GEORGIA DOT RESEARCH PROJECT 14-10

FINAL REPORT

PREFAB BRIDGES FOR GEORGIA CITY AND COUNTY ROADS

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The objective of this study was to develop and deliver a toolkit to help local governments (LGs) in Georgia select and construct bridges using prefabricated modular systems with 40-, 60-, and 80-foot spans. The components of the proposed accelerated bridge construction (ABC) toolkit address: 1) decision-making; 2) design; 3) construction; 4) risk analysis; and 5) cost estimation. It will be an extensive, convenient source of the latest guidelines for ABC applications. It is not intended for developing final design and construction plans but as a source of information to help decision-makers and owners develop an initial design, estimate the material and construction costs, and determine when and where ABC will be most beneficial. It will provide guidelines to assist local governments and third-party designers using GDOT design standards for ABC. With repeated implementation, ABC options will become even more economical and efficient.</td></tr>
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Final Report

PREFAB BRIDGES FOR GEORGIA CITY AND COUNTY ROADS

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EXECUTIVE SUMMARY

Accelerated bridge construction (ABC) differs from conventional cast-in-place methods in that all the members are prefabricated then lifted into place and assembled onsite. As a result of increased interest and use of ABC in Georgia, the Georgia Department of Transportation (GDOT) funded a research project aimed at introducing ABC design and construction to Georgia cities and counties through the use of a toolkit. The research focused specifically on short span bridges for span lengths of 40, 60, and 80 ft. The toolkit is not intended to be used for developing a final design, but rather as an informational source that can help decision makers develop an initial design, estimate the material and construction costs, and determine when and where ABC is most beneficial. This study presents the process used in developing the toolkit and its primary features.

The first phase of the project involved the creation and completion of a survey which was distributed to several state DOTs. It contained questions regarding the organization's experience with ABC, the level of acceptance of ABC techniques in their state, the number of completed projects in recent years, impediments to the use of ABC techniques, and the ongoing research on ABC topics in the entity's state.

The toolkit itself contains construction, design, risk analysis, and cost estimate components. The construction guidelines will encompass most steps in the construction process from the foundation to the paving of the deck. It will also outline the construction process of the offsite prefabrication area, transportation of elements, and setting of the prefabricated bridge elements. The design component of the toolkit will provide design concepts, user friendly pre-design examples, and interactive design flowcharts with design aides such as Mathcad, which will allow readers, such as Georgia city and county engineers, to easily follow the extensive procedures involved in ABC bridge construction. Both steel and concrete girder design examples were developed and modified to allow for easy understanding using GDOT standard criteria for highway bridges, information obtained from a design example created by the Federal Highway Association (FHWA), and the latest AASHTO LRFD Bridge Design Specifications, 6th ed. (2012). The base design examples were taken from "Innovative Bridge Designs for Rapid Renewal" (SHRP2, 2013).

In terms of risk analysis, the risk assessment components of the toolkit focused on the evaluation of the bridge's ability to convey the design and base floods without causing significant damage to the roadway, stream, bridge itself, and other property. The guidelines with an interactive flowchart will be created to assist the potential designer in the collection of the hydrologic data needed to determine the peak discharges for different design year floods and perform a hydraulic analysis. The cost estimate component will provide examples of cost comparisons between corresponding Federal and State requirements to guide local governments in their cost estimation activity.

The proposed ABC toolkit will provide guidelines to assist local governments and third-party designers in employing GDOT design standards for accelerated-built bridges. In the future, sufficient ABC experience in GA will lead to contractor acceptance as well as to savings in schedules and costs, which could diminish the initial additional costs with a consistent and repeated application of ABC practices.

Key Words: Accelerated bridge construction (ABC); Concrete girder; Design; Steel girder, Toolkit.

CHAPTER 1. INTRODUCTION AND RESEARCH APPROACH

Accelerated bridge construction (ABC) encompasses the techniques used in the prefabrication of bridge sections to decrease the closure time required to construct or renovate a bridge. From the Federal Highway Administration (FHWA) website, the definition of ABC is a bridge construction that uses innovative planning, design, materials, and construction methods in a safe and cost effective manner to reduce the onsite construction time that occurs when building new bridges or replacing and rehabilitating existing bridges. In other words, ABC is “building the bridge first before setting up the traffic control cones, and then move it quickly into places, like in hours or a weekend.” (Mistry 2008). As a result of increased interest and use of ABC in Georgia, the Georgia Department of Transportation (GDOT) funded a research project aimed at introducing ABC design and construction to Georgia cities and counties through the use of a toolkit. This study focuses on short span bridges, between 40 and 80 feet, for the state of Georgia. Special attention was given to various areas of ABC, generally regarding design, constructability, and risk analysis, and specifically regarding the use of concrete and steel girders, concrete-decked steel girders, prestressed concrete, as well as cost efficiency, lessons learned reports from other states’ completed projects, and industry surveys. Several specific case studies were also conducted to evaluate the performance and design-to-finish process of a bridge which utilizes prefabricated modular systems.

ABC techniques have been performed in the past and are currently being investigated for more extensive use. Garver, an engineering consultancy out of Arkansas, details the process of a lateral slide, a bridge sliding technique used to replace a bridge superstructure. There is no one ABC technique in use in the United States. Instead, there is a family of ABC construction technologies that are in use that cover the majority of ABC projects. In construction, the foundation and wall element technologies are in the early stages of deployment, while others such as modular superstructure systems are mature and in use on a regular basis. Benefits to employing ABC techniques include:

ABC improves:

- Site constructability;
- Total project delivery time;
- Material quality and product durability; and
- Work-zone safety for the commuters and contractor personnel.

ABC reduces:

- Traffic impacts;
- Onsite construction time; and
- Weather-related time delays.

ABC can minimize:

- Environmental impacts;
- Impacts to existing roadway alignment; and
- Utility relocations and right of way take.

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Commonly, ABC has been employed to reduce the traffic impact since the safety and flow of public travel and the flow of transportation directly correspond to the onsite construction flow of the activities. There are other common and equally viable reasons to use ABC, which range from site constructability issues to time management issues.

Conventional bridge construction, commonly referred to as cast-in-place (CIP) methods, is construction that does not focus on the reduction of onsite construction time. Conventional construction methods involve onsite activities that are time consuming and weather dependent. Conventional construction includes onsite installation of bridge substructures and superstructures, placing reinforcing steel, and concrete placement, followed by concrete curing.

Effectiveness of ABC is determined by two factors: onsite construction time and mobility impact time. Onsite construction time is the period of time from when a contractor alters the project site location until all construction related activity is removed. Some examples involved include maintenance of traffic items, construction materials on site, equipment, and workforce. Mobility impact time is any period of time the traffic flow of the commuters is reduced due to onsite construction activities. The fewer amount of disruptions, the better and least expensive.

The use of prefabricated bridge elements and systems (PBES) is one of the most crucial strategies employed to meet the objectives of ABC. PBES are structural components of a bridge that are built off-site. These elements help reduce the onsite construction time and commuter impact time that occurs from conventional construction methods. Combining PBES with the "Fast Track Contracting" method can create a high-performance and fast paced construction project. Components of PBES include, but are not limited to:

- Precast footings;
- Precast wing walls;
- Precast pile foundations;
- Prefabricated caps and footings; and
- Prefabricated steel/concrete girder beams.

The first phase of the project involved the creation and completion of a survey which was distributed to several state DOTs. It contained questions regarding the organization's experience with ABC, the level of acceptance of ABC techniques in their state, the number of completed projects in recent years, impediments to the use of ABC techniques, and the ongoing research on ABC topics in the entity's state.

The primary objective of this study was to develop and deliver a toolkit for accelerated selection and construction of bridges in place using prefabricated modular systems with 40, 60, and 80 feet span lengths for local governments (LGs) in Georgia. The proposed toolkit itself contains construction, design, risk analysis, cost estimate components, and decision-making tools. The construction guidelines will encompass most steps in the construction process from the foundation to the paving of the deck. It will also outline the construction process of the offsite prefabrication area, transportation of elements, and setting of the prefabricated bridge elements. The design component of the toolkit will provide user friendly pre-design examples and interactive design flowcharts with design aides such as Mathcad, which will allow readers, such as Georgia city and county engineers, to easily follow the extensive procedures involved in ABC bridge construction. Both steel and concrete girder design examples were developed and modified to allow for easy understanding using GDOT standard criteria for highway bridges, information obtained from a design example created by the Federal Highway Association, and the latest AASHTO LRFD Bridge Design Specifications, 6th ed. (2012). The base design examples were taken from the SHRP 2 document, "Innovative Bridge Designs for Rapid Renewal" (SHRP2 2013). In terms of risk analysis, the risk assessment components of the toolkit focused on the evaluation of the bridge's ability to convey the design and base floods without causing significant damage to the roadway, stream, bridge itself, and other property. The guidelines with an interactive flowchart will be created to assist the potential designer in the collection of the hydrologic data needed to determine the peak discharges for different design year floods and perform a hydraulic analysis. The cost estimate component will provide examples of cost comparisons between corresponding Federal and State requirements and survey other state DOTs regarding their conceptual cost estimates.

The toolkit is not intended to be used for developing final design and construction, but rather as an informational source that can help decision makers develop an initial design, estimate the material and construction costs, and determine when and where ABC is most beneficial. The proposed ABC toolkit will provide guidelines to assist LGs and third-party designers in employing GDOT design standards for accelerated-built bridges. With the sufficient and repeated application, ABC option can become more economical and efficient.

CHAPTER 2. SURVEYS

2.1 OVERVIEW OF DOT SURVEYS

The survey was submitted to various agencies in order to inquire as to their experience with ABC. The survey was used to evaluate their successes and to find out what worked, as well as to evaluate their failures to find out what did not. This survey consisted of questions which gauged the experience of bridge owners. Their responses are noted in our research, and state DOTs from all 50 states were contacted regarding their own ABC experiences via a more generalized 7-question survey (taken from SHRP2 2013) by the Georgia Southern ABC Research team. They were asked questions specifically regarding:

- The amount of experience they have had with ABC in recent years and how many projects they have completed.
- The general level of acceptance of ABC in their state.
- Which agency generally engineers the projects to have components of ABC.
- Which impediments, if any, are keeping these agencies from opting to use ABC techniques as opposed to traditional methods.
- The availability of standardized elements and the benefit thereof.
- The condition of ongoing or completed projects.
- Current research regarding ABC.

Results from our surveys were obtained from 45 of the 50 states and are summarized using tables and U.S. maps in Appendix B. With the exception of Arkansas, Nebraska, and North Dakota, all indicated states have completed ABC projects in recent years and ABC has become standard in Utah and Colorado. The following section briefly presents the level and status of ABC application for each state:

2.1.1 Results

Alabama: The Alabama DOT (ALDOT) has completed one project in the past 5 years. The level of acceptance of ABC within the state is low and contractors are doubtful about its use for typical bridges. However, they think ABC could be successful for long structures that require substantial repetition of elements. Alabama is currently involved in the research and testing of four systems of rapid deck replacement on structures in the northern region of the state.

Alaska: The Alaska DOT & Public Facilities (ADOT&PF) has completed several projects in recent years, and the support for ABC is moderate within the state. Projects completed recently use decked bulb-tee girders so that construction is faster and a deck does not need to be cast in place. They have also used precast pier cap beams, full depth deck planks, and other prefabricated bridge elements. Projects are engineered to employ ABC, but contractors on occasion have opted to use ABC. Standardization would help encourage ABC, but training and education would probably help more. Most recently, ADOT&PF has completed research on an all-steel bridge pier system that can be quickly constructed in remote locations. It is reported to have good seismic performance but may not be acceptable in the highest seismic regions.

Arizona: The Arizona DOT (ADOT) has completed one project using PBES connections. ABC is valued and there are plans to use it on future projects. The decision to use is sometimes left up to the contractor, but projects are often designed to utilize ABC. Standardization would help the decision making process of whether or not to use ABC. Currently, ADOT is in the planning stages of a bridge replacement project that will use a geosynthetic reinforced soil integrated bridge system (GRS-IBS) abutment and a bridge slide. Research on Ultra-High Performance Concrete (UHPC) connections is in progress.

Arkansas: The Arkansas State Highway and Transportation Department (AHTD) has no active ABC program. Their perception is that ABC projects will be more expensive and thus counter to their desire for cost savings.

California: The California DOT (Caltrans) described several recent projects in which various ABC methods were employed and indicated that meaningful incentives / disincentives greatly motivated contractors. For example, seismic concerns limit their use of precast pier elements, and they are concerned about long-term durability and the ability to balance the higher costs of ABC projects against user cost-savings.

Colorado: The Colorado Department of Transportation (CDOT) has completed a wide array of projects using ABC, including 73 bridge projects and culverts as of March 2015. The level of acceptance is clearly high. Most bridge projects are designed to employ ABC, but for some design-build projects, the decision is left up to the contractor. Standardization is believed essential; ABC is standard in Colorado and considered on every project, although complex bridge projects will always require a certain level of customization. CDOT is not currently conducting research on ABC.

Connecticut: None reported

Delaware: The Delaware DOT (DelDOT) has completed fewer than 10 projects in the past 5 years. They were all engineered to employ ABC techniques, so acceptance of innovation is generally good. Projects have used precast elements, but no research on ABC is ongoing.

Florida: The Florida Department of Transportation (FDOT) has conducted many ABC projects in recent years, and though it is not a standard practice, it is considered in every project. Florida has access to standardized elements, but contractors tend to avoid subcontracting work to broadcasters because they make their profit from placing steel and concrete. FDOT does not mandate the use of ABC but leaves the decision up to the contractor. Each bridge project has performance specifications that contractors must meet, and because contractors are given more responsibility with ABC, uncertainties about their methods persist.

Georgia: GDOT has completed one ABC project in recent years. The decision to use ABC techniques is left up to the contractor, and GDOT does consider the standardization of prefabricated elements a way to lower costs associated with ABC. GDOT is currently preparing their own prefabricated bridge toolkit to expedite the application of ABC and other prefabricated bridge technologies in GA city and county roads.

Hawaii: The Hawaii DOT (HDOT) has been using ABC concepts since precast-prestressed concrete elements were introduced in around 1959. However, based on current definitions, it started in 2001 with the use of adjacent slab beams made of precast-prestressed concrete. Hawaii has completed over 20 projects since 2001. The level of acceptance is very high, and some ABC projects proved less expensive than CIP projects. Government incentives would encourage further use of ABC in the state, and though standardized elements are available, ABC is only used when it proves economically beneficial. In the field, it saves contractors money in forming, shoring, and stressing tendons or prestressing strands. Time is saved when these elements are prefabricated while other fieldwork is being performed.

Idaho: None reported.

Illinois: The Illinois Department of Transportation (IDOT) has completed several projects using ABC methods in recent years. Bridge projects undergo a "Bridge Planning State", during which ABC is evaluated based on site needs and cost-versus-benefit analyses. Standardized elements would help to curtail ABC costs.

Indiana: The Indiana DOT (INDOT) has completed two ABC projects in recent years; they used the bridge-slide technique. Projects are designed to use ABC techniques, and standards would make ABC more efficient.

Iowa: The Iowa DOT (IOWADOT) has extensive experience with ABC and has completed approximately eleven ABC projects in recent years. Research focusing on substructure is under way with funding from the Iowa Highway Research Board. Acceptance in the state is good, and projects are designed to use ABC.

Kansas: The Kansas DOT (KDOT) used prefabricated materials, including precast concrete girders and deck panels, even before FHWA's "Every Day Counts" initiative. Its first official ABC project was designed in August 2014 and let in November 2014. It is modeled after Iowa's Keg Creek bridge project and used a pre-installed foundation; precast columns, abutments, and pier caps; a conventional weathering steel rolled beam superstructure; and precast, full-depth segmental deck sections post-tensioned together. Bridges are designed to employ ABC concepts. Contractors can also use ABC with KDOT permission, and this would normally happen after the project is let. Kansas law prohibits delivery methods other than design-bid-build. The 2014 project attracted only one bid because of its high price. It will be relet in June 2015.

Kentucky: None reported.

Louisiana: The Louisiana Department of Transportation and Development (LDOTD) has extensive experience with ABC methods, specifically using precast elements, such as span and cap segments, and float-out, float-in construction to erect long-span bridges over its many waterways. It has also used precast flat slab bridges for federal highway system projects. LDOTD reports that though these bridges do not provide the service life of their CIP counterparts, they are easier to construct in remote areas. While the state's soil conditions preclude precasting longer girder spans, standardization would be possible for shorter spans. Contractors often request that crane mats be used on the top of the structure, so a standard element that accounts for crane loads would be ideal. The department plans to continue using ABC techniques in the future.

Maine: The Maine DOT (MaineDOT) has completed several ABC projects in recent years. Acceptance is high, and standardization of elements might help lower precasting costs by encouraging fabricators to invest in standard forms for bridge elements.

Maryland: The Maryland DOT (MDOT) has completed several projects using ABC techniques in recent years and has experienced no significant problems. The major bar to further use is the lack of technique testings.

Massachusetts: The Massachusetts DOT (MassDOT) has not completed any ABC projects but is interested in implementing pilot projects to become familiar with techniques. Standardized elements would be useful in reducing the need to develop custom details, and unfamiliarity could be offset by learning about standardized elements that have been successful in other locations.

Michigan: The Michigan DOT (MDOT) has some experience with ABC methods; it completed projects designed to use ABC concepts and projects accelerated by the contractor. Standardization could make ABC methods more accessible to designers and would help contractors gain meaningful experience, helping to lower costs and improving quality in the long run.

Minnesota: The Minnesota Department of Transportation (MnDOT) has completed approximately 20 projects using ABC techniques. Acceptance is good. In MnDOT design-bid-build projects, the contractor proposes ABC, and use is generally approved. Value Engineering (VE) proposals are also considered during construction. Standardization of elements would help but not substantially. While UHPC addresses precast connection issue, it is expensive and requires a high level of contractor and supplier expertise. MnDOT is participating in a National Cooperative Highway Research Program (NCHRP) project to define tolerances for precast elements and design criteria for lateral slides and self-propelled modular transport (SPMT) moves.

Mississippi: ABC is only applied selectively at this point, reserved for emergency reconstruction or projects with special conditions, such as emergency access or site constraints. MDOT senior management must be convinced of the advantages of acceleration and would appreciate having a catalog of ideas to choose from rather than prescriptive standards when trying to decide whether or how to pursue an ABC project.

Missouri: The Missouri DOT (MoDOT) cited various examples of recent ABC deployment. Although these projects alleviate traffic constraints, they are much more expensive than conventional approaches and must be used judiciously.

Montana: The Montana DOT (MDT) has completed only one ABC project: the Highway 89 Pondera County Marias River Crossing, which used a GRS-IBS abutment system. The design included a wall radial edge as opposed to the more common straight edge design and was an on-site adjustment. The block required for the abutments was made to order, and the manufacturer was not equipped to produce rounded-edge blocks. MDT is still conducting research on GRS-IBS systems.

Nebraska: The Nebraska Department of Roads (NDOR) has not used ABC methods on any completed projects but completed its first GRS-IBS bridge with folded plate girders for a local agency in Boone County, finishing the project within 30 days in 2014. The GRS abutment was built in 15 days, including excavation, to support the 58 ft single-span, modular decked, beam superstructure. Recently, bridge elements have been accelerated. NDOR does not perceive a need for ABC, so it is not widely accepted, and its applications have been limited. However, contractors have used discretionary methods to accelerate construction, such as more man-hours. Standardizing elements is seen as a way to both lower costs and increase the quality and durability of finished bridge projects. Nebraska is currently studying the use of precast deck panels and heavy lifting of remotely assembled superstructure modules.

Nevada: The Nevada DOT (NDOT) has experience in using SPMTs, the bridge-slide technique, and precast arches. ABC is widely accepted when it is used for the right application, but the decision is left up to the contractor. Several NDOT projects have used GRS-IBS abutments and fully prefabricated superstructures.

New Hampshire: Although successful ABC projects are noted, the New Hampshire DOT (NHDOT) staff said not enough people at the agency were interested in ABC as a project delivery tool. They had no questions about its effectiveness, just insufficient motivation to evaluate. The University of New Hampshire continues to conduct research in the area. ABC is generally accepted, but when given the option, contractors seem reluctant to use it. When considered feasible and appropriate, projects are engineered to use ABC techniques, rather than leaving the decision to the contractor.

New Jersey: The New Jersey DOT (NJDOT) provided an extensive interview focused mainly on impediments. ABC has not taken hold because NJDOT engineers, particularly project managers, do not think it is a solution in many situations, based on their past experience and that of other NJDOT units. The agency is generally risk- averse, and ABC raises the level of risk associated with a project. If risk is not shown to be manageable, the concept will not gain traction, and it has not. NJDOT recognizes the need to study and update the user-cost model and its application, but it has no mechanism to screen or to choose projects for ABC and no systematic approach.

New Mexico: The New Mexico DOT (NMDOT) has completed approximately 10 projects in recent years. ABC is moderately accepted, and standardizing elements would help. Two projects used a full-depth precast deck panel system, and one used precast pier caps, abutment caps, and wingwalls.

New York: The New York State DOT (NYSDOT) completed at least 10 projects using ABC methods, and although ABC is the exception rather than the rule, more and more ABC techniques are gaining acceptance, especially in the region around New York City. Projects are generally designed to use ABC, but contractors also submit substitution proposals opting to use ABC methods. Standardization would be less effective because the most beneficial applications tend to be less standard, such as projects in urban areas. Several pilot projects used UHPC for joints between precast components, deck bulb-tee beams for one bridge, and full-depth precast deck panels for another. NYSDOT is investigating fatigue in precast element joints. On June 17, 2015, it successfully placed a 1,100-ton assembly of three curved steel girders between two concrete piers near the Tappan Zee Bridge’s Rockland County side, which took about four hours to complete.

North Carolina: The North Carolina DOT (NCDOT) has recently completed several ABC projects, including 24-hours-a-day construction to replace seven bridges on Ocracoke Island in 90 days. The Washington Bypass project employs an innovative construction gantry that allows complete construction of a new viaduct from the top without any intrusion into environmentally sensitive areas. NCDOT has selection criteria for ABC projects and has discussed a role for the Alternative Project Delivery Unit, which typically enables innovation in several ways: as a proposal from the contractor in design-build contracts, an as-designed solution for special projects, and a VE proposal. NCDOT is currently exploring the use of mechanically stabilized earth (MSE) abutments and GRS-IBS to expedite foundation construction.

North Dakota: The North Dakota DOT (NDDOT) has completed no ABC projects, and the level of acceptance is low. Nonetheless, the decision to use ABC is left up to the contractor.

Ohio: The Ohio DOT (ODOT) has created "permitted lane-closure maps" that define which highway lanes can be closed for construction. ABC is used to reduce closure time in more urban corridors; projects are designed to fit set time frames in accordance with the maps, and contractors are forced to use ABC practices, though they are free to decide which practices they use. Standardization is not necessary because contractors specialize in certain construction practices. ODOT has experience with precast elements as well as SPMT roll-ins. In a first for the state, it used slide-in bridge construction to replace the I-75 bridges over U.S. 6 in Bowling Green in 2015. Traffic was disrupted for just a weekend instead of the months replacing a bridge typically takes.

Oklahoma: None reported.

Oregon: The Oregon DOT (ODOT) has completed 8 projects in the past 5-10 years using ABC techniques. Support for ABC is high, and practice is shifting from contractor-employed ABC techniques to DOT-designed ABC projects.

Pennsylvania: The Pennsylvania DOT (PennDOT) has used precast elements and SPMTs several times over the past 5 years. ABC is considered in every project, but the decision is left up to the contractor unless ABC promises a clear advantage, in which case PennDOT will engineer the project to use it. Past efforts at standardization have not translated into profits for contractors. Once ABC methods become more mainstream, costs and risks are predicted to decrease to the point where their use will be economical. PennDOT is not currently implementing ABC methods on any project but is involved in research on structural details that could be applied to ABC in the future.

Rhode Island: The Rhode Island DOT (RIDOT) replaced I-95 over Route 2 in halves using SPMT and prefabricated approach slabs in August 2014. The work took half the time of conventional construction and, prior to installation, had no impact on interstate traffic.

South Carolina: None reported.

South Dakota: The South Dakota DOT (SDDOT) most recently completed an ABC project in 2001 using SPMT to move a steel-truss superstructure to its abutments. The bridge spanned a railroad yard, so closures and outages had to be kept to a minimum. ABC is viewed favorably if project conditions warrant it. Interest in using ABC methods to construct jointless decks of adequate length for little or no increased cost is high.

Tennessee: The Tennessee DOT (TDOT) has completed one project in recent years that incorporates ABC methods. ABC is always considered for bridge projects but not often used. Standardized elements are considered to be useful along with proven installation and serviceability records.

Texas: The ratio of incentives to disincentives impedes further use. For example, low-bid contractors might not be able to perform ABC, but a suggested solution was to select the contractor who offers the best value, not the lowest bid. Project size is also an important consideration. Since most candidate bridges are either small or medium-sized, contractors will not have time to become efficient in the new methods on an individual project. In addition, precast components used for bridge substructures are only practical when several are needed or the available access makes CIP difficult. Contractors would like to choose whether or not to use ABC, so the Florida approach, laying out the requirements and specifications that have to be met, might be effective.

Utah: In 2010, The Utah DOT (UDOT) standardized ABC. Senior management unanimously supports it, and project selection criteria frequently lead to its use, rather than traditional methods. Presently, UDOT is delivering its ABC program through a combination of design-build contracts and a method known as Construction Manager/General Contractor (CM/GC), both of which have proven successful. At the same time, it is developing ABC standards for such modules as deck panels, precast substructures, and new prestressed beam sections. These standards will increase the flexibility to let contracts using various mechanisms and to communicate ABC intentions to the design and construction community. Once ABC standards become available for engineers to use in creating as-designed plans, UDOT will explore their use in more conventional design-bid-build contracts. Precast elements offer another opportunity for cost savings in substructure construction. During the early phases of implementation, contractors showed reluctance. UDOT held a series of workshops and scan tours to learn from other agency practices, and some contractors made changes to their business practices to compete in the ABC arena. Successful contractors have demonstrated a willingness to get into the precasting business. Projects let to date have demonstrated a 5:1 – 6:1 ratio of user-costs saved to construction-costs incurred, and with repetition, costs have decreased. Recent bridge project lettings indicate that full-depth precast decks are competitive with, and occasionally less expensive than, traditional CIP concrete decks and include time and quality savings.

Vermont: The Vermont DOT (VDOT) acceptance of ABC is generally good, and at least 5 projects using ABC methods have been completed in recent years. Projects are typically engineered to use ABC, but Vermont is considering the Florida approach, which allows the contractor to decide how to meet VDOT design specifications. VDOT is also investigating incentive/disincentive clauses to encourage contractors to use ABC.

Virginia: None reported.

Washington: The Washington DOT (WashDOT) has completed various ABC projects using traditional design-bid-build procurement and redesigning structures to accommodate ABC approaches. Projects included complete bridge prefabrication and large-scale prefabrication of superstructure and substructure elements. In general, the use of prefabrication and ABC techniques did not seem to affect project quality but had a beneficial impact on safety. WashDOT does not specifically require that user impacts be considered components of project cost but has used incentive/disincentive clauses to motivate project completion.

West Virginia: The West Virginia DOT (WVDOT) has completed at least 5 projects in the past several years that were designed to use ABC. Incentive/disincentive clauses were designed to motivate contractors to develop ABC approaches. WVDOT would benefit from ABC specifications and is interested in methods that minimize environmental disruption.

Wisconsin: The Wisconsin DOT (WisDOT) is just beginning to implement ABC practices. Its first project re-decked a major structure with a full-depth precast deck panel system. The level of support for ABC is not very high since the practice is new there and not well established. WisDOT is funding research on precast substructure units and looking for opportunities for a demonstration project (Wisconsin Highway Research Program - 0092-15-02 - Evaluation of Performance of Innovative Bridges in Wisconsin - Iowa State University, PI).

Wyoming: The Wyoming DOT (WYDOT) has completed several projects involving precast elements and decked bulb-tees for country road bridges. ABC is generally well accepted and used where appropriate. Seeing the design standards used by other states would lead to more use in Wyoming.

2.1.2 Impediments

The impediments to ABC are widely noted. Many states cited increased cost as a major factor discouraging its use.

