Report No. KS-18-04 ▪ FINAL REPORT ▪ August 2018

Comparison of NU I-Girders and K-Girders for Use in Kansas Pretensioned Concrete Bridges

Robert J. Peterman, Ph.D., P.E.
Yu-Szu Chen

Kansas State University Transportation Center

AD ASTRA PER ASPERA
Kansas
Department of Transportation

1 Report No. KS-18-04	2 Government Accession No.	3 Recipient Catalog No.
4 Title and Subtitle Comparison of NU I-Girders and K-Girders for Use in Kansas Pretensioned Concrete Bridges		5 Report Date August 2018 6 Performing Organization Code
7 Author(s) Robert J. Peterman, Ph.D., P.E., Yu-Szu Chen		7 Performing Organization Report No.
9 Performing Organization Name and Address Kansas State University Transportation Center Department of Civil Engineering 2118 Fiedler Hall Manhattan, KS 66506-5000		10 Work Unit No. (TRAIS) 11 Contract or Grant No. C2064
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Research 2300 SW Van Buren Topeka, Kansas 66611-1195		13 Type of Report and Period Covered Final Report February 2015–August 2017 14 Sponsoring Agency Code RE-0683-01
15 Supplementary Notes For more information write to address in block 9.		

Over the past five decades, prestressed concrete bridge girders have evolved from traditional bulky shapes to efficient girder cross-sections with long spans and wide, thin top and bottom flanges. The objective of this research study is to provide the Kansas Department of Transportation (KDOT) with the information needed to make an informed decision about possible adoption of NU girders, including the data to determine whether or not wide-scale adoption is warranted.

The investigation compared NU girders and Kansas K-girders in a parametric study of bridge superstructure designs using CONSPAN software, including evaluation of anticipated costs that include material, labor, and transportation. The bridge design procedure was based on the American Association of State Highway and Transportation Officials (AASHTO, 2012) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* (6th edition). Additional design guidelines were referenced from the Precast/Prestressed Concrete Institute's (PCI, 2014) *Precast Prestressed Concrete Bridge Design Manual* (3rd edition), and the KDOT (2015) *Design Manual, Volume III – Bridge Section*.

The overall finding of this study is that K-girders should continue to be used instead of NU girders whenever normal spans and girder spacing allow, as this will likely result in the most economical superstructure. At longer spans (beyond 130–140 ft) NU girders are an excellent option and should become a standard design implementation to extend the applicable range of pretensioned girders to 200 ft and beyond. Additionally, the NU girder system can be used for the purpose of extending the span range (beyond K-girder capabilities) in specific situations where the maximum girder height is fixed. However, as shown previously through analyses, if K-girders can achieve the desired span at a normal spacing, then these will likely provide the most economical option.

17 Key Words Girders, NU Girders, K-Girders, Bridge Design		18 Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service www.ntis.gov.	
19 Security Classification (of this report) Unclassified	20 Security Classification (of this page) Unclassified	21 No. of pages 118	22 Price

Form DOT F 1700.7 (8-72)

This page intentionally left blank.

Comparison of NU I-Girders and K-Girders for Use in Kansas Pretensioned Concrete Bridges

Final Report

Prepared by

Robert J. Peterman, Ph.D., P.E.
Yu-Szu Chen

Kansas State University Transportation Center

A Report on Research Sponsored by

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

and

KANSAS STATE UNIVERSITY TRANSPORTATION CENTER
MANHATTAN, KANSAS

August 2018

NOTICE

The authors and the state of Kansas do not endorse products or manufacturers. Trade and manufacturers names appear herein solely because they are considered essential to the object of this report.

This information is available in alternative accessible formats. To obtain an alternative format, contact the Office of Public Affairs, Kansas Department of Transportation, 700 SW Harrison, 2^{nd} Floor – West Wing, Topeka, Kansas 66603-3745 or phone (785) 296-3585 (Voice) (TDD).

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or the policies of the state of Kansas. This report does not constitute a standard, specification or regulation.

Abstract

Over the past five decades, prestressed concrete bridge girders have evolved from traditional bulky shapes to efficient girder cross-sections with long spans and wide, thin top and bottom flanges. The objective of this research study is to provide the Kansas Department of Transportation (KDOT) with the information needed to make an informed decision about possible adoption of NU girders, including the data to determine whether or not wide-scale adoption is warranted.

The investigation compared NU girders and Kansas K-girders in a parametric study of bridge superstructure designs using CONSPAN software, including evaluation of anticipated costs that include material, labor, and transportation. The bridge design procedure was based on the American Association of State Highway and Transportation Officials (AASHTO, 2012) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* (6th edition). Additional design guidelines were referenced from the Precast/Prestressed Concrete Institute's (PCI, 2014) *Precast Prestressed Concrete Bridge Design Manual* (3rd edition), and the KDOT (2015) *Design Manual, Volume III – Bridge Section*.

The overall finding of this study is that K-girders should continue to be used instead of NU girders whenever normal spans and girder spacing allow, as this will likely result in the most economical superstructure. At longer spans (beyond 130–140 ft) NU girders are an excellent option and should become a standard design implementation to extend the applicable range of pretensioned girders to 200 ft and beyond. Additionally, the NU girder system can be used for the purpose of extending the span range (beyond K-girder capabilities) in specific situations where the maximum girder height is fixed. However, as shown previously through analyses, if K-girders can achieve the desired span at a normal spacing, then these will likely provide the most economical option.

Acknowledgments

First, the authors would like to thank the Kansas Department of Transportation (KDOT) for funding this project, as well as the KDOT Bureau of Structures and Geotechnical Services for assistance provided.

We would also like to thank Dr. Shri Bhide at Bentley Systems who provided two years of access to commercial bridge design software (CONSPAN) for this project. Additionally, we would like to thank Mr. Dilip Matange, engineering consultant at Bentley Systems, for assisting with the use of CONSPAN and troubleshooting user-input issues.

The authors would like to acknowledge Ms. Nancy Turner, Program Manager in Transportation Systems at the Precast/Prestressed Concrete Institute (PCI), who provided the most recent version of the PCI Bridge Design Manual for this research project.

Finally, we would like to thank Engineering Manager Mr. Terry Fleck and Vice President and General Manager Mr. Mark Simpson at Coreslab Structure, Inc. (Kansas), for sharing their firsthand experiences in producing both K-Girders and NU girders, and for the opportunity to visit their manufacturing facility.

Table of Contents

List of Tables

List of Figures

Chapter 1: Introduction

For many years, the state of Kansas has been using K-Girders for pretensioned concrete bridges. The cross-sectional shape of these girders is very similar to the American Association of State Highway and Transportation Officials (AASHTO) girder shapes adopted in the 1950s by the Bureau of Public Roads (known today as the Federal Highway Administration). These shapes reflected state-of-the-art design methods for the time period when pretensioned concrete was relatively new, along with moderately low concrete strength (f'_c =5000-7000 psi). However, as the industry has gained experience, coupled with advances in concrete technology, many states have transitioned from the traditional bulky girder shapes to more efficient girder cross-sections, which allow longer spans and more efficient designs to be achieved.

The efficiency of the new girder shapes has come primarily through making top and bottom flanges both wider and thinner. The added width of the top flange has significantly provided better lateral stability. On the other hand, the added width of the bottom flange has enabled the center of gravity of the prestressing strands to be lower by allowing more prestressing strands to fit into the bottom-most rows.

Accordingly, an increase in the moment-resisting capacity for a given structural height has been achieved, and the concrete compression area in negative-moment regions has also increased. The result has been the extension of the use of pretensioned concrete girders to spans which had previously only been attainable through use of structural steel beams. Several states have taken the lead in developing new girder cross-sections, including Florida, Washington, Kentucky, and Nebraska.

The prestressing sections used in Nebraska were developed by the University of Nebraska in the early 1990s under the direction of Dr. Maher Tadros and were called the “NU” girder sections. These structurally efficient sections have greatly increased in popularity and are currently being used by several states in addition to Nebraska. The purpose of this research is to investigate the possible adoption of these NU girder sections by the state of Kansas for pretensioned concrete bridges.

1.1 Organization of the Report

This report consists of eight chapters. Chapter 1 provides basic concepts about the research. Chapter 2 provides survey results from various state Departments of Transportation (DOTs) in the United States. Chapter 3 introduces information about the NU girder, and the commercial software which is used in the future analysis. Chapter 4 discusses the methodology of analysis using commercially available software (CONSPAN) and the design parameters. Chapter 5 provides the analysis results in single-span prestressed concrete bridges in terms of span length, strand patterns (straight, de-bond, and harp), and girder series with consistent girder spacing of 8 and 10 ft. Chapter 6 presents information pertaining to a visit to the Coreslab Structures, Inc., casting plant in Kansas City, KS. Chapter 7 presents information pertaining to cost analysis and pricing. Chapter 8 presents the summary of the research, and recommendations for utilization of the results.

1.2 Objective

The primary objective of this research is to provide the Kansas Department of Transportation (KDOT) with the information needed to make an informed decision about possible adoption of NU girders, including whether or not wide-scale adoption is warranted.

1.3 Scope

A parametric study of bridge superstructure design was conducted using Bentley Systems CONSPAN software. The study compares the characteristics of bridge superstructures composed of either NU girders or Kansas K-girders. The parametric study evaluated cases where either beam section could be used, as well as investigating terms of spans length which might possibly only be achieved using the more efficient NU girder sections. Structural performance was evaluated by the following criteria:

1. Maximum span achieved by each section for a standard height and spacing.
2. Overall weight of the section for a given span height and spacing.
3. Stiffness of the sections for a given span and spacing.
4. Stiffness versus weight of the different sections.

Furthermore, state DOTs which have already used the NU girder sections on existing projects were contacted about their experiences in using these girders.

In addition to the structural performance, a comparison of anticipated costs was also investigated. Anticipated cost differences were evaluated, in part, from the differences found by the parametric analysis above, and through information gained from discussions with Coreslab personnel who have unique experience in the manufacture and shipping of both K-girders and NU girders.

Cost differences were evaluated in the specific areas of materials, labor, transportation, and installation costs. The Coreslab Kansas City plant has already purchased and installed NU girder forms on three of its prestressing beds. As part of this study, the authors visited the Coreslab facility to meet with key personnel to discuss their experiences and findings in terms of differences in casting operations and costs.

Chapter 2: NU Girder Survey

At the beginning of the project, a survey about NU girders was sent to key persons at each of the state's Departments of Transportation. Key contact persons were selected with the help of KDOT. The purpose of the NU girder survey was to collect each state DOT's firsthand knowledge and experience pertaining to NU girder use. A total of 50 participants were contacted: 49 other states plus the District of Columbia.

The survey question form is presented in Appendix A. Survey results were classified into three categories (Figure 2.1): No response, Yes (currently using), and No (not using). According to results, 42 states do not use NU girders, and two states, Missouri and Nebraska, are currently using NU girders. Six states did not respond to the survey: Colorado, Connecticut, Idaho, Rhode Island, Vermont, and Washington. Survey responses are detailed in the following sections.

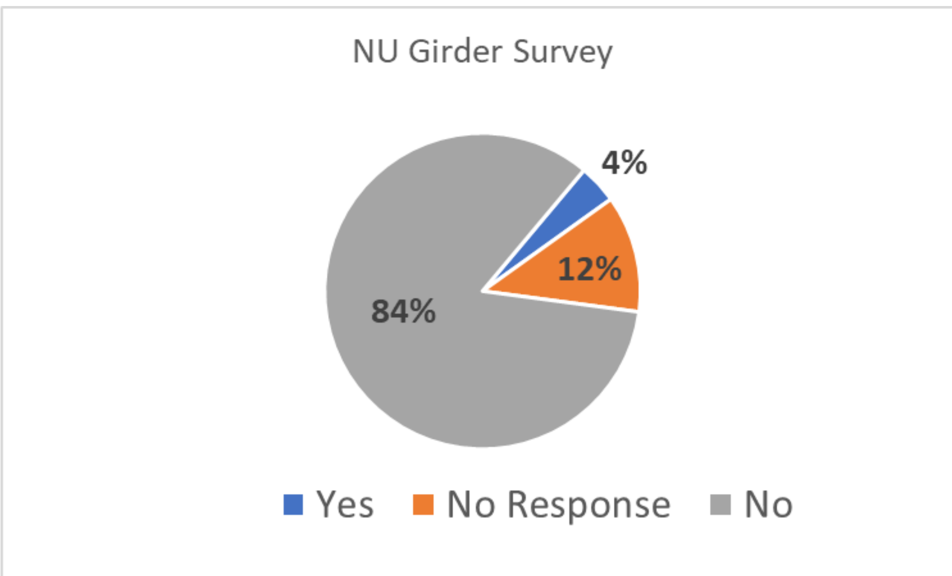


Figure 2.1: NU Girder Survey Results

Although survey results were based on state DOT's answers regarding current NU girder usage, some states, such as Washington and Florida, may have developed girders based on the NU girder due to similarities in girder section properties and published references. In addition, a survey participant may not have had sufficient knowledge about girder history and had difficulty answering the questions, and thus incorrectly concluded that its state does not use NU girders.

2.1 Survey Results Using Current NU Girder System

The NU girder system has been used in the state of Nebraska since 1995 and the state of Missouri began using it in 2007. Neither state adopted the tallest (NU-2400) nor shortest (NU-750) girder, and the state of Missouri did not use the NU-2000 girder.

Nebraska has completed design specifications for application of the NU girder system, and the design aid has details of applicable girders with corresponding span lengths and concrete strengths. The state of Nebraska is experienced in using the NU girder system, resulting in no issues in design, fabrication, transportation, or erection. Even though deck replacement and long-term maintenance issues were an initial concern, these issues were eliminated.

The longest span length ever used is 187 ft without any issue in transportation. In the standard case, the concrete strength used is 5–6 ksi at release, corresponding to 8–10 ksi at service. The highest concrete strength ever used is 7 ksi at release and 12 ksi at service.

The state of Missouri uses similar concrete strength as the state of Kansas, and both have used 10 ksi concrete strength in special cases. The maximum span length of 148 ft was used, as longer girders had issues of stability and security during transporting. Additionally, Missouri had difficulty passing the small radius turning route because of girder size. Moreover, Missouri experienced one girder tipping over on an interstate highway ramp, resulting in extra costs and time delays. The other area related to cost was in fabrication, where one girder had an imperfection from cracking and damages on a section upon form removal or strand release.

In deck placement, the execution method was associated with cost due to formation of a slab cantilever. Furthermore, erection costs increased when larger cranes were needed. Thus, costs may be reduced after the contractor and precaster become more familiar with the NU girder. The detailed survey response is presented below:

- Year state began using the University of Nebraska I-girder system:
  - **Nebraska**: 1995
  - **Missouri**: 2007
- Sizes/depths of University of Nebraska I-girders currently employed:
  - **Nebraska**: NU-900, 1100, 1350, 1600, 1800, 2000
  - **Missouri**: NU-900, 1100, 1350, 1600, 1800

- Maximum span length used for NU girders:
    - **Nebraska**: We have shipped a 187-ft NU-2000. We have design charts showing every section's capabilities.
    - **Missouri**: 148 ft
- Concrete Strength at transfer and at service:
    - **Nebraska**: Typical release strength is 5000 to 6000 psi, and we have gone to 7000 psi. Typical strength is 8000 psi and 10,000 psi (we have gone to 12,000 psi).
    - **Missouri**: As a standard, we use 8 ksi for service and strength limit, and 6.5 ksi for transfer. We have used values of 10 ksi and 8 ksi concrete in special cases.
- Any difficulties experienced by precasters when implementing sections at plants:
    - **Nebraska**: No issues.
    - **Missouri**: We have had issues with our two precasters having slightly different forms. We have looked into how this will affect weight and camber, and found that it is insignificant.
- Impact on costs associated with using the NU girder system compared to other prestressed concrete girder systems in:
    - Design area
        - **Nebraska**: No issues. We found out the NU section is more economical and gives better performance.
        - **Missouri**: The initial learning curve cost us some added design cost. Also, with two forms of reinforcement for the web, we have added design and detailing costs to produce the extra plan sheet(s).
    - Fabrication / Labor
        - **Nebraska**: No issues.
        - **Missouri**: We have experienced significant cracking on the top flange and at the girder ends, along with damaged flanges from

removal/strand release issues. These issues required a follow-up repair proposal by the precaster before we accepted the girder.

- Transportation
  - **Nebraska**: No issues.
  - **Missouri**: The long girders have had some stability issues during transportation. One girder actually tipped over on an Interstate ramp. The size and width of the top flange have posed some issues with securing the load to the trailer. The long girders are hard to get to the site on routes where the turning radius is smaller. We have a lot of these routes with our hilly topography.
- Erection area
  - **Nebraska**: No issues.
  - **Missouri**: These girders require much larger cranes than our standard sections. This drives up the cost somewhat.
- Deck placement
  - **Nebraska**: Might be an issue. NDOR already finished research dealing with this potential issue.
  - **Missouri**: Forming of the slab cantilever has been an issue with our contractors. They add girder inserts for form work and need to place holes in the top flange for form support.
- Long-term maintenance
  - **Nebraska**: No issue, especially when we eliminated the intermediate and end joints.
  - **Missouri**: No issue, but we expect to have significant issues when it comes time for a re-deck. Our contractors tell us it is cheaper to tear off the whole superstructure rather than re-deck prestressed girders. We are hoping hydro-demolition technology will provide a means to selectively remove concrete over the girders without causing damage. We are not aware of this being tried yet.

Chapter 3: Literature Review

Use of precast, prestressed concrete bridges has increased due to new efficient girder shapes that overcame limitations of traditional bulky girders. AASHTO girders, which were adopted by the Federal Highway Administration (FHWA) in 1956, were the most commonly-used girder shapes for many years.

The state of Kansas adopted several AASHTO girders and bulb tees as the official state girders and refers to them as K-girders. Regarding the K-girder series, girder shapes K-2, K-3, and K-4 are traditional bulky shaped girders, while the K-6 girder is a bulb tee girder that is identical to the AASHTO/PCI bulb tee which is slightly more efficient than the original AASHTO girder series (Geren & Tadros, 1994).

Several states including Washington, Florida, Colorado, and Nebraska have developed, adopted, or modified their own standards for precast concrete I-girders based on concrete technology advances and experience gained in use of local materials, resulting in diverse and uncoordinated girder shapes throughout the United States (Geren & Tadros, 1994).

3.1 University of Nebraska Girder

In the early 1990s, the University of Nebraska-Lincoln developed a new shape I-girder under the supervision of Dr. Maher Tadros called the “NU” girder, which has been used in the state of Nebraska since 1995. Three years after NU girders were developed, the Washington State Department of Transportation (WSDOT) modified and adopted it as its state girder (Seguirant, 1998).

The NU girder has several structural advantages over the traditional girder. It has wide, thin top and bottom flanges, and is available in various girder depths: 750, 900, 1100, 1350, 1600, 1800, 2000, and 2400 mm. Its “one set of forms with web extension panels” is adequate for casting an entire NU girder series, because no distinction is necessary for top and bottom flange dimensions. In addition, the wide range of girder depth adds flexibility in replacing AASHTO girders because of similarity in girder height. Increased width on the bottom flanges of NU girders enhances the compressive strength in negative-moment regions, allowing placement of a large amount of pretensioned strands with lower centers of gravity (Geren & Tadros, 1994).

NU girders are also advantageous due to their comparatively long spans, shallow depths, and subsequent economical design. As concrete technology continues to develop, NU girders can be used with high strength concrete up to 15,000 psi. A traditional girder contains sharp angles on the outside of the flange edges and the connection of flange to web, which decreases the girder's attractiveness. The NU girder, however, improves the aesthetics of the girder because it contains a circular curve design at flange and web junctures, which also improves concrete placement and consolidation (Geren & Tadros, 1994).

3.1.1 Section Properties

A standard NU girder cross-section with strand template is shown in Figure 3.1. NU I-girder section properties listed in Table 3.1 are based on NDOR design aids for NU I-girder bridges (Hanna, Morcous, & Tadros, 2010). The K-Girder cross-sections are shown in Figure 3.2 and tabulated in Table 3.2. Figure 3.3 shows the K-Girder cross-sections along with the NU girder cross-sections with the same approximate height, with all girders plotted to the same scale. From this figure, the NU girder system clearly provides more options to the bridge designer, especially for longer spans.

Table 3.1: NU I-Girder Properties

Section	Height in. (mm)	Area in.^{2}	Y_b in.	I in.^{4}	V/S in.	Weight (kips/ft)
NU-750	29.5 (750)	614.0	13.6	69,403	3.11	0.640
NU-900	35.4 (900)	648.1	16.1	110,262	3.10	0.680
NU-1100	43.3 (1100)	694.6	19.6	182,279	3.09	0.724
NU-1350	53.1 (1350)	752.7	24.0	302,334	3.08	0.785
NU-1600	63.0 (1600)	810.8	28.4	458,482	3.07	0.840
NU-1800	70.9 (1800)	857.3	32.0	611,328	3.06	0.894
NU-2000	78.7 (2000)	903.8	35.7	790,592	3.06	0.942
NU-2400	94.5 (2400)	998.0	43.0	1,235,547	3.05	1.040

Source: Hanna, Morcous, and Tadros (2010)

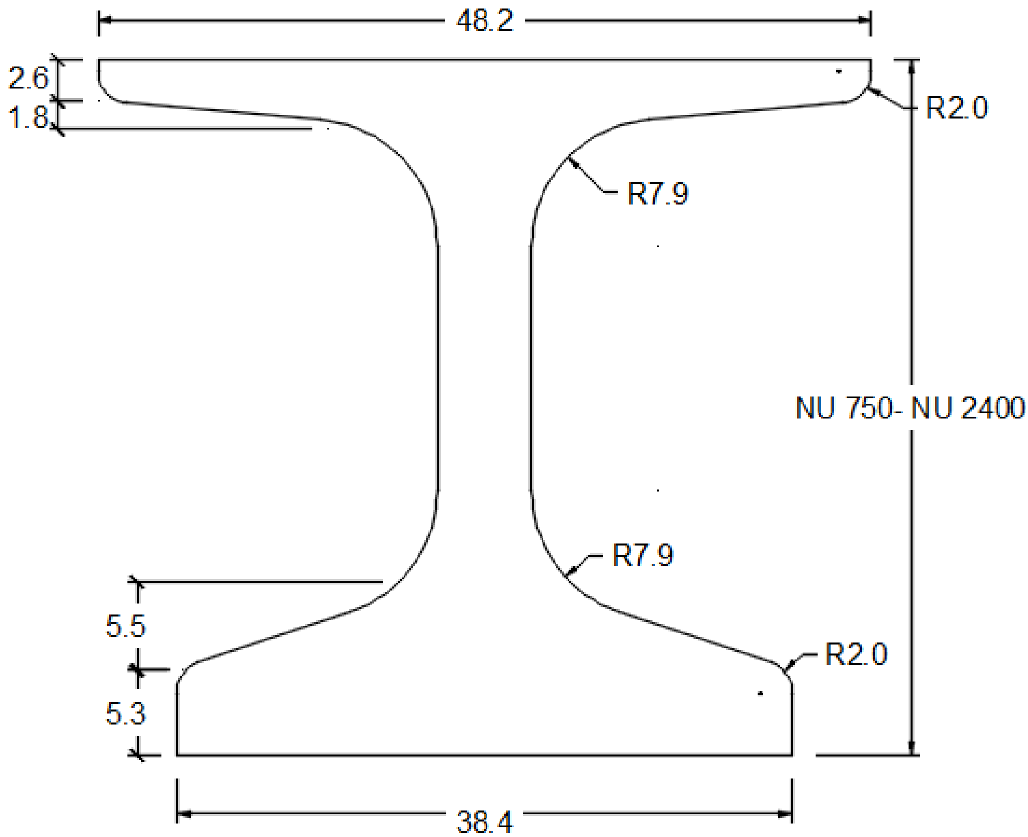


Figure 3.1: NU I-Girder Cross-Section (Dimension in Inches)
Source: Hanna, Morcous, and Tadros (2010)

Table 3.2: K-Girder Properties

Section	Height in. (mm)	Area in.^{2}	Y_b in.	I in.^{4}	V/S in.	Weight (kips/ft)
K-2	36.0 (914)	369.0	15.83	50,979	3.37	0.384
K-3	45.0 (1140)	525.1	21.02	127,487	3.56	0.547
K-4	54.0 (1370)	644.1	25.89	236,105	3.68	0.671
K-6	72.0 (1830)	767.0	36.60	545,857	3.01	0.799

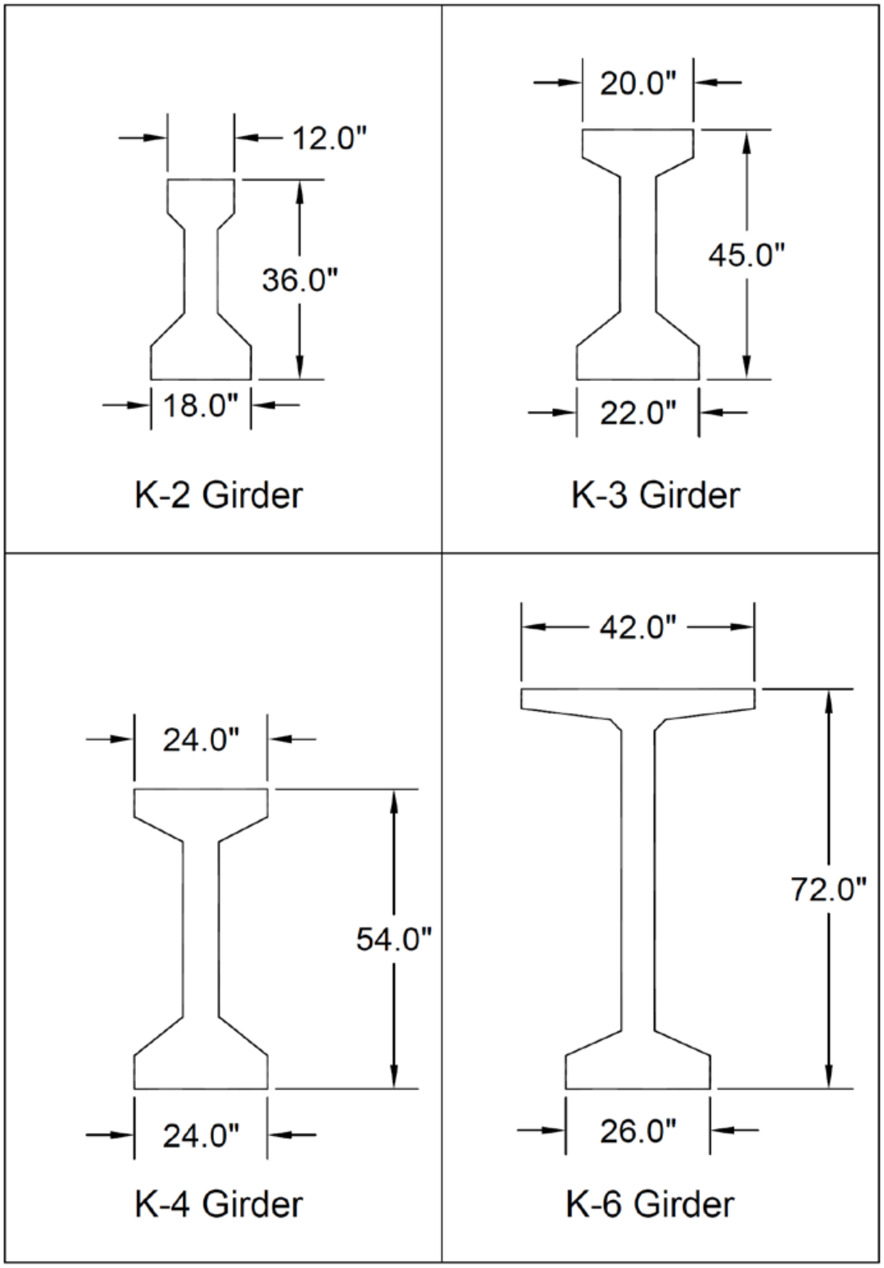


Figure 3.2: K-Girder Cross-Sections

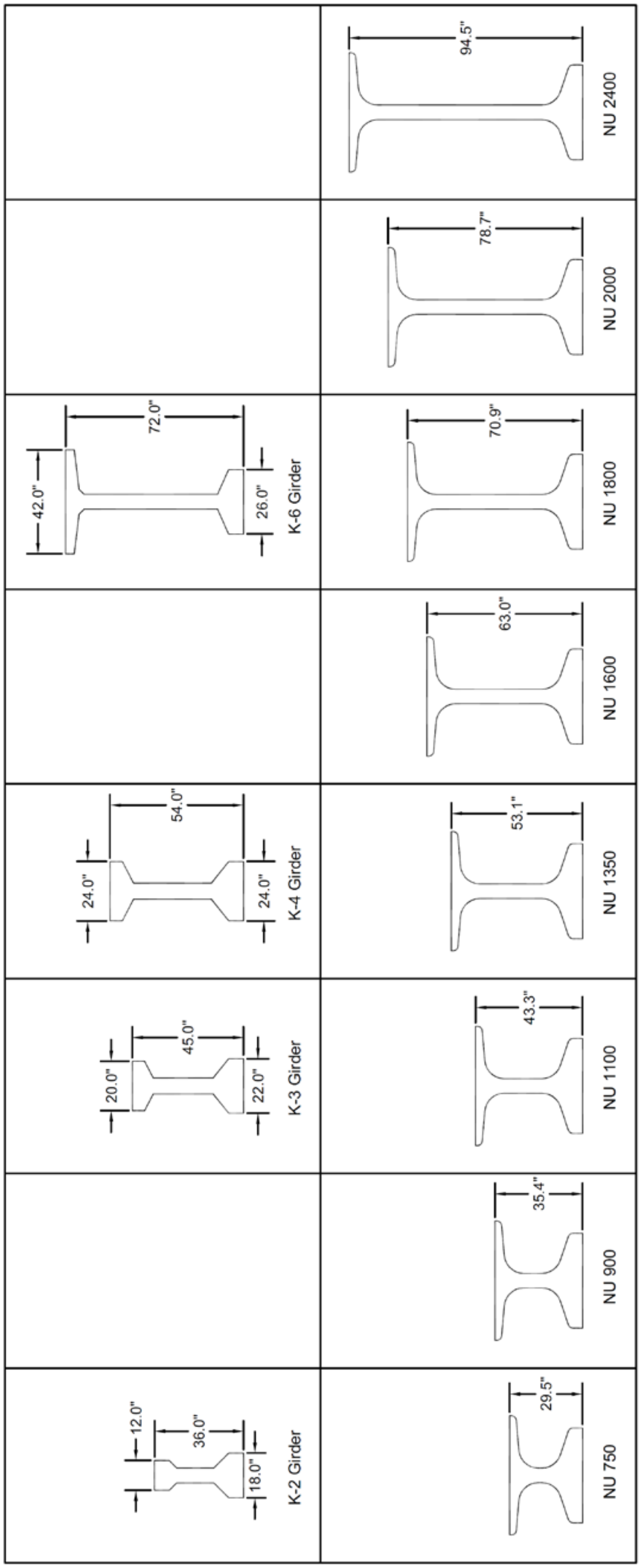


Figure 3.3: Comparison of K-Girders with NU Girders of the Same Approximate Height

3.2 Bridge Design and Analysis Program

Bentley Systems provided the bridge design software, CONSPAN V8i (SELECT series 7), for this project (Bentley Systems, Inc., 2012). CONSPAN, which is used by more than 30 state DOTs, is a comprehensive design program that incorporates both AASHTO LRFD and LFD bridge design specifications and offers automatic design generation options. It provides flexibility when adjusting design parameters and inputting analysis factors, because the user can make changes any time. In some cases, the analysis may need to start over because CONSPAN will not follow the adjustment in design parameter once the analysis is completed.

CONSPAN contains seven primary tab screens (Project, Geometry, Materials, Loads, Analysis, Beam, and Deck) as shown in Figure 3.4, and each tab contains relative input options or results data.

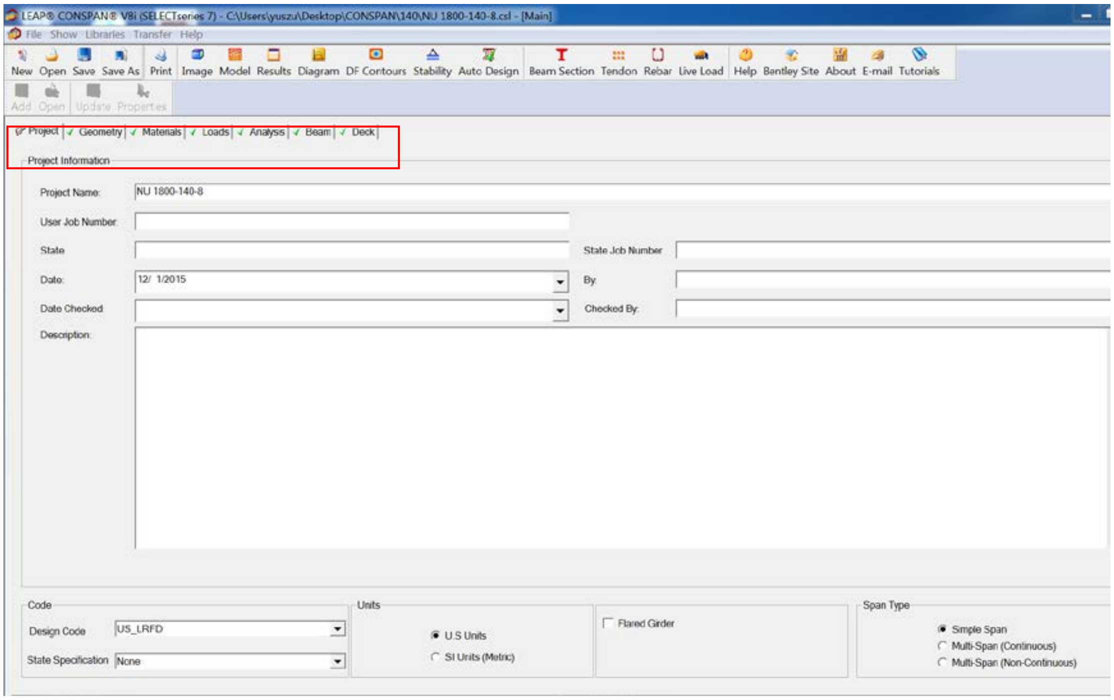


Figure 3.4: CONSPAN Tab Screens

In the project tab, the program contains three span options (simple, continuous multiple, and non-continuous multiple), units (U.S. and metric), and design codes (LRFD and LFD). Bridge layout information is entered under the geometry tab; the input screen varies depending on the span type and design code selected in the project tab. Figure 3.5 shows an example screen view of a simple-span design with NU-1800 girders and LRFD design. When the design information (overall width, skew angle, curb data, lane data, topping data, span data, and beam type/location) is input, CONSPAN generates a two-dimensional (2D) cross-section graph that conveniently illustrates the overall superstructure. Furthermore, CONSPAN offers options to view the plan or elevation using a zoom-in and zoom-out function. The view can be changed by clicking view options.

