment being transported by helicopter. Neither alternative would require a haul road to be constructed. Since both sides of the river at the project location are accessible via existing trails, personnel could be transported between the sides of the creek by returning to the highway on UTV and driving down an alternative trail fork. After all materials are transported to site, the helicopter would be needed to hold the bottom chords in place while each section is connected. After the bottom chords are placed, all other components could be erected without aerial support.

Cost

It was difficult to quantify the cost of each alternative because the chosen construction methods vary significantly and bids were not collected for this work. However, it was determined that $3 per pound of structural steel (material cost only) was a fair approximation. Additionally, for shop fabricated steel, an additional $6 per pound was factored in, and $8 per pound was added for field fabricated steel. This is roughly approximated from a cost breakdown of steel construction from an article on AISC's website (*Construction Costs*, n.d.). An additional factor of 2.5 was included to capture the variability of the real construction cost, transport conditions, and contractor markup.

This resulted in a total cost of $201,060 for the pony truss bridge and a total cost of $133,925 for the through truss bridge. This is largely a function of the raw weight of the bridge, but it indicates that the pony truss bridge would be much more expensive using the unit costs approximated above.

Aesthetics and Functionality

One of the most important functions of this bridge would be to give hikers and trail occupants a good experience and the ability to witness the natural beauty of the Bertha Creek landscape. The bridge should offer an unrestricted view of the creek and surrounding mountains.

The pony truss bridge performed much better on this metric. The top chord of the truss had a very low height that would provide an armrest for people to lean out and view the park. The top chord was at approximate chest height and would not intrude into the eyeline of hikers passing over the bridge.

Alternatively, the through truss had chain link safety mesh that extends up to 10 ft on all sides of the bridge. This would partially restrict the view of anyone on the bridge. The top chord was much too tall to serve as a functional armrest, and hikers would likely grasp the chain link if they were to rest on the bridge and overlook the park.

Conclusion

This bridge was designed with the intent to show feasibility of the construction of a pony truss installed over Bertha Creek, Alaska, at the Iditarod National Historic Trail crossing. Subsequently, the design has sufficiently shown that the construction of a pony truss at this location is possible, and likely the optimal design for a low occupancy bridge at this site. Although a pony truss would likely cost more, it would better fulfill the intended goals of the bridge – to provide an overlook and river crossover bridge for INHT hikers and maintenance personnel, and to serve as a net positive to the beauty of the landscape. Additionally, the pony truss design would be easier and faster to construct in the field, as most connections would be shop fabricated.

At the 65% design level, this report has outlined the types of materials that would be used to construct such a bridge and included all calculations necessary to establish functionality under design loading. If a bridge of this nature is issued for construction in the future, this report could serve as a basis to lower front-end design costs. The report has also explored the design of a through truss, which would also be a viable alternative for allowing river crossing but may slightly interfere with hikers' abilities to enjoy the scenic views.

Additionally, this project has demonstrated the necessity of conducting a thorough elastic buckling analysis in conjunction with the rest of the design calculation outlined in AISC 360-16. Many structures require multiple forms of analysis to have full confidence in the design, and long unbraced pony trusses are no exception.

References

Alaska Snow loads. Structural Engineers Association of Alaska. (n.d.). https://seaak.net/alaska-snow-loads

BLM. (n.d.). National Conservation Lands: National Scenic and Historic Trails: Iditarod: Bureau of Land Management. Iditarod National Historic Trail | Bureau of Land Management. Retrieved from https://www.blm.gov/programs/national-conservation-lands/national-scenic-and-historic-trails/iditarod

Building Code Requirements for Structural Concrete (ACI 318-14). (2014). American Concrete Institute.

Construction Costs. AISC. (n.d.). https://www.aisc.org/why-steel/resources/construction-costs/

Minimum Design Loads and Associated Criteria for Buildings and other Structures (ASCE 7-16). (2016). American Society of Civil Engineers.

Occupational Safety and Health Administration. (1970). Occupational safety and health standards: Duty to have fall protection and falling object protection (Standard No. 1910.28). Retrieved from https://www.osha.gov/laws-regs/regulations/standardnumber/1910/1910.28

RISA Tech, Inc. (n.d.). *RISA-3D* (Version 22.0) [Computer software]. https://risa.com/products/risa-3d

Steel Construction Manual (AISC 360-16). (2017). American Institute of Steel Construction.

Ziemian, R., McGuire, W., & Liu, S. (n.d.). MASTAN2 (Version 5.1) [Computer software]. https://www.mastan2.com/about.html

Appendix A – 65% INHT Calculations Package – Pony Truss

Contents

Scope

The project scope includes the design of a pony truss access bridge located in Turnagain Pass, Kenai Peninsula Bureau, Alaska. This project is further outlined in the scope of work dated 04/25/23. This calculations package includes the development of design loads to be placed on the bridge. The determined loads will be imported into the concept RISA model in order to develop a basis for more in-depth design. The loads will be based primarily on ASCE 7-16 load combinations, with alternative codes and methods used where appropriate.

Note: throughout the calculations, a formula similar to $2>1=1$ will be utilized, this is a true / false equation. A result of 1 is "True" and a "0" is false.

References

- WBDG Structural Load Data Tool (SLDT) for UFC 3-301-01
- IBC 2018 - International Building Code
- ASCE 7-16 - Minimum Design Loads for Buildings and Other Structures
- ACI 318-14 - Building Code Requirements for Structural Concrete
- AISC Steel Construction Manual (15th Ed.)
- AISC Seismic Design Manual (3rd Ed.)

Load Combinations:

ASCE 7 Load and Resistance Factor Design (LRFD) Basic Combinations

LRFD	Principal Load
1) $1.4\ D$	D
2) $1.2\ D+1.6\ L+0.5\ S$	L
3) $1.2\ D+1.6\ S+L$	L_r or S or R
6) $\left(1.2+0.2 \cdot S_{DS}\right)\ D+\rho Q_E+L+0.2\ S$	E
7) $\left(0.9-0.2 \cdot S_{DS}\right)\ D+\rho Q_E$	E

D = Dead Load
L = Live Load
L_r = Roof Live Load
S = Snow Load

ASCE 7 notes on loading

This bridge will be designed to resist all applicable load combinations, including the effects of Dead Load, Seismic Load, Live Load, and Snow Load. The bridge will be open face with no structural or aesthetic sheathing, and therefore, the wind load will be minimal in comparison to weight-based loads like dead and seismic. Therefore, wind has been neglected, as it would needlessly complicate the design without yielding any discernable effect on the design. Additionally, live roof load, as defined by ASCE 7-16, is not applicable to the bridge, and will not be considered.

Dead

The dead load for the bridge will primarily be from self weight. It is assumed that no area on the bridge will be used for storage, and therefore, the only area-based dead load on the functional surface of the bridge will be the weight of the pedestrian travel surface. This will most likely be hot-dip galvanized heavy duty carbon steel bar grating. Based on industry charts, it is assumed that the weight of the bar grating will not exceed 25 psf. This 25 psf will be distributed across the full 8ft width of the bridge, and will be applied to the RISA model as a $\frac{25\ psf \cdot 8\ ft}{2} = 100\ plf$ line load on the bridge stringer.

Live

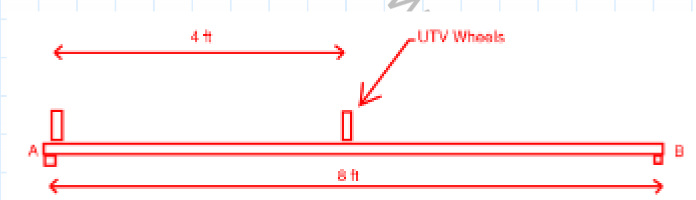
The main live load to consider is the moving load posed by the application of a pedestrian or utility vehicle on the bridge. Based on research conducted on the average and extreme end weights of off-road vehicles, 3000 lbs is an appropriate upper bound estimate for the curb weight of a side by side-style utv with two passengers. An additional complicating factor is that, width-wise, this concentrated load will be distributed between the two sides of the utv, through the tires, and then distributed into the stringers. The most extreme reaction induced in one stringer will occur if the utv has a narrow wheel base and is riding on one side of the bridge. 4 ft has been adopted as a "narrow wheel base" standard for this design vehicle. If the vehicle is driving with one side directly adjacent to the stringer, analysis reveals that the load actually imparted to one stringer will be 2250 lbf. Therefore, a moving load of 2250 lbf will be used in the RISA model.

$\Sigma M_A := 0$

$(1500\ lbf)\ (4\ ft) = R_B \cdot (8\ ft)$

$R_B := 1500\ lbf \cdot \frac{4\ ft}{8\ ft} = 750\ lbf$

$R_A := (2 \cdot 1500\ lbf) - R_B = 2250\ lbf$ (Moving Concentrated Load)



Snow

Snow loads have been based on station data for the Turnagain Pass region, published by Structural Engineers Association of Alaska. 298 psf will be used as the snow load, and will be distributed uniformly across the entire functional surface of the bridge.

$p_g := 298\ psf$

$C_e := 0.7$ (ASCE 7-16 Tbl. 7.3-1, Fully Exposed in Windswept Mountainous Areas)

$C_t := 1.2$ (ASCE 7-16 Tbl. 7.3-2, Open Air Structure)

$I_s := 0.8$ (ASCE 7-16 Tbl. 1.5-2, Risk Category I)

$p_f := 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 140.18\ psf$

$w_{snow.stringer} := \frac{p_f \cdot 8\ ft}{2} = 560.72\ plf$

Seismic

For this portion of the concept design, a seismic load of half the full weight of the structure will be distributed laterally to each stringer. Since the base shear coefficient in most cases is between 0 and 1, using the full structure weight in a lateral direction will be a conservative approximation. Although there is a procedure for the calculation of seismic loads on bridges in the AASHTO Bridge Design Specification, it will not be utilized due to the inability to acquire the document. This conservative approach will be taken, instead.

Unit Weights

$Weight.Stringer := 49\ plf$ (W10x49 Stringer)

$Weight.BarGrate := 25\ psf$ (Assumed from McNicholas Bar Grate Catalog)

$Weight.SmallTruss := 24.05\ plf$ (HSS5.000x0.500)

$Weight.LargeTruss := 50.81\ plf$ (HSS8x6x5/8)

Lengths

$Length.Stringer := 50\ ft \cdot 2 = 100\ ft$

$Area.BarGrate := 8\ ft \cdot 50\ ft = 400\ ft^2$

$Length.SmallTruss := (10 \cdot 3.5\ ft) + (8 \cdot 10.595\ ft) = 119.76\ ft$

$Length.LargeTruss := (2 \cdot 40\ ft) + (6 \cdot 6.103\ ft) = 116.62\ ft$

$$Weight.Bridge := (Length.Stringer \cdot Weight.Stringer) + (Area.BarGrate \cdot Weight.BarGrate) + (Length.SmallTruss \cdot Weight.SmallTruss) + (Length.LargeTruss \cdot Weight.LargeTruss) = 23705.59\ lb$$

$$\omega_{seismic} := \frac{Weight.Bridge}{2 \cdot Length.Stringer} = 118.53\ plf$$

Foundation Design

All anchor design checks have been conducted in Hilti PROFIS, based on applicable embedment calculations from ACI 318-19 Section 17. The loads used in the design were taken from the worst case load combinations output from the RISA 3D bridge model. These loads have been put into profis as factored loads, and overstrength factors were not considered in this design. Minimum concrete reinforcement requirements for temperature and shrinkage are included below.

$d_{pier} := 36\ in - 4.5\ in = 31.5\ in$

$b_{pier} := 18\ in$

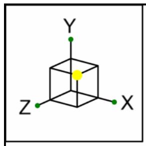
$f'_c := 6\ ksi$ (From PROFIS Model)

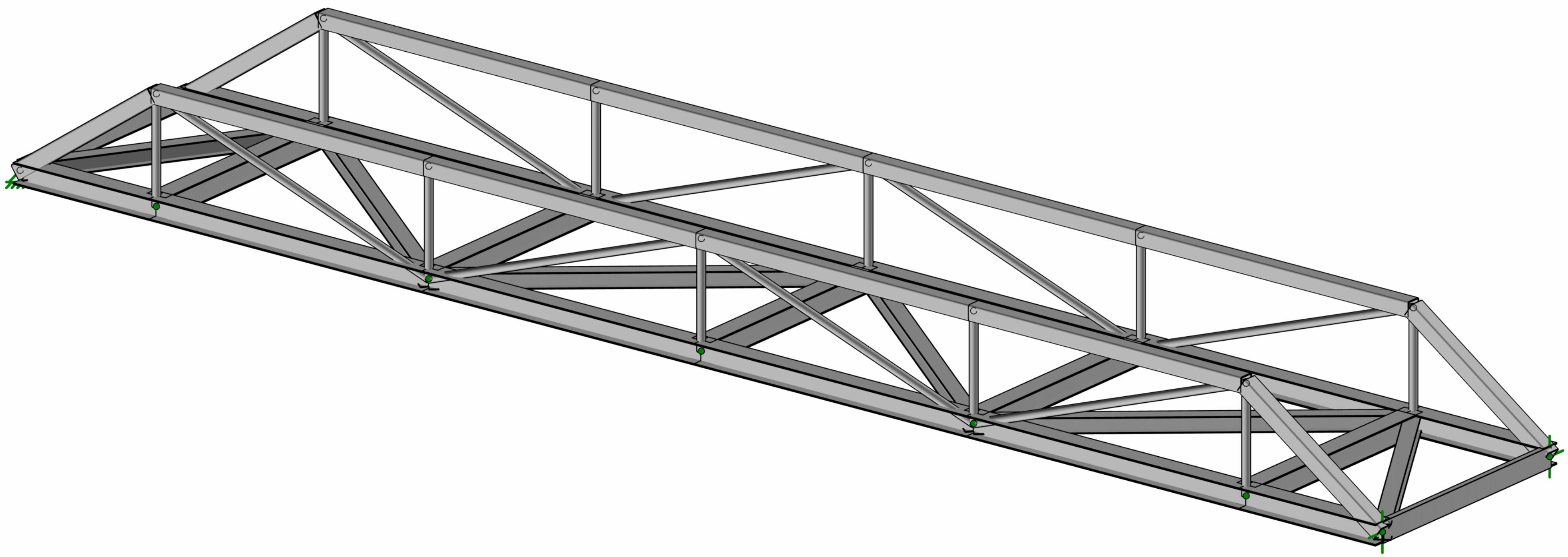
$f_y := 60\ ksi$

$$A_{s.min} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}}}{\frac{f_y}{psi}} \cdot b_{pier} \cdot d_{pier}, \frac{200}{\frac{f_y}{psi}} \cdot b_{pier} \cdot d_{pier}\right) = 2.2\ in^2$$

$A_{s.\#4} := 0.2\ in^2$

$$No.\#4Bars := \mathrm{ceil}\left(\frac{A_{s.min}}{A_{s.\#4}}\right) = 11$$





Envelope Only Solution

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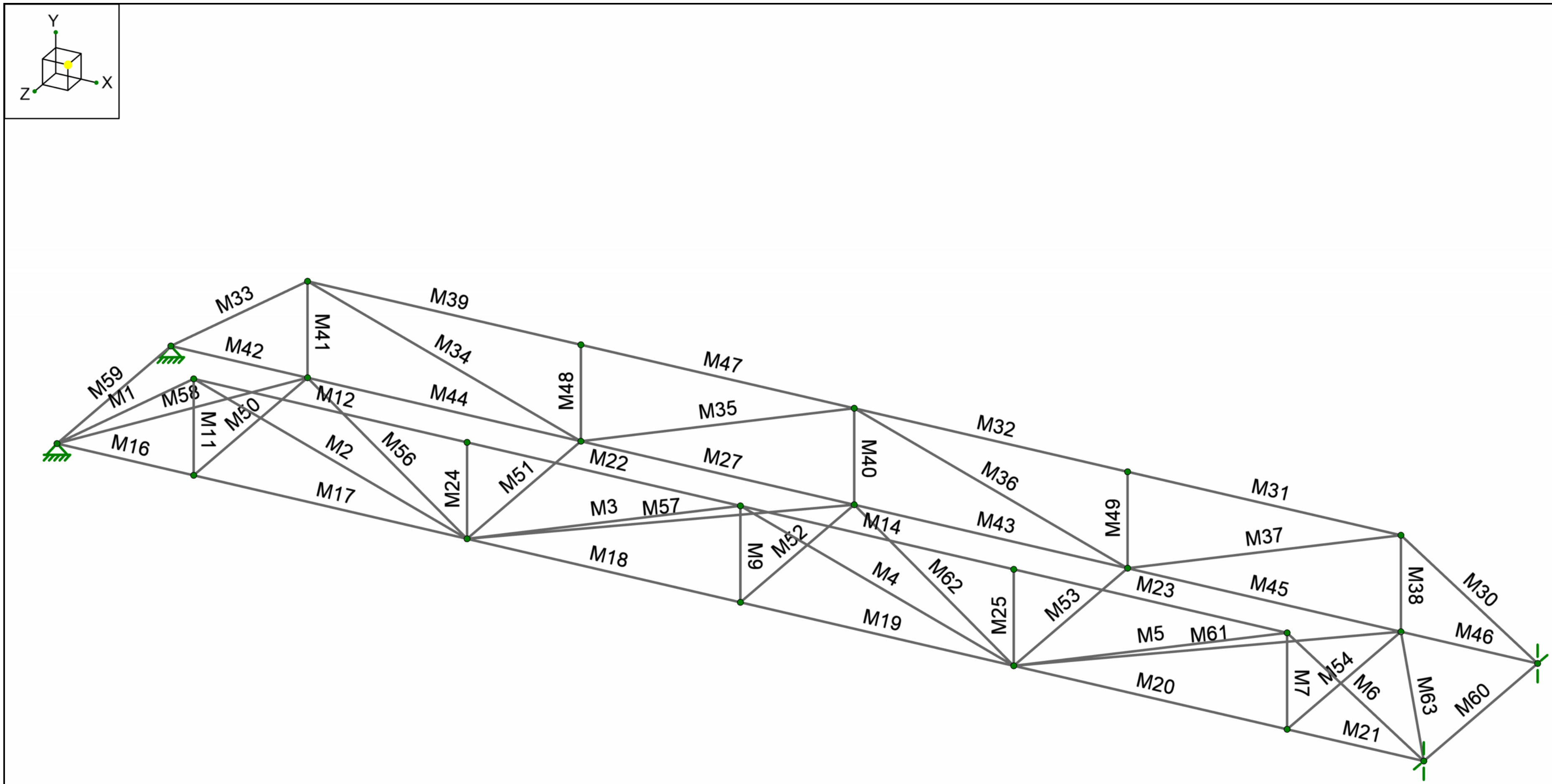
	Nicholas F. Schwantes		Apr 22, 2024 at 01:24 PM
			PonyTruss Rev 1.r3d



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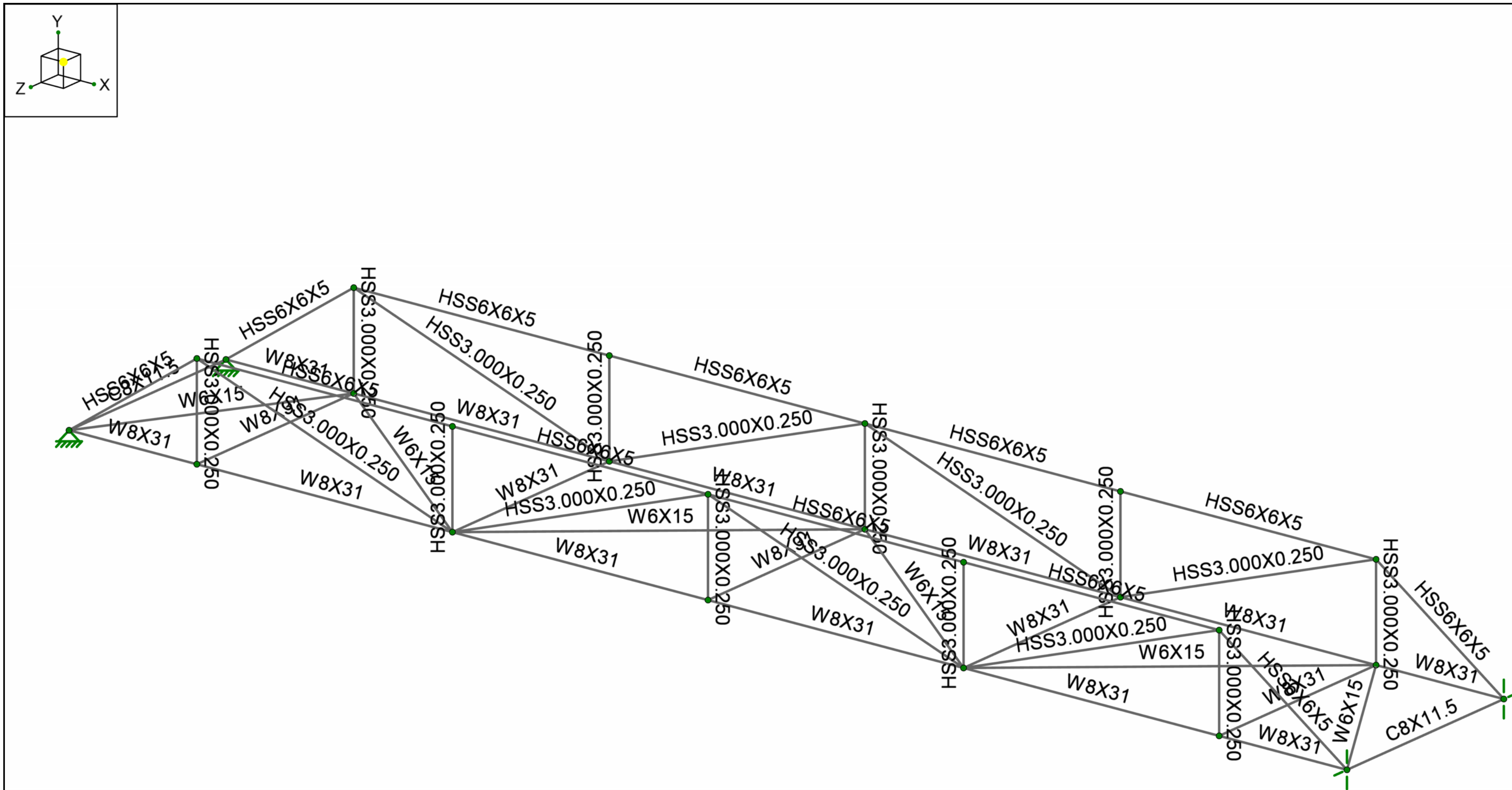

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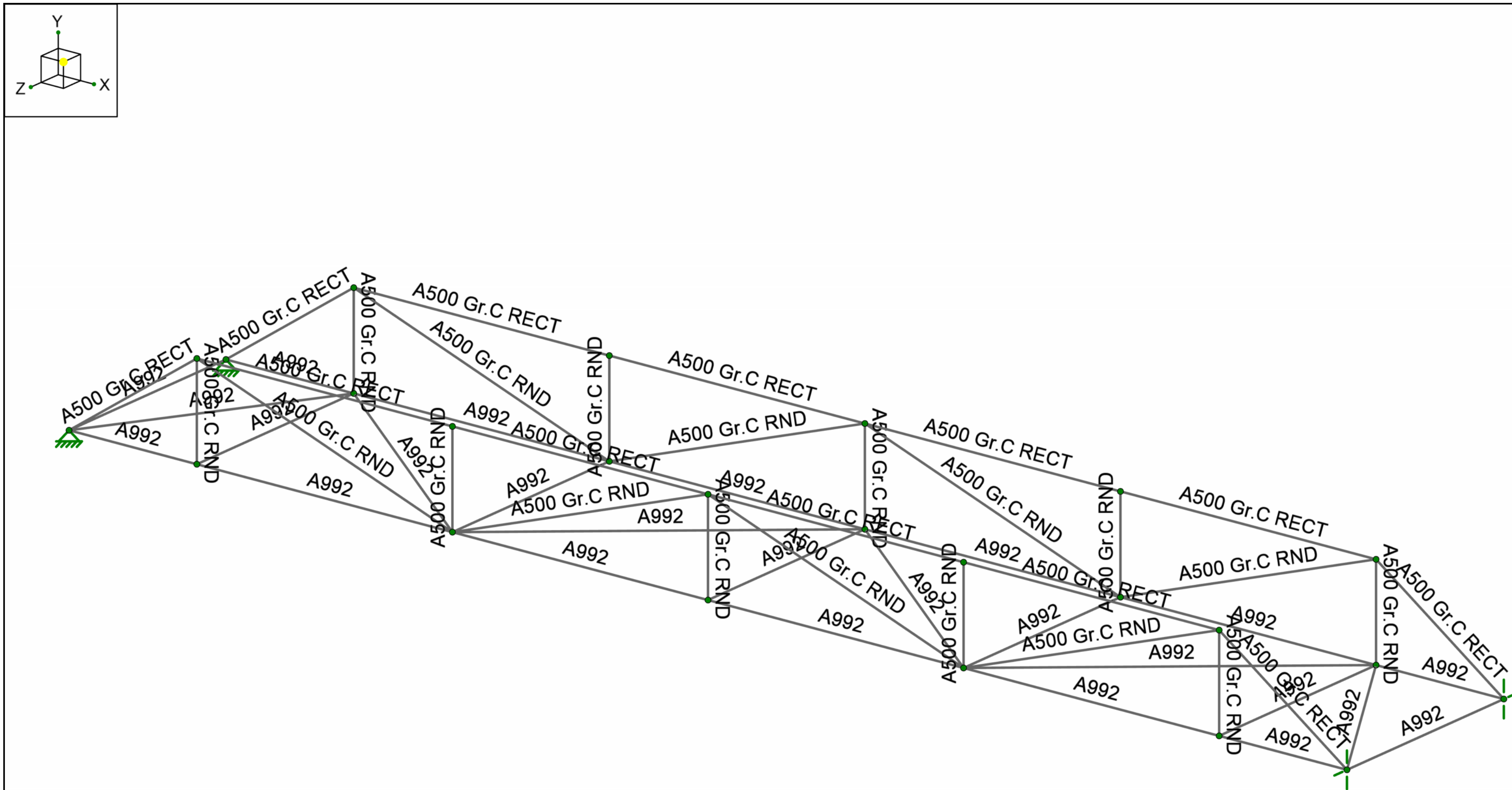