Alabama: Alabama cited increased manpower and other costs.

Alaska: ADOT&PF reported that the high initial cost of using a new technology is a major impediment. Contractor inexperience and the overall conservatism of the state are also hindrances.

Arizona: Currently, ADOT is facing questions about connection durability. Funding for ABC is limited, and contractors are inexperienced.

Arkansas: The primary concern is high initial costs. Incentives to use ABC are limited, and no active program is using ABC.

California: Caltrans is concerned about how precast pier elements will stand up to earthquakes; long-term durability; and elevated initial cost. The cost of ABC is widely known to exceed that of CIP, and although its time savings is also greater, Caltrans considers time secondary to financial savings.

Colorado: The high cost of ABC is the primary impediment, although it has become standard practice.

Connecticut: None reported.

Delaware: DelDOT noted higher initial costs and longer hours for construction workers, which posed a problem for the contractors who have to pay them.

Florida: Though ABC is highly accepted in Florida, many impediments were noted; for example, lack of staging space for SPMTs in urban areas and inexperienced contractors. During the design phase, site traffic constraints must be accounted for since traffic maintenance and phased construction have posed problems. FDOT tries to balance out the higher ABC costs with user costs.

Georgia: GDOT is experiencing some of the same problems as other states, especially higher cost, but interest in using ABC, especially prefabricated bridge elements and PBES, is growing.

Hawaii: The only impediment is encouraging governing agencies to use ABC, although not for all bridge construction projects. These agencies fail to consider such factors as the use of temporary detours when bridges are being replaced or repaired.

Idaho: None reported.

Illinois: The main hindrances are higher cost expectations and that user costs are difficult to quantify.

Indiana: The lack of overall knowledge and proper pricing methods are impediments.

Iowa: Despite high acceptance generally, some contractors are reluctant to adopt ABC because they believe it is less profitable where traffic volumes are low. They believe ABC is too complex and are discouraged by low incentives. Higher level management supports the use of ABC wherever warranted, yet in some cases, production-level engineers find ABC design slow and frustrating. Standard plans and shapes might ease the design process and allow reuse to save money.

Kansas: The biggest obstacle is the cost difference between ABC and CIP methods. CIP bridges have fewer joints and are therefore cheaper and easier to maintain over time.

Kentucky: None reported.

Louisiana: None reported.

Maine: Cost is the biggest impediment; precast elements generally cost more than casting in place.

Maryland: None reported.

Massachusetts: Since MassDOT has not completed any ABC projects, general lack of familiarity with ABC is a major impediment. Contractors have a conservative CIP culture, but increased exposure through pilot projects should overcome it. More experience with ABC may also diminish concerns about financial risk.

Michigan: The main hindrances are cost, constructability, and quality/performance. Life-cycle cost analyses with accurate accounting of public benefit would be useful for addressing higher costs, and constructability and quality concerns can be addressed by more experience in completing ABC projects.

Minnesota: Contractors are concerned about higher cost and that the reduced timeframes will mean tired, overworked staff. It is difficult to decide to use ABC methods late in the design phase.

Mississippi: MDOT is reluctant to use precast columns or footings based on concerns about connection durability and would welcome development of durable connections for these precast elements. They also do not use integral abutments due to concern about approach-slab connection details.

Missouri: MoDOT is concerned about durability and seismic activity and is working with local university partners for assistance in advancing ABC.

Montana: Low traffic volume is the main impediment. Interest in GRS-IBS systems is growing, but the overall perception is that ABC is not needed now.

Nebraska: ABC is primarily hindered by higher costs. Contractors are hesitant to use precast elements because of the amount of work that would have to be subcontracted. Urban areas are associated with higher user-delay costs, but the user costs on lower traffic roads and rural routes do not warrant ABC.

Nevada: The main concern is connection durability in case of seismic activity and questions about the efficiency of ABC methods and elements.

New Hampshire: NHDOT indicated that opportunities where acceleration appears justified are few. It also reported that the Epping project, one of its successful ABC projects, was 2.2 times more expensive than a conventional bridge replacement, and until the cost premium can be cut by at least 25% or less, promoting ABC will be difficult. Contractors hesitate to use the new technology and want to keep their own employees working rather than subcontracting work to precasters.

New Jersey: When NJDOT tried to accelerate earlier projects, their own construction engineering department was reluctant to support the schedule. Schedules are frequently lengthened based on traditional practices. The traffic operations staff also impeded prior efforts, allowing only short closure windows, which prolongs projects. The NJDOT incentive-disincentive opportunity is tied to computation of roadway-user costs, which are typically very low and do not justify acceleration. Designers are reluctant to suggest innovative approaches because they think NJDOT will not accept them. They have no incentive to be creative, and the state does not procure contracts requiring innovative design and construction solutions.

New Mexico: NMDOT considers accelerated techniques for every bridge project, but problems were noted on NMDOT's first full-depth precast deck panel project, the 2013 Eagle Draw Bridge renovation on NM 13. According to the report, the precast deck panels cost approximately 2.5 times the CIP system based on the bidder's prices. The primary pay items for the precast deck panels were the prestressed, posttensioned concrete; the 8.5-inch precast deck panels; and the epoxy urethane overlay used to create a smooth driving surface and to seal the joints between panels. If the job was done using CIP methods, the primary pay items would have been the concrete in which the deck would have only been 8 inches and epoxy-coated rebar. As one of only two such projects constructed by NMDOT, fabricators, contractors, and designers had no prior experience with full-depth, precast deck panels. The shop drawings for the precast deck panels, twice the number required for CIP construction, went through 5 iterations and took 4 months to be approved. In terms of fabrication, the bridge deck had a crown down the center, which meant one panel could not be used across the entire width, and closure pours had to be used at the abutments, piers, and down the center, causing the exposed rebar from the deck panels to come in contact with the rebar from the closure pours. The rebar then had to be field bent to avoid the adjacent reinforcement and shear studs on the prestressed girders. In addition, the precast deck panels had to be moved transversely over the width of the bridge because the posttensioning ducts in adjacent panels did not line up. This uneven alignment was noticeable along the edges of the deck. The strength of the precast girders was questionable, and since the girders had to be set up before the deck panels, the entire project was slowed. As far as construction was concerned, the contractor could only shut down NM 13 for 60 calendar days, but it could not be shut down until all precast elements were fabricated and accepted by NMDOT. Fabrication took longer than expected, so the contractor decided to close NM 13 at his own When the bridge was closed for over 120 days, the contractor was assessed penalties.

New York: Since the state is so heavily developed, staging is a particular problem. Construction costs are also an impediment; specifically, the use of precast, prefabricated elements and offsite construction using roll-in methods. NYSDOT was also concerned about the durability of precast component connections and joints. Local contractors resist using extensive prefabrication because of the large project share subcontracted out to specialists.

North Carolina: None reported.

North Dakota: The primary concerns are high cost, connection details, and the low level of support for ABC in the state.

Ohio: ABC costs more inevitably but are balanced by user costs when ABC is used on bridges with high average daily traffic (ADT) and relatively high importance to public transportation.

Oklahoma: None reported.

Oregon: In addition to the elevated initial costs, connections for seismic activity presented a major problem. Connections in seismic zones must withstand a much higher transverse loading and dynamic, repetitive loading. Most common connections have not been tested under lab conditions simulating seismic forces. Once the testing is completed, peer-reviewed, and reported, ODOT will have more confidence that connections of precast columns, footings, and pier caps can safely withstand the high horizontal and vertical uplift common in seismic events.

Pennsylvania: Contractors are generally unwilling to assume the additional associated risks with ABC. Because they are inexperienced, they have to subcontract work, which leads to inflated bids.

Rhode Island: None reported.

South Carolina: None reported.

South Dakota: SDDOT is another state with low traffic volumes, so user costs do not balance the cost of ABC.

Tennessee: Questions about the durability and quality of precast members and connections, specifically attaching precast bridge decks to beams, impede ABC use.

Texas: The funding structure in the Texas DOT (TxDOT) provides no owner incentive to use rapid renewal methods other than staged construction. TxDOT districts may use only 5 percent of the project's cost and no more than 25 percent of road-user delay costs for incentives. Although road-user costs are considered, the owner has no way of collecting any savings from them. Therefore, if additional funds are spent to reduce road-user costs, fewer funds will be available for other projects. Federal grants to owners, based on the value of savings, would help them to capture savings from user costs and promote rapid construction projects. Moreover, the incentive amount must be sufficient to pay for the additional construction crews and/or special construction equipment needed for ABC and still result in profit. As an alternative, consider milestones with no-excuse bonuses. If the contractor can complete construction without excuses, then he or she is awarded a bonus but will be most likely to submit the bid assuming no bonus will be awarded.

Utah: At the outset of the ABC program, internal middle management was the biggest obstacle, particularly its conservatism, as in New Jersey. Convincing consultants, designers, and the contracting industry of ABC's merits was easier than convincing DOT staff. However, across the business, the core groups willing to try new things prompted a decision, and UDOT moved aggressively to implement trial projects. It still has some unanswered questions and sees areas for improvement; for example, in specifications, connection details and durability, seismic detailing, design considerations for moving structures, and acceptable deformation limits during movement. Nevertheless, UDOT is moving forward with ABC as a standard delivery mechanism.

Vermont: Vermont does not experience high traffic volumes, so road-user costs are often too small to create meaningful incentive/disincentive clauses in contracts that would encourage ABC projects. A way to incorporate savings from ABC methods, such as eliminating the need for temporary bridges, into incentive/disincentive clauses might change the picture.

Virginia: None reported.

Washington: None reported.

West Virginia: West Virginia contractors are inexperienced in ABC, and the state does not have a precasting industry or heavy-lift contractors. Contractors would probably use ABC standards, so ABC specifications and sample contracts would prove useful.

Wisconsin: Contractors using ABC techniques in a project are most concerned about making money. Training would be beneficial.

Wyoming: With lower traffic counts, the main impediment to ABC implementation is justifying the higher costs.

2.2 INDUSTRIAL SURVEYS

The research team contacted various contractors around the country who had experience with ABC. It wanted to know what types of problems they encountered during the construction or design process and how they were resolved.

Hugh Boyle Engineering:

Hugh Boyle Engineering (HBE) reported that the biggest problem on design-bid-build projects was modifying the original designer's details to fit an alternate ABC option or making the original ABC design easier to construct. According to this engineer, whether the owner and/or original engineer will accept HBE-proposed revisions is normally unknown, so HBE prefers design-build projects.

In HBE's experience, ABC designs try to emulate a traditional design as opposed to looking for alternative methods. For example, a bridge would be designed to use a lateral slide, yet its abutments would be designed to be fully integral because the owner wants to use a fully integral bridge. The solution would be to design a semi-integral system.

Precast element connections are also a concern. One of the most common problems is tolerances that are either too tight or unrelated to any functional requirements. Bridge flexibility is not recognized. Flexibility affects how loads are transmitted to equipment or supports used to move the bridge, which is especially serious for SPMT moves where the hydraulic system must balance structural loads.

HBE has also noticed a disconnection between acceptable tolerances and methods used to slide bridges. A specification may allow an elevation difference of up to 1/8" over 10' of a slide slab. A system can be designed to accommodate a significantly greater difference, but some systems need less tolerance. For example, on most of its slides, HBE uses only two supports per abutment because they are determinant. If one support goes up a little, its load barely changes due to a slight twisting of the structure between abutments; these systems can accommodate much more than 1/8" per 10'. However, when designers use a series of very stiff rollers under relatively stiff superstructures that require much less than the 1/8" tolerance, they still use the original 1/8" specs. With more than 2 supports, the system becomes indeterminate, and roller reactions are very sensitive to roller elevation. On these systems, a 1/8" variance over 10 feet might cause the entire bridge to rock over the high point, essentially putting all the load on a single point, which can be dangerous when the designer assumes that the loads will be evenly distributed over 5 supports. HBE is not actively working on any ABC projects; they have a lateral slide under contract, but the owners are considering cancelling due to budget constraints.

Kraemer North America:

Kraemer North America, a privately held general contractor from Wisconsin, has had plenty of experience using ABC in various transportation and rail projects that included such methods as incremental launching, superstructure roll-in, transverse slides, and superstructure float-ins. The most common prefabricated bridge components they used were precast bent caps, columns, abutments, and full-depth panels. Further details on their experience in specific ABC projects are being sought.

Mammoet:
Mammoet does not design or build bridges; they specialize in moving them with SPMTs. They are called in when an ABC project may need to use the skid or transverse-sliding method. A project manager explained, “We install our transporters underneath the prefabricated bridge and lift the drive, drive away with it, and install it in its final location. We design the support structure on top of our trailers but not the actual bridge. The engineer that designs the bridge already takes into account the fact that the bridge will be driven away. He also checks whether the supports that are under the bridge, will not damage the bridge, etc.”

McFarland Johnson:
McFarland Johnson (MJ) is another contractor with ABC experience. One project was the I-93 Exit 14 bridge in Bow-Concord, NH. MJ was involved in the initial phase and studied alternatives for improving the safety, mobility, and capacity of the I-93 bridge. After evaluation, it determined that ABC would be the best option for replacing the bridge. Each half of the superstructure was replaced within a 60-hour period. MJ used full-depth, precast concrete deck panels with high-early concrete and longitudinal posttensioning for long-term durability. MJ believes that ABC’s advantages will provide future benefits.

2.3 SUMMARY

Survey results for owners and contractors show the following impediments to ABC:

- Higher costs;
- Inexperience with the techniques;
- Constructability concerns about connection details, congestion of rebar around joints, and staging area;
- Resistance to innovation; and
- Design-bid-build contracts.

We learned that contractors prefer CIP construction for bridge renewals because the large prefabricated elements diminish their profits (Sivakumar 2014). Moreover, ABC involves a new technology, and contractors prefer to keep their own employees working instead of subcontracting work to precasters. Possible solutions to these and other impediments to ABC adoption are:

- Introduce the industry to precast technology and demonstrate its profitability;
- Use pre-engineered modular systems that can be built with conventional construction equipment, enabling local contractors to bid on rapid replacement projects;
- Bundle several bridge projects with similar requirements into a single construction contract, allowing a local contractor to get more benefits from repetition; and
- Use full-moment connections with UHPC, which will satisfy the criteria for constructability, structural requirements, and durability in prefabricated modular superstructure systems.

CHAPTER 3. ABC DECISION-MAKING TOOLS

The ABC decision-making tools is a section devoted to provide guidance on when to use ABC versus conventional bridge construction. If ABC is found to be the most efficient type of construction, then this section will also serve as a guide as to which ABC method is deemed most appropriate for a specific project.

Appendix F presents a Decision Making Matrix, an ABC Decision Flowchart, and a Decision Making Scoring Chart and descriptions of items that can be used in conjunction with one another to answer whether to use ABC or conventional methods.

The Decision-Making Matrix in Appendix F may be used to determine how applicable ABC is for a specific project. This matrix is utilized by tallying up the total amount of points next to each section and finding the overall score for a project. Each of the sections listed in the Decision Making Matrix is explained in further detail within the Decision Making Scoring Guidance which is also located in Appendix F. After a total score is determined from the Decision Making Matrix, that score is then used to enter the Decision Making Flowchart at the appropriate location. The Decision Making Flowchart is designed to help the user make an intelligible decision on whether ABC or a conventional method is the best decision for the project. Once the correct scoring location is determined the question "Do the overall advantages of ABC negate any additional costs?" is to be answered. These additional costs may include schedule, traffic impacts, funding, road user costs (RUC), etc. This question is to be answered on project-specific basis taking into consideration all engineering components and professional judgement, and also the available project information. This question is a part of the Decision Making Flowchart in order to assist the user of the Decision Making Tools in analyzing and making an intelligent decision on whether ABC is in fact the best form of construction for a project. After answering this question and concluding that ABC is in fact the best method, the part of the Decision Making Flowchart afterwards will help guide the user to the best form of ABC for the project.

These Decision Making Tools can help to provide insight into which method would be deemed most efficient, however they are not considered to give an exact answer. After reviewing the outcome from the Decision Making Tools it is up to the user to decide whether ABC does in fact make the most sense for the specific project at hand. There is no definitive answer provided upon the completion of the Decision Making aspect of ABC versus Conventional, it is up to the user's better judgement.

The Decision Making Flowchart can incorporate a variety of resources that are pertinent which may include (but are not limited to) "Program Initiatives", like research needs, local resources, input from the public, requests of stakeholders, or structure exhibits. These items should be considered on a project-specifics basis.

While the Decision Making Flowchart is designed to lead the user to the best method for ABC, it should be noted that there is room to combine methods listed at the bottom of the flowchart (i.e. PBES, GRS-IBS).

CHAPTER 4. ABC DESIGN CONCEPTS

This chapter presents current developments in ABC design that can be used for future projects in Georgia. It should be noted that ABC projects use innovative designs that are also compatible with innovative construction techniques because they are highly interrelated. ABC design strategies have the following goals (SHRP2 2014):

- As light as possible
  This concept improves the load rating of existing foundations and piers, and can simplify the transportation and erection of bridge components.
- As simple as possible
  To achieve this goal, it is recommended to reduce the number of certain elements, such as girders, field splices, and bracing systems.
- As simple to erect as possible
  Fewer workers and fresh-concrete operations on site are desirable. Additionally, geometry needs to be simple.

The following sections discuss design considerations and concepts for prefabricated systems in detail.

4.1 MODULAR SUPERSTRUCTURE SYSTEMS

Pre-engineered standards include the concrete and steel girders under modular superstructure systems. When researchers evaluated new construction techniques, technologies, and bridge systems, including lab testing, deck bulb tees and decked steel stringer systems received the highest scores, reflecting what has been used in the field for rapid renewal. Due to the quality of the prefabricated superstructure, high-performance concrete, and attention to different connections, the modular system is predicted to have a 75- to 100-year service life. Modular superstructure standards for steel and concrete will include:

- Decked steel stringer system;
- Composite steel tube girder system;
- Concrete deck bulb tees; and
- Deck double tees.

4.1.1 Decked Steel Stringer System

Like concrete deck girder systems, the decked steel stringer system has proven quite economical and quick to construct (See Fig. 4.1). The use of a modular decked steel system for ABC has become quite popular with states that employ rapid construction techniques.

When compared to precast concrete units, the modular steel system is much lighter, easier to construct, less expensive and easier to fabricate. Each aspect of the system, including the length and weight of the module, can be tailored to suit the particular mode of transportation and erection methods for each case. Conventional construction equipment can usually be employed to erect these steel units, and UHPC is used for closure pours to connect each unit.

(a) (b) (c)

Figure 4.1. Decked Steel Stringer System. (a) Steel grid open or filled with concrete (photo: D.S. Brown Co.). (b) Full-depth precast deck panels with and without longitudinal post-tensioning (FHWA, 2015). (c) Partial-depth precast deck panels (photo: Keegan Precast on project in UK by contractor Laing O'Roruke).

Standard designs of common span lengths will assist in gaining acceptance and more widespread use for modular concepts. Full moment connections are preferred in steel girder systems for the same reason they are in concrete girder systems. In many cases, an integral wearing surface, with a thickness between 1.5 in. and 2 in., can be built with the deck to assist in future surface replacements without damaging the structural deck slab.

4.1.2 Composite Steel Tub Girder System

Composite steel tub girder superstructures can be built in the shop in large scale, transported to the site, and then erected by assembling the pieces together with a minimal need of formwork. (See Figs. 4.2 and 4.3). If there is enough room adjacent to the project site, decks can be cast on site. In addition, these systems can be fabricated as longitudinal sections that can be erected piece wise and assembled together using in-place posttensioning, or they can be fabricated as full-width deck systems that can be erected in a single piece.

Trapezoidal steel box girders are very suitable for this type of large construction. They offer light, cost-effective solutions while providing structural efficiency during transportation, erection, and service life. Trapezoidal box girders building blocks can be designed with a single box, two boxes, or as many as needed to complete the width of deck. Twin tub girders, however, are the most popular standard. These bridges can be designed and constructed to function as simple spans or continuous structures. Several connection details are available and can be used to provide continuity for dead and live loads, either as standard splice construction procedures or specific details applicable to the particular situation on the site.

(a) (b) (c)

Figure 4.2. (a) Steel tub girder (photo: Greg Price, DHS Discussion Forum). (b) Concrete tub girder (photo: StressCon Industries, Inc., website). (c) Open trapezoidal composite box girder (photo: SteelConstruction.info).

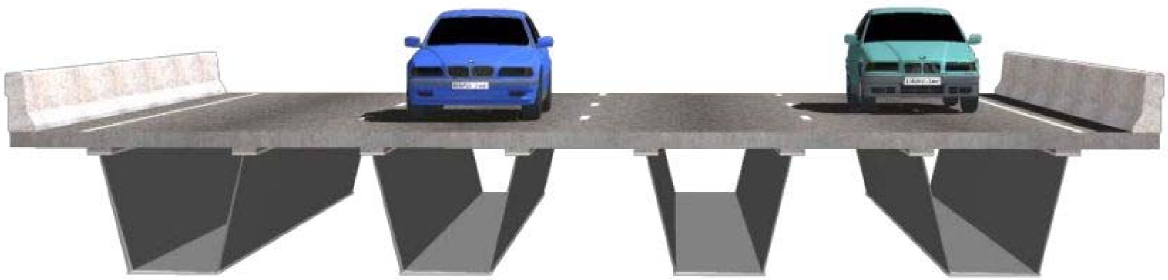
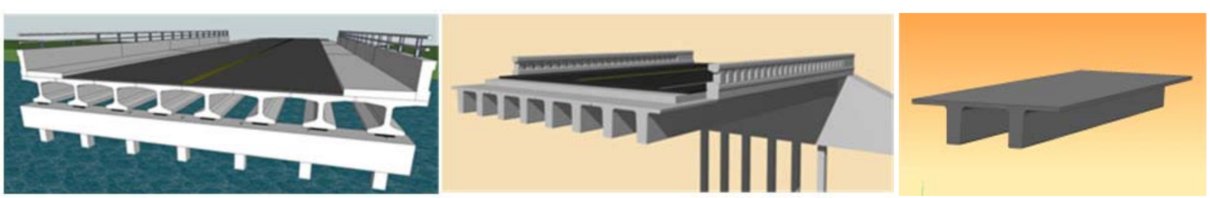


Figure 4.3. Composite Steel Tub Girder (SHRP, 2014).

4.1.3 Precast Concrete Deck Bulb Tee and Double Tee

Conventional precast concrete girders have been perfected and used all over the United States for more than 50 years. Owners and contractors use these types of bridges because they are easy and economical to build and maintain. In most cases, the girders are used with a CIP deck built on site. For ABC, the major difference is that the girders will now have integral decks, eliminating the need for CIP (See Fig. 4.4). This concept of precast decked girders has increased in popularity in several states, but is not used all over the country. The integral wearing surface, which is typically 1.5 in. to 2 in. thick, can be built monolithically or in addition to the deck slab. In the future, the wearing surface concrete can be removed and replaced while preserving the structural deck slab. The precast deck bulb tee girders and double tee girders combine the benefits of eliminating the time it takes to make CIP decked superstructure along with the positive attributes of precast girder construction. This ABC approach should be easily adopted by experienced contractors if they have prior knowledge of conventional precast girder construction.



(a) (b) (c)

Figure 4.4. (a) Adjacent deck bulb tee beams (FHWA, 2015). (b) Adjacent double tee beams (FHWA, 2015). (c) NEXT beam (drawing on High Steel Structures LLC website).

Utah, Washington, and Idaho have proven and standardized deck bulb tee and double tee girders. The Precast/Prestressed Concrete Institute (PCI) Northeast developed the northeast extreme tee (NEXT) beam, a variation of the double tee, to serve the ABC market. This deck girder is expected to be competitive with girder and CIP deck systems. It may also be beneficial for sites where deck-casting operations are constrained. CIP closure pours are typically used to connect girders in the field. These girder flanges can be made to different widths to fit site and transportation requirements.

4.1.4 Pre-Topped Trapezoidal Concrete Tub Beams

Pre-topped trapezoidal concrete tub superstructures have been developed using TxDOT U beams for spans up to 115 ft, which can be transported and erected in one piece. Standards for this system, usually, would be developed to cover span ranging from 60 to 175 ft, with no more than five standard cross sections (See Fig. 4.5). There are two options to construct pre-topped U beams: 1) Spans 60 to 115 ft, transported and erected in one piece; and 2) Spans 60 to 175 ft, transported in 10 ft long, and posttensioned on site. The following design concepts can be considered for pre-topped U beams:

- The use of available standard U sections can minimize fabrication costs.
- Design can be improved to use high-performance materials to reduce weight.
- Lengths less than 115 ft produce sections under 150 tons for shipment in one piece.
- Units would be designed to handle transportation and erection stresses.
- An overlay can be provided with this system and still allow the bridge to be opened within 4 days of the beginning of superstructure erection.
- Limit the number of standardized sections to 5.
- Provide two or three suggested methods of erection, such as cranes, launching, and overhead gantries.
- Edge sections of deck with curb pieces to allow bolting of prefabricated barriers.

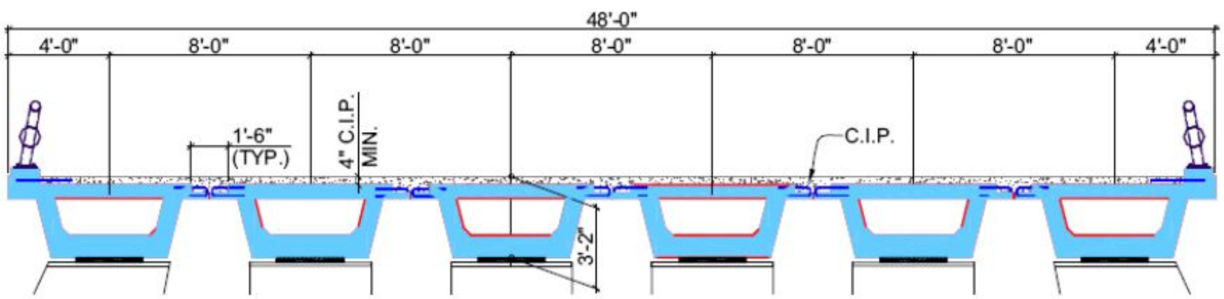


Figure 4.5. Cross Sections of Pre-topped, Trapezoidal, Concrete U Beams (SHRP2, 2014).

4.1.5 Full-Depth Precast Concrete Deck Systems

Full-depth precast concrete deck systems allow the bridge to be reopened to traffic faster, as CIP concrete is needed only at the joints between the prefabricated panels (see Fig. 4.6). The CIP joints can also be replaced by match cast joints, which can save time and efforts. The match cast joints method uses each segment matching cast against its adjacent segment to form a precision fit. The joint width between the installed segments is very small, and any gaps are taken up with epoxy paste. Eliminating the CIP joints with match cast joints accelerates the schedule considerably. The addition of post-tensioning does not increase the time of construction because the posttensioning is required to extrude the epoxy on match cast joints and occurs simultaneously. NCHRP Report 584 (Badie and Tadros 2008) addresses the optimum benefits and opportunities of full-depth precast concrete deck panels.

A fully composite connection between the concrete deck panels and the steel or prestressed concrete girders are the primary concern for a precast deck panel system. There have been several research for the use of higher-capacity shear studs and innovative construction. Full-depth concrete deck panels offer a number of innovative opportunities as listed below:

- Durable transverse panel connection for staged construction. One possibility for this application is the use of UHPC, which has been successfully used by NYSDOT.

- Reduced dead load to simplify installation. The use of lightweight concrete, ultra-high-performance materials, or a waffle-slab configuration offers significant potential.
- Improved riding surface. Improvements in shimming and match casting to provide a smooth surface immediately after placement would be beneficial.

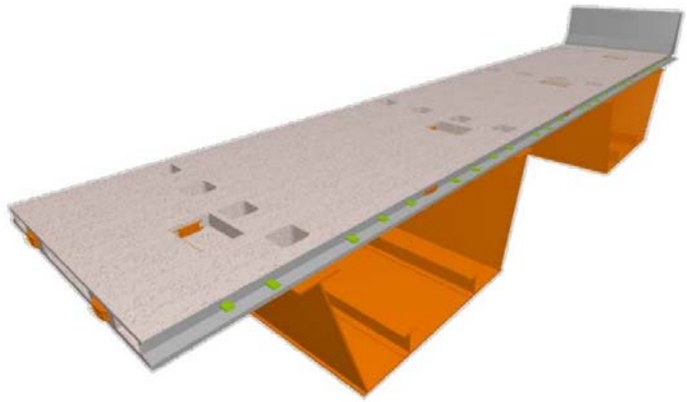


Figure 4.6. Full-depth Precast Concrete Deck System (SHRP2, 2014).