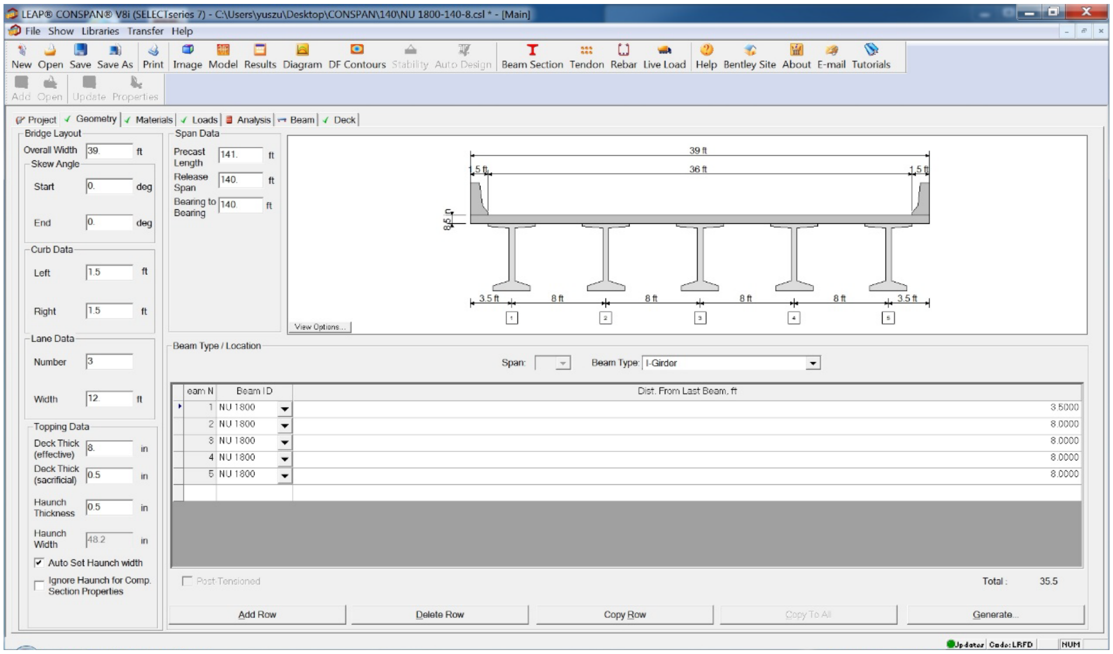


Figure 3.5: Geometry Tab Screen – Simple Span (LRFD)

Under the materials tab, the user can select or input specific concrete, prestressing tendon, rebar, and transformed section parameters (Figure 3.6). The load tab, shown in Figure 3.7, offers two methods to add dead loads: user input option and Wizard option. If the Wizard input is selected, CONSPAN automatically computes dead load into specific categories, according to LRFD specification for dead load classification, after the detailed permanent weights are input (Figure 3.8). Diaphragm, temperature, and live load details can also be modified in the load tab.

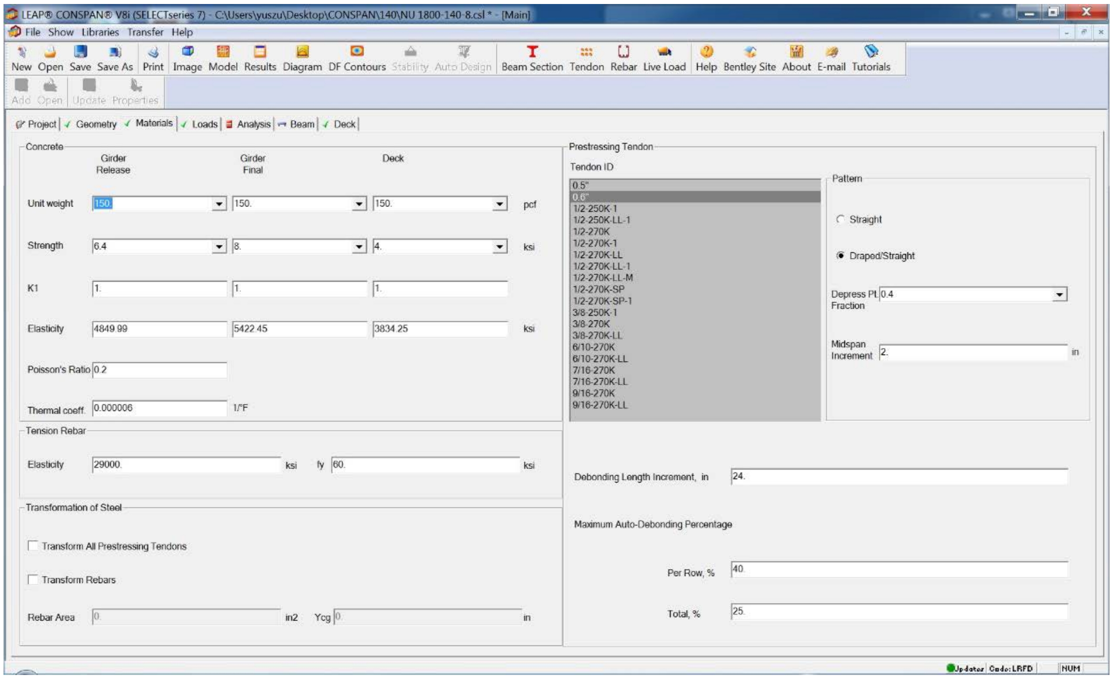


Figure 3.6: Materials Tab Screen

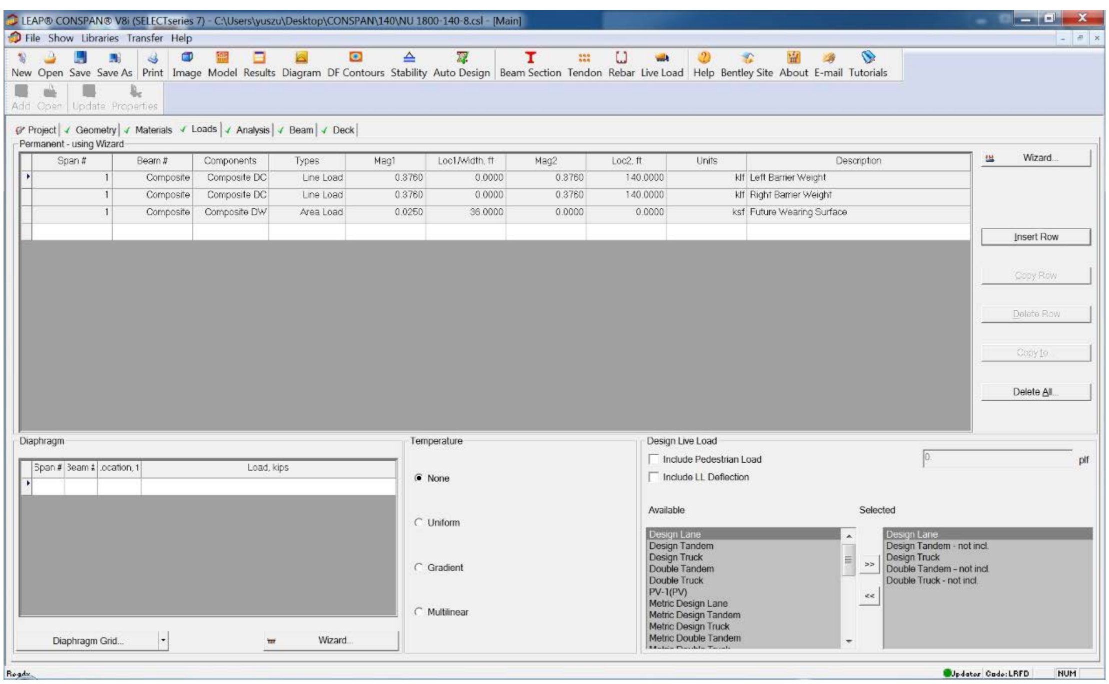


Figure 3.7: Load Tab Screen

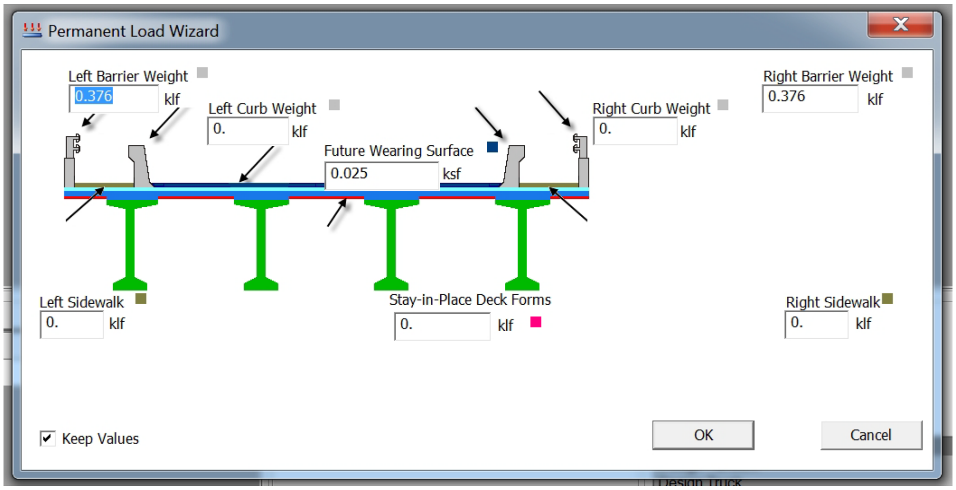


Figure 3.8: Permanent Load Wizard Screen

After all design information is entered, load case results are presented in the Analysis tab (Figure 3.9), in which the user can compute dead and live-load results by selecting "run analysis."

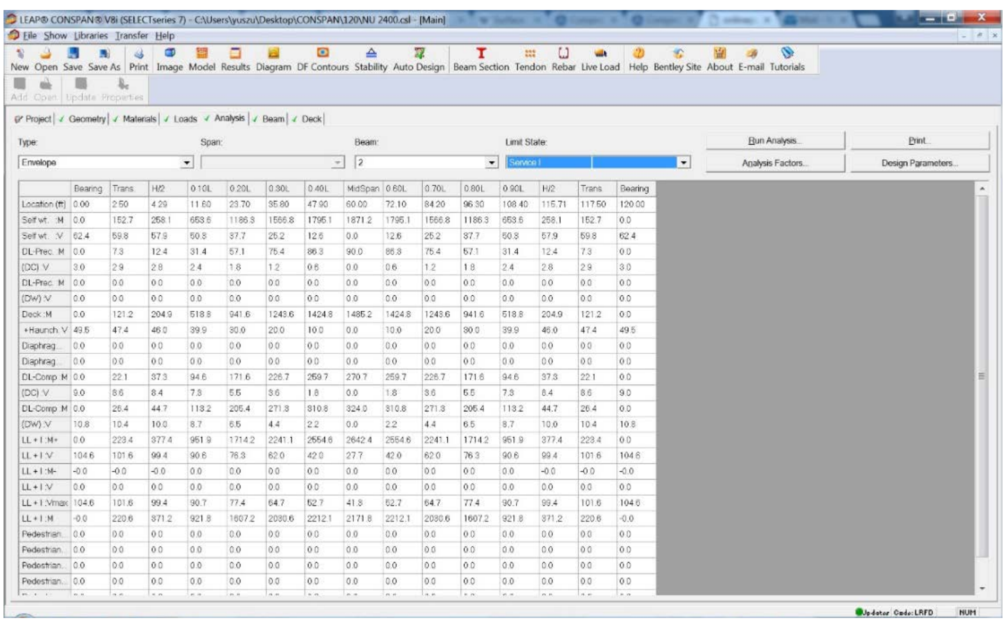


Figure 3.9: Analysis Tab Screen

Loading analysis output can be displayed in both raw and envelope form, and the enveloped results are multiplied by the appropriate safety factors, distribution factors, impact factors, etc. Although basic design methods and factors are built into CONSPAN according to LRFD or LFD design codes, these factors can be modified in the analysis factors tab as shown in Figure 3.10.

Design methods can be adjusted on the design parameters screen, including limiting allowable stress, moment and shear preliminary design method, and resistance factor/losses setting, as shown in Figure 3.11.

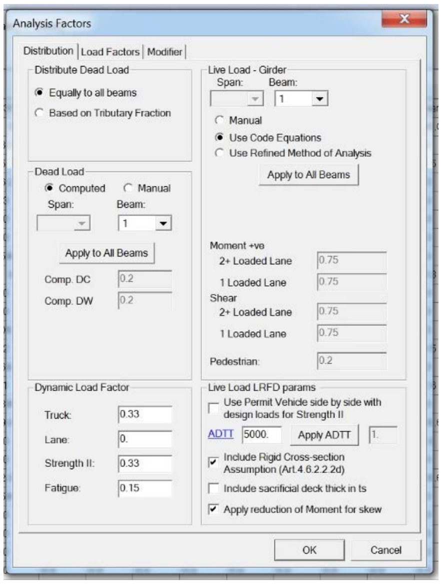


Figure 3.10: Analysis Factors Screen

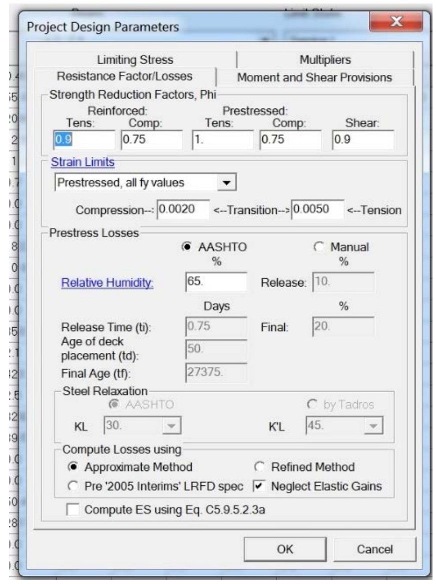


Figure 3.11: Design Parameters Screen

The beam tab (Figure 3.12) shows the number of beams used in the project. The user can double click the beam to place strands. In the strand-pattern design tab, shown in Figure 3.13, concrete stress can be checked on the right side of the screen after strands are added. Moreover, harped and debond design can be selected in this tab.

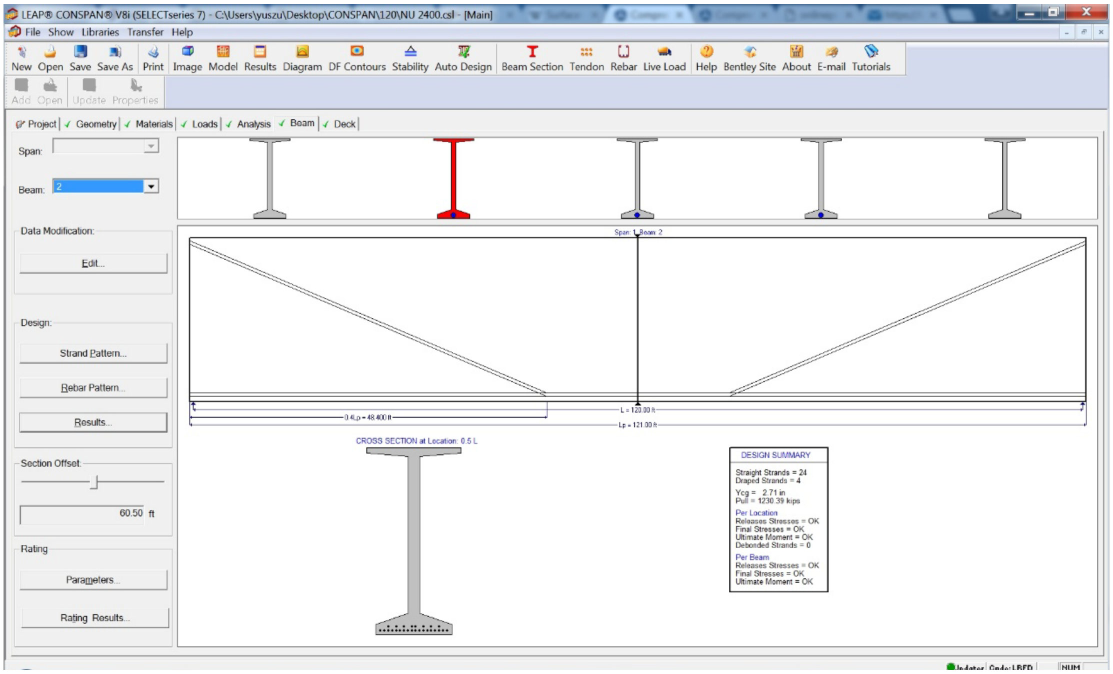


Figure 3.12: Beam Tab Screen

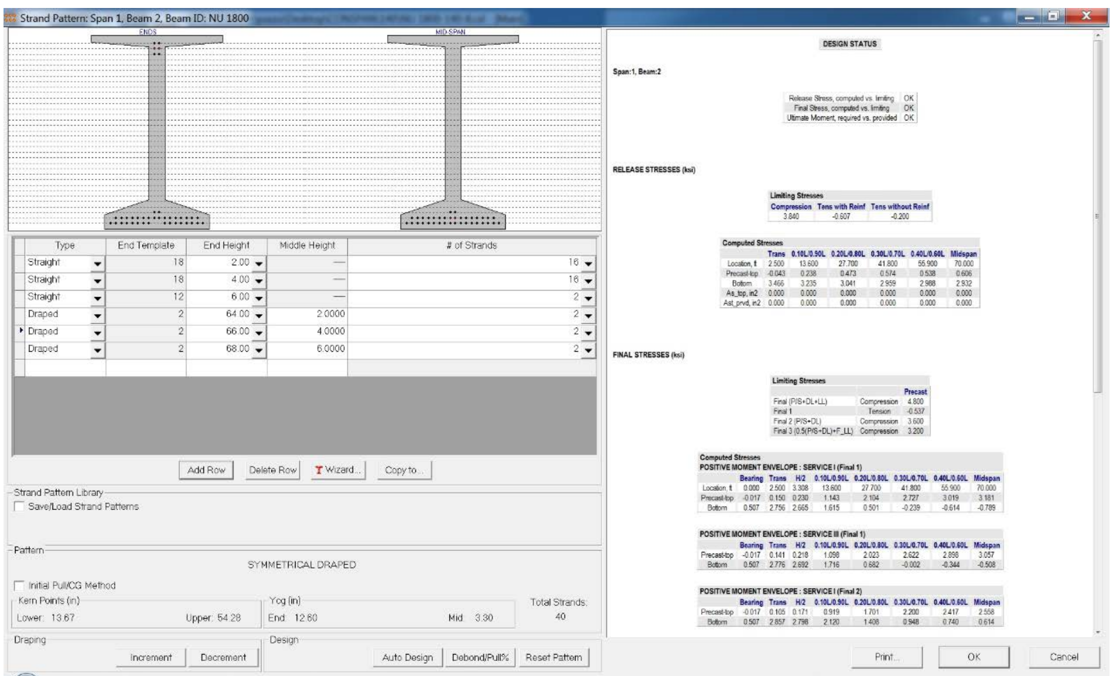


Figure 3.13: Strand Pattern Design Screen

The design results can be viewed with the "results" button by returning to the beam tab. Deck design can be reviewed in the deck tab, which contains two built-in design methods (empirical and approximate).

The libraries section, shown in Figure 3.14, stores information such as beam sections, prestressing tendons, rebar, and live loads; however, the user can add new sections or modify the data in each section. For example, in the beam section library screen (Figure 3.15), the beam section (Figure 3.16) can be created or changed by clicking the edit button at the bottom of the section detail screen (Figure 3.17). The strand pattern can then be set up in the template (Figure 3.18).

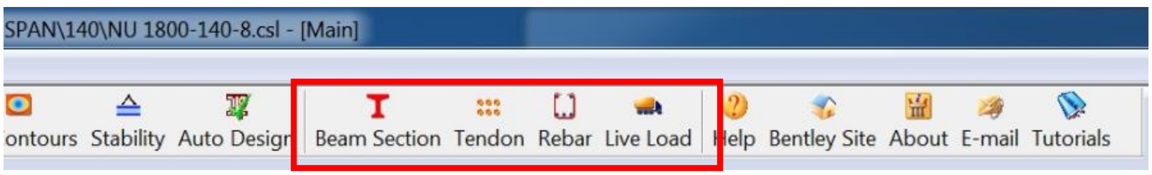


Figure 3.14: Libraries Section

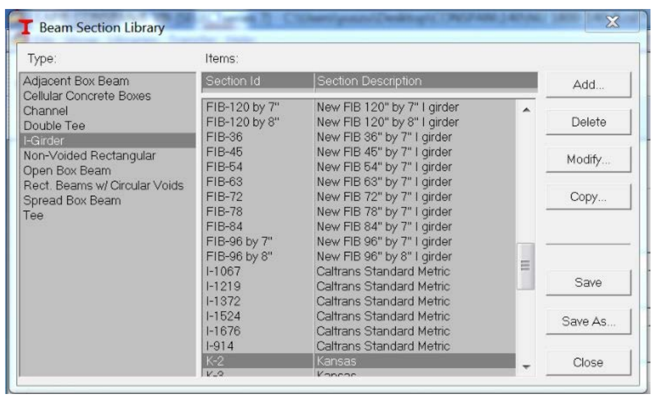


Figure 3.15: Beam Section Library

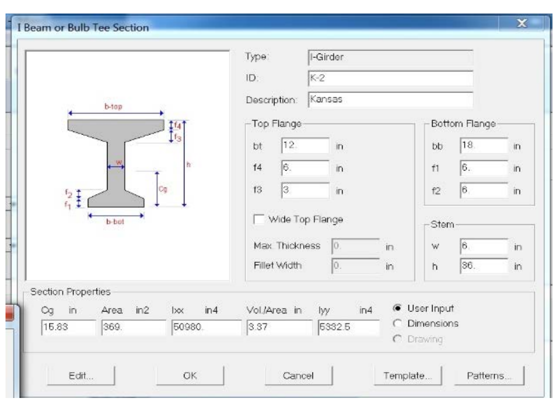


Figure 3.16: Beam Section Detail

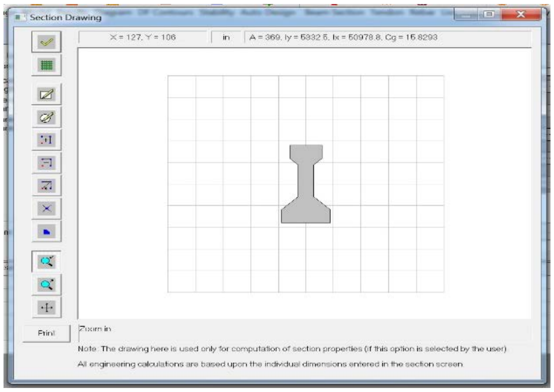


Figure 3.17: Beam Drawing Section

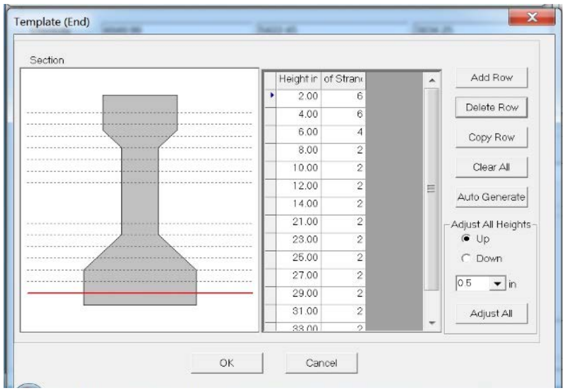


Figure 3.18: Strand Pattern Setting

Chapter 4: Design Methodology and Comparisons

This section discusses design assumptions and differences in design procedures obtained by comparing CONSPAN results with an independent analysis using Microsoft Excel and RISA-3D. This separate, independent analysis was considered a crucial first step in order to understand the embedded calculations being performed by CONSPAN, since the large parametric study relied heavily on this commercial software. Superstructure design in this section is in accordance with the AASHTO (2012) *LRFD Bridge Design Specifications* and the KDOT (2015) *Bridge Design Manual*.

4.1 Design Assumptions of Pretensioned Precast Girders

Design parameters for this study were approved by KDOT and adhered to KDOT LRFD prestressed beam design guidelines. Transformed section and elastic gain were not applied in this project. The NU girder strand placement template is based on NDOR NU I-beam design aids. The design parameters and assumptions are summarized below.

- Design Code:
    - *AASHTO LRFD Bridge Design Specifications*, 6th Edition (AASHTO, 2012)
    - *Kansas Department of Transportation Design Manual, Volume III – Bridge Section* (KDOT, 2015)
    - *Precast Prestressed Concrete Bridge Design Manual*, 3rd Edition (PCI, 2014)
- Design Criteria:
    - Service I: Shear and moment force [un-factored dead load (DL) and live load (LL)] are computed by RISA-3D.
    - Prestress losses:
        - Elastic shortening
        - Time-dependent losses: approximate method
    - Estimate required prestress
    - Service I and Service III: release stress and finial stress
    - Strength I: ultimate-moment limit state
    - Strength I: shear design

- Structural System:
  - Simple span length: targeted spans of 120 ft minimum to 160 ft maximum with others considered for comparison purposes
  - Total deck width: 47 ft
- Girder Sections:
  - NU-750, NU-900, NU-1100, NU-1350, NU-1600, NU-1800, NU-2000, NU-2400
  - K-2, K-3, K-4, K-6
  - Interior girder
  - Concrete unit weight (w_c): 0.15 kcf
- Girder Spacing:
  - 8 ft and 10 ft
- Girder Compressive Strength at Final:
  - 8 ksi
- Girder Compressive Strength at Release:
  - limit f'_{ci} min. = $0.8f'_c = 0.8 * 8 = 6.4$ ksi
- Deck Concrete:
  - 4 ksi
- Deck Thickness:
  - 8.5 in., including 0.5-in.-thick wearing surface
- Haunch:
  - Width = 48.2 in. for NU girder
  - Width varies according to the width of top flange for K-girder
  - Thickness = 0.5 in.
- Strand Type:
  - 0.6 in., grade 270 low-relaxation, E_s = 28,500 ksi
  - Area = 0.217 $in.^2$
  - Yield strength = 0.9 f_{pu} = 243 ksi
  - Jacking stress = 0.75 f_{pu} = 202.5 ksi

- Strand Arrangement:
    - NU girder: 60 strands – 7 rows (18, 18, 12, 6, 2, 2, 2) @ 2 in. × 2 in. grid spacing
    - K-6 girder: 42 strands – 7 rows (12, 12, 8, 4, 2, 2, 2) @ 2 in. × 2 in. grid spacing
    - K-4 girder: 42 strands – 8 rows (10, 10, 8, 6, 2, 2, 2, 2) @ 2 in. × 2 in. grid spacing
    - K-3 girder: 32 strands – 7 rows (8, 8, 6, 4, 2, 2, 2) @ 2 in. × 2 in. grid spacing
    - K-2 girder: 24 strands – 7 rows (6, 6, 4, 2, 2, 2, 2) @ 2 in. × 2 in. grid spacing
    - Straight strands
    - Harp strands at 0.4L and 0.6L
- Dead Load (DL):
    - Girder weight
    - Deck weight
    - Haunch weight
    - 2-ft, 8-in. Kansas corral rail with curb: 376 plf
    - Wearing surface: 25 (psf) for one-course decks with 2.5-inch clearance to top of reinforcing
- Live Load (LL):
    - HL-93:
        - Design lane load of 0.64 kips/ft without dynamic allowance
        - Design truck or design tandem
- Humidity:
    - 65% in Kansas

4.2 Comparison of CONSPAN, Microsoft Excel, and RISA-3D

This section compares the results from CONSPAN along with a separate analysis performed using Microsoft Excel and RISA-3D. A majority of design results obtained using CONSPAN were identical to those generated using Microsoft Excel and RISA-3D. However, a few of the results differed slightly; these differences are discussed in the following sections. Microsoft Excel calculations and RISA-3D analyses were performed independently and were systematically revised in order to understand the design assumptions and calculation methods contained within CONSPAN.

Microsoft Excel 2013 was used for data entry, equation computation, and results indication to compare to CONSPAN. RISA-3D was used for independent live-load analyses, and determination of the values of shear, moment, and reactions for the different span lengths.

In the following section, a comprehensive design of an interior girder is presented. A single-span bridge length of 120 ft was used as selected for the comparison purposes with NU-2400 girders at a spacing of 8 ft. Twenty-eight total strands were utilized, with four of the strands harped, according to the strand pattern shown in Figure 4.1 and Figure 4.2.

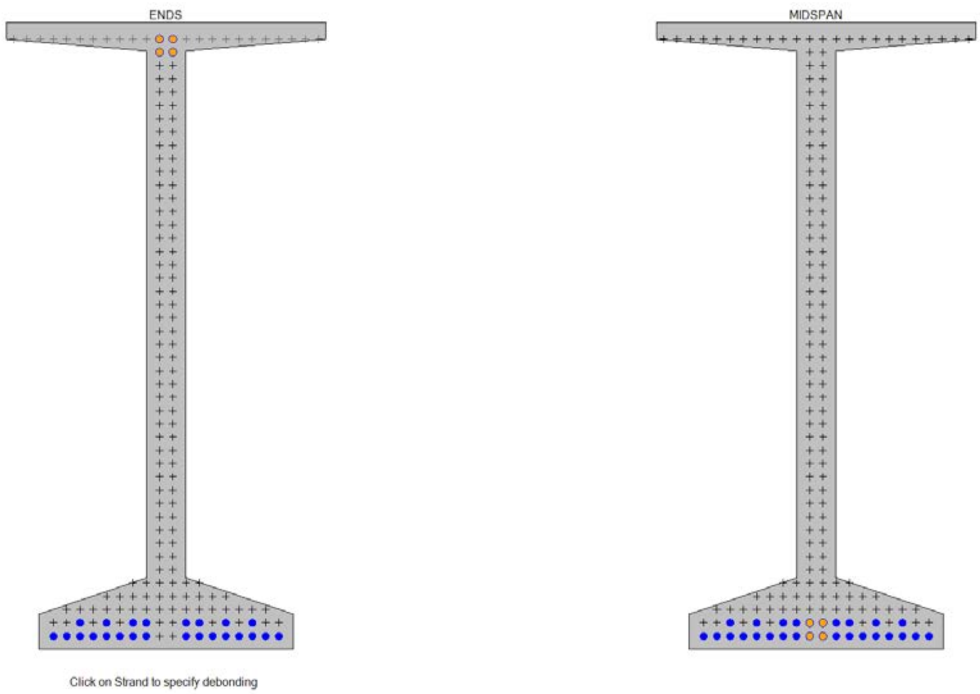
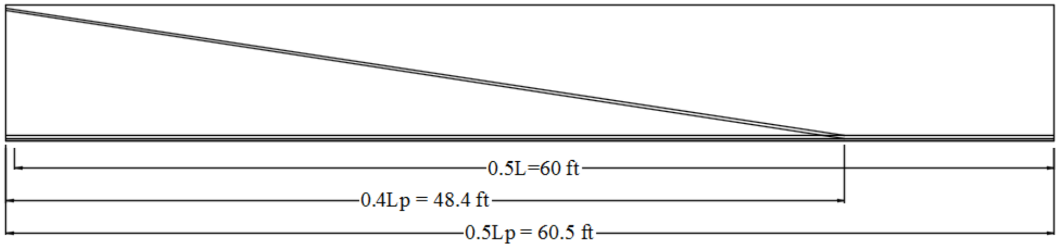


Figure 4.1: Strand Pattern on Selected Beam



Lp = Precast length

L= Design length

Figure 4.2: Strand Pattern on Beam Elevation View

4.2.1 Distribution Factor

This study used AASHTO equations for the hand calculation of the distribution factor for live load moment and shear, and the factors were then compared to CONSPAN's computation. Table 4.1 compares CONSPAN and the calculated distribution factors.