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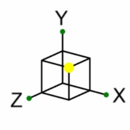

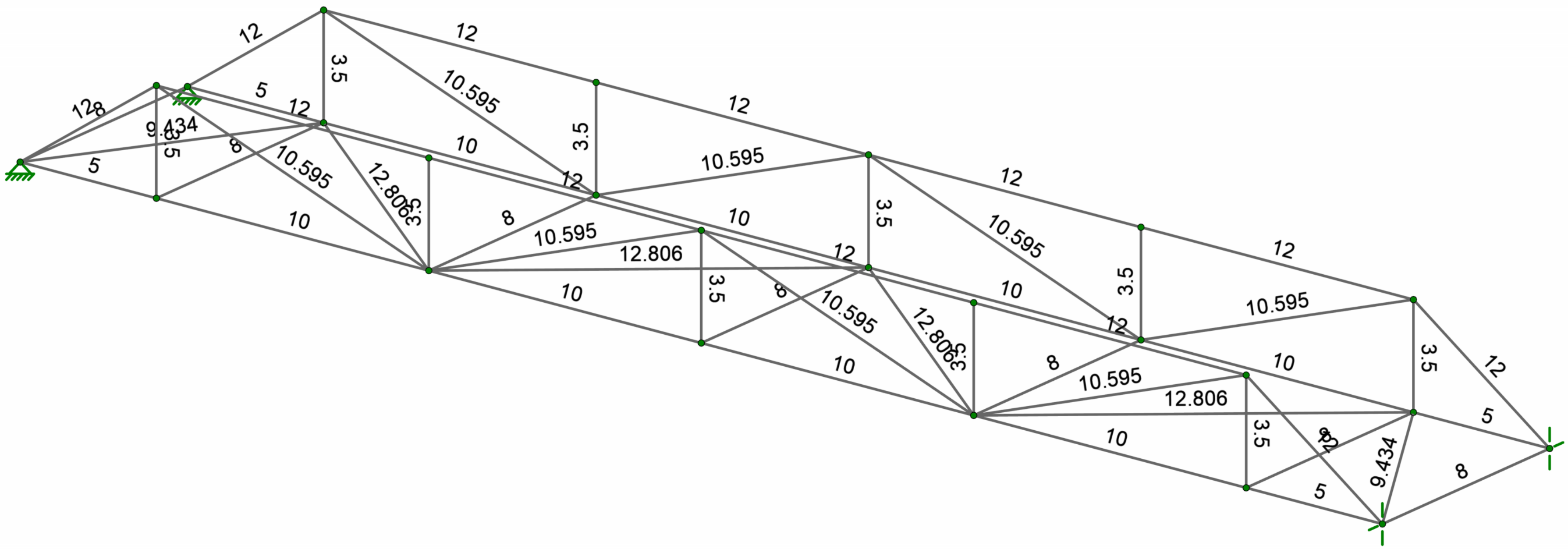

Member Lbyy (ft) Displayed
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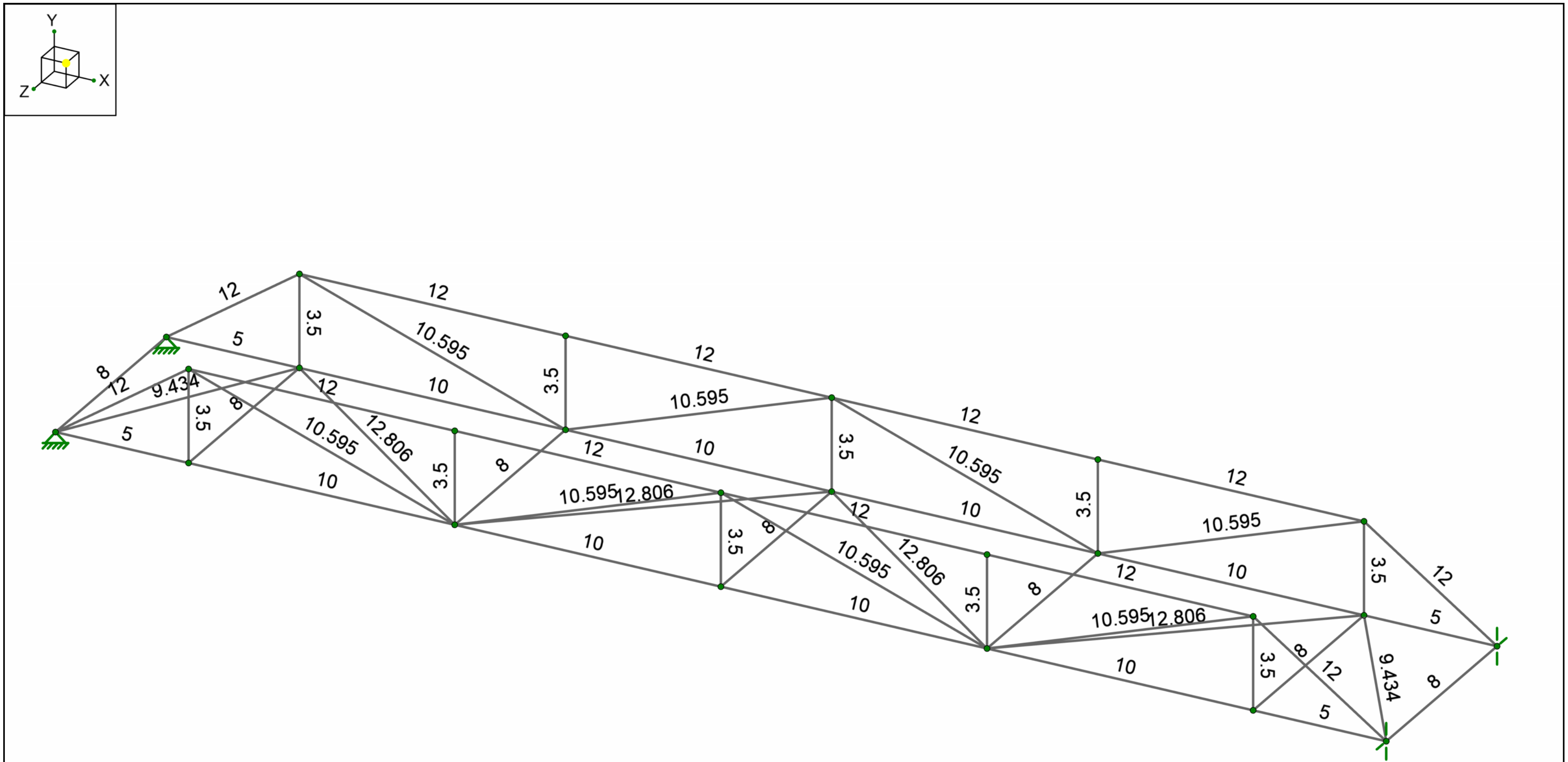

Member Lbzz (ft) Displayed

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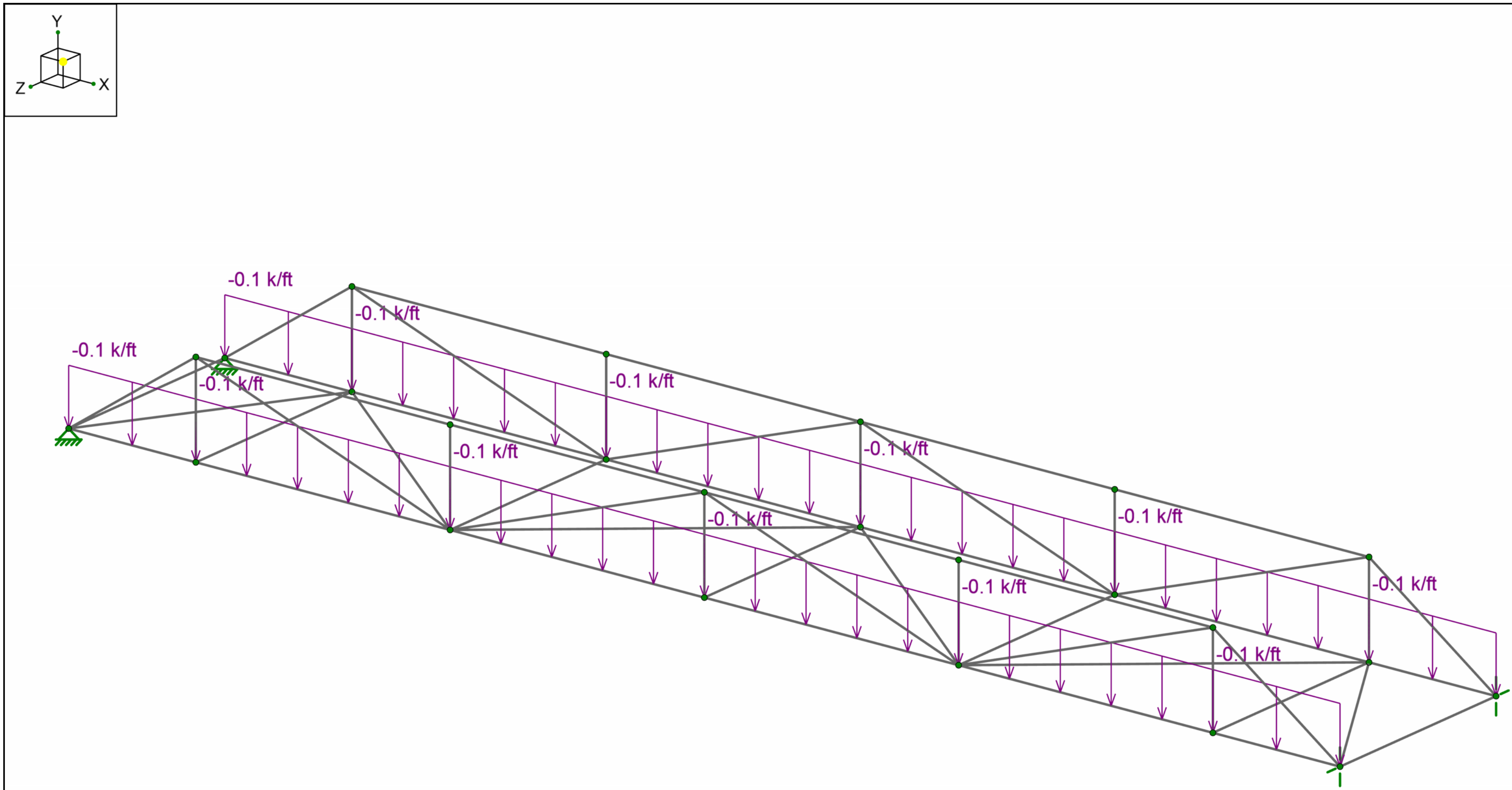

Loads: BLC 1, Bar Grate Load
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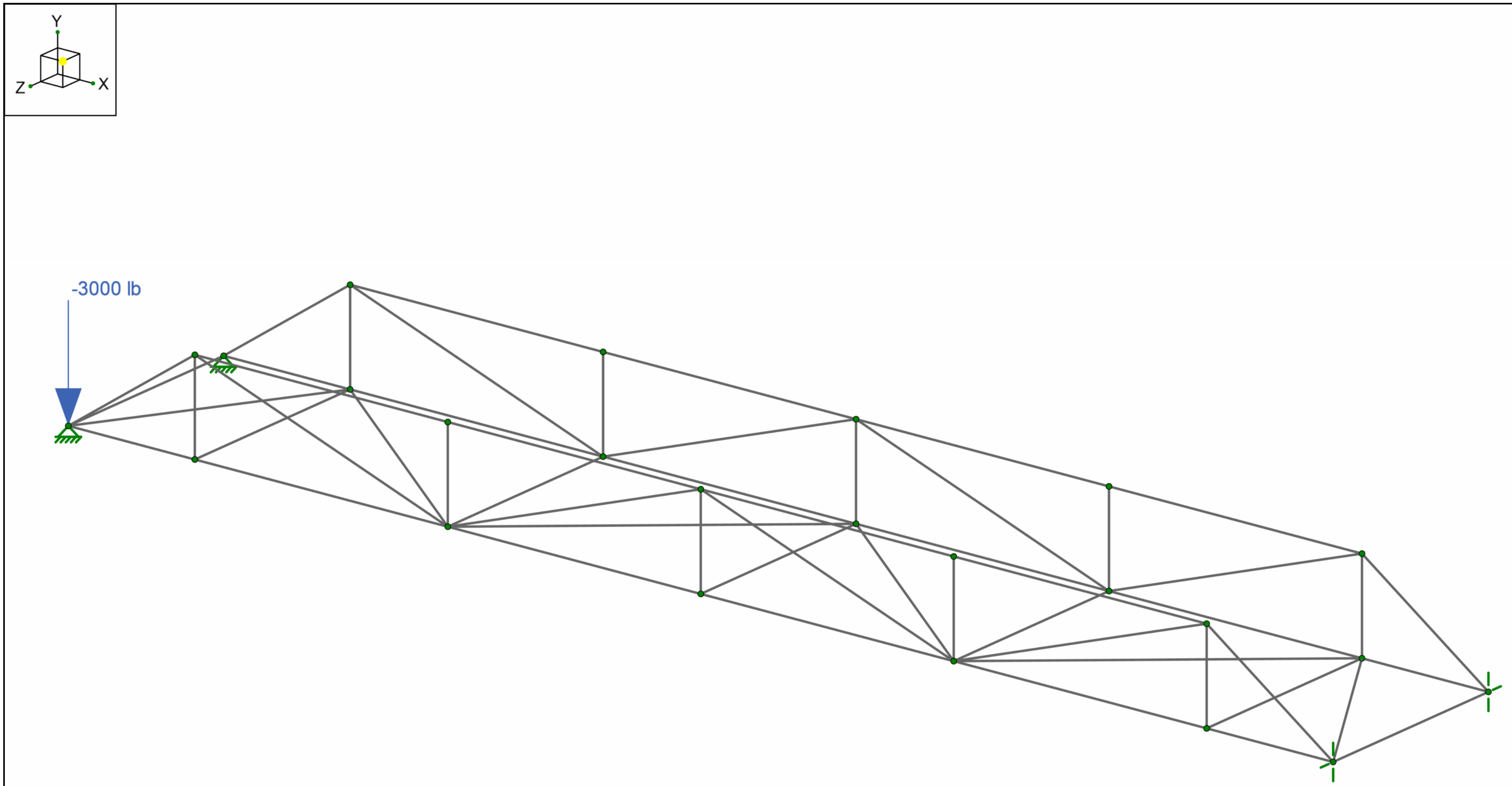
			PonyTruss Rev 1.r3d



Loads: BLC 2, UTV Moving Load
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Enterprise Engineering, Inc.		SK-6
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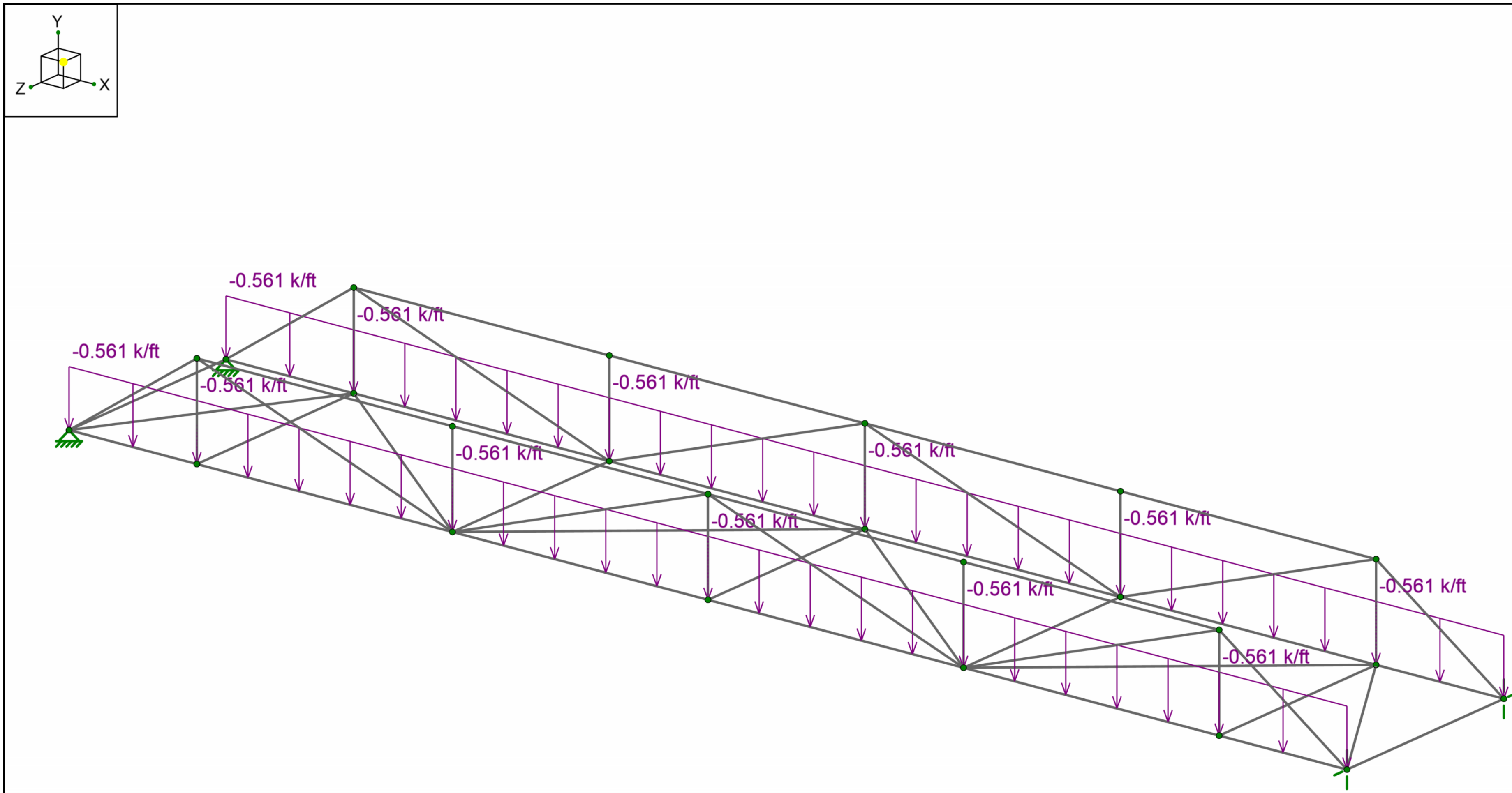
		PonyTruss Rev 1.r3d



Loads: BLC 3, Uniform Snow
Envelope Only Solution

RISA A NEMETSCHEK COMPANY	Enterprise Engineering, Inc.		SK-7
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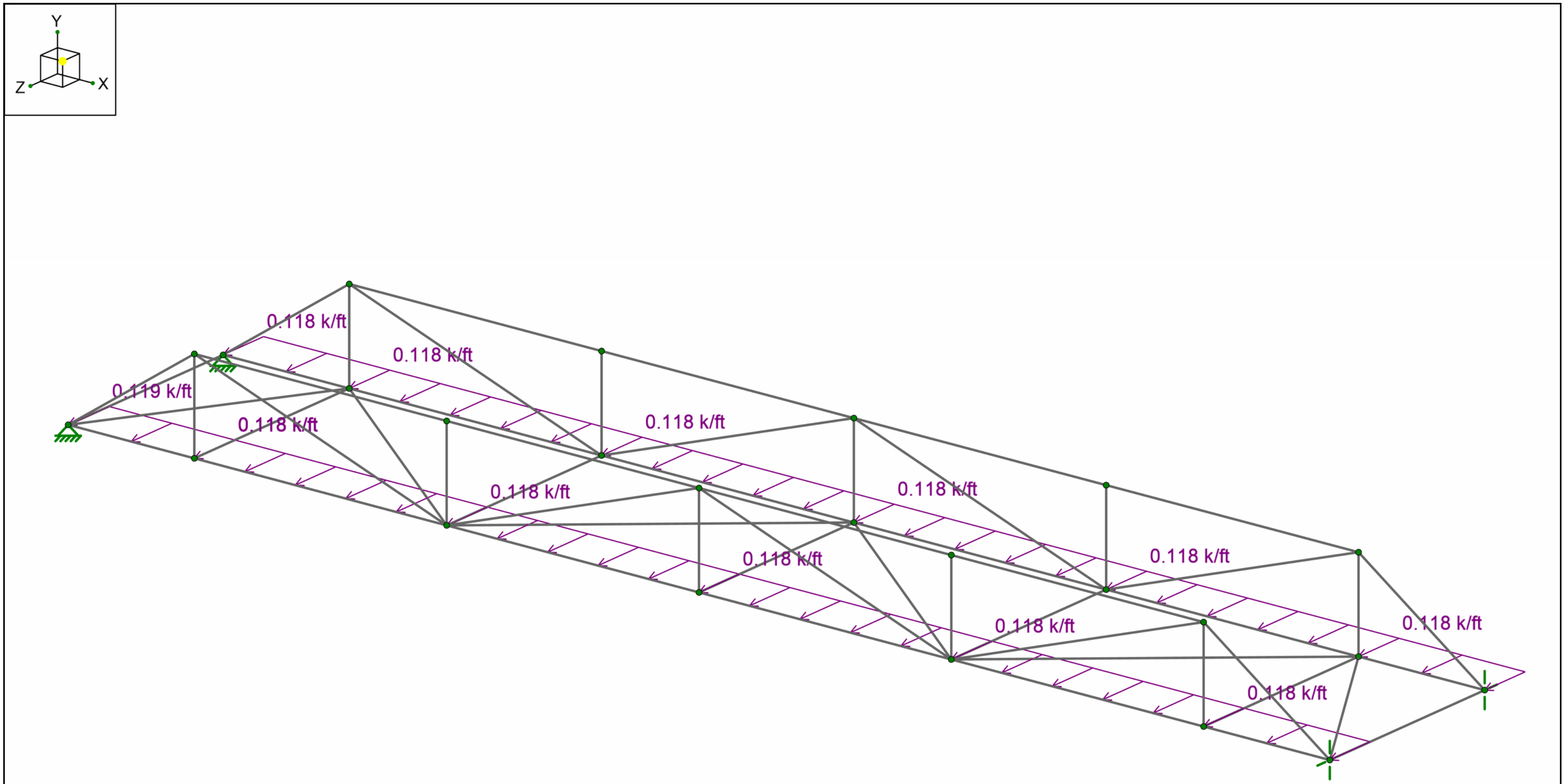