4.1.6 Ultra-High Performance Concrete (UHPC) Superstructures

The extremely high strength and durability of UHPC make it a valid candidate for consideration in standardized ABC components. UHPC is composed of fine sand, cement, and silica fume in a dense, low water-to-cement ratio (0.15). Compressive strengths of 18,000 to 30,000 psi can be achieved, depending on the mixing and curing process. UHPC has an average strength gain of 10 ksi in 48 hours, which is when deck grinding can begin. The material has an extremely non-existing intrusion of chloride-laden water. To improve ductility, steel or polyvinyl alcohol (PVA) fibers (approximately 2% by volume) are added, which replace the use of mild reinforcing steel. Although, however, the UHPC material was designed to function without conventional reinforcing, it might be an option to provide nominal reinforcing as a redundant system.

Full moment connections coupled with UHPC joints are the preferred connection type for ABC purposes regardless of which modular system is used, because of their structural behavior and durability. Prefabricated components with UHPC connections have proven to have increased connection performance over time when compared to conventional construction materials and practices. The properties of UHPC allow for the use of small-width, full-depth closure pour connections between modular components that can withstand the abuse of vehicular impacts and heavy loading. Connection size of UHPC joints compared to conventional concrete are much smaller because of their impressive strength.

The narrow joint width reduces concrete shrinkage and the quantity of UHPC required, while providing a full moment transfer connection. UHPC, however, is not cheap or easy to work with so the less that is required the better. For example, this material is projected to cost three to five times as much as conventional concrete. A longer cycle of casting and heat curing is also required to achieve extremely high compressive strength. Furthermore, the limited number of casting locations in the United States might be a potential impediment.

4.1.7 Connections between Modules

The ease and speed of construction of a prefabricated bridge system in the field have direct correlation to its acceptance as a viable system for rapid renewal. ABC construction time is greatly influenced by the speed with which the connections between modules can be assembled. Connections between modular segments can also affect live load distribution characteristics, seismic performance of the superstructure system, and the superstructure redundancy. Connections play a crucial role when designing with this approach. Often, the time to develop a structural connection is a function of cure times for the closure pour. Joint detail and number are crucial to the speed of construction, to the overall durability and the amount of long-term maintenance the final structure will require. The use of CIP concrete closure joints should be kept to a minimum for accelerated construction methods due to placement, finishing, and curing time.

To enhance load transfer, prevent cracks under live loads and close shrinkage, the use of induced compression in post-tensioned joints is favored. The post-tensioned joints can present a female-female shear key arrangement infilled with grout or match-cast with epoxied joints, but only if precise tolerances can be maintained throughout the lifetime of the bridge. This process will provide long-term performance. Post-tensioning requires an additional step and complexity during on-site construction, therefore, its use may slow down field assembly and compromise long-term durability which makes it unfavorable for ABC.

Design considerations for connections between deck segments include:

- Full moment connections that can be built quickly.
- At least as durable as the precast deck.
- Joints that can suit heavy, moderate, and light truck-traffic sites.
- Ride quality that is at least equal to CIP decks.
- Durability even without overlays on the deck. An integral wearing surface consisting of an extra thickness of concrete over the structural slab can help.
- Post-tensioned connections can be an alternative for ABC construction.
- Details that can accommodate slight differential camber between neighboring modules.
- Quick strength gain, so that traffic can be opened with very little delay.

4.1.8 Summary of Design Considerations for Modular Superstructures

Design considerations for modular superstructure systems include:

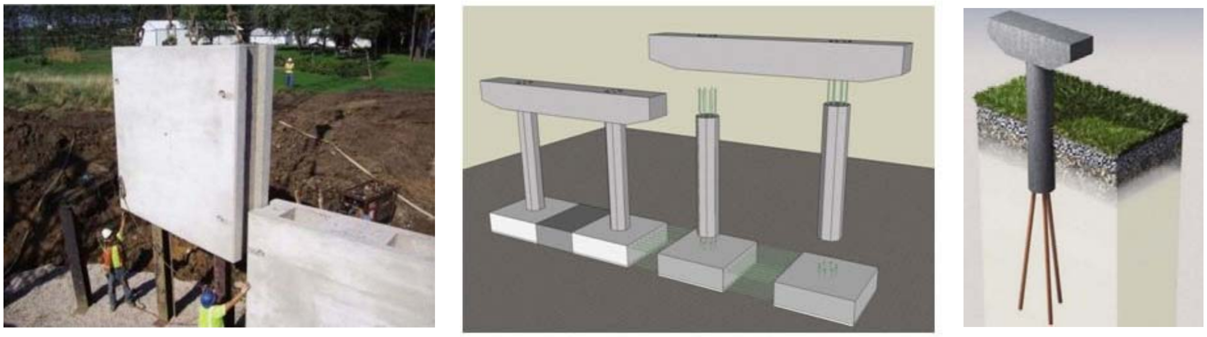
- Pre-engineered standards for modular construction. Designs that can be used for most sites with minimal bridge-specific adjustments.
- Optimize designs for ABC and use of high-performance materials. Simplicity and efficiency of design, availability of sections, and short lead times are key considerations.
- Usually length ≤140 ft, weight ≤100 tons, width ≤8 ft for transportation and erection using conventional construction equipment.
- Able to accommodate moderate skews. For rapid renewal, it would be more beneficial to eliminate skews between bents and the longitudinal axis of the bridge.
- Segments designed for transportation and erection stresses, including lifting inserts. Sweep of longer beams should not be an issue for erection as there is an opening between the beams.

- Segments that can be installed without the need for cross frames or diaphragms between adjacent segments. This improves speed of construction and reduces costs. Use of diaphragms is optional based on owner preference.
- Segments that can be used in simple spans and in continuous spans (simple for dead load and continuous for live load). Details to eliminate deck joints at piers. Details for live load continuity at piers to be included for use as required.
- Use of high-performance materials: High Performance Concrete (HPC)/UHPC, High Performance Steel (HPS), or A588 weathering steel. Consider lightweight concrete for longer spans to reduce weights of deck segments.
- Deck tees and double tees with minimum 8-in. flange to function as decks with integral wearing surface so that an overlay is not required. Use of overlay is optional.
- Cambering of steel sections for longer spans. Control fabrication of concrete sections, time to erection, and curing procedures so that camber differences between adjacent deck sections are minimized. Leveling procedure to be specified to equalize cambers in the field during erection.
- Deck segments when connected in the field should provide acceptable ride quality without the need for an overlay. Deck segments to have ¼-in. concrete overfill that can be diamond ground in the field to obtain a desired surface profile.
- Limit the number of standardized designs for each deck type to five, which should cover span ranges from 40 ft to 140 ft. Consider steel rolling cycles and sections widely available.
- Segments designed to be used with either full moment connection between flanges or with shear-only connections. Each flange edge needs to be designed as a cantilever deck overhang.
- Design for sections that can be transported and erected in one piece. Lengths up to 140 ft may be feasible in certain cases. Provide one method of erection. (Spans longer than 140 ft may be erected by shipping the segments in pieces, splicing on site, and using a temporary launching truss for erection.)
- Design for sections that can be transported in pieces and spliced on site before erection to extend spans to 200 ft and beyond. Develop two alternate erection techniques when conventional lifting with cranes may not be feasible due to weight or site constraints.
- Edge sections of deck with curb piece ready to allow bolting of precast barriers.
- Provide standard details for durable connections between deck segments.

4.2 MODULAR SUBSTRUCTURE SYSTEMS

A significant portion of the on-site construction time is dedicated to building the substructure. Reducing the time for substructure work is critical for all rapid renewal projects. Precasting as much of the substructure as possible will allow for faster construction of the bridge and reduce interference with normal system operation. With this in mind, this section provides details about modular systems for abutments, wingwalls, and piers that are commonly used in routine bridge replacements. These standards include the following:

- Precast modular abutment systems. See Fig. 4.7(a).
- Precast complete pier systems. See Fig. 4.7(b)
- Hybrid drilled shaft/micropile foundation systems. See Fig. 4.7(c).



(a) (b) (c)

Figure 4.7. (a) Precast modular abutment systems (SHRP2, 2014). (b) Precast complete pier system (FHWA, 2015). (c) Hybrid drilled shaft/micropile foundation (SHRP2, 2014).

4.2.1 Integral and Semi-Integral Abutments

Installing roadway expansion joints and expansion bearings can slow down construction considerably, raise lifetime maintenance costs and reduce the life of the structure. Therefore, these expansions tend to be avoided. Besides providing a more maintenance-free durable structure, eliminating joints and expansion bearings can make the bridge design more innovative and may result in cost efficient solutions regarding construction. Providing a bridge design with minimal joints and maintenance liabilities should be an important goal while planning rapid renewal projects. The use of integral or semi-integral abutments allows the joints to be moved beyond the bridge and into the abutments. Integral bridges are bridges where the superstructure is continuous and connected monolithically with the substructure with a moment-resisting connection. Bridges utilizing integral abutments have proven to be cheaper to construct, easier to maintain, and more economical to own over their lifespan. These types of abutments, integral and semi-integral, are preferred in bridge construction by most DOTs.

The downside to eliminating deck joints is that alternative ways must be found to account for creep, shrinkage and temperature change. Usually, provisions are made for thermal movement at the ends of the bridge by using either integral or semi-integral abutments. Along with adding these abutments, there is a need to place a joint in the pavement or at the end of a concrete approach slab. Continuous jointless bridges are generally referred to as “integral bridges” and “integral abutment bridges (IAB)”. Stub or propped-pile end caps are commonly used when designing a bridge due to their superior flexibility. The flexibility offered by these end caps provides little resistance to cyclic thermal movements. A single row of vertical piles is highly recommended to provide a high level of flexibility to combat thermal movements.

The semi-integral abutment bridge (SIAB) is related to the integral abutment design. In SIAB, only the backwall portion of the substructure is directly connected with the superstructure, due to this change there will be no expansion joints within the bridge. The stationary abutment stem holds the bearings which holds the beams so the superstructure and backwall will move together during thermal expansion and contraction.

4.2.2 Jointless Construction

ABC is intended to reduce on-site construction time and eliminate long traffic delays through the use of precast components and innovative construction practices. Eliminating joints results in faster construction and a surplus of money that can be allocated to other aspects of the construction process. Some of the advantages to jointless construction for ABC projects are summarized as follows:

- Issues with tolerances are reduced. The close tolerances required when using expansion bearings and joints are eliminated by using integral abutments. Minor mislocation of the abutments does not create problems with the final fit of the bridge.
- Rapid construction. With integral abutments, only one row of vertical (not battered) piles is used and fewer piles are needed. The backwall and superstructure can be cast together with less forming. This reduces the amount of materials needed, and thus reduces the cost of the project. Fewer issues are encountered with scheduling between suppliers and manufacturers. Integral abutment bridges are faster to erect than bridges with expansion joints, which leads to cost savings. IABs are quicker to construct because the connections involved are simple to form, and there are no expansion joints to slow down construction.
- Reduced removal of existing elements. Integral abutment bridges do not require the complete removal of existing substructures, and can actually be built around existing foundations. Reducing the amount of demolition entailed with the construction process will greatly reduce the overall duration of the project.
- No cofferdams. Integral abutments are generally built with capped pile piers or drilled shaft piers that do not require cofferdams.
- Improved ride quality. Jointless bridges provide a very smooth ride and diminish the impact stress a car experiences. This translates to lower impact loads and, for snow prone areas, less deck damage due to snowplows.
- Integral abutments provide an added element of redundancy in components and capacity for many types of catastrophic events. When seismic events are considered in designs, a significant amount of material can be cut from the design by using integral abutments which do not need enlarged seat widths and restrainers. Integral abutments completely eliminate the loss of girder support, which has proven to be the most common cause of bridge damage in seismic events. The presence of joints creates a much higher potential collapse risk of the overall bridge structure. In the past, integral abutments have consistently outperformed standard CIP bridges during actual seismic events, and have shown very minimal problems with backwall and bearing damage that are associated with seat-type jointed abutments.

4.2.3 Precast Abutments and Wingwalls

Bridge abutments are constructed in several different pieces off site in a factory, shipped to the construction site and then put together in the field (See Fig. 4.8). ABC construction companies have preferred an integral connection of the superstructure and substructure. The different components that are being shipped on site should be designed to be transported over roads and constructed using typical construction equipment. To this point, the precast components are made as light as practicable. Voids can be used in the wall sections of larger elements. This is to reduce their weight and facilitate their fabrication and shipment. Voids are also used to attach drilled shafts or piles to the cap for stub-type abutments. Once the components are constructed into place, the voids and shear keys are filled with self-consolidating concrete. Wingwalls are also precast with a formed pocket to slide over wingwall piles or drilled shaft reinforcing. Once this process has been completed, the wingwall pockets are filled with high early strength concrete or self-consolidating concrete.

(a) (b) (c)

Figure 4.8. (a) and (b) Precast modular abutment systems. (c) Precast wingwall (SHRP2, 2014).

4.2.4 Connections

CIP construction can be eliminated by full-moment connections between modular substructure components. The closure pours are constructed using self-consolidating concrete which makes them easy to construct and results in a highly durable connection. Self-consolidating or self-compacting concrete (SCC) is used to enclose or encapsulate congested reinforcements. This is due to the fact that SCC is highly flowable and non-segregating. It fills formwork without the need to employ any mechanical vibration. SCC is also an ideal material to fill pile pockets in substructure components. This kind of concrete mix can be placed purely by means of its own weight, with little or no vibration. SCC allows easier pumping, flows into complex shapes, transitions through inaccessible spots, and minimizes voids around embedded items to produce a high degree of uniformity.

4.2.5 Precast Complete Piers

Precast complete piers consist of separate components premade off site, shipped, and fabricated onsite (See Fig. 4.9). Piers with single-column and multiple-column configurations are common. Foundations can consist of drilled shafts, which can be extended to form the pier columns. When soil conditions are appropriate, precast spread footings can be employed. However, if soil conditions do not permit these footings, driven piles may be used with precast pile caps. Pier columns are attached to the foundation by grouted splice sleeve connectors. Precast columns can be square or octagonal, the tops of which are connected by grouted splice sleeves to the precast cap. The precast cap is typically rectangular in shape. The pier bents may have single or multiple columns.

States in high seismic regions use integral pier caps. However, the standards in this project were developed only for non-integral piers, which have been found to present more benefits in rapid construction. When using integral pier cap connections, CIP concrete is commonly used. However, the connection can also be made with precast concrete, but it often requires a complicated and lengthy procedure. There are also tight controls over tolerances and grades so the most common form of connection is a CIP concrete closure pour. In a non-integral pier cap, the superstructure and deck will be continuous and jointless over the piers which makes them easier to reuse.

As it is the case with precast modular abutments, the precast piers have been designed to be shipped from the fabrication location to the construction site. To this point, the precast components are made as light as practical for shipping purposes. Precast spread footings can consist of partial precast or complete precast components. To avoid localized point loads, a grout-filled void will be formed beneath the footing to transfer the load to the soil. Column heights and cap lengths will be limited by transportation regulations and erection equipment, but these cap length limitations can be avoided by forming multiple short caps that will function as a single pier cap would. Precast bearing seats can also be used for pier design.

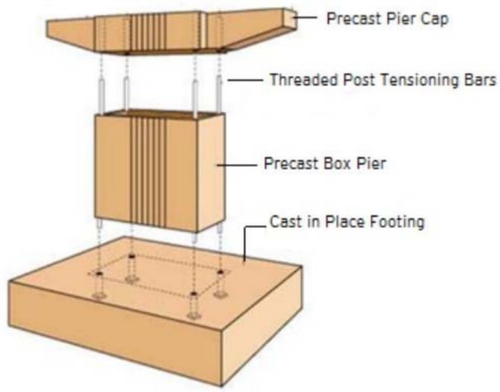


Figure 4.9. Precast Concrete Pier (SHARP2, 2014).

4.2.6 Hybrid Drilled Shaft/Micropile Foundation Systems

A hybrid system is composed of conventional drilled shaft and clusters of micropiles, whereby the upper portion (10 to 20 ft) of the deep foundation is constructed by conventional drilled shaft and the lower portion of the shaft is composed of micropiles. Above grade, the drilled shaft is extended to serve as a circular pier column, eliminating pile cap foundation construction. Below grade, the drilled shaft portion of the hybrid foundation need to extend only to the depth required by design, with due consideration of flexural demands and extreme events relating to scour and seismic design.

Micropile foundation systems have several advantages for ABC. One is the possibility to use low-cost, small-footing, all-terrain drilling rigs for installation, and to employ segmented 5 to 12 in. nominal diameter high-strength steel casings that allow for rapid installation in low head-room conditions. Another advantage associated to the use of micropiles is a reduced construction risk, since a failed micropile can simply be abandoned and replaced with a closely adjacent one.

4.2.7 Steel or Fiber-Reinforced Polymer (FRP) Jacket System for Existing Column

Jacketing has been used to extend the life of bridge columns that may suffer from significant spalling due to corrosion of reinforcing steel, or for columns that must be upgraded for seismic considerations. External jacketing is used to provide the desired level of confinement without the need for expensive, time-consuming replacement. The concept of column jacketing can be used not only as a retrofit for providing additional capacity, but also as a means for accelerated construction without on-site formwork.

The use of steel or FRP jacket systems has achieved success for retrofitting and strengthening of existing concrete piers for many years (See Fig. 4.10). These jacket systems offer a number of advantages for accelerated construction:

- Prefabricated shell components can be easily standardized in a variety of commonly used shapes and sizes.
- Easy transportation and erection on site.

- No on-site formwork to be constructed and stripped.
- Suitable for use with all foundation types, including footings and drilled shafts.

(a) (b) (c)

Figure 4.10. (a) Steel/FRP jacket concept (SHRP2, 2014). (b) Steel jacketed bridge column (Nelson, 2012). (c) FRP jackets in several bridge columns (Buccola, 2011).

CHAPTER 5. RISK ANALYSIS

The risk-assessment component of the toolkit enables the user to determine how to best convey surface water (if applicable) and storm water runoff in order to minimize damage to the roadway, bridge itself, and other property. This process starts by helping the user decide if a culvert or bridge is most appropriate for the site. If a culvert is selected, the toolkit assists in helping the user select the culvert's shape, material, and initial size. If a bridge crossing is most appropriate, the toolkit provides the user with information as how the hydrologic and hydraulic considerations influence the bridge foundation investigation (BFI) and scour analysis components of the bridge design.

5.1 THE ROLE OF RISK IN CULVERT AND BRIDGE DESIGN

Since rainfall events are governed by chance, historical rainfall information and statistical analysis are used to estimate the magnitude of different storm events over different return periods (example: 50 year storm). Knowing this information about a storm, along with information about the physical attributes of the watershed (area, slope, soil type, vegetation cover, percentage of impervious surfaces) allows us to predict how much water will drain from the watershed and eventually drain through the culvert or under the bridge.

Risk is a measure of the probability of occurrence multiplied by the cost associated with repairs/replacement caused by the event. Since a storm's return period (20 year, 50 year, etc.) is inversely related to probability that the storm will occur that year, there is a direct relationship between the storm period and risk. While larger return periods have lower probabilities of occurrence, the tradeoff is that they also have higher construction costs. For example, using a return period at 20 years (probability of equaling or exceed the storm is 1/20 or 5%) might result in a project with a low initial construction project cost but with frequent repair or replacement costs. Conversely, using a large return period of 200 years (probability of equaling or exceed the storm is 1/200 or 0.5%) can result in an overly designed project with an excessive construction cost. Since the selection return period requires careful consideration of several factors (potential damage to highway and property as a result of flooding, potential hazards and inconveniences to the public, and project costs) GDOT specifies the required return periods (flood frequency) that should be used for both culverts and bridges (GDOT, 2014):

- Culverts for state routes and interstate highways shall be designed using a 50 year flood frequency.
- Bridges for state routes and interstates shall be sized so that a 50 year flood is conveyed only through the bridge opening and the 100 year flood is conveyed through the bridge opening and over the roadway.

5.2 CHOOSING BETWEEN A CULVERT AND A BRIDGE CROSSING

Culverts are closed conduits that covey surface water or storm water runoff from one side of a road to the other side. They play a key role in preserving the road base by preventing water from overtopping the road surface and by keeping the sub-base dry by draining water from ditches along the road. Whereas bridges use the bridge deck, superstructure (beams, girders), and substructure (abutments, piers) to support vehicle loads, culverts rely on the structural properties of the conduits and the embankment material covering them and to support these loads.

In cases in which the toolkit user needs to decide between using a culvert or bridge crossing, the following guidelines should be applied:

Cases in which a bridge crossing is recommended:

- Area draining to the crossing exceeds 20 miles^2 (12,800 acres).
- Cases in which the surface water canal is navigable.
- Cases in which the water area at the crossing is undefined.
- If the crossing point is located near an area where flow back up behind the culvert could flood residential areas (Ministry for the Environment, 2004).
- If high debris loads (gravel, trees, logs) passing below the roadway are likely.
- Cases in which the hill catchments is steep (rule of thumb value is 6% or larger) (Ministry for the Environment, 2004).
- When the required culvert size (with minimum soil cover) exceeds the elevation difference between road and canal or drain.

Conversely, culverts can be used when:

- Area draining to the crossing does not exceed 20 miles^2 (12,800 acres).
- Surface water is limited or not present.
- The water area is well defined (i.e. easily be routed through the culvert).
- Cases in which large debris will not pass below the roadway.
- Cases in which any flow backup will not flood adjacent areas.

5.3 SELECTING A CULVERT TYPE AND SIZE

A culvert's shape (Fig. 5.1 for common shapes) and material depends on site-specific characteristics including: the elevation difference between road and canal or drain (available soil height), required span, and bearing capacity of the soil. Other important factors include: material and installation cost, needs of fish and other aquatic organisms, and local preferences. Circular (pipe), box, and arched culverts similar to the ones shown in Fig. 5.2 are most commonly used.

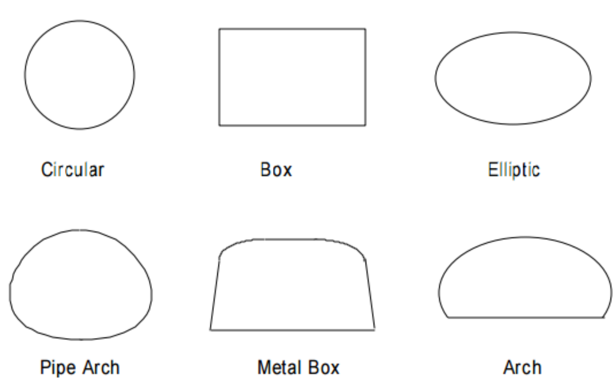


Figure 5.1. Common Culvert Shapes (Purdue University, 2005).

(a)

(b)

(c)

Figure 5.2. Pipe (a), Box (b), and Arch (c) Culverts (Cranberry Township, American Concrete Industries, and Contech, 2015).

Culverts are made using a wide variety of materials. While concrete, reinforced concrete, steel (smooth and corrugated), corrugated aluminum, and plastic (high density polyethylene) are the most common, vitrified clay, bituminous fiber, cast iron, wood and masonry culverts are occasionally used.
By definition, culverts must have a clear span of no more than 20 feet (GDOT, 2014). If the culvert's clear span exceeds 20 feet, then it is designated as a *bridge culvert* (See Fig. 5.3). When multiple culverts are installed side by side, referred to as a *multi-barrel* culvert, then span can exceed 20 feet (and still be designated as a culvert) if the spacing between culverts is less than half the culvert width/diameter (GDOT, 2014). For instance, while the total span of the multi-barrel box culvert in Fig. 5.2(b) exceeds 20 feet, because the spacing between each culvert is less than half the culvert width, this is designated as a culvert and not a bridge culvert.

Figure 5.3. Bridge Culvert (Contech, 2015).

5.4 PROCESS FOR SIZING AND DESIGNING CULVERTS

There are many hydrologic, hydraulic, economic, and site-specific factors that must be considered when designing a culvert. In addition to knowing how much water is required to be transported by the culvert (refer to previous section on risk), factors such as culvert slope, conduit material properties (roughness, strength), soil characteristics, and water velocity must all be considered as part of a hydraulic evaluation. Flow through culverts can be complex depending if either end of the conduit (barrel) is submerged (covered) by either (or both) the headwater or tailwater ends (See Fig. 5.4). As such, GDOT requires that all culvert designs follow the guidelines presented Chapter 8 of the 2014 GDOT Manual on Drainage Design for Highways. This includes a requirement for the designer to utilize either the HY-8 Culvert Hydraulic Analysis Program or the Hydrologic Engineering Centers River Analysis System (HEC-RAS) culvert modules.

In addition to the hydrologic and hydraulic considerations, culvert design is also dependent on the impact of fish and other aquatic organisms, any local preferences, construction and maintenance costs.

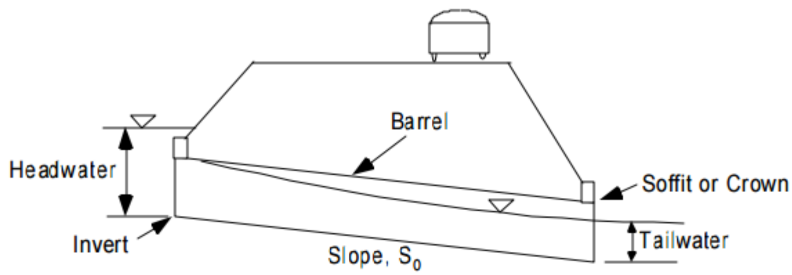


Figure 5.4. Culvert Cross Section showing Headwater and Tailwater Levels (Purdue University, 2005).

5.5 PROCESS FOR SIZING CULVERTS AND REQUIRED BRIDGE OPENINGS

While a detailed culvert design requires the expertise of an engineer familiar with the GDOT guidelines and the necessary software, the toolkit provides the user with the ability to determine an initial culvert size for estimating purposes. The following steps outline the process that is used within the toolkit.

Step 1: Delineating the Watershed

A watershed is an area that drains to a common point of discharge (outlet). Since water flows downhill through gravity, identifying the boundary (delineating) a watershed involves using a topographic map to identify the outlet or downstream point and then locating the boundary at which any rains falling within the boundary will be directed towards the outlet. Fig. 5.5 below shows an example of a delineated watershed boundary.

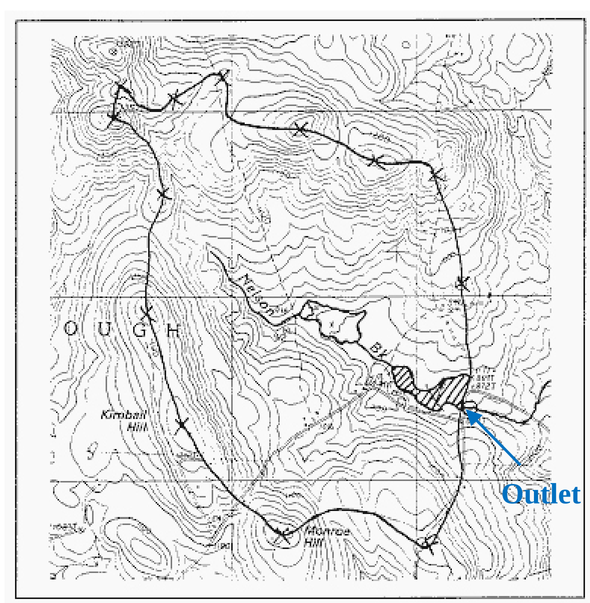


Figure 5.5. Example of a Delineated Watershed Boundary (Natural Resources Conservation Service, 2014).

The area of the watershed can be determined using various techniques. GDOT recommends using the United States Geological Survey (USGS) application StreamStats (http://water.usgs.gov/osw/streamstats/georgia.html) which allows the user to click on the culvert/bridge crossing point and have the software compute the watershed area. Area can also be manually estimated by using a planimeter or by counting the square grids and multiplying by the map scale. For example, for maps with a 1:24,000 scale (1 inch represents 2,000 feet) one square grid represents 4 million square feet or 91.8 acres). At a scale of 1:24,000, the map shown in Fig.5.5 would have an approximate area of 3.5 squares or 321 acres.