Table 4.1: Comparison of Calculated Distribution Factors

	Hand calculation	CONSPAN	Difference, %
1-lane moment	0.499	0.497	0.27%
2+-lane moment	0.724	0.723	0.08%
1-lane shear	0.680	0.681	0.12%
2+-lane shear	0.814	0.825	1.35%

From Table 4.1, the distribution factors resulting from both Excel calculation and CONSPAN are relatively small, with the maximum difference occurring for the two-lane shear distribution factor (a 1.35% difference from CONSPAN-calculated values). It was subsequently determined that these slight differences in the calculated-moment and shear-distribution factor value are attributed to the span length used for calculations. CONSPAN used precast length instead of design length, while the Microsoft Excel and RISA-3D calculations used the span design length, according to AASHTO LRFD specifications (Figure 4.3).

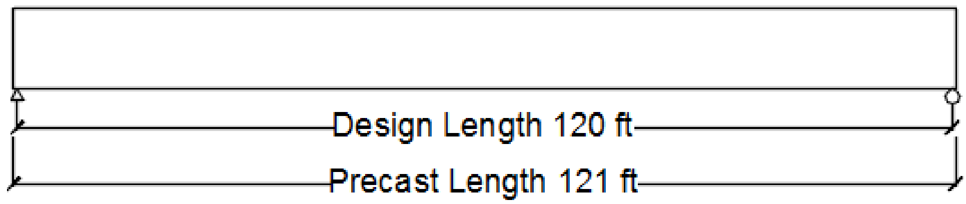


Figure 4.3: Elevation View

4.2.2 Moment and Shear

Loads applied to an interior girder can be classified as non-composite (dead), composite (dead), or live loads. Both composite and non-composite element load are classified as dead load (DL), which are subdivided into DC (dead load of structural component and nonstructural attachments) and DW (dead load of wearing surface and utilities) according to AASHTO (2012) definitions. Live load (LL) is a variable moving load, and it's considered as the worst loading case in the design. In order to compute loads on a structure, component weights were calculated according to overall concrete precast length. Un-factored dead load shear and moment forces were produced by the equations for a simply-supported span and span length from both side-center locations of bearing. The span length from an elevation view is shown in Figure 4.3. LL was computed using RISA, dynamic impact and distribution factors were multiplied in Excel file, and the critical value was then taken following comparisons of the HL-93 combinations. Microsoft Excel, RISA-3D, and CONSPAN analysis results are presented in Table 4.2.

As shown in Table 4.2, dead load forces were nearly identical between CONSPAN and Microsoft Excel. The largest differences occurred (between CONSPAN and RISA analyses) for shear values due to live loading. On the other hand, bending moment under both dead and live loading showed no significant difference. The RISA analysis indicated higher shear values than CONSPAN at sections close to mid-span; however, CONSPAN calculated higher shear forces near the end, suggesting that CONSPAN analyzed live load shear and moment by utilizing a slightly different method.

Table 4.2: Simple Support of Dead Load and Live Load on Interior Girders

Location		Mid-span	0.4L	0.3L	0.2L	0.1L	Trans.	End
Self-weight								
Microsoft Excel	M_g (k-ft)	1817.12	1795.02	1566.71	1186.20	653.48	152.56	0
	V_g (kips)	0	12.58	25.16	37.74	50.32	59.78	62.38
CONSPAN	M_g (k-ft)	1871.25	1795.15	1566.84	1186.33	653.61	152.7	0
	V_g (kips)	0	12.58	25.16	37.74	50.32	59.8	62.37
Deck and haunch								
Microsoft Excel	M_{s+h} (k-ft)	1485.08	1424.68	1243.48	9471.47	518.66	121.08	0
	V_{s+h} (kips)	0	9.98	19.97	29.95	39.94	47.44	49.51
CONSPAN	M_{s+h} (k-ft)	1485.19	1424.79	1243.58	941.57	518.76	121.2	0
	V_{s+h} (kips)	0	9.98	19.97	29.95	39.94	47.7	49.51
Deck wearing								
Microsoft Excel	M_{dw} (k-ft)	89.99	86.33	75.35	57.05	31.43	7.34	0
	V_{dw} (kips)	0	0.61	1.21	1.82	2.42	2.88	3.0
CONSPAN	M_{dw} (k-ft)	90	86.34	75.36	57.06	31.44	7.3	0
	V_{dw} (kips)	0	0.6	1.21	1.81	2.42	2.9	3.0
Barrier and wearing surface								
Microsoft Excel	M_{B+ws} (k-ft)	555.56	532.97	465.18	352.20	194.03	45.30	0
	V_{B+ws} (kips)	0	3.73	7.47	11.20	14.94	17.75	18.52
CONSPAN	M_{B+ws} (k-ft)	555.6	533	465.2	352.2	194.1	45.3	0
	V_{B+ws} (kips)	0	3.7	7.4	11.2	15	17.7	18.5
HL-93 (design lane + trucking with 33% IM)								
RISA	$M+_{LL}$ (k-ft)	2643.41	2556.70	2242.95	1715.63	952.67	180.26	0
	V_{LL} (kips)	32.28	47.03	61.14	75.25	89.35	100.20	102.55
CONSPAN	$M+_{LL}$ (k-ft)	2642.2	2554.6	2241.1	1714.2	951.9	223.4	0
	V_{LL} (kips)	27.7	42	62	76.3	90.6	101.6	104.6

The slight discrepancy in shear values between CONSPAN and RISA analysis is explained in the next sentences. CONSPAN first calculates values at the tenth points of pier-to-pier distance and uses parabolic interpolation between adjacent known values to find the value that is not exactly on a tenth point (Bentley Systems, Inc., 2012). According to Walbrun (2006), CONSPAN may consider variable support locations, such as center of the pier as support or bearing on the pier as support, resulting in the application of different bridge lengths in the analysis.

4.2.3 Prestress Losses

Total loss in prestressing steel includes both elastic shortening losses (immediately after prestress force transfer) and time-dependent losses occurring prior to the analysis stage (this section will not discuss elastic gain in the design procedure since KDOT does not consider it). In computation elastic shortening, KDOT and CONSPAN have different assumptions regarding initial prestress. CONSPAN considers 90% of initial prestress before transfer (AASHTO, 2012, Section C5.9.5.2.3a), whereas KDOT applies 70% of ultimate stress, and iterations are required until appropriate accuracy is achieved. Table 4.3 presents CONSPAN and Excel-calculation results for instantaneous prestress loss.

Table 4.3: Elastic Shortening Analysis

	Excel calculation	**CONSPAN**
Number of strands	24	24
Strand eccentricity (in)	40	40
Prestressing steel stress	$0.7f_{pu}$	90% jacking stress = 0 .675 f_{pu}
Initial pull (kips) – first iteration	984.31	949.16
f_{cgp} **(ksi) – first iteration**	1.53	1.45
Δf_{pes} **(ksi) – first iteration**	9.01	8.54
Initial pull (kips) – finial iteration	1006.14	1006.14
f_{cgp} **(ksi) – finial iteration**	1.58	1.58
Δf_{pes} **(ksi) – finial iteration**	9.31	9.31

Excel-calculated f_{cgp} and Δf_{pes} values matched CONSPAN values after five times of iterative processes, demonstrating a 2.5% difference in initial pull stress, a slight difference from analysis of the first iteration. However, the values have an excellent agreement in the final iterations, suggesting that differences in the initial assumption have no effect on the final elastic shortening computation.

The time-dependent loss is estimated by applying the AASHTO (2012) approximate method (Equation 5.9.5.3-1), including concrete creep, shrinkage, and steel relaxation loss. Time-dependent loss results showed no difference in values obtained from Excel calculations and CONSPAN, as shown in Table 4.4.

Table 4.4: Time-Dependent Loss with Application of Approximate Method

	Excel calculation	CONSPAN
Assumed humidity (%)	65	65
Relaxation of steel (ksi)	2.40	2.4
Shrinkage of concrete (ksi)	8.514	8.51
Creep of concrete (ksi)	7.50	7.50

4.2.4 Girder Stress at Critical Sections

Concrete stress was checked at two different stages (at release and under final service loading) at the mid-span, harp point, and transfer point. Concrete and fatigue stress limit values at the transfer, and service loads were based on AASHTO specifications; however, the allowable tension was considered as the lower limit ($0.0948\sqrt{f'_c}$). Prestressing force at transfer assumed the initial loss determined in the Excel and CONSPAN analyses. Table 4.5 compares concrete stress values determined at release by Excel calculation and CONSPAN; the values were identical in all cases.

Table 4.5: Computed Concrete Stress at Release

	Location	Hand calculation	CONSPAN
Concrete stress in **top** of beam (ksi)	**Mid-span**	0.267	0.267
	0.4L	0.228	0.229
	Trans.	-0.016	-0.016
Concrete stress in **bottom** of beam (ksi)	**Mid-span**	1.627	1.627
	0.4L	1.659	1.695
	Trans.	1.864	1.863

Concrete stress at service was compared for various load combinations such as Service I, Service III, and Fatigue I. Fatigue trucking force was estimated using equations of HL-93 loading from the PCI (2014) Bridge Design Manual (Table 4.6).

Table 4.6: HL-93 Fatigue Trucking Load Maximum-Bending-Moment Equation from the PCI Manual

Load type	x/L^*	Formula for maximum-bending moment, Kip-ft	Minimum	
			x, ft	L, ft
Fatigue truck	0 -0.241	$\frac{72(x)[(L-x)-18.22]}{L}$	0	44
	0.241 -0.5	$\frac{72(x)[(L-x)-11.78]}{L}-112$	14	28

Source: PCI (2014)

Table 4.7: Computed Concrete Stress at Service

	Location	Excel calculation	CONSPAN
Concrete stress in **top** of beam (ksi)			
Service I (Final 1) (P/S + DL + LL)	Mid-span	1.629	1.629
	0.4L	1.542	1.542
	Trans.	0.093	0.1
Service III (Final 1) (P/S + DL + 0.8LL)	Mid-span	1.545	1.545
	0.4L	1.46	1.46
	Trans.	0.087	0.093
Service I (Final 2) (P/S + DL)	Mid-span	1.207	1.207
	0.4L	1.133	1.133
	Trans.	0.064	0.064
Fatigue I (Final 3) (0.5(P/S) + 0.5DL+1.5LL)	Mid-span	0.798	0.789
	0.4L	0.761	0.752
	Trans.	0.05	0.049
Concrete stress in **bottom** of beam (ksi)			
Service I (Final 1) (P/S + DL + LL)	Mid-span	-0.287	-0.286
	0.4L	-0.193	-0.192
	Trans.	1.554	1.54
Service III (Final 1) (P/S + DL + 0.8LL)	Mid-span	-.0117	-0.117
	0.4L	-0.028	-0.029
	Trans.	1.565	1.554
Service I (Final 2) (P/S + DL)	Mid-span	0.562	0.562
	0.4L	0.627	0.627
	Trans.	1.612	1.611
Fatigue I (Final 3) (0.5(P/S) + 0.5DL+1.5LL)	Mid-span	-0.109	-0.092
	0.4L	-0.076	-0.058
	Trans.	0.77	0.772

As shown in Table 4.7, Excel-calculated concrete stress values on the top and bottom of the beam at service showed no differences under load combination Service I (Final 2) compared to CONSPAN. Service I (Final 1) and Service III results had some discrepancy at the transfer point, as influenced by live load shear and moment results. Fatigue-moment stress caused a gradual parting from end to mid-span at both top and bottom of girder, suggesting that CONSPAN calculated fatigue stress differently than the equations in the PCI (2014) manual.

4.2.5 Reinforcement Limits

Since, at the time of this study, the current KDOT (2015) Bridge Design Manual had not been updated to the most recent AASHTO (2012) code, differences in the specifications were reviewed.

In order to determine whether designed tensile reinforcement was sufficient, the factored flexural resistance (M_r) must satisfy at least one of following conditions:

$$\emptyset M_n = M_r \geq 1.2 M_{cr} \ \textbf{or} \ \emptyset M_n = M_r \geq 1.33 M_u$$

Where:

M_n = nominal resistance (k-ft);

M_{cr} = cracking moment (k-ft); and

M_u = factored moment (k-ft).

The following expressions for cracking moment, M_{cr}, and modulus of rupture, f_r, are used:

$$M_{cr} = S_c(f_r + f_{cpe}) - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right) \leq S_c f_r$$

2011 AASHTO

$$f_r = 0.37\sqrt{f'_c}$$

Where:

S_c = composite section modulus for bottom fiber of prestressed beam where tensile stress is caused by an externally applied load, in.^{3};

f_r = allowable cracking tensile stress, ksi;

f_{cpe} = compressive stress in concrete due to effective prestress force only at extreme fiber of the section where tensile stress is caused by external loads, ksi;

M_{dnc} = total unfactored dead-load moment acting on the slab of the noncomposite prestressed beam, k-in.; and

S_{nc} = noncomposite section modulus for bottom fiber of prestressed beam where tensile stress is caused by an externally applied load, in.^{3}

KDOT currently includes this provision in their bridge design specifications. This provision contains an acceptable safety margin of 20% and 33% in cracking moment and required strength, respectively. All sections of a flexural component are required to satisfy it. Thereby additional tensile strength is often needed at sections away from mid-span in order to meet the requirement (Figure 4.4). In order to show difference in the previous two versions of specification, the number of reinforcement and pattern used to plot diagrams in Figure 4.4 hadn't followed previous design assumption. Figure 4.4 presents the moment demand and theoretical cracking moment for a 120-ft-long pretensioned I-beam section with harped strands. At 0.3L and 0.4L, this beam did not have adequate strength to meet the standard. Factored and un-factored required strength and cracking moment are shown by dotted lines in the figure. Required and provided factored flexural resistance is denoted by solid lines.

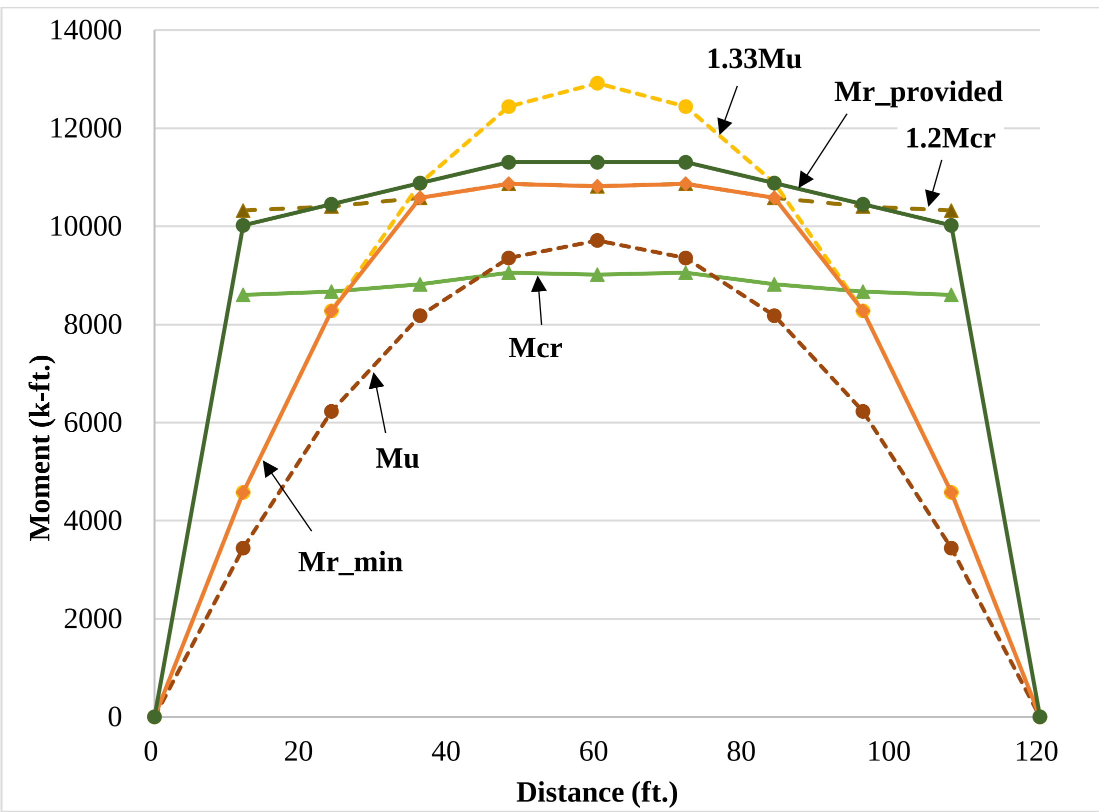


Figure 4.4: AASHTO 2011 Minimum Reinforcement Requirements

As previously noted, AASHTO (2012) significantly revised minimum reinforcement provisions in the 2012 interims. The revised version individually accounts for the safety factor of each component for minimum reinforcement requirements as developed by Freyermuth and Alami in 1997. Application of individual factors not only "achieved appropriate and consistent safety," but also increased the accuracy of cracking-moment estimations by accounting for more variables in design detail (Holombo & Tadros, 2009). Additionally, modulus of rupture was taken as $0.24\sqrt{f_c'}$ instead of $0.37\sqrt{f_c'}$, in order to compute the new version of cracking moment.

The revised specification is written as

M_r at least equal to less of: $M_r \geq M_{cr}$ or $M_r \geq 1.33M_u$

The factored cracking moment is calculated as

$$M_{cr} = \gamma_3\left[(\gamma_1 f_r + \gamma_2 f_{cpe})S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right] \qquad \text{2012 AASHTO}$$

$$f_r = 0.24\sqrt{f_c'}$$

Where:

γ_1 = flexural cracking variability factor

=1.2 for precast segmental structures

=1.6 for all other concrete structures

γ_2 = prestress variability factor

= 1.1 for bonded tendons

= 1.0 for unbonded tendons

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of reinforcement, ${f_y}/{f_u}$

= 0.67 for A615. Grade 60 reinforcement

= 0.75 for A706, Grade 60 reinforcement

= 1.00 for prestressed concrete structures

The flexural cracking stress of the concrete section was revised to $0.24\sqrt{f_c'}$ for normal weight and checking minimum reinforcement, which is recommended by AASHTO (2012). However, the modulus of rupture did not decrease because a safety factor was applied. For the concrete structure, the cracking factor (γ_1) was 1.6 and the modulus of rupture was $0.384\sqrt{f_c'}$, leading to more conservative design compared to use of $0.37\sqrt{f_c'}$. Moreover, γ_3 was an approximate ratio to estimate the ultimate strength, since the brittle response should be measured using ultimate strength instead of nominal strength according to AASHTO (2012) LRFD Bridge Design Specifications.

Factors for prestress in concrete (γ_2) were considered within two prestress conditions: bonded and unbonded. The value for bonded member was 1.1, which accounted for losses in draped and post-tension members due to friction and anchor set. This value was appropriate because friction coefficients were within 0.15 to 0.25 according AASHTO specifications. Unbonded prestress strength was continued even after cracking occurred, indicating that prestress was constant. Therefore, the value for the unbonded tendon was 1.0 in order to eliminate "any unintended increase in prestress" for minimum reinforcement check. This minimum reinforcement provision was recommended by Holombo and Tadros (2009), whose study came to the following conclusion:

> "[It] provides a consistent level of safety for all components in the database of concrete structures. This consistency is largely due to the recognition that the maximum strength, including the effects of strain hardening, should be considered when evaluating minimum reinforcement. Also, each component of the minimum reinforcement evaluation is factored appropriately, resulting in uniform reliability in achieving resistance against brittle failure. Finally, [this] method offers economy, where compression-controlled and transition-region sections are not subject to minimum reinforcement requirements" (Holombo & Tadros, 2009, pp. 63–64).

Figure 4.5 is similar to Figure 4.4; however, Figure 4.5 applied the AASHTO 2012 provision of minimum reinforcement. The same amount and arrangement of reinforcements were placed. The average M_{cr} was significantly reduced compared to the 2011 provision, resulting in less tensile reinforcement in design and increased the control of initial flexural cracking.

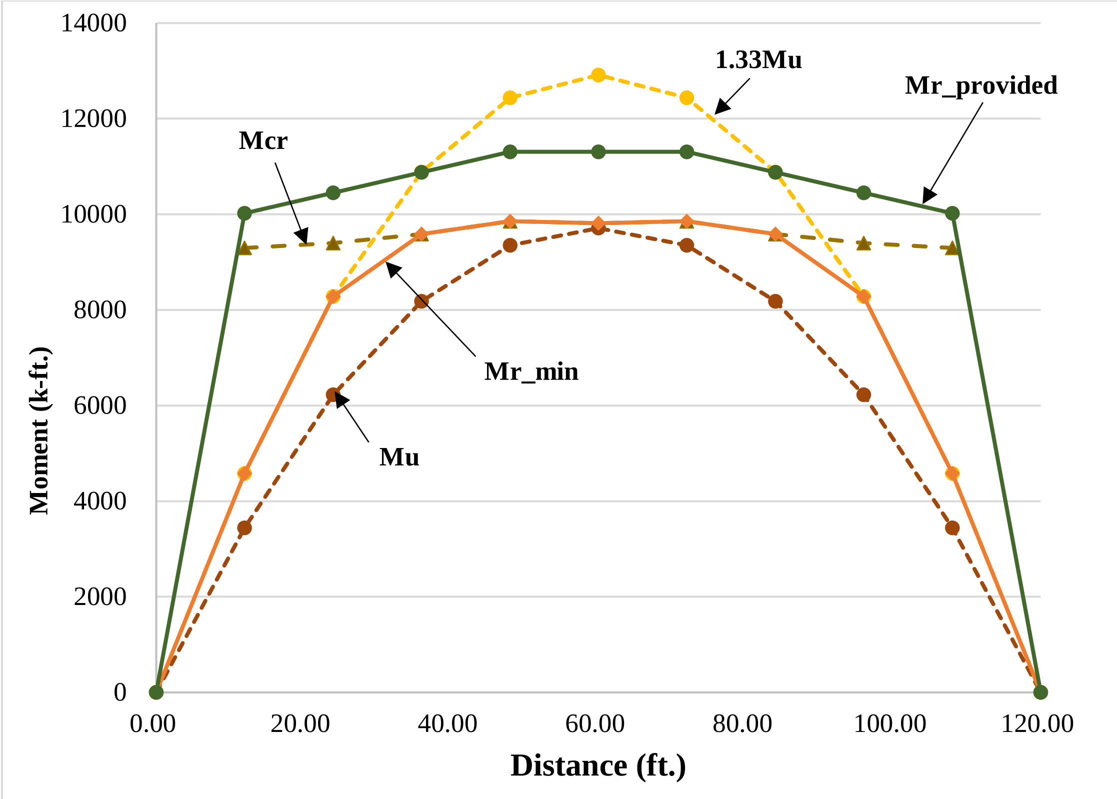


Figure 4.5: AASHTO 2012 Minimum Reinforcement Requirements

The Excel calculation followed the 2012 version of the AASHTO code, and the comparisons followed the previous design assumptions. Table 4.8 shows detailed calculations at mid-span by Excel calculation and CONSPAN, and the diagram is shown in Figure 4.6.

Table 4.8: Girder Minimum Reinforcement Design at Mid-Span

	Excel calculation	CONSPAN
dp (in.)	100	100
A_{ps} (in.^{2})	5.21	5.21
f_s (ksi)	266.22	266.2
C (in.)	5	5
a (in.)	4.25	4.25
M_r_provided (k-ft)	11308.64	11308.6
ε_t	0.057 (T)	0.058 (T)
M_{cr} (k-ft)	9814.85	9815.0
Mu (k-ft)	9710.66	9709.3
1.33Mu	12915.18	12913.37
M_r_min control	M_{cr}	M_{cr}
M_r_provided. ≥ M_r_min	Yes	Yes

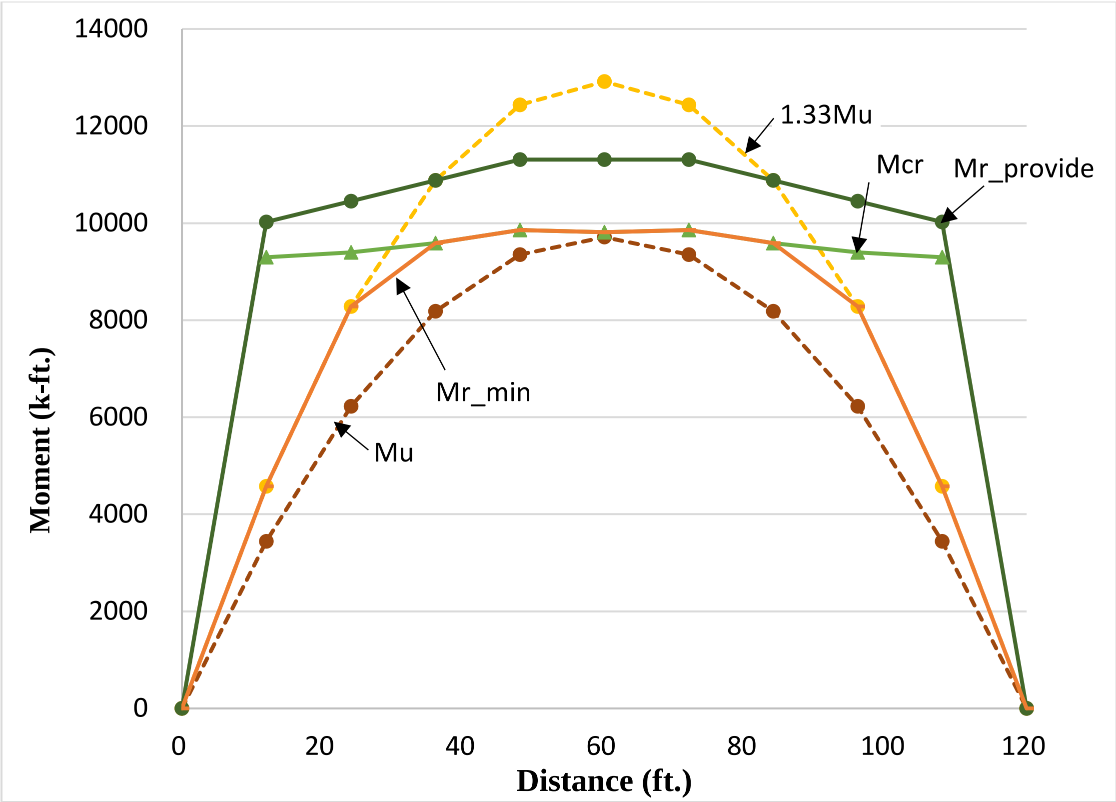


Figure 4.6: Ultimate Moment Requirement Design by Excel Calculation

As shown in Table 4.8, computation values were in an excellent agreement, indicating that CONSPAN adheres to the newest minimum reinforcement provision. Since Kansas has not updated the section of minimum reinforcement in its 2015 bridge design manual, this may lead to a variation in the number of strands required in the final girder design. According to the results presented in this chapter, it is concluded that the CONSPAN had accurate calculations, according to the 2012 minimum reinforcement provision. Therefore, the CONSPAN software was used for all further analyses presented in this report.

Chapter 5: Single Span Design – Required Strands

Chapter 4 verified the accuracy of CONSPAN by comparing its values to the Excel-calculation ones, thereby increasing the understanding of the design procedure. This section used the commercial software, CONSPAN, to determine the minimum number of strands in single-span and two-span girder systems with varied span lengths (120 to 160 ft) and two different girder spacings (8 and 10 ft). In addition, the maximum applicable span length for each girder section type was investigated. According to AASHTO (2012) Table 2.5.2.6.3-1, the I-beam and slab may be checked with minimum depth requirements, but "values may be adjusted to account for changes in relative stiffness of positive and negative moment sections" (AASHTO, 2012). These minimum depth requirements are 0.045 × span length for single span beams, and 0.040 × span length for continuous beams. Table 5.1 lists the AASHTO minimum superstructure depths corresponding to the spans investigated in this study.

Table 5.1: Minimum Depths (Including Deck)

Span length (ft)	Precast I-beam depths	
	Single, ft [in.]	Continuous, ft [in.]
120	5.4 [64.8]	4.8 [57.6]
130	5.85 [70.2]	5.2 [62.4]
140	6.3 [75.6]	5.6 [67.2]
150	6.75 [81]	6.0 [72]
160	7.2 [86.4]	6.4 [76.8]
170	7.65 [91.8]	6.8 [81.6]
180	8.1 [97.2]	7.2 [86.4]

Based on AASHTO minimum depth requirement, K-2, K-3, K-4, NU-750, NU-900, NU-1100, and NU-1350 should not be used for the minimum investigated span length of 120 ft. However, both the NU-1350 girder and K-4 girder types were also investigated to determine their applicability for a span length in the range of 70 to 120 ft. The strand patterns that were investigated in this study were straight, harped, and debonded designs, and the minimum number of required strands in each design type were determined based on the results generated by CONSPAN. Debonded and harped strands were investigated according to KDOT (2015) Bridge Design Manual Section 5.2.1.

The following section discusses strand patterns in the NU girder systems (NU-1350 to NU-2400) and K-6 girders in terms of span length, and also shows comparisons in three strand patterns (straight, harped, and debonded). Additionally, K-4 girder has similar height as NU-1350 girder, therefore, the evaluation on both girders was examined. There were two types of NU girders evaluated, which were the original size and "plus one inch" of concrete added to the top flange (called "plus-1" in the following analyses). The section properties for the different NU girder types are shown in the Table 5.2.

Table 5.2: Section Properties

NU girder	Height, in.	Area, in.^{2}	Y_b, in.	I, in.^{4}	Weight, lb/ft
750	29.5	614.0	13.6	69,403	639.6
900	35.4	648.1	16.1	110,262	675.1
1100	43.3	694.6	19.6	182,279	723.5
1350	53.1	752.7	24.0	302,334	784.1
1600	63.0	810.8	28.4	458,482	844.6
1800	70.9	857.3	32.0	611,328	893.0
2000	78.7	903.8	35.7	790,592	941.5
2400	94.5	998.0	43.0	1,235,547	1039.6
750+1	30.5	662.2	14.8	81,423	689.8
900+1	36.4	696.3	17.5	127,850	725.3
1100+1	44.3	742.8	21.2	208,675	773.8
1350+1	54.1	800.9	25.8	342,023	834.3
1600+1	64.0	859.0	30.4	514,553	894.8
1800+1	71.9	905.5	34.1	682,168	943.2
2000+1	79.7	952.0	37.9	877,181	991.7
2400+1	95.5	1046.2	45.5	1,389,880	1089.8

Design results are shown in both tables and figures. Although the number of tendons can potentially vary due to the placement pattern by rows, the investigation provided a generalization of minimum tensile requirements needed. The design assumptions were discussed in Section 4.1, and the ultimate-moment capacity was checked by both Excel calculations and CONSPAN.

5.1 8-ft Girder Spacing

Span length from 120 to 160 ft has been investigated with different girder sections. The girder spacing was 8 ft in this section and the overall width was 47 ft, as shown in Figure 5.1 which was obtained from CONSPAN.

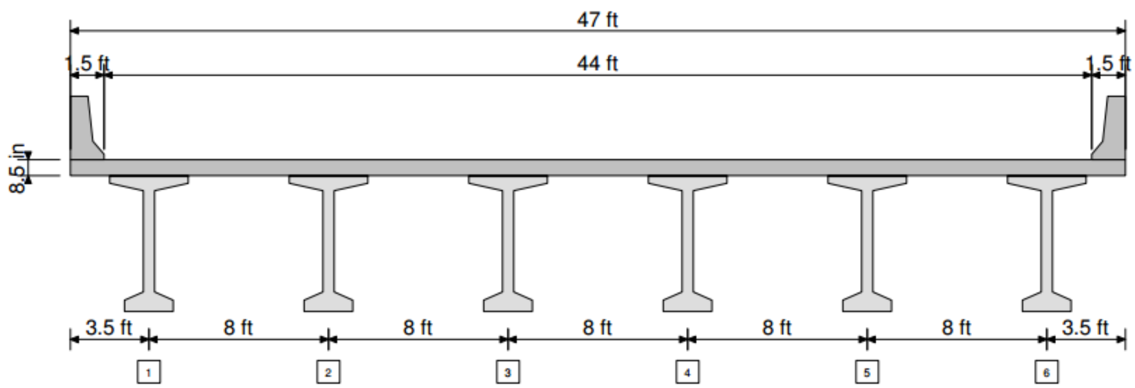


Figure 5.1: Analysis of Bridge Geometry with 8-ft Girder Spacing

The deeper girders proved to be much more flexible in terms of their applicable span length, since shallower members were easily overstressed when attempted for longer span designs, especially for strand patterns that were not harped or debonded. Generally, girder sections with a harped strand pattern required fewer total number of strands because harping allowed placing more strands on the bottom row which resulted in a lower center of gravity of tendons.

Similarly, a girder with a straight strand pattern required more strands since tendon placement must be dispersed between the rows in order to prevent exceeding allowable stresses. Debonded designs were used when the straight pattern reached the maximum stress limitations, which were limited by excessive compressive stresses at prestress transfer. Debonding is typically accomplished by sheathing the strands with a plastic cover and can reduce end stress and also minimize cracking at end region (Burgueño & Sun, 2011).

5.1.1 NU-2400 Girder at 8-ft Spacing

Straight and debonded designs are presented together, and an estimation of minimum reinforcement and required area of shear reinforcements are shown in Table 5.3. In this table, and in similar tables in the remainder of the report, the red span lengths indicate the maximum span which met all design criteria. A chart format is shown in Appendix B Figure B.1, in which the solid line represents the fully bonded straight strand pattern and the dashed line represents the debonded design.