Loads: BLC 4, Seismic
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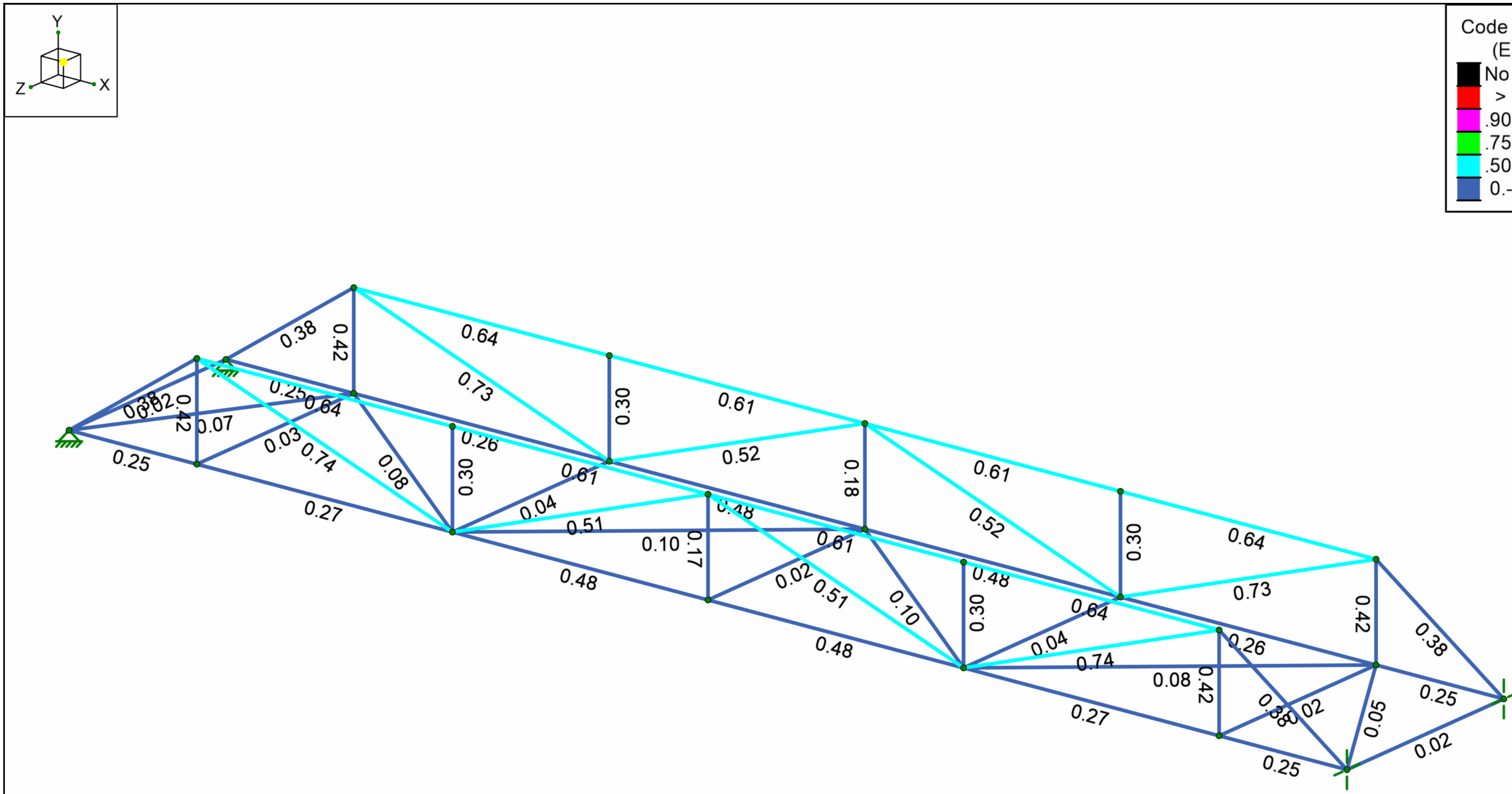

Code Check
(Env)
No Calc
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.90-1.0
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.50-.75
0.-.50

Member Code Checks Displayed (Enveloped)
Envelope Only Solution

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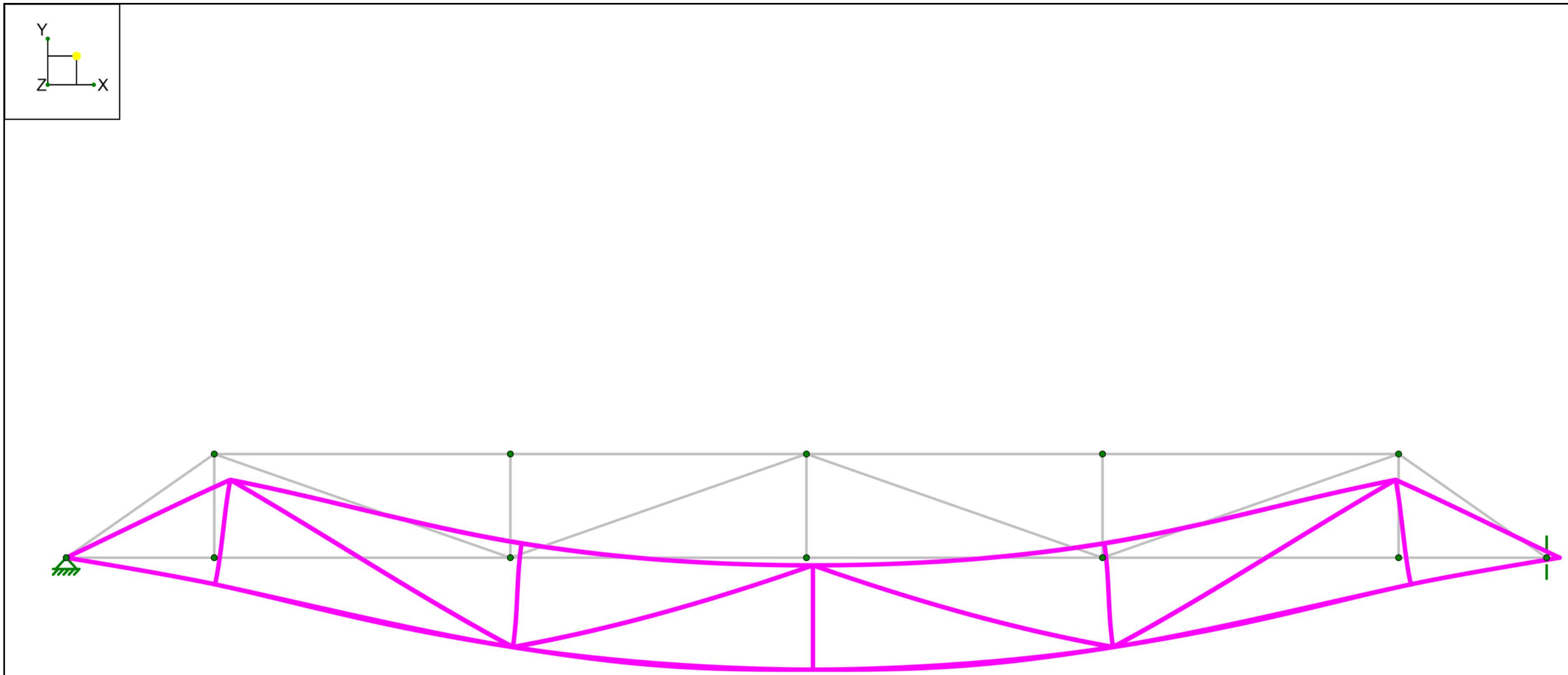


Results for LC 7, ASCE Strength 3

Enterprise Engineering, Inc.		SK-10
Nicholas F. Schwantes		Apr 22, 2024 at 01:30 PM
		PonyTruss Rev 1.r3d

Node Displacements

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	0	0	0	2.024e-5	-2.446e-4	-1.231e-3
2	1	N2	0.002	-0.079	0.013	3.868e-5	-1.939e-4	-1.565e-3
3	1	N3	0.005	-0.269	0.03	2.864e-4	-1.277e-4	-1.212e-3
4	1	N5	0.034	-0.272	0.034	3.435e-4	1.342e-4	1.183e-3
5	1	N4	0.02	-0.342	0.055	3.788e-4	-4.04e-6	-1.187e-5
6	1	N7	0.044	0	0	4.838e-5	3.147e-4	1.324e-3
7	1	N6	0.041	-0.084	0.017	1.259e-4	2.567e-4	1.609e-3
8	1	N8	0.047	-0.076	0.016	7.845e-5	-2.25e-4	-1.41e-3
9	1	N10	0.02	-0.34	0.061	1.578e-4	-2.684e-5	-1.056e-5
10	1	N12	-0.007	-0.081	0.021	1.093e-4	2.645e-4	1.455e-3
11	1	N11	0.034	-0.269	0.043	1.724e-4	-2.165e-4	-1.228e-3
12	1	N13	0.006	-0.272	0.048	1.912e-4	1.903e-4	1.193e-3
13	1	N14	-0.011	-0.068	0.02	6.814e-5	2.651e-4	1.275e-3
14	1	N16	0.026	-0.246	0.035	2.474e-4	1.341e-4	1.092e-3
15	1	N17	0	0	0	5.001e-6	-2.671e-4	-1.157e-3
16	1	N18	0.004	-0.074	0.013	8.18e-5	-1.693e-4	-1.468e-3
17	1	N19	0.008	-0.248	0.031	1.938e-4	-1.279e-4	-1.067e-3
18	1	N20	0.028	-0.07	0.016	1.639e-4	2.361e-4	1.439e-3
19	1	N21	0.018	-0.308	0.054	3.036e-4	-4.088e-6	7.763e-6
20	1	N22	0.03	0	0	4.407e-6	3.322e-4	1.084e-3
21	1	N23	0.044	-0.071	0.016	5.836e-5	-2.21e-4	-1.306e-3
22	1	N24	0.016	-0.305	0.058	1.133e-4	-2.569e-5	1.028e-5
23	1	N25	0.03	-0.248	0.041	1.382e-4	-2.019e-4	-1.072e-3
24	1	N26	0.003	-0.245	0.046	1.463e-4	1.797e-4	1.097e-3
25	2	N1	0	0	0	-1.388e-5	1.845e-4	-1.191e-3
26	2	N2	0.003	-0.075	-0.01	-6.271e-5	2.135e-4	-1.476e-3
27	2	N3	0.01	-0.248	-0.039	-1.395e-4	6.742e-5	-1.05e-3
28	2	N5	0.027	-0.245	-0.043	-1.97e-4	-7.392e-5	1.079e-3
29	2	N4	0.018	-0.304	-0.036	-3.501e-4	4.05e-6	1.214e-5
30	2	N7	0.031	0	0	-4.21e-5	-2.547e-4	1.098e-3
31	2	N6	0.03	-0.071	-0.014	-1.501e-4	-2.764e-4	1.431e-3
32	2	N8	0.045	-0.073	-0.014	-5.262e-5	2.371e-4	-1.324e-3
33	2	N10	0.018	-0.302	-0.05	-1.466e-4	2.693e-5	1.082e-5
34	2	N12	-0.01	-0.068	-0.019	-8.359e-5	-2.767e-4	1.279e-3
35	2	N11	0.031	-0.248	-0.04	-6.782e-5	1.526e-4	-1.038e-3
36	2	N13	0.004	-0.245	-0.046	-8.67e-5	-1.263e-4	1.074e-3
37	2	N14	-0.006	-0.081	-0.019	-6.608e-5	-2.891e-4	1.45e-3
38	2	N16	0.035	-0.27	-0.043	-2.795e-4	-8.078e-5	1.175e-3
39	2	N17	0	0	0	-7.791e-6	2.054e-4	-1.261e-3
40	2	N18	0	-0.08	-0.01	-1.822e-5	1.908e-4	-1.566e-3
41	2	N19	0.007	-0.268	-0.039	-2.255e-4	7.451e-5	-1.2e-3
42	2	N20	0.043	-0.084	-0.014	-1.005e-4	-2.577e-4	1.596e-3
43	2	N21	0.02	-0.34	-0.035	-4.247e-4	4.094e-6	-7.7e-6
44	2	N22	0.045	0	0	-7.173e-6	-2.705e-4	1.335e-3
45	2	N23	0.048	-0.078	-0.015	-5.634e-5	2.449e-4	-1.419e-3
46	2	N24	0.021	-0.338	-0.052	-1.704e-4	2.575e-5	-1.024e-5
47	2	N25	0.035	-0.268	-0.042	-8.034e-5	1.612e-4	-1.203e-3
48	2	N26	0.008	-0.27	-0.047	-8.85e-5	-1.389e-4	1.178e-3
49	3	N1	0	0	0	1.826e-5	-2.263e-4	-4.9e-4
50	3	N2	0	-0.032	0.012	4.706e-5	-1.999e-4	-6.35e-4
51	3	N3	0	-0.111	0.033	2.46e-4	-1.093e-4	-5.208e-4
52	3	N5	0.016	-0.114	0.037	3.037e-4	1.158e-4	4.918e-4
53	3	N4	0.008	-0.145	0.049	3.76e-4	-4.04e-6	-1.207e-5
54	3	N7	0.021	0	0	4.621e-5	2.963e-4	5.832e-4
55	3	N6	0.019	-0.036	0.016	1.344e-4	2.629e-4	6.796e-4

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
56	3	N8	0.019	-0.031	0.015	6.756e-5	-2.279e-4	-5.742e-4
57	3	N10	0.009	-0.144	0.057	1.384e-4	-2.634e-5	-1.074e-5
58	3	N12	-0.002	-0.036	0.02	9.707e-5	2.669e-4	6.194e-4
59	3	N11	0.014	-0.111	0.042	1.292e-4	-1.935e-4	-5.356e-4
60	3	N13	0.003	-0.114	0.047	1.464e-4	1.676e-4	4.999e-4
61	3	N14	-0.006	-0.022	0.019	6.279e-5	2.712e-4	4.389e-4
62	3	N16	0.007	-0.087	0.037	2.622e-4	1.179e-4	3.964e-4
63	3	N17	0	0	0	5.36e-6	-2.483e-4	-4.159e-4
64	3	N18	0.003	-0.027	0.012	6.334e-5	-1.759e-4	-5.381e-4
65	3	N19	0.003	-0.09	0.033	2.079e-4	-1.116e-4	-3.713e-4
66	3	N20	0.006	-0.023	0.016	1.456e-4	2.429e-4	5.085e-4
67	3	N21	0.006	-0.109	0.048	3.47e-4	-4.089e-6	7.923e-6
68	3	N22	0.007	0	0	4.429e-6	3.133e-4	3.425e-4
69	3	N23	0.016	-0.026	0.015	5.436e-5	-2.276e-4	-4.703e-4
70	3	N24	0.005	-0.108	0.056	1.15e-4	-2.517e-5	1.047e-5
71	3	N25	0.01	-0.09	0.041	1.093e-4	-1.863e-4	-3.74e-4
72	3	N26	0	-0.087	0.046	1.158e-4	1.643e-4	3.991e-4
73	4	N1	0	0	0	-1.575e-5	2.029e-4	-4.489e-4
74	4	N2	0.002	-0.028	-0.011	-5.64e-5	2.076e-4	-5.436e-4
75	4	N3	0.005	-0.09	-0.036	-1.89e-4	8.596e-5	-3.558e-4
76	4	N5	0.008	-0.086	-0.04	-2.471e-4	-9.247e-5	3.849e-4
77	4	N4	0.007	-0.106	-0.042	-3.653e-4	4.045e-6	1.234e-5
78	4	N7	0.008	0	0	-4.375e-5	-2.73e-4	3.555e-4
79	4	N6	0.008	-0.023	-0.015	-1.439e-4	-2.706e-4	4.986e-4
80	4	N8	0.017	-0.027	-0.015	-5.756e-5	2.325e-4	-4.857e-4
81	4	N10	0.006	-0.105	-0.053	-1.369e-4	2.64e-5	1.1e-5
82	4	N12	-0.005	-0.022	-0.02	-8.718e-5	-2.715e-4	4.402e-4
83	4	N11	0.011	-0.089	-0.041	-8.994e-5	1.7e-4	-3.428e-4
84	4	N13	0.001	-0.086	-0.046	-1.072e-4	-1.44e-4	3.785e-4
85	4	N14	-0.001	-0.036	-0.019	-6.197e-5	-2.803e-4	6.171e-4
86	4	N16	0.016	-0.112	-0.04	-2.747e-4	-9.719e-5	4.822e-4
87	4	N17	0	0	0	-6.425e-6	2.243e-4	-5.217e-4
88	4	N18	-0.001	-0.033	-0.011	-3.875e-5	1.843e-4	-6.382e-4
89	4	N19	0.002	-0.11	-0.037	-2.201e-4	9.091e-5	-5.073e-4
90	4	N20	0.021	-0.037	-0.015	-1.211e-4	-2.513e-4	6.678e-4
91	4	N21	0.008	-0.142	-0.041	-3.942e-4	4.091e-6	-7.851e-6
92	4	N22	0.022	0	0	-5.474e-6	-2.893e-4	5.95e-4
93	4	N23	0.02	-0.032	-0.015	-5.358e-5	2.366e-4	-5.857e-4
94	4	N24	0.01	-0.141	-0.053	-1.389e-4	2.521e-5	-1.042e-5
95	4	N25	0.015	-0.11	-0.042	-8.745e-5	1.714e-4	-5.075e-4
96	4	N26	0.004	-0.112	-0.046	-9.394e-5	-1.494e-4	4.824e-4
97	5	N1	0	0	0	1.966e-6	-1.818e-5	-7.304e-4
98	5	N2	0.001	-0.046	0.001	-7.25e-6	5.909e-6	-9.168e-4
99	5	N3	0.004	-0.156	-0.003	4.405e-5	-1.82e-5	-6.822e-4
100	5	N5	0.018	-0.156	-0.003	4.401e-5	1.819e-5	6.822e-4
101	5	N4	0.011	-0.195	0.006	8.372e-6	0	0
102	5	N7	0.023	0	0	1.95e-6	1.814e-5	7.304e-4
103	5	N6	0.021	-0.046	0.001	-7.302e-6	-5.947e-6	9.168e-4
104	5	N8	0.028	-0.045	0.001	7.855e-6	3.565e-6	-8.245e-4
105	5	N10	0.011	-0.194	0.003	2.284e-6	1.581e-8	0
106	5	N12	-0.005	-0.045	0.001	7.838e-6	-3.588e-6	8.245e-4
107	5	N11	0.02	-0.156	0.001	3.113e-5	-1.873e-5	-6.836e-4
108	5	N13	0.003	-0.156	0.001	3.112e-5	1.874e-5	6.836e-4
109	5	N14	-0.005	-0.045	0	4.933e-7	-7.176e-6	8.217e-4
110	5	N16	0.018	-0.155	-0.003	-9.83e-6	1.609e-5	6.838e-4

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
111	5	N17	0	0	0	-8.643e-7	-1.866e-5	-7.293e-4
112	5	N18	0.001	-0.046	0.001	1.915e-5	6.516e-6	-9.152e-4
113	5	N19	0.004	-0.155	-0.003	-9.798e-6	-1.61e-5	-6.838e-4
114	5	N20	0.021	-0.046	0.001	1.91e-5	-6.555e-6	9.152e-4
115	5	N21	0.011	-0.195	0.006	-3.683e-5	0	0
116	5	N22	0.023	0	0	-8.638e-7	1.862e-5	7.294e-4
117	5	N23	0.028	-0.045	0	4.986e-7	7.15e-6	-8.217e-4
118	5	N24	0.011	-0.194	0.002	-1.849e-5	1.511e-8	0
119	5	N25	0.02	-0.155	0	1.686e-5	-1.174e-5	-6.86e-4
120	5	N26	0.003	-0.155	0	1.685e-5	1.176e-5	6.86e-4
121	6	N1	0	0	0	5.328e-6	-5.164e-5	-2.089e-3
122	6	N2	0.004	-0.133	0.002	-2.066e-5	1.675e-5	-2.622e-3
123	6	N3	0.012	-0.446	-0.007	1.271e-4	-5.204e-5	-1.951e-3
124	6	N5	0.052	-0.446	-0.007	1.27e-4	5.203e-5	1.951e-3
125	6	N4	0.032	-0.558	0.016	2.607e-5	0	1.329e-8
126	6	N7	0.065	0	0	5.296e-6	5.156e-5	2.089e-3
127	6	N6	0.061	-0.133	0.002	-2.075e-5	-1.682e-5	2.622e-3
128	6	N8	0.08	-0.129	0.001	2.221e-5	1.077e-5	-2.359e-3
129	6	N10	0.032	-0.554	0.01	1.694e-5	3.11e-8	1.181e-8
130	6	N12	-0.015	-0.129	0.001	2.218e-5	-1.081e-5	2.358e-3
131	6	N11	0.056	-0.446	0.002	9.352e-5	-5.844e-5	-1.955e-3
132	6	N13	0.009	-0.446	0.002	9.349e-5	5.847e-5	1.955e-3
133	6	N14	-0.015	-0.129	0.001	1.829e-6	-2.123e-5	2.35e-3
134	6	N16	0.052	-0.445	-0.007	-2.719e-5	4.603e-5	1.955e-3
135	6	N17	0	0	0	-2.423e-6	-5.316e-5	-2.086e-3
136	6	N18	0.004	-0.133	0.002	5.497e-5	1.849e-5	-2.617e-3
137	6	N19	0.012	-0.445	-0.007	-2.713e-5	-4.604e-5	-1.955e-3
138	6	N20	0.061	-0.133	0.002	5.488e-5	-1.857e-5	2.617e-3
139	6	N21	0.032	-0.559	0.016	-1.033e-4	0	0
140	6	N22	0.065	0	0	-2.423e-6	5.308e-5	2.086e-3
141	6	N23	0.08	-0.129	0.001	1.842e-6	2.118e-5	-2.35e-3
142	6	N24	0.032	-0.555	0.006	-4.415e-5	2.98e-8	-1.134e-8
143	6	N25	0.056	-0.444	-0.001	5.18e-5	-3.766e-5	-1.962e-3
144	6	N26	0.009	-0.444	-0.001	5.179e-5	3.769e-5	1.961e-3
145	7	N1	0	0	0	1.139e-5	-1.286e-4	-5.309e-3
146	7	N2	0.011	-0.338	0.006	-5.193e-5	4.149e-5	-6.666e-3
147	7	N3	0.032	-1.133	-0.018	3.306e-4	-1.325e-4	-4.956e-3
148	7	N5	0.133	-1.133	-0.018	3.307e-4	1.325e-4	4.956e-3
149	7	N4	0.082	-1.418	0.041	8.154e-5	0	-1.693e-8
150	7	N7	0.165	0	0	1.144e-5	1.287e-4	5.309e-3
151	7	N6	0.154	-0.338	0.006	-5.18e-5	-4.139e-5	6.666e-3
152	7	N8	0.202	-0.328	0.003	5.412e-5	3.155e-5	-5.996e-3
153	7	N10	0.082	-1.409	0.032	1.161e-4	-4.612e-8	-1.497e-8
154	7	N12	-0.038	-0.328	0.003	5.418e-5	-3.149e-5	5.996e-3
155	7	N11	0.143	-1.133	0.006	2.69e-4	-1.836e-4	-4.969e-3
156	7	N13	0.022	-1.133	0.006	2.69e-4	1.835e-4	4.969e-3
157	7	N14	-0.037	-0.327	0.002	6.522e-6	-5.904e-5	5.971e-3
158	7	N16	0.133	-1.129	-0.018	-6.248e-5	1.173e-4	4.961e-3
159	7	N17	0	0	0	-6.174e-6	-1.335e-4	-5.298e-3
160	7	N18	0.011	-0.337	0.006	1.411e-4	4.599e-5	-6.65e-3
161	7	N19	0.032	-1.129	-0.018	-6.256e-5	-1.173e-4	-4.961e-3
162	7	N20	0.154	-0.337	0.006	1.412e-4	-4.589e-5	6.65e-3
163	7	N21	0.082	-1.418	0.041	-2.483e-4	0	1.067e-8
164	7	N22	0.165	0	0	-6.172e-6	1.336e-4	5.298e-3
165	7	N23	0.202	-0.327	0.002	6.495e-6	5.912e-5	-5.971e-3

Node Displacements (Continued)

	LC	Node Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
166	7	N24	0.082	-1.409	0.02	-5.201e-5	-4.442e-8	1.433e-8
167	7	N25	0.142	-1.129	-0.002	1.558e-4	-1.249e-4	-4.981e-3
168	7	N26	0.022	-1.129	-0.002	1.558e-4	1.248e-4	4.981e-3
169	8	N1	0	0	0	8.251e-6	-8.458e-5	-3.448e-3
170	8	N2	0.007	-0.219	0.004	-3.395e-5	2.736e-5	-4.329e-3
171	8	N3	0.021	-0.736	-0.012	2.117e-4	-8.592e-5	-3.219e-3
172	8	N5	0.086	-0.736	-0.012	2.116e-4	8.592e-5	3.219e-3
173	8	N4	0.054	-0.921	0.027	4.691e-5	0	0
174	8	N7	0.107	0	0	8.229e-6	8.453e-5	3.448e-3
175	8	N6	0.1	-0.219	0.004	-3.401e-5	-2.741e-5	4.329e-3
176	8	N8	0.132	-0.213	0.002	3.612e-5	1.883e-5	-3.893e-3
177	8	N10	0.054	-0.915	0.018	4.665e-5	2.242e-8	0
178	8	N12	-0.024	-0.213	0.002	3.609e-5	-1.886e-5	3.893e-3
179	8	N11	0.093	-0.736	0.004	1.624e-4	-1.053e-4	-3.227e-3
180	8	N13	0.014	-0.736	0.004	1.624e-4	1.054e-4	3.227e-3
181	8	N14	-0.024	-0.212	0.001	3.61e-6	-3.633e-5	3.878e-3
182	8	N16	0.087	-0.734	-0.012	-4.321e-5	7.604e-5	3.225e-3
183	8	N17	0	0	0	-3.965e-6	-8.735e-5	-3.442e-3
184	8	N18	0.007	-0.219	0.004	9.109e-5	3.026e-5	-4.32e-3
185	8	N19	0.021	-0.734	-0.012	-4.317e-5	-7.605e-5	-3.225e-3
186	8	N20	0.1	-0.219	0.004	9.102e-5	-3.031e-5	4.32e-3
187	8	N21	0.054	-0.922	0.027	-1.669e-4	0	0
188	8	N22	0.107	0	0	-3.965e-6	8.73e-5	3.442e-3
189	8	N23	0.131	-0.212	0.001	3.621e-6	3.63e-5	-3.878e-3
190	8	N24	0.054	-0.916	0.011	-5.737e-5	2.155e-8	0
191	8	N25	0.093	-0.733	-0.002	9.177e-5	-6.957e-5	-3.237e-3
192	8	N26	0.015	-0.733	-0.002	9.176e-5	6.959e-5	3.237e-3