Step 2: Determining Peak Flow

If a gauge is installed at the outlet (refer to Fig. 5.5) to record the flow passing through this point over the duration of a storm, the resulting graph (referred to as a hydrograph) would have a shape similar to the one shown in Fig. 5.6. The peak flow (Q_p) is highest point on the hydrograph, representing the largest flow rate. This flow occurs at the time to peak (t_p) which is the time at which the entire watershed is contributing to the runoff.

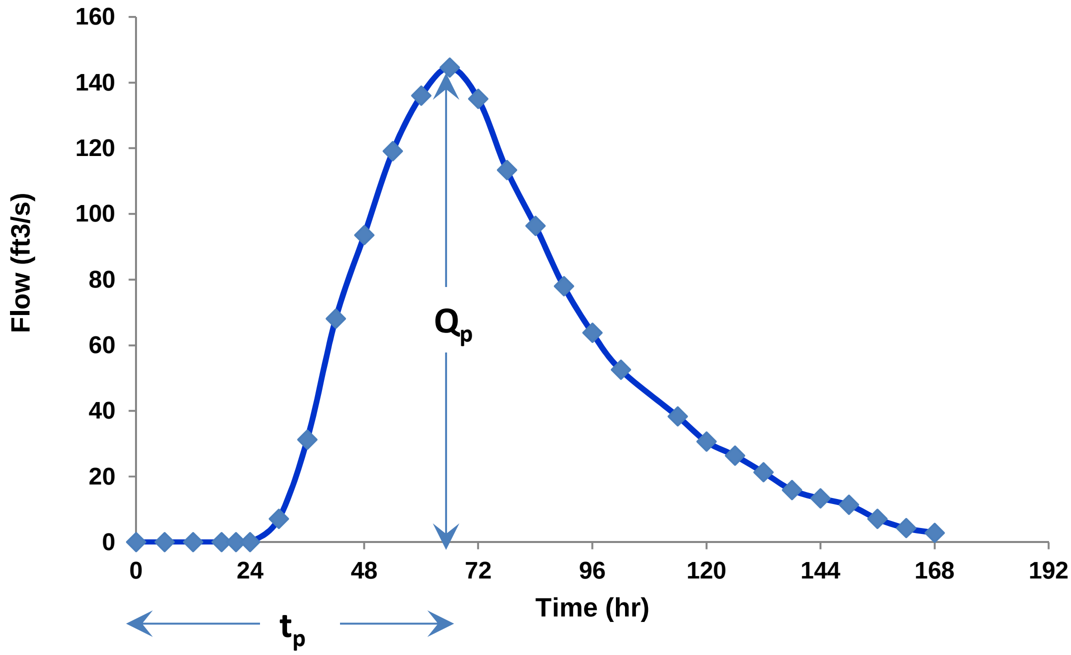


Figure 5.6. Example Hydrograph.

There are several commonly used methods for determining peak runoff. The primary differences between each method relates to: i) the assumptions inherent to each method and ii) the required data (slope, vegetation cover, etc.). Accordingly, the methods presented in this manual (and used within the toolkit) where selected based on their wide acceptance and ease of use. While these methods are acceptable for an initial estimate of culvert or bridge opening sizes, an actual culvert design requires a more detailed hydrologic and hydraulic design as specified in the 2014 GDOT Manual on Drainage Design for Highways.

Method 1: USGS Regression Equations

One option for estimating flow is to apply the regression equations developed by the USGS which take into account the watershed and climatic characteristics within 5 hydrologic regions within the state (See Fig. 5. 7). While the USGS has separate equations for rural and urban watersheds (urban watersheds have impervious areas of 10% or greater), because the urban equations can only be applied to 4 regions within Georgia and require parameters such as mean basin slope and percent developed land which are often not readily available, the toolkit uses the equations for rural watersheds. These equations are shown in Table

5.1 for eight different return periods (2 ~ 500 years) for all five regions within Georgia with watershed areas between 1 and 9,000 miles^2. Note that the Table 5.1 equations can only be applied to a watershed that is entirely within one hydrologic region.

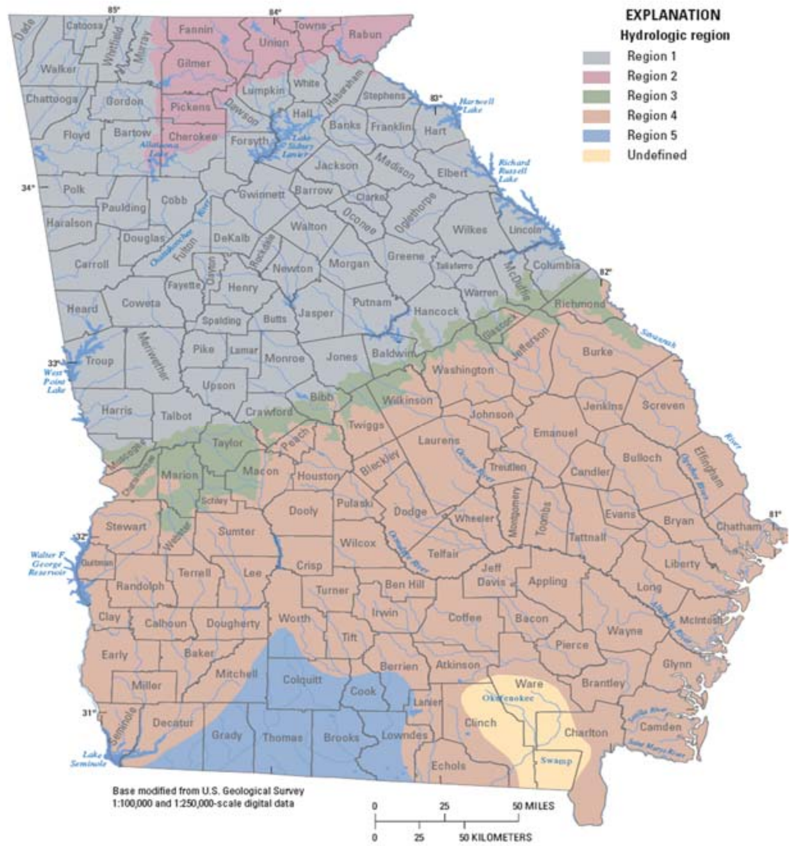


Figure 5.7. Map of the Georgia Flood Frequency Regions (USGS, 2008).

Table 5.1. Regression Equations for Estimating Peak Flow in Rural Ungauged Areas that are Entirely Within One Hydrologic Region (USGS, 2009)

Return Period (years)	Regression Equations for Peak Flow (ft^3/s) in all Hydrologic Regions (DA: Drainage area in square miles)				
	1	2	3	4	5
2	$158(DA)^{0.649}$	$110(DA)^{0.779}$	$25.7(DA)^{0.758}$	$60.3(DA)^{0.649}$	$91.2(DA)^{0.649}$
5	$295(DA)^{0.627}$	$209(DA)^{0.749}$	$44.7(DA)^{0.744}$	$123(DA)^{0.627}$	$200(DA)^{0.627}$
10	$398(DA)^{0.617}$	$288(DA)^{0.736}$	$58.9(DA)^{0.740}$	$174(DA)^{0.617}$	$295(DA)^{0.617}$
25	$537(DA)^{0.606}$	$398(DA)^{0.724}$	$77.6(DA)^{0.736}$	$245(DA)^{0.606}$	$447(DA)^{0.606}$
50	$661(DA)^{0.600}$	$479(DA)^{0.718}$	$91.2(DA)^{0.735}$	$309(DA)^{0.600}$	$575(DA)^{0.600}$
100	$776(DA)^{0.594}$	$575(DA)^{0.713}$	$105(DA)^{0.733}$	$380(DA)^{0.594}$	$724(DA)^{0.594}$
200	$891(DA)^{0.589}$	$661(DA)^{0.709}$	$120(DA)^{0.733}$	$447(DA)^{0.589}$	$891(DA)^{0.589}$
500	$1,072(DA)^{0.583}$	$794(DA)^{0.704}$	$138(DA)^{0.732}$	$550(DA)^{0.583}$	$1,148(DA)^{0.583}$

For watersheds located within multiple regions, the 2009 USGS manual provides a separate set of equations that can be applied. As an example, equations for peak flow for 50 and 100 year storm events are given as:

$$Q_{p\ 50} = 10^{[0.0282(PCT_1)+0.0268(PCT_2)+0.0196(PCT_3)+0.0249(PCT_4)+0.0276(PCT_5)]}\ (DA)^{[0.600+0.001118(PCT_2)+0.00135(PCT_3)]}$$
$$Q_{p,100} = 10^{[0.0289(PCT_1)+0.0276(PCT_2)+0.0202(PCT_3)+0.0258(PCT_4)+0.0286(PCT_5)]}\ (DA)^{[0.5940+0.001119(PCT_2)+0.00139(PCT_3)]}$$

where Q_{p50}, Q_{p50} = Peak flow for the 50 and 100 year storm event, and PCT_1, PCT_2, etc. = Basin percentages in hydrologic regions 1, 2, etc.
As an example, a 130 miles^2 watershed spanning region 2 (90 miles^2) and region 3 (40 miles^2) would have PCT_1 of 69.2% and PCT_2 of 30.8% respectively. Using the equation shown above for the 50 year event, the peak flow would be 9,710 ft^3/s.

Method 2: Rational Method
For cases in which the watershed is clearly urban and less than 200 acres (0.3 miles^2) in area the Rational Method can be applied to estimate peak flow. The area limitation is primarily due to the assumption that the rainfall intensity is constant over the entire basin for a duration of time equal to or greater than the time of concentration. It also assumes that the runoff coefficient (C) is constant during the storm event. The formula estimates the peak flow at any location within the watershed as a function of drainage area, runoff coefficient, and rainfall intensity for a duration equal to the time of concentration (defined as the time required for water to flow from the most remote point in the watershed to the location being analyzed). In equation form:

$$Q_p = CIA$$

where Q_p = peak flow (ft^3/s); C = runoff coefficient (dimensionless); I = rainfall intensity (in/hr); and A = drainage area (acres).

Runoff Coefficient (C)
The runoff coefficient is a unit less number between 0 and 1 that relates the rate of runoff to the total rainfall. The more covered and impervious the land is (such as pavement) the closer to 1 the C value becomes. The coefficient is highly dependent on several factors including: land use, ground slope, topography, and soil factors influencing the rainfall infiltration into the soil. Table 5.2 shows the recommended runoff coefficients for flat, rolling, and hilly terrains. This table corresponds to storms of 5 year to 10 year frequencies. For higher year storms, the coefficients shown in Table 5.2 should be adjusted by multiplying the coefficients by the frequency adjustment factors (f_a) shown in Table 5.3. For example, the runoff coefficient for apartment homes located in a flat area for a 50 year storm event would be calculated as:

$$C = (.50) * (1.25) = 0.625$$

In cases in which there are several different types of surfaces within the drainage area, a composite (weighted) coefficient can be computed by using the percentages of the different land uses. This is done by using the following equation:

$$C_{weighted} = \frac{C_1A_1 + C_2A_2 + \cdots C_nA_n}{A_{total}}$$

where C_1, C_2, etc. = runoff coefficients for surface 1, 2, etc.; and A_1, A_2, etc. = areas 1, 2, etc.

Table 5.2. Runoff Coefficients (C) for the Rational Method (GDOT, 2014)

Type of Cover	Flat (0%-2%)	Rolling (2%-10%)	Hilly (Over 10%)
Pavement and Roofs	0.95	0.95	0.95
Earth Shoulders	0.50	0.50	0.50
Drives and Walks	0.75	0.80	0.85
Gravel Pavement	0.50	0.55	0.60
City Business Areas	0.80	0.85	0.85
Suburban Residential	0.25	0.35	0.40
Apartment Homes	0.50	0.60	0.70
Single Family Residential	0.30	0.40	0.50
Lawns, Very Sandy Soil	0.05	0.07	0.10
Lawns, Sandy Soil	0.10	0.15	0.20
Lawns, Heavy (clay) Soil	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay and Loam	0.50	0.55	0.60
Cultivated Land, Sand and Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	0.90
Parks and Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodlands and Forests	0.10	0.15	0.20
Meadows and Pasture Land	0.25	0.30	0.35
Pasture with Frozen Ground	0.40	0.45	0.50
Unimproved Areas	0.10	0.20	0.30
Water Surfaces	1.00	1.00	1.00

Table 5.3. Frequency Adjustment Factors for Rational Method
(Georgia Stormwater Management Manual, 2001)

Storm Frequency	f_a
25 year	1.1
50 year	1.2
100 year	1.25

<u>Rainfall Intensity (I)</u>

Rainfall intensity is directly related to both the duration of the storm and the return period (frequency) of the storm event. The Georgia Stormwater Management Manual (GSMM) provides rainfall intensity information for 16 locations across Georgia shown in Fig. 5.8. Table 5.4 shows rainfall intensity data for a one-hour storm duration for 50 year and 100 year frequencies at these 16 sites. For areas not tabulated in Table 5.4, the rainfall intensity charts shown in Figs. 5.9 and 5.10 can be used to extract site-specific intensity data for 50 and 100 year events.

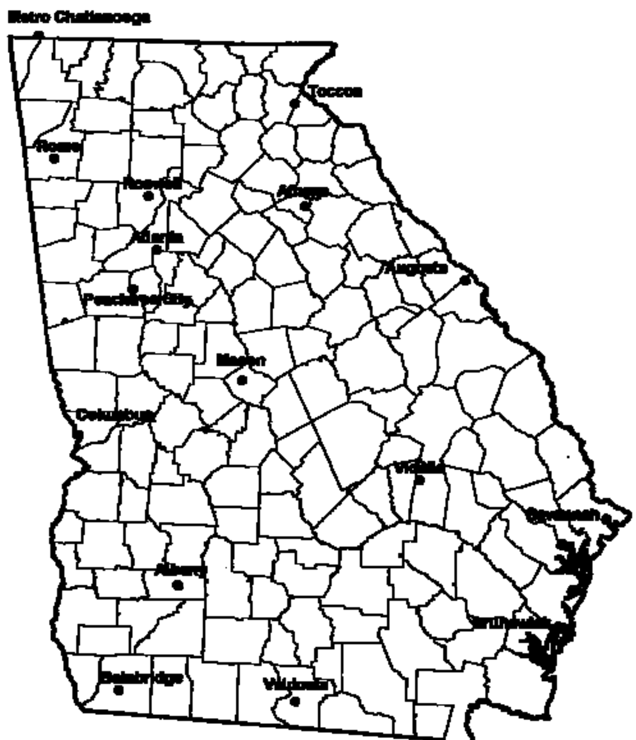


Figure 5.8. Location of the 16 Sites Containing Rainfall Intensity Information (GSMM, 2001).

Table 5.4. Rainfall Intensity Information for One Hour Storms Across Georgia (GSMM, 2001)

Site	**Rainfall Intensity (in/hr.)**	
	50 yr.	**100 yr.**
Albany	3.81	4.20
Atlanta	3.30	3.65
Athens	3.36	3.72
Augusta	3.20	3.50
Bainbridge	3.96	4.36
Brunswick	3.75	4.09
Columbus	3.56	3.93
Macon	3.58	3.95
Metro Chattanooga	3.06	3.38
Peachtree City	3.34	3.69
Rome	3.12	3.46
Roswell	3.25	3.59
Savannah	3.96	4.36
Toccoa	3.53	3.93
Valdosta	3.55	3.88
Vidalia	3.83	4.21

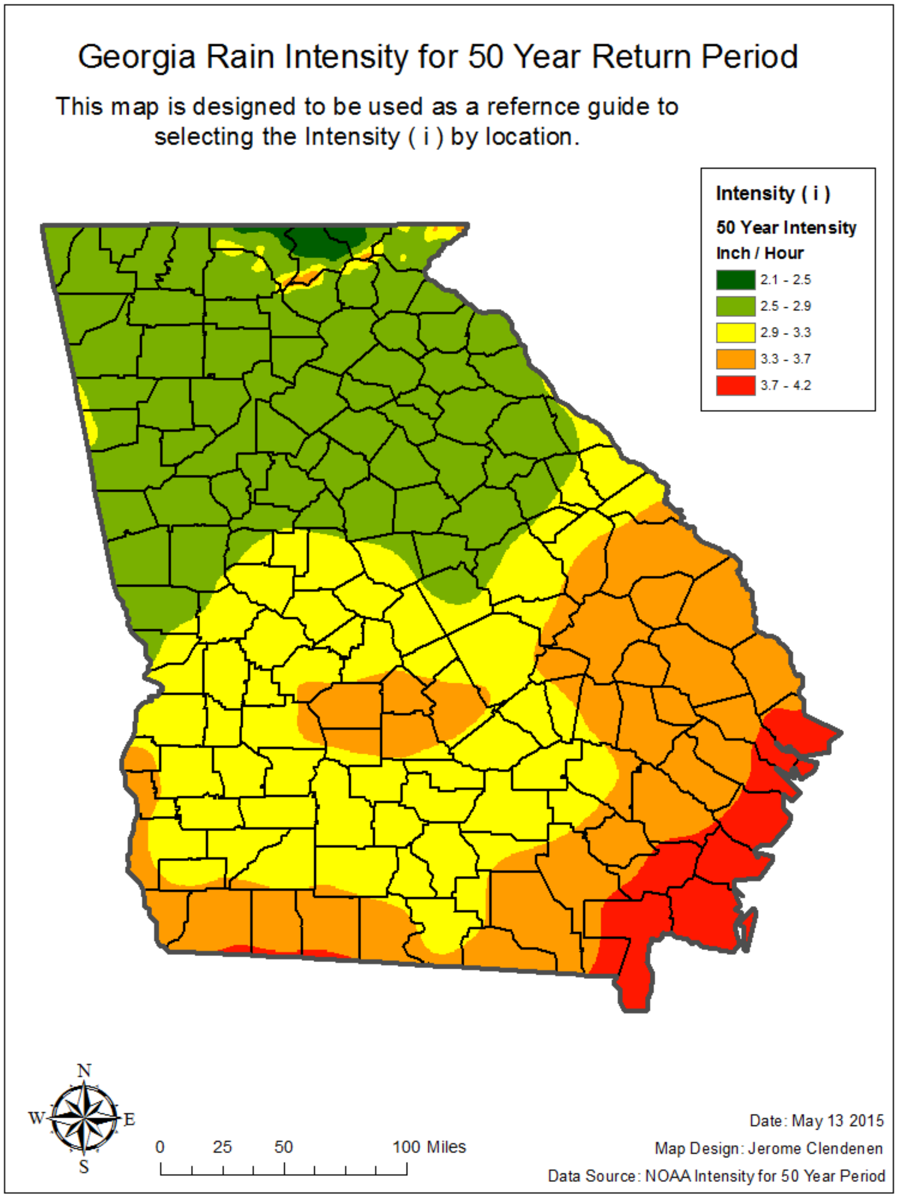


Figure 5.9. Georgia Rainfall Intensity Data for a One Hour Storm, 50 Year Return Period (NOAA, 2015).

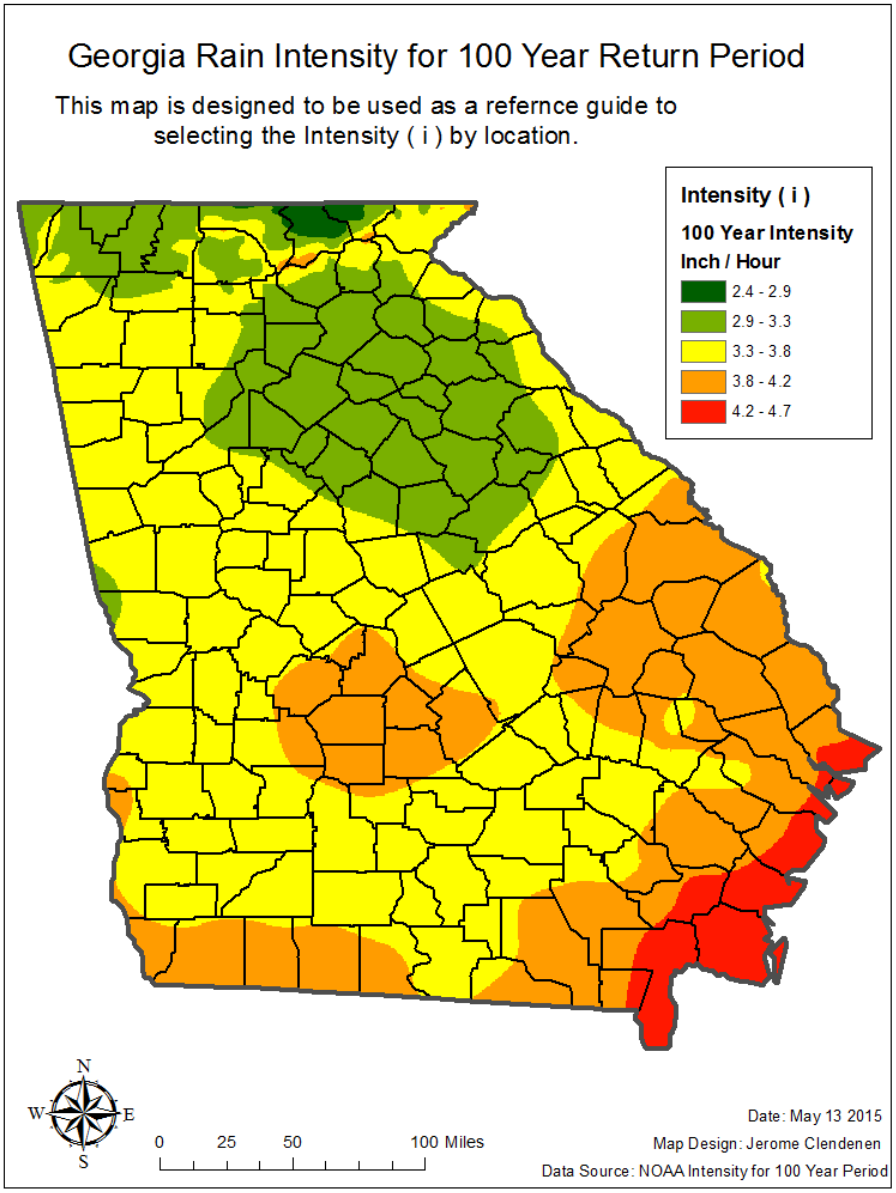


Figure 5.10. Georgia Rainfall Intensity Data for a One Hour Storm, 100 Year Return Period (NOAA, 2015).

Step 3: Computing Waterway Area:

Having computed the anticipated peak flow (runoff), the waterway area is computed using:

$$A = \frac{Q_p}{V}$$

where Q_p = peak flow (ft^3/s), and V = average velocity (ft/s).

For applications involving the flow of water overland or through some form of hydraulic channel, common velocities range from 3 ~ 5 ft/s. Compute a waterway area range by using both V = 3 ft/s and V = 5 ft/s. For bridge crossings, this is the required bridge opening area for the 50 year flood.

Step 4: Sizing Culverts:

Using Table 5.5 for pipe culverts and Table 5.6 for box culverts, the initial culvert size is determined based on the waterway area values computed for both V = 3 and 5 ft/s and then choosing the larger size. Note that the table does not include culvert diameters less than 18 inches since this is the minimum size specified by GDOT (2014). As an example, for a peak flow of 40 ft^3/s the computed waterway areas are 13.33 ft^2 (for V=3 ft/s) and 8 ft^2 (for V=5 ft/s) respectively. Based on these areas, Table 5.5 indicates required pipe culvert diameters of either 54 in or 42 inches, so select 54 inches for an initial sizing. For cases in which site restrictions (example: height from channel bottom to roadway) is not sufficient for a single large culvert, multi-barrel pipe culverts may be applied. Table 5.7 provides equivalent capacities for 2, 3, and 4 multi-barrels culverts.

Table 5.5. Pipe Culvert Sizing Table (Sizes Common for Corrugated Steel Pipe)

Culvert Diameter (inches)	Waterway Area (ft^2)	Diameter of Culvert (inches)	Waterway Area (ft^2)
18	1.767	78	33.183
21	2.405	84	38.484
24	3.142	90	44.179
27	3.976	96	50.265
30	4.909	102	56.745
33	5.94	108	63.617
36	7.069	114	70.882
42	9.621	120	78.540
48	12.566	126	86.590
54	15.904	132	95.033
60	19.635	138	103.869
66	23.758	144	113.097
72	28.274	---	---

Table 5.6. Box Culvert Sizing Table (American Concrete Pipe Association, 2015)

Dimensions ft x ft	Waterway Area (ft^2)	Dimensions ft x ft	Waterway Area (ft^2)	Dimensions ft x ft	Waterway Area (ft^2)
4 x 4	15.65	8 x 5	39.11	10 x 10	98.61
5 x 2	9.50	8 x 6	47.11	11 x 4	42.32
5 x 3	14.50	8 x 7	55.11	11 x 6	53.32
5 x 4	19.50	8 x 8	63.11	11 x 8	64.32
5 x 5	24.50	9 x 4	34.88	11 x 9	97.32
6 x 3	17.32	9 x 5	43.88	11 x 10	108.32
6 x 4	23.32	9 x 6	52.88	11 x 11	119.32
6 x 5	29.32	9 x 7	61.88	12 x 4	46.00
6 x 6	35.32	9 x 8	70.88	12 x 5	58.00
7 x 3	20.11	9 x 9	79.88	12 x 6	70.00
7 x 4	27.11	10 x 4	38.61	12 x 7	82.00
7 x 5	34.11	10 x 5	48.61	12 x 8	94.00
7 x 6	41.11	10 x 6	58.61	12 x 9	106.00
7 x 7	48.11	10 x 7	68.61	12 x 10	118.00
8 X 3	23.11	10 x 8	78.61	12 x 11	130.00
8 x 4	31.11	10 x 9	88.61	12 x 12	142.00

Table 5.7. Equivalent Capacities for Multi-barrel Pipe Culverts

Culvert Diameter	Equivalent to		
21 in.	2 x 18 in.		
24 in.	2 x 18 in.		
27 in.	2 x 21 in.	3 x 18 in.	
30 in.	2 x 24 in.	3 x 18 in.	4 x 18 in.
33 in.	2 x 24 in.	3 x 21 in.	4 x 18 in.
36 in.	2 x 27 in.	3 x 21 in.	4 x 18 in.
42 in.	2 x 30 in.	3 x 27 in.	4 x 21 in.
48 in.	2 x 36 in.	3 x 30 in.	4 x 24 in.
54 in.	2 x 42 in.	3 x 33 in.	4 x 27 in.
60 in.	2 x 48 in.	3 x 36 in.	4 x 30 in.
66 in.	2 x 48 in.	3 x 42 in.	4 x 36 in.
72 in.	2 x 54 in.	3 x 42 in.	4 x 36 in.
78 in.	2 x 60 in.	3 x 48 in.	4 x 42 in.
84 in.	2 x 60 in.	3 x 48 in.	4 x 42 in.
90 in.	2 x 66 in.	3 x 54 in.	4 x 48 in.
96 in.	2 x 72 in.	3 x 60 in.	4 x 48 in.
102 in.	2 x 78 in.	3 x 60 in.	4 x 54 in.
108 in.	2 x 78 in.	3 x 66 in.	4 x 54 in.
114 in.	2 x 84 in.	3 x 66 in.	4 x 60 in.
120 in.	2 x 90 in.	3 x 72 in.	4 x 60 in.
126 in.	2 x 90 in.	3 x 78 in.	4 x 66 in.
132 in.	2 x 96 in.	3 x 78 in.	4 x 66 in.
138 in.	2 x 102 in.	3 x 84 in.	4 x 72 in.
144 in.	2 x 102 in.	3 x 84 in.	4 x 72 in.

5.6 BRIDGE FOUNDATION INVESTIGATION AND SCOUR

For applications in which a bridge crossing is required, a BFI is necessary for the foundation design. The BFI must be performed by a licensed geotechnical engineer in the State of Georgia and contains information collected from borings taken at several points along the proposed bridge location. Borings normally contain information relating to soil types, depth to groundwater, presence of any impenetrable layers (bedrock), and the soil's permeability to water. Aside from providing this baseline information, the BFI also provides design recommendations for different types of foundations including:

- Driven piles for pile bents;
- Caissons (drilled shafts);
- Spread footings; and
- Pile footings.