Table 5.3: Straight and Debonded Design of Minimum Reinforcements for NU-2400 Girder System

Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**142**
	Number of 0.6-in. strands	18	20	24	28*	34*	34*
	As top @ trans., in.^{2}	2.317	2.334	2.367	2.474	2.443	2.436
	Al req. @ bearing, in.^{2}	0.79	0.73	0.47	0.88	0.65	0.67
	A_v/S. @ trans., in.^{2}	0.105	0.12	0.147	0.172	0.199	0.204
	A_{vh} min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482
Plus-1 - Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**143**
	Number of 0.6-in. strands	18	20	24	28*	32*	34*
	As top @ trans., in.^{2}	2.664	2.741	2.63	2.843	2.872	2.851
	Al req. @ bearing, in.^{2}	0.81	0.76	0.5	0.91	0.87	0.72
	A_v/S. @ trans., in.^{2}	0.105	0.117	0.146	0.166	0.193	0.204
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482

*Debonded strands

A_s_top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds the allowable value, in.^{2};

Al-req. : required area of longitudinal reinforcement, in.^{2};

A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};

A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

The maximum span length was slightly above 140 ft for both the NU-2400 and NU-2400 plus-1 girder system, and thirty-four 0.6”-diameter 270 ksi strands were required in this debonded design. Without debonding strands at the ends, 120 ft was the maximum applicable span length, with twenty-four 0.6”-diameter strands required. There were no differences in the number of required strands in the range of the designed span lengths, except for the span length of 140 ft. In this case, the NU-2400 plus-1 girder could have two less strands than the regular NU-2400 girder since the centroid of strand reinforcements was lower by 5.75 in. compared to the regular NU-2400 girder (Y_{cg} = 6.82 in.). The detail reinforcement pattern is shown in Figure 5.2.

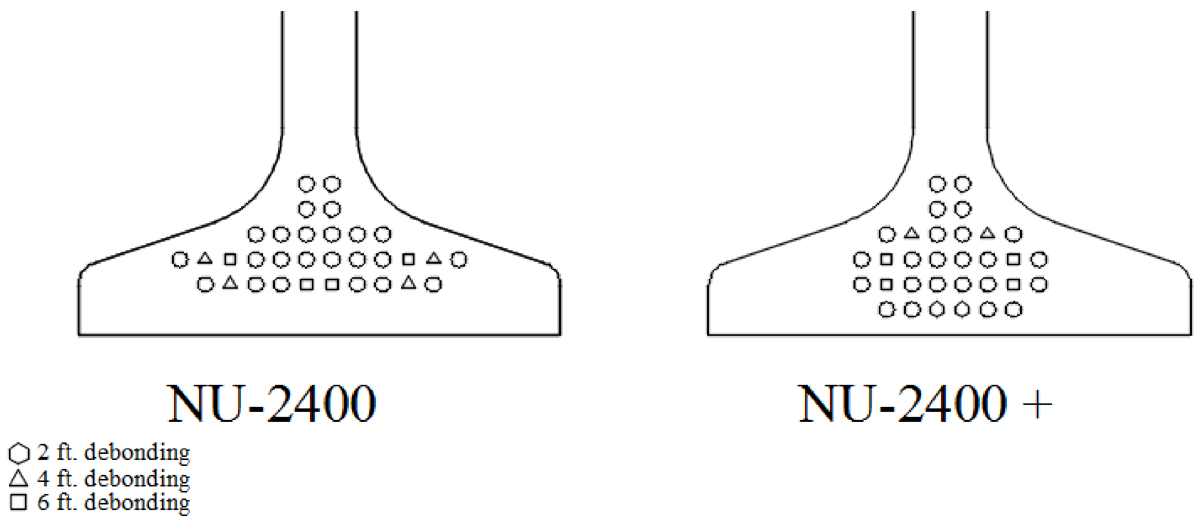


Figure 5.2: Straight Strand Pattern of NU-2400 Girders with 8-ft Spacing @ 140-ft Span Length

Most of the straight and debonded strand patterns required additional compressive/tensile reinforcement at the top flange of the prestressed girders because the net top stress at detensioning exceeded the allowable stress value. Since the strand placement is limited to positions within rows’ restriction (template) at the bottom of the girder, the straight strand design might not be applicable when the span length exceeds 120 ft. Debonded design results could vary due to strand placement pattern and percent of debonding. Similarly, the required shear reinforcement numbers would also vary by the strand number and pattern.

Harped strand patterns are much more efficient in utilizing the NU girder system and can be utilized for span lengths up to 180 ft, which is 40 ft longer than with debonded strands. Meanwhile, increment in reinforcement number is observed. For NU-2400 and NU-2400 plus-1, the detail estimation in harped strands pattern is presented in Table 5.4 and Figure B.2. Plus-1 girders can often use fewer strands for the same span than regular girders, but this is not always the case due to variations in strand pattern, minimum reinforcement specification, and number of released strands. Compression steel is needed for the 150- and 160-ft span lengths but could possibly be eliminated by changing the strand pattern or by increasing the concrete compressive strength.

Table 5.4: Harped Design of Minimum Reinforcements for NU-2400 Girder System

Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**	**160**	**170**	**180**	
	Number of 0.6-in. strands	18	20	22	28	32	38	44	50	58	
	As top @ Trans., in.^{2}	0	0	0	0	0	1.821	0.758	0	0	
	Al reqd @ bearing, in.^{2}	1	1.15	1.1	0.66	0.63	0.03	0	0	0	
	A_v/S @ trans., in.^{2}	0.105	0.105	0.109	0.128	0.144	0.165	0.178	0.19	0.198	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482	
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**	**160**	**170**	**180**	**182**
	Number of 0.6-in. strands	18	18	22	26	32	36	42	50	58	58
	As top @ trans., in.^{2}	0	0	0	0	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	1.03	1.18	1.13	0.88	0.66	0.43	0.04	0	0	0
	A_v/S. @ trans., in.^{2}	0.105	0.105	0.11	0.133	0.145	0.166	0.184	0.193	0.201	0.206
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482

A_s_top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds the allowable value, in.^{2};
Al-req. : required area of longitudinal reinforcement, in.^{2};
A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};
A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

Although the same number of strands were often required at the same span length for NU-2400 girder and NU-2400 plus-1 girder, the plus-1 girder can achieve a 2 ft longer span (182 ft compared to 180 ft) because of the increased girder height. Figure 5.3 shows the strand pattern at the end of each girder type at the maximum span length. The difference is that the plus-1 girder was able to have two more 0.6"-diameter strands in the second-lowest row (lower center of gravity) which provided a higher flexure resistance.

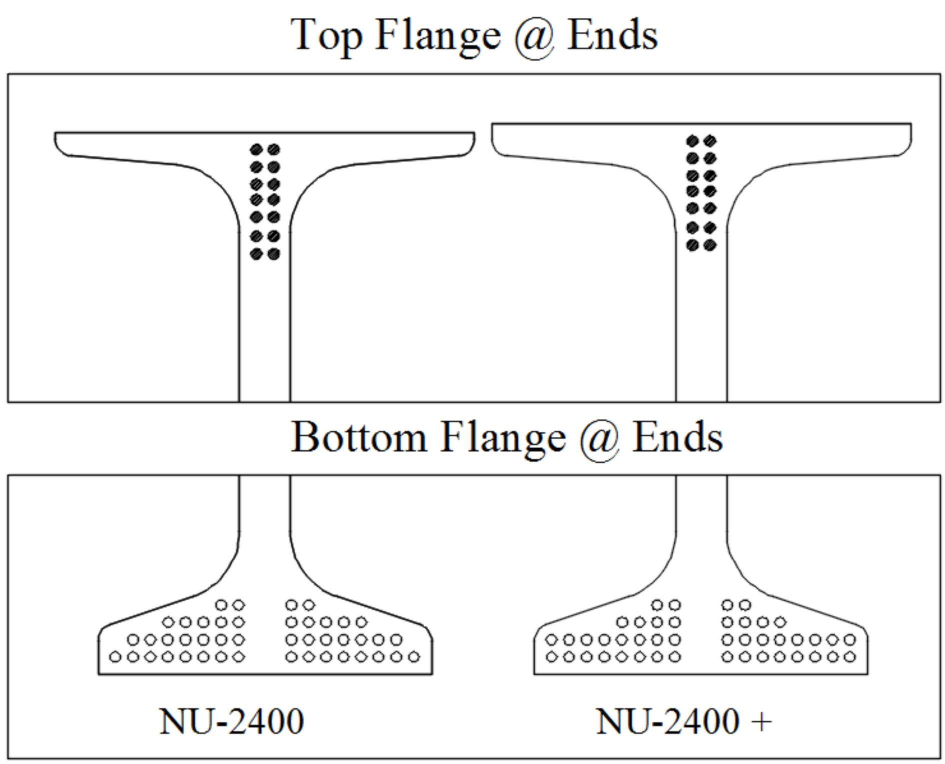


Figure 5.3: Harped Strand Pattern of NU-2400 Girders with 8-ft Spacing @ Maximum Span Length

5.1.2 NU-2000 Girder at 8-ft Spacing

The results from analyses on straight strand patterns with debonding are presented in Table 5.5 and Figure B.3, along with minimum number of 0.6"-diameter strands for each span length and minimum reinforcement requirements, including the required area of shear reinforcement. Strands were placed within the bottom seven rows in order to limit the release stresses at the girder ends. Top reinforcement in the flange was still needed for the straight and debonded designs.

For strands without harping, the regular girder span length was limited to 138 ft, otherwise stresses would exceed the AASHTO specification. However, the NU-2000 plus-1 girder was able to span 142 ft. Figure 5.4 shows the strand pattern in NU-2000 and NU-2000 plus-1 girder at their maximum span lengths of 138 ft and 142 ft, respectively, when strands were straight and debonding was utilized. Both girders used fifty-eight 0.6"-diameter strands, but the strand pattern was different in order to balance the release and final stress limitations and maintain all debonding limitations. Debonding in individual rows is governed either by a maximum of four strands or less than 40% of debonded strands, as well as the outside strands should be fully bonded (AASHTO, 2012). Additional de-tension requirements were introduced following KDOT (2015) Bridge Design Manual Section 5.2.1.

Table 5.5: Straight and Debond Design of Minimum Reinforcements for NU-2000 Girder System

Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**138**	
	Number of 0.6-in. strands	20	24	30	34*	40*	
	As top @ trans., in.^{2}	2.295	2.148	2.06	2.459	2.505	
	Al req. @ bearing, in.^{2}	0.49	0.24	0	0.42	0.19	
	A_v/S. @ trans., in.^{2}	0.151	0.184	0.212	0.239	0.256	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**142**
	Number of 0.6-in. strands	18	24	30	34*	40*	42*
	As top @ trans., in.^{2}	2.649	2.548	2.531	2.744	2.8	2.759
	Al req. @ bearing, in.^{2}	0.73	0.26	0	0.45	0.24	0.08
	A_v/S. @ trans., in.^{2}	0.146	0.18	0.208	0.237	0.261	0.271
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482

* Debonded strands

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};

Al-req. : required area of longitudinal reinforcement, in.^{2};

A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};

A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

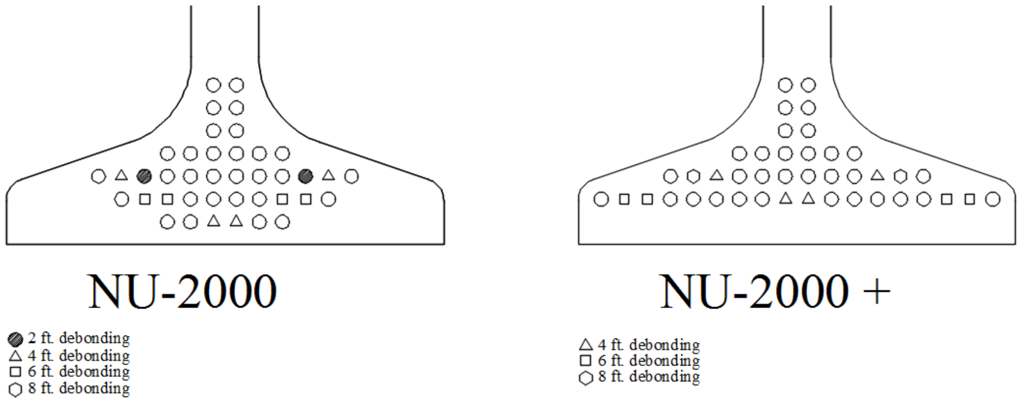


Figure 5.4: Straight Strand Pattern of NU-2000 Girder with 8-ft Spacing @ Maximum Span Length

Girders with harped strands required fewer number of 0.6”-diameter strands than those without straight strands (Table 5.6). Also, these girders met the allowable tensile stress requirements at de-tensioning without needing additional tensile reinforcement in the top flange. An average of four strands were harped in the analysis for span lengths from 120 to 140 ft. However, the number of harped strands increased to 14 when the span length increased to 160 ft. The strand pattern for the NU-2000 girder and NU-2000 plus-1 at maximum spans and 8-ft girder spacing are shown in Figure 5.5. Detailed harped strand analysis results are presented in Table 5.6, and comparisons of minimum number of strands in various patterns are presented in Figure B.4.

Table 5.6: Harped Design of Minimum Reinforcements for NU-2000 Girder System

Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**	**158**
	Number of 0.6-in. strands	18	22	28	32	40	46	54
	As top @ trans., in.^{2}	0	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.91	0.86	0.43	0.4	0	0	0
	A_v/S. @ trans., in.^{2}	0.129	0.147	0.168	0.187	0.203	0.221	0.228
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**	**160**
	Number of 0.6-in. strands	18	22	28	32	38	46	54
	As top @ trans., in.^{2}	0	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.94	0.89	0.45	0.22	0.02	0	0
	A_v/S. @ trans., in.^{2}	129	0.145	0.169	0.194	0.209	0.224	0.238
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};
Al-req. : required area of longitudinal reinforcement, in.^{2};
A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};
A_{vh}_min: required minimum amount of horizontal shear reinforcement, in.^{2}

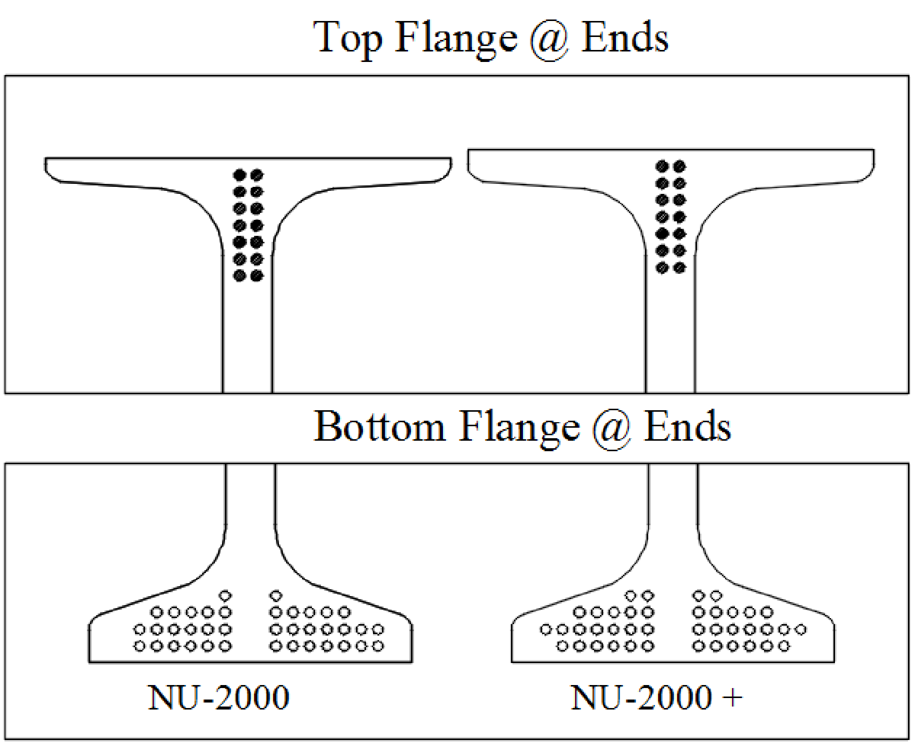


Figure 5.5: Harped Strand Pattern with 8-ft Spacing @ Maximum Span Length

5.1.3 NU-1800 Girder and K-6 Girder at 8-ft Spacing

Direct comparisons between the NU-1800 girder and the K-6 girder were conducted, since the two girders had similar girder depths: 70.92 in. and 72 in., respectively. Analysis results are shown in Table 5.7 and Figure B.5 for straight and debonded design, and the harped design results are shown in Table 5.8 and Figure B.6. In Figure B.6, the results are presented in chart format; the solid line represents straight pattern, and the dashed line displays the debonded design.

NU-1800 plus-1 girder often required two less strands than the regular NU girder at a given span, and both NU-1800 girders had to debond or harp strands for span length above 120 ft. If straight pattern without debonding was used, the available span length would be less than 120 ft. However, span length within 120 to 135 ft was accessible when debonded was attempted.

The smaller cross-section and corresponding moment of inertia of the K-6 girder resulted in higher compression stresses in the bottom flange at detensioning (than the NU-1800 girder) for the same number of strands. This limited the number of 0.6" straight strands that could be accommodated in the K-6 girder as the span length increased. The tallest girder in the K-girder system, K-6, achieved the maximum span length which was 23 ft shorter than NU-1800 girder system (112 ft versus 135 ft) when a straight strand pattern was used.

The K-6 girder had a maximum span of 112 ft when using straight strands, and a maximum span length of 132 ft when harping of the strands was employed. The NU-1800 girder, however, could be used for span lengths up to 135 ft with straight strands and up to 148 ft when harping is employed. The NU-1800 plus-1 girder with harped strands could achieve spans up to 150 ft with harping. In all cases, girders utilizing straight strand patterns at maximum span length required additional tensile reinforcement in the top flanges because of excessive tensile stresses.

It is important to note that the NU-1800 girder required a similar number of strands as the K-6 girder at spans lengths in the 100- to 132-ft span range. This can be seen by examining both Table 5.7 (for straight strands) and Table 5.8 (for harped strands). This means that there would likely be a cost savings for the K-6 girder option in this span range, due to less concrete material and also possibly also due to reduced trucking costs and erection costs. This will be discussed further in Chapter 7.

Table 5.7: Straight and Debond Design of Minimum Reinforcements for NU-1800 and K-6 Girder System

NU-1800						
Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**135**
	Number of strands	22	28	34	40*	42*
	As top @ trans., in.^{2}	2.284	2.027	2.242	2.299	2.481
	Al req. @ bearing, in.^{2}	0.25	0	0	0.03	0
	A_v/S. @ trans., in.^{2}	0.188	0.223	0.246	0.279	0.294
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**135**
	Number of strands	22	26	32	38*	42*
	As top @ trans., in.^{2}	2.569	2.384	2.555	2.663	2.795
	Al req. @ bearing, in.^{2}	0.28	0.02	0	0	0
	A_v/S. @ trans., in.^{2}	0.185	0.221	0.247	0.274	0.293
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482
K-6						
Straight + Debonded	**Span Length, ft**	**100**	**110**	**112**		
	Number of strands	22*	26*	28*		
	As top @ trans., in.^{2}	2.162	2.413	2.512		
	Al req. @ bearing, in.^{2}	0.67	0.64	0.48		
	A_v/S. @ trans., in.^{2}	0.188	0.216	0.216		

* Debonded strands

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};

Al-req. : required area of longitudinal reinforcement, in.^{2};

A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};

A_{vh} min: required minimum amount of horizontal shear reinforcement, in.^{2}

Table 5.8: Harped Design of Minimum Reinforcements for NU-1800 and K-6 Girder System

NU-1800							
Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**148**
	Number of 0.6-in. strands	20	26	30	38	44	52
	As top @ trans., in.^{2}	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.87	0.43	0.2	0	0	0
	A_v/S. @ trans., in.^{2}	0.154	0.181	0.209	0.222	0.245	0.251
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**
	Number of strands	20	26	30	36	44	52
	As top @ trans., in.^{2}	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.69	0.46	0.22	0.02	0	0
	A_v/S. @ trans., in.^{2}	0.165	0.181	0.209	0.228	0.247	0.264
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482
K-6							
Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**132**	
	Number of 0.6-in. strands	20	26	30	38	38	
	As top @ trans., in.^{2}	0	0	0	0	0	
	Al req. @ bearing, in.^{2}	0.87	0.46	0.43	0.26	0.28	
	A_v/S. @ trans., in.^{2}	0.136	0.159	0.181	0.195	0.202	

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};
Al-req. : required area of longitudinal reinforcement, in.^{2};
A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};
A_{vh} min: required minimum amount of horizontal shear reinforcement, in.^{2}

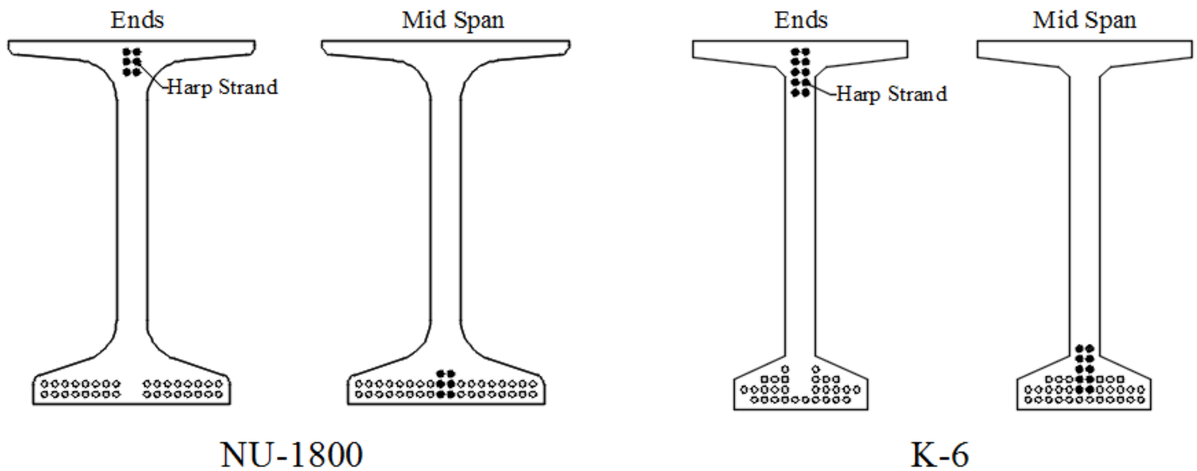


Figure 5.6: Comparisons of Girder Designs for Harped Strand Patterns with 130-ft Span Lengths

5.1.4 NU-1600 Girder at 8-ft Spacing

This section discusses the results from analyses with NU-1600 girders. For the NU-1600 girder with a straight strand pattern, a maximum span length of 125 ft was achieved using forty 0.6”-diameter strands when debonded was assumed. There were no significant variances between NU-1600 girder and NU-1350 plus-1 girder, as the required number of strands was identical except for span length of 110 ft. In this case, the NU-1350 plus-1 girder required two less strands (30 versus 32). Table 5.9 shows detailed analysis results for straight and debonded strand pattern. Figure B.7 presents in chart format with sold line as straight design and dashed line as debonded design.

Table 5.9: Straight and Debond Design of Minimum Reinforcements for NU-1600 Girder System

Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**125**
	Number of 0.6-in. strands	24	32	36*	40*
	As top @ trans., in^2	2.246	2.044	2.199	2.355
	Al req. @ bearing, in.^{2}	0.02	0	0.02	0
	Av/S. @ trans., in.^{2}	0.234	0.267	0.3	0.311
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**125**
	Number of 0.6-in. strands	24	30	36*	40*
	As top @ trans., in.^{2}	2.474	2.417	2.421	2.705
	Al req. @ bearing, in.^{2}	0.04	0	0.04	0
	Av/S. @ trans., in.^{2}	0.232	0.266	0.303	0.315
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482

* Debonded strands

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};

Al-req. : required area of longitudinal reinforcement, in.^{2};

A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};

A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

When using a harped strand pattern, the maximum span length was 135 ft with 48 strands for the NU-1600 girder, and 138 ft with 50 strands for the NU-1600 plus-1 girder. For the harped strand option, no additional tension steel was required in the top flange. Table 5.10 shows detailed analysis results for the harped pattern, and Figure B.8 presents results in chart format.

Table 5.10: Harped Design of Minimum Reinforcements for NU-1600 Girder System

Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**135**	
	Number of 0.6-in. strands	24	30	36	42	48	
	As top @ trans., in.^{2}	0	0	0	0	0	
	Al req. @ bearing, in.^{2}	0.44	0.03	0	0	0	
	A_v/S. @ trans., in.^{2}	0.198	0.226	0.248	0.273	0.283	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**135**	**138**
	Number of 0.6-in. strands	24	28	34	42	46	50
	As top @ trans., in.^{2}	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.46	0.22	0.01	0	0	0
	A_v/S. @ trans., in.^{2}	0.196	0.229	0.251	0.273	0.281	0.281
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};
Al-req. : required area of longitudinal reinforcement, in.^{2};
A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};
A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

5.1.5 NU-1350 and K-4 at 8-ft Spacing

In this section, the NU-1350 girder was compared with K-4 girder because both girders have similar height (54 in. for K-4 versus 53.1 in. for NU-1350). In order to observe the differences between designs using NU-1350 girders and K-4 girders, analyses were conducted for span lengths of 70 ft to maximum length. Analysis results are shown in Table 5.11 and Figure B.9 for straight and debonded pattern.

Table 5.11: Straight and Debond Design of Minimum Reinforcements between NU-1350 and K-4 Girder System

NU-1350							
Straight + Debonded	**Span Length, ft**	**70**	**80**	**90**	**100**	**110**	
	Number of 0.6-in. strands	14	18	24	30	38*	
	As top @ trans., in.^{2}	1.904	2.19	2.222	2.086	1.801	
	Al req. @ bearing, in.^{2}	0.53	0.28	0	0	0	
	Av/S. @ trans., in.^{2}	0.185	0.226	0.268	0.308	0.342	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	
Plus-1 Straight + Debonded	**Span Length, ft**	**70**	**80**	**90**	**100**	**110**	**112**
	Number of strands	14	18	24	30	36*	38*
	As top @ trans., in.^{2}	1.997	2.406	2.366	2.247	2.286	2.157
	Al req. @ bearing, in.^{2}	0.55	0.3	0	0	0	0
	A_v/S. @ trans., in.^{2}	0.182	0.211	0.266	0.306	0.344	0.345
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482
K-4							
Straight + Debonded	**Span Length, ft**	**70**	**80**	**90**	**100**		
	Number of 0.6-in. strands	14	20	24*	32*		
	As top @ trans., in.^{2}	1.587	1.445	1.641	1.414		
	Al req. @ bearing, in.^{2}	0.52	0.07	0.27	0		
	A_v/S. @ trans., in.^{2}	0.162	0.207	0.246	0.285		

* Debonded strands

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};

Al-req. : required area of longitudinal reinforcement, in.^{2};

A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};

A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

Overall, both systems required approximately the same number of strands within the investigated span lengths. The NU-1350 and NU-1350 plus-1 girders required the same number of strands at all span lengths except 110 ft, but K-4 girders required two more strands at spans of 80 ft and 100 ft compared to NU-1300 girders. At a span of 90 ft, the 24 strands were required for both girder systems, although the NU-1350 girders did not require any debonding of strands while the K-4 girders did. With straight strand pattern, the NU-1350 girders had a maximum span of 110 ft, while the K-4 girders had a maximum span of 100 ft.

Figure 5.7 shows the detailed design pattern for NU-1350 girders and K-4 girder when the design span length is 100 ft. The NU-1350 girders and NU-1350 plus-1 girders required 30 strands without debonding, while the K-4 girders required 32 strands with eight strands debonded. For the K-4 girders, the maximum debonding length was up to 20 ft on the bottom row. These girders reached the maximum allowable debonding specification: 25% of total number of strands.

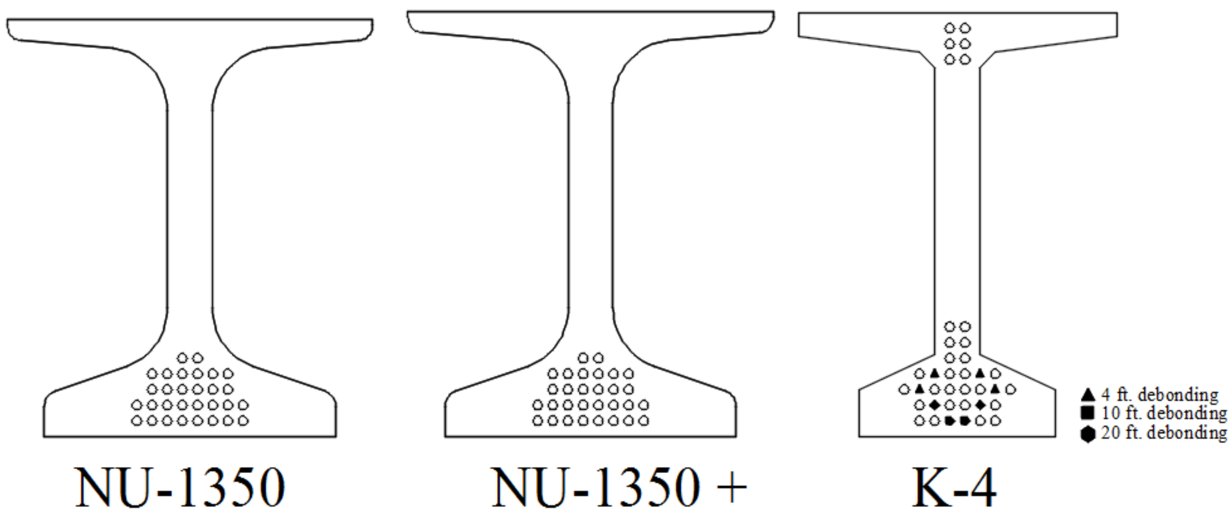


Figure 5.7: Comparison of NU Girder and K-Girder System with 100-ft Span Length

The harped analysis results are shown in Table 5.12 and Figure B.10. When the span length was below 90 ft, the number of strands needed was typically based on providing adequate ultimate flexural moment. Thus, in this case, the same number of strands were often required despite the type of girder and design patterns (straight or harp). However, with longer spans, the design is often limited by allowable stresses at the ends. In this case, harping of strands allows the maximum span to be increased. In addition, harped design often required two less strands (at longer spans) than the straight strand design at a similar span.

The maximum span length that could be used (with harped strands) for the NU-1350 girder was 120 ft, while the NU-1350 plus-1 girder was 122 ft. Both of these designs required forty-six 0.6”-diameter strands. However, the K-4 girder with harped strands had a maximum span length of only 104 ft, and the design utilized thirty-two 0.6”-diameter strands. Note, for the NU-1350 girder at maximum span in Table 5.12, the height did not satisfy AASHTO minimum depth requirement.

Table 5.12: Harped Design of Minimum Reinforcements Between NU-1350 and K-6 Girder System

NU-1350								
Harped	**Span Length, ft**	**70**	**80**	**90**	**100**	**110**	**120**	
	Number of 0.6-in. strands	14	18	22	28	36	46	
	As top @ trans., in.^{2}	0	0	0.898	0	0	0	
	Al req. @ bearing, in.^{2}	0.74	0.49	0.24	0.02	0	0	
	Av/S. @ trans., in.^{2}	0.169	0.207	0.246	0.274	0.298	0.329	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	
Plus-1 Harped	**Span Length, ft**	**70**	**80**	**90**	**100**	**110**	**120**	**122**
	Number of 0.6-in. strands	14	18	22	28	36	44	46
	As top @ trans., in.^{2}	0	0	0.904	0	0.512	0	0
	Al req. @ bearing, in.^{2}	0.76	0.5	0.26	0.04	0	0	0
	A_v/S. @ trans., in.^{2}	0.155	0.204	0.243	0.269	0.298	0.322	0.319
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482
K-4								
Harped	**Span Length, ft**	**70**	**80**	**90**	**100**	**104**		
	Number of strands	14	18	24	30	32		
	As top @ trans., in.^{2}	0	0	0	0	0		
	Al req. @ bearing, in.^{2}	0.74	0.7	0.29	0.09	0.14		
	A_v/S. @ trans., in.^{2}	0.127	0.152	0.186	0.212	0.222		

A_s top : required area of steel at top of precast to resist total tension force in the concrete when net top stress exceeds allowable value, in.^{2};
Al-req. : required area of longitudinal reinforcement, in.^{2};
A_v / S : area of vertical shear reinforcement within distance S=12 in., in.^{2};
A_{vh}_min : required minimum amount of horizontal shear reinforcement, in.^{2}

The primary difference in the harped strand and debonded strand designs was that the harped strand design allowed more strands to be placed in the bottom row, which resulted in a greater moment resistance with fewer number of strands. The comparison is illustrated in

Figure 5.7 and Figure 5.8 which shows the detailed strand pattern with 100 ft span for straight and harped design patterns, respectively.

