Design Aids for Threaded Rod Precast Prestressed Girder Continuity System

Nebraska Department of Roads

Project Number: STPD-92-7(103)

August 2007
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DRAFT FINAL REPORT

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Sponsored by
Nebraska Department of Roads
University of Nebraska-Lincoln

August 2007
### Abstract
A new continuity system recently introduced in Nebraska has received national attention at meetings of the Federal Highway Administration and the Precast/Prestressed Concrete Institute. It is called Threaded Rod Continuity System. The system allows the girders to be continuous for deck weight. The bridge becomes continuous for about two-thirds of the total load. In this system, the precast concrete I-girders work with high strength threaded rods located above the top flange.

The goal of this research project is to provide bridge designers with the necessary tools for the preliminary design of bridges made continuous for deck weight, using high strength threaded rods. In the research, Clarks Viaduct was reviewed. AASHTO LRFD and NDOR BOPP Manual were studied. The strain compatibility analysis procedure for negative moment was established. This report is divided into eight sections and four appendices. Section 1 is the introduction of TR continuity system. Section 2 illustrates the design criteria. Section 3 presents design steps. Section 4 provides production procedure of design charts. Section 5 is the load table for three-span bridges. Section 6 provides preliminary design example using the charts developed in Section 4. Section 7 is simplified charts. Section 8 is the conclusions and recommendations. Appendix A provides hand calculation example for an interior girder. Appendix B includes the programs used for the service design and negative moment calculations (electronic version). Appendix C includes all the charts and tables required for each design step (electronic version). Appendix D presents maximum moment and shear calculations for live loads (electronic version).
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ABSTRACT

A new continuity system recently introduced in Nebraska has received national attention at meetings of the Federal Highway Administration and the Precast/Prestressed Concrete Institute. It is called Threaded Rod Continuity System. The system allows the girders to be continuous for deck weight. The bridge becomes continuous for about two-thirds of the total load. In this system, the precast concrete I-girders work with high strength threaded rods located above the top flange.

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ACKNOWLEDGEMENTS

This project was sponsored by the Nebraska Department of Roads (NDOR) and the University of Nebraska-Lincoln. The support of Lyman Freemon, Bridge Engineer, Sam Fallaha, Assistant Bridge Engineer, is gratefully acknowledged. They spent a lot of time and effort in coordinating this project, discussing its technical direction, and inspiring the university researchers.

Acknowledgement also goes to Dr. Amgad Girgis, research assistant professor, and my graduate student, Ning Wang.
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SECTION 1: INTRODUCTION

BACKGROUND

A new continuity system recently introduced in Nebraska has received national attention at meetings of the Federal Highway Administration (FHWA) and the Precast/Prestressed Concrete Institute (PCI). It is called Threaded Rod (TR) Continuity System. The system allows the girders to be continuous for deck weight. Thus, the bridge becomes continuous for about two-thirds of the total load. The continuous bridge system experience four stages as described below.

Stage 1: Conventional Bridge Continuity

In this system, girders are continuous for superimposed dead load (SIDL) and live load (LL), as shown in Fig. 1. The capacity of negative section is limited by the maximum amount of deck steel.

![Fig. 1 Stage I Conventional Bridge Continuity](image)

Stage 2: Post-tensioning Continuity System

Post-tensioning Continuity System is continuous for deck weight, SIDL, and LL, as shown in Fig. 2.
Stage 3: First Generation TR Continuity System with TR Embedded in Girder Top Flanges

Led and sponsored by Nebraska Department of Roads (NDOR), the University of Nebraska-Lincoln (UNL) research group developed the TR Continuity System, shown in Fig. 3, with precast girders continuous for deck weight and the entire girder line continuous for SIDL and LL. The construction procedure steps include: embedding TR in girder ends, coupling girders over piers, pouring the diaphragm and placing the deck with the continuity deck reinforcement in it. The system was tested by full-scale specimen testing and applied in the actual bridge design.

Stage 4: Second Generation TR Continuity System with TR Placed above Girder Top Flange

During the application of TR continuity system in practice, a new idea was developed to place TR above the girder top flanges as shown in Fig. 4.
Fig. 4 Stage III New Design Idea with TR above Girder Top Flanges

TR helps increase the fatigue capacity and increase the doable reinforcement area and the price is not much higher than that of bars.

RESEARCH OBJECTIVES

The goal of this research project is to provide bridge designers with the necessary tools for preliminary design of bridges made continuous for deck weight with high strength threaded rods placed above girder top flanges.

SECTION 2: DESIGN CRITERIA

The following criteria supplement the latest edition and interims of AASHTO LRFD Specifications, and the provisions of NDOR BOPP Manual. Where there is a conflict, the criteria below supersede those in AASHTO LRFD Specifications and NDOR BOPP Manual.

1) Moments due to deck weight should be determined by using uncracked section continuous beam analysis. To allow for possible moment redistribution due to negative moment area cracking, the positive moment section should be designed for the values obtained from the analysis above plus an increase corresponding to the possible reduction in negative moment. The increase should be calculated assuming a 10% reduction in the negative moment. This would result in an increase in the
positive moment in interior spans equal to the average drop in the negative moments at the two adjacent supports, and in exterior spans equal to 4% of the negative moment at the adjacent pier support. No reduction should be allowed for the design of the negative moment zone as the redistribution could vary from zero to 10%.

2) This system is considered to be conventionally reinforced in the negative moment zone. As such, strength, steel fatigue, concrete fatigue, as well as crack control of the top surface of the deck must be satisfied. Note that the maximum resistance factor for reinforced concrete, \( \phi = 0.9 \), rather than 1.0 for prestressed concrete.

3) For fatigue stress checks of concrete in compression and of steel in tension, the fatigue truck loading specified by AASHTO LRFD, Interim 2006, which was approved by AASHTO-T10 in Nov. 2005, should be used.

4) For fatigue stress check, use the stress limits given in AASHTO LRFD for concrete, mild reinforcing bars, welded wire reinforcement and strand. Grade 150 steel stress limit is currently not covered by AASHTO LRFD. Based in previous research, the following criteria are conservative: Maximum stress due to dead loads only \( f_{\text{min}} = 54 \) ksi and maximum stress range due to live load \( f_r = 36 - f_{\text{min}} / 3 \).

5) The live load deflection limits in the AASHTO LRFD Specifications are required to be satisfied for this system.

6) Crack control of the deck slab reinforcement is the same as given in the LRFD Specs.

7) Maximum amount of threaded rod to be used in the pier area for this system is the equivalent of 10-1 3/8” Grade 150 rods (or 15.8 in\(^2\)). This limit may be increased in the future as more research and full-scale testing indicates.

8) Service I live load factor is 2.0.
**Cast-in-place Haunch Standard:**

The UNL research group conducted three full-scale specimen tests. It helps the group to standardize the cast-in-place haunch size, shown as the following pictures.

![Diagram of threaded rods layout on girder top design detail](image)

**Fig. 5 Threaded Rods Layout on Girder Top Design Detail**

Note: shear reinforcement may be replaced with vertical TR as desired.

The maximum number of 1 3/8” diameter threaded rods which can be put above the girder top is 15. There is 0.75 in. clear spacing below the TR from the girder top flange.

Because no testing with bundled rods has been conducted, the author suggests to use 10-1 3/8” diameter or 6- 1 3/4” diameter unbundled rods as the maximum amount of TR before more testing is made to adjust it.
SECTION 3: DESIGN STEPS

This section gives users brief guidance for bridge design. It includes the following eight steps:

Step 1 Bridge information

Step 2 Load calculation

Step 3 Determine number of strands
Step 4 Determine TR area
Step 5 Determine deck bar area
Step 6 Determine $f_c$
Step 7 Check fatigue limit and crack control
Step 8 Calculate deflection due to LL in Service

**STEP 1: BRIDGE INFORMATION**

Collect the necessary information for design, including bridge width, span length, girder spacing, and the load information. Design should follow AASHTO LRFD Specifications Interim 2005 and NDOR BOPP Manual 2005.

**Bridge Layout**

Unless otherwise specified, the roadway part of the overhang, $d_e$, does not exceed 3 ft. (LRFD 4.6.2.2.1). The distance from girder center to the outside edge of the deck should be less than 4.58 ft, which is equal to the sum of 3’ (roadway width), 5.9”/2 (half of girder web), 14” (barrier width) and 2” (slab edge). Normally, the overhang width that can be used for preliminary design is $0.45S$ ($S = $ Spacing). The bridge width is roughly equal to $0.45S + 0.45S + (N-1) \times S$. Therefore the girder line number, $N$, is equal to the bridge width divided by $S$. 
Critical Section of Negative Moment Area

The critical section near the pier is at the face of the diaphragm. Normally the diaphragm is 24” wide, with 8” gap between two girders, 8” overlap with each girder end. Due to New TR Specimen Testing with 3 ft wide diaphragm, all the calculations are updated according to 3 ft wide diaphragm. The transfer length of strands is $60d_b = 60 \times 0.6 = 36”$. Assume the effective prestress is 160 ksi at the transfer length, therefore the prestress at the face of the diaphragm is $8/36 \times 160 = 36.56$ ksi. The haunch is 3 in. at the negative
section (actually it is from 2.5” to 3.5”) by assuming 2” camber.

Fig. 10 Diaphragm at the Pier

Girder Information

The section properties of the standard NU I-girders are shown in Table 1. The cross section may be changed according to the actual need in design, such as adding 3 in. extra haunch to the top flange, widening the web. Then, the section properties should be modified accordingly.

Table 1 NU-I Girder Section Properties

<table>
<thead>
<tr>
<th></th>
<th>NU 900</th>
<th>NU 1100</th>
<th>NU 1350</th>
<th>NU 1600</th>
<th>NU 1800</th>
<th>NU 2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>h (in)</td>
<td>35.4</td>
<td>43.3</td>
<td>53.1</td>
<td>63</td>
<td>70.9</td>
<td>78.7</td>
</tr>
<tr>
<td>A (in²)</td>
<td>648.1</td>
<td>694.6</td>
<td>752.7</td>
<td>810.8</td>
<td>857.3</td>
<td>903.8</td>
</tr>
<tr>
<td>I (in⁴)</td>
<td>110262</td>
<td>182279</td>
<td>302334</td>
<td>458482</td>
<td>611328</td>
<td>790592</td>
</tr>
<tr>
<td>Y_b (in)</td>
<td>16.1</td>
<td>19.6</td>
<td>24</td>
<td>28.4</td>
<td>32</td>
<td>35.7</td>
</tr>
<tr>
<td>W_g (kip/ft)</td>
<td>0.68</td>
<td>0.724</td>
<td>0.785</td>
<td>0.84</td>
<td>0.894</td>
<td>0.942</td>
</tr>
</tbody>
</table>
**Strands**

In prestressed NU I-girders, 4-0.5” strands are always put at 1.75” from the top fiber. The prestressing force is 2.02 kips per strands. The max number of strands used here is 60. In the calculation, assume none of the strands are draped and 0.4L is where positive section analysis is considered. Also, it is conservative for the negative section analysis without considering draping.

**STEP 2: LOAD CALCULATION**

Then the load table should be built. Determine the moment and shear diagrams for various loading cases with the aid of computer programs like Risa 3D and Conspan. The approximate equation for live load calculation is made by the author (Refer to Appendix D). Use uncracked section properties for member stiffness. Modify the deck weight moment diagram by shifting the entire positive moment diagram vertically by the amount stated in Criterion (1). Corresponding changes in shear are not warranted. Positive moment should first be designed as in other prestressed girders.

The load values conform to LRFD 2005 except Service I live load factor is 2 instead of 1.75 as required by Nebraska. Girder weight works on a simple supported span; deck weight works on continuous spans with a reduction due to 10% drop of the negative moment at pier; rail weight, future wearing surface and live load work on continuous spans. In fatigue load calculation, one truck load with 30 ft spacing between axles is used and the load is multiplied by 1.15 dynamic load factor, 1.5 load factor on live load effect (2*0.75). The two factors should always been used in working stress design as specified.
in AASHTO 2005 Interim. Girder weight and deck weight including haunch works on the precast section. Superimposed dead load (rail and barrier weight plus future wearing surface) and live load work on the composite section. Live load includes Service I live load and fatigue live load. Service I live load includes single truck plus lane load, double trucks, single tandem and double tandem. (3.6.1.2.2)

Dead load is obtained through structural analysis. In the design chart, barrier load is taken as 382 lb/ft/per side. So the barrier weight per girder takes is equal to 0.382×2/girder number. Stay-in-place Forms are taken as 5 lb/ft². The Future Wearing Surface is 25 lb/ft.

Use fatigue truck loading to calculate the threaded rod and deck bar stresses, and the bottom fiber concrete stress. The fatigue truck has a 30 ft fixed distance between two 32 kips axles. There is no distributed lane loading required to be added to the truck loading. The load factor is 0.75×2 = 1.5, representing an infinite life check. The dynamic allowance factor is 1.15 (not 1.33). Only one truck is to be used and one lane loaded distribution factor as given in the LRFD Specifications should be applied. Designers are reminded that multiple presence factor of 1.2, for one-lane-only loaded, is already included in the approximate formulas for distribution factor given in the LRFD Specifications.

The positive section analysis is at 0.4L and negative section analysis is at the face of diaphragm. The load combination factor is:

Service III moment: DL + 0.8(Service I LL)
Fatigue LL multiplied by 1.5

Strength I load at precast section: 1.25(Girder weight + Deck weight)

Strength I load at composite section: 1.25(Girder weight + Deck weight + Rail) + 1.5(FWS) + 2 LL

STEP 3: DETERMINE NUMBER OF STRANDS

The total number of strands is obtained by checking Service III requirement using the NCHRP 18-07 detailed prestress loss method. The critical section is 0.4L from the abutment, where the bottom fiber tension stress named Service III (use Service I LL) and girder top compression fiber named Girder Top Fatigue (use Fatigue load) are calculated. All load factors are equal to 1.0 for this problem, except that the Service I live load is reduced by a factor of 0.8 in Service III checking. Fatigue live load already includes a 1.5 load factor. The transformed section and net section properties are used in the calculation.