In addition to using information relating to the geotechnical (soil) properties of the bridge site, the bridge foundation design must also take into account the effects of scour. Scour occurs when flowing water removes sediment such as sand a rocks, from bridge abutments or piers. The US Department of Agriculture (USDA) reports that scour is the most common cause of highway bridge failures in the United States (USDA, 1998). GDOT addresses the issue of scour by specifying that foundation depths be based on a 500 year storm event. This depth, referred to the scour depth, can be adjusted based on the findings from the BFI (GDOT, 2015). An estimation of the flow associated with the 500 year storm event can be calculated using the USGS regression equations shown in Table 5.1.

CHAPTER 6. CONCEPTUAL COST ESTIMATES

The effort for conceptual cost estimates was concentrated on the unit installed cost approach which combines the cost for time, equipment, manpower, materials, general and project-specific overhead, contingency and profit. A couple of case studies were identified in the literature and the main concern for ABC construction cost estimation was higher cost associated with the projects. This result in higher bid prices due to the complexity of the project and the severity of the time constraints imposed on the contractors. Cost analysis of bridge projects built under Highways for Life (HFL) program, cost premium for deploying ABC ranged from 6 to 21% higher when compared to cost of traditional bridge construction. Also, other cost items that might need to be accounted for would be ancillary items, like railroad flagging, resident engineering time, traffic control devices. Most agencies employ phase-based construction in order to keep traffic flowing through a work zone. Alternatives are considered to close the roadway, establish a detour and build the bridge quickly using ABC method. The additional costs for this process may be offset by the elimination of phased-construction costs. As more ABC projects are built, the costs are trending downward due to construction familiarity with the process which results in lower risks. Road user costs in the work zone are added vehicle operating costs and delay costs to roadway users resulting from construction, maintenance, or rehabilitation activity. They are a function of the timing, duration, frequency, scope, and characteristics of the work zone, the volume and operating characteristics of the traffic affected, and the cost rates assigned to vehicle operations and delays. Overall the conceptual cost estimates are mainly based on specific design, known site conditions, major equipment costs and the probable calculation of user cost savings.

The decision-making matrix described in Chapter 3 and provided in Appendix F may help LGs decide whether to use ABC or conventional construction methods or alternative contracting mechanisms, depending on available funding. A few examples of project costs were initiated corresponding Federal and State requirements to guide LGs in their cost estimation activity; also a short survey was deployed related to conceptual cost estimating activity and major factors impacting cost components for ABC in order to find out how other state DOTs assess cost and its components for ABC. In the followings, contractor cost concerns, cost options, road user costs, and cost accounting options are considered and provided as assistance for local governing bodies in their estimating activity.

6.1 CONTRACTOR COST CONCERNS

Some agencies that have expressed concerns over cost, indicate that they do not see a need to spend extra funds to minimize impacts to the public. In fact, these agencies are already spending additional funds for this purpose. Most agencies employ phased (staged) construction in order to keep traffic flowing through a work zone. It is well known that this construction method is more expensive than construction with a full road closure. The contractors are required to work in a small work zone with adjacent traffic that impedes the work in progress. This approach can increase the cost of the construction. ABC allows owners to take reductions in traffic impacts to the next level by providing even better customer service. In some cases, it may be preferable to close the roadway, establish a detour and build the bridge quickly using ABC. The additional costs of this process may very well be offset by the elimination of phased construction costs. As more ABC projects are built, the costs are trending downward due to construction familiarity with the process which results in lower risks. Risk equates for higher cost in the project.

6.2 COST OPTIONS

The primary concern for ABC construction cost estimation is the higher cost associated with the projects. This has resulted in higher bid prices due to the complexity of the project and the severity of the time constraints imposed on the contractors. Based on the cost analysis of eight bridge projects built under HFL program, the cost premium for deploying ABC ranged from 6 to 21% higher when compared to the cost of traditional construction. Other cost options that might need to be accounted for would be Ancillary Items. Examples of this would be: railroad flagging, police details, resident engineering time, traffic control devices, etc.

6.3 ROAD USER COSTS

Road user costs are costs incurred by a highway network when they are delayed due to construction activities. Road User Costs in the work zone are added vehicle operating costs and delay costs to highway users resulting from construction, maintenance, or rehabilitation activity. They are a function of the timing, duration, frequency, scope, and characteristics of the work zone; the volume and operating characteristics of the traffic affected; and the dollar cost rates assigned to vehicle operations and delays. Daily road user costs (DRUC) are a measure of the daily financial impact of a construction project on the traveling public. The major factors in calculating user costs are out-of-distance travel (OODT), average annual daily traffic (AADT), and average daily truck traffic (ADTT) on the bridge. The Iowa DOT uses the formula:

$$\text{DRUC (\$)} = (\text{AADT} + 2 * \text{ADTT}) * (\text{OODT}) * (\text{Mileage Rate})$$

6.4 SAFETY COSTS

Safety costs are estimated based on crash history, exposure times, and traffic maintenance strategies (if the information is available).

6.5 LIFE CYCLE COST ANALYSIS

A process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future costs, such as maintenance, user costs, reconstruction, rehabilitation, restoration, and resurfacing costs, over the life of the project segment. Higher quality reduces the need for maintenance and extends the lifespan of the structure. This will lead to a reduced life cycle cost for prefabricated structures.

6.6 COST ACCOUNTING OPTIONS

The unit cost approach is a method of accounting that combines the cost for time, equipment, manpower, materials, general and project-specific overhead, contingency and profit.

Cost Based (Bottom-Up Estimating) is a method of accounting that takes into consideration production rates, equipment needs, and manpower for each construction operation.

Appendix E is providing a couple of examples on the conceptual cost estimates with respective major categories of direct costs in both conventional and prefabricated construction. Another example was obtained from an experienced contractor working in one of the northern state and it is a detailed report of cost break down by individual construction items entered the respective bridge replacement job. The job estimate report is presented as a guidance in exhibit E3 of Appendix E.

This section of conceptual cost estimates has a focus in guiding users on which circumstances are most appropriate to use ABC versus conventional bridge construction. Additionally, information on the decision making process for ABC selection can be essentially filtered through the decision-making matrix compiled in Chapter 3. A special survey with essential questions on cost estimation for ABC projects (provided in Appendix E) was deployed to all DOTs that were associated, in multiple occasions, with this type of bridge construction. Separate submissions of the survey were also reaching all other state DOTs. In this final report and to this date, a number of sixteen DOTs responses have been recorded. The effort was closely monitored and a number of iterations of the survey were used in the submission and answers collection process. The results can be used as further guidance to potential users and to complement the decision-making matrix with inclusive items affecting directly the conceptual cost estimates. All the state DOTs responses were recorded in the table belonging to Appendix E, as exhibit E4.

CHAPTER 7. TYPICAL CONSTRUCTION PRACTICES

This chapter presents an overview of ABC practices. Over the years, ABC techniques using PBES have become more and more popular throughout the United States. The desire to reduce traffic impacts for commuters has been a high priority in the recent years. Consequently, in the last 20 years, acute traffic control issues at specific job sites has catalyzed the development of what is now known as ABC.

Early ABC projects focused on specific prefabricated elements such as bridge decks and/or pier caps. Bridge deck construction using full depth precast concrete deck panels has been in use for over 20 years. In recent years, use of PBES has spread to all bridge elements, including substructures and foundations. As structural components of a bridge that are built off site, they are crucial strategies to meet ABC objectives. Combining them with the "Fast-Track Contracting" method can generate a fast, high-performance project. PBES components include, but are not limited to:

- Precast footings;
- Precast wing walls;
- Precast pile foundations;
- Prefabricated caps and footings; and
- Prefabricated steel/concrete girder beams.

Benefits of ABC projects using PBES are:

- Fewer problems for reduced road user impacts;
- Improved worker and motorist safety;
- Expedited project planning process;
- Improved quality/constructability; and
- Reduced cost to society.

7.1 ABC CONSTRUCTION CONCEPTS

7.1.1 Prefabricated Spread Footings

Precast concrete spread footings are a relatively new concept in bridge construction. These footings were placed over a prepared subgrade on leveling bolts and then grouted into place. The joints between the footing elements are made with shear keys that are filled with non-shrink grout. Although the size of footings for bridge piers can get rather large, the columns for 40, 60, and 80 ft bridge spans will allow for reasonably sized footings which will facilitate their transportation. Reinforcing bars can be extended from the precast footings, and a CIP closure pour can be completed during the erection of the remaining portions of the bridge. For compact bridges, smaller footings can be placed under the columns and extended as the pier structure is being constructed. See Fig. 7.1.

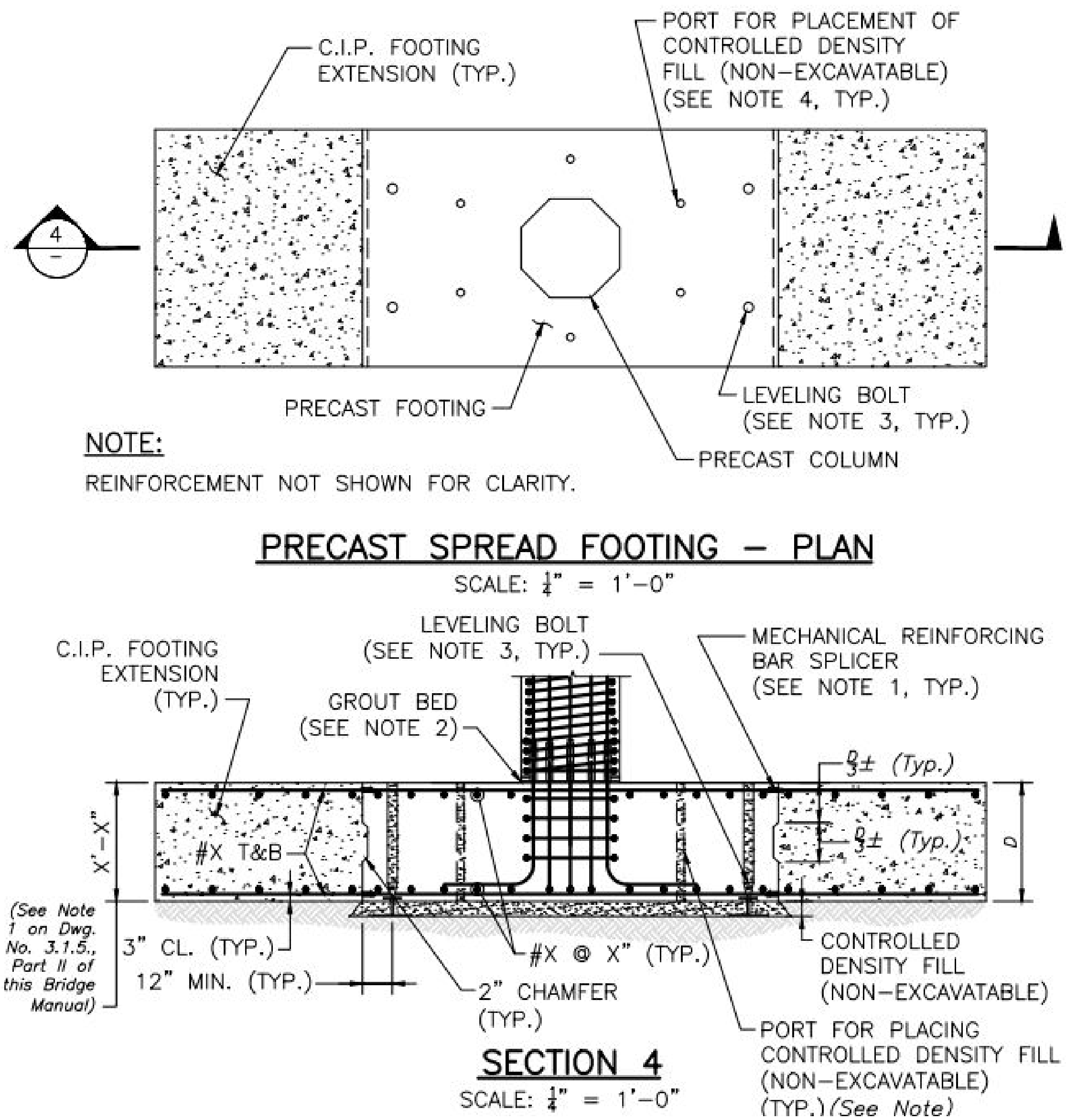


Figure 7.1. Example of Precast Spread Footing Plan and Section (MassDOT, 2013).

7.1.2 Precast Pile Cap Footings

Precast components can be used for concrete pile caps. Recently, new design details have been developed using corrugated steel pipe voids that are based on research completed in Iowa. Results showed that a void made in a precast concrete footing with a corrugated steel pipe can provide very large punching shear resistance. Research on seismic connections has also demonstrated that these voids can develop large moment resistance as well.

7.1.3 Modular Block Systems

Modular block retaining walls are a form of gravity retaining wall in which CIP concrete is replaced by modular reinforced concrete modules that interlock to form a wall (See Fig. 7.2). The mass of the wall and, sometimes, the mass of soil placed within the voids in the blocks. These walls are beneficial to ABC because of the ability to construct them off site and be transported to the site.

Figure 7.2. Modular Block Systems (photo: Redi-Rock.com and ReinforcedEarth.com).

7.1.4 Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS)

This method combines foundation, abutment, and approach embankment into one composite material. GRS is comprised of many thin layers of soil and geosynthetic reinforcement. The internal soil is retained at the face of the abutment with a high quality concrete block facing. The facing simply retains and prevents erosion of the soil near the face of the wall. The composite mass extends into the embankment which allows the abutment, superstructure, and approach into one unit, and eliminating the differential settlement between the abutment seat and the approach backfill, otherwise known as *the bump* (Fig. 7.3). This integrated bridge system does not require approach slabs and, consequently, can drastically save the time associated with forming, pouring, and curing concrete for the approach slab.

GRS carries all loads and movements of the superstructure allowing this method to be utilized with concrete precast slabs. This system can be used for 40, 60, and 80 foot bridge spans and presents the following benefits:

- Low initial cost;
- Low life cycle cost;
- Fast construction;
- Minimal installation labor and equipment; and
- No approach slabs needed.

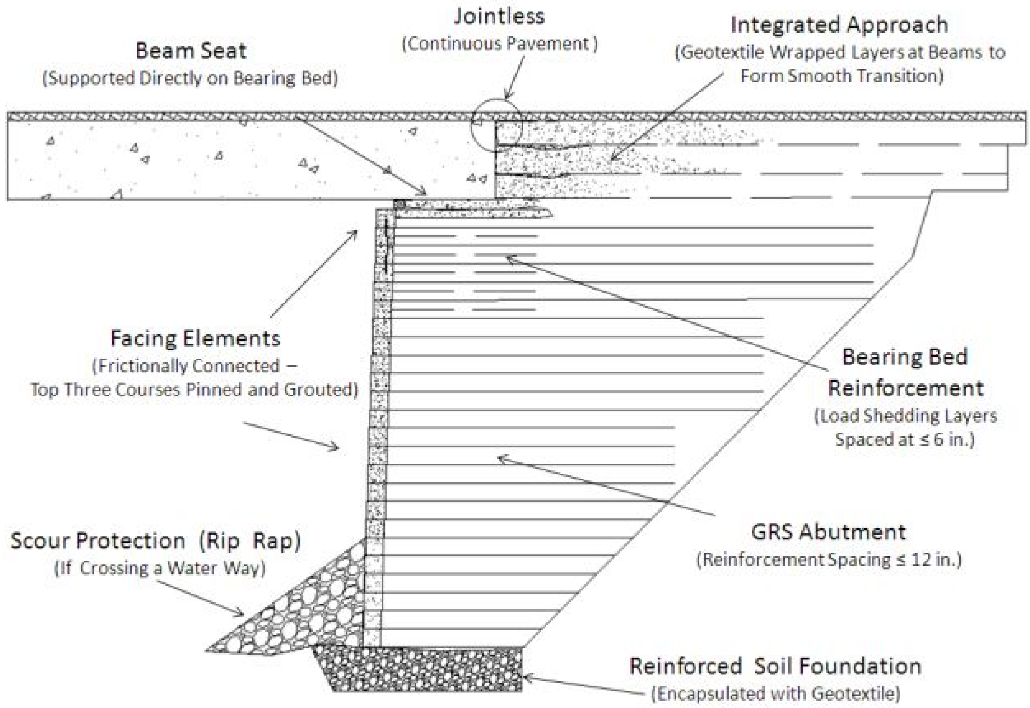


Figure 7.3. Typical GRS-IBS Cross Section at the Bridge Abutment (Adams et. al, 2012).

7.1.5 Expanded Polystyrene (EPS) Geofoam for Rapid Embankment Construction

EPS geofoam is an embankment fill system comprised of large lightweight blocks (1-2 pounds per cubic foot) of expanded polystyrene. This system is not intended to be a structural support system for the bridge abutment. EPS is used to support bridge abutments by placing the blocks behind a conventional abutment or around piles of integral abutments (See Fig. 7.4). A layer of subbase is required below the pavement to distribute wheel loads. The benefits are:

- Fast construction;
- Extremely lightweight; and
- Eliminates pre-load settlement times.

(a) (b) (c)

Figure 7.4. (a) Bridge abutment with geofoam backfill. (b) EPS geofoam in embankment fill. (c) EPS geofoam for embankment widening (Bartlett et. al., 2000).

7.1.6 Abutments or End-Bents

Abutments are the substructure at the end of a bridge span on which the superstructure rests. Single-span bridges have abutments at each end, which provide vertical and lateral support for the bridge, as well as acting as retaining walls to resist lateral movement of the earthen fill of the bridge approach (Fig. 7.5). Multi-span bridges require piers to support ends of spans unsupported by abutments.

(a) (b) (c)

Figure 7.5. (a) Location of abutments at each end of the bridge (image from Benchmark Hunting Wiki). (b) Integral abutment placed behind a mechanically stabilized earth (MSE) wall (Hailat, 2014). (c) Partial precast end abutment (Hailat, 2014).

7.1.7 Prefabricated Superstructure Elements

AASHTO defines a bridge's superstructure as "Structural parts of the bridge that provide the horizontal span." The superstructure can also be defined as the portion of the bridge above the bridge bearings. Superstructure systems include both the deck and primary supporting members integrated in a modular manner such that mobility disruptions occur only as a result of the system being placed. These systems can be rolled, launched, slid, lifted, or transported in place, onto existing or new substructures (abutments and/or piers) that have been built in a manner that does not impact mobility (Fig. 7.6).

Figure 7.6. Prefabricated Superstructure Elements (SHRP2, 2014).

7.1.8 Materials for Prefabricated Bridge Elements

This section provides an overview of the many different types of materials used in prefabrication and discusses the impact of the materials on accelerated construction processes. Most agencies have approval processes for prequalification of proprietary materials. Designers should be aware of the materials that are available for use in a particular agency prior to specifying the material.

Structural Steel

- Steel elements are well suited for prefabrication and accelerated construction. There is a high degree of control over fabrication tolerances, so using steel for connection systems can become easier to use than concrete. Common elements include steel beams and girders, steel grid decks, and steel railings.
- An advantage steel possesses over precast concrete is that it typically weighs less than an equivalent concrete element. This is a critical factor to look at when considering shipping and lifting capacities.
- Steel has the ability to handle large stress reversals. SPMT could be used to move prefabricated large components. SPMT bridge moves typically induce large stress reversals during lifting and transport that some prestressed concrete beams cannot stand.

Ultra-High Performance Concrete (UHPC)

- High performance concrete has emerged in the ABC market in the recent years and combines high quality cement and stone products along with steel or organic fibers. UHPC can achieve very high compressive strengths of 18,000 to 33,000 psi. Not only is it very high in compressive strength, but also very high in flexural strengths between 900 and 7000 psi.
- One downside to using UHPC is the high cost of the material compared to the conventional high performance concrete. UHPC has been successfully used on several bridges in the United States for girders and decks on an experimental basis. Even with its high cost, UHPC has a high potential for use in ABC. The high compressive and tensile strength makes the product ideal for closure pour connections between adjacent elements.

Concrete (Normal Weight and Lightweight)

Concrete is a popular and versatile material for prefabrication and ABC projects. The ability to build elements off site, in virtually any shape, makes this material a prime choice for designers. Common prefabricated concrete elements include beams and girders, full depth deck panels, and pier caps. Several states have built entire bridges using precast concrete elements including pier columns, abutment stems, footings, and retaining walls. Concrete can also be used for making connections between different prefabricated bridge elements. These connections often require the use of high early strength concrete to allow for accelerated construction processes.

Durability is a major concern. The new generation of HPC offers durability that exceeds the performance of past concretes. Plant produced precast concrete also has the advantage of being constructed in a controlled environment with higher production and curing standards than normally found in the field. This benefit of higher quality materials of accelerated construction projects is often overlooked or undervalued by designers.

One obstacle to the use of prefabricated concrete elements is the shipping and handling weight. One way to reduce the weight of the elements is to cast voids in the elements during prefabrication. Once in place, the voids can be filled with concrete. The voids can also be used to make connections between elements. A common cost effective way to make voids is to use corrugated steel pipe to form the void. The corrugations are very effective at transmitting very large forces.

Concrete link-slab technology has also been used on ABC projects at bridge piers to make a jointless bridge without live load continuity. The concept with link slabs is to design the connection of the deck across the pier to accommodate the rotation of the beams without significant cracking. This is done by debonding a small portion of the deck near the pier to allow for a wider spread of the rotation strain. That portion (link) is filled with a highly ductile engineered cementitious composite (ECC) material.

Fiber-Reinforced Polymer (FRP)

There has been much research into the use of FRP in recent years. Many states and universities have experimented with these materials. The development of high strength polymers has made the use of FRP materials practical for many bridge applications

7.2 APPLICATION EXAMPLES OF ABC CONSTRUCTION TECHNOLOGIES

ABC is most effective for projects that require traffic management. This section will cover how ABC can be applied to the various types of bridge projects that transportation agencies typically manage.

7.2.1 Rehabilitation of Existing Bridges

The national bridge inventory is aging; many bridges have deteriorated significantly, especially their superstructure, and concrete deck replacement projects are becoming more common. The supporting girders also experience deterioration due to leaking bridge joints and lack of maintenance.

The substructure is often in better condition when compared to the superstructure. This is especially true for single span bridges and continuous span bridges without deck joints. On many projects, rehabilitation of the substructure combined with replacement of superstructure elements is feasible. The following sections discuss how ABC can be used for the execution for bridge rehabilitation projects.

7.2.2 Deck Replacement

Deck replacement is the most common use of ABC. The installation of a bridge deck is time consuming, requiring significant manpower to form it, place reinforcing, cast and cure the concrete, and strip the forms. This labor intensive work is difficult under the best circumstances, but where traffic management is required, it becomes even more complicated.

Three basic forms of ABC deck replacement strategies have been used in the United States. The first is stay-in-place deck forms. They consist of corrugated metal panels designed to support the reinforcing steel and wet concrete of the deck and eliminate the need to strip the forms after the concrete is cured. However, they still require the placement of reinforcing steel and casting and curing of concrete, which does not result in a significant time savings. Also, the underside of the deck cannot be visually inspected in the future.

The second ABC deck replacement strategy is the use of precast and prestressed concrete partial depth deck panels. These panels are typically cast to half the thickness of the finished deck. The remainder of the deck is made up of one layer of steel reinforcement and on-site cast concrete. The panels are designed to span from girder to girder with reinforcement designed to accommodate the positive deck bending moments. Negative deck bending moments are accommodated by the top layer of field placed reinforcement. The advantages and disadvantages of this system are similar to stay-in-place deck forms except that the panel is a structural part of the deck, and is exposed for future visual inspections.

The fastest form of deck placement uses full depth prefabricated deck panels. Different systems have been used in the United States including open grid steel, exodermic deck panels, fiber reinforced polymer panels, and precast concrete panels.

Most ABC projects use precast panels, and they are the focus of this construction manual. Significant research has examined them, including the composite connections between the panel and the girders and the connection between the panels.

7.2.3 Superstructure Replacement

The use of ABC for superstructure replacement projects is very common. ABC techniques are particularly well suited for them since the time consuming process of building foundations and substructures is not required. Each state has its own technique to accomplish this type of work in a very short time frame.

Figure 7.7. Self-propelled Modular Transporters (AASHTO, 2006).

Several states have used SPMT technology or lateral skidding/sliding technology to remove and install entire bridge superstructure systems (Fig. 7.7). The new superstructures can be built off site and moved into position in only a few hours.

7.2.4 Substructure Replacement

On some bridges, the substructure may be in disrepair due to leaking deck joints and spray from vehicles passing underneath. If abutments have deteriorated, full bridge replacement or abutment patching may be necessary because replacing an abutment is difficult without significant disruption to adjacent areas.

Accelerated replacement of pier columns and caps has more potential. Old pier columns and caps can be removed and replaced with prefabricated pier elements if the footings and foundation are in sound condition and structurally adequate. Closure pours at the base of the columns can be used to connect the old footings to the new pier elements. If an existing pier is supported on a spread footing, the new pier can be built alongside it on rails and jacked into place in a method similar to lateral superstructure skidding/ sliding. Once in place, the footing can be underpinned with grout to seat it on the subgrade.

7.2.5 Replacement of Existing Bridges and New Bridges

Replacing entire bridges and building new bridges differ from deck and superstructure replacement projects because the substructures and foundations must be replaced, which adds a level of complexity. However, ABC methods are often still appropriate.

Replacing a bridge is also different from constructing a new bridge on a new alignment. Often, traffic needs to be accommodated, which requires building the bridge amid and around the traffic or a traffic detour. ABC can minimize the impact of both scenarios. The following sections describe the role that ABC can play in several replacement strategies.

7.2.6 Staging

On many projects, traffic needs to be maintained through the construction site at various stages if a suitable detour is unavailable. ABC can be used to minimize the duration of each construction stage. In some cases, complete superstructure prefabrication may not be possible in stage construction projects. Individual prefabricated elements can be used for all portions of the substructures and superstructures.

On any project where staging is being considered, the project team should investigate the potential for changing from staged construction to ABC with full closure with a detour. This approach will offer the greatest opportunity for accelerating the construction since the contractor will have full access to the site for manpower and equipment. It also offers the safest work zone for workers and inspectors.

7.2.7 Full Closure and New Construction

On many projects, the entire construction site can be made available to a contractor for the construction of an entire bridge. This occurs on new bridges being built on new roadway alignments, and on bridges that are built with traffic detours.

The previous section outlined the benefits of ABC on projects with detours. Full closure of the bridge offers significant benefits to the ABC process. The use of ABC on projects that involve detours is appropriate if the detour is long or has undesirable geometrics. ABC can be used to minimize the length of time that the detour is in place. Virtually any bridge can be built using ABC methods; however, certain bridge features can have an effect on the ABC process.

7.3 ABC CONSTRUCTION TECHNOLOGIES

SHRP2 (2014) provide ABC concepts and sketches for the following technologies:

- Above-deck driven carrier systems;
- Launched temporary truss bridge;
- Wheeled carriers or SPMTs;
- Launching and lateral sliding; and
- Jacking and mining.

This section summarizes these ABC technologies and their advantages and disadvantages to help decision-makers choose the ideal technique for a specific project.

7.3.1 Above-Deck Driven Carrier (ADDC)

Above-deck driven carriers (ADDCs) ride over an existing bridge structure to deliver components. They are ideal where access below the bridge for cranes is limited or the bridge's span is long.

ADDC can be delivered to the construction site shipped on flatbed trucks or towed using mountable axles. Once at the site, they are erected with multiple-axle configurations; after reaching the destination pier, the ADDC is raised to unload the axles. It is then secured and supported at the pier, loaded with gantries, and ready to demolish structure or to deliver girders and slab panels.

On narrow bridges, the ADDC can remove and replace half of the existing structure using counterweights on the gantries. Next, the ADDC can be repositioned over the new half to remove and replace the remaining old half. For shorter bridge lengths, ADDC can be used to provide access from abutment to abutment. For longer bridge lengths, ADDC can be used to provide access over a number of spans concurrently, to allow for complete removal and replacement of the exterior portions of multiple spans of an existing structure.

ADDC may not be ideal for highly curved bridges. The stability of the gantry systems must be studied for the weights of the existing slab panels and girders as well as the weights of the new girders and slab panels.

