Finley Engineering Group, Inc.

BRIDGE DEVELOPMENT REPORT

SR-87 BRIDGES
OVER CLEAR CREEK

February 2013

Santa Rosa County, Florida
FDOT District Three

FPID(s): 416748-3-22-01 and 416748-3-22-02
FINLEY Project No. 09.6015

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EXECUTIVE SUMMARY

This project requires the construction of twin two-lane bridges to carry vehicle traffic and a possible future multi-use trail over Clear Creek as part of a new four-lane segment of SR 87. The new highway will link SR 87S at US 90 east of the city of Milton, Florida with SR 87N north of the city center. As Milton and the surrounding areas continue to be developed, it can be expected that traffic volumes will increase as reflected in projected traffic counts. This project will help alleviate travel demand on portions of US 90 that currently pass through downtown Milton using a shared designation with SR 87. The new corridor will also provide a direct hurricane evacuation route northward from coastal communities located on the Gulf of Mexico. SR 87 throughout this project is classified as a principal arterial. The new portion of SR 87 will be a divided four-lane semi-controlled-access highway based on the ultimate typical section. This report establishes recommended structural systems, material types and the basic constraints necessary to guide the work to be done in the final design for the crossing of Clear Creek.

The recommended structure was determined by developing key criteria applicable for this project. Once the key criteria had been established, several bridge alternatives combining a range of structural systems and construction materials were compared relative to the criteria. The key criteria used to evaluate the recommended structure for this project site included bridge construction economy, long-term maintenance, constructability, site access, channel hydraulics, navigation and aesthetics. The overall required length of the crossing was determined to be 180’-0” based on the Technical Memorandum detailing hydrologic and hydraulic investigation prepared by the Balmoral Group. The proposed bridge will be designed having a length and vertical clearance to provide hydraulic conveyance of storm events affecting Clear Creek. The proposed Southbound Bridge will have an extra wide should, than can later be converted to carry a 12’-0” multi-use trail separated from the traffic lanes by an F-shaped traffic barrier.

Upon evaluating several bridge alternatives it was determined that the preferred structure would consist of two parallel bridges each carrying two lanes of traffic in each direction. Each of the parallel bridges would be comprised of two spans measuring 90’-0” in length at the centerline of construction. The superstructure for the recommended bridge alternative would be comprised of 45” deep Florida-I Beams (FIBs) with a composite cast-in-place concrete deck. The substructure for this alternative uses pile bents supported by 24 inch prestressed concrete piles. The southbound bridge has an overall width of 56’-0$^{1}/_{2}$” including 52’-11 ½“ of clear roadway with shoulders. The northbound bridge has an overall deck width of 43’-1”, including 40’-0” of clear roadway with shoulders. The recommended structure provides effective construction economy and should require minimal life cycle maintenance.

Prestressed concrete Florida-I Beam bridges with prestressed concrete pile supported foundations are commonly constructed in Florida and should not pose any unusual construction difficulties for contractors pre-qualified to perform work for FDOT. Each of the twin bridges can be constructed simultaneously considering no traffic currently exists within the new highway alignment. This eliminates the need for phased construction and special MOT considerations. If funding is not available for the full four lane facility, the southbound bridge can be constructed in an initial phase of construction. The southbound bridge can then be used to carry one lane of traffic in each direction until funding becomes available for the full four lane SR 87 typical section.

SECTION 1

Intent of the Bridge Development Report

The goal of the Bridge Development Report (BDR) is to establish the type of foundation, substructure and superstructure for the proposed crossing of Clear Creek relative to key design criteria. Several bridge alternatives have been investigated based on this objective with the intent of recommending the optimal structure. Key considerations for this project included: construction economy, long-term maintenance, constructability, site access, channel hydraulics, navigation and aesthetics. In order to determine the optimum structure, the expertise of several engineering disciplines including that of structural, geotechnical, drainage and roadway designers have combined efforts to study various aspects of the project to evaluate the proposed bridge alternatives. Upon evaluating each of the bridge alternatives, the recommended bridge alternative will be established for final project development. This report will also establish basic constraints that will guide work to be done in the final design and plans preparation stage of the project.

Project Description and Location

This project will provide a new roadway alignment linking SR 87S with SR 87N northeast of Milton in Santa Rosa County, Florida. SR 87 has a shared route designation with US90 through downtown Milton from the intersection of US 90 with SR 87S east of Milton and the intersection of US 90 with SR 87N north of Milton. A "SR 87 Connector PD&E Study" was initiated by Metric Engineering in December 2010 for the Florida Department of Transportation and the Federal Highway Administration. The PD&E Study clearly demonstrated the need for the new facility based on several factors including; the importance of SR 87 as an emergency evacuation route, the social demand and economic development of Santa Rosa County, the failing level of service of US 90 from Ward Basin Road to SR 87N, and the safety/crash rate of the US 90/SR 87S intersection. Figure 1 shows the project location on a map of the surrounding area.

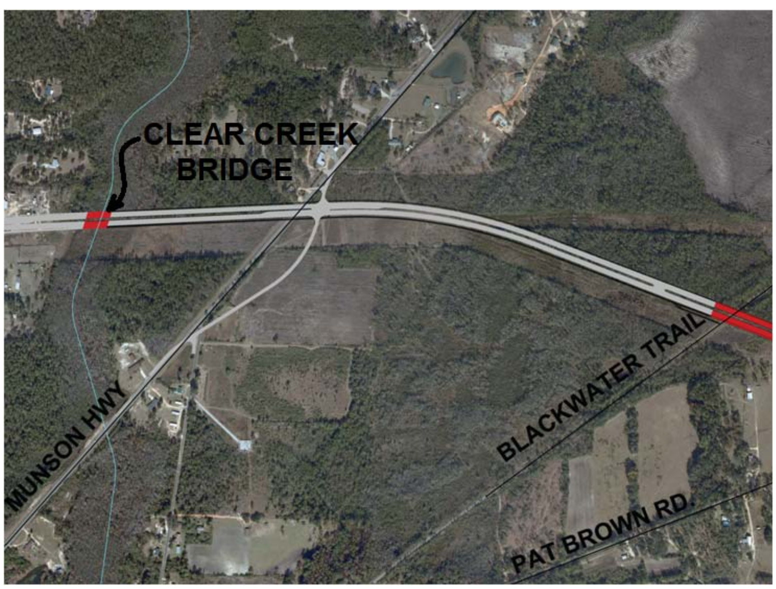


Figure 1 – Bridge Location Map

SECTION 2

Traffic Data and Highway Classification

SR 87 in Santa Rosa County is classified as a principal arterial. Traffic studies for the new alignment of SR 87 estimate the average daily traffic would be 10,731 vehicles daily for an opening year occurring in 2015. The following summarizes the estimated traffic data on the proposed SR 87 connector road:

Traffic AADT -	0	Current Year Estimate (2009) AADT
	10,761	Opening Year Estimate (2015) AADT
	19,746	Design Year Estimate (2035) AADT
Distribution -	K = 9.0%	
	D = 58.7%	
	24 Hour T = 5.0%	

The design speed for the project will be 45 mph with a posted speed limit of 45 mph. The traffic data noted above appears on the Typical Section Package and is included as Appendix D of this report.

Vessel Navigation of Clear Creek

Based on site visits, conversations with locals, and the information provided in the Technical Memorandum by The Balmoral Group, Clear Creek is a non-navigable creek. The low chord elevation of the bridge will be governed by the design flood elevation.

Considering the creek is non-navigable for commercial vessels, it will not be necessary to design bridge foundations and substructure components to be vessel impact resistant.

Proposed Bridge Geometry

The proposed structure will be comprised of twin bridges each carrying two lanes for each direction of traffic. The clear roadway width from the inside face of the traffic barriers will be 52’’-11 ½” on the southbound bridge and 40’-0” on the northbound bridge, including a 6’-0” inside shoulder, two 12’-0” travel lanes, and a 10’-0” outside shoulder for each of the bridges. The southbound bridge will have a 22’-11 ½” shoulder, that can later be modified to a 10’-0” shoulder, providing a corridor for a multi-use trail. The total coping-to-coping width of the southbound bridge will be 56’-0$^{1}/_{2}$” including the two F-shaped NCHRP TL-4 crash tested safety barriers. The total coping-to-coping width of the northbound bridge will be 43’-1” which includes two F-shaped NCHRP TL-4 crash tested safety barriers. The typical section for each bridge requires a cross-slope at a constant rate of 2.00% sloping downward from the median side of the bridges for the full width of the deck. Based on the tangent alignment in combination with the 45 mph design speed, superelevation and associated transitions will not be required throughout the limits of the bridge. The approved Typical Section Package reflecting the roadway section as outlined above is provided as Appendix D of this report.

The profile grade line (PGL) for each of the twin bridges has been set to coincide with the inside edge of the travel lanes at a distance of 6'-0" from the inside face of the median side traffic barriers equal to the inside shoulder width. The PGL for the southbound bridge is offset a distance of 20'-0" from the baseline of construction, and a distance of 20'-0" from the baseline of construction for the northbound bridge.

Preliminary Geotechnical Exploration Information

Environmental and Geotechnical Specialists, Inc. (EGS) conducted a subsurface investigation for the project in November of 2011. Two (2) Standard Penetration Test (SPT) soil borings were performed near the proposed location of the Blackwater Bridge, however no borings were taken at Clear Creek. The results of the preliminary investigation are presented in a separate report in the Phase I Geotechnical Investigation – Bridge Investigation for the SR 87 Connector PD&E Study dated November 30, 2011. Two soil samples were collected from soil boring B-1, and a water sample was collected from the Blackwater River. Based on the results from these samples, the environmental classification for the substructure of the Blackwater River Bridge is moderately aggressive for both concrete and steel. It can be reasonably assumed that a similar classification would be appropriate for the bridge over Clear Creek. The boring logs prepared by EGS were used to evaluate subsurface conditions and develop pile capacity curves for estimating BDR alternative pile lengths at the site. Although these aren't at the exact location of this bridge, the comparison is a relative comparison of alternates, and should be sufficient for this preliminary comparison. This information is included in Appendix F of this report.

The Geotechnical Investigation conducted by EGS determined that shallow foundations were not feasible for the project due to the relatively loose nature of the surface soils and the potential for scour instability. Therefore, only deep foundations were considered, including drilled shafts and driven piles. Drilled shafts could be considered as a viable foundation option only if a limestone bearing stratum was encountered within 100 feet or less of the existing ground surface, or if the axial and lateral loads for the bridge were expected to be high enough to justify the extra costs typically associated with drilled shafts. Since none of these conditions exist for the proposed BDR alternatives, drilled shaft foundations were not considered to be a cost effective foundation option. Consequently, driven piles were recommended as the most appropriate foundation system for this bridge. EGS prepared an axial capacity analysis comparing HP 14x73 H-Piles, 24-inch open-ended steel pipe piles, 18-inch square concrete, and 24-inch square concrete driven piles. EGS noted that steel piles are generally more expensive than concrete piling driven to the same capacity. Therefore, unless more extensive future subsurface investigations found significantly different results than those encountered during the preliminary investigation, EGS recommends square prestressed concrete piles to be the most appropriate and cost effective foundation option for the SR 87 Bridges over the Blackwater River. These same recommendations were used to develop the alternatives for Clear Creek. The BDR alternatives presented herein are based on foundations comprised of 18" or 24" square prestressed concrete piles depending on the pile capacity needed for each alternative.

SECTION 3

Bridge Design Criteria

This report was prepared in accordance with the latest revisions of the AASHTO LFRD Bridge Design Specifications, the FDOT Plans Preparation Manual, the FDOT Structures Design Manual and the desires of District Three as made known to Finley Engineering Group. The following summarizes the criteria that was used to prepare this report and will be used to develop the final plans and contract documents:

1.) Specifications

Construction:

- Florida Department of Transportation Standard Specifications for Road and Bridge Construction.

Design:

- AASHTO LRFD Bridge Design Specifications
- Florida Department of Transportation Structures Manual
- Florida Department of Transportation Plans Preparation Manual
- Florida Department of Transportation Design Standards

2.) Design Loadings

Dead Loads:

- Unit weight of concrete = 150 pcf
- Unit weight of structural steel = 490 pcf
- Future wearing surface = none
- Weight of S.I.P. forms = 20 psf
- Weight of concrete barrier (Index 420) = 420 plf

Live Loads:

- HL-93 Truck with impact and associated lane load.

Wind Loads:

- In accordance with the Florida Department of Transportation Structures Manual.

3.) Environment

- Substructure:
  Concrete = Moderately Aggressive (Soil Resistivity = 2,500 Ohm-cm,
  Water pH = 6.5)
  Steel = Moderately Aggressive (Soil pH = 6.2, Soil Resistivity = 2,500 Ohm-cm)
- Superstructure: Slightly Aggressive

4.) Hydraulic Evaluation

Deck Drainage:
- In accordance with the FDOT Drainage Manual.

Stream Hydraulics:
- In accordance with the FDOT Drainage Manual.

Scour:
- In accordance with the FDOT Drainage Manual.

5.) Roadway Geometry

Horizontal, Vertical and Superelevation:
- In accordance with the FDOT Plans Preparation Manual.

6.) Clearances

Vertical:
- In accordance with the Bridge Hydraulics Report and the FDOT Drainage Manual (No less than 2'-0" above the Design High Water elevation, 50 year event, throughout the length of the structure)
- No less than 6'-0" above the Mean High Water

Horizontal:
- In accordance with the Bridge Hydraulics Report (180'-0" minimum overall length).
- Tangent alignment, with 20° skew

SECTION 4

Superstructure Alternatives

The superstructure alternatives for the proposed bridges over Clear Creek were established such that they were appropriate for the site and incorporate bridge construction methods that are commonly used throughout Florida.

The following superstructure systems were evaluated for this report:

- Florida I-45 Beams with a cast-in-place composite slab. Each span is 90'-0" in length measured at the centerline of construction and is simply supported on the substructure elements of the bridge.
- Florida I-36 Beams with a cast-in-place composite slab. Each span is 90'-0" in length measured at the centerline of construction and is simply supported on the substructure elements of the bridge.
- 1'-10" thick reinforce concrete flat-slab superstructure. Each span is 36'-0" in length measured at the centerline of construction and is simply supported on the substructure elements of the bridge.

Preliminary concrete beam designs for the Florida I-beam alternatives were evaluated using the FDOT LRFD prestressed beam program. Preliminary design of the flat-slab alternate was performed using a Mathcad sheet developed in accordance with AASHTO LRFD and the FDOT SDG.

Substructure Alternatives

Per the Phase I Geotechnical Investigation, substructure alternatives were limited to deep foundations supported by prestressed concrete piling based on subsurface soil conditions, load carrying capacity, construction economy, constructability and long term durability. Drilled shaft foundations and driven steel piles, including open-ended pipe piles and H-piles, were eliminated based on construction economy when compared to prestressed concrete piles.

BDR alternatives considered in this report include preliminary pile bent and interior bridge pier designs in conformance with requirements of the AASHTO LRFD Specification and the FDOT Structures Design Guidelines. Foundation configurations for the proposed bridge BDR alternatives were established based on LRFD Strength Load Combinations I, III, and V..

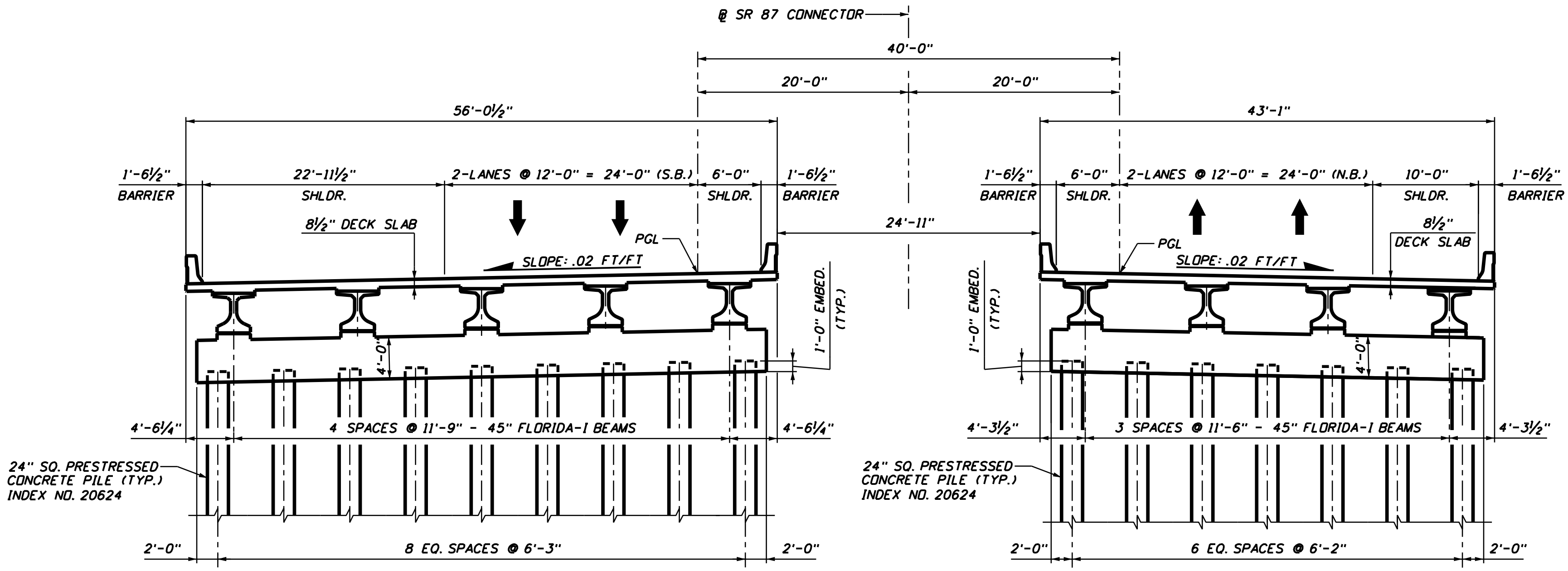
Due to the relatively short spans, all alternates were evaluated with typical pile bents. Large footings would not be economical, and are not necessary due to the small loads associated with the chosen span lengths. Further discussion of the subsurface conditions and foundation systems evaluated for this project is included in the Preliminary Geotechnical Report for Structures provided under a separate cover.

Summary of Alternatives Considered

Based on the superstructure and substructure considerations outlined above, three bridge alternatives were determined to be appropriate for the site and were evaluated for the proposed SR 87 Bridge over Clear Creek. None of the bridge alternatives considered should require elaborate construction techniques or extensive specialty construction engineering to build. It is not anticipated that contractors pre-qualified by FDOT to do bridge construction in the State of Florida will encounter any unusual construction difficulties associated with any of the alternatives considered.

Appendix C of this report includes detailed quantity and cost estimates, along with preliminary design documentation for the following alternatives:

- Alternative A – 2 spans of simply supported 45" Florida-I Beams with an 8.5" composite cast-in-place deck, including a 0.5" sacrificial wearing surface. Each span measures 90'-0" in length at the centerline of construction and consists of five (5) beams spaced at 11'-9" for the southbound bridge, and four (4) beams spaced at 11'-6" for the northbound bridge. The typical overall widths of the southbound and northbound bridges measure 56'-0½" and 43'-1" respectively, including provisions for a future multi-use trail on the southbound bridge. The interior bents are founded on a single line of 24 inch prestressed concrete piles with nine (9) piles for the southbound bridge and eight (8) piles for the northbound bridge. The end bents are comprised of a cast-in-place cap supported on a single line of six (6) 24 inch prestressed piles for the southbound bridge and five (5) 24 inch prestressed piles for the northbound bridge. Figure No. 2 shown at the end of this section provides a cross section of this alternative. Figure No. 3 shows a partial elevation view.
- Alternative B – 2 spans of simply supported 36" Florida-I Beams with an 8.5" composite cast-in-place deck, including a 0.5" sacrificial wearing surface. Each span measures 90'-0" in length at the centerline of construction and consists of seven (7) beams spaced at 8'-1 ½" for the southbound bridge, and five (5) beams spaced at 8'-6" for the northbound bridge. The typical overall widths of the southbound and northbound bridges measure 56'-0½" and 43'-1" respectively including provisions for a future multi-use trail on the southbound bridge. The interior bents are founded on a single line of 24 inch prestressed concrete piles with nine (9) piles for the southbound bridge and seven (7) piles for the northbound bridge. The end bents are comprised of a cast-in-place cap supported on a single line of six (6) 24 inch prestressed piles for the southbound bridge and five (5) 24 inch prestressed piles for the northbound bridge. Figure No. 4 shown at the end of this section provides a cross section of this alternative. Figure No. 5 shows a partial elevation view.
- Alternative C – 5 spans of simply supported 22" cast-in-place flat slabs, including a 0.5" sacrificial wearing surface. Each span measures 36'-0" in length at the centerline of construction. The typical overall widths of the southbound and northbound bridges measure 56'-0½" and 43'-1" respectively provisions for a future multi-use trail on the southbound bridge. The interior bents are founded on a single line of 18 inch prestressed concrete piles with seven (7) piles for the southbound bridge and six (6) piles for the northbound bridge. The end bents are comprised of a cast-in-place cap supported on a single line of five (5) 18 inch prestressed piles for the southbound bridge and four (4) 18 inch prestressed piles for the northbound bridge. Figure No. 6 shown at the end of this section provides a cross section of this alternative. Figure No. 7 shows a partial elevation view.



TYPICAL SECTION
(LOOKING UPSTATION)

FINLEY Engineering Group, Inc.

FIGURE NO. 2

ALTERNATIVE A
SR 87 OVER CLEAR CREEK

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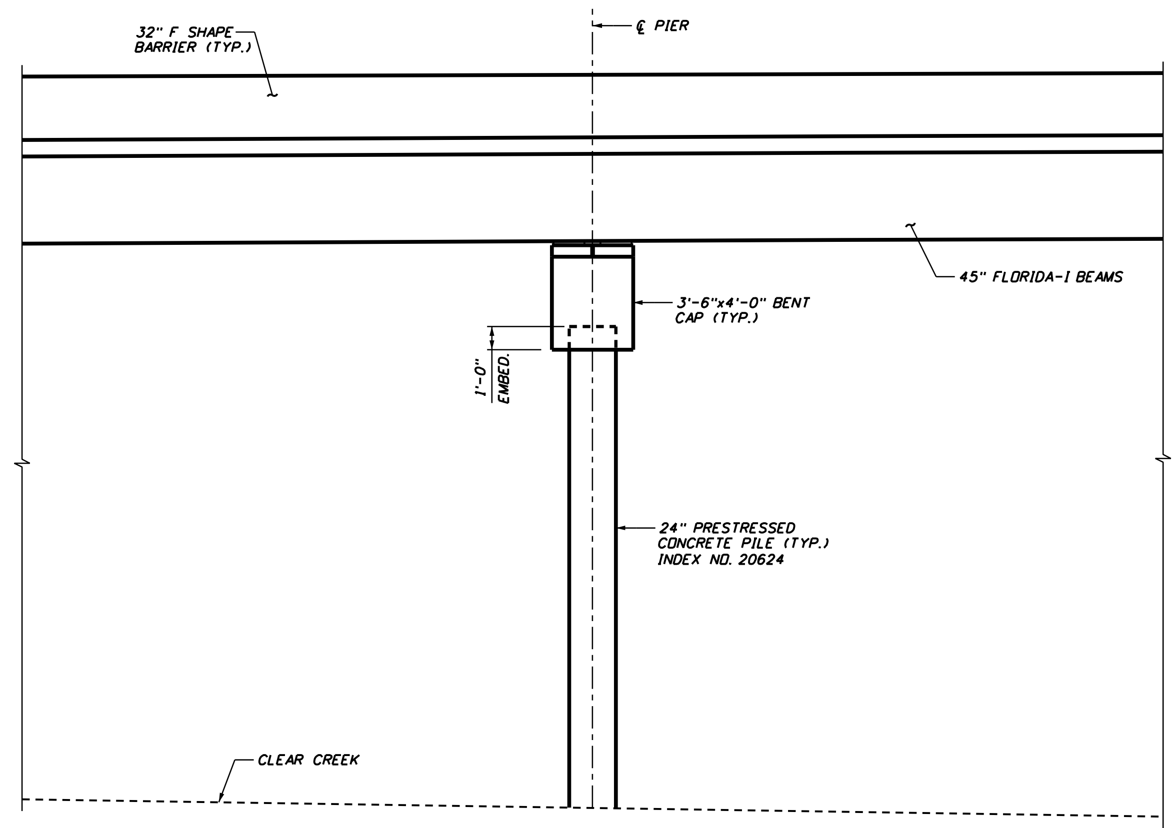

PARTIAL BRIDGE ELEVATION

FIGURE NO. 3

ALTERNATIVE A
SR 87 OVER CLEAR CREEK

FINLEY Engineering Group, Inc.

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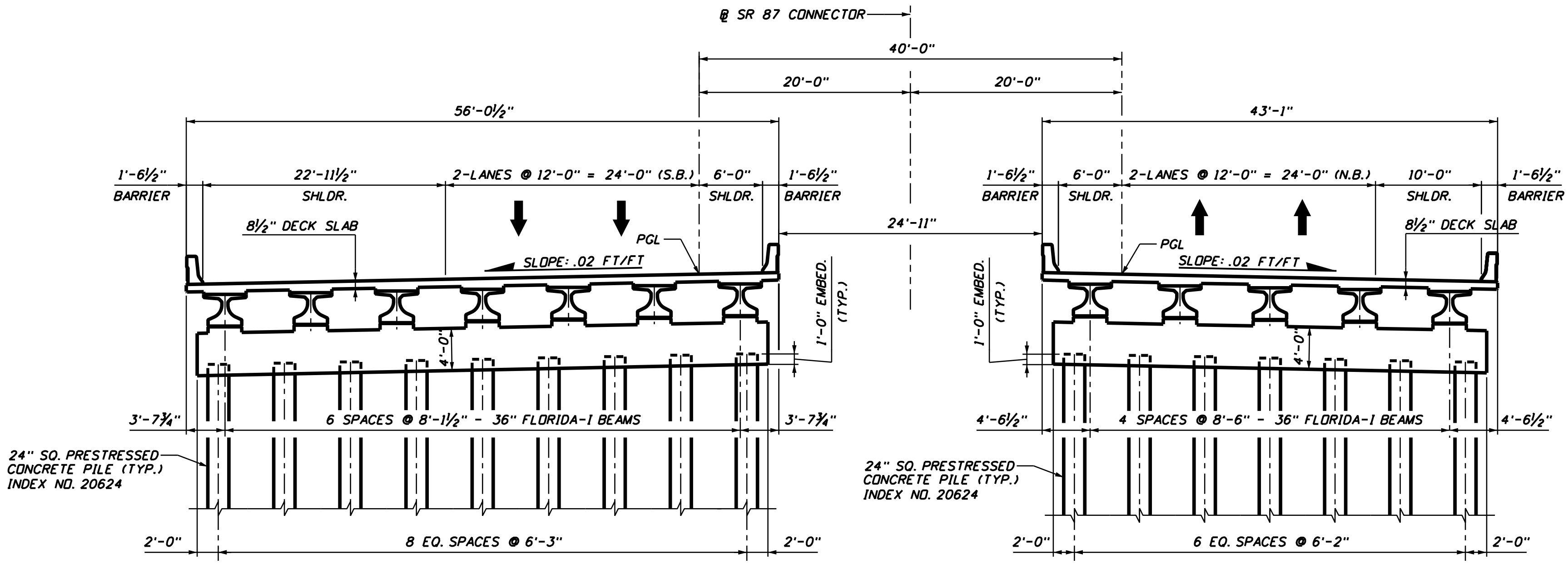

TYPICAL SECTION
(LOOKING UPSTATION)

FINLEY
Engineering Group, Inc.

FIGURE NO. 4

ALTERNATIVE B
SR 87 OVER CLEAR CREEK

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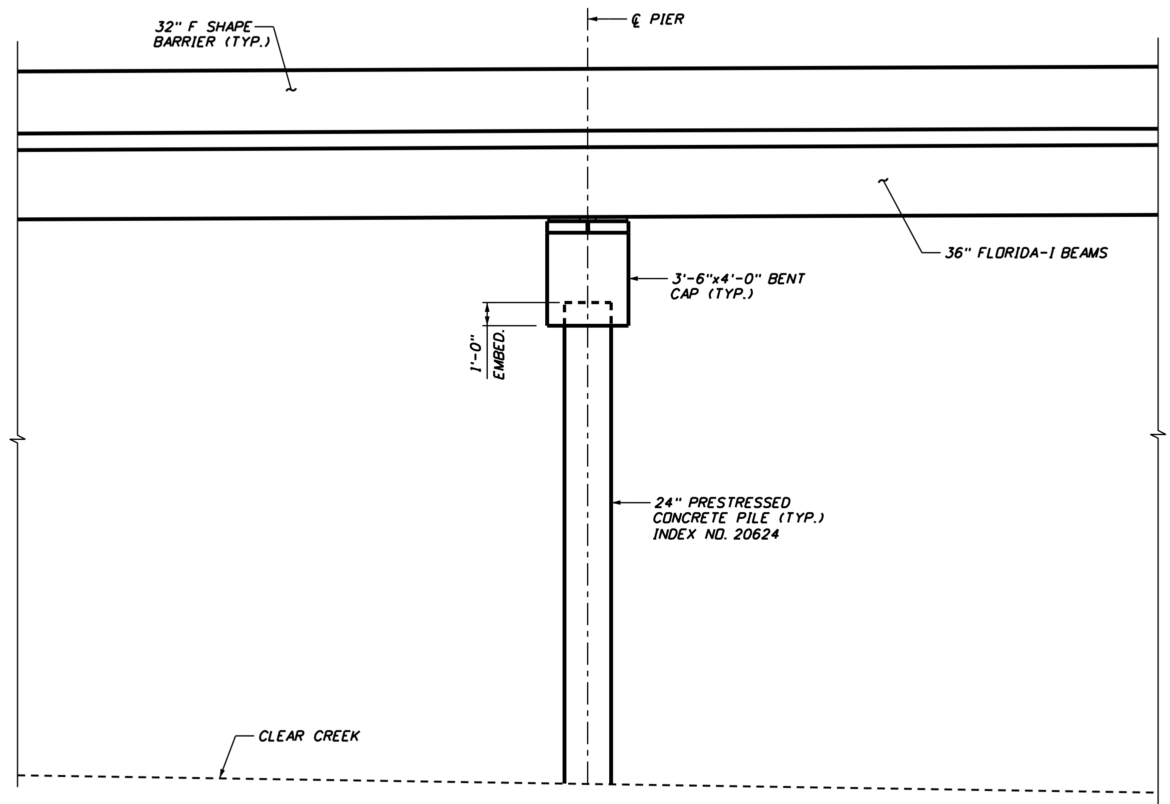

PARTIAL BRIDGE ELEVATION

FINLEY Engineering Group, Inc.

FIGURE NO. 5

ALTERNATIVE B
SR 87 OVER CLEAR CREEK

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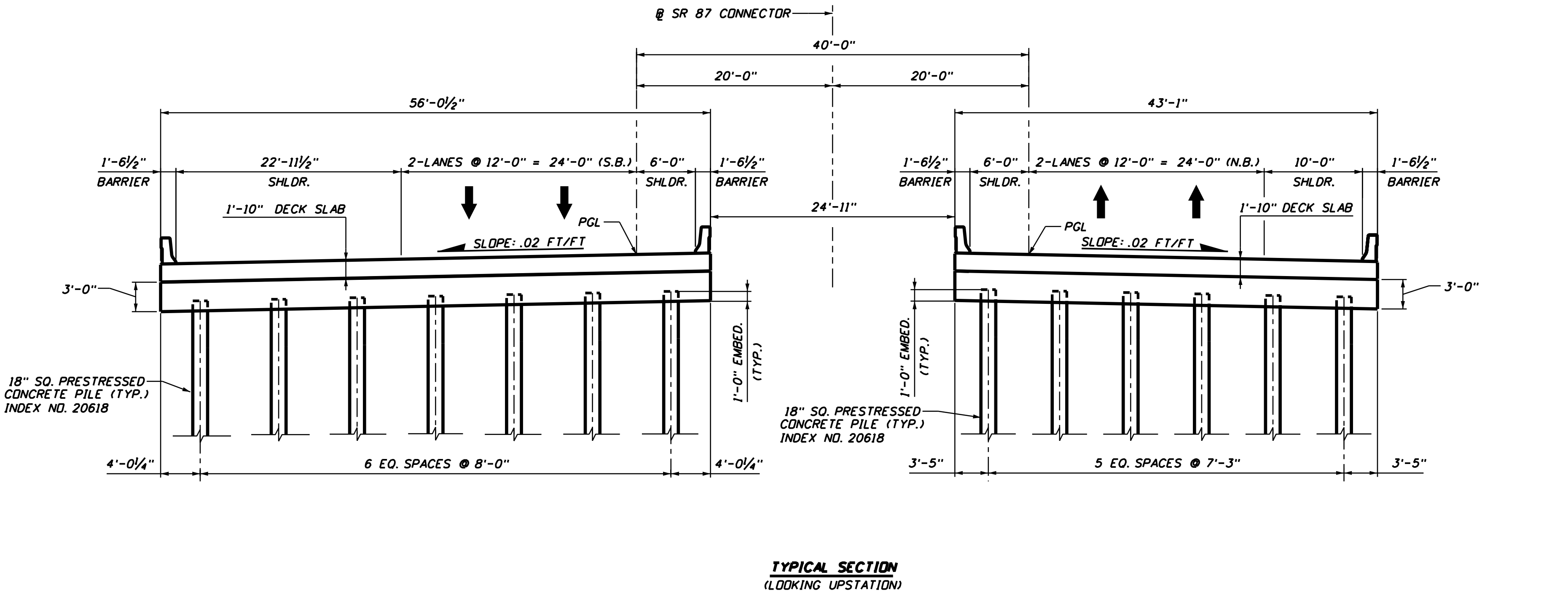

TYPICAL SECTION
(LOOKING UPSTATION)

FINLEY Engineering Group, Inc.

FIGURE NO. 6

ALTERNATIVE C
SR 87 OVER CLEAR CREEK

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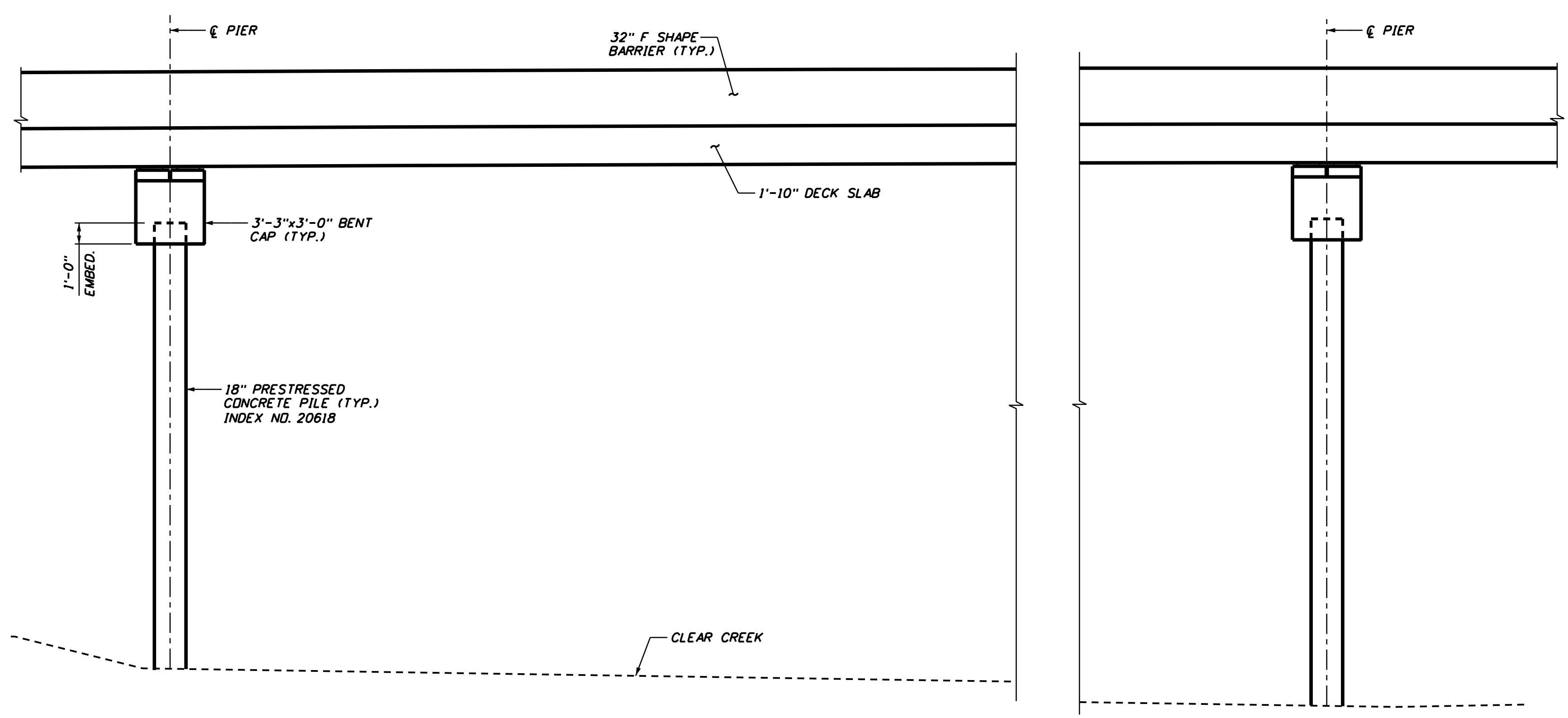

PARTIAL BRIDGE ELEVATION

FINLEY Engineering Group, Inc.

FIGURE NO.7

ALTERNATIVE C
SR 87 OVER CLEAR CREEK

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SECTION 5

Construction Economy

Pay item quantities were developed based on preliminary design of foundation, substructure and superstructure components for each of the BDR bridge alternatives. Relative cost estimates were then developed for each BDR alternative using unit costs from the 2012 BDR Bridge Cost Estimate Excel spreadsheet found in Chapter 9 of the Structures Design Guidelines. Costs for items such as mobilization, approach roadway paving and maintenance of traffic were considered to be the same for each alternative evaluated. Because these costs were assumed to be the same for each alternative, they were not quantified or included in the estimated bridge cost comparison of the alternatives. As a result, the bridge construction cost estimates presented in this report are relative values and do not represent the full construction cost of the proposed bridges. A summary of the relative construction cost established for each alternative is included in the Conclusions and Recommendations section of this report. A detailed summary of the various quantities, unit costs and pay item estimates for items that were compared between alternatives is provided in Appendix C of this report.

Maintenance of Traffic

The proposed SR 87 Bridges spanning Clear Creek will be constructed on a new roadway alignment where no highway traffic currently exists. With this in mind, the new bridge construction may be completed in a sequence as determined by the contractor to be most efficient. It is anticipated that the southbound bridge may be built as a stand-alone structure in an initial phase of construction. Two travel lanes, one for each direction of traffic, could then be maintained on the southbound bridge. When funding became available for expansion to the full four-lane facility, the northbound bridge could then be completed in a single phase of construction while both directions of traffic were maintained on the southbound bridge.

Constructability

FDOT pre-qualified contractors should not encounter unusual construction difficulties associated with any of the bridge alternatives studied in the preparation of this report. Florida-I Beam and flat-slab construction is common in the State of Florida. Construction of these types of bridge components should not require special construction engineering, elaborate formwork or specialized erection equipment.

Foundations incorporating prestressed concrete piles are used extensively for bridge construction in the State of Florida. Pile installation performed by an experienced contractor should not pose any unique problems for the proposed bridge construction considering subsurface conditions encountered in the subsurface geotechnical exploration performed by EGS. Pile driving for this bridge will not affect any existing structures since the proposed SR 87 alignment is located in a relatively undeveloped part of Santa Rosa County.

Prefabricated items such as prestressed beams and piling required for the recommended BDR alternative in this report can be fabricated in lengths and sizes such that they should not pose critical difficulties associated with delivery to the site. Upon delivery to the site, prestressed concrete piles can be spliced at the site if required lengths exceed trucking lengths.

Site Access

The site where the new SR 87 Bridge over Clear Creek is located consists of a combination of open water channels and marshlands with varying degrees of vegetation. Contract documents should include plan sheets denoting limits of various zones within the construction limits that will be classified as either marshlands or open water areas. Based on regulatory agency permitting requirements, criteria will be established informing the Contractor as to limitations of what will be permissible in each zone. It is anticipated that marshlands will be able to be temporarily impacted with various methods of stabilization provided that they are completely removed upon completion of the project. Open channel areas and locations with deep muck may require use of temporary work trestle to maintain hydraulic conveyance.

Life Cycle Maintenance

Properly detailed concrete bridges have historically required less maintenance efforts and expenses when compared to bridges incorporating structural steel components. It should be anticipated that any of the alternatives considered in this report will require minimal maintenance, such as bearing pad replacement and expansion joint repair.

Right-of-Way

Right-of-way and TIITF easements for State sovereign lands will be required addressing both the completed bridges over Clear Creek and as required to provide a reasonable work area for the bridge construction.

Utility Considerations

Based on survey information and visits to the site, it was noted that the following utilities were located in the vicinity of the proposed bridge construction:

- Gulf Power Company Transmission Lines located parallel to proposed bridges on the east side of the SR 87 alignment. The SR 87 alignment will be located outside of the Gulf Power Company right-of-way/easement and at a sufficient distance from the transmission lines such that construction of the bridge can be completed without disruption to the power service.

The proposed structure will incorporate two 2” diameter conduits located internally within the traffic railing barrier on each side of the roadway to accommodate future utilities.

Lighting Requirements

It is not anticipated that roadway or aesthetic lighting will be attached or hung from the proposed bridge. Lighting on the bridge for navigational guidance is not anticipated considering the terrain at the proposed crossing.

Bridge Deck Drainage

The bridge deck cross slope will be 2.00% throughout the limits of the bridge from the face of the median side barrier downward to the outside traffic barrier. Water will drain toward the shoulder, then off the bridge toward the roadway shoulders, and ultimately into the ditches adjacent to the rural typical section.

ADA Considerations

The future multi-use trail is required to be in compliance with all applicable Americans with Disabilities Act (ADA) requirements. ADA regulations require that accessible ramps have cross slopes no greater than 2.00% and profile grades not exceeding 5.00% without the use of intermittent landings. The profile grade of the proposed bridges spanning Clear Creek are less than 2.0%. The profile grade slope is therefore well within ADA limits of 5.000% for grades without intermittent landings. The cross slope of the bridge will be 2.00% throughout the limits of the bridge from the face of the median side barrier downward to the outside traffic barrier. The deck cross slope is therefore also compliant with ADA limits to not exceed 2.00% cross slope.

Aesthetics

The proposed SR 87 Bridge Over Clear Creek is designated as a Level One Bridge from an aesthetics standpoint. No special aesthetic treatments will be required for construction of the proposed bridge.

A Class V applied finish coating will be applied to all faces of the barrier and the deck fascia in accordance with the Structures Detailing Manual. This will enhance the look of the concrete elements substantially over that of plain concrete finishing.

SECTION 6

Conclusions and Recommendations

Twin bridges will be constructed to carry two travel lanes for each direction of traffic on a new alignment of SR 87 crossing Clear Creek. A minimum bridge length of 180 feet was established to span Clear Creek. The bridge profile was set such that the low member for each of the BDR alternatives would convey the design storm flood elevation including 2 feet of freeboard throughout the length of the bridge. Additionally, the vertical clearance for the bridge was increased to provide no less than 6 feet of clearance above the mean high water elevation at the creek. With this in mind, feasible alternatives were developed for the new twin bridge facility including cost estimates. Three feasible alternatives were evaluated in this report. The alternatives that were evaluated include the following structural systems:

- Alternative A – A (2) span 45" Florida-I Beam system with a cast-in-place composite slab founded on prestressed concrete pile supported bents.
- Alternative B – A (2) span 36" Florida-I Beam system with a cast-in-place composite slab founded on prestressed concrete pile supported bents.
- Alternative C – A (6) span 22" flat slab superstructure founded on prestressed concrete pile supported bents.

Each of the above listed alternatives has been determined to be feasible based on preliminary design calculations to verify their structural soundness. In selecting the BDR alternatives, close attention was given to constructability to ensure FDOT prequalified Contractors should not encounter unreasonable difficulties building the bridges. Key criteria for evaluation of the BDR alternatives were established based on the site conditions at the proposed crossing site. The key criteria used to evaluate the recommended structure for this project included bridge construction economy, long-term maintenance, constructability, site access, hydraulics, navigation and aesthetics. A systematic scoring system was used to determine the preferred BDR alternative for the proposed SR 87 crossing of Clear Creek. Table 1 shown at the end of this section summarizes the scoring of each of the alternatives evaluated in this report.

Upon comparison of the three BDR alternatives, it was determined that the optimum structure for this project would be Alternative A, the two (2) span bridge with 45" Florida-I Beams supported by pile bents. Alternative A is recommended as the preferred option based on the following considerations:

- Based on construction economy, the Florida-I Beam bridge alternative supported by pile bents provides better initial construction economy than the flat-slab alternative, which requires man more intermediate bents.
- The Florida-I Beam bridge alternatives minimal long term maintenance.
- The 90 foot length of the Florida-I Beams used for Alternative A can be efficiently delivered to the construction site. This beam length will not require special permits for delivery by truck on the state highway system.

Construction of twin bridges allows for initial construction of the southbound bridge which can be used to carry one lane of traffic in each direction. This structure could be used as a two-lane facility for the new SR 87 alignment until funding becomes available to build the full four-lane facility. Considering no vehicle traffic currently exists at the proposed bridge site, the contractor can build the bridges in a single phase of construction completed in a sequence as determined by the contractor to be most efficient.

Table 1 - Comparison of SR 87 Bridge Over Clear Creek BDR Alternatives

Rating Category	Possible Rating Score	Alternatives		
		Alt. A (2 Span - 45" FIB)	Alt. B (2 Span 36" FIB)	Alt. C (5 Span Flat Slab)
Unit Construction Cost of Bridge ($/SF)		**71.23**	76.17	87.22
Total Deck Area (SF)		**17,843**	17,843	17,843
Relative Bridge Construction Cost ($)		**$1,270,991**	$1,359,036	$1,639,515
Construction Economy	60	60.0	56.1	46.5
Long Term Maintenance	15	**15**	13	11
Constructability	15	**15**	13	11
Hydraulics (Piers)	5	**5**	5	3
Aesthetics	5	**5**	4	3
Total Score	100	**100.0**	91.1	77.0

Alternative A - 2 Span 45" Florida I-Beams on Pile Bents supported by 24" Prestressed Concrete Piles
Alternative B - 2 Span 36" Florida I-Beams on Pile Bents supported by 24" Prestressed Concrete Piles
Alternative C - 5 Span Flat Slab on Pile Bents supported by 18" Prestressed Concrete Piles

Note: Relative costs do not include items such as mobilization, approach roadway paving, permanent walls, temporary walls, and any other construction costs deemed to be the same for each of the alternatives considered.

APPENDIX A
BDR Submittal Checklist

BRIDGE DEVELOPMENT REPORT (BDR) SUBMITTAL CHECKLIST

Project Name SR 87 Over Clear Creek

Financial Project ID 416748-3-22-01

FA No. ________ FHWA Oversight (yes no) [yes circled] NHS (yes no) [no circled]

Date February 2013 FDOT Project Manager Peggy Kelley

	ITEMS	STATUS[b]		
1.	Typical Sections for Roadway and Bridge[a]	P	NA	C
2.	Roadway Plans in Vicinity of Bridge[a]	(P)	NA	C
3.	Maintenance of Traffic Requirements[a]	(P)	NA	C
4.	Bridge Hydraulics Report[c]	(P)	NA	C
5.	Geotechnical Report[c]	(P)	NA	C
6.	Bridge Corrosion Environmental Report[c]	(P)	NA	C
7.	Existing Bridge Plans	P	(NA)	C
8.	Existing Bridge Inspection Report	P	(NA)	C
9.	Utility Requirements	(P)	NA	C
10.	Railroad Requirements	P	(NA)	C
11.	Retaining Wall and Bulkhead Requirements	P	(NA)	C
12.	Lighting Requirements	(P)	NA	C
13.	ADA Access Requirements	(P)	NA	C
14.	Other – USCG Bride Project Questionnaire	P	(NA)	C

(a) Must be approved by District before BDR submittal
(b) Circle appropriate status:
P – Provided NA – Not Applicable C – Comments attached
(c) See approval requirements for these documents in Chapter 26 of the PPM.

Comments:
Items 4, 5, and 6 provided under separate cover and included in Appendix for reference.

APPENDIX B
Bridge Alternatives – Cost Estimates

Table 1 - Comparison of SR 87 Bridge Over Clear Creek BDR Alternatives

Rating Category	Possible Rating Score	Alternatives		
		Alt. A (2 Span - 45" FIB)	Alt. B (2 Span 36" FIB)	Alt. C (5 Span Flat Slab)
Unit Construction Cost of Bridge ($/SF)		**71.23**	76.17	87.22
Total Deck Area (SF)		**17,843**	17,843	17,843
Relative Bridge Construction Cost ($)		**$1,270,991**	$1,359,036	$1,639,515
Construction Economy	60	60.0	56.1	46.5
Long Term Maintenance	15	**15**	13	11
Constructability	15	**15**	13	11
Hydraulics (Piers)	5	**5**	5	3
Aesthetics	5	**5**	4	3
Total Score	100	**100.0**	91.1	77.0

Alternative A - 2 Span 45" Florida I-Beams on Pile Bents supported by 24" Prestressed Concrete Piles
Alternative B - 2 Span 36" Florida I-Beams on Pile Bents supported by 24" Prestressed Concrete Piles
Alternative C - 5 Span Flat Slab on Pile Bents supported by 18" Prestressed Concrete Piles

Note: Relative costs do not include items such as mobilization, approach roadway paving, permanent walls, temporary walls, and any other construction costs deemed to be the same for each of the alternatives considered.

Alternate A

$ 71.23

Bridge Development Report Relative Cost Estimate

Multiple Span - Prestressed Concrete Florida-I Beam 45"

Alternative A

	SB		NB
General Provisions			
Number of Typical Spans	**2**		2
Typical Span Length (Measured @ ℄ of construction)	**90.00**	ft	**90.00**
Number of Beams per Span	**5**		**4**
Bridge Length (FFBW to FFBW measured @ ℄ of construction)	180.0	ft	180.0
Bridge Width	**56.04**	ft	**43.08**
Bridge Clear Width (Used only for no. of lanes calculation)	52.96		40.00
Beam Spacing	**11.75**	ft	**11.50**
Overhang Width	4.52	ft	4.29
Deck Thickness	**8**	in	8
Sacrificial Deck Thickness	**0.5**	in	0.5
Average Haunch Thickness	**1.5**	in	1.5
Typical Deck Cross Slope	**2%**		2%

$ 71.23

A. Bridge Substructure			
Prestressed Concrete Piling			
Pile Size	**24**	in	24
End Bent			
Number of Piles	**6**		**5**
Pile Spacing	**11.75**	ft	**11.5**
Length of Piles	**90**	ft	**90**
Pile Embedment on Cap	**1**	ft	**1**
Intermediate Bent			
Number of Piles	**9**		**7**
Length of Piles	**110**	ft	**110**
Pile Embedment on Cap	**1**	ft	**1**
Total Pile Length (All Foundations)	**2070**	ft	**1670**
Substructure Concrete			
End Bent			
Cap			
Length	56.04	ft	43.08
Width	**3.50**	ft	3.50
Depth	**3.50**	ft	3.50
Volume	24.5	CY	18.8
Pedestals			
Minimum Height	**0.50**	ft	**0.50**
Width	3.17	ft	3.17
Length	2.50	ft	2.50
Volume	0.8	CY	0.6
Back Wall			
Height (Average)	4.13	ft	4.13
Width	**1.00**	ft	**1.00**
Length	54.54	ft	41.58
Volume	8.3	CY	6.4
Curtain Wall			
Height	4.52	ft	4.52
Width	**0.75**	ft	**0.75**
Length	3.50	ft	3.50
Volume	0.9	CY	0.9
Total Volume per End Bent	34.5	CY	26.7
Total Volume for the Two End Bents	69.1	CY	53.3
Intermediate Bent			
Cap			
Length	51.50	ft	39.50
Width	**3.50**	ft	3.50
Depth	**4.00**	ft	4.00
Volume	25.4	CY	19.4
Pedestals			
Minimum Height	**0.50**	ft	**0.50**
Width	3.17	ft	3.17
Length	3.50	ft	3.50
Volume	1.1	CY	0.9
Total Volume per Intermediate Bent	26.5	CY	20.3
Total Volume for all Intermediate Bents	26.5	CY	20.3

				$ 71.23
Substructure Total Concrete Volume		**95.5**	CY	**73.7**
Reinforcing Steel				
Weight per End Bent (135 lb/CY)		4662	lb	3600
Weight per Intermediate Bent (145 lb/CY)		3837	lb	2946
Substructure Total Reinforcing Steel Weight		**13161**	lb	**10146**
B. Bridge Superstructure				
Neoprene Bearing Pad				
Type		**E**		**E**
Width		**32**	in	**32**
Length		**10**	in	**10**
Thickness		**1.91**	in	**1.91**
Volume		0.353	CF	0.353
Number of Pads		20		16
Total Volume		**7.06**	CF	**5.65**
Prestressed Concrete Girders				
Florida-I Beam Type		**45**		**45**
Top Flange Width		**4**	ft	**4**
Total Length (Average measured @ ℄ of construction)		**900**	ft	**720**
Deck Concrete				
Superstructure Total Concrete Volume		**285.1**	CY	**219.6**
Reinforcing Steel				
SuperstructureTotal Reinforcing Steel Weight (205 lb/CY)		**58446**	lb	**45018**
Railing and Barriers				
Traffic Railing				
Type 32" F Shape	No. of Railing	**2**		**2**
Total Length (Average measured @ ℄ of construction)		**360**	ft	**360**
Pedestrian Railing		**No**		**No**
Concrete Parapet 27"				
Total Length (Average measured @ ℄ of construction)		**0**	ft	**0**
Bullet Railing				
Total Length (Average measured @ ℄ of construction)		**0**	ft	**0**
Expansion Joints				
Strip Seal				
Number of Joints		**2**		**2**
Length		56.04	ft	43.08
Total Length		**112.1**	ft	**86.2**

Bridge Development Report Pile Loads

End Bent

	SB		NB
General Provisions			
Number of Beams	5		4
Span Length (Measured @ ℄ of construction)	90.0	ft	90.0
Bridge Width	56.04	ft	43.08
Deck Thickness	8.0	in	8.0
Sacrificial Deck Thickness	0.5	in	0.5
Average Haunch Thickness	1.5	in	1.5
Beam Top Flange Width	4.0	ft	4.0
Beam Spacing	11.75	ft	11.50
Beam Weight	**906.0**	lb/ft	**906.0**
Traffic Railing Weight	**420.0**	lb/ft	**420.0**
Pedestrian Railing with Bullet Railing Weight	**0.0**	lb/ft	**0.0**
SIP Forms Weight	**20.0**	lb/ft^2	**20.0**
A. Live Load Reaction at End Bent			
Number of Design Lanes	4		3
Multiple Presence Factor	**0.65**		**0.85**
HL-93			
Design Truck Reaction	167.8	kip	164.6
Design Tandem Reaction	127.1	kip	124.7
Design Lane Load	74.9	kip	73.4
Total End Bent Live Load	**242.7**	kip	**238.0**
B. End Bent Dead Loads			
Self-Weight			
Cap	99.4	kip	76.2
Pedestals	3.2	kip	2.5
Back Wall	33.8	kip	25.7
Curtain Wall	3.6	kip	3.6
Total End Bent Self-Weight Dead Load	**139.9**	kip	**108.0**
Superstructure Weight			
Beams	203.9	kip	163.1
Deck	267.9	kip	206.0
Haunch	16.9	kip	13.5
Thickened Slab End	3.9	kip	2.8
SIP Forms	27.9	kip	20.3
Traffic Railing	37.8	kip	37.8
Pedestrian Railing	0.0	kip	0.0
Total End Bent Superstructure Dead Load	**558.2**	kip	**443.4**
C. Pile Loads			
Factored Reaction at Bent (Strength I) Note: Increased by 15% for preliminary design	1491.9	kip	1271.7
Number of Piles	6		5
Factored Individual Pile Load	248.7	kip	254.3
Downdrag Force	**0.0**	kip	**0.0**
Phi factor for pile driving	**0.65**		**0.65**
Required driving resistance	191	tons	196

Bridge Development Report Pile Loads

Intermediate Bent

	SB		NB
General Provisions			
Number of Beams	5		4
Span Length (Measured @ ℄ of construction)	90.0	ft	90.0
Bridge Width	56.04	ft	43.08
Deck Thickness	8.0	in	8.0
Sacrificial Deck Thickness	0.5	in	0.5
Average Haunch Thickness	1.5	in	1.5
Beam Top Flange Width	4.0	ft	4.0
Beam Spacing	11.75	ft	11.50
Beam Weight	906.0	lb/ft	906.0
Traffic Railing Weight	420.0	lb/ft	420.0
Pedestrian Railing Weight	0.0	lb/ft	0.0
SIP Forms Weight	20.0	lb/ft^2	20.0
A. Live Load Reaction at Intermediate Bent			
Number of Design Lanes	4		3
Multiple Presence Factor	0.65		0.85
HL-93			
Design Truck Reaction	190.9	kip	187.3
Design Tandem Reaction	127.1	kip	124.7
Design Lane Load	149.8	kip	146.9
Total Intermediate Bent Live Load	**325.7**	kip	**319.5**
B. Intermediate Bent Dead Loads			
Self-Weight			
Pier Cap	102.8	kip	78.8
Pedestals	4.4	kip	3.5
Total Intermediate Bent Self-Weight Dead Load	**107.2**	kip	**82.3**
Superstructure Weight			
Beams	407.7	kip	326.2
Deck	535.9	kip	412.0
Haunch	33.8	kip	27.0
Thickened Slab End	7.8	kip	5.6
SIP Forms	55.8	kip	40.5
Traffic Railing	75.6	kip	75.6
Pedestrian Railing	0.0	kip	0.0
Total Intermediate Bent Superstructure Dead Load	**1116.5**	kip	**886.9**
C. Pile Loads			
Factored Reaction at Bent (Strength I) Note: Increased by 15% for preliminary design	2414.6	kip	2036.1
Number of Piles	9		7
Factored Individual Pile Load	268.3	kip	290.9
Scour Resistance	**5.00**	kip	**5.00**
Phi factor for pile driving	**0.65**		**0.65**
Required driving resistance	210	tons	228

Bridge Development Report Cost Estimating

Effective 1/01/2012

Step One: Estimate Component Items

Utilizing the cost provided herein, develop the cost estimate for each bridge type under consideration.

A. Bridge Substructure

1. Prestressed Concrete Piling, (furnished and installed)			
Size of Piling	**Cost per Lin. Foot [1]**	**Quantity**	**Cost**
18" (Driven Plumb or 1" Batter)	$65		
18" (Driven Battered)	$75		
24" (Driven Plumb or 1" Batter)	$85	3740	$317,900
24" (Driven Battered)	$95		
30" (Driven Plumb or 1" Batter)	$120		
30" (Driven Battered)	$140		
Heavy mild steel reinforcing in pile head (each)	$250		
Embedded Data Collector (each)	$2,000		
		Subtotal	$317,900

1 When silica fume, metakaolin or ultrafine fly ash is used add $6/LF to the piling cost.

2. Steel Piling, (furnished and installed)			
Size of Piling	**Cost per Lin. Foot**	**Quantity**	**Cost**
14 x 73 H Section	$70		
14 x 89 H Section	$90		
20" Pipe Pile	$105		
24" Pipe Pile	$114		
30" Pipe Pile	$160		
		Subtotal	

3. Drilled Shaft (Total in-place cost)			
Dia. (on land, casing salvaged)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$250		
4 ft	$430		
5 ft	$510		
6 ft	$630		
7 ft	$750		
Dia. (in water, casing salvaged)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$320		
4 ft	$500		
5 ft	$600		
6 ft	$690		
7 ft	$800		
8 ft	$1,100		
Dia. (in water, permanent casing)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$460		
4 ft	$625		
5 ft	$750		
6 ft	$950		
7 ft	$1,100		
8 ft	$1,500		
9 ft	$1,800		
		Subtotal	

A. Bridge Substructure (continued)

4. Sheet Piling Walls			
Size (Prestressed Concrete)	**Cost per Lin. Foot**	**Quantity**	**Cost**
10" x 30"	$100		
12" x 30"	$110		
Type (Steel)	**Cost per Sq. Foot**	**Quantity**	**Cost**
Permanent Cantilever Wall	$24		
Permanent Anchored Wall [1]	$36		
Temporary Cantilever Wall	$14		
Temporary Anchored Wall [1]	$22		
Soil Anchors	**Cost per Anchor**	**Quantity**	**Cost**
Permanent	$3,200		
Temporary	$2,800		
		Subtotal	

1 Includes the cost of waler steel, miscellaneous steel for permanent/temporary walls and concrete face for permanent walls.

5. Cofferdam Footing (Cofferdam and Seal Concrete[1])			
Prorate the cost provided herein based on area and depth of water. A cofferdam footing having the following attributes cost $600,000: Area 63 ft x 37.25 ft; Depth of seal 5 ft; Depth of water over footing 16 ft			
Type	**Cost per Footing**	**Quantity**	**Cost**
Cofferdam Footing			
		Subtotal	

1 Cost of seal concrete included in pay item 400-3-20 or 400-4-200.

6. Substructure Concrete			
Type	**Cost per Cubic Yard**	**Quantity**	**Cost**
Concrete [1]	$575	169.2	$97,290
Mass Concrete [1]	$512		
Seal Concrete [1]	$412		
Bulkhead Concrete [1]	$925		
Shell Fill [1]	$30		
		Subtotal	$97,290

1 Admixtures: For Calcium Nitrite add $40/cy (@4.5 gal/cy) and for silica fume, metakaolin or ultrafine fly ash add $40/cy (@ 60 lb./cy)

7. Reinforcing Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Reinforcing Steel	$0.90	23307	$20,976
		Subtotal	$20,976

Substructure Subtotal $436,166

B. Bridge Superstructure

1. Bearing Material			
Type	Cost per Cubic Foot	Quantity	Cost
Neoprene Bearing Pads	$900	12.71	$11,438
Multirotational Bearings (kips)	Cost per Each	Quantity	Cost
1- 250	$6,000		
251- 500	$7,000		
501- 750	$8,000		
751-1000	$9,500		
1001-1250	$9,900		
1251-1500	$10,000		
1501-1750	$11,000		
1751-2000	$12,500		
>2000	$15,000		
		Subtotal	$11,438

2. Bridge Girders			
Structural Steel (includes coating)	Cost per Pound	Quantity	Cost
Rolled Wide Flange Sections, straight [1]	$1.35		
Rolled Wide Flange Sections, curved [1]	$1.70		
Plate Girders, Straight [1]	$1.50		
Plate Girders, Curved [1]	$1.70		
Box Girders, Straight [1]	$1.75		
Box Girders, Curved [1]	$1.85		
Prestressed Concrete Girders	Cost per Lin. Foot	Quantity	Cost
Fl. Inverted Tee 16" [2]	$80		
Fl. Inverted Tee 20"	$90		
Fl. Inverted Tee 24" [2]	$105		
Fl. Tub (U-Beam) 48" [2]	$700		
Fl. Tub (U-Beam) 54"	$750		
Fl. Tub (U-Beam) 63"	$800		
Fl. Tub (U-Beam) 72"	$900		
Solid Flat Slab (<48"x12")	$150		
Solid Flat Slab (<48"x15")	$160		
Solid Flat Slab (48"x12")	$160		
Solid Flat Slab (48"x15")	$170		
Solid Flat Slab (60"x12")	$170		
Solid Flat Slab (60"x15")	$180		
Florida-I; 36	$175		
Florida-I; 45	$185	1620	$299,700
Florida-I; 54	$200		
Florida-I; 63	$225		
Florida-I; 72	$250		
Florida-I; 78	$265		
Florida-I; 84	$320		
Florida-I; 96	$400		
Haunched Florida-I; 78	$700		
Haunched Florida-I; 84	$800		
		Subtotal	$299,700

1 When weathering steel (uncoated) is used reduce the price by $0.04 per pound. Inorganic zinc coating systems have an expected life cycle of 20 years.
2 Price is based on ability to furnish products without any conversions of casting beds and without pu

B. Bridge Superstructure (continued)

3. Cast-in-Place Superstructure Concrete			
Type	**Cost per Cubic Yard**	**Quantity**	**Cost**
Box Girder Concrete, Straight	$950		
Box Girder Concrete, Curved	$1,100		
Deck Concrete	$600	504.7	$302,820
Precast Deck Overlay Concrete Class IV	$600		
		Subtotal	$302,820

4. Concrete for Precast Segmental Box Girders, Cantilever Construction			
Concrete Cost by Deck Area	**Cost per Cubic Yard**	**Quantity**	**Cost**
≤ 300,000 SF	$1,250		
> 300,000 SF AND ≤ 500,000 SF	$1,200		
> 500,000 SF	$1,150		
		Subtotal	

5. Reinforcing Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Reinforcing Steel	$0.60	103464	$62,078
		Subtotal	$62,078

6. Post-Tensioning Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Strand, Longitudinal	$2.50		
Strand, Transverse	$4.00		
Bars	$6.00		
		Subtotal	

7. Railings and Barriers			
Type	**Cost per Lin. Foot**	**Quantity**	**Cost**
Traffic Railing[1]	$70	720	$50,400
Pedestrian/Bicycle Railings:			
Concrete Parapet (27")[1]	$65		
Single Bullet Railing[1]	$27		
Double Bullet Railing[1]	$36		
Triple Bullet Railing[1]	$45		
Picket Railing (42") steel	$65		
Picket Railing (42") aluminum	$50		
Picket Railing (54") steel	$95		
Picket Railing (54") aluminum	$60		
1 Combine cost of Bullet Railings with Concrete Parapet or Traffic Railing, as appropriate.		**Subtotal**	$50,400

8. Expansion Joints			
Type	**Cost per Lin. Foot**	**Quantity**	**Cost**
Strip Seal	$360	198.3	$71,370
Finger Joint <6"	$850		
Finger Joint >6"	$1,500		
Modular 6"	$500		
Modular 8"	$700		
Modular 12"	$900		
		Subtotal	$71,370

Superstructure Subtotal $797,806

JB

C. Miscellaneous Items

1. MSE Walls

Type	Cost per Sq. Foot	Quantity	Cost
Permanent	$26		
Temporary	$14		
		Walls Subtotal	

2. Sound Barriers

Type	Cost per Sq. Foot	Quantity	Cost
Post and Panel Sound Barriers	$25		
	Sound Barrier Subtotal		

3. Detour Bridges

Type	Cost per Sq. Foot	Quantity	Cost
Acrow Detour Bridge [1]	$55		
	Detour Bridge Subtotal		

1 Using FDOT supplied components. The cost is for the bridge proper and does not include approach work, surfacing, or guardrail.

Unadjusted Total $1,233,972

Step Two: Estimate Conditional Variables and Cost per Square Foot

After developing the total cost estimate utilizing the unit cost, modify the cost to account for site condition variables. If appropriate, the cost will be modified by the following variables:

Conditional Variables	% Increase/ Decrease	Cost (+/-)
For construction over water, increase cost by 3 %.	3%	$37,019
Phased construction or widening, increase by 20 %. [1]		
	3%	$37,019

1 Phased construction is defined as construction over traffic or construction requiring multiple phases to complete the construction of the entire cross section of the bridge. The 20 percent premium is applied to the affected units of the superstructure

Substructure Subtotal	$436,166
Superstructure Subtotal	$797,806
Walls Subtotal	
Sound Barrier Subtotal	
Detour Bridge Subtotal	
Conditional Variables	$37,019
Total Cost	$1,270,991
Total Square Feet of Deck	17843
Cost per Square Foot	$71

Design Aid for Determination of Reinforcing Steel

In the absence of better information, use the following quantities of reinforcing steel per cubic yard of concrete.

Location	Pounds of Steel	Cubic Yds.	Tot. Pounds
Pile Abutments	135		
Pile Bents	145		
Single Column Piers >25'	210		
Single Column Piers <25'	150		
Multiple Column Piers >25'	215		
Multiple Column Piers <25'	195		
Bascule Piers	110		
Standard Deck Slabs	205		
Isotropic Deck Slabs	125		
Concrete Box Girders, Pier Seg	225		
Concrete Box Girders, Typ. Seg	165		
Flat Slabs @ 30ft & 15" Deep	220		

Step Three: Cost Estimate Comparison to Historical Bridge Cost

The final step is a comparison of the cost estimate by comparison with historic bridge cost based on a cost per square foot. These total cost numbers are calculated exclusively for the bridge cost as defined in the General Section of this chapter. Price

Bridge Superstructure Type	Total Cost per Square Foot Low	High
Short Span Bridges:		
Reinforced Concrete Flat Slab- Simple Span [1]	$92	$160
Pre-cast Concrete Slab - Simple Span [1]	$81	$200
Medium Span Bridges:		
Concrete Deck / Steel Girder - Simple Span [1]	$125	$142
Concrete Deck / Steel Girder - Continuous Span [1]	$135	$170
Concrete Deck / Prestressed Girder - Simple Span [1]	$66	$145
Concrete Deck / Prestressed Girder - Continuous Span [1]	$83	$211
Concrete Deck / Steel Box Girder [1] - Span range from 150' to 280' (for curvature, add 15% premium)	$100	$165
Segmental Concrete Box Girders - Cantilever Construction Span range from 150' to 280'	$130	$160
Movable Bridge - Bascule Spans & Piers	$1,800	$2,000
Demolition Costs:		
Typical	$35	$60
Bascule	$60	$70
Project Type		
Widening (Construction Only)	$85	$160

1 Increase the cost by twenty percent for phased construction

Estimated Cost per Square Foot | **$71**

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

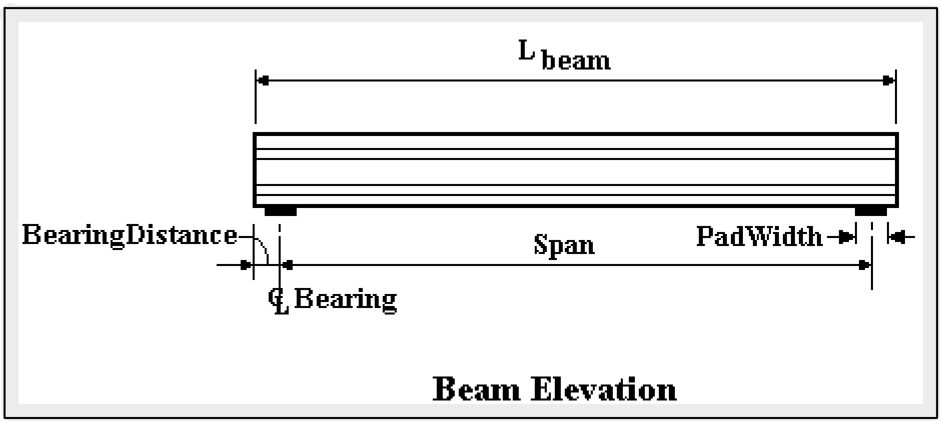
filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltA_ 90'_FIB-45_SE

Comment = "Alt. A - SB Interior"

Legend

TanHighlight = DataEntry | YellowHighlight = CheckValues | GreyHighlight = UserComments + Graphs
BlackText = ProgramEquations | *Maroon Text = Code Reference* | *Blue Text = Commentary*

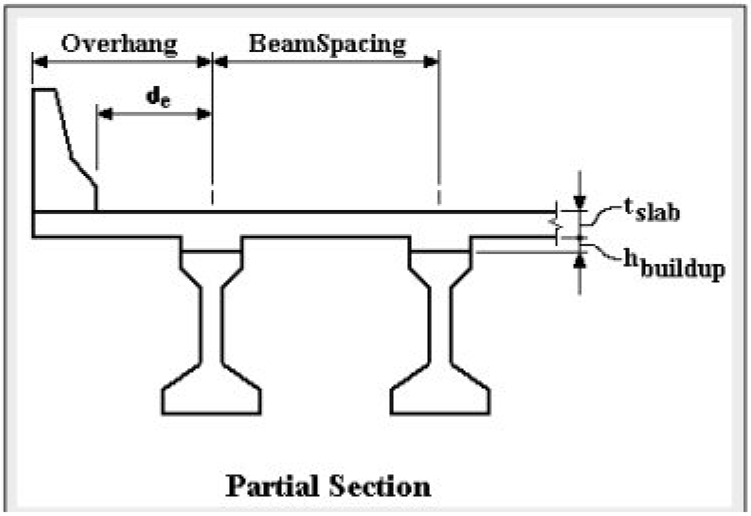
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ Span = $88.5 \cdot ft$ BearingDistance = $9 \cdot in$ PadWidth = $10 \cdot in$

BeamTypeTog = "FIB45"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



$Overhang = 4.52 \cdot ft$ | $BeamSpacing = 11.75 \cdot ft$ | $t_{slab} = 8 \cdot in$ | $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ | $t_{integral.ws} = 0.5 \cdot in$ | $NumberOfBeams = 5$ | $t_{slab.delta} = 0 \cdot in$

$de = 2.75\ ft$

BeamPosition = "interior" *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 11.75\ ft$ *effective slab width* *LRFD 4.6.2.6*

$t_{slab} := \text{if}(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

$SectTog := \text{if}(SectionType = \text{"transformed"}, 1, 0)$ $SectTog = 1$

Concrete:

Corrosion Classification Environment = "moderately"

density of slab concrete $\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$

strength of slab concrete $f_{c.slab} = 4.5 \cdot ksi$

density of beam concrete $\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$

strength of beam concrete $f_{c.beam} = 8.5 \cdot ksi$

weight of future wearing surface $Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$

release beam strength $f_{ci.beam} = 6 \cdot ksi$

type of course aggregate, "Florida" *or* "Standard" AggregateType = "Florida"

relative humidity $H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ *used in distribution calculation* $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9 \cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor \cdot \sqrt{f_{ci.beam} \cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012 \cdot ksi$

$E_c := AggFactor \cdot \sqrt{f_{c.beam} \cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776 \cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270 \cdot ksi$

tendon modulus of elasticity $E_p = 28500 \cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60 \cdot ksi$

mild steel modulus of elasticity $E_s = 29000 \cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62 \cdot \frac{in^2}{ft}$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0 \cdot in^2$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4 \cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0 \cdot in$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \quad 53.33 \quad 53.33) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0) \cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.91$ $tmp\_g_{shear} = 1.13$

user value overrides (optional):

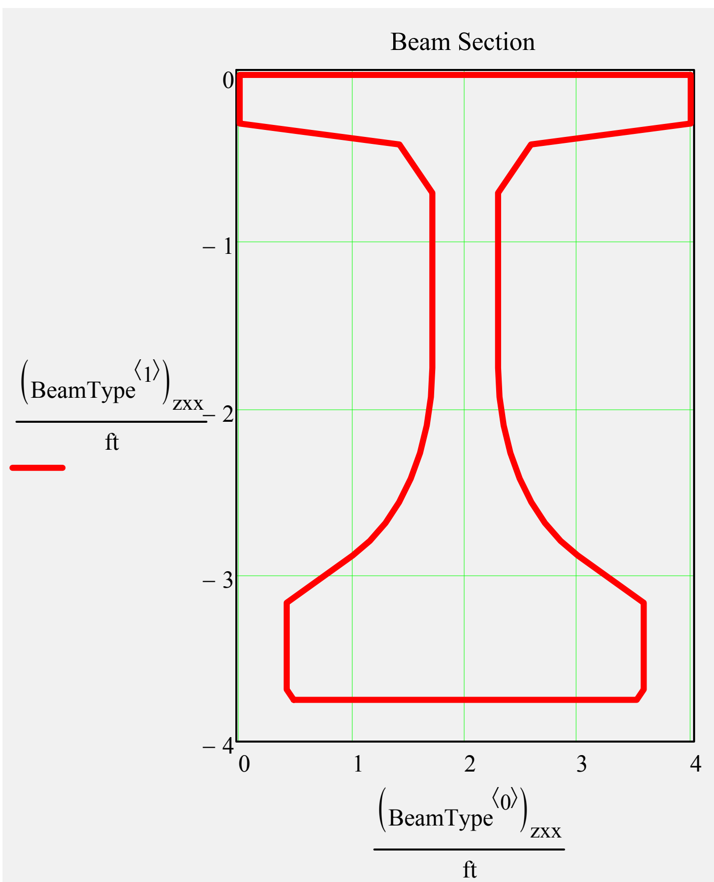
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

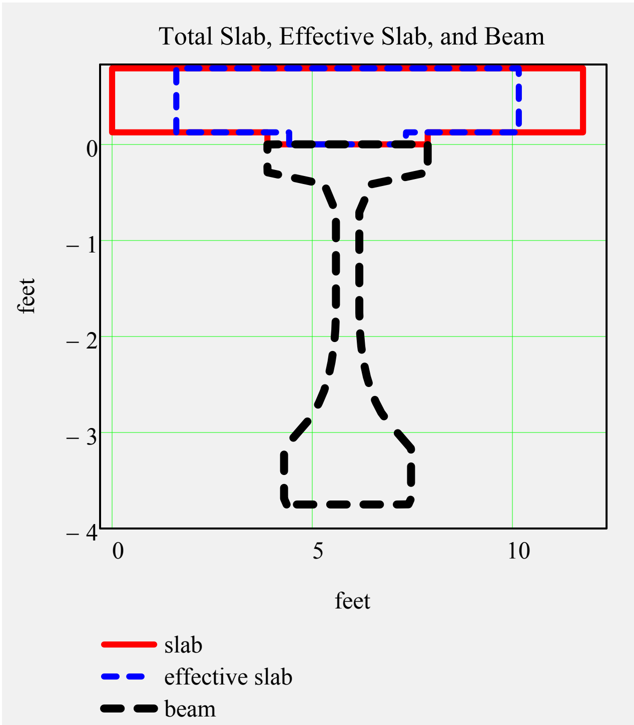
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.91$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 1.13$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 1.323 \cdot \frac{kip}{ft}$ $w_{beam} = 0.907 \cdot \frac{kip}{ft}$ $w_{forms} = 0.155 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$ *additional non composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 2.385 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 1.478 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

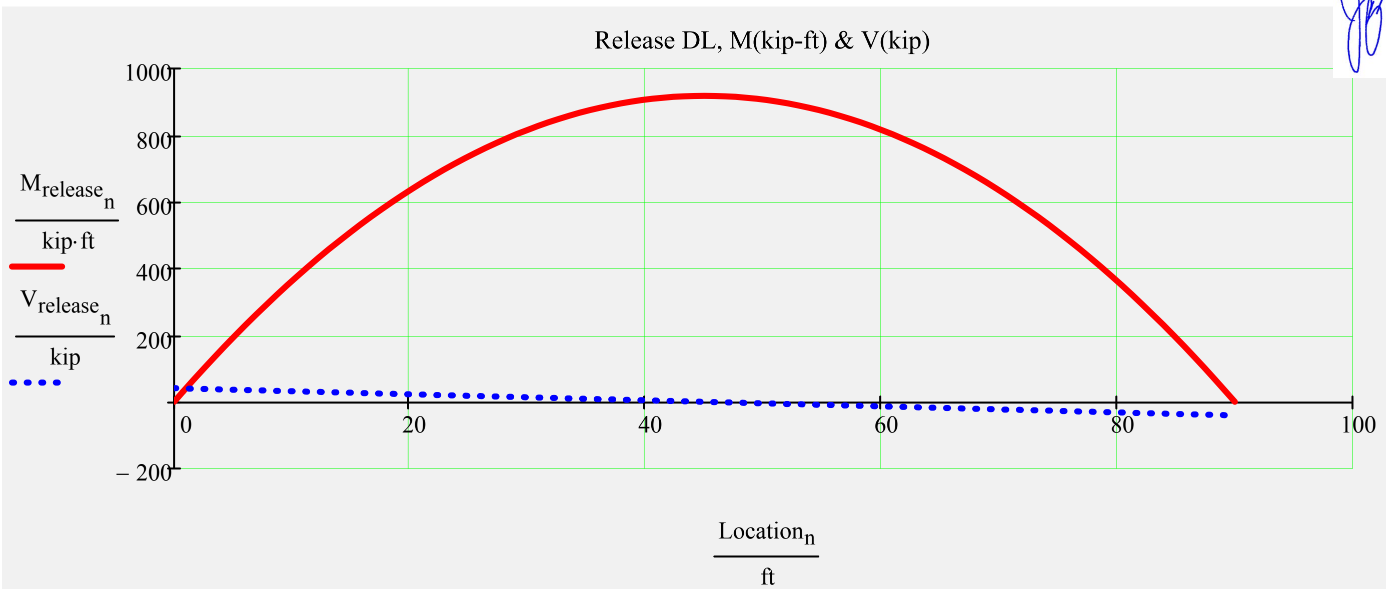
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.168 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.168 \cdot \frac{kip}{ft}$

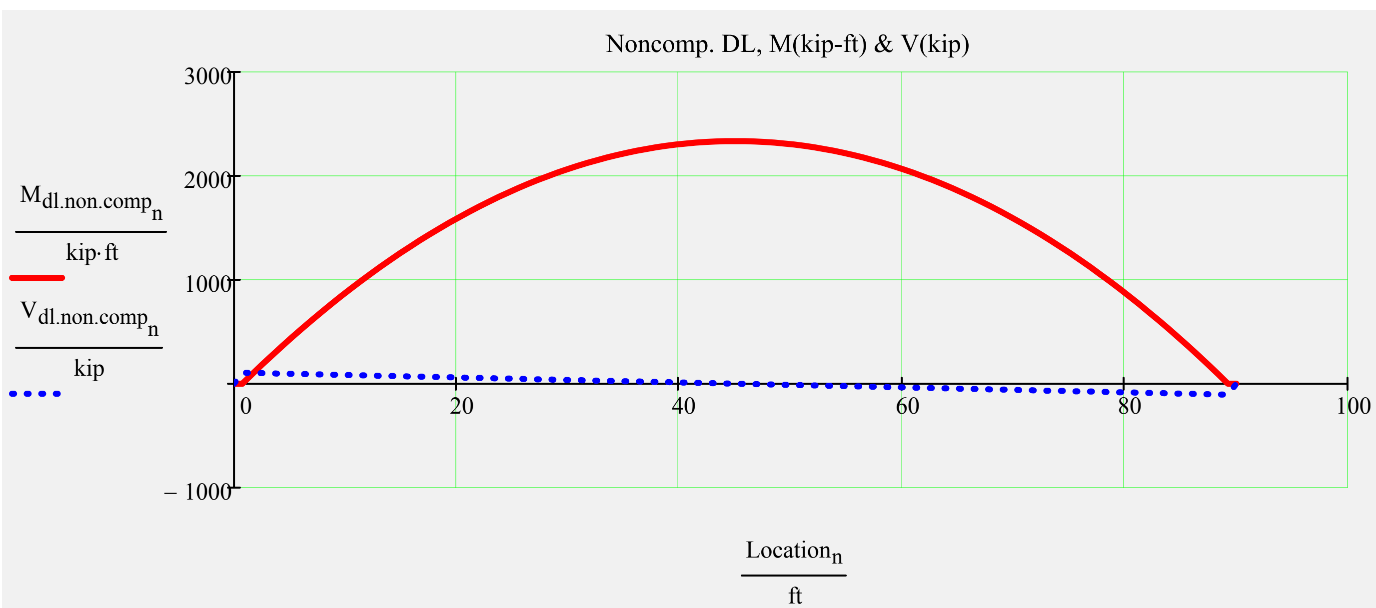
$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.168 \cdot \frac{kip}{ft}$

Release Dead Load Moments and Shear



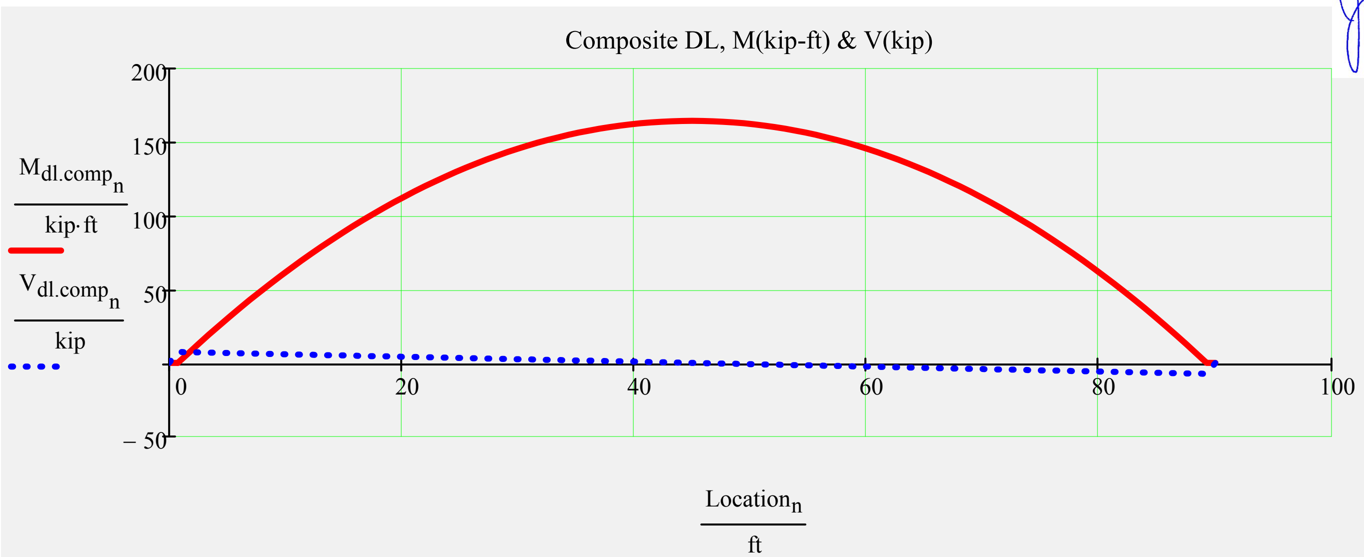
$\max(M_{release}) = 918 \cdot kip \cdot ft$ $\max(V_{release}) = 40.8 \cdot kip$

Noncomposite Dead Load Moments and Shear



$\max(M_{dl.non.comp}) = 2334.4 \cdot kip \cdot ft$ $\max(V_{dl.non.comp}) = 105.5 \cdot kip$

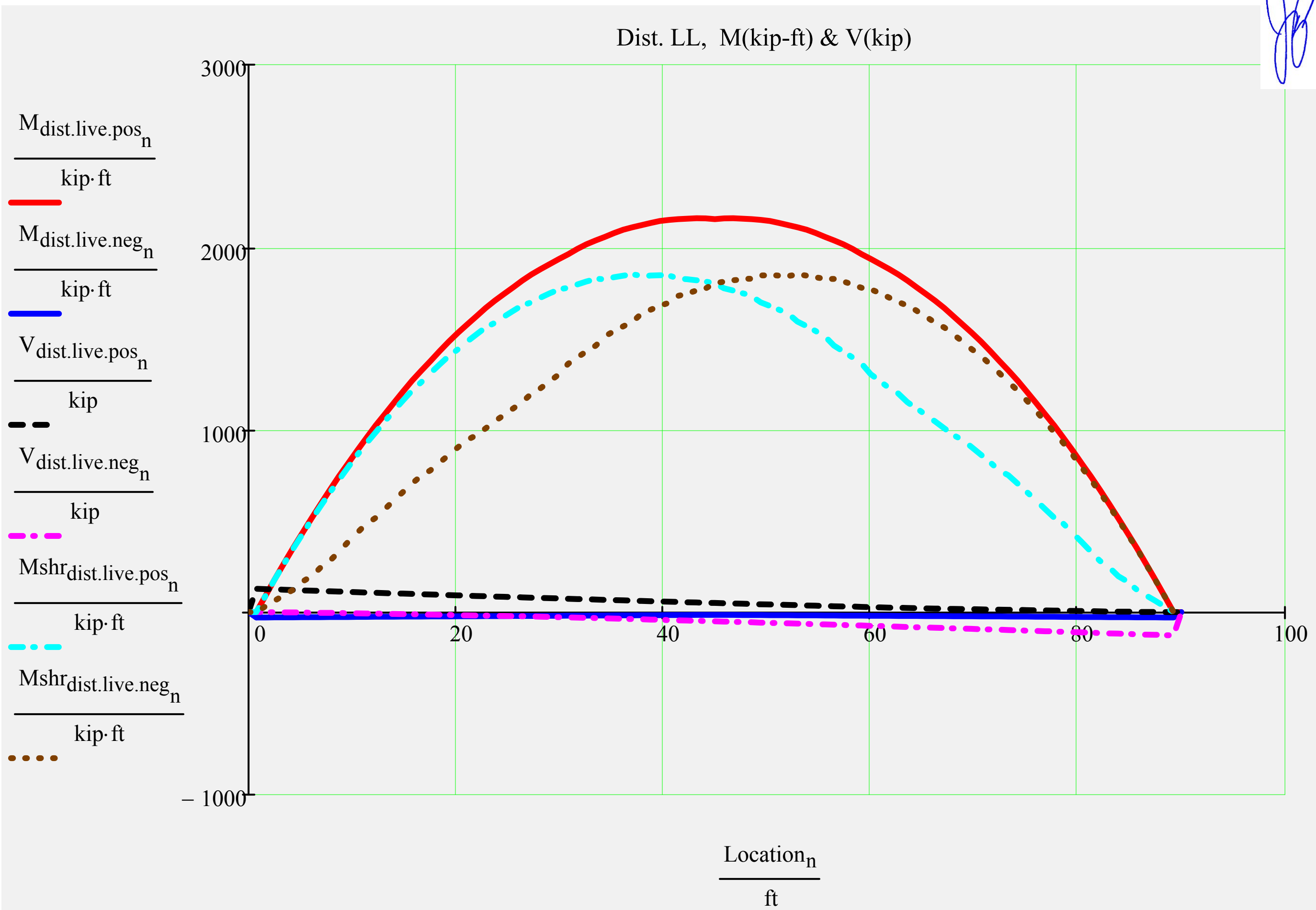
Composite Dead Load Moments and Shear



$max(M_{dl.comp}) = 164.4 \cdot kip \cdot ft$

$max(V_{dl.comp}) = 7.4 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 2158.9 \cdot kip \cdot ft$

$\min(M_{dist.live.neg}) = -29.2 \cdot kip \cdot ft$

$Reaction_{LL} = 129.68 \cdot kip$

$\max(V_{dist.live.pos}) = 128.2 \cdot kip$

$\max(Mshr_{dist.live.pos}) = 1850.8 \cdot kip \cdot ft$

$Reaction_{DL} = 114.89 \cdot kip$

Prestress Strand Layout Input

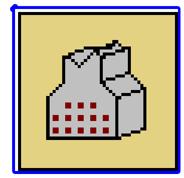
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate :=
- Standard
- Custom

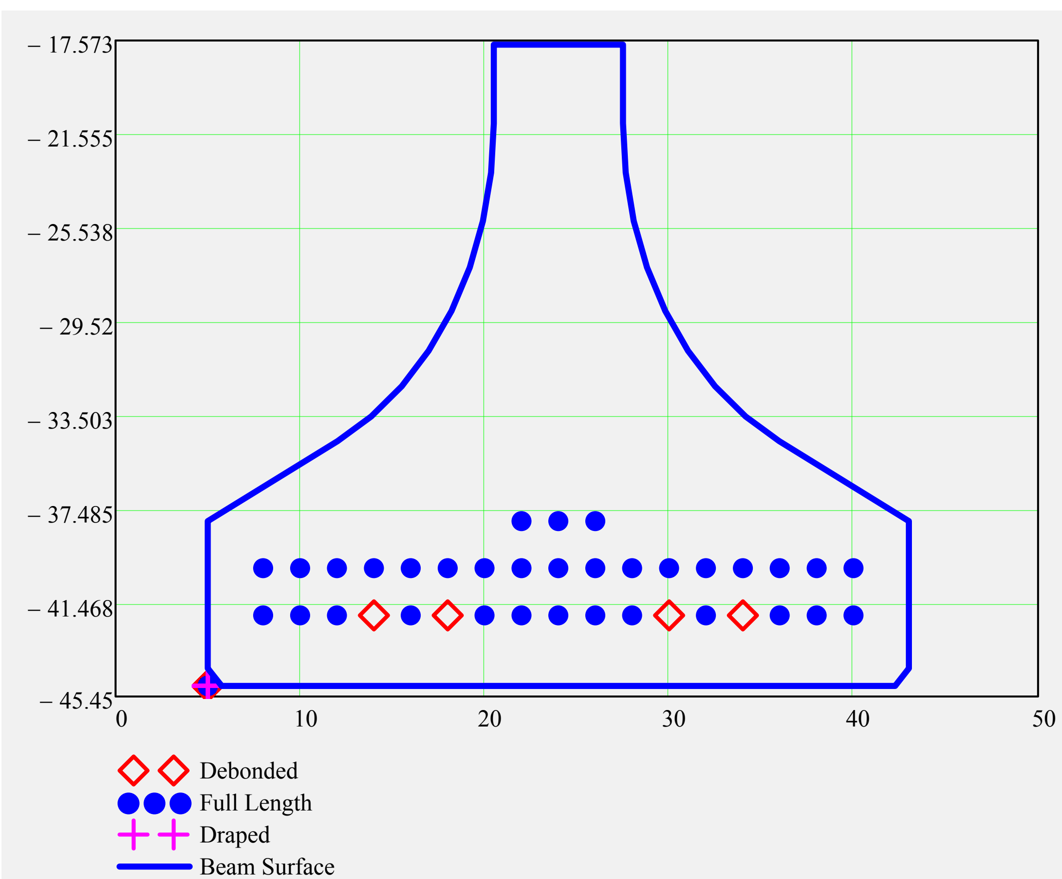
Strand Pattern Generator:



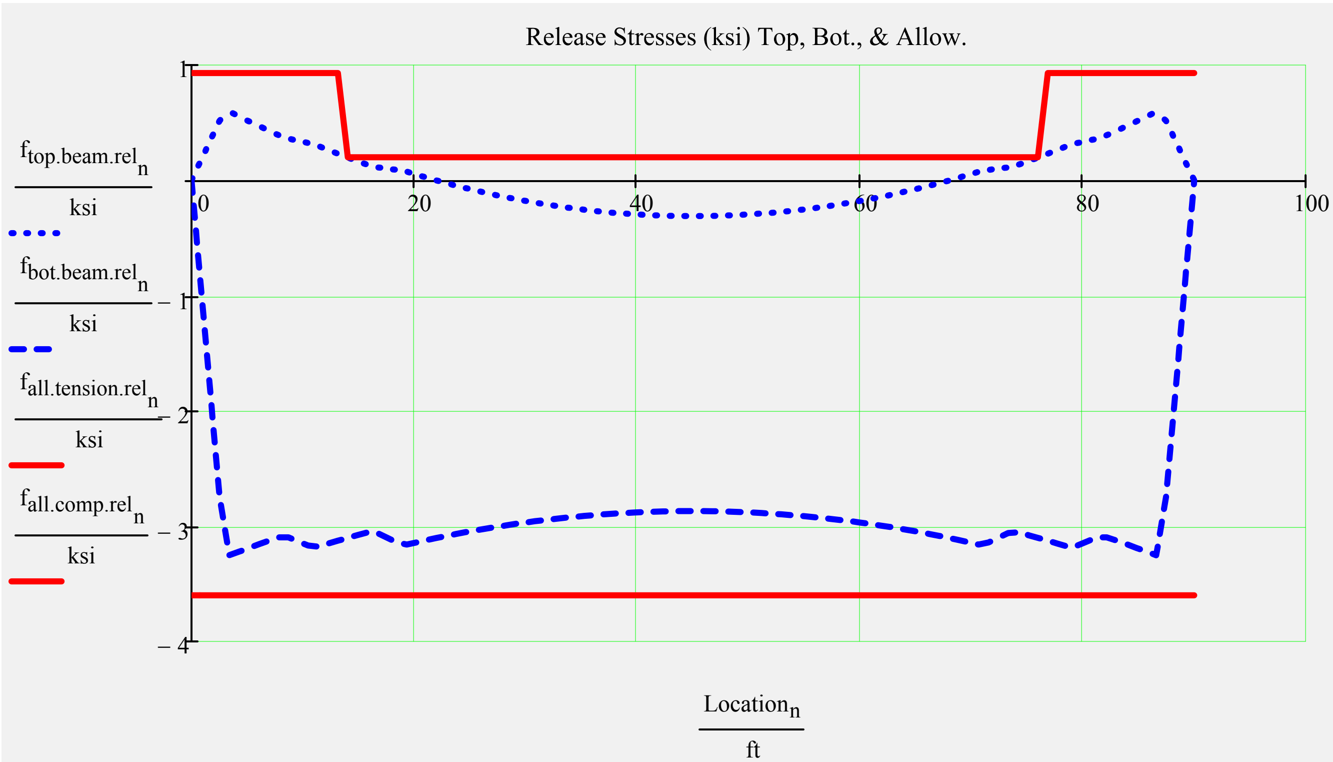
Collapsed Region for Custom Strand Sizes...

$\text{CheckPattern}_0 = \text{"OK"}$	***check 0*** *- no debonded tendon in outside row*
$\text{CheckPattern}_1 = \text{"OK"}$	***check 1*** *- less than 25% debonded tendons total*
$\text{CheckPattern}_2 = \text{"OK"}$	***check 2*** *- less than 40% debonded tendons in any row*
$\text{CheckPattern}_3 = \text{"OK"}$	***check 3*** *- less than 40% of debonded tendons terminated at same section* (LRFD 5.11.4.3)
$\text{CheckPattern}_4 = \text{"OK"}$	***check 4*** *- more than half beam depth debond length* (SDG 4.3.1.E)

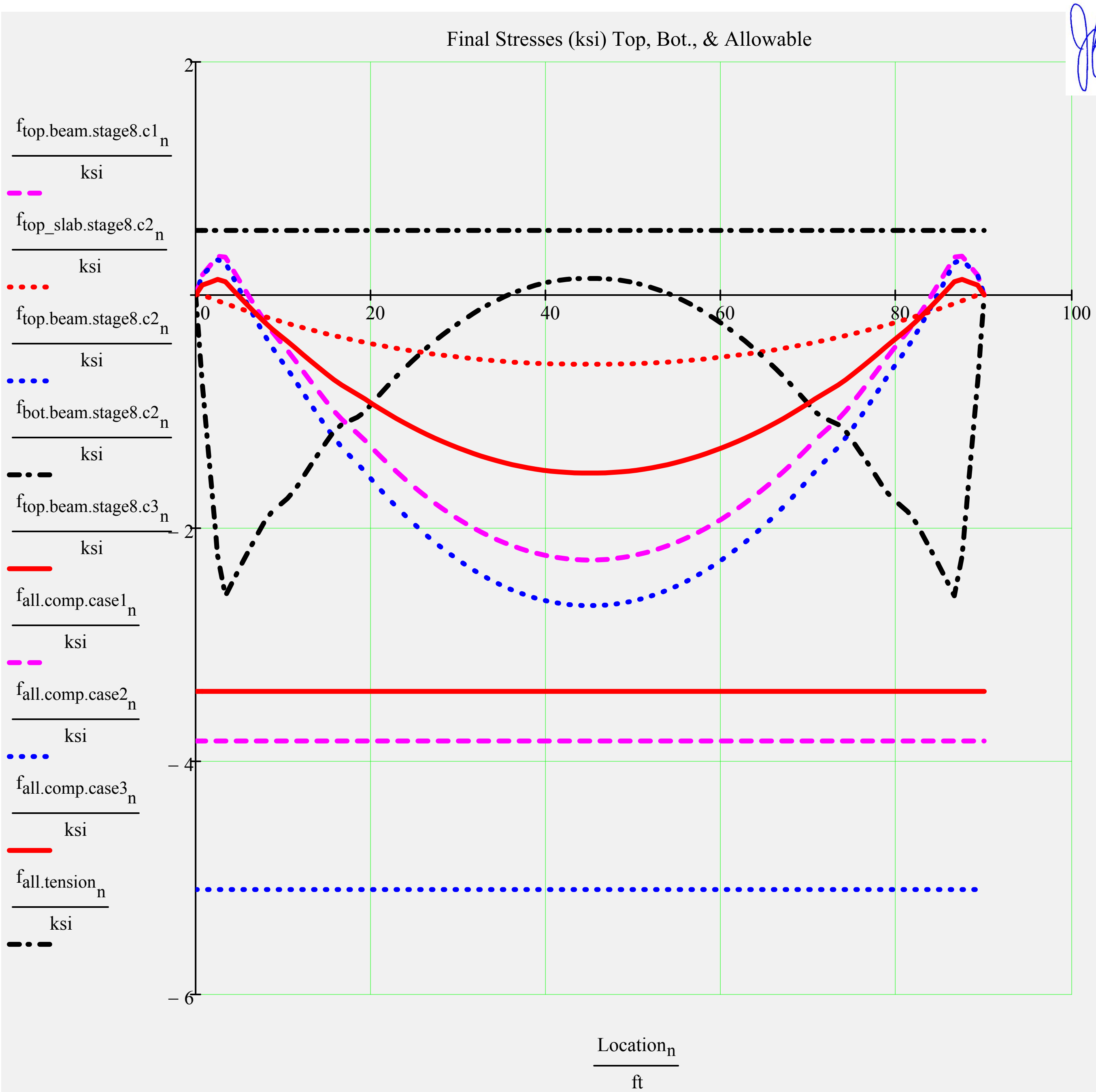
Tendon Layout



Release Stresses



Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.01$ | $Check\_f_{tension.rel}$ = "OK" | (Release tension)

$\min(CR\_f_{comp.rel}) = 1.11$ | $Check\_f_{comp.rel}$ = "OK" | (Release compression)

$\min(CR\_f_{tension.stage8}) = 3.89$ | $Check\_f_{tension.stage8}$ = "OK" | (Service III , PS + DL +LL*0.8)

$\min(CR\_f_{comp.stage8.c1}) = 1.68$ | $Check\_f_{comp.stage8.c1}$ = "OK" | (Service I , PS + DL)

$\min(CR\_f_{comp.stage8.c2}) = 1.91$ | $Check\_f_{comp.stage8.c2}$ = "OK" | (Service I , PS + DL +LL)

$\min(CR\_f_{comp.stage8.c3}) = 2.23$ | $Check\_f_{comp.stage8.c3}$ = "OK" | (Service I , (PS + DL)*0.5 +LL)

Section and Strand Properties Summary

$A_{beam} = 870.4 \cdot in^2$ — Concrete area of beam

$I_{beam} = 226606.0804 \cdot in^4$ — Gross Moment of Inertia of Beam about CG

$y_{comp} = -9.76 \cdot in$ — Dist. from top of beam to CG of gross composite section

$I_{comp} = 624522.259 \cdot in^4$ — Gross Moment of Inertia Composite Section about CG

$A_{deck} = 873.13 \cdot in^2$ — Concrete area of deck slab

$A_{ps} = 8 \cdot in^2$ — total area of strands

$d_{b.ps} = 0.6 \cdot in$ — diameter of Prestressing strand

$\min(PrestressType) = 0$ — 0 - low lax 1 - stress relieved

$f_{py} = 243 \cdot ksi$ — tendon yield strength

$f_{pj} = 203 \cdot ksi$ — prestress jacking stress

$L_{shielding}^{T} = (8 \quad 16 \quad 0 \quad 0 \quad 0) \cdot ft$

$A_{ps.row}^{T} = (0.4 \quad 0.4 \quad 2.8 \quad 3.7 \quad 0.7) \cdot in^2$

$d_{ps.row} =$ (·in)

	0	1	2	3	4	5	6	7	8	9
0	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
1	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
2	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
3	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40
4	-38	-38	-38	-38	-38	-38	-38	-38	-38	...