The detailed procedure is to get the strand number decided by Service III first with given $f'_c$. Then concrete fatigue is checked. Revise $f'_c$ if needed and check flexural strength at Strength I. Revise the strand number if Strength I requirement cannot be met. Then Strength I at precast section is checked. Normally, it does not control the design. Also, the strand number should meet the minimum reinforcement requirement (refer to Appendix B for details). After Service III checking, fatigue checking is needed at beam compression fiber.
STEP 4: DETERMINE TR AREA

Use Strength I moment due to loading combination just after deck placement and use precast section only to determine required threaded rod area, $A_{tr}$, and the required diaphragm concrete strength at the time of deck placement. Although the section over the pier centerline will likely control in this step, it would be advisable to check the section at the face of the diaphragm and the section at transfer length away from the pier. Select the number of 1 3/8” or 1 3/4” rods to give equal or higher value than $A_{tr}$. The negative moment section at pier is analyzed as reinforced section. Strength I requirement ($\phi M_u \geq M_u$) at precast section should be met at design.

<table>
<thead>
<tr>
<th>Nominal TR Diameter (in.)</th>
<th>Net Area (in^2)</th>
<th>Ultimate Strength (kips)</th>
<th>Yield Strength (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.85</td>
<td>127.5</td>
<td>102</td>
</tr>
<tr>
<td>1.25</td>
<td>1.25</td>
<td>187.5</td>
<td>150</td>
</tr>
<tr>
<td>1.375</td>
<td>1.58</td>
<td>237</td>
<td>189.6</td>
</tr>
<tr>
<td>1.75</td>
<td>2.6</td>
<td>400</td>
<td>320</td>
</tr>
<tr>
<td>2.5</td>
<td>5.19</td>
<td>778</td>
<td>622.4</td>
</tr>
</tbody>
</table>

The length of threaded rod is cut off with a development length beyond the location where it is required for the moment due to combined deck weight and girder weight.
STEP 5: DETERMINE DECK BAR AREA

Strength I requirement \( (\phi M_u \geq M_u) \) at composite section should be met at design. First, try to increase deck area to get the requirement satisfied. When area is increased to the maximum value, TR area has to be increased. Then, \( f' \) is increased if necessary. The length of deck bars is extended to provide for adequate capacity at all sections due to full Strength I loading.

The detailed procedure is: use Strength I for full loads, and the value of \( A_{tr} \) from Step 4, to determine the area of bars in the deck, \( A_s \). The maximum area of steel in the deck should correspond to 1#5 plus 2#8 in the bottom mat, and 1#4 plus 2#8 in the top mat per foot. If that area of steel is not adequate, gradually increase the threaded rods to a maximum amount of 10-1 3/8” diameter TR until the capacity is reached. If the capacity cannot be reached with maximum threaded rod and deck bars, the section depth is not adequate. In some cases, low concrete strength could result in low \( \phi \). This could be overcome by increasing the concrete strength up to the practical limit, currently to 11 ksi, or by adding compression reinforcement in the compression zone.

Deck reinforcement area

Actually, the maximum bar size can be #9 as shown in Fig. 11.

![Fig. 11 Maximum Deck Bar Size](image_url)
LRFD Section 9.7.2.5 requires that minimum deck reinforcement is #4 @ 12” placed on the top layer and #5 @ 12” placed on the bottom layer. One or two #5, #6, #7 or #8 bar can be placed between each bar. The maximum deck bar area depends on girder spacing. The following table is the total deck reinforcement area varying with girder spacing.

Table 3 Total Deck Reinforcement Area

<table>
<thead>
<tr>
<th>Bar inserted between Min deck steel</th>
<th>Girder Spacing, ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girder Spacing, ft</td>
</tr>
<tr>
<td>Size</td>
<td>6</td>
</tr>
<tr>
<td>0</td>
<td>3.06</td>
</tr>
<tr>
<td>#5</td>
<td>6.78</td>
</tr>
<tr>
<td>2#5</td>
<td>10.5</td>
</tr>
<tr>
<td>#6</td>
<td>8.34</td>
</tr>
<tr>
<td>2#6</td>
<td>13.62</td>
</tr>
<tr>
<td>#7</td>
<td>10.26</td>
</tr>
<tr>
<td>2#7</td>
<td>17.46</td>
</tr>
<tr>
<td>#8</td>
<td>12.54</td>
</tr>
<tr>
<td>2#8</td>
<td>22.02</td>
</tr>
</tbody>
</table>

STEP 6: DETERMINE $f_c$.

In other positive sections from girder end to 0.4L, if the concrete strength is not adequate to meet Strength I and fatigue, debond some of the straight strands in the bottom flange to satisfy the positive moment section conditions. If debonding (up to LRFD limits of 25%
total and 40% per row) does not reduce $f_e$ to an acceptable level, the section depth should be increased, a steel plate placed in the bottom of the section, or compression bar used in the bottom flange, if room is available for such bars. The effective prestress may be assumed the same as in the positive moment section and assumed to gradually develop over 60 strand diameters from girder end.

Increase $f_e$ in each design step to meet the relative requirement. The required $f_e$ will be obtained after negative section design. It may be increased by the following service checking.

**STEP 7 CHECK FATIGUE LIMIT AND CRACK CONTROL**

**Fatigue Limit**

Service design criteria require that fatigue limit for steel and concrete should be met and crack control limit should be met. The fatigue requirement includes concrete fatigue checking and steel fatigue checking.

Fatigue for concrete, i.e., girder compression fiber stress:

$$0.5(f_{DL} + f_{eff. prestress}) + f_{fatigue LL} \leq 0.4f_c$$

Deck bar stress due to fatigue Live load moment, $f_r \leq 24 - 0.33 f_{min}$ ksi

Because there are not enough previous experiments, the author tried to use the information for fatigue in reinforcing bars. The author is testing Grade 150 ksi threaded rod fatigue assuming that the formula $f_r = 36 - f_{min} / 3$ will give conservative fatigue limits. It allows us to use the results for the research project and for an actual bridge. The
author is going one step further. The stress range set in the machine is going to be the limit used in bridge design, as long as a great majority of the specimens do not break under the stress range being identified. In the consideration of TR fatigue criteria, TR should not get to the yield point in fatigue loading. Therefore \( f_{\text{min}} + f_r \leq 120 \text{ ksi} \). Using a safety factor of 1.2, the author gets \( f_{\text{min}} + f_r \leq 100 \text{ ksi} \). The author is testing three fatigue group data by using five million cycles as the limit defining endurance.

<table>
<thead>
<tr>
<th>( f_{\text{min}} ) (ksi)</th>
<th>( f_r ) (ksi)</th>
<th>( (f_{\text{min}}+f_r)/f_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>54</td>
<td>18</td>
<td>0.60</td>
</tr>
<tr>
<td>72</td>
<td>12</td>
<td>0.70</td>
</tr>
<tr>
<td>96</td>
<td>4</td>
<td>0.83</td>
</tr>
</tbody>
</table>

Table 4 TR Fatigue Testing Data

For the current project of TR aid project, it is conservative to get \( f_r = 36 - f_{\text{min}}/3 \), \( f_{\text{min}} \leq 54 \text{ ksi} \).

Use precast cracked section analysis for girder weight plus deck loads, and composite section analysis for superimposed dead loads and fatigue live load. If the limit is exceeded, gradually increase the rod area up to the maximum limit (10-1 3/8” diameter) or increase the concrete strength.

After all the fatigue criteria are checked, the reinforcement should meet the minimum requirement (Refer to Appendix B for details).
Crack Control

Use the information from Service I analysis above to check crack control requirements as given in the LRFD Specifications (Art. 5.7.3.4):

\[ s \leq \frac{700 \gamma_c}{\beta_s f_s} - 2d_c < (1.5 \text{ slab thickness}) \text{ or } 18, \text{ whichever is smaller.} \]

\( d_c = \) thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto, normally 2.5 clear cover + 0.5 for transverse reinforcement +0.5 (radius of #8) = 3.5” for typical design in Nebraska with 2.5” clear concrete cover.

\( h = \) overall thickness or depth of the component. Use conservative value, \( h=7.5”(\text{deck})+1”(\text{haunch})+43.3”(\text{height of NU750})=38” \)

\( \beta_s = \) ratio of distance between top fibers and neutral axis to distance between top layer of steel and neutral axis. \( \beta_s = 1 + \frac{d_c}{0.7(h - d_c)} = 1 + \frac{3.5}{0.7(38 - 3.5)} = 1.14 \) This can be conservatively taken here as \( \beta_s = 1.2 \). \( f_s = \) tensile stress in top layer of rebar at Service I, and \( \gamma_c = \) exposure factor, taken here = 0.75 for class 2 when there is increased concern of appearance and/or corrosion. When these constants are used:

\[ s \leq \frac{700 \times 0.75}{1.2 \times f_s} - 2(3.5) \]

\[ = \frac{437.5}{f_s} - 7 \]

\(< 1.5 \text{ slab thickness or } 18”. \)

Accordingly, this criterion is very unlikely to control the design.
STEP 8: CALCULATE DEFLECTION DUE TO LL IN SERVICE

Meeting the deflection limit recommended by AASHTO LRFD is good in a bridge design although AASHTO only suggests it. Users are suggested to check deflection, which must be less than span/800 for vehicular traffic according to criteria in AASHTO 2.5.2.6.2.

When investigating the maximum absolute deflection, all design lanes should be loaded, and all supporting components should be assumed to deflect equally (criteria for deflection, AASHTO 2.5.2.6.2). For composite design, the stiffness of the design cross-section used for the determination of deflection should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers. For a straight girder system, the composite bending stiffness of an individual girder may be taken as the stiffness determined as specified above, divided by the number of girders. When investigating maximum relative displacement, the number and position of loaded lanes should be selected to provide the worst different effect (AASHTO 3.6.1.3.2). The load for live load deflection is the larger of that resulting from the design truck alone or that resulting from 25% of the design truck taken together with the design lane load. In deflection calculation, always use positive live load.

After searching on the comparison between the loads above and Service I live load, the author found that the former is always less than the latter. To simplify the design, a conservative approach for deflection estimation is to use the positive moment live load
envelope per lane developed for Service I. This envelope should be adjusted for two
effects: (a) distribution factor = No. of lanes / No. of girder lines, and (b) multiple
presence factor = 1.2, 1.0, 0.85, or 0.65 for one, two and three, or more than three lanes
loaded. This approach would avoid the special loading for deflection specified in
AASHTO 3.6.1.3.2, and gives conservative results. Hence the Distribution Factor for live
load of one lane loaded is (N of lanes)/(N of girders)×(Multiple Presence Factor).

SECTION 4: DESIGN CHARTS
This section gives the details of the making procedure for the charts and tables in
Appendix A. The designed bridge is a two span continuous bridge. The bridge width is
46.67 ft. The spacing is 6, 8, 10 and 12 ft. The spans are 80, 100, 120, 140, 160, 180 and
200 ft. The girder concrete strength is 8, 9, 10 and 11 ksi, and the deck concrete is 4 ksi.
In the design, structural thickness of 7 in. is used in the analysis. In the calculation of
deck weight, 7.5 in. is used. The haunch is 1 in. at positive section and 3 in at negative
section.

STEP 1: LOAD (APPENDIX C, SHEET 1)
The load tables give all loads needed for positive section and negative section at the face
of the diaphragm, including shear load, moment for deflection calculation and moment
for Service and Strength Design. Users should pick the number needed for each step of
analysis. Normally, for preliminary design, 0.4L location and diaphragm face are the two
locations that need to be considered.
STEP 2: SERVICE DESIGN AT POSITIVE SECTION (APPENDIX C, SHEET 2)

From the analysis, the author found that strand number required in Service III checking is not sensitive to concrete strength. The relationship between Total Service III moment and strand number is almost on one line for each girder spacing. See 9 ksi concrete group for example (see Charts 2-1 to 2-6). Therefore, for each girder size, the data for different girder spacing can be drawn on one line (see Chart 2-7). The minimum reinforcement needs to be checked if the strand number needed is less than the number noted on the charts. The strands number should be equal to or larger than the minimum required value.

The service design tables give maximum fatigue load and the actual fatigue load. Users can calculate the fatigue load or pick it from the table and compare with the maximum value limited in the chart (see Charts 2-8 to 2-23). The fatigue live load moment should be equal to or larger than the maximum value.

STEP 3: STRENGTH DESIGN AT POSITIVE COMPOSITE SECTION (APPENDIX C, SHEET 3)

Effective prestress is assumed to be 160 ksi in the calculation. Use minimum deck reinforcement which is #4@12" for the top layer and #5@12" for the bottom layer. The flexural strength is not sensitive to concrete strength or to girder spacing (refer to Charts 3-1 to 3-6). Therefore the charts for each girder section can be drawn in one chart (see Chart 3-7).
STEP 4: STRENGTH DESIGN OF PRECAST POSITIVE SECTION (APPENDIX C, SHEET 4)

Effective prestressing stress is assumed to be 160 ksi. From the charts, it can be seen that \( \phi M_u \) is not sensitive to concrete strength when strand number is less than 32. When strand number is equal to 32 or larger, the larger \( f'_c \) is used and the larger \( \phi M_u \) can be achieved. But the ratio between the maximum value and the minimum value is almost a constant value ranging from 1.35 to 1.37 (see Charts 4-1 to 4-6).

STEP 5: STRENGTH DESIGN OF PRECAST NEGATIVE SECTION (APPENDIX C, SHEET 5)

These tables give the flexural moment capacity for each girder size with different concrete strength and TR area. Based on UNL testing, the maximum amount of Grade 150 steel is 10 – 1 3/8” diameter Threaded Rod above the girders. Here prestress is ignored since the number of strands varies and the prestress is not large at negative section. It is found that \( \phi M_u \) is not sensitive to the concrete strength (see Charts 5-1 to 5-6). Thus 9 ksi concrete is chosen to draw all girders’ curve in one chart (See Chart 5-7).

STEP 6: STRENGTH DESIGN OF COMPOSITE NEGATIVE SECTION (APPENDIX A, SHEET 6)

These tables give the flexural moment capacity for each girder size with different concrete strength, TR area, and deck bar area. The maximum deck steel \( A'_{sv} = 3.67 \text{ in.}^2/\text{ft.} \). Based on UNL tests, the maximum amount of Grade 150 steel is 10 – 1 3/8” diameter Threaded Rod above the girders. For preliminary design, prestress is ignored because the
number strands varies and the prestress is not large at negative section. In Strength I design at composite section, maximum deck steel is not needed in some cases. However, deck reinforcement may need to be increased based on fatigue criteria checking (see Charts from App. 6-1 to App. 6-24).