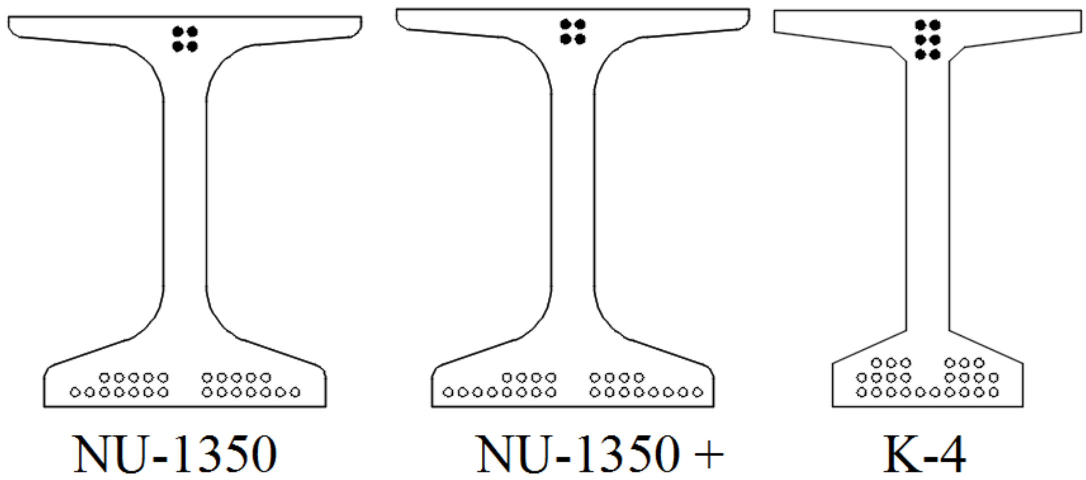


Figure 5.8: Comparison of Harped NU Girder and K-Girder System with 100-ft Span Length

5.1.6 Summary of 8-ft Girder Spacing

For the 8-ft girder spacing study, analyses of girder heights below 53.1 in. (NU-1350) were limited to 70 ft and longer in order to investigate the maximum span lengths possible with each girder section. In the span ranges investigated and with the assumed concrete strengths, the NU-plus-1 girder system had very similar requirements (in terms of minimum number of strands) as the standard NU girder system. However, in some specific cases, girders with plus-1 inch of top flange thickness required two less strands than the standard girders at the same span. There was no specific pattern observed where the plus-1 girder system would achieve a more economical design.

Combined analysis results are summarized in Figure 5.9 and Figure 5.10. When a larger size of girder was applied on shorter span, a certain minimum number of strands were required to reach the minimum reinforcement specification. In Figure 5.9 and Figure 5.10, there was no difference in quantity of strands for the NU-1600 and NU-1800 girders unless the span length was above 80 ft and 90 ft, respectively. In addition, NU-2000 and NU-2400 girders required the same minimum number of strands for spans up to 100 ft long.

The general trend was noted that, within the applicable span ranges of K-girders, the K-girders and NU girders of similar height required approximately the same number of strands at a given span length. Therefore, although the NU girders have an increased span capability, there is no significant structural benefit to using NU girders within the applicable span range of K-girders when the same girder spacing is used. In fact, if the same girder spacing is used, then the NU girder system would likely cost more due to higher material costs, and possibly increase shipping and erection costs.

Additionally, as span lengths increase, the designs are often governed by stresses at detensioning and the need for harping becomes increasingly important. For shallower girders at shorter spans, the effect of harping serves to extend the span length in the 10- to 15-ft range. However, for the tallest girder (NU-2400) debonding was not an efficient means to satisfy allowable stresses at detensioning. Thus, the maximum span length was relatively short for the straight strand with debonding design compared to the harped strand design (refer to Figure 5.9 and Figure 5.10).

Note that these results correspond to an assumed maximum girder concrete release strength of 6.4 ksi, a 28-day girder strength of 8.0 ksi, and a deck concrete strength of 4.0 ksi. According to NDOR, in the example of a simple-span bridge using the NU-900 girder, the girder was capable of a maximum span length of approximately 118 ft with 8-ft girder spacing, but the concrete strength was assumed at 12.0 ksi at service and 9.0 ksi at detensioning. Concrete deck strength was also assumed at 5 ksi and strength limited was $0.196\sqrt{f'c}$ (Hanna, Morcous, & Tadros, 2010). This average concrete strength was significantly higher than the design assumptions in this research program.

Moreover, research results from NDOR indicated a significant increase in span length when concrete strength increased from 8 ksi to 12 ksi. The NDOR research concluded that there was an approximate 4% increased span length applicable to the NU-2000 girder, and a 24% increase in the NU-900 girder, leading to the conclusion that the main influence of the span-length option was “due to the strength at release limit state” (Hanna, Morcous, & Tadros, 2010). Although NU girders have a wide flange and various height, spacing or concrete strength may need to be altered in order to efficiently utilize the NU girder system, particularly at shallow depths.

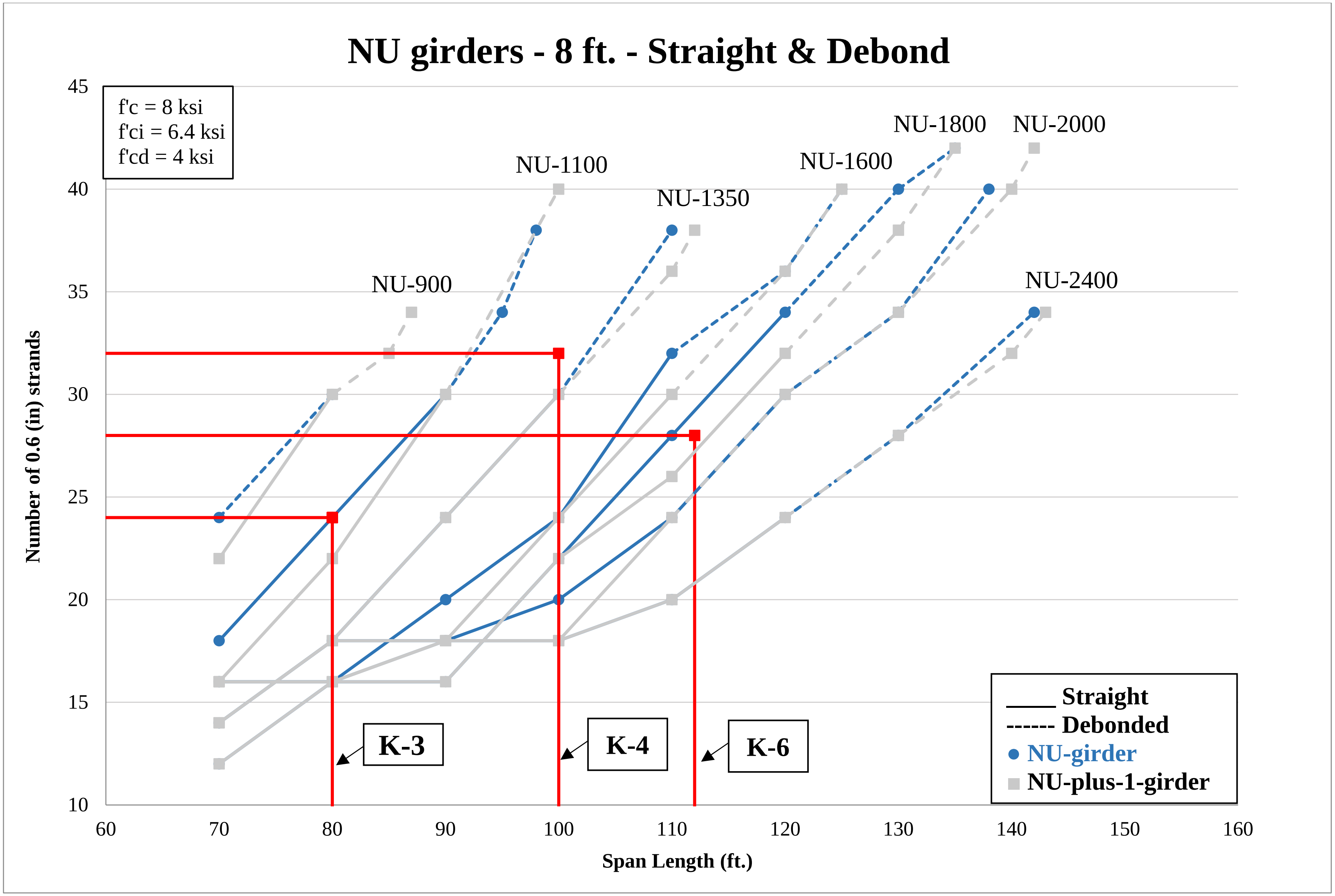


Figure 5.9: 8-ft Spacing Summary of NU Girders Straight and Debonded Strand Minimum Reinforcement Estimation

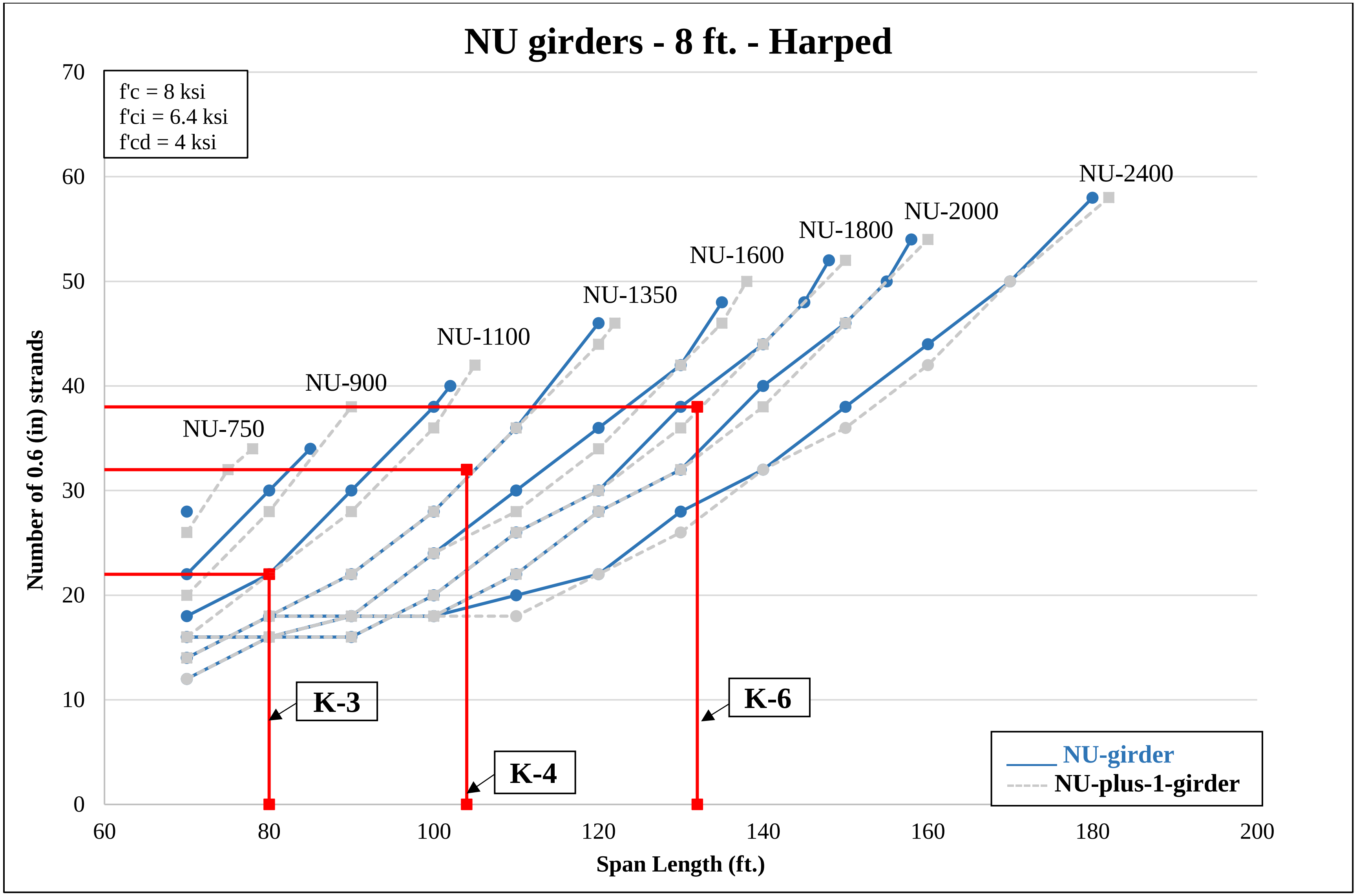


Figure 5.10: 8-ft Spacing Summary of NU Girders Harped Strand Minimum Reinforcement Estimation

5.2 10-ft Girder Spacing

The girder systems were investigated with same criteria as Section 5.1. The difference was that girder spacing was 10 ft and the overall width was 57 ft to keep the same number of girders and overhang (3.5 ft) as 8 ft spacing analysis, as shown in Figure 5.11 which was obtained from LEAP CONSPAN.

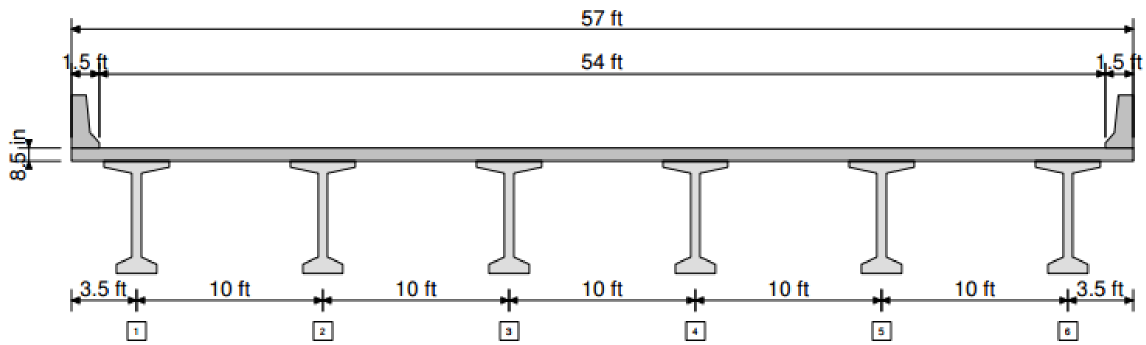


Figure 5.11: Analysis Bridge Geometry with 10-ft Girder Spacing

The investigated results are shown in the following sections in table format, and the chart format is presented in Appendix B, including minimum required number of strands, compression/tension steel on top of flange, and area of shear reinforcements with specific strand patterns (straight, de-tension, and harp). The comparison between NU girder and K-girder system was conducted according to the height of the girder; for example, NU-1800 versus K-6 and NU-1350 versus K-4. Additionally, 1-inch increment on the top flange of the NU girder plus-1 system was included in the analysis.

5.2.1 NU-2400 Girder at 10-ft Spacing

The analysis results are shown in Table 5.13, Figure B.11, and Figure B.12, which included straight, debonded, and harped strand pattern. When using a straight strand pattern, the longest possible spans were 133 ft with NU-2400 girders and 135 ft with the NU-2400 plus-1 girders. With a harped strand design, these maximum spans were significantly extended to 165 ft with NU-2400 girders and 170 ft with the NU-2400 plus-1 girders.

Table 5.13: Design Minimum Reinforcements for NU-2400 Girder System with 10-ft Spacing

Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**133**			
	Number of 0.6-in. strands	18	24	28*	34*	34*			
	As top @ trans., in.^{2}	2.422	2.405	2.452	2.569	2.517			
	Al req. @ bearing, in.^{2}	1.15	0.7	1.13	0.93	0.98			
	A_v/S. @ trans., in.^{2}	0.15	0.184	0.21	0.238	0.252			
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482			
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**135**			
	Number of 0.6-in. strands	20	22	26*	32*	36*			
	As top @ trans., in.^{2}	2.73	2.757	2.857	2.984	2.95			
	Al req. @ bearing, in.^{2}	0.97	0.93	1.36	1.16	0.86			
	A_v/S. @ trans., in.^{2}	0.149	0.18	0.206	0.237	0.254			
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482			
Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**	**160**	**165**
	Number of 0.6-in. strands	18	22	26	32	38	44	50	56
	As top @ trans., in.^{2}	0	0	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	1.36	1.33	1.11	0.91	0.54	0.37	3.12	0.18
	A_v/S. @ trans., in.^{2}	0.132	0.142	0.169	0.183	0.206	0.224	0.241	0.243
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**150**	**160**	**170**
	Number of 0.6-in. strands	20	22	26	32	36	42	50	58
	As top @ trans., in.^{2}	0	0	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	1.38	1.36	1.14	0.93	0.74	0.37	0.05	0.12
	A_v/S. @ trans., in.^{2}	0.111	0.142	0.169	0.184	0.21	0.233	0.248	0.257
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482	0.482	0.482

NU-2400 required more strands within the 110- to 130-ft span length, compared to girders with 1 inch added on top of the flange. For shorter span length (100 ft), the NU girder plus-1 required more reinforcement to produce sufficient nominal moment strength (M_r) due to increased girder size and corresponding cracking moment.

Per AASHTO Specification, the factored flexural resistance (M_r) should be greater than the minimum value between the cracking moment (M_{cr}) and the required factored moment ($1.33M_u$) based on reinforcement limitation discussed in Section 4.2.5. In the case of NU-2400 plus-1 girder with the straight strands, eighteen 0.6”-diameter prestressing strands were initially arranged as shown in Figure 5.12. The provided M_r was 8602.6 kips-ft, and minimum M_r was 8656.8 kips-ft which was governed by cracking moment (M_{cr}). Thus, NU-plus-1 girder needed two additional prestressing reinforcements (20 total) for the design span length of 100 ft in order to satisfy the specification requirement ($M_r > M_{cr}$).

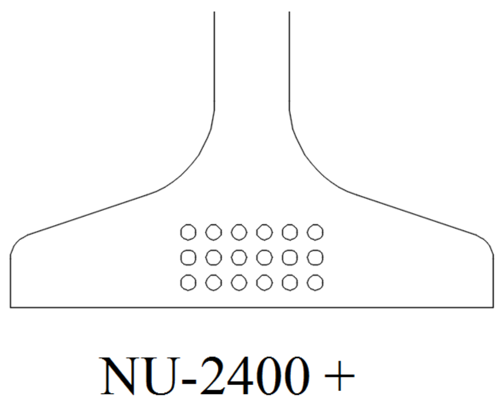


Figure 5.12: Designed Straight Strand Pattern with 100-ft Span Length

At spans less than 130 ft, the NU-2400 plus-1 girder with straight debonded strands could have the same number of strands as the harped design if additional steel is provided in the top flange due to excessive tension forces. For harped strand, a difference was observed when the span length was above 140 ft in which the plus-1 girder typically required two less strands. Figure 5.13 shows the strand pattern for 130-ft span length, with both debonded and harped strands. Note, the harped strand design had a lower strand centroid and fewer strands compared to the straight pattern.

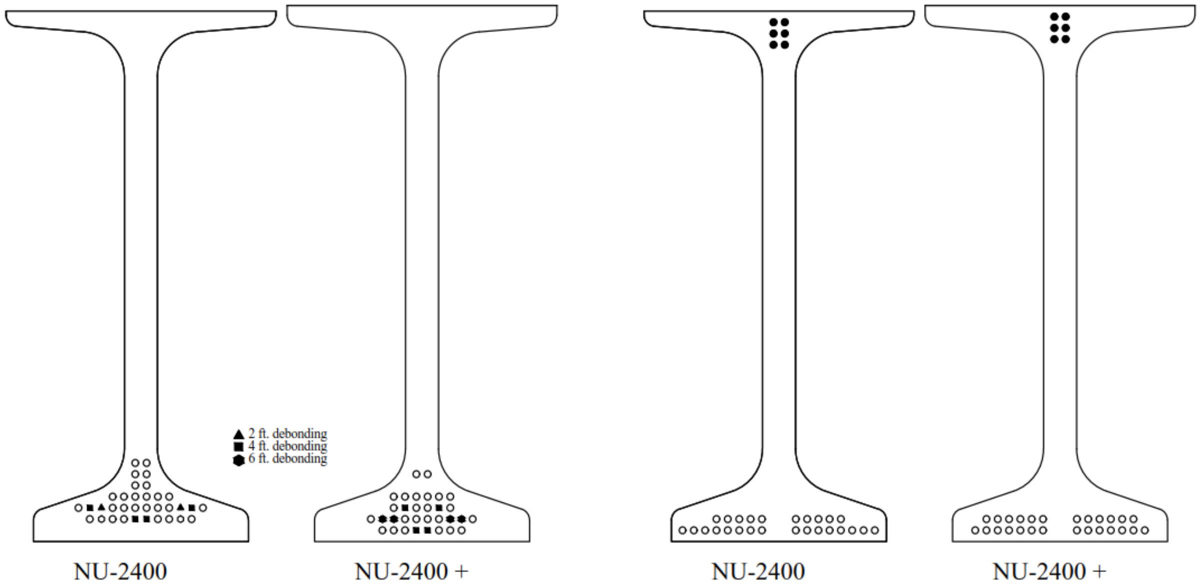


Figure 5.13: Design Strands with Span Length of 130 ft

5.2.2 NU-2000 Girder at 10-ft Spacing

The NU-2000 girder system's analysis results are presented in Table 5.14, including the straight, debonded, and harped designs. The chart format results are illustrated in Figure B.13 and Figure B.14. For the straight and debonded strand designs, additional top reinforcement was needed to resist the additional top stress. For the NU-2000 girders, there was no consistent difference observed in the number of strands when the top flange thickness increased within the investigated span lengths.

NU-2000 girder design at a span of 120 ft utilized thirty-four 0.6”-diameter straight strands (Figure 5.14) and eight strands were detensioned. For the NU-2000 plus-1 girder, only 32 strands were needed, as a lower center of gravity of prestressing force was achieved (Figure 5.14).

Table 5.14: Design Minimum Reinforcements for NU-2000 with 10-ft Spacing

Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**132**	
	Number of 0.6-in. strands	22	28	34*	40*	42*	
	As top @ trans., in.^{2}	2.393	2.348	2.555	2.535	2.606	
	Al req. @ bearing, in.^{2}	0.66	0.245	0.69	0.51	0.37	
	A_v/S. @ trans., in.^{2}	0.22	0.255	0.284	0.317	0.325	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**130**	**134**	
	Number of 0.6-in. strands	22	28	32*	40*	42*	
	As top @ trans., in.^{2}	2.672	2.646	2.862	2.84	2.872	
	Al req. @ bearing, in.^{2}	0.68	0.26	0.91	0.53	0.42	
	A_v/S. @ trans., in.^{2}	0.217	0.253	0.283	0.314	0.327	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	
Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**146**
	Number of 0.6-in. strands	22	26	32	38	46	52
	As top @ trans., in.^{2}	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	1.08	0.86	0.47	0.3	0	0
	A_v/S. @ trans., in.^{2}	0.18	0.212	0.239	0.258	0.277	0.286
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**140**	**148**
	Number of 0.6-in. strands	22	26	32	38	46	52
	As top @ trans., in.^{2}	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	1.1	0.88	0.49	0.32	0	0.18
	A_v/S. @ trans., in.^{2}	0.18	0.212	0.239	0.258	0.285	0.291
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482

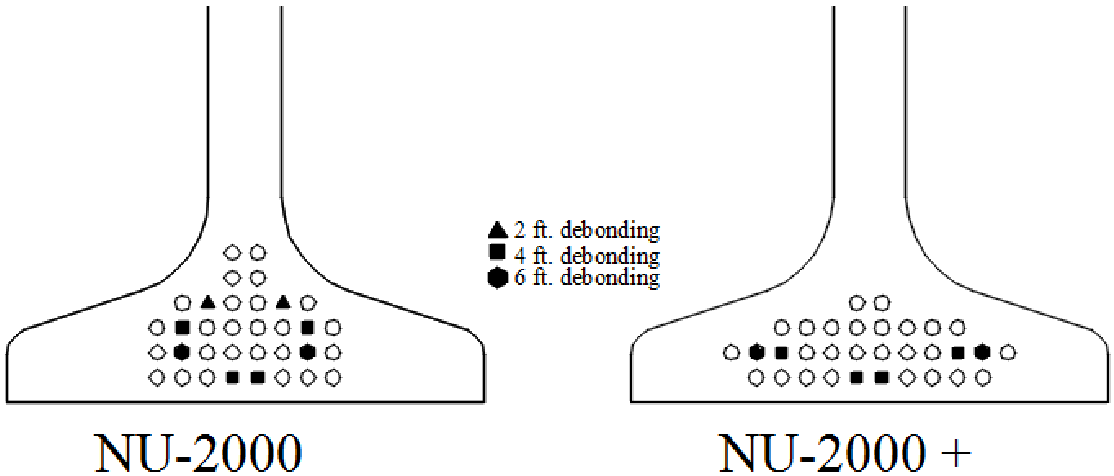


Figure 5.14: NU-2000 Girder Design Debonded Strands with Span Length of 120 ft

5.2.3 NU-1800 Girder and K-6 Girder at 10-ft Spacing

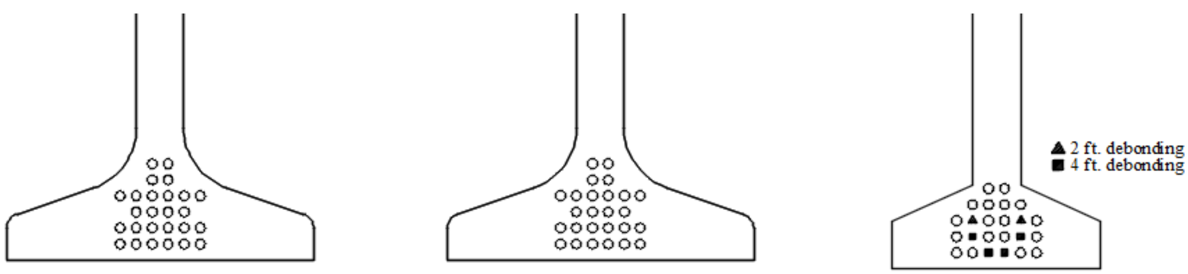
In this section, the NU-1800 girder system was compared with K-6 girder due to similarity in height (72.0 in. for K-6 and 70.9 in. for NU-1800). The straight and debonded analysis results are presented in Table 5.15 and Figure B.15. The K-6 girders were able to achieve a span length of 105 ft with twenty-eight 0.6”-diameter strands, with 21.4% out of an allowable 25% of the strands debonded at the ends. On the other hand, NU-1800 plus-1 girder achieved a span length of 128 ft with forty-two 0.6”-diameter strands, and there were five pairs of debonded strands (23.8% of the total strands).

Forty-two 0.6”-diameter strands were also used for NU-1800 girder (same as NU-1800 plus-1 girder), with 125 ft as the maximum applicable span. For 120-ft span length, the NU-1800 girder required two more strands than the NU-1800 plus-1 girder. When shorter span length was considered, the K-6 girder had a more economical design compared to the NU-1800 girder system based on the number of strands required and area of concrete according to Table 5.15.

Additionally, K-girders may require less shipping costs than the heavier NU girder for the same span length. The strand patterns for 100-ft span are presented in Figure 5.15 for the three different girder types with straight strands. Note, the three girders had a similar centroid of strands which were 5.17 in. and 5.85 in. from the bottom for K-6 and NU-1800 girder system, respectively. Note that K-6 girder had total of 25% of the strands debonded, while the NU girders did not require any debonding at this span length.

Table 5.15: Straight and Debond Design of Minimum Reinforcements Between NU-1800 and K-6 with 10-ft Spacing

NU-1800					
Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**125**
	Number of 0.6-in. strands	26	32	38*	42*
	As top @ trans., in.^{2}	2.271	2.349	2.5	2.462
	Al req. @ bearing, in.^{2}	0.23	0	0.29	0.22
	A_v/S. @ trans., in.^{2}	0.265	0.299	0.335	0.349
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**120**	**128**
	Number of strands	26	32	36*	42*
	As top @ trans., in.^{2}	2.432	2.535	2.81	2.898
	Al req. @ bearing, in.^{2}	0.25	0	0.49	0.29
	A_v/S. @ trans., in.^{2}	0.264	0.298	0.332	0.356
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482
K-6					
Straight + Debonded	**Span Length, ft**	**100**	**105**		
	Number of 0.6-in. strands	24*	28*		
	As top @ trans., in.^{2}	2.748	2.545		
	Al req. @ bearing, in.^{2}	1.06	0.76		
	A_v/S. @ trans., in.^{2}	0.252	0.275		



NU-1800 NU-1800 + K-6

Figure 5.15: NU-1800 and K-6 Girder Design Debonded Strands with Span Length of 100 ft

The harped design results for NU-1800 and K-6 girders at 10-ft spacing are reported in Table 5.16 and Figure B.16. Using harped strands, both girder systems had similar number of required strands at the same span length. However, the NU-1800 plus-1 girder reached 138 ft which was 3 ft longer than the NU-1800 girder. The K-6 girder had a maximum span length of 120 ft. The detailed strand patterns (with harping) corresponding to a 120-ft span and 10-ft spacing are shown in Figure 5.16.

Table 5.16: Harped Design of Minimum Reinforcements Between NU-1800 and K-6 with 10-ft Spacing

NU-1800						
Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**135**
	Number of 0.6-in. strands	24	30	36	44	50
	As top @ trans., in.^{2}	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.84	0.45	0.28	0	0.09
	A_v/S. @ trans., in.^{2}	0.224	0.255	0.277	0.303	0.306
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**130**	**138**
	Number of strands	24	30	36	44	50
	As top @ trans., in.^{2}	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.86	0.47	0.29	0	0.14
	A_v/S. @ trans., in.^{2}	0.223	0.253	0.277	0.302	0.317
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482
K-6						
Harped	**Span Length, ft**	**100**	**110**	**120**		
	Number of strands	24	30	36		
	As top @ trans., in.^{2}	0	0	0		
	Al req. @ bearing, in.^{2}	0.86	0.68	0.7		
	A_v/S. @ trans., in.^{2}	0.204	0.227	0.249		

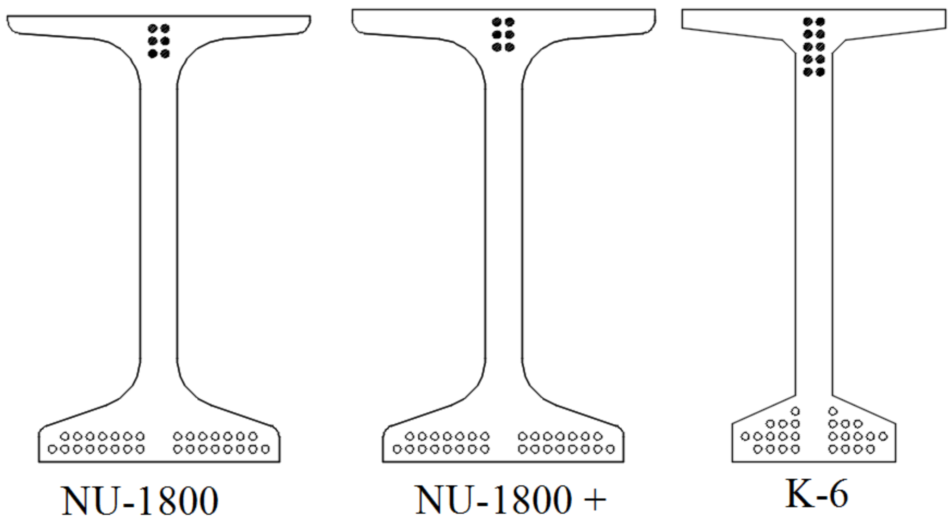


Figure 5.16: NU-1800 and K-6 Girder Harp Design with Span Length of 120 ft

5.2.4 NU-1600 Girder at 10-ft Spacing

The results of analyses with NU-1600 girders at 10-ft spacing are presented in this section. Using straight debonded strands, the NU-1600 can span up to 115 ft, while the plus-1 girder's span length could be extended to 117 ft. Debonding was only required when spans approached the maximum length. In all cases with straight strands, additional mild steel must be provided in the top flange at the member ends due to excessive top tension. Figure 5.17 shows the strand patterns used for the NU-1600 and NU-1600 plus-1 girders at a span of 110 ft.

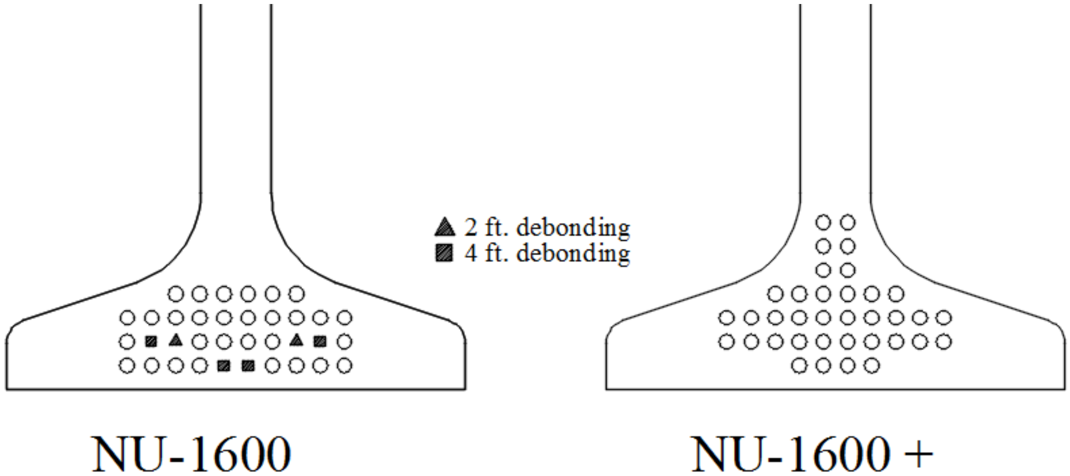


Figure 5.17: NU-1600 Girder Straight or Debond Design with Span Length of 110 ft

Table 5.17: Design Minimum Reinforcements for NU-1600 Girder System with 10-ft Spacing

Straight + Debonded	**Span Length, ft**	**100**	**110**	**115**	
	Number of 0.6-in. strands	30	36*	40*	
	As top @ trans., in.^{2}	2.219	2.392	2.385	
	Al req. @ bearing, in.^{2}	0	0.28	0	
	A_v/S. @ trans., in.^{2}	0.318	0.353	0.372	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	
Plus-1 Straight + Debonded	**Span Length, ft**	**100**	**110**	**117**	
	Number of 0.6-in. strands	28	36	40*	
	As top @ trans., in.^{2}	2.598	2.57	2.676	
	Al req. @ bearing, in.^{2}	0.03	0	0	
	A_v/S. @ trans., in.^{2}	0.314	0.35	0.379	
	A_{vh}_min, in.^{2}	0.482	0.482	0.483	
Harped	**Span Length, ft**	**100**	**110**	**120**	**125**
	Number of 0.6-in. strands	28	34	42	48
	As top @ trans., in.^{2}	0	0	0	0
	Al req. @ bearing, in.^{2}	0.43	0.26	0	0.05
	A_v/S. @ trans., in.^{2}	0.275	0.302	0.328	0.346
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482
Plus-1 Harped	**Span Length, ft**	**100**	**110**	**120**	**125**
	Number of 0.6-in. strands	28	34	42	46
	As top @ trans., in.^{2}	0	0	0	0
	Al req. @ bearing, in.^{2}	0.45	0.08	0	0
	A_v/S. @ trans., in.^{2}	0.273	0.306	0.329	0.339
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482

For harped strand pattern, the span can be extended to 125 ft. Additionally, no additional top steel is required when using the harped strand patterns. Comparison of NU-1600 and plus-1 girders in detail strands design are shown in Figure 5.17. For harp design, there were no difference between NU-1600 and plus-1 girders in number of strands required within the investigated span length range. However, the plus-1 girders required two less strands at a span length of 125 ft. The harp design strand patterns are shown in Figure 5.18 for the span length of 125 ft.