$TotalNumberOfTendons = 37$

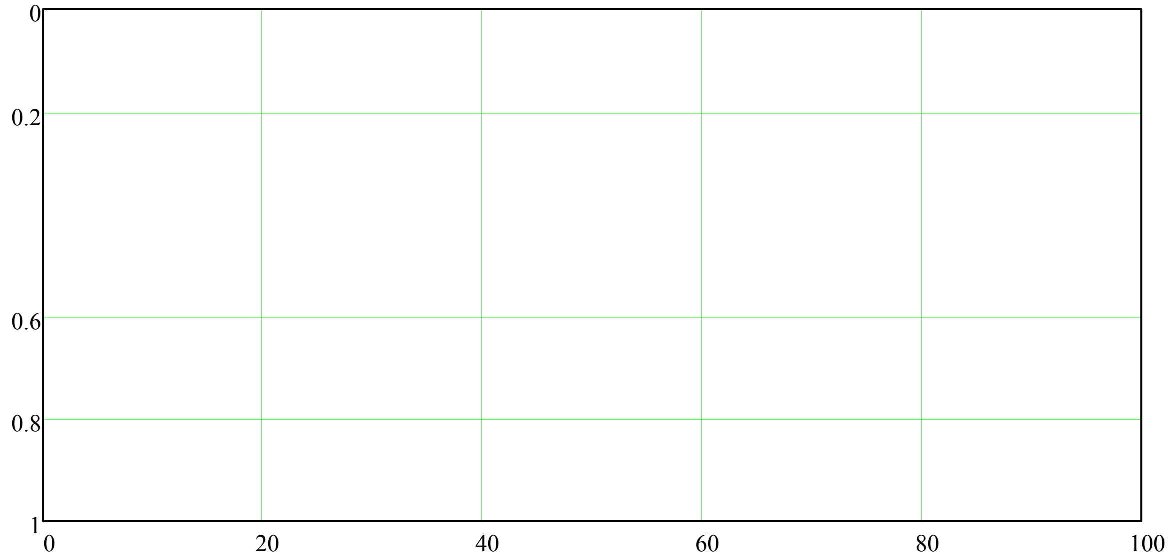
$NumberOfDebondedTendons = 4$

$NumberOfDrapedTendons = 0$

$StrandSize = $ "0.6 in low lax"

$StrandArea = 0.22 \cdot in^2$

$JackingForce_{per.strand} = 43.94 \cdot kip$



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

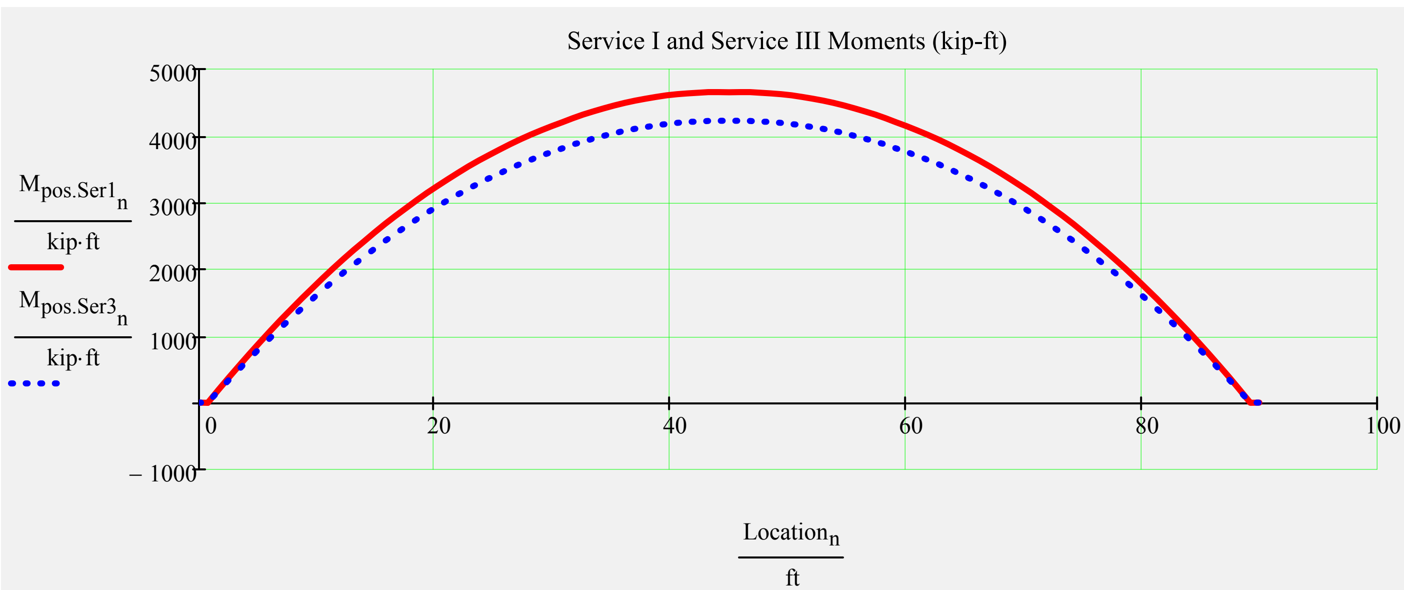
$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

$f_{pe} = 179 \cdot ksi$ $\Delta f_{pTot} = -23 \cdot ksi$

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.47 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.53 \cdot \%$

Check_f_{pt} = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f_{pe} = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 4655.1 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 4223.7 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.31	−2.87
2	−0.41	−2.44
4	−0.37	−2.47
6	−2.24	−1.03
8	−2.66	0.14

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−2183.5
"Final (about composite centroid)"	−1439.5	−1831.4

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	862.37	224527.31	−24.64
"Transformed Beam (initial)"	919.4	238511.11	−25.64
"Transformed Beam "	910.29	236392.94	−25.49
"Composite "	1820.37	671261.2	−10.13

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	918
"Non-composite (includes bm wt.)"	2334.4
"Composite"	164.4
"Distributed Live Load"	2153.8

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

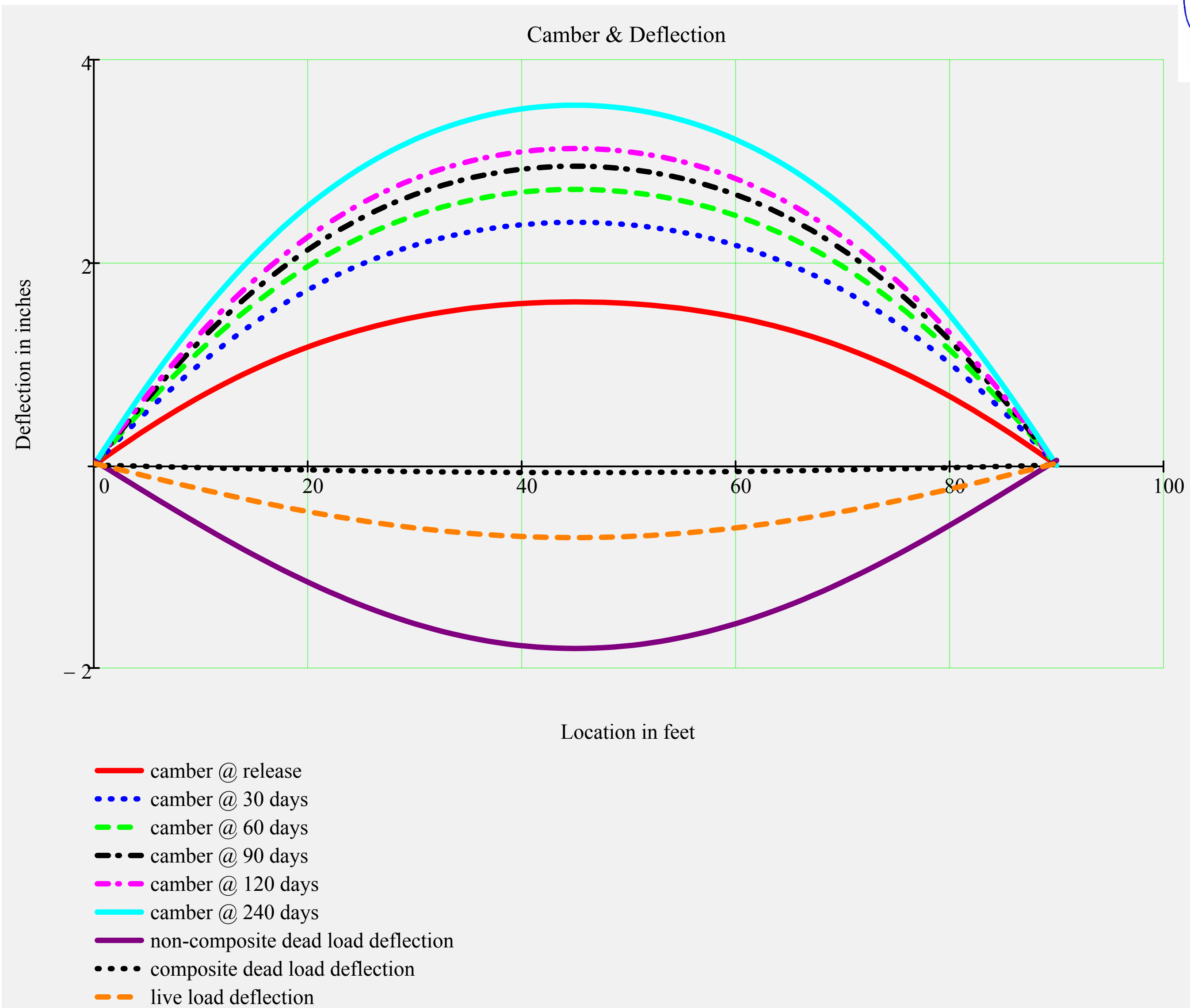
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

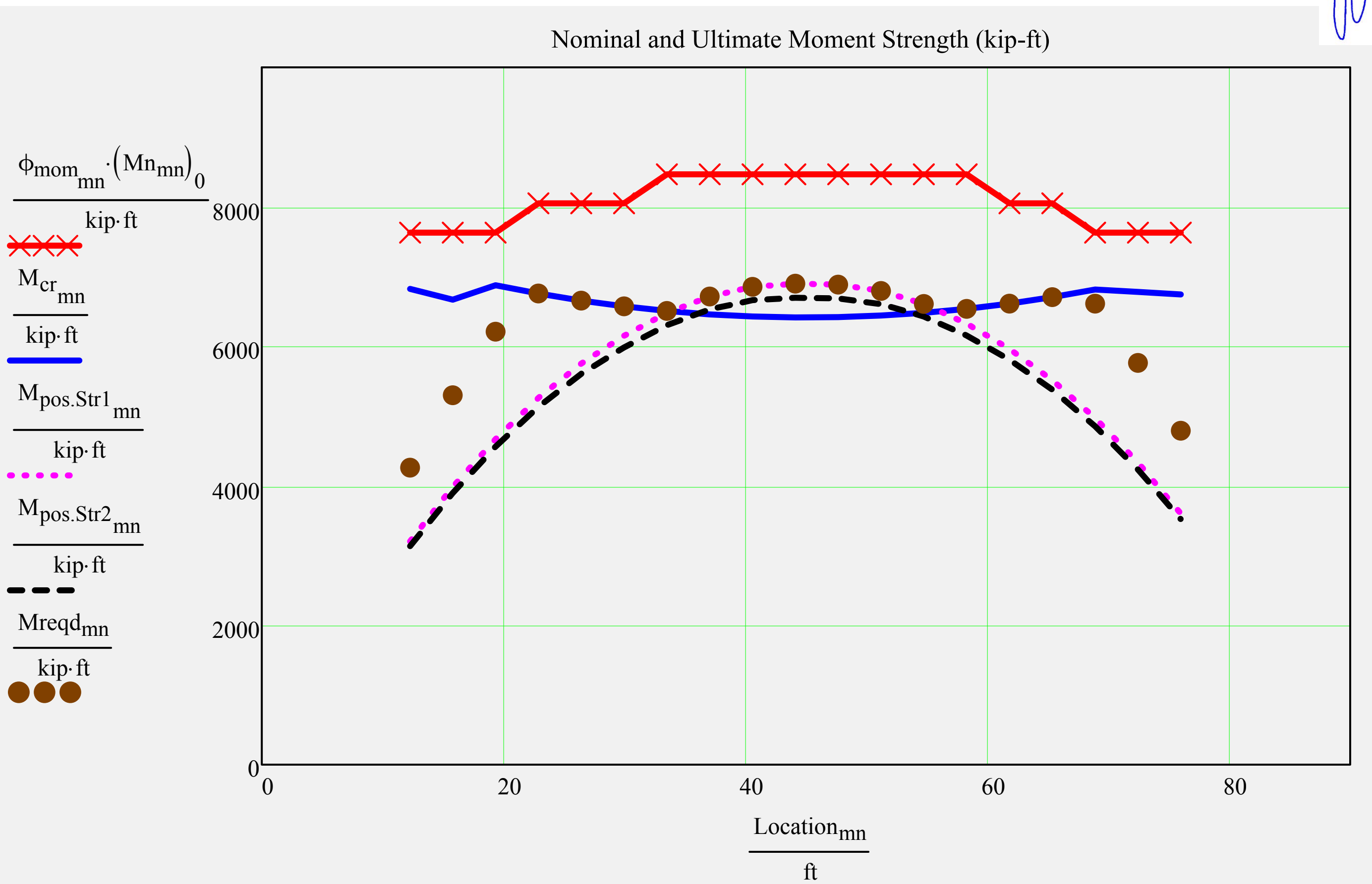
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	0.0105	−0.8054	0.3825	1.6109
"30 Days"	−0.1453	−1.4271	0.6214	2.3965
"60 Days"	−0.201	−1.6496	0.7148	2.7208
"90 Days"	−0.2307	−1.7681	0.7646	2.9497
"120 Days"	−0.2492	−1.8418	0.7955	3.124
"240 Days"	−0.2833	−1.9778	0.8526	3.5512
"non-comp DL"	−0.2776	0.2127	−0.3123	−1.8074
"comp DL"	−0.0044	0.0152	−0.0125	−0.0723
"LL"	−0.0439	0.1512	−0.1243	−0.7135

Strength Limit State Moments



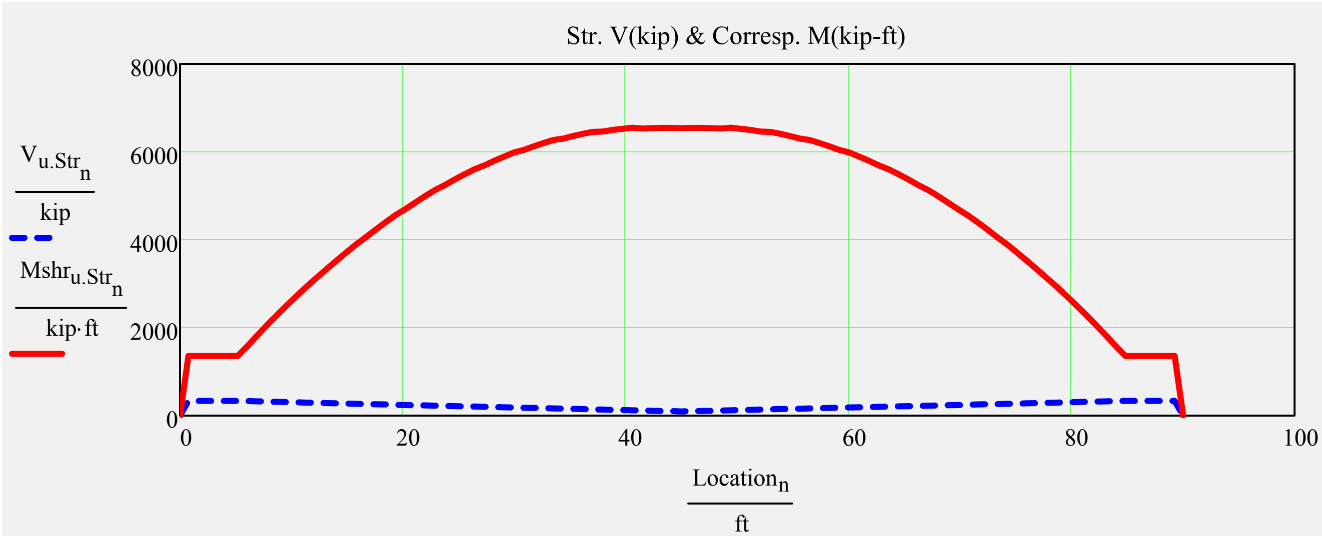
$CR_{Str.mom_n} := 10$ $\qquad CR_{Str.mom_{mn}} := \frac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ $\qquad$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.15$

$\max(Mreqd) = 6897.6 \cdot kip \cdot ft$

$CheckMomentCapacity := if(\min(CR_{Str.mom}) > 0.99, "OK", "No Good!")$ $\qquad CheckMomentCapacity = "OK"$

Strength Shear and Associated Moments



$\max(V_{u.Str}) = 335.8 \cdot kip$ $\max(Mshr_{u.Str}) = 6547.8 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing: tmp_s = (in)	Number of Spaces: tmp_NumberSpaces =	area per stirrup: $tmp\_A_{stirrup}$ = (in²)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

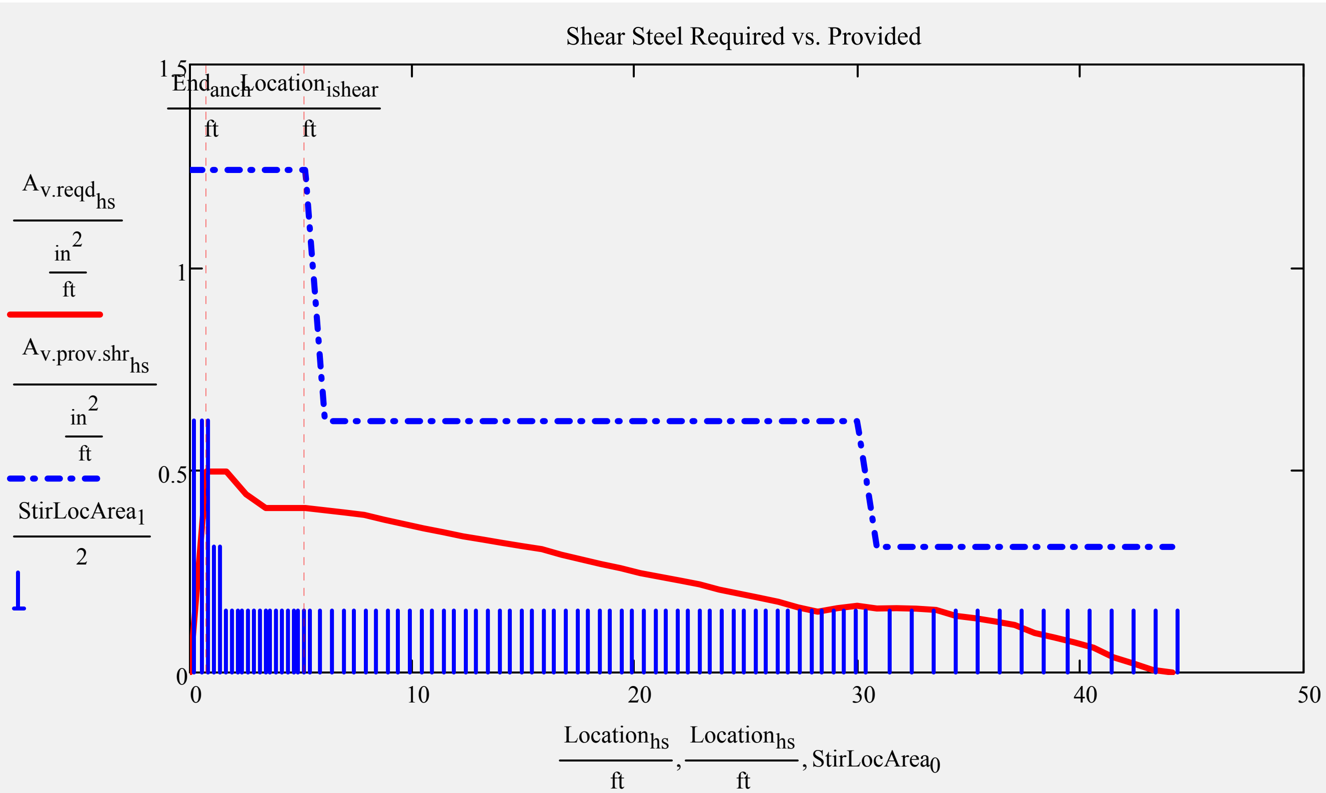
Location	$user\_s_{nspacings}$:=	$user\_NumberSpaces_{nspacings}$:=	$user\_A_{stirrup_{nspacings}}$:=	$interface\_factor_{nspacings}$:=
A1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.25
A2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.5
A3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S4 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

Stirrup sizes and spacings used in analysis

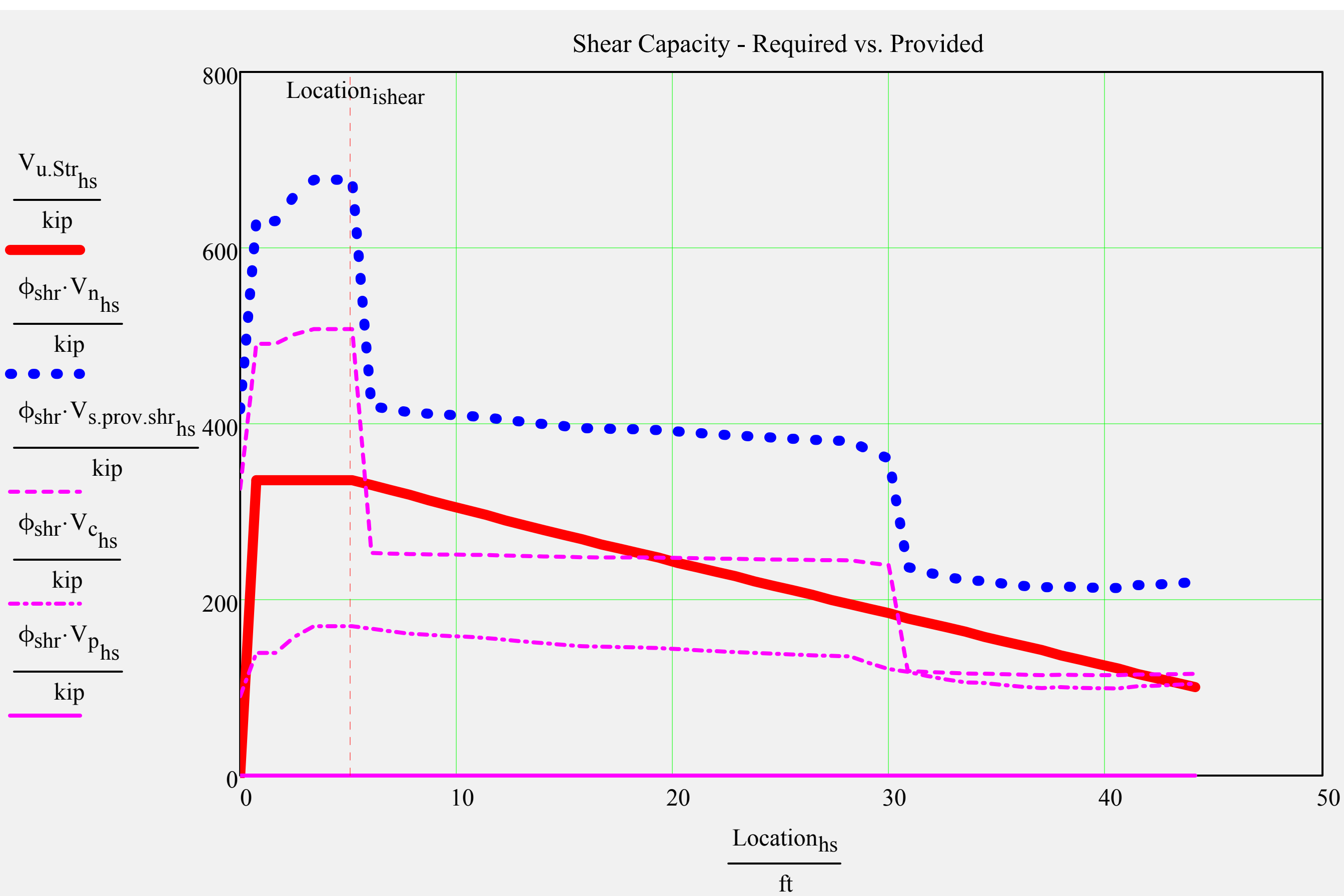
The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s = (in)	NumberSpaces =	$A_{stirrup}$ = (in²)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

EndCover = 2.5·in



Shear Steel Required vs. Provided
End_anch / ft
Location_ishear / ft
A_v.reqd_hs / (in^2/ft)
A_v.prov.shr_hs / (in^2/ft)
StirLocArea_1 / 2
1.5
1
0.5
0
0
10
20
30
40
50
Location_hs / ft, Location_hs / ft, StirLocArea_0



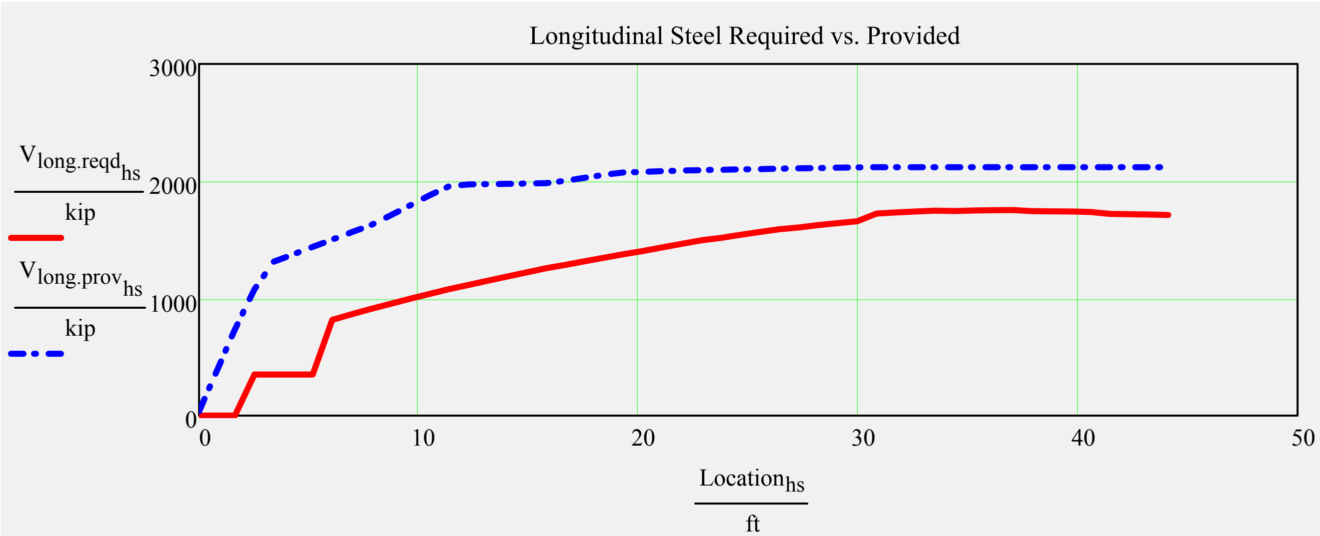
Shear Capacity - Required vs. Provided
Location_ishear
V_u.Str_hs / kip
ϕ_shr · V_n_hs / kip
ϕ_shr · V_s.prov.shr_hs / kip
ϕ_shr · V_c_hs / kip
ϕ_shr · V_p_hs / kip
800
600
400
200
0
0
10
20
30
40
50
Location_hs / ft

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

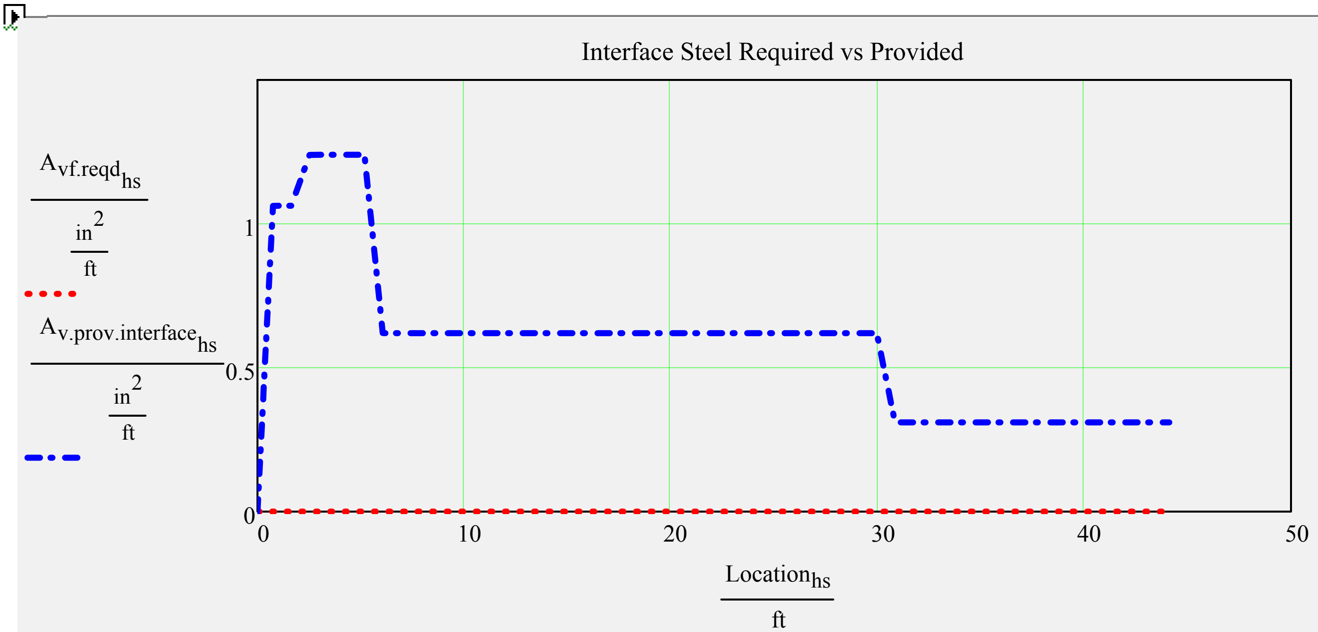


$$CR_{LongSteel_{hs}} := \text{if}\left(V_{long.reqd_{hs}} < .01\text{kip}, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$$

$$\min(CR_{LongSteel}) = 1.21$$

$$\text{CheckLongSteel} := \text{if}\left(\min(CR_{LongSteel}) > 1, \text{"OK"}, \text{"No Good, add steel!"}\right)$$

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, \text{"OK"}, \text{"No Good"}\right)$

CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A." CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0 := AcceptAASHTO$

$check_1 := AcceptSDG$

$check_2 := AcceptOntario$

$check_3 := Check\_f_{pt}$

$check_4 := Check\_f_{pe}$

$check_5 := Check\_f_{tension.rel}$

$check_6 := Check\_f_{comp.rel}$

$check_7 := Check\_f_{tension.stage8}$

$check_8 := Check\_f_{comp.stage8.c1}$

$check_9 := Check\_f_{comp.stage8.c2}$

$check_{10} := Check\_f_{comp.stage8.c3}$

$check_{11} := CheckMomentCapacity$

$check_{12} := CheckMaxCapacity$

$check_{13} := CheckStirArea$

$check_{14} := CheckShearCapacity$

$check_{15} := CheckMinStirArea$

$check_{16} := CheckMaxStirSpacing$

$check_{17} := CheckLongSteel$

$check_{18} := CheckInterfaceSpacing$

$check_{19} := CheckSplittingSteel$

$check_{20} := CheckMaxPrestressingForce$

$check_{21} := CheckPattern_0$

$check_{22} := CheckPattern_1$

$check_{23} := CheckPattern_2$

$check_{24} := CheckPattern_3$

$check_{25} := CheckPattern_4$

$check_{26} := CheckInterfaceSteel$

$check_{27} := CheckStrandFit$

$check_{28} := Check\_SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T =$

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8:
FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

Moment (Strength) or Stress (Service)   Shear (Strength)

$LRFR_{loadrating}$ =

"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.91	1.42	"N/A"	43.37	1.13	1.44	"N/A"	5.31	HL-93
"Strength I(Op)"	0.91	1.83	"N/A"	43.37	1.13	1.86	"N/A"	5.31	HL-93
"Service III(Inv)"	0.91	1.38	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.91	1.51	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.91	1.50	89.71	43.37	1.13	1.44	86.27	30.09	*Permit
"Service III"	0.91	1.41	84.39	45.13	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.25$ $CR_{LongSteel.Permit} = 1.21$ $CheckLongSteel_{loadrating}$ = "OK"

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltA_ 90'_FIB-45_SE

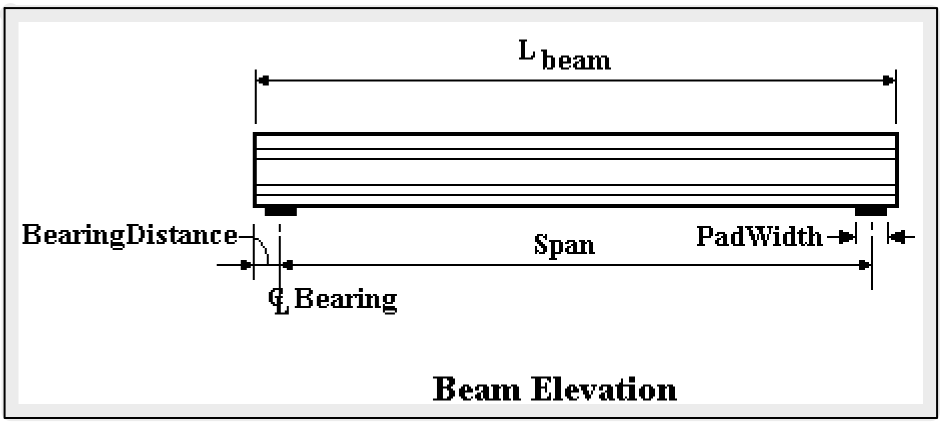
Comment = "Alt. A - SB Exterior"

Legend

TanHighlight **=** DataEntry   YellowHighlight **=** CheckValues   GreyHighlight **=** UserComments + Graphs

BlackText **=** ProgramEquations   *Maroon Text = Code Reference*   *Blue Text = Commentary*

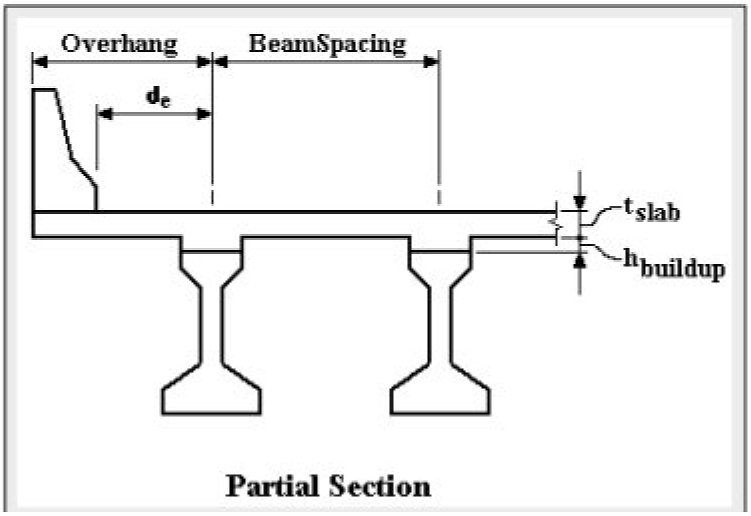
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ $Span = 88.5 \cdot ft$ $BearingDistance = 9 \cdot in$ $PadWidth = 10 \cdot in$

BeamTypeTog = "FIB45"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



Partial Section

$Overhang = 4.52 \cdot ft$ $\quad$ $BeamSpacing = 11.75 \cdot ft$ $\quad$ $t_{slab} = 8 \cdot in$ $\quad$ $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ $\quad$ $t_{integral.ws} = 0.5 \cdot in$ $\quad$ $NumberOfBeams = 5$ $\quad$ $t_{slab.delta} = 0 \cdot in$

$de = 2.75\ ft$

BeamPosition = "exterior" *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 10.4\ ft$ *effective slab width* *LRFD 4.6.2.6*

$t_{slab} := if(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

SectTog := if(SectionType = "transformed", 1, 0) $\quad$ SectTog = 1

Concrete:

Corrosion Classification	Environment = "moderately"	*density of slab concrete*	$\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$
strength of slab concrete	$f_{c.slab} = 4.5 \cdot ksi$	*density of beam concrete*	$\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$
strength of beam concrete	$f_{c.beam} = 8.5 \cdot ksi$	*weight of future wearing surface*	$Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$
release beam strength	$f_{ci.beam} = 6 \cdot ksi$		
type of course aggregate, "Florida" *or* "Standard"	AggregateType = "Florida"	*relative humidity*	$H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ *used in distribution calculation* $\quad$ $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9\cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor\cdot\sqrt{f_{ci.beam}\cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012\cdot ksi$

$E_c := AggFactor\cdot\sqrt{f_{c.beam}\cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776\cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270\cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

tendon modulus of elasticity $E_p = 28500\cdot ksi$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60\cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4\cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0\cdot in$

mild steel modulus of elasticity $E_s = 29000\cdot ksi$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62\cdot\frac{in^2}{ft}$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0\cdot in^2$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \quad 53.33 \quad 53.33)\cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0)\cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.97$ $tmp\_g_{shear} = 1.02$

user value overrides (optional):

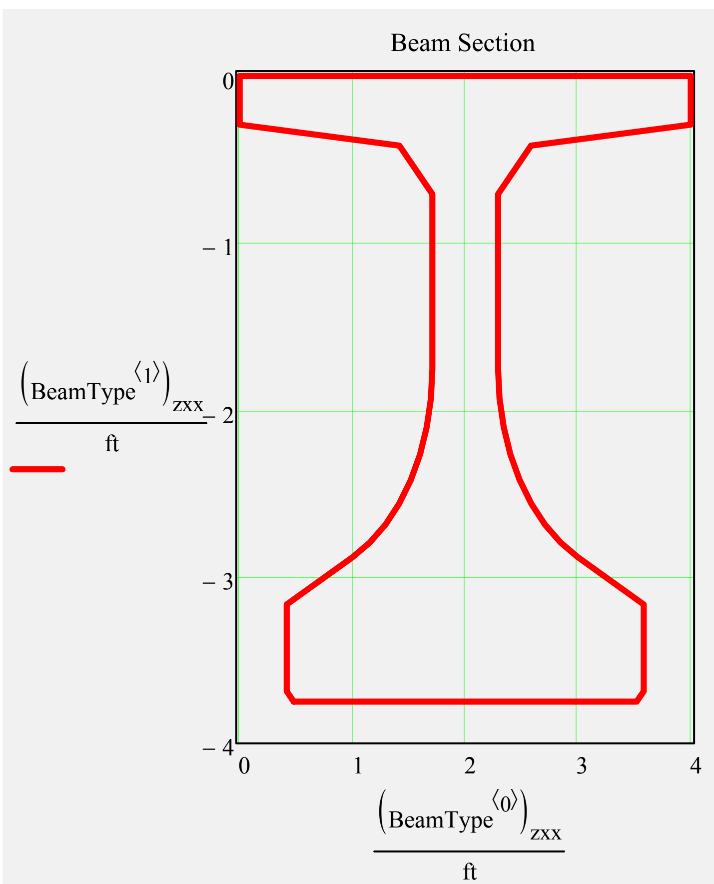
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

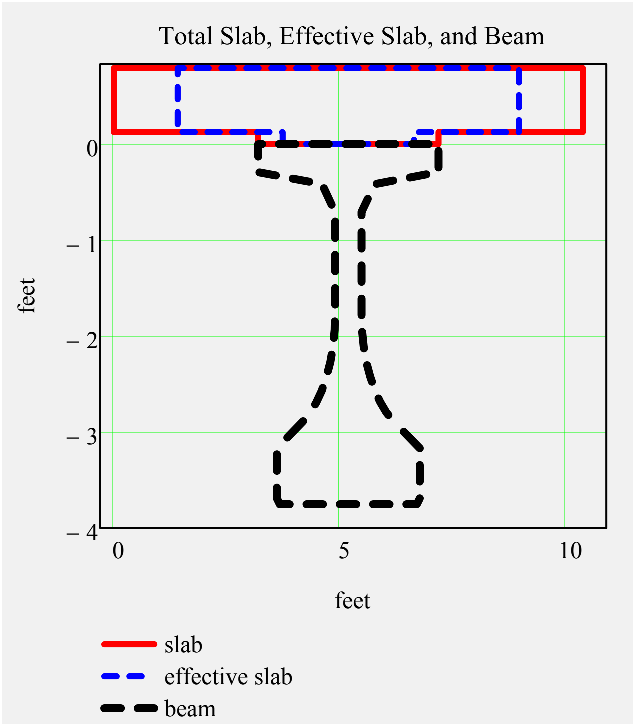
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.97$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 1.02$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 1.179 \cdot \frac{kip}{ft}$ $w_{beam} = 0.907 \cdot \frac{kip}{ft}$ $w_{forms} = 0.078 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$

additional non composite dead load (positive or negative)
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 2.164 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 1.257 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

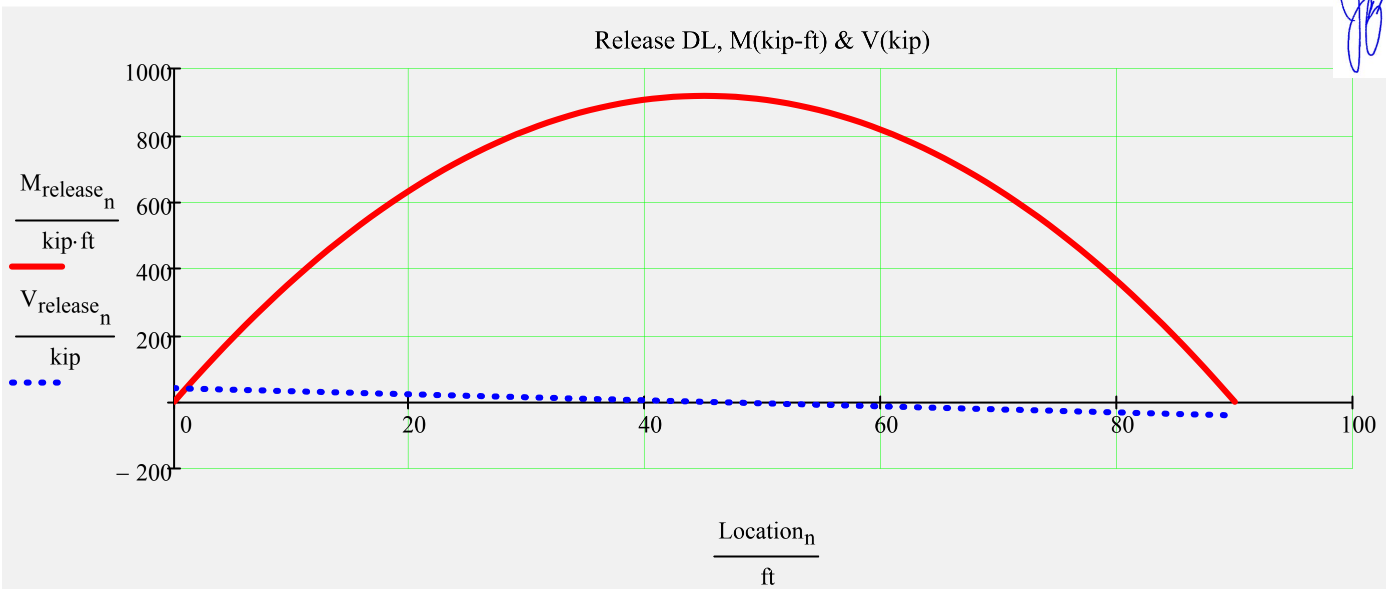
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.168 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.168 \cdot \frac{kip}{ft}$

$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.168 \cdot \frac{kip}{ft}$

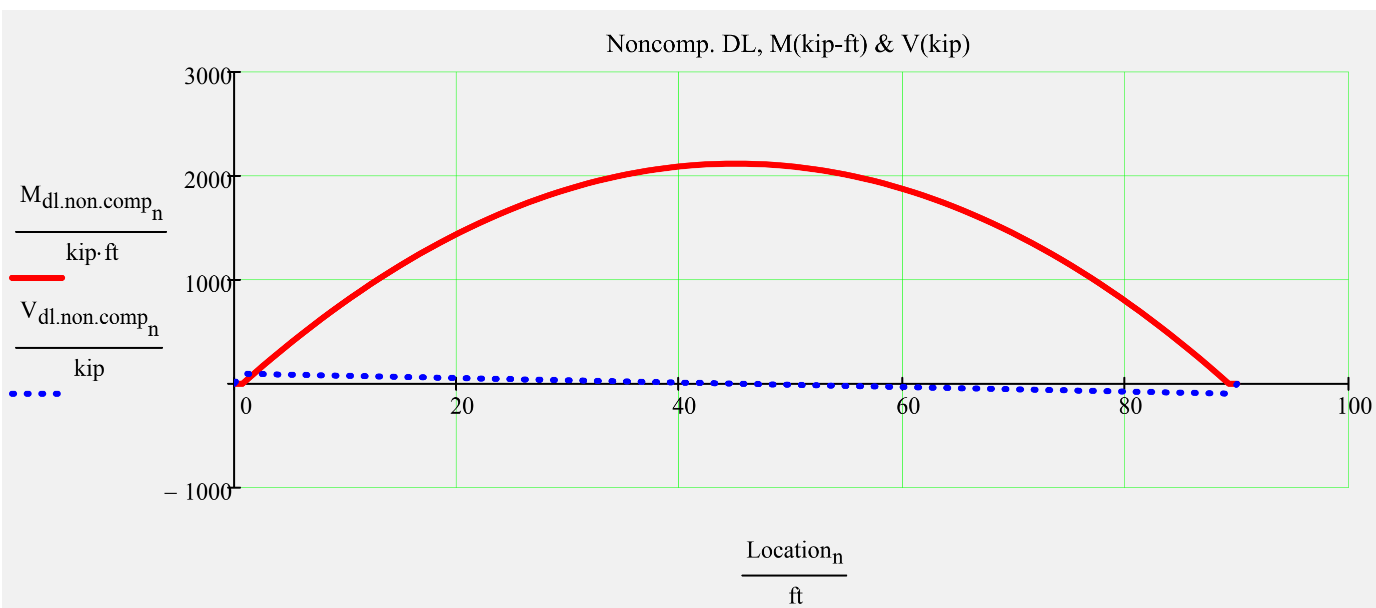
Release Dead Load Moments and Shear



$\max(M_{release}) = 918 \cdot kip \cdot ft$

$\max(V_{release}) = 40.8 \cdot kip$

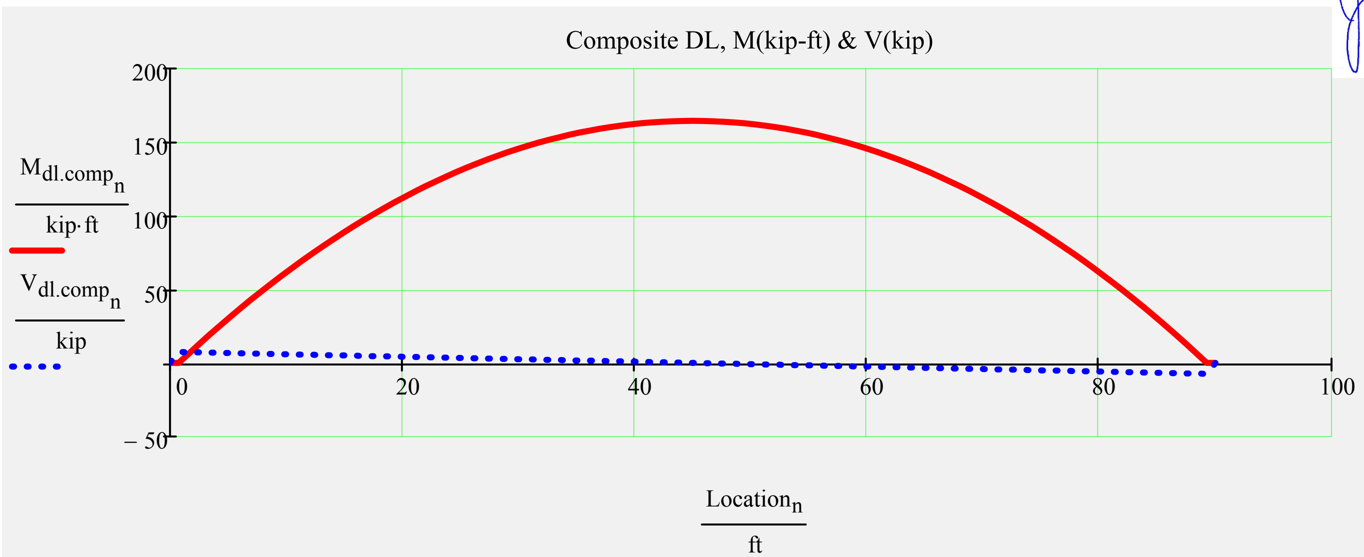
Noncomposite Dead Load Moments and Shear



$\max(M_{dl.non.comp}) = 2117.7 \cdot kip \cdot ft$

$\max(V_{dl.non.comp}) = 95.7 \cdot kip$

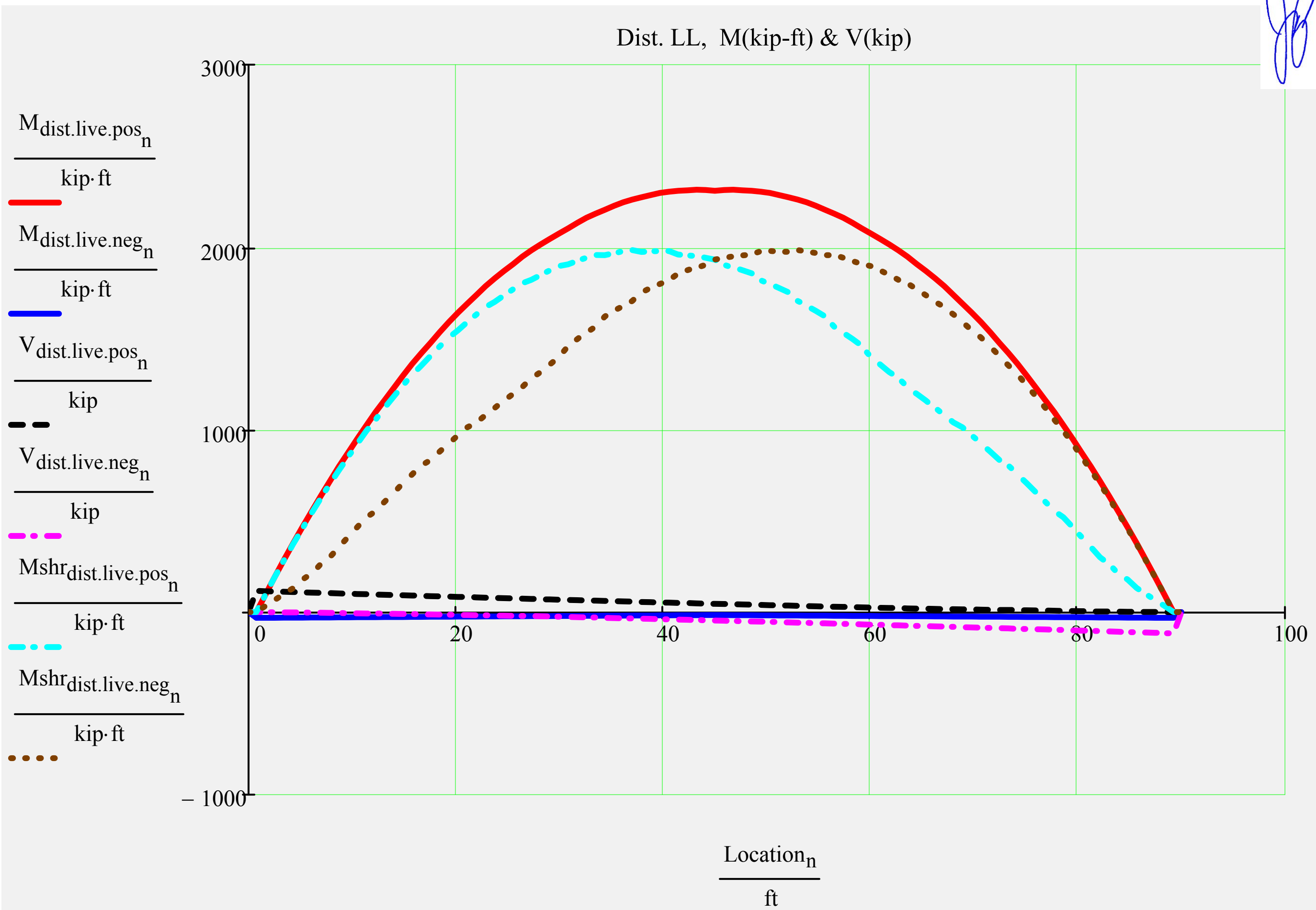
Composite Dead Load Moments and Shear



$max(M_{dl.comp}) = 164.4 \cdot kip \cdot ft$

$max(V_{dl.comp}) = 7.4 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 2314.4 \cdot kip \cdot ft$ $\min(M_{dist.live.neg}) = -31.2 \cdot kip \cdot ft$ $Reaction_{LL} = 118.04 \cdot kip$

$\max(V_{dist.live.pos}) = 116.7 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 1984.1 \cdot kip \cdot ft$ $Reaction_{DL} = 104.92 \cdot kip$

Prestress Strand Layout Input

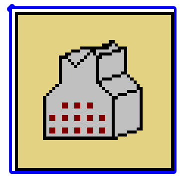
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate := Standard / Custom

Strand Pattern Generator:



Collapsed Region for Custom Strand Sizes...

$CheckPattern_0 =$ "OK" — ***check 0*** *- no debonded tendon in outside row*

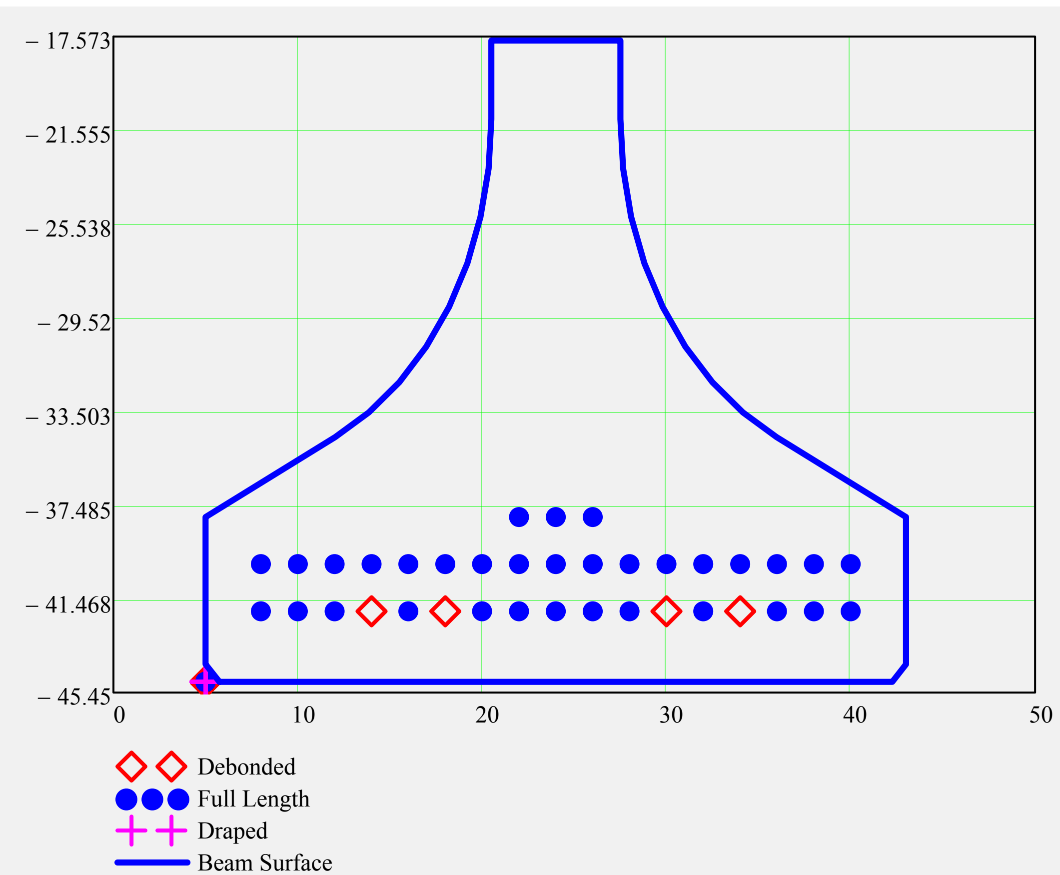
$CheckPattern_1 =$ "OK" — ***check 1*** *- less than 25% debonded tendons total*

$CheckPattern_2 =$ "OK" — ***check 2*** *- less than 40% debonded tendons in any row*

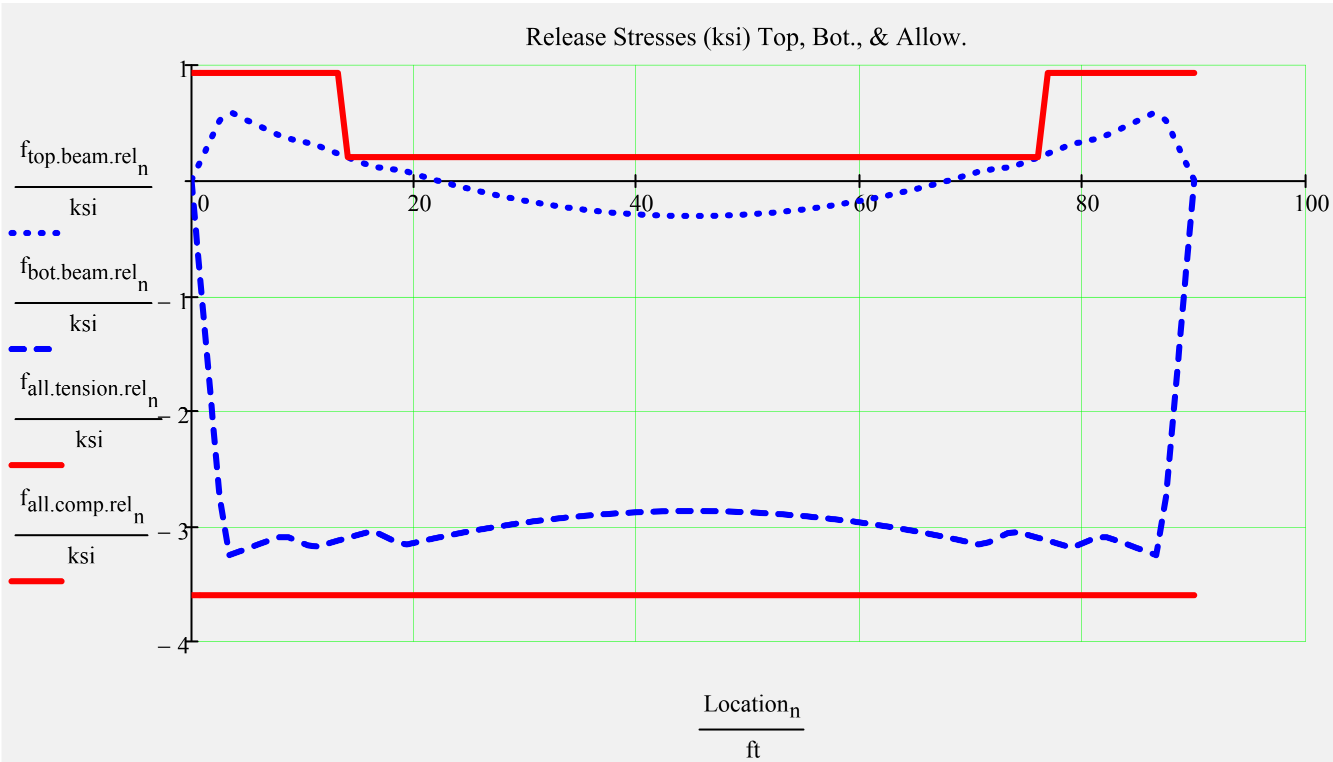
$CheckPattern_3 =$ "OK" — ***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*

$CheckPattern_4 =$ "OK" — ***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

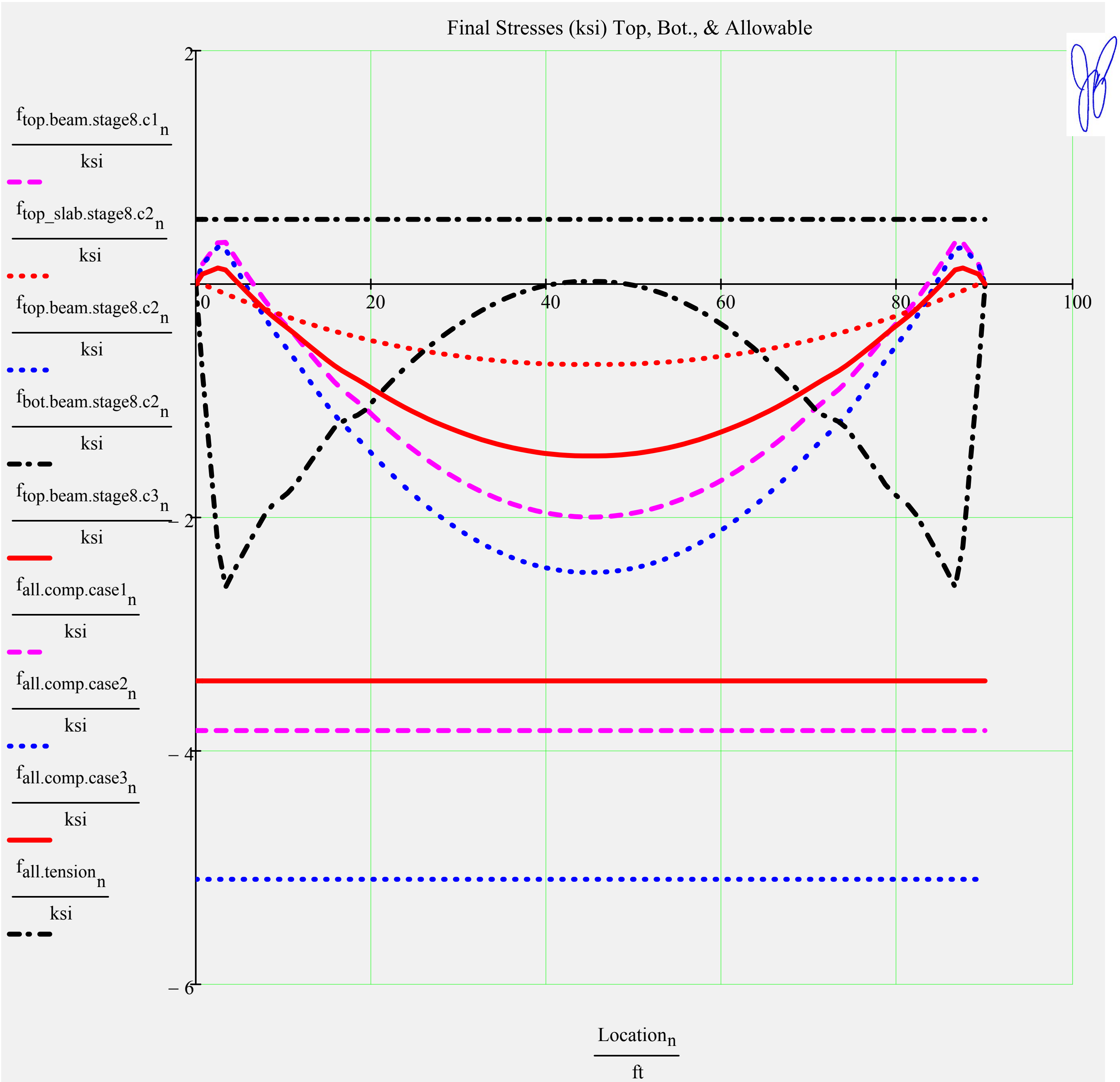
Tendon Layout



Release Stresses



Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.01$ Check_$f_{tension.rel}$ = "OK" (Release tension)

$\min(CR\_f_{comp.rel}) = 1.11$ Check_$f_{comp.rel}$ = "OK" (Release compression)

$\min(CR\_f_{tension.stage8}) = 10$ Check_$f_{tension.stage8}$ = "OK" (Service III , PS + DL +LL*0.8)

$\min(CR\_f_{comp.stage8.c1}) = 1.92$ Check_$f_{comp.stage8.c1}$ = "OK" (Service I , PS + DL)

$\min(CR\_f_{comp.stage8.c2}) = 2.06$ | $Check\_f_{comp.stage8.c2} = \text{"OK"}$ | *(Service I , PS + DL +LL)*

$\min(CR\_f_{comp.stage8.c3}) = 2.31$ | $Check\_f_{comp.stage8.c3} = \text{"OK"}$ | *(Service I , (PS + DL)*0.5 +LL)*

Section and Strand Properties Summary

$A_{beam} = 870.4 \cdot in^2$ — *Concrete area of beam*

$I_{beam} = 226606.0804 \cdot in^4$ — *Gross Moment of Inertia of Beam about CG*

$y_{comp} = -10.64 \cdot in$ — *Dist. from top of beam to CG of gross composite section*

$I_{comp} = 600792.9432 \cdot in^4$ — *Gross Moment of Inertia Composite Section about CG*

$A_{deck} = 778.83 \cdot in^2$ — *Concrete area of deck slab*

$A_{ps} = 8 \cdot in^2$ — *total area of strands*

$d_{b.ps} = 0.6 \cdot in$ — *diameter of Prestressing strand*

$\min(PrestressType) = 0$ — *0 - low lax 1 - stress relieved*

$f_{py} = 243 \cdot ksi$ — *tendon yield strength*

$f_{pj} = 203 \cdot ksi$ — *prestress jacking stress*

$L_{shielding}{}^T = (8 \;\; 16 \;\; 0 \;\; 0 \;\; 0) \cdot ft$

$A_{ps.row}{}^T = (0.4 \;\; 0.4 \;\; 2.8 \;\; 3.7 \;\; 0.7) \cdot in^2$

$d_{ps.row} =$ (values in $\cdot in$)

	0	1	2	3	4	5	6	7	8	9
0	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
1	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
2	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
3	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40
4	-38	-38	-38	-38	-38	-38	-38	-38	-38	...

$TotalNumberOfTendons = 37$

$StrandSize = \text{"0.6 in low lax"}$

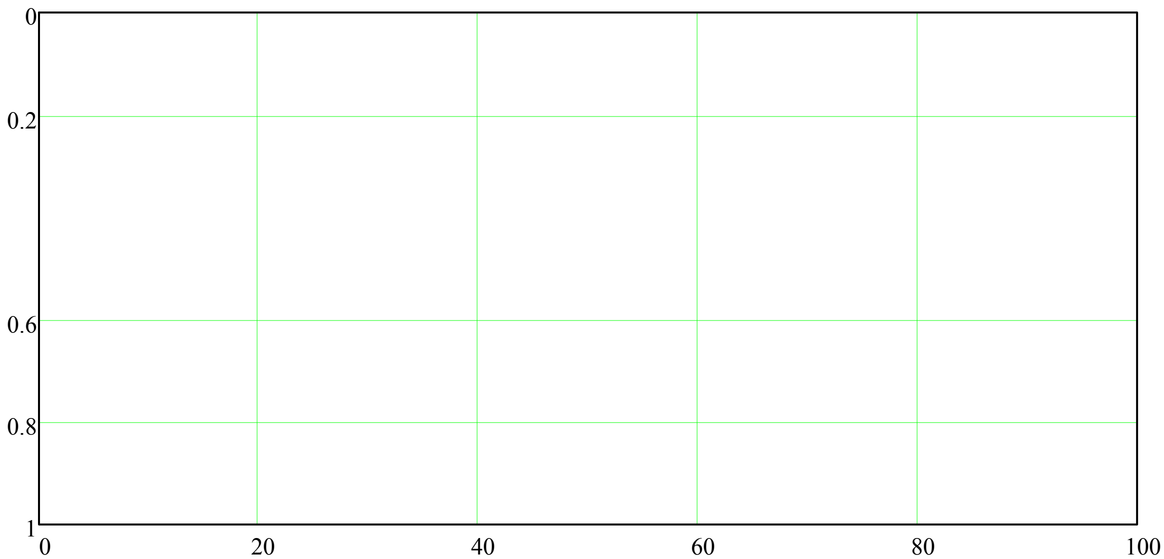
$NumberOfDebondedTendons = 4$

$StrandArea = 0.22 \cdot in^2$

$NumberOfDrapedTendons = 0$

$JackingForce_{per.strand} = 43.94 \cdot kip$

Location of Depressed Strands



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

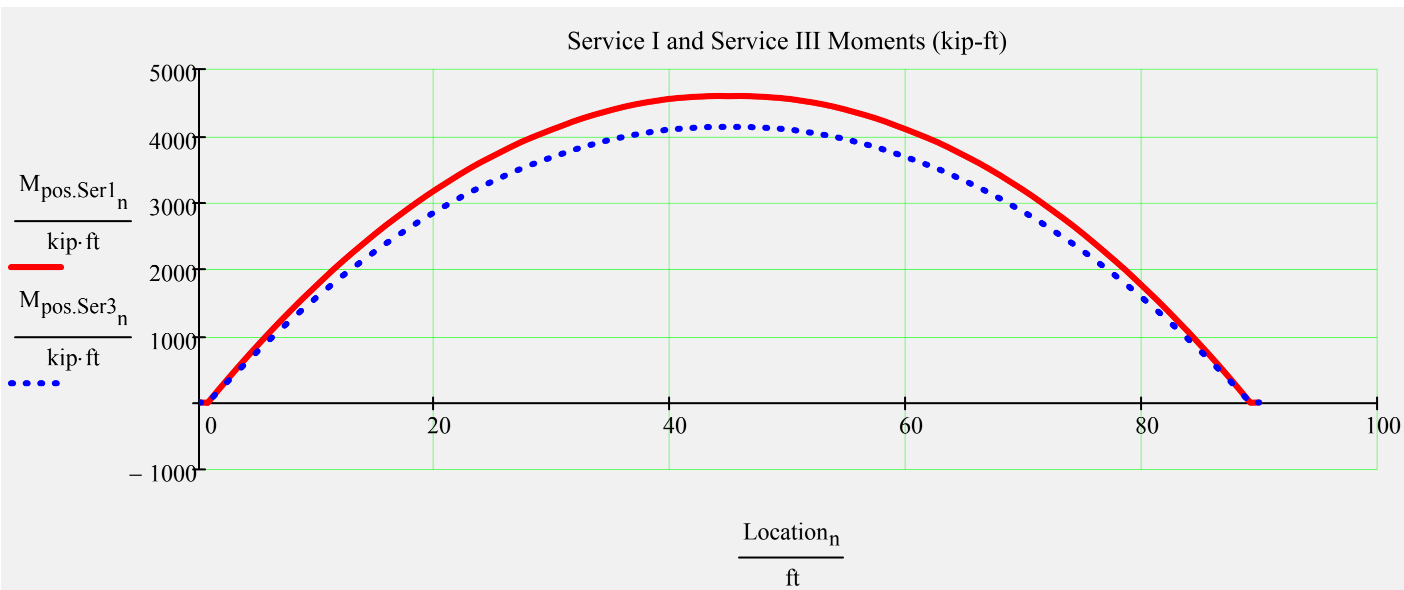
$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

$f_{pe} = 179 \cdot ksi$ $\Delta f_{pTot} = -23 \cdot ksi$

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.47 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.53 \cdot \%$

Check_f_pt = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f_pe = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 4593.8 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 4131.3 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.31	−2.87
2	−0.41	−2.44
4	−0.37	−2.47
6	−1.96	−1.25
8	−2.47	0.02

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−2183.5
"Final (about composite centroid)"	−1439.5	−1831.4

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	862.37	224527.31	−24.64
"Transformed Beam (initial)"	919.4	238511.11	−25.64
"Transformed Beam "	910.29	236392.94	−25.49
"Composite "	1721.83	645296.26	−11.03

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	918
"Non-composite (includes bm wt.)"	2117.7
"Composite"	164.4
"Distributed Live Load"	2309

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

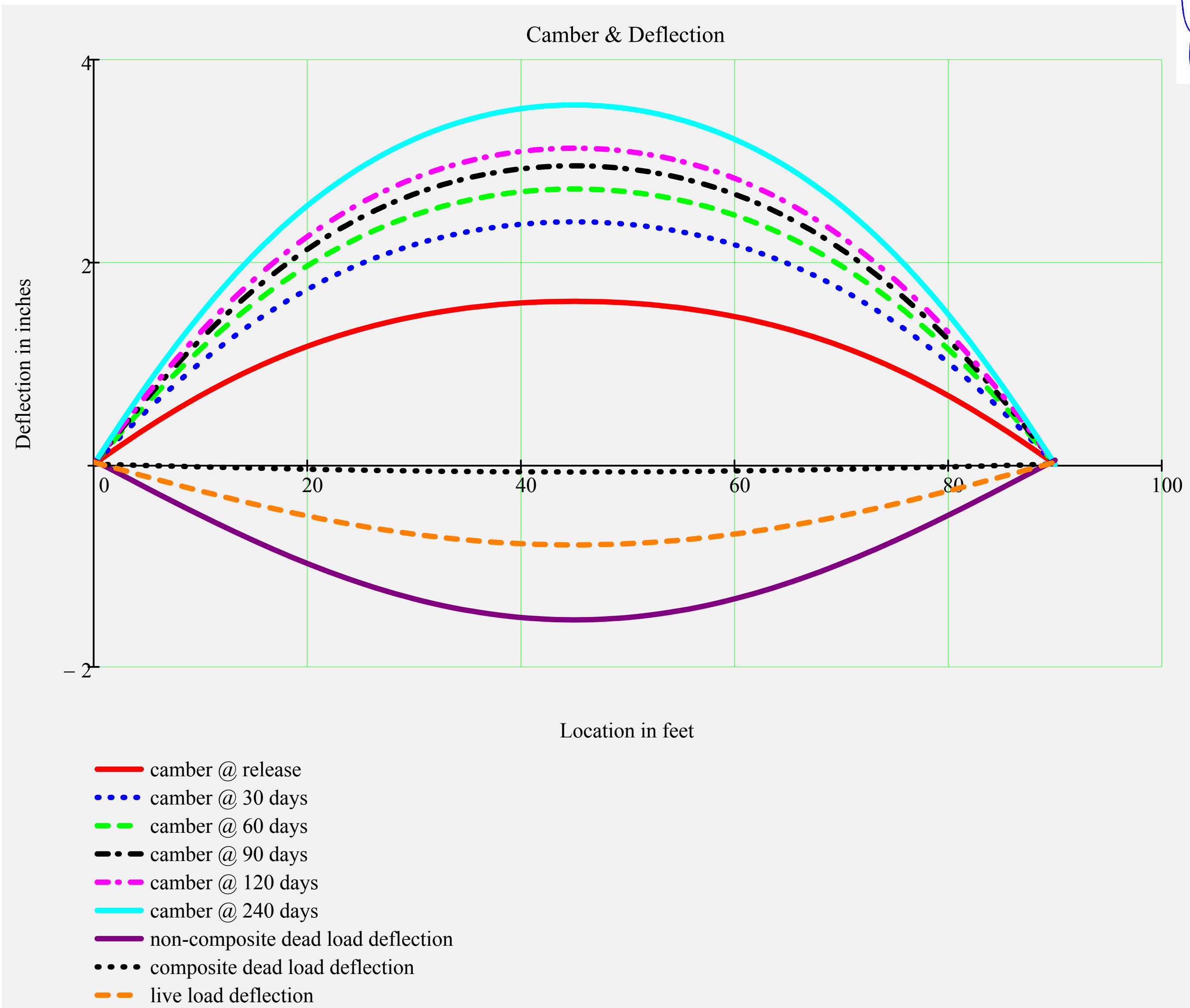
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

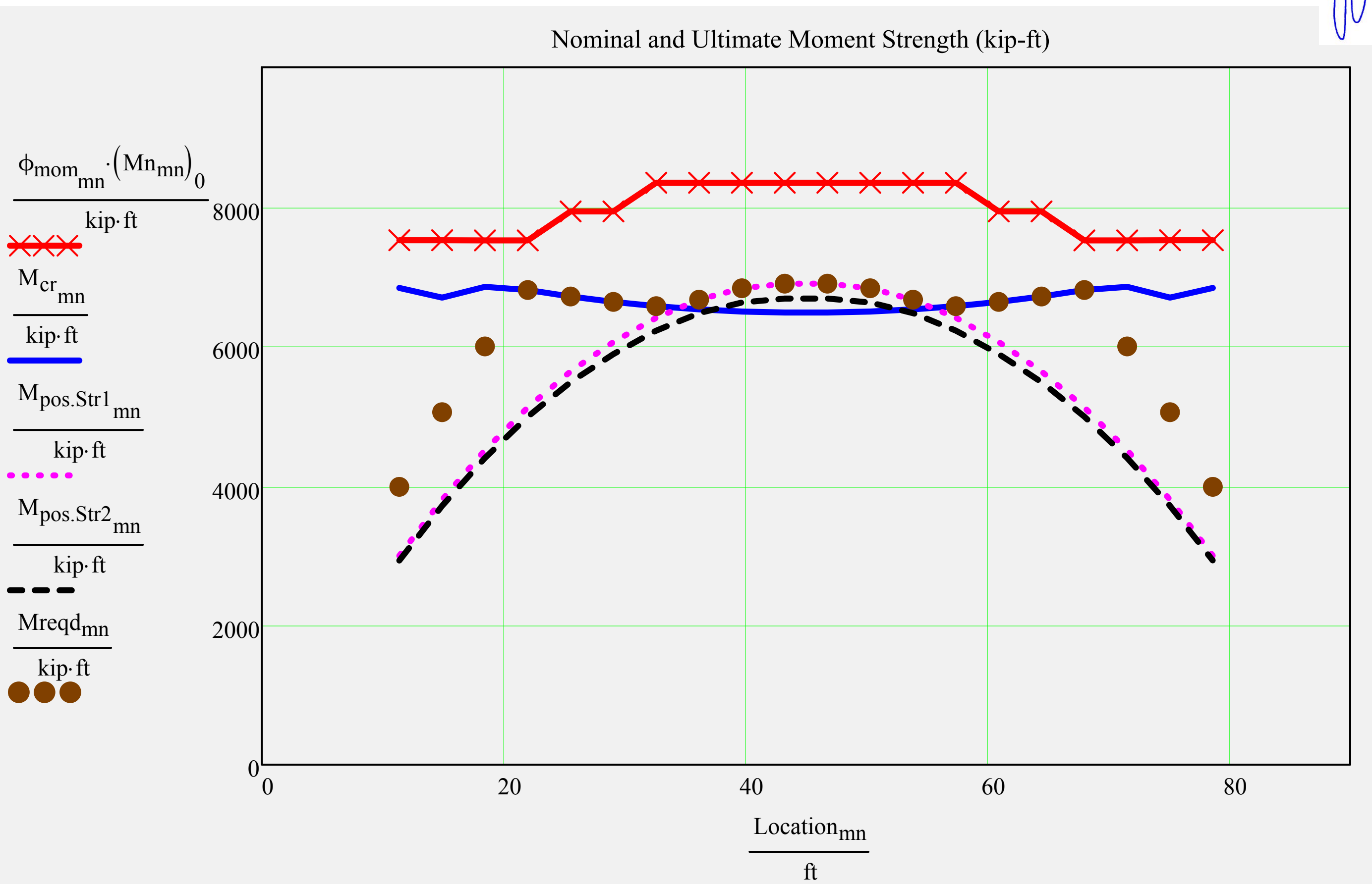
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	0.0105	−0.8054	0.3825	1.6109
"30 Days"	−0.1453	−1.4271	0.6214	2.3965
"60 Days"	−0.201	−1.6496	0.7148	2.7208
"90 Days"	−0.2307	−1.7681	0.7646	2.9497
"120 Days"	−0.2492	−1.8418	0.7955	3.124
"240 Days"	−0.2833	−1.9778	0.8526	3.5512
"non-comp DL"	−0.236	0.1808	−0.2655	−1.5367
"comp DL"	−0.005	0.0154	−0.013	−0.0752
"LL"	−0.0533	0.1643	−0.1386	−0.7956

Strength Limit State Moments



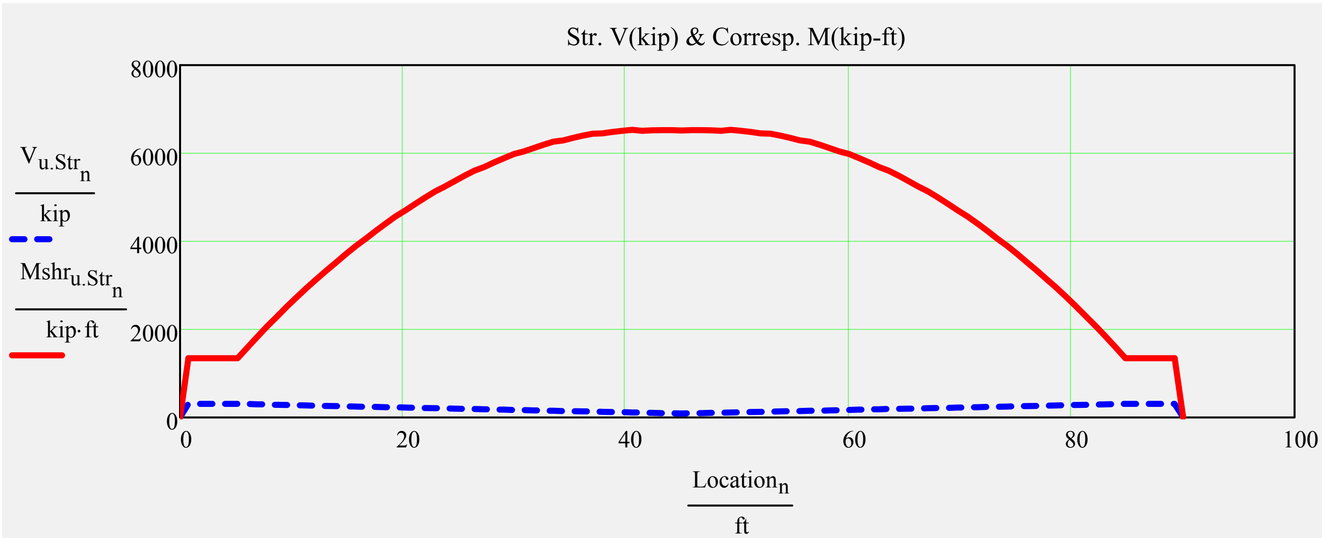
$CR_{Str.mom_n} := 10$ $\qquad CR_{Str.mom_{mn}} := \frac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ $\qquad$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.1$

$\max(Mreqd) = 6898.2 \cdot kip \cdot ft$

CheckMomentCapacity := if(min($CR_{Str.mom}$) > 0.99, "OK" , "No Good!") $\qquad$ CheckMomentCapacity = "OK"

Strength Shear and Associated Moments



$\max(V_{u.Str}) = 306.1 \cdot kip$ $\max(Mshr_{u.Str}) = 6528.4 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing	Number of Spaces	area per stirrup
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$tmp\_s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in$ $\quad tmp\_NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix}$ $\quad tmp\_A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

Location	$user\_s_{nspacings} :=$	$user\_NumberSpaces_{nspacings} :=$	$user\_A_{stirrup_{nspacings}} :=$	$interface\_factor_{nspacings} :=$
A1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.25
A2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.5
A3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S4 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

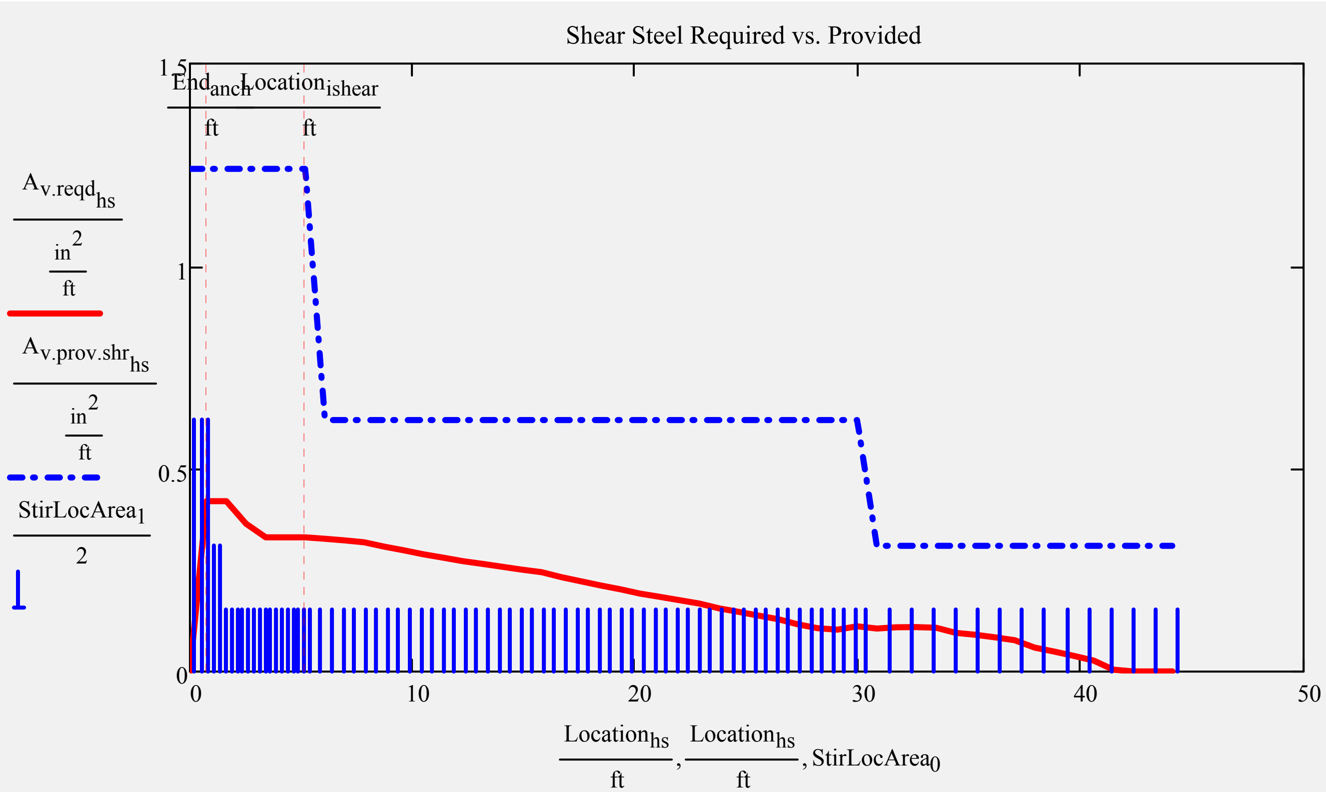
Stirrup sizes and spacings used in analysis

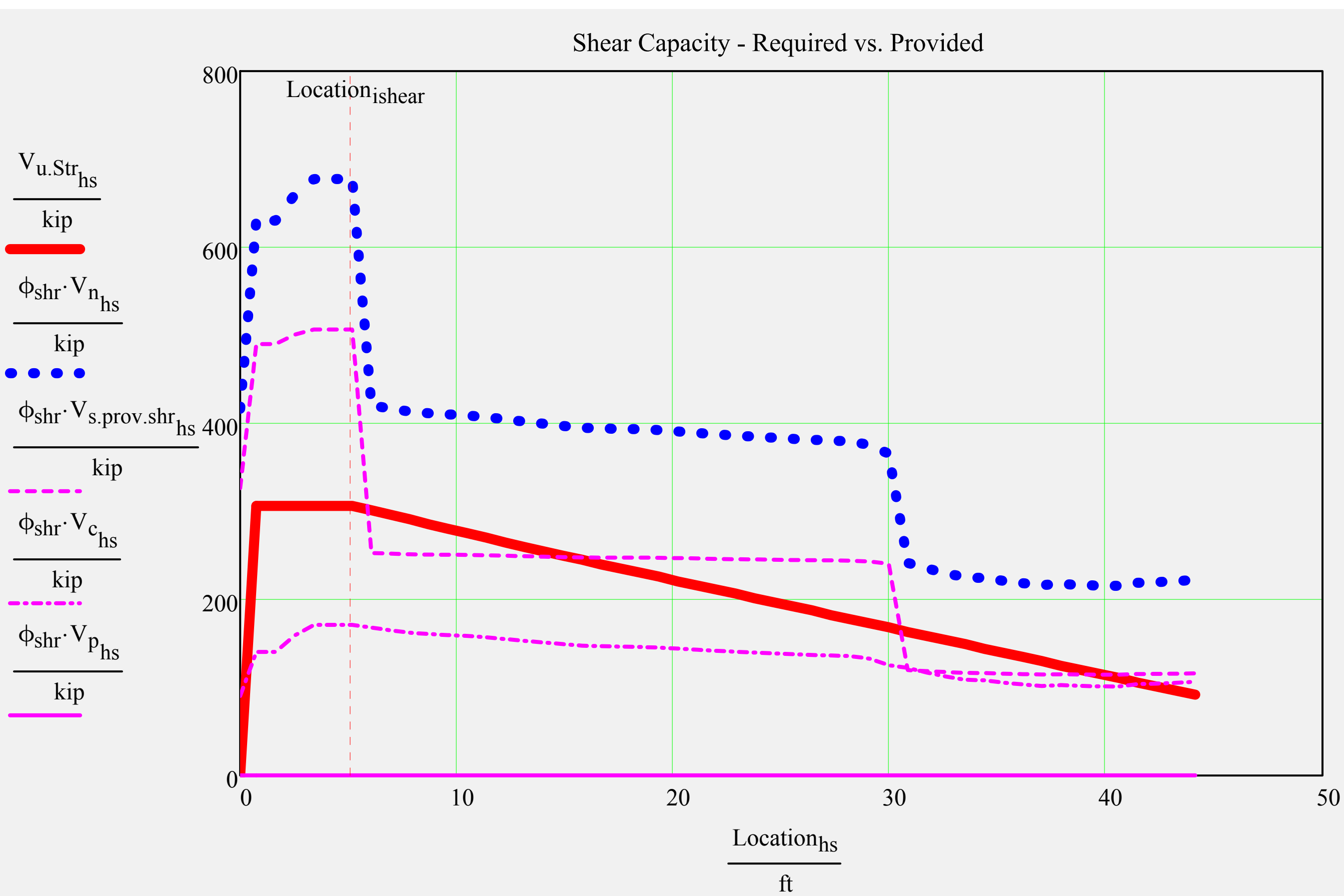
The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s (in)	NumberSpaces	$A_{stirrup}$ (in^2)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in$ $\quad NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix}$ $\quad A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

$EndCover = 2.5 \cdot in$


Shear Steel Required vs. Provided
$\frac{End_{anch}}{ft}$
$\frac{Location_{ishear}}{ft}$
$\frac{A_{v.reqd_{hs}}}{\frac{in^2}{ft}}$
$\frac{A_{v.prov.shr_{hs}}}{\frac{in^2}{ft}}$
$\frac{StirLocArea_1}{2}$
1.5
1
0.5
0
0
10
20
30
40
50
$\frac{Location_{hs}}{ft}, \frac{Location_{hs}}{ft}, StirLocArea_0$

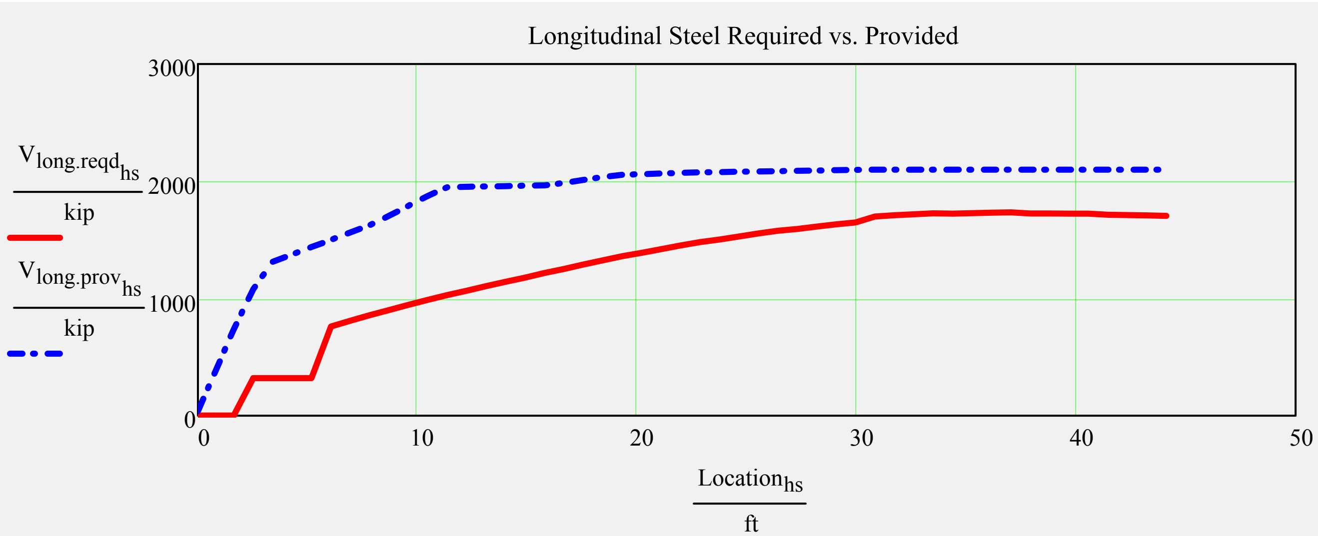

Shear Capacity - Required vs. Provided
$Location_{ishear}$
$\frac{V_{u.Str_{hs}}}{kip}$
$\frac{\phi_{shr} \cdot V_{n_{hs}}}{kip}$
$\frac{\phi_{shr} \cdot V_{s.prov.shr_{hs}}}{kip}$
$\frac{\phi_{shr} \cdot V_{c_{hs}}}{kip}$
$\frac{\phi_{shr} \cdot V_{p_{hs}}}{kip}$
800
600
400
200
0
0
10
20
30
40
50
$\frac{Location_{hs}}{ft}$

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

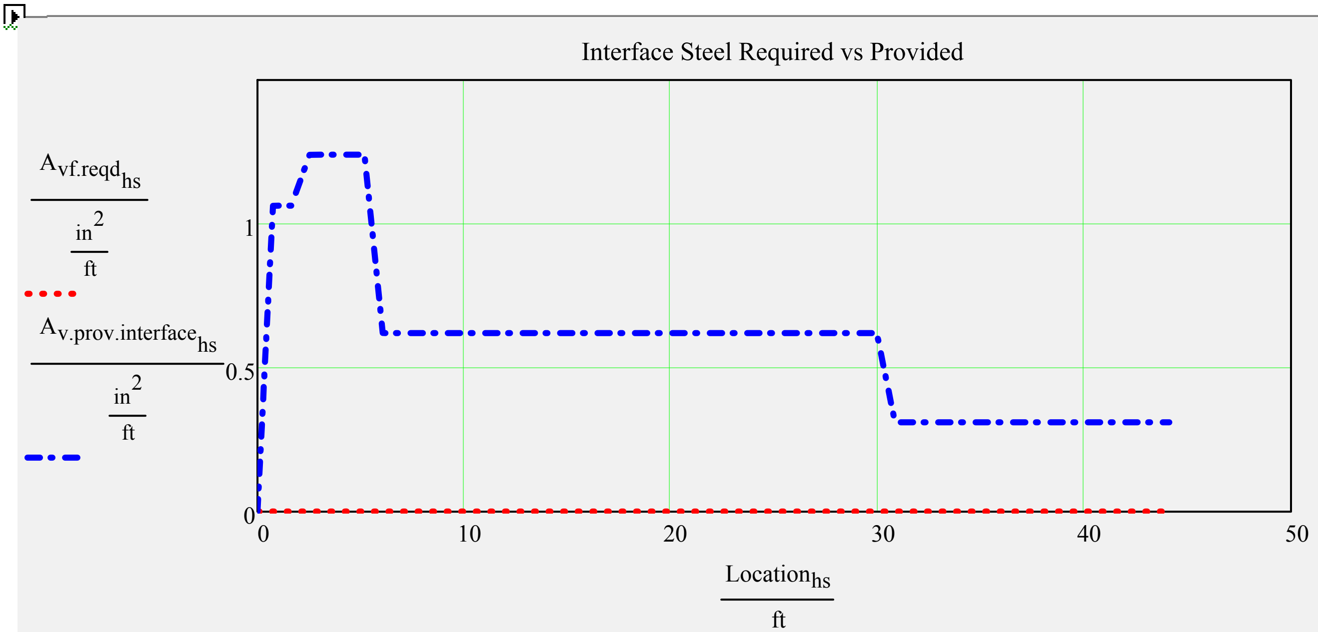


$$CR_{LongSteel_{hs}} := \text{if}\left(V_{long.reqd_{hs}} < .01kip, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$$

$$\min(CR_{LongSteel}) = 1.21$$

CheckLongSteel := if(min(CR_LongSteel) > 1 ,"OK" ,"No Good, add steel!")