If girder spacing is known first, users need to go to Table 3 to get the possible deck reinforcement before using the chart directly. If girder spacing is an unknown value, use the chart directly to get a rough design first.

STEP 7: MAXIMUM LIVE LOAD DUE TO TR FATIGUE (APPENDIX C, SHEET 7)

\[ f_r = 36 - \frac{f_{\text{min}}}{3}, \]  
If \( f_{\text{min}} \) is limited to 54 ksi, \( f_r = 18 \) ksi. Therefore 18 ksi is used to get maximum fatigue live load moment with composite section properties. After researching on the design with different concrete strength, the author found that maximum fatigue live load moment that lets Rod fatigue stress get to 18 ksi is not sensitive to concrete strength. Therefore the author calculated the fatigue live load capacity for each girder size with 8 ksi girder concrete based on 18 ksi of TR fatigue limit (see Charts 7-1 to 7-6).

STEP 8: MAXIMUM DEAD LOAD DUE TO TR DEAD LOAD STRESS LIMIT (APPENDIX C, SHEET 8)

TR can take 54 ksi due to dead loads including deck weight on precast section and SIDL on the composite section.  
\[ \sigma_{\text{deck}} = n \frac{M_{\text{deck}}}{S_{b \cdot \text{rod}}}, \] in which \( S_{b \cdot \text{rod}} = \frac{y_{S, A} - y_{\text{rod}}}{I_{nc}} \), n is the Modulus of Elasticity ratio of TR over that of concrete.  
\[ \sigma_{\text{SIDL}} = n \frac{M_{\text{SIDL}}}{S_{bc \cdot \text{rod}}}, \] in which
Bridge width is 46.67 ft as assumed earlier. There are two rails on each side with 0.382 k/ft. The average load is applied on 46.67 ft wide bridge with a value of \(0.382 \times 2 / 46.67\). Plus Future Wearing Surface of 0.025 psf, the linear uniform distributed SIDL each girder takes is \(W_{SIDL} = (0.382 \times 2 / 46.67 + 0.025) \times S\), \(S\) is girder spacing. Ignoring haunch weight, the deck weight plus Stay-in-place Forms (5 lb/ft\(^2\)) each girder takes is \((7.5/12 \times 0.15 + 0.005) \times S\). Therefore,

\[
\frac{W_{SIDL}}{W_{deck}} = \frac{(0.382 \times 2 / 46.67 + 0.025)}{(7.5/12 \times 0.15 + 0.005)} = 0.42
\]

\[
\frac{\sigma_{SIDL}}{\sigma_{deck}} = \frac{M_{SIDL}}{M_{deck}} \times \frac{S_{b-rods}}{S_{bc-rods}} = \frac{W_{SIDL}}{W_{deck}} \times \frac{S_{b-rods}}{S_{bc-rods}} = (0.42) \frac{S_{b-rods}}{S_{bc-rods}}
\]

The TR stress ratio of stress due to SIDL over stress due to deck weight is not sensitive to concrete strength for all the sections. After researching each section by changing the rod’s area and deck reinforcement area (see Charts 8-1 to 8-3 and the related tables), it is found that the ratio of rod stress due to SIDL over the stress due to deck weight can be assumed to be a constant value of 0.08 because in most cases the deck steel area is larger than 30 square in. \(((\#4 + \#5 + 4 \#8) \times 8)\). Divide 54 ksi into 50 ksi \((54/1.08 = 50)\) and 4 ksi \((54-50 = 4)\), the maximum deck weight that each section can take can be obtained by using non-composite section only with 50 ksi rod stress limit due to deck weight (see Chart 8-4). And the maximum SIDL is taken by composite section with 4 ksi rod stress limit. Then the total dead load that the composite section can take is the summation of the two moments (see Chart 8-4).

The maximum SIDL taken by composite section with 4 ksi of rod stress limit can be
calculated from this equation: $4 = n \frac{M_{SIDL} (y_{cN.A} - y_{rod})}{I_c}$. The maximum fatigue LL moment causes 18 ksi stress in TR, that is, $18 = n \frac{M_{LL} (y_{cN.A} - y_{rod})}{I_c}$. To avoid repeating the charts, charts 7-1 to 7-6 can be used for the maximum SIDL calculation by multiplying a factor of 2/9.

Because deck reinforcement cannot work by itself with large spans and large spacing without the help of TR in most cases, therefore the author does not research on it.

**STEP 9: MAXIMUM LIVE LOAD DUE TO CONCRETE FATIGUE (APPENDIX C, SHEET 9)**

The author did sensitivity research about fatigue live load stress over concrete strength by checking fatigue limit at service stage. The average ratio is 0.1 as shown Table 5. Although concrete strength required is higher for shallower sections and lower for larger sections, it is good enough for the designers to choose their system in the preliminary design stage. After that, every criteria needs to be met.
Table 5 Average of Fatigue LL Stress over Concrete Strength Ratio

<table>
<thead>
<tr>
<th></th>
<th>NU900</th>
<th>NU1100</th>
<th>NU1350</th>
<th>NU1600</th>
<th>NU1800</th>
<th>NU2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span (ft)</td>
<td>100</td>
<td>120</td>
<td>140</td>
<td>160</td>
<td>180</td>
<td>200</td>
</tr>
<tr>
<td>Spacing (ft)</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>$A_{tr}$ (in.$^2$)</td>
<td>7.9</td>
<td>9.48</td>
<td>9.48</td>
<td>11.06</td>
<td>12.64</td>
<td>14.22</td>
</tr>
<tr>
<td>$A_{ds}$ (in.$^2$)</td>
<td>33.03</td>
<td>33.03</td>
<td>33.03</td>
<td>33.03</td>
<td>33.03</td>
<td>33.03</td>
</tr>
<tr>
<td>Number of 0.6&quot; strands</td>
<td>30</td>
<td>36</td>
<td>40</td>
<td>44</td>
<td>50</td>
<td>56</td>
</tr>
<tr>
<td>$f_c$ (1.0 Deck wt) (ksi)</td>
<td>2.23</td>
<td>2.27</td>
<td>2.31</td>
<td>2.26</td>
<td>2.35</td>
<td>2.47</td>
</tr>
<tr>
<td>$f_c$ (1.0 SIDL) (ksi)</td>
<td>0.49</td>
<td>0.55</td>
<td>0.58</td>
<td>0.61</td>
<td>0.66</td>
<td>0.71</td>
</tr>
<tr>
<td>$f_c$ (1.0 fatigue LL) (ksi)</td>
<td>0.61</td>
<td>0.57</td>
<td>0.51</td>
<td>0.47</td>
<td>0.45</td>
<td>0.43</td>
</tr>
<tr>
<td>$f_c$ (0.5DL+1.0LL) (ksi)</td>
<td>1.97</td>
<td>1.97</td>
<td>1.96</td>
<td>1.91</td>
<td>1.96</td>
<td>2.03</td>
</tr>
<tr>
<td>Required $f'_c$ (ksi)</td>
<td>4.91</td>
<td>4.94</td>
<td>4.90</td>
<td>4.77</td>
<td>4.89</td>
<td>5.07</td>
</tr>
<tr>
<td>$f_c$ (1.0LL)/ $f'_c$</td>
<td>0.12</td>
<td>0.11</td>
<td>0.11</td>
<td>0.10</td>
<td>0.09</td>
<td>0.09</td>
</tr>
<tr>
<td>Average ratio of $f_c$  (1.0LL) / $f'_c$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.10</td>
<td></td>
</tr>
</tbody>
</table>

Composite section properties are used to calculate maximum fatigue live load moment based on $0.1f_c$ stress limit on compression fiber (see Charts 9-1 to 9-6).

**STEP 10: SIMPLIFIED DESIGN CHARTS (APPENDIX C, SHEET 10)**

For each girder size, the author got the maximum span capacity for each spacing (6 ft to 12 ft) and plot a point on the chart. The strands number (0.6” Diameter) and TR number (1 3/8” Diameter) are put beside each point on the chart. Therefore the charts provide
information for both positive sections and negative sections for concrete from 8 ksi to 11 ksi. The minimum reinforcement and girder shipping and handling issues needs to be checked in the detailed design (see Charts 10-1 to 10-4).

SECTION 5: THREE SPAN LOADS (APPENDIX C, SHEET 11)

The span ratio of three-span bridges is 0.8-1.0-0.8. The table gives all loads needed for analysis. The girder weight is a uniform distributed load on a simple span. The deck weight is a uniform distributed load on three spans. It is easy to get by using any commercial software, such as Risa 3D. In addition, “Three-Moment Equation” Method can be used to easily get the deck weight.

Service I live loads multiplied with Distribution Factors are obtained from Conspan. In the deflection calculation, Service I load is used with the deflection Distribution Factor (Refer to Section 3 Step 8 for details). It belongs to two lanes loaded. The equation used is

$$\text{DF} = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{Kf}{12Lt^3} \right)^{0.1}$$

Fatigue live load moment of one lane loaded is obtained from Risa 3D. Then the data are mutilated by the distribution factor. For fatigue load, the Distribution Factor calculation uses one lane loaded equation:

$$\text{DF} = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{Kf}{12Lt^3} \right)^{0.1}$$

SECTION 6: PRELIMINARY DESIGN EXAMPLE

Given: A bridge is 46.67 ft wide with 100ft -100ft two continuous span. The girder size
is NU1100 and the girder spacing is 6ft. The other information on loads is the same as Section 3. The following is the step-by-step design.

**STEP 1: LOADS**

The critical positive section is 0.4L; the critical negative section is 1.5 ft from the pier center line. The load needed for preliminary design is shown below.

<table>
<thead>
<tr>
<th>0.4L</th>
<th>Service III moment M(_{(DL+0.8LL)})</th>
<th>2499 k.ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Fatigue LL M(_u)</td>
<td>706 k.ft</td>
</tr>
<tr>
<td></td>
<td>Strength I Composite Section M(_u)</td>
<td>4370 k.ft</td>
</tr>
<tr>
<td></td>
<td>Strength I Precast Section M(_u)</td>
<td>1689 k.ft</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Value at 1.5 ft from pier center line (Face of diaphragm)</th>
<th>Maximum V</th>
<th>282 k</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Strength I Precast Section M(_u)</td>
<td>874 k.ft</td>
</tr>
<tr>
<td></td>
<td>Strength I Composite Section M(_u)</td>
<td>3563 k.ft</td>
</tr>
<tr>
<td></td>
<td>Fatigue Load M</td>
<td>410 k.ft</td>
</tr>
<tr>
<td></td>
<td>Deck Weight M</td>
<td>748 k.ft</td>
</tr>
<tr>
<td></td>
<td>SIDL</td>
<td>256 k.ft</td>
</tr>
</tbody>
</table>

**STEP 2: BEAM SIZE BASED ON THE MAXIMUM SHEAR CAPACITY**

\[ V_u = 0.9 \left( 0.25 f' \_c \_b \_d \_v \right) + V_p \]

Assume \( V_p = 0 \), \( d \_v = 0.72H \)
H = 43.3 + 1 + 7 = 51.3”

\[ V_u = 0.9 \times (0.25 \times 8 \times 5.9 \times (0.72 \times 51.3)) = 392.3 \text{ k} > V_u \text{ OK!} \]

The section can take maximum shear force.

**STEP 3: SERVICE III AT 0.4L**

Use M = 2499 k.ft to get the strand number from the chart below. 18 strands are needed to satisfy the capacity requirement. The minimum reinforcement requires at least 18 strands. Therefore it is fine.

![Service III Moment vs. Strands Number Needed](image)

Fig. 12 Get Strands Number from Service III
STEP 4: FATIGUE CHECK AT 0.4L

Using a 100 ft span, 6 ft spacing, by using Chart 2-8, it is easy to get the maximum fatigue LL as 5386 k.ft. The actual fatigue LL is 706 k.ft. Proceed to next step.

![Max Fatigue Moment vs. Girder Span, 6 ft Girder Spacing, 8 ksi Concrete](image)

Note: Strands Number determined by Service III

Fig. 13 Maximum Fatigue Moment

STEP 5: FLEXURAL DESIGN STRENGTH OF POSITIVE COMPOSITE SECTION

Using 18-0.6” strands, from Chart 3-7, $\phi M_u = 4158$ k.ft < $M_u = 4370$ k.ft. Increase strand number to 20.
Fig. 14 Check Flexural Capacity of Composite Section

STEP 6: FLEXURAL DESIGN STRENGTH OF POSITIVE PRECAST SECTION

Using 20-0.6” strands, from Chart 4-2, $\phi M_n = 3768$ k.ft $> M_u = 1689$ k.ft. Proceed to the next step.
STEP 7: FLEXURAL STRENGTH OF NEGATIVE PRECAST SECTION

Choose TR Number to meet the flexural design requirement of the negative precast section. First, try to use 2- 1 3/8” diameter TR, $A_{tr} = 3.16$ in.$^2$. From the chart below, $\phi M_u = 1558$ k.ft > $M_u= 874$ k.ft. Proceed to the next step.
STEP 8: FLEXURAL STRENGTH OF THE NEGATIVE COMPOSITE SECTION

$M_u = 3563$ k.ft. TR area in Step 7 is 3.16 in.$^2$. From Table 3, it is known that for 6 ft girder spacing, the deck area could be 3.06, 6.78, 8.34, 10.26, 10.5, 13.62, 12.54, 17.46, 22.02 in.$^2$.

Increase the deck bar area among the value above little by little, until the flexural strength is larger than or equal to the ultimate moment. When deck reinforcement is equal to 10.26 in.$^2$ $\phi M_n = 3589$ k.ft $M_u = 3563$ k.ft. Proceed to the next step.
Fig. 17 Check Flexural Capacity of Negative Composite Section

STEP 9: CHECK FATIGUE LL TO SATISFY 18 KSI STRESS LIMIT IN TR AT NEGATIVE MOMENT SECTION

From the last step, the deck reinforcement area is 10.26 in.$^2$. The fatigue live load moment, $M_u = 410$ k.ft. From the chart, fatigue moment capacity is $1030$ k.ft $> M_u = 410$ k.ft. Ok!
STEP 10: CHECK DL CAPACITY BASED ON TR STRESS

From Step 9, deck reinforcement is 10.26 in.$^2$. Fatigue moment capacity is 1030 k.ft. SIDL moment capacity is 2/9 of fatigue moment capacity (refer to Section 4 Step8).

Therefore the capacity of SIDL is $1030(2/9) = 229$ k.ft < SIDL moment of 256 k.ft. Not Good!