7.3.2 Launched Temporary Truss Bridge

Launched temporary truss bridges (LTTBs) are used to transport girders, materials, or equipment over spans, especially when:

- Minimal disruption to traffic is desired,
- Traditional crane access and picks are limited, or
- Temporary access over waterways is restricted.

LTTBs can be shipped to sites on flatbed trucks or towed with mountable axles. They are erected on site and launched; after reaching the destination pier or temporary bent, they are secured and ready to deliver girders and equipment.

7.3.3 Self-Propelled Modular Transports (SPMTs)

SPMTs can be used to remove entire spans or full-length span strips of existing bridges for replacement with new units. They are cost-effective, easily transported, and can be set up and taken down quickly. Trolley movement and steering are governed by hydraulic motors powered by diesel engines. For highway bridge applications, the spans are lighter when made with prestressed concrete and much lighter when made from steel girders and a concrete deck slab. The spans can also be divided into longitudinal strips to diminish the weight to be lifted and the cost of the wheeled carriers.

7.3.4 Launching and Lateral Shifting

This method involves building a bridge at a single construction location and launching it incrementally as each section is completed. Prestressed concrete bridges are constructed in a small casting yard behind an abutment. The first bridge segment is equipped with a light steel extension to control the launch stresses. The segment and the steel extension are launched forward onto the piers until it clears the formwork. A second bridge segment is match cast and prestressed against the first one, and the entire bridge section is launched again. This process is repeated until the bridge completed. Similar operations can be applied to steel girder bridges, where the form is replaced with adjustable supports that sustain the girder segments during assembly.

CHAPTER 8. ABC TOOLKIT

Our proposed toolkit has three unique, superior features over other ABC toolkits:

- Extensive
  The proposed toolkit covers, not only design and construction components, but also risk analysis and cost estimation (Table 8.1). These extensive contents assist owners, designers, contractors, and decision-makers to make appropriate decisions, and start ABC projects without any additional resources.

- Convenient
  Enhanced and detailed design examples minimize the need for other design aides such as finite element programs or structural analysis software by providing additional Mathcad design aides to calculate design loadings on superstructures, which help pre-design ABC bridges conveniently.

- Current
  The proposed toolkit includes current state-of-the-art development of ABC applications through comprehensive literature reviews and latest surveys.

Table 8.1. Comparison of the SHRP2 R04 ABC Toolkit and the Proposed Toolkit

ABC Components	Proposed Toolkit	Existing SHRP2 R04 Toolkit
Decision-Making Tool	• Decision-making matrix • Decision-making flowchart	N/A
Design	• Design concepts • Design examples & aides	• Design concepts • Design examples • Design specifications (recommendation)
Construction	• Construction guidelines • Construction flowcharts	• Construction concepts • Construction specifications (recommendation)
Risk Analysis	• Risk analysis guidelines • Interactive flowcharts	N/A
Cost Estimates	• Cost estimates guidelines • Examples of cost estimates	N/A

The Components will include the followings:

- ABC decision-making Component
- ABC design Component
    - Design Concepts
    - Pre-design Examples
    - Design Aides (Mathcad Examples) to calculate design loadings (moments, shears, and reactions)
- ABC Construction Component
- Hydrological and Hydraulic Component
- Conceptual Cost Component

The toolkit will be provided in a word-processor format with hyperlinks to referenced sites, or the team will create a template for a web-based system to facilitate ABC design implementation; a final version of the web site can be built from the template. The web site and its server will contain links to the main design documents required for the proper selection of accelerated bridge techniques and a user form generated by a software application for cost analyses.

CHAPTER 9. SUMMARY AND CONCLUSIONS

Accelerated Bridge Construction (ABC) techniques are highly effective in remediating traffic disruptions during bridge renewals, promoting traffic and worker safety, and improving the quality and durability of bridges. However, their higher initial cost has prevented widespread and sustained implementation.

Our survey results revealed that local contractors prefer conventional CIP construction for bridge renewals because modular construction cuts into profits. Furthermore, designers hesitate to risk using new technologies. However, comprehensive national studies of ABC practices found that accumulated experience and repeated use could lead to contractor acceptance as well as savings in construction costs and time.

The primary objective of this study was to develop and deliver a toolkit for accelerated selection and construction of bridges in place using prefabricated modular systems with 40, 60, and 80 ft span lengths for LGs in Georgia. The components of the proposed ABC toolkit address: 1) decision-making; 2) design; 3) construction; 4) risk analysis; and 5) cost estimation. It will be an extensive, convenient source of the latest guidelines for ABC applications. It is not intended for developing final design and construction plans but as a source of information to help decision-makers and owners develop an initial design, estimate the material and construction costs, and determine when and where ABC will be most beneficial. It will provide guidelines to assist LGs and third-party designers in employing GDOT design standards for ABC. With repeated implementation, ABC options will become even more economical and efficient.

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LIST OF USEFUL ABC WEBSITES

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APPENDIX A

SURVEY RESULTS

APPENDIX A - SURVEY RESULTS

A1- DOT Surveys

State	Past experience with ABC	Level of Acceptance of ABC	Whether projects engineered by DOT's or Contractor	Impediments to ABC employment	Availability of Standardized elements	Ongoing or recent projects	Ongoing or recent research
Alabama	Yes, one project in the past 5 years	Low, doubtful for typical bridges but may be successful for long structures that require substantial repetition of elements	Contractor	lack of man power, elevated cost	N/A	None	four systems of rapid deck replacement on structures in the northern region of the state
Alaska	Yes, several projects in recent years	Moderate	DOT	High costs, lack of experience, lack of support	Standardization would help but training and education would also be required	Most new bridges in Alaska incorporate ABC features.	Most recently, research on an all-steel bridge pier system
Arizona	Yes, one project completed	Moderate	DOT & Contractors	Connection issues, lack of funding, lack of experience	Standardization would help with decision making	Bridge replacement project utilizing GRS-IBS and a bridge slide	Pursuing research in using UHPC for bridge connections.
Arkansas	None	Should be done when it's possible to reduce costs	N/A	N/A	N/A	N/A	None
California	Yes, 8 projects in the past 5 years	Strong acceptance among engineers when funding is available	DOT	Seismicity, suitable staging in urban areas, cost	Standardization would further encourage ABC	I - 40 Marble Wash Bridge, Oakland Bay Bridge	seismic performance of precast elements
Colorado	Yes, 73 projects that utilize ABC	High acceptance	DOT & Contractors	High cost	Standardization would further encourage ABC	Projects are always ongoing and ABC is a primary consideration	None
Connecticut							
Delaware	Yes, less than 10 in the past 5 years	Good acceptance	DOT & Contractors	Higher costs, extended hours	N/A	None	None
Florida	Yes, multiple projects in the past 5 years	Not adopted as a standard method but is considered for every project	Contractor	Lack of staging space for SPMTs, inexperienced contractors, traffic maintenance, costs	Elements are availale but contractors avoid precasting since they make profits placing steel and concrete	Graves Avenue Bridge (SPMT usage)	None

State	Past experience with ABC	Level of Acceptance of ABC	Whether projects engineered by DOT's or Contractor	Impediments to ABC employment	Availability of Standardized elements	Ongoing or recent projects	Ongoing or recent research
Georgia	Yes, 1 project in the past 5 years	Growing acceptance and interest	Contractor	Higher costs, extended hours	N/A	None	standardized prefabricated elements
Hawaii	Very experienced, 20 projects using ABC techniques since 2001	Very high acceptance, lower cost than cast in place methods	DOT	Initiatives by governing bodies encourage using ABC techniques just to use them instead of using ABC techniques where it most makes sense	Yes but ABC should be used when it is most economically efficient or environmentally beneficial	None	None
Idaho	2 ongoing projects		DOT		Used precast concrete deck panels	Northside Blvd I-84, Union Pacific Railroad E. Lateral Canal Bridge, Highway 75	
Illinois	Yes, multiple projects in the past 5 years	Low accpetance because of high cost	DOT	Higher costs	Would help cut costs	None	None
Indiana	Yes, 2 projects, one involved sliding	Low acceptance	DOT	Lack of knowledge, cost efficiency	Availability of standards would make ABC easier	One unspecified project	None
Iowa	Yes, multiple projects over the past 5 years	Good acceptance	DOT	Low traffic volumes, contractors say it's less profitable and more complex	Standar plans and shapes would ease the design process and save money	I-92 Cass County Bridge, US 6 Keg Creek Bridge (completely prefabricated)	None
Kansas	Yes, one project in the past 5 years	Unknown but a shift to ABC as a standard is doubtful	DOT	Methods other than design-bid-build are prohibited by state law	Kansas gets very low prices on cast-in-place short span bridges that are very low maintenance. Kansas is relectant to use standardized prefabricated elements	Project similar to Iowa's Keg Creek project (completely prefabricated)	precast concrete bridge elements
Kentucky							
Louisiana	Extensive experience with several ABC methods	Widely accepted and currently used	DOT	Precast bridges have a shorter service life than cast in place bridges	Common use of precast girders for short spans	Maree Michael Bridge and Creek Bridge (Vermilion Parish, LA)	None
Maine	Yes, several projects over the past 5 years	High accpetance	DOT	Cost of precasting elements as opposed to casting in place	Desired standardization for lower costs	None	None

State	Past experience with ABC	Level of Acceptance of ABC	Whether projects engineered by DOT's or Contractor	Impediments to ABC employment	Availability of Standardized elements	Ongoing or recent projects	Ongoing or recent research
Maryland	Yes, over 20 projects in the last 5-10 years	Good acceptance	Contractors	None; ABC is standard	Standardization may help but ABC is employed where it makes sense3	Prestressed slab deck replacement is routine	None
Massachusetts	Yes, one project completed in the past 5 years	Interested in pilot projects to gain familiarity with ABC techniques	DOT	Lack of familiarity due to inexperience	Desired to reduce customization	I-93 Fast 14 (Salem St, Boston)	None
Michigan	Some experience within the past couple of years	Moderate acceptance that could be improved on upon standardization	Contractors	Cost, constructability, quality/performance issues	N/A	None	None
Minnesota	Yes, 20 various projects	High	Contractors	Reduced time frame results in overworked and fatigued staff, decision making process	Stadardization may not help because all tools are created at home in the state	Full depth precast deck with superstructure lateral slide	NCHRP Project, determining tolerances for precast elements
Mississippi	Yes, several reconstruction projects following Katrina	Low acceptance, timidity about connection durability of precast columns, profitability of large precast elements,	DOT	Lack of regional consensus, doubts about joint and connection durability	Local fabricators would embrace new technologies if a commitment to a large number of projects was made	None	Joint and connection durability between precast elements
Missouri	Yes, multiple ABC methods have been applied.	Good acceptance	DOT	High cost, seismic durability issues	MSE wall abutments	New Mississippi River Bridge Crossing (St. Louis)	Innovations in substructure construction
Montana	None yet, considering a pilot project	Growing	DOT	Low traffic volume	N/A	Highway 89 Pondera County South Fork/Dry Fork Marias River Crossing	GRS-IBS
Nebraska	No experience but elements of bridge designs were accelerated	Not widely accepted; no need	Contractor	High cost, precast elements would need to get subcontracted	Standardization is seen as a way to reduce costs and increase the quality and durability of finished projects	N/A	use of precast deck panels, heavy lifting of remotely assembled superstructure
Nevada	Experience utilizing SPMT, bridge slide technique, and precast arches	Widely accepted when used for the right application	Contractor	Questionable efficiency	More durable ABC connection details would be beneficial because of Nevada's high seismicity	Unspecified projects involving GRS-IBS abutments and fully prefabricated superstrectures	None
New Hampshire	Yes, several projects completed but DOT interest is low	Accepted but contractors are reluctant to utilize it	Contractor	No real impediments, contractors don't want to hire subcontractors to precast	Extensive use of precast elements	Main Street Bridge (Epping, NH)	unspecified research being conducted at the University of New Hampshire

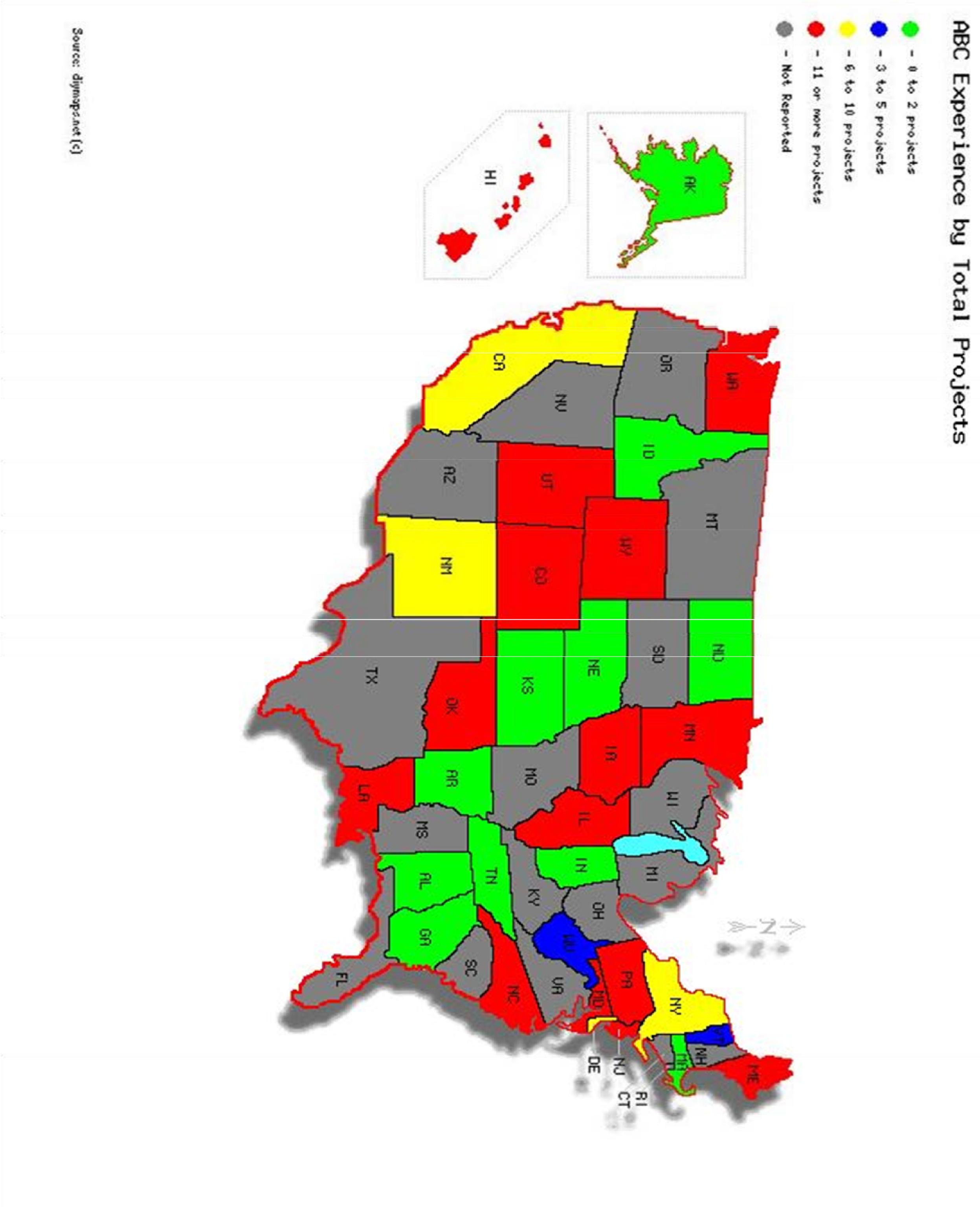
State	Past experience with ABC	Level of Acceptance of ABC	Whether projects engineered by DOT's or Contractor	Impediments to ABC employment	Availability of Standardized elements	Ongoing or recent projects	Ongoing or recent research
New Jersey	Many projects over the past 5 years	Good acceptance but high reluctance on the thesis that ABC is too high risk	DOT	DOT and contractors don't think of it to be a solution in many cases, no incentive to be creative	Precast elements used	Route 70 bridge over the Manasquan River,	None
New Mexico	Yes, 10 projects in the past 5 to 10 years	ABC is moderately accepted	DOT	Cost and effectiveness, lack of contractor experience and knowledge, lack of construction personnel	Standardization would help	2 projects that utilized full depth precast deck panels, 1 project that utilizes precast pier caps, abutment caps, and wingwalls	None
New York	Yes, 10 total projects	Gaining acceptance	DOT	Construction costs, lack of staging areas	Precast elements used	Van Wyck Expressway on Long Island	UHPC research for fatigue in precast element joints
North Carolina	Several projects over the past couple of years,	Good accpetance	DOT	N/A	N/A	Washington Bypass Project	MSE abutments, geosynthetic reinforced soil abutments
North Dakota	None	N/A	Contractor	Low acceptance, high cost, connection issues	Standardization may help	None	Very little (topics unspecified)
Ohio	Many projects completed in the past 14 years	High	DOT	Connection issues	Standardization is not necessary		
Oklahoma	Several projects over the past 5 years	Good acceptance	DOT		Extensive use of precast elements	Highway 51 over Cottonwood Creek Bridge (Mannford,OK)	
Oregon	8 projects in the past couple of years	Very high	DOT	High cost, seismic connections		UHPC for connections of full depth deck panels	None
Pennsylvania	Yes, several projects over the past 5 years	Accepted, considered on a project by project basis	Contractor	Contractors are unwilling to assume additional risks, low experience, reluctance to subcontract,	Yes, used precast elements and launching using SPMTs	None	structural details
Rhode Island							

State	Past experience with ABC	Level of Acceptance of ABC	Whether projects engineered by DOT's or Contractor	Impediments to ABC employment	Availability of Standardized elements	Ongoing or recent projects	Ongoing or recent research
South Carolina							
South Dakota	Yes	Widely accepted, seen as favorable if project conditions warrant it		Low traffic volumes that don't balance high cost of ABC methods		2001 project over a railroad yard	Construction of jointless decks without increasing cost significantly or at all
Tennessee	Little experience, one project in the past 5 years	Limited acceptance, always considered in bridge projects but not often used	DOT	Questions about durability of precast elements, connection issues with precast decks to beams	Precast memebers and elements used	None	None
Texas		Low acceptance because of low incentives	Contractor	No financial incentive, bridge projects are too small to sufficiently expose contractors to the new methods	Only practical for use when lack of access makes cast-in-place components hard to construct or when there is a lot of repetition	None	None
Utah	Yes, extensive use of ABC techniques over the past 10 years	High acceptance, standard practice since 2010	DOT	No impediments. ABC is standard	Available for use and currently being implemented	Standard current practice	Standards for deck panels, precast substructures, new presrressed beam sections, seismic detailing, acceptable deformation limits, connection details and durability
Vermont	Yes, completed 5 projects in the past 5 years	Good acceptance	Contractor	Low traffic, user costs don't balance out costs of ABC methods,	Yes	None	Incentive/dicincentive clauses to help encourage ABC methods
Virginia							
Washington	Yes, several projects that incorporated ABC techniques	Good acceptance	DOT	None documented	Completed projects using complete prefabrication or superstructure and substructure elements	None	None
West Virginia	Yes, 5 projects in the past 5 years	Accepted but traffic volume is too low to make it feasible	DOT	Underdeveloped ABC contracting industry, lack of heavy lift contractors and local contractors,	No precasting industry in the state	None	methods that minimize environmental disruption
Wisconsin	Just beginning to impliment ABC practices	Low acceptance and support because the program is so new, yet no strong opposition	Contractor	New process, low support and lack of experience	Precasting available	Re-decking of major structure with full depth precast deck panels	precast substructure units

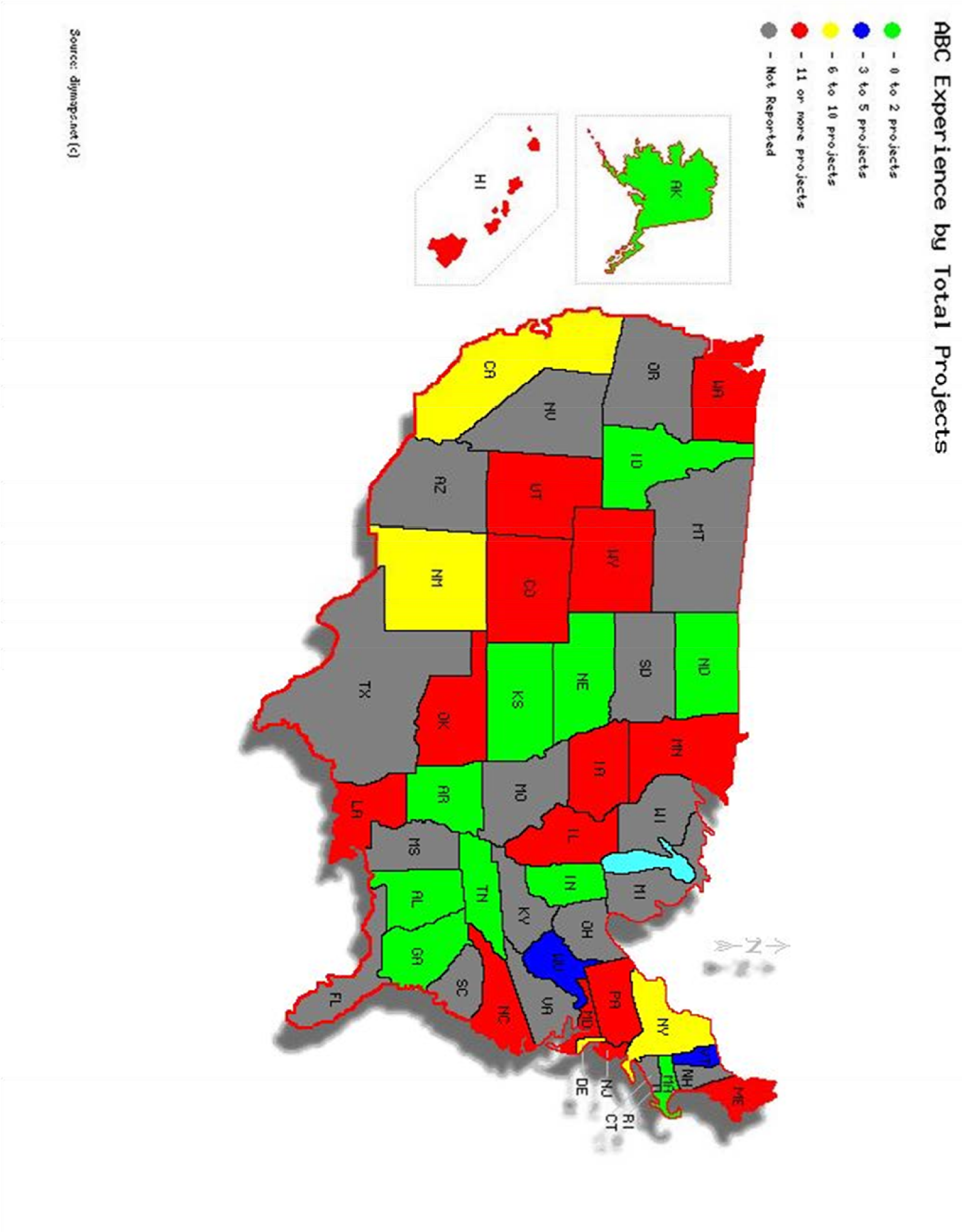
State	Past experience with ABC	Level of Acceptance of ABC	Whether projects engineered by DOT's or Contractor	Impediments to ABC employment	Availability of Standardized elements	Ongoing or recent projects	Ongoing or recent research
Wyoming	Yes, several completed projects	Good acceptance, ABC used where appropriate		Justification of higher costs since traffic is low	Extensive use of precast decked bulb-tees for country road bridges	None	None

A2- ABC Map

Map based off 2009 survey results (SHRP2 2013). Displays total completed projects reported.



Map based off 2015 survey results. Displays total completed projects reported to date.



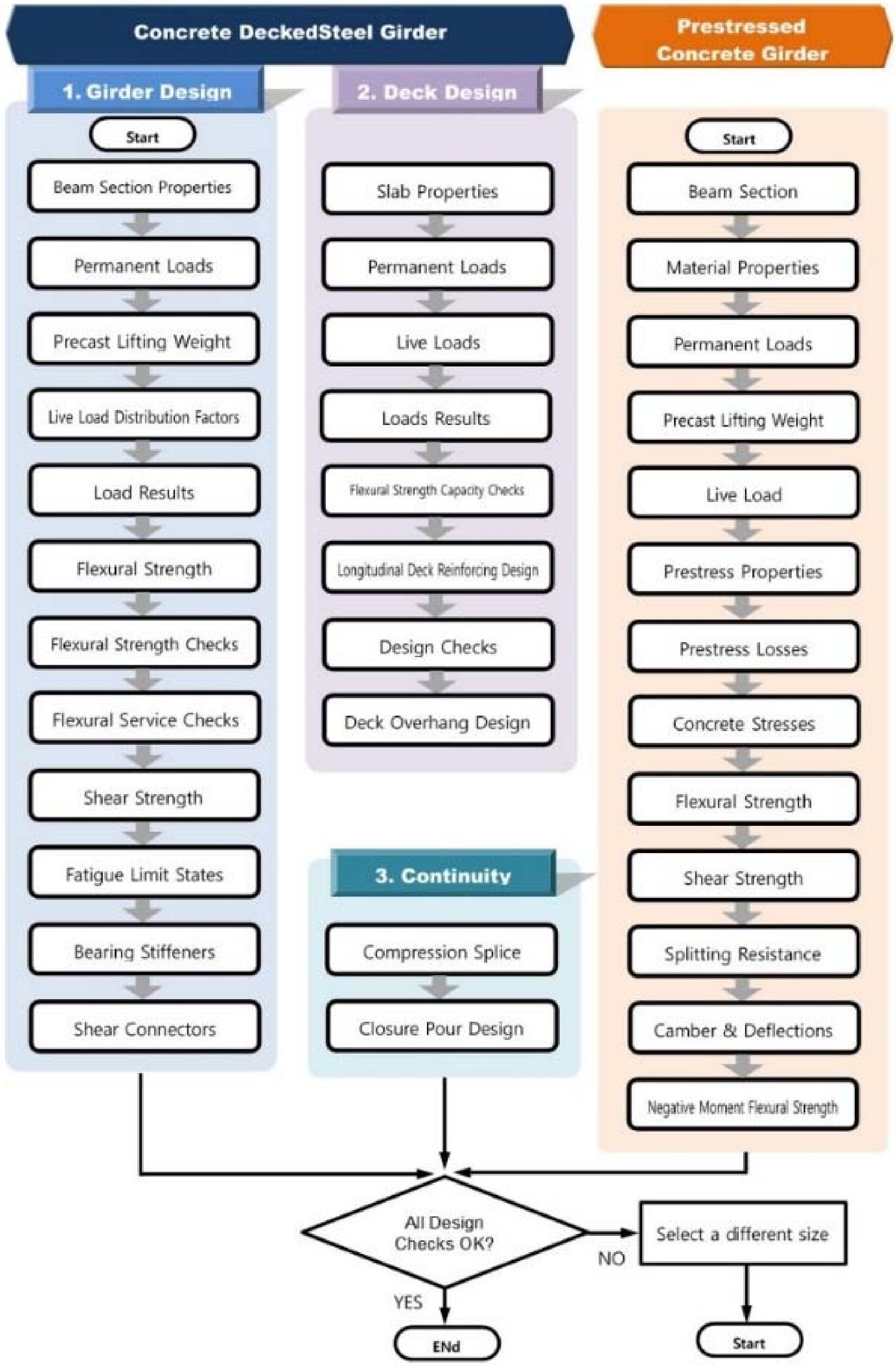
APPENDIX B

ABC Sample Design Examples and Flowcharts: Using Mathcad

APPENDIX B - ABC Sample Design Examples and Flowcharts: Using Mathcad

The design component of the toolkit is composed of design concepts and examples. This APPENDIX B provides user friendly pre-design examples and interactive design flowcharts with design aides such as Mathcad. Both steel and concrete girder design examples were developed for 40, 60, 80 ft span lengths, and modified to allow for easy understanding. The base design examples were taken from the SHRP 2 document "Innovative Bridge Designs for Rapid Renewal" (SHRP2 2013). Modifications were made to the original design document by using GDOT standard criteria for highway bridges, information obtained from a design example created by the Federal Highway Association, and the latest AASHTO LRFD Bridge Design Specifications, 6th Ed. (2012). All design examples in this project were created using Mathcad, which allows readers such as Georgia city or county engineers to easily follow the extensive procedures involved in ABC bridge design. Simplicity is stressed throughout the examples and even in the standard drawings in this project.