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, \text{"OK"}, \text{"No Good"}\right)$$

$$CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = \text{"FLT"}, \text{"N.A."}, CheckInterfaceSteel)$$

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A."   CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0$:= AcceptAASHTO

$check_1$:= AcceptSDG

$check_2$:= AcceptOntario

$check_3$:= Check_f_{pt}

$check_4$:= Check_f_{pe}

$check_5$:= Check_$f_{tension.rel}$

$check_6$:= Check_$f_{comp.rel}$

$check_7$:= Check_$f_{tension.stage8}$

$check_8$:= Check_$f_{comp.stage8.c1}$

$check_9$:= Check_$f_{comp.stage8.c2}$

$check_{10}$:= Check_$f_{comp.stage8.c3}$

$check_{11}$:= CheckMomentCapacity

$check_{12}$:= CheckMaxCapacity

$check_{13}$:= CheckStirArea

$check_{14}$:= CheckShearCapacity

$check_{15}$:= CheckMinStirArea

$check_{16}$:= CheckMaxStirSpacing

$check_{17}$:= CheckLongSteel

$check_{18}$:= CheckInterfaceSpacing

$check_{19}$:= CheckSplittingSteel

$check_{20}$:= CheckMaxPrestressingForce

$check_{21}$:= $CheckPattern_0$

$check_{22}$:= $CheckPattern_1$

$check_{23}$:= $CheckPattern_2$

$check_{24}$:= $CheckPattern_3$

$check_{25}$:= $CheckPattern_4$

$check_{26}$:= CheckInterfaceSteel

$check_{27}$:= CheckStrandFit

$check_{28}$:= Check_$SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T$ =

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

*Structure Manual (SM) Vol-8:
FDOT Modifications to LRFR*

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

$LRFR_{loadrating}$ =

	Moment (Strength) or Stress (Service)				Shear (Strength)				
"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.97	1.36	"N/A"	46.02	1.02	1.64	"N/A"	5.31	HL-93
"Strength I(Op)"	0.97	1.76	"N/A"	46.02	1.02	2.12	"N/A"	5.31	HL-93
"Service III(Inv)"	0.97	1.45	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.97	1.57	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.97	1.43	85.98	46.02	1.02	1.64	98.55	30.98	*Permit
"Service III"	0.97	1.46	87.86	46.02	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.25$ $CR_{LongSteel.Permit} = 1.21$ $CheckLongSteel_{loadrating}$ = "OK"

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltA_ 90'_FIB-45_NI

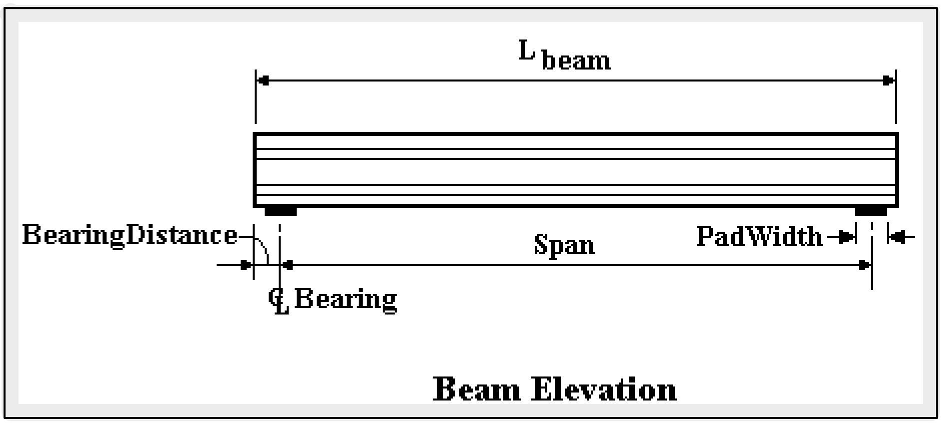
Comment = "Alt. A - NB Interior"

Legend

TanHighlight = DataEntry   YellowHighlight = CheckValues   GreyHighlight = UserComments + Graphs

BlackText = ProgramEquations   *Maroon Text = Code Reference*   *Blue Text = Commentary*

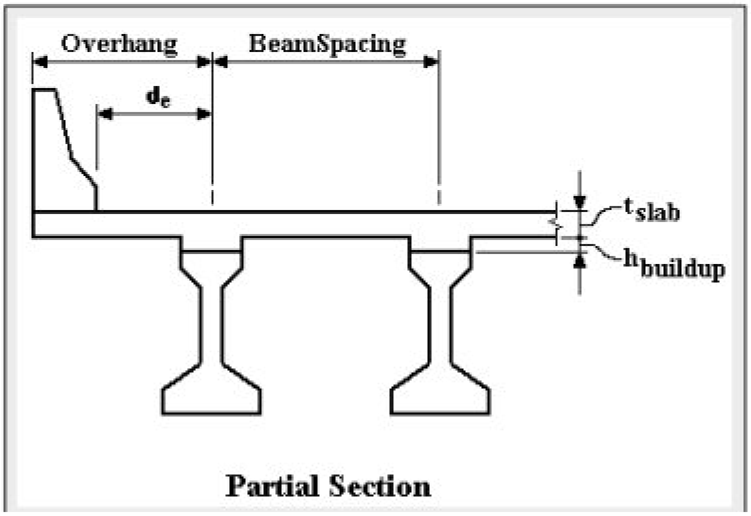
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ $Span = 88.5 \cdot ft$ $BearingDistance = 9 \cdot in$ $PadWidth = 10 \cdot in$

BeamTypeTog = "FIB45"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



$Overhang = 4.29 \cdot ft$ $BeamSpacing = 11.5 \cdot ft$ $t_{slab} = 8 \cdot in$ $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ $t_{integral.ws} = 0.5 \cdot in$ $NumberOfBeams = 4$ $t_{slab.delta} = 0 \cdot in$

$de = 2.75\,ft$

BeamPosition = "interior" *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 11.5\,ft$ *effective slab width* *LRFD 4.6.2.6*

$t_{slab} := if(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

$SectTog := if(SectionType = "transformed", 1, 0)$ $SectTog = 1$

Concrete:

Corrosion Classification Environment = "moderately"

density of slab concrete $\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$

strength of slab concrete $f_{c.slab} = 4.5 \cdot ksi$

density of beam concrete $\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$

strength of beam concrete $f_{c.beam} = 8.5 \cdot ksi$

weight of future wearing surface $Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$

release beam strength $f_{ci.beam} = 6 \cdot ksi$

type of course aggregate, "Florida" *or* "Standard" AggregateType = "Florida"

relative humidity $H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ *used in distribution calculation* $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9 \cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor \cdot \sqrt{f_{ci.beam} \cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012 \cdot ksi$

$E_c := AggFactor \cdot \sqrt{f_{c.beam} \cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776 \cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270 \cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

tendon modulus of elasticity $E_p = 28500 \cdot ksi$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60 \cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4 \cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0 \cdot in$

mild steel modulus of elasticity $E_s = 29000 \cdot ksi$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62 \cdot \frac{in^2}{ft}$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0 \cdot in^2$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \;\; 53.33 \;\; 53.33) \cdot kip$

$PermitAxleSpacing^T = (0 \;\; 14 \;\; 14 \;\; 0) \cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.89$ $tmp\_g_{shear} = 1.11$

user value overrides (optional):

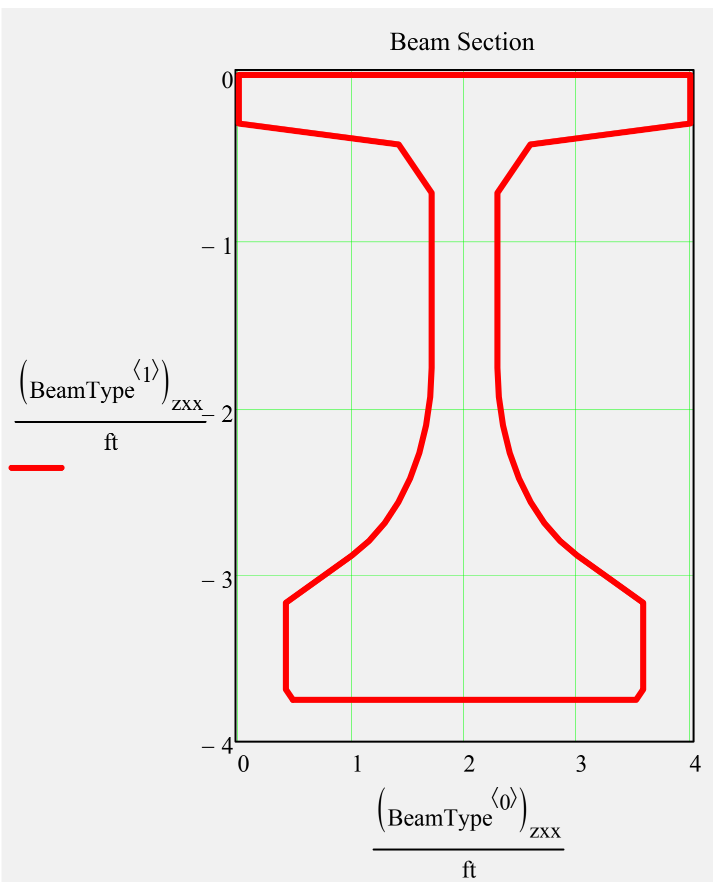
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

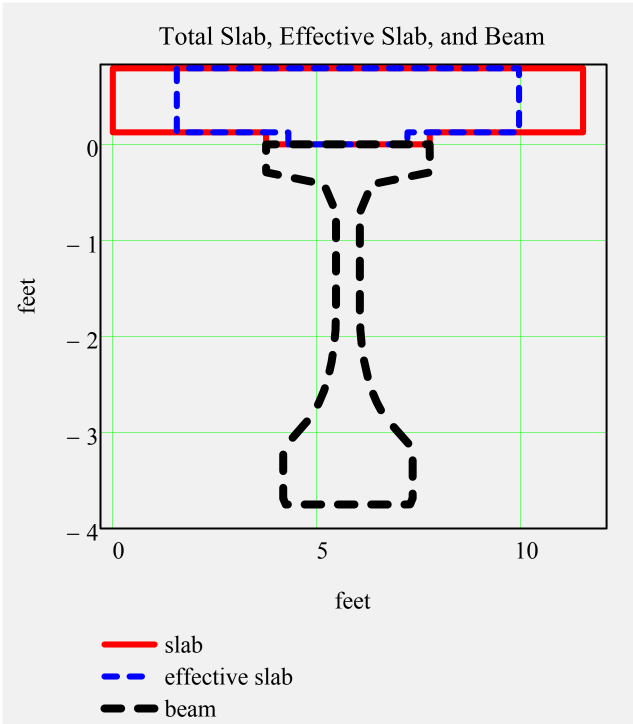
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.89$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 1.11$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 1.297 \cdot \frac{kip}{ft}$ $\quad w_{beam} = 0.907 \cdot \frac{kip}{ft}$ $\quad w_{forms} = 0.15 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$

additional non composite dead load (positive or negative)
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $\quad w_{noncomposite} = 2.354 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $\quad wb_{noncomposite} = 1.447 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.21 \cdot \frac{kip}{ft}$

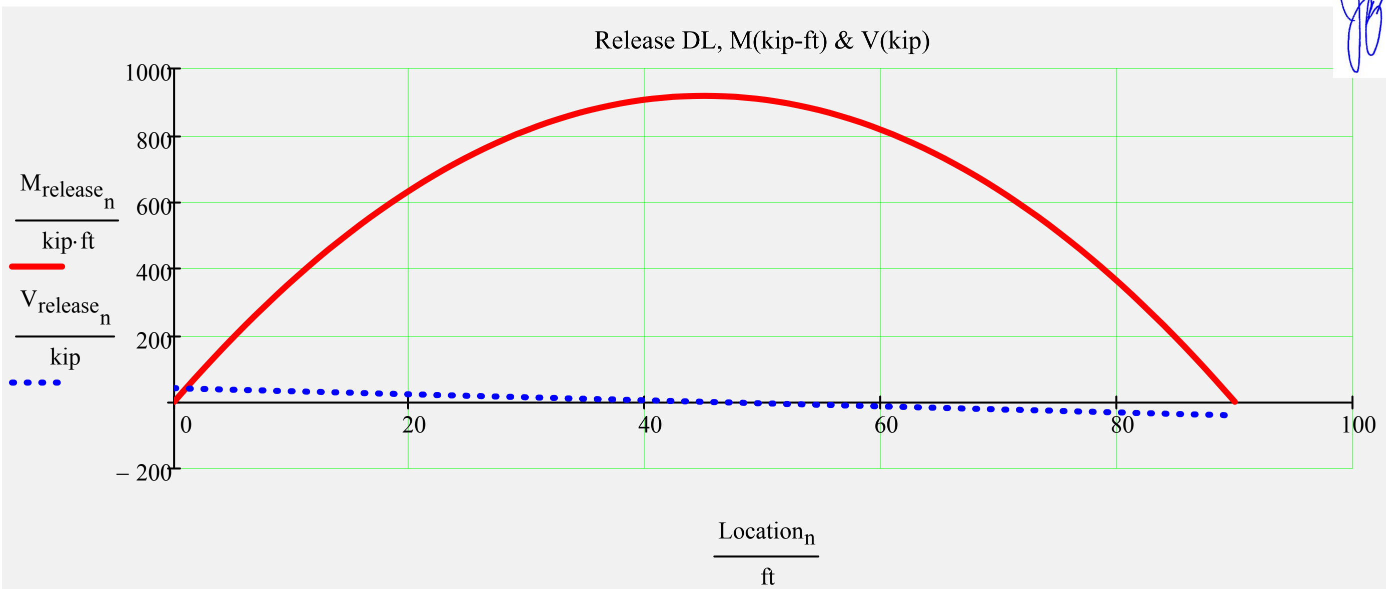
$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$

additional composite dead load (positive or negative)
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.21 \cdot \frac{kip}{ft}$

$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.21 \cdot \frac{kip}{ft}$

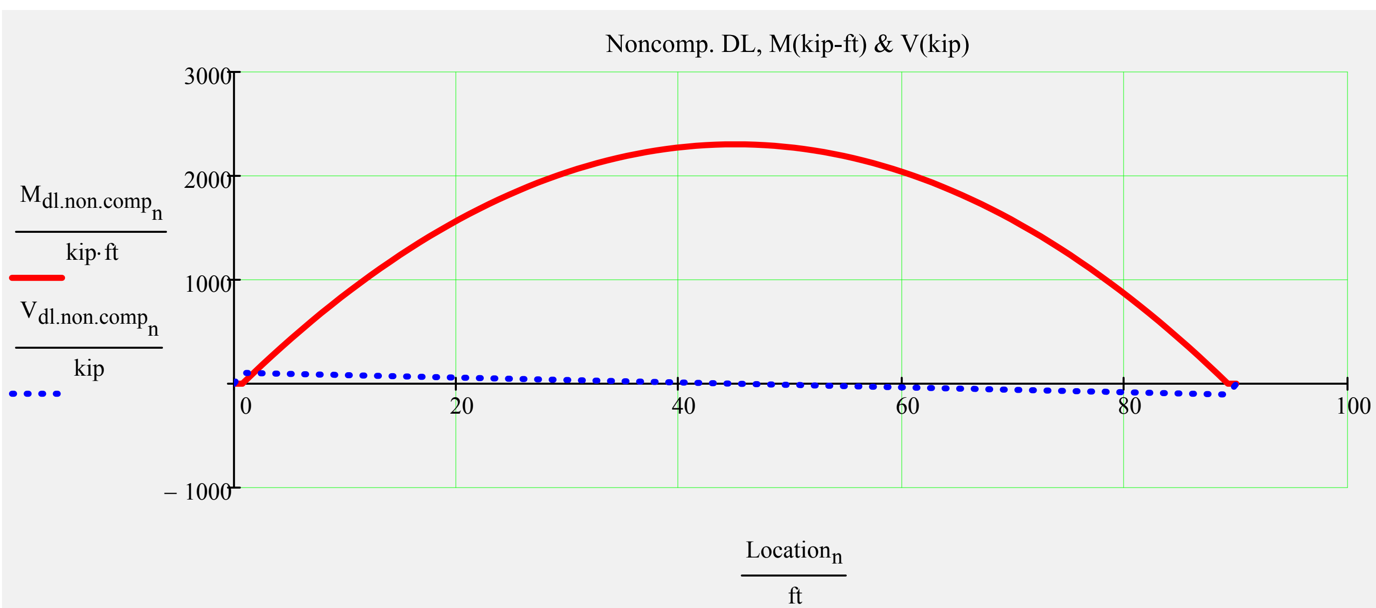
Release Dead Load Moments and Shear



$\max(M_{release}) = 918 \cdot kip \cdot ft$

$\max(V_{release}) = 40.8 \cdot kip$

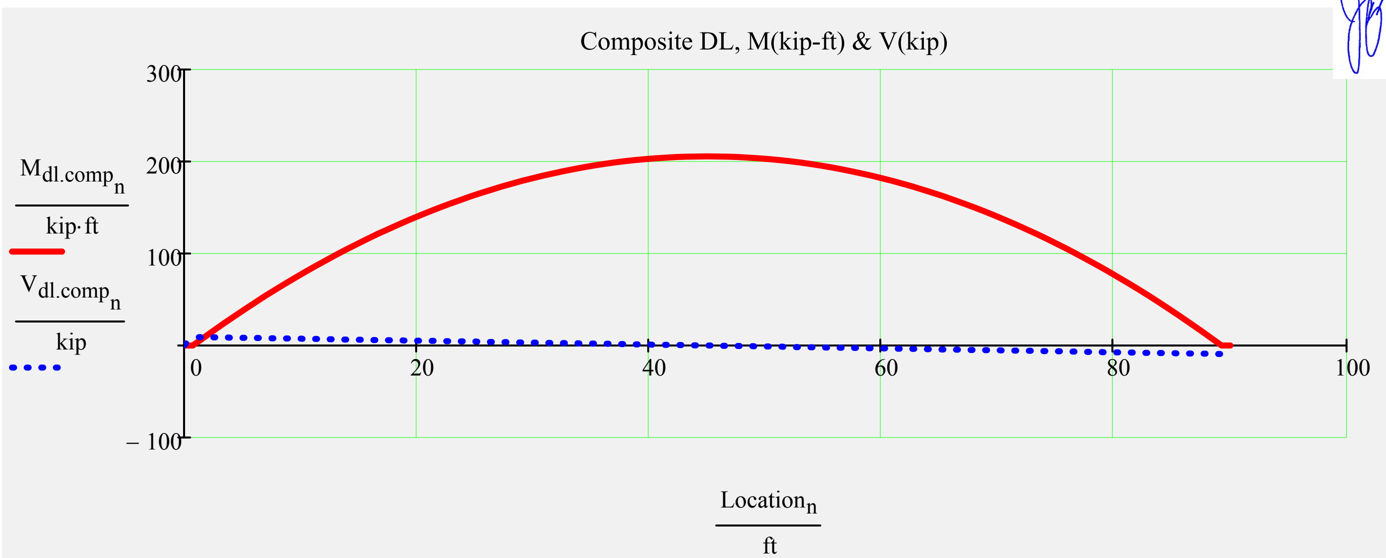
Noncomposite Dead Load Moments and Shear



$\max(M_{dl.non.comp}) = 2303.5 \cdot kip \cdot ft$

$\max(V_{dl.non.comp}) = 104.1 \cdot kip$

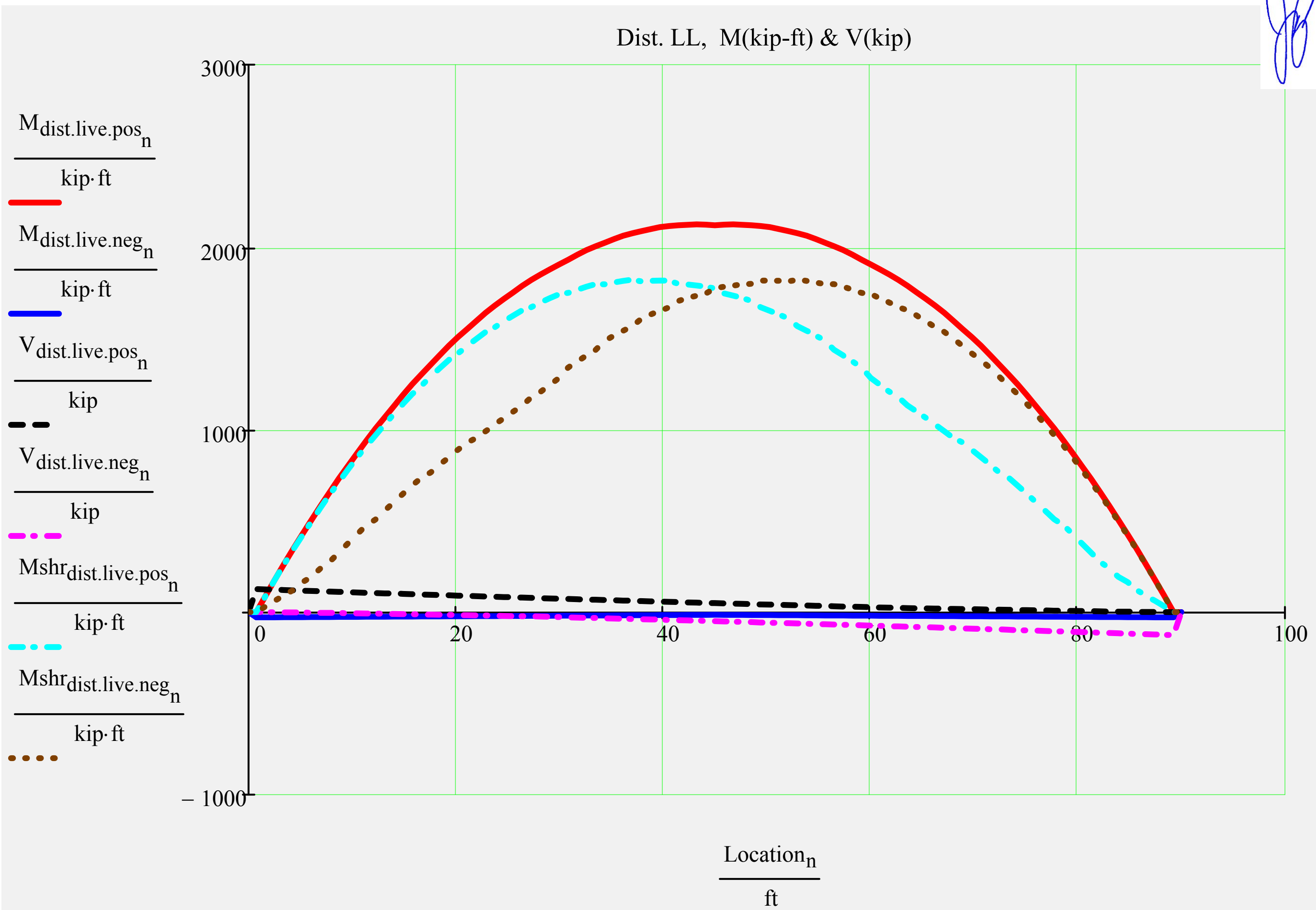
Composite Dead Load Moments and Shear



$\max(M_{dl.comp}) = 205.5 \cdot kip \cdot ft$

$\max(V_{dl.comp}) = 9.3 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 2125.1 \cdot kip \cdot ft$ $\min(M_{dist.live.neg}) = -28.7 \cdot kip \cdot ft$ $Reaction_{LL} = 127.72 \cdot kip$

$\max(V_{dist.live.pos}) = 126.2 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 1821.8 \cdot kip \cdot ft$ $Reaction_{DL} = 115.36 \cdot kip$

Prestress Strand Layout Input

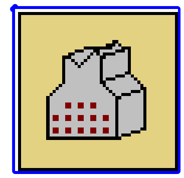
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate :=
- Standard
- Custom

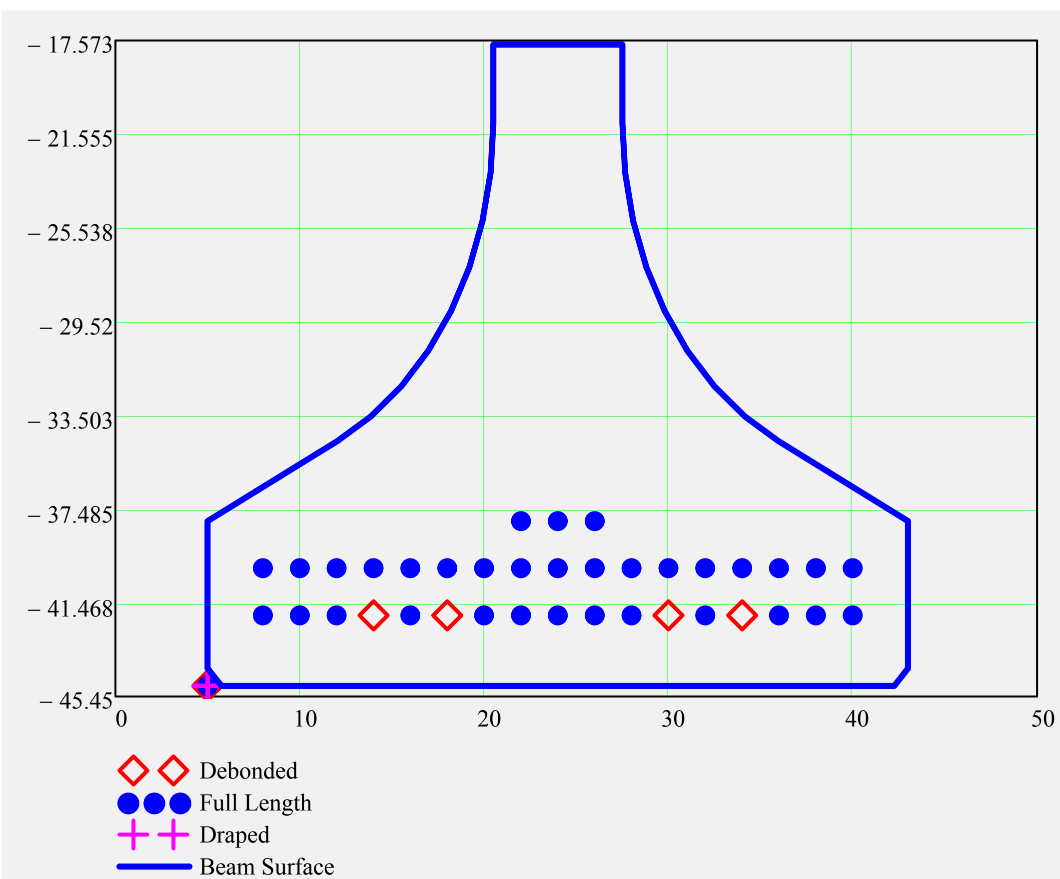
Strand Pattern Generator:



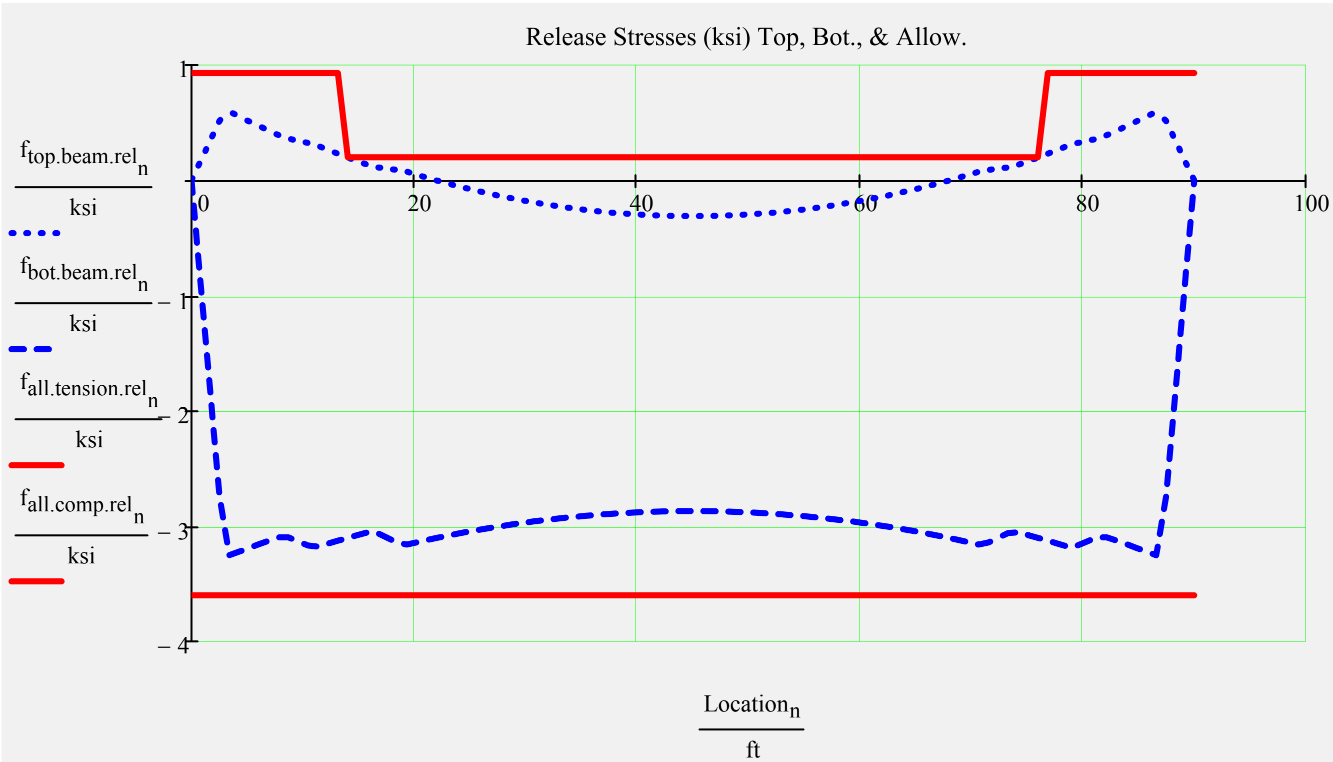
Collapsed Region for Custom Strand Sizes...

$\text{CheckPattern}_0 = \text{"OK"}$	***check 0*** *- no debonded tendon in outside row*
$\text{CheckPattern}_1 = \text{"OK"}$	***check 1*** *- less than 25% debonded tendons total*
$\text{CheckPattern}_2 = \text{"OK"}$	***check 2*** *- less than 40% debonded tendons in any row*
$\text{CheckPattern}_3 = \text{"OK"}$	***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*
$\text{CheckPattern}_4 = \text{"OK"}$	***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

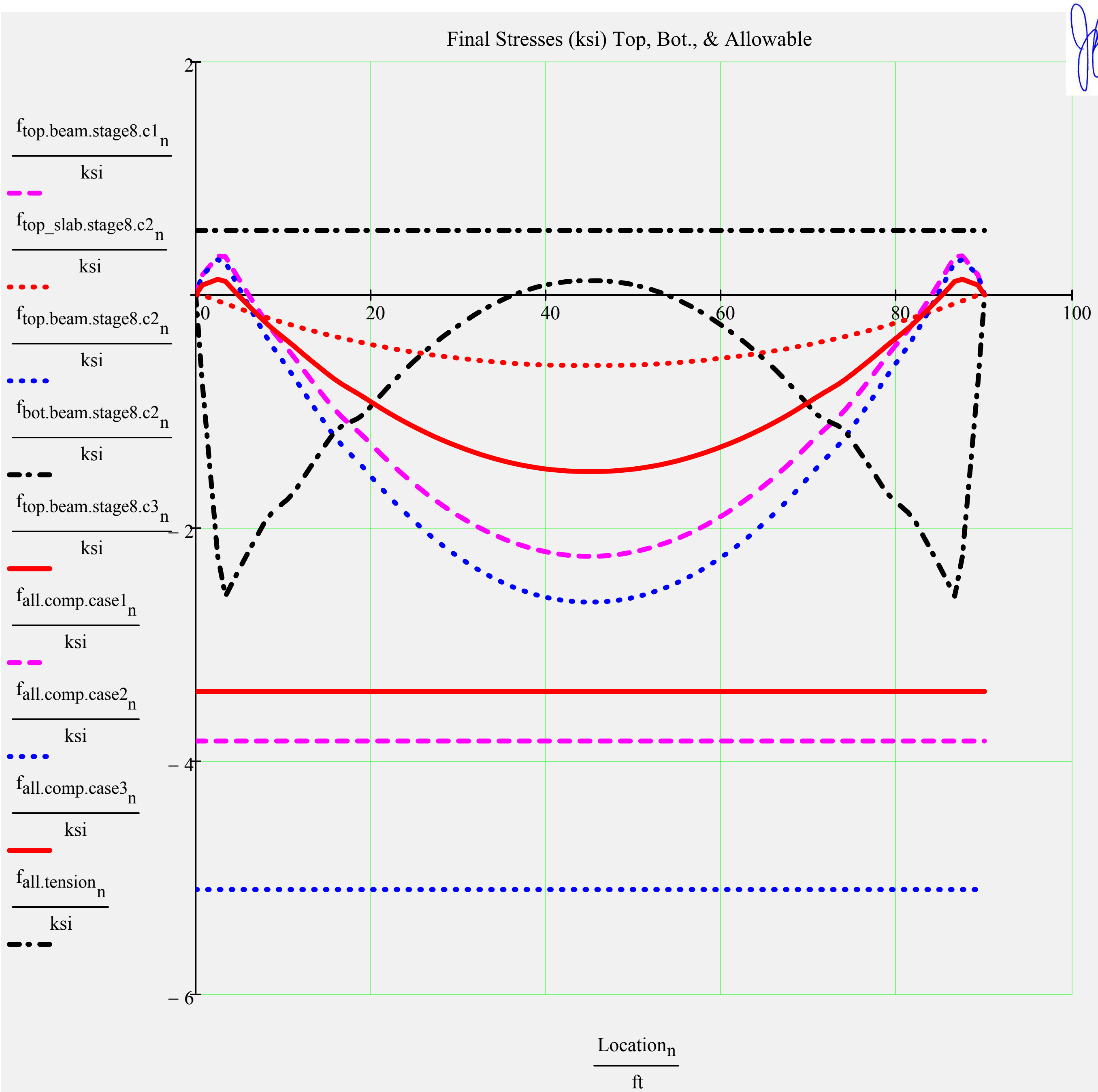
Tendon Layout



Release Stresses



Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.01$ | Check_$f_{tension.rel}$ = "OK" | (Release tension)

$\min(CR\_f_{comp.rel}) = 1.11$ | Check_$f_{comp.rel}$ = "OK" | (Release compression)

$\min(CR\_f_{tension.stage8}) = 4.49$ | Check_$f_{tension.stage8}$ = "OK" | (Service III , PS + DL +LL*0.8)

$\min(CR\_f_{comp.stage8.c1}) = 1.71$ | Check_$f_{comp.stage8.c1}$ = "OK" | (Service I , PS + DL)

$\min(CR\_f_{comp.stage8.c2}) = 1.94$ | Check_$f_{comp.stage8.c2}$ = "OK" | (Service I , PS + DL +LL)

$\min(CR\_f_{comp.stage8.c3}) = 2.25$ | Check_$f_{comp.stage8.c3}$ = "OK" | (Service I , (PS + DL)*0.5 +LL)

Section and Strand Properties Summary

$A_{beam} = 870.4 \cdot in^2$ Concrete area of beam

$I_{beam} = 226606.0804 \cdot in^4$ Gross Moment of Inertia of Beam about CG

$y_{comp} = -9.92 \cdot in$ Dist. from top of beam to CG of gross composite section

$I_{comp} = 620319.407 \cdot in^4$ Gross Moment of Inertia Composite Section about CG

$A_{deck} = 855.67 \cdot in^2$ Concrete area of deck slab

$A_{ps} = 8 \cdot in^2$ total area of strands

$d_{b.ps} = 0.6 \cdot in$ diameter of Prestressing strand

$\min(PrestressType) = 0$ 0 - low lax 1 - stress relieved

$f_{py} = 243 \cdot ksi$ tendon yield strength

$f_{pj} = 203 \cdot ksi$ prestress jacking stress

$L_{shielding}{}^T = (8 \quad 16 \quad 0 \quad 0 \quad 0) \cdot ft$

$A_{ps.row}{}^T = (0.4 \quad 0.4 \quad 2.8 \quad 3.7 \quad 0.7) \cdot in^2$

$d_{ps.row} =$ (·in)

	0	1	2	3	4	5	6	7	8	9
0	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
1	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
2	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
3	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40
4	-38	-38	-38	-38	-38	-38	-38	-38	-38	...

TotalNumberOfTendons = 37

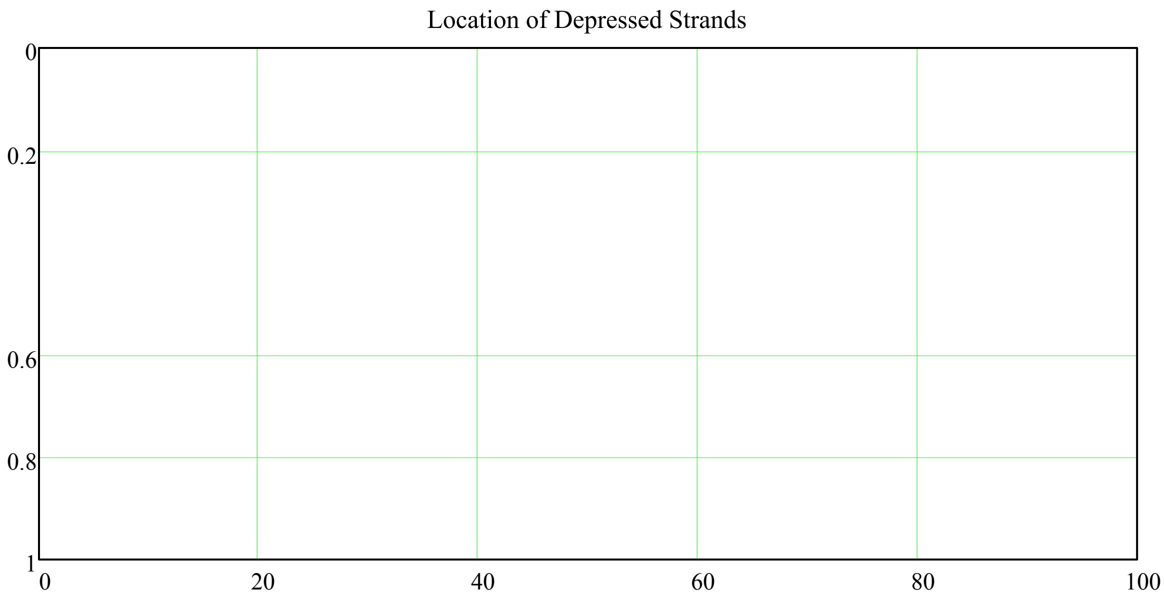
NumberOfDebondedTendons = 4

NumberOfDrapedTendons = 0

StrandSize = "0.6 in low lax"

$\text{StrandArea} = 0.22 \cdot \text{in}^2$

$\text{JackingForce}_{\text{per.strand}} = 43.94 \cdot \text{kip}$



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

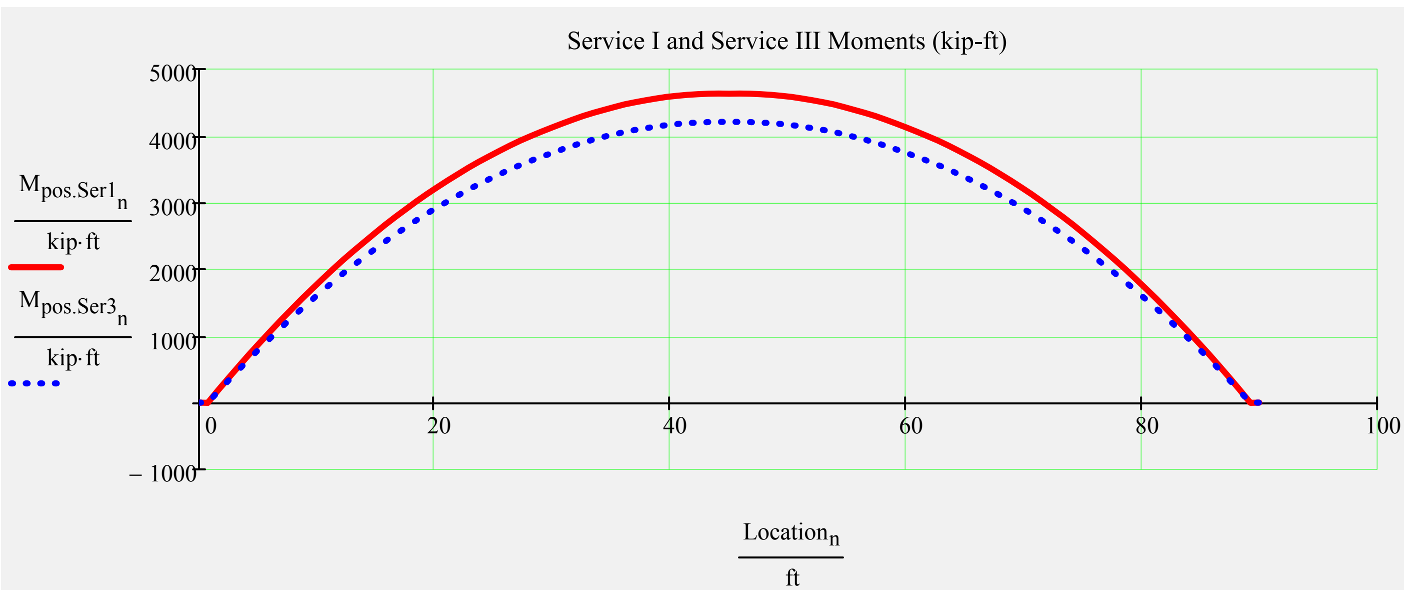
$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

$f_{pe} = 179 \cdot ksi$ $\Delta f_{pTot} = -23 \cdot ksi$

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.47 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.53 \cdot \%$

Check_f_{pt} = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f_{pe} = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 4631.6 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 4206.9 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.31	−2.87
2	−0.41	−2.44
4	−0.37	−2.47
6	−2.2	−1.07
8	−2.63	0.12

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−2183.5
"Final (about composite centroid)"	−1439.5	−1831.4

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	862.37	224527.31	−24.64
"Transformed Beam (initial)"	919.4	238511.11	−25.64
"Transformed Beam "	910.29	236392.94	−25.49
"Composite "	1802.12	666662.96	−10.29

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	918
"Non-composite (includes bm wt.)"	2303.5
"Composite"	205.5
"Distributed Live Load"	2120.2

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

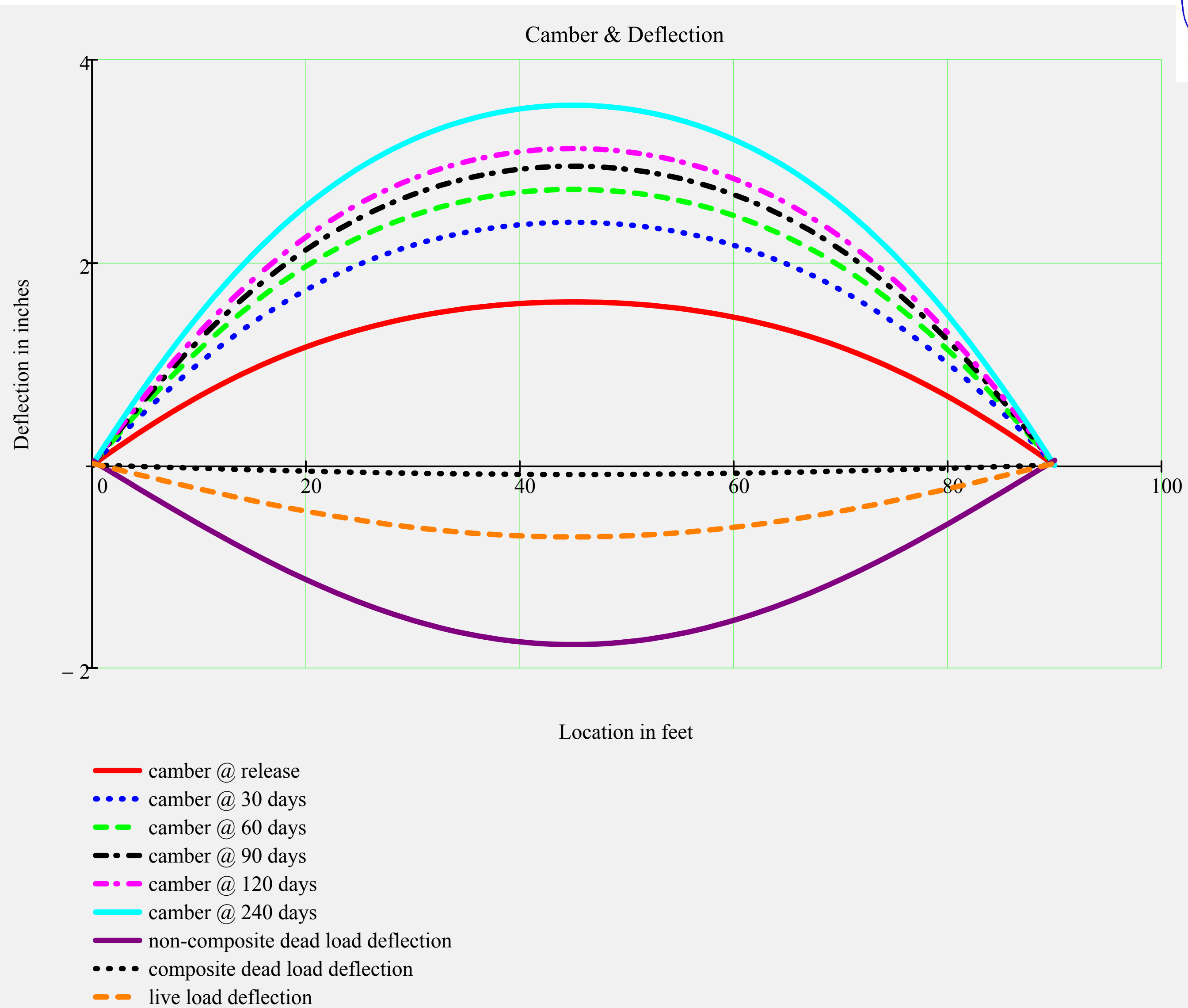
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

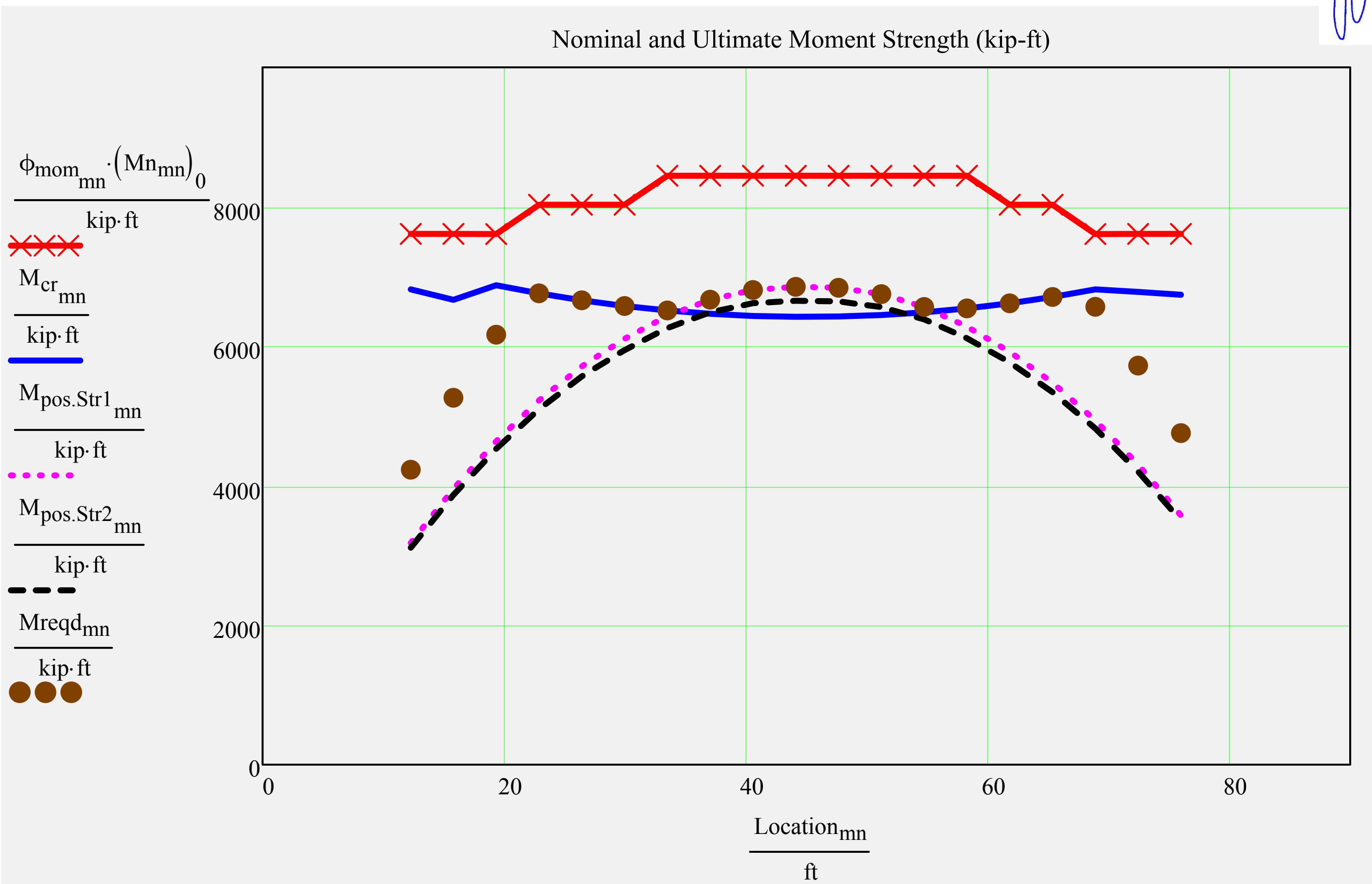
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	0.0105	−0.8054	0.3825	1.6109
"30 Days"	−0.1453	−1.4271	0.6214	2.3965
"60 Days"	−0.201	−1.6496	0.7148	2.7208
"90 Days"	−0.2307	−1.7681	0.7646	2.9497
"120 Days"	−0.2492	−1.8418	0.7955	3.124
"240 Days"	−0.2833	−1.9778	0.8526	3.5512
"non-comp DL"	−0.2717	0.2081	−0.3056	−1.7688
"comp DL"	−0.0056	0.0191	−0.0157	−0.091
"LL"	−0.0442	0.1492	−0.1232	−0.7072

Strength Limit State Moments



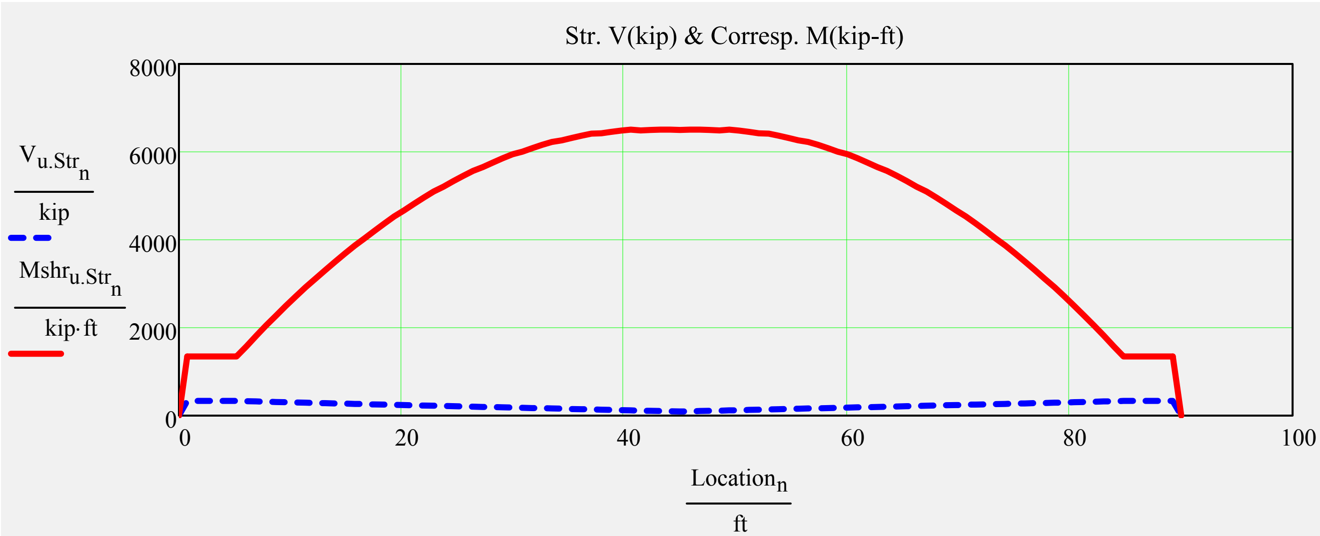
$CR_{Str.mom_n} := 10$ $\qquad CR_{Str.mom_{mn}} := \frac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ $\qquad$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.16$

$\max(Mreqd) = 6851.3 \cdot kip \cdot ft$

$CheckMomentCapacity := if(\min(CR_{Str.mom}) > 0.99, "OK", "No Good!")$ $\qquad CheckMomentCapacity = "OK"$

Strength Shear and Associated Moments



$max(V_{u.Str}) = 333.2 \cdot kip$ $max(Mshr_{u.Str}) = 6506.4 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing	Number of Spaces	area per stirrup
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$$tmp\_s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in \qquad tmp\_NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix} \qquad tmp\_A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

Location	$user\_s_{nspacings} :=$	$user\_NumberSpaces_{nspacings} :=$	$user\_A_{stirrup_{nspacings}} :=$	$interface\_factor_{nspacings} :=$
A1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.25
A2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.5
A3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S4 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

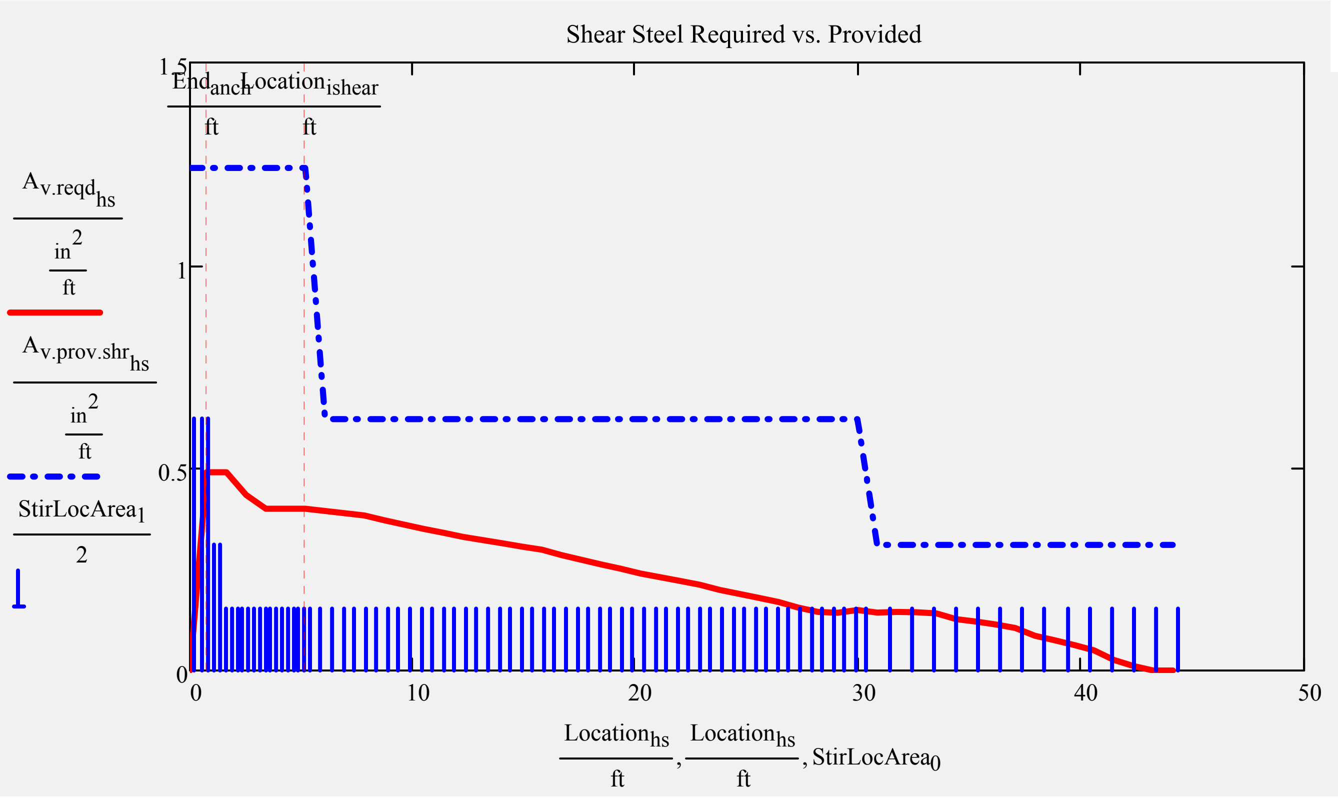
Stirrup sizes and spacings used in analysis

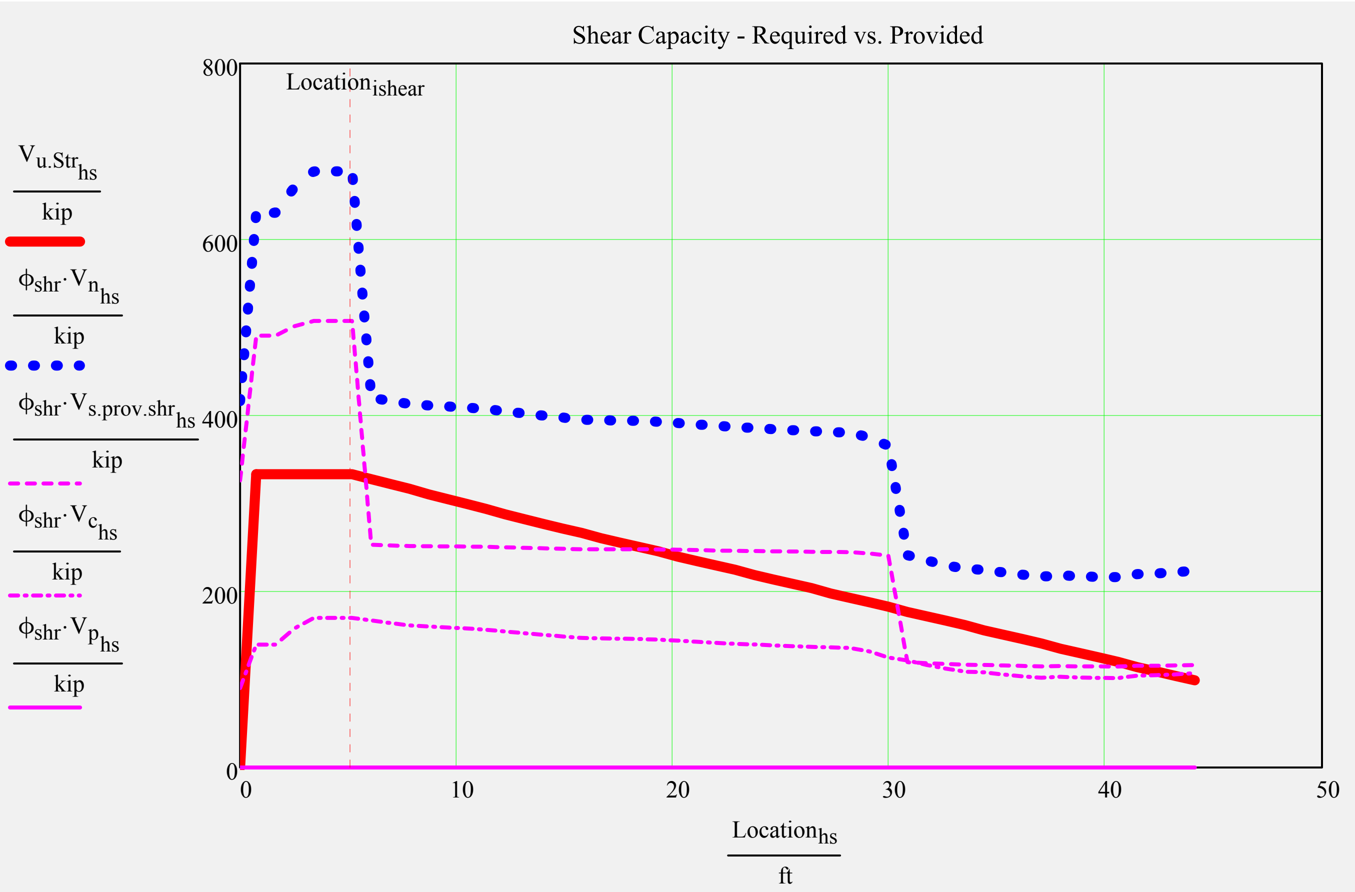
The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s	NumberSpaces	$A_{stirrup}$
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$$s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in \qquad NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix} \qquad A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

$EndCover = 2.5 \cdot in$


Shear Steel Required vs. Provided
End_anch / ft
Location_ishear / ft
A_v.reqd_hs / (in^2/ft)
A_v.prov.shr_hs / (in^2/ft)
StirLocArea_1 / 2
1.5
1
0.5
0
0
10
20
30
40
50
Location_hs / ft, Location_hs / ft, StirLocArea_0

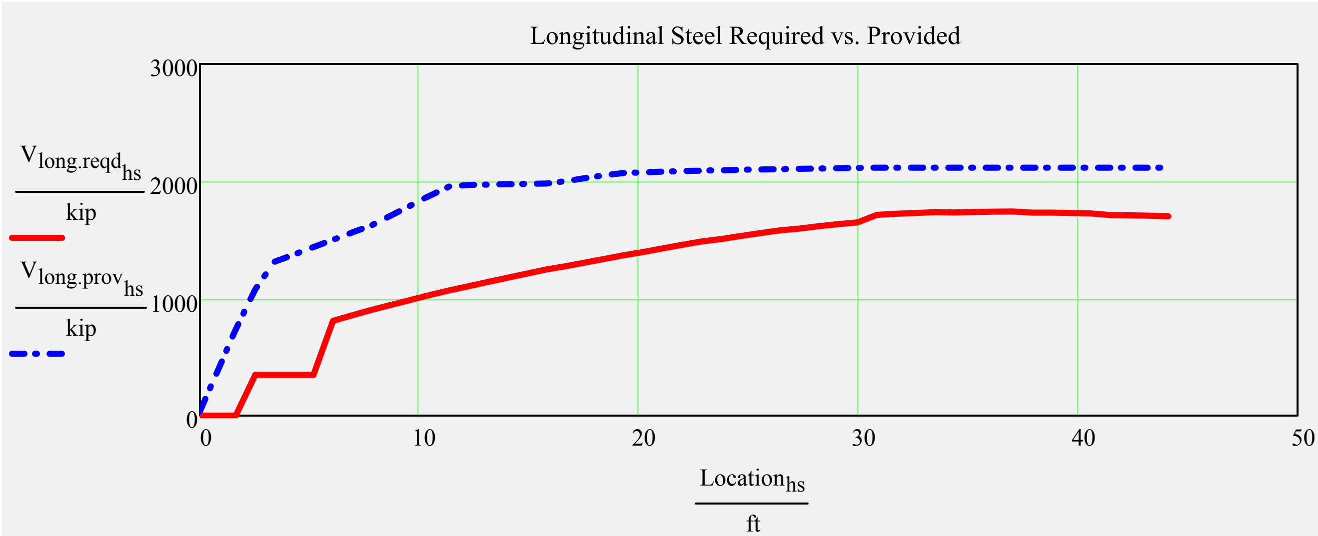

Shear Capacity - Required vs. Provided
Location_ishear
V_u.Str_hs / kip
φ_shr · V_n_hs / kip
φ_shr · V_s.prov.shr_hs / kip
φ_shr · V_c_hs / kip
φ_shr · V_p_hs / kip
800
600
400
200
0
0
10
20
30
40
50
Location_hs / ft

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

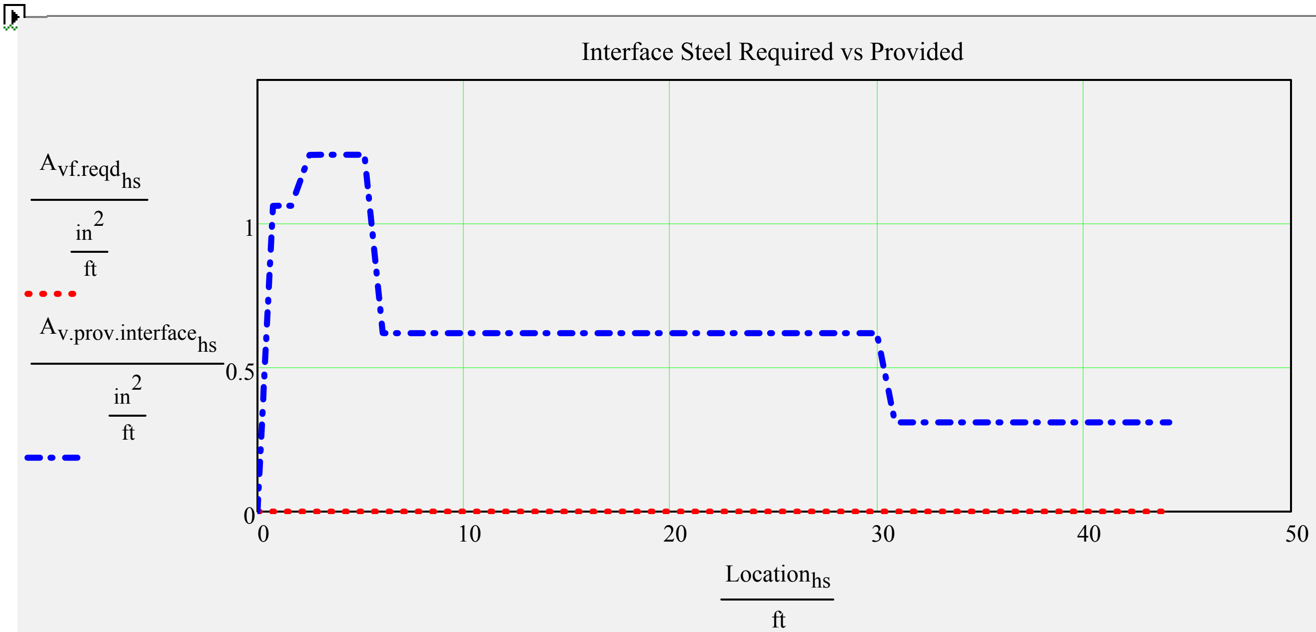


$$CR_{LongSteel_{hs}} := \text{if}\left(V_{long.reqd_{hs}} < .01kip, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$$

$\min(CR_{LongSteel}) = 1.21$

CheckLongSteel := if(min(CR_LongSteel) > 1 ,"OK" ,"No Good, add steel!")

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$

$\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, \text{"OK"}, \text{"No Good"}\right)$$

CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A."

CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0 := AcceptAASHTO$

$check_1 := AcceptSDG$

$check_2 := AcceptOntario$

$check_3 := Check\_f_{pt}$

$check_4 := Check\_f_{pe}$

$check_5 := Check\_f_{tension.rel}$

$check_6 := Check\_f_{comp.rel}$

$check_7 := Check\_f_{tension.stage8}$

$check_8 := Check\_f_{comp.stage8.c1}$

$check_9 := Check\_f_{comp.stage8.c2}$

$check_{10} := Check\_f_{comp.stage8.c3}$

$check_{11} := CheckMomentCapacity$

$check_{12} := CheckMaxCapacity$

$check_{13} := CheckStirArea$

$check_{14} := CheckShearCapacity$

$check_{15} := CheckMinStirArea$

$check_{16} := CheckMaxStirSpacing$

$check_{17} := CheckLongSteel$

$check_{18} := CheckInterfaceSpacing$

$check_{19} := CheckSplittingSteel$

$check_{20} := CheckMaxPrestressingForce$

$check_{21} := CheckPattern_0$

$check_{22} := CheckPattern_1$

$check_{23} := CheckPattern_2$

$check_{24} := CheckPattern_3$

$check_{25} := CheckPattern_4$

$check_{26} := CheckInterfaceSteel$

$check_{27} := CheckStrandFit$

$check_{28} := Check\_SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T =$

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8: FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

Moment (Strength) or Stress (Service) — Shear (Strength)

$LRFR_{loadrating} =$

"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.89	1.43	"N/A"	43.37	1.11	1.46	"N/A"	5.31	HL-93
"Strength I(Op)"	0.89	1.85	"N/A"	43.37	1.11	1.89	"N/A"	5.31	HL-93
"Service III(Inv)"	0.89	1.40	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.89	1.54	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.89	1.51	90.55	43.37	1.11	1.49	89.41	30.98	*Permit
"Service III"	0.89	1.43	85.78	45.13	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.25$ $CR_{LongSteel.Permit} = 1.21$ $CheckLongSteel_{loadrating} =$ "OK"

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltA_ 90'_FIB-45_NI

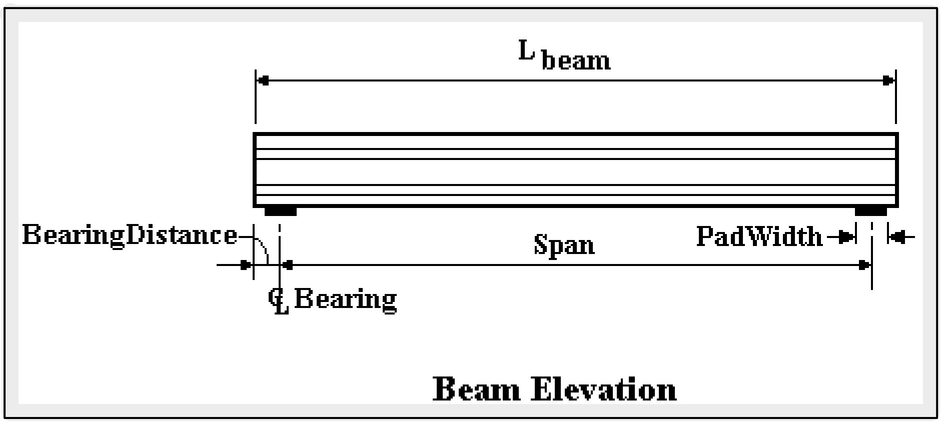
Comment = "Alt. A - NB Exterior"

Legend

TanHighlight = DataEntry   YellowHighlight = CheckValues   GreyHighlight = UserComments + Graphs

BlackText = ProgramEquations   *Maroon Text = Code Reference*   *Blue Text = Commentary*

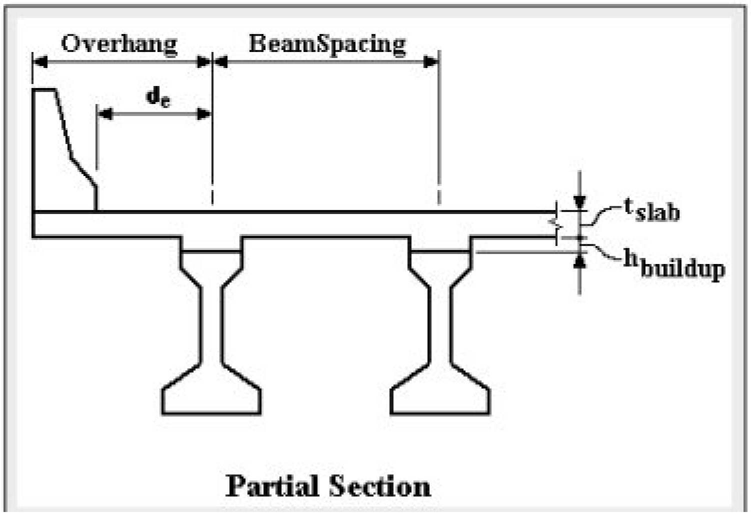
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ $Span = 88.5 \cdot ft$ $BearingDistance = 9 \cdot in$ $PadWidth = 10 \cdot in$

BeamTypeTog = "FIB45"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



Partial Section

$Overhang = 4.29 \cdot ft$ | $BeamSpacing = 11.5 \cdot ft$ | $t_{slab} = 8 \cdot in$ | $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ | $t_{integral.ws} = 0.5 \cdot in$ | $NumberOfBeams = 4$ | $t_{slab.delta} = 0 \cdot in$

$de = 2.75\ ft$

BeamPosition = "exterior" — *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" — *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 10.04\ ft$ — *effective slab width* — *LRFD 4.6.2.6*

$t_{slab} := if(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ — *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

$SectTog := if(SectionType = \text{"transformed"}, 1, 0)$ $SectTog = 1$

Concrete:

Corrosion Classification — Environment = "moderately"

density of slab concrete — $\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$

strength of slab concrete — $f_{c.slab} = 4.5 \cdot ksi$

density of beam concrete — $\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$

strength of beam concrete — $f_{c.beam} = 8.5 \cdot ksi$

weight of future wearing surface — $Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$

release beam strength — $f_{ci.beam} = 6 \cdot ksi$

type of course aggregate, "Florida" *or* "Standard" — AggregateType = "Florida"

relative humidity — $H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ — *used in distribution calculation* — $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9 \cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor \cdot \sqrt{f_{ci.beam} \cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012 \cdot ksi$

$E_c := AggFactor \cdot \sqrt{f_{c.beam} \cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776 \cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270 \cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

tendon modulus of elasticity $E_p = 28500 \cdot ksi$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60 \cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4 \cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0 \cdot in$

mild steel modulus of elasticity $E_s = 29000 \cdot ksi$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62 \cdot \frac{in^2}{ft}$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0 \cdot in^2$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \quad 53.33 \quad 53.33) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0) \cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.97$ $tmp\_g_{shear} = 1.02$

user value overrides (optional):

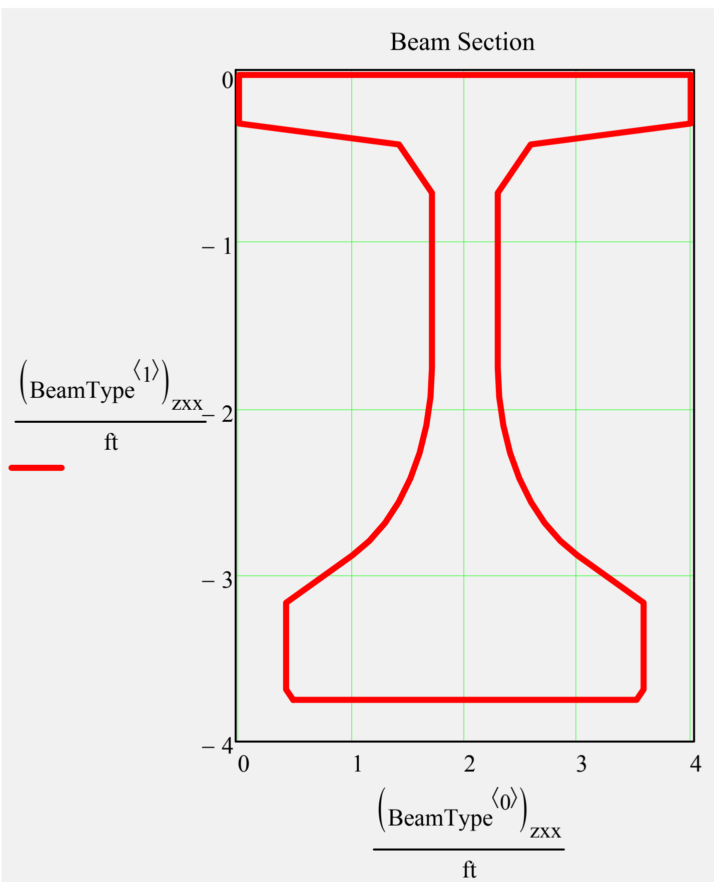
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

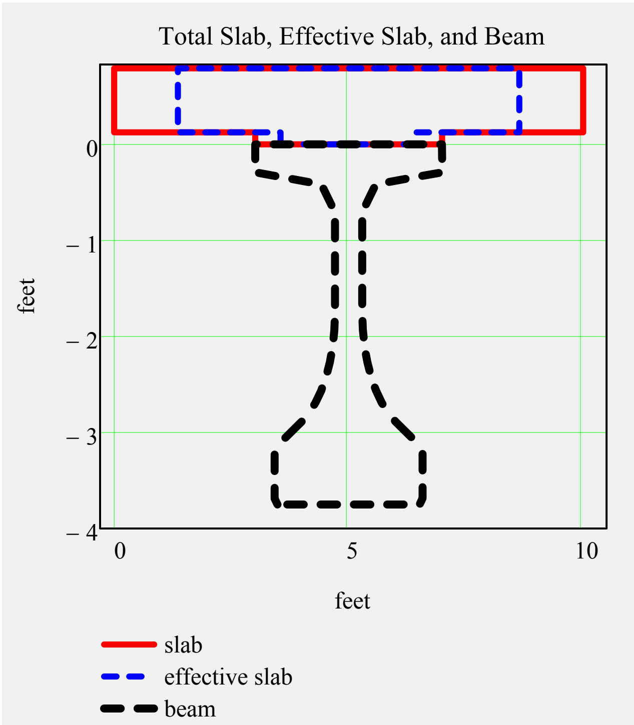
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.97$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 1.02$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 1.142 \cdot \frac{kip}{ft}$ $w_{beam} = 0.907 \cdot \frac{kip}{ft}$ $w_{forms} = 0.075 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$

additional non composite dead load (positive or negative)
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 2.123 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 1.217 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

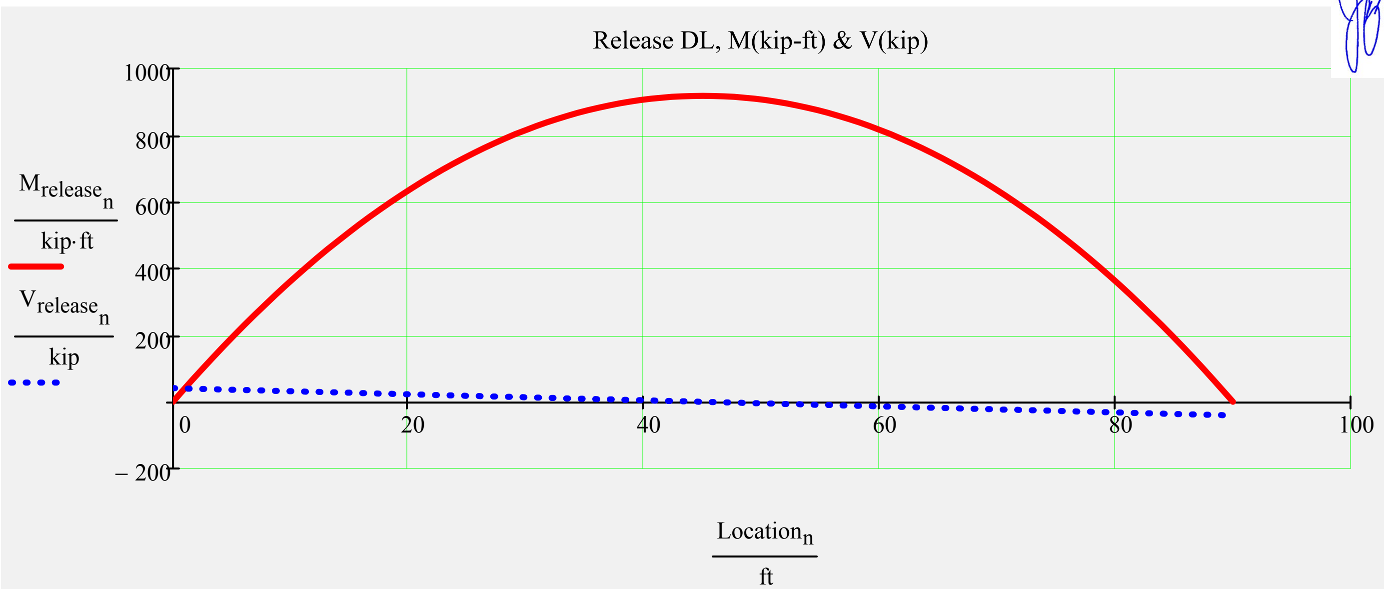
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.21 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.21 \cdot \frac{kip}{ft}$

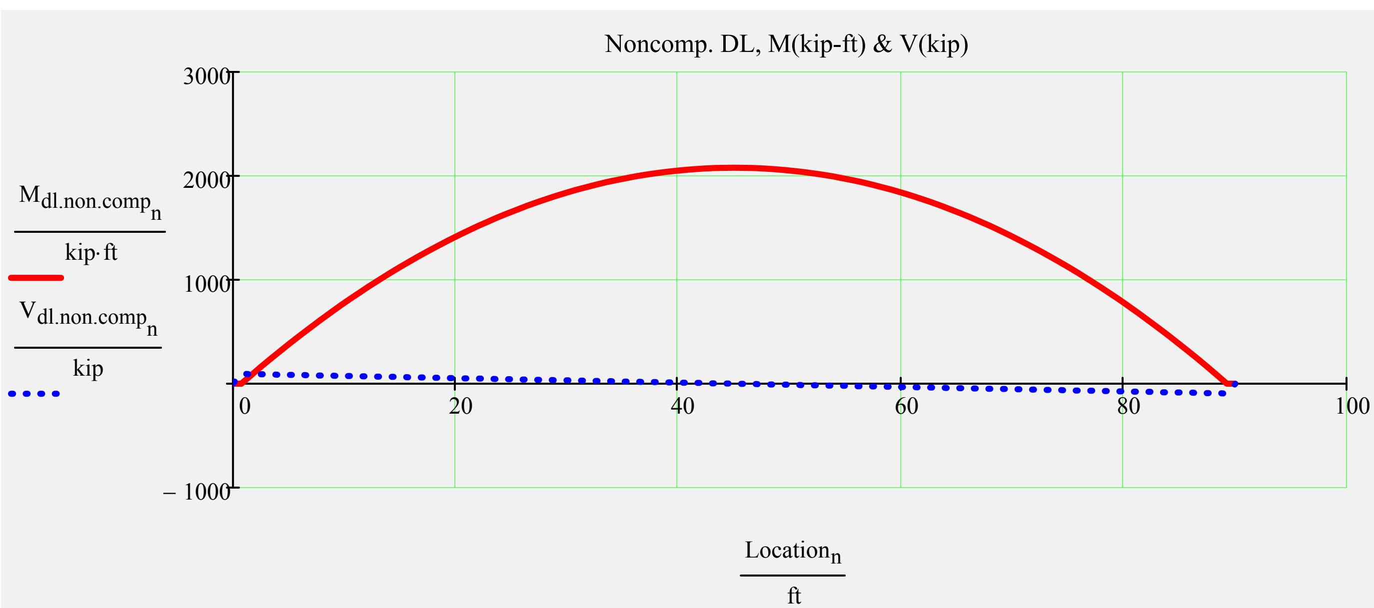
$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.21 \cdot \frac{kip}{ft}$

Release Dead Load Moments and Shear



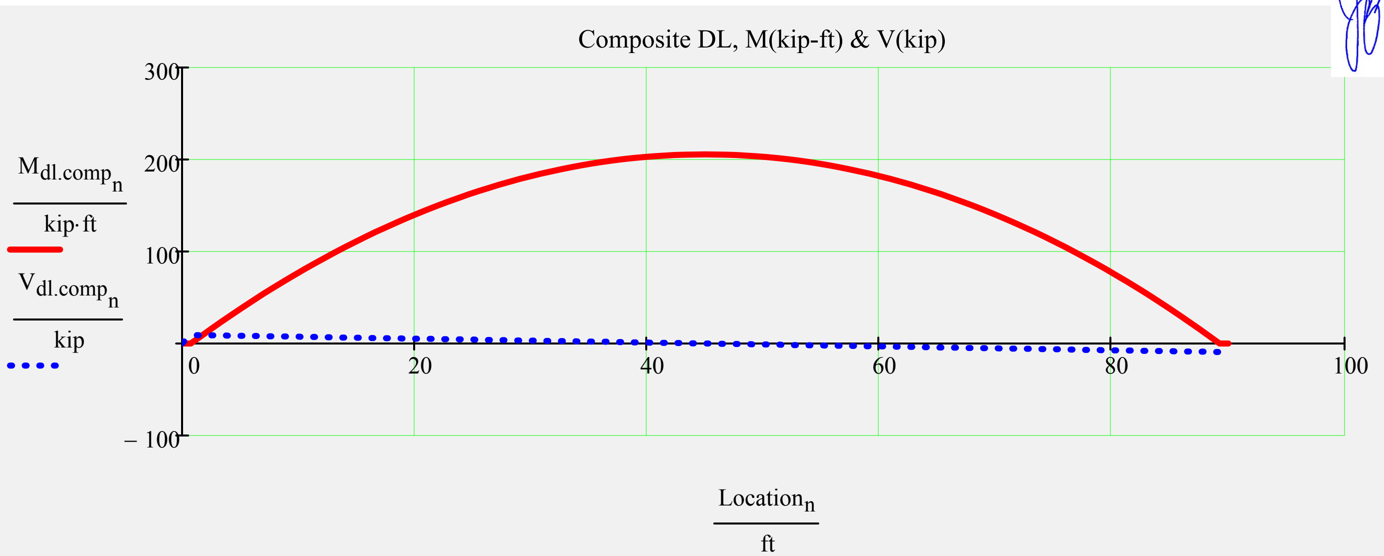
$max(M_{release}) = 918 \cdot kip \cdot ft$ $\qquad$ $max(V_{release}) = 40.8 \cdot kip$

Noncomposite Dead Load Moments and Shear



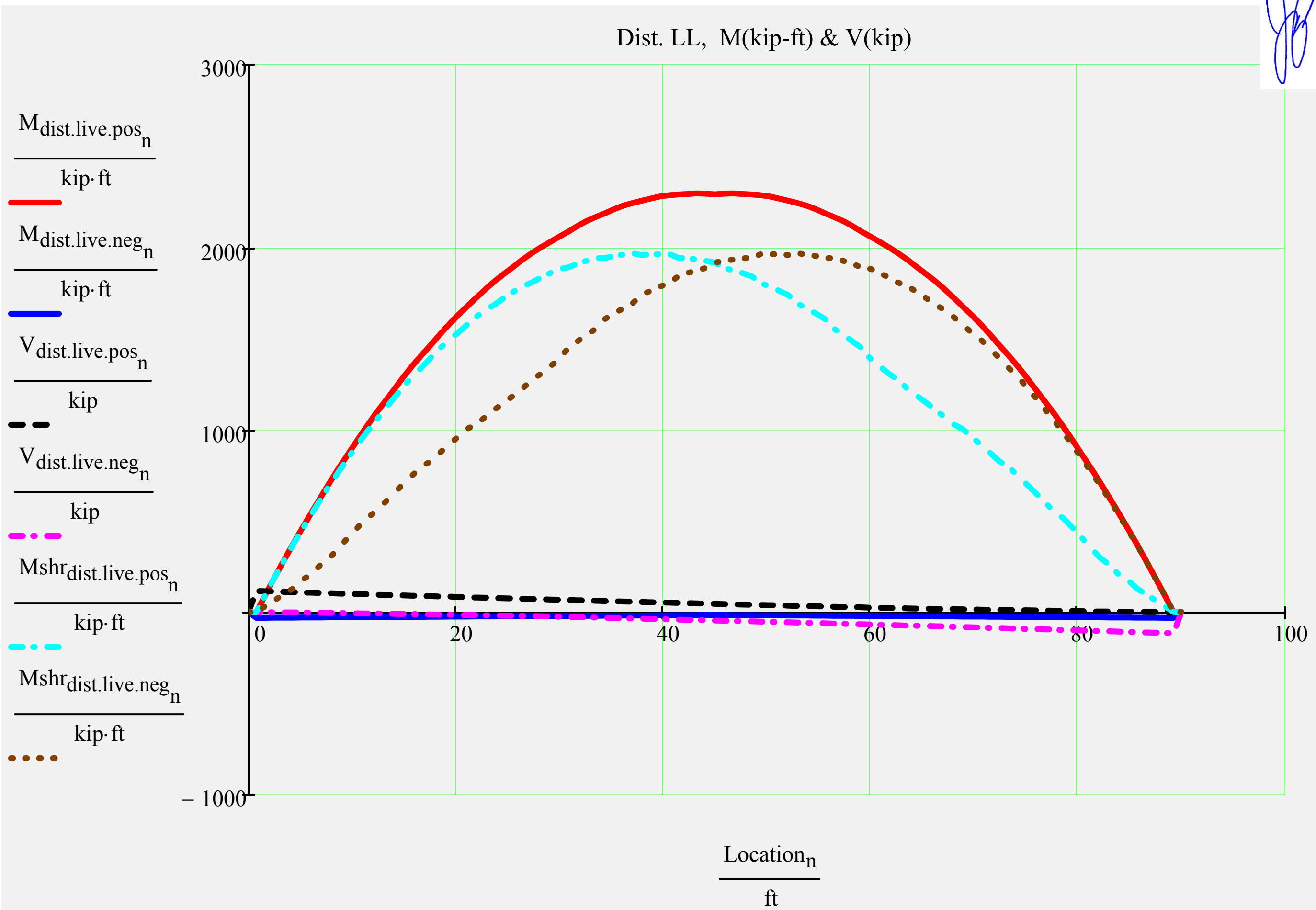
$max(M_{dl.non.comp}) = 2078.3 \cdot kip \cdot ft$ $\qquad$ $max(V_{dl.non.comp}) = 94 \cdot kip$

Composite Dead Load Moments and Shear



$max(M_{dl.comp}) = 205.5 \cdot kip \cdot ft$ $\qquad$ $max(V_{dl.comp}) = 9.3 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 2294.6 \cdot kip \cdot ft$ $\min(M_{dist.live.neg}) = -31 \cdot kip \cdot ft$ $Reaction_{LL} = 117.35 \cdot kip$

$\max(V_{dist.live.pos}) = 116 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 1967.2 \cdot kip \cdot ft$ $Reaction_{DL} = 105 \cdot kip$

Prestress Strand Layout Input

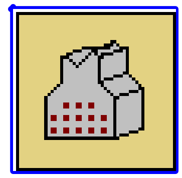
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate := Standard / Custom

Strand Pattern Generator:



Collapsed Region for Custom Strand Sizes...

$CheckPattern_0$ = "OK" — ***check 0*** *- no debonded tendon in outside row*

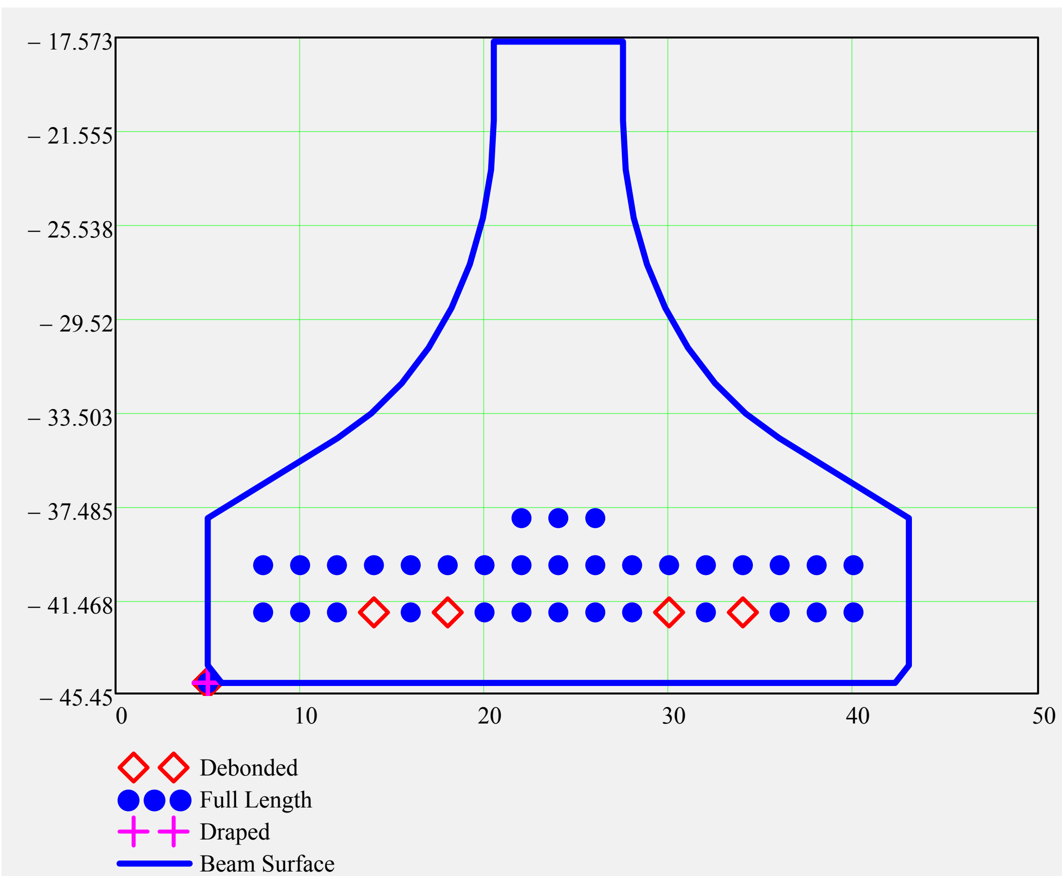
$CheckPattern_1$ = "OK" — ***check 1*** *- less than 25% debonded tendons total*

$CheckPattern_2$ = "OK" — ***check 2*** *- less than 40% debonded tendons in any row*

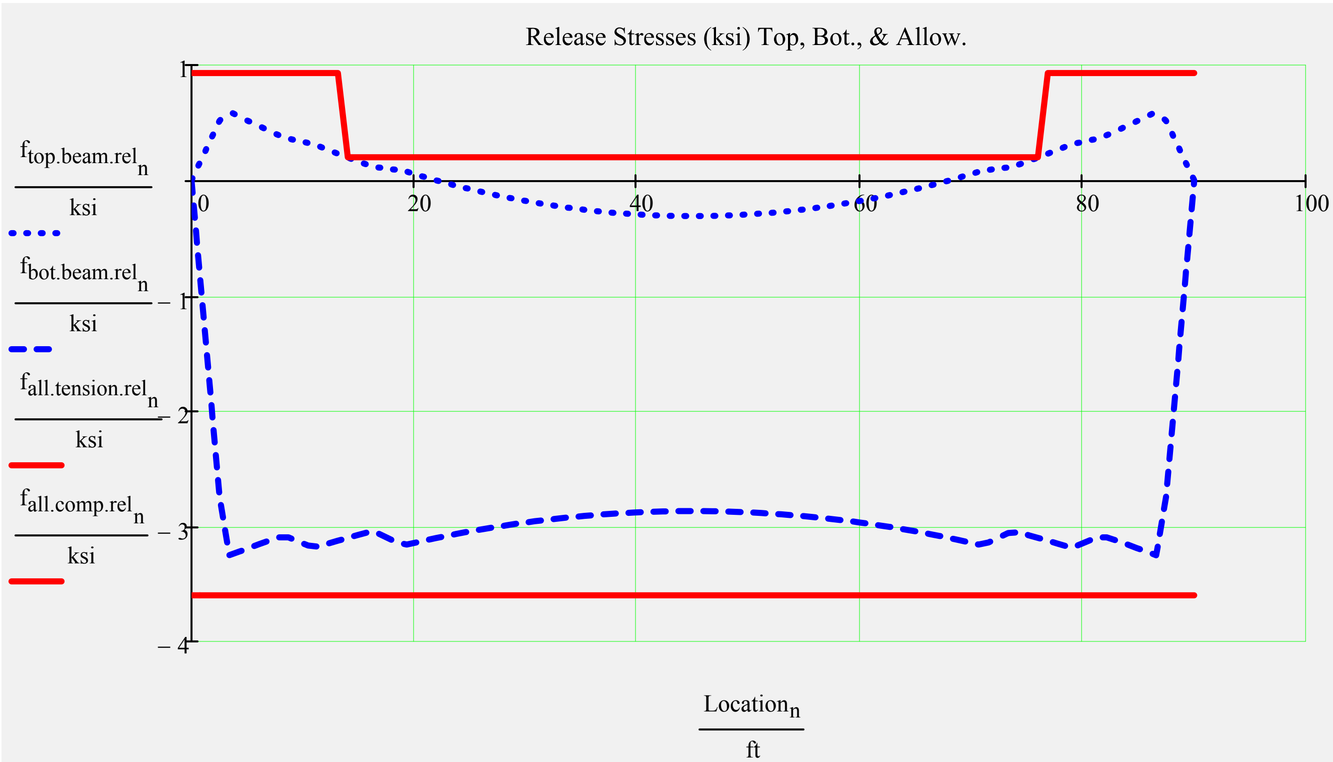
$CheckPattern_3$ = "OK" — ***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*

$CheckPattern_4$ = "OK" — ***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

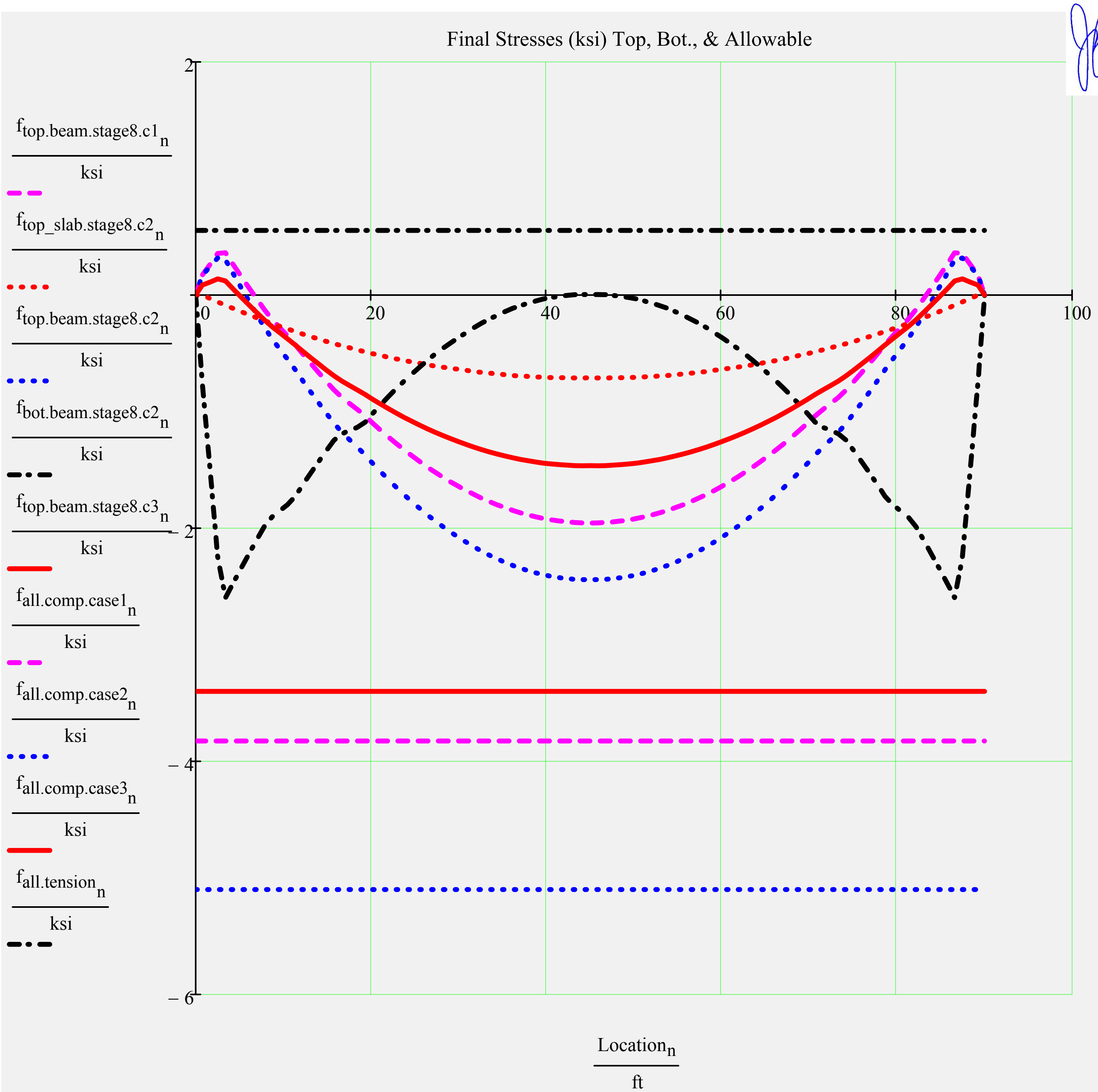
Tendon Layout



Release Stresses



Final Stresses



Stress Checks

$min(CR\_f_{tension.rel}) = 1.01$ | Check_$f_{tension.rel}$ = "OK" | *(Release tension)*

$min(CR\_f_{comp.rel}) = 1.11$ | Check_$f_{comp.rel}$ = "OK" | *(Release compression)*

$min(CR\_f_{tension.stage8}) = 10$ | Check_$f_{tension.stage8}$ = "OK" | *(Service III , PS + DL +LL*0.8)*

$min(CR\_f_{comp.stage8.c1}) = 1.96$ | Check_$f_{comp.stage8.c1}$ = "OK" | *(Service I , PS + DL)*

$min(CR\_f_{comp.stage8.c2}) = 2.09$ | Check_$f_{comp.stage8.c2}$ = "OK" | *(Service I , PS + DL +LL)*

$min(CR\_f_{comp.stage8.c3}) = 2.32$ | Check_$f_{comp.stage8.c3}$ = "OK" | *(Service I , (PS + DL)*0.5 +LL)*

Section and Strand Properties Summary

$A_{beam} = 870.4 \cdot in^2$ *Concrete area of beam*

$I_{beam} = 226606.0804 \cdot in^4$ *Gross Moment of Inertia of Beam about CG*

$y_{comp} = -10.89 \cdot in$ *Dist. from top of beam to CG of gross composite section*

$I_{comp} = 594009.6075 \cdot in^4$ *Gross Moment of Inertia Composite Section about CG*

$A_{deck} = 753.68 \cdot in^2$ *Concrete area of deck slab*

$A_{ps} = 8 \cdot in^2$ *total area of strands*

$d_{b.ps} = 0.6 \cdot in$ *diameter of Prestressing strand*

min(PrestressType) = 0 *0 - low lax 1 - stress relieved*

$f_{py} = 243 \cdot ksi$ *tendon yield strength*

$f_{pj} = 203 \cdot ksi$ *prestress jacking stress*

$L_{shielding}^T = (8\ \ 16\ \ 0\ \ 0\ \ 0) \cdot ft$

$A_{ps.row}^T = (0.4\ \ 0.4\ \ 2.8\ \ 3.7\ \ 0.7) \cdot in^2$

$d_{ps.row} =$ (·in)

	0	1	2	3	4	5	6	7	8	9
0	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
1	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
2	-42	-42	-42	-42	-42	-42	-42	-42	-42	-42
3	-40	-40	-40	-40	-40	-40	-40	-40	-40	-40
4	-38	-38	-38	-38	-38	-38	-38	-38	-38	...

TotalNumberOfTendons = 37

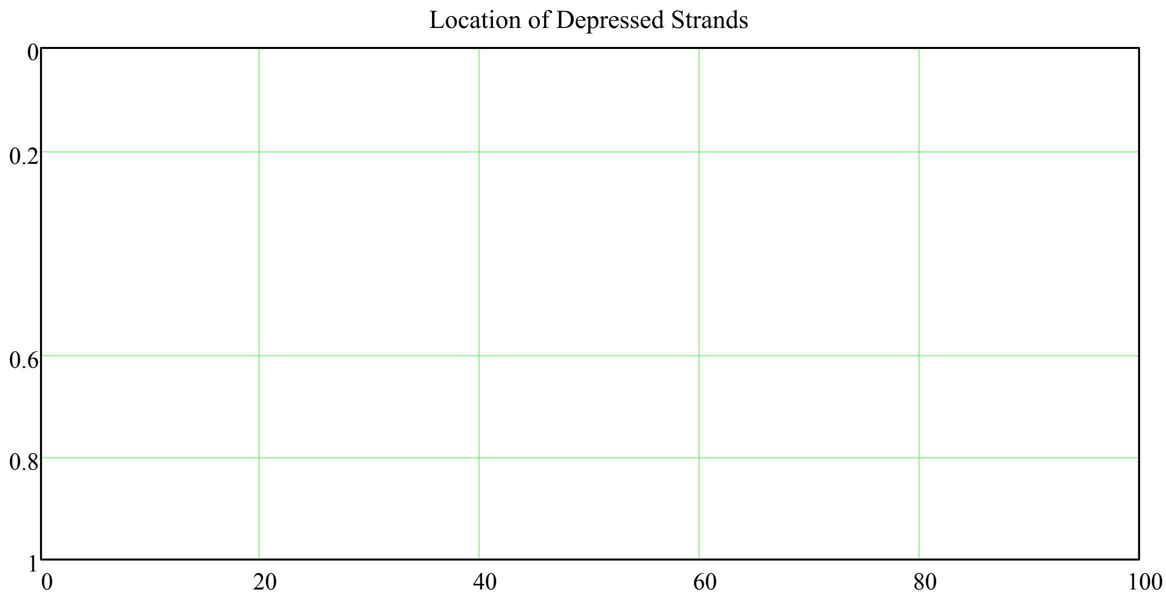
NumberOfDebondedTendons = 4

NumberOfDrapedTendons = 0

StrandSize = "0.6 in low lax"

$StrandArea = 0.22 \cdot in^2$

$JackingForce_{per.strand} = 43.94 \cdot kip$



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

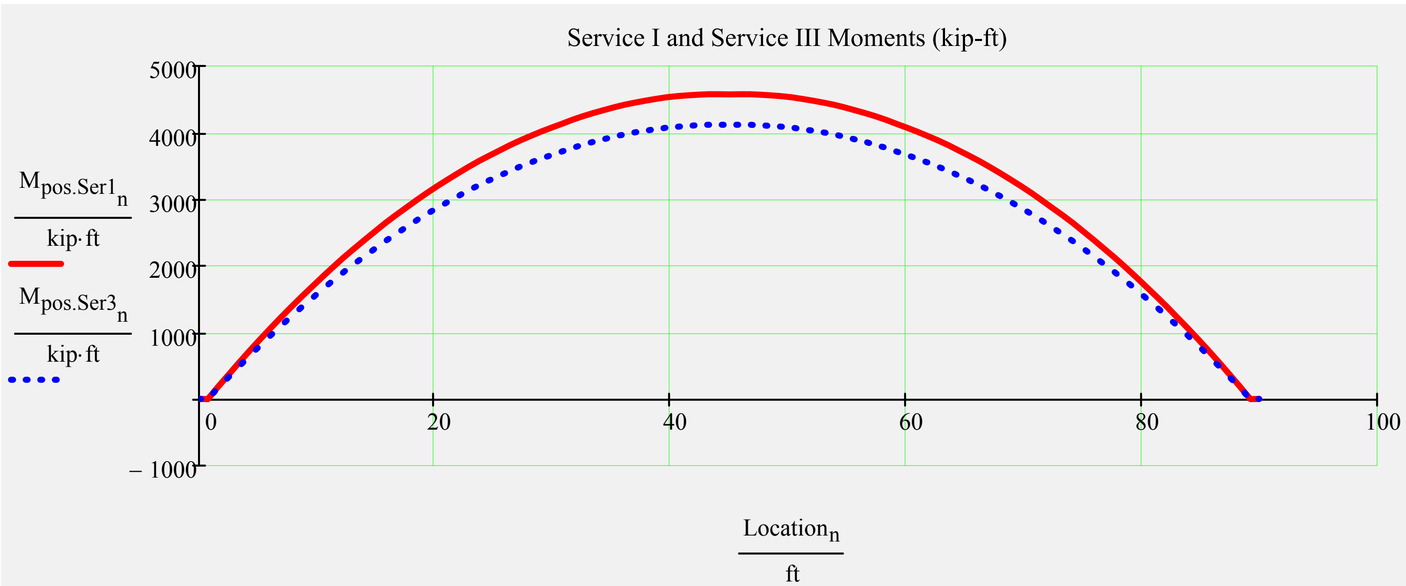
$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

$f_{pe} = 179 \cdot ksi$ $\Delta f_{pTot} = -23 \cdot ksi$

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.47 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.53 \cdot \%$

Check_f_{pt} = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f_{pe} = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 4575.9 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 4117.3 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.31	−2.87
2	−0.41	−2.44
4	−0.37	−2.47
6	−1.91	−1.29
8	−2.44	0

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−2183.5
"Final (about composite centroid)"	−1439.5	−1831.4

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	862.37	224527.31	−24.64
"Transformed Beam (initial)"	919.4	238511.11	−25.64
"Transformed Beam "	910.29	236392.94	−25.49
"Composite "	1695.55	637872.43	−11.28

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	918
"Non-composite (includes bm wt.)"	2078.3
"Composite"	205.5
"Distributed Live Load"	2289.3

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

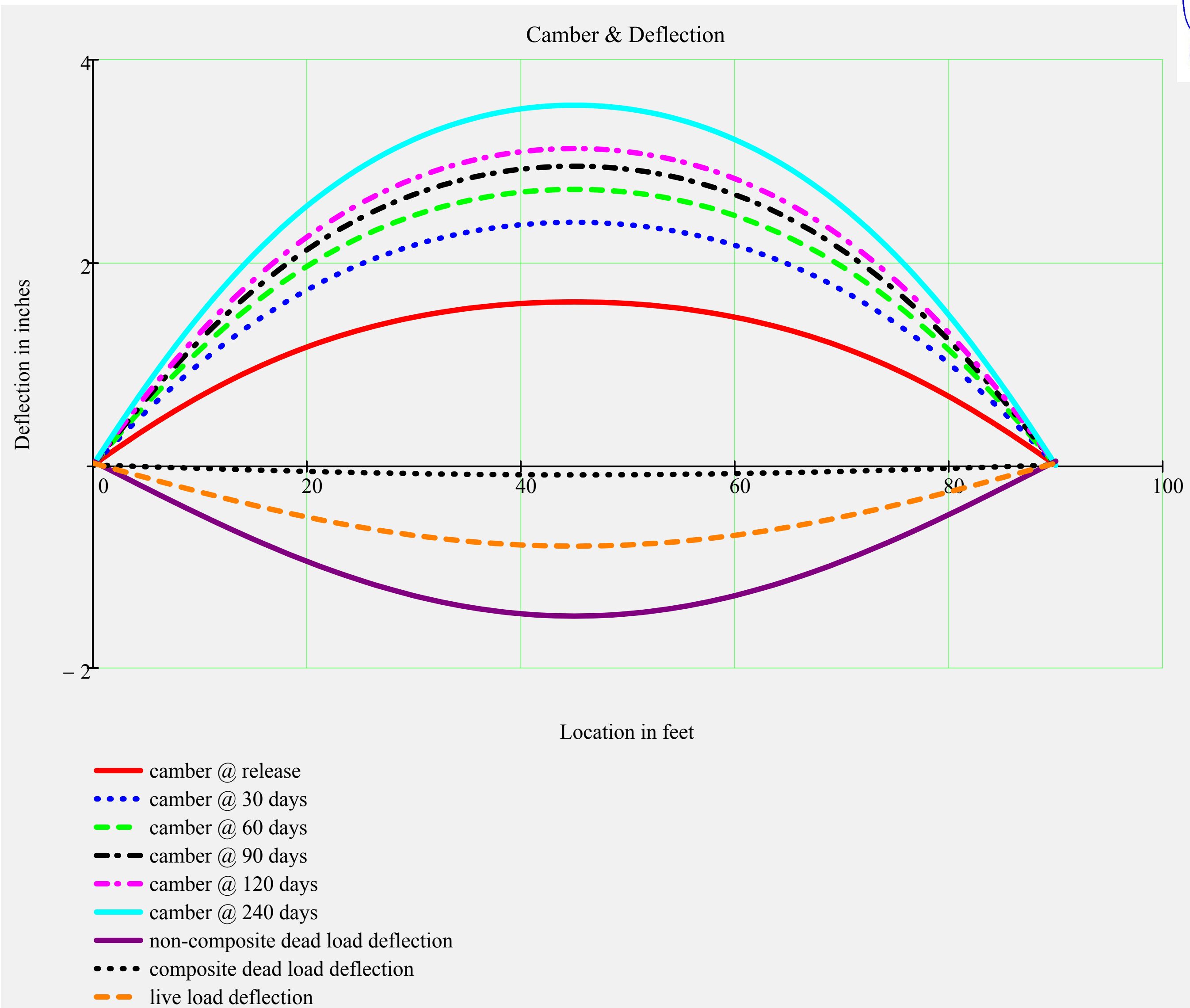
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

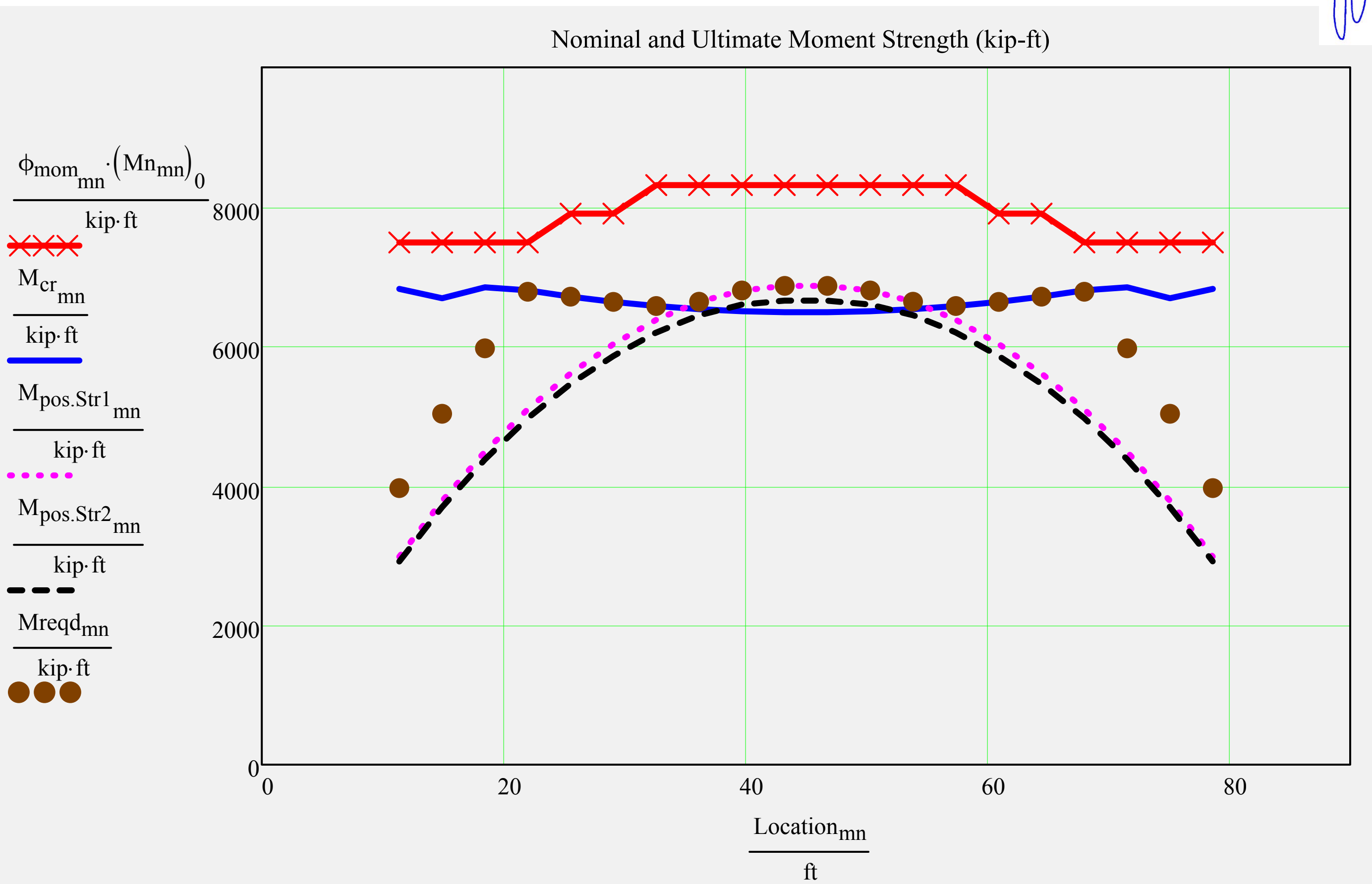
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	0.0105	−0.8054	0.3825	1.6109
"30 Days"	−0.1453	−1.4271	0.6214	2.3965
"60 Days"	−0.201	−1.6496	0.7148	2.7208
"90 Days"	−0.2307	−1.7681	0.7646	2.9497
"120 Days"	−0.2492	−1.8418	0.7955	3.124
"240 Days"	−0.2833	−1.9778	0.8526	3.5512
"non-comp DL"	−0.2284	0.175	−0.257	−1.4875
"comp DL"	−0.0065	0.0193	−0.0164	−0.0951
"LL"	−0.0547	0.1635	−0.139	−0.798

Strength Limit State Moments



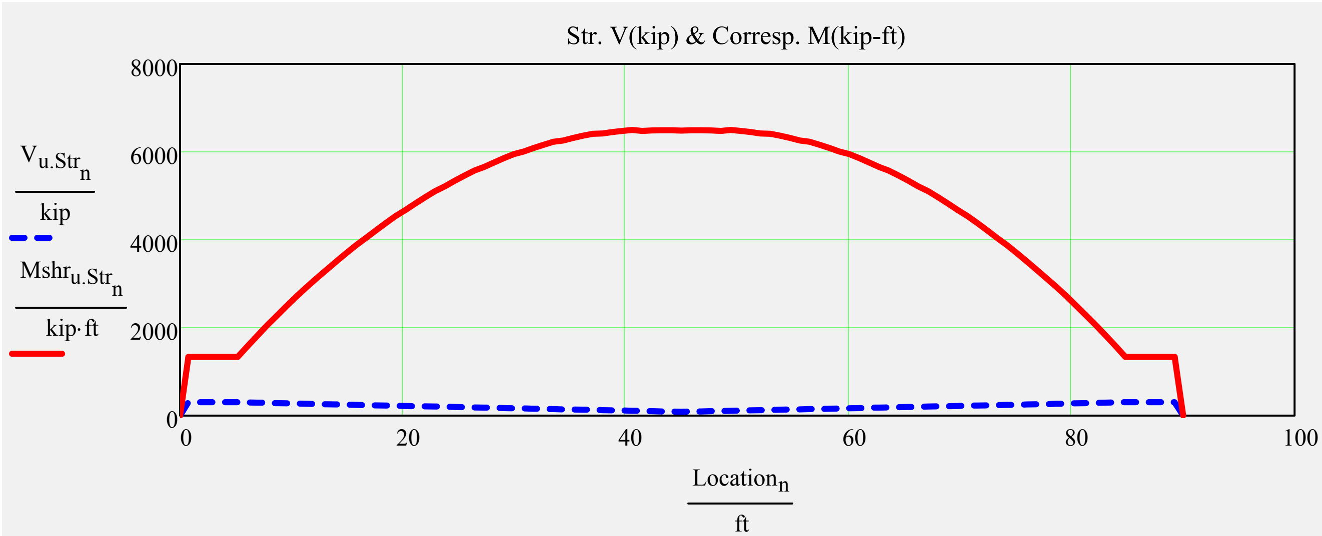
$CR_{Str.mom_n} := 10$ $CR_{Str.mom_{mn}} := \frac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.1$

$\max(Mreqd) = 6865.8 \cdot kip \cdot ft$

$CheckMomentCapacity := if(\min(CR_{Str.mom}) > 0.99, "OK", "No Good!")$ $CheckMomentCapacity = "OK"$

Strength Shear and Associated Moments



$max(V_{u.Str}) = 305.1 \cdot kip$ $max(Mshr_{u.Str}) = 6499.0 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing	Number of Spaces	area per stirrup
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$$tmp\_s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in \qquad tmp\_NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix} \qquad tmp\_A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

Location	$user\_s_{nspacings} :=$	$user\_NumberSpaces_{nspacings} :=$	$user\_A_{stirrup_{nspacings}} :=$	$interface\_factor_{nspacings} :=$
A1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.25
A2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.5
A3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S4 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

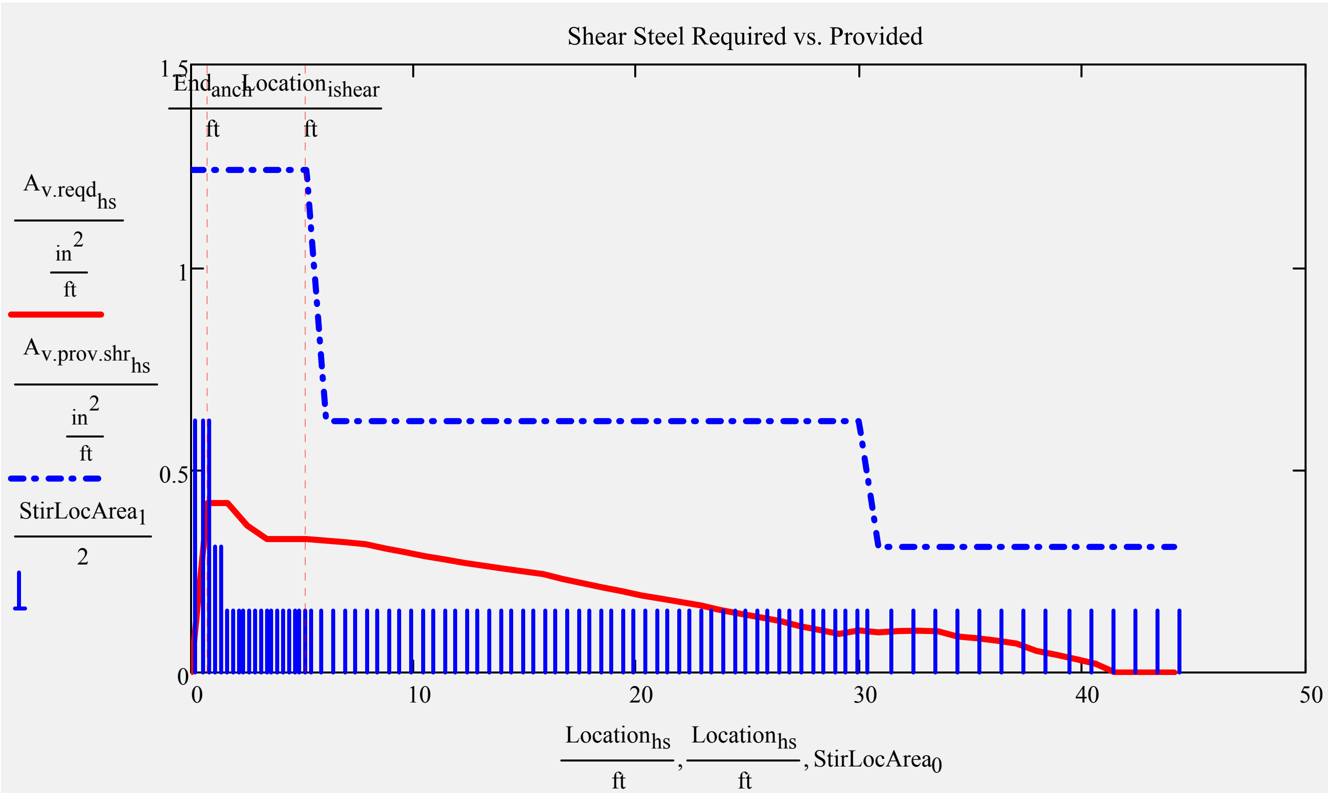
Stirrup sizes and spacings used in analysis

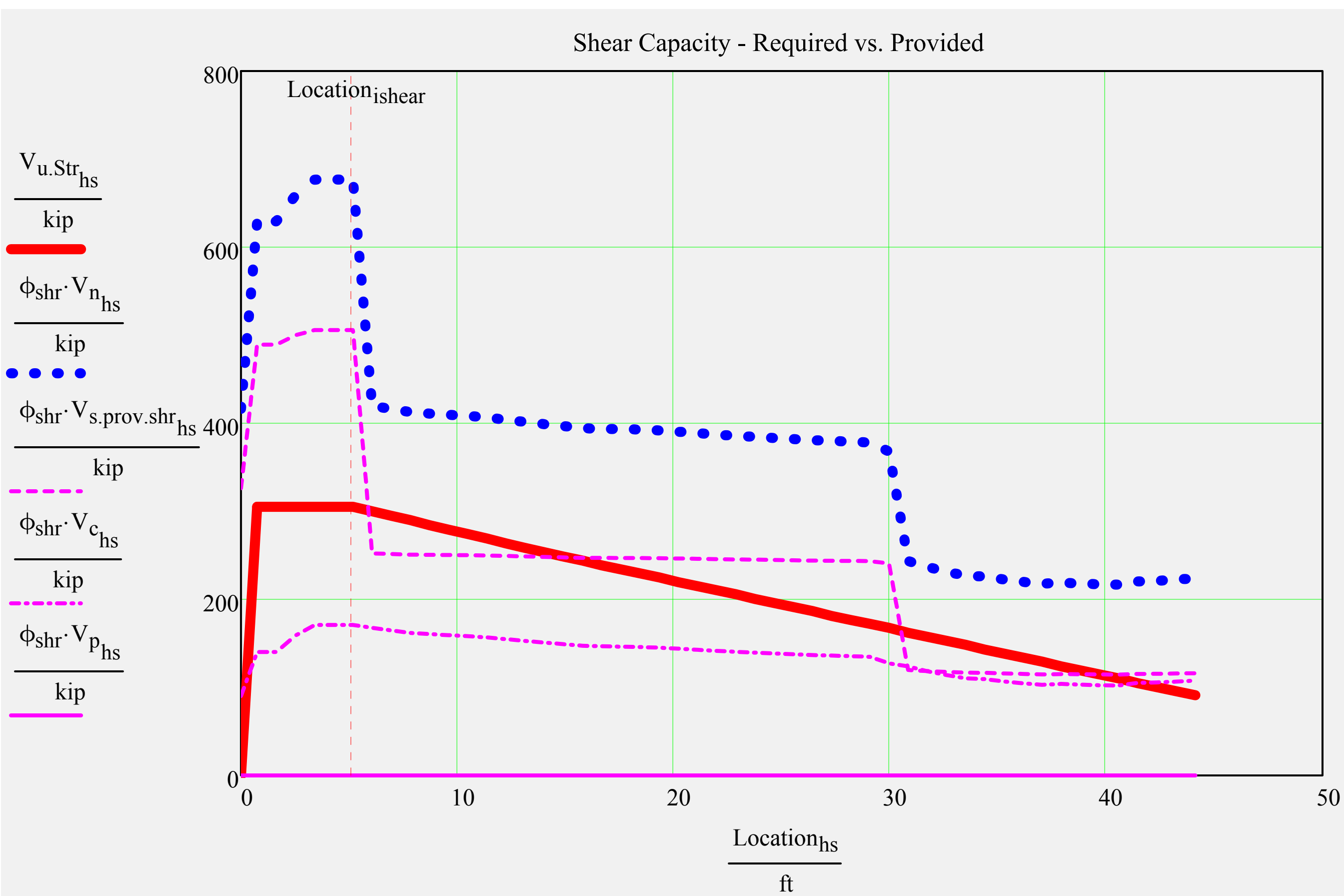
The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	spacing	Number of Spaces	area per stirrup
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$$s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in \qquad NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix} \qquad A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$$

$EndCover = 2.5 \cdot in$


Shear Steel Required vs. Provided
Endanch / ft
Locationishear / ft
Av.reqdhs / (in2/ft)
Av.prov.shrhs / (in2/ft)
StirLocArea1 / 2
1.5
1
0.5
0
0
10
20
30
40
50
Locationhs / ft , Locationhs / ft , StirLocArea0

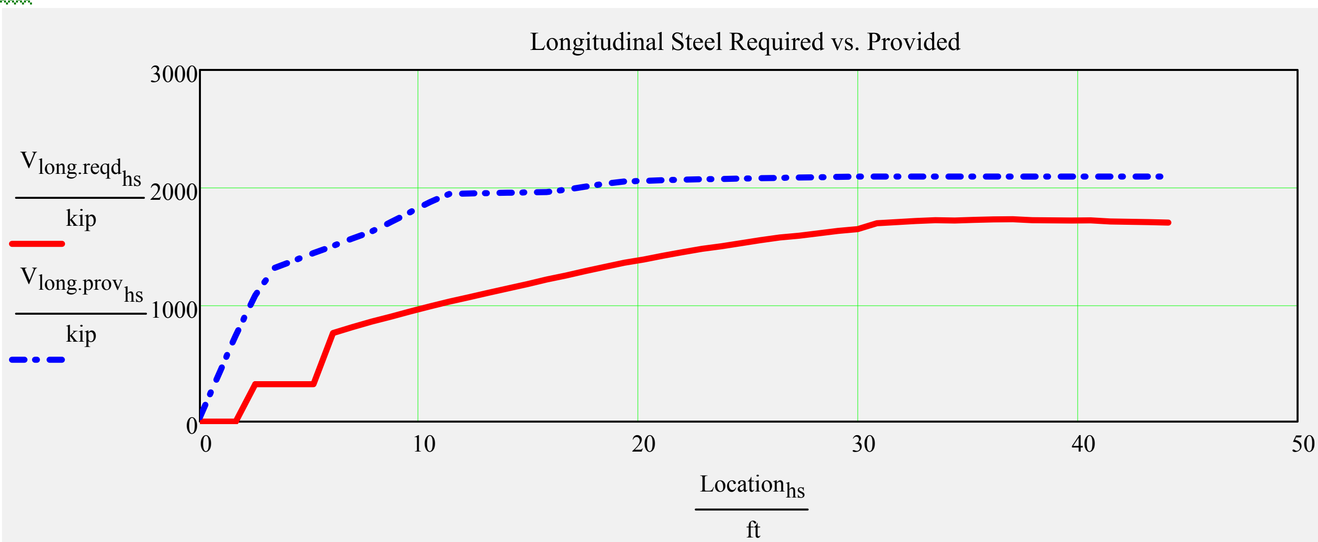

Shear Capacity - Required vs. Provided
Locationishear
Vu.Strhs / kip
ϕshr·Vnhs / kip
ϕshr·Vs.prov.shrhs / kip
ϕshr·Vchs / kip
ϕshr·Vphs / kip
800
600
400
200
0
0
10
20
30
40
50
Locationhs / ft

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

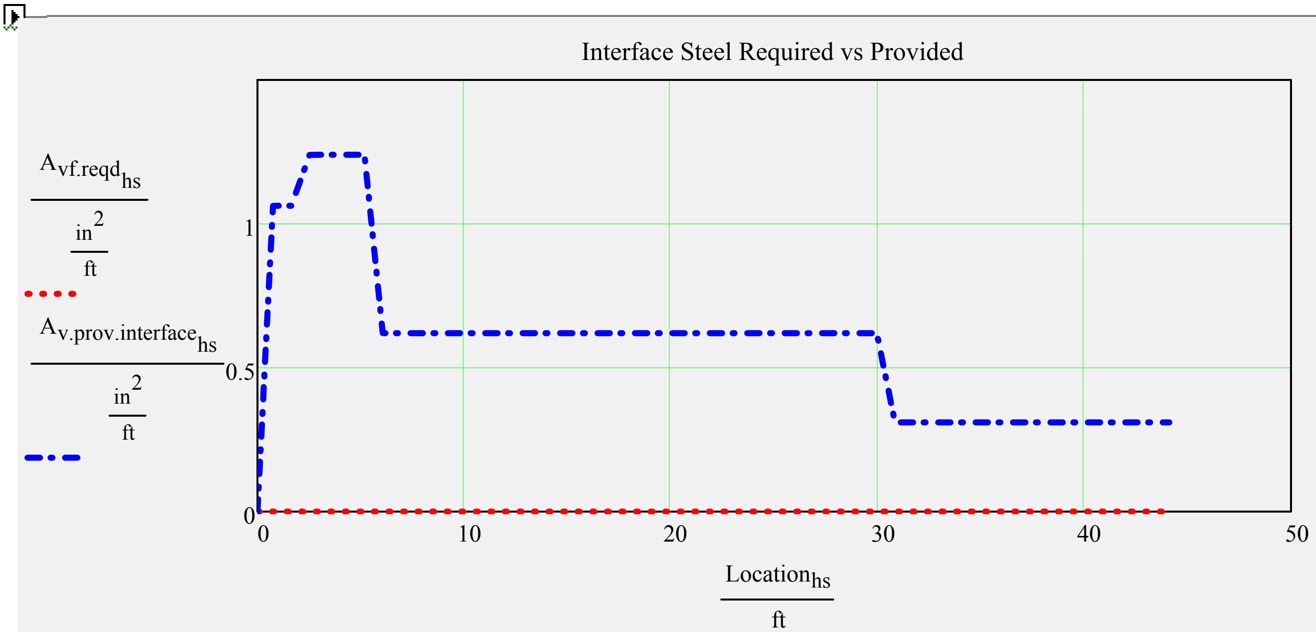


$CR_{LongSteel_{hs}} := \text{if}\left(V_{long.reqd_{hs}} < .01\text{kip}, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$

$\min(CR_{LongSteel}) = 1.21$

CheckLongSteel := if(min($CR_{LongSteel}$) > 1 ,"OK" ,"No Good, add steel!")