Increase the deck area to the maximum area of 22.02 in.$^2$. From the chart below, the max fatigue limit is increased to 1957 k.ft. SIDL moment capacity is $1957(2/9) = 435$ k.ft > 256 k.ft. Proceed to the next step.
Max. Fatigue Live Load Moment to Satisfy the 18 ksi Stress Limit in the Threaded Rods, NU1100

Fig. 19 Increase Deck Steel to Get Larger Fatigue Moment Capacity and SIDL Moment Capacity

With 3.16 in.\(^2\) TR, from the chart below, the maximum deck weight that the non composite section can take is 563 k.ft \(< M_{\text{deck}} = 748\) k.ft. Increase TR area to 4.74 in.\(^2\) TR, deck weight capacity is 837 k.ft \(> M_{\text{deck}} = 748\) k.ft. Proceed to the next step.
STEP 11: FATIGUE LL CAPACITY BASED ON CONCRETE FATIGUE LIMIT

Use deck reinforcement of 22.02 in.\(^2\) and 4.74 in.\(^2\) TR area. The sum of them is equal to 26.76 in.\(^2\). From the chart below, the LL fatigue moment capacity is 804 k.ft > \(M_u = 470\) k.ft. Proceed to the next step.
Fig. 21 Check LL Fatigue Capacity Based on Concrete Fatigue Limit

The concrete strength at release should be calculated by using the corresponding Excel spreadsheet in Appendix C. The code requires $f'_{ci} \leq 0.8f'_c$ (LRFD 5.4.2.3.2). This is fine in most cases. The live load deflection also should be checked with the limit of $L/800$ by using the corresponding Excel spreadsheet. From the preliminary design above, the design result of this bridge is summarized in Table 7.
### Table 7 Design Result

<table>
<thead>
<tr>
<th>Girder</th>
<th>NU 1100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Span</td>
<td>100 ft</td>
</tr>
<tr>
<td>Spacing</td>
<td>6.00 ft</td>
</tr>
<tr>
<td>Number Strands</td>
<td>20 0.6&quot; Dia.</td>
</tr>
<tr>
<td>Deck bar area</td>
<td>22.02 in.$^2$</td>
</tr>
<tr>
<td>TR (1 3/8&quot;) N</td>
<td>3</td>
</tr>
<tr>
<td>Final $f'_c$</td>
<td>8.000 ksi</td>
</tr>
<tr>
<td>$f'_{ci}$</td>
<td>2.186 ksi</td>
</tr>
<tr>
<td>Live load deflection</td>
<td>0.59 in.</td>
</tr>
<tr>
<td>TR location</td>
<td>above top flange</td>
</tr>
</tbody>
</table>

If the girder concrete strength is increased up to 11.0 ksi and the section cannot meet all the criteria above, a larger girder size should be tried by repeating the same steps as above.

**SECTION 7: SIMPLIFIED PRELIMINARY DESIGN CHARTS**

The designed bridge is a two-span continuous bridge. The bridge width is 46.67 ft. The spacing is 6, 8, 10 and 12 ft. The spans are 80, 100, 120, 140, 160, 180 and 200 ft. The girder concrete strength is 8, 9, 10 and 11 ksi and the deck concrete is 4 ksi. In the design, structural thickness of 7 in. is used in the analysis. In the calculation of the deck weight, 7.5 in. is used. The haunch is 1 in. at the positive section and 3 in at the negative section. From the charts below, it is very easy for designers to get the number of 0.6"\(\phi\) strands
and TR needed given girder concrete strength, bridge span, girder spacing, and NU-I girder size.

![P/C NU-I Girder Preliminary Design, 8 ksi](image)

Fig. 22 Preliminary Design Chart Aid $f'_c = 8$ ksi

![P/C NU-I Girder Preliminary Design, 9 ksi](image)

Fig. 23 Preliminary Design Chart Aid $f'_c = 9$ ksi
Fig. 24 Preliminary Design Chart Aid $f'_c = 10$ ksi

Fig. 25 Preliminary Design Chart Aid $f'_c = 11$ ksi
SECTION 8: CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

1. The design by placing TR above girder top flanges is very effective and allows larger span and larger spacing to be used.

2. The chart design aids are very convenient and conservative for bridge engineers to use in the preliminary design. It gives a guide for users to choose the girder size, girder span, girder spacing and concrete strength.

3. The charts and the example describe a clear and brief picture in each design step and point out a right direction for users who are new in the bridge design field.

RECOMMENDATIONS

This aid is a starting point for bridge design. After that, users need to go over a detailed design, run Conspan to get accurate Service I live load and run Risa to get accurate fatigue live load, check minimum reinforcement, design every 0.1L, including flexural strength design, service design, shear design, end zone design, prestress loss and concrete strength at release.
LIST OF APPENDICES

APPENDIX A: HAND CALCULATION EXAMPLE FOR DESIGN OF AN INTERIOR GIRDER LINE WITH TR CONTINUITY

APPENDIX B: PROGRAMS FOR SERVICE DESIGN OF NEGATIVE MOMENT AND LIVE LOAD MOMENT AND SHEAR CALCULATIONS (SEE SEPARATE ELECTRONIC EXCEL FILE ON CD)

APPENDIX C- CHARTS AND TABLES (SEE SEPARATE ELECTRONIC EXCEL FILE ON CD)

APPENDIX D- MOMENT AND SHEAR CALCULATIONS DUE TO LIVE LOADS (SEE SEPARATE ELECTRONIC EXCEL FILE ON CD)
APPENDIX A: HAND CALCULATION EXAMPLE FOR DESIGN OF AN INTERIOR GIRDER LINE WITH TR CONTINUITY

INPUT BRIDGE INFORMATION

The designed bridge is a two-span continuous bridge. Bridge width is 46.67 ft. The girder spacing is 6 ft. The span is 100 ft-100 ft. Concrete strength is assumed to be 8 ksi at the beginning and deck concrete is 4 ksi. 20 -0.6” diameter G270 strands, 4.74 in.² G150 TR, and 22.02 in.² G60 bars are used. The other input data is given below:

Table A-1 Input Data

<table>
<thead>
<tr>
<th></th>
<th>f’ci</th>
<th>f’c-deck</th>
<th>Current wearing thickness</th>
<th>Haunch of positive section</th>
<th>Haunch of negative section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5.5 Ksi</td>
<td>4 Ksi</td>
<td>0.5 in.</td>
<td>1 in.</td>
<td>3 in.</td>
</tr>
</tbody>
</table>

Table A-2 Girder Section Properties

<table>
<thead>
<tr>
<th>Girder Size</th>
<th>NU 1100</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>h</td>
<td>43.3</td>
<td>in.</td>
</tr>
<tr>
<td>A</td>
<td>694.6</td>
<td>in.²</td>
</tr>
<tr>
<td>I</td>
<td>182279</td>
<td>in.⁴</td>
</tr>
<tr>
<td>Yb</td>
<td>19.6</td>
<td>in.</td>
</tr>
<tr>
<td>Wg</td>
<td>0.724</td>
<td>k/ft</td>
</tr>
</tbody>
</table>
Girder Line Number $N_{\text{girders}} = \frac{Bridge\text{Widt} h}{Girder\text{Spac ing}} = \frac{46.67}{6} \approx 8$

Lane Number $N_{\text{lanes}} = \frac{Bridge\text{Widt} h}{12 \text{ ft}} = \frac{46.67}{12} \approx 3$

**Deck**

Maximum deck bar area is 22.02 in.$^2$.

**Cross Section**

The effective deck width (conform to LRFD 4.6.2.6.1) is the smallest value of $1/4L_w$, $12t_s$ plus MAX ($b_d/2$ or $t_w$), and girder spacing.

$$b = MIN \left\{ \begin{array}{l} 1/4 (100)(12) = 300 " \\ b = MIN \left\{ b (7) + MAX (48.2 / 2, 5.9) = 108 " \\ 6(12) = 72 " (Controls ) \end{array} \right. \right. $$

The girder is divided into five rectangular layers. From bottom to top, the section includes two layers at the bottom flange, one layer at the web, two layers at the top flange, one layer of haunch, and one layer of deck as shown below.

<table>
<thead>
<tr>
<th>Width, W</th>
<th>Thick., T</th>
</tr>
</thead>
<tbody>
<tr>
<td>48.200</td>
<td>2.56</td>
</tr>
<tr>
<td>27.050</td>
<td>1.75</td>
</tr>
</tbody>
</table>

Table A-3 Concrete Layers
For each girder line, the load calculation is as below:

Deck weight = \((7\text{” structural deck thickness} + 0.5\text{” current wearing surface})/12\times6’\text{ girder spacing} + \text{48.2” haunch width} \times1”\text{ haunch/144}) \times 0.15 = 0.61 \text{ k/ft}\)

Stay-in-place form = 0.005\times6 \text{ ft girder spacing} = 0.03 \text{ k/ft}

Adding the weight of stay-in-place form into the deck weight, the total weight is

\[ w_d = 0.61 + 0.03 = 0.64 \text{ k.ft} \]

Rail & barrier, \( w_{rail} = 0.382\times2/8 = 0.096 \text{ k/ft} \)

Future wearing surface, \( w_{FWS} = 0.025\times6’\text{ (girder spacing)} = 0.15 \text{ k/ft} \)
### Table A-4 Load Combination

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Total Enacted Service ( S_{\text{E}} ) M</th>
<th>Load combination</th>
<th>Composite section</th>
<th>Service Live Load</th>
<th>Fatigue load</th>
<th>Force</th>
<th>Total ( M )</th>
<th>Straight Section ( M )</th>
<th>Straight Composite Section ( M )</th>
<th>max ( V )</th>
<th>Value at face of diaphragm</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DUDL</td>
<td>DUDL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Legend:**
- DUDL: Design Ultimate Limit State Load
- FVSL-L: Fatigue Limit State Load - Low Fatigue Class
- FVSL-H: Fatigue Limit State Load - High Fatigue Class
- V: Shear force
- M: Bending moment
Use structural analysis method to get the moment. The following hand calculation is for the section at 0.4L. The analysis of other sections should follow the same procedure.

a.) Girder weight

At 0.4L = \( \frac{w_g a b}{2} = \frac{(0.724)(0.4 \times 100)(0.6 \times 100)}{2} = 869 \text{ k.ft} \) (Cell C24)

b.) Deck weight calculation

From structural analysis, for two continuous spans, the maximum deck weight at the pier center line, \( M^- = -w_d \left( \frac{L^2}{8} \right) = -0.64 \left( \frac{100^2}{8} \right) = -803 \text{ k.ft} \) (Cell E18)

The support force R at girder end away from pier, \( R = \frac{3}{8} w_d L \). Upward is positive.

Deck weight in continuous span (CS) at xL location (x is number of tenth point),

\[
M_{xL} = Rw_d(xL) - \frac{1}{2} w_d(xL)^2 = w_d(x) \left( \frac{3}{8} - \frac{x}{2} \right) L^2
\]

Take deck weight moment at 0.4L for example,

\[
M_{0.4L} = w_d(0.4) \left( \frac{3}{8} - \frac{0.4}{2} \right) L^2 = 0.64 \left( \frac{3}{8} - \frac{0.4}{2} \right) \left(100\right)^2 = 450 \text{ k.ft} \) (Cell E24)

The modified moment,

\[
M_{0.4L} = M_{0.4L} + (0.4)\| 0 \% M^- \| = 450 + (0.4)(803)(10 \%) = 482 \text{ k.ft} \) (Cell G24)

Choose the larger absolute value between CS and CS + Change for conservation on each location. Shear is not affected by the change. The diagram of the moment is as the following chart below.
### Table A-5 Deck Moment Redistribution

<table>
<thead>
<tr>
<th>Location from end</th>
<th>Deck weight M</th>
<th>Moment redistribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-803</td>
<td>-723</td>
</tr>
<tr>
<td>0.9</td>
<td>-434</td>
<td>-362</td>
</tr>
<tr>
<td>0.8</td>
<td>-129</td>
<td>-64</td>
</tr>
<tr>
<td>0.7</td>
<td>112</td>
<td>169</td>
</tr>
<tr>
<td>0.6</td>
<td>289</td>
<td>337</td>
</tr>
<tr>
<td>0.5</td>
<td>402</td>
<td>442</td>
</tr>
<tr>
<td>0.4</td>
<td>450</td>
<td>482</td>
</tr>
<tr>
<td>0.3</td>
<td>434</td>
<td>458</td>
</tr>
<tr>
<td>0.2</td>
<td>353</td>
<td>370</td>
</tr>
<tr>
<td>0.1</td>
<td>209</td>
<td>217</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

**Fig. A-1 Deck Weight Moment**
c.) SIDL

Moment due to SIDL should not be modified as deck weight. They simply work on continuous span. Use the result of the deck weight moment on continuous span without redistribution.

Rail moment at 0.4L, \( M_{rail} = M_{deck} \left( \frac{w_{rail}}{w_{deck}} \right) = 450 \left( \frac{0.096}{0.64} \right) = 67 \text{ k.ft (Cell I24)} \)

d.) FWS moment at 0.4L

\[ M_{FWS} = M_{deck} \left( \frac{w_{FWS}}{w_{deck}} \right) = 450 \left( \frac{0.15}{0.64} \right) = 105 \text{ k.ft (Cell K24)} \]

e.) Live load

Following LRFD, the author researched on all the girder sections at the span range from 80 ft to 200 ft and spacing range from 6 ft to 12 ft that are used in the chart design, gives a series of convenient equations for calculation of live load as the following (refer to Appendix D “Live load M_V per lane” for details).

**Fatigue LL**

With 1.15 impact Live load factor,

\( M^+ \) at 0.4L: \( M^+ = aL^2 + bL + c = 0.003L^2 + 15.908L - 427.91 \)

\( M^- \) at pier centerline: \( M^- = aL^2 + bL + c = -0.0041L^2 + 9.7041L - 233.31 \)

\( M^- \) at 0.9L: \( M^- = aL^2 + bL + c = -0.0037L^2 + 8.7337L - 209.97 \)

Load distribution factor for fatigue load (LRFD Table 4.6.2.2.2b):
\[ DF = 0.06 + \left( \frac{S}{14} \right)^{0.4} \left( \frac{S}{L} \right)^{0.3} \left( \frac{K_s}{12Lt^3} \right)^{0.1} \]

The detailed calculation is shown below.