Envelope AISC 15TH (360-16): ASD Member Steel Code Checks

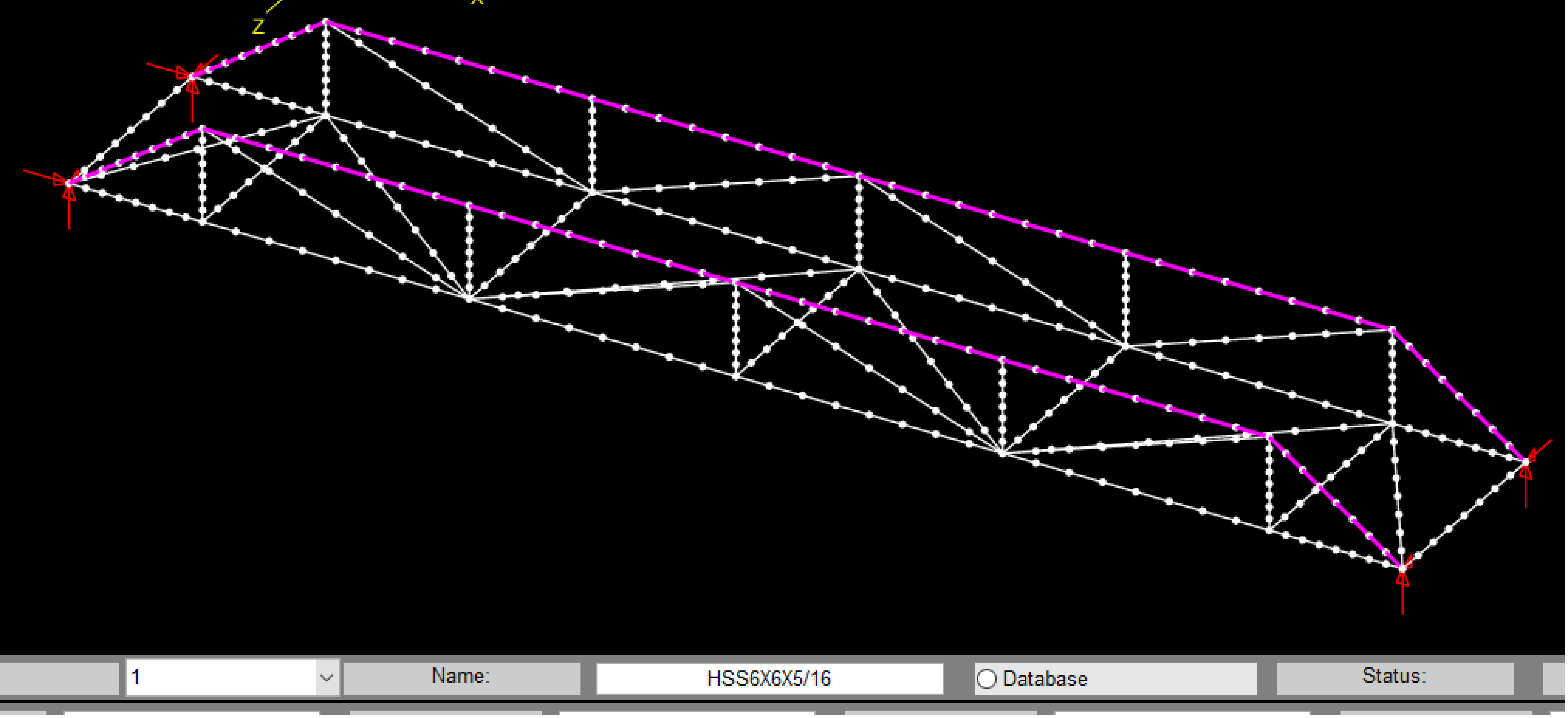
	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [lb]	Pnt/om [lb]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
1	M1	HSS6X6X5	0.38	6.103	7	0.016	6.103	y	7	144886.29	192514.97	33.932	33.932	2.181	H1-1a
2	M2	HSS3.000X0.250	0.739	10.595	7	0.004	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
3	M3	HSS3.000X0.250	0.507	0	7	0.002	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
4	M4	HSS3.000X0.250	0.507	10.595	7	0.002	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
5	M5	HSS3.000X0.250	0.739	0	7	0.004	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
6	M6	HSS6X6X5	0.38	0	7	0.016	6.103	y	7	144886.29	192514.97	33.932	33.932	2.181	H1-1a
7	M7	HSS3.000X0.250	0.42	3.5	7	0.032	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1a
8	M12	HSS6X6X5	0.639	10	7	0.018	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
9	M9	HSS3.000X0.250	0.173	3.5	7	0.006	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1b*
10	M11	HSS3.000X0.250	0.42	3.5	7	0.032	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1a
11	M14	HSS6X6X5	0.612	10	7	0.005	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
12	M16	W8X31	0.249	5	7	0.104	5	y	7	265973.555	273353.293	35.124	75.767	2.113	H1-1b
13	M19	W8X31	0.483	4.583	7	0.116	0	y	7	245008.357	273353.293	35.124	75.767	1.134	H1-1a
14	M17	W8X31	0.269	0	7	0.146	0	y	7	245008.357	273353.293	35.124	75.767	1.919	H1-1b
15	M18	W8X31	0.483	5.417	7	0.116	10	y	7	245008.357	273353.293	35.124	75.767	1.134	H1-1a
16	M20	W8X31	0.269	10	7	0.146	10	y	7	245008.357	273353.293	35.124	75.767	1.919	H1-1b
17	M21	W8X31	0.249	0	7	0.104	0	y	7	265973.555	273353.293	35.124	75.767	2.113	H1-1b
18	M22	HSS6X6X5	0.612	0	7	0.005	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
19	M23	HSS6X6X5	0.639	0	7	0.018	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
20	M24	HSS3.000X0.250	0.303	3.5	7	0.043	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1b
21	M25	HSS3.000X0.250	0.303	3.5	7	0.043	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1b
22	M27	W8X31	0.485	5.521	7	0.115	10	y	7	245008.357	273353.293	35.124	75.767	1.133	H1-1a
23	M30	HSS6X6X5	0.381	0	7	0.016	6.103	y	7	144886.29	192514.97	33.932	33.932	2.182	H1-1a
24	M31	HSS6X6X5	0.636	0	7	0.017	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
25	M32	HSS6X6X5	0.609	10	7	0.005	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a

Envelope AISC 15TH (360-16): ASD Member Steel Code Checks (Continued)

	Member	Shape	Code Check	Loc[ft]	LC	Shear Check	Loc[ft]	Dir	LC	Pnc/om [lb]	Pnt/om [lb]	Mnyy/om [k-ft]	Mnzz/om [k-ft]	Cb	Eqn
26	M33	HSS6X6X5	0.381	6.103	7	0.016	6.103	y	7	144886.29	192514.97	33.932	33.932	2.182	H1-1a
27	M34	HSS3.000X0.250	0.735	10.595	7	0.002	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
28	M35	HSS3.000X0.250	0.519	0	7	0.001	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
29	M36	HSS3.000X0.250	0.519	10.595	7	0.001	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
30	M37	HSS3.000X0.250	0.735	0	7	0.002	10.595		7	34660.988	55916.168	4.109	4.109	1	H1-1a
31	M38	HSS3.000X0.250	0.42	3.5	7	0.032	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1a
32	M39	HSS6X6X5	0.636	10	7	0.017	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
33	M40	HSS3.000X0.250	0.179	3.5	7	0.007	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1b*
34	M41	HSS3.000X0.250	0.42	3.5	7	0.032	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1a
35	M42	W8X31	0.251	5	7	0.104	5	y	7	265973.555	273353.293	35.124	75.767	2.118	H1-1b
36	M43	W8X31	0.484	4.479	7	0.115	0	y	7	245008.357	273353.293	35.124	75.767	1.133	H1-1a
37	M44	W8X31	0.265	0	7	0.145	0	y	7	245008.357	273353.293	35.124	75.767	1.915	H1-1b
38	M45	W8X31	0.265	10	7	0.145	10	y	7	245008.357	273353.293	35.124	75.767	1.915	H1-1b
39	M46	W8X31	0.251	0	7	0.104	0	y	7	265973.555	273353.293	35.124	75.767	2.118	H1-1b
40	M47	HSS6X6X5	0.609	0	7	0.005	10	y	7	144886.29	192514.97	33.932	33.932	1	H1-1a
41	M48	HSS3.000X0.250	0.305	0	7	0.042	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1b
42	M49	HSS3.000X0.250	0.305	0	7	0.042	3.5		7	53072.701	55916.168	4.109	4.109	1	H1-1b
43	M50	W8X31	0.026	8	2	0.002	4	y	7	254856.977	273353.293	35.124	75.767	1.58	H1-1b
44	M51	W8X31	0.043	0	7	0.006	4.083	y	7	254856.977	273353.293	35.124	75.767	1.802	H1-1b
45	M52	W8X31	0.023	8	7	0.005	5.25	y	7	254856.977	273353.293	35.124	75.767	1.784	H1-1b
46	M53	W8X31	0.043	0	7	0.006	4.083	y	7	254856.977	273353.293	35.124	75.767	1.802	H1-1b
47	M54	W8X31	0.022	8	2	0.002	4	y	7	254856.977	273353.293	35.124	75.767	1.726	H1-1b
48	M56	W6X15	0.083	12.806	7	0.008	6.403	y	7	93768.55	132634.731	10.834	25.364	2.159	H1-1b
49	M57	W6X15	0.104	0	7	0.006	6.403	z	7	93768.55	132634.731	10.834	21.603	1.07	H1-1b
50	M61	W6X15	0.083	12.806	7	0.008	6.403	y	7	93768.55	132634.731	10.834	25.364	2.159	H1-1b
51	M62	W6X15	0.104	0	7	0.006	6.403	z	7	93768.55	132634.731	10.834	21.603	1.07	H1-1b
52	M58	W6X15	0.07	9.434	3	0.005	4.717	y	7	109882.663	132634.731	10.834	25.364	1.711	H1-1b*
53	M59	C8X11.5	0.015	8	1	0.001	4	y	1	48507.242	100898.204	3.098	24.027	2.202	H1-1b
54	M60	C8X11.5	0.015	8	1	0.002	4	y	1	48507.242	100898.204	3.098	24.027	2.182	H1-1b
55	M63	W6X15	0.046	9.434	3	0.005	4.717	y	7	109882.663	132634.731	10.834	25.364	1.164	H1-1b*

Material Take-Off

	Material	Size	Pieces	Length[ft]	Weight[K]
1	Hot Rolled Steel				
2	A500 Gr.C RECT	HSS6X6X5	12	104.4	2.457
3	A500 Gr.C RND	HSS3.000X0.250	18	119.8	0.89
4	A992	C8X11.5	2	16	0.183
5	A992	W6X15	6	70.1	1.057
6	A992	W8X31	17	140	4.349
7	Total HR Steel		55	450.3	8.936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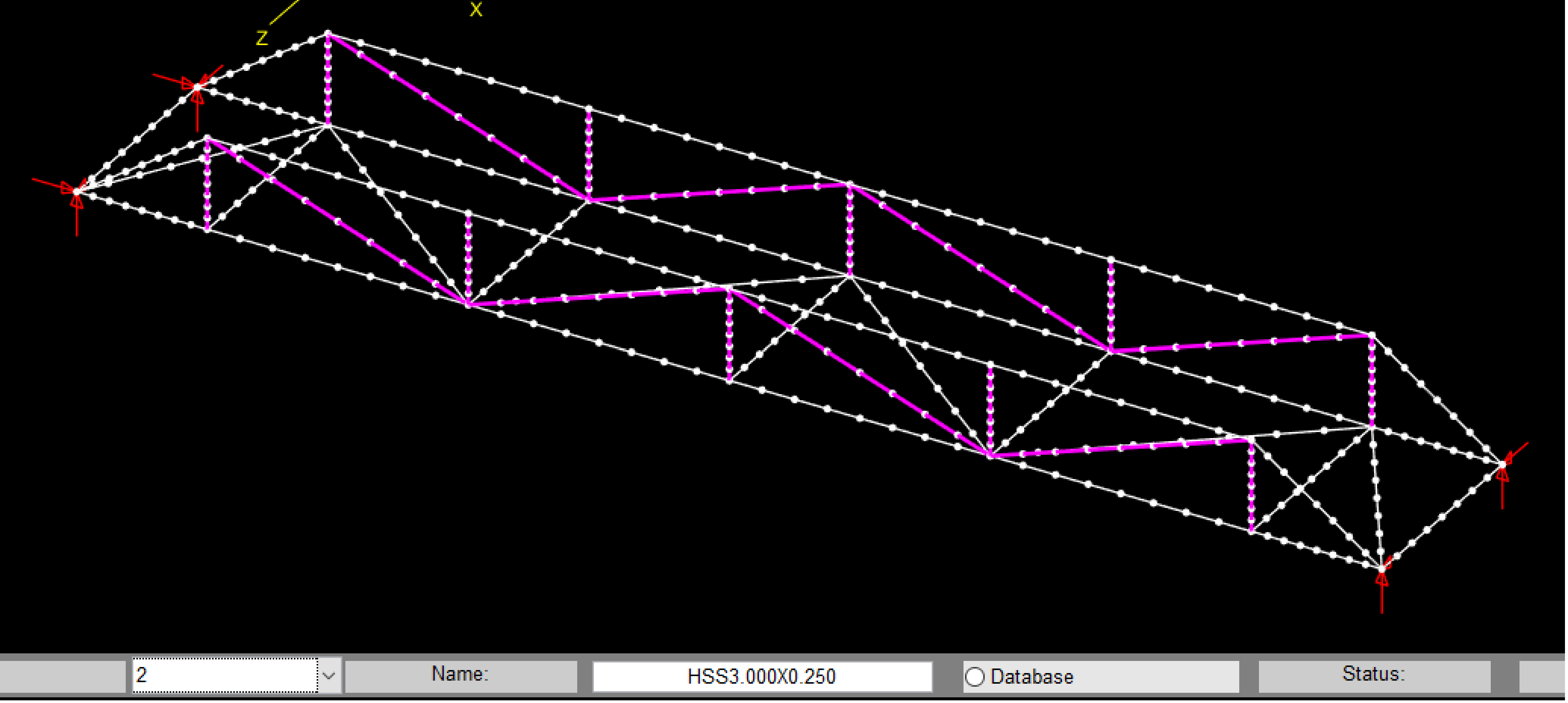
X
1
Name:
HSS6X6X5/16
Database
Status: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X
2
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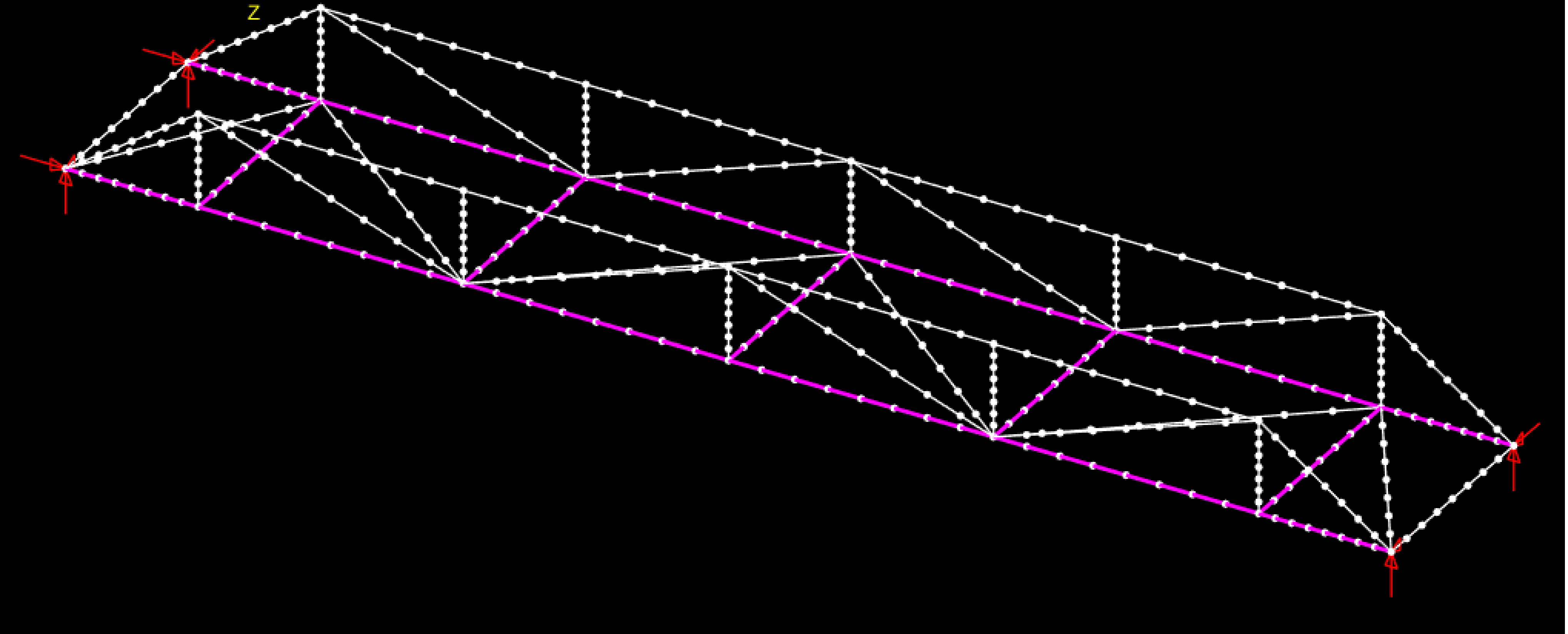

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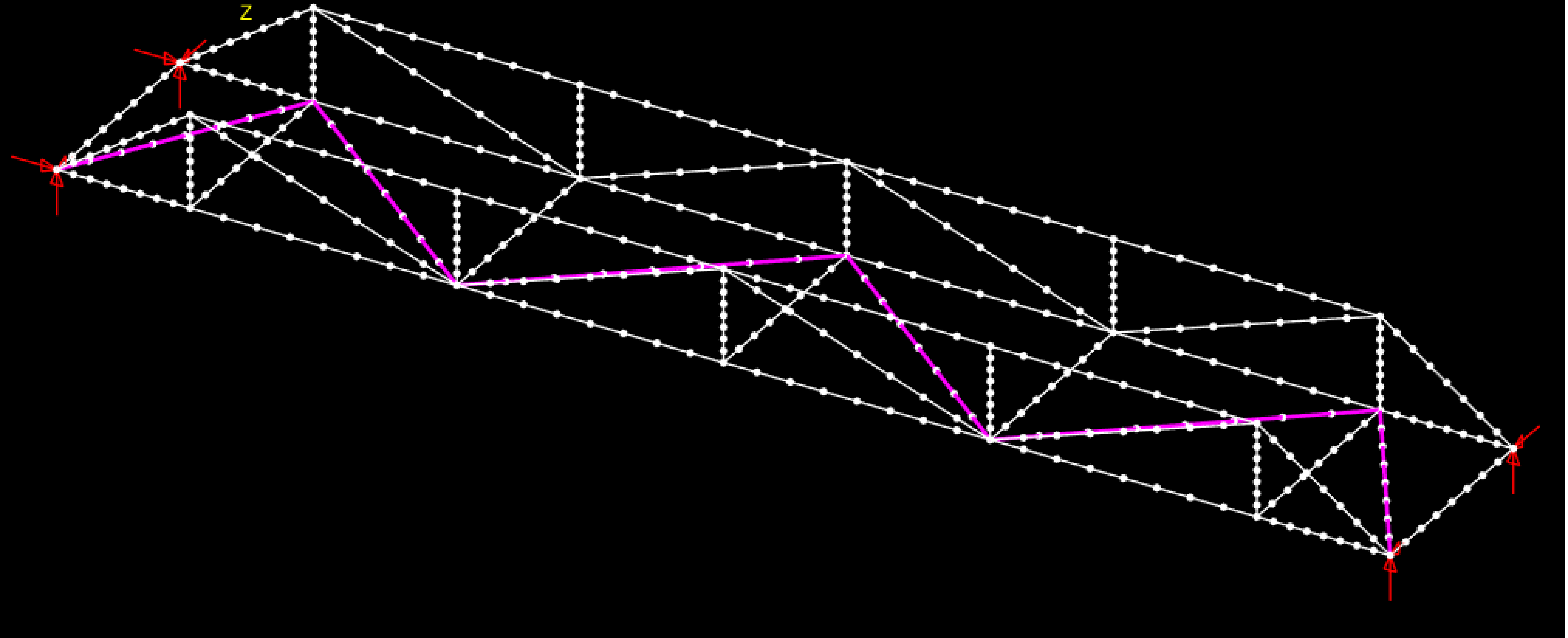

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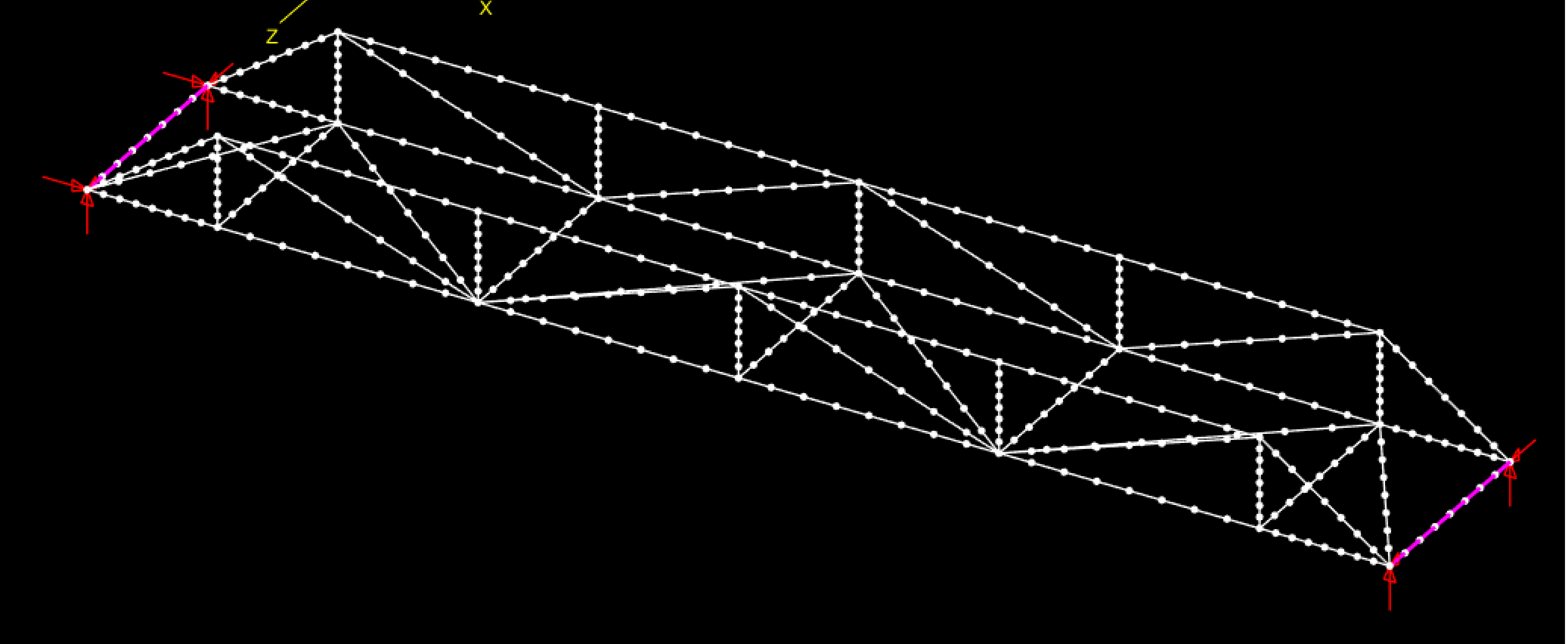