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, "OK", "No Good"\right)$

CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A." CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0 := AcceptAASHTO$

$check_1 := AcceptSDG$

$check_2 := AcceptOntario$

$check_3 := Check\_f_{pt}$

$check_4 := Check\_f_{pe}$

$check_5 := Check\_f_{tension.rel}$

$check_6 := Check\_f_{comp.rel}$

$check_7 := Check\_f_{tension.stage8}$

$check_8 := Check\_f_{comp.stage8.c1}$

$check_9 := Check\_f_{comp.stage8.c2}$

$check_{10} := Check\_f_{comp.stage8.c3}$

$check_{11} := CheckMomentCapacity$

$check_{12} := CheckMaxCapacity$

$check_{13} := CheckStirArea$

$check_{14} := CheckShearCapacity$

$check_{15} := CheckMinStirArea$

$check_{16} := CheckMaxStirSpacing$

$check_{17} := CheckLongSteel$

$check_{18} := CheckInterfaceSpacing$

$check_{19} := CheckSplittingSteel$

$check_{20} := CheckMaxPrestressingForce$

$check_{21} := CheckPattern_0$

$check_{22} := CheckPattern_1$

$check_{23} := CheckPattern_2$

$check_{24} := CheckPattern_3$

$check_{25} := CheckPattern_4$

$check_{26} := CheckInterfaceSteel$

$check_{27} := CheckStrandFit$

$check_{28} := Check\_SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T =$

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8: FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

$LRFR_{loadrating} =$

	Moment (Strength) or Stress (Service)				Shear (Strength)				
"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.97	1.36	"N/A"	46.02	1.02	1.64	"N/A"	5.31	HL-93
"Strength I(Op)"	0.97	1.76	"N/A"	46.02	1.02	2.13	"N/A"	5.31	HL-93
"Service III(Inv)"	0.97	1.47	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.97	1.59	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.97	1.44	86.15	46.02	1.02	1.67	100.15	30.98	*Permit
"Service III"	0.97	1.48	88.86	46.02	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.25$ $CR_{LongSteel.Permit} = 1.21$ $CheckLongSteel_{loadrating} =$ "OK"

Alternate B

$ 76.17

Bridge Development Report Relative Cost Estimate

Multiple Span - Prestressed Concrete Florida-I Beam 36"

Alternative B

	SB		NB
General Provisions			
Number of Typical Spans	**2**		2
Typical Span Length (Measured @ ℄ of construction)	**90.00**	ft	**90.00**
Number of Beams per Span	**7**		**5**
Bridge Length (FFBW to FFBW measured @ ℄ of construction)	180.0	ft	180.0
Bridge Width	**56.04**	ft	**43.08**
Bridge Clear Width (Used only for no. of lanes calculation)	52.96		40.00
Beam Spacing	**8.13**	ft	**8.50**
Overhang Width	3.65	ft	4.54
Deck Thickness	**8**	in	8
Sacrificial Deck Thickness	**0.5**	in	0.5
Average Haunch Thickness	**1.5**	in	1.5
Typical Deck Cross Slope	**2%**		2%

$ 76.17

A. Bridge Substructure

Prestressed Concrete Piling			
Pile Size	**24**	in	24
End Bent			
Number of Piles	**6**		**5**
Pile Spacing	**11.75**	ft	**11.5**
Length of Piles	**90**	ft	**90**
Pile Embedment on Cap	**1**	ft	**1**
Intermediate Bent			
Number of Piles	**9**		**7**
Length of Piles	**110**	ft	**110**
Pile Embedment on Cap	**1**	ft	**1**
Total Pile Length (All Foundations)	**2070**	ft	**1670**
Substructure Concrete			
End Bent			
Cap			
Length	56.04	ft	43.08
Width	**3.50**	ft	3.50
Depth	**3.50**	ft	3.50
Volume	24.5	CY	18.8
Pedestals			
Minimum Height	**0.50**	ft	**0.50**
Width	3.17	ft	3.17
Length	2.50	ft	2.50
Volume	1.1	CY	0.8
Back Wall			
Height (Average)	3.38	ft	3.38
Width	**1.00**	ft	**1.00**
Length	54.54	ft	41.58
Volume	6.8	CY	5.2
Curtain Wall			
Height	3.77	ft	3.77
Width	**0.75**	ft	**0.75**
Length	3.50	ft	3.50
Volume	0.7	CY	0.7
Total Volume per End Bent	33.2	CY	25.5
Total Volume for the Two End Bents	66.4	CY	51.0
Intermediate Bent			
Cap			
Length	51.92	ft	39.50
Width	**3.50**	ft	3.50
Depth	**4.00**	ft	4.00
Volume	25.6	CY	19.4
Pedestals			
Minimum Height	**0.50**	ft	**0.50**
Width	3.17	ft	3.17
Length	3.50	ft	3.50
Volume	1.5	CY	1.1
Total Volume per Intermediate Bent	27.1	CY	20.5
Total Volume for all Intermediate Bents	27.1	CY	20.5

			$ 76.17
Substructure Total Concrete Volume	**93.5**	CY	**71.6**
Reinforcing Steel			
Weight per End Bent (135 lb/CY)	4480	lb	3445
Weight per Intermediate Bent (145 lb/CY)	3932	lb	2978
Substructure Total Reinforcing Steel Weight	**12892**	lb	**9868**

B. Bridge Superstructure

Neoprene Bearing Pad				
Type		**E**		**E**
Width		**32**	in	**32**
Length		**10**	in	**10**
Thickness		**1.91**	in	**1.91**
Volume		0.353	CF	0.353
Number of Pads		28		20
Total Volume		**9.88**	CF	**7.06**
Prestressed Concrete Girders				
Florida-I Beam Type		**36**		**36**
Top Flange Width		**4**	ft	**4**
Total Length (Average measured @ ℄ of construction)		**1260**	ft	**900**
Deck Concrete				
Superstructure Total Concrete Volume		**291.0**	CY	**222.3**
Reinforcing Steel				
SuperstructureTotal Reinforcing Steel Weight (205 lb/CY)		**59655**	lb	**45572**
Railing and Barriers				
Traffic Railing				
Type 32" F Shape	No. of Railing	**2**		**2**
Total Length (Average measured @ ℄ of construction)		**360**	ft	**360**
Pedestrian Railing		**No**		**No**
Concrete Parapet 27"				
Total Length (Average measured @ ℄ of construction)		**0**	ft	**0**
Bullet Railing				
Total Length (Average measured @ ℄ of construction)		**0**	ft	**0**
Expansion Joints				
Strip Seal				
Number of Joints		**2**		**2**
Length		56.04	ft	43.08
Total Length		**112.1**	ft	**86.2**

Bridge Development Report Pile Loads

End Bent

	SB		NB
General Provisions			
Number of Beams	7		5
Span Length (Measured @ ℄ of construction)	90.0	ft	90.0
Bridge Width	56.04	ft	43.08
Deck Thickness	8.0	in	8.0
Sacrificial Deck Thickness	0.5	in	0.5
Average Haunch Thickness	1.5	in	1.5
Beam Top Flange Width	4.0	ft	4.0
Beam Spacing	8.13	ft	8.50
Beam Weight	**840.0**	lb/ft	**840.0**
Traffic Railing Weight	**420.0**	lb/ft	**420.0**
Pedestrian Railing with Bullet Railing Weight	**0.0**	lb/ft	**0.0**
SIP Forms Weight	**20.0**	lb/ft^2	**20.0**
A. Live Load Reaction at End Bent			
Number of Design Lanes	4		3
Multiple Presence Factor	**0.65**		**0.85**
HL-93			
Design Truck Reaction	167.8	kip	164.6
Design Tandem Reaction	127.1	kip	124.7
Design Lane Load	74.9	kip	73.4
Total End Bent Live Load	**242.7**	kip	**238.0**
B. End Bent Dead Loads			
Self-Weight			
Cap	99.4	kip	76.2
Pedestals	4.4	kip	3.2
Back Wall	27.6	kip	21.1
Curtain Wall	3.0	kip	3.0
Total End Bent Self-Weight Dead Load	**134.4**	kip	**103.4**
Superstructure Weight			
Beams	264.6	kip	189.0
Deck	267.9	kip	206.0
Haunch	23.6	kip	16.9
Thickened Slab End	3.1	kip	2.3
SIP Forms	22.3	kip	16.2
Traffic Railing	37.8	kip	37.8
Pedestrian Railing	0.0	kip	0.0
Total End Bent Superstructure Dead Load	**619.3**	kip	**468.1**
C. Pile Loads			
Factored Reaction at Bent (Strength I) Note: Increased by 15% for preliminary design	1571.9	kip	1300.5
Number of Piles	6		5
Factored Individual Pile Load	262.0	kip	260.1
Downdrag Force	**0.0**	kip	**0.0**
Phi factor for pile driving	**0.65**		**0.65**
Required driving resistance	202	tons	200

Bridge Development Report Pile Loads

Intermediate Bent

	SB		NB
General Provisions			
Number of Beams	7		5
Span Length (Measured @ ℄ of construction)	90.0	ft	90.0
Bridge Width	56.04	ft	43.08
Deck Thickness	8.0	in	8.0
Sacrificial Deck Thickness	0.5	in	0.5
Average Haunch Thickness	1.5	in	1.5
Beam Top Flange Width	4.0	ft	4.0
Beam Spacing	8.13	ft	8.50
Beam Weight	840.0	lb/ft	840.0
Traffic Railing Weight	420.0	lb/ft	420.0
Pedestrian Railing Weight	0.0	lb/ft	0.0
SIP Forms Weight	20.0	lb/ft^2	20.0
A. Live Load Reaction at Intermediate Bent			
Number of Design Lanes	4		3
Multiple Presence Factor	0.65		0.85
HL-93			
Design Truck Reaction	190.9	kip	187.3
Design Tandem Reaction	127.1	kip	124.7
Design Lane Load	149.8	kip	146.9
Total Intermediate Bent Live Load	**325.7**	kip	**319.5**
B. Intermediate Bent Dead Loads			
Self-Weight			
Pier Cap	103.6	kip	78.8
Pedestals	6.2	kip	4.4
Total Intermediate Bent Self-Weight Dead Load	**109.8**	kip	**83.2**
Superstructure Weight			
Beams	529.2	kip	378.0
Deck	535.9	kip	412.0
Haunch	47.3	kip	33.8
Thickened Slab End	6.2	kip	4.5
SIP Forms	44.6	kip	32.4
Traffic Railing	75.6	kip	75.6
Pedestrian Railing	0.0	kip	0.0
Total Intermediate Bent Superstructure Dead Load	**1238.7**	kip	**936.2**
C. Pile Loads			
Factored Reaction at Bent (Strength I) Note: Increased by 15% for preliminary design	2594.0	kip	2108.3
Number of Piles	9		7
Factored Individual Pile Load	288.2	kip	301.2
Scour Resistance	**5.00**	kip	**5.00**
Phi factor for pile driving	**0.65**		**0.65**
Required driving resistance	226	tons	236

Bridge Development Report Cost Estimating

Effective 1/01/2012

Step One: Estimate Component Items

Utilizing the cost provided herein, develop the cost estimate for each bridge type under consideration.

A. Bridge Substructure

1. Prestressed Concrete Piling, (furnished and installed)

Size of Piling	Cost per Lin. Foot [1]	Quantity	Cost
18" (Driven Plumb or 1" Batter)	$65		
18" (Driven Battered)	$75		
24" (Driven Plumb or 1" Batter)	$85	3740	$317,900
24" (Driven Battered)	$95		
30" (Driven Plumb or 1" Batter)	$120		
30" (Driven Battered)	$140		
Heavy mild steel reinforcing in pile head (each)	$250		
Embedded Data Collector (each)	$2,000		
		Subtotal	$317,900

1 When silica fume, metakaolin or ultrafine fly ash is used add $6/LF to the piling cost.

2. Steel Piling, (furnished and installed)

Size of Piling	Cost per Lin. Foot	Quantity	Cost
14 x 73 H Section	$70		
14 x 89 H Section	$90		
20" Pipe Pile	$105		
24" Pipe Pile	$114		
30" Pipe Pile	$160		
		Subtotal	

3. Drilled Shaft (Total in-place cost)

Dia. (on land, casing salvaged)	Cost per Lin. Foot	Quantity	Cost
3 ft	$250		
4 ft	$430		
5 ft	$510		
6 ft	$630		
7 ft	$750		
Dia. (in water, casing salvaged)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$320		
4 ft	$500		
5 ft	$600		
6 ft	$690		
7 ft	$800		
8 ft	$1,100		
Dia. (in water, permanent casing)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$460		
4 ft	$625		
5 ft	$750		
6 ft	$950		
7 ft	$1,100		
8 ft	$1,500		
9 ft	$1,800		
		Subtotal	

A. Bridge Substructure (continued)

4. Sheet Piling Walls			
Size (Prestressed Concrete)	**Cost per Lin. Foot**	**Quantity**	**Cost**
10" x 30"	$100		
12" x 30"	$110		
Type (Steel)	**Cost per Sq. Foot**	**Quantity**	**Cost**
Permanent Cantilever Wall	$24		
Permanent Anchored Wall [1]	$36		
Temporary Cantilever Wall	$14		
Temporary Anchored Wall [1]	$22		
Soil Anchors	**Cost per Anchor**	**Quantity**	**Cost**
Permanent	$3,200		
Temporary	$2,800		
		Subtotal	

1 Includes the cost of waler steel, miscellaneous steel for permanent/temporary walls and concrete face for permanent walls.

5. Cofferdam Footing (Cofferdam and Seal Concrete[1])

Prorate the cost provided herein based on area and depth of water. A cofferdam footing having the following attributes cost $600,000: Area 63 ft x 37.25 ft; Depth of seal 5 ft; Depth of water over footing 16 ft

Type	**Cost per Footing**	**Quantity**	**Cost**
Cofferdam Footing			
		Subtotal	

1 Cost of seal concrete included in pay item 400-3-20 or 400-4-200.

6. Substructure Concrete			
Type	**Cost per Cubic Yard**	**Quantity**	**Cost**
Concrete [1]	$575	165.1	$94,933
Mass Concrete [1]	$512		
Seal Concrete [1]	$412		
Bulkhead Concrete [1]	$925		
Shell Fill [1]	$30		
		Subtotal	$94,933

1 Admixtures: For Calcium Nitrite add $40/cy (@4.5 gal/cy) and for silica fume, metakaolin or ultrafine fly ash add $40/cy (@ 60 lb./cy)

7. Reinforcing Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Reinforcing Steel	$0.90	22760	$20,484
		Subtotal	$20,484

Substructure Subtotal $433,317

B. Bridge Superstructure

1. Bearing Material

Type	Cost per Cubic Foot	Quantity	Cost
Neoprene Bearing Pads	**$900**	**16.94**	**$15,250**
Multirotational Bearings (kips)	**Cost per Each**	**Quantity**	**Cost**
1- 250	**$6,000**		
251- 500	**$7,000**		
501- 750	**$8,000**		
751-1000	**$9,500**		
1001-1250	**$9,900**		
1251-1500	**$10,000**		
1501-1750	**$11,000**		
1751-2000	**$12,500**		
>2000	**$15,000**		
		Subtotal	$15,250

2. Bridge Girders

Structural Steel (includes coating)	Cost per Pound	Quantity	Cost
Rolled Wide Flange Sections, straight [1]	**$1.35**		
Rolled Wide Flange Sections, curved [1]	**$1.70**		
Plate Girders, Straight [1]	**$1.50**		
Plate Girders, Curved [1]	**$1.70**		
Box Girders, Straight [1]	**$1.75**		
Box Girders, Curved [1]	**$1.85**		
Prestressed Concrete Girders	**Cost per Lin. Foot**	**Quantity**	**Cost**
Fl. Inverted Tee 16" [2]	**$80**		
Fl. Inverted Tee 20"	**$90**		
Fl. Inverted Tee 24" [2]	**$105**		
Fl. Tub (U-Beam) 48" [2]	**$700**		
Fl. Tub (U-Beam) 54"	**$750**		
Fl. Tub (U-Beam) 63"	**$800**		
Fl. Tub (U-Beam) 72"	**$900**		
Solid Flat Slab (<48"x12")	**$150**		
Solid Flat Slab (<48"x15")	**$160**		
Solid Flat Slab (48"x12")	**$160**		
Solid Flat Slab (48"x15")	**$170**		
Solid Flat Slab (60"x12")	**$170**		
Solid Flat Slab (60"x15")	**$180**		
Florida-I; 36	**$175**	**2160**	**$378,000**
Florida-I; 45	**$185**		
Florida-I; 54	**$200**		
Florida-I; 63	**$225**		
Florida-I; 72	**$250**		
Florida-I; 78	**$265**		
Florida-I; 84	**$320**		
Florida-I; 96	**$400**		
Haunched Florida-I; 78	**$700**		
Haunched Florida-I; 84	**$800**		
		Subtotal	**$378,000**

1 When weathering steel (uncoated) is used reduce the price by $0.04 per pound. Inorganic zinc coating systems have an expected life cycle of 20 years.
2 Price is based on ability to furnish products without any conversions of casting beds and without pu

B. Bridge Superstructure (continued)

3. Cast-in-Place Superstructure Concrete			
Type	**Cost per Cubic Yard**	**Quantity**	**Cost**
Box Girder Concrete, Straight	$950		
Box Girder Concrete, Curved	$1,100		
Deck Concrete	$600	513.3	$307,980
Precast Deck Overlay Concrete Class IV	$600		
		Subtotal	$307,980

4. Concrete for Precast Segmental Box Girders, Cantilever Construction			
Concrete Cost by Deck Area	**Cost per Cubic Yard**	**Quantity**	**Cost**
≤ 300,000 SF	$1,250		
> 300,000 SF AND ≤ 500,000 SF	$1,200		
> 500,000 SF	$1,150		
		Subtotal	

5. Reinforcing Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Reinforcing Steel	$0.60	105227	$63,136
		Subtotal	$63,136

6. Post-Tensioning Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Strand, Longitudinal	$2.50		
Strand, Transverse	$4.00		
Bars	$6.00		
		Subtotal	

7. Railings and Barriers			
Type	**Cost per Lin. Foot**	**Quantity**	**Cost**
Traffic Railing[1]	$70	720	$50,400
Pedestrian/Bicycle Railings:			
Concrete Parapet (27")[1]	$65		
Single Bullet Railing[1]	$27		
Double Bullet Railing[1]	$36		
Triple Bullet Railing[1]	$45		
Picket Railing (42") steel	$65		
Picket Railing (42") aluminum	$50		
Picket Railing (54") steel	$95		
Picket Railing (54") aluminum	$60		
1 Combine cost of Bullet Railings with Concrete Parapet or Traffic Railing, as appropriate.		**Subtotal**	$50,400

8. Expansion Joints			
Type	**Cost per Lin. Foot**	**Quantity**	**Cost**
Strip Seal	$360	198.3	$71,370
Finger Joint <6"	$850		
Finger Joint >6"	$1,500		
Modular 6"	$500		
Modular 8"	$700		
Modular 12"	$900		
		Subtotal	$71,370

Superstructure Subtotal $886,136

C. Miscellaneous Items

1. MSE Walls

Type	Cost per Sq. Foot	Quantity	Cost
Permanent	$26		
Temporary	$14		
		Walls Subtotal	

2. Sound Barriers

Type	Cost per Sq. Foot	Quantity	Cost
Post and Panel Sound Barriers	$25		
	Sound Barrier Subtotal		

3. Detour Bridges

Type	Cost per Sq. Foot	Quantity	Cost
Acrow Detour Bridge [1]	$55		
	Detour Bridge Subtotal		

1 Using FDOT supplied components. The cost is for the bridge proper and does not include approach work, surfacing, or guardrail.

Unadjusted Total $1,319,452

Step Two: Estimate Conditional Variables and Cost per Square Foot

After developing the total cost estimate utilizing the unit cost, modify the cost to account for site condition variables. If appropriate, the cost will be modified by the following variables:

Conditional Variables	% Increase/ Decrease	Cost (+/-)
For construction over water, increase cost by 3 %.	3%	$39,584
Phased construction or widening, increase by 20 %. [1]		
	3%	$39,584

1 Phased construction is defined as construction over traffic or construction requiring multiple phases to complete the construction of the entire cross section of the bridge. The 20 percent premium is applied to the affected units of the superstructure

Substructure Subtotal	$433,317
Superstructure Subtotal	$886,136
Walls Subtotal	
Sound Barrier Subtotal	
Detour Bridge Subtotal	
Conditional Variables	$39,584
Total Cost	$1,359,036
Total Square Feet of Deck	17843
Cost per Square Foot	$76

JB

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltB_ 90'_FIB-36_SB

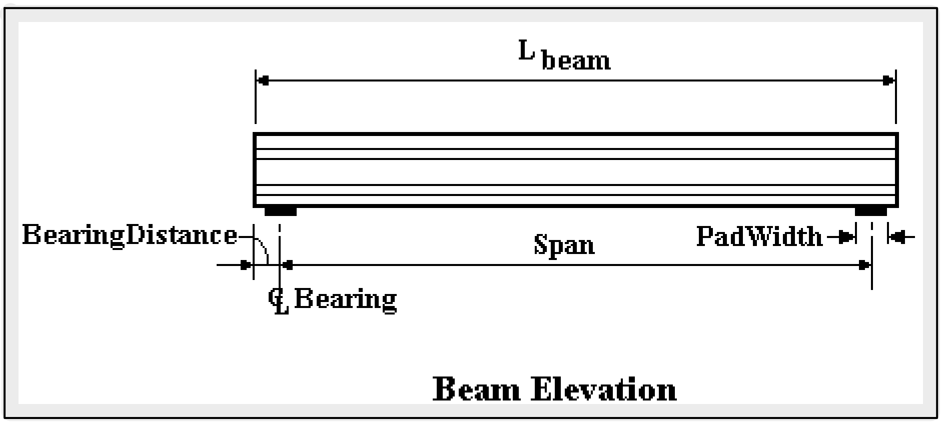
Comment = "Alt. B - SB Interior"

Legend

TanHighlight = DataEntry   YellowHighlight = CheckValues   GreyHighlight = UserComments + Graphs

BlackText = ProgramEquations   *Maroon Text = Code Reference*   *Blue Text = Commentary*

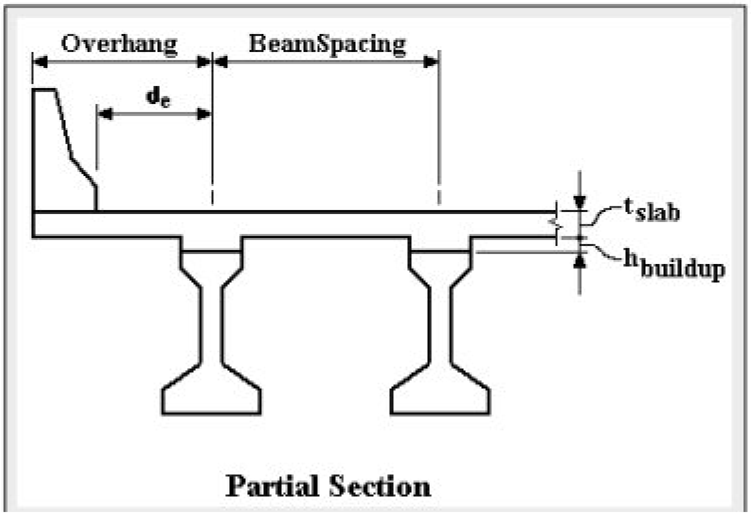
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ $Span = 88.5 \cdot ft$ $BearingDistance = 9 \cdot in$ $PadWidth = 10 \cdot in$

BeamTypeTog = "FIB36"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



$Overhang = 3.65 \cdot ft$ | $BeamSpacing = 8.13 \cdot ft$ | $t_{slab} = 8 \cdot in$ | $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ | $t_{integral.ws} = 0.5 \cdot in$ | $NumberOfBeams = 7$ | $t_{slab.delta} = 0 \cdot in$

$de = 2.75\ ft$

BeamPosition = "interior" — *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" — *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 8.13\ ft$ — *effective slab width* — *LRFD 4.6.2.6*

$t_{slab} := if(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ — *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

SectTog := if(SectionType = "transformed", 1, 0) | SectTog = 1

Concrete:

Corrosion Classification	Environment = "moderately"	*density of slab concrete*	$\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$
strength of slab concrete	$f_{c.slab} = 4.5 \cdot ksi$	*density of beam concrete*	$\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$
strength of beam concrete	$f_{c.beam} = 8.5 \cdot ksi$	*weight of future wearing surface*	$Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$
release beam strength	$f_{ci.beam} = 6 \cdot ksi$		
type of course aggregate, "Florida" *or* "Standard"	AggregateType = "Florida"	*relative humidity*	$H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ — *used in distribution calculation* — $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9 \cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor \cdot \sqrt{f_{ci.beam} \cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012 \cdot ksi$

$E_c := AggFactor \cdot \sqrt{f_{c.beam} \cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776 \cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270 \cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

tendon modulus of elasticity $E_p = 28500 \cdot ksi$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60 \cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = ▮$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4 \cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0 \cdot in$

mild steel modulus of elasticity $E_s = 29000 \cdot ksi$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = ▮ \cdot \frac{in^2}{ft}$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0 \cdot in^2$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \quad 53.33 \quad 53.33) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0) \cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.67$ $tmp\_g_{shear} = 0.88$

user value overrides (optional):

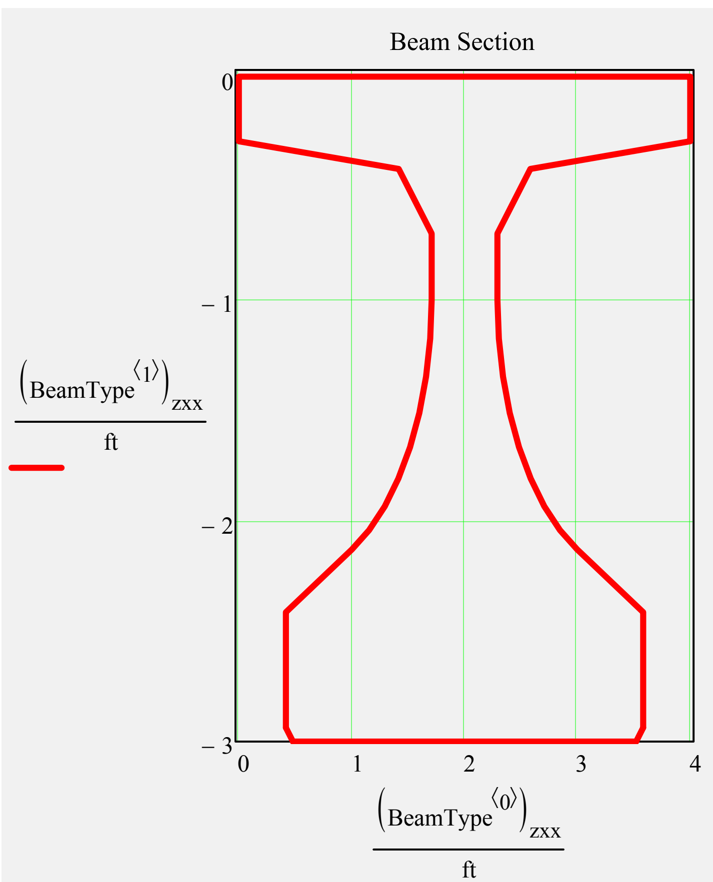
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

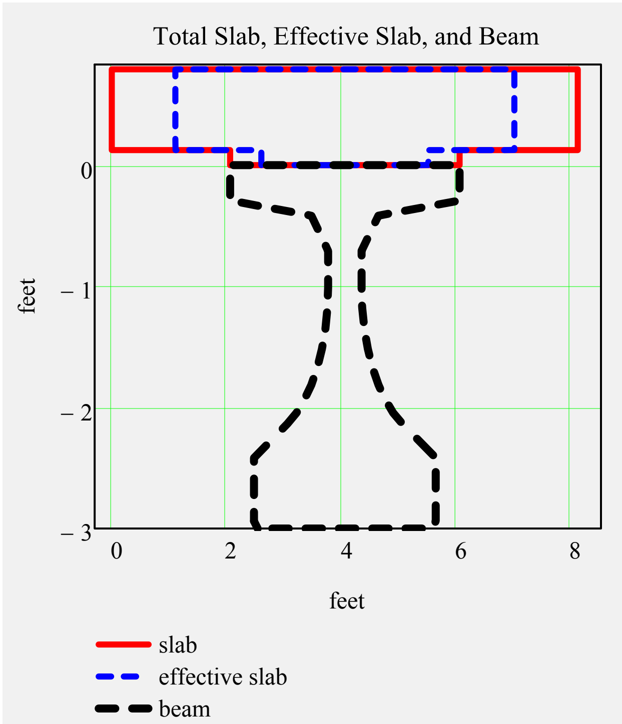
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.67$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 0.88$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 0.938 \cdot \frac{kip}{ft}$ $w_{beam} = 0.841 \cdot \frac{kip}{ft}$ $w_{forms} = 0.083 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$

additional non composite dead load (positive or negative)
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 1.862 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 1.021 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

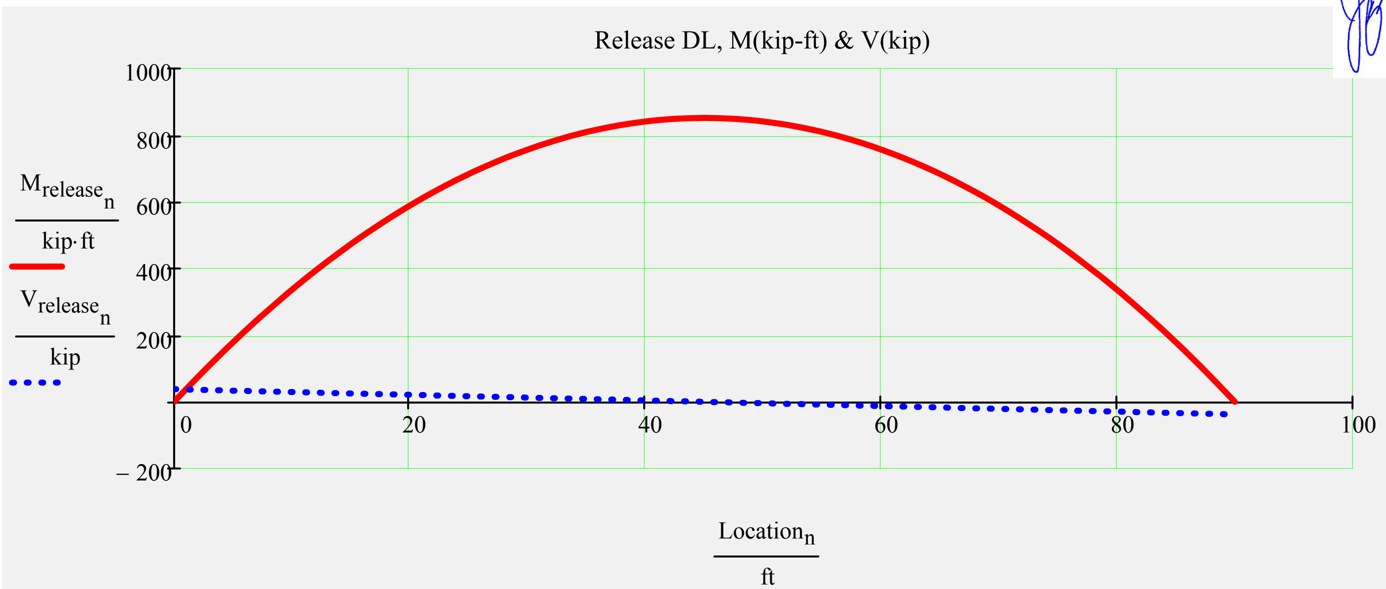
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.12 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.12 \cdot \frac{kip}{ft}$

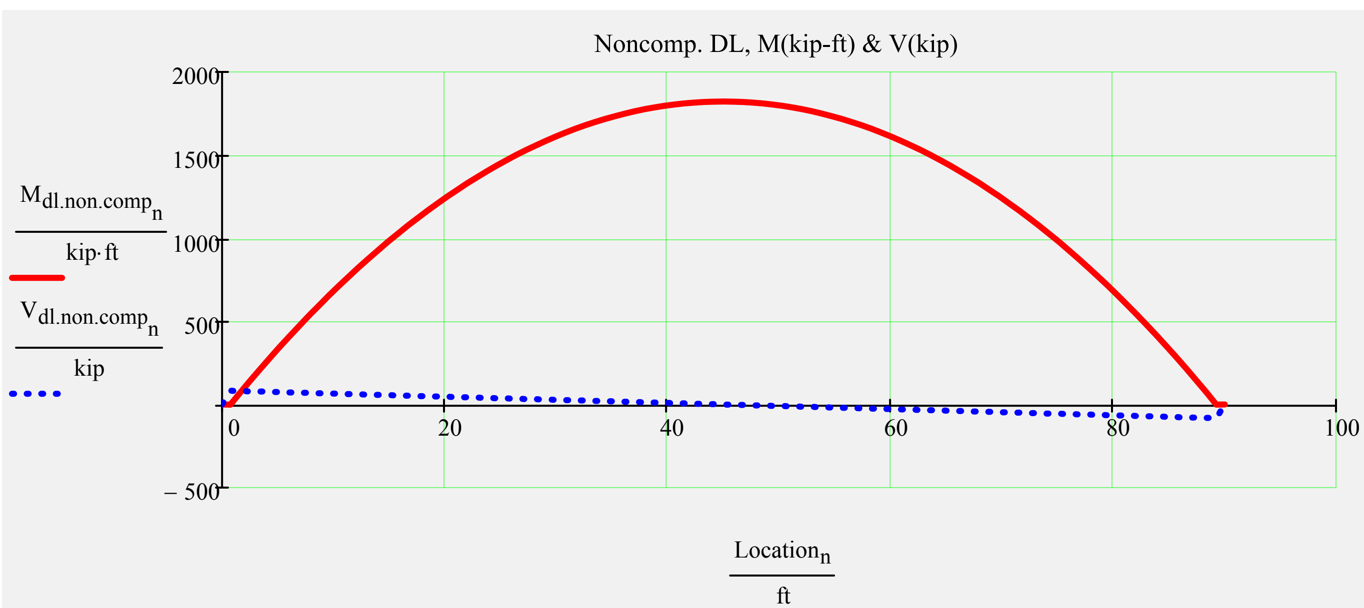
$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.12 \cdot \frac{kip}{ft}$

Release Dead Load Moments and Shear



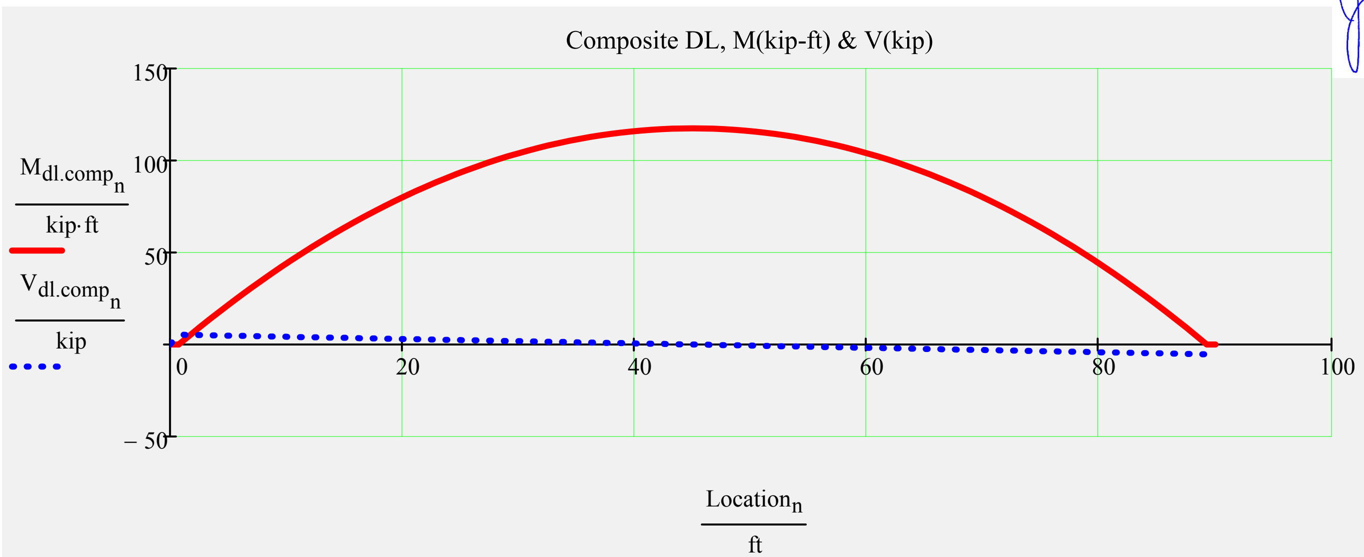
$max(M_{release}) = 851.6 \cdot kip \cdot ft$ $max(V_{release}) = 37.8 \cdot kip$

Noncomposite Dead Load Moments and Shear



$max(M_{dl.non.comp}) = 1822.3 \cdot kip \cdot ft$ $max(V_{dl.non.comp}) = 82.4 \cdot kip$

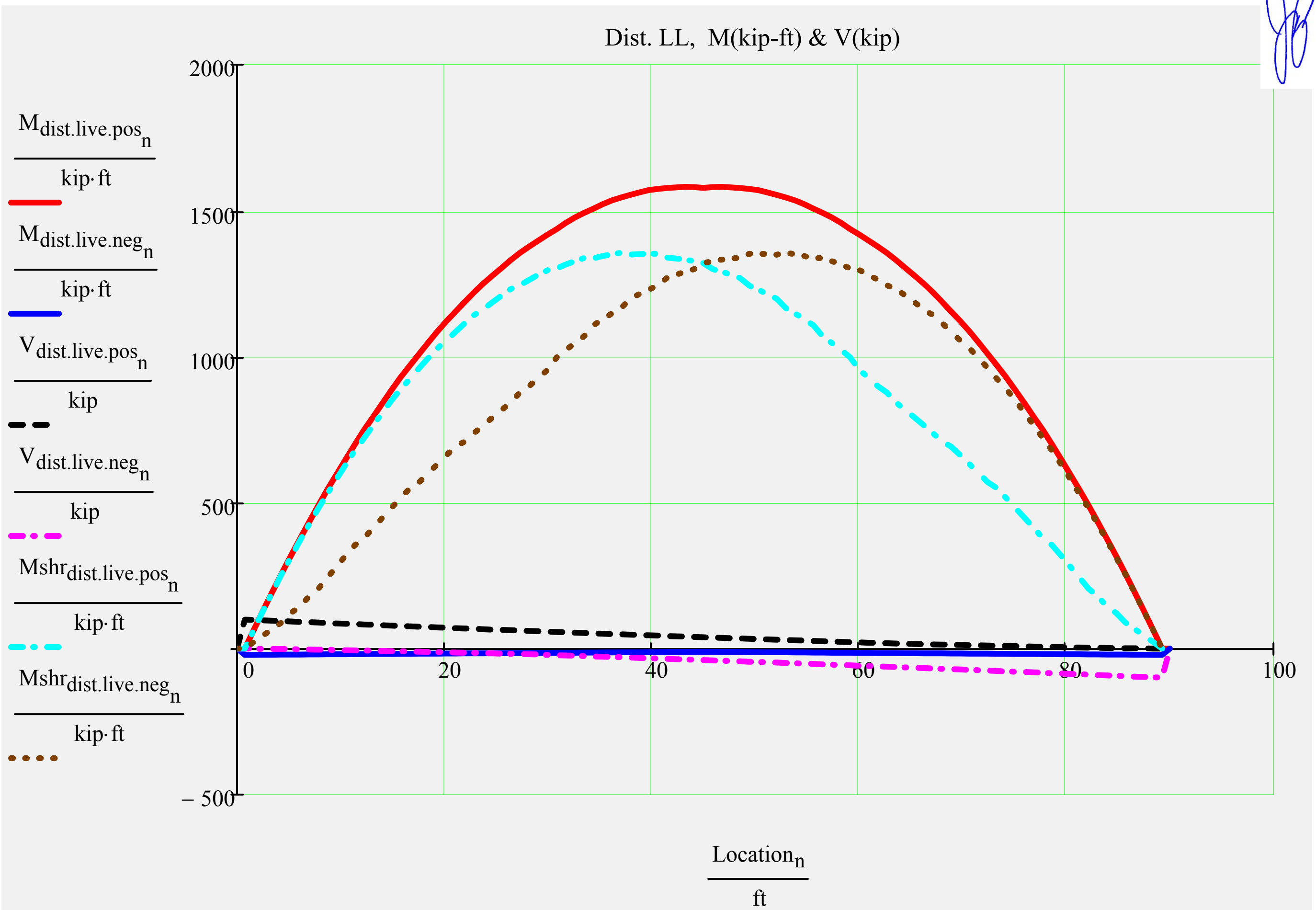
Composite Dead Load Moments and Shear



$\max(M_{dl.comp}) = 117.5 \cdot kip \cdot ft$

$\max(V_{dl.comp}) = 5.3 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 1581.7 \cdot kip \cdot ft$ $\min(M_{dist.live.neg}) = -21.4 \cdot kip \cdot ft$ $Reaction_{LL} = 100.94 \cdot kip$

$\max(V_{dist.live.pos}) = 99.8 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 1355.9 \cdot kip \cdot ft$ $Reaction_{DL} = 89.18 \cdot kip$

Prestress Strand Layout Input

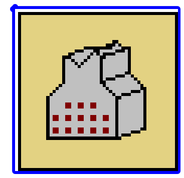
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate :=
Standard
Custom

Strand Pattern Generator:



Collapsed Region for Custom Strand Sizes...

$CheckPattern_0$ = "OK" — ***check 0*** *- no debonded tendon in outside row*

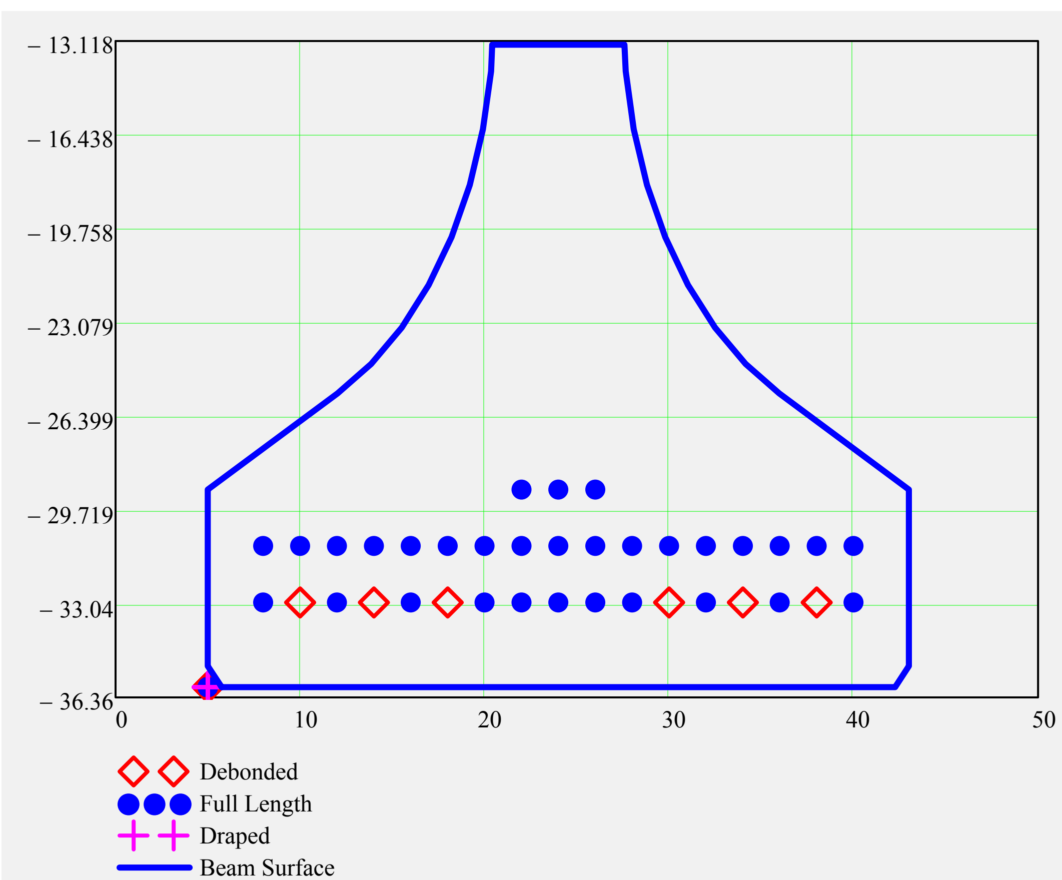
$CheckPattern_1$ = "OK" — ***check 1*** *- less than 25% debonded tendons total*

$CheckPattern_2$ = "OK" — ***check 2*** *- less than 40% debonded tendons in any row*

$CheckPattern_3$ = "OK" — ***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*

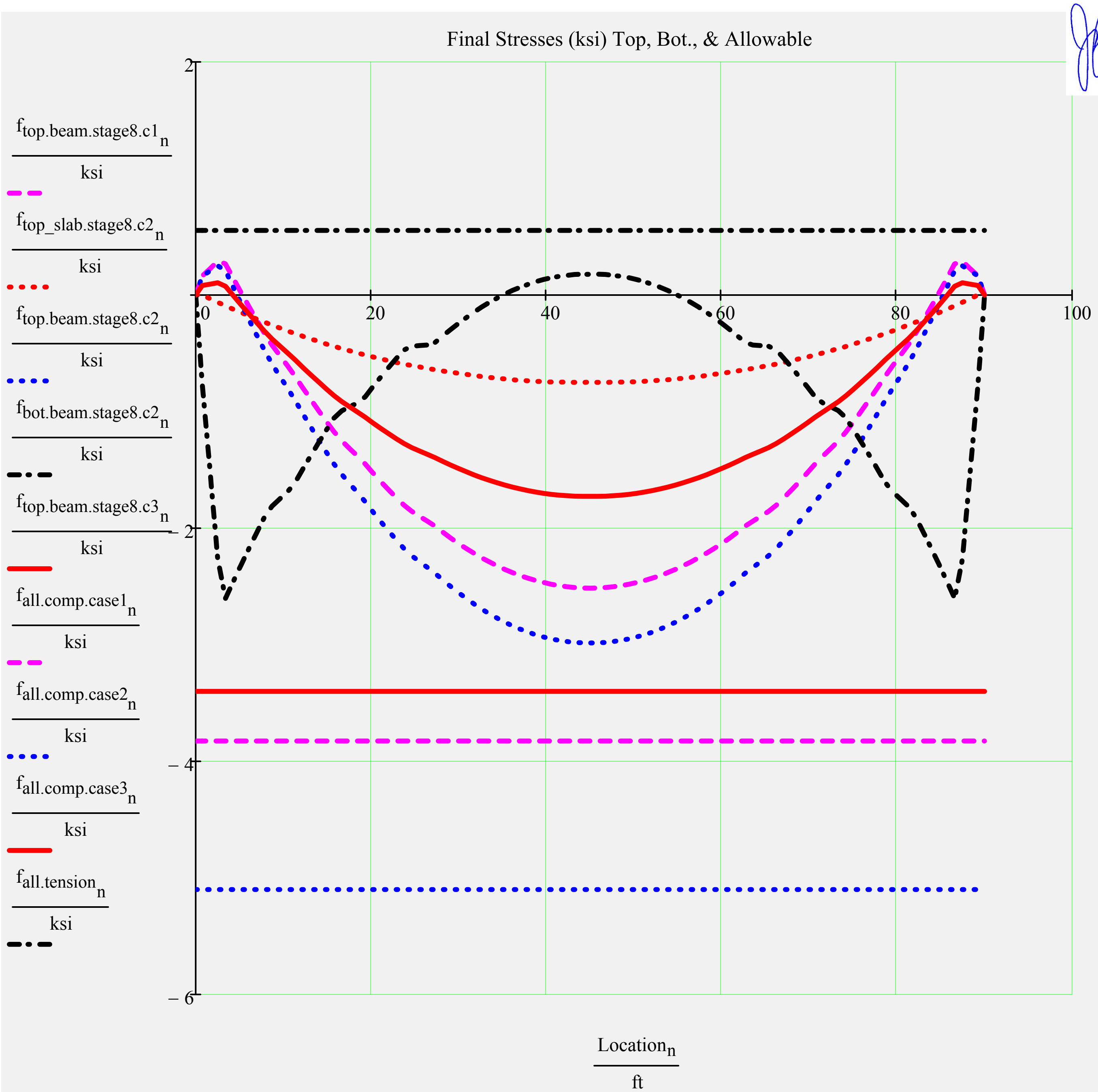
$CheckPattern_4$ = "OK" — ***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

Tendon Layout



Release Stresses

Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.79$	$Check\_f_{tension.rel}$ = "OK"	*(Release tension)*
$\min(CR\_f_{comp.rel}) = 1.09$	$Check\_f_{comp.rel}$ = "OK"	*(Release compression)*
$\min(CR\_f_{tension.stage8}) = 3.1$	$Check\_f_{tension.stage8}$ = "OK"	*(Service III , PS + DL +LL*0.8)*
$\min(CR\_f_{comp.stage8.c1}) = 1.52$	$Check\_f_{comp.stage8.c1}$ = "OK"	*(Service I , PS + DL)*
$\min(CR\_f_{comp.stage8.c2}) = 1.71$	$Check\_f_{comp.stage8.c2}$ = "OK"	*(Service I , PS + DL +LL)*
$\min(CR\_f_{comp.stage8.c3}) = 1.97$	$Check\_f_{comp.stage8.c3}$ = "OK"	*(Service I , (PS + DL)*0.5 +LL)*

Section and Strand Properties Summary

$A_{beam} = 807.4 \cdot in^2$	*Concrete area of beam*	$I_{beam} = 127557.7893 \cdot in^4$	*Gross Moment of Inertia of Beam about CG*
$y_{comp} = -8.82 \cdot in$	*Dist. from top of beam to CG of gross composite section*	$I_{comp} = 344044.9102 \cdot in^4$	*Gross Moment of Inertia Composite Section about CG*
$A_{deck} = 619.92 \cdot in^2$	*Concrete area of deck slab*	$A_{ps} = 8 \cdot in^2$	*total area of strands*
$d_{b.ps} = 0.6 \cdot in$	*diameter of Prestressing strand*	$\min(PrestressType) = 0$	*0 - low lax 1 - stress relieved*
$f_{py} = 243 \cdot ksi$	*tendon yield strength*	$f_{pj} = 203 \cdot ksi$	*prestress jacking stress*

$L_{shielding}^{T} = (8 \quad 16 \quad 24 \quad 0 \quad 0 \quad 0) \cdot ft$

$A_{ps.row}^{T} = (0.4 \quad 0.4 \quad 0.4 \quad 2.4 \quad 3.7 \quad 0.7) \cdot in^2$

$d_{ps.row} =$ (in)

	0	1	2	3	4	5	6	7	8	9
0	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
1	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
2	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
3	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
4	-31	-31	-31	-31	-31	-31	-31	-31	-31	-31
5	-29	-29	-29	-29	-29	-29	-29	-29	-29	...

$\cdot in$

TotalNumberOfTendons = 37

NumberOfDebondedTendons = 6

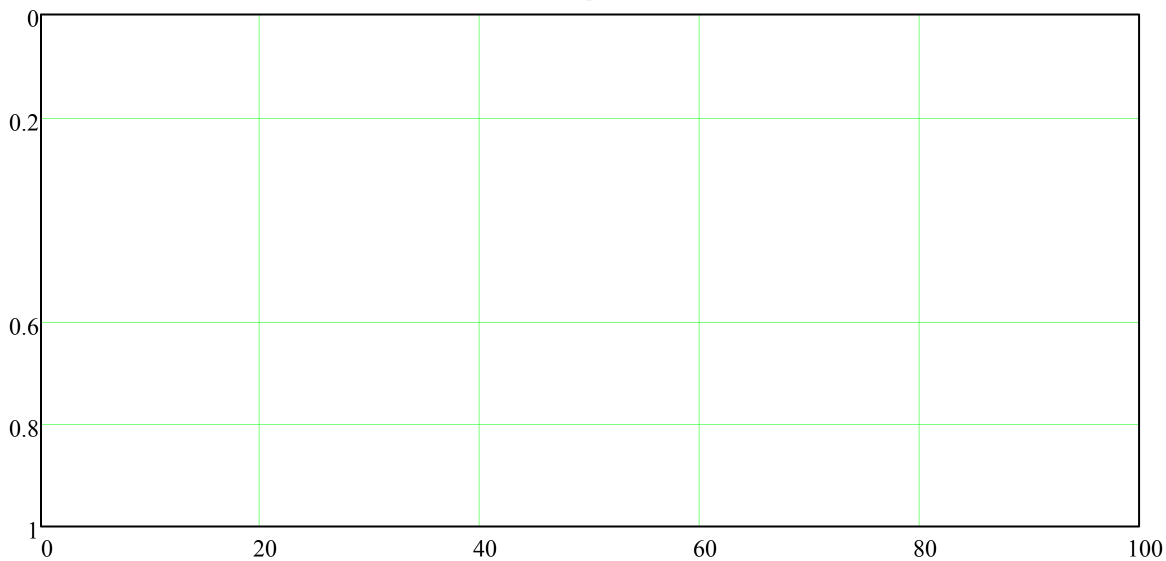
NumberOfDrapedTendons = 0

StrandSize = "0.6 in low lax"

$StrandArea = 0.22 \cdot in^2$

$JackingForce_{per.strand} = 43.94 \cdot kip$

Location of Depressed Strands



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$ *Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3*

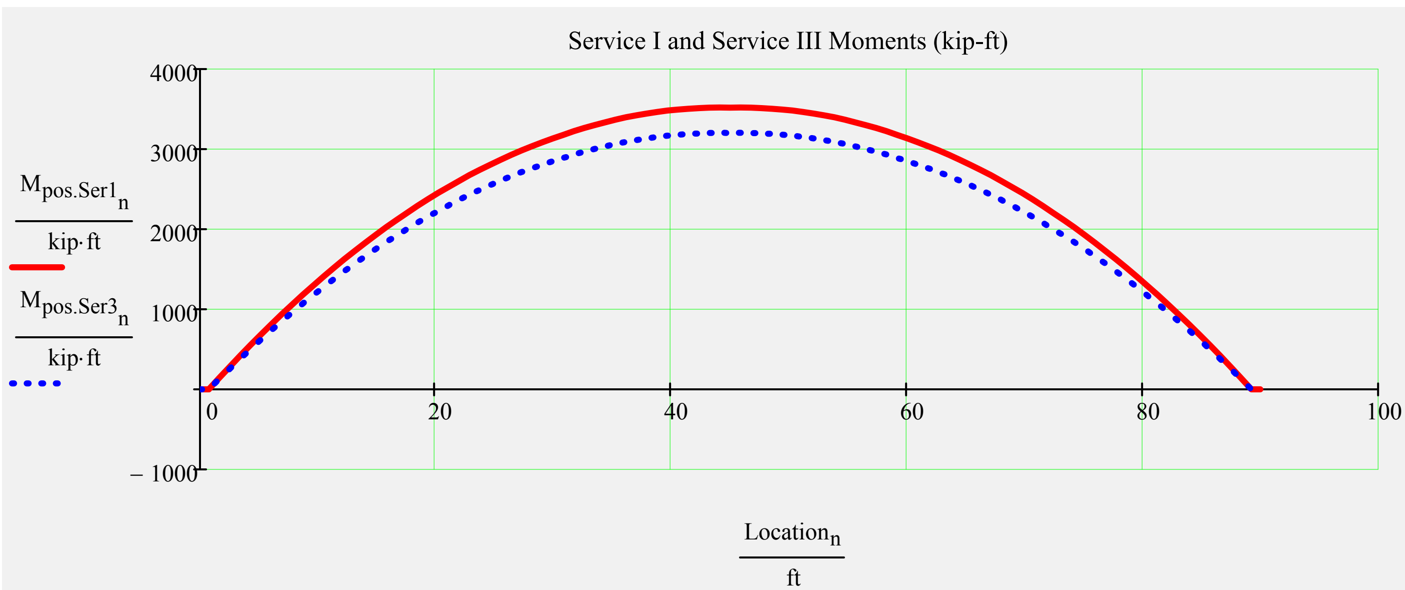
$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

$f_{pe} = 178 \cdot ksi$ $\Delta f_{pTot} = -24 \cdot ksi$

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.95 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.05 \cdot \%$

Check_f_{pt} = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f_{pe} = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 3519.4 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 3203.3 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.61	−2.9
2	−0.72	−2.41
4	−0.67	−2.45
6	−2.48	−1.02
8	−2.98	0.18

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−1675.9
"Final (about composite centroid)"	−1431.5	−1392.1

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	799.37	126328.79	−19.39
"Transformed Beam (initial)"	856.4	134564.33	−20.21
"Transformed Beam "	847.29	133321.64	−20.09
"Composite "	1492.76	370128.72	−9.19

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	851.6
"Non-composite (includes bm wt.)"	1822.3
"Composite"	117.5
"Distributed Live Load"	1578

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

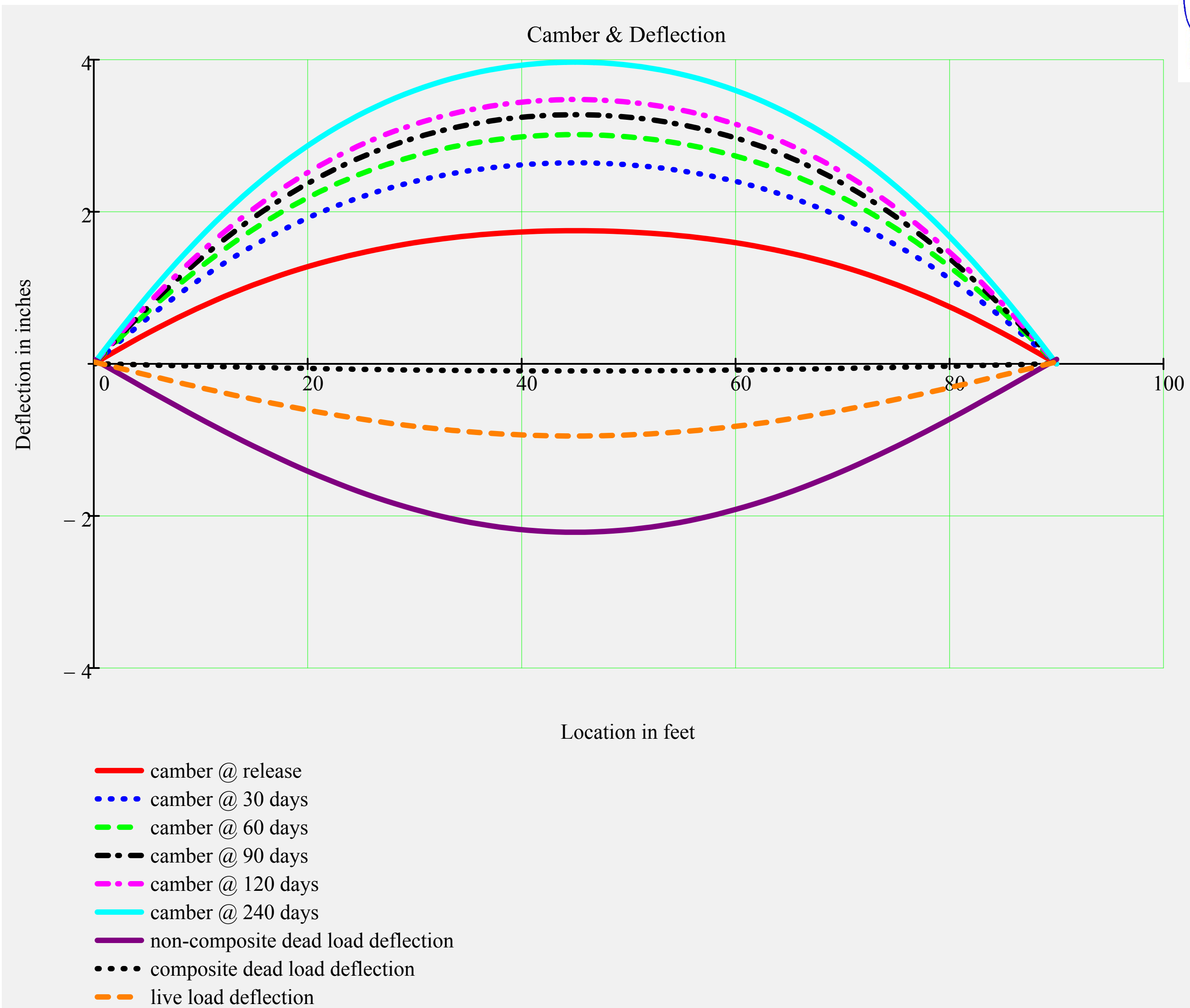
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

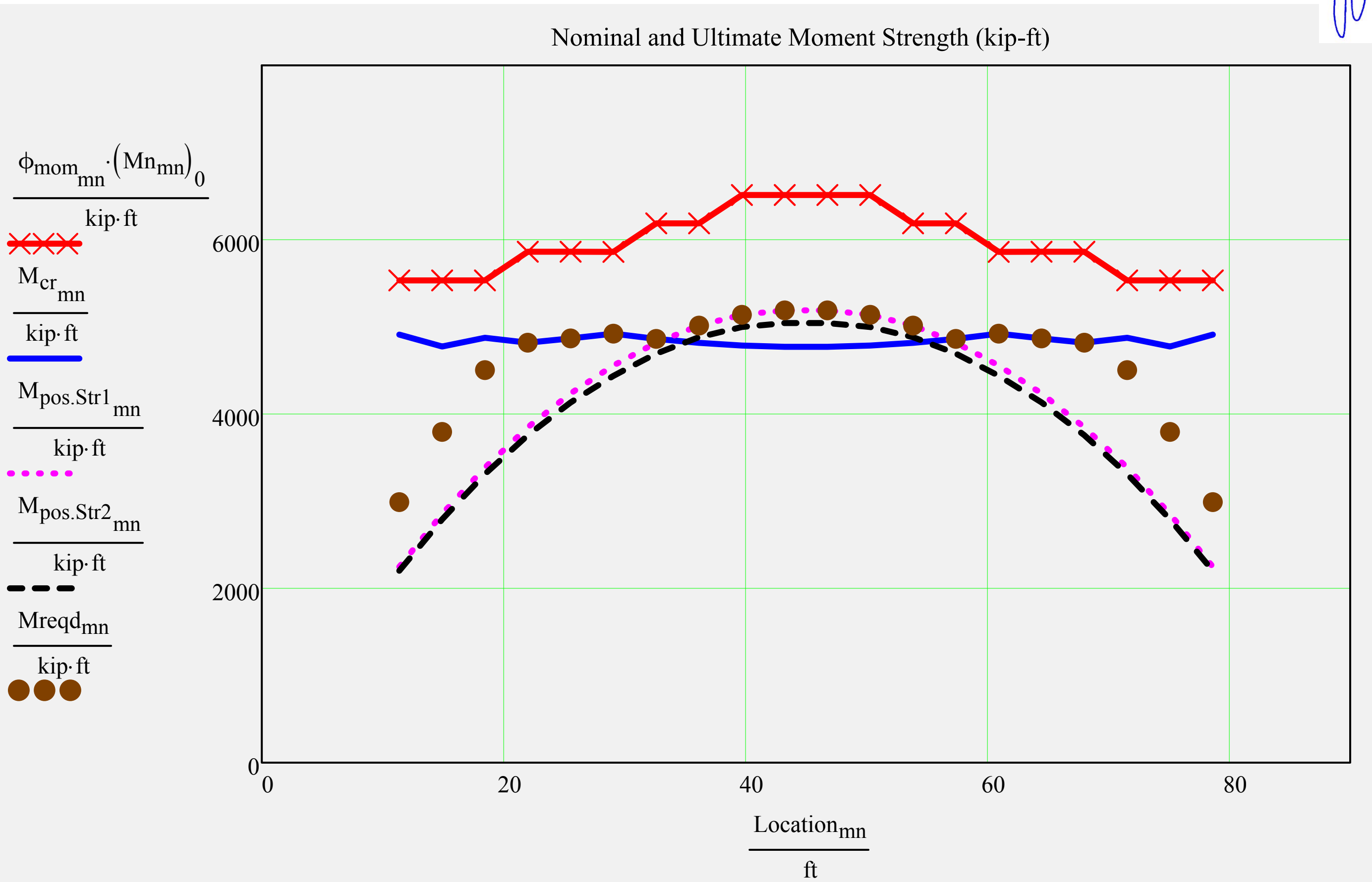
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	−0.0497	−0.8058	0.4274	1.7515
"30 Days"	−0.2398	−1.4277	0.7048	2.6438
"60 Days"	−0.3078	−1.6502	0.8132	3.0143
"90 Days"	−0.3441	−1.7688	0.871	3.2764
"120 Days"	−0.3666	−1.8425	0.9069	3.4763
"240 Days"	−0.4082	−1.9786	0.9732	3.9676
"non-comp DL"	−0.2679	0.2128	−0.3827	−2.2141
"comp DL"	−0.0052	0.0152	−0.0162	−0.0938
"LL"	−0.0529	0.1548	−0.1653	−0.9487

Strength Limit State Moments



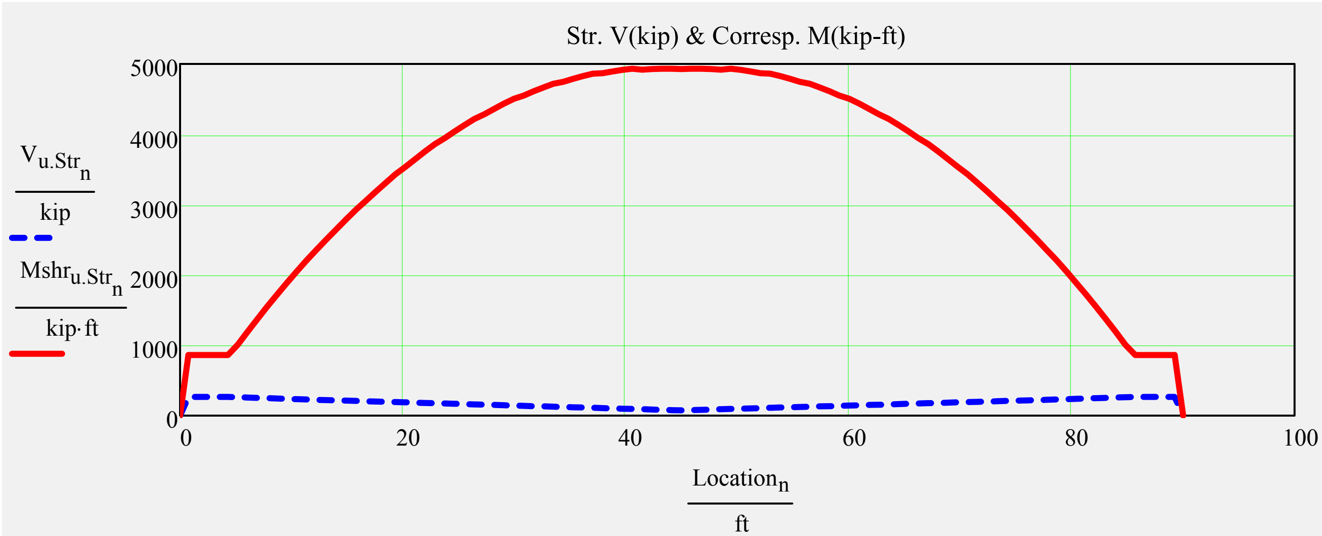
$CR_{Str.mom_n} := 10$ $\qquad$ $CR_{Str.mom_{mn}} := \dfrac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ $\qquad$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.19$

$\max(Mreqd) = 5188.7 \cdot kip \cdot ft$

CheckMomentCapacity := if(min($CR_{Str.mom}$) > 0.99, "OK" , "No Good!") $\qquad$ CheckMomentCapacity = "OK"

Strength Shear and Associated Moments



$\max(V_{u.Str}) = 266.1 \cdot kip$ $\max(Mshr_{u.Str}) = 4931.9 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing	Number of Spaces	area per stirrup
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

tmp_s = (above) ·in   tmp_NumberSpaces = (above) $tmp\_A_{stirrup}$ = (above) $\cdot in^2$

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

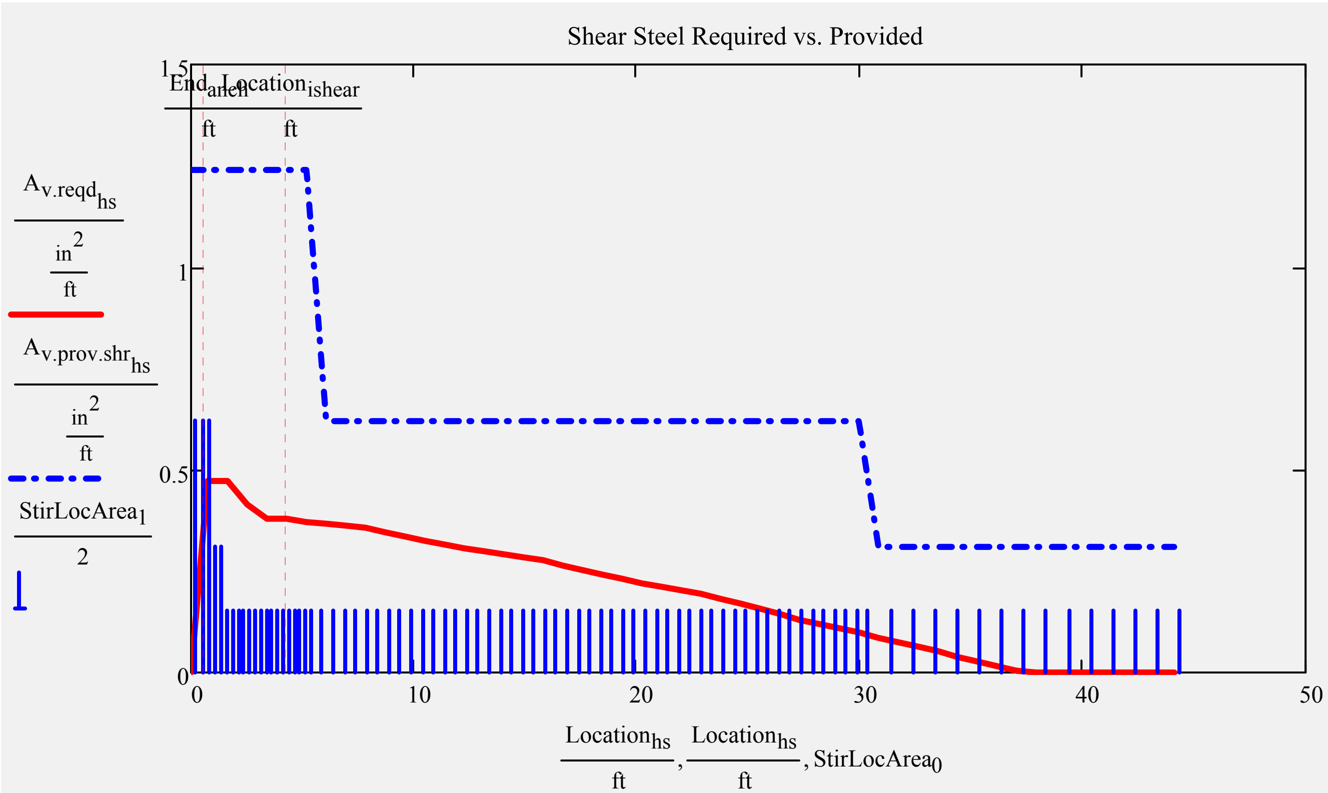
Location	$user\_s_{nspacings}$:=	$user\_NumberSpaces_{nspacings}$:=	$user\_A_{stirrup\,nspacings}$:=	$interface\_factor_{nspacings}$:=
A1 stirrup	−1·in	−1	$-1 \cdot in^2$	0.25
A2 stirrup	−1·in	−1	$-1 \cdot in^2$	0.5
A3 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S1 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S2 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S3 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S4 stirrup	−1·in	−1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

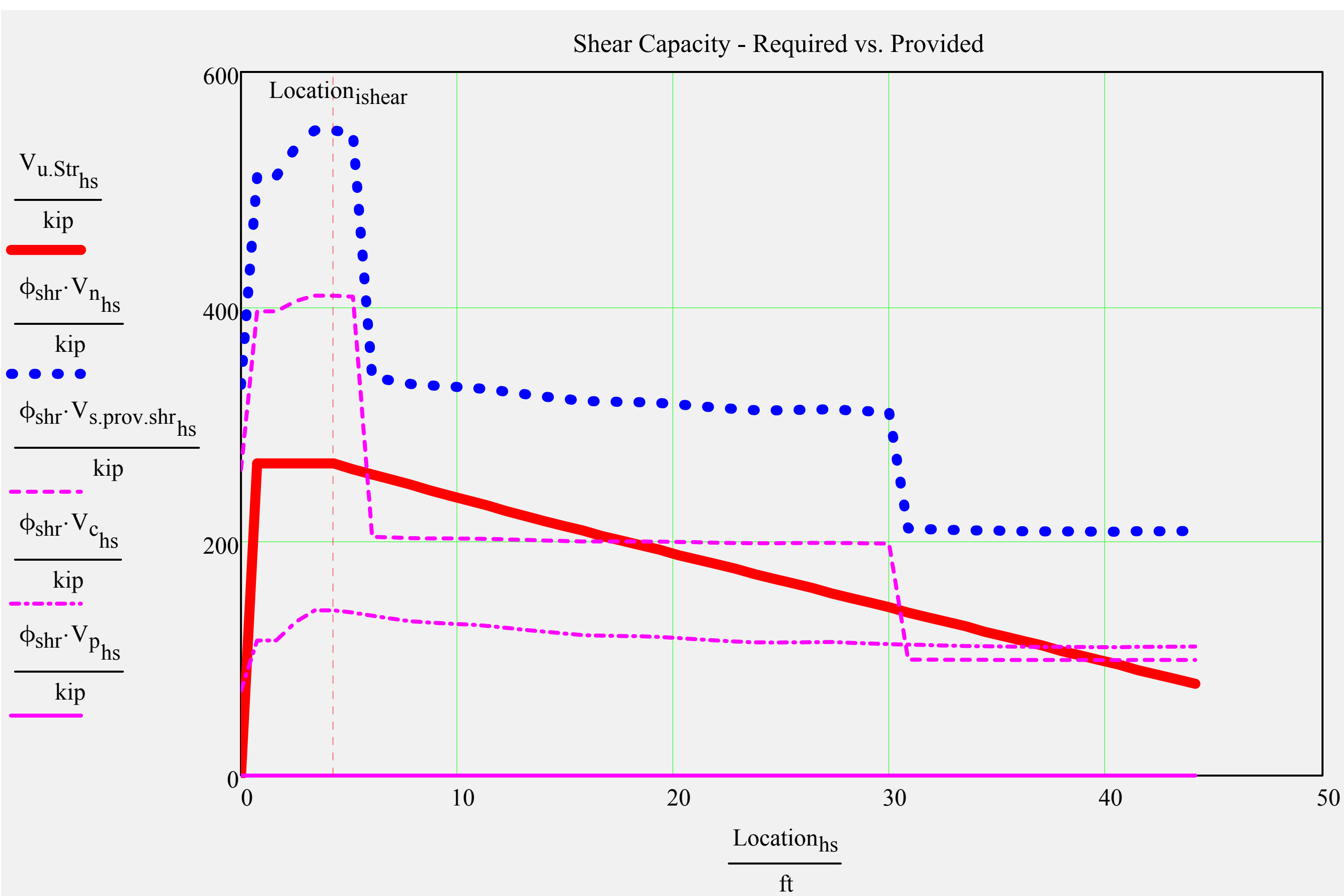
Stirrup sizes and spacings used in analysis

The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s = (·in)	NumberSpaces =	$A_{stirrup}$ = ($\cdot in^2$)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

EndCover = 2.5·in


Shear Steel Required vs. Provided
End_area / ft
Location_ishear / ft
A_v.reqd_hs / (in^2/ft)
A_v.prov.shr_hs / (in^2/ft)
StirLocArea_1 / 2
1.5
1
0.5
0
0
10
20
30
40
50
Location_hs / ft , Location_hs / ft , StirLocArea_0

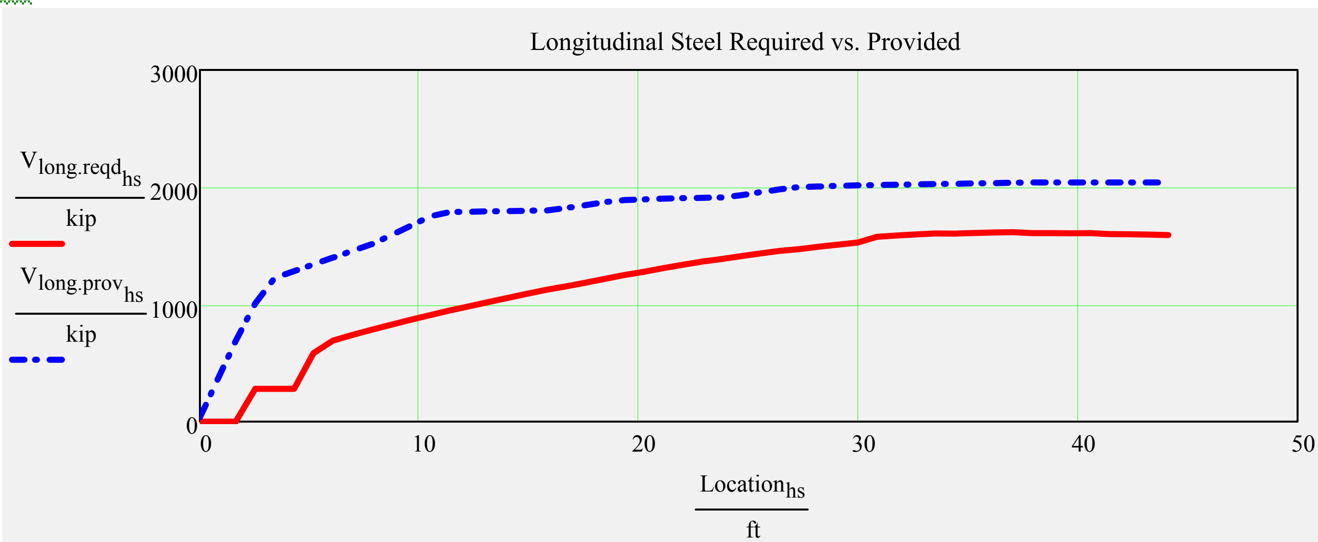

Shear Capacity - Required vs. Provided
Location_ishear
V_u.Str_hs / kip
φ_shr · V_n_hs / kip
φ_shr · V_s.prov.shr_hs / kip
φ_shr · V_c_hs / kip
φ_shr · V_p_hs / kip
600
400
200
0
0
10
20
30
40
50
Location_hs / ft

$CheckShearCapacity = "OK"$

$CheckMinStirArea = "OK"$

$CheckStirArea = "OK"$

$CheckMaxStirSpacing = "OK"$

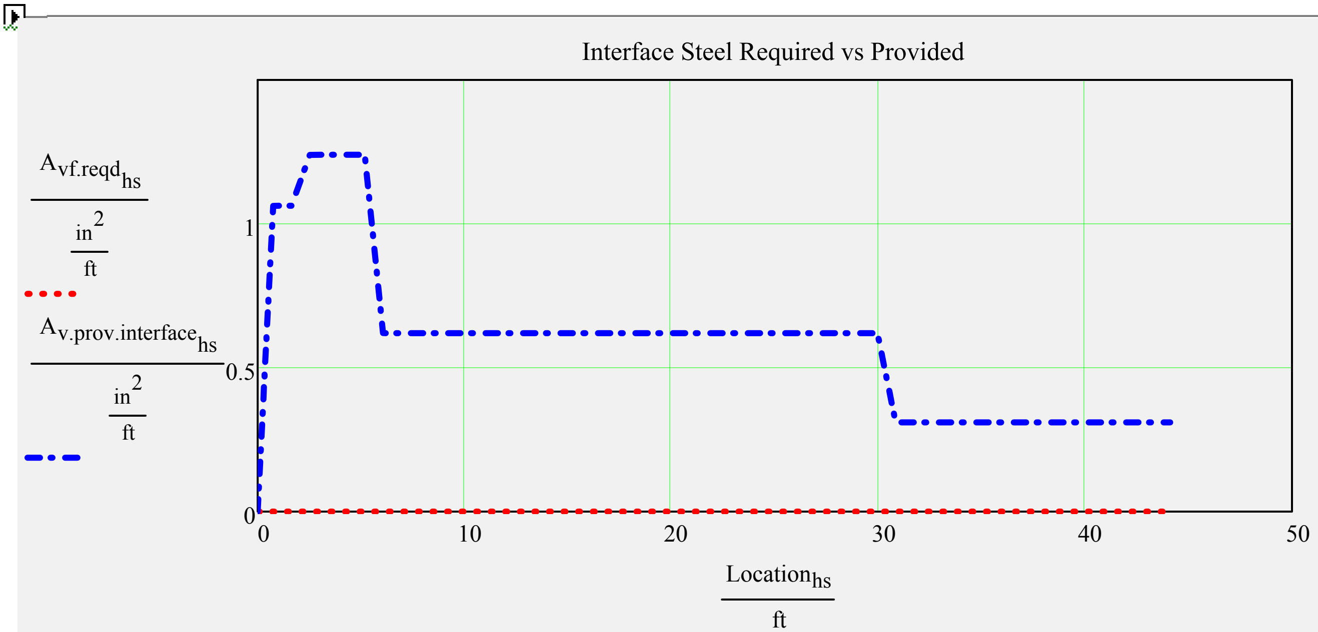


$$CR_{LongSteel_{hs}} := if\left(V_{long.reqd_{hs}} < .01kip, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$$

$$\min(CR_{LongSteel}) = 1.26$$

$$CheckLongSteel := if(\min(CR_{LongSteel}) > 1, "OK", "No Good, add steel!")$$

$CheckLongSteel = "OK"$



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \ge 1, "OK", "No Good"\right)$

CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A." CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0$:= AcceptAASHTO

$check_1$:= AcceptSDG

$check_2$:= AcceptOntario

$check_3$:= Check_f_{pt}

$check_4$:= Check_f_{pe}

$check_5$:= Check_$f_{tension.rel}$

$check_6$:= Check_$f_{comp.rel}$

$check_7$:= Check_$f_{tension.stage8}$

$check_8$:= Check_$f_{comp.stage8.c1}$

$check_9$:= Check_$f_{comp.stage8.c2}$

$check_{10}$:= Check_$f_{comp.stage8.c3}$

$check_{11}$:= CheckMomentCapacity

$check_{12}$:= CheckMaxCapacity

$check_{13}$:= CheckStirArea

$check_{14}$:= CheckShearCapacity

$check_{15}$:= CheckMinStirArea

$check_{16}$:= CheckMaxStirSpacing

$check_{17}$:= CheckLongSteel

$check_{18}$:= CheckInterfaceSpacing

$check_{19}$:= CheckSplittingSteel

$check_{20}$:= CheckMaxPrestressingForce

$check_{21}$:= $CheckPattern_0$

$check_{22}$:= $CheckPattern_1$

$check_{23}$:= $CheckPattern_2$

$check_{24}$:= $CheckPattern_3$

$check_{25}$:= $CheckPattern_4$

$check_{26}$:= CheckInterfaceSteel

$check_{27}$:= CheckStrandFit

$check_{28}$:= Check_$SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T$ =

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8:
FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

$LRFR_{loadrating}$ =

	Moment (Strength) or Stress (Service)				Shear (Strength)				
"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.67	1.44	"N/A"	53.10	0.88	1.52	"N/A"	5.31	HL-93
"Strength I(Op)"	0.67	1.86	"N/A"	53.10	0.88	1.97	"N/A"	5.31	HL-93
"Service III(Inv)"	0.67	1.34	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.67	1.47	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.67	1.51	90.71	53.10	0.88	1.55	93.28	5.31	*Permit
"Service III"	0.67	1.37	81.95	45.13	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.3$ $CR_{LongSteel.Permit} = 1.26$ $CheckLongSteel_{loadrating}$ = "OK"

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltB_ 90'_FIB-36_SB

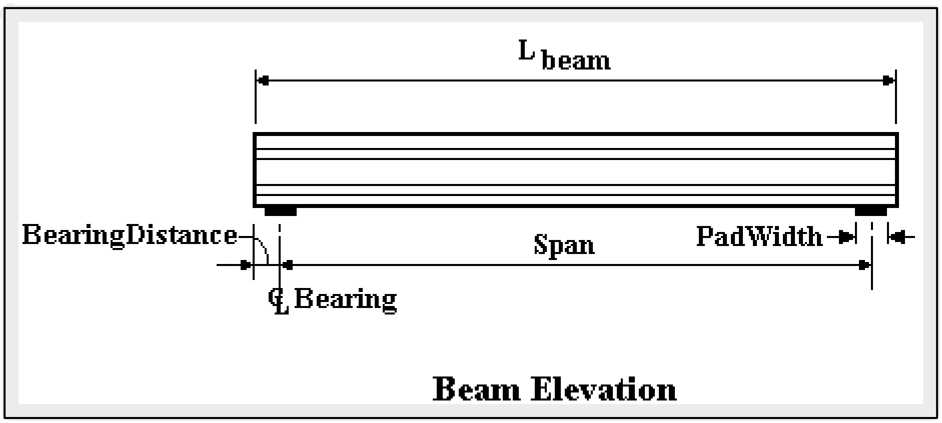
Comment = "Alt. B - SB Exterior"

Legend

TanHighlight = DataEntry   YellowHighlight = CheckValues   GreyHighlight = UserComments + Graphs

BlackText = ProgramEquations   *Maroon Text = Code Reference*   *Blue Text = Commentary*

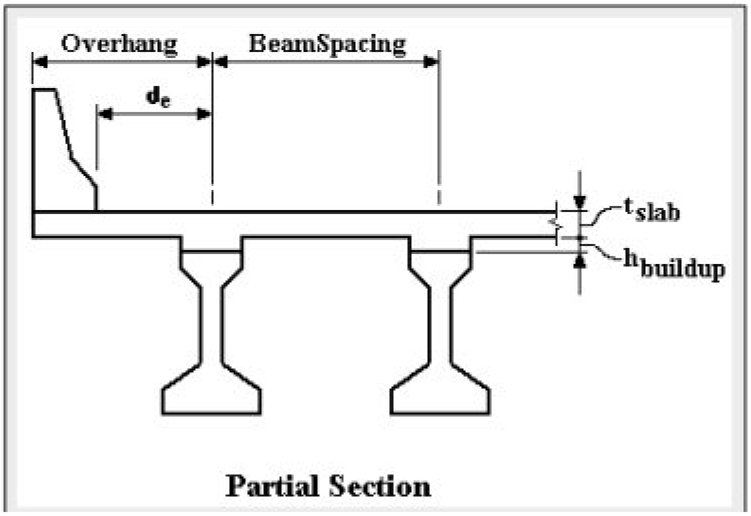
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ $Span = 88.5 \cdot ft$ $BearingDistance = 9 \cdot in$ $PadWidth = 10 \cdot in$

BeamTypeTog = "FIB36"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



$Overhang = 3.65 \cdot ft$ $BeamSpacing = 8.13 \cdot ft$ $t_{slab} = 8 \cdot in$ $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ $t_{integral.ws} = 0.5 \cdot in$ $NumberOfBeams = 7$ $t_{slab.delta} = 0 \cdot in$

$de = 2.75\ ft$

BeamPosition = "exterior" *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 7.71\ ft$ *effective slab width* *LRFD 4.6.2.6*

$t_{slab} := \text{if}\left(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab}\right)$ *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

$SectTog := \text{if}(SectionType = \text{"transformed"}, 1, 0)$ $SectTog = 1$

Concrete:

Corrosion Classification Environment = "moderately"

density of slab concrete $\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$

strength of slab concrete $f_{c.slab} = 4.5 \cdot ksi$

density of beam concrete $\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$

strength of beam concrete $f_{c.beam} = 8.5 \cdot ksi$

weight of future wearing surface $Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$

release beam strength $f_{ci.beam} = 6 \cdot ksi$

type of course aggregate, "Florida" *or* "Standard" AggregateType = "Florida"

relative humidity $H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ *used in distribution calculation* $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9\cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor\cdot\sqrt{f_{ci.beam}\cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012\cdot ksi$

$E_c := AggFactor\cdot\sqrt{f_{c.beam}\cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776\cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270\cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

tendon modulus of elasticity $E_p = 28500\cdot ksi$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60\cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4\cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0\cdot in$

mild steel modulus of elasticity $E_s = 29000\cdot ksi$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62\cdot\frac{in^2}{ft}$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0\cdot in^2$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33\ \ 53.33\ \ 53.33)\cdot kip$

$PermitAxleSpacing^T = (0\ \ 14\ \ 14\ \ 0)\cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.87$ $tmp\_g_{shear} = 0.92$

user value overrides (optional):

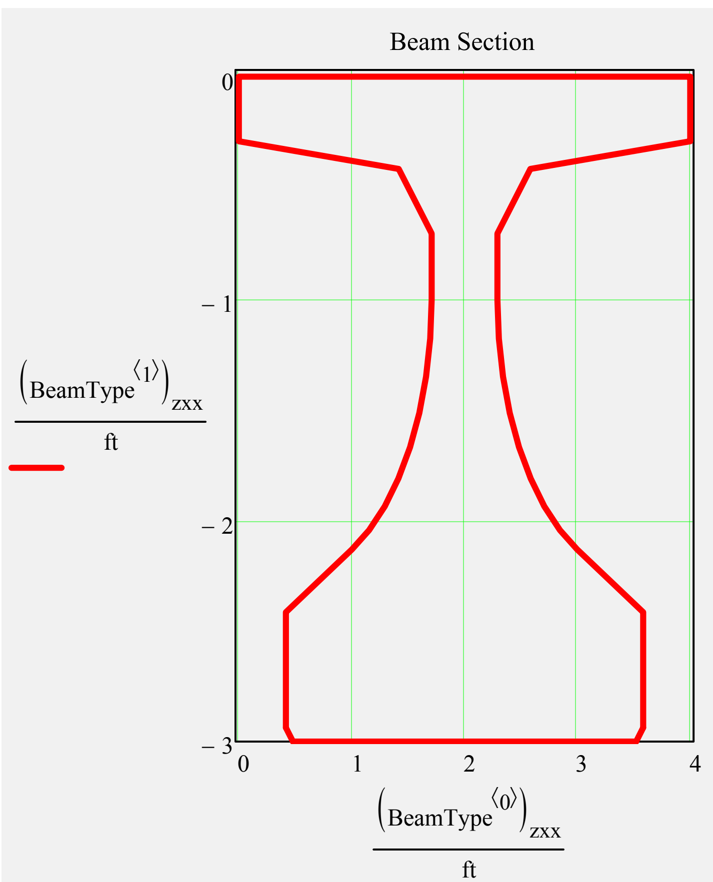
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

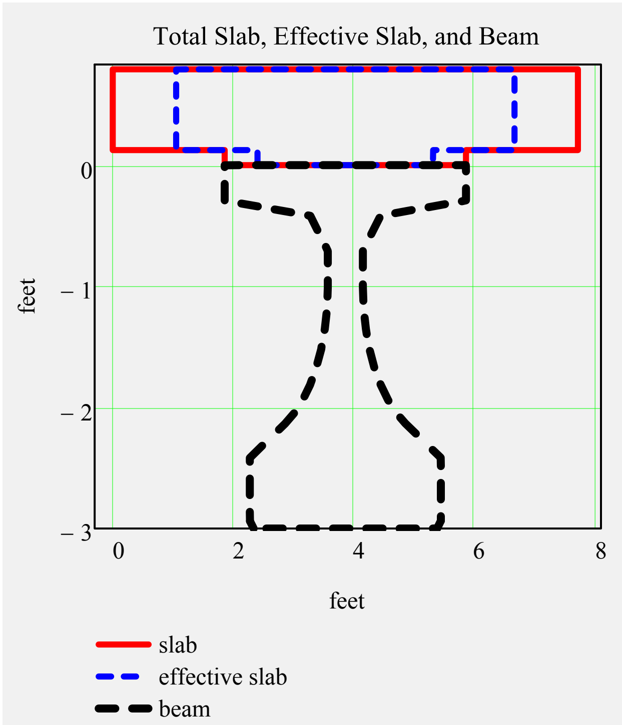
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.87$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 0.92$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 0.894 \cdot \frac{kip}{ft}$ $w_{beam} = 0.841 \cdot \frac{kip}{ft}$ $w_{forms} = 0.041 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$

additional non composite dead load (positive or negative)
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 1.777 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 0.936 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

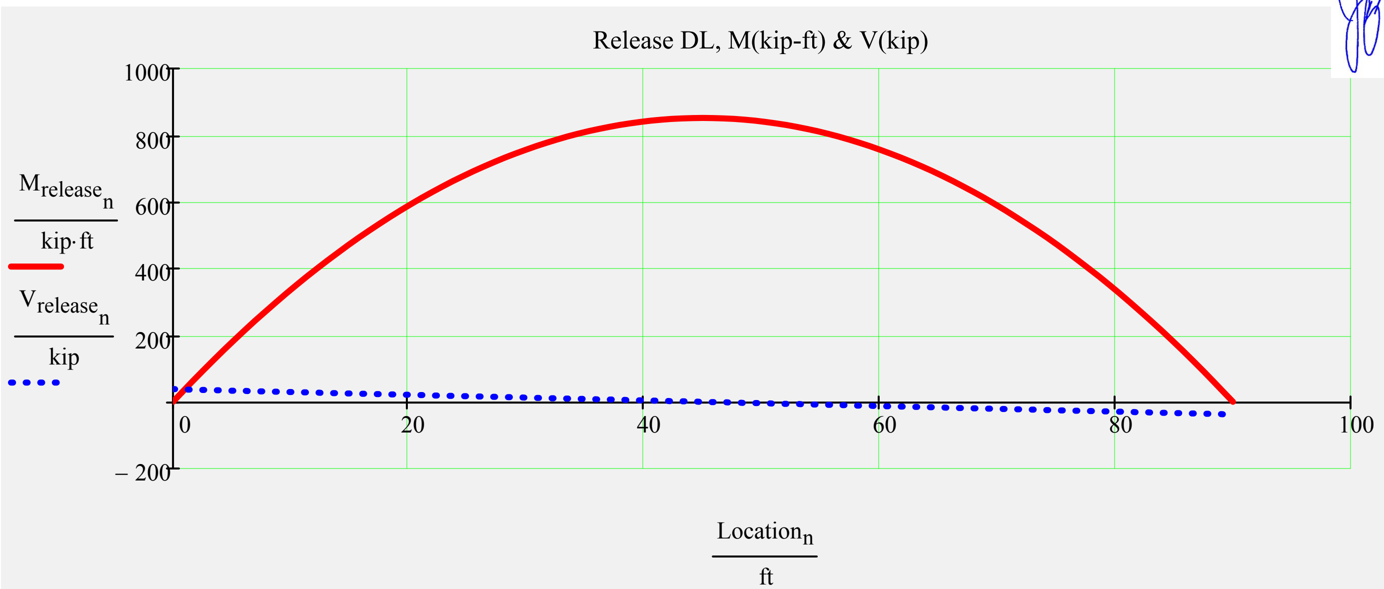
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.12 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.12 \cdot \frac{kip}{ft}$

$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.12 \cdot \frac{kip}{ft}$

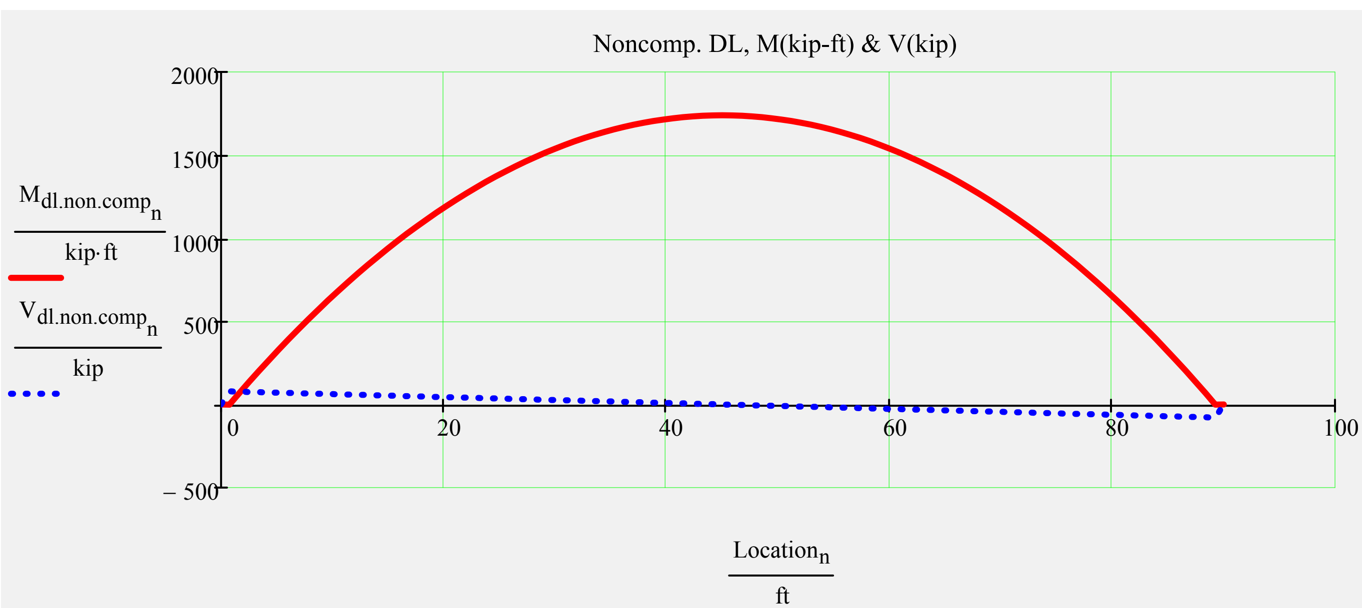
Release Dead Load Moments and Shear



$max(M_{release}) = 851.6 \cdot kip \cdot ft$

$max(V_{release}) = 37.8 \cdot kip$

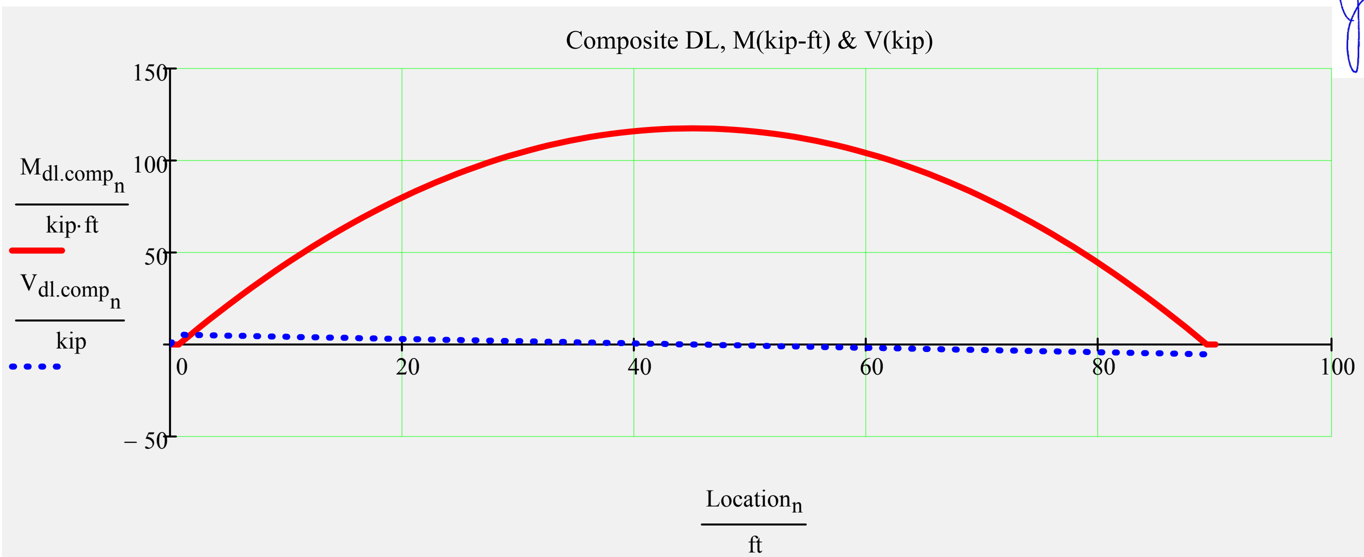
Noncomposite Dead Load Moments and Shear



$max(M_{dl.non.comp}) = 1739 \cdot kip \cdot ft$

$max(V_{dl.non.comp}) = 78.6 \cdot kip$

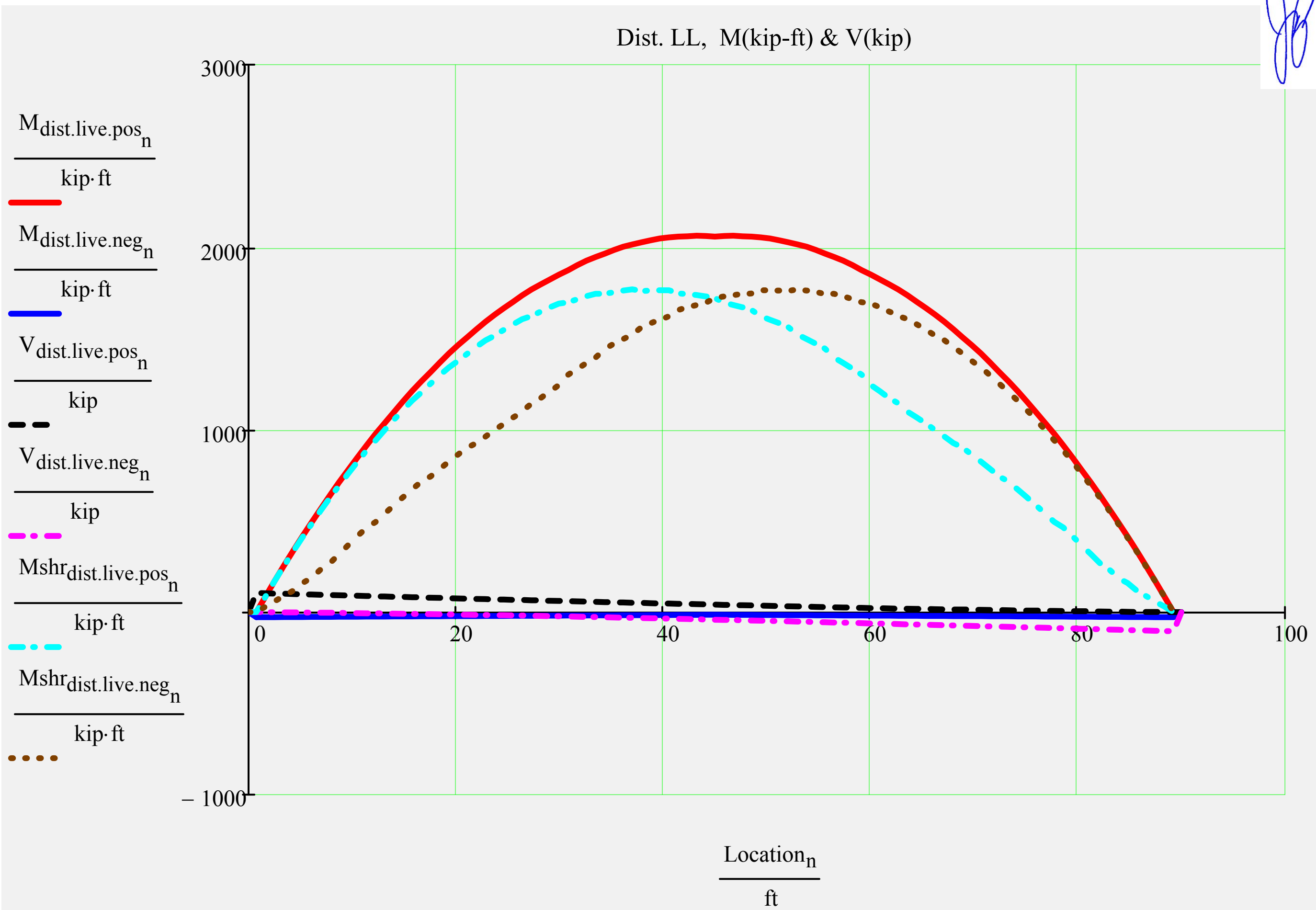
Composite Dead Load Moments and Shear



$\max(M_{dl.comp}) = 117.5 \cdot kip \cdot ft$

$\max(V_{dl.comp}) = 5.3 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 2062.8 \cdot kip \cdot ft$ $\qquad \min(M_{dist.live.neg}) = -27.9 \cdot kip \cdot ft$ $\qquad Reaction_{LL} = 106.4 \cdot kip$

$\max(V_{dist.live.pos}) = 105.1 \cdot kip$ $\qquad \max(Mshr_{dist.live.pos}) = 1768.4 \cdot kip \cdot ft$ $\qquad Reaction_{DL} = 85.35 \cdot kip$

Prestress Strand Layout Input

Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

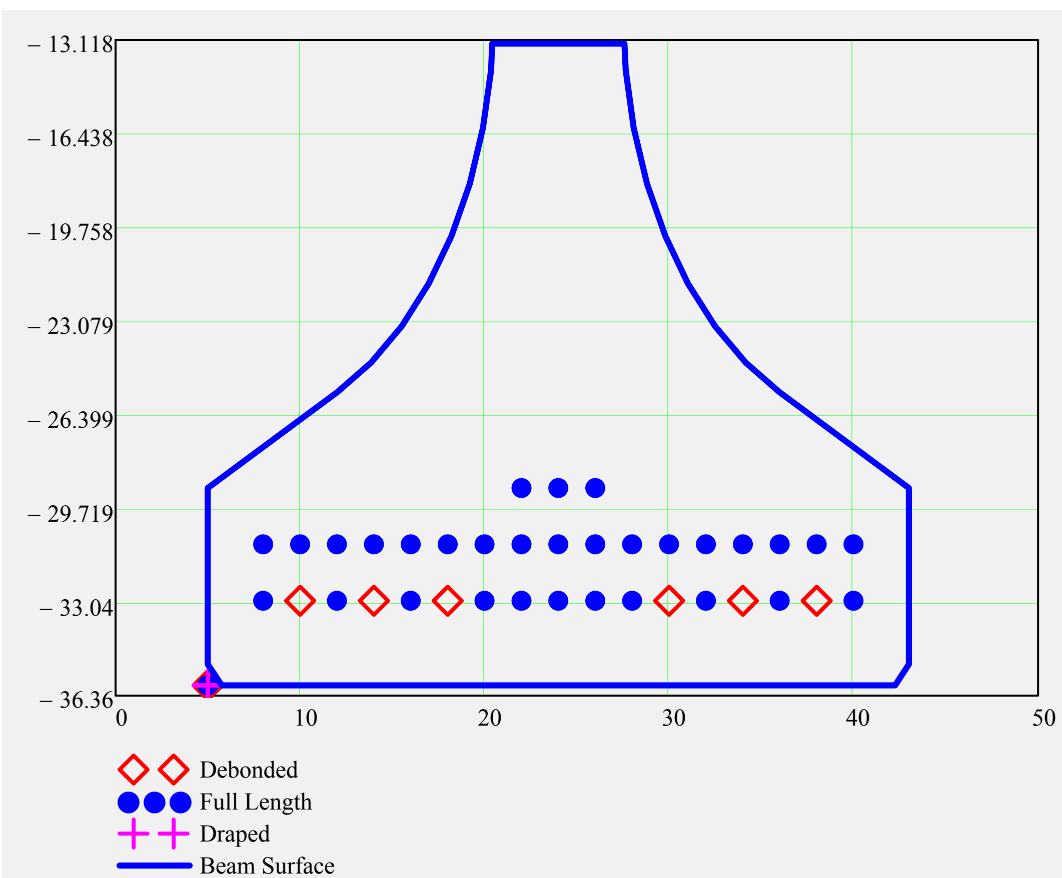
StrandTemplate := Standard / Custom

Strand Pattern Generator:

Collapsed Region for Custom Strand Sizes...