**Table A-6 Distribution Factor Calculation of One Lane Loaded**

<table>
<thead>
<tr>
<th></th>
<th>M+ section</th>
<th>M- section</th>
</tr>
</thead>
<tbody>
<tr>
<td>L for M-</td>
<td>100</td>
<td>100 ft</td>
</tr>
<tr>
<td>( f'_{c-deck} )</td>
<td>4</td>
<td>4 ksi</td>
</tr>
<tr>
<td>( f'_{c} )</td>
<td>8</td>
<td>8 ksi</td>
</tr>
<tr>
<td>( t_{slab} )</td>
<td>7</td>
<td>7 in.</td>
</tr>
<tr>
<td>( t_{hauanch} )</td>
<td>1</td>
<td>3 in.</td>
</tr>
<tr>
<td>( h )</td>
<td>43.3</td>
<td>43.3</td>
</tr>
<tr>
<td>( I_{non-comp} )</td>
<td>182279</td>
<td>182279 in²</td>
</tr>
<tr>
<td>( A_{non-comp} )</td>
<td>694.6</td>
<td>694.6 in²</td>
</tr>
<tr>
<td>( Y_t )</td>
<td>23.7</td>
<td>23.7 in</td>
</tr>
<tr>
<td>( n=E_c/E_{cd} )</td>
<td>1.357</td>
<td>1.357</td>
</tr>
<tr>
<td>( e_g )</td>
<td>28.2</td>
<td>30.2 in</td>
</tr>
<tr>
<td>( k_g=n(I+A e_g^2) )</td>
<td>997122</td>
<td>1107236 in²</td>
</tr>
<tr>
<td>DF</td>
<td>0.395</td>
<td>0.398</td>
</tr>
</tbody>
</table>

Fatigue live load with its load combination factor is

\[ M_{ul} = 1.5 \left( 0.003L^2 + 15.908L - 427.91 \right) = 1.5 \left( 0.003 (100)^2 + 15.908 (100) - 427.91 \right) = 1789.3 \]

Multiplied by one lane load distribution factor, \( M_{ul} = 1789.3 \times 0.395 = 706 \) k.ft (Cell P24).
Service I Live load:

\[ M^+ \text{ at } 0.4L: M^+ = aL^2 + bL + c = 0.0616L^2 + 19.445L - 331.09 \]

\[ M^- \text{ at pier centerline: } M^- = aL^2 + bL + c = 0.0712L^2 + 16.997L - 216.31 \]

\[ M^- \text{ at } 0.9L: M^- = aL^2 + bL + c = 0.0895L^2 + 0.7696L - 395.08 \]

Load distribution factor for Service I Live load (LRFD Table 4.6.2.2.2b):

\[ DF = 0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_x}{12L^3} \right)^{0.1} \]

Using the data in Table A-6, the result is:

<table>
<thead>
<tr>
<th>Location</th>
<th>M+ section</th>
<th>M- section</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF</td>
<td>0.547</td>
<td>0.552</td>
</tr>
</tbody>
</table>

Take Service I live load moment at 0.4L as an example,

\[ M_{LL} = 0.062L^2 + 19.445L - 331.09 \]

\[ = 0.062 \times 100^2 + 19.445 \times 100 - 331.09 = 2229.41 \text{ k.ft} \]

Multiply by the Distribution Factor for Service Design and Flexural Strength Design,

\[ M_{LL} = 2229.41 \times 0.547 = 1229 \text{ k.ft} \text{ (Cell M24)} \]

The distribution factor for deflection calculation is

\[ DF = \frac{N_{\text{lane}}}{N_{\text{girders}}} \times \text{Multiple Presence Factor} \]
= 3/8 \times 0.85 = 0.319 \text{ (Cell B14)}

Multiply Service I live load moment by Distribution Factor for deflection calculation,

\[ M_{LL} = 2229.41 \times 0.319 = 711 \text{ k.ft (Cell L24)} \]

**Load Combination**

Strength I at precast section,

\[ M_u = 1.25 \, DC = 1.25 \times (869+482) = 1689 \text{ ft (Cell R24)} \]

Strength I at composite section is

\[ M_u = 1.25 \, DC + 1.5 \, DW + 2 \times (LL + IM) \]

\[ = 1.25 \times (869+482+67) + 1.5 \times (105) + 2.0 \times (1229) = 4388 \text{ k.ft (Cell S24)} \]

Strength I at composite section is

\[ M_u = 1.25 \, DC = 1.25 \times (869+482) = 1689 \text{ k.ft (Cell R24)} \]

The critical section of negative moment area is at face of diaphragm in the calculation.

The moment value is obtained from interpolation between \( M' \) at the pier centerline and \( M \) at 0.9L. The values are put on the bottom line in the load table.

**BEAM SIZE BASED ON THE MAXIMUM SHEAR CAPACITY**

\[ d_v = 0.72 \, h = 0.72 \times (43.3 + 7 + 1) = 36.9 \text{ in.} \]

In the chart, draping is not considered because the worst case is considered in the preliminary design. Draping makes design easier.

Nominal shear resistance \( V_n = 0.25 \, f'c \, b_v \, d_v + V_p \) (AASHTO 5.8.3.3)

\[ \phi V_n = 0.9(0.25 \times 8 \times 5.9 \times 36.9) = 392 \text{ k} > V_u = 282 \text{ k. Ok!} \]
If the section shear capacity at 0.4L is acceptable, the other sections are fine to take shear load because at the other sections, draping strands can help resist the shear force.

**SERVICE III DETERMINE NUMBER OF STRANDS USING DETAILED NCHRP 18-07 METHOD**

The unit for force and moment is kips and kips.in. The strands number is increased layer by layer to make it meet Service III requirements. By increasing the strand number little by little to meet Service III requirement, the Excel spreadsheet gives the total number of bottom strands as 20-0.6 in. Grade 270 strands. The strand centroid is 2.20” from the bottom fiber. $A_{pe} = 4.34$ in.$^2$.

If draping is needed in actual bridge design, to avoid excessive uplift force on the prestressing bed, a maximum of 12 strands can be harped at any point, such as 0.2L, 0.3L, and 0.4 L. Further design checks may require altering this arrangement. In addition, 4 – 0.5” Grade 270 straight top strands are needed for mild reinforcement support and for control of top cracking at prestress release. For the top strands, the tension force is specified at 2.02 kips for each strand. The effect of the top strands on the total prestress force will be ignored.
Table A-8 Assumed Input Values

<table>
<thead>
<tr>
<th></th>
<th>65</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ci}$</td>
<td>5.500</td>
<td>ksi</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>8.000</td>
<td>ksi</td>
</tr>
<tr>
<td>$f'_{cd}$</td>
<td>4.000</td>
<td>ksi</td>
</tr>
<tr>
<td>$t_i$ (release)</td>
<td>1</td>
<td>days</td>
</tr>
<tr>
<td>$t$ (deck pour)</td>
<td>60</td>
<td>days</td>
</tr>
<tr>
<td>$t_f$ (final)</td>
<td>20000</td>
<td>days</td>
</tr>
<tr>
<td>V/S (Beam)</td>
<td>2.95</td>
<td>in</td>
</tr>
<tr>
<td>V/S (Deck)</td>
<td>3.50</td>
<td>in</td>
</tr>
</tbody>
</table>

The result and the calculation details are shown in the Excel spreadsheet below.
Fig. A-2 Prestress Loss Using NCHRP 18-07 Detailed Method

**Transformed Section Properties**

Unify all the material into beam concrete by using Elastic Modulus ratio, and then transformed Section Properties can be calculated.

Steel area = \( (E_p / E_c - 1)A_p \)

Deck area = \( (E_d / E_c - 1)A_d \)

The centroid is calculated by Area Moment Method, \( Y_{bi} = \frac{\sum AY}{\sum A} \)

The total Moment of Inertia is calculated by summarizing the Moment of Inertia of every material. The results are listed in the following table.
**Table A-9 Transformed Section Properties**

<table>
<thead>
<tr>
<th>Section Properties</th>
<th>Precast Beam</th>
<th>Transformed Deck</th>
<th>Composite Beam &amp; Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Gross</td>
<td>Net (-$A_{ps}$)</td>
<td>Tr.- initial</td>
</tr>
<tr>
<td>A (in.$^2$)</td>
<td>694.60</td>
<td>690.26</td>
<td>718.33</td>
</tr>
<tr>
<td>y_b (in.)</td>
<td>19.60</td>
<td>19.71</td>
<td>19.03</td>
</tr>
<tr>
<td>I (in.$^4$)</td>
<td>182279.0</td>
<td>180957</td>
<td>189226</td>
</tr>
<tr>
<td>$c_p$ (in.)</td>
<td>17.40</td>
<td>17.51</td>
<td>16.83</td>
</tr>
<tr>
<td>$c_d$ (in.)</td>
<td>-18.09</td>
<td>-17.98</td>
<td>2.1537</td>
</tr>
<tr>
<td>$\alpha = 1 + (A \cdot c_p^2)/I$</td>
<td>2.2996</td>
<td>2.3164</td>
<td>2.2152</td>
</tr>
<tr>
<td>$\alpha_t = 1 + A \cdot c_p \cdot y_b/I$</td>
<td>2.996</td>
<td>3.014</td>
<td>2.915</td>
</tr>
<tr>
<td>h/I</td>
<td>-0.5714</td>
<td>-0.5756</td>
<td>-0.5505</td>
</tr>
<tr>
<td>$\alpha_d = 1 + A \cdot c_d \cdot e_p/I$</td>
<td>2</td>
<td>0.00038</td>
<td>0.0003</td>
</tr>
</tbody>
</table>

**Prestress Loss due to Elastic Shortening**

From the load table, it is easy to get the load needed for Service III calculation.

**Table A-10 Load for Service III and Fatigue Checking**

<table>
<thead>
<tr>
<th>Moment Type</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>10426</td>
<td>kip-in</td>
</tr>
<tr>
<td>Deck, haunch, diaphragms</td>
<td>5784</td>
<td>kip-in</td>
</tr>
<tr>
<td>SIDL</td>
<td>2062</td>
<td>kip-in</td>
</tr>
</tbody>
</table>
Note: In the following calculation, gaining prestress is positive when the load makes the beam tend to become shorter; losing prestress is negative when the load makes the beam tend to become longer.

Initial elastic loss due to prestressing force and self-weight moment is

\[
\Delta f_{p,E} = n \left( f_{ps} A_{ps} \left( \frac{1}{A_{gdr-rel}} + \frac{e_{y-rel}^2}{I_{gdr-rel}} \right) + \frac{M_{gdr} e_{y-rel}}{I_{gdr-rel}} \right) \\
= 6.47 \left( -202.5 \times 4.34 \left( \frac{1}{716} + \frac{16.83^2}{188697} \right) + 10426 \left( \frac{16.83}{188697} \right) \right) \\
= -10.42 \text{ ksi}
\]

Elastic loss due to deck weight:

\[
\Delta f_{D,E} = n \left( \frac{M e_{y-final-rel}}{I_{g-final-rel}} \right) \\
= 5.36 \left( \frac{5784 \left( 16.94 \right)}{187860} \right) \\
= 2.8 \text{ ksi}
\]

Elastic loss due to SIDL (on composite section):

\[
\Delta f_{SIDL,E} = n \left( \frac{M_{SIDL} e_{y-final}}{I_{c-final}} \right) \\
= 5.36 \left( \frac{2062 \left( 26.69 \right)}{386692} \right) \\
= 0.8 \text{ ksi}
\]

Creep
Creep Coefficient, $\psi_i = 1.9 k_{vs} k_{hc} k_i k_{cf} t_i^{-0.118}$ (LRFD 5.4.2.3.2)

where $k_{vs} = 1.45 - 0.13 (V / S) \geq 1.0$

$k_{hc} = 1.56 - 0.008 H$

$k_f = \frac{5}{1 + f_{ci}^i}$

$k_{cf} = \left( \frac{t}{61 - 4f_{ci}^i + t} \right)$ in which $t = t_{\text{considered}} - t_i$

Combine the equations above together, it becomes

$$\psi_i = 1.9 \text{MAX} \left( 1.45 - 0.13 (V / S), 1 \right) (1.56 - 0.008 H) \left( \frac{5}{1 + f_{ci}^i} \right) \left( \frac{t}{61 - 4f_{ci}^i + t} \right) t_i^{-0.118}$$

Input the assumed values in Table A-8 to the combined equation above, the creep coefficient is got, as shown in the table below.

<table>
<thead>
<tr>
<th>Beam</th>
<th>initial to final</th>
<th>$\psi$ bij</th>
<th>1.618</th>
<th>$t$: from prestress release</th>
</tr>
</thead>
<tbody>
<tr>
<td>initial to deck placement</td>
<td>$\psi$ bid</td>
<td>0.976</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck placement to final</td>
<td>$\psi$ bdf</td>
<td>0.998</td>
<td></td>
<td>$t$: from deck placement</td>
</tr>
<tr>
<td>Deck</td>
<td>Deck placement to final</td>
<td>$\psi$ ddf</td>
<td>2.594</td>
<td>$t$: from deck placement</td>
</tr>
</tbody>
</table>

Transformed section factors, $K = \left(1 + n \alpha_{\text{net}} (A_{pu} / A_g)(1 + 0.7\psi)\right)^{-1}$ (5.9.5.4.2a-2)

There are two “K” factors. One is for the period of initial to deck placement, marked as $K_{id}$; the other is for deck placement to final, marked as $K_{df}$. In the calculation, gross...
section properties are used.

\[ K_{id} = (1 + 6.47 (2.2)(4.34/694.6)(1 + 0.7(1.624 )))^{-1} = 0.843 \]

\[ K_{if} = (1 + 6.47 (3.1)(4.34/1069.3)(1 + 0.7(1.624 )))^{-1} = 0.852 \]

Using all the factors above, creep between release and deck placement due to initial prestress and girder weight is

\[ \Delta f_{bad} = \psi_{bad} \Delta f_{p&mg-E} K_{id} = 0.976(-10.42) (0.843) = -8.6 \text{ ksi} \]

Creep Coefficient, \( \psi_{f} \) is not a linear variable. Creep due to initial prestress and girder weight between deck placement and final is

\[ \Delta f_{bad} = (\psi_{bad} - \psi_{if}) \Delta f_{p&gdr-E} K_{if} = (1.618-0.976)(-10.42)(0.852) = -5.7 \text{ ksi} \]

Therefore creep due to prestress and girder weight is -8.6-5.7 = -14.3 ksi

Creep due to prestress loss (between release and deck placement), deck weight and SIDL:
The total prestress loss between release and deck placement, called first-term loss, includes -6.2 ksi due to shrinkage (see the calculation on shrinkage), -8.6 ksi due to creep, -1.2 due to relaxation.

\[ \Delta f_{p-loss} = -6.2 - 8.6 - 1.2 = -16.0 \text{ ksi} \]

The elastic stress caused by first-term loss can be expressed as:

\[
\Delta f_{p-loss-E} = n \left( - \Delta f_{p-loss} A_p \left( \frac{1}{A_{gdr-gross}} + \frac{e_{gross}^2}{I_{gdr-gross}} \right) \right)
\]

\[
= 5.36 \left( 16.0 \times 4.34 \left( \frac{1}{694.6} + \frac{19.6^2}{182279} \right) \right)
\]

\[
= 1.154 \text{ ksi}
\]
Therefore the total creep loss in this part is

\[
\Delta f_{CR3} = (\Delta f_{p-loss-E} + \Delta f_{D-E} + \Delta f_{SIDL-E})/bdf K_{df}
\]

\[
= (1.154 + 2.8 + 0.8) \times 0.998 \times 0.852
\]

\[
= 4.0 \text{ ksi}
\]

**Shrinkage**

The prestress loss due to shrinkage of girder concrete between time of transfer and deck placement is \( \Delta f_{pSR} = E_{sp} K_{ad} \). The prestress loss between time of deck placement and final time is \( \Delta f_{pSD} = E_{sp} K_{df} \).