B1 - Design Flowchart for Concrete Decked Steel Girder and Precast/Prestressed Concrete Girder



B2- Design Examples for Concrete Decked Steel Girder and Precast/Prestressed Concrete Girder Design using Mathcad

1. Concrete decked steel girder examples for 40ft, 60ft, 80ft spans
- Steel Girder-80ft.xmcd
- Steel Girder-60ft.xmcd
- Steel Girder-40ft.xmcd

2. Precast/prestressed concrete girder examples for 40ft, 60ft, 80ft spans
- Prestressed Concrete Girder-80ft.xmcd
- Prestressed Concrete Girder-60ft.xmcd
- Prestressed Concrete Girder-40ft.xmcd

Note:
The electronic files of these Mathcad examples are provided through an external hard drive or email.

Summary of changes from SHRP2:

- Adapted the AASHTO LRFD Bridge Design Specifications, 6th Edition (2012) and GDOT Standards
- MathCAD Design Aides provided in Appendix of the final report
  - Design loadings calucation (moment, shear, and reaction) for girders
  - Design loadings calucation for deck
- List of variable definitions added
- Enhanced the descriptions for all design steps
- Expansion of detail regarding girder sizing
- New cross-section drawings
- Load combination explanations
- 12 ft travel lanes, 6 ft shoulders and 2% slope from crown to comply with GDOT standards

File Name: Steel Girder-80 ft.xmcd

CONCRETE DECKED STEEL GIRDER DESIGN FOR ABC

The following example details the design of a steel girder bridge accompanied by precast concrete deck panels. This particular example was created in accordance with Accelerated Bridge Construction (ABC) principles. The example shown here is presented for a Georgia Department of Transportation research endeavour into ABC technology, and is intended to simplify the design procedure of ABC style bridges. This example was taken from the SHRP 2 Manual (S2-R04-RR-2), and modified by a Georgia Southern University research team working for the Georgia Department of Transportation.

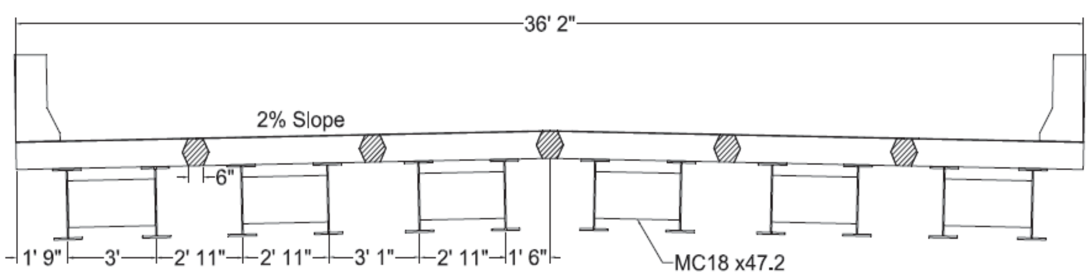
Note: These calculations do not consider every aspect of the bridge design process, and should not be condsidered exhaustive.

Note: All user inputs are highlighted in yellow for easy identification.

AASHTO LRFD Bridge Design Specifications (Sixth Edition with 2012 interims) was used to formulate this example. Located throughout this example are direct references to the AASHTO LRFD Bridge Design Specifications, which are found to the right side of their affiliated calculation.

Before beginning this example, a structural modeling program was used to analyze the superstructure. Although the calculations are not shown, the outputs are used for the design moments, shears and reactions in the example.

BRIDGE GEOMETRY:



Design member parameters:

Deck Width:	$w_{deck} := 36ft + 2in$	C. to C. Piers:	$Length := 80ft$
Roadway Width:	$w_{roadway} := 33ft$	C. to C. Bearings	$L_{span} := 77ft + 10in$
Skew Angle:	$Skew := 0deg$	Bridge Length:	$L_{total} := 3 \cdot Length = 240\ ft$
Deck Thickness	$t_d := 10.5in$	Stringer	W33x118
Haunch Thickness	$t_h := 2in$	Stringer Weight	$w_{s1} := 118plf$
Haunch Width	$w_h := 10.5in$	Stringer Length	$L_{str} := Length - 6 \cdot in = 79.5\ ft$
Girder Spacing	$spacing_{int} := 2ft + 11in$	Average spacing of adjacent beams. This value is used so that effective deck width is not overestimated.	
	$spacing_{ext} := 3ft$		

TABLE OF CONTENTS:

List of Variable Definitions

A = plan area of ice floe (ft^2); depth of temperature gradient (in.) (C3.9.2.3) (3.12.3)
AEP = apparent earth pressure for anchored walls (ksf) (3.4.1)
AF = annual frequency of bridge element collapse (number/yr.) (C3.14.4)
AS = peak seismic ground acceleration coefficient modified by short-period site factor (3.10.4.2)
B = notional slope of backfill (degrees) (3.11.5.8.1)
B′ = equivalent footing width (ft) (3.11.6.3)
Be = width of excavation (ft) (3.11.5.7.2b)
BM = beam (width) for barge, barge tows, and ship vessels (ft) (C3.14.5.1)
Bp = width of bridge pier (ft) (3.14.5.3)
BR = vehicular braking force; base rate of vessel aberrancy (3.3.2) (3.14.5.2.3)
b = braking force coefficient; width of a discrete vertical wall element (ft) (C3.6.4) (3.11.5.6)
bf = width of applied load or footing (ft) (3.11.6.3)
C = coefficient to compute centrifugal forces; constant for terrain conditions in relation to wind approach (3.6.3) (C3.8.1.1)
CD = drag coefficient (s^2 lbs./ft^4) (3.7.3.1)
CH = hydrodynamic mass coefficient (3.14.7)
CL = lateral drag coefficient (C3.7.3.1)
Csm = elastic seismic response coefficient for the m^{th} mode of vibration (3.10.4.2)
c = soil cohesion (ksf) (3.11.5.4)
cf = distance from back of a wall face to the front of an applied load or footing (ft) (3.11.6.3)
D = depth of embedment for a permanent nongravity cantilever wall with discrete vertical wall elements (ft) (3.11.5.6)
DE = minimum depth of earth cover (ft) (3.6.2.2)
Do = calculated embedment depth to provide equilibrium for nongravity cantilevered with continuous vertical elements by the simplified method (ft) (3.11.5.6)
D1 = effective width of applied load at any depth (ft) (3.11.6.3)
d = depth of potential base failure surface below base of excavation (ft); horizontal distance from the back of a wall face to the centerline of an applied load (ft) (3.11.5.7.2b) (3.11.6.3)
dc = total thickness of cohesive soil layers in the top 100 ft (3.10.3.1)
ds = total thickness of cohesionless soil layers in the top 100 ft (3.10.3.1)
E = Young's modulus (ksf) (C3.9.5)
EB = deformation energy (kip-ft) (C3.14.11)
e′ = eccentricity of load on footing (ft) (3.11.6.3)
F1 = lateral force due to earth pressure (kip/ft) (3.11.6.3)
F2 = lateral force due to traffic surcharge (kip/ft) (3.11.6.3)
f = constant applied in calculating the coefficient C used to compute centrifugal forces, taken equal to 4/3 for load combinations other than fatigue and 1.0 for fatigue (3.6.3)
f′c = specified compressive strength of concrete for use in design (ksi) (3.5.1)
g = gravitational acceleration (ft/s^2) (3.6.3)
H = ultimate bridge element strength (kip); final height of retaining wall (ft); total excavation depth (ft); resistance of bridge component to a horizontal force (kip) (C3.11.1) (3.11.5.7.1) (3.14.5.4)
Hp = ultimate bridge pier resistance (kip) (3.14.5.4)
Hs = ultimate bridge superstructure resistance (kip) (3.14.5.4)
H1 = distance from ground surface to uppermost ground anchor (ft) (3.11.5.7.1)
Hn+1 = distance from base of excavation to lowermost ground anchor (ft) (3.11.5.7.1)
h = notional height of earth pressure diagram (ft) (3.11.5.7)
heq = equivalent height of soil for vehicular load (ft) (3.11.6.4)
IM = dynamic load allowance (C3.6.1.2.5)
k = coefficient of lateral earth pressure; number of cohesive soil layers in the top 100 ft (3.11.6.2) (3.10.3.1)
ka = coefficient of active lateral earth pressure (3.11.5.1)
ko = coefficient of at rest lateral earth pressure (3.11.5.1)
kp = coefficient of passive lateral earth pressure (3.11.5.1)
ks = coefficient of earth pressure due to surcharge (3.11.6.1)
L = perimeter of pier (ft); length of soil reinforcing elements in an MSE wall (ft); length of footing (ft);

expansion length (in.) (3.9.5) (3.11.5.8) (3.11.6.3) (3.12.2.3)
ℓ = characteristic length (ft); center-to-center spacing of vertical wall elements (ft) (C3.9.5) (3.11.5.6)
m = multiple presence factor; number of cohesionless soil layers in the top 100 ft (3.6.1.1.2) (3.10.3.1)
N = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile (3.10.3.1)

Nch = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for cohesive soil layers in the upper 100 ft of the soil profile and us for cohesive soil layers (PI > 20) in the top 100 ft (us method) (3.10.3.1)
Nchi = blowcount for a cohesionless soil layer (not to exceed 100 blows/ft in the above expression) (3.10.3.1)
Ni = Standard Penetration Test blow count of a layer (not to exceed 100 blows/ft in the above expression). Note that when using Method B, N values are for cohesionless soils and cohesive soil and rock layers within the upper 100 ft Where refusal is met for a rock layer, Nishould be taken as 100 blows/ft (3.10.3.1)
Ns = stability number (3.11.5.6)
OCR = overconsolidation ratio (3.11.5.2)
P = maximum vertical force for single ice wedge (kip); load resulting from vessel impact (kip); concentrated wheel load (kip); live load intensity; point load (kip) (C3.9.5) (3.14.5.4) (C3.6.1.2.5) (C3.11.6.2) (3.11.6.1)
Pa = force resultant per unit width of wall (kip/ft) (3.11.5.8.1)
PC = probability of bridge collapse (3.14.5)
PD = design wind pressure (ksf) (3.8.1.2.1)
PGA = peak seismic ground acceleration coefficient on rock (Site Class B) (3.10.2.1) (3.10.4.2)
PH = lateral force due to superstructure or other concentrated lateral loads (kip/ft) (3.11.6.3)
Ph = horizontal component of resultant earth pressure on wall (kip/ft) (3.11.5.5)
PI = plasticity index (ASTM D4318) (3.10.3.1)
Pp = passive earth pressure (kip/ft) (3.11.5.4)
Pv = vertical component of resultant earth pressure on wall (kip/ft); load per linear foot of strip footing (kip/ft) (3.11.5.5) (3.11.6.3)
P′v = load on isolated rectangular footing or point load (kip) (3.11.6.3)
p = effective ice crushing strength (ksf); stream pressure (ksf); basic earth pressure (psf); fraction of truck traffic in a single lane; load intensity (ksf) (3.9.2.2) (3.7.3.1) (3.11.5.1) (3.6.1.4.2) (3.11.6.1)
pa = apparent earth pressure (ksf); maximum ordinate of pressure diagram (ksf) (3.11.5.3) (3.11.5.7.1)
pp = passive earth pressure (ksf) (3.11.5.4)
Q = total factored load; load intensity for infinitely long line loading (kip/ft) (3.4.1) (3.11.6.2)
Qi = force effects (3.4.1)
q = surcharge pressure (ksf) (3.11.6.3)
qs = uniform surcharge pressure (ksf) (3.11.6.1)
R = radius of curvature (ft); radius of circular pier (ft); seismic response modification factor; reduction factor of lateral passive earth pressure; radial distance from point of load application to a point on the wall (ft); reaction force to be resisted by subgrade below base of excavation (kip/ft) (3.6.3) (3.9.5) (3.10.7.1) (3.11.5.4) (3.11.6.1) (3.11.5.7.1)
Sm = shear strength of rock mass (ksf) (3.11.5.6)
Su = undrained shear strength of cohesive soil (ksf) (3.11.5.6)
Sub = undrained strength of soil below excavation base (ksf) (3.11.5.7.2b)
Sv = vertical spacing of reinforcements (ft) (3.11.5.8.1)
us = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile (3.10.3.1)
sui = undrained shear strength for a cohesive soil layer (not to exceed 5.0 ksf in the above expression) (3.10.3.1)
S1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock (Site Class B) (3.10.2.1) (3.10.4.2)
T = mean daily air temperature (°F) (C3.9.2.2)
TF = period of fundamental mode of vibration of bridge (s) (3.10.2.2)
Thi = horizontal load in anchor i (kip/ft) (3.11.5.7.1)
Tm = period of vibration for mth mode (s) (3.10.4.2)
Tmax = applied load to reinforcement in a mechanically stabilized earth wall (kip/ft) (3.11.5.8.2)
TMaxDesign= maximum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
TMinDesign = minimum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
TS = corner period at which acceleration response spectrum changes from being independent of period to being inversely proportional to period (s) (3.10.4.2)
T0 = reference period used to define shape of acceleration response spectrum (s) (3.10.4.2)

t = thickness of ice (ft); thickness of deck (in.) (3.9.2.2) (3.12.3)
V = design velocity of water (ft/s); design impact speed of vessel (ft/s) (3.7.3.1) (3.14.6)
V_B = base wind velocity taken as 100 mph (3.8.1.1)
V_{DZ} = design wind velocity at design Elevation Z (mph) (3.8.1.1)
V_{MIN} = minimum design impact velocity taken not less than the yearly mean current velocity for the bridge location (ft/s) (3.14.6)
V_0 = friction velocity, a meteorological wind characteristic for various upwind surface characteristics (mph) (3.8.1.1)
V_{30} = wind speed at 30.0 ft above low ground or water level (mph) (3.8.1.1)
v = highway design speed (ft/s) (3.6.3)
$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile (3.10.3.1)
W = displacement weight of vessel (tonne) (C3.14.5.1)
w = width of clear roadway (ft); width of clear pedestrian and/or bicycle bridge (ft); width of pier at level of ice action (ft); specific weight of water (kcf); moisture content (ASTM D2216) (3.6.1.1.1) (3.6.1.6) (3.9.2.2) (C3.7.3.1) (3.10.3.1)
X = horizontal distance from back of wall to point of load application (ft); distance to bridge element from the centerline of vessel transit path (ft) (3.11.6.2) (3.14.6)
X_1 = distance from the back of the wall to the start of the line load (ft) (3.11.6.2)
X_2 = length of the line load (ft) (3.11.6.2)
Z = structure height above low ground or water level > 30.0 ft (ft); depth below surface of soil (ft); depth from the ground surface to a point on the wall under consideration (ft); vertical distance from point of load application to the elevation of a point on the wall under consideration (ft) (3.8.1.1) (3.11.6.3) (3.11.6.2)
Z_0 = friction length of upstream fetch, a meteorological wind characteristic (ft) (3.8.1.1)
Z_2 = depth where effective width intersects back of wall face (ft) (3.11.6.3)
z = depth below surface of backfill (ft) (3.11.5.1)
α = constant for terrain conditions in relation to wind approach; coefficient for local ice condition; inclination of pier nose with respect to a vertical axis (degrees); inclination of back of wall with respect to a vertical axis (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing nearest to the wall (rad); coefficient of thermal expansion (in./in./°F) (C3.8.1.1) (C3.9.2.2) (3.9.2.2) (C3.11.5.3) (3.11.6.2) (3.12.2.3)
β = safety index; nose angle in a horizontal plane used to calculate transverse ice forces (degrees); slope of backfill surface behind retaining wall; {+ for slope up from wall; − for slope down from wall} (degrees) (C3.4.1) (3.9.2.4.1) (3.11.5.3)

β' = slope of ground surface in front of wall {+ for slope up from wall; − for slope down from wall} (degrees) (3.11.5.6)
γ = load factors; unit weight of materials (kcf); unit weight of water (kcf); unit weight of soil (kcf) (C3.4.1) (3.5.1) (C3.9.5) (3.11.5.1)
γ_s = unit weight of soil (kcf) (3.11.5.1)
γ'_s = effective soil unit weight (kcf) (3.11.5.6)
γ_{EQ} = load factor for live load applied simultaneously with seismic loads (3.4.1)
γ_{eq} = equivalent-fluid unit weight of soil (kcf) (3.11.5.5)
γ_i = load factor (3.4.1)
γ_p = load factor for permanent loading (3.4.1)
γ_{SE} = load factor for settlement (3.4.1)
γ_{TG} = load factor for temperature gradient (3.4.1)
Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft) (C3.11.1) (3.11.5.5)
Δ_p = constant horizontal earth pressure due to uniform surcharge (ksf) (3.11.6.1)
Δ_{ph} = constant horizontal pressure distribution on wall resulting from various types of surcharge loading (ksf) (3.11.6.2)
ΔT = design thermal movement range (in.) (3.12.2.3)
$\Delta\sigma_H$ = horizontal stress due to surcharge load (ksf) (3.11.6.3)
$\Delta\sigma_v$ = vertical stress due to surcharge load (ksf) (3.11.6.3)
δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing furthest from the wall (rad) (C3.9.5) (3.11.5.3) (3.11.6.2)
η_i = load modifier specified in Article 1.3.2; wall face batter (3.4.1) (3.11.5.9)

θ = angle of back of wall to the horizontal (degrees); angle of channel turn or bend (degrees); angle between direction of stream flow and the longitudinal axis of pier (degrees) (3.11.5.3) (3.14.5.2.3) (3.7.3.2)
θ_f = friction angle between ice floe and pier (degrees) (3.9.2.4.1)
σ = standard deviation of normal distribution (3.14.5.3)
σ_T = tensile strength of ice (ksf) (C3.9.5)
ν = Poisson's Ratio (dim.) (3.11.6.2)
φ = resistance factors (C3.4.1)
φ_f = angle of internal friction (degrees) (3.11.5.4)

φ'_f = effective angle of internal friction (degrees) (3.11.5.2)
φ_r = internal friction angle of reinforced fill (degrees) (3.11.6.3)
φ'_s = angle of internal friction of retained soil (degrees) (3.11.5.6)

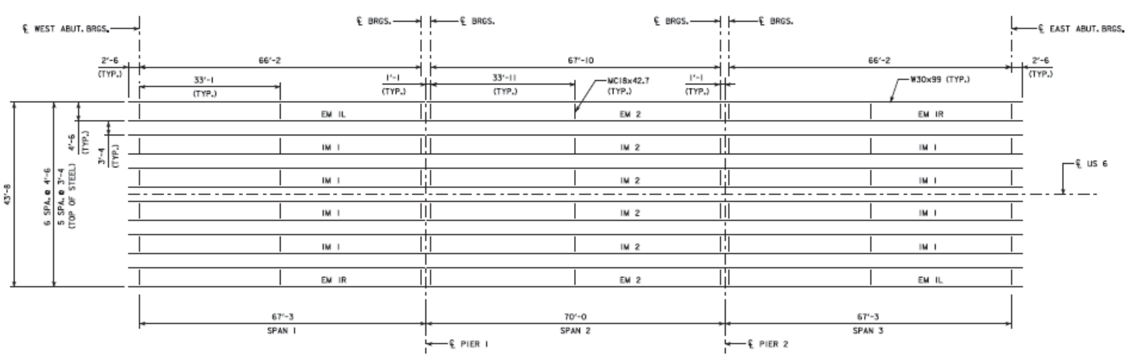
• Permanent Loads
CR = force effects due to creep
DD = downdrag force
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

• Transient Loads

EQ = earthquake load
FR = friction load
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = force effect due to settlement
TG = force effect due to temperature gradient
TU = force effect due to uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

1. INTRODUCTION

AASHTO LRFD principles were used in the design of this superstructure. The example is designed for a bridge with three even spans, and has no skew. The bridge has two 12-foot wide lanes and two 6-foot wide shoulders, for a total roadway width of 36' from curb to curb. The bridge deck is precast reinforced concrete with overhangs at the outermost girders. The longitudinal girders are placed as simply supported modules, and made continuous with connection plates and cast-in-place deck joints. The design of the continuity at the deck joint is addressed in final sections of this example.



FRAMING PLAN

The cross-section consists of six modules. The interior modules are identical and consist of two steel girders and a 6'-0" precast composite deck slab. Exterior modules include two steel girders and a 6'-1" precast composite deck slab, with F-shape barriers. Grade 50 steel is used throughout, and the deck concrete has a compressive strength of 5,000 psi.

The closure pour joints between the modules use Ultra High Performance Concrete with a strength of 21,000 psi.

Steel girder design steps, including constructability checks, fatigue design for infinite fatigue lift (unless otherwise noted), and bearing stiffener design comprise the majority of the example. Diaphragm and deck design procedures are present, but not detailed.

Tips for reading this Design Example:

This calculation was prepared with Mathcad version 14. Mathcad was used in this instance to provide a clear representation of formulas, and their execution. Design software other than Mathcad is recommended for a speedier and more accurate design.

Mathcad is not a design software. Mathcad executes user mathematical and simple logic commands.

Example 1: User inputs are noted with dark shaded boxes. Shading of boxes allows the user to easily find the location of a desired variable. Given that equations are written in mathcad in the same fashion as they are on paper, except that they are interactive, shading input cells allows the user to quicly locate inputs amongst other data on screen. Units are user inputs.

Height of Structure: $H_{structure} := 25ft$

Example 2: Equations are typed directly into the workspace. Mathcad then reads the operators and executes the calculations.

Panels are 2.5' $N_{panels} := \frac{H_{structure}}{2.5ft}$ $N_{panels} = 10$

Example 3: If Statements are an important operator that allow for the user to dictate a future value with given parameters. They are marked by a solid bar and operate with the use of program specific logic commands.

Operator offers discount per volume of panels

$$Discount := \left| \begin{array}{l} .75 \ \text{ if } \ N_{panels} \geq 6 \\ .55 \ \text{ if } \ N_{panels} \geq 10 \\ 1 \ \text{ otherwise} \end{array} \right.$$

$Discount = 0.6$

Example 4: True or False Verification Statements are an important operator that allow for the user to verify a system criteria that has been manually input. They are marked by lighter shading to make a distinction between the user inputs. True or false statements check a single or pairs of variables and return a Zero or One.

Owner to proceed if discounts on retail below 60% $Discount \leq .55 = 1$

2. DESIGN PHILOSOPHY

The superstructure of the bridge in this example consists of modules, which are two rolled steel girders supporting a bridge deck panel along their length. The girders are assumed to be simply supported under the weight of the deck panels. In each module, one girder is assumed to support half the weight of its respective deck panel.

The barrier wall is added to exterior modules once the deck and girders are joined. When working with the barrier dead load, the weight is assumed to be evenly distributed between the two modules. Under the additional barrier dead load, the girders are again assumed to be simply supported.

Concerning transportation of modules, it is assumed that the deck has reached 28-day concrete strength, and the deck is fully composite with the girders. The self-weight of the module during lifting and placement is assumed as evenly distributed to four pick points (two per girder).

The modules are placed such that there is a bearing on each end and are again simply supported. The continuous span

configuration, which includes two bearings per pier on either side of the UHPC joints, is analyzed for positive and negative bending and shear (using simple or refined methods). The negative bending moment above the pier is used to find the force couple for continuity design, between the compression plates at the bottom of the girders and the closure joint in the deck.

The deck design utilizes the equivalent strip method.

3. DESIGN CRITERIA

The first step for any bridge design is to establish the design criteria. The following is a summary of the primary design criteria for this design example:

Governing Specifications: AASTHO LRFD Bridge Design Specifications (6th Edition with 2012 interims)

Design Methodology: Load and Resistance Factor Design (LRFD)

Live Load Requirements: HL-93 S S3.6

Section Constraints:

$W_{mod.max} := 200 \cdot kip$ Upper limit on the weight of the modules, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

4. MATERIAL PROPERTIES

Structural Steel Yield Strength:	$F_y := 50ksi$		STable 6.4.1-1
Structural Steel Tensile Strength:	$F_u := 65ksi$		STable 6.4.1-1
Concrete 28-day Compressive Strength:	$f_c := 5ksi$	$f_{c\_uhpc} := 21ksi$	S5.4.2.1
Reinforcement Strength:	$F_s := 60ksi$		S5.4.3 & S6.10.3.7
Steel Density:	$w_s := 490pcf$		STable 3.5.1-1
Concrete Density:	$w_c := 150pcf$		STable 3.5.1-1
Modulus of Elasticity - Steel:	$E_s := 29000ksi$		
Modulus of Elasticity - Concrete:	$E_c := 33000 \cdot \left(\frac{w_c}{1000pcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 4286.8 \cdot ksi$		
Modular Ratio:	$n := \text{ceil}\left(\frac{E_s}{E_c}\right) = 7$		
Future Wearing Surface Density:	$W_{fws} := 140pcf$		STable 3.5.1-1
Future Wearing Surface Thickness:	$t_{fws} := 2.5in$	(Assumed)	

5. LOAD COMBINATIONS

The following load combinations will be used in this design example, in accordance with Table 3.4.1-1.

Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.

Strength III—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

Strength V—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

Service I—Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.

Service II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

Fatigue I—Fatigue and fracture load combination related to infinite load-induced fatigue life.

Strength I = 1.25DC + 1.5DW + 1.75(LL+IM), where IM = 33%

Strength III = 1.25DC + 1.5DW + 1.40WS

Strength V = 1.25DC + 1.5DW + 1.35(LL+IM) + 0.40WS + 1.0WL, where IM = 33%

Service I = 1.0DC + 1.0DW + 1.0(LL+IM) + 0.3WS + 1.0WL, where IM = 33%

Service II = 1.0DC + 1.0DW + 1.3(LL+IM), where IM = 33%

Fatigue I = 1.5(LL+IM), where IM = 15%

6. BEAM SECTION

Determining the proper girder depth and dimensions is a vital part of any bridge design process. The size of the girder is a major factor in the cost of the bridge. From Table 2.5.2.6.3-1, the suggested minimum overall depth of the composite I-section in a continuous span is equal to 0.032L.

Thus we have, (.032*80ft) = 2.56' = 30.72" (round up to 33", for common sizing)

The following girder dimensions were taken from the AISC Steel Construction Manual (14th Edition).

Determine Beam Section Properties:

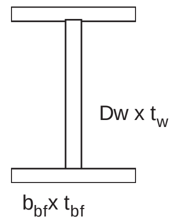
Girder W33x118 $b_{tf} x\ t_{tf}$

Top Flange $b_{tf} := 11.5in$ $t_{tf} := 0.74in$

Bottom Flange $b_{bf} := 11.5in$ $t_{bf} := 0.74in$

Web $D_w := 31.4in$ $t_w := 0.55in$

Girder Depth $d_{gird} := 32.9in$



Check Flange Proportion Requierements Met: S 6.10.2.2

$$\frac{b_{tf}}{2 \cdot t_{tf}} \le 12.0 = 1 \qquad \frac{b_{bf}}{2 \cdot t_{bf}} \le 12.0 = 1$$

$$b_{tf} \ge \frac{D_w}{6} = 1 \qquad b_{bf} \ge \frac{D_w}{6} = 1$$

$$t_{tf} \ge 1.1 \cdot t_w = 1 \qquad t_{bf} \ge 1.1 \cdot t_w = 1$$

$$0.1 \le \frac{\dfrac{t_{bf}^{3} \cdot b_{bf}}{12}}{\dfrac{t_{tf}^{3} \cdot b_{tf}}{12}} \le 10 = 1 \qquad \frac{\dfrac{t_{bf} \cdot b_{bf}}{12}}{\dfrac{t_{tf} \cdot b_{tf}}{12}} \ge 0.3 = 1$$

Properties for use when analyzing under beam self weight (steel only):

$A_{tf} := b_{tf} \cdot t_{tf}$ $A_{bf} := b_{bf} \cdot t_{bf}$ $A_w := D_w \cdot t_w$

$A_{steel} := A_{bf} + A_{tf} + A_w$ $A_{steel} = 34.3 \cdot in^2$ Total steel area.

$$y_{steel} := \frac{A_{tf} \cdot \frac{t_{tf}}{2} + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf}\right)}{A_{steel}} \qquad y_{steel} = 16.4 \cdot in$$

Steel centroid from top.