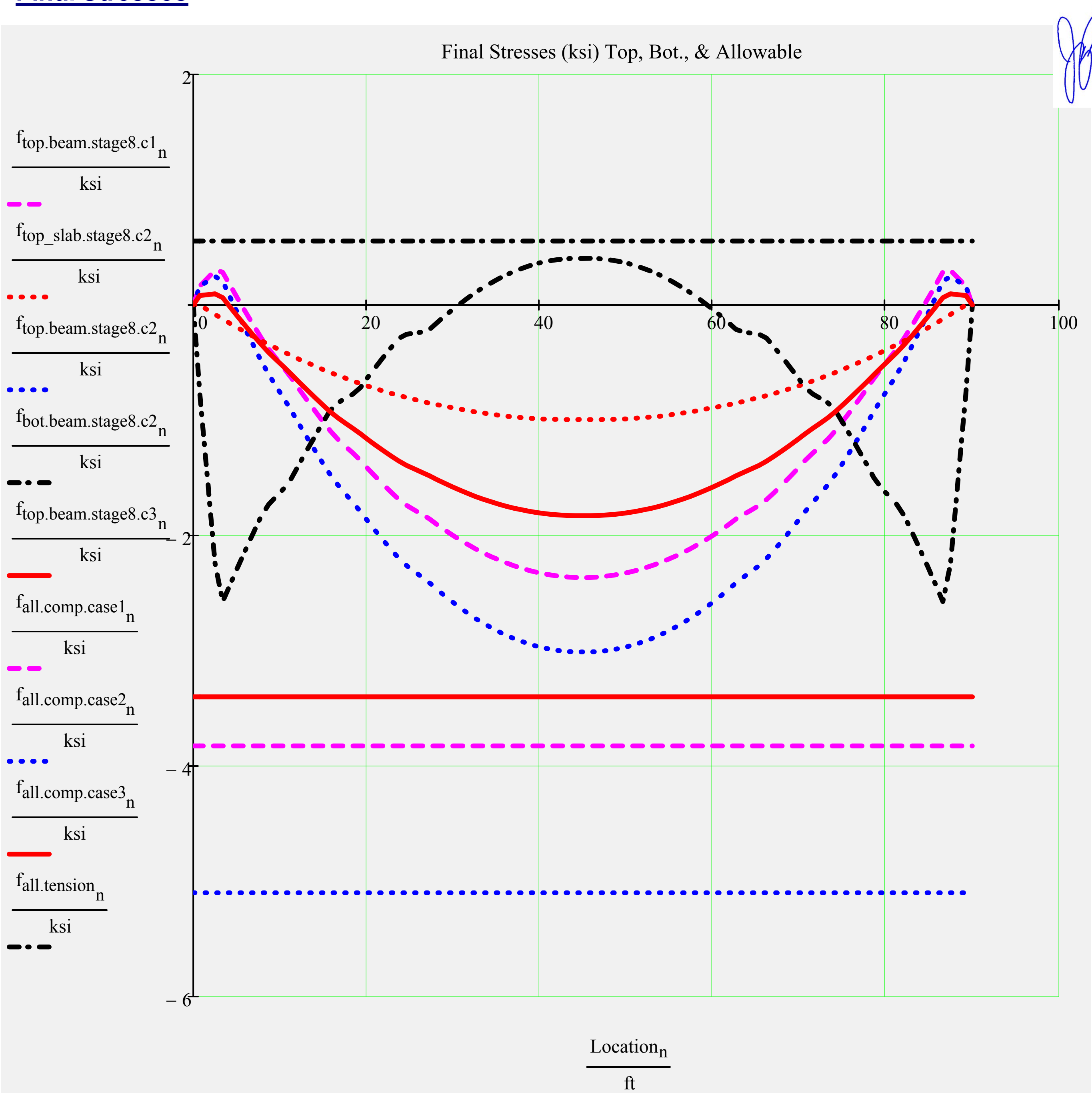
CheckPattern_0 = "OK"	***check 0*** *- no debonded tendon in outside row*
CheckPattern_1 = "OK"	***check 1*** *- less than 25% debonded tendons total*
CheckPattern_2 = "OK"	***check 2*** *- less than 40% debonded tendons in any row*
CheckPattern_3 = "OK"	***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*
CheckPattern_4 = "OK"	***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

Tendon Layout



Release Stresses

Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.79$ — $Check\_f_{tension.rel}$ = "OK" — (Release tension)

$\min(CR\_f_{comp.rel}) = 1.09$ — $Check\_f_{comp.rel}$ = "OK" — (Release compression)

$\min(CR\_f_{tension.stage8}) = 1.37$ — $Check\_f_{tension.stage8}$ = "OK" — (Service III , PS + DL +LL*0.8)

$\min(CR\_f_{comp.stage8.c1}) = 1.62$ — $Check\_f_{comp.stage8.c1}$ = "OK" — (Service I , PS + DL)

$\min(CR\_f_{comp.stage8.c2}) = 1.69$ — $Check\_f_{comp.stage8.c2}$ = "OK" — (Service I , PS + DL +LL)

$\min(CR\_f_{comp.stage8.c3}) = 1.86$ — $Check\_f_{comp.stage8.c3}$ = "OK" — (Service I , (PS + DL)*0.5 +LL)

Section and Strand Properties Summary

$A_{beam} = 807.4 \cdot in^2$ — Concrete area of beam

$I_{beam} = 127557.7893 \cdot in^4$ — Gross Moment of Inertia of Beam about CG

$y_{comp} = -9.12 \cdot in$ — Dist. from top of beam to CG of gross composite section

$I_{comp} = 337866.8867 \cdot in^4$ — Gross Moment of Inertia Composite Section about CG

$A_{deck} = 591.14 \cdot in^2$ — Concrete area of deck slab

$A_{ps} = 8 \cdot in^2$ — total area of strands

$d_{b.ps} = 0.6 \cdot in$ — diameter of Prestressing strand

$\min(PrestressType) = 0$ — 0 - low lax 1 - stress relieved

$f_{py} = 243 \cdot ksi$ — tendon yield strength

$f_{pj} = 203 \cdot ksi$ — prestress jacking stress

$L_{shielding}{}^T = (8 \;\; 16 \;\; 24 \;\; 0 \;\; 0 \;\; 0) \cdot ft$

$A_{ps.row}{}^T = (0.4 \;\; 0.4 \;\; 0.4 \;\; 2.4 \;\; 3.7 \;\; 0.7) \cdot in^2$

$d_{ps.row} =$ (·in)

	0	1	2	3	4	5	6	7	8	9
0	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
1	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
2	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
3	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
4	-31	-31	-31	-31	-31	-31	-31	-31	-31	-31
5	-29	-29	-29	-29	-29	-29	-29	-29	-29	...

TotalNumberOfTendons = 37

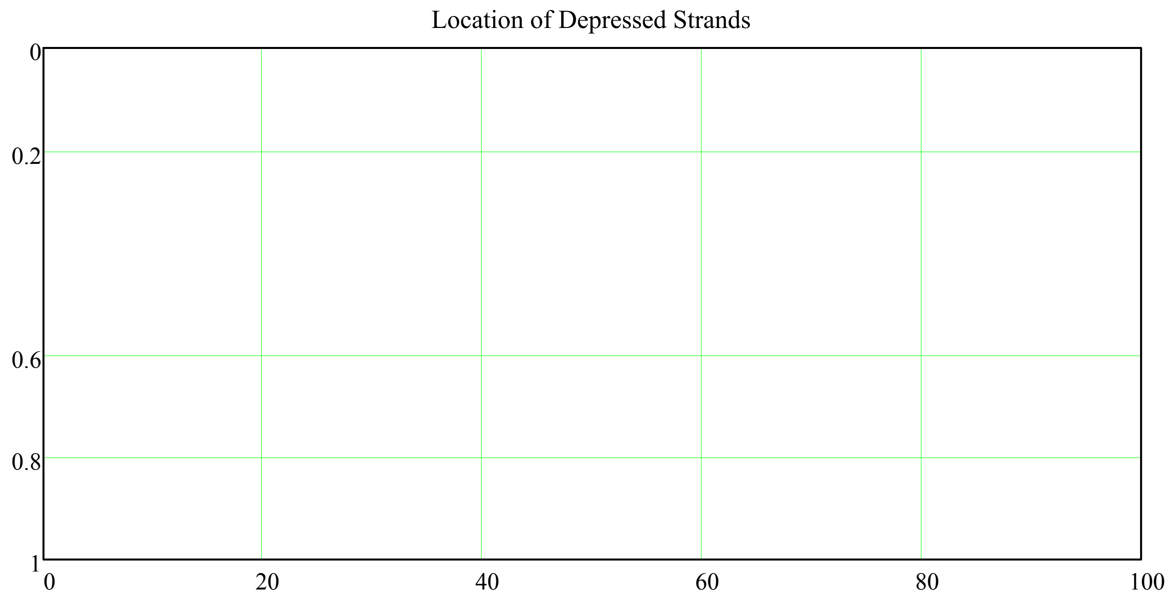
NumberOfDebondedTendons = 6

NumberOfDrapedTendons = 0

StrandSize = "0.6 in low lax"

$StrandArea = 0.22 \cdot in^2$

$JackingForce_{per.strand} = 43.94 \cdot kip$



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$

$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

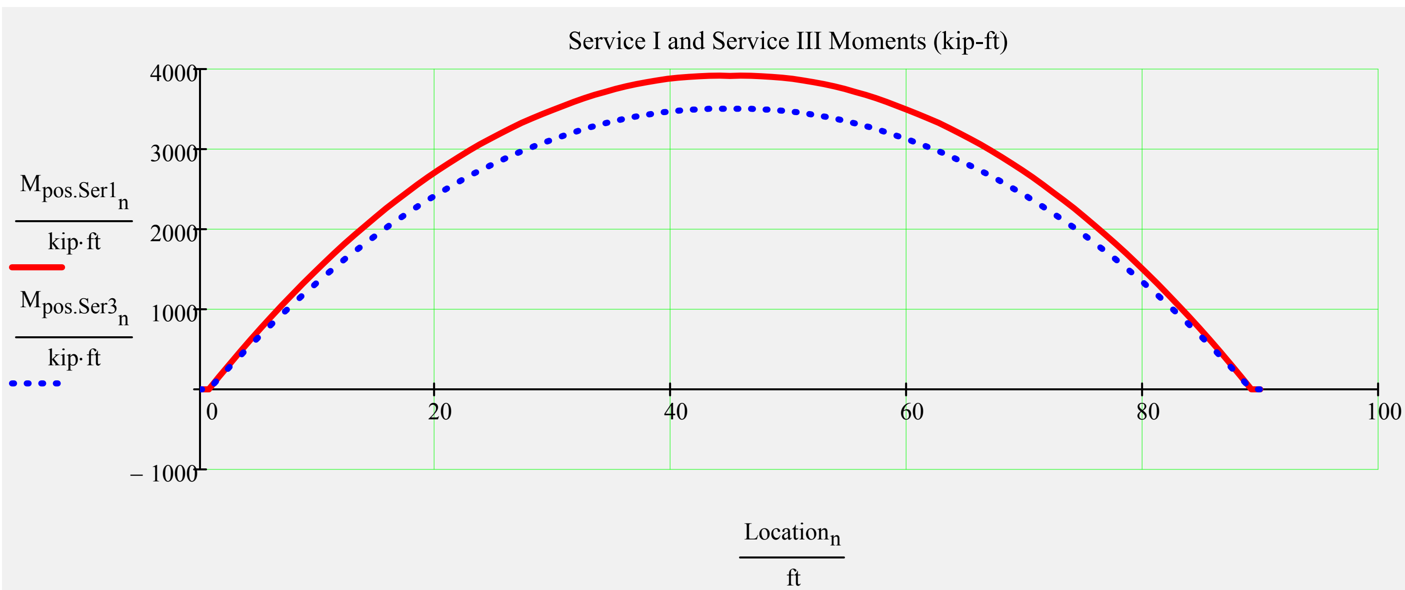
$f_{pe} = 178 \cdot ksi$ $\Delta f_{pTot} = -24 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.95 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.05 \cdot \%$

Check_f_pt = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f_pe = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 3917 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 3504.7 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.61	−2.9
2	−0.72	−2.41
4	−0.67	−2.45
6	−2.33	−1.14
8	−3.01	0.4

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−1675.9
"Final (about composite centroid)"	−1431.5	−1392.1

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	799.37	126328.79	−19.39
"Transformed Beam (initial)"	856.4	134564.33	−20.21
"Transformed Beam "	847.29	133321.64	−20.09
"Composite "	1462.69	363352.17	−9.49

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	851.6
"Non-composite (includes bm wt.)"	1739
"Composite"	117.5
"Distributed Live Load"	2058

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

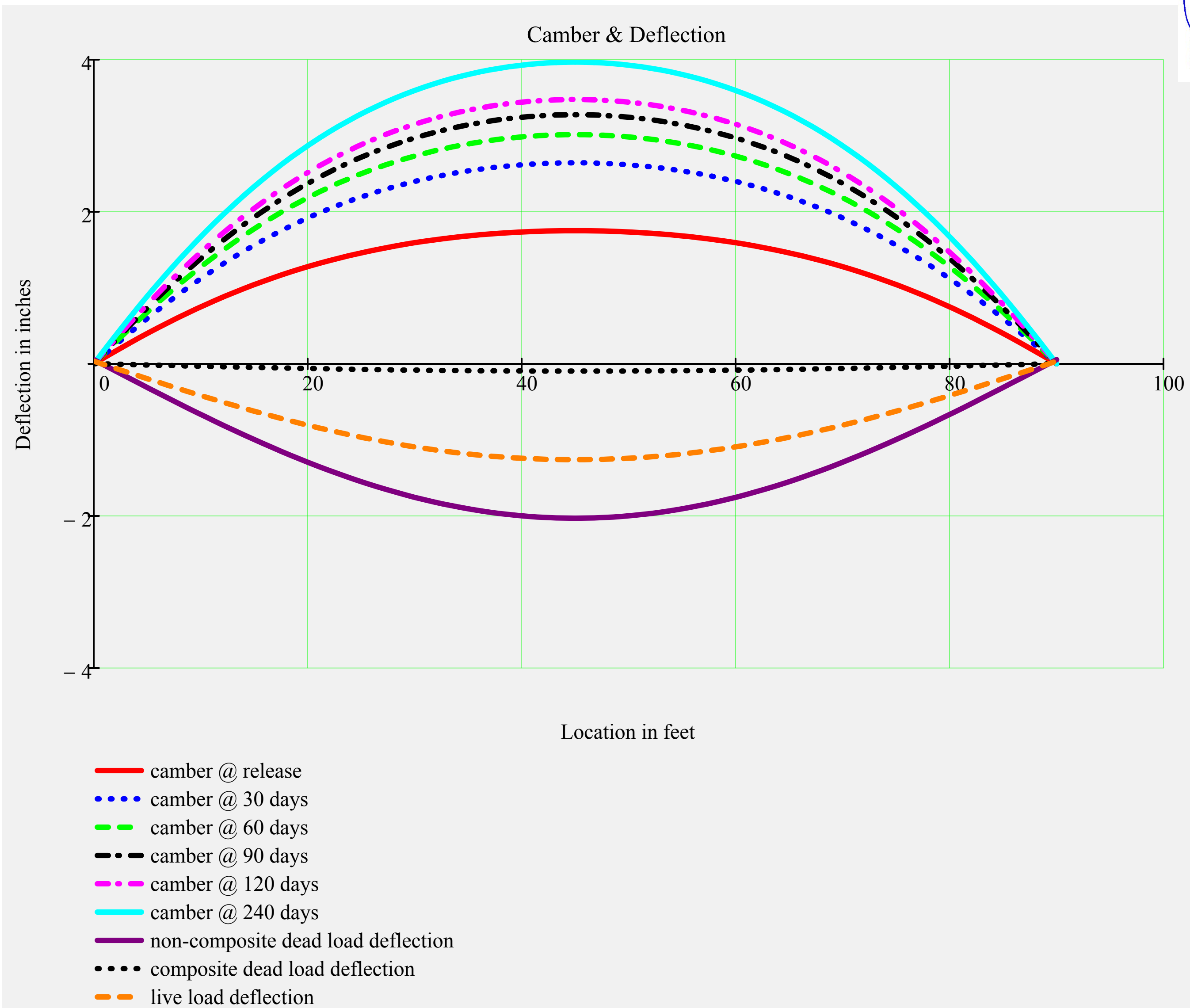
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

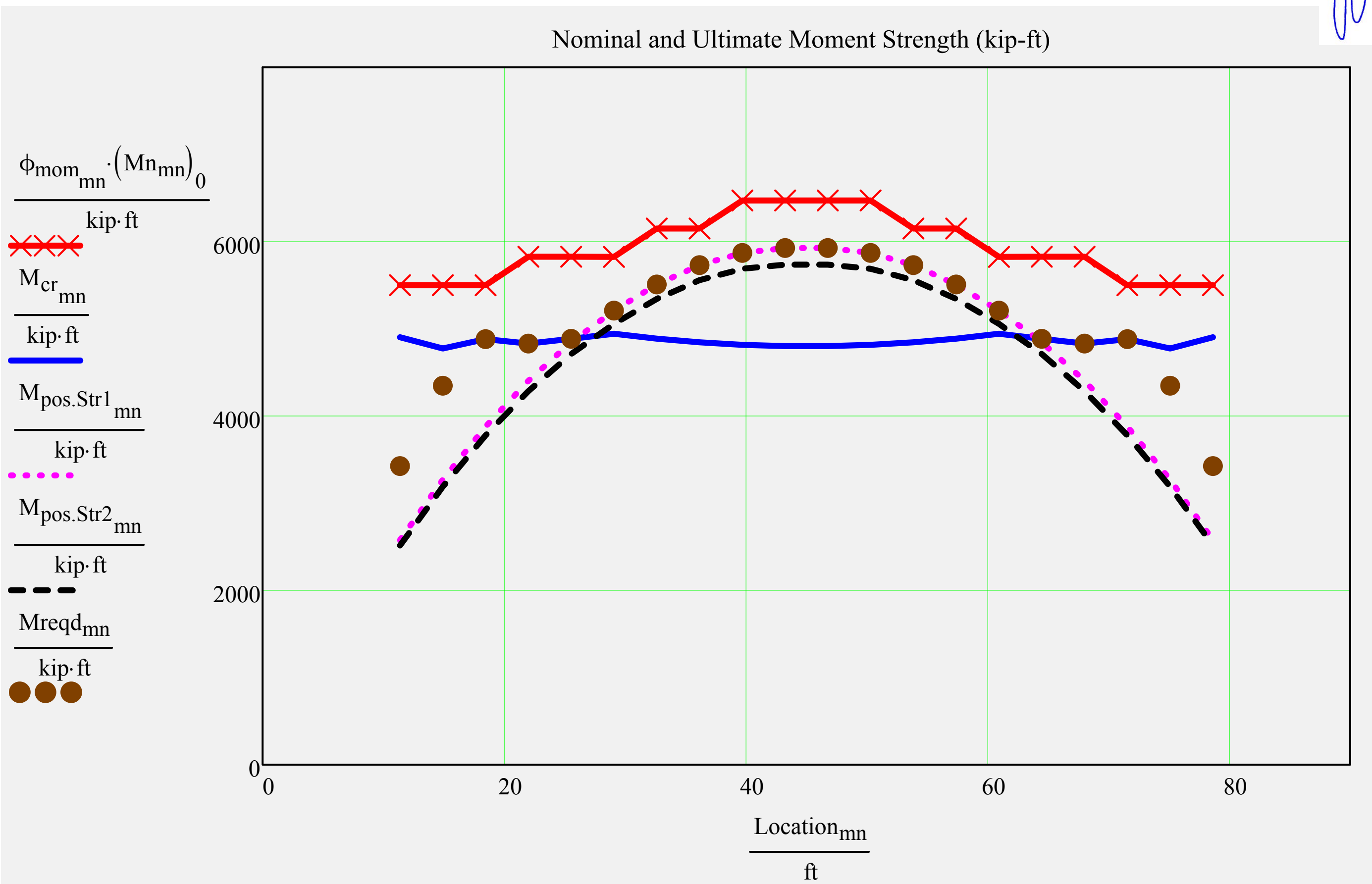
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	−0.0497	−0.8058	0.4274	1.7515
"30 Days"	−0.2398	−1.4277	0.7048	2.6438
"60 Days"	−0.3078	−1.6502	0.8132	3.0143
"90 Days"	−0.3441	−1.7688	0.871	3.2764
"120 Days"	−0.3666	−1.8425	0.9069	3.4763
"240 Days"	−0.4082	−1.9786	0.9732	3.9676
"non-comp DL"	−0.2456	0.195	−0.3508	−2.0296
"comp DL"	−0.0055	0.0153	−0.0165	−0.0955
"LL"	−0.0725	0.2033	−0.2197	−1.2604

Strength Limit State Moments



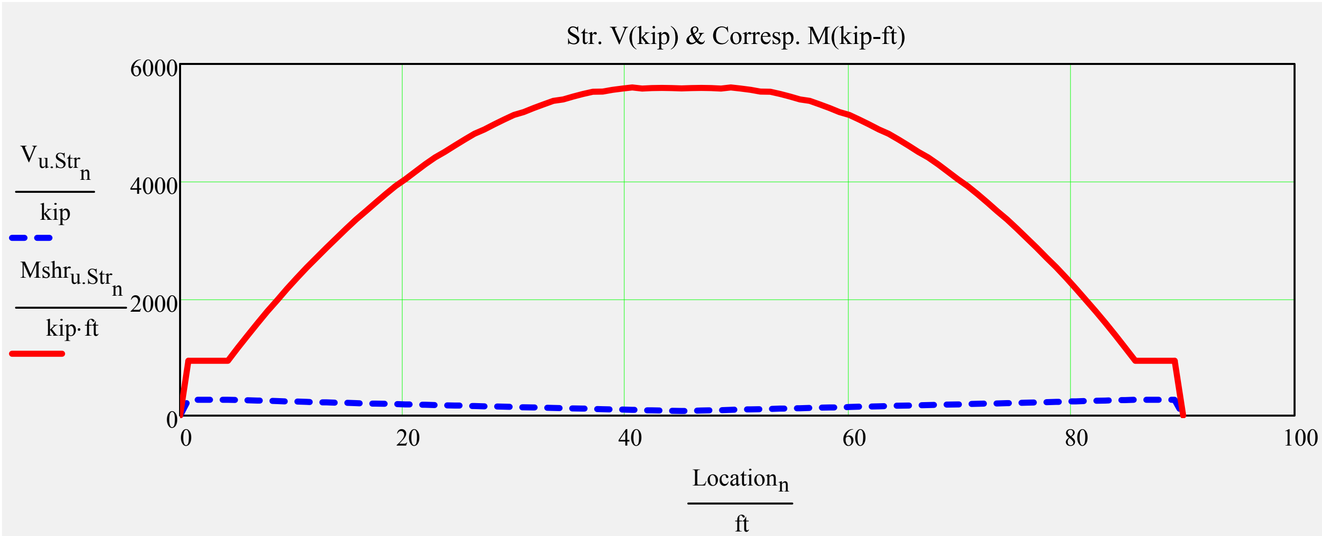
$CR_{Str.mom_n} := 10$ $\quad CR_{Str.mom_{mn}} := \dfrac{\phi_{mom_{mn}} \cdot \left(Mn_{mn}\right)_0}{Mreqd_{mn}}$ *(LRFD 5.7.3.3.2)*

$\min\left(CR_{Str.mom}\right) = 1.07$

$\max(Mreqd) = 5926.8 \cdot kip \cdot ft$

CheckMomentCapacity := if(min($CR_{Str.mom}$) > 0.99, "OK", "No Good!") CheckMomentCapacity = "OK"

Strength Shear and Associated Moments



$max(V_{u.Str}) = 270.7 \cdot kip$ $max(Mshr_{u.Str}) = 5599.1 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing	Number of Spaces	area per stirrup
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$tmp\_s = \begin{pmatrix} 3.5 \\ 3.5 \\ 3 \\ 6 \\ 12 \\ 12 \\ 12 \end{pmatrix} \cdot in$ $tmp\_NumberSpaces = \begin{pmatrix} 2 \\ 2 \\ 16 \\ 50 \\ 3 \\ 3 \\ 0 \end{pmatrix}$ $tmp\_A_{stirrup} = \begin{pmatrix} 1.24 \\ 0.62 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \\ 0.31 \end{pmatrix} \cdot in^2$

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

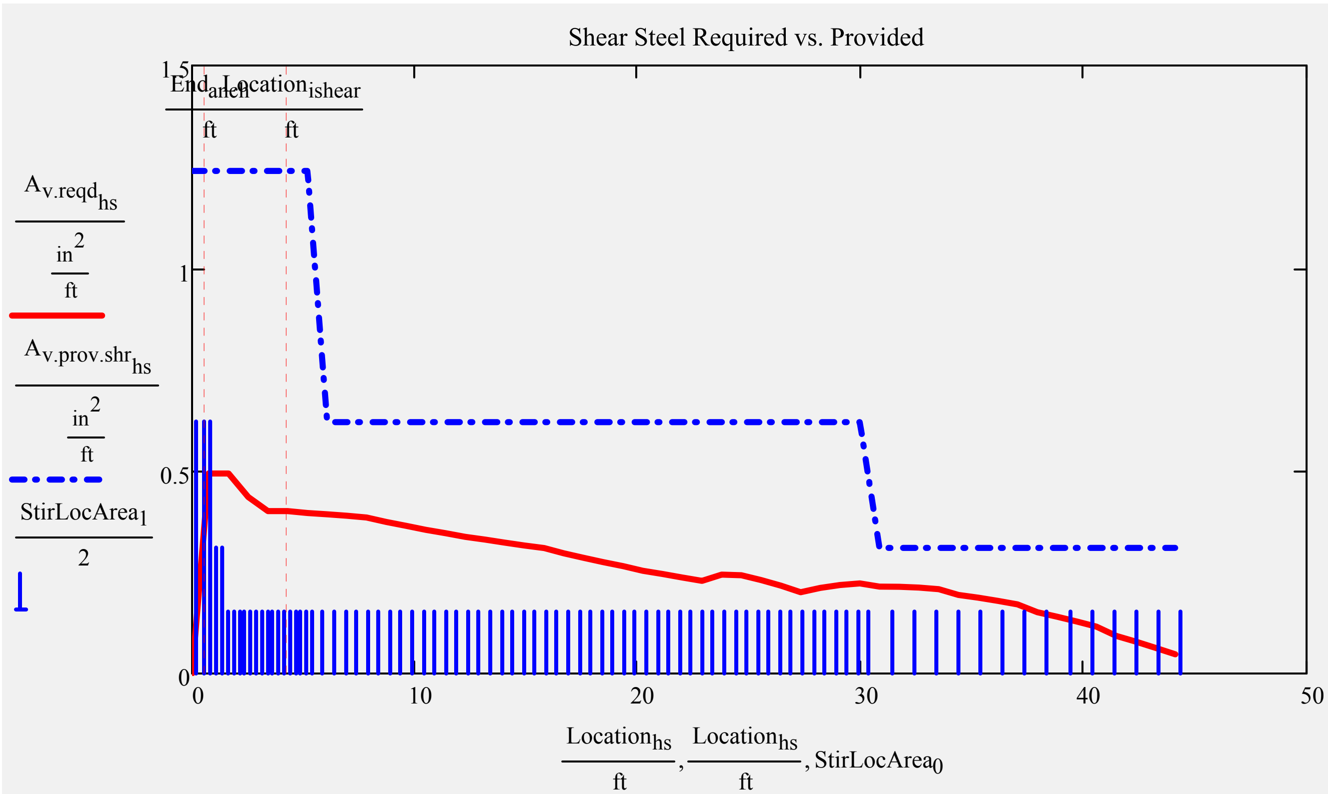
	$user\_s_{nspacings} :=$	$user\_NumberSpaces_{nspacings} :=$	$user\_A_{stirrup_{nspacings}} :=$	$interface\_factor_{nspacings} :=$
A1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.25
A2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	0.5
A3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S1 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S2 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S3 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
S4 stirrup	$-1 \cdot in$	-1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

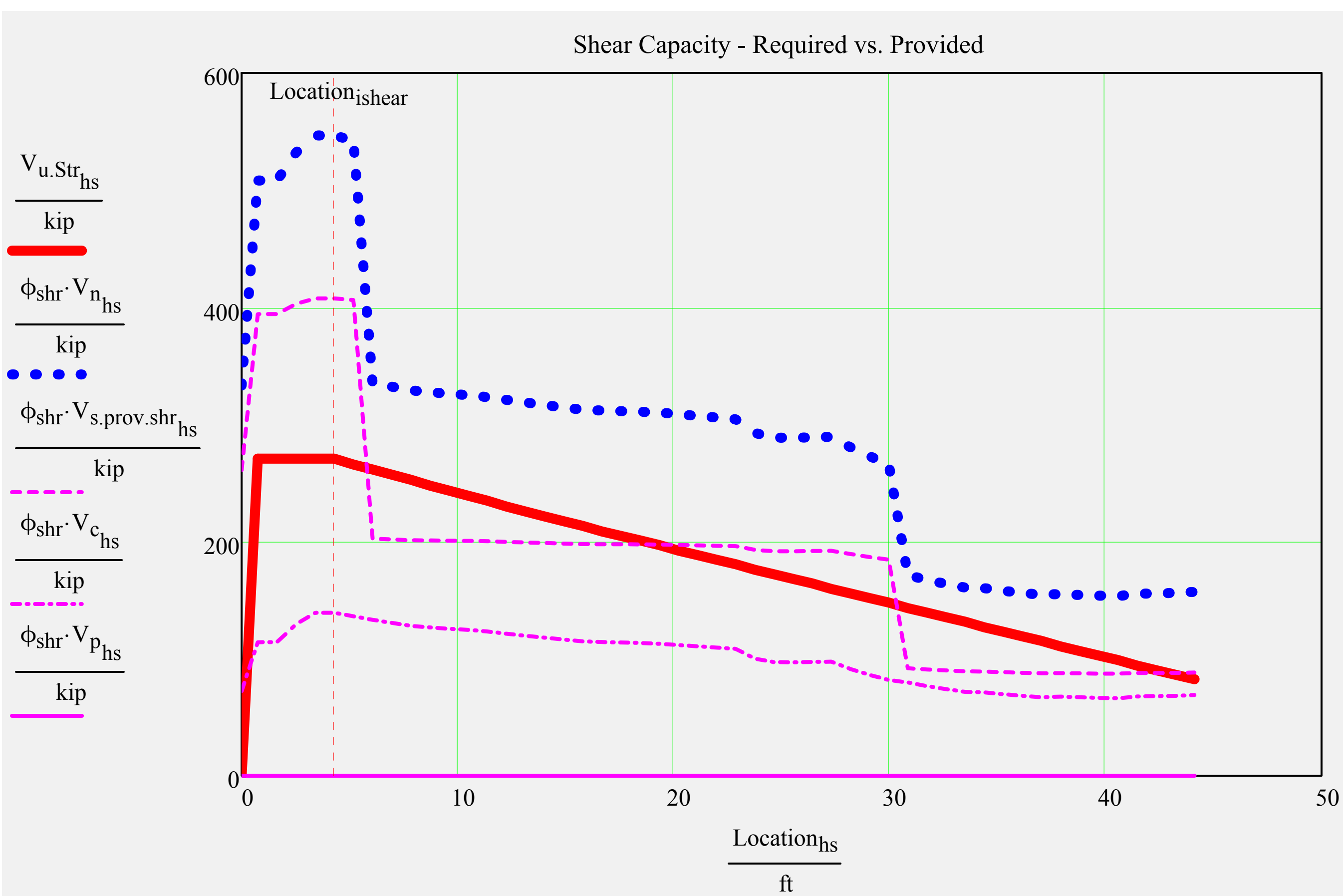
Stirrup sizes and spacings used in analysis

The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s (in)	NumberSpaces	$A_{stirrup}$ (in^2)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

$EndCover = 2.5 \cdot in$


Shear Steel Required vs. Provided
End_arch/ft
Location_ishear/ft
A_v.reqd_hs / (in^2/ft)
A_v.prov.shr_hs / (in^2/ft)
StirLocArea_1 / 2
Location_hs/ft, Location_hs/ft, StirLocArea_0
0
0.5
1
1.5
0
10
20
30
40
50

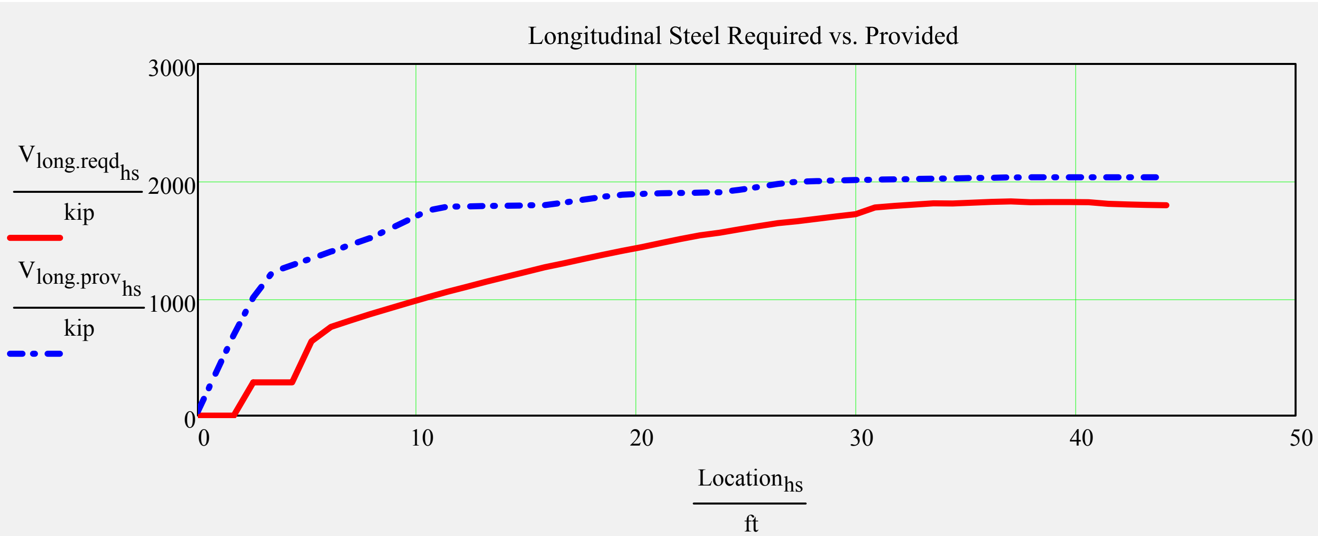

Shear Capacity - Required vs. Provided
Location_ishear
V_u.Str_hs / kip
ϕ_shr·V_n_hs / kip
ϕ_shr·V_s.prov.shr_hs / kip
ϕ_shr·V_c_hs / kip
ϕ_shr·V_p_hs / kip
Location_hs / ft
0
200
400
600
0
10
20
30
40
50

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

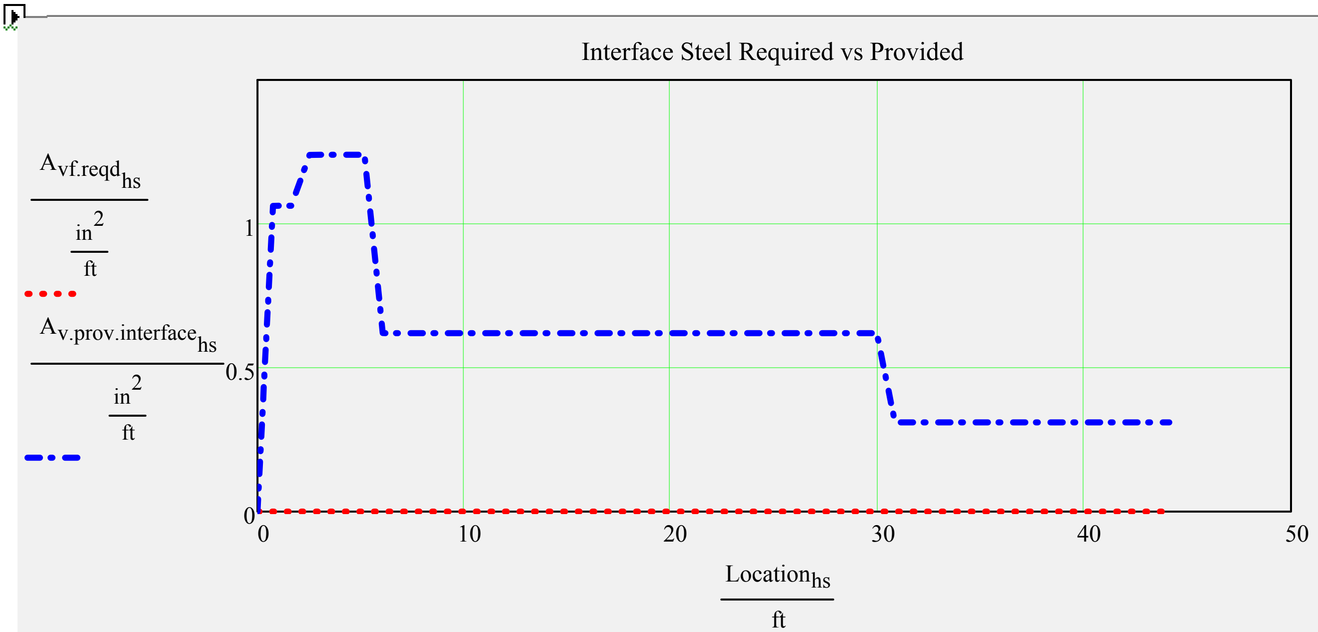


$$CR_{LongSteel_{hs}} := \text{if}\left(V_{long.reqd_{hs}} < .01\text{kip}, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$$

$$\min(CR_{LongSteel}) = 1.11$$

CheckLongSteel := if(min($CR_{LongSteel}$) > 1 ,"OK" ,"No Good, add steel!")

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, "OK", "No Good"\right)$$

CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A." CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0 := AcceptAASHTO$

$check_1 := AcceptSDG$

$check_2 := AcceptOntario$

$check_3 := Check\_f_{pt}$

$check_4 := Check\_f_{pe}$

$check_5 := Check\_f_{tension.rel}$

$check_6 := Check\_f_{comp.rel}$

$check_7 := Check\_f_{tension.stage8}$

$check_8 := Check\_f_{comp.stage8.c1}$

$check_9 := Check\_f_{comp.stage8.c2}$

$check_{10} := Check\_f_{comp.stage8.c3}$

$check_{11} := CheckMomentCapacity$

$check_{12} := CheckMaxCapacity$

$check_{13} := CheckStirArea$

$check_{14} := CheckShearCapacity$

$check_{15} := CheckMinStirArea$

$check_{16} := CheckMaxStirSpacing$

$check_{17} := CheckLongSteel$

$check_{18} := CheckInterfaceSpacing$

$check_{19} := CheckSplittingSteel$

$check_{20} := CheckMaxPrestressingForce$

$check_{21} := CheckPattern_0$

$check_{22} := CheckPattern_1$

$check_{23} := CheckPattern_2$

$check_{24} := CheckPattern_3$

$check_{25} := CheckPattern_4$

$check_{26} := CheckInterfaceSteel$

$check_{27} := CheckStrandFit$

$check_{28} := Check\_SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T =$

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8:
FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

$LRFR_{loadrating} =$

	Moment (Strength) or Stress (Service)				Shear (Strength)				
"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.87	1.12	"N/A"	53.10	0.92	1.42	"N/A"	30.09	HL-93
"Strength I(Op)"	0.87	1.45	"N/A"	53.10	0.92	1.84	"N/A"	30.09	HL-93
"Service III(Inv)"	0.87	1.10	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.87	1.20	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.87	1.18	70.69	53.10	0.92	1.26	75.50	30.09	*Permit
"Service III"	0.87	1.12	66.97	45.13	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.15$ $CR_{LongSteel.Permit} = 1.11$ $CheckLongSteel_{loadrating} =$ "OK"

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltB_ 90'_FIB-36_NE

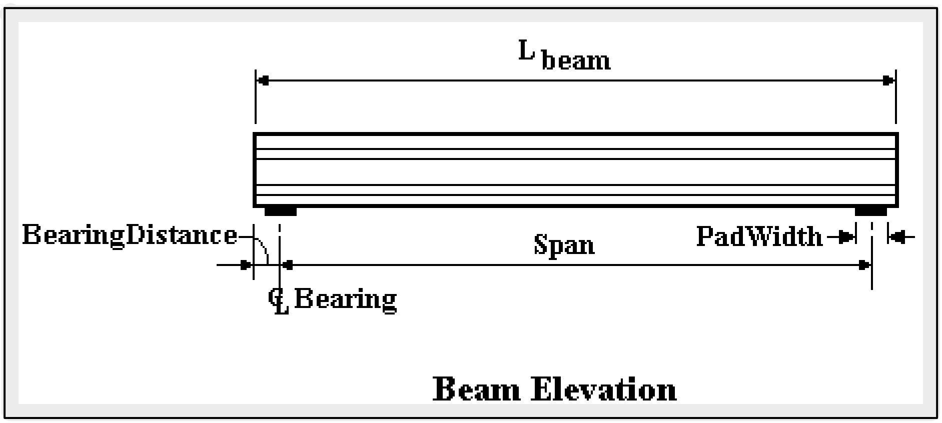
Comment = "Alt. B - NB Interior"

Legend

TanHighlight = DataEntry | YellowHighlight = CheckValues | GreyHighlight = UserComments + Graphs

BlackText = ProgramEquations | *Maroon Text = Code Reference* | *Blue Text = Commentary*

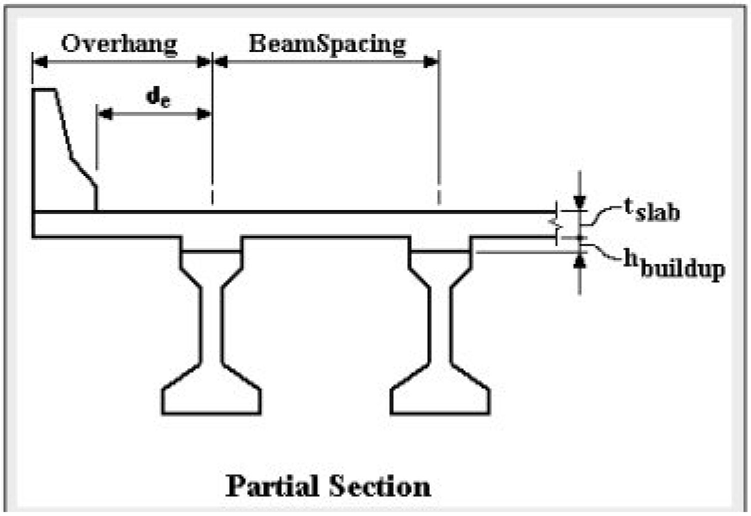
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ Span = 88.5·ft BearingDistance = 9·in PadWidth = 10·in

BeamTypeTog = "FIB36"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



Overhang = 4.54·ft BeamSpacing = 8.5·ft t_{slab} = 8·in $h_{buildup}$ = 1.5·in

Skew = 20·deg $t_{integral.ws}$ = 0.5·in NumberOfBeams = 5 $t_{slab.delta}$ = 0·in

de = 2.75 ft

BeamPosition = "interior" *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed" *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

b_e = 8.5 ft *effective slab width* *LRFD 4.6.2.6*

$t_{slab} := \text{if}(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

SectTog := if(SectionType = "transformed", 1, 0) SectTog = 1

Concrete:

Corrosion Classification Environment = "moderately"

density of slab concrete $\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$

strength of slab concrete $f_{c.slab} = 4.5 \cdot ksi$

density of beam concrete $\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$

strength of beam concrete $f_{c.beam} = 8.5 \cdot ksi$

weight of future wearing surface $Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$

release beam strength $f_{ci.beam} = 6 \cdot ksi$

type of course aggregate, "Florida" *or* "Standard" AggregateType = "Florida"

relative humidity H = 75

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ *used in distribution calculation* $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9 \cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor \cdot \sqrt{f_{ci.beam} \cdot ksi}$ *initial beam concrete modulus of elasticity(LRFD 5.4.2.4)* $E_{ci} = 4012 \cdot ksi$

$E_c := AggFactor \cdot \sqrt{f_{c.beam} \cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776 \cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270 \cdot ksi$

tendon modulus of elasticity $E_p = 28500 \cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60 \cdot ksi$

mild steel modulus of elasticity $E_s = 29000 \cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62 \cdot \frac{in^2}{ft}$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0 \cdot in^2$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4 \cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0 \cdot in$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \quad 53.33 \quad 53.33) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0) \cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.69$ $tmp\_g_{shear} = 0.9$

user value overrides (optional):

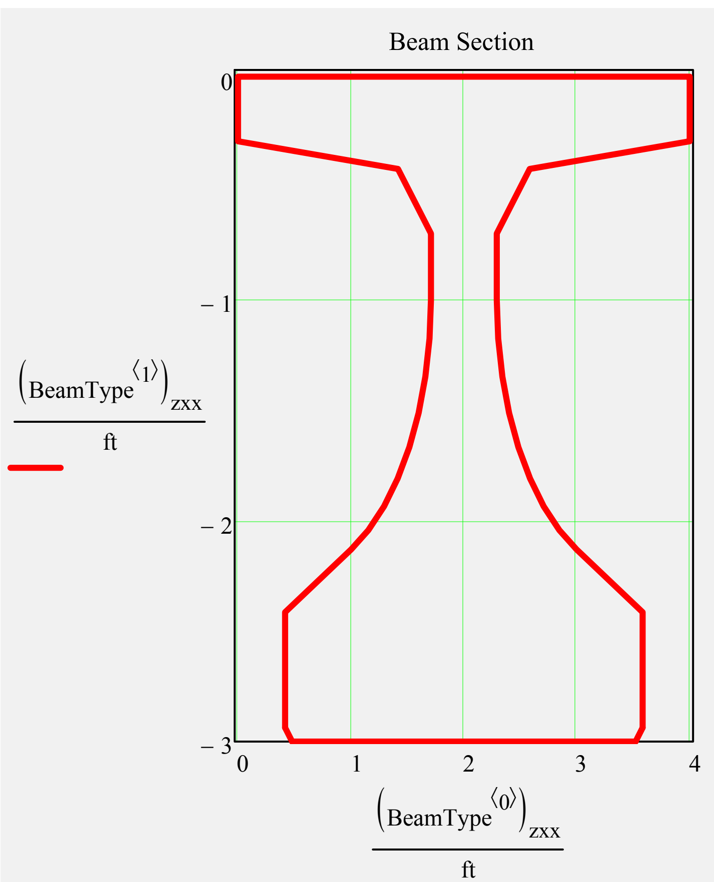
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

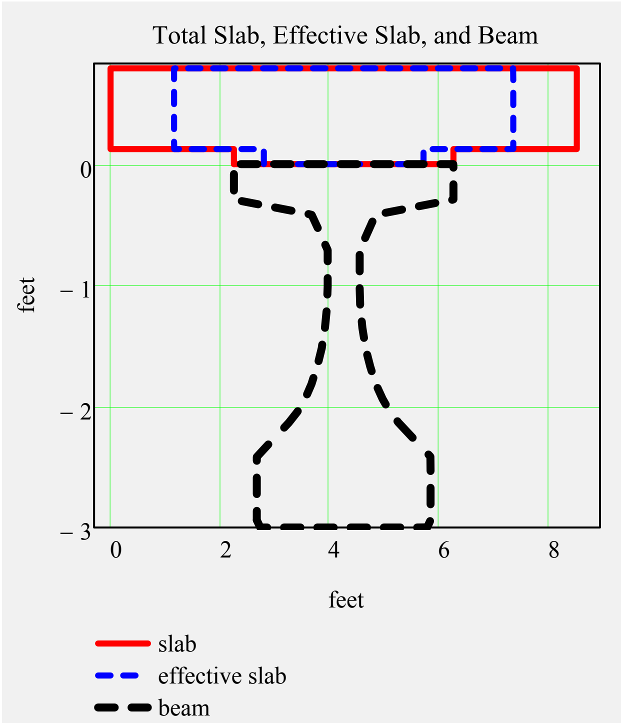
value check

$g_{mom} := \text{if}(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.69$

$g_{shear} := \text{if}(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 0.9$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 0.978 \cdot \frac{kip}{ft}$ $w_{beam} = 0.841 \cdot \frac{kip}{ft}$ $w_{forms} = 0.09 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$ *additional non composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 1.909 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 1.068 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

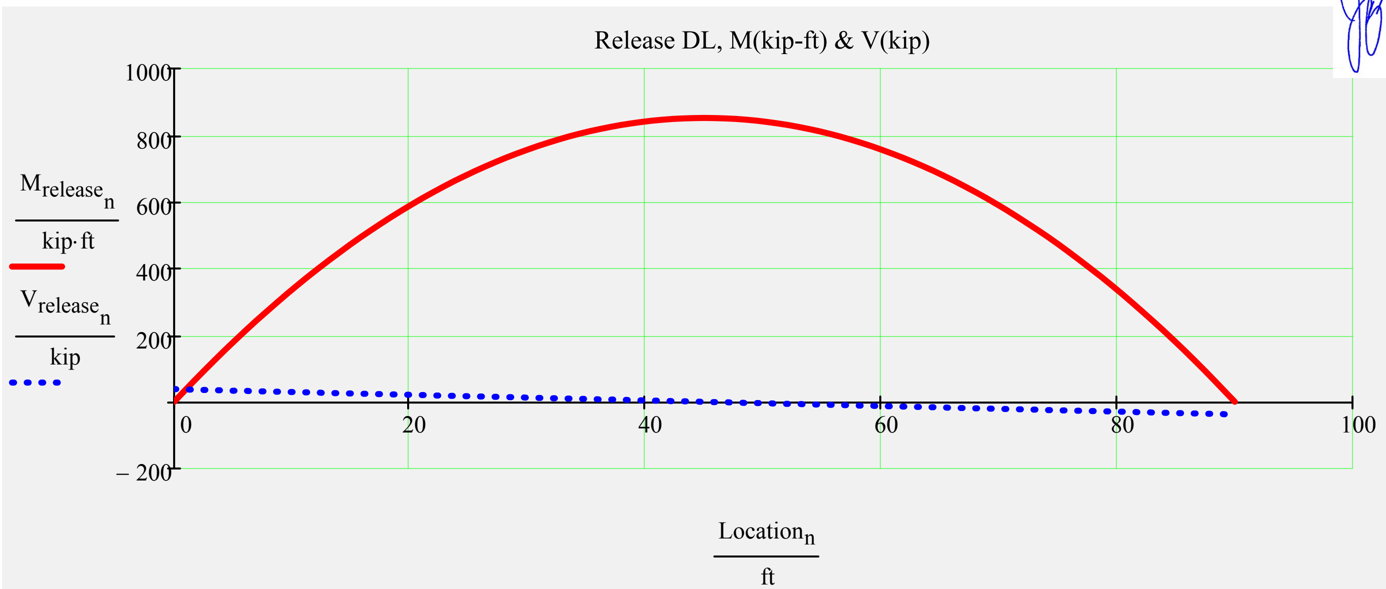
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.168 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.168 \cdot \frac{kip}{ft}$

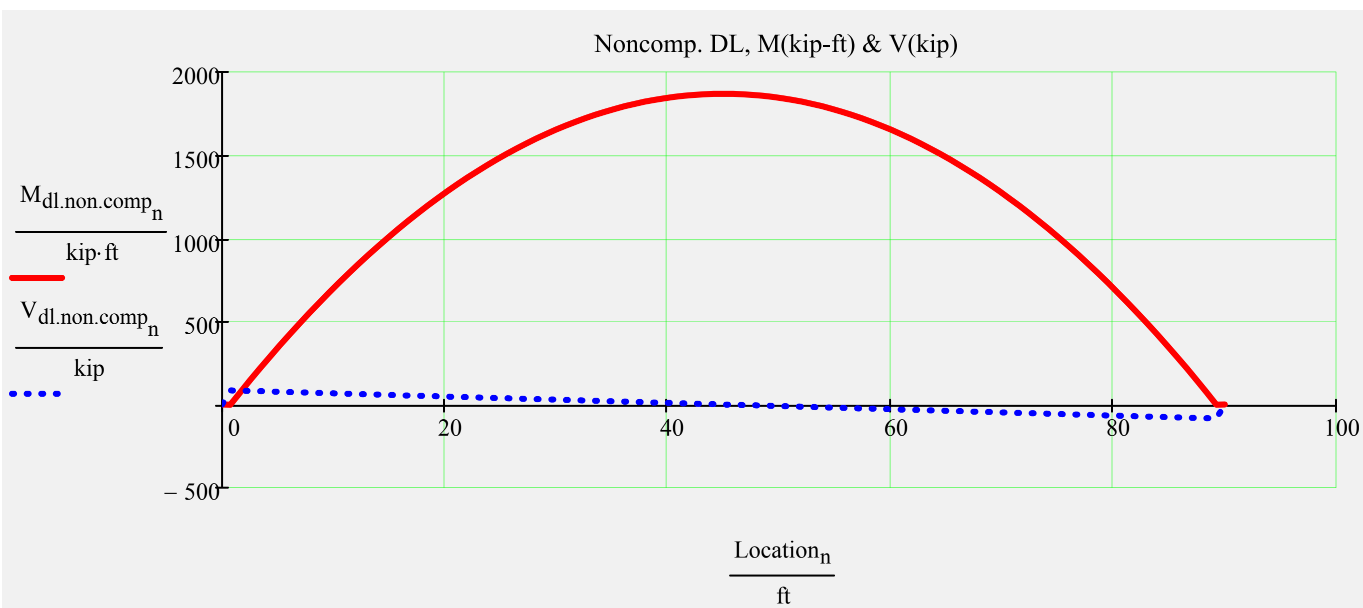
$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.168 \cdot \frac{kip}{ft}$

Release Dead Load Moments and Shear



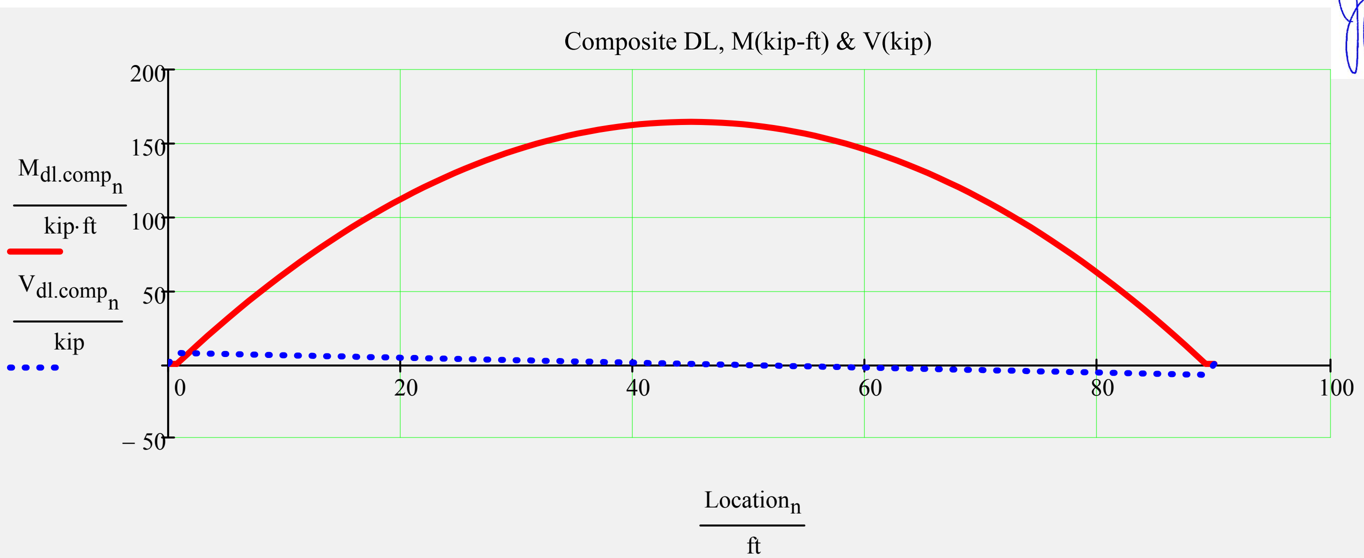
$\max(M_{release}) = 851.6 \cdot kip \cdot ft$ $\max(V_{release}) = 37.8 \cdot kip$

Noncomposite Dead Load Moments and Shear



$\max(M_{dl.non.comp}) = 1868.6 \cdot kip \cdot ft$ $\max(V_{dl.non.comp}) = 84.5 \cdot kip$

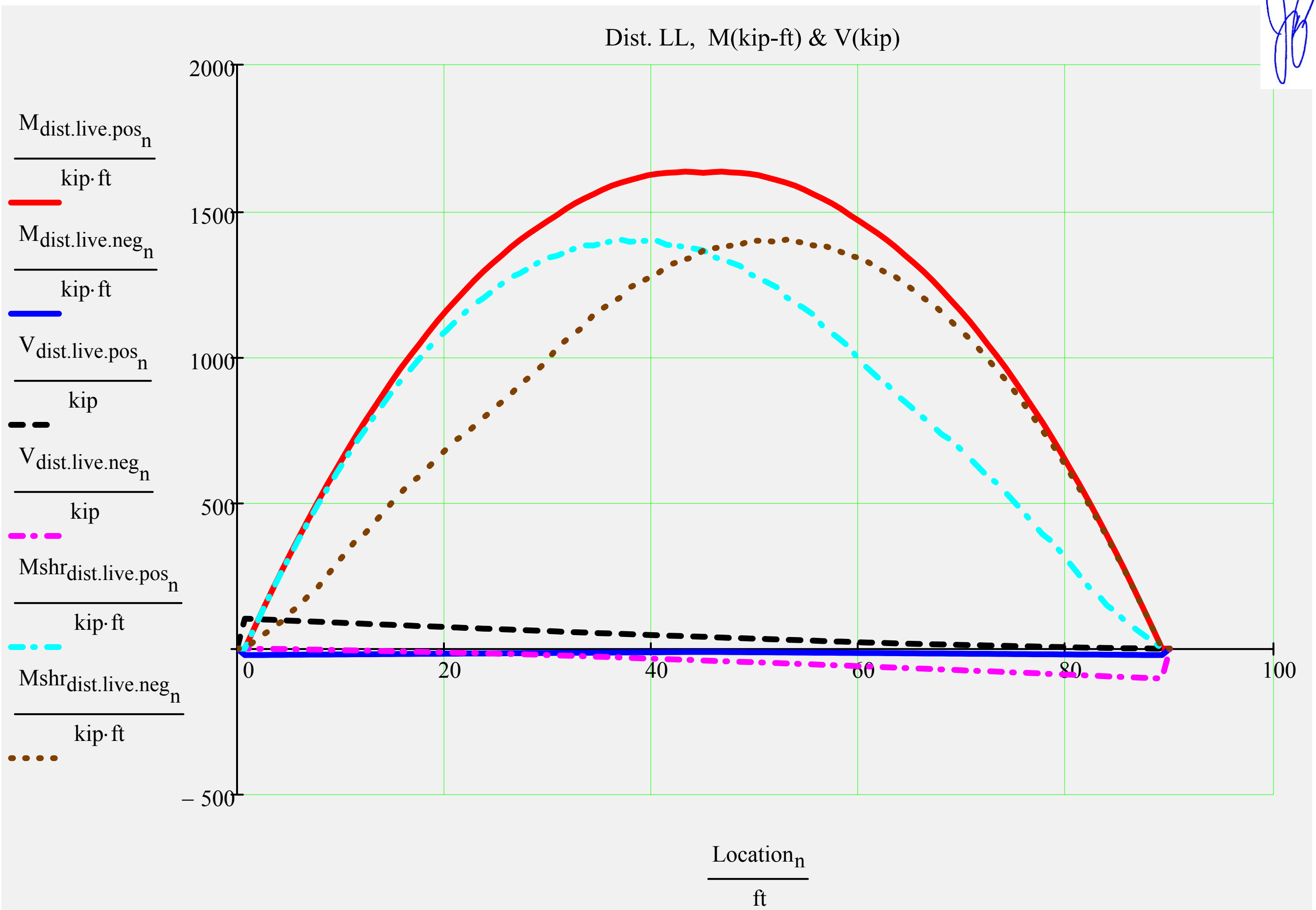
Composite Dead Load Moments and Shear



$max(M_{dl.comp}) = 164.4 \cdot kip \cdot ft$

$max(V_{dl.comp}) = 7.4 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 1633.5 \cdot kip \cdot ft$

$\min(M_{dist.live.neg}) = -22.1 \cdot kip \cdot ft$

$Reaction_{LL} = 104.15 \cdot kip$

$\max(V_{dist.live.pos}) = 102.9 \cdot kip$

$\max(Mshr_{dist.live.pos}) = 1400.4 \cdot kip \cdot ft$

$Reaction_{DL} = 93.47 \cdot kip$

Prestress Strand Layout Input

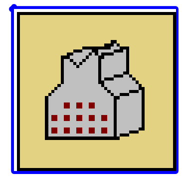
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate := Standard / Custom

Strand Pattern Generator:



Collapsed Region for Custom Strand Sizes...

$CheckPattern_0$ = "OK" — ***check 0*** *- no debonded tendon in outside row*

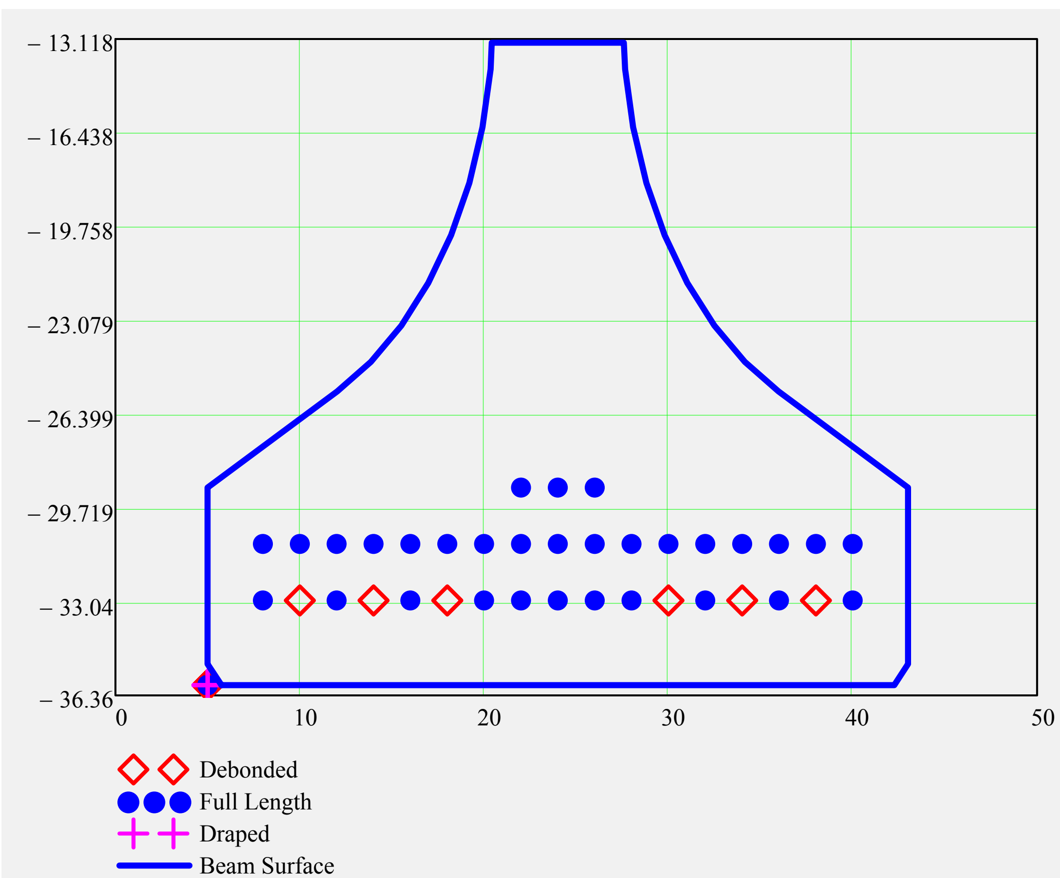
$CheckPattern_1$ = "OK" — ***check 1*** *- less than 25% debonded tendons total*

$CheckPattern_2$ = "OK" — ***check 2*** *- less than 40% debonded tendons in any row*

$CheckPattern_3$ = "OK" — ***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*

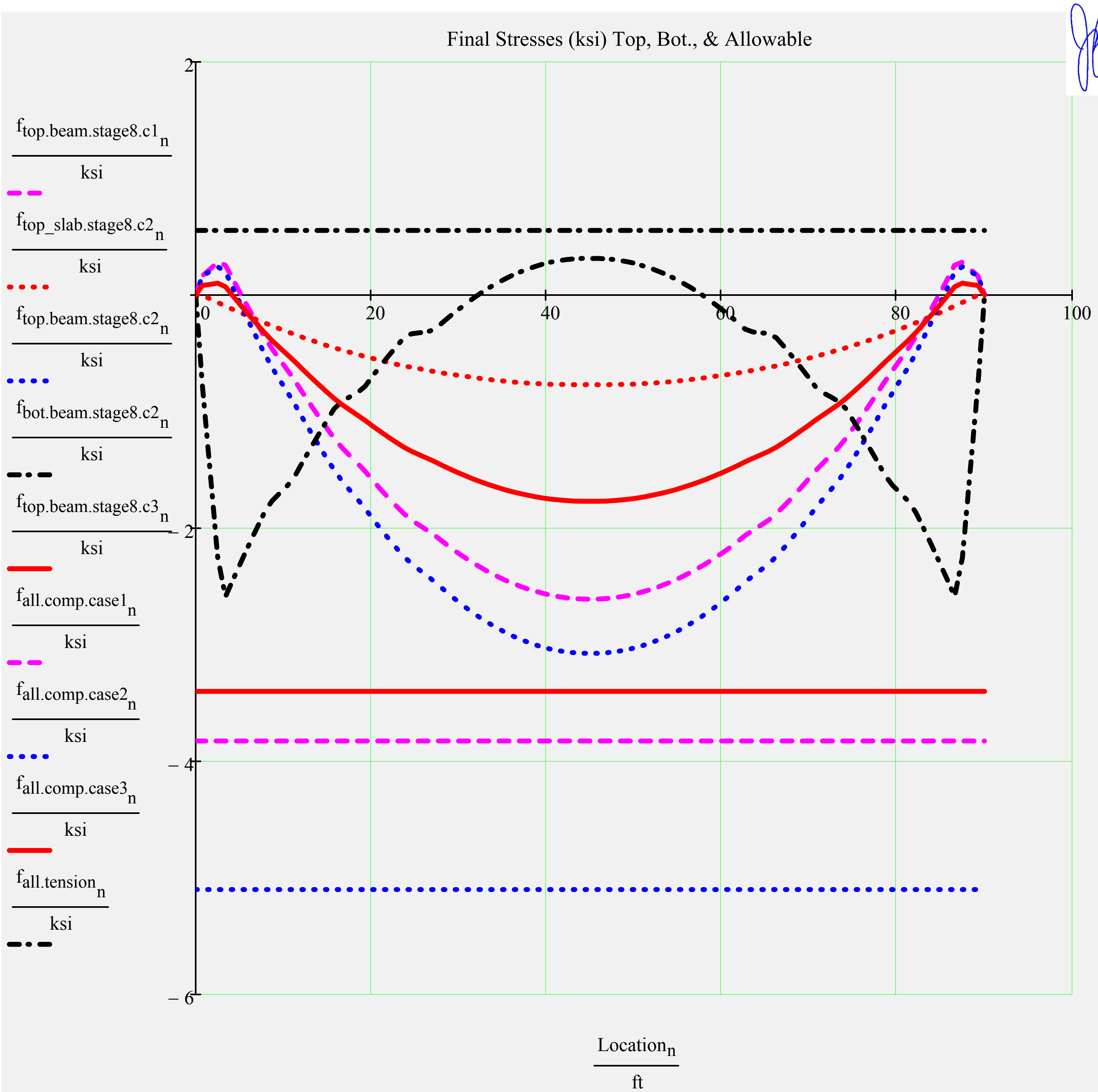
$CheckPattern_4$ = "OK" — ***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

Tendon Layout



Release Stresses

Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.79$ — Check_$f_{tension.rel}$ = "OK" — (Release tension)

$\min(CR\_f_{comp.rel}) = 1.09$ — Check_$f_{comp.rel}$ = "OK" — (Release compression)

$\min(CR\_f_{tension.stage8}) = 1.76$ — Check_$f_{tension.stage8}$ = "OK" — (Service III , PS + DL +LL*0.8)

$\min(CR\_f_{comp.stage8.c1}) = 1.47$ — Check_$f_{comp.stage8.c1}$ = "OK" — (Service I , PS + DL)

$\min(CR\_f_{comp.stage8.c2}) = 1.66$ — Check_$f_{comp.stage8.c2}$ = "OK" — (Service I , PS + DL +LL)

$\min(CR\_f_{comp.stage8.c3}) = 1.92$ — Check_$f_{comp.stage8.c3}$ = "OK" — (Service I , (PS + DL)*0.5 +LL)

Section and Strand Properties Summary

$A_{beam} = 807.4 \cdot in^2$ — Concrete area of beam

$I_{beam} = 127557.7893 \cdot in^4$ — Gross Moment of Inertia of Beam about CG

$y_{comp} = -8.56 \cdot in$ — Dist. from top of beam to CG of gross composite section

$I_{comp} = 349460.7246 \cdot in^4$ — Gross Moment of Inertia Composite Section about CG

$A_{deck} = 646.11 \cdot in^2$ — Concrete area of deck slab

$A_{ps} = 8 \cdot in^2$ — total area of strands

$d_{b.ps} = 0.6 \cdot in$ — diameter of Prestressing strand

min(PrestressType) = 0 — 0 - low lax 1 - stress relieved

$f_{py} = 243 \cdot ksi$ — tendon yield strength

$f_{pj} = 203 \cdot ksi$ — prestress jacking stress

$L_{shielding}^T = (8 \quad 16 \quad 24 \quad 0 \quad 0 \quad 0) \cdot ft$

$A_{ps.row}^T = (0.4 \quad 0.4 \quad 0.4 \quad 2.4 \quad 3.7 \quad 0.7) \cdot in^2$

$d_{ps.row} =$ (·in)

	0	1	2	3	4	5	6	7	8	9
0	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
1	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
2	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
3	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
4	-31	-31	-31	-31	-31	-31	-31	-31	-31	-31
5	-29	-29	-29	-29	-29	-29	-29	-29	-29	...

TotalNumberOfTendons = 37

NumberOfDebondedTendons = 6

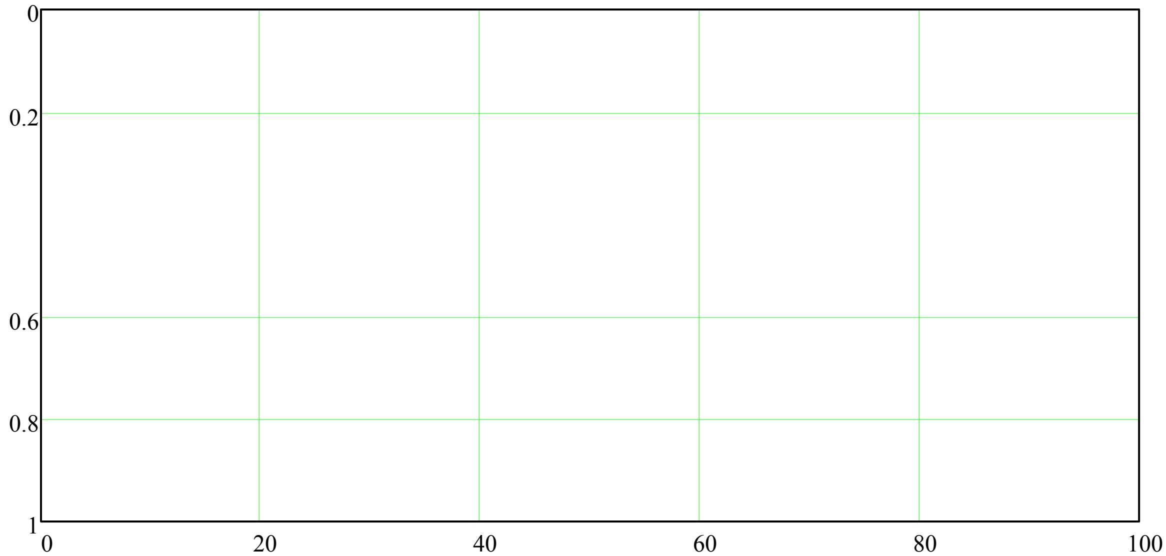
NumberOfDrapedTendons = 0

StrandSize = "0.6 in low lax"

StrandArea = $0.22 \cdot in^2$

$JackingForce_{per.strand}$ = $43.94 \cdot kip$

Location of Depressed Strands



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$

$\Delta f_{pES} = 0 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

$f_{pi} = 203 \cdot ksi$

$\Delta f_{pi} = 0 \cdot ksi$

$f_{pe} = 178 \cdot ksi$

$\Delta f_{pTot} = -24 \cdot ksi$

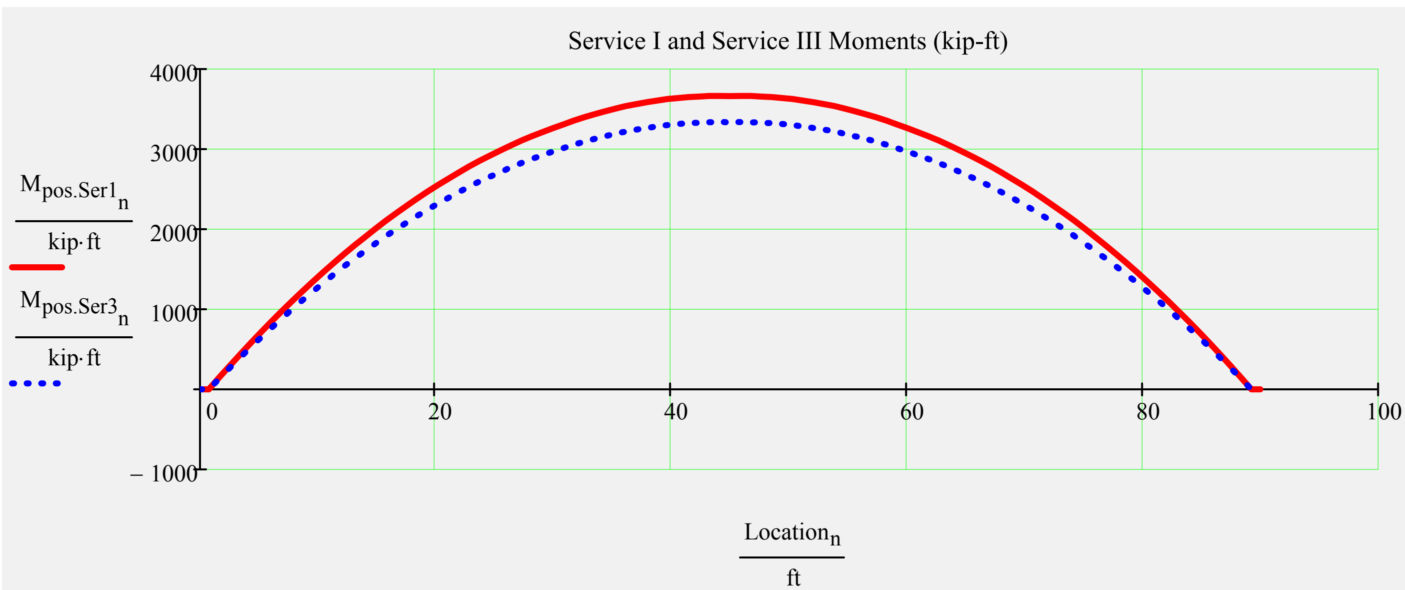
percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -11.95 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 88.05 \cdot \%$

Check_f_{pt} = "OK"

$0.8 \cdot f_{py} = 194 \cdot ksi$

Check_f_{pe} = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 3664.5 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 3338 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.61	−2.9
2	−0.72	−2.41
4	−0.67	−2.45
6	−2.56	−0.96
8	−3.07	0.31

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1625.9	−1675.9
"Final (about composite centroid)"	−1431.5	−1392.1

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	799.37	126328.79	−19.39
"Transformed Beam (initial)"	856.4	134564.33	−20.21
"Transformed Beam "	847.29	133321.64	−20.09
"Composite "	1520.14	376068.89	−8.93

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	851.6
"Non-composite (includes bm wt.)"	1868.6
"Composite"	164.4
"Distributed Live Load"	1629.7

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

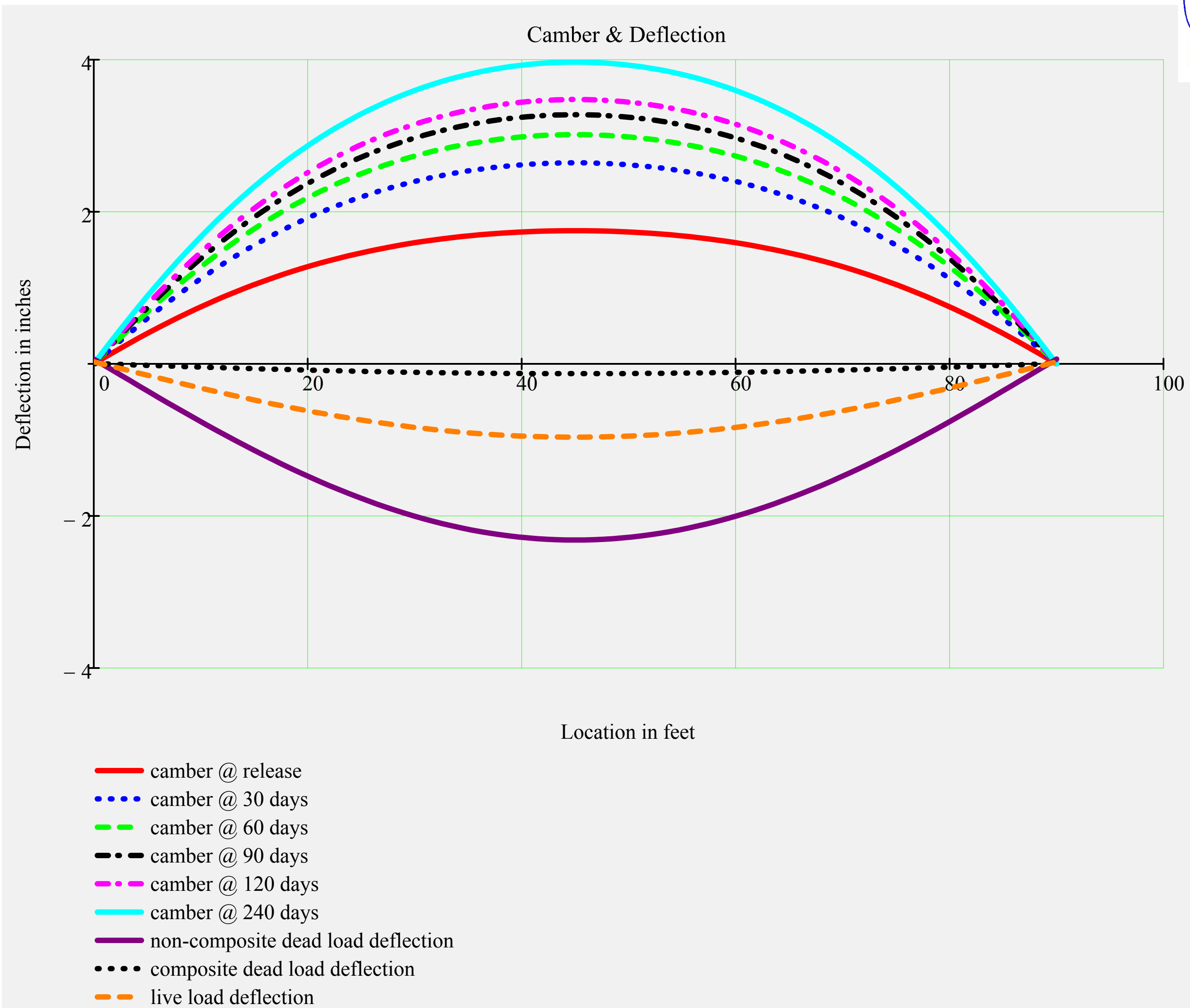
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

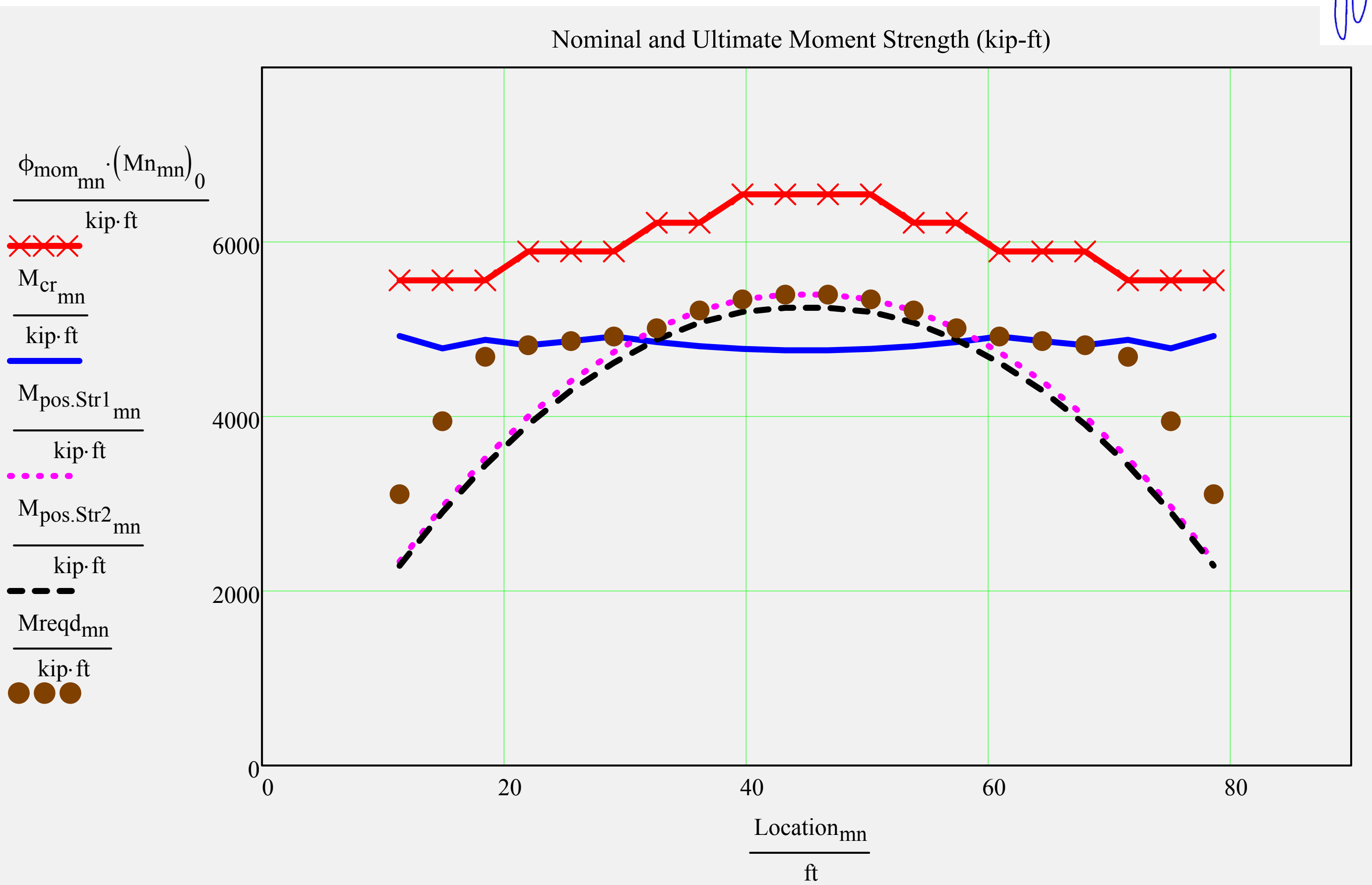
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	–0.0497	–0.8058	0.4274	1.7515
"30 Days"	–0.2398	–1.4277	0.7048	2.6438
"60 Days"	–0.3078	–1.6502	0.8132	3.0143
"90 Days"	–0.3441	–1.7688	0.871	3.2764
"120 Days"	–0.3666	–1.8425	0.9069	3.4763
"240 Days"	–0.4082	–1.9786	0.9732	3.9676
"non-comp DL"	–0.2803	0.2226	–0.4005	–2.3168
"comp DL"	–0.0069	0.0211	–0.0223	–0.1292
"LL"	–0.0522	0.1589	–0.1681	–0.9643

Strength Limit State Moments



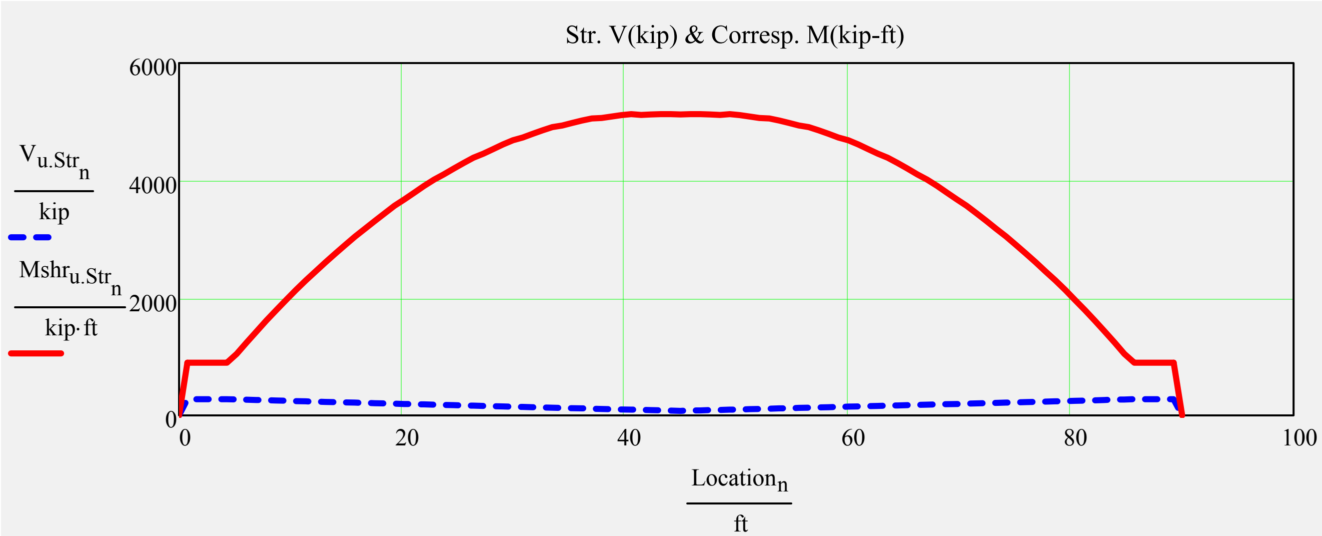
$CR_{Str.mom_n} := 10$ $\qquad CR_{Str.mom_{mn}} := \frac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.19$

$\max(Mreqd) = 5395.8 \cdot kip \cdot ft$

CheckMomentCapacity := if$\left(\min(CR_{Str.mom}) > 0.99, \text{"OK"}, \text{"No Good!"}\right)$ CheckMomentCapacity = "OK"

Strength Shear and Associated Moments



$max(V_{u.Str}) = 276.2 \cdot kip$ $\qquad$ $max(Mshr_{u.Str}) = 5130.4 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing, tmp_s (in)	Number of Spaces, tmp_NumberSpaces	area per stirrup, $tmp\_A_{stirrup}$ (in^2)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

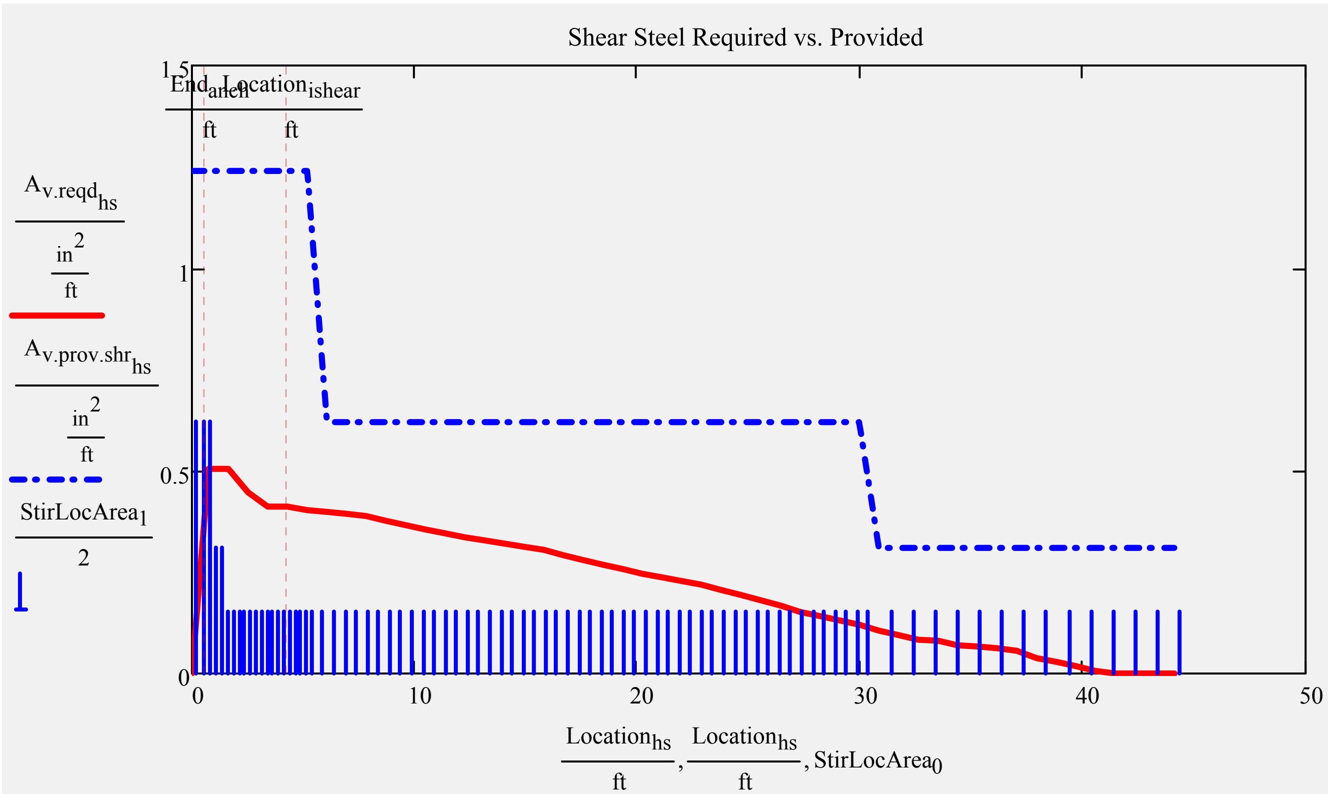
Location	$user\_s_{nspacings}$:=	$user\_NumberSpaces_{nspacings}$:=	$user\_A_{stirrup_{nspacings}}$:=	$interface\_factor_{nspacings}$:=
A1 stirrup	−1·in	−1	$-1 \cdot in^2$	0.25
A2 stirrup	−1·in	−1	$-1 \cdot in^2$	0.5
A3 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S1 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S2 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S3 stirrup	−1·in	−1	$-1 \cdot in^2$	1
S4 stirrup	−1·in	−1	$-1 \cdot in^2$	1
			$-1 \cdot in^2$	

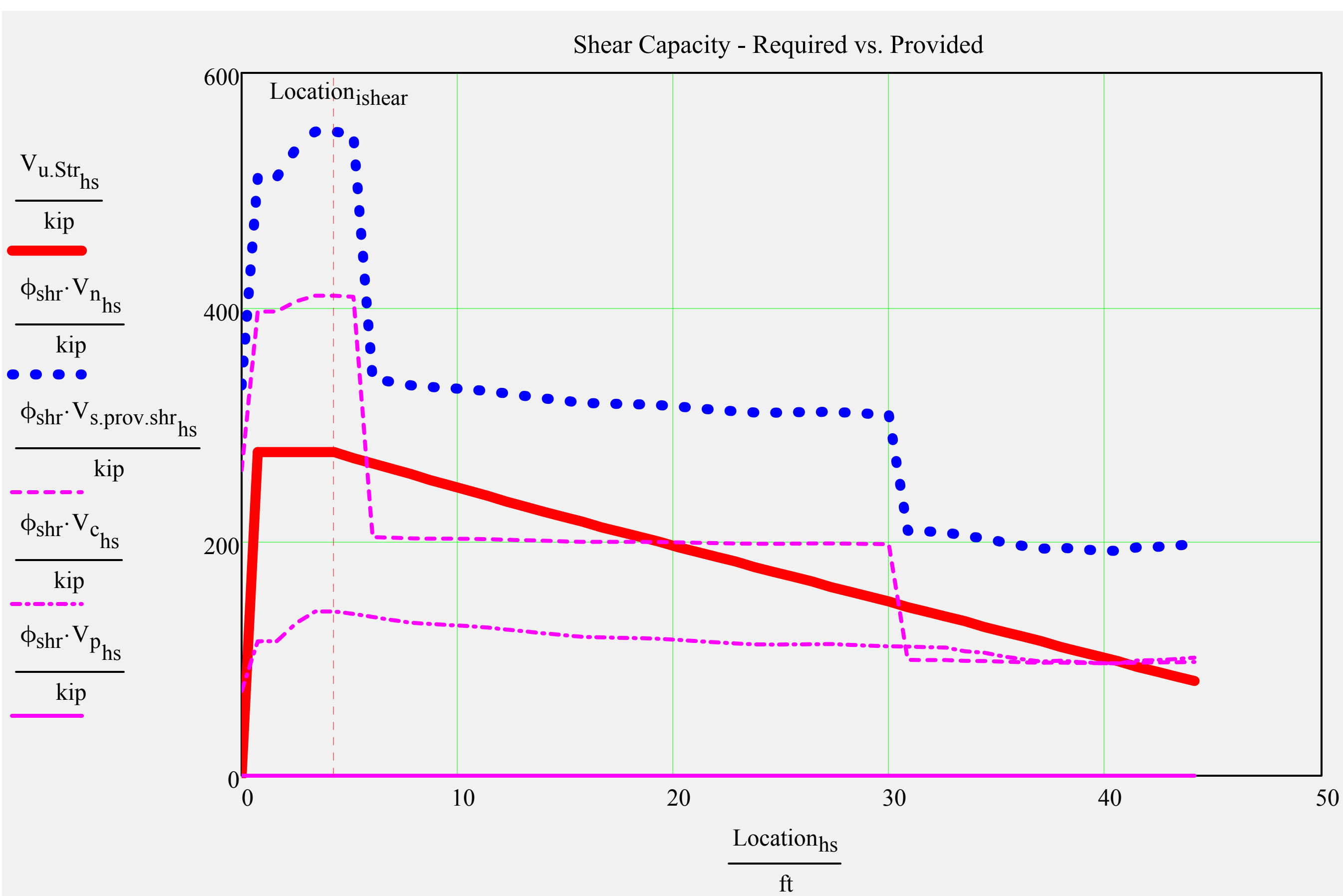
Stirrup sizes and spacings used in analysis

The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s (in)	NumberSpaces	$A_{stirrup}$ (in^2)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

EndCover = 2.5·in


Shear Steel Required vs. Provided
End_arch / ft
Location_ishear / ft
A_v.reqd_hs / (in^2 / ft)
A_v.prov.shr_hs / (in^2 / ft)
StirLocArea_1 / 2
1.5
1
0.5
0
0
10
20
30
40
50
Location_hs / ft , Location_hs / ft , StirLocArea_0

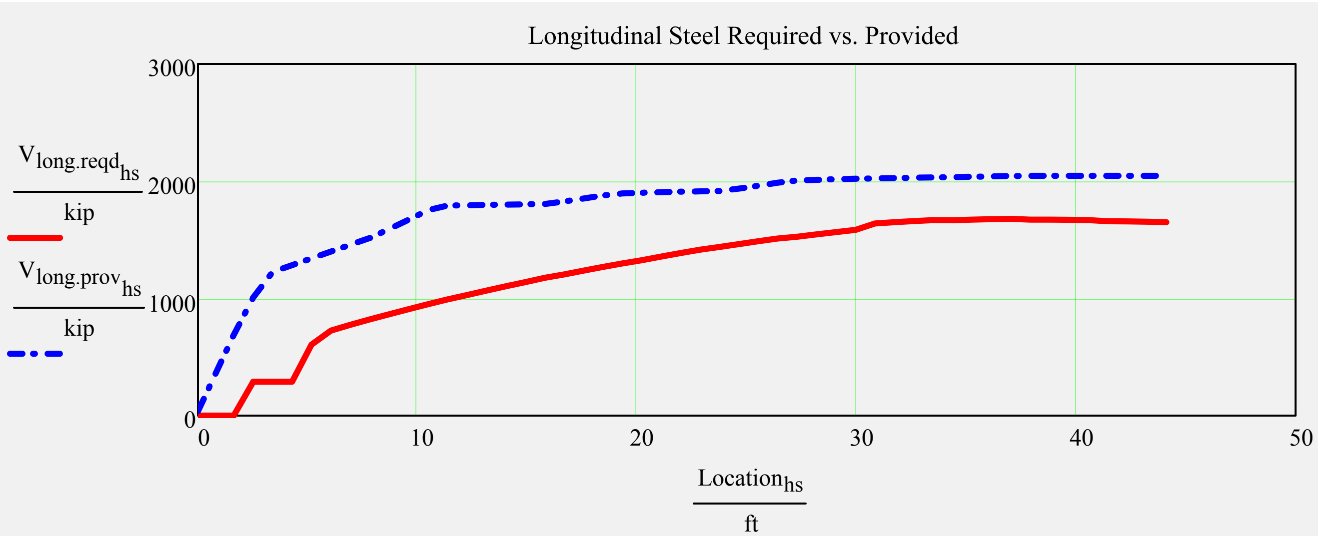

Shear Capacity - Required vs. Provided
Location_ishear
V_u.Str_hs / kip
φ_shr · Vn_hs / kip
φ_shr · V_s.prov.shr_hs / kip
φ_shr · Vc_hs / kip
φ_shr · Vp_hs / kip
600
400
200
0
0
10
20
30
40
50
Location_hs / ft

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

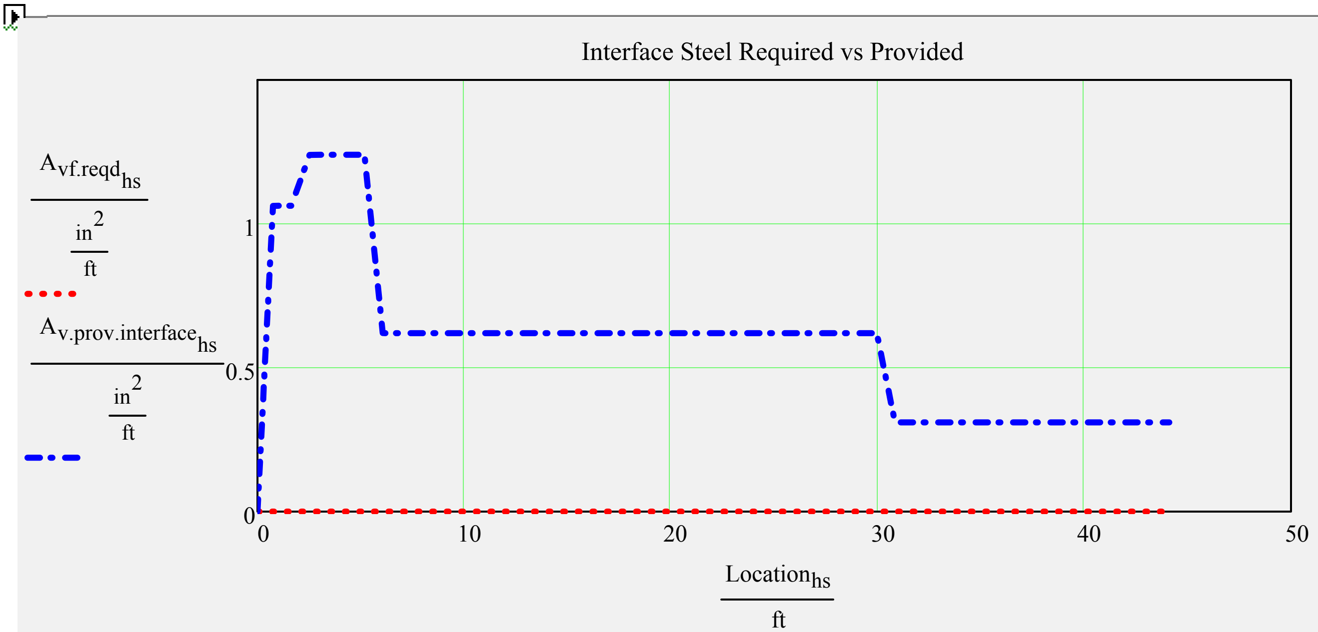


$CR_{LongSteel_{hs}} := if\left(V_{long.reqd_{hs}} < .01kip, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$

$min(CR_{LongSteel}) = 1.22$

CheckLongSteel := if(min($CR_{LongSteel}$) > 1, "OK", "No Good, add steel!")