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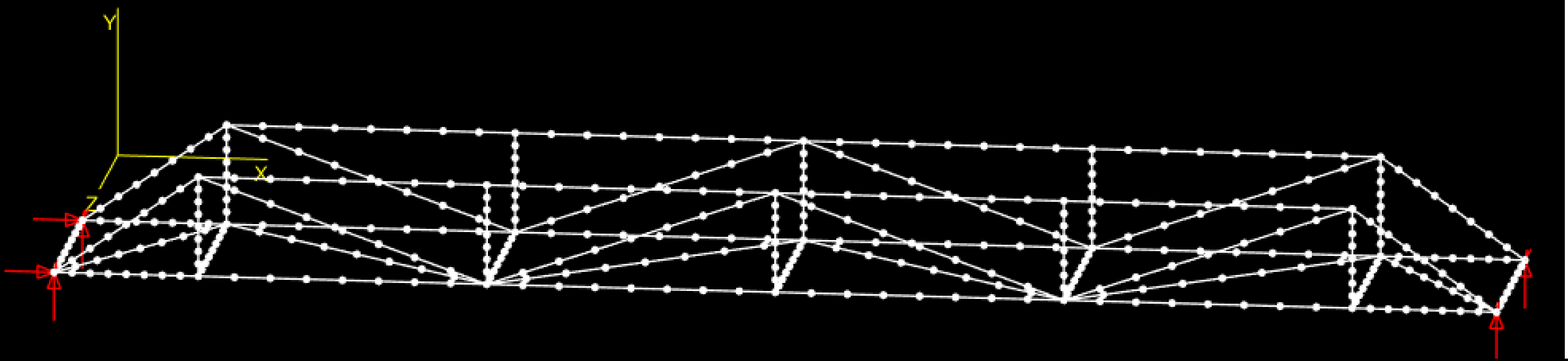
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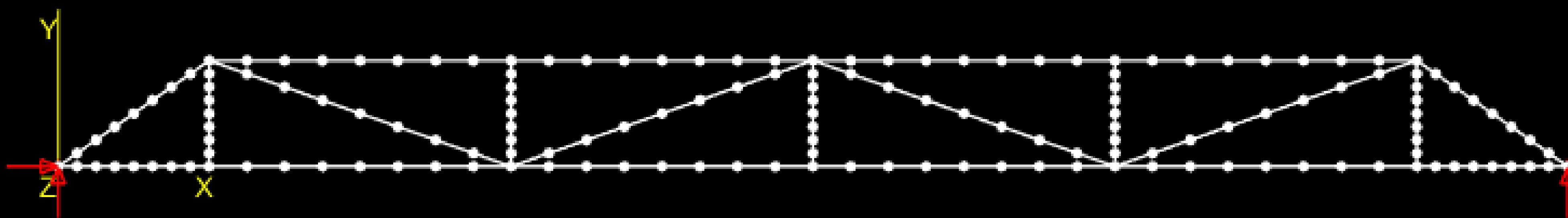
X
Z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Z
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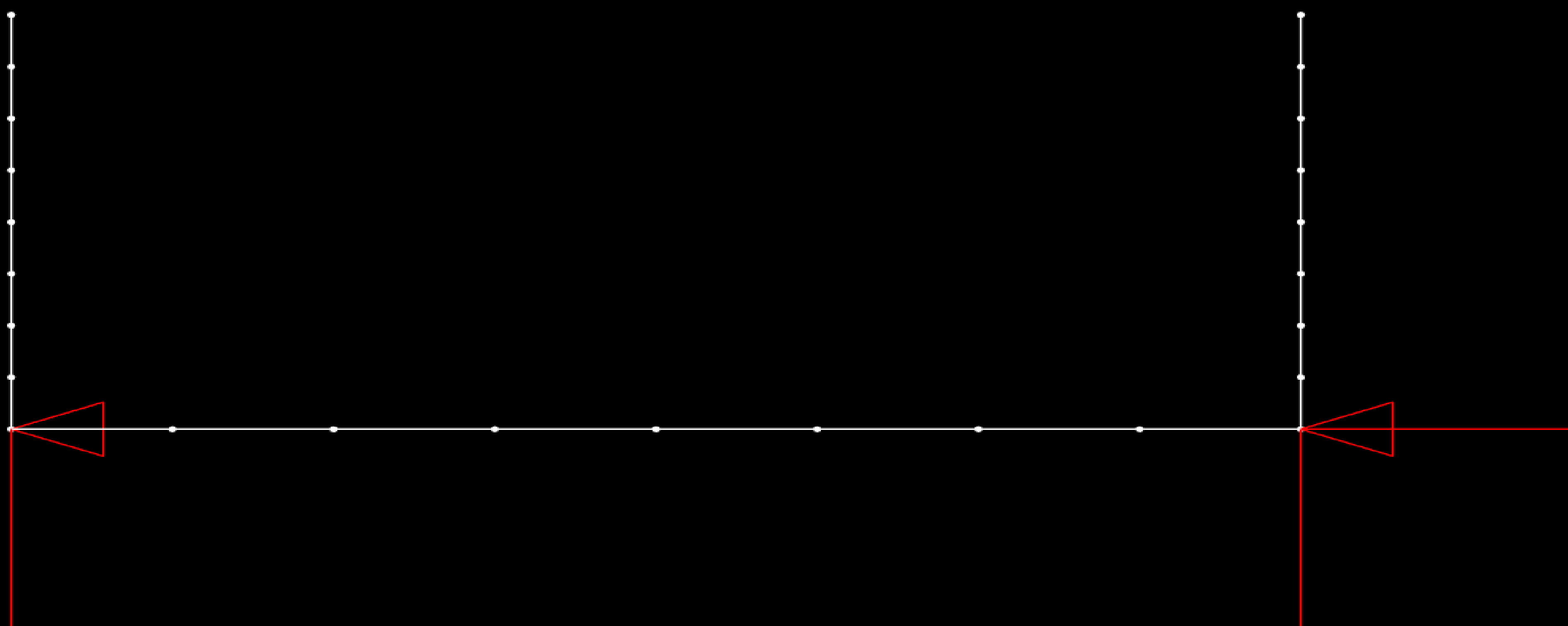
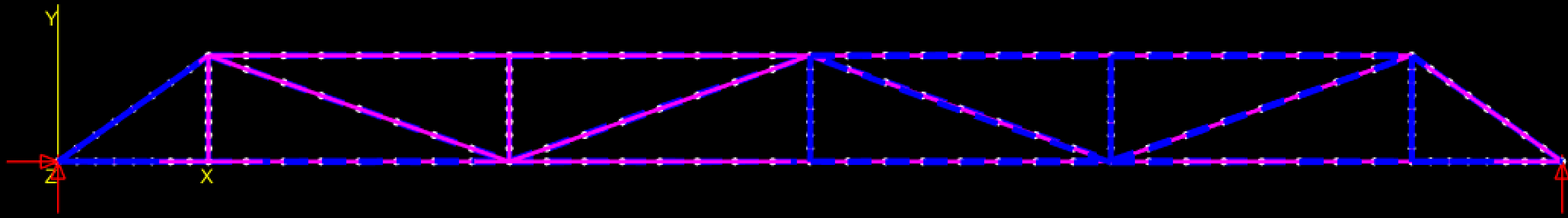
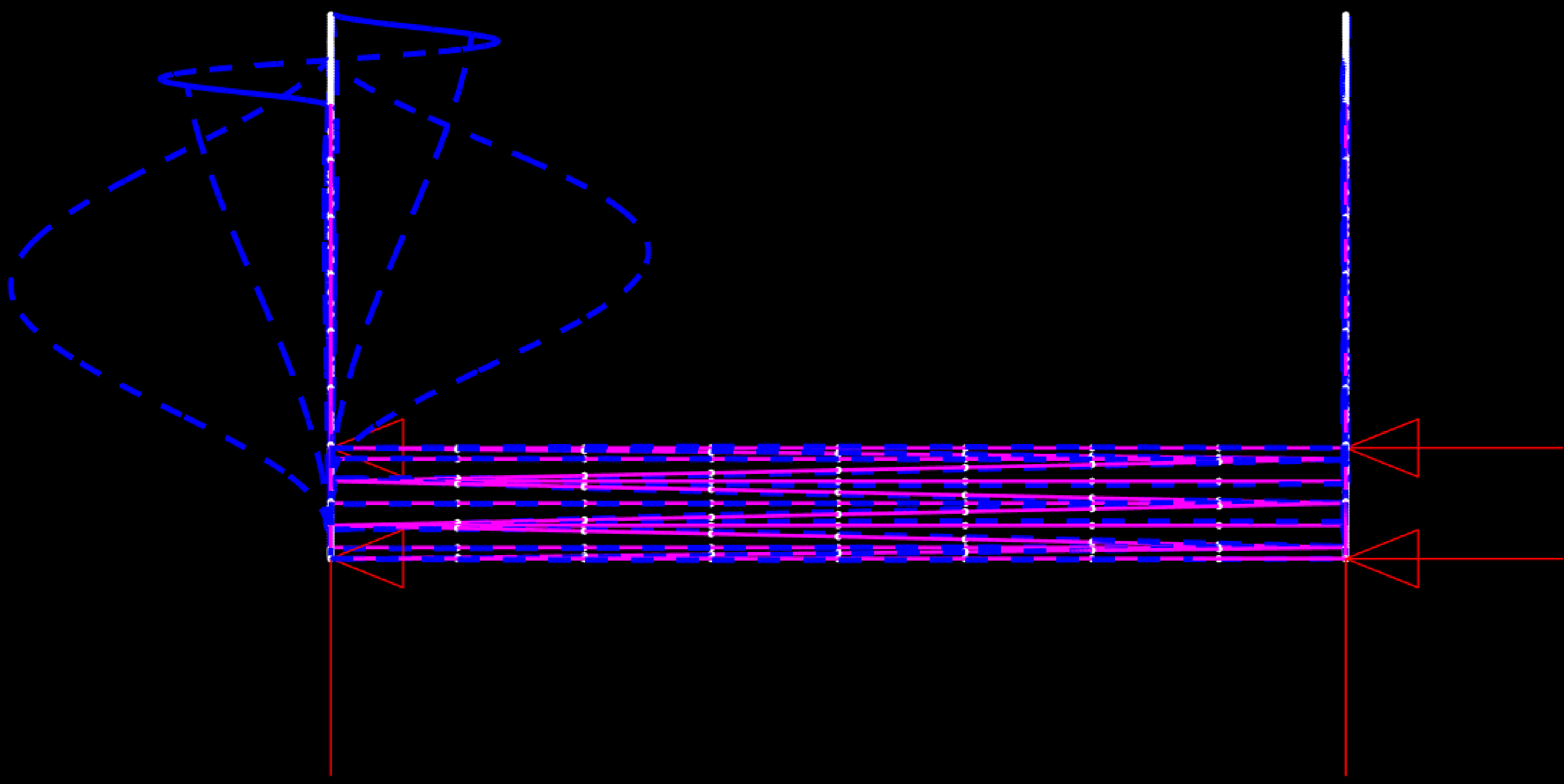


Deflected Shape: Elastic Critical Load, Mode # 1, Applied Load Ratio = 5.5539



Deflected Shape: Elastic Critical Load, Mode # 1, Applied Load Ratio = 5.5539



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***********   MASTAN2 v5.1.26 (August 22, 2022)   ***********

Time:  14:04:49      Date:  04/22/2024

Problem Title:   not provided
**************

=============================
Input for Structural Analysis
=============================

General Information Categories:

(i)   Number of Nodes = 409
(ii)  Number of Elements = 440
(iii) Number of Sections = 5
(iv)  Number of Materials = 3
(v)   Number of Supports = 4
(vi)  Applied Loads



(iii) Section Information
```

Part 1: Properties

Number	Area	Izz	Iyy	J	Cw
1	6.4300e+00	3.4300e+01	3.4300e+01	5.5400e+01	0.0000e+00
2	2.0300e+00	1.9500e+00	1.9500e+00	3.9000e+00	0.0000e+00
3	9.1300e+00	1.1000e+02	3.7100e+01	5.3600e-01	5.3000e+02
4	4.4300e+00	2.9100e+01	9.3200e+00	1.0100e-01	7.6500e+01
5	3.3700e+00	3.2500e+01	1.3100e+00	1.3000e-01	1.6500e+01

Part 2: Properties (cont.)

Number	Ysc	Zsc	Betay	Betaz	Betaw
1	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00
2	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00
3	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00
4	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00
5	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00	0.0000e+00

Part 3: Properties (cont.)

Number	Zzz	Zyy	Ayy	Azz	Phi
1	1.3600e+01	1.3600e+01	Inf	Inf	0.0000e+00
2	1.7900e+00	1.7900e+00	Inf	Inf	0.0000e+00
3	3.0400e+01	1.4100e+01	Inf	Inf	0.0000e+00
4	1.0800e+01	4.7500e+00	Inf	Inf	0.0000e+00
5	9.6300e+00	1.5700e+00	Inf	Inf	0.0000e+00

Part 4: Properties (cont.)

Number	Name
1	HSS6X6X5/16
2	HSS3.000X0.250
3	W8X31
4	W6X15
5	C8X11.5

Part 5: Yield Surface Maximum Values

Number	P/Py	Mz/Mpz	Mz/Mpy
1	1.0000e+00	1.0000e+00	1.0000e+00
2	1.0000e+00	1.0000e+00	1.0000e+00
3	1.0000e+00	1.0000e+00	1.0000e+00
4	1.0000e+00	1.0000e+00	1.0000e+00
5	1.0000e+00	1.0000e+00	1.0000e+00

(iv) Material Information

Number	E	v	Fy	Wt	Name
1	2.9000e+07	3.0000e-01	5.0000e+04	2.8358e-01	ASTM A992
2	2.9000e+07	3.0000e-01	5.0000e+04	2.8358e-01	ASTM A500 Gr. C RECT
3	2.9000e+07	3.0000e-01	4.6000e+04	2.8358e-01	ASTM A500 Gr. C RND

====================================

End of Input for Structural Analysis

```
====================================

##############################
Results of Structural Analysis
##############################

General Information:
      Structure Analyzed as:  Space Frame
      Analysis Type:  Elastic Critical Load

Analytical Results:


(iii) Reactions at Step # 1, Applied Load Ratio = 5.5539

  Forces
     Node        Rx             Ry             Rz
       1    -1.6185e+04    1.5287e+05   -8.7413e+02
       7       FREE        1.5287e+05   -8.7584e+02
      13     1.6185e+04    1.6121e+05   -1.7155e+03
      24       FREE        1.6120e+05    3.4655e+03