The shrinkage strain at time \( t \) is

\[
\varepsilon_{sh} = -0.00048 \times k_{vs} \times k_{hs} \times k_{ds} \quad \text{(LRFD 5.4.2.3.3)}
\]

(Sign convention is considered in the calculation.)

in which, \( k_{sh} = 2 - 0.014 \times H \). The other items are the same as the item in creep calculation. Take each factor in the equation for \( \varepsilon_{sh} \) to get the results shown below.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Initial to final</th>
<th>Initial to deck placement</th>
<th>Deck placement to final</th>
<th>Deck</th>
<th>Deck placement to final</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \varepsilon_{bif} )</td>
<td>( \varepsilon_{bid} )</td>
<td>( \varepsilon_{bdf} )</td>
<td>( \varepsilon_{ddf} )</td>
<td>( \varepsilon_{bid} - \varepsilon_{bdf} = 0.000170 )</td>
</tr>
<tr>
<td></td>
<td>0.000428</td>
<td>0.000258</td>
<td></td>
<td>0.000687</td>
<td></td>
</tr>
</tbody>
</table>

Prestress loss due to beam shrinkage between release and deck placement is

\[
\Delta f_{pSR} = -E_{sp} K_{ad} = -0.000258 \times 28500 \times 0.843 = -6.2 \text{ ksi}
\]
Prestress loss due to beam shrinkage of beam between deck place and final is

\[ \Delta f_{psd} = -\varepsilon_{bf} E_y K_{gf} = -0.000170 \times 28500 \times 0.852 = -4.1 \text{ ksi} \]

The elastic stress of strands due to deck shrinkage is

\[ \Delta f_{ds-e} = n \left( \varepsilon_{adj} E_{cd} A_c \left( \frac{1}{A_c} + \frac{e_d e_p c}{I_c} \right) \right) \]

\[ = 5.36 \left( 0.000687 \times 3607 \times 552.2 \times 0.0004 \right) \]

\[ = 2.804 \text{ ksi} \]

where

\[ \frac{1}{A_c} + \frac{e_d e_p c}{I_c} = \frac{1}{1069.3} + \frac{-18.09 (27.16)}{372967} = 0.0004 \]

Prestress loss due to deck shrinkage is

\[ \Delta f_{pss} = \Delta f_{ds-e} \frac{K_{adj}}{(1 + 0.7 \varepsilon_{bdf})} \]

\[ = 2.804 \left( \frac{0.852 \times (1 + 0.7 \times 0.998)}{(1 + 0.7 \times 2.594)} \right) \]

\[ = 1.4 \text{ ksi} \]

**Relaxation**

Relaxation between release and deck placement and between deck placement to final are both assumed to be (-1.2) ksi for low-relaxation strands. (LRFD 5.9.5.4.2c)

Total long-term prestress loss from deck placement to final is

\[ \Delta f_{p-loss} = -4.1 - 5.7 - 4.0 - 1.2 + 1.4 = -5.6 \text{ ksi} \]

**Bottom Fiber Stress at Service III Checking**
Different force and moment works on different section properties as shown in the table below. Compression stress is positive.

**Table A-13 Service Stage Stress**

<table>
<thead>
<tr>
<th>Force and Moment</th>
<th>Section</th>
<th>Concrete Stage</th>
<th>properties used in the calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestressing force and girder self weight</td>
<td>precast</td>
<td>At release</td>
<td>transformed</td>
</tr>
<tr>
<td>Time depended prestress loss from release to deck placement</td>
<td>precast</td>
<td>------</td>
<td>Gross</td>
</tr>
<tr>
<td>Deck weight</td>
<td>precast</td>
<td>At final</td>
<td>transformed</td>
</tr>
<tr>
<td>SIDL</td>
<td>composite</td>
<td>At final</td>
<td>transformed</td>
</tr>
<tr>
<td>Time depended prestress loss from deck placement to final</td>
<td>composite</td>
<td>------</td>
<td>Gross</td>
</tr>
<tr>
<td>Live load</td>
<td>composite</td>
<td>At final</td>
<td>transformed</td>
</tr>
</tbody>
</table>
\[ f_c = P_i \left( \frac{1}{A_{p,i}} + \frac{e_p y_{b-p,i}}{I_{p,i}} \right) - \frac{M_g^* y_b}{I_g^*} \]

\[-\Delta P_{p-lost_1} \left( \frac{1}{A_{p,gross}} + \frac{e_p y_b}{I_{p,gross}} \right) + \frac{M_{dl} y_{b-p}}{I_{p}} \]

\[+ \frac{M_{sidl} y_{b-p-c}}{I_{p-c}} \]

\[= 0.261 \text{ ksi} \]

\[ \leq -0.19 \sqrt{f_c} = -0.537 \text{ ksi Ok!} \]

**Minimum Reinforcement**

Minimum reinforcement is checked after getting the strand number from Service III for a certain span, spacing and girder type. LRFD 5.7.3.3.2 requires that at any section of a flexural component, the amount of prestressed and non pretressed tensile reinforcement shall be adequate to meet:

\[ \phi M_u \geq 1.33 M_u \]

or \[ \phi M_u \geq 1.2 \ M_{cr} \]

in which, \[ M_{cr} = S_c \left( f_c + f_{vpe} \right) - M_{nc} \left( \frac{S_c}{S_{nc}} - 1 \right) \]

Precast section is assumed uncracked. \[ \phi M_u > 1.33 M_u \] should be met. Flexural Strength I at precast section (refer to the following section for hand calculation)

gives \[ \phi M_u = 3775 \text{ k.ft} > 1.33 M_u = 1.33(1689) = 2245 \text{ k.ft Ok!} \]

For composite section, Flexural Strength I at composite section gives

\[ \phi M_u = 4588 \text{ k.ft} > 1.33 M_u = 1.33(4370) = 5812 \text{ k.ft Ok!} \]
If this requirement cannot be met, then check $\phi M_u > 1.2 M_{cs}$. At least one of the two requirements should be met.

**Girder Top Fatigue**

Fatigue live load moment with impact is 8475 kip-in. Using the data from Service III calculation, it is easy to check the girder top compression fatigue by using fatigue load. Compression stress is positive in the equation below.

$$f_c = P_1 \left( \frac{1}{A_{n-i}} - \frac{e_p y_{t-n-i}}{I_{n-i}} \right) + \frac{M_g * y_t}{l_{g}}$$

$$- \Delta P_{p-loss 1} \left( \frac{1}{A_{gross}} - \frac{e_p y_t}{I_{gross}} \right)$$

$$+ \frac{M_d y_{t-n}}{I_{n}}$$

$$+ \frac{M_{sid}}{I_{n-c}}$$

$$- \Delta P_{p-loss 2} \left( \frac{1}{A_{gross-c}} - \frac{e_p net \cdot y_{t-gross-c}}{I_{gross-c}} \right)$$

$$+ \frac{M_{ll} * y_{t-n-c}}{I_{n-c}}$$

$$= 1.088 \text{ ksi}$$

$$< 0.4f'_c = 3.2 \text{ ksi OK!}$$

**FLEXURAL STRENGTH I AT POSITIVE SECTION**

The critical section is 0.4L from the abutment. No distribution is made between moments and shears applied to the non-composite or composite sections for strength computations. The factored loads, equal to 4370 k-ft, are applied to the composite section. The reinforcement bars in the deck are taken as the minimum deck steel area, which is #4 bar
@ 12” at the top and #5 bar @ 12” at the bottom. The effective prestress is 178.2 ksi from Service III calculation. The value of the compression block depth, a, is found by iteration. From the spreadsheet, a = 4.835 in. This indicates that the compression block is within the deck.

![Fig. A-3 Strength I at Composite Section](image)

Deck $\beta_1 = 0.85-0.05 \times (f'_c-4) = 0.85-0.05 \times (4-4) = 0.85$

c = a/\beta_1 = 4.835 / 0.85 = 5.688 \text{ in.}$

The effective deck width is 72” (Refer to Page 29 about cross section).

The compression force and moment in the deck slab is

$F_{c1} = -0.85 \times 4 \times (72 \times 4.835) = -1183.5 \text{ kips}$

$M_{c1} = -1183.5 \times 4.835 / 2 = -2861.0 \text{ k.in.}$

Bottom steel: There are 20 – 0.6” Grade 270 strands at the bottom flange. For the bottom
layer strands,

\[
\varepsilon_i = 0.003 \times \left( \frac{y_i}{c} - 1 \right) + \frac{f_{\mu}}{E_{\mu}} = 0.003 \times \left( \frac{49.3}{5.688} - 1 \right) + \frac{178}{28500} = 0.029
\]

Using the “Power formula” PCI Bridge Design Manual Section (8.2.2.5-1)

\[
f_{ps} = \varepsilon_{ps} \left[ 887 + \frac{27613}{1 + (112 \cdot 4 \varepsilon_{ps})^{3.36}} \right]^{1/3.36} \leq 270
\]

\[f_{ps1} = 270 \text{ ksi}\]

\[F_{s1} = 270 \times 3.906 = 1054.6 \text{ kips}\]

Moment based on compression fiber is \(M_{s1} = 1054.6 \times 49.3 = 41992.8 \text{ k.in.}\)

The calculation procedure for second bottom strands and top strands are the same as the bottom strands. The result of second layer bottom strands is

\[F_{s2} = 117.2 \text{ kips};\]

\[M_{s2} = 117.2 \times 47.3 = 5542.6 \text{ kip-in.}\]

The result of top strands is

\[F_{s3} = 45.5 \text{ kips};\]

\[M_{s3} = 45.5 \times 9.8 = 443.1 \text{ kip-in.}\]

The stress of #4 deck bars is \(f_s = E_s \varepsilon_s = 29000 \times 0.003 (3 / 5.688 - 1) = -41.1 \text{ ksi}\)

These bars are in the effective compression depth. The area needs to be reduced from the concrete area. Therefore the modified stress is \(-41.1 + 0.85 \times 4 = -37.7 \text{ ksi}\)

\[A_s = 0.2 \times 6 = 1.2 \text{ in.}^2\]

\[F_{s3} = -41.1 \times 1.2 = -45.3 \text{ kips}\]

\[M_{s3} = -45.3 \times 3 = -135.8 \text{ k.in.}\]

The calculation for bottom #5 bar is the same as above.
Adding steel force and concrete force together, equilibrium can be achieved. Add all the moment together, the flexural moment capacity is obtained: \( \phi M_u = 4588 \text{ k.ft} > M_u = 4370 \text{ k.ft} \)

with \( \phi = 1.0 \) which follows Equation \( 0.75 \leq \phi = \left( 1.75 + 250 e_{extreme} \right)/3 \leq 1.0 \) (LRFD 5.5.4.2). The strength reduction factor is one because the tension strain is larger than 0.005, called “tension control” in LRFD.

**CONCRETE STRENGTH AT RELEASE AT 0.4L**

Required by NDOR policy, girder release design should follow Strength Design Method in which the strength design for prestress transfer can be approached in a manner similar to that for non-prestressed reinforced concrete. The member can be treated as a “reinforced concreted column subjected to moment combined with axial compression force equal to the force in the prestressing steel just before prestress transfer.” Therefore, the author can solve for the neutral axis location “c” and \( f_{ci} \) by using the equilibrium equations.
Moment–curvature method is used in the force equilibrium calculation method. The equations that were used in the analysis are as follows:

1. Strain calculation: \( \Delta \varepsilon_{ci} = -0.003 \times \left( 1 - \frac{\gamma_{ci}}{c} \right) \)

2. Force equilibrium:

   If the effective concrete includes the strands inside, the exact force of concrete should be \( 0.85\gamma_{ci}(A_c-A_s)=0.85 \gamma_{ci}A_c-0.85 \gamma_{ci}A_s \). 0.85\gamma_{ci}A_s is considered when the author calculated the force of strands in order to avoid the calculation of the centroid of the concrete area.

\[
F_{\text{int prior}} = F_{\text{external}}
\]

\[
F_c - F_s = \frac{\gamma_p}{\phi} A_{pi} f_{pi}
\]

\[
\sum 0.85 \gamma_{ci} (A_c - A_s) - \sum A_s \Delta f_{ci} = \frac{\gamma_p}{\phi} A_{pi} f_{pi}
\]
Rearranging the items in the equation, the author gets

\[
\sum F = 0.85 f_{ci} \sum A_{ij} - \sum A_i \left( \Delta f_u + \frac{\gamma f_u}{\phi} f_u + 0.85 f_{ci} w_i \right) = 0 \quad \begin{cases} 
\gamma_u > a & w_i = 1 \\
\gamma_u < a & w_i = 0
\end{cases}
\]

3. Moment equilibrium:

\[
\sum M = 0.85 f_{ci} \sum A_{ij} y_{ij} - \sum A_i \left( \Delta f_u + \frac{\gamma f_u}{\phi} f_u + 0.85 f_{ci} w_i \right) y_u - \frac{\gamma f_u}{\phi} M = 0 \quad \begin{cases} 
\gamma_u > a & w_i = 1 \\
\gamma_u < a & w_i = 0
\end{cases}
\]

4. Power formula to get strand stress:

\[
f_u = \varepsilon_u E_u \left[ Q + \frac{1 - Q}{\left\{ 1 + \left( \frac{\varepsilon_u E_u}{k f_y} \right)^n \right\}^{1/R}} \right]
\]

By solving the two equilibrium equations, the two variables “c” and \( f_{ci} \) can be obtained.