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, "OK", "No\ Good"\right)$$

$$CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)$$

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A."   CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0 := AcceptAASHTO$

$check_1 := AcceptSDG$

$check_2 := AcceptOntario$

$check_3 := Check\_f_{pt}$

$check_4 := Check\_f_{pe}$

$check_5 := Check\_f_{tension.rel}$

$check_6 := Check\_f_{comp.rel}$

$check_7 := Check\_f_{tension.stage8}$

$check_8 := Check\_f_{comp.stage8.c1}$

$check_9 := Check\_f_{comp.stage8.c2}$

$check_{10} := Check\_f_{comp.stage8.c3}$

$check_{11} := CheckMomentCapacity$

$check_{12} := CheckMaxCapacity$

$check_{13} := CheckStirArea$

$check_{14} := CheckShearCapacity$

$check_{15} := CheckMinStirArea$

$check_{16} := CheckMaxStirSpacing$

$check_{17} := CheckLongSteel$

$check_{18} := CheckInterfaceSpacing$

$check_{19} := CheckSplittingSteel$

$check_{20} := CheckMaxPrestressingForce$

$check_{21} := CheckPattern_0$

$check_{22} := CheckPattern_1$

$check_{23} := CheckPattern_2$

$check_{24} := CheckPattern_3$

$check_{25} := CheckPattern_4$

$check_{26} := CheckInterfaceSteel$

$check_{27} := CheckStrandFit$

$check_{28} := Check\_SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T =$

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8: FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

Moment (Strength) or Stress (Service) | Shear (Strength)

$LRFR_{loadrating} =$

"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.69	1.36	"N/A"	53.10	0.90	1.44	"N/A"	5.31	HL-93
"Strength I(Op)"	0.69	1.77	"N/A"	53.10	0.90	1.87	"N/A"	5.31	HL-93
"Service III(Inv)"	0.69	1.21	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.69	1.33	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.69	1.43	86.01	53.10	0.90	1.47	88.47	5.31	*Permit
"Service III"	0.69	1.24	74.57	45.13	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.25$ $CR_{LongSteel.Permit} = 1.22$ $CheckLongSteel_{loadrating} = \text{"OK"}$

LRFD Prestressed Beam Program

Project = "SR87 Over Clear Creek"
DesignedBy = "RAA"
Date = "02.13"

filename = "G:\SR87\Engineering\BDR_ClearCreek_Feb2013\LRFDPBeam3.3\Program Files\Beam Data Files_AltB_ 90'_FIB-36_NE

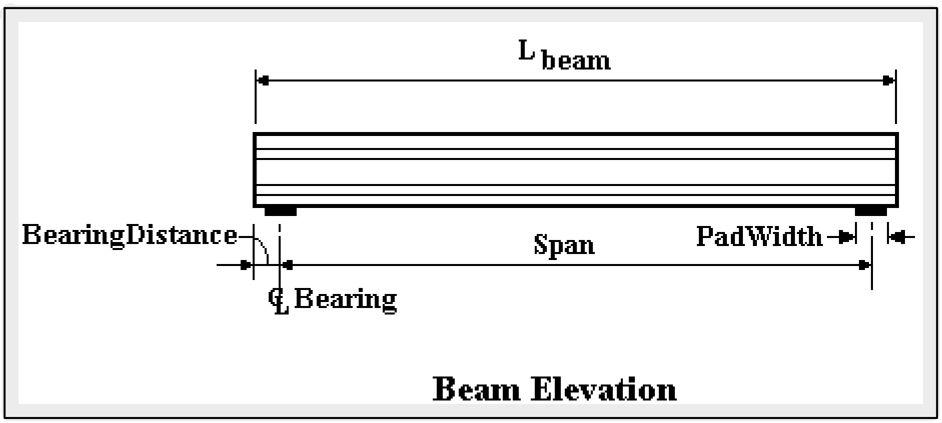
Comment = "Alt. B - NB Exterior"

Legend

TanHighlight = DataEntry   YellowHighlight = CheckValues   GreyHighlight = UserComments + Graphs

BlackText = ProgramEquations   *Maroon Text = Code Reference*   *Blue Text = Commentary*

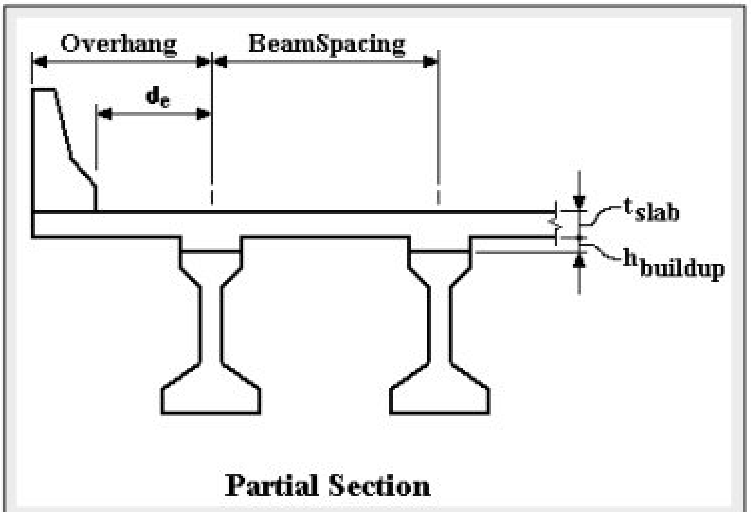
Bridge Layout and Dimensions



$L_{beam} = 90 \cdot ft$ $Span = 88.5 \cdot ft$ $BearingDistance = 9 \cdot in$ $PadWidth = 10 \cdot in$

BeamTypeTog = "FIB36"

These are typically the FDOT designations found in our standards. The user can also create a coordinate file for a custom shape. In all cases the top of the beam is at the y=0 ordinate.



$Overhang = 4.54 \cdot ft$ $BeamSpacing = 8.5 \cdot ft$ $t_{slab} = 8 \cdot in$ $h_{buildup} = 1.5 \cdot in$

$Skew = 20 \cdot deg$ $t_{integral.ws} = 0.5 \cdot in$ $NumberOfBeams = 5$ $t_{slab.delta} = 0 \cdot in$

$de = 2.75\ ft$

BeamPosition = "exterior"   *For calculating distribution factors must be either interior or exterior*

SectionType = "transformed"   *Can be either gross or transformed. Gross uses the section properties of the concrete only ignoring the steel and does not include elastic gains. Transformed includes the effect of the transformed steel areas on the section properties.*

$b_e = 8.79\ ft$ *effective slab width*   *LRFD 4.6.2.6*

$t_{slab} := if(t_{slab} \le 0 \cdot in, 0.00001 \cdot in, t_{slab})$ *Provide a minimum slab thickness to prevent divide by zero errors*

Material Properties

$SectTog := if(SectionType = "transformed", 1, 0)$ $SectTog = 1$

Concrete:

Corrosion Classification   Environment = "moderately"

density of slab concrete $\gamma_{slab} = 0.15 \cdot \frac{kip}{ft^3}$

strength of slab concrete $f_{c.slab} = 4.5 \cdot ksi$

density of beam concrete $\gamma_{beam} = 0.15 \cdot \frac{kip}{ft^3}$

strength of beam concrete $f_{c.beam} = 8.5 \cdot ksi$

weight of future wearing surface $Weight_{future.ws} = 0 \cdot \frac{kip}{ft^2}$

release beam strength $f_{ci.beam} = 6 \cdot ksi$

type of course aggregate, "Florida" *or* "Standard"   AggregateType = "Florida"

relative humidity $H = 75$

$n_d := \sqrt{\frac{f_{c.beam}}{f_{c.slab}}}$ *used in distribution calculation* $n_d = 1.37$

$AggFactor := if[AggregateType = "Florida", (0.9 \cdot 1820), 1820]$ $AggFactor = 1638$

$E_{ci} := AggFactor \cdot \sqrt{f_{ci.beam} \cdot ksi}$ *initial beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_{ci} = 4012 \cdot ksi$

$E_c := AggFactor \cdot \sqrt{f_{c.beam} \cdot ksi}$ *beam concrete modulus of elasticity (LRFD 5.4.2.4)* $E_c = 4776 \cdot ksi$

Prestressing Tendons:

tendon ultimate tensile strength $f_{pu} = 270 \cdot ksi$

time in days between jacking and transfer $t_j = 1.5$

tendon modulus of elasticity $E_p = 28500 \cdot ksi$

ratio of tendon modulus to initial beam concrete modulus $n_{pi} := \frac{E_p}{E_{ci}}$

ratio of tendon modulus to beam concrete modulus $n_p := \frac{E_p}{E_c}$

Mild Steel:

mild steel yield strength $f_y = 60 \cdot ksi$

ratio of rebar modulus to initial beam concrete modulus $n_{mi} := \frac{E_s}{E_{ci}}$ $n_{mi} = 7.23$

ratio of rebar modulus to beam concrete modulus $n_m := \frac{E_s}{E_c}$ $n_m = 6.07$

d distance from top of slab to centroid of slab reinf. $d_{slab.rebar} = 4 \cdot in$

d distance from top of beam to centroid of mild flexural tension reinf. $d_{long} = 0 \cdot in$

mild steel modulus of elasticity $E_s = 29000 \cdot ksi$

area per unit width of longitudinal slab reinf. $A_{slab.rebar} = 0.62 \cdot \frac{in^2}{ft}$

area of mild reinf lumped at centroid of bar locations $A_{s.long} = 0 \cdot in^2$

Size of bar used create used to calculate development length $BarSize = 5$

Permit Loads

This is the number of wheel loads that comprise the truck, max for DLL is 11 $PermitAxles = 3$

Indexes used to identify values in the P and d vectors $q := 0 .. (PermitAxles - 1)$ $qt := 0 .. PermitAxles$

$PermitAxleLoad^T = (13.33 \quad 53.33 \quad 53.33) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0) \cdot ft$

Distribution Factors

DataMessage = "This is a Single Web Beam Design, AASHTO distribution factors used"

calculated values:

$tmp\_g_{mom} = 0.88$ $tmp\_g_{shear} = 0.94$

user value overrides (optional):

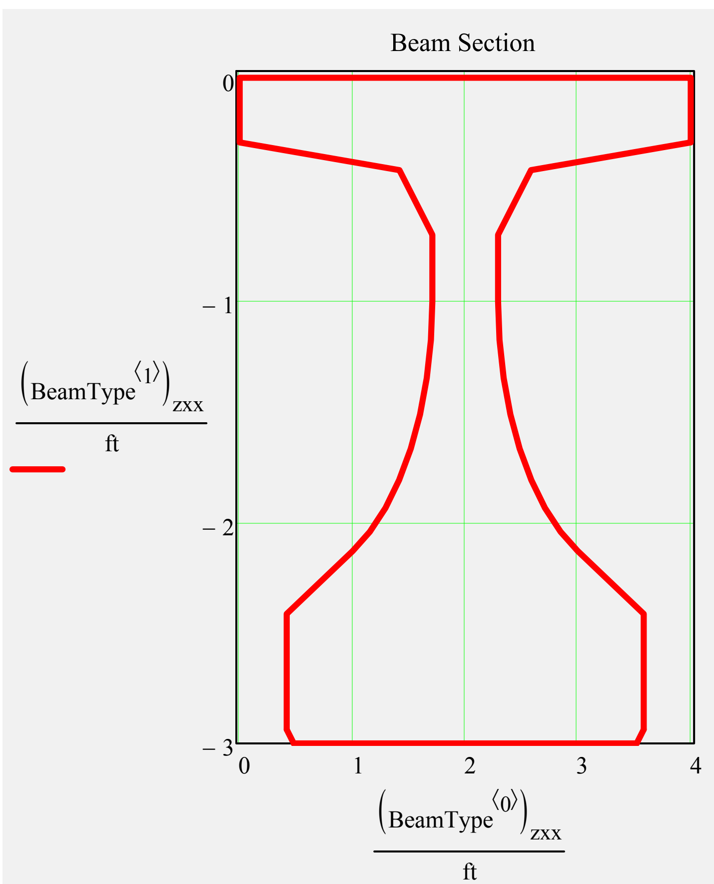
$user\_g_{mom} := 0$ $user\_g_{shear} := 0$

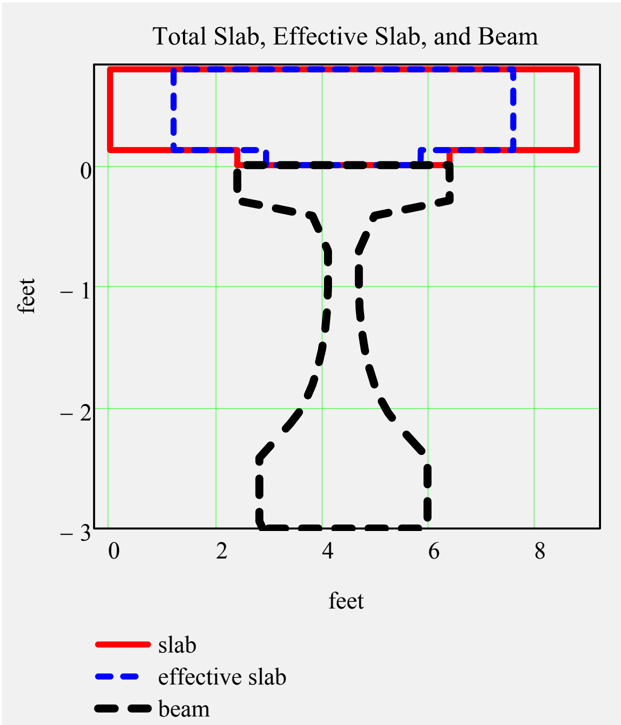
value check

$g_{mom} := if(user\_g_{mom} \neq 0, user\_g_{mom}, tmp\_g_{mom})$ $g_{mom} = 0.88$

$g_{shear} := if(user\_g_{shear} \neq 0, user\_g_{shear}, tmp\_g_{shear})$ $g_{shear} = 0.94$

Section Views





Non-Composite Dead Load Input:

$w_{slab} = 1.009 \cdot \frac{kip}{ft}$ $w_{beam} = 0.841 \cdot \frac{kip}{ft}$ $w_{forms} = 0.045 \cdot \frac{kip}{ft}$

$Add\_w_{noncomp} := 0.0 \cdot \frac{kip}{ft}$ *additional non composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet.

$w_{noncomposite} := w_{slab} + w_{beam} + w_{forms} + Add\_w_{noncomp}$ $w_{noncomposite} = 1.895 \cdot \frac{kip}{ft}$

$wb_{noncomposite} := w_{slab} + w_{forms} + Add\_w_{noncomp}$ $wb_{noncomposite} = 1.054 \cdot \frac{kip}{ft}$

Diaphragms/Point Load Input

End Diaphragms or Misc. Point Loads over bearing... included in bearing reaction calculation only

$EndDiaphragmA := 0 \cdot kip$ *begin bridge*

$EndDiaphragmE := 0 \cdot kip$ *end bridge*

Intermediate Diaphragms or Misc. Point Loads... included in shear, moment, and bearing reaction calculations

$IntDiaphragmB := 0 \cdot kip$ *input load is per beam*

$DistB := 0 \cdot ft$

Longitudinal Distance B, C, & D - Measured from CL Bearing at begin bridge

$IntDiaphragmC := 0 \cdot kip$

$DistC := 0 \cdot ft$

$IntDiaphragmD := 0 \cdot kip$

$DistD := 0 \cdot ft$

Composite Dead Load Input:

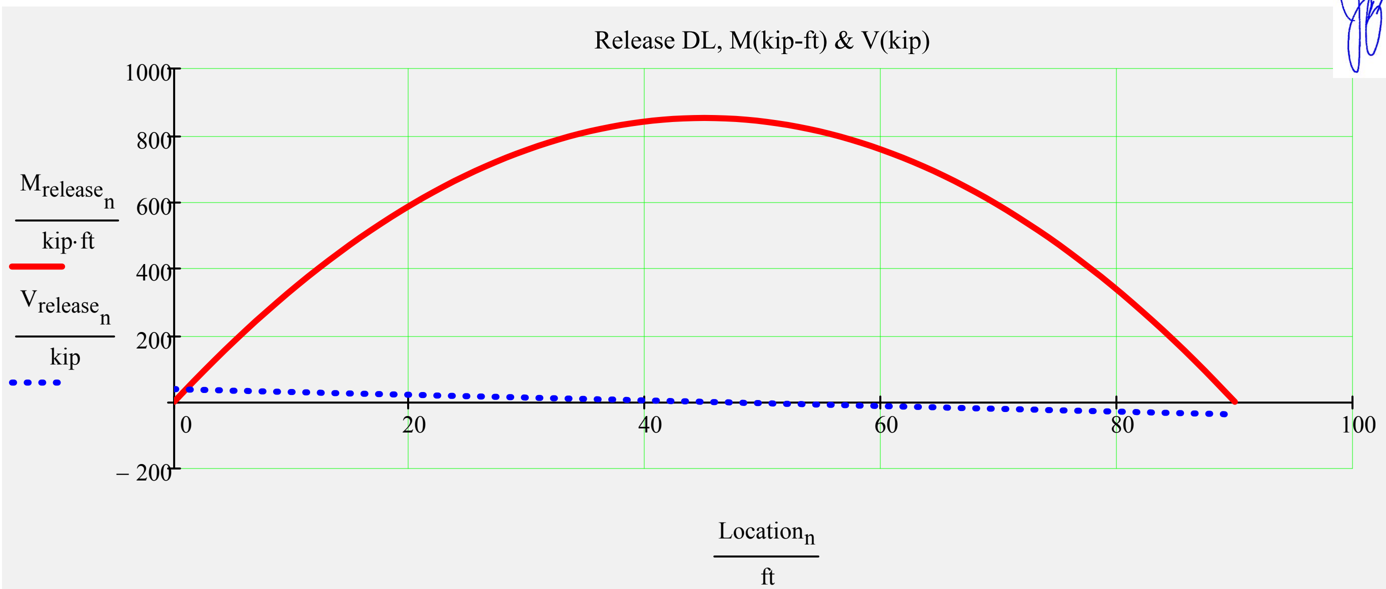
$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.168 \cdot \frac{kip}{ft}$

$Add\_w_{comp} := 0.0 \cdot \frac{kip}{ft}$ *additional composite dead load (positive or negative)*
note: not saved to data file, may be saved to Mathcad worksheet

$w_{composite} := w_{future.ws} + w_{barrier} + Add\_w_{comp}$ $w_{composite} = 0.168 \cdot \frac{kip}{ft}$

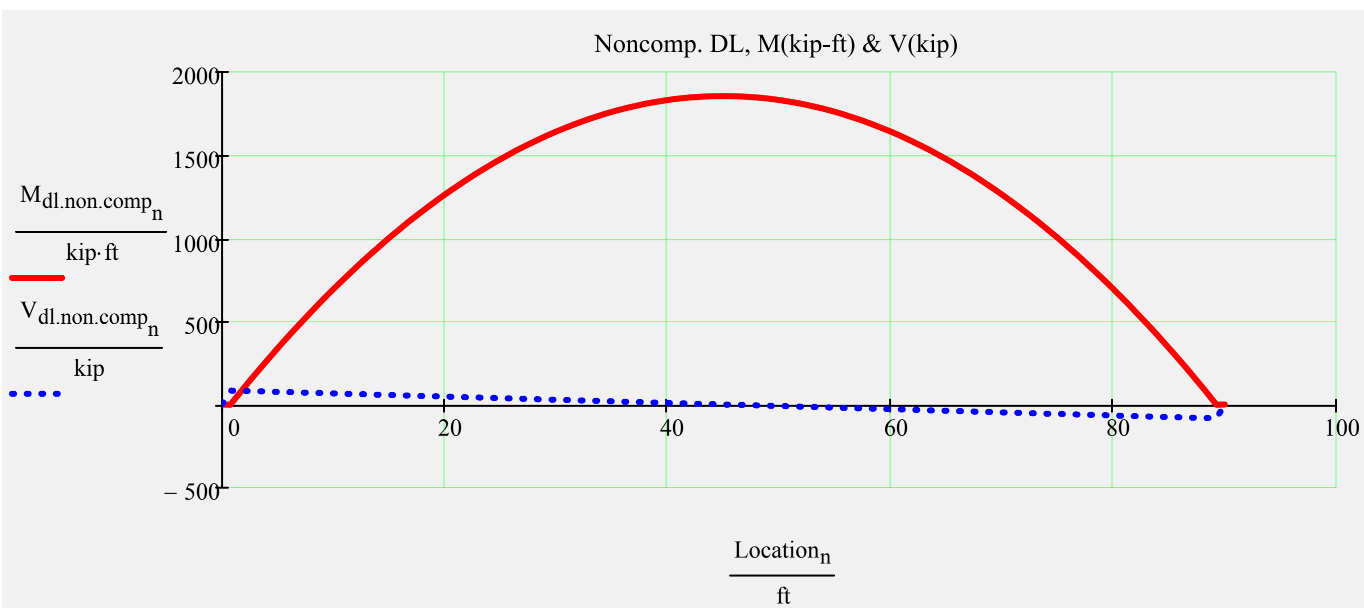
$w_{comp.str} := w_{barrier} + Add\_w_{comp}$ $w_{comp.str} = 0.168 \cdot \frac{kip}{ft}$

Release Dead Load Moments and Shear



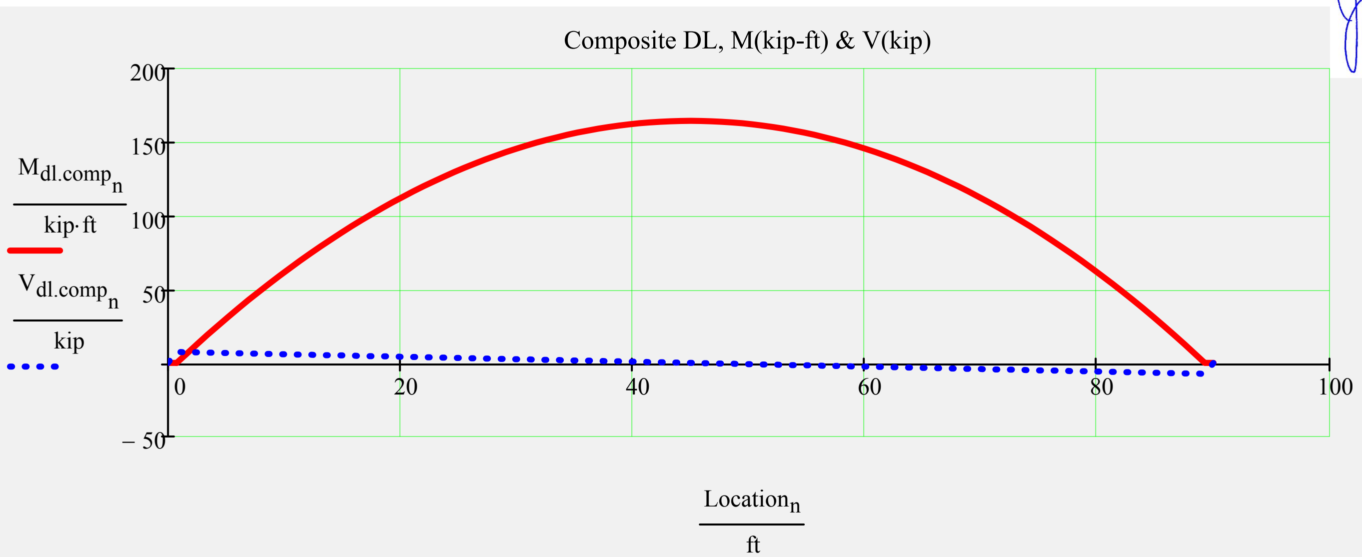
$\max(M_{release}) = 851.6 \cdot kip \cdot ft$ $\max(V_{release}) = 37.8 \cdot kip$

Noncomposite Dead Load Moments and Shear



$\max(M_{dl.non.comp}) = 1854.7 \cdot kip \cdot ft$ $\max(V_{dl.non.comp}) = 83.9 \cdot kip$

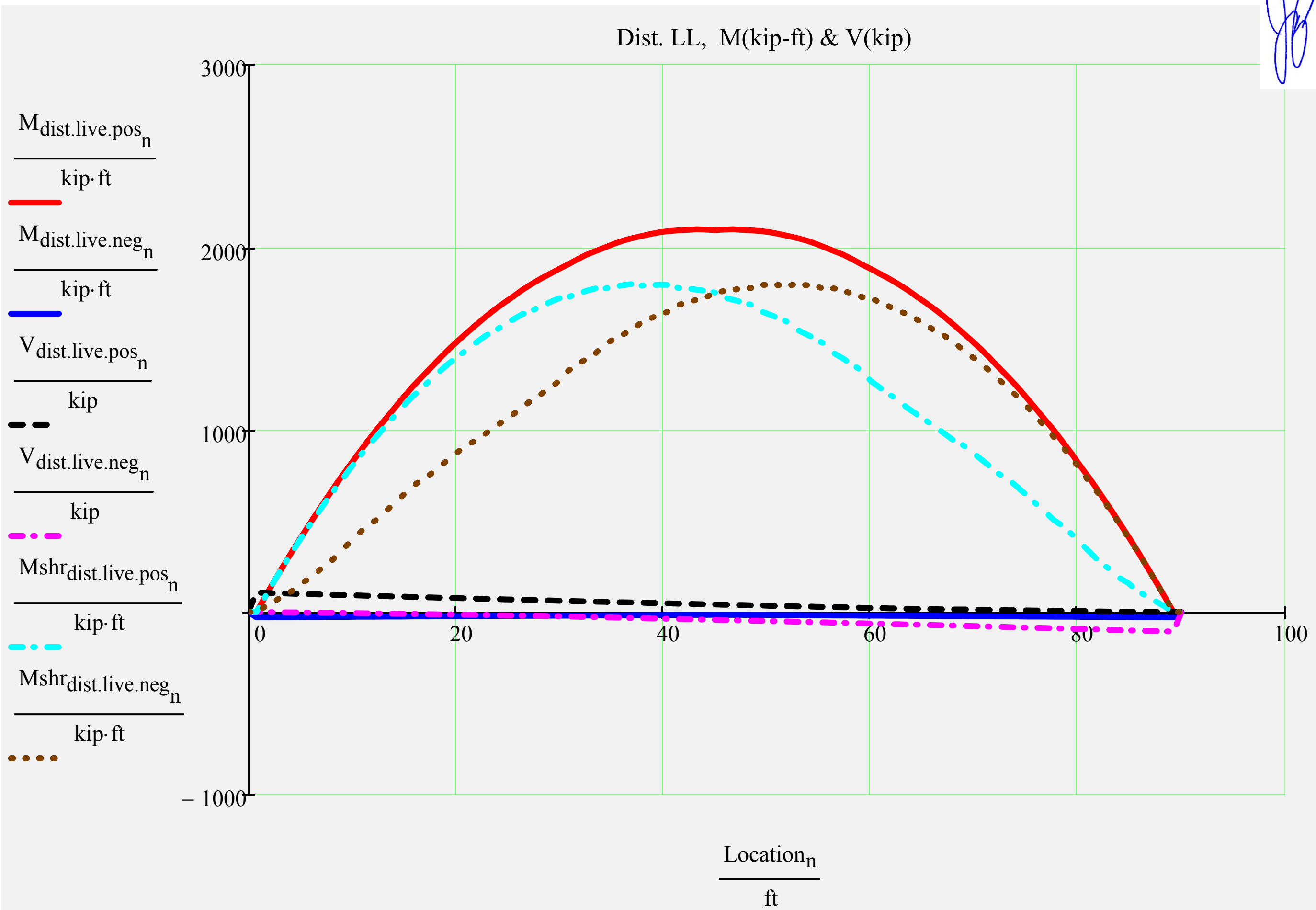
Composite Dead Load Moments and Shear



$max(M_{dl.comp}) = 164.4 \cdot kip \cdot ft$

$max(V_{dl.comp}) = 7.4 \cdot kip$

Distributed Live Load Moments and Shear



Beam End Reactions...
with IM factor only

$\max(M_{dist.live.pos}) = 2097.8 \cdot kip \cdot ft$ $\min(M_{dist.live.neg}) = -28.3 \cdot kip \cdot ft$ $Reaction_{LL} = 108.18 \cdot kip$

$\max(V_{dist.live.pos}) = 106.9 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 1798.4 \cdot kip \cdot ft$ $Reaction_{DL} = 92.83 \cdot kip$

Prestress Strand Layout Input

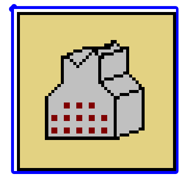
Instructions:

Double click the icon to open the 'Strand Pattern Generator'. Specify the type, location, size, and debonding of strands. When finished, press the 'Continue' button. Calculate worksheet(ctrl+F9) to update strand pattern (see Tendon Layout below).

Strand Pattern Input Mode:

StrandTemplate :=
Standard
Custom

Strand Pattern Generator:



Collapsed Region for Custom Strand Sizes...

$CheckPattern_0$ = "OK" — ***check 0*** *- no debonded tendon in outside row*

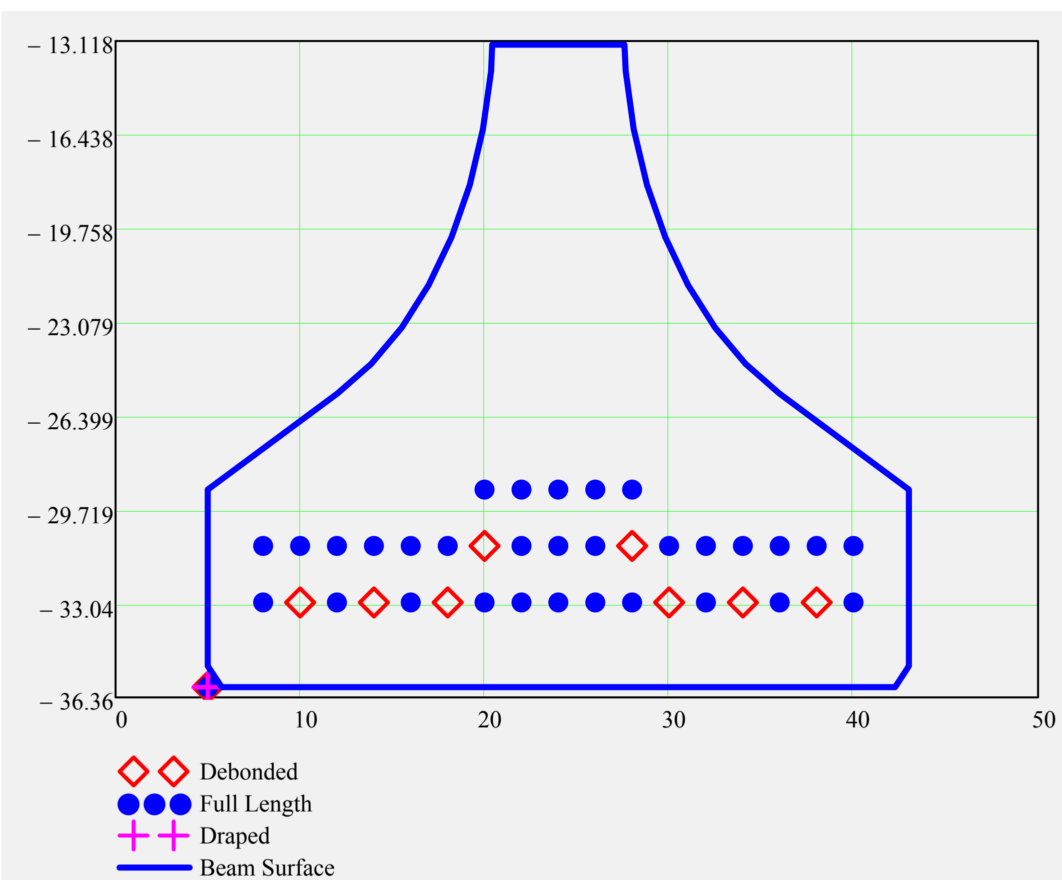
$CheckPattern_1$ = "OK" — ***check 1*** *- less than 25% debonded tendons total*

$CheckPattern_2$ = "OK" — ***check 2*** *- less than 40% debonded tendons in any row*

$CheckPattern_3$ = "OK" — ***check 3*** *- less than 40% of debonded tendons terminated at same section* *(LRFD 5.11.4.3)*

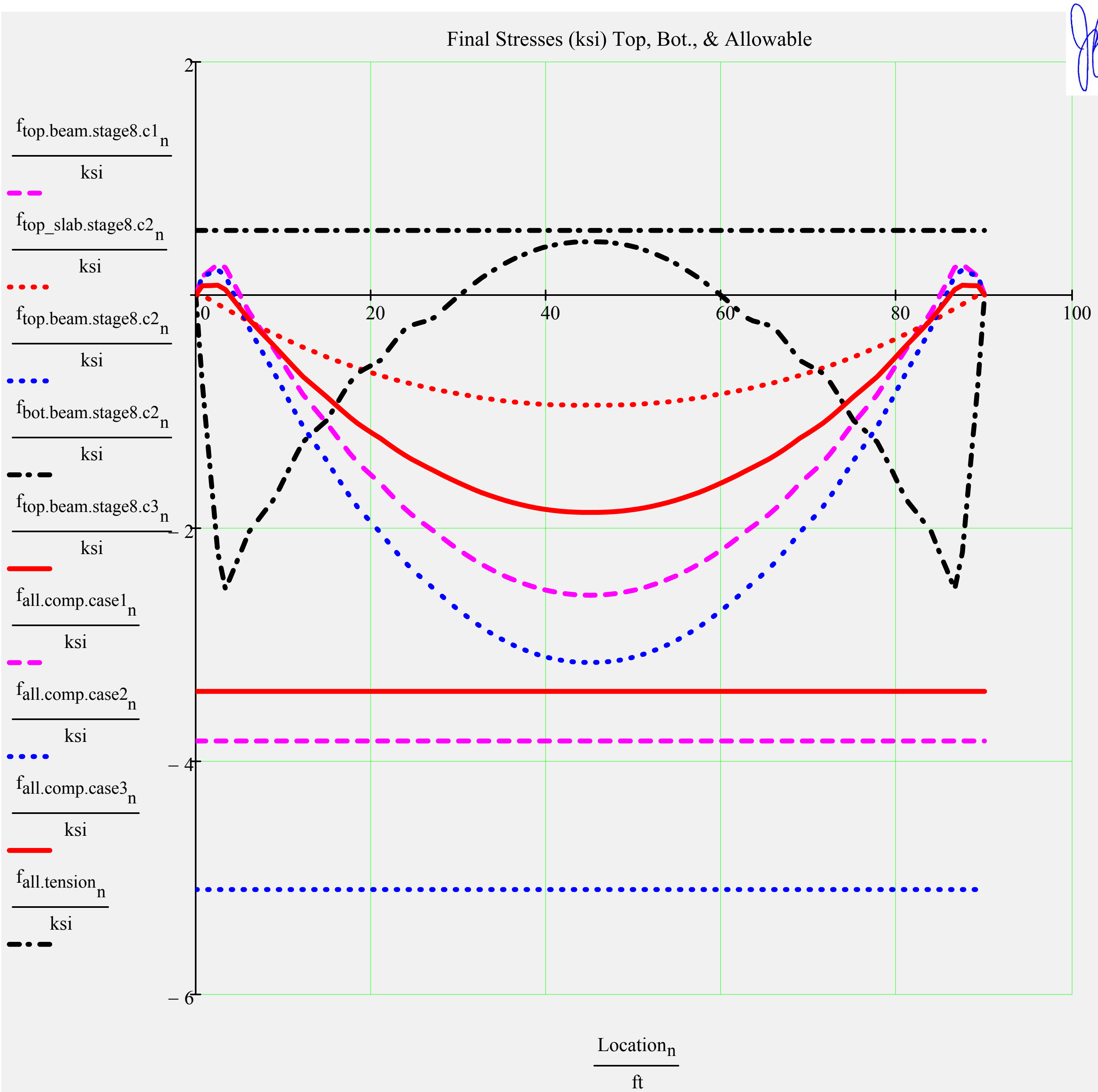
$CheckPattern_4$ = "OK" — ***check 4*** *- more than half beam depth debond length* *(SDG 4.3.1.E)*

Tendon Layout



Release Stresses

Final Stresses



Stress Checks

$\min(CR\_f_{tension.rel}) = 1.88$	$Check\_f_{tension.rel} =$ "OK"	(Release tension)
$\min(CR\_f_{comp.rel}) = 1.1$	$Check\_f_{comp.rel} =$ "OK"	(Release compression)
$\min(CR\_f_{tension.stage8}) = 1.21$	$Check\_f_{tension.stage8} =$ "OK"	(Service III , PS + DL +LL*0.8)
$\min(CR\_f_{comp.stage8.c1}) = 1.49$	$Check\_f_{comp.stage8.c1} =$ "OK"	(Service I , PS + DL)
$\min(CR\_f_{comp.stage8.c2}) = 1.62$	$Check\_f_{comp.stage8.c2} =$ "OK"	(Service I , PS + DL +LL)
$\min(CR\_f_{comp.stage8.c3}) = 1.82$	$Check\_f_{comp.stage8.c3} =$ "OK"	(Service I , (PS + DL)*0.5 +LL)

Section and Strand Properties Summary

$A_{beam} = 807.4 \cdot in^2$	Concrete area of beam	$I_{beam} = 127557.7893 \cdot in^4$	Gross Moment of Inertia of Beam about CG
$y_{comp} = -8.37 \cdot in$	Dist. from top of beam to CG of gross composite section	$I_{comp} = 353520.3547 \cdot in^4$	Gross Moment of Inertia Composite Section about CG
$A_{deck} = 666.37 \cdot in^2$	Concrete area of deck slab	$A_{ps} = 8.5 \cdot in^2$	total area of strands
$d_{b.ps} = 0.6 \cdot in$	diameter of Prestressing strand	$\min(PrestressType) = 0$	0 - low lax 1 - stress relieved
$f_{py} = 243 \cdot ksi$	tendon yield strength	$f_{pj} = 203 \cdot ksi$	prestress jacking stress

$$L_{shielding}^{T} = (6 \quad 12 \quad 18 \quad 0 \quad 24 \quad 0 \quad 0) \cdot ft$$

$$A_{ps.row}^{T} = (0.4 \quad 0.4 \quad 0.4 \quad 2.4 \quad 0.4 \quad 3.3 \quad 1.1) \cdot in^2$$

$d_{ps.row} =$ (·in)

	0	1	2	3	4	5	6	7	8	9
0	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
1	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
2	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
3	-33	-33	-33	-33	-33	-33	-33	-33	-33	-33
4	-31	-31	-31	-31	-31	-31	-31	-31	-31	-31
5	-31	-31	-31	-31	-31	-31	-31	-31	-31	-31
6	-29	-29	-29	-29	-29	-29	-29	-29	-29	...

TotalNumberOfTendons = 39

NumberOfDebondedTendons = 8

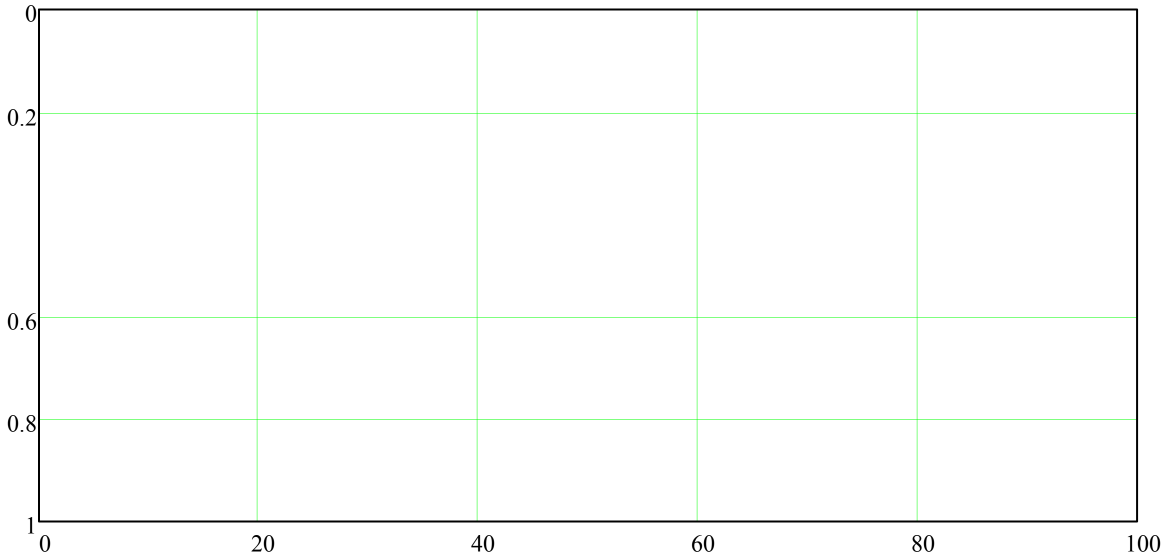
NumberOfDrapedTendons = 0

StrandSize = "0.6 in low lax"

$StrandArea = 0.22 \cdot in^2$

$JackingForce_{per.strand} = 43.94 \cdot kip$

Location of Depressed Strands



Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$ $\Delta f_{pES} = 0 \cdot ksi$

$f_{pi} = 203 \cdot ksi$ $\Delta f_{pi} = 0 \cdot ksi$

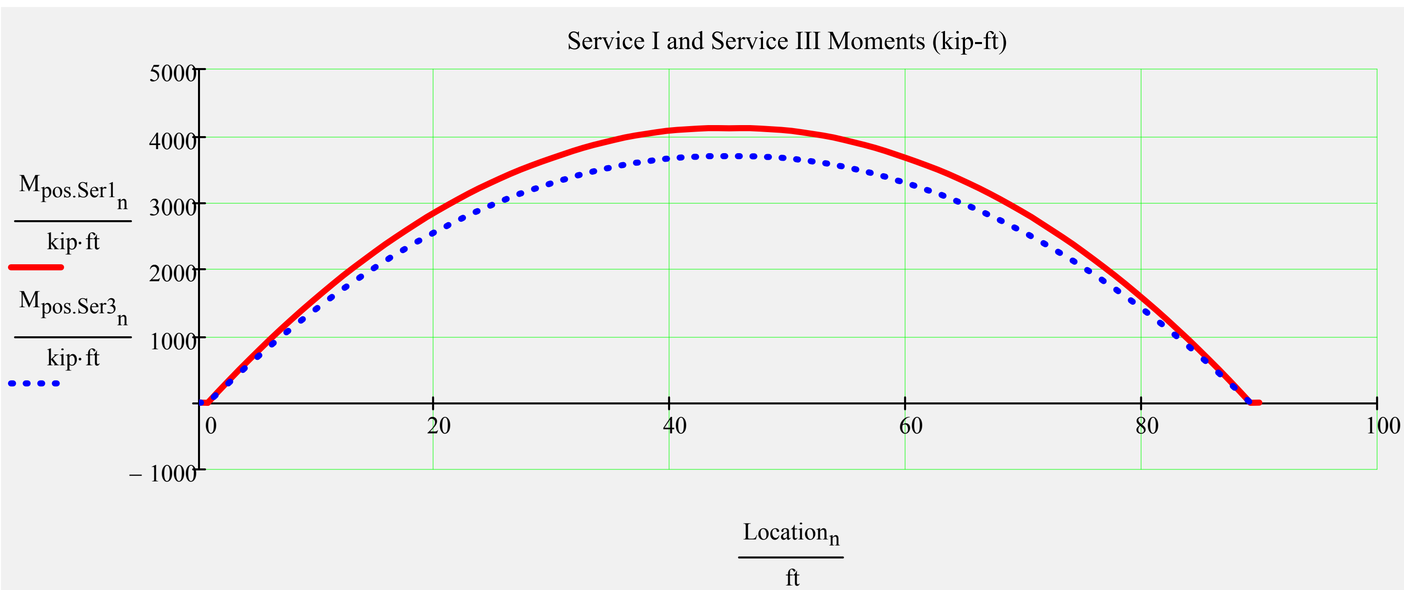
$f_{pe} = 178 \cdot ksi$ $\Delta f_{pTot} = -25 \cdot ksi$

Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per LRFD 5.9.5.2.3

percentages $\frac{\Delta f_{pi}}{f_{pj}} = 0 \cdot \%$ $\frac{f_{pi}}{f_{pj}} = 100 \cdot \%$ $\frac{\Delta f_{pTot}}{f_{pj}} = -12.32 \cdot \%$ $\frac{f_{pe}}{f_{pj}} = 87.68 \cdot \%$

Check_f$_{pt}$ = "OK" $0.8 \cdot f_{py} = 194 \cdot ksi$ Check_f$_{pe}$ = "OK"

Service Limit State Moments



$\max(M_{pos.Ser1}) = 4114.6 \cdot kip \cdot ft$ $\max(M_{pos.Ser3}) = 3695.3 \cdot kip \cdot ft$

Summary of Values at Midspan

Stresses =

"Stage "	"Top of Beam (ksi) "	"Bott of Beam (ksi)"
1	−0.6	−3.08
2	−0.72	−2.55
4	−0.67	−2.59
6	−2.53	−1.12
8	−3.15	0.46

PrestressForce =

"Condition "	"Axial (kip)"	"Moment (kip*ft)"
"Release"	−1713.8	−1747.1
"Final (about composite centroid)"	−1502.6	−1440.8

Properties =

"Section "	"Area (in^2) "	"Inertia (in^4) "	"distance to centroid from top of bm (in)"
"Net Beam "	798.94	126288.66	−19.38
"Transformed Beam (initial)"	859.05	134768.3	−20.24
"Transformed Beam "	849.45	133492.5	−20.11
"Composite "	1543.46	381406.23	−8.76

ServiceMoments =

"Type "	"Value (kip*ft)"
"Release"	851.6
"Non-composite (includes bm wt.)"	1854.7
"Composite"	164.4
"Distributed Live Load"	2092.9

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

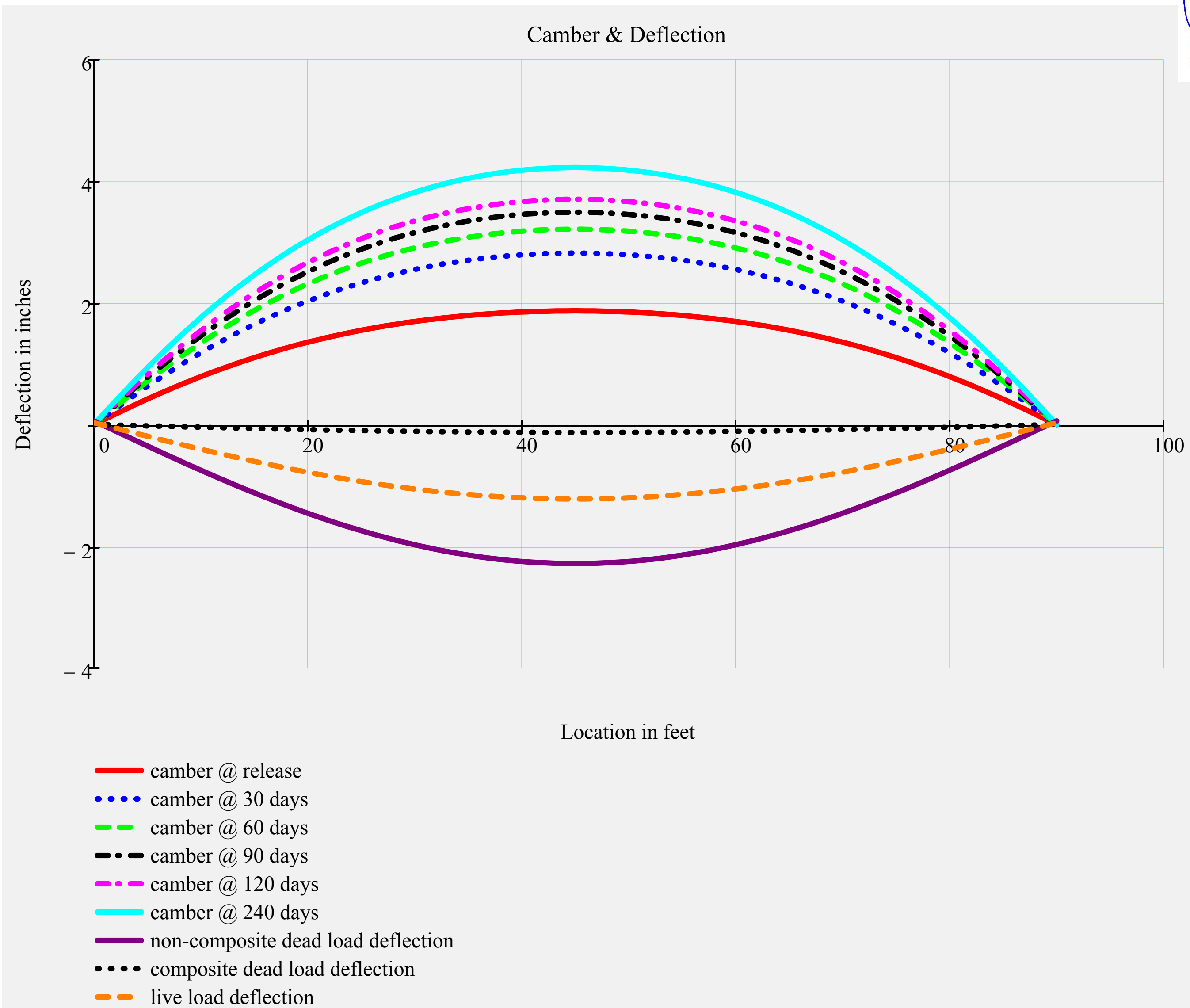
Stage 2 ---> Same as release with the addition of the remaining prestress losses applied to the transformed beam

Stage 4 ---> Same as stage 2 with supports changed from the end of the beam to the bearing locations

Stage 6 ---> Stage 4 with the addition of non-composite dead load excluding beam weight which has been included since Stage 1

Stage 8 ---> Stage 6 with the addition of composite dead load and live loads applied to the composite section

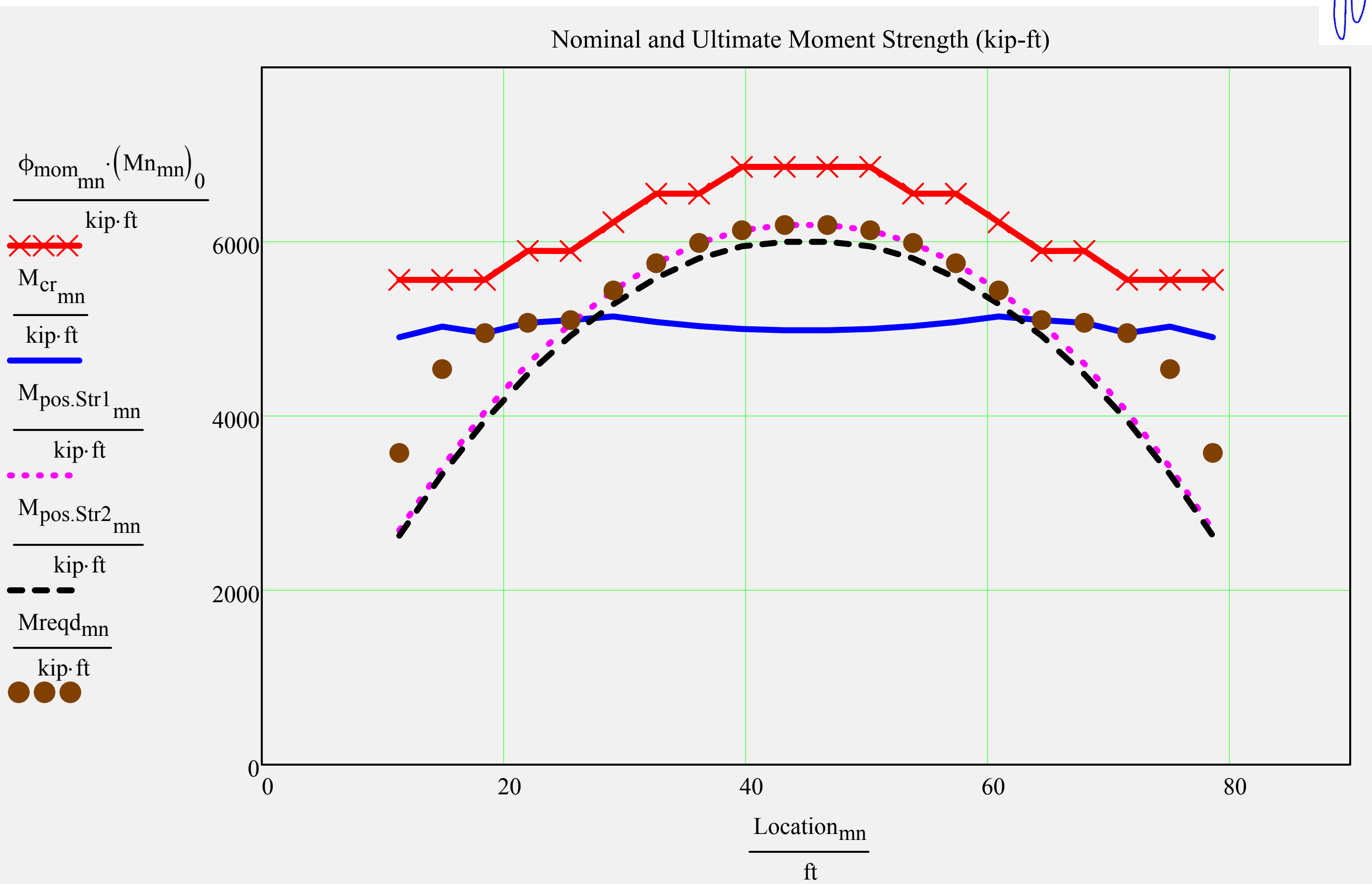
Camber, Shrinkage, and Dead Load Deflection Components



SlopeData =

"Stage"	"Change in L @ Top (in)"	"Change in L @ Bot. (in)"	"Slope at End (deg)"	"midspan defl (in)"
"Release"	−0.0472	−0.8405	0.4475	1.8714
"30 Days"	−0.2359	−1.4823	0.7377	2.8199
"60 Days"	−0.3034	−1.7119	0.8511	3.2137
"90 Days"	−0.3394	−1.8343	0.9116	3.4923
"120 Days"	−0.3618	−1.9104	0.9491	3.7048
"240 Days"	−0.403	−2.0508	1.0185	4.227
"non-comp DL"	−0.2766	0.2191	−0.3947	−2.2829
"comp DL"	−0.0067	0.021	−0.022	−0.1274
"LL"	−0.0648	0.2025	−0.2128	−1.2211

Strength Limit State Moments



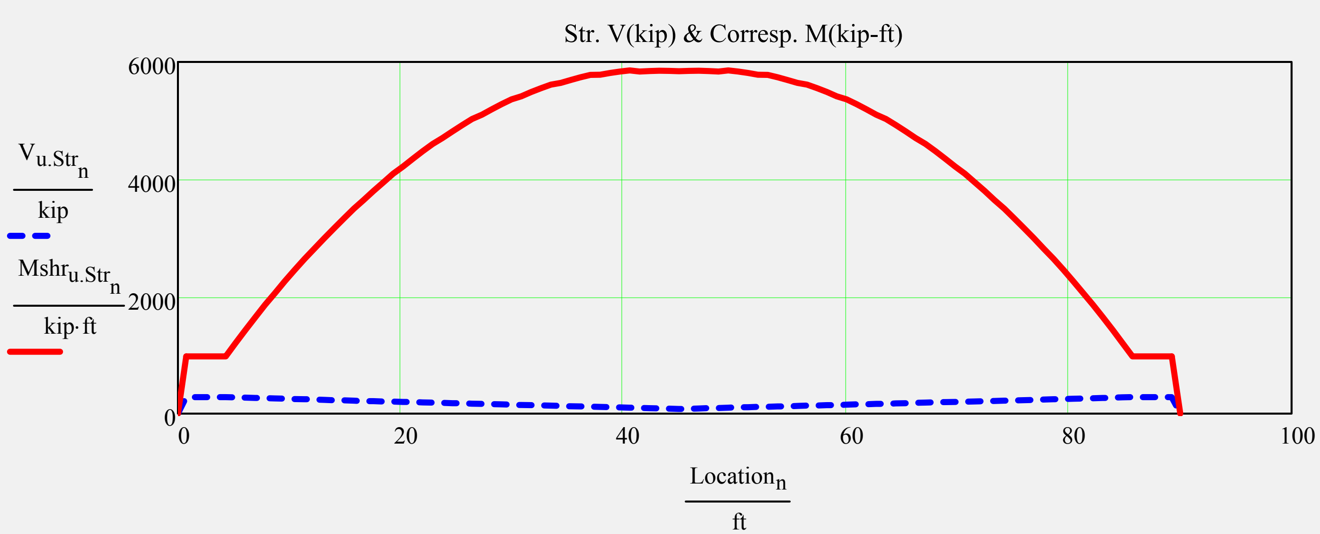
$CR_{Str.mom_n} := 10$ $\qquad$ $CR_{Str.mom_{mn}} := \frac{\phi_{mom_{mn}} \cdot (Mn_{mn})_0}{Mreqd_{mn}}$ $\qquad$ *(LRFD 5.7.3.3.2)*

$\min(CR_{Str.mom}) = 1.09$

$\max(Mreqd) = 6191.0 \cdot kip \cdot ft$

CheckMomentCapacity := if$\left(\min(CR_{Str.mom}) > 0.99, \text{"OK"}, \text{"No Good!"}\right)$ $\qquad$ CheckMomentCapacity = "OK"

Strength Shear and Associated Moments



$\max(V_{u.Str}) = 282.1 \cdot kip$ $\max(Mshr_{u.Str}) = 5856.3 \cdot kip \cdot ft$

Design Shear, Longitudinal, Interface and Anchorage Reinforcement

Stirrup sizes and spacings assigned in input file

Location	spacing tmp_s (in)	Number of Spaces tmp_NumberSpaces	area per stirrup $tmp\_A_{stirrup}$ (in^2)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

Locally assigned stirrup sizes and spacings

To change the values from the input file enter the new values into the vectors below. Input only those that you wish to change. Values less than 0 are ignored.

The interface_factor accounts for situations where not all of the shear reinforcing is embedded in the poured in place slab.

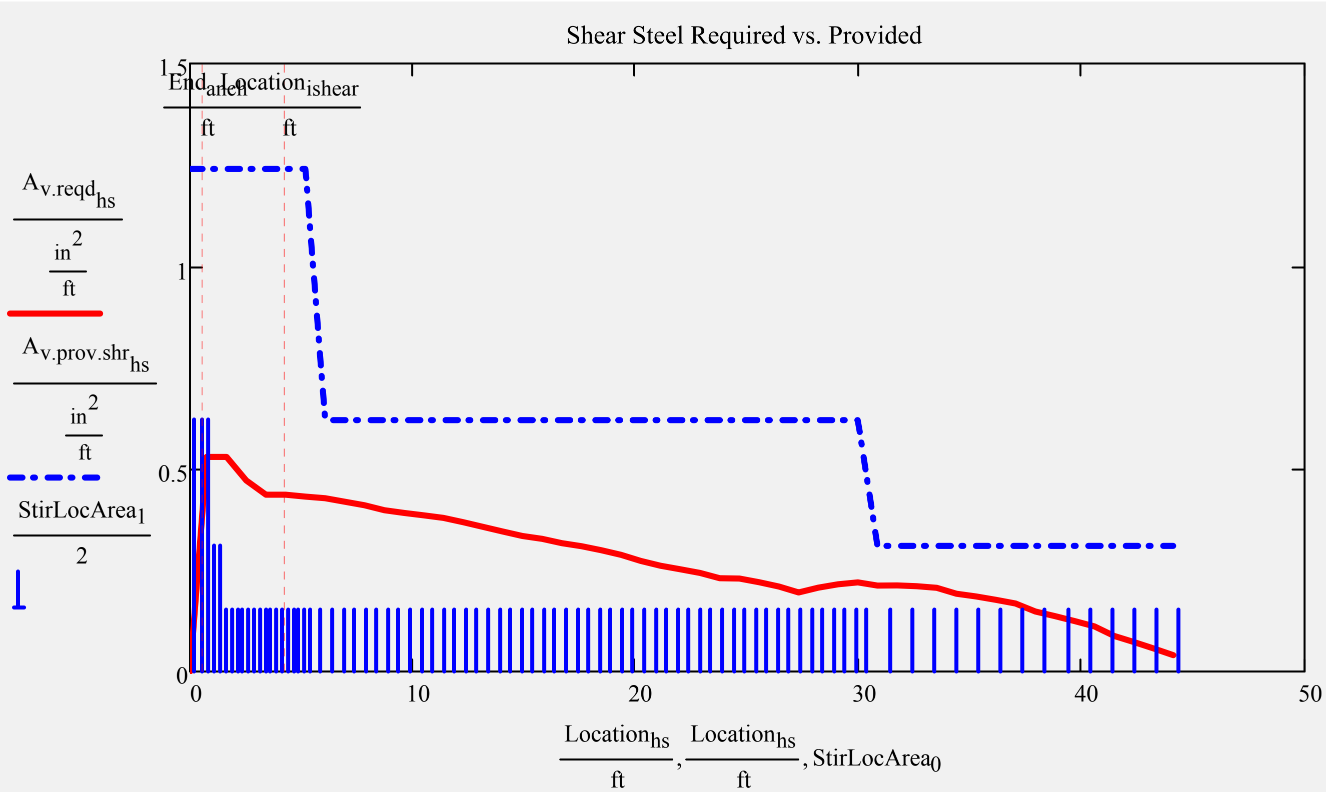
Location	$user\_s_{nspacings}$:=	$user\_NumberSpaces_{nspacings}$:=	$user\_A_{stirrup_{nspacings}}$:=	$interface\_factor_{nspacings}$:=
A1 stirrup	−1·in	−1	−1·in²	0.25
A2 stirrup	−1·in	−1	−1·in²	0.5
A3 stirrup	−1·in	−1	−1·in²	1
S1 stirrup	−1·in	−1	−1·in²	1
S2 stirrup	−1·in	−1	−1·in²	1
S3 stirrup	−1·in	−1	−1·in²	1
S4 stirrup	−1·in	−1	−1·in²	1
			−1·in²	

Stirrup sizes and spacings used in analysis

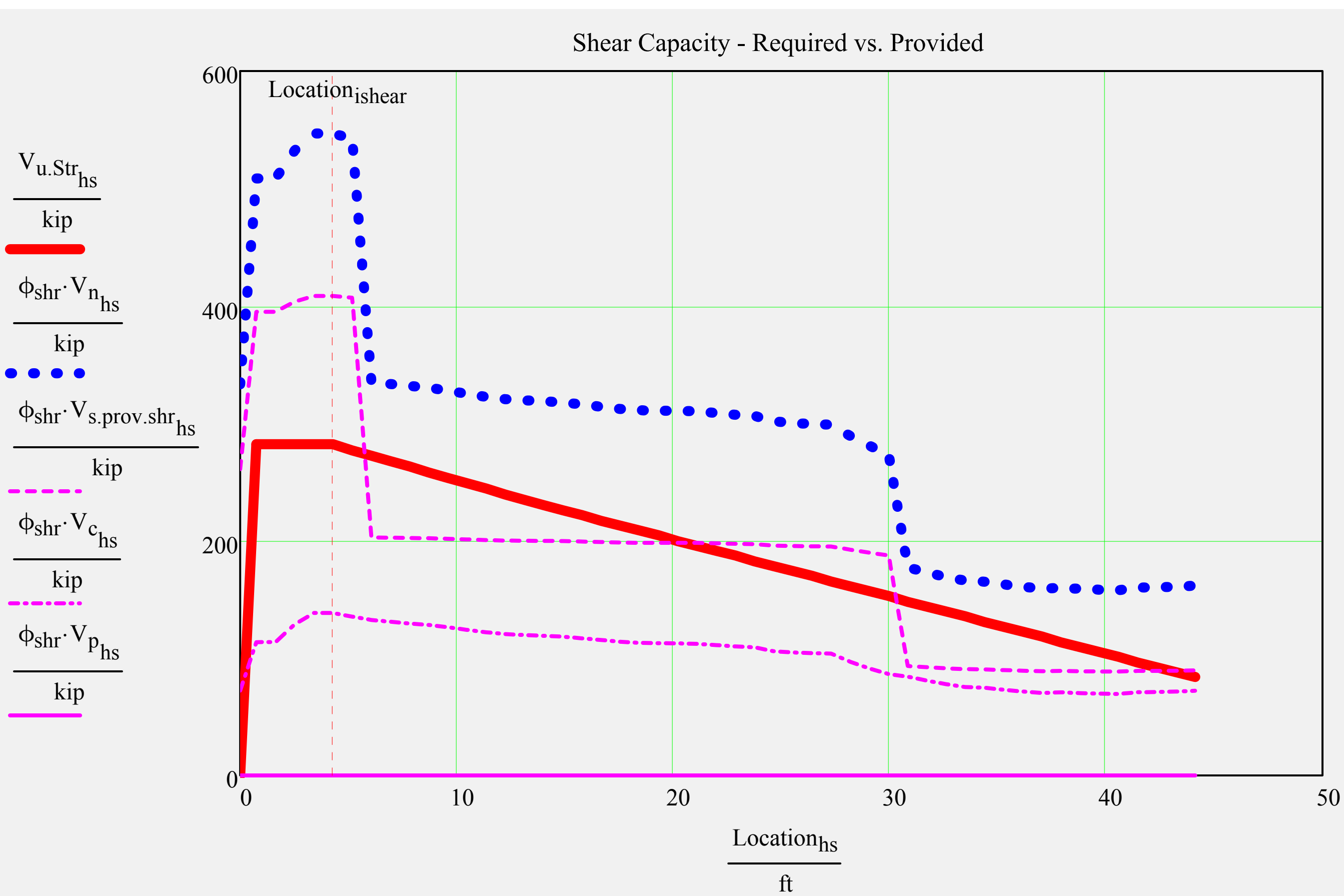
The number of spaces for the S4 stirrup is calculated by the program to complete the half beam length.

Location	s (in)	NumberSpaces	$A_{stirrup}$ (in^2)
A1 stirrup	3.5	2	1.24
A2 stirrup	3.5	2	0.62
A3 stirrup	3	16	0.31
S1 stirrup	6	50	0.31
S2 stirrup	12	3	0.31
S3 stirrup	12	3	0.31
S4 stirrup	12	0	0.31

EndCover = 2.5·in



Shear Steel Required vs. Provided
End_areq / ft
Location_ishear / ft
A_v.reqd_hs / (in^2/ft)
A_v.prov.shr_hs / (in^2/ft)
StirLocArea_1 / 2
1.5
1
0.5
0
0
10
20
30
40
50
Location_hs / ft, Location_hs / ft, StirLocArea_0



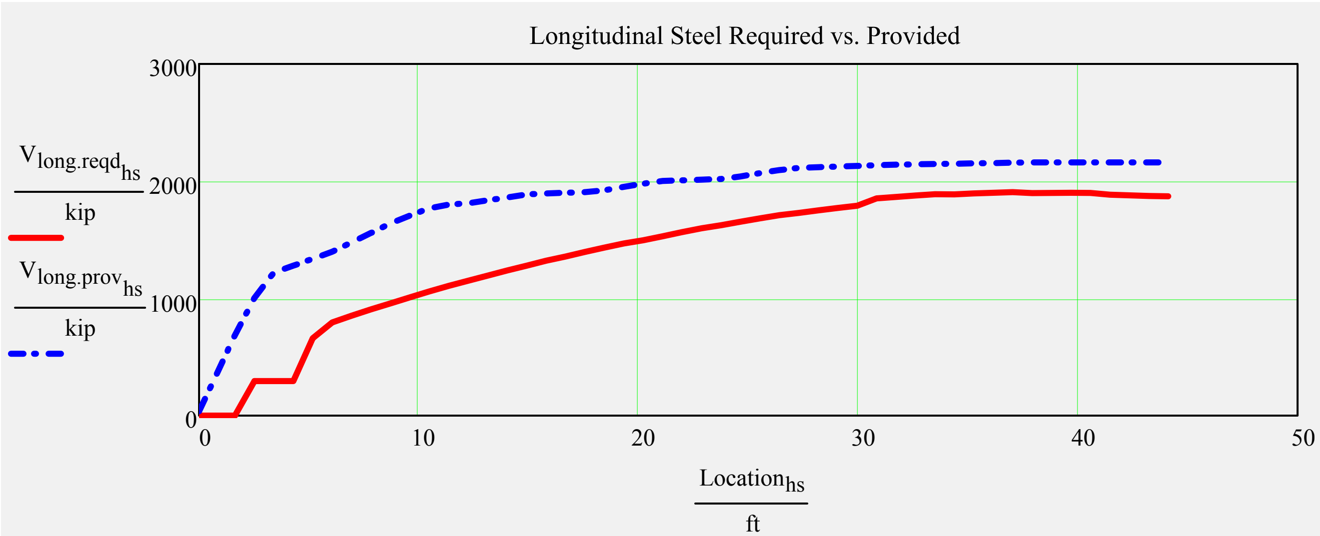
Shear Capacity - Required vs. Provided
Location_ishear
V_u.Str_hs / kip
φ_shr · V_n_hs / kip
φ_shr · V_s.prov.shr_hs / kip
φ_shr · V_c_hs / kip
φ_shr · V_p_hs / kip
600
400
200
0
0
10
20
30
40
50
Location_hs / ft

CheckShearCapacity = "OK"

CheckMinStirArea = "OK"

CheckStirArea = "OK"

CheckMaxStirSpacing = "OK"

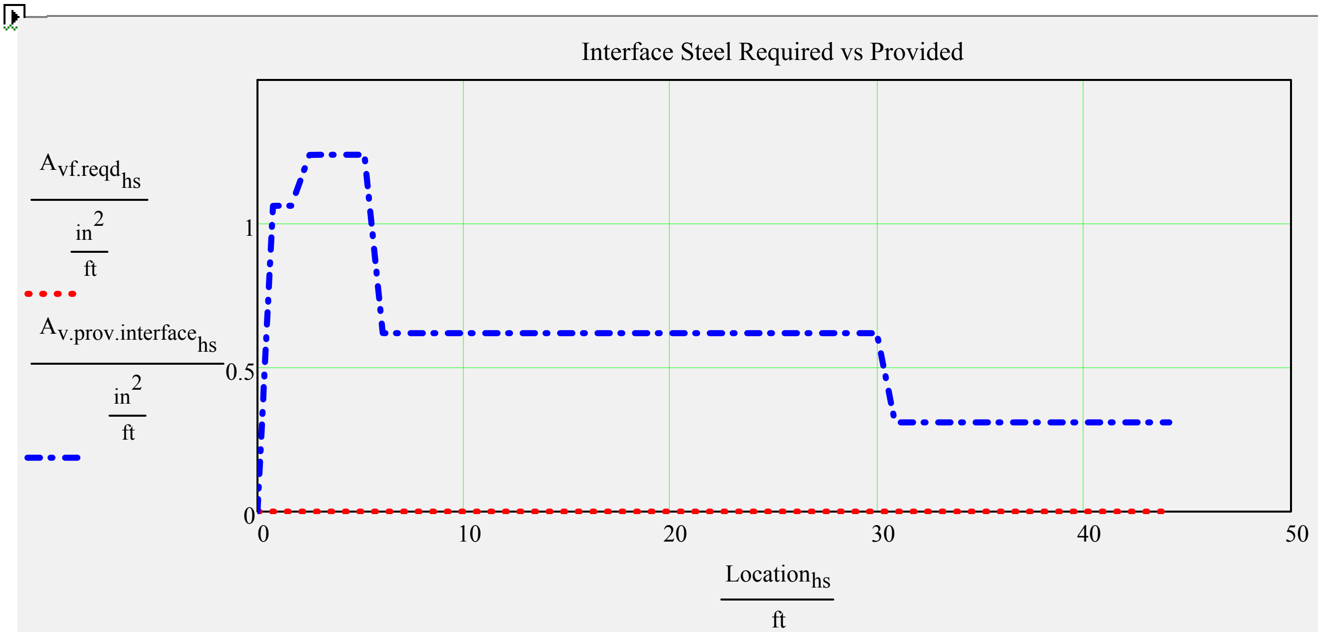


$$CR_{LongSteel_{hs}} := \text{if}\left(V_{long.reqd_{hs}} < .01kip, 100, \frac{V_{long.prov_{hs}}}{V_{long.reqd_{hs}}}\right)$$

$$\min\left(CR_{LongSteel}\right) = 1.13$$

CheckLongSteel := if(min($CR_{LongSteel}$) > 1 ,"OK" ,"No Good, add steel!")

CheckLongSteel = "OK"



Typically shear steel is extended up into the deck slab.
These calculations are based on shear steel functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

$\max(A_{vf.min}) = 0 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface steel is required.

CheckInterfaceSpacing = "OK"

$CheckInterfaceSteel := if\left(\frac{TotalInterfaceSteelProvided}{TotalInterfaceSteelRequired + 0.001 \cdot in^2} \geq 1, "OK", "No Good"\right)$

CheckInterfaceSteel := if(substr(BeamTypeTog, 0, 3) = "FLT", "N.A.", CheckInterfaceSteel)

CheckInterfaceSteel = "OK"

Anchorage Reinforcement and Maximum Prestressing Force

Was FDOT Design Standard splitting reinforcing used? (bars Y,K, & Z)

StandardSplittingReinforcing := yes

if yes-> checks max allowable standard prestress force
if no-> checks stirrup area given input prestress force

CheckSplittingSteel = "N.A." CheckMaxPrestressingForce = "OK"

Summary of Design Checks

$check_0 := AcceptAASHTO$

$check_1 := AcceptSDG$

$check_2 := AcceptOntario$

$check_3 := Check\_f_{pt}$

$check_4 := Check\_f_{pe}$

$check_5 := Check\_f_{tension.rel}$

$check_6 := Check\_f_{comp.rel}$

$check_7 := Check\_f_{tension.stage8}$

$check_8 := Check\_f_{comp.stage8.c1}$

$check_9 := Check\_f_{comp.stage8.c2}$

$check_{10} := Check\_f_{comp.stage8.c3}$

$check_{11} := CheckMomentCapacity$

$check_{12} := CheckMaxCapacity$

$check_{13} := CheckStirArea$

$check_{14} := CheckShearCapacity$

$check_{15} := CheckMinStirArea$

$check_{16} := CheckMaxStirSpacing$

$check_{17} := CheckLongSteel$

$check_{18} := CheckInterfaceSpacing$

$check_{19} := CheckSplittingSteel$

$check_{20} := CheckMaxPrestressingForce$

$check_{21} := CheckPattern_0$

$check_{22} := CheckPattern_1$

$check_{23} := CheckPattern_2$

$check_{24} := CheckPattern_3$

$check_{25} := CheckPattern_4$

$check_{26} := CheckInterfaceSteel$

$check_{27} := CheckStrandFit$

$check_{28} := Check\_SDG_{1.2.Display_2}$

click table to reveal scroll bar...

$check^T =$

	0	1	2	3	4
0	"OK"	"N.A."	"N.A."	"OK"	...

Link to Note- Checks, 0, 1 & 2

TotalCheck = "OK"

LRFR Load Rating Analysis

Structures Manual (SM) Vol-8: FDOT Modifications to LRFR

(SM Vol-8 G.6)

(Load Rating Summary Details for Prestressed Concrete Bridges - Flat Slab and Deck/Girder - Sheet 4)

Moment (Strength) or Stress (Service) — Shear (Strength)

$LRFR_{loadrating}$ =

"Limit State"	"DF"	"Rating"	"Tons"	"Dim(ft)"	"DF"	"Rating"	"Tons"	"Dim(ft)"	
"Strength I(Inv)"	0.88	1.16	"N/A"	53.10	0.94	1.37	"N/A"	5.31	HL-93
"Strength I(Op)"	0.88	1.50	"N/A"	53.10	0.94	1.77	"N/A"	5.31	HL-93
"Service III(Inv)"	0.88	1.07	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Service III(Op)"	0.88	1.16	"N/A"	45.13	"N/A"	"N/A"	"N/A"	"N/A"	HL-93
"Strength II"	0.88	1.22	73.14	53.10	0.94	1.26	75.84	30.09	*Permit
"Service III"	0.88	1.08	65.00	45.13	"N/A"	"N/A"	"N/A"	"N/A"	*Permit

*note: default permit load is FL120 per input worksheet

Longitudinal Steel Check:

$CR_{LongSteel.HL93} = 1.17$ $CR_{LongSteel.Permit} = 1.13$ $CheckLongSteel_{loadrating}$ = "OK"

Alternate C

$ 91.89

Bridge Development Report Relative Cost Estimate

Multiple Span - Continuous Flat Slab

Alternative C

	SB		NB
General Provisions			
Number of Spans	5		5
Typical Span Length	36.0	ft	36.0
Bridge Length (FFBW to FFBW)	180.0	ft	180.0
Bridge Width	56.04	ft	43.08
Bridge Clear Width (Used only for no. of lanes calculation)	52.96		40.00
Design Slab Thickness	21.5	in	21.5
Sacrificial Slab Thickness	0.5	in	0.5
A. Bridge Substructure			
Prestressed Concrete Piling			
Pile Size	18	in	18
End Bent			
Number of Piles	5		4
Pile Spacing	12	ft	12
Length of Piles	85	ft	85
Pile Embedment on Cap	1	ft	1
Intermediate Bent			
Number of Piles	7		6
Pile Spacing	8	ft	7.25
Length of Piles	120	ft	120
Pile Embedment on Cap	1	ft	1
Total Pile Length (All Foundations)	**4210**	ft	**3560**
Substructure Concrete			
End Bent			
Cap			
Length	56.13	ft	43.17
Width	3.25	ft	3.25
Depth	3.00	ft	3.00
Volume	19.9	CY	15.3
Curtain Wall			
Height	22.00	in	22.00
Width	1.00	ft	1.00
Length	6.00	ft	6.00
Volume	0.8	CY	0.8
Total Volume per End Bent	20.7	CY	16.1
Total Volume for the Two End Bents	41.4	CY	32.2
Intermediate Bent			
Cap			
Length	56.04	ft	43.08
Width	3.25	ft	3.25
Depth	3.00	ft	3.00
Volume	19.7	CY	15.1
Total Volume per Intermediate Bent	19.7	CY	15.1
Total Volume for all Intermediate Bents	78.8	CY	60.4
Substructure Total Concrete Volume	**120.2**	CY	**92.6**

			$ 91.89
Reinforcing Steel			
Weight per End Bent (135 lb/CY)	2795	lb	2174
Weight per Intermediate Bent (145 lb/CY)	2857	lb	2190
Substructure Total Reinforcing Steel Weight	**17018**	lb	**13108**
B. Bridge Superstructure			
Slab Concrete			
Minimum Thickness (AASHTO LRFD Requirement)	18.4	in	18.4
Check Slab Thickness	OK		OK
Superstructure Total Concrete Volume	**685.0**	CY	**526.6**
Reinforcing Steel			
SuperstructureTotal Reinforcing Steel Weight (220 lb/CY)	**150690**	lb	**115846**
Railing and Barriers			
Traffic Railing			
Type 32" F Shape			
Total Length	**360**	ft	**360**
Pedestrian Railing	**No**		**No**
Concrete Parapet 27"			
Total Length	**0**	ft	**0**
Bullet Railing			
Total Length	**0**	ft	**0**

Bridge Development Report Pile Loads

End Bent

	SB		NB
General Provisions			
Number of Spans	5		5
Span Length	36.0	ft	36.0
Bridge Width	56.0	ft	43.1
Design Slab Thickness	21.5	in	21.5
Sacrificial Slab Thickness	0.5	in	0.5
Traffic Railing Weight	**420.0**	lb/ft	**420.0**
Pedestrian Railing with Bullet Railing Weight	**235.0**	lb/ft	**235.0**
A. Live Load Reaction at End Bent (based on tributary area)			
Number of Design Lanes	4		3
Multiple Presence Factor	**0.65**		**0.85**
HL-93			
Design Truck Reaction	138.7	kip	136.0
Design Tandem Reaction	122.8	kip	120.4
Design Lane Load	30.0	kip	29.4
Total End Bent Live Load	**168.6**	kip	**165.4**
B. End Bent Dead Loads (based on tributary area)			
Self-Weight			
Cap	80.4	kip	61.8
Curtain Wall	3.3	kip	3.3
Total End Bent Self-Weight Dead Load	**83.7**	kip	**65.1**
Superstructure Weight			
Slab	277.4	kip	213.3
Traffic Railing	15.1	kip	15.1
Pedestrian Railing	0.0	kip	0.0
Total End Bent Superstructure Dead Load	**292.5**	kip	**228.4**
C. Pile Loads			
Factored Reaction at Bent (Strength I) Note: Increased by 15% for preliminary design	880.2	kip	754.7
Number of Piles	5		4
Factored Individual Pile Load	176.0	kip	188.7
Downdrag Force	**0.0**	kip	**0.0**
Phi factor for pile driving	**0.65**		**0.65**
Required driving resistance	135	tons	145

Bridge Development Report Pile Loads

Typical Intermediate Bent

	SB		NB
General Provisions			
Number of Spans	5		5
Span Length	36.0	ft	36.0
Bridge Width	56.0	ft	43.1
Design Slab Thickness	21.5	in	21.5
Sacrificial Slab Thickness	0.5	in	0.5
Traffic Railing Weight	420.0	lb/ft	420.0
Pedestrian Railing with Bullet Railing Weight	235.0	lb/ft	235.0
A. Live Load Reaction at Interior Bent (based on tributary area)			
Number of Design Lanes	4		3
Multiple Presence Factor	0.65		0.85
HL-93			
Design Truck Reaction	146.8	kip	143.9
Design Tandem Reaction	122.8	kip	120.4
Design Lane Load	59.9	kip	58.8
Total Interior Bent Live Load	**206.7**	kip	**202.7**
B. Interior Bent Dead Loads (based on tributary area)			
Self-Weight			
Cap	79.8	kip	61.2
Total Interior Bent Self-Weight Dead Load	**79.8**	kip	**61.2**
Superstructure Weight			
Deck	554.8	kip	426.5
Traffic Railing	30.2	kip	30.2
Pedestrian Railing	0.0	kip	0.0
Total Interior Bent Superstructure Dead Load	**585.1**	kip	**456.8**
C. Pile Loads			
Factored Reaction at Bent (Strength I) Note: Increased by 15% for preliminary design	1371.6	kip	1152.4
Number of Piles	7		6
Factored Individual Pile Load	195.9	kip	192.1
Scour Resistance	**5.00**	kip	**5.00**
Phi factor for pile driving	**0.65**		**0.65**
Required driving resistance	155	tons	152

Bridge Development Report Cost Estimating

Effective 1/01/2012

Step One: Estimate Component Items

Utilizing the cost provided herein, develop the cost estimate for each bridge type under consideration.

A. Bridge Substructure

1. Prestressed Concrete Piling, (furnished and installed)			
Size of Piling	**Cost per Lin. Foot [1]**	**Quantity**	**Cost**
18" (Driven Plumb or 1" Batter)	$65	7770	$505,050
18" (Driven Battered)	$75		
24" (Driven Plumb or 1" Batter)	$85		
24" (Driven Battered)	$95		
30" (Driven Plumb or 1" Batter)	$120		
30" (Driven Battered)	$140		
Heavy mild steel reinforcing in pile head (each)	$250		
Embedded Data Collector (each)	$2,000		
		Subtotal	$505,050

1 When silica fume, metakaolin or ultrafine fly ash is used add $6/LF to the piling cost.

2. Steel Piling, (furnished and installed)			
Size of Piling	**Cost per Lin. Foot**	**Quantity**	**Cost**
14 x 73 H Section	$70		
14 x 89 H Section	$90		
20" Pipe Pile	$105		
24" Pipe Pile	$114		
30" Pipe Pile	$160		
		Subtotal	

3. Drilled Shaft (Total in-place cost)			
Dia. (on land, casing salvaged)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$250		
4 ft	$430		
5 ft	$510		
6 ft	$630		
7 ft	$750		
Dia. (in water, casing salvaged)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$320		
4 ft	$500		
5 ft	$600		
6 ft	$690		
7 ft	$800		
8 ft	$1,100		
Dia. (in water, permanent casing)	**Cost per Lin. Foot**	**Quantity**	**Cost**
3 ft	$460		
4 ft	$625		
5 ft	$750		
6 ft	$950		
7 ft	$1,100		
8 ft	$1,500		
9 ft	$1,800		
		Subtotal	

A. Bridge Substructure (continued)

4. Sheet Piling Walls			
Size (Prestressed Concrete)	**Cost per Lin. Foot**	**Quantity**	**Cost**
10" x 30"	$100		
12" x 30"	$110		
Type (Steel)	**Cost per Sq. Foot**	**Quantity**	**Cost**
Permanent Cantilever Wall	$24		
Permanent Anchored Wall [1]	$36		
Temporary Cantilever Wall	$14		
Temporary Anchored Wall [1]	$22		
Soil Anchors	**Cost per Anchor**	**Quantity**	**Cost**
Permanent	$3,200		
Temporary	$2,800		
		Subtotal	

1 Includes the cost of waler steel, miscellaneous steel for permanent/temporary walls and concrete face for permanent walls.

5. Cofferdam Footing (Cofferdam and Seal Concrete[1])			
Prorate the cost provided herein based on area and depth of water. A cofferdam footing having the following attributes cost $600,000: Area 63 ft x 37.25 ft; Depth of seal 5 ft; Depth of water over footing 16 ft			
Type	**Cost per Footing**	**Quantity**	**Cost**
Cofferdam Footing			
		Subtotal	

1 Cost of seal concrete included in pay item 400-3-20 or 400-4-200.

6. Substructure Concrete			
Type	**Cost per Cubic Yard**	**Quantity**	**Cost**
Concrete [1]	$575	212.8	$122,360
Mass Concrete [1]	$512		
Seal Concrete [1]	$412		
Bulkhead Concrete [1]	$925		
Shell Fill [1]	$30		
		Subtotal	$122,360

1 Admixtures: For Calcium Nitrite add $40/cy (@4.5 gal/cy) and for silica fume, metakaolin or ultrafine fly ash add $40/cy (@ 60 lb./cy)

7. Reinforcing Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Reinforcing Steel	$0.90	30126	$27,113
		Subtotal	$27,113

Substructure Subtotal **$654,523**

B. Bridge Superstructure

1. Bearing Material			
Type	**Cost per Cubic Foot**	**Quantity**	**Cost**
Neoprene Bearing Pads	**$900**		
Multirotational Bearings (kips)	**Cost per Each**	**Quantity**	**Cost**
1- 250	$6,000		
251- 500	$7,000		
501- 750	$8,000		
751-1000	$9,500		
1001-1250	$9,900		
1251-1500	$10,000		
1501-1750	$11,000		
1751-2000	$12,500		
>2000	$15,000		
		Subtotal	

2. Bridge Girders			
Structural Steel (includes coating)	**Cost per Pound**	**Quantity**	**Cost**
Rolled Wide Flange Sections, straight [1]	$1.35		
Rolled Wide Flange Sections, curved [1]	$1.70		
Plate Girders, Straight [1]	$1.50		
Plate Girders, Curved [1]	$1.70		
Box Girders, Straight [1]	$1.75		
Box Girders, Curved [1]	$1.85		
Prestressed Concrete Girders	**Cost per Lin. Foot**	**Quantity**	**Cost**
Fl. Inverted Tee 16" [2]	$80		
Fl. Inverted Tee 20"	$90		
Fl. Inverted Tee 24" [2]	$105		
Fl. Tub (U-Beam) 48" [2]	$700		
Fl. Tub (U-Beam) 54"	$750		
Fl. Tub (U-Beam) 63"	$800		
Fl. Tub (U-Beam) 72"	$900		
Solid Flat Slab (<48"x12")	$150		
Solid Flat Slab (<48"x15")	$160		
Solid Flat Slab (48"x12")	$160		
Solid Flat Slab (48"x15")	$170		
Solid Flat Slab (60"x12")	$170		
Solid Flat Slab (60"x15")	$180		
Florida-I; 36	$175		
Florida-I; 45	$185		
Florida-I; 54	$200		
Florida-I; 63	$225		
Florida-I; 72	$250		
Florida-I; 78	$265		
Florida-I; 84	$320		
Florida-I; 96	$400		
Haunched Florida-I; 78	$700		
Haunched Florida-I; 84	$800		
		Subtotal	

1 When weathering steel (uncoated) is used reduce the price by $0.04 per pound. Inorganic zinc coating systems have an expected life cycle of 20 years.
2 Price is based on ability to furnish products without any conversions of casting beds and without pu

B. Bridge Superstructure (continued)

3. Cast-in-Place Superstructure Concrete			
Type	**Cost per Cubic Yard**	**Quantity**	**Cost**
Box Girder Concrete, Straight	$950		
Box Girder Concrete, Curved	$1,100		
Deck Concrete	$600	1211.5	$726,917
Precast Deck Overlay Concrete Class IV	$600		
		Subtotal	$726,917

4. Concrete for Precast Segmental Box Girders, Cantilever Construction			
Concrete Cost by Deck Area	**Cost per Cubic Yard**	**Quantity**	**Cost**
≤ 300,000 SF	$1,250		
> 300,000 SF AND ≤ 500,000 SF	$1,200		
> 500,000 SF	$1,150		
		Subtotal	

5. Reinforcing Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Reinforcing Steel	$0.60	266536	$159,922
		Subtotal	$159,922

6. Post-Tensioning Steel			
Type	**Cost per Pound**	**Quantity**	**Cost**
Strand, Longitudinal	$2.50		
Strand, Transverse	$4.00		
Bars	$6.00		
		Subtotal	

7. Railings and Barriers			
Type	**Cost per Lin. Foot**	**Quantity**	**Cost**
Traffic Railing[1]	$70	720	$50,400
Pedestrian/Bicycle Railings:			
Concrete Parapet (27")[1]	$65		
Single Bullet Railing[1]	$27		
Double Bullet Railing[1]	$36		
Triple Bullet Railing[1]	$45		
Picket Railing (42") steel	$65		
Picket Railing (42") aluminum	$50		
Picket Railing (54") steel	$95		
Picket Railing (54") aluminum	$60		
1 Combine cost of Bullet Railings with Concrete Parapet or Traffic Railing, as appropriate.		**Subtotal**	$50,400

8. Expansion Joints			
Type	**Cost per Lin. Foot**	**Quantity**	**Cost**
Strip Seal	$360		
Finger Joint <6"	$850		
Finger Joint >6"	$1,500		
Modular 6"	$500		
Modular 8"	$700		
Modular 12"	$900		
		Subtotal	

Superstructure Subtotal $937,238

C. Miscellaneous Items

1. MSE Walls

Type	Cost per Sq. Foot	Quantity	Cost
Permanent	$26		
Temporary	$14		
		Walls Subtotal	

2. Sound Barriers

Type	Cost per Sq. Foot	Quantity	Cost
Post and Panel Sound Barriers	$25		
	Sound Barrier Subtotal		

3. Detour Bridges

Type	Cost per Sq. Foot	Quantity	Cost
Acrow Detour Bridge [1]	$55		
	Detour Bridge Subtotal		

1 Using FDOT supplied components. The cost is for the bridge proper and does not include approach work, surfacing, or guardrail.

Unadjusted Total $1,591,762

Step Two: Estimate Conditional Variables and Cost per Square Foot

After developing the total cost estimate utilizing the unit cost, modify the cost to account for site condition variables. If appropriate, the cost will be modified by the following variables:

Conditional Variables	% Increase/ Decrease	Cost (+/-)
For construction over water, increase cost by 3 %.	3%	$47,753
Phased construction or widening, increase by 20 %. [1]		
	3%	$47,753

1 Phased construction is defined as construction over traffic or construction requiring multiple phases to complete the construction of the entire cross section of the bridge. The 20 percent premium is applied to the affected units of the superstructure

Substructure Subtotal	$654,523
Superstructure Subtotal	$937,238
Walls Subtotal	
Sound Barrier Subtotal	
Detour Bridge Subtotal	
Conditional Variables	$47,753
Total Cost	$1,639,515
Total Square Feet of Deck	17843
Cost per Square Foot	$92

Design Aid for Determination of Reinforcing Steel

In the absence of better information, use the following quantities of reinforcing steel per cubic yard of concrete.

Location	Pounds of Steel	Cubic Yds.	Tot. Pounds
Pile Abutments	135		
Pile Bents	145		
Single Column Piers >25'	210		
Single Column Piers <25'	150		
Multiple Column Piers >25'	215		
Multiple Column Piers <25'	195		
Bascule Piers	110		
Standard Deck Slabs	205		
Isotropic Deck Slabs	125		
Concrete Box Girders, Pier Seg	225		
Concrete Box Girders, Typ. Seg	165		
Flat Slabs @ 30ft & 15" Deep	220		

Step Three: Cost Estimate Comparison to Historical Bridge Cost

The final step is a comparison of the cost estimate by comparison with historic bridge cost based on a cost per square foot. These total cost numbers are calculated exclusively for the bridge cost as defined in the General Section of this chapter. Price

Bridge Superstructure Type	Total Cost per Square Foot Low	High
Short Span Bridges:		
Reinforced Concrete Flat Slab- Simple Span [1]	$92	$160
Pre-cast Concrete Slab - Simple Span [1]	$81	$200
Medium Span Bridges:		
Concrete Deck / Steel Girder - Simple Span [1]	$125	$142
Concrete Deck / Steel Girder - Continuous Span [1]	$135	$170
Concrete Deck / Prestressed Girder - Simple Span [1]	$66	$145
Concrete Deck / Prestressed Girder - Continuous Span [1]	$83	$211
Concrete Deck / Steel Box Girder [1] - Span range from 150' to 280' (for curvature, add 15% premium)	$100	$165
Segmental Concrete Box Girders - Cantilever Construction Span range from 150' to 280'	$130	$160
Movable Bridge - Bascule Spans & Piers	$1,800	$2,000
Demolition Costs:		
Typical	$35	$60
Bascule	$60	$70
Project Type		
Widening (Construction Only)	$85	$160

1 Increase the cost by twenty percent for phased construction

Estimated Cost per Square Foot $92

Change Log

Reference:G:\0Production\reference.xmcd

Flat Slab Design
Southbound Bridge

1.0 GEOMETRY

Overall bridge length......................... $L_{bridge} \equiv 180\cdot ft$

Length of spans.............................. $L := 36\cdot ft$

Overall bridge width......................... $W_{bridge} := 56\cdot ft + 0.5\cdot in$

Skew.. $\theta := 20\cdot deg$

Span to Depth Ratio

[AASHTO LRFD 2.5.2.6.3]

For continuous reinforced slabs with main reinforcement parallel to traffic

$$t_{min} = \frac{S + 10}{30} \ge 0.54\cdot ft$$

Minimum slab thickness

$$t_{min} := \max\left(\frac{L + 10\cdot ft}{30}, 0.54\cdot ft\right) \qquad t_{min} = 18.4\cdot in$$

Preliminary design thickness of flat slab........ $t_{slab} := 21.5\cdot in$
(exluding 0.5 in sacrificial for deck planning)

2.0 LOADS

2.1 Dead Loads

Weight of future wearing surface (SDG 2.2 & 4.2) **[FDOT SDG 2.2 & 4.2]**

$$\rho_{fws} := \begin{cases} 15\cdot psf & \text{if } L_{bridge} \le 100ft \\ 0\cdot psf & \text{otherwise} \end{cases} \qquad \rho_{fws} = 0\cdot psf$$

Sacrificial Slab Thickness

$$t_{sac} := \begin{cases} 0.5\cdot in & \text{if } L_{bridge} > 100ft \\ 0\cdot in & \text{otherwise} \end{cases} \qquad t_{sac} = 0.5\cdot in$$

Weight of traffic railing barrier............. $W_{barrier} := 420\cdot plf$

32" F-Shape (SDG 2.2)

Unit weight of concrete...................... $\gamma_{conc} := 150\cdot pcf$

2.2 Live Loads

The design is based on the HL-93 Design Load.

2.2.1 Design Lanes

Current lane configurations show two striped lanes and two shoulders. Using the roadway clear width between barriers, $Rdwy_{width}$, the number of design traffic lanes per roadway, N_{lanes}, can be calculated as:

Barrier width.................................... $b_{barrier} := 18.5\cdot in$

Roadway clear width

$$Rdwy_{width} := W_{bridge} - 2\cdot b_{barrier} \qquad Rdwy_{width} = 53\,ft$$

Number of design traffic lanes per roadway

$$N_{lanes} := floor\left(\frac{Rdwy_{width}}{12\cdot ft}\right) \qquad N_{lanes} = 4$$

2.2.2 Distribution

Based on the LRFD Section 4 "Structural Analysis and Evaluation"

The superstructure is designed on a per foot basis longitudinally. However, in order to distribute the live loads, equivalent strips of flat slab deck widths are calculated. The moment and shear effects of a single HL-93 vehicle or multiple vehicles are divided by the appropriate equivalent strip width. The equivalent strips account for the transverse distribution of LRFD wheel loads. Multiple presence factors are already taken into consideration in the following equations

One design lane

The equivalent width of longitudinal strips per lane for both shear and moment with one lane loaded:

$E = 10 + 5.0\sqrt{L_1 \cdot W_1}$ **[AASHTO LRFD 4.6.2.3-1]**

where

L_1, modified span length taken equal to the lesser of the actual span or 60 feet

$L_1 := \min(L, 60.0 \cdot ft)$ $L_1 = 36\,ft$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 30 feet for single lane loading

$W_1 := \min(W_{bridge}, 30.0 \cdot ft)$ $W_1 = 30\,ft$

The equivalent distribution width for one lane loaded is given as:

$$E_{OneLane} := \left(10 + 5.0\sqrt{\frac{L_1}{ft} \frac{W_1}{ft}}\right) \cdot in \qquad E_{OneLane} = 14.5\,ft$$

Two or more design lanes

The equivalent width of longitudinal strips per lane for both shear and moment with more than one lane loaded:

$E = 84 + 1.44\sqrt{L_1 \cdot W_1} \le \frac{12.0W}{N_L}$ **[AASHTO LRFD 4.6.2.3-2]**

where

L_1, modified span length taken equal to the lesser of the actual span or 60 feet

$L_1 := \min(L, 60.0 \cdot ft)$ $L_1 = 36\,ft$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 60 feet for single lane loading

$W_1 := \min(W_{bridge}, 60.0 \cdot ft)$ $W_1 = 56\,ft$

N_L, number of design lanes

$N_L := N_{lanes}$ $N_L = 4$

The equivalent distribution width for more than one lane loaded is given as:

$$E_{TwoLane} := \min\left[\left(84 + 1.44\sqrt{\frac{L_1}{ft} \frac{W_1}{ft}}\right), \frac{12.0\left(\frac{W_{bridge}}{ft}\right)}{N_L}\right] \cdot in \qquad E_{TwoLane} = 12.4\,ft$$

Longitudinal force effects reduction factor for skewed bridges

$r := \min(1.05 - 0.25\tan(\theta), 1)$ **[AASHTO LRFD 4.6.2.3-3]** $r = 0.96$

The design strip width to use would be the one that causes the maximum effects. In this case, it would be the minimum value of the two equivalent strip widths

$$E := \frac{\min(E_{OneLane}, E_{TwoLane})}{r}$$

$E = 12.9\ ft$

3.0 LOAD ANALYSIS

3.1 Dead Loads

Barrier weight is assumed to be equally distributed thought the entire width of the bridge. Then, the distributed dead load per 1-foot wide slab strip is as follows:

Slab self weight

$$w_{slab} := (t_{slab} + t_{sac}) \cdot \gamma_{conc} \cdot (1 \cdot ft) \qquad w_{slab} = 0.275 \cdot klf$$

Barrier weight

$$w_{barrier} := \frac{W_{barrier} \cdot 2}{W_{bridge}} \cdot (1 \cdot ft) \qquad w_{barrier} = 0.015 \cdot klf$$

Total distributed dead load, DC

$$w_{DC} := w_{slab} + w_{barrier} \qquad w_{DC} = 0.290 \cdot klf$$

The following shears and moments are determined using beam equations for a 5 span continuous system.