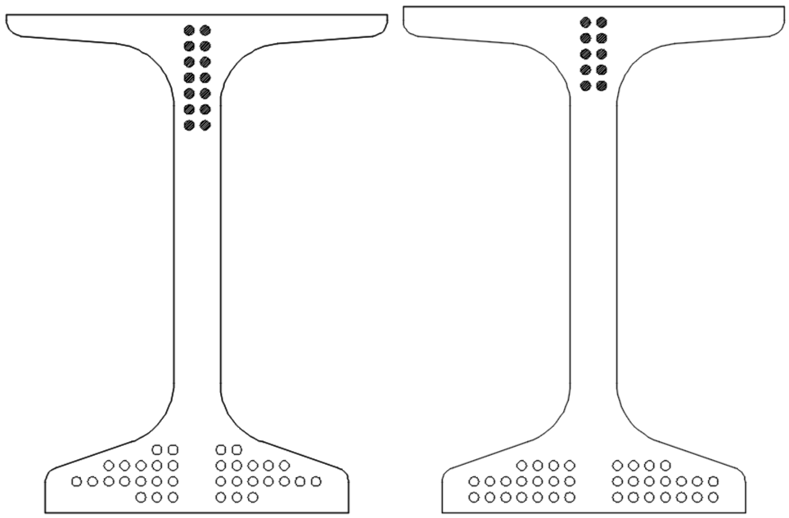


Figure 5.18: NU-1600 Girder Harp Design with Span Length of 125 ft

5.2.5 NU-1350 Girder and K-4 Girders at 10-ft Spacing

As previously noted, the K-4 and NU-1350 girders did not satisfy AASHTO minimum depth requirement (Table 2.5.2.6.3-1) for use in the targeted span range of 120–160 ft, and analyses also proved that they were not capable of this span range. Still, the authors have included the analysis of NU-1350 and K-4 girders at span ranges from 70 ft to maximum span length. Thus, in this section, NU-1350 girders were compared with K-4 girders using straight, debonded, and harped strand designs.

For straight and debonded design, the analysis results are presented in Table 5.18 and Figure B.19. No differences in the number of required strands were observed in NU-1350 and NU-1300 plus-1 girders except at 100-ft span length. Here, two less straight strands (36 total) were required for the NU-1350 plus-1 girder for a design that did not require debonding.

However, if debonding were used for the NU-1350 plus-1 system, then an even smaller number of strands would be possible. Figure 5.19 shows two possible strand patterns (at 100-ft span length) for the NU-1350 plus-1 girder. The figure on the right is the case where 36 strands are used and no debonding is required. The figure on the left, however, shows that if debonding were used, then only 34 strands would be required.

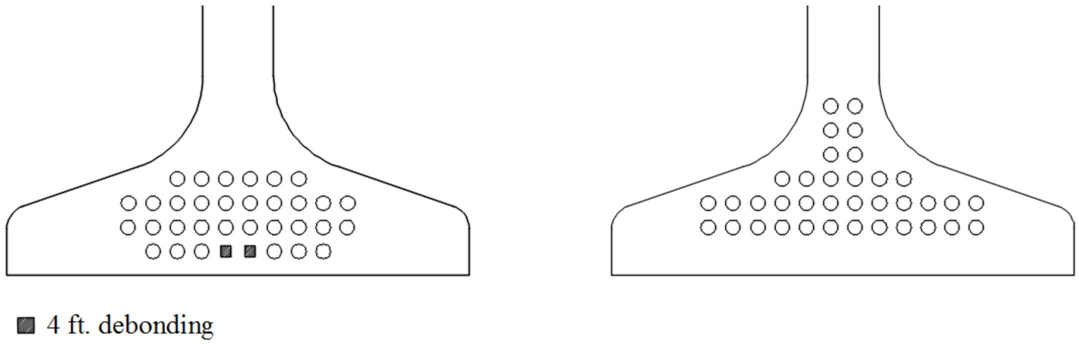


Figure 5.19: NU-1350 Plus-1 Girder, Minimum Strands Design With and Without Debonding for Span Length of 100 ft

For the NU-1350 plus-1 girder, the maximum span length was 106 ft when forty 0.6”-diameter strands were used with 15% of total strands debonded. On the other hand, the NU-1350 girder required debonding when the span length was above 100 ft, and the span could reach 105 ft with six debonded strands out of 40 total. Compared to NU-1350 girders, K-4 girder was limited to a span length of 90 ft with debonding. Figure 5.20 shows the minimum number of required strands and corresponding strand patterns at 90-ft span length for NU-1350, NU-1350 plus-1, and K-4 girders. At this span, the total number of required strands was similar (28 for NU-1350 girders and 30 for K-4 girders).

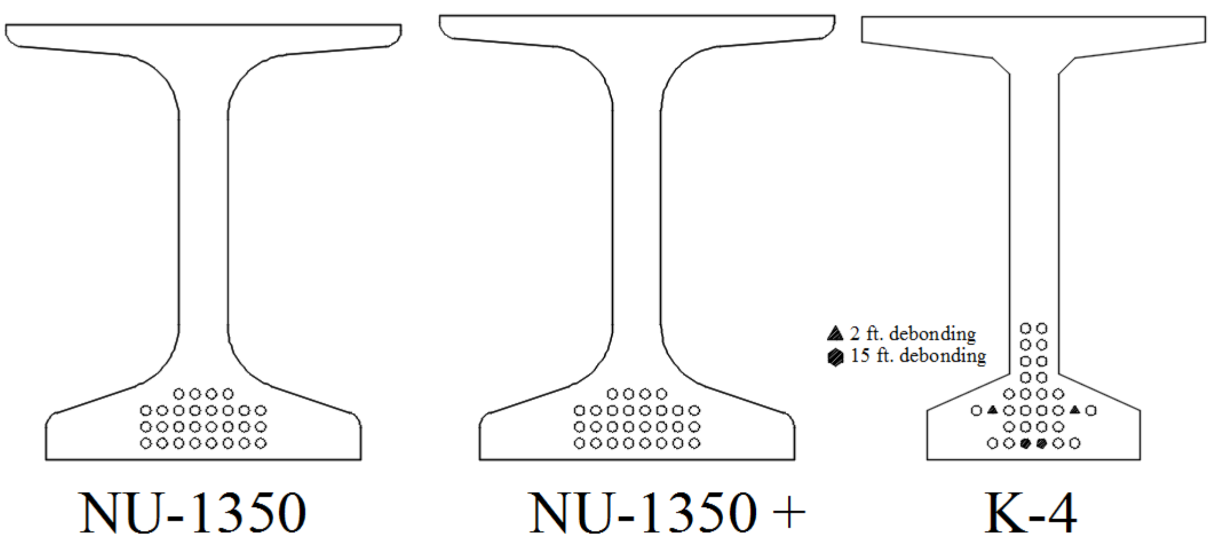


Figure 5.20: NU-1350 and K-4 Girder Strand Patterns (Straight Strands) at a Design Span of 90 ft

Table 5.18: Straight and Debond Design of Minimum Reinforcements Between NU-1350 and K-4 Girder System with 10-ft Spacing

NU-1350						
Straight + Debonded	**Span Length, ft**	**70**	**80**	**90**	**100**	**105**
	Number of 0.6-in. strands	16	22	28	38*	40*
	As top @ trans., in.^{2}	2.224	2.248	2.286	1.516	2.067
	Al req. @ bearing, in.^{2}	0.63	0.21	0	0	0
	A_v/S. @ trans., in.^{2}	0.258	0.312	0.359	0.409	0.424
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482
Plus-1 Straight + Debonded	**Span Length, ft**	**70**	**80**	**90**	**100**	**106**
	Number of strands	16	20	28	36	40*
	As top @ trans., in.^{2}	2.335	2.444	2.449	1.499	2.601
	Al req. @ bearing, in.^{2}	0.65	0.42	0	0	0
	A_v/S. @ trans., in.^{2}	0.254	0.306	0.355	0.412	0.424
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482
K-4						
Straight + Debonded	**Span Length, ft**	**70**	**80**	**90**		
	Number of 0.6-in. strands	18	22*	30*		
	As top @ trans., in.^{2}	1.556	1.739	1.638		
	Al req. @ bearing, in.^{2}	0.42	0.64	0.08		
	A_v/S. @ trans., in.^{2}	0.245	0.287	0.331		

When the strands were harped instead of using debonding, the maximum span length was 3 ft longer for both the NU-1350 and K-4 girders. Furthermore, NU-1350 plus-1 girder extended the span length from 106 ft to 112 ft with 40 strands required. These results are shown in Table 5.19 and presented in chart format in Figure B.20.

Table 5.19: Harped Design of Minimum Reinforcements Between NU-1600 and K-6 Girder System with 10-ft Spacing

NU-1350							
Harped	**Span Length, ft**	**70**	**80**	**90**	**100**	**108**	
	Number of 0.6-in. strands	16	20	28	34	42	
	As top @ trans., in.^{2}	0	0.757	0	0	0	
	Al req. @ bearing, in.^{2}	0.83	0.61	0.23	0	0	
	A_v/S. @ trans., in.^{2}	0.242	0.288	0.318	0.359	0.372	
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	
Plus-1 Harped	**Span Length, ft**	**70**	**80**	**90**	**100**	**110**	**112**
	Number of 0.6-in. strands	16	20	26	34	42	44
	As top @ trans., in.^{2}	0	0	0	0	0	0
	Al req. @ bearing, in.^{2}	0.85	0.83	0.42	0	0	0
	A_v/S. @ trans., in.^{2}	0.237	0.27	0.318	0.354	0.383	0.39
	A_{vh}_min, in.^{2}	0.482	0.482	0.482	0.482	0.482	0.482
K-4							
Harped	**Span Length, ft**	**70**	**80**	**90**	**93**		
	Number of 0.6-in. strands	16	22	28	30		
	As top @ trans., in.^{2}	0	0	0	0		
	Al req. @ bearing, in.^{2}	1.04	0.83	0.84	0.35		
	A_v/S. @ trans., in.^{2}	0.19	0.231	0.258	0.281		

5.2.6 Summary of 10-ft Girder Spacing

A summary of the different girder depths and number of required strands at 10-ft spacing are presented in Figure 5.21 and Figure 5.22. In these figures, the maximum possible spans for the different K-girder sections are shown, along with their corresponding number of required strands. The NU-2400 plus-1 girder with harped design strands had a maximum span length of 170 ft (Figure 5.22). Due to the limitation on the maximum number of debonded strands, the maximum span lengths for straight strands (Figure 5.21) were significantly shorter than the spans achieved with a harped design, particularly for the larger girders.

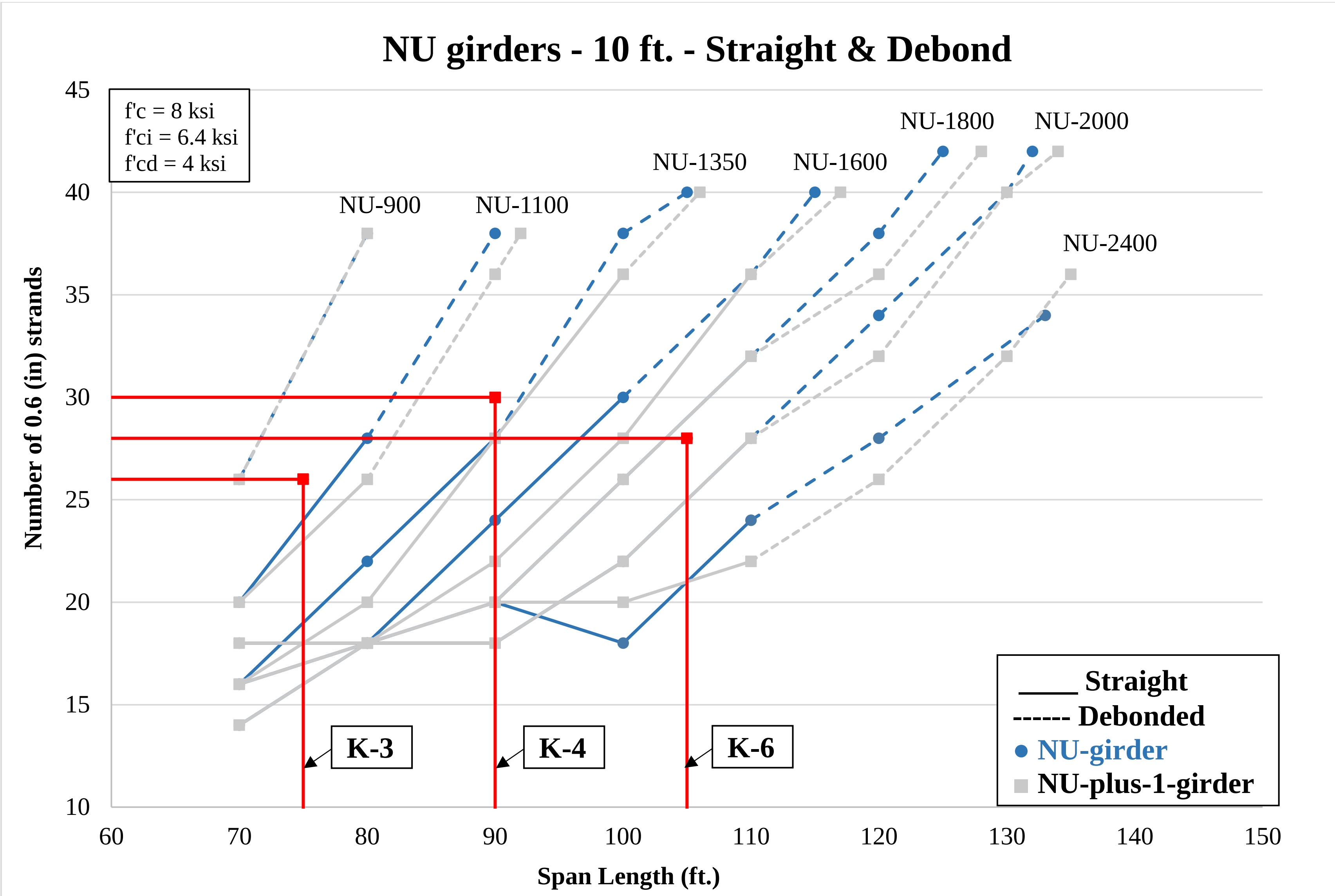


Figure 5.21: 10-ft Spacing Summary of NU Girders Straight and Debonded Strand Minimum Reinforcement Estimation

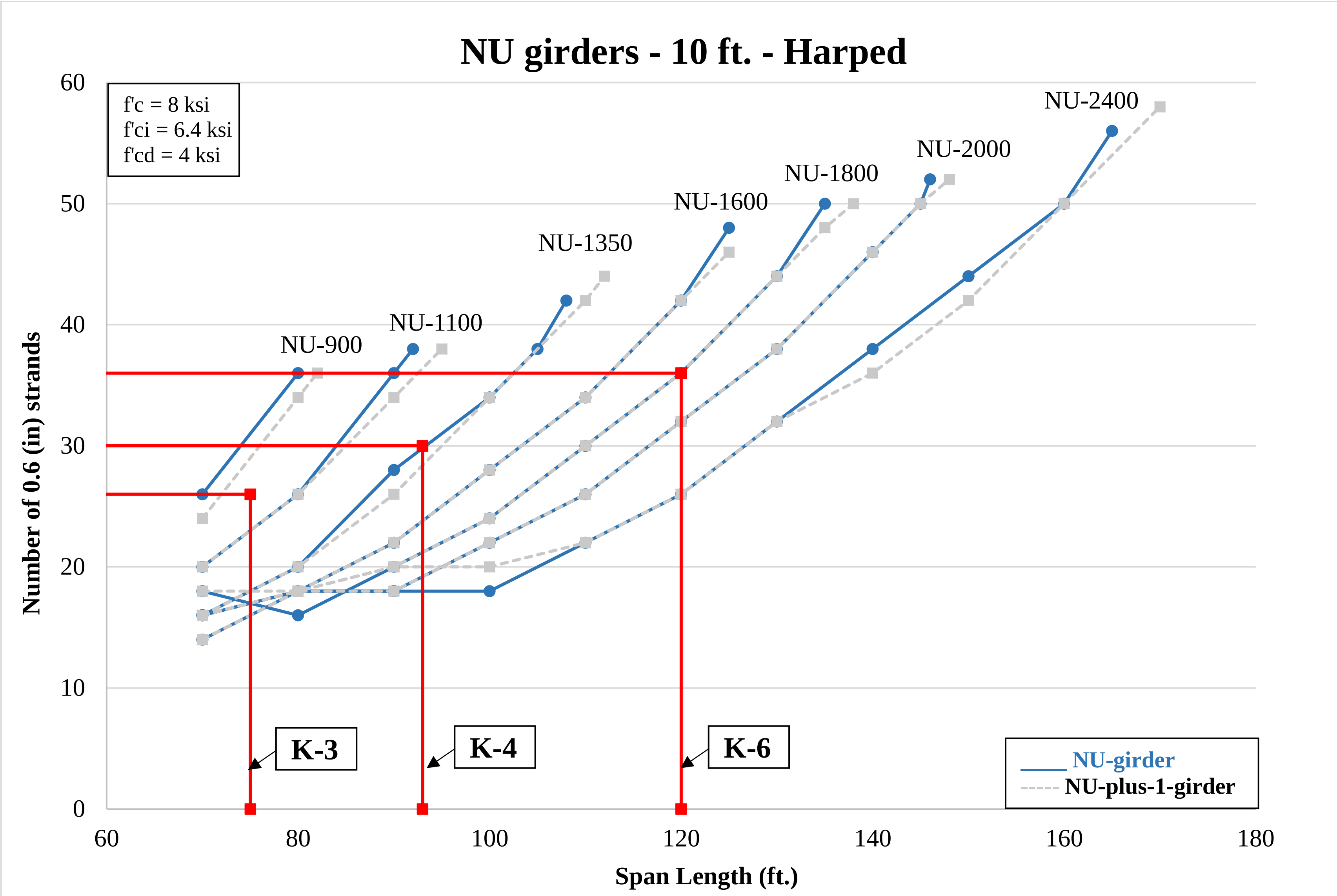


Figure 5.22: 10-ft Spacing Summary of NU Girders Straight and Debonded Strand Minimum Reinforcement Estimation

Chapter 6: Site Visit to Coreslab Structures, Inc.

The researchers traveled to Coreslab Structures, Inc., in Kansas City in July 2016 to tour the production facility and to meet with Engineering Manager Terry Fleck and V.P. and General Manager Mark Simpson. Coreslab has over 40 years of experience producing precast/prestressed concrete products, and they have been producing K-Girders for over 25 years and NU girders for over 7 years. The reason the researchers chose to visit the Coreslab facility is because it is the only pretensioned concrete plant that regularly manufactures both K-Girders and NU girders, and we are very thankful for the opportunity to discuss the experience that Coreslab has had with these two girder types.

At the time of the meeting, Coreslab had two full sets of NU-900 and NU-2000 casting forms and were planning to purchase 350 ft of forms for casting NU-2400 girders, which are expected to last for 30 years. Coreslab personnel estimated that the cost for the new forms would be about $250,000. Figure 6.1 shows the side of NU girder casting forms (NU-900 on the left, and NU-2000 on the right).

Figure 6.1: NU Girder System Casting Form at Coreslab Structures, Inc.

At the time of the meeting, the NU girder casting forms at Kansas City were not designed to cast the largest NU girder (NU-2400) so Coreslab has plans to modify their existing NU girder forms by stiffening bottom region of the forms to resist the additional form pressure (due to weight of the fresh concrete) associated with casting the deeper girder size. The casting bed with NU-900 forms is shown in Figure 6.2.

Figure 6.2: Casting Bed with NU-900 Forms at Coreslab Structures, Inc.

Mr. Terry Fleck and Mr. Mark Simpson confirmed that both Nebraska and Missouri are using NU girders. They are also aware of other states that have created and use shapes similar to the NU girders, but that are not actual NU girders. Coreslab recommends the NU girder system when it makes sense from an economical viewpoint, and they predict increasing demand in the future. They said that NU girders are an appropriate replacement of steel girders because of the efficient shape and relative low cost.

According to their company's experience with NU girders, span lengths from 100 to 140 ft are used most commonly. Mr. Fleck noted, "we have actually done quite a few NU35 girders from mid-80 ft length up to a 100 ft as well." He noted that these jobs were for the Missouri Department of Transportation (MoDOT) and not KDOT. Mr. Fleck said that they had used NU63 (NU-1600) girders that were 161 ft long and spaced at 8-ft, 6-in. centers. Additionally, they have previously made 168-ft girders using NU-2000 forms with 3 inches of concrete added to the top flange (NU-2000 +3"). These girders weighed around 190,000 pounds. However, there were some difficulties that occurred during shipment of these girders and one of them "rolled over" during transport. This issue was attributed to inexperience of the driver and not anything specifically related to the NU girder system, as longer girders generally require more care in shipping.

Mr. Fleck recommended girder spacing to be around the 10-ft spacing range for the NU girders to be more cost effective. He noted that designers sometimes treat NU girders as a 1-to-1 replacement for K-girders. However, this can result in a more expensive bridge overall since the NU girders require more materials to produce and are heavier to ship. Economy is achieved by taking advantage of the wider top flange and spacing the girders farther apart than is possible with K-girders.

In order to produce quality bridge girders, Coreslab typically takes 1 ½ days to prepare and perform a detailed check of the forms and reinforcement prior to casting the concrete. This detailed checking of forms prior to casting was credited for the fact that Coreslab has not had to re-pour/re-make any NU girders due to improper setup. Therefore, bridge girders are cast every other day with a possibility of two to three castings per week. This is true for both K-girders and NU girders. Figure 6.3 shows the overall girder configuration during reinforcement placement and setup, while Figure 6.4 shows the steel bulkhead and placement of mild steel reinforcement.

After all strands are tensioned and mild reinforcement is set, the concrete batches are prepared and mixed in the plant. In order to produce hardened concrete with the desired properties, two variables Coreslab must regularly deal with are fluctuations in weather and raw materials, as these can affect concrete strength, bonding of reinforcement, and durability. Tension in the prestressing strands is released individually (by torch) after the concrete reaches the required compressive strength. The strand cutting is performed by well-trained personnel for safety purposes.

Detensioning often occurs about 14 to 16 hours after casting, when the concrete release strength reaches the required threshold based on engineering design. If the concrete strength does not reach the desired strength at the anticipated time, the prestress release time is postponed and this can affect schedules. Subsequently, shrinkage cracks can also occur while the girders stay on the prestressing bed for extra days without being detensioned.

Figure 6.3: Preparation of NU-2000 Form at Coreslab Structures, Inc.

Figure 6.4: Steel Bulkhead for Pretensioned Strand (left) and Placement of Mild Steel Reinforcement at Girder End (right)

Once the strands are detensioned, the camber is measured and the girders are inspected for possible cracks. Camber of the girders is typically maintained within an acceptable range. Mr. Fleck noted that they had previously experienced up to 7.5 in. of camber on NU-1100 girders with a design span length of 115 ft, but that camber is no longer an issue since they have gained more experience with the NU girder system. Figure 6.5 shows NU-2000 girders in the storage yard at Coreslab. In this figure, the typical bowing of the girders due to long-term camber can be observed.

Mr. Fleck noted that another area of possible concern in using NU girders has been the very thin top flange, as the flange can be damaged when the side casting forms are removed. Coreslab gave some possible recommendations to prevent this issue (such as using additional reinforcement on the back side of the top flange or increasing the thickness by using a "plus-1" or "plus-2" option) and noted that it is sometimes very challenging to control cracks in this region.

Figure 6.5: Camber in NU-2000 Girders at the Coreslab Plant in Kansas City

From the design standpoint, Mr. Fleck has observed that engineers are committed to pushing the span length to the maximum achievable level based on theory, while at the same time making sure all specifications are satisfied. However, it is often very challenging for girder producers to push the span-length limits due to variables such as existing bed lengths and handling equipment, and storage locations within the facility that can accommodate the longer members. As noted previously, Coreslab has succeeded in casting NU-2000 plus-3” girders that had a 168-ft span length, and they have a positive attitude about casting longer girders in the future. With their investment in NU-2400 girder forms, Coreslab expects that 200-ft span lengths will become commonly used since the NU girders are capable and suitable for larger spans.

Lateral stability must be considered during lifting and hauling, especially when larger and longer girders are used. Beam material properties, quantity of prestressing, hauling equipment, and roadway condition all influenced the stability of these girders.

For transporting the beam, the beam generally has “sufficient lateral bending strength to withstand greater angle of inclination” but flexible supports with less roll stiffness result in a beam tending to camber laterally (Mast, 1993). These concerns, lateral stability, can be improved by adjusting lifting location, adding extra prestressing reinforcement in the top flange, selecting desire lifting method, and modifying the roll axis.

Coreslab highly recommended using NU girders for longer spans, and said that it was currently not a problem to cast and ship NU girders within 185-ft span length. Coreslab also recommended that the existing K-girder system should not be eliminated because of the lower costs associated with these bridges. Mr. Fleck also noted that the state of Missouri is currently using both NU girders (for longer spans) and K-girder sections when possible (they had several recent jobs with K-5 and K-6 girders). He said that this approach made a lot of sense from a cost standpoint.

Chapter 7: Cost Analysis

Cost of pretensioned concrete girders can vary significantly, depending on girder sizes and design details. A general cost is discussed in this section, and the price will be influenced by the area of concrete, the amount of prestressing steel (number of strands), general casting costs, labor, and transportation. From the producer standpoint, the costs are critically influenced by delivery location (bridge site) and bed fitting.

If the casting bed can be nearly filled by the required girder lengths, then this will result in the lowest cost-per-foot. Alternatively, if the required girder sizes lead to only a partially-full bed being cast each day, then the cost-per-foot can increase significantly. Also, the cost-per-foot can vary significantly for longer spans, since other factors such as larger cranes and special shipping and handling devices may be required.

According to Coreslab, labor rates in the Kansas City area in Summer 2016 were approximately $28 per hour. Additionally, the number of labor hours can vary greatly between small and large girders because of different details and equipment needed for setup. Delivery costs had reached as much as $60 per foot for long girders in the past, and this cost depends on job-site location, road conditions, size of girder, and lateral stability requirement.

Mr. Terry Fleck, Engineering Manager at Coreslab Structures (Kansas), reviewed their records and provided typical cost ranges for various girder sizes which are listed in Table 7.1. These typical prices reflect the time period from 2015 to 2017 and are general guidelines that do not encompass all situations. The price is an estimated per-foot cost that includes labor, material, and delivery. In Table 7.1, the girders are arranged according to their height. Note, MoDOT refers to the NU girders according to the height in inches instead of the height in millimeters.

Table 7.1: Typical Delivered Price per Foot for 2015–2017 Time Period

Girder	Depth (Inches)	Missouri Name	Cost per foot
NU-900	35.4	**NU35**	\$180 – \$205
NU-1100	43.3	**NU43**	\$190 – \$215
NU-1350	53.1	**NU53**	\$220 – \$245
K-4	54.0		\$140 – \$155
NU-1600	63.0	**NU63**	\$230 – \$255
NU-1800	70.9	**NU70**	\$240 – \$270
K-6	72.0		\$225 – \$250

Note: these were typical delivered prices for the specific time period (2015–2017) and current pricing may lie outside these ranges due to many factors such as bridge location, span length and strand pattern, number of spans in a project, etc. Furthermore, this pricing included girders that Coreslab fabricated for MoDOT bridges which may be quite different than KDOT design.

From Table 7.1, the per-foot cost of K-girders are lower than NU girders of similar height, with the most significant difference occurring for the K-4 girders. However, as noted previously, the most efficient designs are achieved by taking advantage of the fact that NU girders can be spaced farther apart than K-girders with similar height.

Therefore, to get a better cost comparison, the quantity of K-girders were increased by reducing girder spacing with the same design assumption in Chapter 5. Specifically, when referring to the comparison of NU-1350 with K-4 girders with harped strands (Table 5.12), the K-4 girders had an ultimate span length of 104 ft at an 8-ft spacing, while NU-1350 girders were able to span up to 120 ft at a similar spacing of 8 ft.

In this analysis, the spacing of K-4 girders was decreased until the capable span length was extended to 120 ft. Consequently, nine K-4 girders were used at a 5.4-ft spacing, compared with six NU-1350 girders at an 8-ft spacing, and the bridge superstructure geometry for the K-4 option is shown in Figure 7.1. Note, a typical superstructure geometry for spacing at 8 ft was previously presented in Figure 5.1.

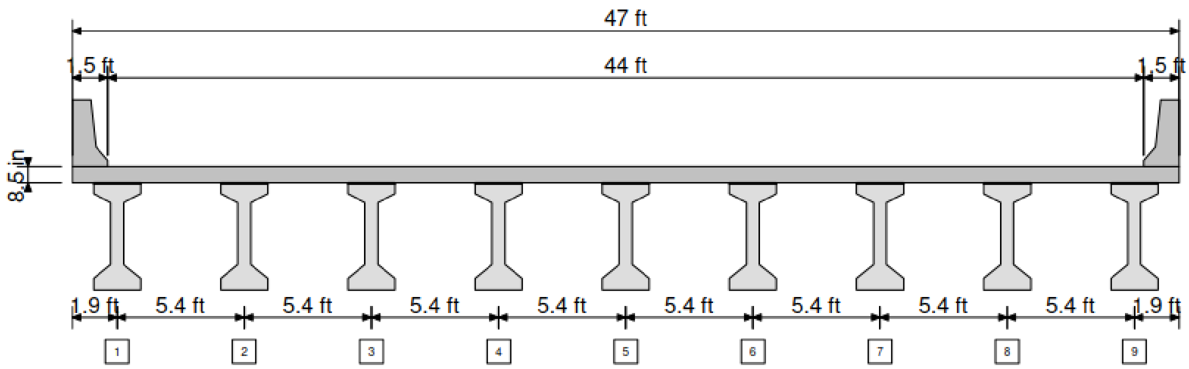


Figure 7.1: 122-ft Span Length Bridge Geometry by Using K-4 Girder

Using the typical cost information from Table 7.1, the average cost per foot for K-4 girders would be around $148, while the average cost per foot for NU-1350 girders would be approximately $233. Hence, for the K-4 option, the total girder cost per foot of bridge span would be 9 girders × $148/foot = $1,332 per foot of span length. For the NU-1350 option, the total girder cost per foot of bridge span would be 6 girders × $233/ft = $1,398 per foot of span length. Hence, even with a reduced spacing the K-4 option would still be competitive.

The researchers shared the results of this K-4 cost comparison with Terry Fleck at Coreslab and he cautioned that this is likely not an appropriate use of the numbers provided in Table 7.1, which are based on typical spans for the sections used. Mr. Fleck noted that part of the cost advantage of the K-4 girders is that they are almost never over 100 ft in length which allows Coreslab to cast three girders at a time. The NU-1350 prices listed in Table 7.1 were based on two girders in the bed at a time, thus not providing the same efficiency.

Mr. Fleck also noted that if the NU-1350 girders were spaced at 10-ft centers and a similar comparison conducted, then the NU-1350 option would be more cost effective based on the average prices in Table 7.1. Hence, Mr. Fleck noted "it is very difficult to use average prices in a comparison like that and say the K-4 is more cost effective at that span length. I really don't ever see pushing the K-4 past 110 ft and that is a stretch in my opinion."

Other factors which are not taken into account include the additional erection cost (for three more girders). However, a smaller crane could possibly be used since the K-4 girders would weigh less. Additionally, the bridge deck would have a smaller span with the K-4 option.

A similar comparison was conducted using the K-6 girder option. From Table 5.8, the K-6 girders had an ultimate span length of 132 ft at an 8-ft spacing, while NU-1800 girders were able to span up to 148 ft at a similar spacing of 8 ft. Analyses were performed to determine the required spacing of K-6 girders to span 140 ft. It was established that K-6 girders with a 7-ft spacing and 40 harped strands could span 140 ft, compared to NU-1800 girders at 8-ft spacing with 44 harped strands.

Using the typical cost information from Table 7.1, the average cost per foot for K-6 girders would be around \$237, while the average cost per foot for NU-1800 girders would be approximately \$255. Hence, for the K-6 option, the total girder cost per foot of bridge span would be 7 girders × \$237/foot = \$1659 per foot of span length. For the NU-1800 option, the total girder cost per foot of bridge span would be 6 girders × \$255/ft = \$1530 per foot of span length.