  Moments
     Node        Mx             My             Mz
```

*** No Reaction Moments Exist ***

######################################

End of Results of Structural Analysis

######################################

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Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 26, 2023	Date:	4/22/2024
Fastening point:			

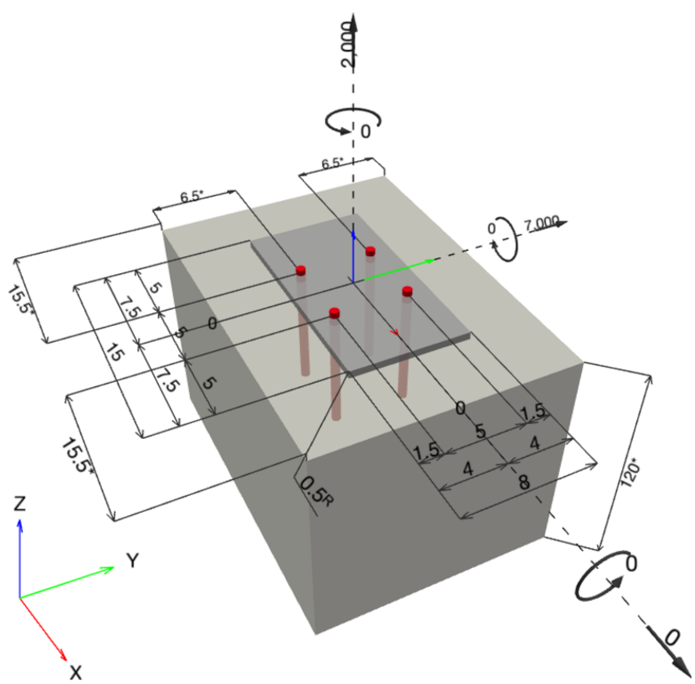
Specifier's comments: This design is for the anchor bolts that are holding the longitudinal beams down to the embedded concrete piers of this bridge.

1 Input data

Anchor type and diameter:	**Heavy Hex Head ASTM F 1554 GR. 36 5/8**
Item number:	not available
Effective embedment depth:	h_{ef} = 8.000 in.
Material:	ASTM F 1554
Evaluation Service Report:	Hilti Technical Data
Issued I Valid:	- | -
Proof:	Design Method ACI 318-19 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate[R] :	l_x x l_y x t = 15.000 in. x 8.000 in. x 0.500 in.; (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete, Custom, f_c' = 4,500 psi; h = 120.000 in.
Reinforcement:	tension: not present, shear: present; anchor reinforcement: shear
	edge reinforcement: > No. 4 bar with stirrups

[R] - The anchor calculation is based on a rigid anchor plate assumption.

Geometry [in.] & Loading [lb, in.lb]



Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 26, 2023	Date:	4/22/2024
Fastening point:			

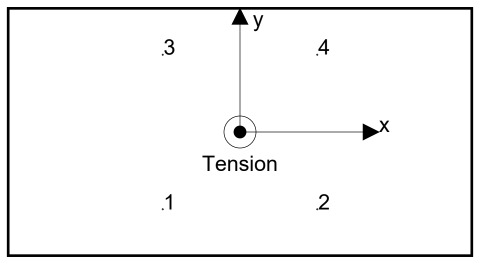
1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	RISA Reaction Output	N = 2,000; V_x = 0; V_y = 7,000; M_x = 0; M_y = 0; M_z = 0;	no	35

2 Load case/Resulting anchor forces

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	500	1,750	0	1,750
2	500	1,750	0	1,750
3	500	1,750	0	1,750
4	500	1,750	0	1,750



max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 2,000 [lb]
resulting compression force in (x/y)=(-/-): 0 [lb]

Anchor forces are calculated based on the assumption of a rigid anchor plate.

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi$ N_n	Status
Steel Strength*	500	9,831	6	OK
Pullout Strength*	500	16,909	3	OK
Concrete Breakout Failure**	2,000	19,932	11	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-19 Eq. (17.6.1.2)

$\phi\ N_{sa} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.23	58,000

Calculations

N_{sa} [lb]
13,108

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
13,108	0.750	9,831	500

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-19 Eq. (17.6.3.1)

$N_p = 8\ A_{brg} f'_c$ ACI 318-19 Eq. (17.6.3.2.2a)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-19 Table 17.5.2

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	λ_a	f'_c [psi]
1.000	0.67	1.000	4,500

Calculations

N_p [lb]
24,156

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
24,156	0.700	16,909	500

3.3 Concrete Breakout Failure

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-19 Eq. (17.6.2.1b)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e_N'}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f_c'}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
8.000	0.000	0.000	6.500	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
-	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
522.00	576.00	1.000	1.000	0.863	1.000	36,429

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
28,475	0.700	19,932	2,000

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi$ V_n	Status
Steel Strength*	1,750	5,112	35	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	7,000	39,865	18	OK
Concrete edge failure in direction **[1]	N/A	N/A	N/A	N/A

* highest loaded anchor **anchor group (relevant anchors)

[1] Shear Anchor Reinforcement has been selected!

4.1 Steel Strength

$V_{sa} = 0.6\ A_{se,V}\ f_{uta}$ ACI 318-19 Eq. (17.7.1.2b)

$\phi\ V_{steel} \geq V_{ua}$ ACI 318-19 Table 17.5.2

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.23	58,000

Calculations

V_{sa} [lb]
7,865

Results

V_{sa} [lb]	ϕ_{steel}	ϕ V_{sa} [lb]	V_{ua} [lb]
7,865	0.650	5,112	1,750

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}} \right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b \right]$ ACI 318-19 Eq. (17.7.3.1b)

$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-19 Table 17.5.2

A_{Nc} see ACI 318-19, Section 17.6.2.1, Fig. R 17.6.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-19 Eq. (17.6.2.1.4)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.3.1)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.4.1b)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}} \right) \leq 1.0$ ACI 318-19 Eq. (17.6.2.6.1b)

$N_b = k_c\ \lambda_a\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-19 Eq. (17.6.2.2.1)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
2	8.000	0.000	0.000	6.500

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	∞	24	1.000	4,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
522.00	576.00	1.000	1.000	0.863	1.000	36,429

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
56,949	0.700	39,865	7,000

5 Combined tension and shear loads, per ACI 318-19 section 17.8

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.100	0.342	5/3	19	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

Company:		Page:	7
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 26, 2023	Date:	4/22/2024
Fastening point:			

6 Warnings

- The anchor design methods in PROFIS Engineering require rigid anchor plates per current regulations (AS 5216:2021, ETAG 001/Annex C, EOTA TR029 etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Engineering calculates the minimum required anchor plate thickness with CBFEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid anchor plate assumption is valid is not carried out by PROFIS Engineering. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies where the potential concrete failure surfaces are crossed by supplementary reinforcement proportioned to tie the potential concrete failure prism into the structural member. Condition B applies where such supplementary reinforcement is not provided, or where pullout or pryout strength governs.
- For additional information about ACI 318 strength design provisions, please go to https://submittals.us.hilti.com/PROFISAnchorDesignGuide/
- The design of Anchor Reinforcement is beyond the scope of PROFIS Engineering. Refer to ACI 318-19, Section 17.5.2.1 (b) for information about Anchor Reinforcement.
- Anchor Reinforcement has been selected as a design option, calculations should be compared with PROFIS Engineering calculations.

Fastening meets the design criteria!

Company:		Page:	8
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 26, 2023	Date:	4/22/2024
Fastening point:			

7 Installation data

Profile: no profile

Hole diameter in the fixture: d_f = - in.

Plate thickness (input): 0.500 in.

Recommended plate thickness: not calculated

Anchor type and diameter: Heavy Hex Head ASTM F 1554 GR. 36 5/8

Item number: not available

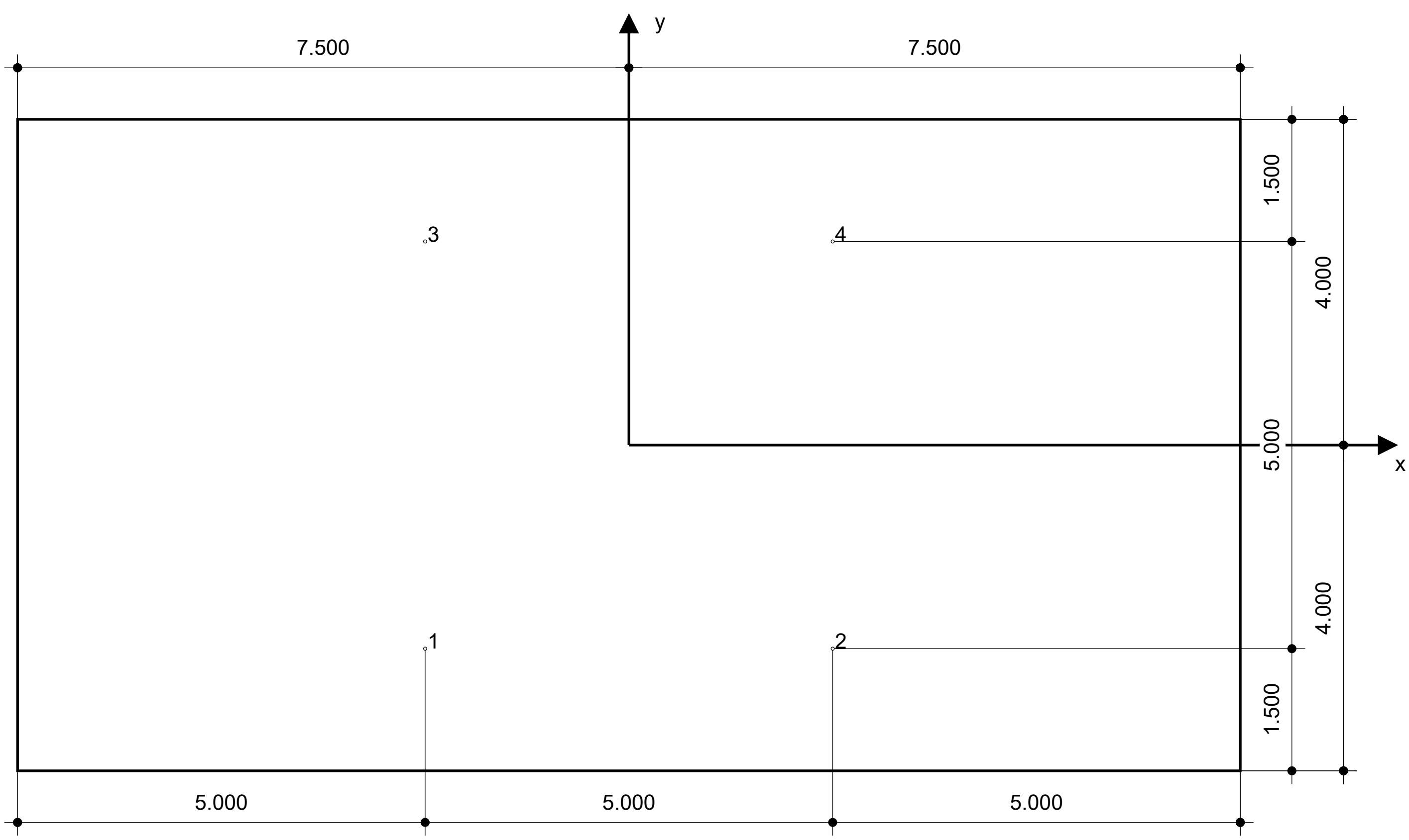
Maximum installation torque: -

Hole diameter in the base material: - in.

Hole depth in the base material: 8.000 in.

Minimum thickness of the base material: 8.922 in.

Hilti Heavy Hex Head headed stud anchor with 8 in embedment, 5/8, Steel galvanized, installation per instruction for use



Coordinates Anchor [in.]