Maximum Support Reactions

$$R_{DC} := \frac{43}{38} \cdot w_{DC} \cdot L \qquad R_{DC} = 11.8 \cdot kip$$

Maximum Shear

$$V_{DC} := \frac{23}{38} \cdot w_{DC} \cdot L \qquad V_{DC} = 6.3 \cdot kip$$

Maximum Moment

$$M_{DC} := .105 \cdot w_{DC} \cdot L^2 \qquad M_{DC} = 39.5 \cdot kip \cdot ft$$

3.2 Live Loads

$w_{Lane} := 640plf$

$P_{Truck} := 72kip$

Maximum Support Reactions

$R_{LL} := \frac{43}{38} \cdot w_{Lane} \cdot L + 1.33P_{Truck}$ $\quad R_{LL} = 121.8 \cdot kip$

Maximum Shear

$V_{LL} := \frac{23}{38} \cdot w_{Lane} \cdot L + 1.33P_{Truck}$ $\quad V_{LL} = 109.7 \cdot kip$

Maximum Moment

$M_{LLpos} := .078 \cdot w_{Lane} \cdot L^2 + 1.33P_{Truck} \cdot L \cdot .171$ $\quad M_{LLpos} = 654.2 \cdot kip \cdot ft$

$M_{LLneg} := .105 \cdot w_{Lane} \cdot L^2 + 1.33P_{Truck} \cdot L \cdot .158$ $\quad M_{LLneg} = 631.8 \cdot kip \cdot ft$

3.3 Strength I Factored Loads

Based on LRFD Tables 3.4.1-1 and 3.4.1-2, the load combination for the Strength I limit state is as follows:

$StrengthI = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot (LL + IM)$ **[AASHTO LRFD 3.4.1]**

Factored Shear

$V_{strI} := 1.25V_{DC} + 1.75 \frac{V_{LL} \cdot ft}{E}$ $\quad \max(V_{strI}) = 22.8 \cdot kip$

Factored Moments

$M_{strI.pos} := 1.25 \cdot M_{DC} + 1.75 \frac{M_{LLpos} \cdot ft}{E}$ $\quad \max(M_{strI.pos}) = 137.9 \cdot kip \cdot ft$

$M_{strI.neg} := 1.25M_{DC} + 1.75 \frac{M_{LLneg} \cdot ft}{E}$ $\quad \min(M_{strI.neg}) = 134.9 \cdot kip \cdot ft$

4.0 FLEXURAL DESIGN

The flexure resistance factor for reinforced concrete is given in LRFD 5.5.4.2

For tension-controlled **[AASHTO LRFD 5.5.4.2.1]** $\phi := 0.9$

Factored resistance

$$M_r = \phi \cdot M_n$$

Minimum 28-day compressive strength of concrete
Class II (Bridge Deck) $f_{c.slab} := 4.5 \cdot ksi$

Reinforcing Steel Grade 60 $f_y := 60 \cdot ksi$

Modulus of Elasticity of Reinforcing Steel $E_s := 29000 \cdot ksi$

Minimum Concrete Cover

concrete cover for the slab **[FDOT SDG 1.4.2]**

Concrete cover for the top of slab.......

$$top\_cover_{slab} := \begin{cases} 2 \cdot in & \text{if } L_{bridge} < 100ft \\ 2.5 \cdot in & \text{otherwise} \end{cases}$$

$top\_cover_{slab} = 2.5 \cdot in$

Concrete cover for the bottom of slab.. $bottom\_cover_{slab} := 2 \cdot in$

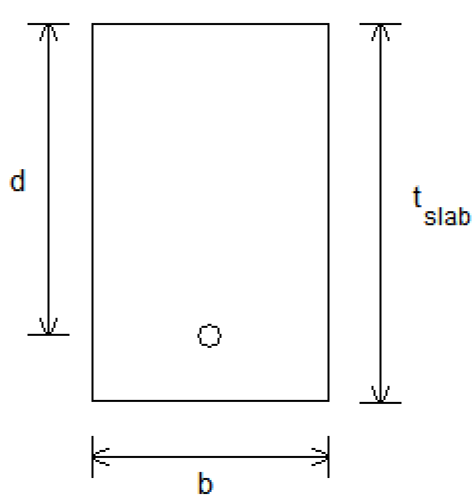
4.1 Positive Moment

Maximum moment for Strength I Limit State

$$M_{r.pos} := \max\left(M_{strI.pos}\right)$$

$M_{r.pos} = 137.9 \cdot kip \cdot ft$

Simplified nominal flexural resistance



$$M_n = A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \quad \text{where} \quad a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

Substituting....

$$M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$$

$$d = t_{slab} - bottom\_cover_{slab} - \frac{d_{bar}}{2}$$

Design Strip Width...... $b := 12 \cdot in$

First, the bar size and the spacing are assumed

Bar Size $d_{bar\_pos} := db8E$

Area of the Bar $A_{bar\_pos} := As8E$

Bar Spacing $s_{bottom} := 5 \cdot in$

Area of Steel per Strip Width

$$A_{s.pos} := A_{bar\_pos} \cdot \frac{b}{s_{bottom}} = 1.9 \cdot in^2 \qquad A_{s.pos} = 1.9 \cdot in^2$$

$$d_p := t_{slab} - bottom\_cover_{slab} - \frac{d_{bar\_pos}}{2} \qquad d_p = 19 \cdot in$$

Given
$$M_{r.pos} = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_p - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$$

$$A_{s.pos.reqd} := Find(A_{s.pos}) \qquad A_{s.pos.reqd} = 1.71 \cdot in^2$$

$$if(A_{s.pos} < A_{s.pos.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}) = \text{"OK"}$$

Check the strain in the reinforcement based on a rectangular stress distribution to ensure that the resistance factor was appropriately assumed

Rectangular distribution factor **[AASHTO LRFD 5.7.2.2]**

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_{c.slab}}{ksi} - 4 \right) \qquad \beta_1 = 0.825$$

Distance between the neutral axis and the compressive face

$$c = \frac{a}{\beta_1}$$

Substituting,
$$c_p := \left(\frac{A_{s.pos} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b} \right) \qquad c_p = 3 \cdot in$$

Based on an ultimate stress in the concrete of 0.003, the strain in the steel is:

$$\frac{0.003}{c} = \frac{\varepsilon_s}{d - c} \quad \text{or} \quad \varepsilon_s = \frac{0.003 \cdot (d - c)}{c}$$

Substituting,
$$\varepsilon_{sp} := \frac{0.003 \cdot (d_p - c_p)}{c_p} \qquad \varepsilon_{sp} = 0.016$$

[AASHTO LRFD 5.7.2.1]

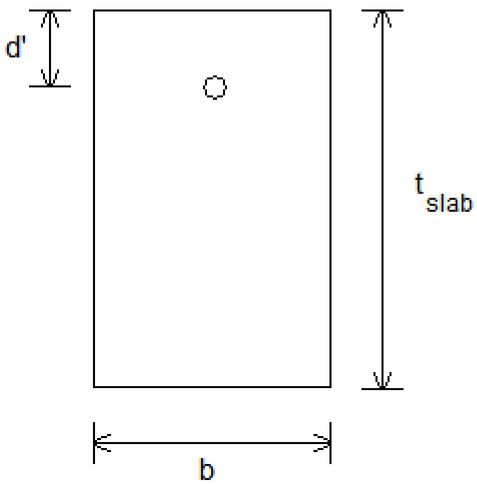
$$if(\varepsilon_{sp} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}) = \text{"Tension Controlled"}$$

4.2 Negative Moment

Maximum moment for Strength I Limit State

$M_{r.neg} := \left|\min\left(M_{strI.neg}\right)\right|$ $\qquad M_{r.neg} = 134.9 \cdot kip \cdot ft$

Similar to positive moment:



$$M_r = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[\left(t_{slab} - d'\right) - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$$

$$d' = \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{bar}}{2}$$

First, the bar size and the spacing are assumed

Bar Size $\qquad d_{bar\_neg} := db8E$

Area of the Bar $\qquad A_{bar\_neg} := As8E$

Bar Spacing $\qquad s_{top} := 5 \cdot in$

Area of Steel per Strip Width

$A_{s.neg} := A_{bar\_neg} \cdot \frac{b}{s_{top}} = 1.9 \cdot in^2$ $\qquad A_{s.neg} = 1.9 \cdot in^2$

$d' := \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{bar\_neg}}{2}$ $\qquad d' = 2.5 \cdot in$

$d_n := t_{slab} - d'$ $\qquad d_n = 19 \cdot in$

Given $\qquad M_{r.neg} = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_n - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

$A_{s.neg.reqd} := Find\left(A_{s.neg}\right)$ $\qquad A_{s.neg.reqd} = 1.67 \cdot in^2$

$if\left(A_{s.neg} < A_{s.neg.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}\right) = \text{"OK"}$

Check the strain in the reinforcement based on a rectangular stress distribution (LRFD 5.7.2.2) to ensure that the resistance factor is appropriate

$$c_n := \left(\frac{A_{s.neg} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b} \right) \qquad c_n = 3 \cdot in$$

The strain in the steel is:

$$\varepsilon_{sn} := \frac{0.003 \cdot \left(d_n - c_n\right)}{c_n} \qquad \varepsilon_{sn} = 0.016$$

$$\text{if}\left(\varepsilon_{sn} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}\right) = \text{"Tension Controlled"}$$

5.0 EDGE BEAM DESIGN

Strip width based on LRFD 4.6.2.1.4b "Longitudinal Edges" which states that "edge beams shall be assumed to support one line of wheels and, where appropriate, a tributary portion of the design lane load"

Distribution

$$E_{EB} = \frac{E_{OneLane}}{4} + b_{barrier} + 12 \cdot in \leq \frac{E_{OneLane}}{2} \leq 72 \cdot in$$

$$E_{EB} := \frac{\min\left(\frac{E_{OneLane}}{4} + b_{barrier} + 12 \cdot in, \frac{E_{OneLane}}{2}, 72 \cdot in\right)}{r} \qquad E_{EB} = 6.3\,ft$$

The tributary portion of the design lane and the truck load are as follows:

$$Factor_{LL} := \frac{E_{EB}}{E} \qquad Factor_{LL} = 0.48$$

5.1 Load Analysis

Dead Load

Edge strip is assumed to carry the weight of the traffic barrier

Total Distributed Load

$$w_{DC.e} := w_{slab} + \frac{W_{barrier} \cdot ft}{E_{EB}} \qquad w_{DC.e} = 0.342 \cdot klf$$

Maximum Support Reactions

$$R_{DC.e} := \frac{43}{38} \cdot w_{DC.e} \cdot L \qquad R_{DC.e} = 13.9 \cdot kip$$

Maximum Shear

$$V_{DC.e} := \frac{23}{38} \cdot w_{DC.e} \cdot L \qquad V_{DC.e} = 7.5 \cdot kip$$

Maximum Moment

$$M_{DC.e} := .105 \cdot w_{DC.e} \cdot L^2 \qquad M_{DC.e} = 46.6 \cdot kip \cdot ft$$

Live Load

Use the same live load moments shown in Section 3.2.

5.2 Strength I Factored Loads

Factored Moments

$$M_{EB.sI.pos} := 1.25 M_{DC.e} + 1.75 \left(\frac{M_{LLpos} \cdot ft \cdot Factor_{LL}}{E_{EB}} \right) \qquad M_{EB.sI.pos} = 146.8 \cdot kip \cdot ft$$

$$M_{EB.sI.neg} := 1.25 M_{DC.e} + 1.75 \left(\frac{M_{LLneg} \cdot ft \cdot Factor_{LL}}{E_{EB}} \right) \qquad M_{EB.sI.neg} = 143.8 \cdot kip \cdot ft$$

Check if design is required for edge beams

Positive Moment

$$Pos_{EB} := if\left(M_{r.pos} > max\left(M_{EB.sI.pos}\right), \text{"Use Interior"}, \text{"Design Edge Beam"}\right)$$

$$Pos_{EB} = \text{"Design Edge Beam"}$$

Negative Moment

$$Neg_{EB} := if\left(\left|M_{r.neg}\right| > \left|min\left(M_{EB.sI.neg}\right)\right|, \text{"Use Interior"}, \text{"Design Edge Beam"}\right)$$

$$Neg_{EB} = \text{"Design Edge Beam"}$$

6.0 FLEXURAL DESIGN

Similar to interior strip

6.1 Positive Moment

Maximum moment for Strength I Limit State

$M_{r.pos.e} := M_{EB.sI.pos}$ $\quad M_{r.pos.e} = 146.8 \cdot kip \cdot ft$

Similar to the interior strip design:

$$M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right] \qquad d = t_{slab} - bottom\_cover_{slab} - \frac{d_{bar}}{2}$$

First, the bar size and the spacing are assumed

Bar Size $\quad d_{e.bar\_pos} := db8E$

Area of the Bar $\quad A_{e.bar\_pos} := As8E$

Bar Spacing $\quad s_{e.bottom} := 5 \cdot in$

Area of Steel per Strip Width

$A_{s.e.pos} := A_{e.bar\_pos} \cdot \frac{b}{s_{e.bottom}}$ $\quad A_{s.e.pos} = 1.9 \cdot in^2$

$d_{p.e} := t_{slab} - bottom\_cover_{slab} - \frac{d_{e.bar\_pos}}{2}$ $\quad d_{p.e} = 19 \cdot in$

Given $\quad M_{r.pos.e} = \phi \cdot A_{s.e.pos} \cdot f_y \cdot \left[d_{p.e} - \frac{1}{2} \cdot \left(\frac{A_{s.e.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

$A_{s.e.pos.reqd} := Find(A_{s.e.pos})$ $\quad A_{s.e.pos.reqd} = 1.83 \cdot in^2$

$if(A_{s.e.pos} < A_{s.e.pos.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}) = \text{"OK"}$

Check the strain in the reinforcement based on a rectangular stress distribution (LRFD 5.7.2.2) to ensure that the resistance factor is appropriate

$c_{p.e} := \left(\frac{A_{s.e.pos} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b}\right)$ $\quad c_{p.e} = 3 \cdot in$

The strain in the steel is:

$\varepsilon_{sp.e} := \frac{0.003 \cdot (d_{p.e} - c_{p.e})}{c_{p.e}}$ $\quad \varepsilon_{sp.e} = 0.016$

$if(\varepsilon_{sp.e} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}) = \text{"Tension Controlled"}$

6.2 Negative Moment

Maximum moment for Strength I Limit State

$$M_{r.neg.e} := \left|\min\left(M_{EB.sI.neg}\right)\right| \qquad M_{r.neg.e} = 143.8\cdot kip\cdot ft$$

Similar to positive moment:

$$M_r = \phi\cdot A_{s.neg}\cdot f_y\cdot\left[\left(t_{slab} - d'\right) - \frac{1}{2}\cdot\left(\frac{A_{s.neg}\cdot f_y}{0.85\cdot f_{c.slab}\cdot b}\right)\right] \qquad d' = \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{bar}}{2}$$

First, the bar size and the spacing are assumed

Bar Size $d_{e.bar\_neg} := db8E$

Area of the Bar $A_{e.bar\_neg} := As8E$

Bar Spacing $s_{e.top} := 5\cdot in$

Area of Steel per Strip Width

$$A_{s.e.neg} := A_{e.bar\_neg}\cdot\frac{b}{s_{e.top}} \qquad A_{s.e.neg} = 1.9\cdot in^2$$

$$d'_e := \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{e.bar\_neg}}{2} \qquad d'_e = 2.5\cdot in$$

$$d_{n.e} := t_{slab} - d'_e \qquad d_{n.e} = 19\cdot in$$

Given $$M_{r.neg.e} = \phi\cdot A_{s.e.neg}\cdot f_y\cdot\left[d_{n.e} - \frac{1}{2}\cdot\left(\frac{A_{s.e.neg}\cdot f_y}{0.85\cdot f_{c.slab}\cdot b}\right)\right]$$

$$A_{s.e.neg.reqd} := Find\left(A_{s.e.neg}\right) \qquad A_{s.e.neg.reqd} = 1.79\cdot in^2$$

$\text{if}\left(A_{s.e.neg} < A_{s.e.neg.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}\right) = \text{"OK"}$

Check the strain in the reinforcement based on a rectangular stress distribution (LRFD 5.7.2.2) to ensure that the resistance factor is appropriate

$$c_{n.e} := \left(\frac{A_{s.e.neg}\cdot f_y}{\beta_1\cdot 0.85\cdot f_{c.slab}\cdot b}\right) \qquad c_{n.e} = 3\cdot in$$

The strain in the steel is:

$$\varepsilon_{sn.e} := \frac{0.003\cdot\left(d_{n.e} - c_{n.e}\right)}{c_{n.e}} \qquad \varepsilon_{sn.e} = 0.016$$

$\text{if}\left(\varepsilon_{sn.e} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}\right) = \text{"Tension Controlled"}$

7.0 DESIGN CHECKS AND SECONDARY REINFORCEMENT DESIGN

7.1 Crack Control by Distribution of Reinforcement

The bar spacing in the reinforcement is limited to control flexural cracking, LRFD 5.7.3.4. The analysis is based on service loads and applies to sections in which tension in the cross-section exceeds 80% of the modulus of rupture.

7.1.1 Interior Strip

Positive Moment

For service load

$$M_{SerI.pos} := \max\left(1.00M_{DC} + 1.00\frac{M_{LLpos}\cdot ft}{E}\right) \qquad M_{SerI.pos} = 90.1\cdot kip\cdot ft$$

Tensile stress in concrete

$$f_{c.pos} := \frac{6M_{SerI.pos}}{b\cdot t_{slab}^{\ 2}} \qquad f_{c.pos} = 1.17\cdot ksi$$

Modulus of rupture, LRFD 5.4.2.6

$$f_r := 0.24\sqrt{f_{c.slab}\cdot ksi} \qquad f_r = 0.51\cdot ksi$$

$$\text{if}\left(f_{c.pos} > 0.8\cdot f_r, \text{"Check Bar Spacing"}, \text{"LRFD 5.7.3.4 does NOT apply"}\right) = \text{"Check Bar Spacing"}$$

Maximum bar spacing

$$s \le \frac{700\cdot\gamma_e}{\beta_s\cdot f_{ss}} - 2\cdot d_c$$

where,

Strain ratio............................... $\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$

Exposure factor........................ $\gamma_e := 0.75$

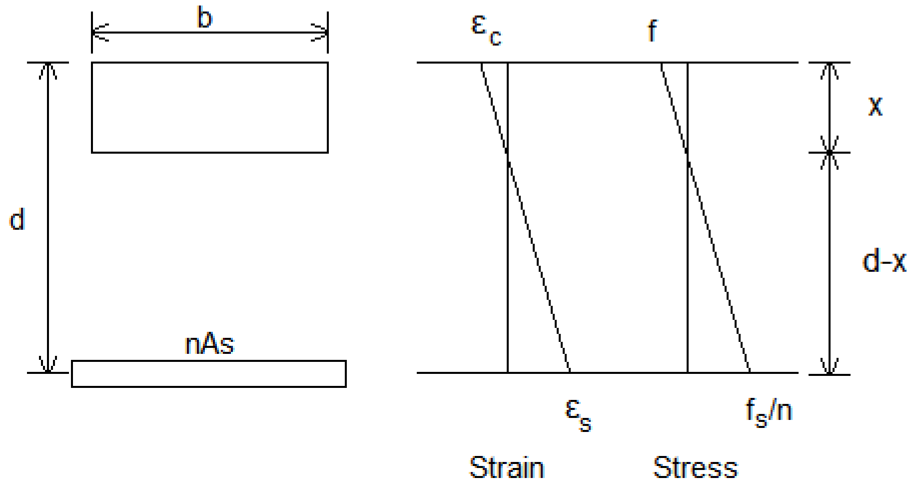
Concrete cover.......................... $d_c = cover_{slab} + \frac{d_{bar}}{2}$

Thickness of the component...... $h := t_{slab}$

Tensile stress in steel................ $f_{ss} = \frac{M}{S_{cr}}$

The stress in the reinforcement is based on the elastic-cracked section and the moment based in the Service I load combination

Elastic-cracked section



Modular Ratio **[AASHTO LRFD 5.7.1 & 5.4.2.4]**

$$n = \frac{E_s}{E_c}$$

where,

Modulusus of Elasticity of Concrete

$$E_c = 1820 \cdot K_1 \cdot \sqrt{f_{c.slab}}$$

Correction factor for Florida Limerock **[FDOT SDG 1.4.1]** $K_1 := 0.9$

$E_c := 1820 \cdot K_1 \cdot \sqrt{f_{c.slab} \cdot ksi}$ $E_c = 3475 \cdot ksi$

$n := \text{round}\left(\frac{E_s}{E_c}\right)$ $n = 8$

Transformed steel area

$A_{trans.p} := n \cdot A_{s.pos}$ $A_{trans.p} = 15.2 \cdot in^2$

Location of neutral axis on transformed section

Assume location

$x_{pre} := 4 \cdot in$

Given $0.5 \cdot b \cdot (x_{pre})^2 = A_{trans.p} \cdot (d_p - x_{pre})$

$x_p := \text{Find}(x_{pre})$ $x_p = 5.78 \cdot in$

Moment of inertia of cracked section

$$I_{cr.pos} := \frac{b \cdot x_p^3}{3} + A_{trans.p} \cdot \left(d_p - x_p\right)^2 \qquad I_{cr.pos} = 3423 \cdot in^4$$

Steel stress

$$f_{ss.pos} := \frac{M_{SerI.pos} \cdot n \cdot \left(d_p - x_p\right)}{I_{cr.pos}} \qquad f_{ss.pos} = 33.4 \cdot ksi$$

if($f_{ss.pos} > 60 \cdot ksi$, "Check Reinforcement" , "Stress is Acceptable") = "Stress is Acceptable"

Strain ratio

$$d_{c.pos} := bottom\_cover_{slab} + \frac{d_{bar\_pos}}{2} \qquad d_{c.pos} = 2.5 \cdot in$$

$$\beta_{s.pos} := 1 + \frac{d_{c.pos}}{0.7\left(h - d_{c.pos}\right)} \qquad \beta_{s.pos} = 1.2$$

Maximum reinforcement spacing

$$s_{cm.pos} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.pos} \cdot f_{ss.pos} \cdot in} - 2 \cdot \left(d_{c.pos}\right) \qquad s_{cm.pos} = 8.2 \cdot in$$

if($s_{cm.pos} > s_{bottom}$, "Spacing is Acceptable" , "Check Spacing") = "Spacing is Acceptable"

Negative Moment

For service load

$$M_{SerI.neg} := \left| \min\left(1.00 M_{DC} + 1.00 \frac{M_{LLneg} \cdot ft}{E}\right) \right| \qquad M_{SerI.neg} = 88.4 \cdot kip \cdot ft$$

Tensile stress in concrete

$$f_{c.neg} := \frac{6 M_{SerI.neg}}{b \cdot t_{slab}^2} \qquad f_{c.neg} = 1.15 \cdot ksi$$

if($f_{c.neg} > 0.8 \cdot f_r$, "Check Bar Spacing" , "LRFD 5.7.3.4 does NOT apply") = "Check Bar Spacing"

Maximum bar spacing

Transformed steel area

$$A_{trans.n} := n \cdot A_{s.neg} \qquad A_{trans.n} = 15.2 \cdot in^2$$

Location of neutral axis on transformed section

Assume location

$x_{pre} := 4 \cdot in$

Given $0.5 \cdot b \cdot (x_{pre})^2 = A_{trans.n} \cdot (d_n - x_{pre})$

$x_n := Find(x_{pre})$ $x_n = 5.78 \cdot in$

Moment of inertia of cracked section

$I_{cr.neg} := \frac{b \cdot x_n^3}{3} + A_{trans.n} \cdot (d_n - x_n)^2$ $I_{cr.neg} = 3423 \cdot in^4$

Steel stress

$f_{ss.neg} := \frac{M_{SerI.neg} \cdot n \cdot (d_n - x_n)}{I_{cr.neg}}$ $f_{ss.neg} = 32.76 \cdot ksi$

$if(f_{ss.neg} > 60 \cdot ksi, \text{"Check Reinforcement"}, \text{"Stress is Acceptable"}) = \text{"Stress is Acceptable"}$

Strain ratio

$d_{c.neg} := d'$ $d_{c.neg} = 2.5 \cdot in$

$\beta_{s.neg} := 1 + \frac{d_{c.neg}}{0.7(h - d_{c.neg})}$ $\beta_{s.neg} = 1.2$

Maximum reinforcement spacing

$s_{cm.neg} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.neg} \cdot f_{ss.neg} \cdot in} - 2 \cdot (d_{c.neg})$ $s_{cm.neg} = 8.5 \cdot in$

$if(s_{cm.neg} > s_{top}, \text{"Spacing is Acceptable"}, \text{"Check Spacing"}) = \text{"Spacing is Acceptable"}$

7.1.2 Edge Strip

Positive Moment

For service load

$M_{SerI.pos.e} := max\left[1.00M_{DC.e} + 1.00\left(\frac{M_{LLpos} \cdot ft \cdot Factor_{LL}}{E_{EB}}\right)\right]$

$M_{SerI.pos.e} = 97.2 \cdot kip \cdot ft$

Tensile stress in concrete

$$f_{c.pos.e} := \frac{6M_{SerI.pos.e}}{b \cdot t_{slab}^{2}} \qquad f_{c.pos.e} = 1.26 \cdot ksi$$

$\text{if}\left(f_{c.pos.e} > 0.8 \cdot f_r, \text{"Check Bar Spacing"}, \text{"LRFD 5.7.3.4 does NOT apply"}\right) = \text{"Check Bar Spacing"}$

Maximum bar spacing

Transformed steel area

$$A_{trans.pe} := n \cdot A_{s.e.pos} \qquad A_{trans.pe} = 15.2 \cdot in^{2}$$

Location of neutral axis on transformed section

Assume location

$$x_{pre} := 4 \cdot in$$

Given $\quad 0.5 \cdot b \cdot \left(x_{pre}\right)^{2} = A_{trans.pe} \cdot \left(d_p - x_{pre}\right)$

$$x_{pe} := \text{Find}\left(x_{pre}\right) \qquad x_{pe} = 5.78 \cdot in$$

Moment of inertia of cracked section

$$I_{cr.pos.e} := \frac{b \cdot x_{pe}^{3}}{3} + A_{trans.p} \cdot \left(d_{p.e} - x_{pe}\right)^{2} \qquad I_{cr.pos.e} = 3423 \cdot in^{4}$$

Steel stress

$$f_{ss.pos.e} := \frac{M_{SerI.pos.e} \cdot n \cdot \left(d_{p.e} - x_{pe}\right)}{I_{cr.pos.e}} \qquad f_{ss.pos.e} = 36.03 \cdot ksi$$

$\text{if}\left(f_{ss.pos.e} > 60 \cdot ksi, \text{"Check Reinforcement"}, \text{"Stress is Acceptable"}\right) = \text{"Stress is Acceptable"}$

Strain ratio

$$d_{c.pos.e} := bottom\_cover_{slab} + \frac{d_{e.bar\_pos}}{2} \qquad d_{c.pos.e} = 2.5 \cdot in$$

$$\beta_{s.pos.e} := 1 + \frac{d_{c.pos.e}}{0.7\left(h - d_{c.pos.e}\right)} \qquad \beta_{s.pos.e} = 1.2$$

Maximum reinforcement spacing

$$s_{cm.pos.e} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.pos.e} \cdot f_{ss.pos.e} \cdot in} - 2 \cdot \left(d_{c.pos.e}\right) \qquad s_{cm.pos.e} = 7.3 \cdot in$$

$\text{if}\left(s_{cm.pos.e} > s_{e.bottom}, \text{"Spacing is Acceptable"}, \text{"Check Spacing"}\right) = \text{"Spacing is Acceptable"}$

Negative Moment

For service load

$$M_{SerI.neg.e} := 1.00M_{DC.e} + 1.00\left(\frac{M_{LLneg}\cdot ft\cdot Factor_{LL}}{E_{EB}}\right)$$

$M_{SerI.neg.e} = 95.5\cdot kip\cdot ft$

Tensile stress in concrete

$$f_{c.neg.e} := \frac{6M_{SerI.neg.e}}{b\cdot t_{slab}^{2}}$$

$f_{c.neg.e} = 1.24\cdot ksi$

if$\left(f_{c.neg.e} > 0.8\cdot f_r\right.$, "Check Bar Spacing" , "LRFD 5.7.3.4 does NOT apply" $\left.\right)$ = "Check Bar Spacing"

Maximum bar spacing

Transformed steel area

$A_{trans.ne} := n\cdot A_{s.e.neg}$

$A_{trans.ne} = 15.2\cdot in^2$

Location of neutral axis on transformed section

Assume location

$x_{pre} := 4\cdot in$

Given $\quad 0.5\cdot b\cdot\left(x_{pre}\right)^2 = A_{trans.ne}\cdot\left(d_{n.e} - x_{pre}\right)$

$x_{ne} := Find\left(x_{pre}\right)$

$x_{ne} = 5.78\cdot in$

Moment of inertia of cracked section

$$I_{cr.neg.e} := \frac{b\cdot x_{ne}^{3}}{3} + A_{trans.ne}\cdot\left(d_{n.e} - x_{ne}\right)^2$$

$I_{cr.neg.e} = 3423\cdot in^4$

Steel stress

$$f_{ss.neg.e} := \frac{M_{SerI.neg.e}\cdot n\cdot\left(d_{n.e} - x_{ne}\right)}{I_{cr.neg.e}}$$

$f_{ss.neg.e} = 35.39\cdot ksi$

if$\left(f_{ss.neg.e} > 60\cdot ksi\right.$, "Check Reinforcement" , "Stress is Acceptable" $\left.\right)$ = "Stress is Acceptable"

Strain ratio

$d_{c.neg.e} := d'_e$

$d_{c.neg.e} = 2.5\cdot in$

$$\beta_{s.neg.e} := 1 + \frac{d_{c.neg.e}}{0.7(h - d_{c.neg.e})} \qquad \beta_{s.neg.e} = 1.2$$

Maximum reinforcement spacing

$$s_{cm.neg.e} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.neg.e} \cdot f_{ss.neg.e} \cdot in} - 2 \cdot (d_{c.neg.e}) \qquad s_{cm.neg.e} = 7.5 \cdot in$$

$if(s_{cm.neg.e} > s_{e.top}, \text{"Spacing is Acceptable"}, \text{"Check Spacing"}) = \text{"Spacing is Acceptable"}$

7.2 Fatigue

Fatigue I Load Combination is used to evaluate the fatigue limit state provisions presented on LRFD 5.5.3.1.

$$FatigueI = 1.5 \cdot (LL + IM)$$

Factored Fatigue I Load Combination

Positive .... $M_{fatigueI.pos} := 1.5 \cdot 1.15 P_{Truck} \cdot L \cdot .171$

Negative .... $M_{fatigueI.neg} := 1.5 \cdot 1.15 P_{Truck} \cdot L \cdot .158$

For fatigue considerations:

$$\Delta f \le \Delta F_{TH}$$

where,

Δf is the live load stress range due to the passage of the fatigue load

ΔF_{TH} is the constant-amplitude fatigue threshold, LRFD 5.5.3.2

$$\Delta F_{TH} = 24 - 0.33 f_{min}$$

where,

f_{min} is the minimum live-load stress due to Fatigue I load combined with the permanent loads

Fatigue Load Moment Range

$$M_{range} := \frac{r \cdot (M_{fatigueI.pos} - M_{fatigueI.neg}) \cdot ft}{E_{OneLane}}$$

Positive Moment Regions

Minimum Stress

$$M_{min.pos} = M_{DC.array} + \frac{r \cdot M_{fatigueI.neg} \cdot ft}{E_{OneLane}}$$

The stress based on the cracked section

Minimum stress.......................... $f_{min.p} = \frac{n \cdot M_{min.pos} \cdot (d_p - x_p)}{I_{cr.pos}}$

Stress range.............................. $\Delta_{f.p} = \frac{n \cdot M_{range} \cdot (d_p - x_p)}{I_{cr.pos}}$

Limit for the stress range............. $\Delta F_{TH.p} = 24 \cdot ksi - 0.33 f_{min.p}$

Negative Moment Regions

Minimum Stress

$$M_{min.neg} = -M_{DC.array} - \frac{r \cdot M_{fatigueI.pos} \cdot ft}{E_{OneLane}}$$

The stress based on the cracked section

Minimum stress.......................... $f_{min.n} = \frac{n \cdot M_{min.neg} \cdot (d_n - x_n)}{I_{cr.neg}}$

Stress range.............................. $\Delta_{f.n} = \frac{n \cdot M_{range} \cdot (d_n - x_n)}{I_{cr.neg}}$

Limit for the stress range............. $\Delta F_{TH.n} = 24 \cdot ksi - 0.33 f_{min.n}$

7.3 Minimum Reinforcement

Check the minimum reinforcement according to the provisions on LRFD 5.7.3.3.2. The factored flexural resistance must be at least the minimum of 1.2 M_{cr} or 1.33 M_u

7.3.1 Interior Strip

Positive Moment

Cracking Moment

$M_{cr} = S_{nc} \cdot f_r$ where, $S_{nc} = \dfrac{b \cdot h^2}{6}$

Substituting,

$$M_{cr} := \frac{b \cdot h^2}{6} \cdot f_r \qquad M_{cr} = 39.2 \cdot kip \cdot ft$$

Flexural resistance

$$\phi M_{n.pos} := \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_p - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.pos} = 151.5 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.pos} := \min(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.pos}) \qquad M_{mr.pos} = 47.1 \cdot kip \cdot ft$$

$$\text{if}\left(\phi M_{n.pos} > M_{mr.pos}, \text{"Requirement Met"}, \text{"Check Reinforcement"}\right) = \text{"Requirement Met"}$$

Negative Moment

Flexural resistance

$$\phi M_{n.neg} := \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_n - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.neg} = 151.5 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.neg} := \min(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.neg}) \qquad M_{mr.neg} = 47.1 \cdot kip \cdot ft$$

$$\text{if}\left(\phi M_{n.neg} > M_{mr.neg}, \text{"Requirement Met"}, \text{"Check Reinforcement"}\right) = \text{"Requirement Met"}$$

7.3.2 Edge Strip

Positive Moment

Flexural resistance

$$\phi M_{n.pos.e} := \phi \cdot A_{s.e.pos} \cdot f_y \cdot \left[d_{p.e} - \frac{1}{2} \cdot \left(\frac{A_{s.e.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.pos.e} = 151.5 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.pos.e} := \min\left(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.pos.e}\right) \qquad M_{mr.pos.e} = 47.1 \cdot kip \cdot ft$$

$$\text{if}\left(\phi M_{n.pos.e} > M_{mr.pos.e}, \text{"Requirement Met"}, \text{"Check Reinforcement"}\right) = \text{"Requirement Met"}$$

Negative Moment

Flexural resistance

$$\phi M_{n.neg.e} := \phi \cdot A_{s.e.neg} \cdot f_y \cdot \left[d_{n.e} - \frac{1}{2} \cdot \left(\frac{A_{s.e.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.neg.e} = 151.5 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.neg.e} := \min\left(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.neg.e}\right) \qquad M_{mr.neg.e} = 47.1 \cdot kip \cdot ft$$

$$\text{if}\left(\phi M_{n.neg.e} > M_{mr.neg.e}, \text{"Requirement Met"}, \text{"Check Reinforcement"}\right) = \text{"Requirement Met"}$$

7.4 Distribution Reinforcement

Placed in the bottom of the slab and may be taken as a percentage of the main reinforcement required for positive moment, LRFD 5.14.4.1.

For reinforced concrete $\frac{100}{\sqrt{L}} \le 50\%$

For simplicity, use the same reinforcement throughout the entire width of the bridge by selecting the critical positive moment out of the interior and edge strips

$A_{sl.dr} := \max\left(A_{s.pos}, A_{s.e.pos}\right)$ $\quad A_{sl.dr} = 1.9 \cdot in^2$

$\%A_{s.dr} := \min\left(\frac{100\%}{\sqrt{\frac{L}{ft}}}, 50\%\right)$ $\quad \%A_{s.dr} = 16.7 \cdot \%$

$A_{s.dr.req} := \%A_{s.dr} \cdot A_{sl.dr}$ $\quad A_{s.dr.req} = 0.32 \cdot in^2$

Determine the size and number of bars to meet requirement

Bar Size $\quad d_{bar\_dr} := db5E$

Area of the Bar $\quad A_{bar\_dr} := As5E$

Bar Spacing $\quad s_{dr} := 9 \cdot in$

$A_{s.dr} := A_{bar\_dr} \cdot \frac{b}{s_{dr}}$ $\quad A_{s.dr} = 0.41 \cdot in^2$

$\text{if}\left(A_{s.dr} < A_{s.dr.req}, \text{"Increase Reinforcement"}, \text{"OK"}\right) = \text{"OK"}$

7.5 Temperature and Shrinkage Reinforcement

Placed in the top of the slab perpendicular to the traffic based on LRFD 5.10.8

$$A_s \ge \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} \quad \text{and} \quad 0.11 \le A_s \le 0.6$$

where,

b is the least width of the span length or bridge width
h is the thickness of the slab
A_s is the area of reinforcement per foot

$b_{ts} := \min(W_{bridge}, L)$ $\qquad b_{ts} = 432 \cdot in$

$A_{s.ts.req} := \dfrac{1.30 \cdot b_{ts} \cdot h \cdot kip}{2 \cdot (b_{ts} + h) \cdot f_y \cdot in}$ $\qquad A_{s.ts.req} = 0.22 \cdot in^2$

Determine the size and number of bars to meet requirement. According to SDG 4.2.11, maximum spacing is 12-inch and the minimum bar size is No. 4.

Bar Size $\qquad d_{bar\_ts} := db5E$

Area of the Bar $\qquad A_{bar\_ts} := As5E$

Bar Spacing $\qquad s_{ts} := 9 \cdot in$

$A_{s.ts} := A_{bar\_ts} \cdot \dfrac{b}{s_{ts}}$ $\qquad A_{s.ts} = 0.413 \cdot in^2$

$\text{if}(A_{s.ts} < A_{s.ts.req}, \text{"Increase Reinforcement"}, \text{"OK"}) = \text{"OK"}$

$\text{if}(A_{s.ts} \le 0.6 \cdot in^2, \text{if}(A_{s.ts} \ge 0.11 \cdot in^2, \text{"OK""}, \text{"Increase Reinforcement"}), \text{"Reduce Reinforcement"}) = \text{"OK""}$

Transverse steel to be used at the top of the slab should be the controlling between distribution and temperature and shrinkage steel. In this case, use No. 5 @ 9-inch.

Change Log

Reference:G:\0Production\reference.xmcd

Flat Slab Design
Northbound Bridge

1.0 GEOMETRY

Overall bridge length........................ $L_{bridge} \equiv 180 \cdot ft$

Length of spans.............................. $L := 36 \cdot ft$

Overall bridge width......................... $W_{bridge} := 43 \cdot ft + 1 \cdot in$

Skew.. $\theta := 20 \cdot deg$

Span to Depth Ratio

[AASHTO LRFD 2.5.2.6.3]

For continuous reinforced slabs with main reinforcement parallel to traffic

$$t_{min} = \frac{S + 10}{30} \geq 0.54 \cdot ft$$

Minimum slab thickness

$$t_{min} := \max\left(\frac{L + 10 \cdot ft}{30}, 0.54 \cdot ft\right)$$

$t_{min} = 18.4 \cdot in$

Preliminary design thickness of flat slab........ $t_{slab} := 21.5 \cdot in$
(exluding 0.5 in sacrificial for deck planning)

2.0 LOADS

2.1 Dead Loads

Weight of future wearing surface (SDG 2.2 & 4.2) **[FDOT SDG 2.2 & 4.2]**

$$\rho_{fws} := \begin{cases} 15 \cdot psf & \text{if } L_{bridge} \le 100ft \\ 0 \cdot psf & \text{otherwise} \end{cases} \qquad \rho_{fws} = 0 \cdot psf$$

Sacrificial Slab Thickness

$$t_{sac} := \begin{cases} 0.5 \cdot in & \text{if } L_{bridge} > 100ft \\ 0 \cdot in & \text{otherwise} \end{cases} \qquad t_{sac} = 0.5 \cdot in$$

Weight of traffic railing barrier............. $W_{barrier} := 420 \cdot plf$

32" F-Shape (SDG 2.2)

Unit weight of concrete..................... $\gamma_{conc} := 150 \cdot pcf$

2.2 Live Loads

The design is based on the HL-93 Design Load.

2.2.1 Design Lanes

Current lane configurations show two striped lanes and two shoulders. Using the roadway clear width between barriers, $Rdwy_{width}$, the number of design traffic lanes per roadway, N_{lanes}, can be calculated as:

Barrier width.................................. $b_{barrier} := 18.5 \cdot in$

Roadway clear width

$$Rdwy_{width} := W_{bridge} - 2 \cdot b_{barrier} \qquad Rdwy_{width} = 40\,ft$$

Number of design traffic lanes per roadway

$$N_{lanes} := \text{floor}\left(\frac{Rdwy_{width}}{12 \cdot ft}\right) \qquad N_{lanes} = 3$$

2.2.2 Distribution

Based on the LRFD Section 4 "Structural Analysis and Evaluation"

The superstructure is designed on a per foot basis longitudinally. However, in order to distribute the live loads, equivalent strips of flat slab deck widths are calculated. The moment and shear effects of a single HL-93 vehicle or multiple vehicles are divided by the appropriate equivalent strip width. The equivalent strips account for the transverse distribution of LRFD wheel loads. Multiple presence factors are already taken into consideration in the following equations

One design lane

The equivalent width of longitudinal strips per lane for both shear and moment with one lane loaded:

$$E = 10 + 5.0\sqrt{L_1 \cdot W_1}$$ **[AASHTO LRFD 4.6.2.3-1]**

where

L_1, modified span length taken equal to the lesser of the actual span or 60 feet

$$L_1 := \min(L, 60.0 \cdot ft) \qquad L_1 = 36\,ft$$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 30 feet for single lane loading

$$W_1 := \min(W_{bridge}, 30.0 \cdot ft) \qquad W_1 = 30\,ft$$

The equivalent distribution width for one lane loaded is given as:

$$E_{OneLane} := \left(10 + 5.0\sqrt{\frac{L_1}{ft}\frac{W_1}{ft}}\right) \cdot in \qquad E_{OneLane} = 14.5\,ft$$

Two or more design lanes

The equivalent width of longitudinal strips per lane for both shear and moment with more than one lane loaded:

$$E = 84 + 1.44\sqrt{L_1 \cdot W_1} \le \frac{12.0W}{N_L}$$ **[AASHTO LRFD 4.6.2.3-2]**

where

L_1, modified span length taken equal to the lesser of the actual span or 60 feet

$$L_1 := \min(L, 60.0 \cdot ft) \qquad L_1 = 36\,ft$$

W_1, modified edge to edge width of bridge taken as the lesser of the actual width, W_{bridge}, or 60 feet for single lane loading

$$W_1 := \min(W_{bridge}, 60.0 \cdot ft) \qquad W_1 = 43.1\,ft$$

N_L, number of design lanes

$$N_L := N_{lanes} \qquad N_L = 3$$

The equivalent distribution width for more than one lane loaded is given as:

$$E_{TwoLane} := \min\left[\left(84 + 1.44\sqrt{\frac{L_1}{ft}\frac{W_1}{ft}}\right), \frac{12.0\left(\frac{W_{bridge}}{ft}\right)}{N_L}\right] \cdot in \qquad E_{TwoLane} = 11.7\,ft$$

Longitudinal force effects reduction factor for skewed bridges

$r := \min(1.05 - 0.25\tan(\theta), 1)$ **[AASHTO LRFD 4.6.2.3-3]** $r = 0.96$

The design strip width to use would be the one that causes the maximum effects. In this case, it would be the minimum value of the two equivalent strip widths

$$E := \frac{\min(E_{OneLane}, E_{TwoLane})}{r}$$ $E = 12.2\ ft$

3.0 LOAD ANALYSIS

3.1 Dead Loads

Barrier weight is assumed to be equally distributed thought the entire width of the bridge. Then, the distributed dead load per 1-foot wide slab strip is as follows:

Slab self weight

$$w_{slab} := (t_{slab} + t_{sac}) \cdot \gamma_{conc} \cdot (1 \cdot ft) \qquad w_{slab} = 0.275 \cdot klf$$

Barrier weight

$$w_{barrier} := \frac{W_{barrier} \cdot 2}{W_{bridge}} \cdot (1 \cdot ft) \qquad w_{barrier} = 0.019 \cdot klf$$

Total distributed dead load, DC

$$w_{DC} := w_{slab} + w_{barrier} \qquad w_{DC} = 0.294 \cdot klf$$

The following shears and moments are determined using beam equations for a 5 span continuous system.

Maximum Support Reactions

$$R_{DC} := \frac{43}{38} \cdot w_{DC} \cdot L \qquad R_{DC} = 12.0 \cdot kip$$

Maximum Shear

$$V_{DC} := \frac{23}{38} \cdot w_{DC} \cdot L \qquad V_{DC} = 6.4 \cdot kip$$

Maximum Moment

$$M_{DC} := .105 \cdot w_{DC} \cdot L^2 \qquad M_{DC} = 40.1 \cdot kip \cdot ft$$

3.2 Live Loads

$w_{Lane} := 640plf$

$P_{Truck} := 72kip$

Maximum Support Reactions

$R_{LL} := \frac{43}{38} \cdot w_{Lane} \cdot L + 1.33P_{Truck}$ $\qquad R_{LL} = 121.8 \cdot kip$

Maximum Shear

$V_{LL} := \frac{23}{38} \cdot w_{Lane} \cdot L + 1.33P_{Truck}$ $\qquad V_{LL} = 109.7 \cdot kip$

Maximum Moment

$M_{LLpos} := .078 \cdot w_{Lane} \cdot L^2 + 1.33P_{Truck} \cdot L \cdot .171$ $\qquad M_{LLpos} = 654.2 \cdot kip \cdot ft$

$M_{LLneg} := .105 \cdot w_{Lane} \cdot L^2 + 1.33P_{Truck} \cdot L \cdot .158$ $\qquad M_{LLneg} = 631.8 \cdot kip \cdot ft$

3.3 Strength I Factored Loads

Based on LRFD Tables 3.4.1-1 and 3.4.1-2, the load combination for the Strength I limit state is as follows:

$StrengthI = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot (LL + IM)$ **[AASHTO LRFD 3.4.1]**

Factored Shear

$V_{strI} := 1.25V_{DC} + 1.75 \frac{V_{LL} \cdot ft}{E}$ $\qquad max(V_{strI}) = 23.7 \cdot kip$

Factored Moments

$M_{strI.pos} := 1.25 \cdot M_{DC} + 1.75 \frac{M_{LLpos} \cdot ft}{E}$ $\qquad max(M_{strI.pos}) = 143.7 \cdot kip \cdot ft$

$M_{strI.neg} := 1.25M_{DC} + 1.75 \frac{M_{LLneg} \cdot ft}{E}$ $\qquad min(M_{strI.neg}) = 140.5 \cdot kip \cdot ft$

4.0 FLEXURAL DESIGN

The flexure resistance factor for reinforced concrete is given in LRFD 5.5.4.2

For tension-controlled **[AASHTO LRFD 5.5.4.2.1]** $\phi := 0.9$

Factored resistance

$$M_r = \phi \cdot M_n$$

Minimum 28-day compressive strength of concrete
Class II (Bridge Deck) $f_{c.slab} := 4.5 \cdot ksi$

Reinforcing Steel Grade 60 $f_y := 60 \cdot ksi$

Modulus of Elasticity of Reinforcing Steel $E_s := 29000 \cdot ksi$

Minimum Concrete Cover

concrete cover for the slab **[FDOT SDG 1.4.2]**

Concrete cover for the top of slab.......

$$top\_cover_{slab} := \begin{cases} 2 \cdot in & \text{if } L_{bridge} < 100ft \\ 2.5 \cdot in & \text{otherwise} \end{cases}$$

$top\_cover_{slab} = 2.5 \cdot in$

Concrete cover for the bottom of slab.. $bottom\_cover_{slab} := 2 \cdot in$

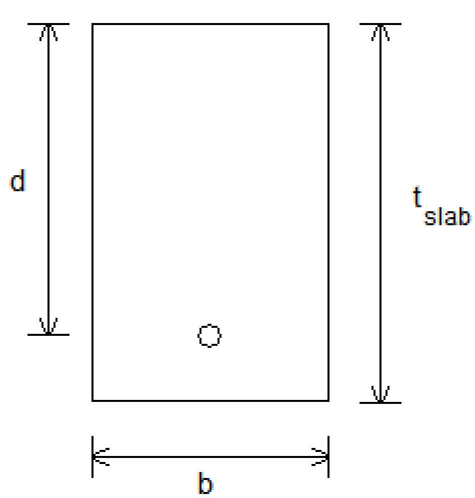
4.1 Positive Moment

Maximum moment for Strength I Limit State

$$M_{r.pos} := \max(M_{strI.pos})$$

$M_{r.pos} = 143.7 \cdot kip \cdot ft$

Simplified nominal flexural resistance



$$M_n = A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \quad \text{where} \quad a = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b}$$

Substituting....

$$M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$$

$$d = t_{slab} - bottom\_cover_{slab} - \frac{d_{bar}}{2}$$

Design Strip Width...... $b := 12 \cdot in$

First, the bar size and the spacing are assumed

Bar Size $d_{bar\_pos} := db8E$

Area of the Bar $A_{bar\_pos} := As8E$

Bar Spacing $s_{bottom} := 5.25 \cdot in$

Area of Steel per Strip Width

$$A_{s.pos} := A_{bar\_pos} \cdot \frac{b}{s_{bottom}} = 1.8 \cdot in^2 \qquad A_{s.pos} = 1.81 \cdot in^2$$

$$d_p := t_{slab} - bottom\_cover_{slab} - \frac{d_{bar\_pos}}{2} \qquad d_p = 19 \cdot in$$

Given

$$M_{r.pos} = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_p - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right]$$

$$A_{s.pos.reqd} := Find(A_{s.pos}) \qquad A_{s.pos.reqd} = 1.79 \cdot in^2$$

$$if(A_{s.pos} < A_{s.pos.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}) = \text{"OK"}$$

Check the strain in the reinforcement based on a rectangular stress distribution to ensure that the resistance factor was appropriately assumed

Rectangular distribution factor **[AASHTO LRFD 5.7.2.2]**

$$\beta_1 := 0.85 - 0.05 \cdot \left(\frac{f_{c.slab}}{ksi} - 4 \right) \qquad \beta_1 = 0.825$$

Distance between the neutral axis and the compressive face

$$c = \frac{a}{\beta_1}$$

Substituting,

$$c_p := \left(\frac{A_{s.pos} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b} \right) \qquad c_p = 2.9 \cdot in$$

Based on an ultimate stress in the concrete of 0.003, the strain in the steel is:

$$\frac{0.003}{c} = \frac{\varepsilon_s}{d - c} \quad \text{or} \quad \varepsilon_s = \frac{0.003 \cdot (d - c)}{c}$$

Substituting,

$$\varepsilon_{sp} := \frac{0.003 \cdot (d_p - c_p)}{c_p} \qquad \varepsilon_{sp} = 0.017$$

[AASHTO LRFD 5.7.2.1]

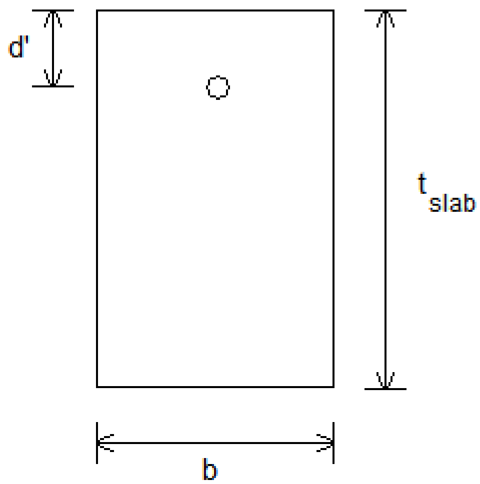
$$if(\varepsilon_{sp} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}) = \text{"Tension Controlled"}$$

4.2 Negative Moment

Maximum moment for Strength I Limit State

$M_{r.neg} := \left|\min\left(M_{strI.neg}\right)\right|$ $\qquad M_{r.neg} = 140.5 \cdot kip \cdot ft$

Similar to positive moment:



$$M_r = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[\left(t_{slab} - d'\right) - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$$

$$d' = \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{bar}}{2}$$

First, the bar size and the spacing are assumed

Bar Size $\qquad d_{bar\_neg} := db8E$

Area of the Bar $\qquad A_{bar\_neg} := As8E$

Bar Spacing $\qquad s_{top} := 5.25 \cdot in$

Area of Steel per Strip Width

$A_{s.neg} := A_{bar\_neg} \cdot \frac{b}{s_{top}} = 1.8 \cdot in^2$ $\qquad A_{s.neg} = 1.81 \cdot in^2$

$d' := \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{bar\_neg}}{2}$ $\qquad d' = 2.5 \cdot in$

$d_n := t_{slab} - d'$ $\qquad d_n = 19 \cdot in$

Given $\qquad M_{r.neg} = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_n - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

$A_{s.neg.reqd} := \mathrm{Find}\left(A_{s.neg}\right)$ $\qquad A_{s.neg.reqd} = 1.75 \cdot in^2$

$\mathrm{if}\left(A_{s.neg} < A_{s.neg.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}\right) = \text{"OK"}$

Check the strain in the reinforcement based on a rectangular stress distribution (LRFD 5.7.2.2) to ensure that the resistance factor is appropriate

$$c_n := \left(\frac{A_{s.neg} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b} \right) \qquad c_n = 2.9 \cdot in$$

The strain in the steel is:

$$\varepsilon_{sn} := \frac{0.003 \cdot (d_n - c_n)}{c_n} \qquad \varepsilon_{sn} = 0.017$$

$$\text{if}\left(\varepsilon_{sn} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}\right) = \text{"Tension Controlled"}$$

5.0 EDGE BEAM DESIGN

Strip width based on LRFD 4.6.2.1.4b "Longitudinal Edges" which states that "edge beams shall be assumed to support one line of wheels and, where appropriate, a tributary portion of the design lane load"

Distribution

$$E_{EB} = \frac{E_{OneLane}}{4} + b_{barrier} + 12 \cdot in \le \frac{E_{OneLane}}{2} \le 72 \cdot in$$

$$E_{EB} := \frac{\min\left(\frac{E_{OneLane}}{4} + b_{barrier} + 12 \cdot in, \frac{E_{OneLane}}{2}, 72 \cdot in\right)}{r} \qquad E_{EB} = 6.3\,ft$$

The tributary portion of the design lane and the truck load are as follows:

$$Factor_{LL} := \frac{E_{EB}}{E} \qquad Factor_{LL} = 0.51$$

5.1 Load Analysis

Dead Load

Edge strip is assumed to carry the weight of the traffic barrier

Total Distributed Load

$$w_{DC.e} := w_{slab} + \frac{W_{barrier} \cdot ft}{E_{EB}} \qquad w_{DC.e} = 0.342 \cdot klf$$

Maximum Support Reactions

$$R_{DC.e} := \frac{43}{38} \cdot w_{DC.e} \cdot L \qquad R_{DC.e} = 13.9 \cdot kip$$

Maximum Shear

$$V_{DC.e} := \frac{23}{38} \cdot w_{DC.e} \cdot L \qquad V_{DC.e} = 7.5 \cdot kip$$

Maximum Moment

$$M_{DC.e} := .105 \cdot w_{DC.e} \cdot L^2 \qquad M_{DC.e} = 46.6 \cdot kip \cdot ft$$

Live Load

Use the same live load moments shown in Section 3.2.

5.2 Strength I Factored Loads

Factored Moments

$$M_{EB.sI.pos} := 1.25M_{DC.e} + 1.75\left(\frac{M_{LLpos} \cdot ft \cdot Factor_{LL}}{E_{EB}}\right)$$

$$M_{EB.sI.pos} = 151.8 \cdot kip \cdot ft$$

$$M_{EB.sI.neg} := 1.25M_{DC.e} + 1.75\left(\frac{M_{LLneg} \cdot ft \cdot Factor_{LL}}{E_{EB}}\right)$$

$$M_{EB.sI.neg} = 148.6 \cdot kip \cdot ft$$

Check if design is required for edge beams

Positive Moment

$$Pos_{EB} := if\left(M_{r.pos} > max\left(M_{EB.sI.pos}\right), \text{"Use Interior"}, \text{"Design Edge Beam"}\right)$$

$$Pos_{EB} = \text{"Design Edge Beam"}$$

Negative Moment

$$Neg_{EB} := if\left(\left|M_{r.neg}\right| > \left|min\left(M_{EB.sI.neg}\right)\right|, \text{"Use Interior"}, \text{"Design Edge Beam"}\right)$$

$$Neg_{EB} = \text{"Design Edge Beam"}$$

6.0 FLEXURAL DESIGN

Similar to interior strip

6.1 Positive Moment

Maximum moment for Strength I Limit State

$M_{r.pos.e} := M_{EB.sI.pos}$ $\qquad M_{r.pos.e} = 151.8 \cdot kip \cdot ft$

Similar to the interior strip design:

$$M_r = \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right] \qquad d = t_{slab} - bottom\_cover_{slab} - \frac{d_{bar}}{2}$$

First, the bar size and the spacing are assumed

Bar Size $\qquad d_{e.bar\_pos} := db8E$

Area of the Bar $\qquad A_{e.bar\_pos} := As8E$

Bar Spacing $\qquad s_{e.bottom} := 5 \cdot in$

Area of Steel per Strip Width

$$A_{s.e.pos} := A_{e.bar\_pos} \cdot \frac{b}{s_{e.bottom}} \qquad A_{s.e.pos} = 1.896 \cdot in^2$$

$$d_{p.e} := t_{slab} - bottom\_cover_{slab} - \frac{d_{e.bar\_pos}}{2} \qquad d_{p.e} = 19 \cdot in$$

Given $\qquad M_{r.pos.e} = \phi \cdot A_{s.e.pos} \cdot f_y \cdot \left[d_{p.e} - \frac{1}{2} \cdot \left(\frac{A_{s.e.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

$$A_{s.e.pos.reqd} := Find(A_{s.e.pos}) \qquad A_{s.e.pos.reqd} = 1.900 \cdot in^2$$

if($A_{s.e.pos} < A_{s.e.pos.reqd}$, "Increase Reinforcement" , "OK") = "Increase Reinforcement"

1% sayok

Check the strain in the reinforcement based on a rectangular stress distribution (LRFD 5.7.2.2) to ensure that the resistance factor is appropriate

$$c_{p.e} := \left(\frac{A_{s.e.pos} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b}\right) \qquad c_{p.e} = 3 \cdot in$$

The strain in the steel is:

$$\varepsilon_{sp.e} := \frac{0.003 \cdot (d_{p.e} - c_{p.e})}{c_{p.e}} \qquad \varepsilon_{sp.e} = 0.016$$

if($\varepsilon_{sp.e} < 0.005$, "Check Resistance Factor" , "Tension Controlled") = "Tension Controlled"

6.2 Negative Moment

Maximum moment for Strength I Limit State

$M_{r.neg.e} := \left|\min\left(M_{EB.sI.neg}\right)\right|$ $\qquad M_{r.neg.e} = 148.6 \cdot kip \cdot ft$

Similar to positive moment:

$$M_r = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[\left(t_{slab} - d'\right) - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right] \qquad d' = \left(top\_cover_{slab} - t_{sac}\right) + \frac{d_{bar}}{2}$$

First, the bar size and the spacing are assumed

Bar Size $\qquad d_{e.bar\_neg} := db8E$

Area of the Bar $\qquad A_{e.bar\_neg} := As8E$

Bar Spacing $\qquad s_{e.top} := 5 \cdot in$

Area of Steel per Strip Width

$A_{s.e.neg} := A_{e.bar\_neg} \cdot \dfrac{b}{s_{e.top}}$ $\qquad A_{s.e.neg} = 1.9 \cdot in^2$

$d'_e := \left(top\_cover_{slab} - t_{sac}\right) + \dfrac{d_{e.bar\_neg}}{2}$ $\qquad d'_e = 2.5 \cdot in$

$d_{n.e} := t_{slab} - d'_e$ $\qquad d_{n.e} = 19 \cdot in$

Given $\qquad M_{r.neg.e} = \phi \cdot A_{s.e.neg} \cdot f_y \cdot \left[d_{n.e} - \dfrac{1}{2} \cdot \left(\dfrac{A_{s.e.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b}\right)\right]$

$A_{s.e.neg.reqd} := Find\left(A_{s.e.neg}\right)$ $\qquad A_{s.e.neg.reqd} = 1.86 \cdot in^2$

$if\left(A_{s.e.neg} < A_{s.e.neg.reqd}, \text{"Increase Reinforcement"}, \text{"OK"}\right) = \text{"OK"}$

Check the strain in the reinforcement based on a rectangular stress distribution (LRFD 5.7.2.2) to ensure that the resistance factor is appropriate

$c_{n.e} := \left(\dfrac{A_{s.e.neg} \cdot f_y}{\beta_1 \cdot 0.85 \cdot f_{c.slab} \cdot b}\right)$ $\qquad c_{n.e} = 3 \cdot in$

The strain in the steel is:

$\varepsilon_{sn.e} := \dfrac{0.003 \cdot \left(d_{n.e} - c_{n.e}\right)}{c_{n.e}}$ $\qquad \varepsilon_{sn.e} = 0.016$

$if\left(\varepsilon_{sn.e} < 0.005, \text{"Check Resistance Factor"}, \text{"Tension Controlled"}\right) = \text{"Tension Controlled"}$

7.0 DESIGN CHECKS AND SECONDARY REINFORCEMENT DESIGN

7.1 Crack Control by Distribution of Reinforcement

The bar spacing in the reinforcement is limited to control flexural cracking, LRFD 5.7.3.4. The analysis is based on service loads and applies to sections in which tension in the cross-section exceeds 80% of the modulus of rupture.

7.1.1 Interior Strip

Positive Moment

For service load

$$M_{SerI.pos} := \max\left(1.00M_{DC} + 1.00\frac{M_{LLpos}\cdot ft}{E}\right) \qquad M_{SerI.pos} = 93.6\cdot kip\cdot ft$$

Tensile stress in concrete

$$f_{c.pos} := \frac{6M_{SerI.pos}}{b\cdot t_{slab}^{\ 2}} \qquad f_{c.pos} = 1.21\cdot ksi$$

Modulus of rupture, LRFD 5.4.2.6

$$f_r := 0.24\sqrt{f_{c.slab}\cdot ksi} \qquad f_r = 0.51\cdot ksi$$

$$\text{if}\left(f_{c.pos} > 0.8\cdot f_r, \text{"Check Bar Spacing"}, \text{"LRFD 5.7.3.4 does NOT apply"}\right) = \text{"Check Bar Spacing"}$$

Maximum bar spacing

$$s \le \frac{700\cdot\gamma_e}{\beta_s\cdot f_{ss}} - 2\cdot d_c$$

where,

Strain ratio................................ $\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$

Exposure factor......................... $\gamma_e := 0.75$

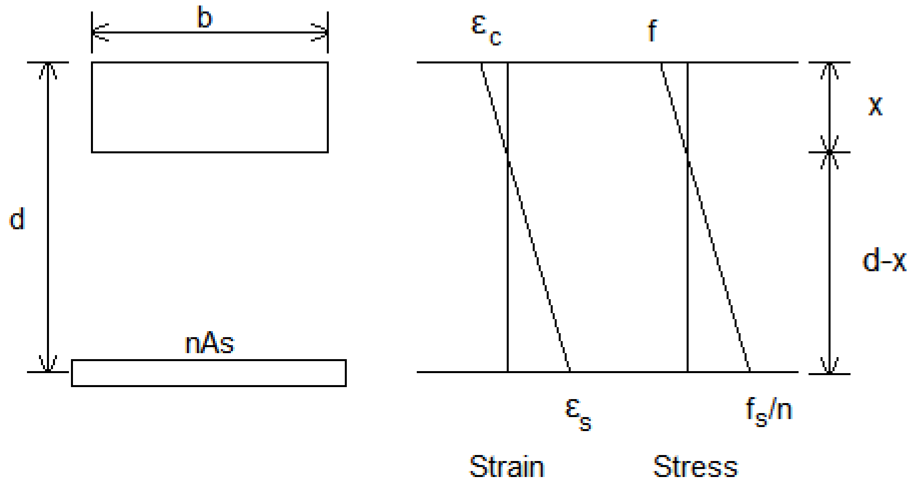
Concrete cover........................... $d_c = cover_{slab} + \frac{d_{bar}}{2}$

Thickness of the component...... $h := t_{slab}$

Tensile stress in steel................ $f_{ss} = \frac{M}{S_{cr}}$

The stress in the reinforcement is based on the elastic-cracked section and the moment based in the Service I load combination

Elastic-cracked section



Modular Ratio **[AASHTO LRFD 5.7.1 & 5.4.2.4]**

$$n = \frac{E_s}{E_c}$$

where,

Modulusus of Elasticity of Concrete

$$E_c = 1820 \cdot K_1 \cdot \sqrt{f_{c.slab}}$$

Correction factor for Florida Limerock **[FDOT SDG 1.4.1]** $K_1 := 0.9$

$E_c := 1820 \cdot K_1 \cdot \sqrt{f_{c.slab} \cdot ksi}$ $E_c = 3475 \cdot ksi$

$n := round\left(\frac{E_s}{E_c}\right)$ $n = 8$

Transformed steel area

$A_{trans.p} := n \cdot A_{s.pos}$ $A_{trans.p} = 14.4 \cdot in^2$

Location of neutral axis on transformed section

Assume location

$x_{pre} := 4 \cdot in$

Given $0.5 \cdot b \cdot \left(x_{pre}\right)^2 = A_{trans.p} \cdot \left(d_p - x_{pre}\right)$

$x_p := Find\left(x_{pre}\right)$ $x_p = 5.67 \cdot in$

Moment of inertia of cracked section

$$I_{cr.pos} := \frac{b \cdot x_p^{\,3}}{3} + A_{trans.p} \cdot \left(d_p - x_p\right)^2$$ $I_{cr.pos} = 3296 \cdot in^4$

Steel stress

$$f_{ss.pos} := \frac{M_{SerI.pos} \cdot n \cdot \left(d_p - x_p\right)}{I_{cr.pos}}$$ $f_{ss.pos} = 36.34 \cdot ksi$

$\text{if}\left(f_{ss.pos} > 60 \cdot ksi, \text{"Check Reinforcement"}, \text{"Stress is Acceptable"}\right) = \text{"Stress is Acceptable"}$

Strain ratio

$$d_{c.pos} := bottom\_cover_{slab} + \frac{d_{bar\_pos}}{2}$$ $d_{c.pos} = 2.5 \cdot in$

$$\beta_{s.pos} := 1 + \frac{d_{c.pos}}{0.7\left(h - d_{c.pos}\right)}$$ $\beta_{s.pos} = 1.2$

Maximum reinforcement spacing

$$s_{cm.pos} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.pos} \cdot f_{ss.pos} \cdot in} - 2 \cdot \left(d_{c.pos}\right)$$ $s_{cm.pos} = 7.2 \cdot in$

$\text{if}\left(s_{cm.pos} > s_{bottom}, \text{"Spacing is Acceptable"}, \text{"Check Spacing"}\right) = \text{"Spacing is Acceptable"}$

Negative Moment

For service load

$$M_{SerI.neg} := \left| \min\left(1.00 M_{DC} + 1.00 \frac{M_{LLneg} \cdot ft}{E}\right) \right|$$ $M_{SerI.neg} = 91.7 \cdot kip \cdot ft$

Tensile stress in concrete

$$f_{c.neg} := \frac{6 M_{SerI.neg}}{b \cdot t_{slab}^{\,2}}$$ $f_{c.neg} = 1.19 \cdot ksi$

$\text{if}\left(f_{c.neg} > 0.8 \cdot f_r, \text{"Check Bar Spacing"}, \text{"LRFD 5.7.3.4 does NOT apply"}\right) = \text{"Check Bar Spacing"}$

Maximum bar spacing

Transformed steel area

$$A_{trans.n} := n \cdot A_{s.neg}$$ $A_{trans.n} = 14.4 \cdot in^2$

Location of neutral axis on transformed section

Assume location

$x_{pre} := 4 \cdot in$

Given $0.5 \cdot b \cdot (x_{pre})^2 = A_{trans.n} \cdot (d_n - x_{pre})$

$x_n := Find(x_{pre})$ $x_n = 5.67 \cdot in$

Moment of inertia of cracked section

$$I_{cr.neg} := \frac{b \cdot x_n^{\,3}}{3} + A_{trans.n} \cdot (d_n - x_n)^2 \qquad I_{cr.neg} = 3296 \cdot in^4$$

Steel stress

$$f_{ss.neg} := \frac{M_{SerI.neg} \cdot n \cdot (d_n - x_n)}{I_{cr.neg}} \qquad f_{ss.neg} = 35.63 \cdot ksi$$

$if(f_{ss.neg} > 60 \cdot ksi, \text{"Check Reinforcement"}, \text{"Stress is Acceptable"}) = \text{"Stress is Acceptable"}$

Strain ratio

$d_{c.neg} := d'$ $d_{c.neg} = 2.5 \cdot in$

$$\beta_{s.neg} := 1 + \frac{d_{c.neg}}{0.7(h - d_{c.neg})} \qquad \beta_{s.neg} = 1.2$$

Maximum reinforcement spacing

$$s_{cm.neg} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.neg} \cdot f_{ss.neg} \cdot in} - 2 \cdot (d_{c.neg}) \qquad s_{cm.neg} = 7.4 \cdot in$$

$if(s_{cm.neg} > s_{top}, \text{"Spacing is Acceptable"}, \text{"Check Spacing"}) = \text{"Spacing is Acceptable"}$

7.1.2 Edge Strip

Positive Moment

For service load

$$M_{SerI.pos.e} := max\left[1.00 M_{DC.e} + 1.00\left(\frac{M_{LLpos} \cdot ft \cdot Factor_{LL}}{E_{EB}}\right)\right]$$

$M_{SerI.pos.e} = 100.1 \cdot kip \cdot ft$

Tensile stress in concrete

$$f_{c.pos.e} := \frac{6M_{SerI.pos.e}}{b \cdot t_{slab}^{2}} \qquad f_{c.pos.e} = 1.30 \cdot ksi$$

if$\left(f_{c.pos.e} > 0.8 \cdot f_r\right.$, "Check Bar Spacing" , "LRFD 5.7.3.4 does NOT apply") = "Check Bar Spacing"

Maximum bar spacing

Transformed steel area

$$A_{trans.pe} := n \cdot A_{s.e.pos} \qquad A_{trans.pe} = 15.2 \cdot in^{2}$$

Location of neutral axis on transformed section

Assume location

$$x_{pre} := 4 \cdot in$$

Given $\quad 0.5 \cdot b \cdot \left(x_{pre}\right)^{2} = A_{trans.pe} \cdot \left(d_p - x_{pre}\right)$

$$x_{pe} := \text{Find}\left(x_{pre}\right) \qquad x_{pe} = 5.78 \cdot in$$

Moment of inertia of cracked section

$$I_{cr.pos.e} := \frac{b \cdot x_{pe}^{3}}{3} + A_{trans.p} \cdot \left(d_{p.e} - x_{pe}\right)^{2} \qquad I_{cr.pos.e} = 3297 \cdot in^{4}$$

Steel stress

$$f_{ss.pos.e} := \frac{M_{SerI.pos.e} \cdot n \cdot \left(d_{p.e} - x_{pe}\right)}{I_{cr.pos.e}} \qquad f_{ss.pos.e} = 38.51 \cdot ksi$$

if$\left(f_{ss.pos.e} > 60 \cdot ksi\right.$, "Check Reinforcement" , "Stress is Acceptable") = "Stress is Acceptable"

Strain ratio

$$d_{c.pos.e} := bottom\_cover_{slab} + \frac{d_{e.bar\_pos}}{2} \qquad d_{c.pos.e} = 2.5 \cdot in$$

$$\beta_{s.pos.e} := 1 + \frac{d_{c.pos.e}}{0.7\left(h - d_{c.pos.e}\right)} \qquad \beta_{s.pos.e} = 1.2$$

Maximum reinforcement spacing

$$s_{cm.pos.e} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.pos.e} \cdot f_{ss.pos.e} \cdot in} - 2 \cdot \left(d_{c.pos.e}\right) \qquad s_{cm.pos.e} = 6.5 \cdot in$$

if$\left(s_{cm.pos.e} > s_{e.bottom}\right.$, "Spacing is Acceptable" , "Check Spacing") = "Spacing is Acceptable"

Negative Moment

For service load

$$M_{SerI.neg.e} := 1.00 M_{DC.e} + 1.00\left(\frac{M_{LLneg} \cdot ft \cdot Factor_{LL}}{E_{EB}}\right)$$

$M_{SerI.neg.e} = 98.2 \cdot kip \cdot ft$

Tensile stress in concrete

$$f_{c.neg.e} := \frac{6 M_{SerI.neg.e}}{b \cdot t_{slab}^{2}}$$

$f_{c.neg.e} = 1.27 \cdot ksi$

if$\left(f_{c.neg.e} > 0.8 \cdot f_r\right.$, "Check Bar Spacing" , "LRFD 5.7.3.4 does NOT apply" $\left.\right)$ = "Check Bar Spacing"

Maximum bar spacing

Transformed steel area

$A_{trans.ne} := n \cdot A_{s.e.neg}$

$A_{trans.ne} = 15.2 \cdot in^{2}$

Location of neutral axis on transformed section

Assume location

$x_{pre} := 4 \cdot in$

Given $0.5 \cdot b \cdot (x_{pre})^{2} = A_{trans.ne} \cdot (d_{n.e} - x_{pre})$

$x_{ne} := Find(x_{pre})$

$x_{ne} = 5.78 \cdot in$

Moment of inertia of cracked section

$$I_{cr.neg.e} := \frac{b \cdot x_{ne}^{3}}{3} + A_{trans.ne} \cdot (d_{n.e} - x_{ne})^{2}$$

$I_{cr.neg.e} = 3423 \cdot in^{4}$

Steel stress

$$f_{ss.neg.e} := \frac{M_{SerI.neg.e} \cdot n \cdot (d_{n.e} - x_{ne})}{I_{cr.neg.e}}$$

$f_{ss.neg.e} = 36.41 \cdot ksi$

if$\left(f_{ss.neg.e} > 60 \cdot ksi\right.$, "Check Reinforcement" , "Stress is Acceptable" $\left.\right)$ = "Stress is Acceptable"

Strain ratio

$d_{c.neg.e} := d'_{e}$

$d_{c.neg.e} = 2.5 \cdot in$

$$\beta_{s.neg.e} := 1 + \frac{d_{c.neg.e}}{0.7(h - d_{c.neg.e})} \qquad \beta_{s.neg.e} = 1.2$$

Maximum reinforcement spacing

$$s_{cm.neg.e} := \frac{700 \cdot \gamma_e \cdot kip}{\beta_{s.neg.e} \cdot f_{ss.neg.e} \cdot in} - 2 \cdot (d_{c.neg.e}) \qquad s_{cm.neg.e} = 7.1 \cdot in$$

$if(s_{cm.neg.e} > s_{e.top}, \text{"Spacing is Acceptable"}, \text{"Check Spacing"}) = \text{"Spacing is Acceptable"}$

7.2 Fatigue

Fatigue I Load Combination is used to evaluate the fatigue limit state provisions presented on LRFD 5.5.3.1.

$$FatigueI = 1.5 \cdot (LL + IM)$$

Factored Fatigue I Load Combination

Positive .... $M_{fatigueI.pos} := 1.5 \cdot 1.15 P_{Truck} \cdot L \cdot .171$

Negative .... $M_{fatigueI.neg} := 1.5 \cdot 1.15 P_{Truck} \cdot L \cdot .158$

For fatigue considerations:

$$\Delta f \le \Delta F_{TH}$$

where,

Δf is the live load stress range due to the passage of the fatigue load

ΔF_{TH} is the constant-amplitude fatigue threshold, LRFD 5.5.3.2

$$\Delta F_{TH} = 24 - 0.33 f_{min}$$

where,

f_{min} is the minimum live-load stress due to Fatigue I load combined with the permanent loads

Fatigue Load Moment Range

$$M_{range} := \frac{r \cdot (M_{fatigueI.pos} - M_{fatigueI.neg}) \cdot ft}{E_{OneLane}}$$

Positive Moment Regions

Minimum Stress

$$M_{min.pos} = M_{DC.array} + \frac{r \cdot M_{fatigueI.neg} \cdot ft}{E_{OneLane}}$$

The stress based on the cracked section

Minimum stress.......................... $f_{min.p} = \dfrac{n \cdot M_{min.pos} \cdot (d_p - x_p)}{I_{cr.pos}}$

Stress range.............................. $\Delta_{f.p} = \dfrac{n \cdot M_{range} \cdot (d_p - x_p)}{I_{cr.pos}}$

Limit for the stress range............. $\Delta F_{TH.p} = 24 \cdot ksi - 0.33 f_{min.p}$

Negative Moment Regions

Minimum Stress

$$M_{min.neg} = -M_{DC.array} - \frac{r \cdot M_{fatigueI.pos} \cdot ft}{E_{OneLane}}$$

The stress based on the cracked section

Minimum stress.......................... $f_{min.n} = \dfrac{n \cdot M_{min.neg} \cdot (d_n - x_n)}{I_{cr.neg}}$

Stress range.............................. $\Delta_{f.n} = \dfrac{n \cdot M_{range} \cdot (d_n - x_n)}{I_{cr.neg}}$

Limit for the stress range............. $\Delta F_{TH.n} = 24 \cdot ksi - 0.33 f_{min.n}$

7.3 Minimum Reinforcement

Check the minimum reinforcement according to the provisions on LRFD 5.7.3.3.2. The factored flexural resistance must be at least the minimum of 1.2 M_{cr} or 1.33 M_u

7.3.1 Interior Strip

Positive Moment

Cracking Moment

$M_{cr} = S_{nc} \cdot f_r$ where, $S_{nc} = \frac{b \cdot h^2}{6}$

Substituting,

$$M_{cr} := \frac{b \cdot h^2}{6} \cdot f_r \qquad M_{cr} = 39.2 \cdot kip \cdot ft$$

Flexural resistance

$$\phi M_{n.pos} := \phi \cdot A_{s.pos} \cdot f_y \cdot \left[d_p - \frac{1}{2} \cdot \left(\frac{A_{s.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.pos} = 144.8 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.pos} := \min(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.pos}) \qquad M_{mr.pos} = 47.1 \cdot kip \cdot ft$$

$$\text{if}(\phi M_{n.pos} > M_{mr.pos}, \text{"Requirement Met"}, \text{"Check Reinforcement"}) = \text{"Requirement Met"}$$

Negative Moment

Flexural resistance

$$\phi M_{n.neg} := \phi \cdot A_{s.neg} \cdot f_y \cdot \left[d_n - \frac{1}{2} \cdot \left(\frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.neg} = 144.8 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.neg} := \min(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.neg}) \qquad M_{mr.neg} = 47.1 \cdot kip \cdot ft$$

$$\text{if}(\phi M_{n.neg} > M_{mr.neg}, \text{"Requirement Met"}, \text{"Check Reinforcement"}) = \text{"Requirement Met"}$$

7.3.2 Edge Strip

Positive Moment

Flexural resistance

$$\phi M_{n.pos.e} := \phi \cdot A_{s.e.pos} \cdot f_y \cdot \left[d_{p.e} - \frac{1}{2} \cdot \left(\frac{A_{s.e.pos} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.pos.e} = 151.5 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.pos.e} := \min\left(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.pos.e}\right) \qquad M_{mr.pos.e} = 47.1 \cdot kip \cdot ft$$

$$\text{if}\left(\phi M_{n.pos.e} > M_{mr.pos.e}, \text{"Requirement Met"}, \text{"Check Reinforcement"}\right) = \text{"Requirement Met"}$$

Negative Moment

Flexural resistance

$$\phi M_{n.neg.e} := \phi \cdot A_{s.e.neg} \cdot f_y \cdot \left[d_{n.e} - \frac{1}{2} \cdot \left(\frac{A_{s.e.neg} \cdot f_y}{0.85 \cdot f_{c.slab} \cdot b} \right) \right] \qquad \phi M_{n.neg.e} = 151.5 \cdot kip \cdot ft$$

Moment controlling minimum reinforcement

$$M_{mr.neg.e} := \min\left(1.2 \cdot M_{cr}, 1.33 \cdot M_{r.neg.e}\right) \qquad M_{mr.neg.e} = 47.1 \cdot kip \cdot ft$$

$$\text{if}\left(\phi M_{n.neg.e} > M_{mr.neg.e}, \text{"Requirement Met"}, \text{"Check Reinforcement"}\right) = \text{"Requirement Met"}$$

7.4 Distribution Reinforcement

Placed in the bottom of the slab and may be taken as a percentage of the main reinforcement required for positive moment, LRFD 5.14.4.1.

For reinforced concrete $\frac{100}{\sqrt{L}} \le 50\%$

For simplicity, use the same reinforcement throughout the entire width of the bridge by selecting the critical positive moment out of the interior and edge strips

$A_{sl.dr} := \max(A_{s.pos}, A_{s.e.pos})$ $\qquad A_{sl.dr} = 1.9 \cdot in^2$

$\%A_{s.dr} := \min\left(\frac{100\%}{\sqrt{\frac{L}{ft}}}, 50\%\right)$ $\qquad \%A_{s.dr} = 16.7 \cdot \%$

$A_{s.dr.req} := \%A_{s.dr} \cdot A_{sl.dr}$ $\qquad A_{s.dr.req} = 0.32 \cdot in^2$

Determine the size and number of bars to meet requirement

Bar Size $\qquad d_{bar\_dr} := db5E$

Area of the Bar $\qquad A_{bar\_dr} := As5E$

Bar Spacing $\qquad s_{dr} := 9 \cdot in$

$A_{s.dr} := A_{bar\_dr} \cdot \frac{b}{s_{dr}}$ $\qquad A_{s.dr} = 0.41 \cdot in^2$

$\text{if}(A_{s.dr} < A_{s.dr.req}, \text{"Increase Reinforcement"}, \text{"OK"}) = \text{"OK"}$

7.5 Temperature and Shrinkage Reinforcement

Placed in the top of the slab perpendicular to the traffic based on LRFD 5.10.8

$$A_s \geq \frac{1.30 \cdot b \cdot h}{2 \cdot (b + h) \cdot f_y} \qquad \text{and} \qquad 0.11 \leq A_s \leq 0.6$$

where,

b is the least width of the span length or bridge width
h is the thickness of the slab
A_s is the area of reinforcement per foot

$b_{ts} := \min(W_{bridge}, L)$ $\qquad b_{ts} = 432 \cdot in$

$A_{s.ts.req} := \frac{1.30 \cdot b_{ts} \cdot h \cdot kip}{2 \cdot (b_{ts} + h) \cdot f_y \cdot in}$ $\qquad A_{s.ts.req} = 0.22 \cdot in^2$

Determine the size and number of bars to meet requirement. According to SDG 4.2.11, maximum spacing is 12-inch and the minimum bar size is No. 4.

Bar Size $\qquad d_{bar\_ts} := db5E$

Area of the Bar $\qquad A_{bar\_ts} := As5E$

Bar Spacing $\qquad s_{ts} := 9 \cdot in$

$A_{s.ts} := A_{bar\_ts} \cdot \frac{b}{s_{ts}}$ $\qquad A_{s.ts} = 0.413 \cdot in^2$

if($A_{s.ts} < A_{s.ts.req}$, "Increase Reinforcement", "OK") = "OK"

if($A_{s.ts} \leq 0.6 \cdot in^2$, if($A_{s.ts} \geq 0.11 \cdot in^2$, "OK"", "Increase Reinforcement"), "Reduce Reinforcement") = "OK""

Transverse steel to be used at the top of the slab should be the controlling between distribution and temperature and shrinkage steel. In this case, use No. 5 @ 9-inch.

APPENDIX C
Roadway and Bridge Typical Sections

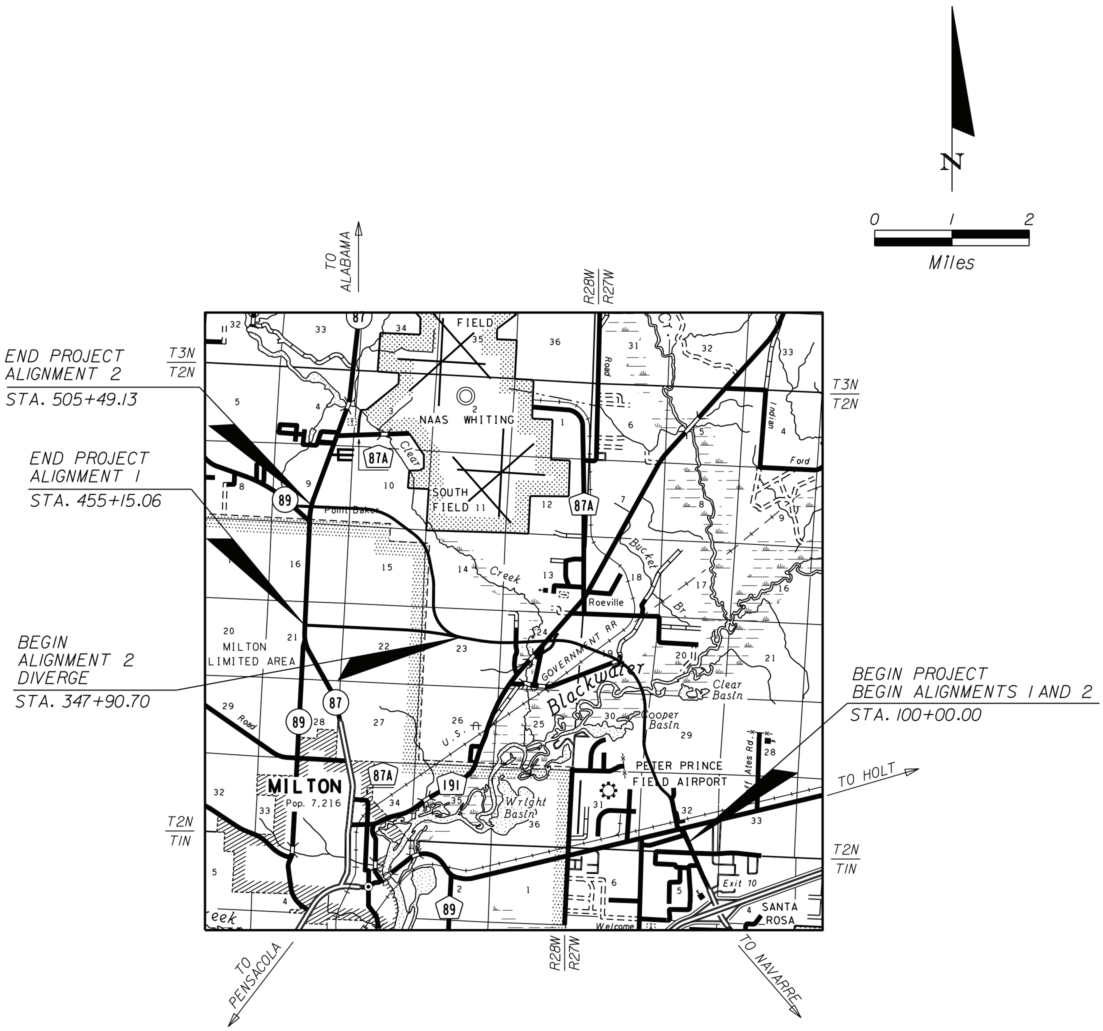
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

TYPICAL SECTION PACKAGE

FINANCIAL PROJECT ID
416748-3-22-01 AND 416748-3-22-02

SR 87 CONNECTOR FROM
SR 87S @ SR 10 (US 90) TO SR 87N

SANTA ROSA COUNTY



PREPARED BY

METRIC ENGINEERING, INC.
ENGINEERS • PLANNERS

2616 JENKS AVE
PANAMA CITY, FLORIDA 32405
TEL. (850) 872-8044
FAX. (850) 872-8704
FLORIDA CERT. NO. EB- 0002294

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID 416748-3-22-01 AND 416748-3-22-02 COUNTY (SECTION) SANTA ROSA (58040)

PROJECT DESCRIPTION
ALIGN. 1 AND 2; STA. 100+00 - 253+60 (FROM S. OF US 90 TO THE BLACKWATER RIVER BRIDGE)
ALIGN. 1: STA. 435+29 - 455+15 (AT CONNECTION TO SR 87N)
ALIGN. 2: STA. 464+44 - 505+49 (AT CONNECTION TO SR 87N)

PROJECT CONTROLS

FUNCTIONAL CLASSIFICATION

() RURAL
(X) URBAN

() FREEWAY/EXPWY. () MAJOR COLL.
(X) PRINCIPAL ART. () MINOR COLL.
() MINOR ART. () LOCAL

HIGHWAY SYSTEM

Yes	No	
( )	(X)	NATIONAL HIGHWAY SYSTEM
( )	(X)	FLORIDA INTRASTATE HIGHWAY SYSTEM
( )	(X)	STRATEGIC INTERMODAL SYSTEM
(X)	( )	STATE HIGHWAY SYSTEM
( )	(X)	OFF STATE HIGHWAY SYSTEM

ACCESS CLASSIFICATION

() 1 - FREEWAY
() 2 - RESTRICTIVE w/Service Roads
(X) 3 - RESTRICTIVE w/660 ft. Connecting Spacing
() 4 - NON-RESTRICTIVE w/2640 ft. Signal Spacing
() 5 - RESTRICTIVE w/440 ft. Connection Spacing
() 6 - NON- RESTRICTIVE w/1320 ft. Signal Spacing
() 7 - BOTH MEDIAN TYPES

TRAFFIC

	YEAR	AADT
CURRENT	2009	0
OPENING	2015	10,731
DESIGN	2035	19,746

DESIGN SPEED 45
POSTED SPEED 45

DISTRIBUTION
K 9.0%
D 58.7%
T_{24} 5%

CRITERIA

(X) NEW CONSTRUCTION / RECONSTRUCTION
() RRR INTERSTATE / FREEWAY
() RRR NON-INTERSTATE / FREEWAY
() TDLC / NEW CONSTRUCTION / RECONSTRUCTION
() TDLC / RRR
() MANUAL OF UNIFORM MINIMUM STANDARDS (FLORIDA GREENBOOK)(OFF-STATE HIGHWAY ONLY)

DESIGN SPEED APPROVALS

JOHN S. GOLDEN, P.E.
DISTRICT DESIGN ENGINEER DATE

JARED PERDUE, P.E.
DISTRICT TRAFFIC OPERATIONS ENGINEER DATE

LIST ANY POTENTIAL EXCEPTIONS AND VARIATIONS RELATED TO TYPICAL SECTION ELEMENTS:

ACCESS MANAGEMENT: CONNECTION SPACING - DRIVEWAY TURNOUTS JUST NORTH OF US 90 - ALIGNMENTS 1 AND 2
CONNECTION SPACING - DRIVEWAY TURNOUTS JUST EAST OF SR 87N - ALIGNMENT 1
MEDIAN OPENING SPACING - BOBBY BROWN ROAD AT BEGINNING OF ALIGNMENTS 1 AND 2
MEDIAN OPENING SPACING - SEASON DRIVE AT THE END OF ALIGNMENT 2

LIST MAJOR STRUCTURES LOCATION/DESCRIPTION - REQUIRING INDEPENDENT STRUCTURE DESIGN:

BRIDGE OVER BLACKWATER RIVER, BLACKWATER HERITAGE TRAIL AND WETLANDS

LIST MAJOR UTILITIES WITHIN PROJECT CORRIDOR:

AT&T, AT&T DISTRIBUTION, CITY OF MILTON, CSX RAILROAD, EAST MILTON WATER SYSTEM, GULF POWER, MCI, MEDIACOM, OKALOOSA GAS, POINT BAKER WATER SYSTEM, QWEST, SOUTHERN LIGHT, SPRINT/NEXTEL

LIST OTHER INFORMATION PERTINENT TO DESIGN OF PROJECT:

SR 87 HAS BEEN DESIGNATED AS A "HURRICANE EVACUATION ROUTE"

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID 416748-3-22-01 AND 416748-3-22-02 COUNTY (SECTION) SANTA ROSA (58040)

PROJECT DESCRIPTION ALIGN. 1: STA. 253+60 - 435+29 AND ALIGN. 2: STA. 253+60 - 464+44 (FROM N. OF THE BLACKWATER RIVER BRIDGE TO E. OF SR 87N CONNECTION)

PROJECT CONTROLS

FUNCTIONAL CLASSIFICATION

(X) RURAL
( ) URBAN

( ) FREEWAY/EXPWY. ( ) MAJOR COLL.
(X) PRINCIPAL ART. ( ) MINOR COLL.
( ) MINOR ART. ( ) LOCAL

HIGHWAY SYSTEM

Yes	No	
( )	(X)	NATIONAL HIGHWAY SYSTEM
( )	(X)	FLORIDA INTRASTATE HIGHWAY SYSTEM
( )	(X)	STRATEGIC INTERMODAL SYSTEM
(X)	( )	STATE HIGHWAY SYSTEM
( )	(X)	OFF STATE HIGHWAY SYSTEM

ACCESS CLASSIFICATION

( ) 1 - FREEWAY
( ) 2 - RESTRICTIVE w/Service Roads
(X) 3 - RESTRICTIVE w/660 ft. Connecting Spacing
( ) 4 - NON-RESTRICTIVE w/2640 ft. Signal Spacing
( ) 5 - RESTRICTIVE w/440 ft. Connection Spacing
( ) 6 - NON- RESTRICTIVE w/1320 ft. Signal Spacing
( ) 7 - BOTH MEDIAN TYPES

CRITERIA

(X) NEW CONSTRUCTION / RECONSTRUCTION
( ) RRR INTERSTATE / FREEWAY
( ) RRR NON-INTERSTATE / FREEWAY
( ) TDLC / NEW CONSTRUCTION / RECONSTRUCTION
( ) TDLC / RRR
( ) MANUAL OF UNIFORM MINIMUM STANDARDS (FLORIDA GREENBOOK)(OFF-STATE HIGHWAY ONLY)

TRAFFIC

	YEAR	AADT
CURRENT	2009	0
OPENING	2015	10,731
DESIGN	2035	19,746

DESIGN SPEED 65
POSTED SPEED 60

DISTRIBUTION
K 9.0%
D 58.7%
T_{24} 5%

DESIGN SPEED APPROVALS

JOHN S. GOLDEN, P.E.
DISTRICT DESIGN ENGINEER DATE

JARED PERDUE, P.E.
DISTRICT TRAFFIC OPERATIONS ENGINEER DATE

LIST ANY POTENTIAL EXCEPTIONS AND VARIATIONS RELATED TO TYPICAL SECTION ELEMENTS:

NONE

LIST MAJOR STRUCTURES LOCATION/DESCRIPTION - REQUIRING INDEPENDENT STRUCTURE DESIGN:

BRIDGE OVER CLEAR CREEK

LIST MAJOR UTILITIES WITHIN PROJECT CORRIDOR:

AT&T, AT&T DISTRIBUTION, CITY OF MILTON, CSX RAILROAD, EAST MILTON WATER SYSTEM, GULF POWER, MCI, MEDIACOM, OKALOOSA GAS, POINT BAKER WATER SYSTEM, QWEST, SOUTHERN LIGHT, SPRINT/NEXTEL

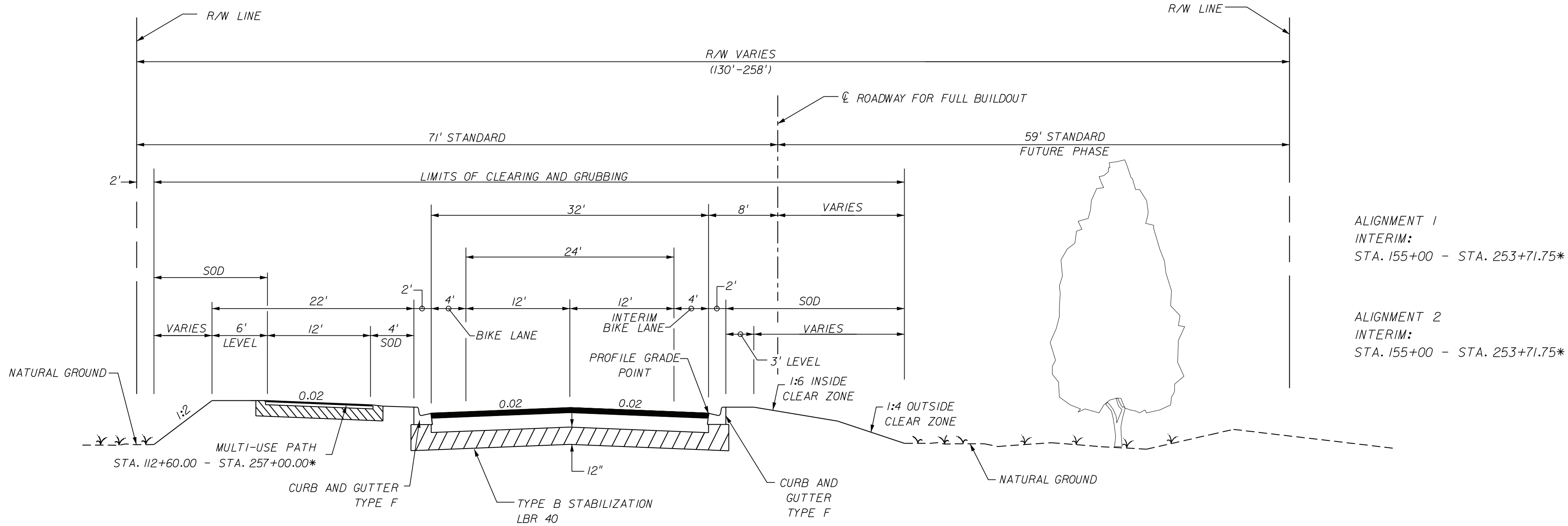
LIST OTHER INFORMATION PERTINENT TO DESIGN OF PROJECT:

SR 87 HAS BEEN DESIGNATED AS A "HURRICANE EVACUATION ROUTE"

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID	416748-3-22-01 AND 416748-3-22-02	FEDERAL AID PROJECT NO.	SFTI 296 R AND S129 348 R	COUNTY NAME	SANTA ROSA
SECTION NO.	58040	ROAD DESIGNATION	SR 87 CONNECTOR	LIMITS/MILEPOST	ALIGNMENT 1 - STA 100+00 - STA 455+15 ALIGNMENT 2 - STA 100+00 - STA 505+49
PROJECT DESCRIPTION	SR 87 CONNECTOR FROM SR 10 (US 90) TO SR 87 NORTH				

PROPOSED INTERIM URBAN ROADWAY TYPICAL SECTION



*FOR AREAS WITHOUT THE MULTI-USE PATH, THE LEFT SIDE WILL MIRROR THE RIGHT SIDE

DESIGN SPEED = 45 MPH
CLEAR ZONE = 24'

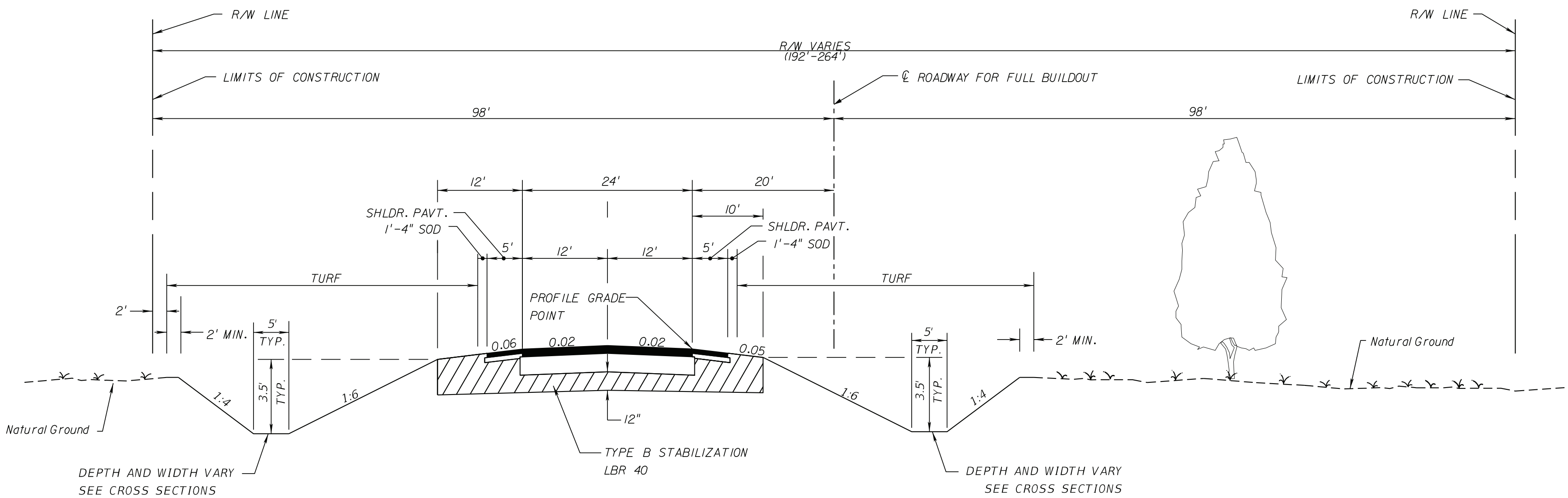
*INCLUDES BLACKWATER RIVER BRIDGE

APPROVED BY JESSICA BLOOMFIELD, P.E.		FDOT CONCURRENCE		FHWA CONCURRENCE	
Engineer Of Record	DATE	JOHN S. GOLDEN, P.E. FDOT District Design Engineer	DATE	FHWA Transportation Engineer	DATE

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID	416748-3-22-01 AND 416748-3-22-02	FEDERAL AID PROJECT NO.	SFTI 296 R AND S129 348 R	COUNTY NAME	SANTA ROSA
SECTION NO.	58040	ROAD DESIGNATION	SR 87 CONNECTOR	LIMITS/MILEPOST	ALIGNMENT 1 - STA 100+00 - STA 455+15 ALIGNMENT 2 - STA 100+00 - STA 505+49
PROJECT DESCRIPTION	SR 87 CONNECTOR FROM SR 10 (US 90) TO SR 87 NORTH				

PROPOSED INTERIM RURAL ROADWAY TYPICAL SECTION



DESIGN SPEED = 65 MPH
CLEAR ZONE = 36'

ALIGNMENT 1
BUILDOUT:
STA. 253+71.75 - STA. 441+89*

ALIGNMENT 2
BUILDOUT:
STA. 253+71.75 - STA. 480+00*

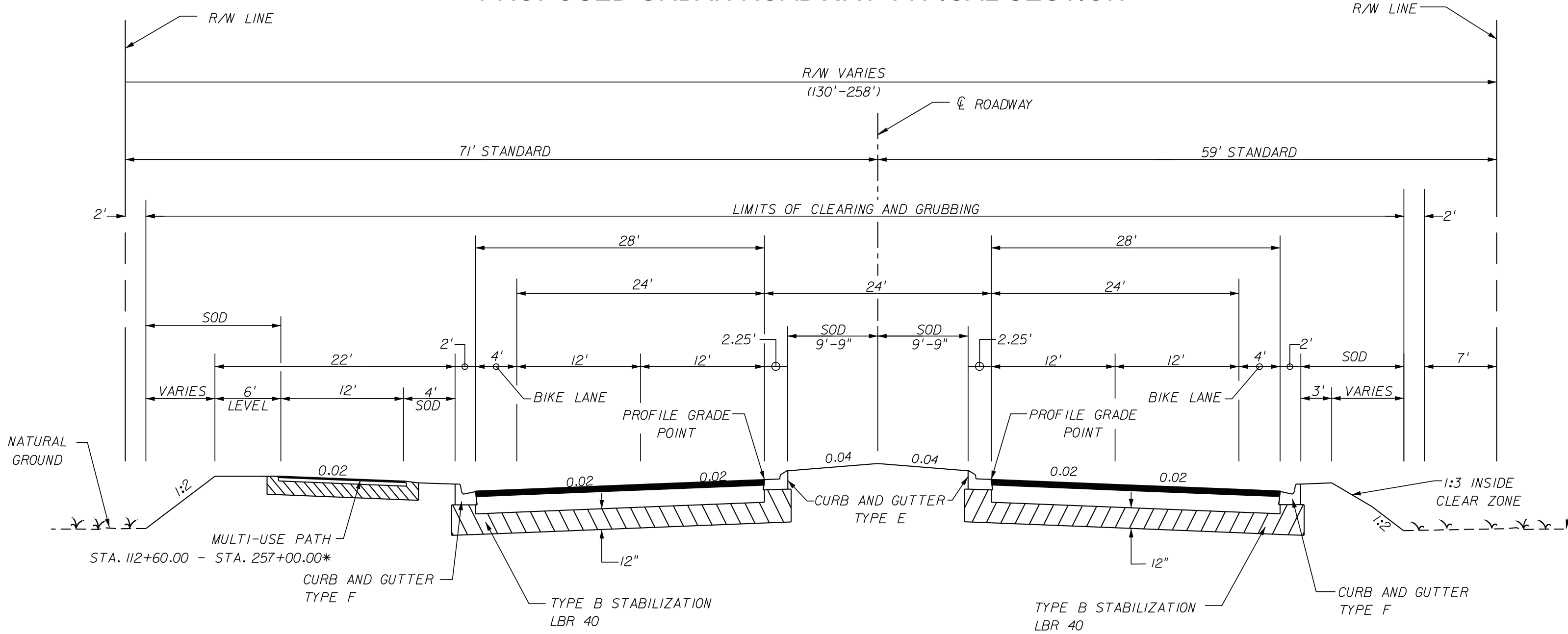
*INCLUDES CLEAR CREEK BRIDGE

APPROVED BY JESSICA BLOOMFIELD, P.E.		FDOT CONCURRENCE		FHWA CONCURRENCE	
Engineer Of Record	DATE	JOHN S. GOLDEN, P.E. FDOT District Design Engineer	DATE	FHWA Transportation Engineer	DATE

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID	416748-3-22-01 AND 416748-3-22-02	FEDERAL AID PROJECT NO.	SFTI 296 R AND S129 348 R	COUNTY NAME	SANTA ROSA
SECTION NO.	58040	ROAD DESIGNATION	SR 87 CONNECTOR	LIMITS/MILEPOST	ALIGNMENT 1 - STA 100+00 - STA 455+15 ALIGNMENT 2 - STA 100+00 - STA 505+49
PROJECT DESCRIPTION	SR 87 CONNECTOR FROM SR 10 (US 90) TO SR 87 NORTH				

PROPOSED URBAN ROADWAY TYPICAL SECTION



ALIGNMENT 1
BUILDOUT:
STA. 100+00 - STA. 253+71.75*
STA. 441+89 - STA. 455+15

INTERIM:
STA. 100+00 - STA. 155+00
STA. 441+89 - STA. 455+15

ALIGNMENT 2
BUILDOUT:
STA. 100+00 - STA. 253+71.75*
STA. 480+00 - STA. 505+64

INTERIM:
STA. 100+00 - STA. 155+00
STA. 480+00 - STA. 505+64

*FOR AREAS WITHOUT THE MULTI-USE PATH, THE LEFT SIDE WILL MIRROR THE RIGHT SIDE

DESIGN SPEED = 45 MPH
CLEAR ZONE = 24'

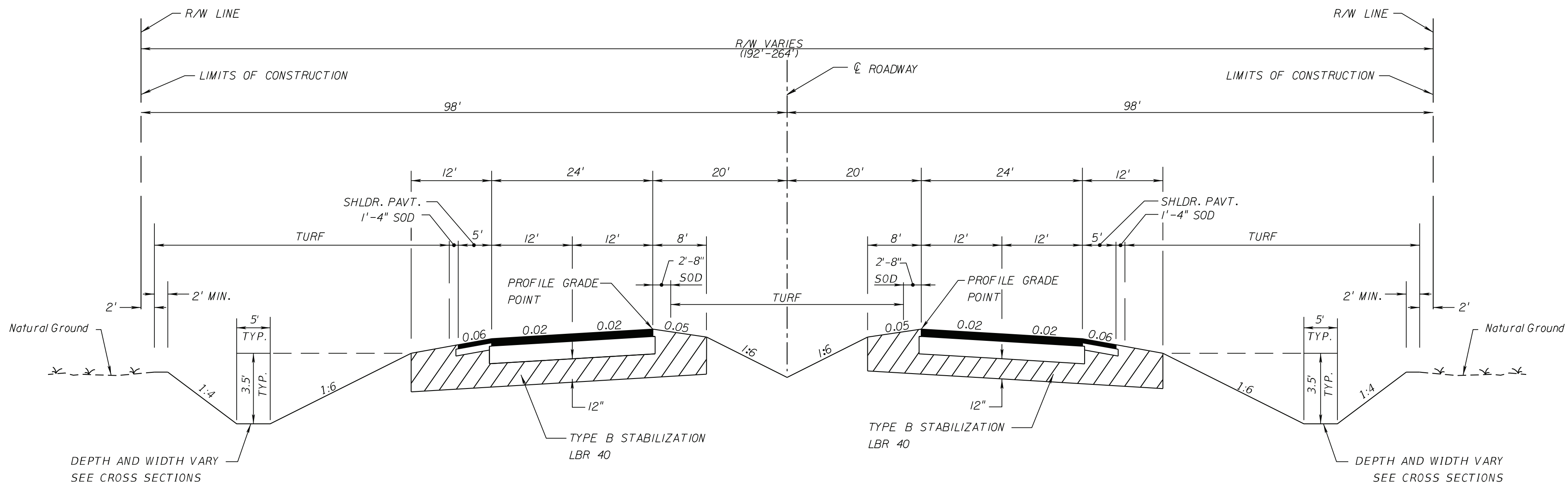
*INCLUDES BLACKWATER RIVER BRIDGE

APPROVED BY JESSICA BLOOMFIELD, P.E.	FDOT CONCURRENCE	FHWA CONCURRENCE
Engineer Of Record — DATE	JOHN S. GOLDEN, P.E. FDOT District Design Engineer — DATE	FHWA Transportation Engineer — DATE

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID	416748-3-22-01 AND 416748-3-22-02	FEDERAL AID PROJECT NO.	SFTI 296 R AND S129 348 R	COUNTY NAME	SANTA ROSA
SECTION NO.	58040	ROAD DESIGNATION	SR 87 CONNECTOR	LIMITS/MILEPOST	ALIGNMENT 1 - STA 100+00 - STA 455+15 ALIGNMENT 2 - STA 100+00 - STA 505+49
PROJECT DESCRIPTION	SR 87 CONNECTOR FROM SR 10 (US 90) TO SR 87 NORTH				

PROPOSED RURAL ROADWAY TYPICAL SECTION



DESIGN SPEED = 65 MPH
CLEAR ZONE = 36'

ALIGNMENT 1
BUILDOUT:
STA. 253+71.75 - STA. 441+89*

ALIGNMENT 2
BUILDOUT:
STA. 253+71.75 - STA. 480+00*

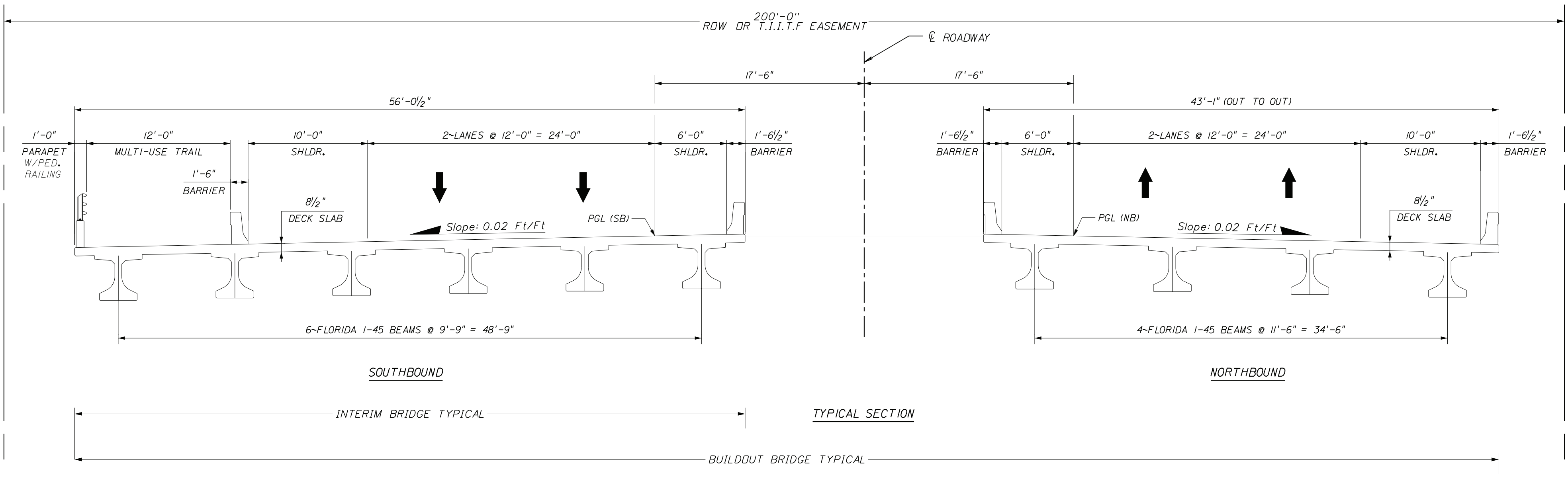
*INCLUDES CLEAR CREEK BRIDGE

APPROVED BY JESSICA BLOOMFIELD, P.E.	FDOT CONCURRENCE	FHWA CONCURRENCE
Engineer Of Record — DATE	JOHN S. GOLDEN, P.E. FDOT District Design Engineer — DATE	FHWA Transportation Engineer — DATE

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID	416748-3-22-01 AND 416748-3-22-02	FEDERAL AID PROJECT NO.	SFTI 296 R AND S129 348 R	COUNTY NAME	SANTA ROSA
SECTION NO.	58040	ROAD DESIGNATION	SR 87 CONNECTOR	LIMITS/MILEPOST	ALIGNMENT 1 - STA 100+00 - STA 455+15 ALIGNMENT 2 - STA 100+00 - STA 505+49
PROJECT DESCRIPTION	SR 87 CONNECTOR FROM SR 10 (US 90) TO SR 87 NORTH				

PROPOSED BRIDGE TYPICAL SECTION - BLACKWATER RIVER BRIDGE - INTERIM AND BUILDOUT



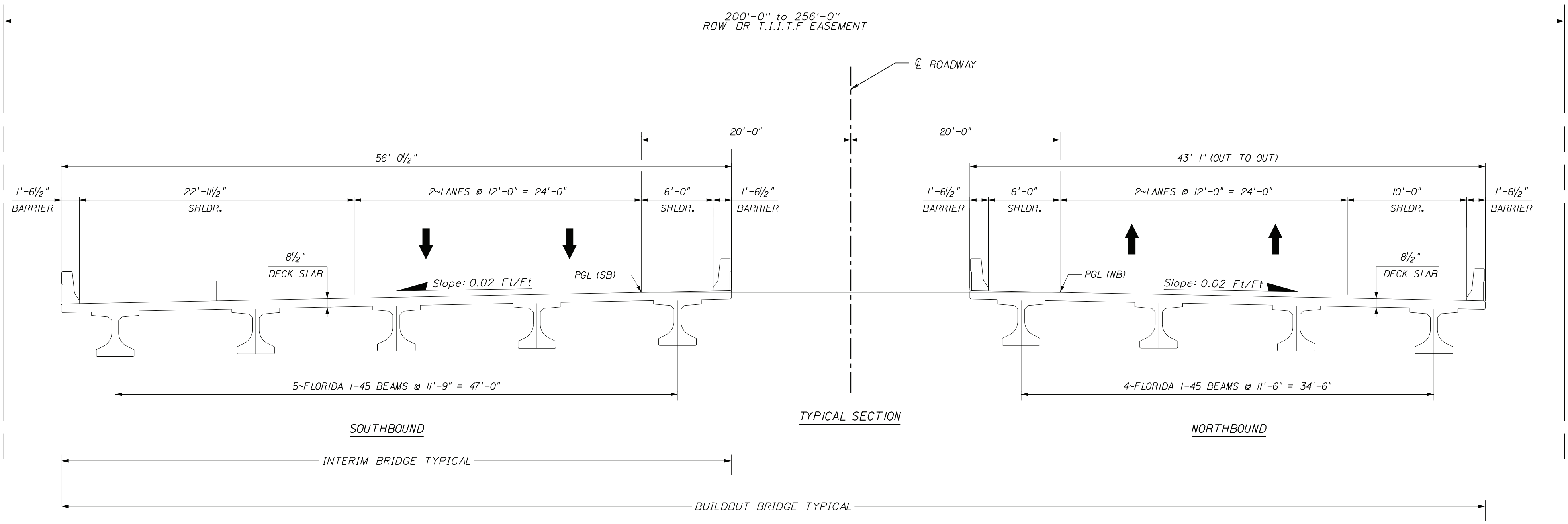
ALIGNMENTS 1 AND 2
BUILDOUT AND INTERIM:
STA. 198+00 - STA. 253+71.75 (BLACKWATER RIVER BRIDGE)

APPROVED BY JESSICA BLOOMFIELD, P.E.	FDOT CONCURRENCE	FHWA CONCURRENCE
Engineer Of Record — DATE	JOHN S. GOLDEN, P.E. FDOT District Design Engineer — DATE	FHWA Transportation Engineer — DATE

PROJECT IDENTIFICATION

FINANCIAL PROJECT ID	416748-3-22-01 AND 416748-3-22-02	FEDERAL AID PROJECT NO.	SFTI 296 R AND SI29 348 R	COUNTY NAME	SANTA ROSA
SECTION NO.	58040	ROAD DESIGNATION	SR 87 CONNECTOR	LIMITS/MILEPOST	ALIGNMENT 1 - STA 100+00 - STA 455+15 ALIGNMENT 2 - STA 100+00 - STA 505+49
PROJECT DESCRIPTION	SR 87 CONNECTOR FROM SR 10 (US 90) TO SR 87 NORTH				

PROPOSED BRIDGE TYPICAL SECTION - CLEAR CREEK BRIDGE - INTERIM AND BUILDOUT



ALIGNMENTS 1 AND 2 BUILDOUT AND INTERIM:
STA. 300+25 - STA. 302+05 (CLEAR CREEK BRIDGE)

APPROVED BY JESSICA BLOOMFIELD, P.E.		FDOT CONCURRENCE		FHWA CONCURRENCE	
Engineer Of Record	DATE	JOHN S. GOLDEN, P.E. FDOT District Design Engineer	DATE	FHWA Transportation Engineer	DATE

APPENDIX D
Bridge Hydraulics Recommendations

Technical Memorandum

This technical memorandum details the results of Hydrologic and preliminary Hydraulic investigation for the proposed bridge crossings of the SR 87 Connector over the Blackwater River and Clear Creek.