The analysis can be done by trying “c” and \( f_{ci} \) and checking the equilibrium equations.

Actually, the critical section is the transfer length at 60d_b or the lifting point. In the chart design, \( f_{ci} \) at 0.4L is calculated because at this section the strands cannot be draped. If this section is fine, the analysis can be continued. The following is the last trial of the analysis:

\[c = 48.0 \text{ in., } f_{ci} = 2.186 \text{ ksi}\]

\[\beta = 0.85 - 0.05 \times (2.186 - 4) = 0.85 - 0.05 \times (2.186 - 4) > 0.85, \text{ use } 0.85.\]

\[a = c \times \beta = 48.0 \times 0.85 = 40.8 \text{ in.}\]

**Concrete Force Calculation**

Divide NU1100 into 5 layers (the more layers, the more exact).
Table A-14 Concrete Layers Division

<table>
<thead>
<tr>
<th>Concrete layer</th>
<th>Width (in.)</th>
<th>Thickness (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>38.4</td>
<td>5.3</td>
</tr>
<tr>
<td>2</td>
<td>22.15</td>
<td>5.5</td>
</tr>
<tr>
<td>3</td>
<td>5.9</td>
<td>28.2</td>
</tr>
<tr>
<td>4</td>
<td>27.05</td>
<td>1.75</td>
</tr>
<tr>
<td>5</td>
<td>48.2</td>
<td>2.56</td>
</tr>
</tbody>
</table>

All the five concrete layers are in compression. Concrete cracks a little in the fifth layer.

1. Concrete layer 1

\[ F_{c1} = 0.85 \times 2.186 \times (38.4 \times 5.3) = 378 \text{ k} \]

All the moment is calculated based on the extreme compression fiber.

\[ M_{c1} = 378 \times 5.3/2 = 1002 \text{ Kips-IN} \]

2. Concrete layer 2

\[ F_{c2} = 0.85 \times 2.186 \times (22.15 \times 5.5) = 226 \text{ k} \]

\[ M_{c2} = 226 \times (5.5/2+5.3) = 1822 \text{ Kips-IN} \]

3. Concrete layer 3

\[ F_{c3} = 0.85 \times 2.186 \times (5.9 \times 28.2) = 309 \text{ k} \]

\[ M_{c3} = 309 \times [28.2/2+5.3+5.5] = 7694 \text{ Kips-IN} \]

4. Concrete layer 4
\[ F_{c4} = 0.85 \times 2.186 \times (27.05 \times 1.75) = 88 \text{ k} \]
\[ M_{c4} = 88 \times [1.75/2 + 5.3 + 5.5 + 28.2] = 3506 \text{ Kips-IN} \]

5. Concrete layer 5

In this layer, concrete cracks. The uncracked layer thickness is:
\[ 40.8 - (1.75 + 5.3 + 5.5 + 28.2) = 0.06 \text{ in.} \]
\[ F_{c5} = 0.85 \times 2.186 \times (48.2 \times 0.06) = 5.0 \text{ k} \]
\[ M_{c5} = 5 \times (0.06/2 + 1.75 + 5.3 + 5.5 + 28.2) = 220 \text{ Kips-IN} \]

Adding all the force together, \( F = 1007 \text{ k} \).

Adding all the moment together, \( M = 14244 \text{ Kips-IN} \)

**Steel Force Calculation**

There are 4 – 0.5” 270 ksi strands on the top flange of the girder, which is 1.75” from the top edge. \( \Delta \varepsilon_s = -0.003 \times \left( 1 - \frac{41.55}{48.0} \right) = -0.0004 \)

Total strain is \( \varepsilon_s = \frac{13.2}{28500} - 0.0004 = 0.0001 \)
\[ f_{si} = \varepsilon_{si} E_s \left[ Q + \frac{1 - Q}{1 + \left( \frac{\varepsilon_{ps} E_s}{k f_{ps}} \right)^{1/R}} \right] \]

\[ = -0.0001 \times 28500 \left[ 0.031 + \frac{1 - 0.031}{1 + \left( \frac{0.0001 \times 28500}{1.04 \times 270} \right)^{2.36}} \right] \]

\[ = -12.1 \text{ ksi} \]

The top strands are not in the effective concrete zone, which means that they do not affect the pure concrete area calculation.

Hence, \[ F_s = A_i \left( \Delta f_{si} + \frac{r_p}{\phi} f_s \right) = 0.61(-12.1+0.75/1.15 \times 202.5) \approx 5 \text{ k.} \]

\[ M_s = 5 \times 41.55 = 200 \text{ Kips-IN} \]

There are 18 strands in the first bottom layer and 2 in the second bottom layer in the tension side. The calculations are the same as above except that the strands are in the effective concrete. The force of 18 strands is

\[ F_s = F_s = A_i \left( \Delta f_{si} + \frac{r_p}{\phi} f_s + 0.85 f_{sci} w_i \right) = 3.91(-82+0.75/1.15 \times 202.5+0.85 \times 2.186) \approx 900 \text{ k.} \]

The other calculation is omitted here. The second bottom strands are the same.

The total force and moment of strands are

\[ \sum F_s = 1007 \text{ k} \]

\[ \sum M_s = 2428 \text{ k.in} \]

\[ \sum F_c - \sum F_s = 1007 - 1007 = 0 \text{ k} \]

The external moment is the girder’s self-weight at 0.4L. The load factor is 0.85 because
the girder’s self-weight helps resist the high prestressing force. If the girder’s self-weight is negative in lifting action, the load factor is 1.15 because the moment worsens the situation.

\[ M = \frac{0.4 \times 0.6 \times 100^2}{2} \times 12 = 10426 \text{ k.in.} \]

\[ \sum M_e - \sum M_i - \frac{Y_n}{\phi} M = 14244 - 2428 - \frac{0.85}{0.75} \times 10426 = 0 \text{ k.in} \]

**STRENGTH I AT PRECAST POSITIVE SECTION AT 0.4L**

The precast section takes the load from the deck weight and girder self-weight. A strain-compatibility approach was used to calculate the flexural strength (refer to Fig. A-5 for details).

**Fig. A-5 Strength I Calculation at Precast Section**

The cross section is still divided into five rectangular blocks. The compression extreme fiber is the girder top flange edge. The last cycle of the iterative analysis will be redone...
by longhand calculation below to explain the spreadsheet analysis and to check its results:

The value of the compression block depth, from the spreadsheet, \( a = 4.126 \) in. This indicates that the compression block is within the top flange of the girder.

\[
\beta_1 = 0.85 - 0.05 \times (f'c - 4) = 0.85 - 0.05 \times (8 - 4) = 0.65
\]

\[
c = \frac{a}{\beta_1} = \frac{4.126}{0.65} = 6.347 \text{ in.}
\]

The compression force in the first concrete layer is

\[
F_{c1} = 0.85 \times 8 \times (48.2 \times 2.56) = -839.1 \text{ k}
\]

\[
M_{c1} = -839.1 \times 2.56/2 = -1074 \text{ Kips-IN}
\]

Similarly, the compression force in the second layer is \(-288.0\) kips and \(-962.6\) kip-in. The effective prestress stress in the strands is 178 ksi from Service III analysis.

Bottom steel: \( \varepsilon = 0.003 \times \left( \frac{41.3}{c} - 1 \right) + \frac{178}{28500} = 0.0228 \)

Using the “Power formula” PCI Bridge Design Manual Section (8.2.2.5-1)

\[
f_{ps1} = \varepsilon_{ps} \left[ 887 + \frac{27613}{\left( 1 + \left( 112 \times 0.4 \varepsilon_{ps} \right)^{1/7} \right)^{1/7} \text{ksi} } \right] \leq 270
\]

\( f_{ps1} = 265.7 \text{ ksi} \)

\( F_{s1} = 265.7 \times 3.906 = 1037.8 \text{ kips} \)

Moment based on compression fiber is \( M_{s1} = 1037.8 \times 41.3 = 42859.0 \text{ k.in.} \)

The calculation procedure for top strands and second layer strands is the same as the bottom strands. If the strands are within the compressed concrete, the steel area needs to be subtracted from the relative concrete area.

Adding steel force and concrete force together, equilibrium can be achieved. The strength
reduction factor is 1.0 because the tension strain is larger than 0.005, called “tension control” in LRFD. Adding all the moment together, the flexural moment capacity is obtained:

$$\phi M_n = 3775 > M_u = 1689 \text{ k.ft}$$

with $\phi = 1.0$ which follows Equation $0.75 \leq \phi = (1.75 + 250 \varepsilon_{\text{extreme}}) / 3 \leq 1.0$ (LRFD 5.5.4.2)

STRENGTH I AT THE PRECAST NEGATIVE SECTION

The negative moment section near the pier is analyzed as a reinforced section. Strength I requirement ($\phi M_n \geq M_u$) at the precast section should be met at design. The calculation is the same as Strength I at the precast positive section at 0.4L. But the compression is at girder’s bottom flange. The strength reduction factor, $\phi$, follows

$$0.75 \leq \phi = 0.65 + 50 \varepsilon_{\text{extreme}} \leq 0.9 \text{ (LRFD 5.5.4.2).}$$

The prestress at the face of diaphragm is

$$(1.5 \times 12) / (60 \times 0.6) \times 178 = 69.3 \text{ ksi.}$$

From the calculation, $\phi M_n = 2192 \text{ k.ft} \geq 1.33 M_u = 1.33 (874 \text{ k.ft}) = 1162.4 \text{ k.ft.}$ Therefore the minimum requirement is met.
STRENGTH I AT THE COMPOSITE NEGATIVE SECTION

The negative moment section at the pier is analyzed as a reinforced section. Strength I requirement ($\phi M_n \geq M_n$) at the composite section should be met in the design. The ideal design policy is that the joint steel takes the deck weight only, and the deck steel takes the superimposed dead load plus the live load (SIDL+LL). Due to the room limit, designers cannot put enough bars to take SIDL+LL. Therefore designers use maximum standard quantity of bars (2#8 between the minimum reinforcement) in the deck and use TR to take the rest of the moment. The calculation is the same as Flexural Strength I at the positive section. But the compression is at the girder’s bottom flange. The strength reduction factor, $\phi$, follows $0.75 \leq \phi = 0.65 + 50 e_{\text{extreme}} \leq 0.9$ (LRFD 5.5.4.2).
Using 4.74 in.\(^2\) TR and 22.02 in.\(^2\) deck reinforcement, \(\phi M_u = 6324\) k.ft

\[ \geq 1.33 M_u = 1.33(3563) = 4738.8 \text{ k.ft. Therefore the minimum requirement is met.} \]

---

**Fig. A-7 Strength I at Composite Negative Section**

**FATIGUE AT THE NEGATIVE SECTION**

This calculation starts with the same data of negative reinforcement and \(f'_c\) as in the last step, analysis of Flexural Strength I at negative section. If any limit as shown above cannot be met, keep TR area and \(f'_c\) unchanged, and increase deck reinforcement up to the maximum value. If it does not work, TR will be increased up to 10 1-3/8 TR. If it still does not work, \(f'_c\) will be increased up to 11 ksi. After iteration, the result is:

\[ f'_c = 8 \text{ ksi} \]

TR area is 4.74 ksi

Deck bar area is 22.02 in.\(^2\)
Non-composite Section Analysis

Precast section is composed of strands, concrete and TR. The moment of Area method is used to calculate precast section properties. Iteration to get the sum of moment of area equal to zero gives Neutral axis (N.A.) depth (kd) equal to 7.28 in. Therefore two concrete layers remain uncracked. Ignore the prestressing force. It is easy to get the moment of inertia, as 45537.8 in.\(^4\) based on N.A. The data in calculation of kd are shown in the following table.

Table A-15 Cracked Precast Section Properties

<table>
<thead>
<tr>
<th>(f'_c)</th>
<th>Width, b</th>
<th>Thick., T</th>
<th>Revised T</th>
<th>N</th>
<th>(A_{tr})</th>
<th>(y-y_{NA})</th>
<th>I</th>
<th>Moment of area</th>
</tr>
</thead>
<tbody>
<tr>
<td>ksi</td>
<td>in.</td>
<td>in.</td>
<td>in.</td>
<td>in.2</td>
<td>in</td>
<td>in.4</td>
<td>k.in</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>38.4</td>
<td>5.3</td>
<td>5.3</td>
<td>1</td>
<td>203.5</td>
<td>-4.6</td>
<td>4835.7</td>
<td>-941.91</td>
</tr>
<tr>
<td>8</td>
<td>22.15</td>
<td>5.5</td>
<td>1.978095</td>
<td>1</td>
<td>43.8</td>
<td>-1.0</td>
<td>57.1</td>
<td>-43.33494</td>
</tr>
<tr>
<td>8</td>
<td>5.9</td>
<td>28.19</td>
<td>0</td>
<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>27.05</td>
<td>1.75</td>
<td>0</td>
<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>8</td>
<td>48.2</td>
<td>2.56</td>
<td>0</td>
<td>1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Area</th>
<th>Depth y</th>
<th>(E_s)</th>
<th>(n)</th>
<th>(A_{tr})</th>
<th>(y-y_{NA})</th>
<th>I</th>
<th>Moment of area</th>
</tr>
</thead>
<tbody>
<tr>
<td>in.(^2)</td>
<td>in.</td>
<td>ksi.</td>
<td>in.2</td>
<td>in</td>
<td>in.4</td>
<td>k.in</td>
<td></td>
</tr>
<tr>
<td>0.612</td>
<td>41.55</td>
<td>28500</td>
<td>5.36</td>
<td>3.3</td>
<td>34.3</td>
<td>3855.0</td>
<td>112.5</td>
</tr>
<tr>
<td>0.434</td>
<td>4</td>
<td>28500</td>
<td>5.36</td>
<td>1.9</td>
<td>-3.3</td>
<td>20.3</td>
<td>-6.2</td>
</tr>
<tr>
<td>3.906</td>
<td>2</td>
<td>28500</td>
<td>5.36</td>
<td>17.0</td>
<td>-5.3</td>
<td>474.7</td>
<td>-89.9</td>
</tr>
<tr>
<td>4.74</td>
<td>44.7</td>
<td>29000</td>
<td>5.46</td>
<td>25.9</td>
<td>37.5</td>
<td>36295.0</td>
<td>968.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>295.4</td>
<td>45537.8</td>
</tr>
</tbody>
</table>
Concrete

In the concrete layer, the stress is trapezoid.

\[ F = \int_{b}^{\Delta t} \sigma w ds \]
\[ = w \int_{b}^{\Delta t} \sigma ds \]
\[ = w \frac{\Delta t (\sigma_1 + \sigma_2)}{2} \]
\[ = A \frac{\Delta t (\sigma_1 + \sigma_2)}{2} \]

The force center is located at \( \frac{\Delta t (2\sigma_2 + \sigma_1)}{3(\sigma_2 + \sigma_1)} \) from the top fiber of the layer (PCI Design Handbook P11-30). Therefore the distance from the force centerline to the layer centerline is \( \frac{\Delta t}{2} - \frac{\Delta t (2\sigma_2 + \sigma_1)}{3(\sigma_2 + \sigma_1)} = - \frac{\Delta t (\sigma_2 - \sigma_1)}{6(\sigma_2 + \sigma_1)} \). The moment from the force centerline to the layer centerline is \( M = F \left( - \frac{\Delta t (\sigma_2 - \sigma_1)}{6(\sigma_2 + \sigma_1)} \right) \).