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.500	-2.500	15.500	20.500	6.500	11.500
2	2.500	-2.500	20.500	15.500	6.500	11.500
3	-2.500	2.500	15.500	20.500	11.500	6.500
4	2.500	2.500	20.500	15.500	11.500	6.500

Company:		Page:	9
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Concrete - Sep 26, 2023	Date:	4/22/2024
Fastening point:			

8 Remarks; Your Cooperation Duties

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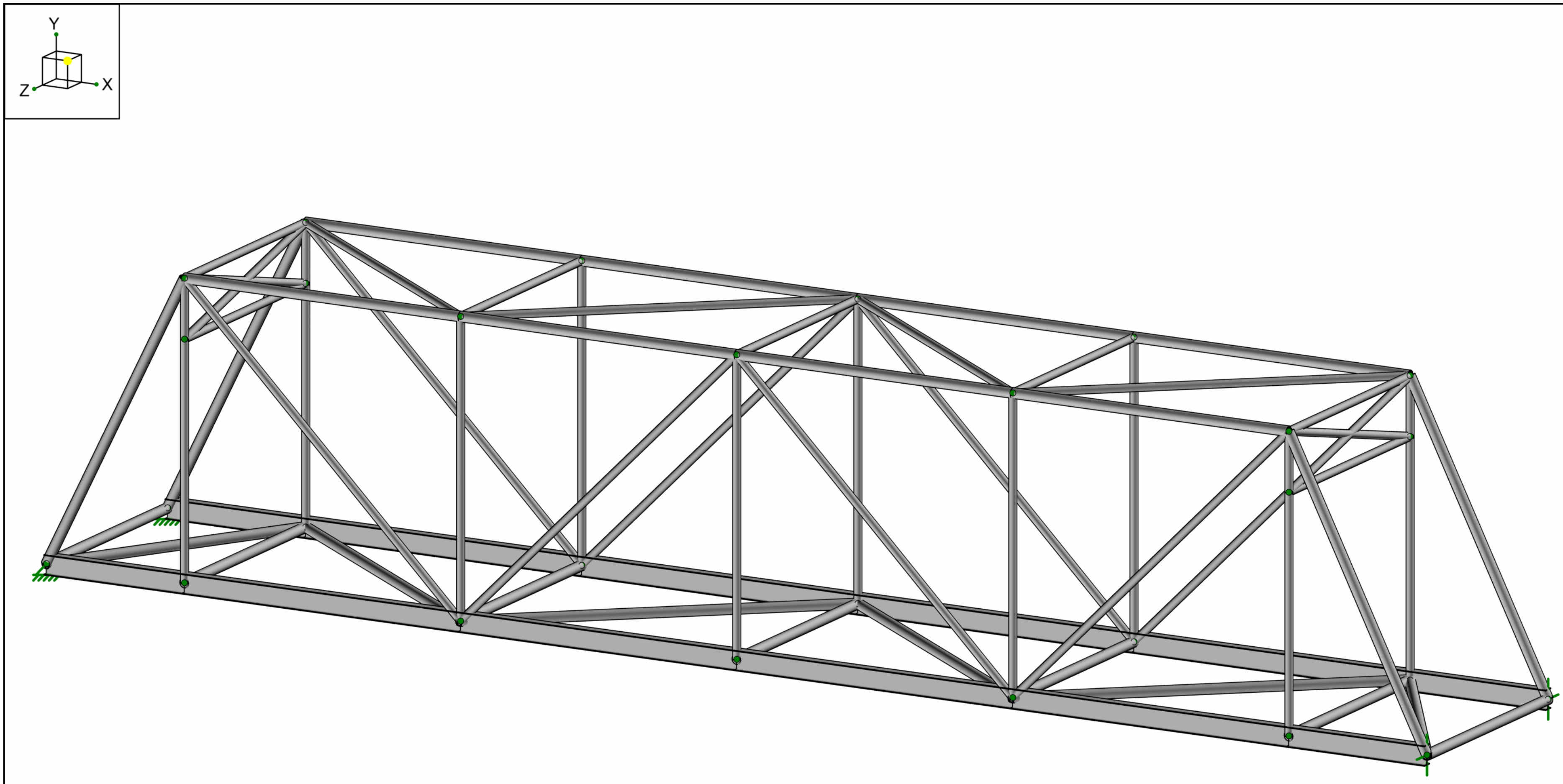
Appendix B – 65% INHT Calculations Package – Through Truss

Contents



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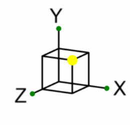

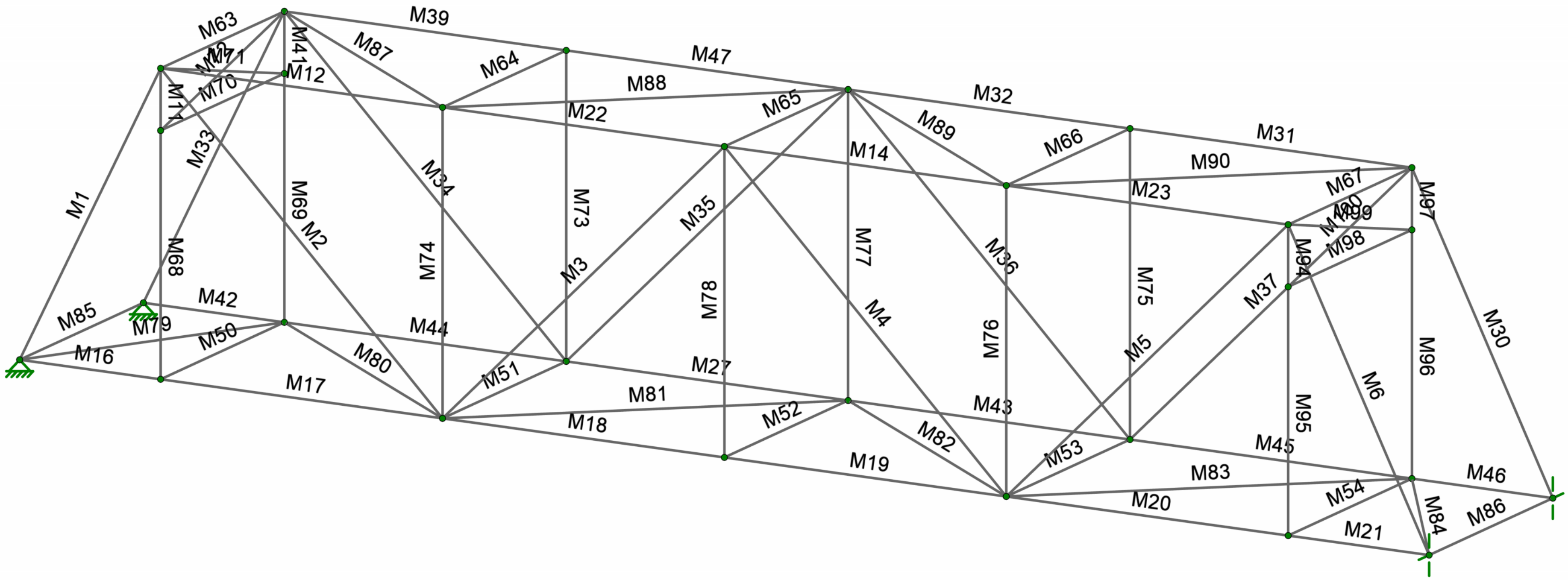

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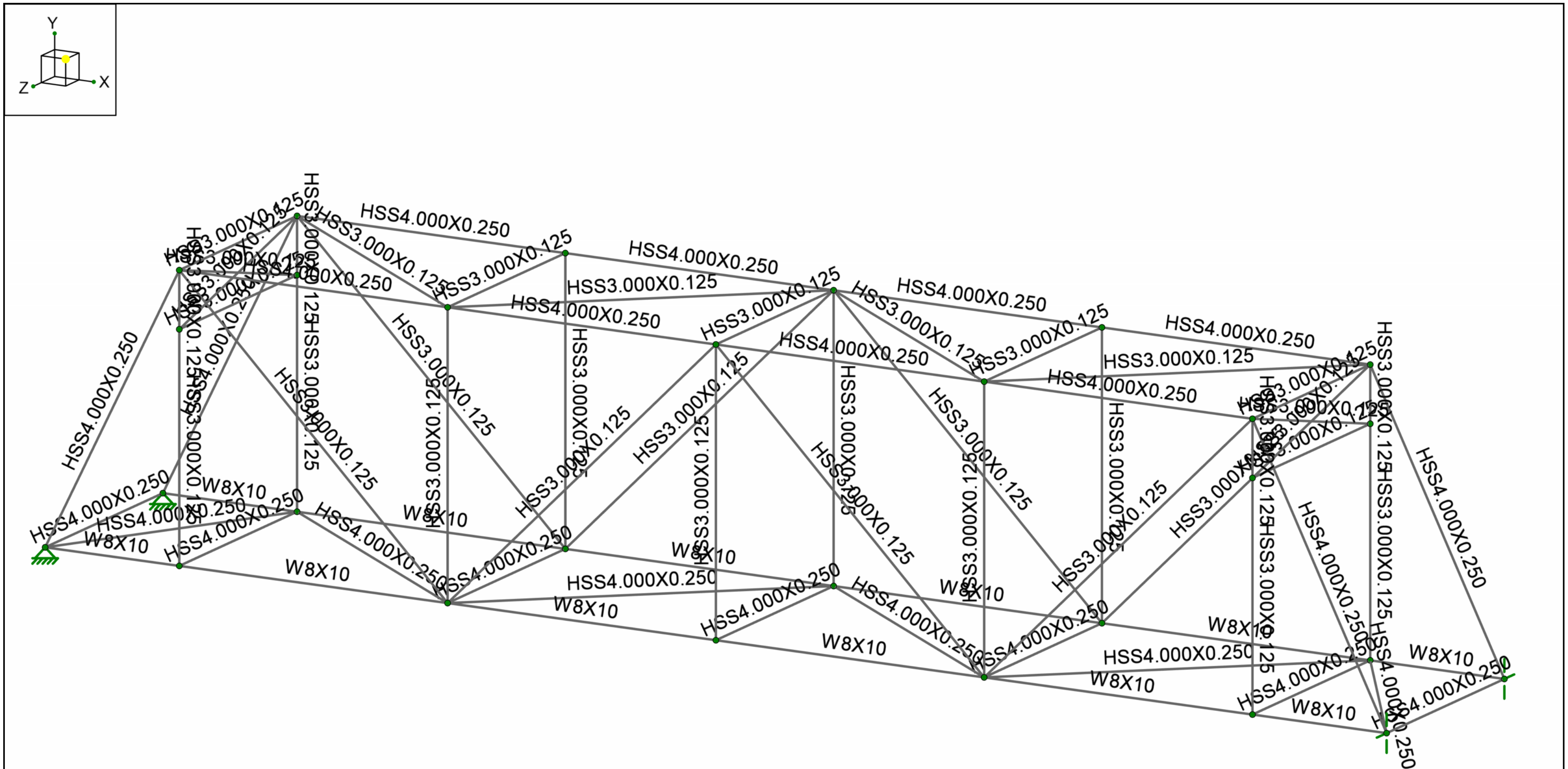

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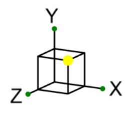

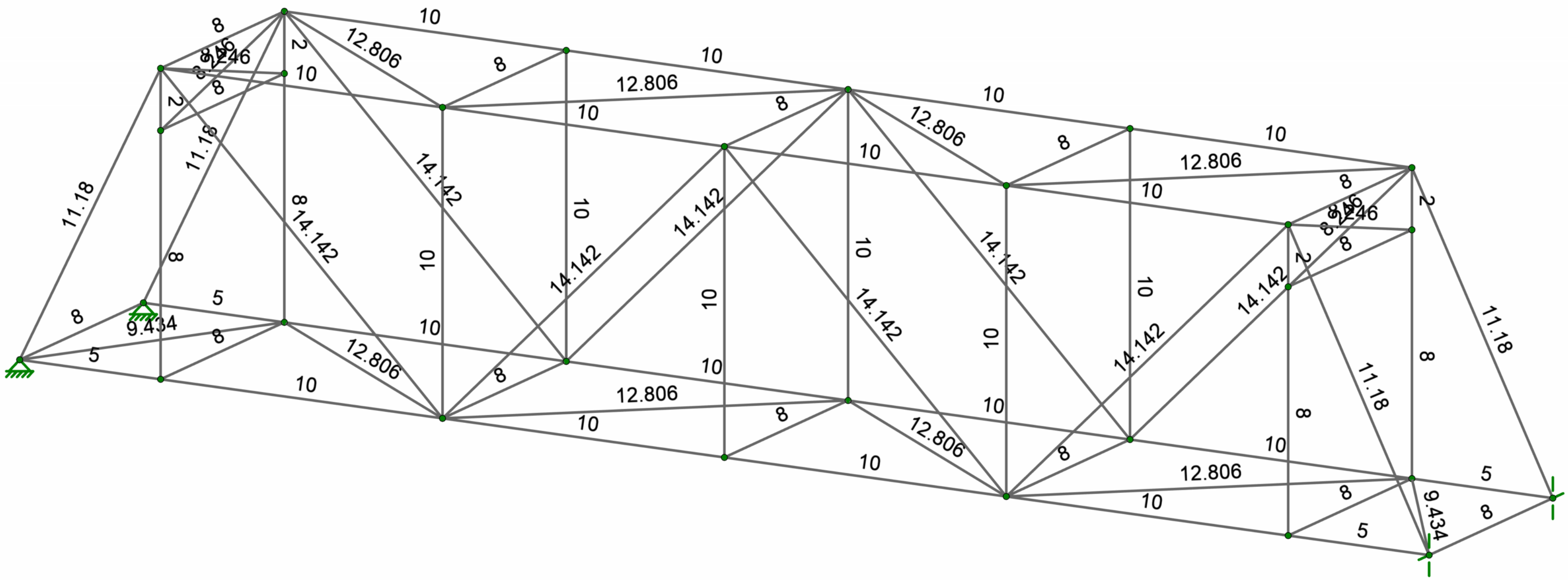

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RISA A NEMETSCHEK COMPANY	Enterprise Engineering, Inc.		SK-4
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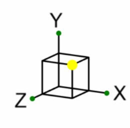

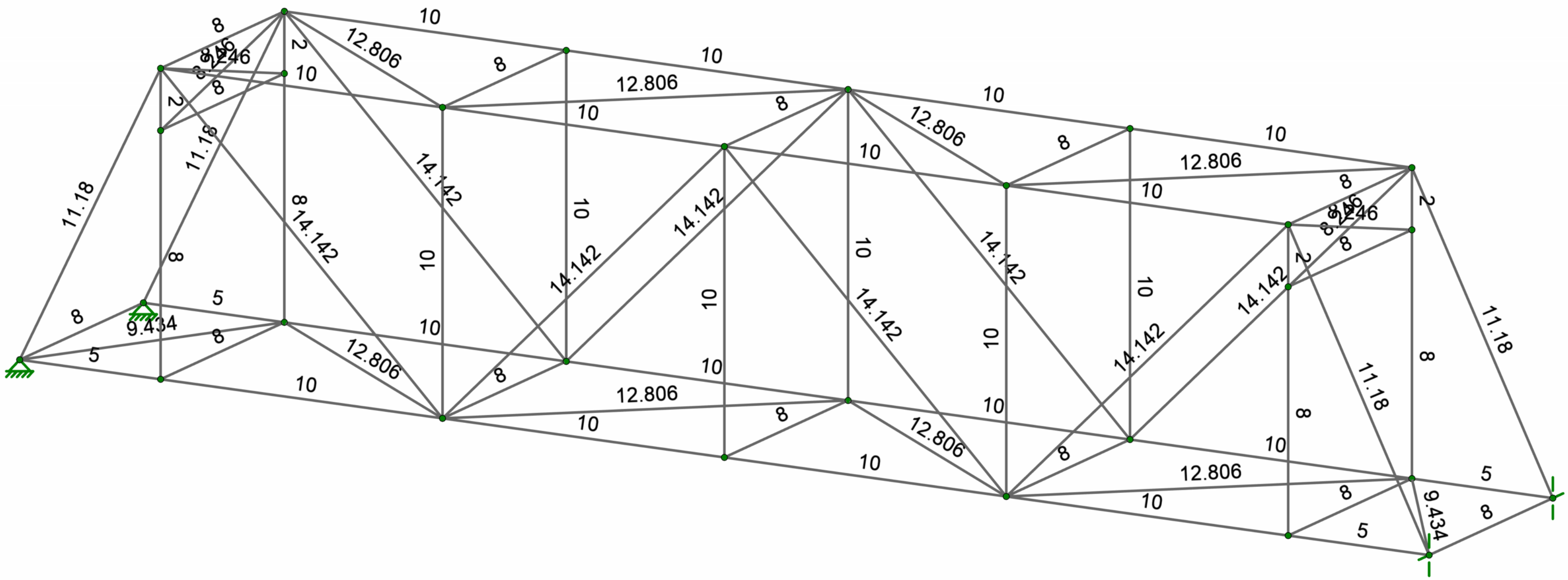

Member Lbzz (ft) Displayed

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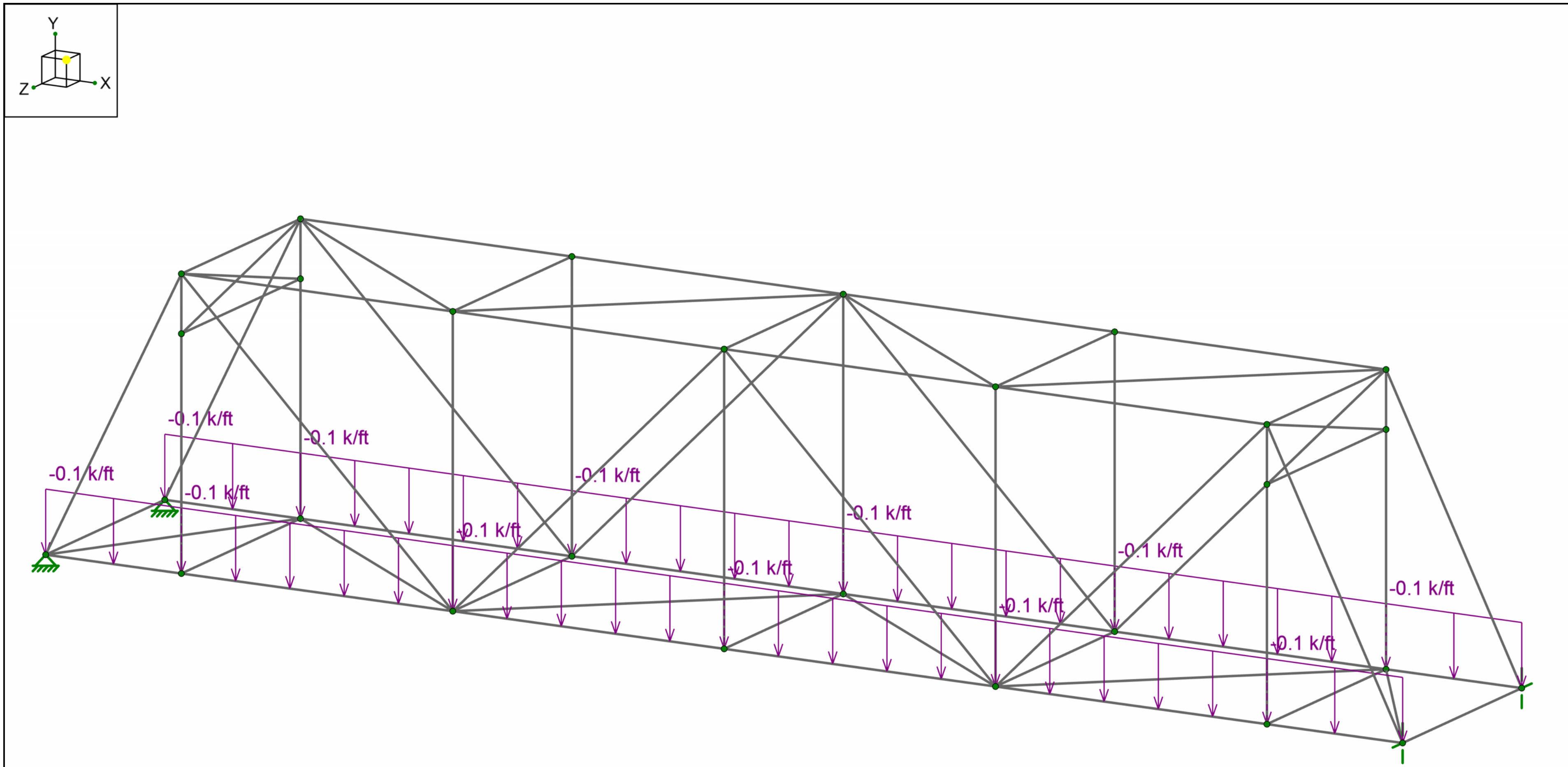

Loads: BLC 1, Bar Grate Load

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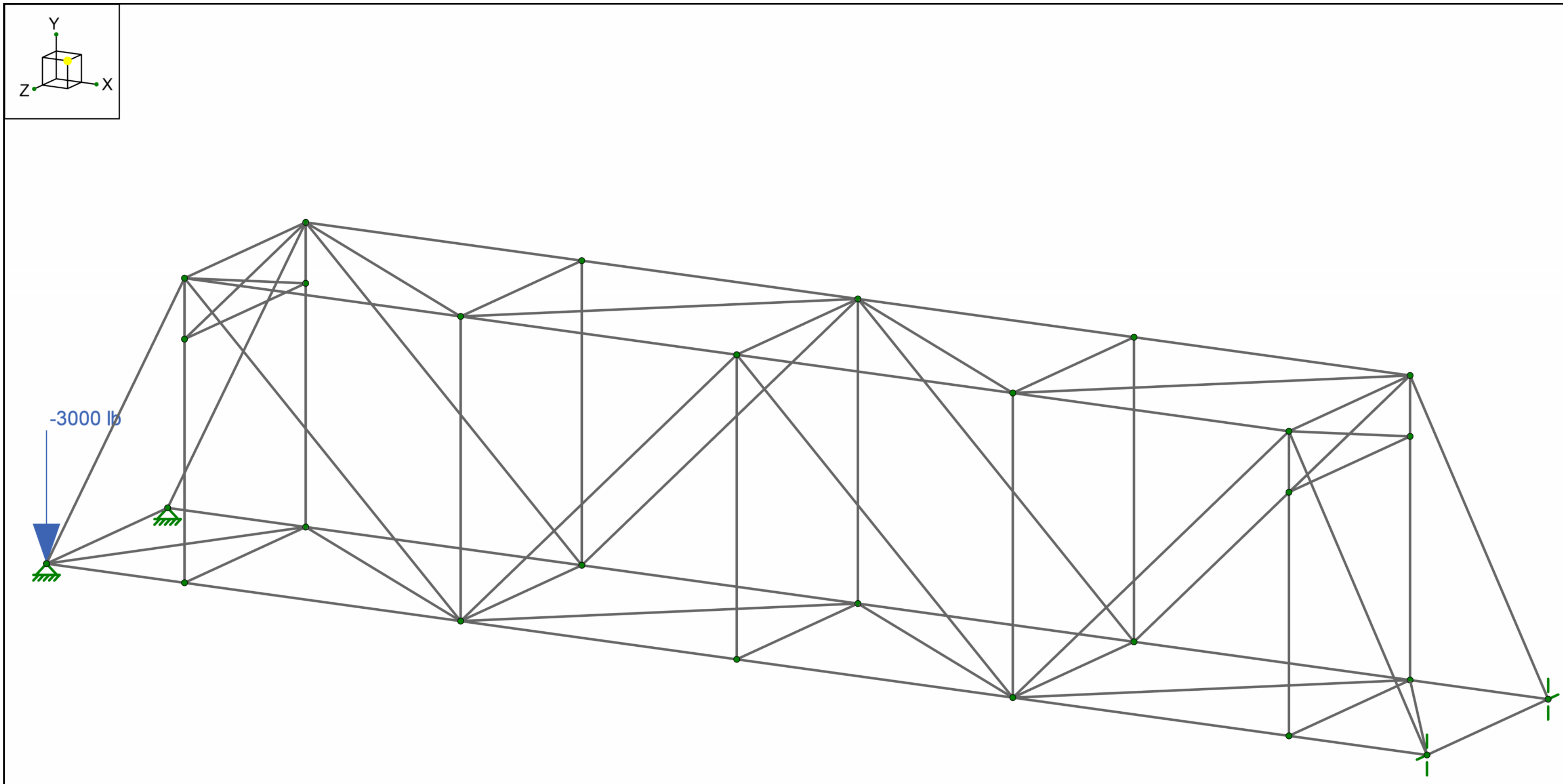

Loads: BLC 2, UTV Moving Load

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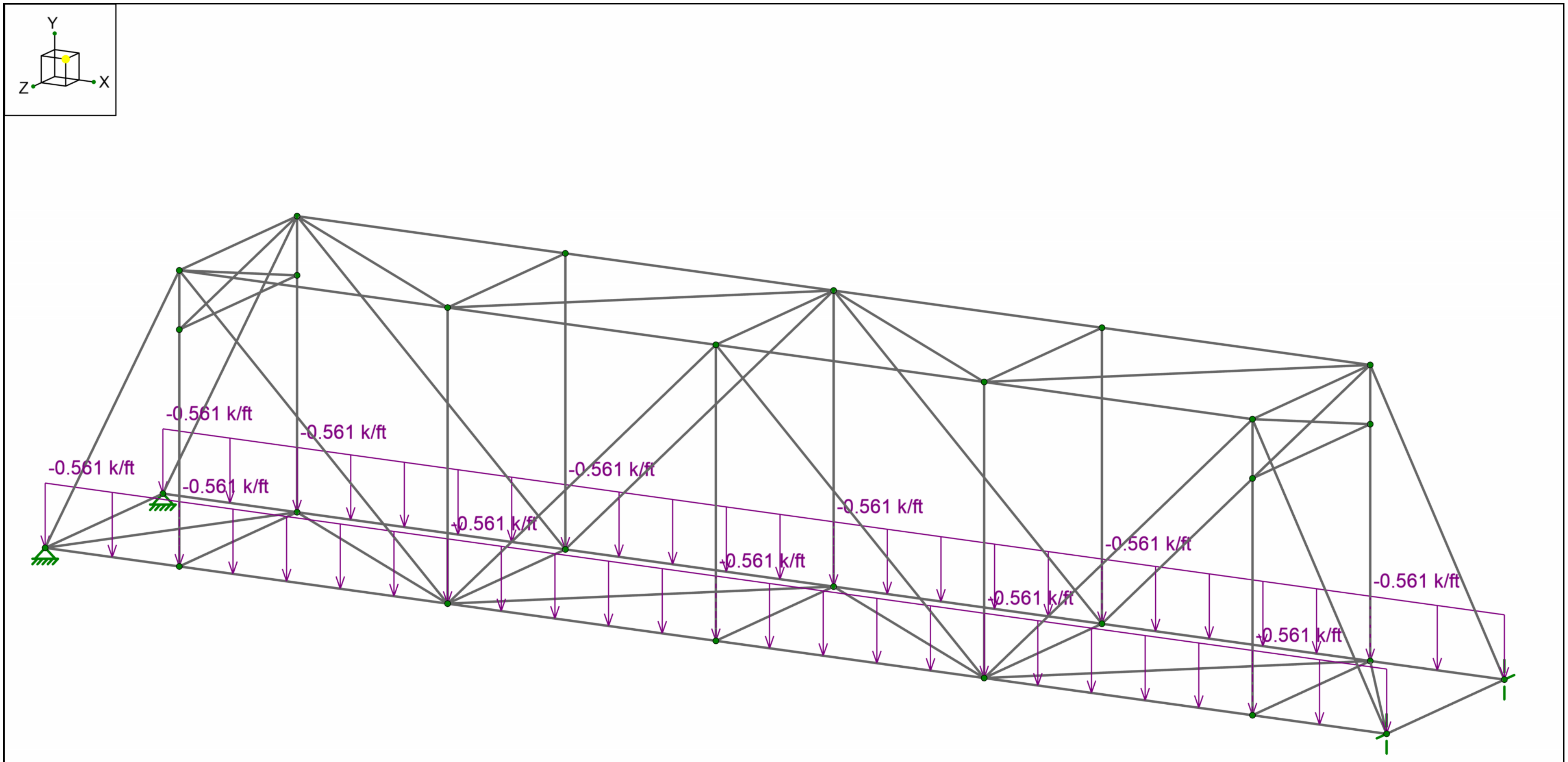

Loads: BLC 3, Uniform Snow

RISA A NEMETSCHEK COMPANY	Enterprise Engineering, Inc.		SK-8
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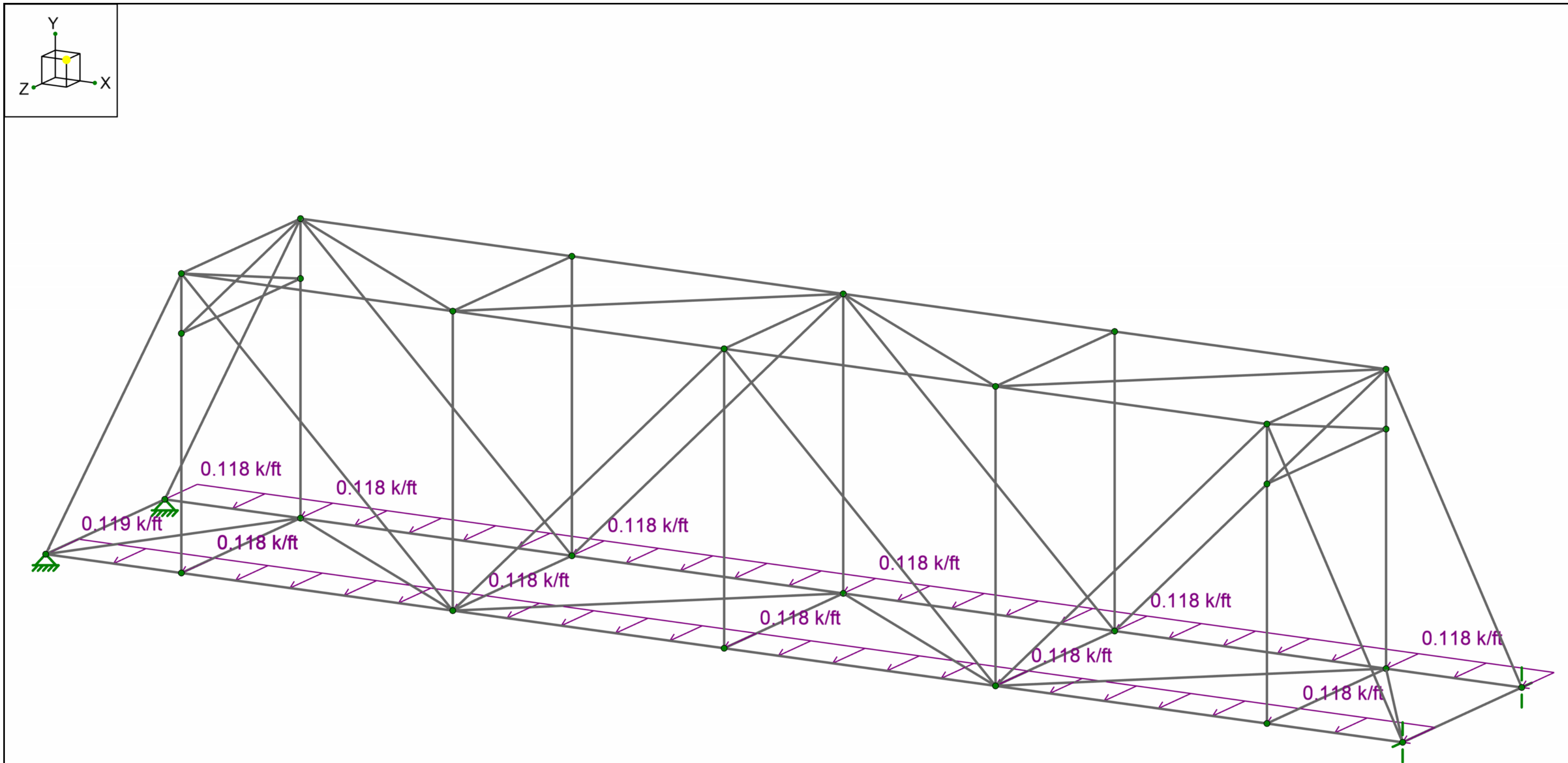

Loads: BLC 4, Seismic

RISA A NEMETSCHEK COMPANY	Enterprise Engineering, Inc.		SK-9
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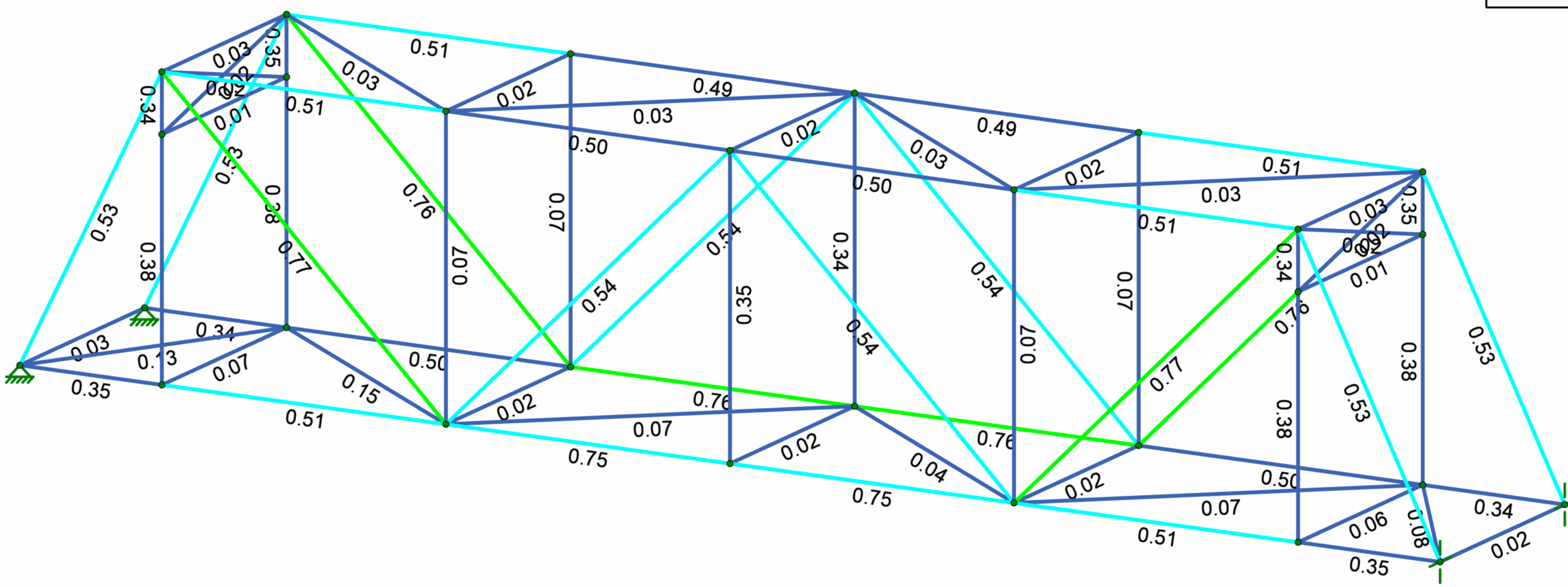
			ThroughTruss10Percent.r3d

Y
Z X

Code Check
(Env)
No Calc
> 1.0
.90-1.0
.75-.90
.50-.75
0.-.50



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RISA A NEMETSCHEK COMPANY	Enterprise Engineering, Inc.		SK-10
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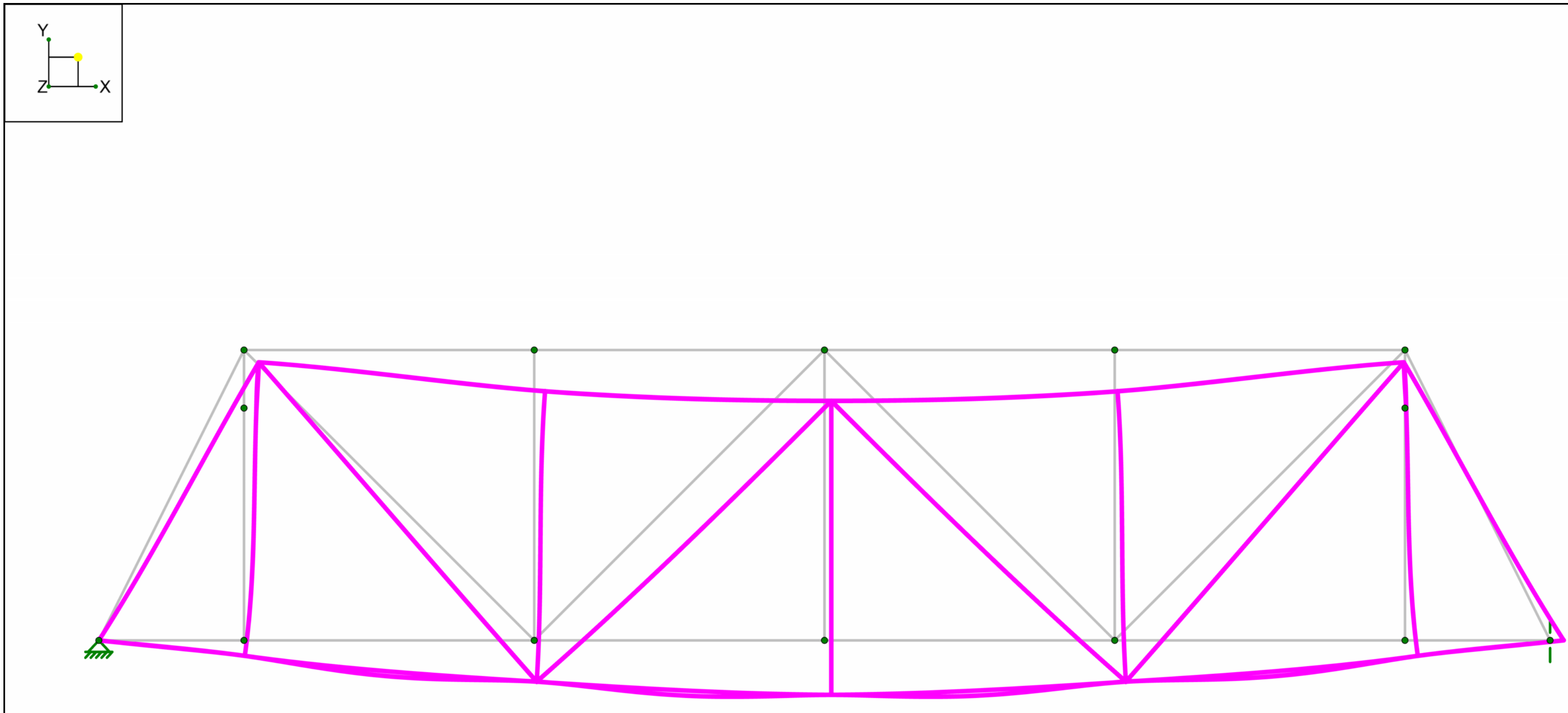

Results for LC 7, ASCE Strength 3

RISA A NEMETSCHEK COMPANY	Enterprise Engineering, Inc.		SK-11
	Nicholas F. Schwantes		May 05, 2024 at 12:58 PM
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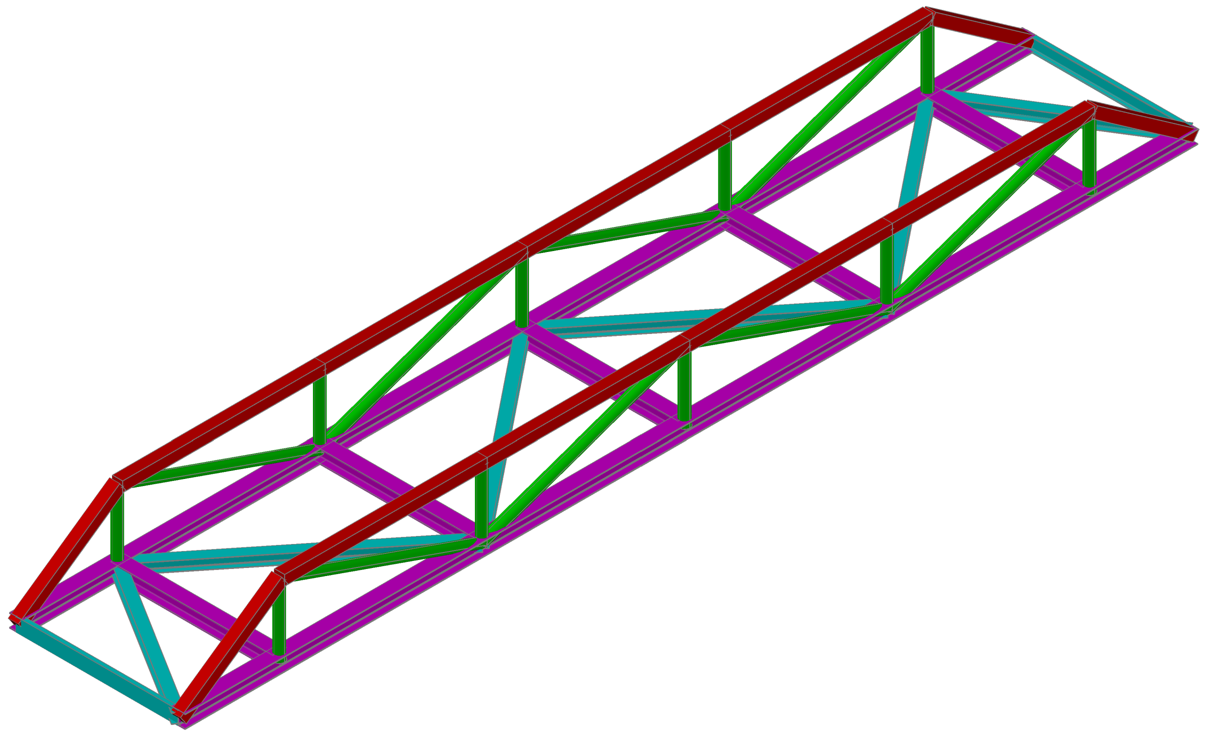
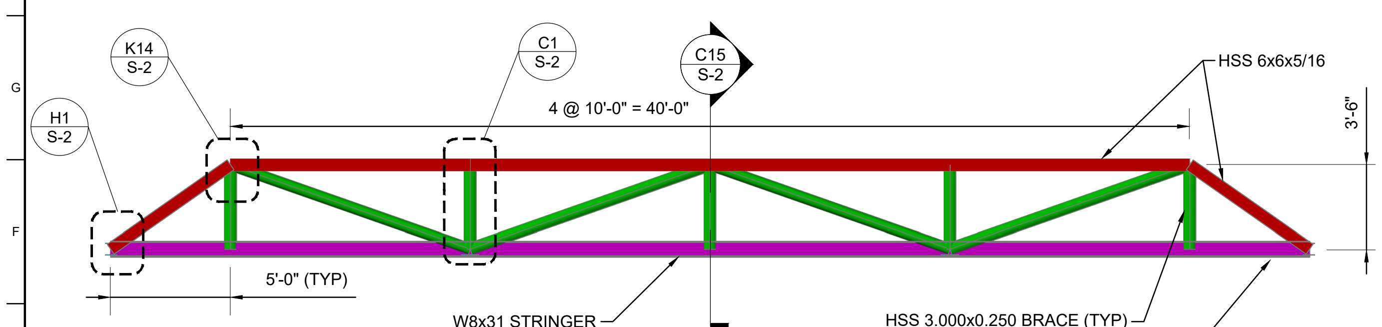
Appendix C – 65% INHT Drawings Package – Pony Truss

Contents



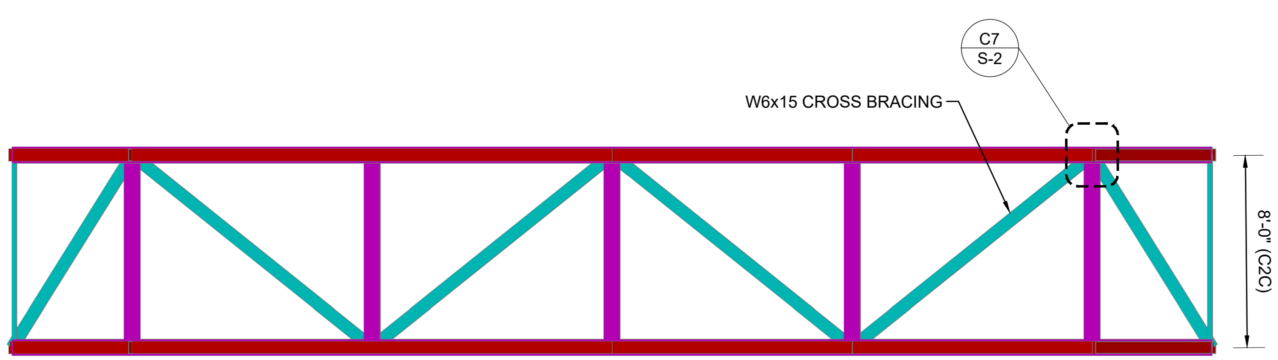
PONY TRUSS - ISOMETRIC

SCALE: NTS



PONY TRUSS - ELEVATION 1

SCALE: 1/4" = 1'-0"



PONY TRUSS - PLAN

SCALE: 1/4" = 1'-0"

STRUCTURAL STEEL
UNLESS SPECIFIED OTHERWISE, STRUCTURAL STEEL MEMBERS MUST ADHERE TO THE FOLLOWING CRITERIA:

STRUCTURAL SECTION	REQUIRED GRADE	YIELD STRENGTH (KSI)
CHANNELS AND PLATES	ASTM A36	36
RECTANGULAR HSS	ASTM A500 GR. B	46
ROUND HSS	ASTM A500 GR. C	50
W SECTIONS	ASTM A992	50

FABRICATION AND ERECTION MUST BE CONDUCTED PER AISC SPECIFICATIONS.

GROUT:
GROUT ASTM C1107, GRADE C, PREMIXED COMPOUND CONSISTING OF NONMETALLIC AGGREGATE, CAPABLE OF DEVELOPING A MINIMUM COMPRESSION STRENGTH OF 5,000 PSI IN 28 DAYS. ICC-ES CERTIFICATION REQUIRED. USE SPECIFIC GROUT MIX RECOMMENDED BY THE MANUFACTURER FOR EACH GROUT APPLICATION AND FOLLOW MANUFACTURER'S INSTRUCTIONS.

ANCHORS:
CAST-IN-PLACE
CAST-IN-PLACE ANCHORS MUST BE RATED ASTM F1554 GR. 36 OR STRONGER. DIAMETERS AND MINIMUM EMBEDMENTS MUST ADHERE TO THE REQUIREMENTS CONTAINED IN THE CONTRACT DRAWINGS.
POST-INSTALLED
POST-INSTALLED ANCHORS MUST NOT BE USED.

WELDING:
WELDING OF CARBON STEEL MUST BE IN ACCORDANCE WITH AWS D1.1.
WELDING ELECTRODES AND FILLER METAL MUST BE COMPATIBLE FOR THE MATERIAL AND POSITION BEING WELDED.

WELDING ELECTRODES FOR CARBON STEEL WELDING MUST CONFORM TO THE FOLLOWING:

PROCESS	ELECTRODE
FCAW	AWS A5.20, E7XT-X CLASSIFICATION
GMAW	AWS A5.18, E70S-X CLASSIFICATION
SAW	AWS A5.17, F7AT-EXXX CLASSIFICATION
SMAW	AWS A5.1, E70XX CLASSIFICATION

MISCELLANEOUS:
VERIFY ALL DIMENSIONS AND CONDITIONS AT THE PROJECT SITE PRIOR TO STARTING WORK AND NOTIFY THE ENGINEER, IMMEDIATELY OF ANY DISCREPANCIES.
SUBMIT ALL REQUIRED SHOP DRAWINGS AND RECEIVE THEIR SATISFACTORY REVIEW FROM THE ENGINEER, PRIOR TO FABRICATION.
PROVIDE TEMPORARY ERECTION BRACING AND SHORING AS REQUIRED FOR STABILITY OF THE BRIDGE DURING ALL PHASES OF CONSTRUCTION.
ALL SOILS DISTURBED DURING CONSTRUCTION MUST BE BACKFILLED AND COMPACTED TO MINIMUM COMPACTION OF TYPICAL UNDISTURBED SOIL.

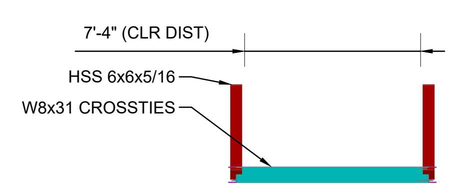
SPECIAL INSPECTIONS:
SPECIAL INSPECTIONS, INCLUDING ALL APPLICABLE PROVISIONS OF IBC SECTION 17, ARE WAIVED PURSUANT TO DESIGNER DISCRETION FOR THE DURATION OF THIS PROJECT WITH THE FOLLOWING JUSTIFICATION:
1. THIS IS AN ACADEMIC PROJECT (NOT FOR CONSTRUCTION).

REINFORCED CONCRETE
1. ALL CONCRETE WORK AND REINFORCING STEEL DETAILS MUST CONFORM TO ACI 318-14 AND ACI 315 LATEST EDITION.
2. MATERIALS MUST MEET THE FOLLOWING:
   a. PORTLAND CEMENT — ASTM C150, TYPE I / II
   b. WATER — POTABLE
   c. AGGREGATES — ASTM C33, 3/4 INCH
   d. AIR ENTRAINMENT ADMIXTURES — ASTM C260
   e. REINFORCING STEEL — ASTM A615, GRADE 60
3. MIX DESIGNS MUST BE PREPARED IN ACCORDANCE WITH ACI 211 AND ACI 301. READY MIX CONCRETE MUST CONFORM TO ASTM C94. CONCRETE MIXES MUST MEET THE FOLLOWING REQUIREMENTS:
   a. MINIMUM 28-DAY COMPRESSIVE STRENGTH — 4,500 PSI
   b. CONCRETE EXPOSURE CLASSES — F2, S0, W1, C1
   c. MAXIMUM WATER CEMENT RATIO (W/CM) — 0.45
   d. MAXIMUM AGGREGATE SIZE — 3/4 INCH
   e. TARGET AIR CONTENT — 5% ± 1.0%
   f. MAXIMUM WATER-SOLUBLE CHLORIDE ION CONTENT BY WEIGHT OF CEMENT 0.30%
4. ALL CONCRETE MUST BE PLACED IN ACCORDANCE WITH ACI 304R.
5. MINIMUM CONCRETE COVER FOR REINFORCING STEEL, UNLESS NOTED OTHERWISE, MUST BE AS FOLLOWS:
   a. CONCRETE CAST AGAINST EARTH — 3 INCHES
   b. ALL OTHER LOCATIONS — 2 INCHES

CHAMFER EXPOSED EDGES 3/4-INCH UNLESS NOTED OTHERWISE.

ABBREVIATIONS

' / "	FEET/INCHES
ACI	AMERICAN CONCRETE INSTITUTE
AISC	AMERICAN INSTITUTE OF STEEL CONSTRUCTION
ASTM	AMERICAN SOCIETY FOR TESTING AND MATERIALS
AWS	AMERICAN WELDING SOCIETY
BF	BOTTOM FLANGE
C2C	CENTER TO CENTER
℄	CENTER LINE
CLR	CLEAR/CLEARANCE
CONC	CONCRETE
DIA	DIAMETER
DIST	DISTANCE
ELEV	ELEVATION
EMBED	EMBEDMENT
ETC	ET CETERA
EX	EXISTING
FCAW	FLUX-CORED ARC WELDING
FT / IN	FEET/INCHES
GMAW	GAS METAL ARC WELDING
GR.	GRADE
HSS	HOLLOW STRUCTURAL SECTION
IBC	INTERNATIONAL BUILDING CODE
LLH	LONG LEG HORIZONTAL
KSI	KIPS PER SQUARE INCH
MAX	MAXIMUM
MIN	MINIMUM
NS/FS	NEARSIDE/FARSIDE
NTS	NOT TO SCALE
OC	ON CENTER
PSI	POUNDS PER SQUARE INCH
PTFE	POLYTETRAFLUOROETHYLENE
RECT	RECTANGULAR
REINF	REINFORCEMENT
SAW	SUBMERGED-ARC WELDING
SMAW	SHIELDED METAL ARC WELDING
T/	TOP OF
TF	TOP FLANGE
TYP	TYPICAL



PONY TRUSS - ELEVATION 2

SCALE: 1/4" = 1'-0"

UNIVERSITY OF ALASKA GRADUATE PROJECT

IDITAROD NATIONAL HISTORIC TRAIL IMPROVEMENT
KENAI PENINSULA, ALASKA

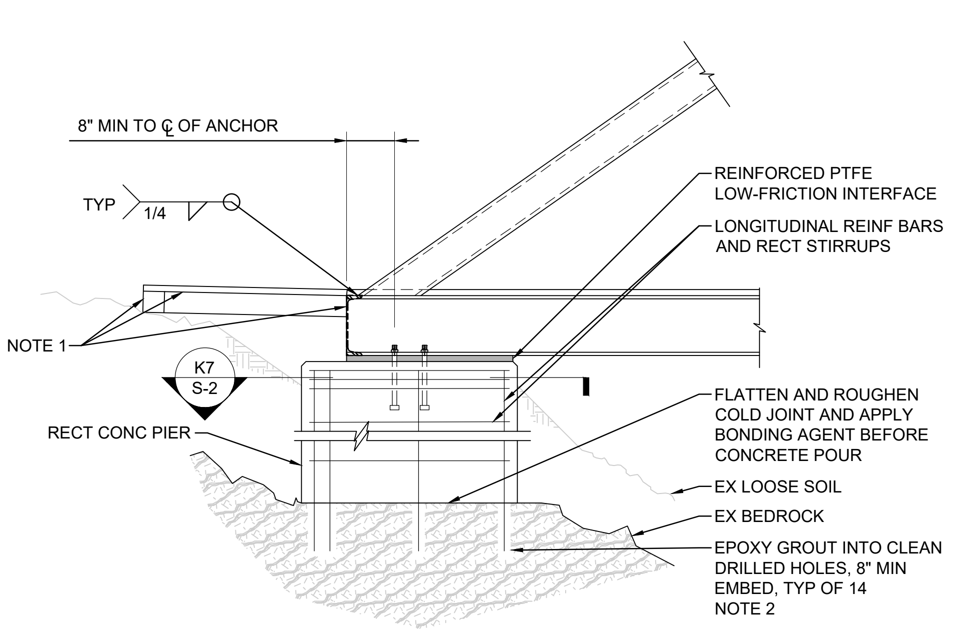
REV	DATE	BY	DESCRIPTION
2	4/22/24	NFS	FINAL 65% SUBMISSION
1	12/23/23	NFS	DRAFT 65% SUBMISSION
0	7/31/23	NFS	35% CONCEPT SUBMISSION

DESIGN: NFS
DRAWN: NFS
CHECKED: SEH
APPROVED: SEH
SCALE: AS NOTED
SEAL

CLIENT/PROJECT
UNIVERSITY OF ALASKA GRADUATE PROJECT
KENAI PENINSULA, ALASKA

DRAWING TITLE