Santa Rosa County
Florida

Financial Project No.'s:
416748-3-22-01, 416748-3-22-02,
416748-4-22-01, 416748-4-22-02,
And 416748-4-22-90
ETDM No.:12597
Federal Aid Project No.:
SFT1296R, S129348R

January 2013

Prepared For:
Florida Department of Transportation
District Three
Chipley, FL

Prepared by:
The Balmoral Group.
165 Lincoln Avenue
Winter Park, Florida 32789
Phone: 407.629.2185

PROFESSIONAL ENGINEER CERTIFICATION

I hereby certify that I am a registered professional engineer in the State of Florida practicing engineering with **The Balmoral Group** and that I have supervised the preparation of and approve the analysis, findings, opinions, conclusions and technical advice hereby reported for:

PROJECT: Technical Memorandum

SR 87 Connector Proposed Bridge Crossings of Clear Creek and Blackwater River

Financial Project ID: 416748-3-32-01

Santa Rosa County, Florida

The engineering work represented by this document was performed through the following duly authorized engineering business:

The Balmoral Group

165 Lincoln Avenue

Winter Park, Florida 32789

Telephone: (407) 629-2185

Certificate of Authorization No. **26123**

This technical memorandum provides the preliminary results of the Project Design and Environmental (PD&E) investigation into the construction of the proposed SR 87 connector bridges over Clear Creek, and the Blackwater River, in Santa Rosa County. I acknowledge that the procedures and references used to develop the results contained in this report are standard to the professional practice of hydrologic analysis and hydraulic engineering as applied through professional judgment and experience.

Any engineering analysis, documents, conclusions or recommendations relied upon from other professional sources or provided with responsibility by the client are referenced accordingly in the following report.

FLORIDA REGISTERED ENGINEER:

Gregory S. Seidel, P.E.

Name

REGISTRATION NUMBER: FL #47571

SIGNATURE: ____

DATE: ______________

Executive Summary

This report forms a Technical Memorandum for the proposed construction of bridges over Clear Creek, and the Blackwater River as part of the SR 87 Connector, in Santa Rosa County, Florida. This report was conducted in a 'desktop' format, which indicates that neither detailed field investigation, nor detailed hydraulic bridge design has taken place, and conclusions should be interpreted within this context. This report forms the basis of a more detailed Bridge Hydraulics Report that should take place prior to design of either of the SR 87 connector bridges.

Hydrologic analysis of the basins draining to the location of the two proposed bridges was undertaken and verified against previously published investigations. The final adopted peak discharges for the 50 year ARI (Average Recurrence Interval) flood are shown in **Table ES1** below.

Table ES1: Summary of Design Peak 50 year ARI Flows at the proposed bridge crossings of the SR 87 Connector

SR 87 Bridge Crossing	ARI	Final Peak Streamflow (cfs)
Blackwater River	50	71,400
Clear Creek	50	5,640

Preliminary investigations were completed for this report determine that the bridge to span Clear Creek should have a minimum width of approximately 180 feet to meet hydraulic criteria, however due to the parallel alignment of the channel at the position of the proposed bridge, the opening width may have to be increased or another management method incorporated. The bridge is to have a low chord no lower than 19.17 feet NAVD with the minimum opening width.

The preliminary proposed bridge to span Blackwater River should have a length of 5,560 feet, and have a low chord no lower than 21 feet NAVD over the river. A minimum low chord of 27.70 feet NAVD is required to span the Blackwater Heritage State Trail. The length and low chord specification will ensure that the proposed bridges do not adversely impact the flood stages for the 100 year ARI flood by more than 1 foot, achieve environmental elements and meet minimum requirements for clear span over the Blackwater Heritage State Trail and Pat Brown Road. The preliminary design stages for the 50 year ARI Flood are shown below in Table ES2 for both proposed bridge crossing locations.

Table ES2: Summary of Preliminary Design Peak 50 year ARI Stages at the proposed bridge crossings of the SR 87 Connector

SR 87 Bridge Crossing	ARI	Peak Stages with No Bridge (feet NAVD)	Peak Stages with Proposed Bridge (feet NAVD)	Minimum bridge low chord elevation (feet NAVD)	Recommended Bridge Length (feet)
Blackwater River	50	18.00	<19.00	21.00 over river and floodplain and 27.70 over the Blackwater Heritage State Trail	5,560
Clear Creek	50	15.95	16.95	18.95	180

Clear Creek has shown channel variation over the last 50 years. The channel banks should be stabilized adjacent to the roadway within the right-of-way using rubble rip-rap.

General and Aggradation/Degradation Scour was considered and it was found that there is no indication of long term bed elevation shift, nor lateral movement for Blackwater River at the location of the proposed bridge. Given the large peak flow rate and sandy soils at the proposed bridge location, a detailed 2-D flow model is recommended to be completed during final design to better quantify peak stages and scour depths.

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1. General Information

1.1. INTRODUCTION

The Florida Department of Transportation has proposed the construction of an additional section of SR 87 to better facilitate vehicular movement in the area (including freight movement) which currently must use a portion of US 90. The construction will also serve as a more direct hurricane evacuation route from coastal areas into northern areas, including Alabama. Additionally, the new segment of SR 87 will reduce the vehicular travel currently required to pass through the nearby town of Milton.

The construction of this new segment of SR 87 will require two new bridges to be constructed, one of which will need to cross the Blackwater River, and the second, Clear Creek, a tributary of Blackwater River. This report aims to provide details on the current hydrologic conditions at the site of both proposed bridge crossings and provide preliminary requirements for bridge length and low chord elevation, evaluate environmental factors that exist, as well as carry out lateral and long term aggregation/degradation analysis, to ensure an appropriate and environmentally sensitive outcome is achieved.

1.2. PROJECT LOCATION AND DATUM

The locations of the proposed bridges over Blackwater River and Clear Creek are located approximately 4 miles and 3 miles, respectively, North-East of the city of Milton, within the Santa Rosa County, Florida. The proposed Clear Creek Bridge is located in Section 24, Township 2 and Range 28, and the proposed Blackwater River Bridge is situated in Sections 19 and 30 of Township 2 and Range 27. The locations of both bridges are shown in **Figure 1** and **Figure 3** enclosed in **Appendix A**. The site of the Clear Creek Bridge is approximately 1.4 miles upstream of the confluence with Blackwater River, which then drains into Blackwater Bay. The location of the proposed bridge crossing of Blackwater Creek is approximately 2.4 miles upstream from the confluence of Clear Creek and 11 miles upstream from Blackwater Bay.

This project uses the North American Vertical Datum of 1988 *(NAVD88)*, and the horizontal datum for the project is Florida State Plane *(NAD 1983)*, Northern Zone.

1.3. PURPOSE OF THIS TECHNICAL MEMORANDUM

The primary purpose of this technical memorandum is to review, and compare all current work that has been conducted in relation to the hydrologic and hydraulic investigations of the sites, as well as provide results of an independent investigation into the hydrology and preliminary hydraulics of the two proposed bridge crossings.

This Technical Memorandum will provide critical hydrologic and hydraulic information that can be used to assist in the design of the SR 87 bridge crossings of Blackwater River and Clear Creek. In particular, it will establish design peak discharges at the two sites, and provide design stage estimates to allow the minimum bridge low chord to be established and utilized in the preliminary design plans.

1.4. EXISTING DRAINAGE OVERVIEW

Clear Creek, at the site of the proposed bridge, drains an area of approximately 23 square miles, and Blackwater River an area of 704 square miles. **Plate 1** below, as well as **Figure 2** in **Appendix A** shows the basins for both proposed bridge crossings.

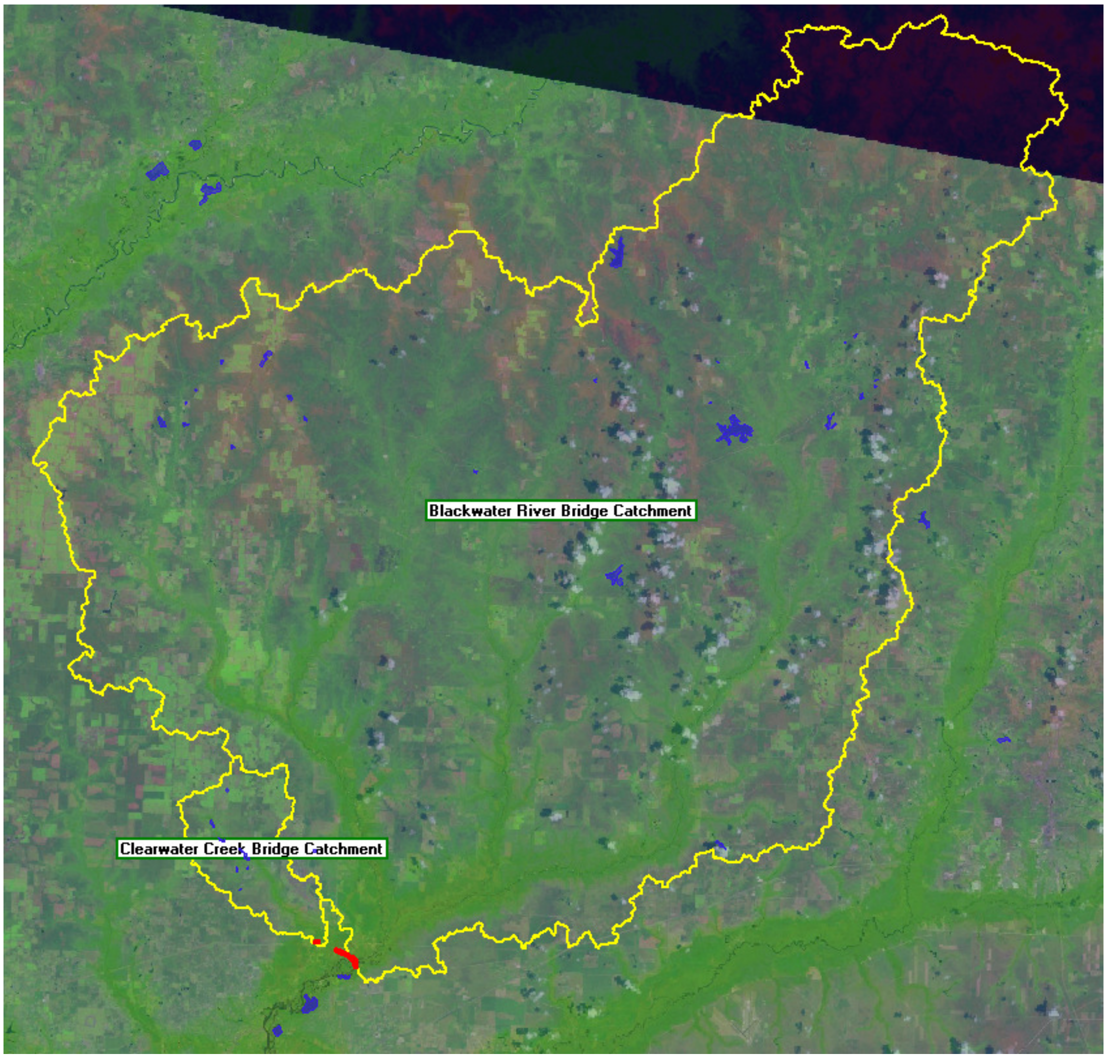


Plate 1: Location and Basins for the two proposed bridge crossings

Clear Creek generally drains from northwest to southeast, and Blackwater River drains from northeast to southwest and meanders considerably in some sections; however, the river has numerous tributaries, such as Big Coldwater Creek.

As shown in **Plate 2 and 3** below, the area around both proposed bridge crossing sites is undeveloped and comprises dense vegetation and tree coverage. The trees and ground cover help to maintain the integrity of the natural channel during low flows and floods. It should be noted, that an area adjacent to both bridges has been cleared, and contains short shrubs (as seen in **Plate 2**), due to a power line easement.

Plate 2: Common vegetation in the clearing adjacent to the proposed Clear Creek bridge site, and the site from the air.

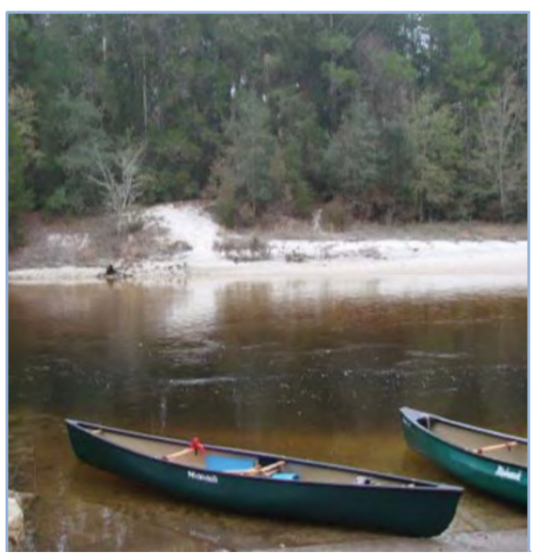


Plate 3: Proposed Bridge location over Blackwater River, and the normal vessels traversing the river.

1.5. TAILWATER

1.5.1. Clear Creek

Gage stations were investigated to provide a suitable tailwater elevation; however, no gages exist at the proposed site of the bridge, and the Clear Creek Gage (USGS 02370550, Clear Creek near Milton, FL) has only intermittent stage data from between 1983 and 1998, and hence does not include sufficient data to determine the design flood peak stages.

As a result, flood behavior in the vicinity of the proposed SR 87 connector bridge crossing over Clear Creek was defined using a HEC-RAS hydraulic computer model of Clear Creek that was developed specifically for this investigation. In order to ensure that flood behavior in the vicinity of the bridge is being reliably defined by the HEC-RAS model, it was necessary to establish reliable tailwater estimates.

As the confluence of Clear Creek with Blackwater River is located only 1.4 miles downstream of the proposed bridge crossing, it was considered that backwater impacts from Blackwater River would impact stages along Clear Creek. As a result, tailwater elevations published in Figure 01P (enclosed in **Appendix E**) by FEMA in the 1996 Flood Insurance Study were utilized to set the tailwater in the HEC-RAS model. These elevations were 13 feet NAVD for the 50 year ARI (Average Recurrence Interval) flood and 17 feet in the 100year ARI flood.

It should be noted that this is a conservative approach to tailwater derivation, as flooding in different sized basins will peak at different times. As the Clear Creek Basin is significantly smaller than the Blackwater River Basin, the relative timing of peak flows will undoubtedly vary, and hence a 50 year ARI rainfall event in the Clear Creek Basin, may only yield a 20 year ARI flood peak at the confluence of the Blackwater River. The opposite is also possible; however, as lower flows would be moving from the Clear Creek Basin at the time of this larger tailwater peak, it is most likely that this would not form the critical scenario.

1.5.2. Blackwater River

Gage stations were investigated to provide a suitable tailwater elevation; however, as previously stated, no gages exist at the proposed site of the bridge, and there are no gages downstream, nor upstream for a significant distance of the proposed site on the Blackwater River. Similarly to the tailwater for Clear Creek, details are available from the 1996 FEMA Flood Insurance Study regarding flood stages along Blackwater River, including a transect at the approximate location of the proposed bridge crossing.

The FEMA stages were evaluated for appropriateness of use. It was found that the stages were estimated using a USACE HEC-2 Model developed using surveyed field data. The results presented in the study are considered to provide a reliable representation of design stages along the river for planning purposes. As a result, stages can be read from Figure 01P (enclosed in **Appendix E**) in the study and utilized to estimate the required bridge clearance. The stages will be adopted as 18 feet NAVD for the 50 year ARI flood, and 20 feet for the 100 year ARI flood.

1.6. WETLAND AND FLOODPLAIN IMPACTS AND MITIGATION

As the two bridges will be constructed on sites that do not currently have any structures, impacts on the wetlands and forested areas will occur. Mitigation will be required to account for these impacts. Remediation techniques that have been outlined for possible use for these bridges include a mitigation bank credit purchase, or a Senate Bill Mitigation; however, the form of mitigation will be determined during permitting by the Interagency Review Team (IRT).

Section 60.3(c) (10) of Title 44 of the code of Federal Regulations requires that the proposed bridge not increase peak 100 year water surface elevations by more than 1 foot relative to the natural (i.e.: no bridge) condition at any location. The preliminary hydraulic analysis for the proposed Clear Creek Bridge in Section 3.1.5 demonstrates that the proposed bridge satisfies this criterion. The proposed bridge crossing of Blackwater River is shown to satisfy this criterion by spanning the FEMA delegated Zone AE regulated floodway as well as the northern floodplain.

1.7. HYDRAULIC DESIGN CRITERIA

The Florida Department of Transportation (FDOT) Drainage Manual (2012) stipulates a range of criteria that must be satisfied for any new or replacement structures. A summary of these criteria is provided below for the SR 87 Connector Bridges:

Design Frequency = 50 year (projected 20 year ADT greater than 1,500 and required for emergency access);

Vertical clearance = 2 feet above peak design flood stage for drift clearance / 6 feet above normal high water for navigation clearance (not applicable as both Clear Creek and Blackwater River are not navigatable by vessel other than canoe/kayak)

The ten feet berm to facilitate construction, reduce scour potential, and provide for abutment stability shall be provided between the top edge of main channel and the toe of spill through abutments;

Scour protection must be designed to withstand the worst case scour condition up to and including the 100 year event (not covered in this investigation); and,

Scour must be checked during the worst case scour conditions up to and including the 500 year event to ensure structural integrity is maintained (not covered in this investigation).

2 Hydrologic Analysis

2.1 General

In order to be able to reliably define flood behavior in the vicinity of the proposed SR 87 connector bridges over both Clear Creek and Blackwater River, it is first necessary to establish reliable design discharge estimates. The following sections describe the hydrologic procedures that were employed to derive the design discharges.

2.2 Drainage Basin

2.2.1. Clear Creek

Clear Creek, at the site of the proposed SR 87 connector bridge, drains an area of approximately 23 square miles. The basin varies in elevation from 240 feet NAVD in the upper basin to 10 feet NAVD at the site of the proposed bridge. There are two major storage dams located in the basin; however the land use within the basin is predominantly rural, agricultural and natural wooded area. The basin is presented in **Figure 2** in **Appendix A**.

Clear Creek drains into the Blackwater River. As the Blackwater River is potentially liable to tidal influence (due to the channel invert being below sea level), it was considered necessary to investigate whether there was the potential for Clear Creek to also be tidally influenced. USGS gage 02370550 (Clear Creek near Milton FL) is located just downstream of the proposed bridge crossing of Clear Creek, and analysis of the minimum water level yielded a stage of 3.84 feet NAVD, with a channel invert at the proposed bridge site approximately equal to this, in which is well above any possible normal tidal influence, and as a result, it was determined that the site is not subject to tidal flows *(i.e.: freshwater flows in one direction only).*

2.2.2. Blackwater River

The Blackwater River at the proposed bridge site drains an area of approximately 704 square miles. The basin varies in elevation from 280 feet NAVD to 3 feet NAVD at the location of the proposed bridge site. There exist a number of large dams and wetland areas within the basin; however the land use is predominantly rural, agricultural and has a large proportion of naturally wooded area. The basin is presented in **Figure 2** and an Aerial view is shown in **Figure 7**. Additionally, the basin headwaters, found in Southern Alabama, flow through Okaloosa County and drain 56.6 miles into the Blackwater Bay, approximately 11 miles downstream of the proposed bridge site.

In an effort to quantify if the proposed bridge location would be tidally influenced, an investigation into the tidal levels within Pensacola Bay (the eventual receiving body for flows from Blackwater River) was undertaken. This investigation utilized data from Station id 8729840, located at Pensacola, Pensacola Bay, and provided 19 years of data, which was considered appropriate for this investigation. The gage location, and project vicinity can be seen in **Plate 4**.

Plate 4: Location of Pensacola Bay Tide Gage and Project Area.

The Mean Higher High Water Level (MHHWL) was extracted from the NOAA National Ocean Service records and found to be 1.327 feet NAVD88 (NOAA, 2012). As the elevation of the channel bed at the proposed site of the Blackwater River Bridge is -11feet NAVD88, well below sea level, it was considered that there is the possibility of a tidal influence at the Blackwater River Bridge. As such, further investigation was undertaken, including derivation of the minimum basin flow that could be expected at the proposed bridge site.

As previously stated, no gage exists at the site, however, the gage upstream from the bridge site along the Blackwater River could be used to estimate constant low flows. It was found that a minimum mean annual flow of 130cfs was experienced. Factoring this up by the catchment area ratio (3.5x) of the gaged site to the proposed bridge site, gives a mean annual flow of 455cfs. It was considered that a flow of this magnitude would provide a sufficiently high energy grade line to prevent saltwater intrusion up the river system to the proposed bridge site. Additionally, as the proposed bridge site is located 11 miles upstream of Blackwater Bay, dampening effects on the tide would be significant and hence maintain a constant downstream flow of freshwater at the proposed bridge site, and the site is not considered to be subject to tidal flows *(i.e.: freshwater flows in one direction only).*

It must be noted that the above conclusion is only valid for normal tide situations, and extremely high or low tides may alter the regime. The Pensacola Bay gage has recorded a maximum tide of 8.771 feet NAVD, and a minimum tide of -2.528 feet NAVD, which indicates that extreme tides can occur, most likely due to hurricane surges, and should be considered in future investigations. (Data extracted from http://tidesandcurrents.noaa.gov/data_menu.shtml?stn=8729840 Pensacola, FL&type=Bench Mark Sheets)

2.3 History of Flooding

Both the proposed bridge crossings of Clear Creek and Blackwater River are located in un-developed rural areas and hence there is no documentation of historic flooding in the direct vicinity of the proposed bridges. Gages located on the watercourses are either too far from, or have a very short period of record to be of sufficient use in determining flood behavior at the location of the proposed bridges.

Additionally, FDOT Maintenance has no reoccurring flooding issues within the limits of this project. There has been some record of major flooding during large storms and hurricanes in the vicinity of the Blackwater River Bridge. It is known from previous investigations and discussion with Public Works Officers that the power easement, located adjacent to the proposed Blackwater River Bridge crossing location, and Pat Brown Road, repeatedly floods to the 100 year flood zone line.

An investigation of storm surge risk, carried out from the National Oceanic and Atmospheric Administration's (NOAA's) Storm Surge Interactive Risk Maps resulted in an acknowledgement of a risk of storm surges at the proposed location of the Blackwater River Bridge. The storm surge elevations associated with a Category 3 through 5 hurricane are between 2 and 10 feet, and a Category 1 hurricane had the storm surge potential of 2 feet just downstream of the bridge location, and as such, there exists the possibility of storm surges in a hurricane of any category. The location of the proposed Clear Creek Bridge did not yield any risk of hurricane surge.

2.4 Previous Studies

A number of previous studies have been carried out in the vicinity of the two proposed bridge sites. The major reports are listed below and a brief description follows:

- FEMA Flood Insurance Study, FEMA, 1996
- Draft BHR Blackwater River, Metric Engineering, 2012
- Draft BHR Clear Creek, Metric Engineering, 2012
- BHR FDOT SR 87 Over Clear Creek, Project Development and Environmental Phase, Volkert Inc, August 2010

<u>FEMA Flood Insurance Study</u>

Although not done to investigate the construction of the two proposed bridges, the FEMA Flood Insurance Study provides an insight into the flooding behavior that occurs within both Blackwater River and Clear Creek. It provides a guide to the peak flows that could be expected, appropriate stages to adopt in hydraulic models and allow verification of results. The FEMA FIRM for Blackwater River is provided in **Figure 4** enclosed in **Appendix A**.

<u>Draft Bridge Hydraulics Report, Blackwater River</u>

The Blackwater River BHR was prepared by Metric Engineering on behalf of the FDOT to investigate the feasibility, design requirements, and environmental considerations pertaining to the construction of the new bridge over the Blackwater River. Data available from this report includes a hydrologic and hydraulic assessment of the proposed site, and design of the bridge, including impacts and remediation plans for any adverse impacts and coordination with local agencies.

Draft Bridge Hydraulics Report, Clear Creek

The Clear Creek BHR was prepared by Metric Engineering on behalf of the FDOT to investigate the feasibility, design requirements, and environmental considerations pertaining to the construction of new bridges over the Blackwater River, and Clear Creek, respectively. Data available from this report includes a hydrologic and hydraulic assessment of the proposed site, and design of the bridge, including impacts and remediation plans for any adverse impacts and coordination with local agencies.

Bridge Hydraulics Report, FDOT SR 87 Over Clear Creek

This Bridge Hydraulic Report was prepared for the FDOT in the Project Development and Environmental (PD&E) stage for the replacement of the existing SR 87 Bridge over Clear Creek and recommends replacement bridge specifications, as well as covers some hydrology and hydraulics of the Clear Creek basin draining to the location. Comparisons between hydrologic conditions and expected scour can be carried out with data presented in this report.

2.5 Peak Flow Analysis

2.5.1. Flood Frequency Analysis

In order to generate reliable design stages for the proposed SR 87 bridges, it was necessary to compute reliable peak flow estimates for the Blackwater River and Clear Creek at the site of the proposed bridges.

The 2012 FDOT Drainage Manual suggests that design discharge estimates be determined utilizing a flood frequency analysis of gages with a suitable length of stream-flow record. As no stream gage is located at the exact location of the proposed bridges, a search for nearby gages was undertaken, and two gages within the basin were identified. These gages are namely USGS 02370500 – Big Coldwater Creek near Milton FL, and USGS 02370000 – Blackwater River near Baker, FL. Although other gages are also located within the basin, the period of record and geographic location within the basin were deemed inappropriate to supply meaningful stream flow records over an appropriate period of time.

A Flood Frequency Analysis was undertaken using the peak streamflow records for these two gages utilizing the USGS PeakFQ software and using input data gained from the USGS National Water Information System. The PeakFQ software uses the methods established by the U.S. Water Resources Council Bulletin 17A (U.S. Water Resources Council, 1977).

The basin areas, slope, and proportion of lakes for the basins of the two before-mentioned gages were derived for input into the National Streamflow Statistics (NSS) program, in which utilizes the USGS regression equations to provide an estimate of design peak stream flow for catchments throughout the United States.

Table 1 below provides the details of the two before-mentioned gages including the period of record, and basin area. **Plate 5** also identifies their location within the Blackwater River basin. The required input parameters were generated by using the CatchmentSIM software, and validated against basin areas stated by the USGS Water Resources Stream Site description.

Table 1 – gages with appropriate data for use in a flood frequency analysis.

Gage ID	Watercourse	Period of Record	Basin Area (sq.miles)	Slope (ft/mile)	Lakes (%)
02370500	Big Coldwater Creek	1939-2011	238	6.84	0.09
02370000	Blackwater River	1951-2011	206	7.92	0.34

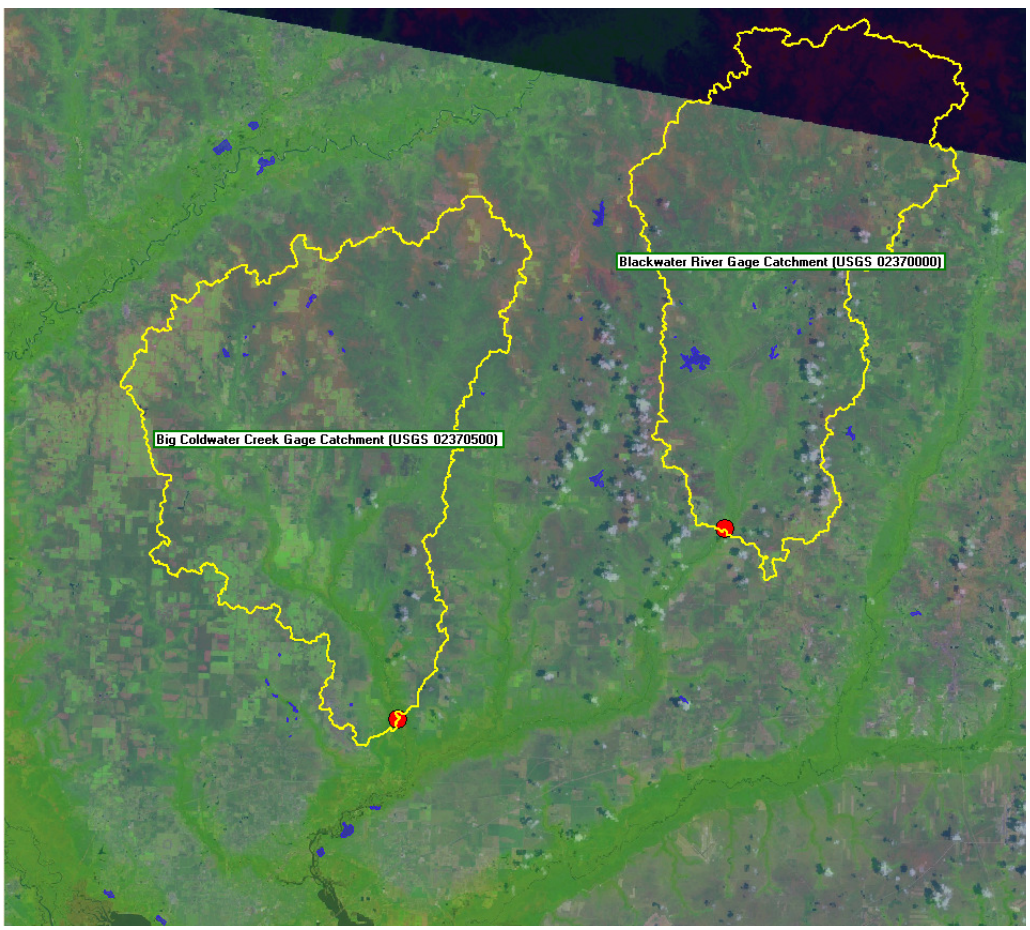


Plate 5: Location and basins for the gages within the Blackwater River basin

The aim of utilizing a Flood Frequency Analysis (FFA) of these upstream gages was initially to carry out the NSS Rural Flood-Probability Estimating Technique of utilizing a weighting for ungaged sites on gaged streams. It was however, determined that this procedure cannot be utilized as the drainage area for both of the gaging stations was less than half the drainage area for the ungaged site (effective range for this method is between 0.5 and 1.5 times the gaged drainage area).

As such, the USGS (NSS v6 2012) regression equations were utilized to estimate peak design flows at both of the gaging site locations. This aimed to verify the suitability of the NSS discharge estimates at the gage locations and, therefore, to infer a level of confidence with the NSS discharge estimates at the bridge locations. As shown in **Table 2**, there is a significant disparity between peak FFA design flows and design flows predicted by the NSS regression analysis. **Table 2** shows that the NSS regression analysis typically produced peak discharge estimates that were 40% lower than the corresponding FFA peak discharge estimate.

Table 2 – Flood Frequency Analysis and USGS regression results and comparison

	ARI	FFA peak streamflow (cfs)	Regression peak streamflow (cfs)	Calibrated regression peak streamflow (cfs)
Big Coldwater Creek				
	5	11,800	9,060	12,100
	10	17,570	12,900	17,900
	25	27,420	19,200	27,400
	50	36,960	25,200	36,700
	100	48,720	31,800	47,300
	200	63,120	39,700	59,900
	500	87,110	51,900	80,300
Blackwater River				
	5	8,970	7,930	10,100
	10	13,330	11,200	14,700
	25	20,640	16,600	22,200
	50	27,610	21,600	29,500
	100	36,070	27,100	37,700
	200	46,280	33,600	47,300
	500	62,980	43,700	62,600

As a result of this variation, adjustment of the input parameters was undertaken by refining the basin slope and % lakes until the peak design discharge estimates generated by the regression analysis agreed (as close as possible) with the design discharge estimates using the flood frequency analysis. Factors of the originally derived parameters for both gages were then calculated, and averaged to provide final factors of 1.9 for the slope parameter, and 1.6 for the % lakes parameter. These factors were the applied to the raw regression analysis discharges to gain 'calibrated' discharge estimates that closely agreed with discharges gained from the flood frequency analysis.

The outcomes of the application of these adjustment factors are shown in **Table 2** above. Derivation of the factors and a summary of the peak flows for all locations is shown in **Appendix B**.

The results of the above process were then compared to a 2006 study by the USGS. The USGS study was completed to determine procedures for estimating flood magnitudes and quantities at ungaged sites. As a result, the peak flows attained through the process outlined above were compared to the results published in the USGS report, and a comparison is shown below in **Table 3**.

The design flows presented below in **Table 3** show that some variation is occurring between the 2006 USGS study and the 'calibrated' NSS regression peak streamflow. Differences can be accounted for by the fact that the analysis done for this project includes an additional 5 years of data, including data from 2009, in which represents a significant flood event. Additionally, only significant water bodies were considered as lakes in order to maintain a conservative approach to determining peak flows in significant flood events.

Table 3: USGS Magnitude and Frequency of Floods for Rural Streams in Florida Study comparison to derived peak discharges

	ARI	USGS peak streamflow (cfs)	Calibrated NSS regression peak streamflow (cfs)
Big Coldwater Creek			
	5	11,300	12,100
	10	16,500	17,900
	25	24,800	27,400
	50	32,600	36,700
	100	41,600	47,300
	200	52,300	59,900
	500	69,400	80,300
Blackwater River			
	5	8680	10,100
	10	12,500	14,700
	25	18,400	22,200
	50	23,600	29,500
	100	29,500	37,700
	200	36,100	47,300
	500	46,400	62,600

2.5.2. Peak Design Flows

The USGS (NSS v6 2012) regression analysis was then carried out at the site of the proposed bridges, using parameters gained from basin analysis using CatchmentSIM. These parameters are shown below in **Table 4** for both bridge crossings of Blackwater River, and Clear Creek.

Table 4: Regression analysis inputs for the two proposed bridge crossings

SR 87 Bridge	Basin Area	Slope	Lakes (%)
Blackwater River	703.77	4.75	0.2
Clear Creek	22.88	15.31	0.48

As the basins draining to these two bridge crossings were within the same geographic vicinity of the previously analyzed gage basins, it was decided that the previously determined slope and % lakes 'calibration' factors could be appropriately applied to the two bridge crossings to gain peak streamflow values. The results of this application are shown below in **Table 5**, and **Appendix B** contains the derivation calculations.

Table 5: Results of regression analysis and final flow estimates for the two bridge sites.

SR 87 Bridge Crossing	ARI	Raw Regression peak stream flow (cfs)	Calibrated Regression peak stream flow (cfs)
Blackwater River			
	5	18,300	24,000
	10	25,900	34,900
	25	38,500	53,300
	50	50,400	71,400
	100	64,000	92,200
	200	80,400	117,000
	500	106,000	158,000
Clear Creek			
	5	1,630	2,040
	10	2,300	2,940
	25	3,320	4,330
	50	4,250	5,640
	100	5,220	7,020
	200	6,310	8,570
	500	7,950	11,000

To validate the peak stream flows, and the applied factors, a further NSS regression analysis was conducted for the basin draining to the ‘Louisville and Nashville Railroad’ crossing of the Blackwater River. This was chosen as the 1996 FEMA Flood Insurance Study (FEMA, 1996) has published peak flows to this location and could hence allow a comparison at this location. Again, the CatchmentSIM regression derived slope and % lakes parameters were multiplied by the previously determined factors, and ‘calibrated’ flows computed. **Table 6** below provides details of the parameters input for this regression analysis as well as a comparison of these ‘calibrated’ flows with those published in the FEMA Flood Insurance Study.

Table 6: regression analysis inputs and results for the Louisville and Nashville Railroad crossing of Blackwater River

Just downstream of the Louisville and Nashville Railroad	FEMA derived parameters	CatchmentSIM derived parameters
Basin Area (sq.miles)	747.4	748.9
Slope (ft/mile)		4.69
Lakes (%)		0.23
	FEMA peak streamflow (cfs)	Calibrated NSS regression peak streamflow (cfs)
5		24,700
10	35,900	36,000
25		54,900
50	69,900	73,400
100	89,900	94,700
200		121,000
500	152,900	162,000

As can be seen from **Table 6**, a close replication of the FEMA peak streamflow has been attained, which allows a greater confidence in the use of the adjustment factors. Therefore; flows obtained for both the Blackwater River and Clear Creek bridge crossing sites are considered appropriate for use in design.

A further check was undertaken by comparing the FEMA Flood Insurance Rate Map (FIRM) transects at the location of the bridge to the above calculated streamflow. This was conducted by multiplying the cross-sectional area of the transect by the average velocity through the transect (extracted from the FEMA Flood Insurance Study) to gain a 100 year peak streamflow value.

This procedure can only be used as a general comparison due to the use of the average velocity to compute the streamflow, and the fact that the transect area is provided only for the portion of flow that falls within the FEMA criteria of Floodway (obstruction would cause an increase in stage by more than 1 foot). As the floodway carries the vast majority of event streamflow, the comparisons between computed flows should be significantly close, however, stream flow generated by this method should underestimate slightly the total streamflow across the transect as a small proportion will be conveyed in the flood fringe .

The comparison is shown below in **Table 7**, and indicates a fairly close reproduction of the FEMA transect values at the exact location of the proposed bridge crossing on Blackwater River. As can be seen from the results, the FEMA streamflow is slightly below that calculated previously in this study, which as explained, is expected when considering that the FEMA transect area and velocity excludes the conveyance in the flood fringe.

Table 7: Regression flow comparison against FEMA transect 'L' flow (from Flood Insurance Rate Map, Panel 340 of 657)

Floodway section area (Sq.ft)	52105	
Mean Velocity (ft/second)	1.7	
	FEMA peak streamflow (cfs)	Calibrated NSS regression peak streamflow (cfs)
100 year ARI	88,579	92,200

No such transect exists at the site of the proposed Clear Creek bridge crossing, so no comparison is able to occur.

3. Hydraulic Analysis

3.1 Clear Creek

3.1.1. General

A one dimensional steady state HEC-RAS hydraulic model was created for Clear Creek in the vicinity of the proposed bridge. The cross sections were created by sampling a NOAA lidar derived DEM and allowed numerous cross sections to be extracted. These cross sections extended about 600 feet upstream and 1200 feet downstream of the proposed site (measured along the main channel). NAVD 88 datum was utilized, along with the Energy Equation for the modeling approach. The positions of the HEC-RAS cross-sections are shown in **Figure 6** in **Appendix A**, and **Appendix C** provides details of the HEC-RAS Project and Outputs.

The intention of this HEC-RAS model was to try and determine an appropriate preliminary bridge opening length and low member elevation. These parameters would also need to meet the criteria of the NWFWMD (North West Florida Water Management District), that being, an increase in stages upstream of the bridge no greater than 1 foot in the 100 year ARI flood.

3.1.2. DEM

The cross sections utilized in the hydraulic model were extracted from a DEM for the area around the proposed site of the Clear Creek Bridge, and was supplemented with survey data from a previous investigation of Clear Creek. The DEM was generated by interpolating between lidar ground strikes and then creating a 2 foot raster grid representation of the ground surface. The lidar was sourced from the National Oceanic and Atmospheric Administration (NOAA), however, as lidar has difficulty providing elevation data in areas of dense vegetation, or within water bodies, lidar point data in vegetated areas are sparser than in open/clear areas. Additionally, no creek invert elevations were able to be extracted from the lidar, and were instead interpolated from survey from previous studies in the general vicinity of the proposed bridge. (Bridge Hydraulics Report, FDOT ST87 over Clear Creek, Volkert INC, August 2010). This data was deemed acceptable for this preliminary analysis.

3.1.3. Mannings Roughness

The Mannings 'n' values used in the HEC-RAS model cross-sections were determined using the FHWA's (Federal Highway Administration) "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains" (FHWA, 1984). Appropriate parameters were selected based on examination of aerial photography and a limited number of field photographs, and hence are limited in accuracy to the attributes visible in this photography. The adopted Mannings 'n' values are shown below in **Table 8**, and full computations are presented in **Appendix C**.

Table 8: Mannings 'n' values adopted in the HEC-RAS Model (values computated using the FHWA's "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains")

Surface	Adopted Mannings 'n'
Creek channel	0.04
Flood Plain	0.10

3.1.4. Boundary Conditions

Downstream boundary conditions were investigated from multiple sources that were considered likely to impact stages at the proposed bridge crossing. The first of these was the potential for backwater impacts from the Blackwater River. This was investigated by analysis of FEMA Flood Insurance Rate Maps (FIRM) and Figure 01P in the 2006 FEMA study, in which shows that at the confluence of the Clear Creek and Blackwater River, a stage of 13 feet is reached in the 50 year ARI flood, and 17 feet in the 100 year ARI flood. This was utilized as the tailwater in the HEC-RAS model. As previously discussed, this application is a conservative approach as the relative timing between peaks of such largely different basin areas will vary and lead to lower flows from the Clear Creek Basin at the time of the adopted downstream stages on the Blackwater River.

Additionally, a downstream bridge crossing at the Munson Hwy was investigated for any hydraulic backwater impact on the proposed bridge. As no details of this bridge were known, a 'desktop' approach of analysis was conducted to attempt to quantify the potential impacts of this bridge. This approach required the modeling of the bridge as a 180 feet opening, and routing the previously determined flows through it. The impact on upstream stages was quantified, and then added to the backwater effects within Clear Creek. The distance downstream and creek bed slope were then also considered and it was found that this bridge had a small impact on stages at the location of the proposed SR 87 bridge crossing, and these were included in the design model as a known water surface. As this is a Project Development and Environmental (PD&E) phase technical memorandum, detailed analysis of this interaction has not taken place, and hence the Munson Highway bridge should be carefully considered in any further investigations.

Additionally, two further downstream bridges (Pat Brown Rd and Blackwater Heritage State Trail) were again considered for their possible impact on stages at the site of the proposed bridge, however this was quickly ruled out due to the backwater impacts of Blackwater River which would inundate the vicinity of these two downstream bridges, and hence control the water surface elevation in these lower areas of Clear Creek. A more rigorous analysis should be completed in the final design.

3.1.5. Preliminary Design Flood Stages

The process involved in the preliminary design of the SR 87 Bridge over Clear Creek required the modeling of pre-construction conditions along the creek alignment to gain a baseline stage during the 50 year and 100 year ARI flood events. The flows previously described were utilized in the developed HEC-RAS model, and yielded stages of 15.95 feet in the 50 year, and 18.42 feet in the 100 year event. The calculated stage at the proposed bridge site is similar to the FIRM 100 year stage shown on the FIRM map, that being ~18 feet. (It should be noted that the FIRM stages are a whole number rounding and hence allow for up to 0.5 feet variation in stage values).

Next, a post construction scenario was modeled, and consisted of the addition of a bridge in the position of the proposed bridge alignment. Various bridge opening lengths were evaluated and the stages gained compared to the baseline scenario in an attempt to minimize the bridge opening, but still meet the requirements of the NWFWMD in relation to the maximum allowable stage increase due to construction (max 1 foot increase in the 100 year ARI flood).

The outcome of this analysis led to the adoption of a 180 feet bridge, with 1:2 sloping abutments to span the major Clear Creek alignment. The upstream stages that are produced with the above described bridge characteristics are 16.95 feet in the 50 year event, and 19.16 in the 100 year event. This bridge opening size ensures that less than a 1 foot increase in stage in the 100 year event occurs upstream of the proposed bridge, however, as this was only a preliminary design, no bridge piers were included, and hence, upstream stages may increase slightly. As a result, the preliminary minimum low chord should be set at an elevation of 18.95 feet NAVD. The proposed bridge location and length can be seen on **Figure 10** in **Appendix A**.

It is important to note that the Clear Creek channel at the site of the proposed bridge site moves in an east to west direction along the proposed alignment of the roadway, and this can be seen in **Figure 6**. This east to west movement of the channel extends for a distance of over 400 feet, and the required design bridge length is 180 feet (for stage increase criteria). As such, the 180 foot opening length should be seen as the minimum bridge opening criteria, and a number of options exist to manage the potential problems associated with the current creek alignment.

Firstly, a re-alignment of the creek channel could be undertaken, and a skew angle of piers and abutment would be required in order for effective flow through the bridge opening. A second alternative would be to extend the bridge opening to a sufficient width to account for any future channel lateral movement, as well as the current alignment of the creek. The described options should be considered in the final design as well as any other viable alternatives to ensure an optimum solution is gained.

3.2 Blackwater River

The proposed bridge over Blackwater River is located in a position which has received greater attention from regulatory agencies in relation to expected flooding behavior. Additionally, due to the meandering nature of the Blackwater River upstream and downstream of the proposed bridge site, it was decided that a HEC-RAS model would not be appropriate to model the behavior that may occur within the river and the adjacent floodplains. It would be recommended that any further investigations into flood behavior in the vicinity of the proposed bridge utilize a 2D model.

As such, the design of the proposed bridge length and low chord elevation took place utilizing already derived data. However, there were still many factors requiring consideration in which will impact both the length and minimum height of the bridge deck. A summary of these major factors are described below;

- The ability for watercraft to pass under the bridge and navigate the river. It was determined by prior field investigation that the only vessel navigation that occurs is canoes/kayaks, some small motorized flat bottom boats, and personal watercraft and hence requires a minimum horizontal clearance of 10 feet and a minimum vertical clearance of six feet above the mean high water (MHW) to accommodate these vessels.

- The Blackwater River has been studied by FEMA using a USACE HEC-2 step-backwater model and the results are presented on FIRM map 0340G. These results show that a regulated floodway exists as a "Floodway Area" with a zone categorization of AE, indicating that it will be inundated by the 100 year ARI flood. As a result, the proposed bridge will need to be sufficiently sized to span this floodway to ensure flood stage increases upstream of the proposed bridge do not exceed 1 foot.

- The bridge will also be required to provide an overpass route past Pat Brown Road, and the Blackwater Heritage State Trail, and this will require a sufficient height to provide access along these routes. It has been prescribed that a minimum 12 feet clearance be provided between the Blackwater Heritage State Trail and the low member of the proposed bridge.

- As with the Clear Creek Bridge, the 50 year ARI flood stage with an additional two feet debris clearance will be used as the major factor setting the required minimum low member elevation.

With the above factors considered, and the sources of data that are available, design lengths and minimum low chord elevations were able to be estimated for the preliminary design.

In the vicinity of Pat Brown Road, the low member elevation will need to provide sufficient clearance for vehicular movement. Additionally, a 12 feet clearance is required over the Blackwater Heritage State Trail, and hence, a minimum low member elevation of 27.70 feet NAVD is required.

The bridge length will be required to span the entire regulated floodway of the Blackwater River, and additionally, span to ensure that clearance of Pat Brown Road and the Blackwater Heritage State Trail occurs. As such, the bridge length can be set to a design length of 5,560 feet. The proposed bridge location and length can be seen on **Figure 11** in **Appendix A**.

As detailed analysis of the Blackwater River has occurred by FEMA, and the peak flows have been determined as being comparable to those derived in this study, the 50 year stage at the site of the proposed bridge was read from Figure 01P of the FEMA Flood Study (FEMA, 2006). As the proposed bridge crossing is located approximately 2.3 miles (12,100 feet) upstream of the Confluence of Clear Creek, and at the approximate location of Transect 'L', the stage was adopted as 18 feet. As the proposed bridge will span the regulated floodway and an allowance of a maximum 1 foot stage increase could occur with blockage of the floodplain, the post bridge scenario was taken as a stage of 19 feet NAVD. With the required 2 feet debris clearance, the minimum bridge deck low member elevation over the river should be set as 21 feet NAVD.

As the construction of the bridge embankment will cause some obstruction to flow area on the Southern end of the bridge, some Flood Fringe designated area and wetland will be lost. Remediation techniques that have been outlined for use includes a mitigation bank credit purchase, or a Senate Bill Mitigation for wetland impacts. Additionally, an area of floodplain constructed to a lower elevation will also be constructed to account for the lost volume of floodplain by the roadway/bridge embankment. This may be offset by pre-post modeling during the design phase.

4 Scour Analysis

4.1 General

Bridge scour refers to the lowering/movement of the streambed in the vicinity of bridge crossings. It is the biggest cause of bridge failure in the United States *(Florida Department of Transportation, May 2005)*. Therefore, it is important that the potential for scour is analyzed during the design of any bridge so that the bridge foundations can be designed accordingly and such failures can be prevented.

Bridge scour can generally be divided into the following categories:

1. Lateral channel movement;
2. Long term aggradation / degradation;
3. Contraction scour; and,
4. Local pier and abutment scour.

Due to the limited scope of this preliminary design analysis, only item 1 will be evaluated in detail and items 2-4 will be reviewed for scour potential.

4.2 Soil Description

A NRCS SSURGO soils map for the project area is provided in **Figure 5** in **Appendix A**. Key properties for each soil unit in the vicinity of the proposed SR 87 Connector Bridges are also summarized in **Table 9**. **Figure 5** and **Table 9** indicate that the soils immediately adjoining both Clear Creek and Blackwater River generally comprise sand.

The soil properties provided in **Table 9** include the erosion factor, K, which provides an indication of the susceptibility of the soil to sheet and rill erosion from water flow. The soils adjoining the proposed bridge sites are mainly map units 1, 3, 21, and 34. As can be seen from **Table 9** below, these soils generally comprise sand, and have a high Erosion Factor (K), which indicates high erosion potential.

Table 9: Existing Soils Properties based on NRCS Soil Survey

Map Unit Symbol	Soil Name	Hydrologic Soil Group	Erosion Factor K
1	Albany loamy sand, 0 to 5 percent slopes	290.6	11.2%
3	Bibb-Kinston association	763.2	29.5%
5	Bonifay loamy sand, 0 to 5 percent slopes	186.5	7.2%
8	Dothan fine sandy loam, 0 to 2 percent slopes	10.9	0.4%
9	Dothan fine sandy loam, 2 to 5 percent slopes	9.0	0.3%
14	Fuquay loamy sand, 0 to 5 percent slopes	19.4	0.7%
18	Johns fine sandy loam	64.8	2.5%
19	Kalmia loamy fine sand, 2 to 5 percent slopes	85.4	3.3%
21	Lakeland sand, 0 to 5 percent slopes	227.5	8.8%
22	Lakeland sand, 5 to 12 percent slopes	10.1	0.4%
27	Lynchburg fine sandy loam	153.5	5.9%
34	Pactolus loamy sand, 0 to 5 percent slopes	383.2	14.8%
37	Rains fine sandy loam	53.2	2.1%
40	Rutlege loamy sand	148.8	5.8%
44	Troup loamy sand, 0 to 5 percent slopes	64.5	2.5%
46	Troup loamy sand, 8 to 12 percent slopes	22.0	0.8%
47	Troup-Orangeburg-Cowarts complex, 5 to 12	2.3	0.1%

Detailed geotechnical information was also obtained for the project. This included soil borings at two locations along the proposed Blackwater River Bridge alignment, and adjacent to the Blackwater River, and a further boring adjacent to the SR 87 alignment over Clear Creek The geotechnical information was collected by Environmental and Geotechnical Specialists, INC in 2011 and 2012, and a summary of the borings is presented below. The bore positions can be seen on **Figure 7** in **Appendix A**, and the core boring results are provided in **Appendix F**.

<u>Soil Boring B-1</u> (Blackwater River floodplain adjacent to proposed SR 87 alignment)

- 0.0 - 32.5 feet – Loose to medium Dense Medium to Fine Sand (**SP-SM**)
- 32.5 - 65.0 feet – Loose to Medium Dense Silty Fine to Clayey Sand (**SM** to **SC**)
- 65.0 - 82.5 feet - Medium Dense to Sense Medium to Fine Sand (**SP-SM**)
- 82.5 - 100.0 feet - Loose to Medium Dense SIlty Fine Sand (**SM**)

Soil Boring B-2 (adjacent to Blackwater River and proposed SR 87 alignment)

- 0.0 - 25.0 feet – Loose Sand and Fibrous Organics (**SP-SM** & **MUCK**)
- 25.0 - 55.0 feet - Loose to Medium Dense Medium to Fine Sand (**SP-SM**)
- 55.0 - 65.0 feet - Dense to Very Dense Medium to Fine Sand (**SP-SM**)
- 65.0 - 100.0 feet - Loose to Medium Dense Silty Fine Sand (**SM**)

Soil Boring CC-1 (adjacent to Clear Creek and proposed SR 87 alignment)

- 0.0 - 25.0 feet – Loose to Medium Dense Medium to Silty Fine Sand (**SM**)
- 25.0 - 33.0 feet – Medium Dense to Dense Silty Medium Sand with Gravel (**SM**)
- 33.0 - 40.0 feet – Dense Fine Sand (**SP-SM**)
- 40.0 - 75.0 feet – Medium Dense Fine to Silty Fine Sand (**SP-SM** to **SM**)
- 75.0 - 100.0 feet – Very Dense Fine to Silty Fine Sand (**SP-SM** to **SM**)
- 100.0 - 110.0 feet – Medium Dense Silty Fine Sand (**SM**)

The results of examination of the two soil borings confirm that the soil around the Blackwater River Bridge alignment is primarily sand, and a high level of erodibility can be expected on exposed ground. However, as the banks of the river are densely vegetated, little erosion is expected to occur in the present state. However, if the vegetation density was to be altered, by means of clearing or a natural process, then significant erosion during flood events could be expected. Consideration of this should be made during the subsequent design phases and appropriate precautions and rehabilitation implemented.

The soil boring at the location of the proposed Clear Creek Bridge, as well as the close similarities in soil properties from the NRCS soil survey indicate that soil properties are similar to those found at the proposed Blackwater River Bridge, and identical precautions and rehabilitation should be implemented at the Clear Creek Bridge site.

4.3 General Scour/Aggradation and Degradation

General scour refers to bed elevation changes associated with the long-term lateral movement of the river channel. Aggradation and degradation refers to the vertical raising and lowering, respectively, of an entire river reach over extended time-frames.

The potential for general scour and aggradation and degradation in the vicinity of the two proposed SR 87 connector bridges was assessed based on procedures outlined in the Federal Highway Administration Hydraulic Engineering Circular No. 20 *(HEC-20)*, titled "Stream Stability at Highway Structures" *(March 2001).*

A 'desktop' geomorphic assessment was conducted for both the proposed bridge crossings of Clear Creek and Blackwater River using procedures outlined in the Federal Highway Administration Hydraulic Engineering Circular No. 20 (HEC-20), titled "Stream Stability at Highway Structures" (March 2001). The assessment provides a summary of the geomorphic characteristics of the basin. The assessment was completed using available online data sources such as digital elevation models, land use mapping, soils mapping and aerial photographs. The outcomes of this assessment are summarized in **Plate 6 and 7** for Clear Creek and Blackwater River respectively (the section numbers refer to the HEC-20 document).

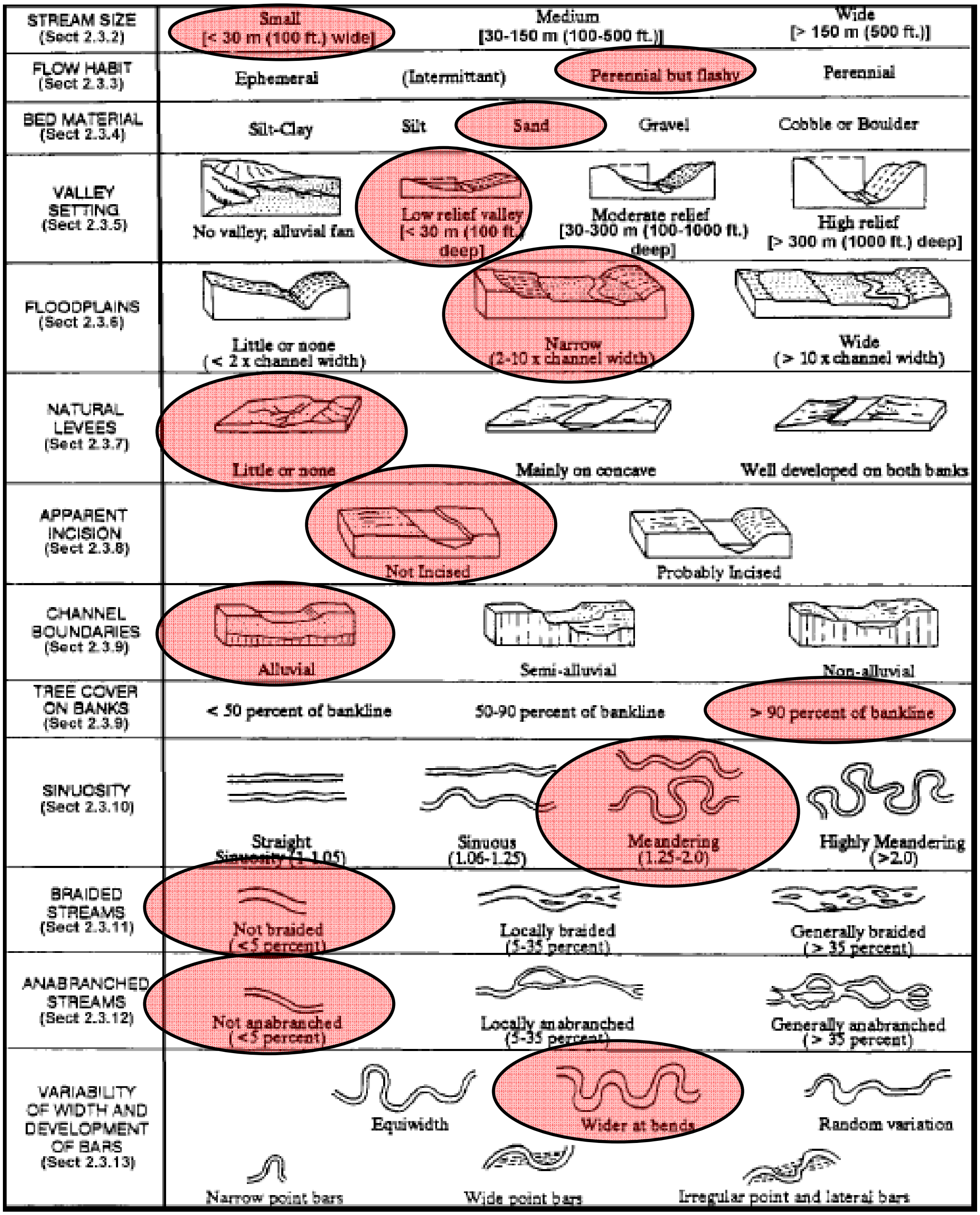


Plate 6: Assessment of Clear Creek geomorphic characteristics at the proposed bridge site

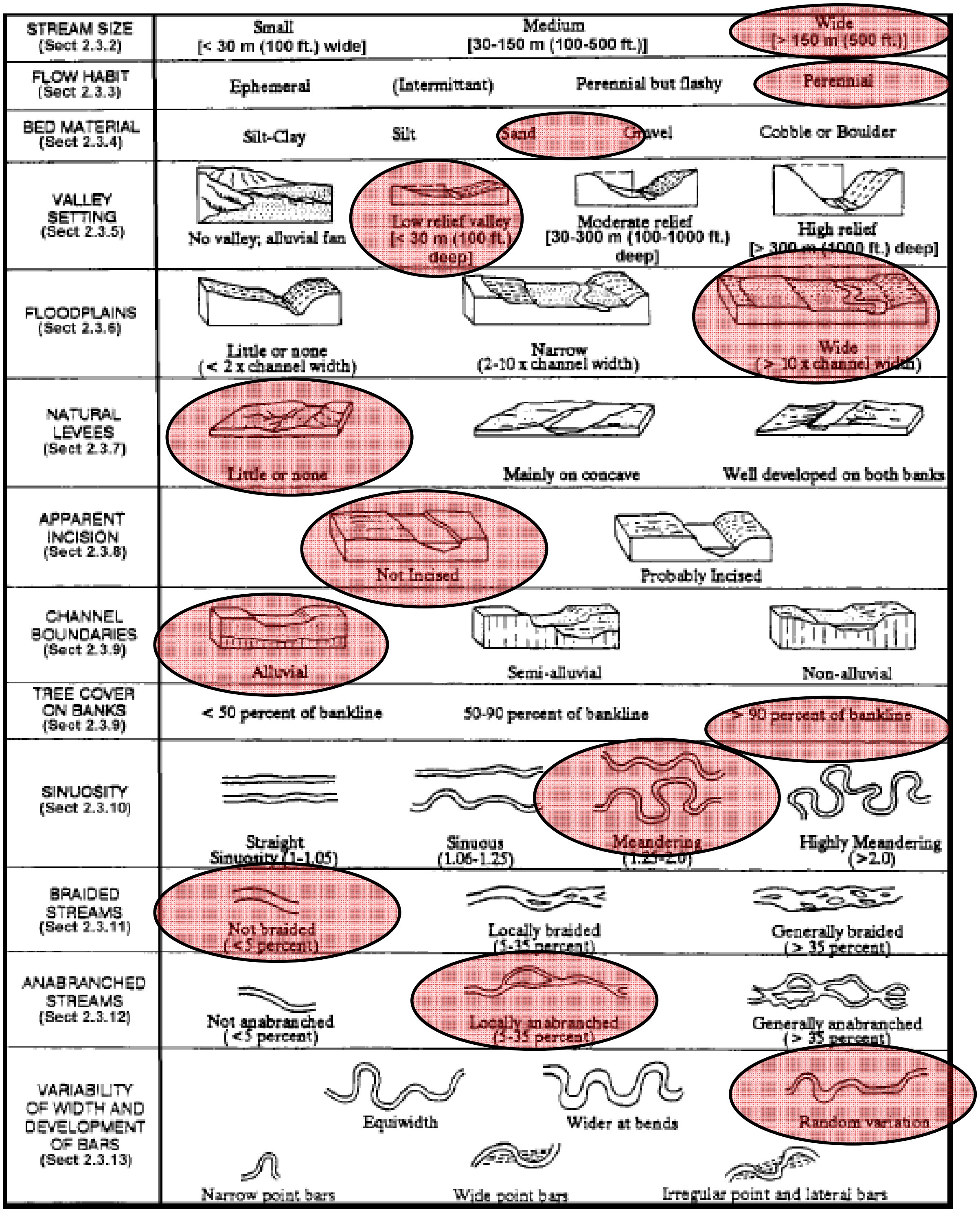


Plate 7: Assessment of Blackwater River geomorphic characteristics at the proposed bridge site

General scour as well as aggradation and degradation are natural geomorphic processes associated with the natural evolution and development of a river and its associated floodplain over extended time periods. Both scour mechanisms can occur without the presence of a bridge. That is, this scour type is not restricted to the vicinity of bridge crossings.

An assessment of general scour has been undertaken for Clear Creek and Blackwater River based on a review of historic aerial photographs dating back to 1966. The outcomes of this assessment are presented in **Figure 8 and 9** in **Appendix A**. As shown in **Figure 8 and 9**, no significant migration of either watercourse has occurred over the past 56 years. This indicates the channels are relatively stable and there is unlikely to be any significant lateral channel movement over the design life of the bridges, if current vegetation conditions are maintained.

Additionally, a review of geomorphic characteristics of both the Clear Creek and Blackwater River basin was completed (refer Section 2.3). The "bed material", "channel boundaries", "valley setting", "natural levee" and "apparent incision" indicate that there is potential for channel scour to occur. However, the "tree cover" and lack of any "anabranched" or "braided" streams tend to illustrate that there is only limited potential for lateral movement of the two channels.

In order to evaluate the potential for aggradation and degradation at the site of the proposed bridges, investigation into previous studies in the locality was undertaken to attempt to determine if aggradation/degradation is likely to occur. The Bridge Hydraulic Report for SR 87 over Clear Creek by Volkert INC (Volkert, 1996) studies a bridge replacement for the crossing of SR 87 in a position upstream of the current proposed location. This report states that through inspection reports and field reviews, there was no indication that long term changes in bed elevations have occurred or are expected to occur in the future.

FDOT has prepared design surge hydrographs based on surge estimates prepared by the Florida Department of Environmental Protection, the US Army Corps of Engineers Waterways Experiment Station and the National Oceanic and Atmospheric Administration. "In 2003, Dr. Sheppard was commissioned by FDOT to investigate the various design storm surge guidance and the methodologies supporting the guidance. His report and a spreadsheet documenting his recommendations for locations around the state have been adopted as policy for design hurricane boundary conditions for Florida DOT." (www.dot.state.fl.us/rddesign/dr/DHSH.shtm). This project is located at reference number 103. The storm surge peak elevations are 9.40/9.08 feet and 10.80/10.48 feet (NGVD 1929/NAVD 1988), respectively, for the 50 and 100 year floods.

As a result, a storm surge can be expected to impact on the proposed location of the SR 87 connector bridge over Blackwater River, and further consideration during design should reflect this. Additionally, a wind induced receding tide in Blackwater Bay may produce the deepest scour potential at the proposed bridge locations. This is associated with a lower tailwater level in Blackwater Bay potentially producing a steeper energy grade line along Blackwater River and consequently Clear Creek.

As a result of the investigations outlined above, it is considered that both the Clear Creek and Blackwater River channels are fairly stable in terms of General and

Aggradation/Degradation Scour, and hence these mechanisms are not considered critical to design of the bridges. Items 3 and 4 will be evaluated with more detailed borings, D50 analysis and the output of a 2D model. The low tailwater, high flow condition scenario should also be investigated as a steeper energy grade line will exist, and may lead to higher velocities, and consequently, higher scour potential.

5 Summary and Conclusion

This report has presented the outcomes of investigations that were completed to determine design flows at the two proposed bridge sites and determine a preliminary minimum low chord elevation and bridge span lengths for the SR 87 connector bridge crossings of Blackwater River and Clear Creek.

A detailed hydrologic analysis has been undertaken and presented, providing design flows for floods between the 5 and 500 year ARI event. These flows are considered the best estimate and as such were utilized in the hydraulic modeling to determine the required low chord elevation and width of the bridges over the Blackwater River and Clear Creek.

Based on the outcomes of the hydraulic investigations and for planning purposes, it is recommended that the proposed bridge spanning Clear Creek comprise a span length of 180 feet, and have a minimum low member elevation of 18.95 feet NAVD. This will ensure that the bridge is elevated sufficiently high to allow debris clearance in the design 50 year ARI flood, and ensure stages do not increase more than 1 foot upstream of the proposed bridge. Realignment of the creek will need to occur to ensure the span length can be minimized and to help ensure water is distributed through the bridge opening more efficiently.

It is recommended that the proposed bridge to span Blackwater River be 5,560 feet long. The bridge should have a minimum low member elevation of 21 feet NAVD over the river and floodplain, and a minimum low member elevation of 27.70 feet NAVD over the Blackwater Heritage State Trail. Similarly to the proposed Clear Creek Bridge, these low chord elevations and span lengths make allowance for 2 feet debris clearance, as well as ensuring upstream stage increases are less than 1 foot. In addition, the length of the bridge will also allow for the spanning of Pat Brown Road, and the Blackwater Heritage State Trail.

As this is a preliminary study, these parameters may vary after a more detailed hydraulic investigation is undertaken. Due to the meandering nature of the Blackwater River in the vicinity of the proposed Blackwater River Bridge site, a 2-dimensional model should be utilized in order to gain a greater understanding of flood behavior, and more specifically, provide accurate stage and velocity parameters in which will define the majority of design requirements. Greater investigation into appropriate tailwater and the variation in the tailwater during extreme events should be undertaken and considered in design and scour calculations.

The detailed investigation of the Clear Creek Bridge should utilize tailwater estimates produced from the Blackwater River model. Additionally, consideration of the hydraulic impacts of all structures downstream of the proposed bridge site to the confluence of Blackwater River should be included. It may be prudent to include the Clear Creek design within the Blackwater River 2-dimension model. An environmentally sensitive method of dealing with the parallel channel alignment with the proposed Clear Creek Bridge should also be identified and may require spanning of the entire channel, or a re-alignment through the bridge opening.

The design stage of both proposed bridges should utilize surveyed cross-section data and more detailed Mannings 'n' values derived from analysis of vegetation and bank conditions at each proposed bridge site.

6 References

1. Environmental and Geotechnical Specialists, INC (November 2011). *Phase I Geotechnical Investigation*: Bridge Investigation SR 87 Connector PD&E Study Santa Rosa County Florida

2. Environmental and Geotechnical Specialists, INC (December 2012). Report Of Geotechnical Bridge Investigation. SR 87 Over Clear Creek Connector Bridge PD&E Study, Santa Rosa County Florida

3. Federal Emergency Management Agency (December 19, 2006). *Flood Insurance Rate Maps, Santa Rosa County, Florida and incorporated Areas*. Community-Panel Number 340 of 657.

4. Florida Department of Transportation (2012). *Drainage Manual*.

5. Florida Department of Transportation (May, 2005). *Bridge Scour Manual*.

6. Federal Emergency Management Agency (1996). *Flood Insurance Study, Santa Rosa County, Florida And Incorporated Areas.*

7. Federal Highway Administration (1984). *Guide for Selecting Manning's Roughness Coefficient For Natural Channels and Flood Plains.*

8. Metric Engineering (2012). *Draft Bridge Hydraulics Report for Blackwater River*

9. Metric Engineering (2012). *Draft Bridge Hydraulics Report for Clear Creek*

10. Volkert Inc (August 2010). *Bridge Hydraulics Report FDOT SR 87 Over Clear Creek, Project Development and Environmental Phase Santa Rosa County, Florida*

APPENDIX E
Report of SPT Borings and Pile Capacity Curves

NOTES

1. NUMBERS LEFT OF BORINGS INDICATE STANDARD PENETRATION TEST (SPT) N-VALUES FOR 12 IN. PENETRATION (UNLESS OTHERWISE NOTED)
2. WATER ELEVATIONS SHOWN ARE THE WATER ELEVATIONS ENCOUNTERED. FLUCATIONS IN THE ELEVATION OF WATER SHOULD BE EXPECTED.
3. SOIL DESCRIPTIONS, TEST DATA, AND STANDARD PENETRATION VALUES SHOWN ARE FOR THE SOIL BORING ONLY AND MAY NOT APPLY TO ANY OTHER LOCATIONS EXCEPT AT THE LOCATION OF THE SOIL BORING. EXTRAPOLATION OF THE SOIL BORING DATA TO OTHER LOCATIONS IS THE SOLE RESPONSIBILITY OF THE PERSON PERFORMING THE EXTRAPOLATION.

GRANULAR MATERIALS RELATIVE DENSITY	SPT (BLOWS/12 IN.)	SILTS AND CLAYS CONSISTENCY	SPT (BLOWS/12 IN.)
VERY LOOSE	LESS THAN 3	VERY SOFT	LESS THAN 1
LOOSE	3 - 8	SOFT	1 - 3
MEDIUM DENSE	8 - 24	FIRM	3 - 6
DENSE	24 - 40	STIFF	6 - 12
VERY DENSE	GREATER THAN 40	VERY STIFF	12 - 24
		HARD	GREATER THAN 24

SPLIT-SPOON: INSIDE DIAMETER: 1.375 IN
OUTSIDE DIAMETER: 2.0 IN
AVG. HAMMER DROP: 30.0 IN
HAMMER WEIGHT: 140 LBS

ENVIRONMENTAL CLASSIFICATION

SUPERSTRUCTURE: SLIGHTLY AGGRESSIVE
SUBSTRUCTURE: MODERATELY AGGRESSIVE FOR STEEL (pH 6.2)
MODERATELY AGGRESSIVE FOR CONCRETE

LEGEND

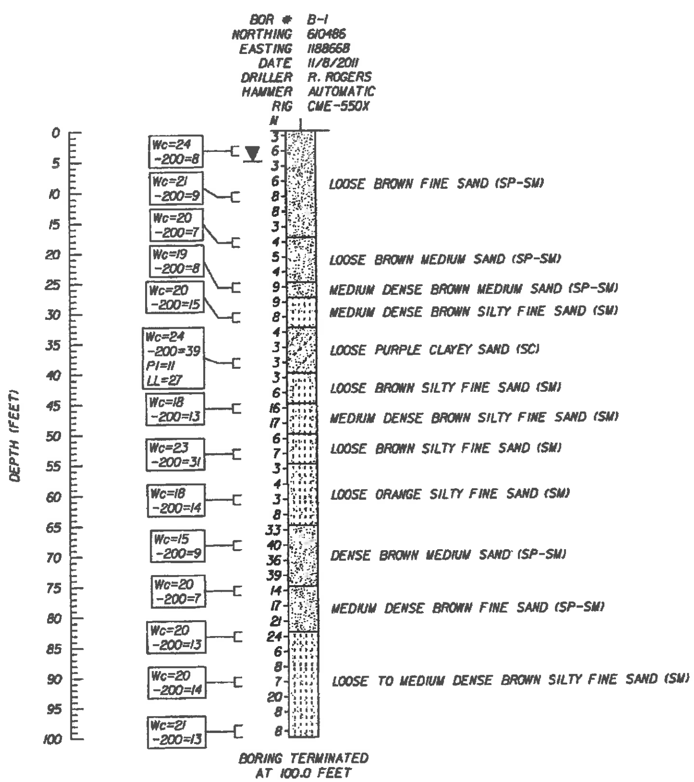
MEASURED GROUNDWATER

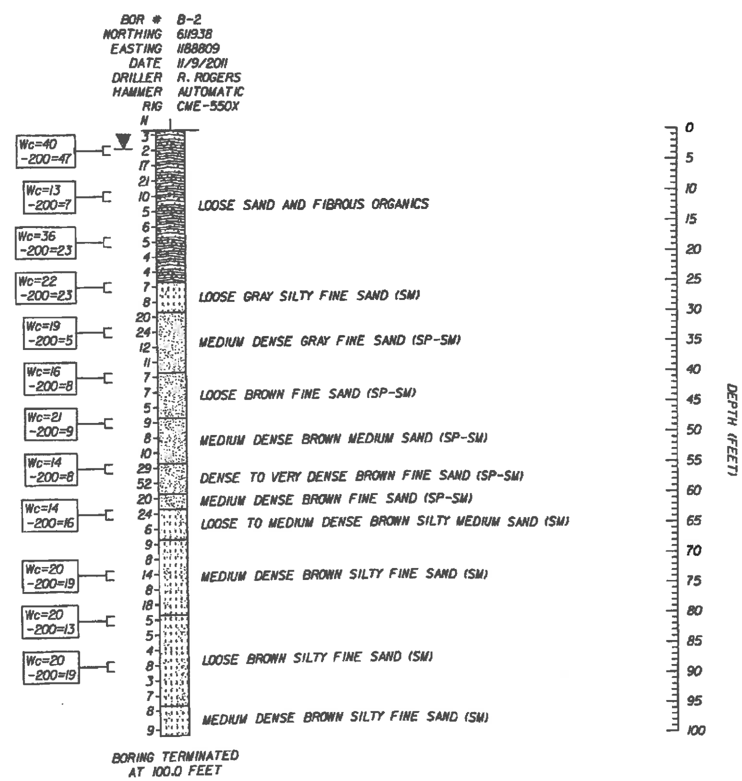
LABORATORY TESTING RESULTS

WATER CONTENT (%) — Wc=
-200 SIEVE (%) — -200=
LIQUID LIMIT — LL=
PLASTICITY INDEX — PI=

(SM) UNIFIED SOIL CLASSIFICATION GROUP SYMBOL

MEDIUM TO FINE SAND (SP-SM)
SILTY MEDIUM TO SILTY FINE SAND (SM)
SAND AND FIBROUS ORGANICS
CLAYEY SAND (SC)





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REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

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Certificate of Authorization: 6222

DRAWN BY: AMR 11-11
CHECKED BY: KJC 11-11
DESIGNED BY: DCS 11-11
CHECKED BY: MJH 11-11

STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 87	SANTA ROSA	416748-3-22-01

SHEET TITLE: REPORT OF CORE BORINGS

PROJECT NAME: SR 87 CONNECTOR PD&E STUDY

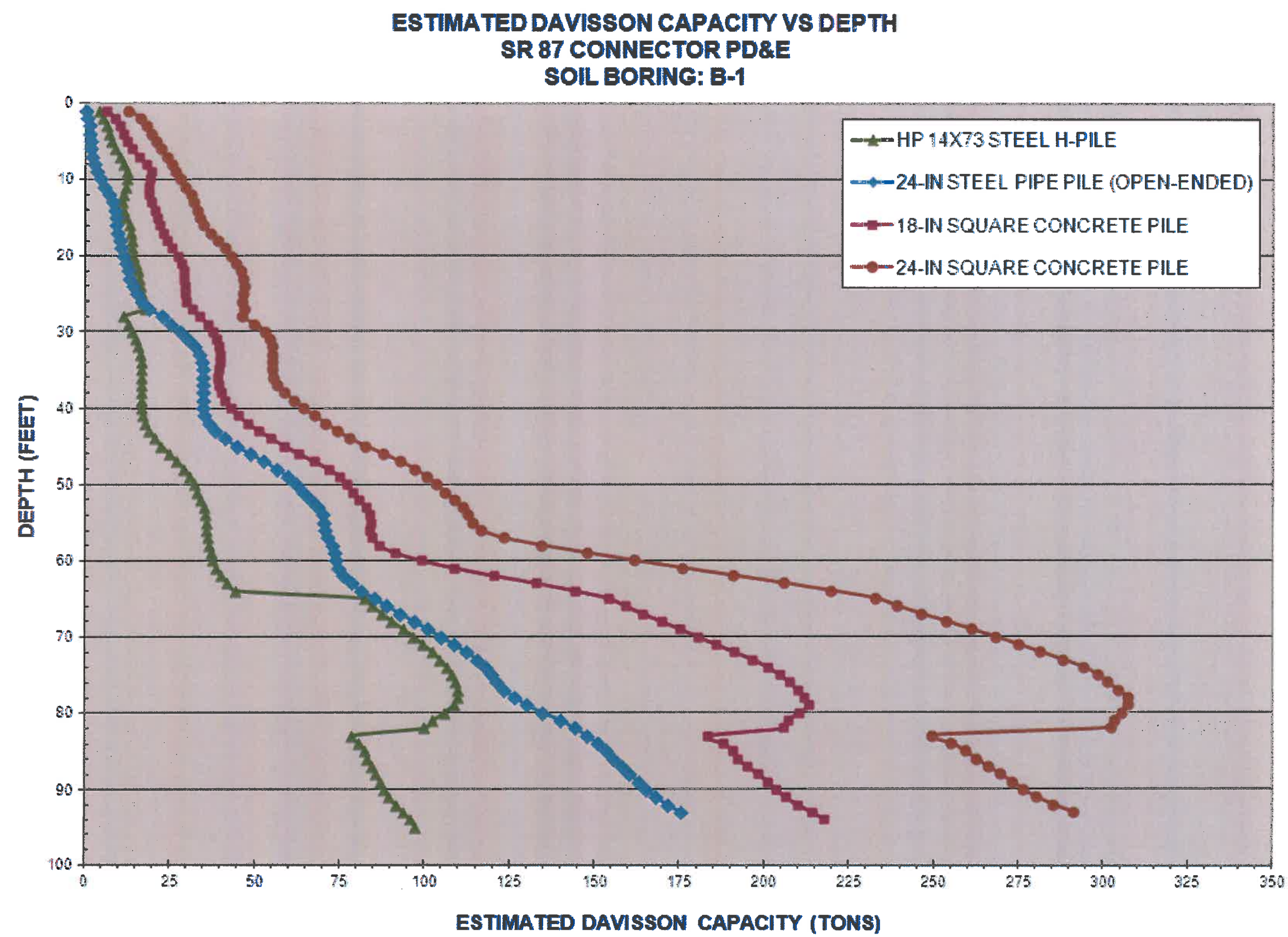
REF. DWG. NO.

SHEET NO.

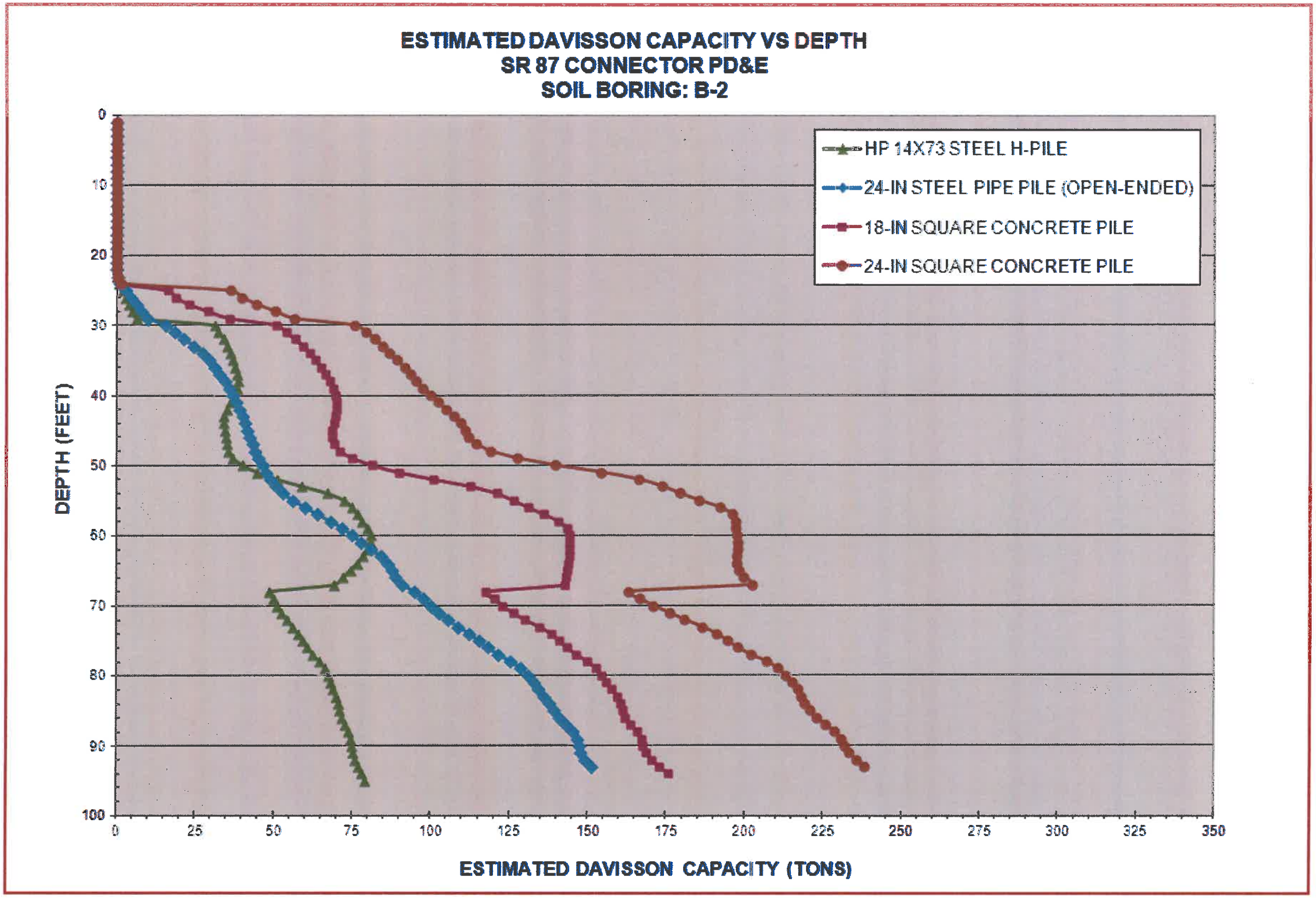
$USER$ $DATE$ $TIME$ $FILE$

E2

DRAWN: A. ROMANELLI, E.I.	CHECKED: M. HAYDEN, P.E.		TITLE: AERIAL PHOTOGRAPH OF PROJECT LOCATION BRIDGE INVESTIGATION SR 87 CONNECTOR PD&E STUDY SANTA ROSA COUNTY, FLORIDA	
ENGINEER: D. SHEPPARD, P.E.		EGS Environmental & Geotechnical Specialists, Inc. 3154 Eliza Road Tallahassee, Florida 32308 Office #: (850) 386-1253 Fax #: (850) 385-8050		
CLIENT: METRIC ENGINEERING, INC.				
PROJ. NO.: 28-09-09-04	SCALE:		DATE: NOVEMBER 2011	FIGURE NO.: 3



DRAWN BY: K. CRONIN, E.I.	CHECKED: M. HAYDEN, P.E.	EGS	TITLE: ESTIMATED DAVISSON CAPACITY VS. DEPTH SOIL BORING B-1 BRIDGE INVESTIGATION SR 87 CONNECTOR PD&E STUDY SANTA ROSA COUNTY, FLORIDA	
ENGINEER: D. SHEPPARD, P.E.		Environmental & Geotechnical Specialists, Inc. 3154 Eliza Road Tallahassee, Florida 32308 Office #: (850) 386-1253 Fax #: (850) 385-8050		
CLIENT: METRIC ENGINEERING, INC.				
PROJECT NO.: 28-09-09-04	SCALE:		DATE: NOVEMBER 2011	FIGURE NO.: 4



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CLIENT: METRIC ENGINEERING, INC.	
PROJECT NO.: 28-09-09-04	SCALE:

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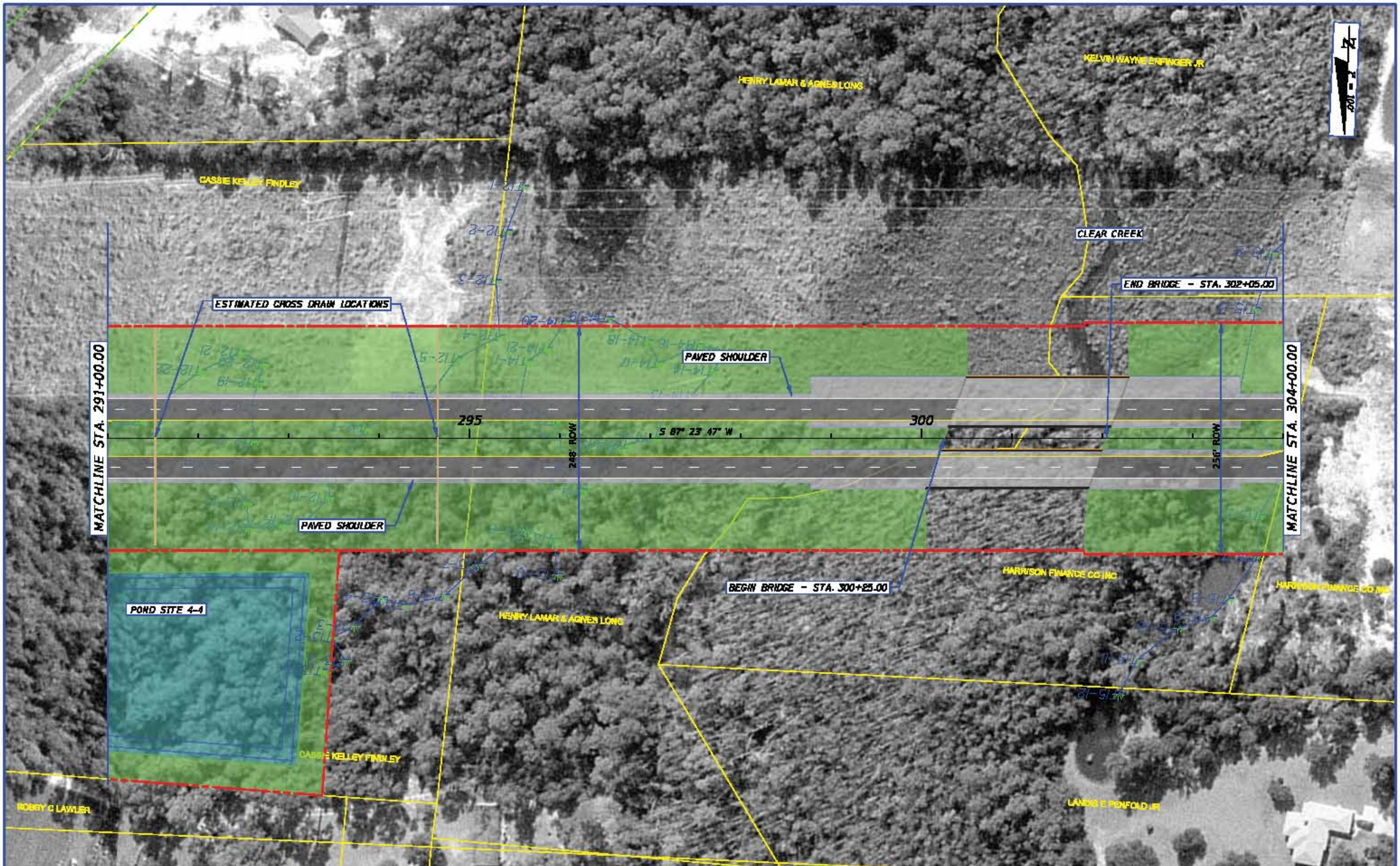
Office #: (850) 386-1253
Fax #: (850) 385-8050

TITLE: ESTIMATED DAVISSON CAPACITY VS. DEPTH
SOIL BORING B-2
BRIDGE INVESTIGATION
SR 87 CONNECTOR PD&E STUDY
SANTA ROSA COUNTY, FLORIDA

DATE: NOVEMBER 2011	FIGURE NO.: 5

APPENDIX F
Preliminary Roadway Plans in the Vicinity of the Bridge



REVISIONS			
DATE	DESCRIPTION	DATE	DESCRIPTION

METRIC ENGINEERING, INC.
2616 JENKS AVENUE
PANAMA CITY, FLORIDA 32405
TEL. (850) 872-8044
FAX. (850) 872-8704
FLORIDA CERT. NO. EB-0002294

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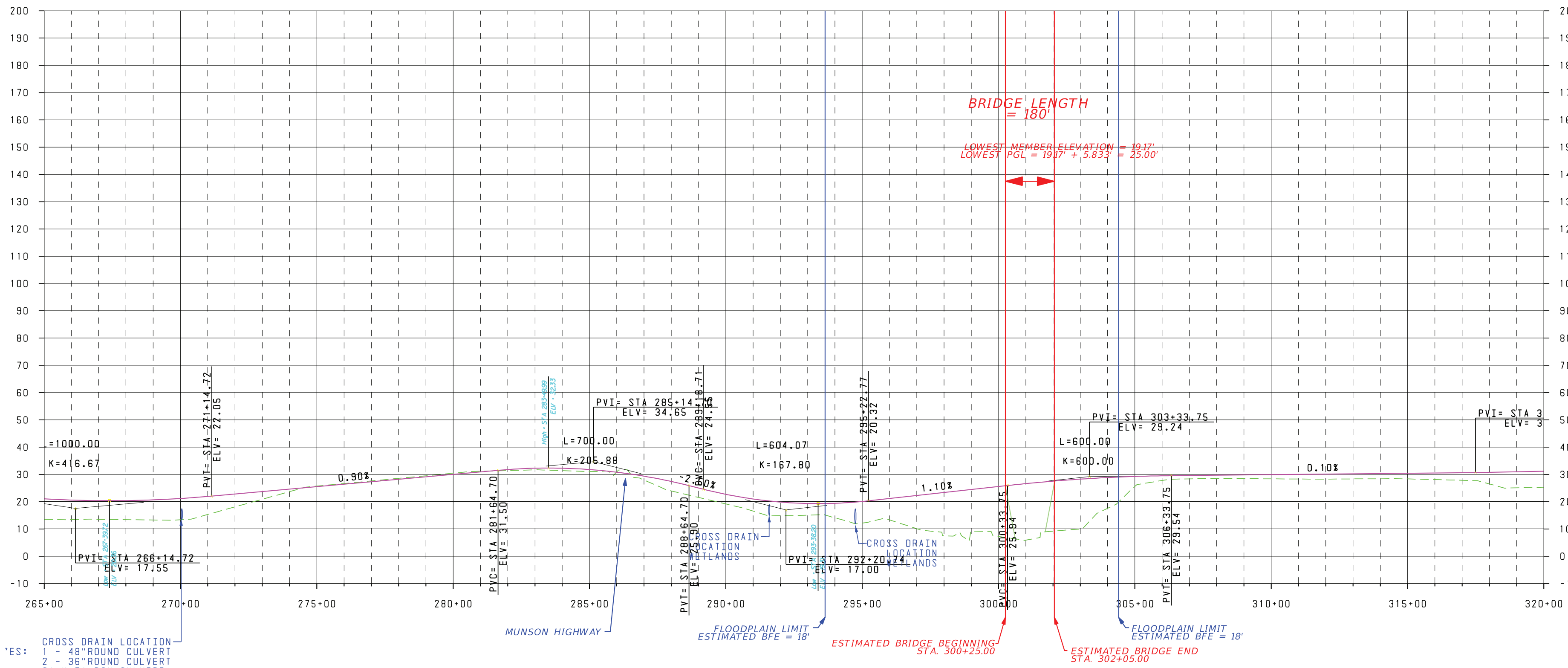
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
SR 87	SANTA ROSA	416748-3-22-01, ETC.

PLAN SHEET
ALIGNMENT 1

SHEET NO.
24

F2



CROSS DRAIN LOCATION
1 - 48" ROUND CULVERT
2 - 36" ROUND CULVERT

EXISTING PROFILE
WORKING MINIMUM PROFILE

V: 1" = 40'
H: 1" = 400'

REVISIONS			
DATE	DESCRIPTION	DATE	DESCRIPTION

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DEPARTMENT OF TRANSPORTATION

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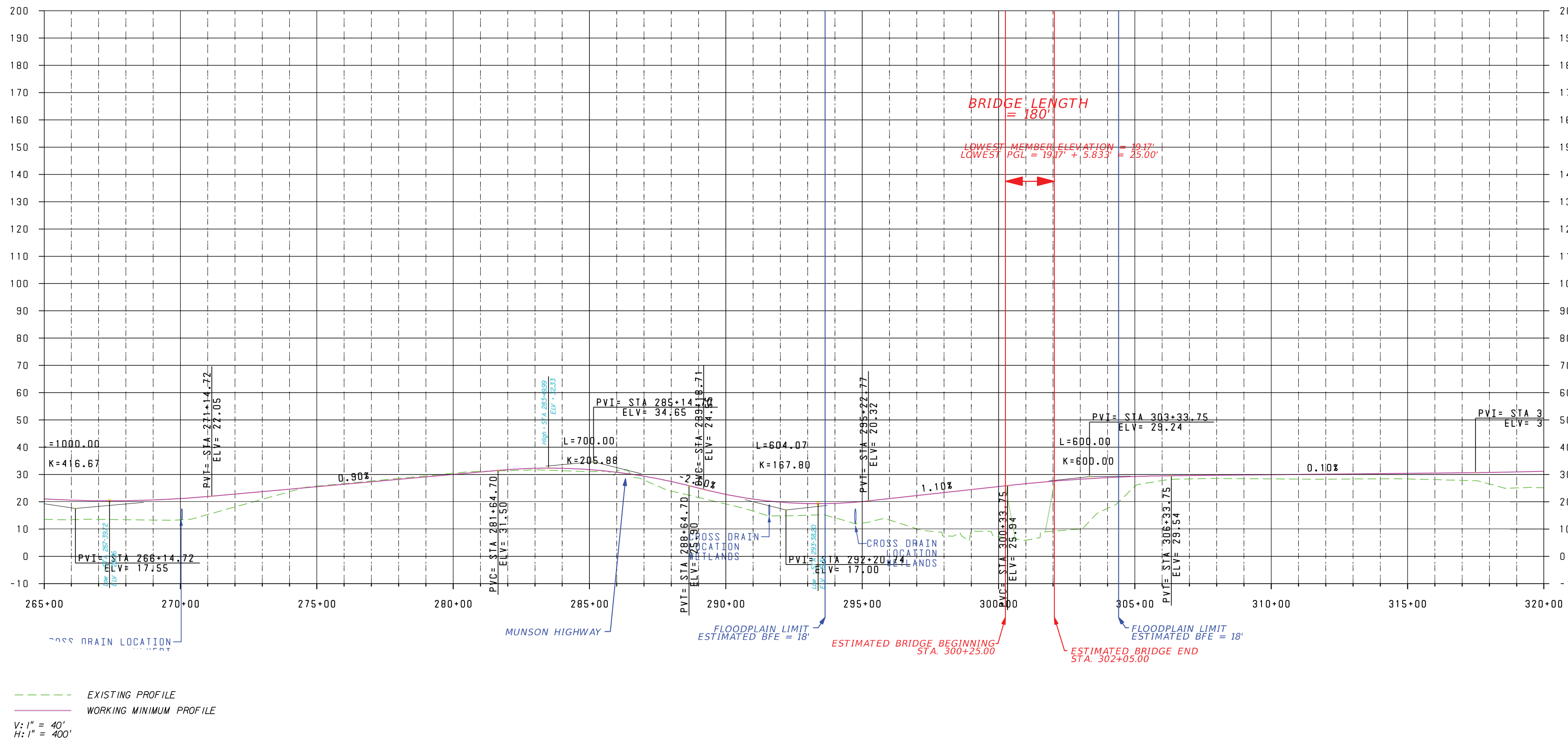
SR 87	SANTA ROSA	416748-3-22-01, ETC.

PROFILE SHEET
ALIGNMENT 1

SHEET NO. 44

NMauntler 2/8/2013 8:19:20 AM F:\E\projects\41674832202\roadway\PROFRD01.DGN



REVISIONS			
DATE	DESCRIPTION	DATE	DESCRIPTION

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SR 87	SANTA ROSA	416748-3-22-01, ETC.

PROFILE SHEET
ALIGNMENT 2

SHEET NO. 49

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F4