According to similar triangle theory, \( \frac{\sigma_1}{\sigma_2} = \frac{y_{N.A} - y - \frac{\Delta t}{2}}{y_{N.A} - y + \frac{\Delta t}{2}}, \quad \frac{\sigma_2 - \sigma_1}{\sigma_2 + \sigma_1} = \frac{\Delta t}{2(y_{N.A} - y)} \)

Therefore, \( M = \frac{F (\Delta t)^2}{12 (y - y_{N.A})} \)

The force caused by strands, TR and concrete can be calculated. Equilibrium can be achieved as long as the moment of area is equal to zero. The following is the detailed checking.

Concrete Layer 1

The product of curvature and Modulus of Elasticity, \( \phi E = 0.18 \text{ ksi} \)
Stress, \( \sigma = \phi E (y_{\text{centroid}} - y_{N_A}) = 0.18(5.3/2 - 7.28) = -0.85 \text{ ksi} \)

Force, \( F = -0.85 \times 38.4 \times 5.3 = -173.5 \text{ k} \)

Moment from force center to layer center, \( M_1 = \frac{F (\Delta t)^2}{12 (y - y_{N, A})} = \frac{-173.5 (5.3^2)}{12 (5.3/2 - 7.28)} = 88 \text{ k.in} \)

Moment on force center is \( M_2 = -173.5 (5.3/2 - 7.28) = 891 \text{ k.in} \).

The total moment of this layer is \( M = M_1 + M_2 = 88 + 891 = 941.9 \text{ k.in} \).

**Concrete Layer 2**

Thickness, \( T = 1.978 \text{ in} \).

Stress, \( \sigma = \phi E (y_{\text{centroid}} - y_{N_A}) = 0.18(5.3 + 1.978/2 - 7.28) = 0.18 \times (-0.991) = -0.18 \text{ ksi} \)

Force, \( F = -0.18(22.15 \times 1.978) = -8.0 \text{ k} \)

Moment, \( M = M_1 + M_2 = -8.0 (-1.0) + \frac{-8.0 (1.978^2)}{12 (-1.0)} = 11 \text{ k.in} \).

**TR**

Ratio of Modulus of Elasticity, \( n = \frac{E_s}{E_c} = \frac{29000}{5324} = 5.46 \)

Concrete stress at TR centroid, \( \sigma = \phi E (y_{\text{centroid}} - y_{N_A}) = 0.18(44.7 - 7.28)(5.46) = 6.9 \text{ ksi} \)

TR stress, \( f = 6.9 \times 5.46 = 37.66 \text{ ksi} \)

Force, \( F = \sigma A_s = 6.69 \times 4.74 = 178.5 \text{ k} \)

Moment, \( M = 178.5 (44.74 - 7.28) = 6686 \text{ k.in} \).

**Strands**
Ratio of Modulus of Elasticity, \( n = \frac{E_s}{E_c} = \frac{28500}{5324} = 5.36 \)

Concrete Stress at steel centroid, \( \sigma = \phi E (y_{\text{centroid}} - y_{sa}) = 0.18(2 - 7.28) = -1.0 \) ksi

Because the stands are in the compression zone, \( F = \sigma A_s (n - 1) = -1.0 (3.906)(5.36 - 1) = -16.6 \) k

Moment, \( M = -16.6(2 - 7.28) = 87 \) k.in.

The calculation for the other strand layer is the same as above.

Adding all the force together, it will come to zero. Therefore will be the moment.

The calculation for composite section is the same as for the precast section above. The only differences are: Bars are taken into consideration and deck works together with the beam. The relative calculation is the same for strands and TR. The procedure is omitted here.

\( \phi E = 0.04 \) ksi

and \( k_d = 17.5 \) in.

**Concrete Fatigue Limit Checking**

In the precast section, the total moment is the sum of moment caused by eccentric prestress force and deck weight. \( M_{\text{total}} = 1099 + 9197 = 10296 \) k.in. Stress due to deck weight at compression fiber, \( \sigma = \frac{M_{\text{deck}} \sigma_{\text{extreme}}}{M_{\text{total}}} = (0.18)(7.28) = 1.34 \) ksi

In the composite section, the stress due to SIDL is \( \sigma = \frac{(3429)(0.04)(17.5)}{8343} = 0.29 \) ksi;
the stress due to fatigue LL is \( \sigma = \frac{(4925)(0.04)(17.5)}{8343} = 0.42 \text{ksi} \)

\[ 0.5f_{DL} + f_{LL} = 0.5(1.34 + 0.29) + 0.42 = 1.24 \text{ ksi} \leq 0.4f' = 3.2 \text{ ksi} \text{ Ok!} \]

**TR Fatigue Limit Checking**

At the non-composite section, TR stress is \( f = 37.66 \text{ ksi} \)

At the composite section, TR stress is \( f = 1.1 \times 5.46 = 6.07 \text{ ksi} \)

\( f_{min} = 37.66 + 6.07(3428/8343) = 40.15 \text{ ksi} < 54 \text{ ksi limit, Okay!} \)

\( f_r = \frac{M_{LL} \sigma_{TR}}{M_{total}} = \frac{(4915)(6.07)}{8343} = 3.58 \leq 18 \text{ ksi OK!} \)

**Bar Fatigue Limit Checking**

In the composite section, the total stress of the top layer bars is 7.3 ksi. The stress due to SIDL is \( f_{min} = \frac{(3428)(7.3)}{8343} = 3.01 \text{ ksi} \)

The stress due to fatigue live load moment is

\[ f_r = \frac{(4915)(7.3)}{8343} = 4.31 \leq 24 - 0.33 f_{min} = 24 - 0.33(7.3) = 23.0 \text{ ksi} \]

**CRACK CONTROL NEGATIVE SECTION**

The calculation of moment of area, curvature, force, moment, etc, is the same as in the Fatigue Negative Section calculation. The only difference is the LL calculation. Here, Service I live load is used instead of fatigue live load. After everything is set, iteration is used for both the precast section and the composite section to calculate N.A. depth, which
makes the moment of area equal to zero. The top stress of bars is 15.0 ksi.

\[ s \leq \frac{437.5}{15.0} - 2(3.5) = \frac{437.5}{15.0} - 7 = 22.2'' \]

and \( s \leq 1.5t_s = 1.5(7) = 10.5'' \) controls

and \( s \leq 18'' \)

The top bar spacing is 3 inches, which is less than the maximum spacing limit of 10.5 inches. Ok!

**LIVE LOAD DEFLECTION**

Simply use Service I live load at positive section per 0.1L as the deflection moment.

Calculation of deflection at each 0.1L location is based on Elastic weight or Moment-area method. The method is shown as Fig. A-8.

![Moment-area Method to Calculate Deflection](image)

Fig. A-8 Moment-area Method to Calculate Deflection
The deflection equations for each 0.1L are

\[
\Delta_{0.1L} = \frac{L^2}{6000} \left(9\Phi_0 + 44\Phi_1 + 48\Phi_2 + 42\Phi_3 + 36\Phi_4 + 30\Phi_5 + 24\Phi_6 + 18\Phi_7 + 12\Phi_8 + 6\Phi_9 + \Phi_{10}\right)
\]
\[
\Delta_{0.2L} = \frac{L^2}{3000} \left(4\Phi_0 + 24\Phi_1 + 43\Phi_2 + 42\Phi_3 + 36\Phi_4 + 30\Phi_5 + 24\Phi_6 + 18\Phi_7 + 12\Phi_8 + 6\Phi_9 + \Phi_{10}\right)
\]
\[
\Delta_{0.3L} = \frac{L^2}{6000} \left(7\Phi_0 + 42\Phi_1 + 84\Phi_2 + 116\Phi_3 + 108\Phi_4 + 90\Phi_5 + 72\Phi_6 + 54\Phi_7 + 36\Phi_8 + 18\Phi_9 + 3\Phi_{10}\right)
\]
\[
\Delta_{0.4L} = \frac{L^2}{3000} \left(3\Phi_0 + 18\Phi_1 + 36\Phi_2 + 54\Phi_3 + 67\Phi_4 + 60\Phi_5 + 48\Phi_6 + 36\Phi_7 + 24\Phi_8 + 12\Phi_9 + 2\Phi_{10}\right)
\]
\[
\Delta_{0.5L} = \frac{L^2}{1200} \left(\Phi_0 + 6\Phi_1 + 12\Phi_2 + 18\Phi_3 + 24\Phi_4 + 28\Phi_5 + 24\Phi_6 + 18\Phi_7 + 12\Phi_8 + 6\Phi_9 + \Phi_{10}\right)
\]
\[
\Delta_{0.6L} = \frac{L^2}{3000} \left(2\Phi_0 + 12\Phi_1 + 24\Phi_2 + 36\Phi_3 + 48\Phi_4 + 60\Phi_5 + 67\Phi_6 + 54\Phi_7 + 36\Phi_8 + 18\Phi_9 + 3\Phi_{10}\right)
\]
\[
\Delta_{0.7L} = \frac{L^2}{6000} \left(3\Phi_0 + 18\Phi_1 + 36\Phi_2 + 54\Phi_3 + 72\Phi_4 + 90\Phi_5 + 108\Phi_6 + 116\Phi_7 + 84\Phi_8 + 42\Phi_9 + 7\Phi_{10}\right)
\]
\[
\Delta_{0.8L} = \frac{L^2}{3000} \left(\Phi_0 + 6\Phi_1 + 12\Phi_2 + 18\Phi_3 + 24\Phi_4 + 30\Phi_5 + 36\Phi_6 + 42\Phi_7 + 43\Phi_8 + 24\Phi_9 + 4\Phi_{10}\right)
\]
\[
\Delta_{0.9L} = \frac{L^2}{6000} \left(\Phi_0 + 6\Phi_1 + 12\Phi_2 + 18\Phi_3 + 24\Phi_4 + 30\Phi_5 + 36\Phi_6 + 42\Phi_7 + 48\Phi_8 + 44\Phi_9 + 9\Phi_{10}\right)
\]

**Curvature Calculation**

Curvature is defined as \( \phi = \frac{M}{EI} \). Take the calculation of curvature at 0.4L as an example.

The section properties are the same as the prestress loss in Service III hand calculation.

The service I live load is not multiplied by 0.8. The upward curvature is positive. For example, the curvature caused by live load moment at 0.4L is calculated as shown below.

Due to Live load \( \phi = \frac{-M_g}{E_I I_{final-composite}} = \frac{-8527}{5314 (386692)} = -4.1E-06 \)

Calculate the curvature at each 0.1L with result shown below.
**Table A-16 Curvature at Each 0.1L**

<table>
<thead>
<tr>
<th>Location</th>
<th>0.0L</th>
<th>0.1L</th>
<th>0.2L</th>
<th>0.3L</th>
<th>0.4L</th>
<th>0.5L</th>
<th>0.6L</th>
<th>0.7L</th>
<th>0.8L</th>
<th>0.9L</th>
<th>1.0L</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live load φ (× E-06)</td>
<td>0</td>
<td>-1.7</td>
<td>-3.0</td>
<td>-3.8</td>
<td>-4.1</td>
<td>-4.1</td>
<td>-3.7</td>
<td>-2.9</td>
<td>-1.7</td>
<td>-6.4</td>
<td>-4.6</td>
</tr>
</tbody>
</table>

\[ \Delta_{0.5L} = \frac{L^2}{1200}(\Phi_0 + 6\Phi_1 + 12\Phi_2 + 18\Phi_3 + 24\Phi_4 + 28\Phi_5 + 24\Phi_6 + 18\Phi_7 + 12\Phi_8 + 6\Phi_9 + \Phi_{10}) \]

\[ = \frac{(100)^2}{1200} \begin{pmatrix} 0 - 6(1.7) - 12(3.0) - 18(3.8) - 24(4.1) - 28(4.1) \\ - 24(3.7) - 18(2.9) - 12(1.7) - 6(6.4) - 4.6 \end{pmatrix} \times 10^{-6} \]

\[ = -0.59 \text{ in.} \]

Therefore the deflection due to live load is -0.59 in. downward (refer to Excel spreadsheet attached for detailed calculation).
APPENDIX B: PROGRAMS FOR SERVICE DESIGN OF NEGATIVE MOMENT AND LIVE LOAD MOMENT AND SHEAR CALCULATIONS (SEE SEPARATE ELECTRONIC EXCEL FILE ON CD)
APPENDIX C- CHARTS AND TABLES (SEE SEPARATE ELECTRONIC EXCEL FILE ON CD)
APPENDIX D- MOMENT AND SHEAR CALCULATIONS DUE TO LIVE LOADS (SEE SEPARATE ELECTRONIC EXCEL FILE ON CD)
REFERENCES

   “Restrained Moments in Precast/Prestressed Concrete Continuous Bridges,” PCI
   JOURNAL, V. 43, No. 6, November-December 1998, pp. 40-57
4. Panya Noppakunwijai, et al., “Strength Design of Pretensioned Flexural Concrete