Therefore, in the case where spans are extended to the point of requiring more K-6 girders than NU girders, the K-6 girder may not be more economical than NU girders. Still, because of the large price ranges in Table 7.1, and because of other factors such as possibly smaller crane and sub-structure requirements due to the reduced weight of K-6 girders, it is possible that the K-6 girders may still be cost-competitive at spans approaching 140 ft. Table 7.2 lists the superstructure assumptions in the comparison of K-6 and NU-1800 girders used at 140-ft span.

When reviewing the above cost comparison of the K-6 versus NU-1800 option, Terry Fleck cautioned that all of the pricing for K-6 girders in Table 7.2 are really based on the MoDOT Type 8 girder which is the same section. However, Mr. Fleck noted that the MoDOT and KDOT designs are not really comparable, since MoDOT designs tend to be less conservative. Although the K-6 girders do have less concrete, the concrete only accounts for about 15% of the girder cost.

Therefore, the limited cost analyses performed above are only a rough estimation and are based on typical pricing obtained from Coreslab Structures (Kansas City) based on typical delivered pricing from 2015 to 2017. Furthermore, the average pricing includes MoDOT designs and this pricing could vary significantly for KDOT designs which tend to be more conservative. Still, the pricing information from Coreslab is believed to be most beneficial since they are the only plant that regularly produces both K-girders and NU girders.

The limited analysis cannot conclude which girder system is definitely more economic since the final cost could be dramatically different by design, as different substructure requirements may be employed, varying material and hauling expenses, etc. Still, it gives an idea in cost for selecting girder system with relatively similar design span length.

Table 7.2: Design Span Length 140 ft Comparison in NU-1800 and K-6 Girder with Varied Girder Spacing

Bridge Information			Bridge Information		
NU Girder type	**1800**		**K-Girder type**	**6**	
No. of Girders	6		**No. of Girders**	7	
Girder Spacing	8	ft	**Girder Spacing**	7	ft
Length of girders	140	ft	**Length of girders**	140	ft
Total Deck Width	47	ft	**Total Deck Width**	47	ft
Deck thickness	8.5	in.	**Deck thickness**	8.5	in.
Haunch Thickness	0.5	in.	**Haunch Thickness**	0.5	in.
Girder height	70.9	in.	**Girder height**	72	in.
Compute Concrete Weight of Bridge			**Compute Concrete Weight of Bridge**		
Girder			**Girder**		
total area	857.3	in.^{2}	total area	767	in.^{2}
	5.95	ft^2		5.32	ft^2
concrete unit weight	150	pcf	concrete unit weight	150	pcf
Girder Weight	893.02	plf	Girder Weight	798.96	plf
	125,023	lbs / beam		**111,854**	lbs / beam
Total weight	**750,138**	lbs	Total weight	**782,979**	lbs
Deck			**Deck**		
Total volume	4660.83	ft^3	Total volume	4660.83	ft^3
concrete unit weight	150	pcf	concrete unit weight	150	pcf
Total Weight	**699,125**	lbs	Total Weight	**699,125**	lbs
Haunch			**Haunch**		
Haunch Width	4.017	ft	Haunch Width	4	ft
Total volume	23.43	ft^3	Total volume	20.42	ft^3
concrete unit weight	150	pcf	concrete unit weight	150	pcf
Total Weight	**21,088**	lbs	Total Weight	**21,438**	lbs
Barriers			**Barriers**		
No. of barrier	2		No. of barrier	2	
Barrier Unit Weight	0.376	klf/barrier	Barrier Unit Weight	0.376	klf/barrier
Total Weight	**105,280**	lbs	Total Weight	**105,280**	lbs
Total Concrete	**1,575,630**	lbs	**Total Concrete**	**1,608,822**	lbs
	787.82	tons		**804.41**	tons
Strand			**Strand**		
# of Strands	44		# of Strands	40	
Weight	32.56	lbs/ft	Weight	29.6	lbs/ft
Total Strand weight	**27,350**	lbs	**Total Strand weight**	**29,008**	lbs

Chapter 8: Conclusions and Recommendations

This report presented the parametric analyses that were used to determine the applicable spans of both NU girders and K-girders and investigated the usefulness of possibly adopting the more efficient NU girders in the state of Kansas. The parametric investigation was preceded by a thorough review of designs using current AASHTO and KDOT standards, understanding the calculations behind commercial software CONSPAN, and then determining the ultimate achievable single-span length and minimum number of 0.6”-diameter 270 ksi prestressing strands required for the different girder sections at specific span lengths and lateral spacing.

The investigations were conducted for both K-girders and NU girders with straight, debonded, and harped strand patterns. Furthermore, this report included surveying U.S. state DOTs who have already adopted the NU girder system, and visiting Coreslab Structures (Kansas City) to learn from their experience and knowledge in producing both K-girders and NU girders. Based on the analytical studies conducted and persons interviewed, the following conclusions and recommendations can be drawn:

- The commercial software CONSPAN was very useful in the superstructure design process, and the internal calculations were verified within the scope of this study.
- The NU girder system was developed in order to provide a pretensioned concrete girder solution that could extend the applicable range of the traditional girder systems such as K-girders. It does this primarily by providing a larger bottom flange to allow for more strands and a much wider top flange that provides increased stiffness and stability. There is no question that the more efficient NU girder system should be used in order to extend the span range of pretensioned concrete girders beyond the practical limits of K-girders. This upper practical limit for K-6 girders is likely in the range of 130- to 140-ft spans based on the concrete release strengths and stress limitations assumed in this study.

- However, the advantages of the NU girder system occur primarily at longer spans and for specific cases to increase spans where maximum girder height is limited. For shorter spans, stability is not a concern and the wider top flange is not needed. In fact, the wider top flange and increased stiffness often necessitates using more strands in order to satisfy minimum reinforcement requirements based on the higher cracking moment. This offsets the benefit of the more efficient cross-section and often can result in a more expensive superstructure design since NU girders are heavier and cost more to produce than K-girders at a similar depth.
- A site visit to Coreslab Structures in Kansas City confirmed that some designers are using NU girders as a 1-to-1 replacement for K-girders. This results in a net cost increase for the overall bridge. For shorter spans with standard spacing, K-girders are almost always the more economical option.
- The study found that harped strand designs are more efficient than using straight strands with debonding. Harped strand options typically can achieve longer spans than debonded options due to specification limits on the number of strands that can be debonded. This was true for both K-girders and NU girders.
- Generally, NU-plus-1 girders required either the same number or at most two fewer strands than the standard NU girders at the same span. The maximum span length of the NU girder plus-1 option was typically about 2–3 ft longer than with the standard NU girder.
- Even though the plus-1 option may not make much difference from a structural analysis standpoint, Coreslab noted that the very thin top flanges of NU girders are often prone to cracking when side-forms are removed. For this reason, and also to provide a more robust member in the event of future re-decking, the authors recommend using either a plus-1 or plus-2 option when NU girders are utilized, and to provide additional mild steel reinforcement in the top flange area.

- When evaluating K-girders and NU girders at the same span and spacing, K-girders typically required the same number of strands (within two strands) as the NU girders. Therefore, the K-girders would be the more economical choice. For this reason, the authors recommend that KDOT continue using K-girders whenever possible, as these will likely result in the most cost-effective structure. This may also be true for limited cases where additional K-girders could be used at a smaller spacing to gain additional span length.
- The relatively low cost in producing K-4 girders is a clear advantage over NU-1350 girders for spans in the 80- to 100-ft range, and K-4 girders may be competitive for spans up to about 110 ft. In fact, the NU girder system does not appear to have an advantage at these shorter spans. Here, minimum strand requirements often offset the efficiency of the NU girder cross-section and the increased top flange is not needed for stability.

Hence, the overall finding of this study is that K-girders should continue to be used instead of NU girders whenever normal spans and girder spacing allow, as this will likely result in the most economical superstructure. At longer spans (beyond 130–140 ft) NU girders are an excellent option and should become a standard design implementation to extend the applicable range of pretensioned girders to 200 ft and beyond.

Additionally, the NU girder system can be used for the purpose of extending the span range (beyond K-girder capabilities) in specific situations where the maximum girder height is fixed. However, as shown previously through analyses, if K-girders can achieve the desired span at a normal spacing, then these will likely provide the most economical option.

References

American Association of State Highway and Transportation Officials (AASHTO). (2012). *AASHTO LRFD bridge design specifications* (6th ed.). Washington, DC: Author.

Bentley Systems, Inc. (2012). *LEAP CONSPAN user manual*. Exton, PA: Author.

Burgueño, R., & Sun, Y. (2011). *Effects of debonded strands on the production and performance of prestressed concrete beams* (Report No. RC-1546). Lansing, MI: Michigan Department of Transportation. Retrieved from https://www.michigan.gov/documents/mdot/MDOT_Research_Report_RC-1546_354418_7.pdf

Geren, K. L., & Tadros, M. K. (1994). The NU precast/prestressed concrete bridge I-girder series. *PCI Journal, 39*(3), 26–39.

Hanna, K. E., Morcous, G., & Tadros, M. K. (2010). *Design aids of NU I-Girder bridges*. Lincoln, NE: Nebraska Department of Roads.

Holombo, J., & Tadros, M. (2009). *Recommended LRFD minimum flexural reinforcement requirements* (NCHRP Web-Only Document 149). Washington, DC: Transportation Research Board. Retrieved from http://www.trb.org/Publications/Blurbs/162915.aspx

Kansas Department of Transportation (KDOT). (2015). *Design manual, Volume III – Bridge section*. Topeka, KS: Author.

Mast, R. F. (1993). Lateral stability of long prestressed concrete beams, Part 2. *PCI Journal, 38*(1), 70–88.

Precast/Prestressed Concrete Institute (PCI). (2014). *Precast prestressed concrete bridge design manual* (3rd ed.). Chicago, IL: Author.

Seguirant, S. J. (1998). New deep WSDOT standard sections extend spans of prestressed concrete girders. *PCI Journal*, *43*(4), 92–119.

Walbrun, A. M. (2006). *Comprehensive analysis of automated LRFD bridge superstructure and substructure design software compared against hand calculations and design* (Master's thesis). University of Wisconsin, Madison, WI.

Appendix A: Survey Questions

<u>Survey of University of Nebraska I-Girders Used by State DOTs</u>

1. Name of State:
2. Do you currently use the University of Nebraska I-girder system in your state?

   ☐ **Yes** ☐ **No**

   *If your answer is "**yes**" in question 2, please provide additional information by answering questions 3-8. If your answer is "**no**," the survey is ended and **thank you for your time**.*
3. When (year) did your state begin using the University of Nebraska I-girder system?
4. What sizes/depths of University of Nebraska I-girders do you currently employ?
5. What is the maximum span length your state is using for University of Nebraska I-girder sections?
6. What is the concrete strength at transfer and at services that your state uses for University of Nebraska I-girders?
7. Any difficulties that precasters have caused in implementing the section at the

   ☐ **Yes** ☐ **No**

   If your answer is "yes," please detail the issues below.
8. Any impact on costs associated with using the NU girder system versus other prestressed concrete girder systems in the **following areas**:

8.1 <u>Design area</u> ☐ **Yes** ☐ **No**

If your answer is "yes," then please detail the issues below.

8.2 <u>Fabrication / Labor</u> ☐ **Yes** ☐ **No**

If your answer is "yes," then please detail the issues below.

8.3 <u>Transportation</u> ☐ **Yes** ☐ **No**

If your answer is "yes," then please detail the issues below.

8.4 <u>Erection area</u> ☐ **Yes** ☐ **No**

If your answer is "yes,", then please detail the issues below.

8.5 <u>Deck placement</u> ☐ **Yes** ☐ **No**

If your answer is "yes," then please detail the issues below.

8.6 <u>Long-term maintenance</u> ☐ **Yes** ☐ **No**

If your answer is "yes," then please detail the issues below.

Appendix B: Parametric Results of Minimum Reinforcements

Maximum span achieved by NU girder system for specific height and spacing is plotted in Figure B.1 through Figure B.20.

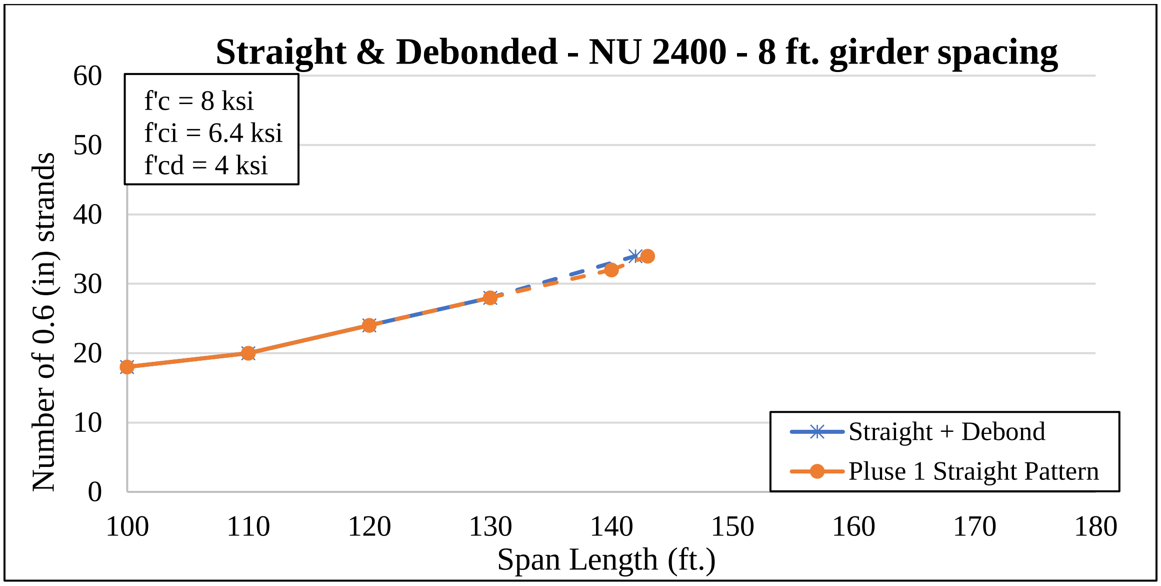


Figure B.1: Comparison of Minimum Reinforcements for NU-2400 with 8-ft Spacing

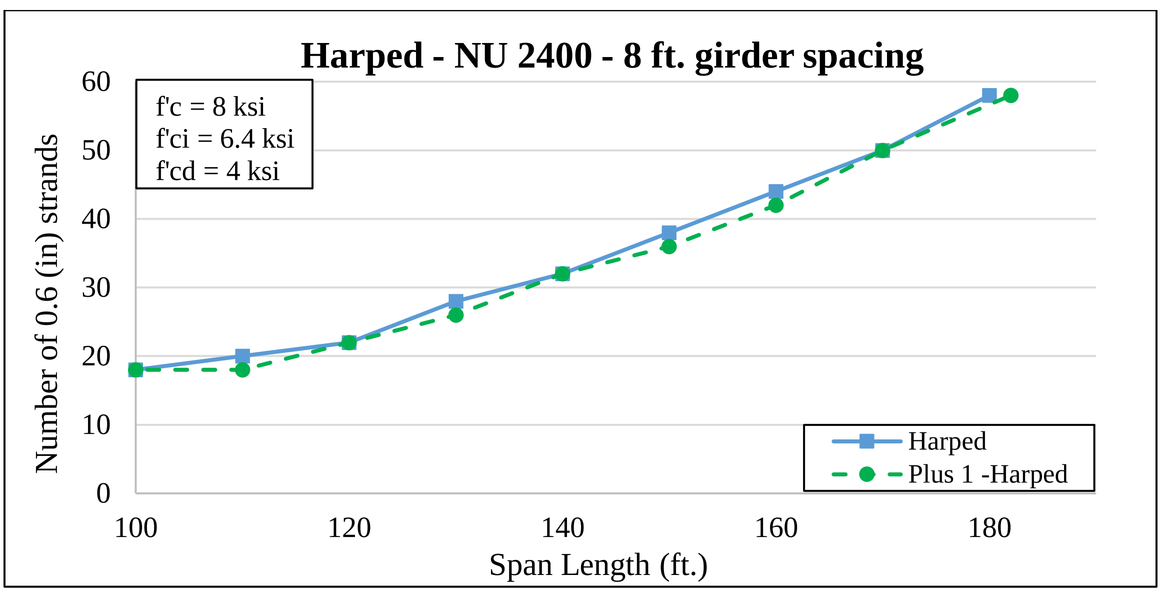


Figure B.2: Comparison of Minimum Reinforcements for Harped NU-2400 with 8-ft Spacing

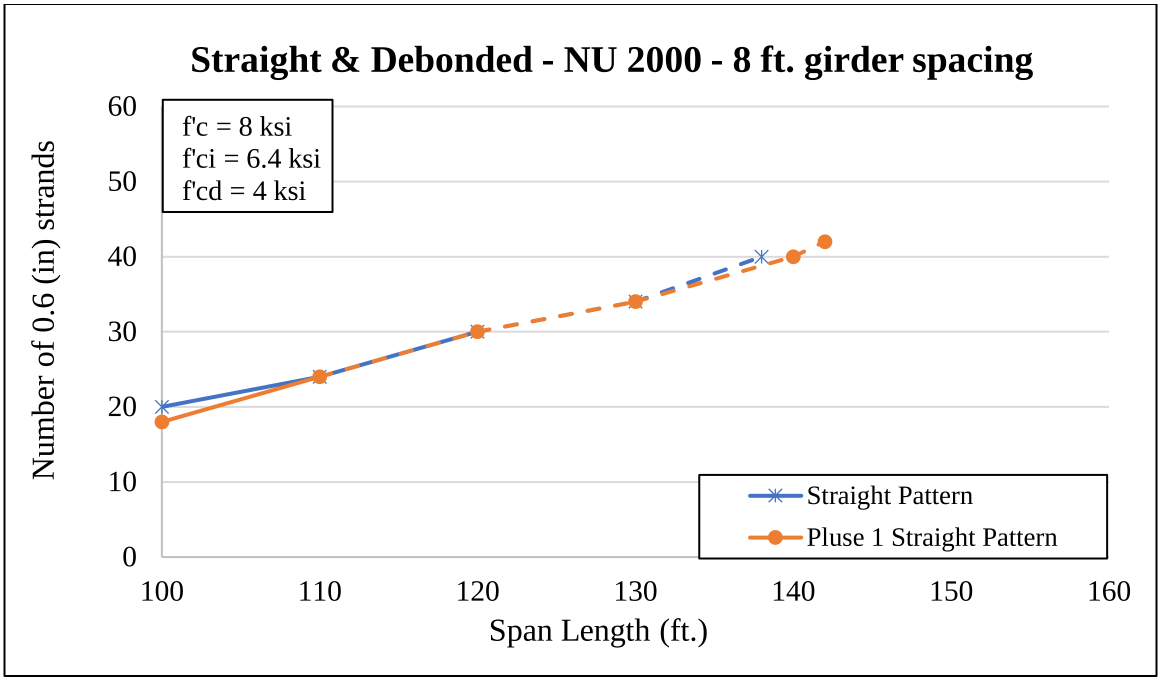


Figure B.3: Comparison of Minimum Reinforcements for NU-2000 with 8-ft Spacing

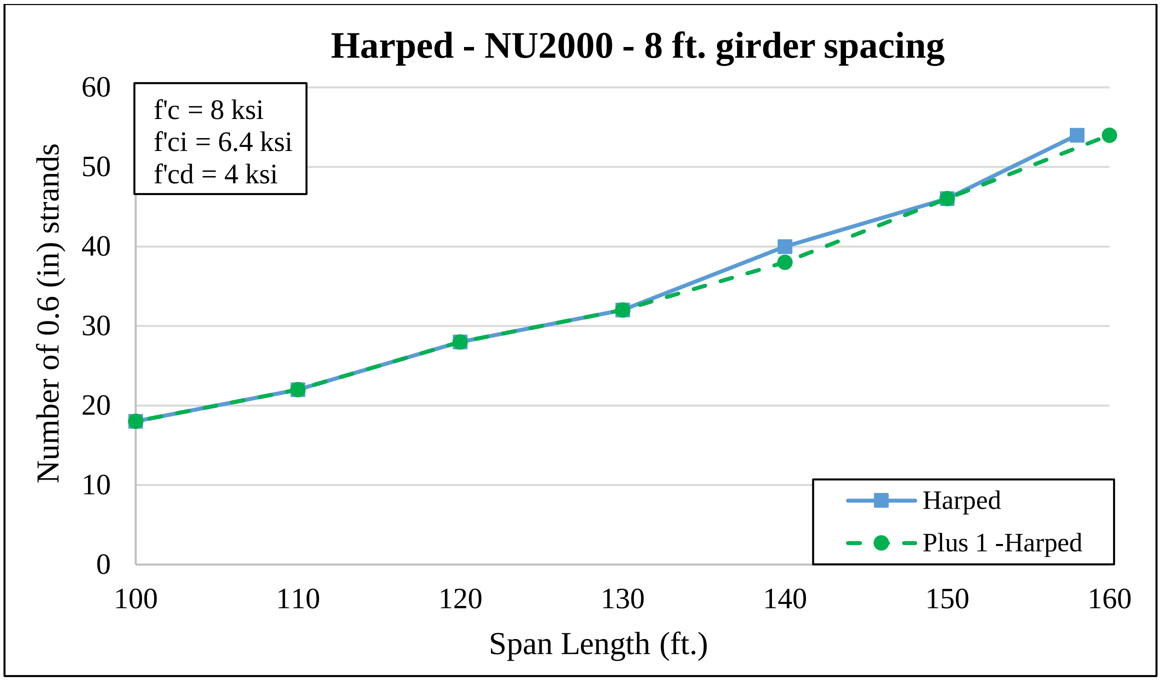


Figure B.4: Comparison of Minimum Reinforcements for Harped NU-2000 with 8-ft Spacing

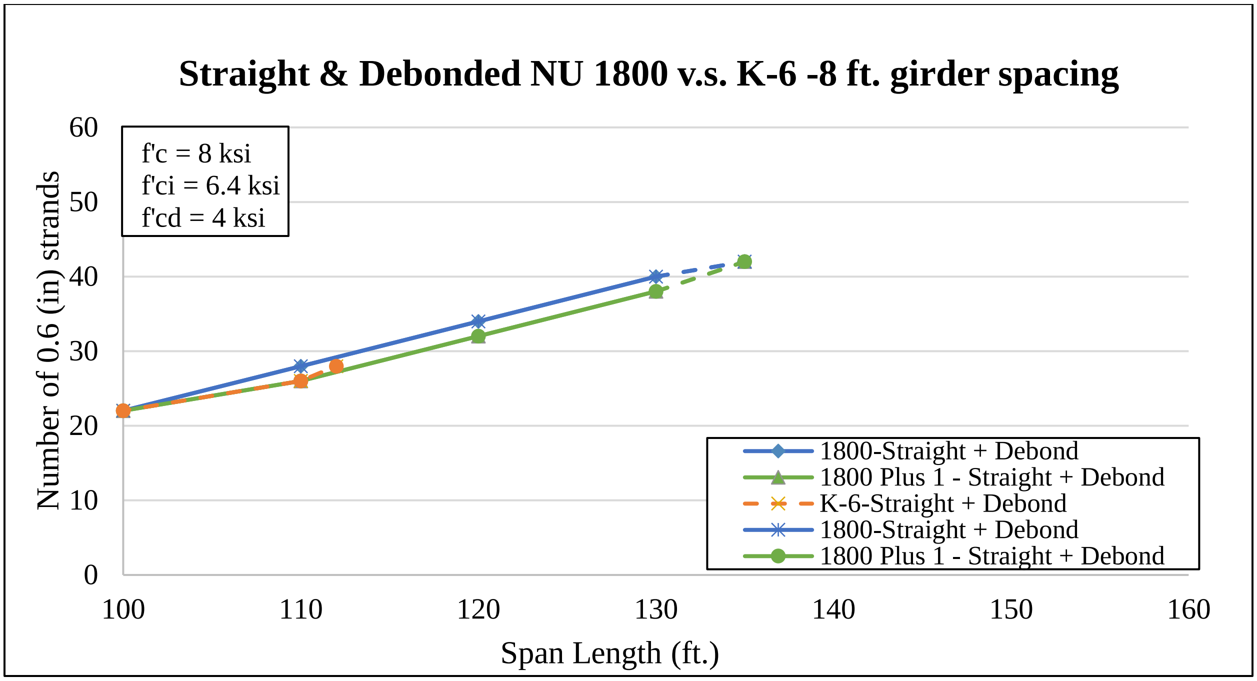


Figure B.5: Comparison of Minimum Reinforcements Between NU-1800 and K-6 with 8-ft Spacing

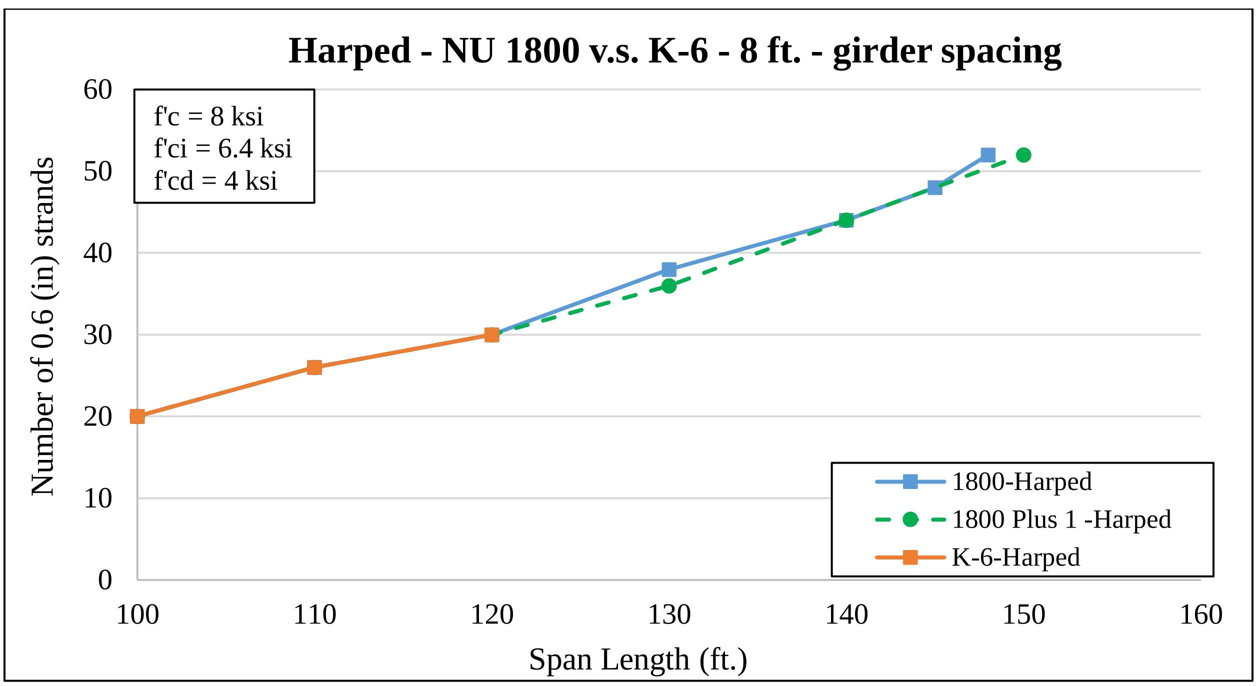


Figure B.6: Comparison of Minimum Reinforcements, Harped Design, Between NU-1800 and K-6 with 8-ft Spacing

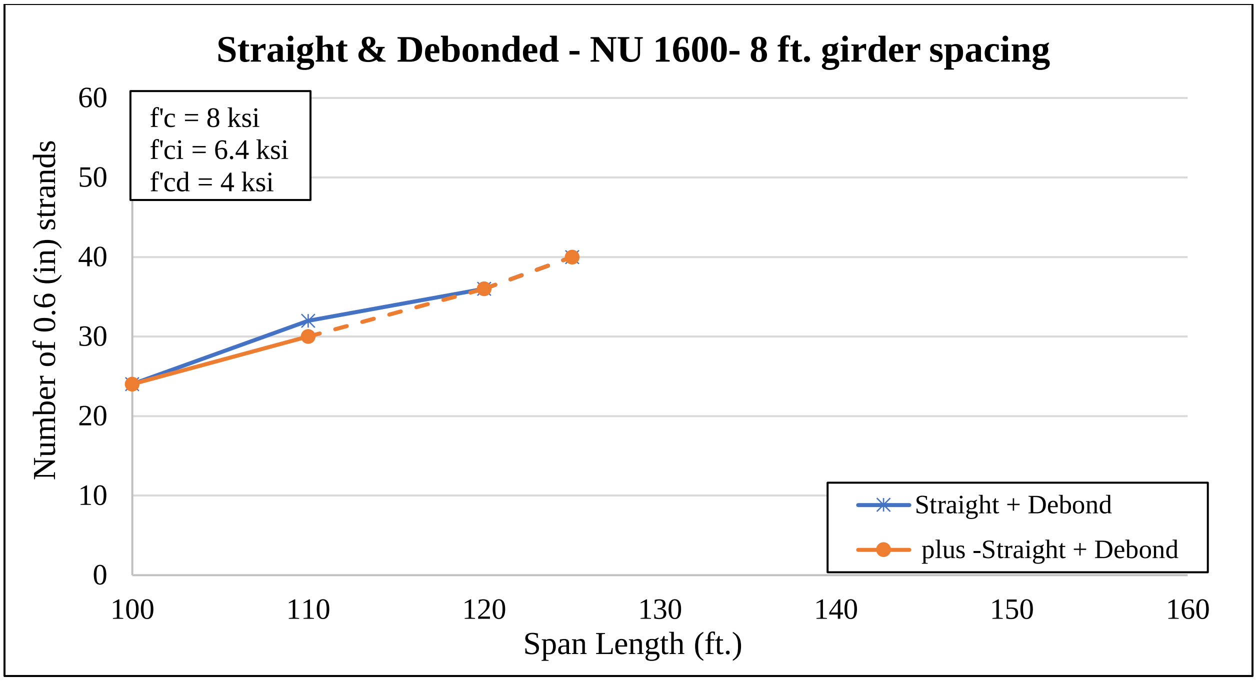


Figure B.7: Comparison of Minimum Reinforcements for NU-1600 with 8-ft Spacing

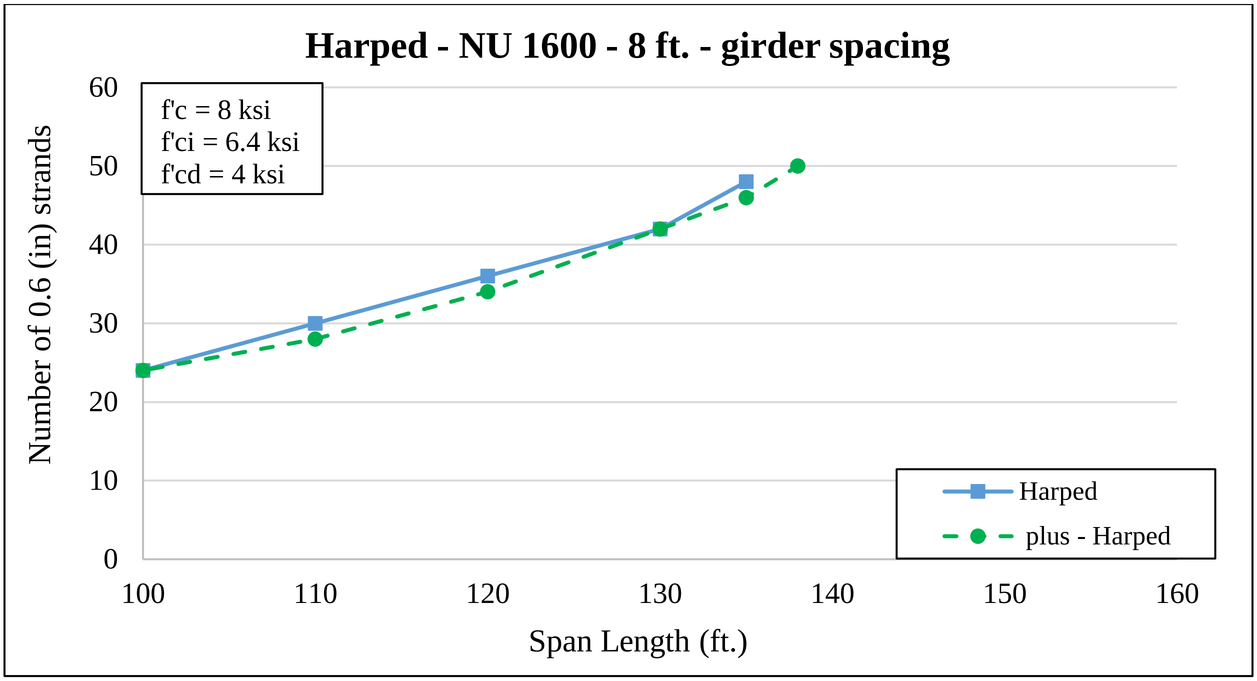


Figure B.8: Comparison of Minimum Reinforcements for Harped NU-1600 with 8-ft Spacing