MACRO VIEWS AND GENERAL NOTES

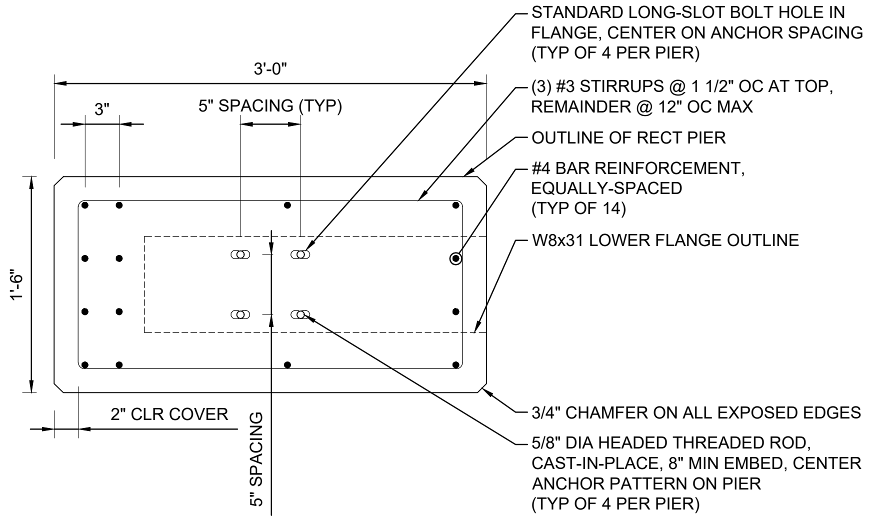
DWG NO.: S-1
REV
PROJ NO.: CE A686
FILE
SHEET: 1 OF 3



NOTE:
1. BAR GRATE LANDING PANEL MUST MATCH MAIN BAR GRATE MATERIAL. BAR GRATE MUST BE SUPPORTED BY HSS 2x2x1/4 (FULL PERIMETER, 1 FT MAX SPACING ALONG RAMP WIDTH). HSS SUPPORT MUST BE BUTT WELDED TO C10x15.3 BRIDGE END CAP. RUN BEARING BARS PERPENDICULAR TO TRUSS.
2. PIER REINFORCEMENT BARS MUST EXTEND A MINIMUM OF 8 INCHES INTO THE UNDERLYING BEDROCK. SET T/ PIER ELEV BY LASER. HEIGHT OF EACH PIER VARIES, DEPENDING ON THE DEPTH OF SECURE BEDROCK. IF PIER HEIGHT IS EXPECTED TO EXCEED 10'-0", STOP WORK AND NOTIFY THE DESIGN ENGINEER FOR FURTHER GUIDANCE.

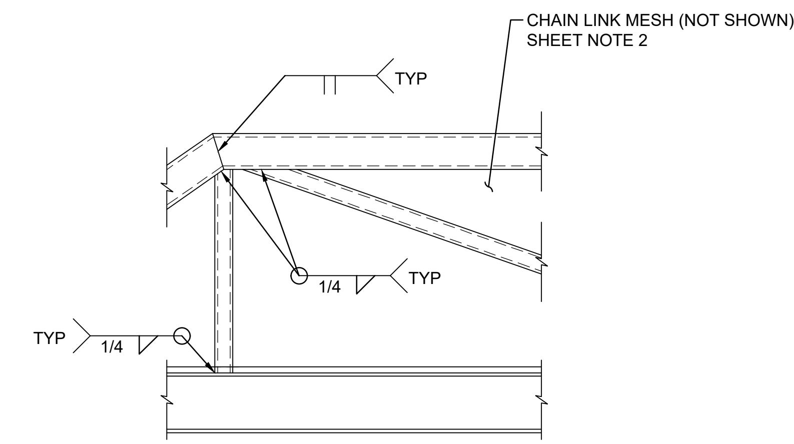
H1 - END CONNECTION AND FOUNDATION DETAIL

SCALE: 3/4" = 1'-0" 0' 1' 2' 3'



K7 - TYPICAL CONC PIER SECTION

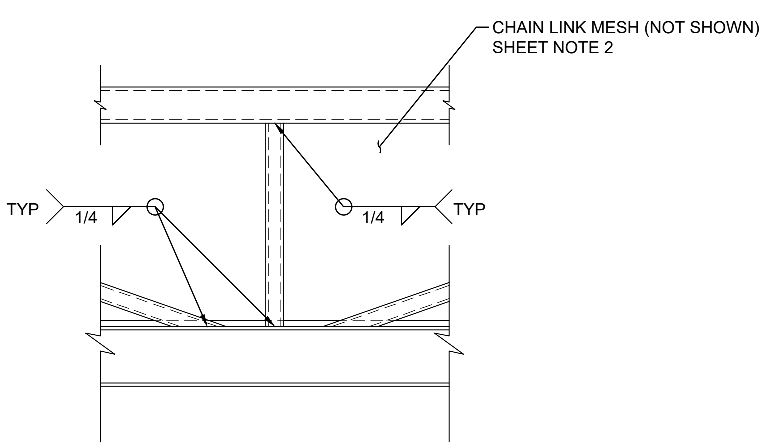
SCALE: 1 1/2" = 1'-0" 0' 1' 2'



K14 - TRUSS CONNECTION DETAIL 1

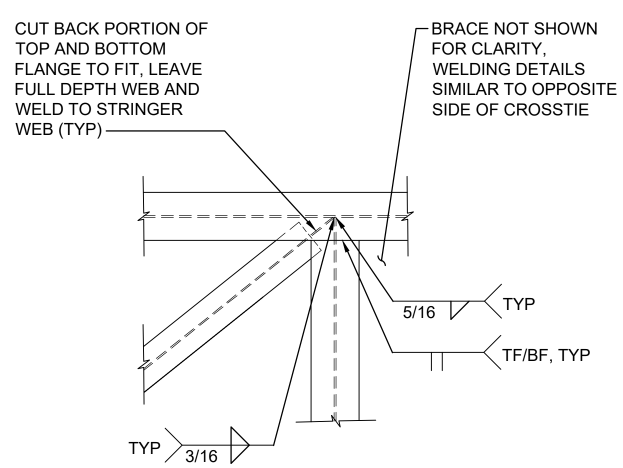
SCALE: 3/4" = 1'-0" 0' 1' 2' 3'

SHEET NOTES
1. PROVIDE GALVANIZED SERRATED WELDED STEEL BANDED BAR GRATING WITH 2" x 1/4" BEARING BARS @ 1 3/16" OC AND CROSS BARS @ 4" OC. GRATING MUST EXTEND TO THE CENTERLINE OF EACH SUPPORT BEAM. PROVIDE FACTORY BANDED COPING TO BYPASS VERTICAL STRUCTURAL STEEL AFFIXED TO BEAM TOP FLANGES. SECURE GRATING TO SUPPORTS WITH SADDLE CLIPS. RUN BEARING BARS PERPENDICULAR TO TRUSS.
2. PROVIDE GALVANIZED CHAIN LINK MESH AT ALL OPENINGS IN TRUSS PLANE. PROVIDE DOCUMENTATION SHOWING CONFORMANCE WITH FALL PROTECTION REQUIREMENTS OF 29 CFR 1910.29. PROVIDE MESH INBOARD OF EACH TRUSS (ON ROAD DECK SIDE) AND SECURE MESH TO SIDE OF EACH TRUSS MEMBER WITH SADDLE CLIPS (MAXIMUM 12" OC).
3. ALL STRUCTURAL STEEL MEMBERS MUST BE FULLY COATED WITH HIGH PERFORMANCE CORROSION RESISTANT COATING BEFORE INSTALLATION OF GALVANIZED CHAIN LINK MESH AND BAR GRATE ROAD DECK. PROVIDE AND INSTALL PER MANUFACTURER INSTRUCTIONS. SUBMIT PRODUCT DATA FOR APPROVAL PRIOR TO COATING ARRIVAL ONSITE.



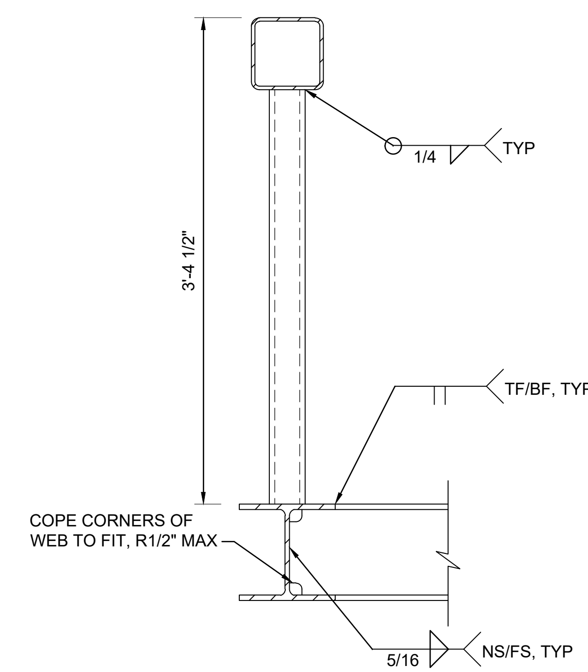
C1 - TRUSS CONNECTION DETAIL 2

SCALE: 3/4" = 1'-0" 0' 1' 2' 3'



C7 - SUBFLOOR BRACING DETAIL

SCALE: 3/4" = 1'-0" 0' 1' 2' 3'



C15 - BRIDGE PARTIAL SECTION

SCALE: 1 1/2" = 1'-0" 0' 1' 2'

UNIVERSITY OF ALASKA GRADUATE PROJECT

IDITAROD NATIONAL HISTORIC TRAIL IMPROVEMENT KENAI PENINSULA, ALASKA

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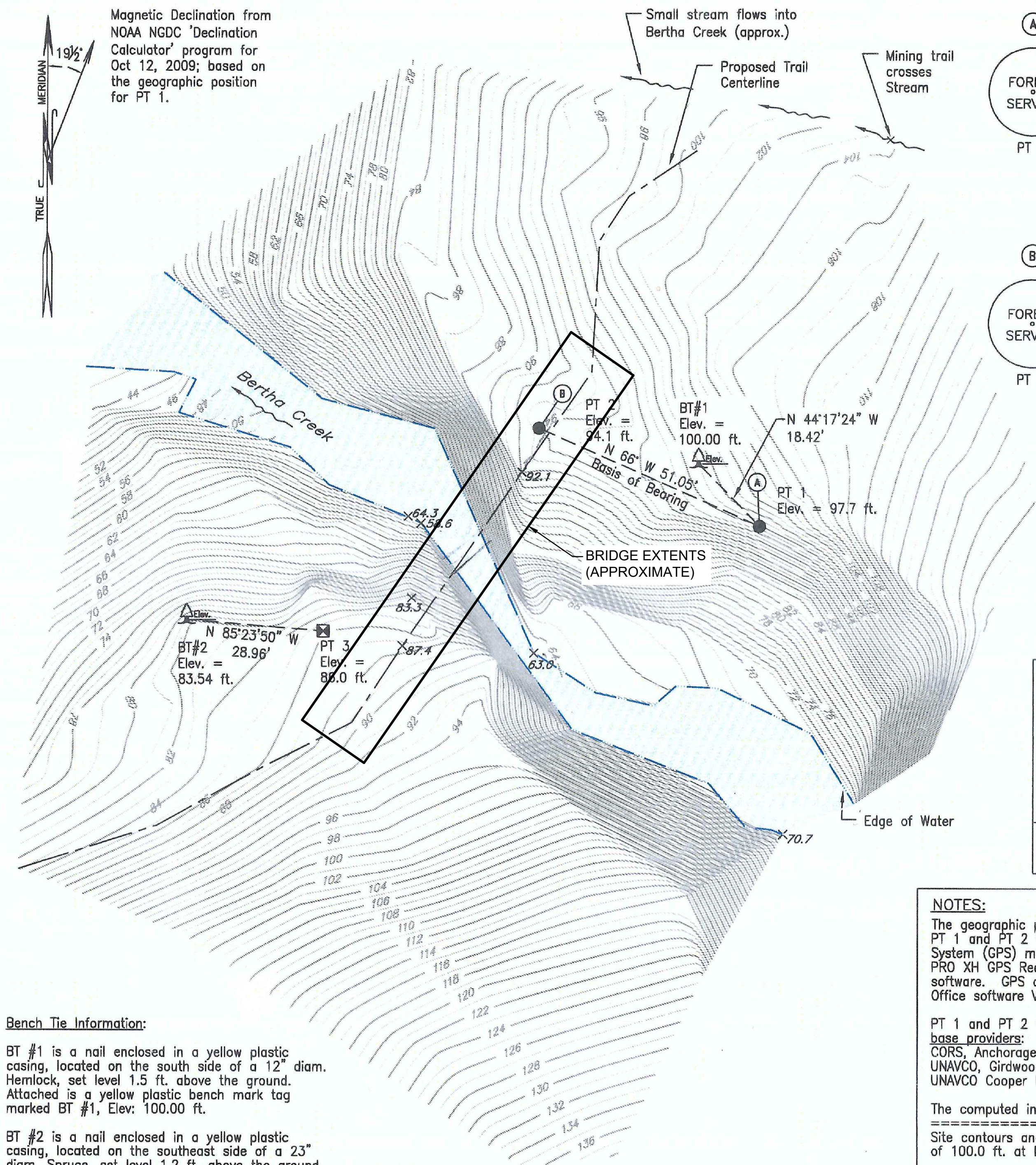
1	12/23/23	NFS	DRAFT 65% SUBMISSION
0	7/31/23	NFS	35% CONCEPT SUBMISSION

DESIGN: NFS
DRAWN: NFS
CHECKED: SEH
APPROVED: SEH
SCALE: AS NOTED
SEAL

CLIENT/PROJECT
UNIVERSITY OF ALASKA GRADUATE PROJECT
KENAI PENINSULA, ALASKA

DRAWING TITLE
STRUCTURAL DETAILS

DWG NO. S-2
REV
PROJ NO. CE A686
FILE
SHEET 2 OF 3



Ⓐ FOREST SERVICE PT 1

Set a ⅝" diam. x 30" long rebar with Forest Service yellow plastic cap, flush with the ground on the right bank of Bertha Creek on a small rock ledge overlooking the creek and waterfall.

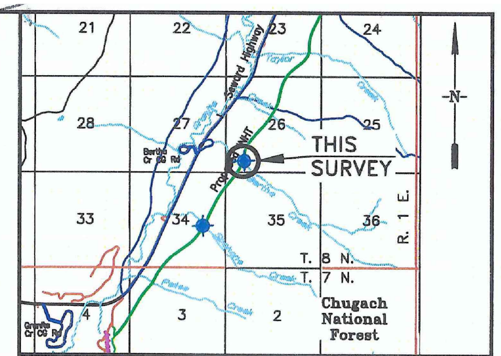
Lat: 60° 45' 01.3" N } NAD83
Long: 149° 14' 20.6" W
Elevation = 97.7 feet (Assumed)
N = 5000.00 } (Assumed)
E = 10000.00

The inverse (Basis of Bearing) between PT 1 and PT 2 is N 66° W, 51.05 ft. distance.

Ⓑ FOREST SERVICE PT 2

Set a ⅝" diam. x 30" long rebar with Forest Service yellow plastic cap, 1" above the ground near the top of the right bank of Bertha Creek and near the proposed bridge crossing site.

Lat: 60° 45' 01.5" N } NAD83
Long: 149° 14' 21.5" W
Elevation = 94.1 feet



Compilation of Seward (D-6), Alaska 1994 Quadrangle and Chugach National Forest GIS Database.
Land ownership shown is current as of March 2009

VICINITY MAP
Scale: 1" = 1-1/2 Miles

Control Point Table

PT #	Bearing	Dist in ft.	Elev* in ft.	Northing*	Easting*
1			97.7	5000.00	10000.00
	N 66°00'00" W	51.05			
2			94.1	5020.76	9953.36
	S 46°44'15" W	62.81			
3			86.0	4977.72	9907.62

*Assumed

NOTES:

1. The purpose of this survey was to collect topographic and site information for the design of a bridge across Upper Bertha Creek as part of the planned construction on the Iditarod National Historic Trail.

2. The field crew consisted of Randy Schrank, RLS and William Nielson, RLS, land surveyors with the USDA Forest Service, Region 10, AK.

3. The monuments depicted hereon actually exist as shown.

4. All distances on this plat are calculated and shown in U.S. Survey Feet.

5. The contour interval used in this project is 1.0 foot.

6. The bearings shown on this plat are True unless otherwise noted.

7. Soils near control points are shallow with underlying bedrock.

8. Vegetation consists of Hemlock and Spruce with scattered Alder and Willow.

LEGEND:

- 2x2x10" long wooden hub with tack, set this survey
- ⅝" diam. rebar with yellow plastic FS Cap, set this survey
- Bench Tie (BT), set this survey
- ×77.2 Elevation in feet
- Proposed Trail Flagline Centerline
- Bertha Creek

Upper Bertha Creek Nonmotorized Bridge Crossing is located on Bertha Creek approx. 3000 feet upstream from the Seward Highway Bertha Creek Bridge at approx. MP 65.4

GRAPHIC SCALE
25 20 15 10 5 0 25 feet
1 inch = 25 ft
1 METER = 3.2808333 U.S. SURVEY FEET

NOTES:

The geographic positions of PT 1 and PT 2 along with the bearing between PT 1 and PT 2 was computed using mapping grade Global Positioning System (GPS) methods. GPS data was collected using Trimble Pathfinder PRO XH GPS Receiver and Trimble Nomad Data Collector using Terrasync software. GPS data was differentially corrected using Trimble Pathfinder Office software Version 4.10.

PT 1 and PT 2 were differentially corrected using data from the following base providers:
CORS, Anchorage (TSEA)
UNAVCO, Girdwood, AK (ac20)
UNAVCO Cooper Landing, AK (ac15)

The computed inverse between 'PT 1' and 'PT 2' is N 66° W.

Site contours and feature information are relative to the assumed elevation of 100.0 ft. at Bench Tie #1 (BT #1).

Survey adjustment and contouring were completed using Tripod Data Systems (TDS) Foresight Software (Ver 2.2.5).

Bench Tie Information:

BT #1 is a nail enclosed in a yellow plastic casing, located on the south side of a 12" diam. Hemlock, set level 1.5 ft. above the ground. Attached is a yellow plastic bench mark tag marked BT #1, Elev: 100.00 ft.

BT #2 is a nail enclosed in a yellow plastic casing, located on the southeast side of a 23" diam. Spruce, set level 1.2 ft. above the ground. Attached is a yellow plastic bench mark tag marked BT #2, Elev: 83.54 ft.

DATE OF SURVEY:
BEGINNING: Oct. 12, 2009
ENDING: Oct. 12, 2009

GLACIER RANGER DISTRICT

EQUIPMENT: Nikon NPL-362 Total Station Nomad Data Collector with Survey Pro

SITE SURVEY

Upper Bertha Creek NonMotorized Bridge Crossing (Part of the Iditarod National Historic Trail)

Located in unsurveyed Section 26, T. 8 N., R. 1 E., Seward Meridian

CHUGACH NATIONAL FOREST
3301 C STREET, SUITE 300
ANCHORAGE, ALASKA 99503

DRAWN BY/DATE:	CHECKED BY/DATE:	PROJECT NO:	SHEET NO.
R.Barnes/11-20-2009	R. Schrank / 12-1-2009	7013-10	1 of 1
FILE NAME: upper_bertha_crk_INHTbridge.dwg			

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DESIGN: NFS
DRAWN: NFS
CHECKED: SEH
APPROVED: SEH
SCALE: AS NOTED
SEAL

CLIENT/PROJECT
UNIVERSITY OF ALASKA GRADUATE PROJECT
KENAI PENINSULA, ALASKA

DRAWING TITLE
SURVEY

DWG NO. S-3
REV

PROJ NO. CE A686
FILE
SHEET 3 OF 3