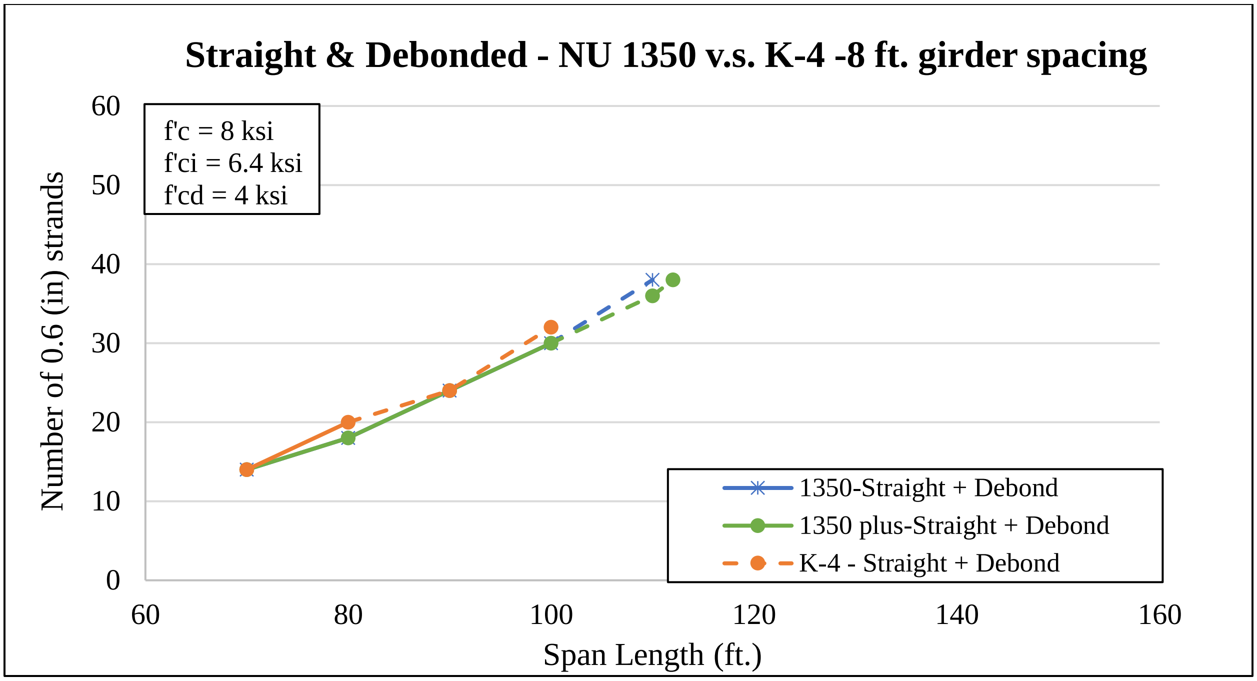


Figure B.9: Comparison of Minimum Reinforcements Between NU-1350 and K-4 with 8-ft Spacing

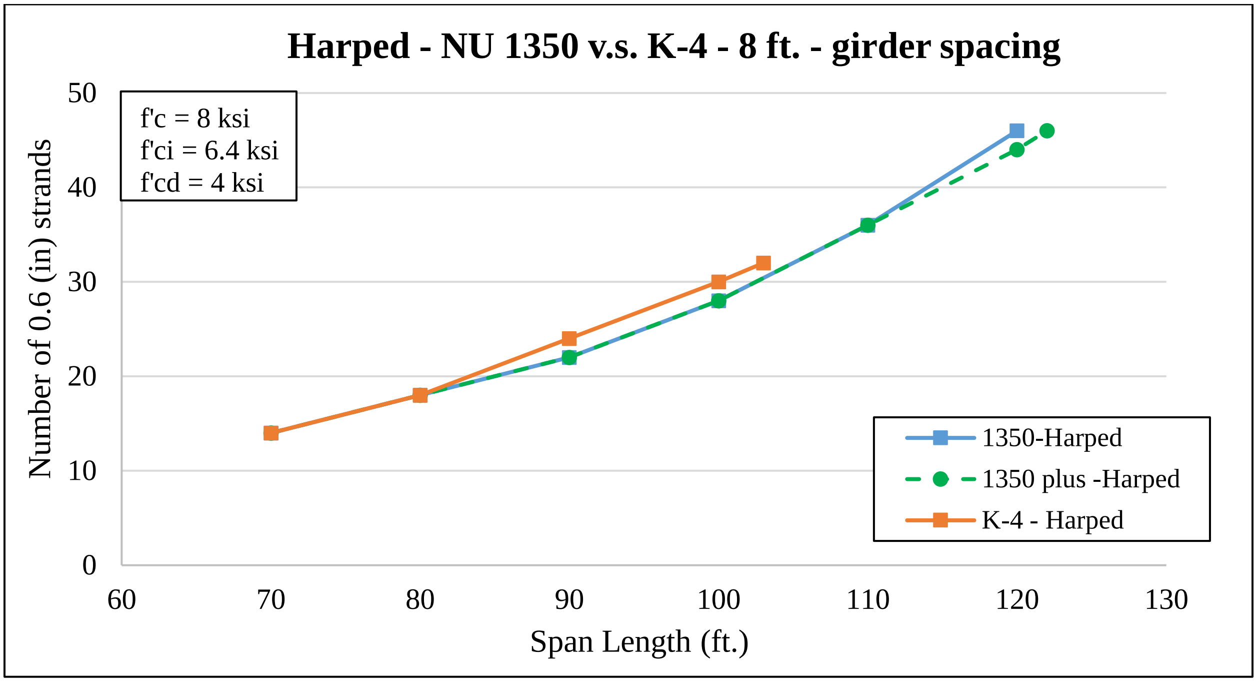


Figure B.10: Comparison of Minimum Reinforcements, Harped Design, Between NU-1350 and K-4 with 8-ft Spacing

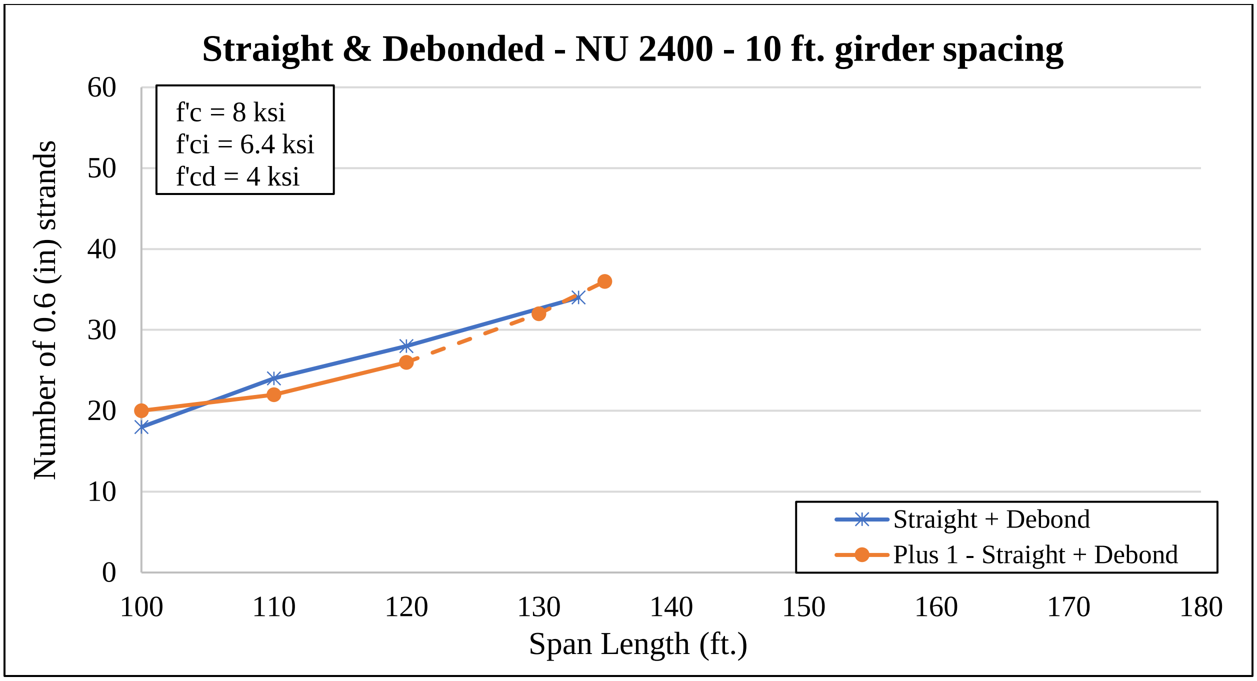


Figure B.11: Comparison of Minimum Reinforcements for NU-2400 with 10-ft Spacing

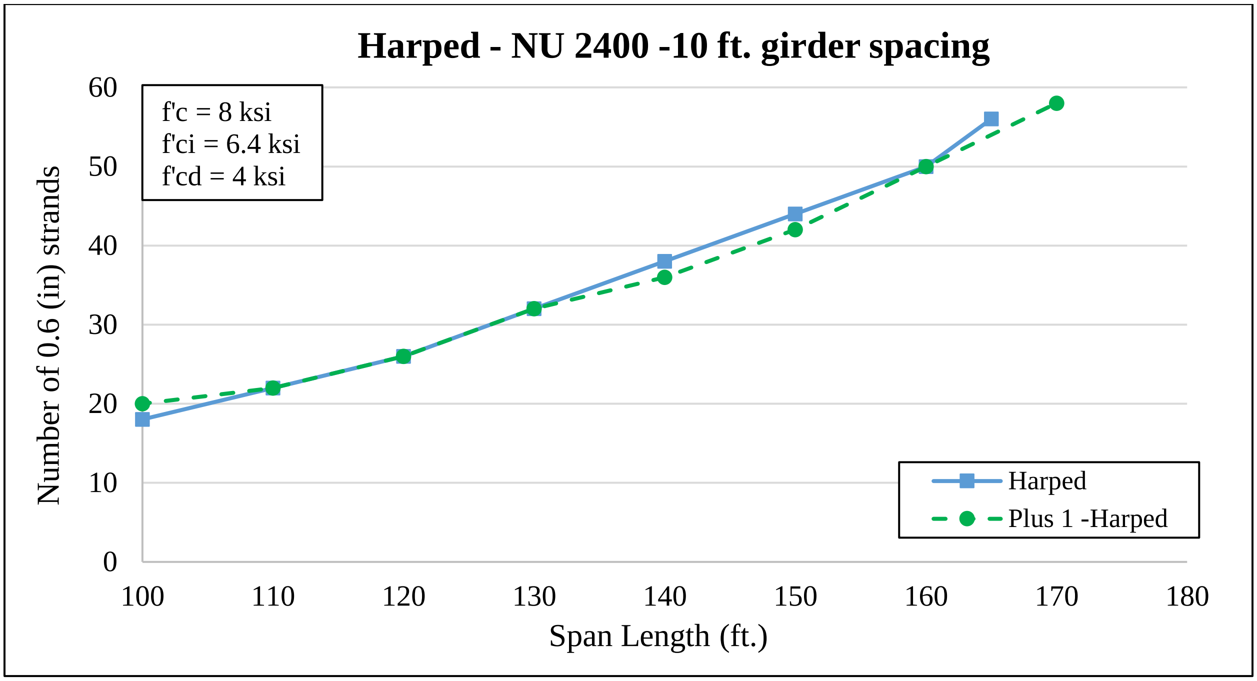


Figure B.12: Comparison of Minimum Reinforcements for Harped NU-2400 with 10-ft Spacing

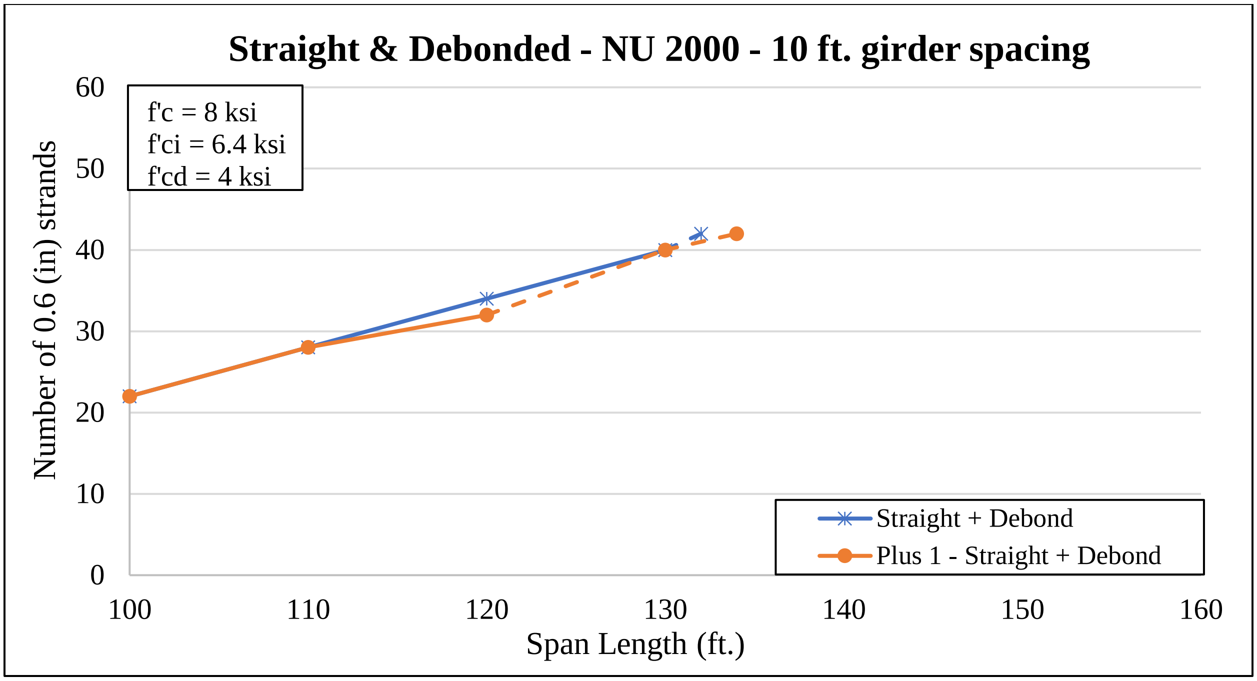


Figure B.13: Comparison of Minimum Reinforcements for NU-2000 with 10-ft Spacing

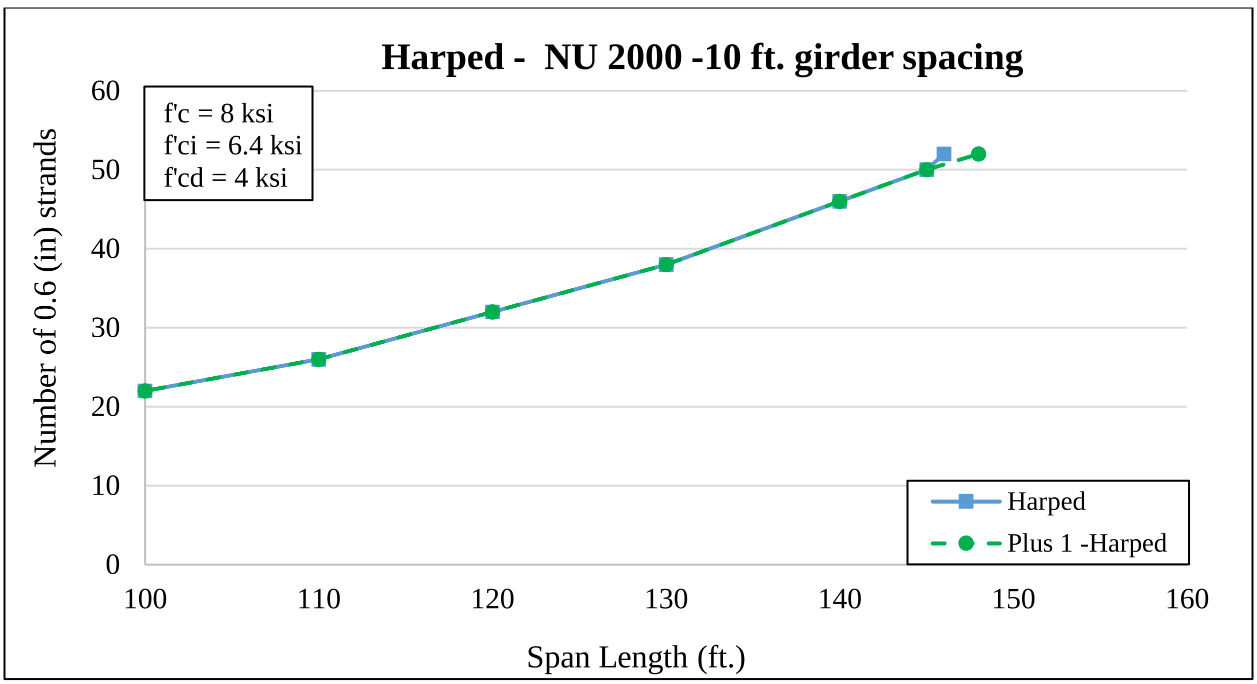


Figure B.14: Comparison of Minimum Reinforcements for Harped NU-2000 with 10-ft Spacing

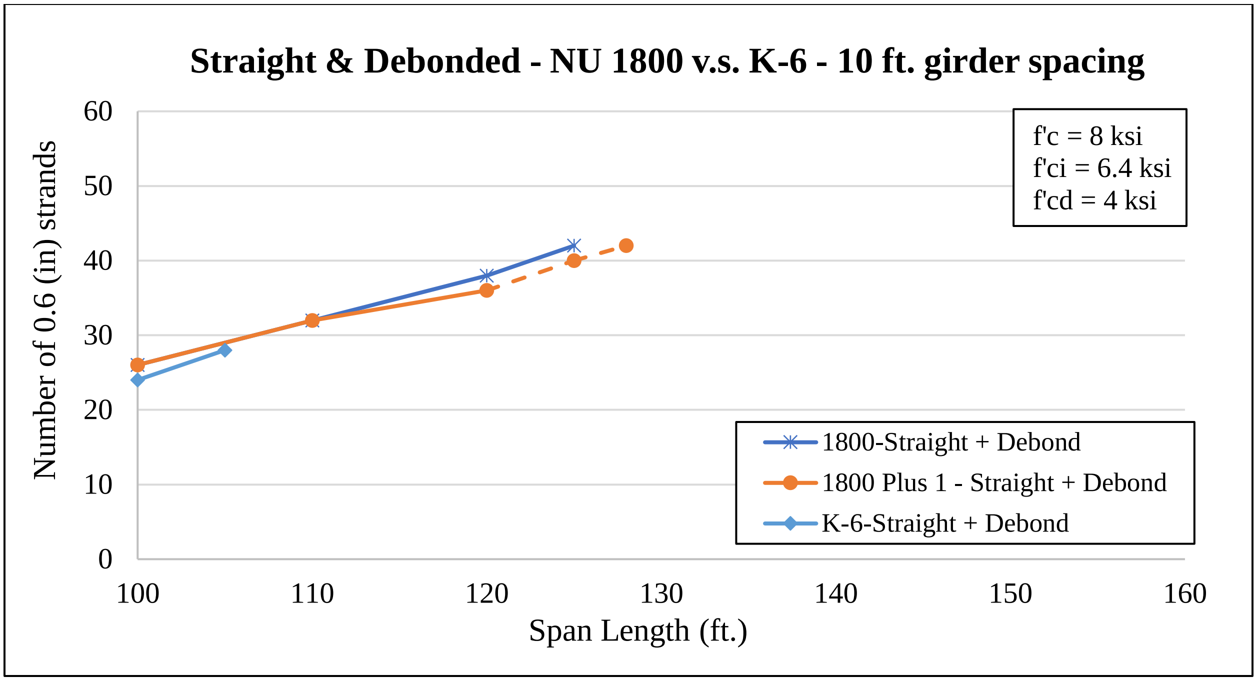


Figure B.15: Comparison of Minimum Reinforcements Between NU-1800 and K-6 with 10-ft Spacing

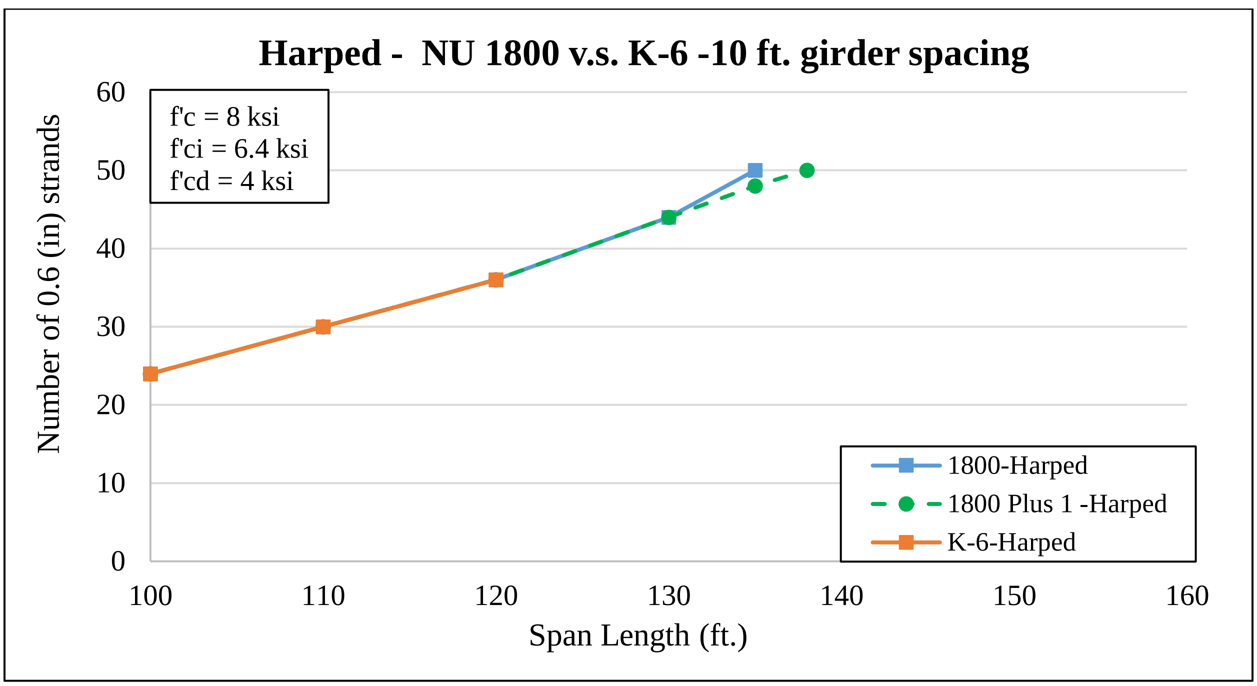


Figure B.16: Comparison of Minimum Reinforcements Between Harped NU-1800 and K-6 with 10-ft Spacing

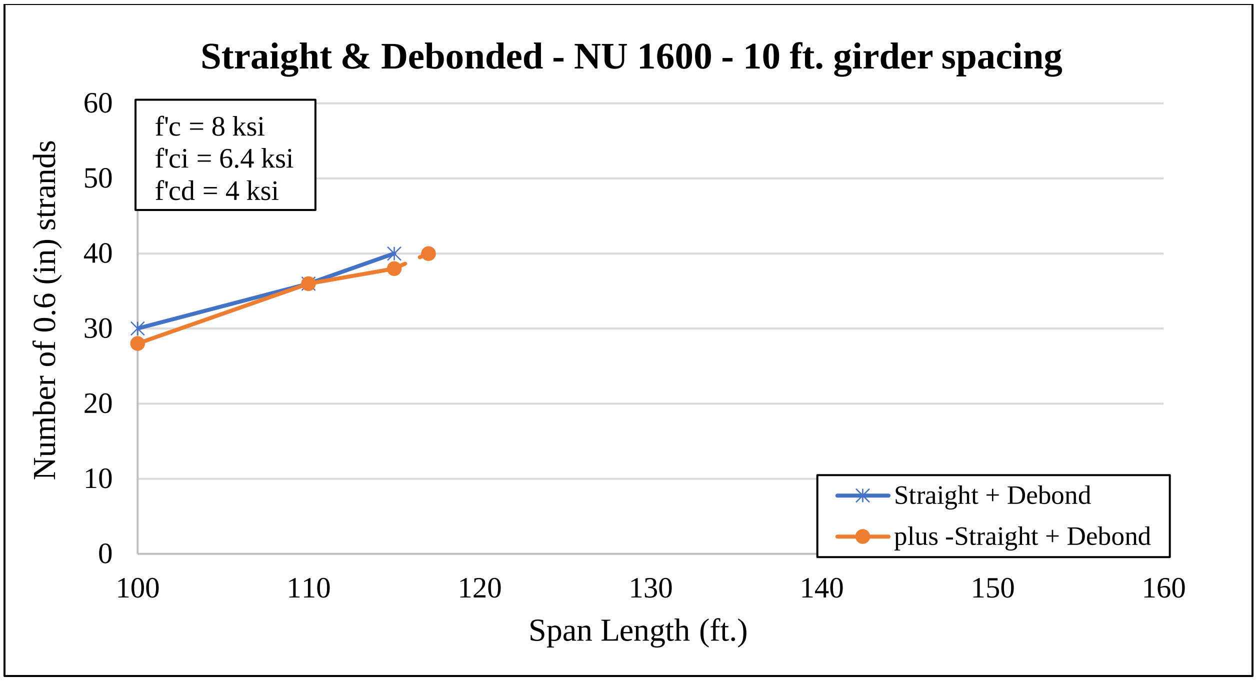


Figure B.17: Comparison of Minimum Reinforcements for NU-1600 with 10-ft Spacing

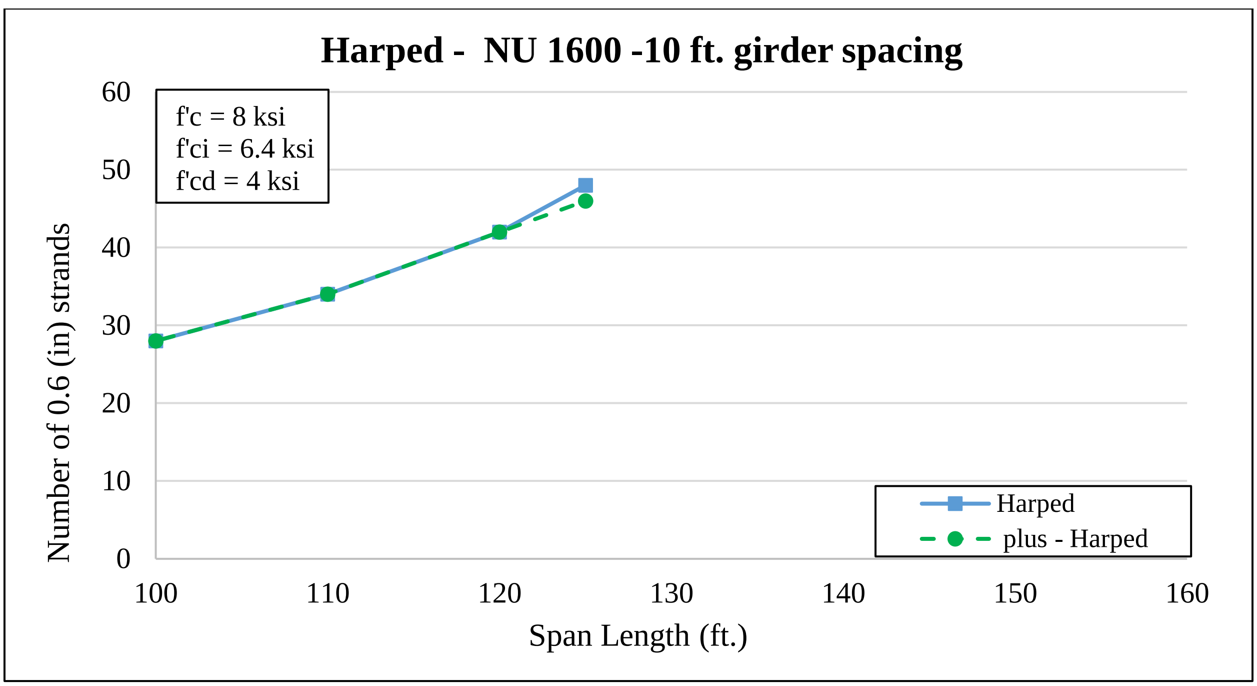


Figure B.18: Comparison of Minimum Reinforcements for Harped NU-1600 with 10-ft Spacing

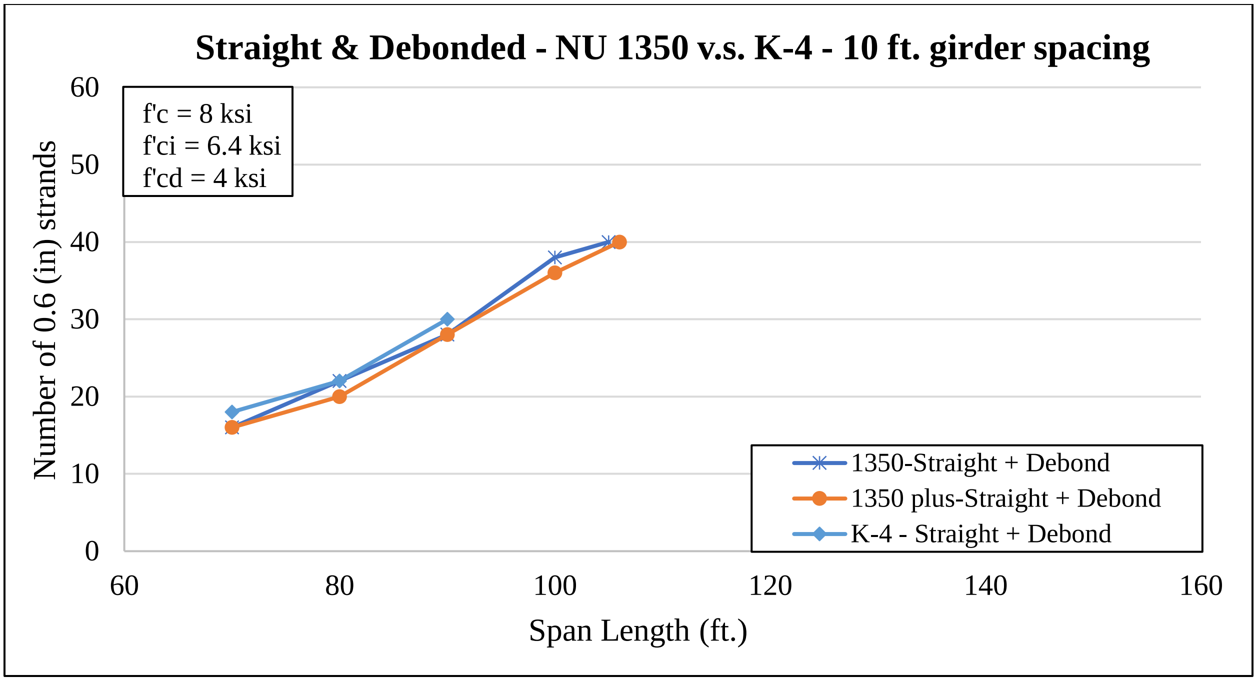


Figure B.19: Comparison of Minimum Reinforcements Between NU-1350 and K-4 with 10-ft Spacing

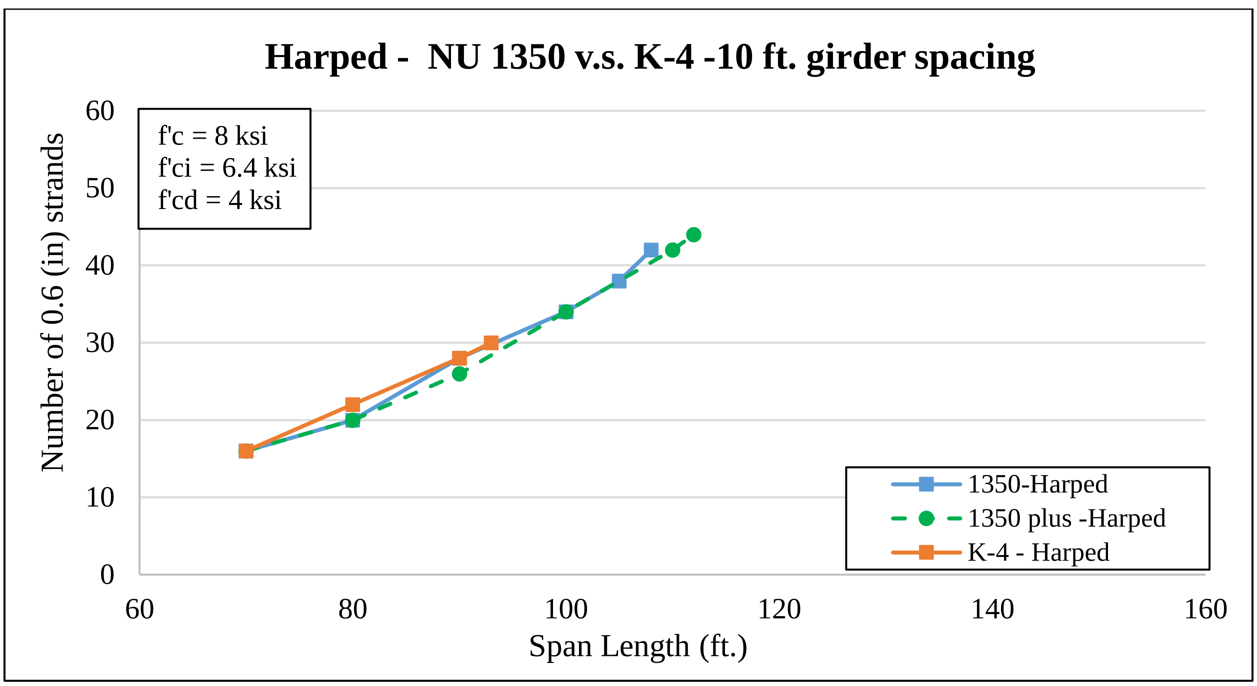


Figure B.20: Comparison of Minimum Reinforcements, Harped Design, Between Harped NU-1350 and K-4 with 10-ft Spacing


AD ASTRA PER ASPERA
Kansas
Department of Transportation