Calculate Iz: Moment of inertia about Z axis.

$$I_{zsteel} := \frac{t_w \cdot D_w^{3}}{12} + \frac{b_{tf} \cdot t_{tf}^{3}}{12} + \frac{b_{bf} \cdot t_{bf}^{3}}{12} + A_w \cdot \left(\frac{D_w}{2} + t_{tf} - y_{steel}\right)^2 + A_{tf} \cdot \left(y_{steel} - \frac{t_{tf}}{2}\right)^2 + A_{bf} \cdot \left(D_w + \frac{t_{bf}}{2} + t_{tf} - y_{steel}\right)^2$$

Calculate Iy:

$$I_{ysteel} := \frac{D_w \cdot t_w^{3} + t_{tf} \cdot b_{tf}^{3} + t_{bf} \cdot b_{bf}^{3}}{12}$$

Moment of inertia about Y axis.

Calculate Ix:

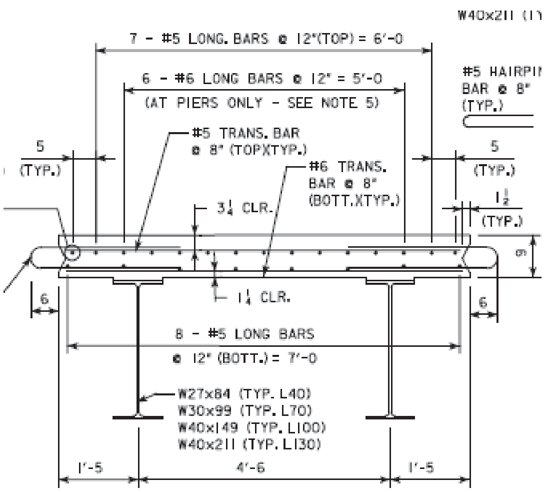
$$I_{xsteel} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{3} + b_{bf} \cdot t_{bf}^{3} + D_w \cdot t_w^{3}\right)$$

Moment of inertia about X axis.

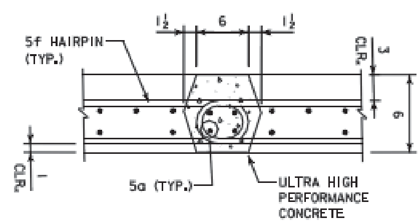
$I_{zsteel} = 5815.066 \cdot in^4$ $I_{ysteel} = 188.010 \cdot in^4$ $I_{xsteel} = 4.8 \cdot in^4$ $A_{steel} = 34.3 \cdot in^2$

Composite Section Properties (Uncracked Section - used for barrier dead load and live load positive bending):

Determine composite slab and reinforcing properties



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL

(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

Slab thickness assumes some sacrificial thickness; use: $t_{slab} := 8in$

$D_t := \left(t_{slab} + t_{tf} + D_w + t_{bf}\right) = 40.9 \cdot in$ Total section depth

$b_{eff} := spacing_{int} \quad b_{eff} = 35 \cdot in$ Effective width. S 4.6.2.6.1 LRFD

$b_{tr} := \dfrac{b_{eff}}{n}$ Transformed slab width as steel.

$I_{zslab} := b_{tr} \cdot \dfrac{t_{slab}^{\ 3}}{12}$ Transformed slab moment of inertia about z axis as steel.

$A_{slab} := b_{tr} \cdot t_{slab}$ Transformed slab area as steel.

Slab reinforcement: (Use #5 @ 8" top, and #6 @ 8" bottom; additional bar for continuous segments of #6 @ 12")

Typical Cross Section

$A_{rt} := 0.465 \dfrac{in^2}{ft} \cdot b_{eff} = 1.4 \cdot in^2$

Cross Section Over Support

$A_{rb} := 0.66 \dfrac{in^2}{ft} \cdot b_{eff} = 1.9 \cdot in^2 \quad A_{rtadd} := 0.44 \cdot \dfrac{in^2}{ft} \cdot b_{eff} = 1.3 \cdot in^2$

$A_r := A_{rt} + A_{rb} = 3.3 \cdot in^2$

$c_{rt} := 2.5in + 0.625in + \left(\frac{5}{16}\right)in = 3.4 \cdot in$

$c_r := \frac{\left(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb}\right)}{A_r} = 4.9 \cdot in$

$A_{rneg} := A_r + A_{rtadd} = 4.6 \cdot in^2$

$c_{rb} := t_{slab} - 1.75in - \left(\frac{6}{16}\right)in = 5.9 \cdot in$ ref from top of slab

$c_{rneg} := \frac{\left(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb} + A_{rtadd} \cdot c_{rt}\right)}{A_{rneg}} = 4.5 \cdot in$

Find composite section centroid:

$A_x := A_{steel} + \frac{A_r \cdot (n-1)}{n} + A_{slab}$ $\quad y_{slab} := \frac{t_{slab}}{2}$

$$y_{st} := \frac{A_{tf} \cdot \left(\frac{t_{tf}}{2} + t_{slab}\right) + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf} + t_{slab}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf} + t_{slab}\right)}{A_{steel}}$$

Centroid of steel from top of slab.

$$y_c := \frac{y_{st} \cdot A_{steel} + \frac{c_r \cdot A_r \cdot (n-1)}{n} + A_{slab} \cdot y_{slab}}{A_x} \qquad y_c = 13.1 \cdot in$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Iz for composite section:

$$I_z := I_{zsteel} + A_{steel} \cdot \left(y_{st} - y_c\right)^2 + I_{zslab} + A_{slab} \cdot \left(y_{slab} - y_c\right)^2 + \frac{A_r \cdot (n-1)}{n} \cdot \left(c_r - y_c\right)^2$$

Transformed moment of inertia about the z axis.

Calculate Transformed Iy for composite section:

$t_{tr} := \frac{t_{slab}}{n}$ Transformed slab thickness.

$I_{yslab} := \frac{t_{tr} \cdot b_{eff}^{\ 3}}{12}$ Transformed moment of inertia about y axis of slab.

$I_y := I_{ysteel} + I_{yslab}$ Transformed moment of inertia about the y axis (ignoring reinforcement).

Calculate Transformed Ix for composite section:

$$I_x := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\ 3} + b_{bf} \cdot t_{bf}^{\ 3} + D_w \cdot t_w^{\ 3} + b_{tr} \cdot t_{slab}^{\ 3}\right)$$

Transformed moment of inertia about the x axis.

Results: $A_x = 77.1 \cdot in^2$ $\quad I_y = 4271.3 \cdot in^4$ $\quad I_z = 13940.9 \cdot in^4$ $\quad I_x = 858.2 \cdot in^4$

Composite Section Properties (Uncracked Section - used for live load negative bending):

Find composite section area and centroid:

$$A_{xneg} := A_{steel} + \frac{A_{rneg} \cdot (n-1)}{n} + A_{slab}$$

$$y_{cneg} := \frac{y_{steel} \cdot A_{steel} + \frac{c_{rneg} \cdot A_{rneg} \cdot (n-1)}{n} + A_{slab} \cdot y_{slab}}{A_{xneg}} \qquad y_{cneg} = 9.5 \cdot in$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Izneg for composite negative moment section:

$$I_{zneg} := I_{zsteel} + A_{steel}\cdot\left(y_{steel} - y_{cneg}\right)^2 + I_{zslab} + A_{slab}\cdot\left(y_{slab} - y_{cneg}\right)^2 + \frac{A_{rneg}\cdot(n-1)}{n}\cdot\left(c_{rneg} - y_{cneg}\right)^2$$

Transformed moment of inertia about the z axis.

$$I_{zneg} = 8989.1\cdot in^4$$

Composite Section Properties (Cracked Section - used for live load negative bending):

Find cracked section area and centroid:

$$A_{cr} := A_{steel} + A_{rneg} = 38.9\cdot in^2$$

$$y_{cr} := \frac{\left(A_{steel}\cdot y_{steel} + A_{rneg}\cdot c_{rneg}\right)}{A_{cr}} = 15\cdot in \qquad y_{crb} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} = 25.8\cdot in$$

Find cracked section moments of inertia and section moduli:

$$I_{zcr} := I_{zsteel} + A_{steel}\cdot\left(y_{steel} - y_{cr}\right)^2 + A_r\cdot\left(c_r - y_{cr}\right)^2 \qquad I_{zcr} = 6222\cdot in^4$$

$$I_{ycr} := I_{ysteel} \qquad I_{ycr} = 188\cdot in^4$$

$$I_{xcr} := \frac{1}{3}\cdot\left(b_{tf}\cdot t_{tf}^{\ 3} + b_{bf}\cdot t_{tf}^{\ 3} + D_w\cdot t_w^{\ 3}\right) \qquad I_{xcr} = 4.8\cdot in^4$$

$$d_{topcr} := y_{cr} - c_{rt} \qquad d_{topcr} = 11.6\cdot in$$

$$d_{botcr} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} \qquad d_{botcr} = 25.8\cdot in$$

$$S_{topcr} := \frac{I_{zcr}}{d_{topcr}} \qquad S_{topcr} = 536.6\cdot in^3$$

$$S_{botcr} := \frac{I_{zcr}}{d_{botcr}} \qquad S_{botcr} = 240.7\cdot in^3$$

7. PERMANENT LOADS

Phase 1: Steel girders are simply supported, and support their self-weight plus the weight of the slab. Steel girders in each module for this example are separated by three diaphragms - one at each bearing location, and one at midspan. Other module span configurations may require an increase or decrease in the number of diaphragms.

$$W_{deck\_int} := w_c\cdot spacing_{int}\cdot t_d \qquad W_{deck\_int} = 382.8\cdot plf$$

$$W_{deck\_ext} := w_c\cdot spacing_{ext}\cdot t_d \qquad W_{deck\_ext} = 393.8\cdot plf$$

$$W_{haunch} := w_c\cdot w_h\cdot t_h \qquad W_{haunch} = 21.9\cdot plf$$

$$W_{stringer} := w_{s1} \qquad W_{stringer} = 118\cdot plf$$

Diaphragms:	MC18x42.7	Thickness Conn. Plate	$t_{conn} := \frac{5}{8}in$
Diaphragm Weight	$w_{s2} := 42.7plf$	Width Conn. Plate	$w_{conn} := 5in$
Diaphragm Length	$L_{diaph} := 4ft + 2.5in$	Height Conn. Plate	$h_{conn} := 28.5in$

$$W_{diaphragm} := w_{s2}\cdot\frac{L_{diaph}}{2} \qquad W_{diaphragm} = 89.8\cdot lbf$$

$W_{conn} := 2 \cdot w_s \cdot t_{conn} \cdot w_{conn} \cdot h_{conn}$ $\qquad W_{conn} = 50.5 \cdot lbf$

$W_{DCpoint} := \left(W_{diaphragm} + W_{conn}\right) \cdot 1.05$ $\qquad W_{DCpoint} = 147.4 \cdot lbf$

Equivalent distributed load from DC point loads: $\quad w_{DCpt\_equiv} := \dfrac{3 \cdot W_{DCpoint}}{L_{str}} = 5.6 \cdot plf$

Interior Uniform Dead Load, Phase 1: $\quad W_{DCuniform1\_int} := W_{deck\_int} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 528.2 \cdot plf$

Exterior Uniform Dead Load, Phase 1: $\quad W_{DCuniform1\_ext} := W_{deck\_ext} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 539.2 \cdot plf$

Moments due to Phase 1 DL: $\quad M_{DC1\_int}(x) := \dfrac{W_{DCuniform1\_int} \cdot x}{2} \cdot \left(L_{str} - x\right) \qquad M_{DC1\_ext}(x) := \dfrac{W_{DCuniform1\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shear due to Phase 1 DL: $\quad V_{DC1\_int}(x) := W_{DCuniform1\_int} \cdot \left(\dfrac{L_{str}}{2} - x\right) \qquad V_{DC1\_ext}(x) := W_{DCuniform1\_ext} \cdot \left(\dfrac{L_{str}}{2} - x\right)$

Phase 2: Steel girders are simply supported and composite with the deck slab, and support their self-weight plus the weight of the slab in addition to barriers on exterior modules. Barriers are assumed to be evenly distributed between the two exterior module girders.

Barrier Area $\quad A_{barrier} := 2.89 ft^2$

Barrier Weight $\quad W_{barrier} := \dfrac{\left(w_c \cdot A_{barrier}\right)}{2} \qquad W_{barrier} = 216.8 \cdot plf$

Interior Dead Load, Phase 2: $\quad W_{DCuniform\_int} := W_{DCuniform1\_int} = 528.2 \cdot plf$

Exterior Dead Load, Phase 2: $\quad W_{DCuniform\_ext} := W_{DCuniform1\_ext} + W_{barrier} = 755.9 \cdot plf$

Moments due to Phase 2 DL: $\quad M_{DC2\_int}(x) := \dfrac{W_{DCuniform\_int} \cdot x}{2} \cdot \left(L_{str} - x\right) \qquad M_{DC2\_ext}(x) := \dfrac{W_{DCuniform\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shear due to Phase 2 DL: $\quad V_{DC2\_int}(x) := W_{DCuniform\_int} \cdot \left(\dfrac{L_{str}}{2} - x\right) \qquad V_{DC2\_ext}(x) := W_{DCuniform\_ext} \cdot \left(\dfrac{L_{str}}{2} - x\right)$

Phase 3: Girders are composite and have been made continuous. Utilities and future wearing surface are applied.

Unit Weight Overlay $\quad w_{ws} := 30 psf$

$W_{ws\_int} := w_{ws} \cdot spacing_{int}$ $\qquad W_{ws\_int} = 87.5 \cdot plf$

$W_{ws\_ext} := w_{ws} \cdot \left(spacing_{ext} - 1 \cdot ft - 7 in\right)$ $\qquad W_{ws\_ext} = 42.5 \cdot plf$

Unit Weight Utilities $\quad W_u := 15 plf$

$W_{DWuniform\_int} := W_{ws\_int} + W_u$ $\qquad W_{DWuniform\_int} = 102.5 \cdot plf$

$W_{DWuniform\_ext} := W_{ws\_ext} + W_u$ $\qquad W_{DWuniform\_ext} = 57.5 \cdot plf$

Moments due to DW: $\quad M_{DW\_int}(x) := \dfrac{W_{DWuniform\_int} \cdot x}{2} \cdot \left(L_{str} - x\right) \qquad M_{DW\_ext}(x) := \dfrac{W_{DWuniform\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shears due to DW: $\quad V_{DW\_int}(x) := W_{DWuniform\_int} \cdot \left(\dfrac{L_{str}}{2} - x\right) \qquad V_{DW\_ext}(x) := W_{DWuniform\_ext} \cdot \left(\dfrac{L_{str}}{2} - x\right)$

8. PRECAST LIFTING WEIGHTS AND FORCES

This section addresses the construction loads for lifting the module into place. The module is lifted from four points, at some distance, D_{lift} from each end of each girder.

Distance from end of lifting point: $D_{lift} := 8.75ft$

Assume weight uniformly distributed along girder, with 30% Dynamic Dead Load Allowance:

Dynamic Dead Load Allowance: $DLIM := 30\%$

Interior Module:

Total Interior Module Weight: $W_{int} := \left(L_{str} \cdot W_{DCuniform\_int} + 3 \cdot W_{DCpoint}\right) \cdot 2 \cdot (1 + DLIM) = 110.3 \cdot kip$

Vertical force at lifting point: $F_{lift\_int} := \dfrac{W_{int}}{4} = 27.6 \cdot kip$

Equivalent distributed load: $w_{int\_IM} := \dfrac{W_{int}}{\left(2 \cdot L_{str}\right)} = 694 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_int} := -w_{int\_IM} \cdot \dfrac{\left(D_{lift}{}^2\right)}{2}$ $\qquad M_{lift\_neg\_max\_int} = -26.6 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting:

$$M_{lift\_pos\_max\_int} := \left| \begin{array}{l} 0 \ \text{ if } \ \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} < 0 \\ \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} \end{array} \right.$$

$M_{lift\_pos\_max\_int} = 306.9 \cdot kip \cdot ft$

Exterior Module:

Total Exterior Module Weight: $W_{ext} := \left(L_{str} \cdot W_{DCuniform\_ext} + 3 \cdot W_{DCpoint} + W_{barrier} \cdot L_{str}\right) \cdot 2 \cdot (1 + DLIM) = 202.2 \cdot kip$

Vertical force at lifting point: $F_{lift\_ext} := \dfrac{W_{ext}}{4} = 50.6 \cdot kip$

Equivalent distributed load: $w_{ext\_IM} := \dfrac{W_{ext}}{2 \cdot L_{str}} = 1271.7 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_ext} := -w_{ext\_IM} \cdot \dfrac{D_{lift}{}^2}{2}$ $\qquad M_{lift\_neg\_max\_ext} = -48.7 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting: $M_{lift\_pos\_max\_ext} := \begin{cases} 0 \text{ if } \dfrac{w_{ext\_IM}\cdot\left(L_{str} - 2\cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} < 0 \\ \dfrac{w_{ext\_IM}\cdot\left(L_{str} - 2\cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} \end{cases}$

$M_{lift\_pos\_max\_ext} = 562.4\cdot kip\cdot ft$

Max Shear during lifting: $V_{lift} := \max\left(w_{ext\_IM}\cdot D_{lift}, F_{lift\_ext} - w_{ext\_IM}\cdot D_{lift}\right) = 39.4\cdot kip$

9. LIVE LOAD DISTRIBUTION FACTORS

These factors represent the distribution of live load from the deck to the girders in accordance with AASHTO Section 4, and assumes the deck is fully continuous across the joints.

Girder Section Modulus: $I_{zsteel} = 5815.1\cdot in^4$

Girder Area: $A_{steel} = 34.3\cdot in^2$

Girder Depth: $d_{gird} = 32.9\cdot in$

Distance between centroid of deck and centroid of beam: $e_g := \dfrac{t_d}{2} + t_h + \dfrac{d_{gird}}{2} = 23.7\cdot in$

Modular Ratio: $n = 7$

Multiple Presence Factors: $MP_1 := 1.2$ $\quad MP_2 := 1.0$ S3.6.1.1.2-1

Interior Stringers for Moment:

One Lane Loaded: $K_g := n\cdot\left(I_{zsteel} + A_{steel}\cdot e_g^{\,2}\right) = 175527.9\cdot in^4$ S4.6.2.2.1-1

$$g_{int\_1m} := \left[0.06 + \left(\frac{spacing_{int}}{14ft}\right)^{0.4}\cdot\left(\frac{spacing_{int}}{L_{span}}\right)^{0.3}\cdot\left(\frac{K_g}{L_{span}\cdot t_d^{\,3}}\right)^{0.1}\right] = 0.226$$

Two Lanes Loaded: $$g_{int\_2m} := \left[0.075 + \left(\frac{spacing_{int}}{9.5ft}\right)^{0.6}\cdot\left(\frac{spacing_{int}}{L_{span}}\right)^{0.2}\cdot\left(\frac{K_g}{L_{span}\cdot t_d^{\,3}}\right)^{0.1}\right] = 0.288$$

Governing Factor: $g_{int\_m} := \max\left(g_{int\_1m}, g_{int\_2m}\right) = 0.288$

Interior Stringers for Sheai

One Lane Loaded: $$g_{int\_1v} := \left(0.36 + \frac{spacing_{int}}{25ft}\right) = 0.477$$

Two Lanes Loaded: $$g_{int\_2v} := \left[0.2 + \frac{spacing_{int}}{12ft} + -\left(\frac{spacing_{int}}{35ft}\right)^2\right] = 0.436$$

Governing Factor: $g_{int\_v} := \max\left(g_{int\_1v}, g_{int\_2v}\right) = 0.477$

Exterior Stringers for Moment:

One Lane Loaded: Use Lever Rule. Wheel is 2' from barrier; barrier is 2" beyond exterior stringer.

$d_e := 2in$

$L_{spa} := 4.5ft \qquad r := L_{spa} + d_e - 2ft = 2.7 \cdot ft$

$g_{ext\_1m} := MP_1 \cdot \frac{0.5r}{L_{spa}} = 0.356$

Two Lanes Loaded: $e_{2m} := 0.77 + \frac{d_e}{9.1ft} = 0.7883$

$g_{ext\_2m} := e_{2m} \cdot g_{int\_2m} = 0.227$

Governing Factor: $g_{ext\_m} := \max(g_{ext\_1m}, g_{ext\_2m}) = 0.356$

Exterior Stringers for Shear:

One Lane Loaded: Use Lever Rule.

$g_{ext\_1v} := g_{ext\_1m} = 0.356$

Two Lanes Loaded: $e_{2v} := 0.6 + \frac{d_e}{10ft} = 0.62$

$g_{ext\_2v} := e_{2v} \cdot g_{int\_2v} = 0.269$

Governing Factor: $g_{ext\_v} := \max(g_{ext\_1v}, g_{ext\_2v}) = 0.356$

FACTOR TO USE FOR SHEAR: $g_v := \max(g_{int\_v}, g_{ext\_v}) = 0.477$

FACTOR TO USE FOR MOMENT: $g_m := \max(g_{int\_m}, g_{ext\_m}) = 0.356$

10. LOAD RESULTS

Case 1: Dead Load on Steel Only (calculated in Section 7). Negative moments are zero and are not considered. Because the girder is simply supported, the maximum moment is at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder

$M_{DC1int} := M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 417.3 \cdot kip \cdot ft \qquad M_{DW1int} := 0 \cdot kip \cdot ft \qquad M_{LL1int} := 0 kip \cdot ft$

$V_{DC1int} := V_{DC1\_int}(0) = 21 \cdot kip \qquad V_{DW1int} := 0 \cdot kip \qquad V_{LL1int} := 0 \cdot kip$

Exterior Girder

$M_{DC1ext} := M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 426 \cdot kip \cdot ft \qquad M_{DW1ext} := 0 \cdot kip \cdot ft \qquad M_{LL1ext} := 0 \cdot kip \cdot ft$

$V_{DC1ext} := V_{DC1\_ext}(0) = 21.4 \cdot kip \qquad V_{DW1ext} := 0 \cdot kip \qquad V_{LL1ext} := 0 \cdot kip \cdot ft$

Load Cases:

$M_{1\_STR\_I} := \max(1.25 \cdot M_{DC1int} + 1.5 \cdot M_{DW1int} + 1.75 \cdot M_{LL1int}, 1.25 \cdot M_{DC1ext} + 1.5 \cdot M_{DW1ext} + 1.75 \cdot M_{LL1ext}) = 532.5 \cdot kip \cdot$

$V_{1\_STR\_I} := \max(1.25 \cdot V_{DC1int} + 1.5 \cdot V_{DW1int} + 1.75 \cdot V_{LL1int}, 1.25 \cdot V_{DC1ext} + 1.5 \cdot V_{DW1ext} + 1.75 \cdot V_{LL1ext}) = 26.8 \cdot kip$

Case 2: Dead Load on Composite Section (calculated in Section 7). Negative moments are zero and are not considered. Again, the maximum moment occur at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder

$M_{DC2int} := M_{DC2\_int}\left(\frac{L_{str}}{2}\right) = 417.3 \cdot kip \cdot ft \qquad M_{DW2int} := 0 \cdot kip \cdot ft \qquad M_{LL2int} := 0 \cdot kip \cdot ft$

$V_{DC2int} := V_{DC2\_int}(0) = 21 \cdot kip \qquad V_{DW2int} := 0 \cdot kip \qquad V_{LL2int} := 0 \cdot kip$

Exterior Girder

$M_{DC2ext} := M_{DC2\_ext}\left(\frac{L_{str}}{2}\right) = 597.2 \cdot kip \cdot ft \qquad M_{DW2ext} := 0 \cdot kip \cdot ft \qquad M_{LL2ext} := 0 \cdot kip \cdot ft$

$V_{DC2ext} := V_{DC2\_ext}(0) = 30 \cdot kip \qquad V_{DW2ext} := 0 \cdot kip \qquad V_{LL2ext} := 0 \cdot kip$

Load Cases:

$M_{2\_STR\_I} := \max(1.25 \cdot M_{DC2int} + 1.5 \cdot M_{DW2int} + 1.75 \cdot M_{LL2int}, 1.25 \cdot M_{DC2ext} + 1.5 \cdot M_{DW2ext} + 1.75 \cdot M_{LL2ext}) = 746.5 \cdot kip \cdot$

$V_{2\_STR\_I} := \max\left(1.25 \cdot V_{DC2int} + 1.5 \cdot V_{DW2int} + 1.75 \cdot V_{LL2int}, 1.25 \cdot V_{DC2ext} + 1.5 \cdot V_{DW2ext} + 1.75 \cdot V_{LL2ext}\right) = 37.6 \cdot kip$

Case 3: Composite girders are lifted into place from lifting points located distance D_{lift} from the girder edges. Maximum moments and shears were calculated in Section 8.

Interior Girder

$M_{DC3int} := M_{lift\_pos\_max\_int} = 306.9 \cdot kip \cdot ft$ $\quad M_{DW3int} := 0 \cdot kip \cdot ft$ $\quad M_{LL3int} := 0 \cdot kip \cdot ft$

$M_{DC3int\_neg} := \left|M_{lift\_neg\_max\_int}\right| = 26.6 \cdot kip \cdot ft$ $\quad M_{DW3int\_neg} := 0 \cdot kip \cdot ft$ $\quad M_{LL3int\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3int} := V_{lift} = 39.4 \cdot kip$ $\quad V_{DW3int} := 0 \cdot kip$ $\quad V_{LL3int} := 0 \cdot kip$

Exterior Girder

$M_{DC3ext} := M_{lift\_pos\_max\_ext} = 562.4 \cdot kip \cdot ft$ $\quad M_{DW3ext} := 0 \cdot kip \cdot ft$ $\quad M_{LL3ext} := 0 \cdot kip \cdot ft$

$M_{DC3ext\_neg} := \left|M_{lift\_neg\_max\_ext}\right| = 48.7 \cdot kip \cdot ft$ $\quad M_{DW3ext\_neg} := 0 \cdot kip \cdot ft$ $\quad M_{LL3ext\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3ext} := V_{lift} = 39.4 \cdot kip$ $\quad V_{DW3ext} := 0 \cdot kip$ $\quad V_{LL3ext} := 0 \cdot kip$

Load Cases:

$M_{3\_STR\_I} := \max\left(1.5 \cdot M_{DC3int} + 1.5 \cdot M_{DW3int}, 1.5 \cdot M_{DC3ext} + 1.5 \cdot M_{DW3ext}\right) = 843.6 \cdot kip \cdot ft$

$M_{3\_STR\_I\_neg} := \max\left(1.5 \cdot M_{DC3int\_neg} + 1.5 \cdot M_{DW3int\_neg}, 1.5 \cdot M_{DC3ext\_neg} + 1.5 \cdot M_{DW3ext\_neg}\right) = 73 \cdot kip \cdot ft$

$V_{3\_STR\_I} := \max\left(1.5 \cdot V_{DC3int} + 1.5 \cdot V_{DW3int}, 1.5 \cdot V_{DC3ext} + 1.5 \cdot V_{DW3ext}\right) = 59.1 \cdot kip$

Case 4: Composite girders made continuous. Utilities and future wearing surface are applied, and live load. Maximum moment and shear results are from a finite element analysis not included in this design example. The live load value includes the lane fraction calculated in Section 9, and impact.

Governing Loads:

$M_{DC4} := 387 \cdot kip \cdot ft$ $\quad M_{DW4} := 52.5 \cdot kip \cdot ft$ $\quad M_{LL4} := 537.98 \cdot kip \cdot ft$

$M_{WS4} := 0 kip \cdot ft$ $\quad M_{W4} := 0 kip \cdot ft$

$M_{DC4neg} := -483.8 \cdot kip \cdot ft$ $\quad M_{DW4neg} := -65.6 \cdot kip \cdot ft$ $\quad M_{LL4neg} := -541.16 kip \cdot ft$

$M_{WS4neg} := 0 \cdot kip \cdot ft$ $\quad M_{WL4neg} := 0 \cdot kip \cdot ft$

$V_{DC} := 36.3 kip$ $\quad V_{DW} := 4.9 kip$ $\quad V_{LL} := 107.7 kip$

$V_u := 1.25 \cdot V_{DC} + 1.5 \cdot V_{DW} + 1.75 \cdot V_{LL} \cdot g_v = 142.6 \cdot kip$

Load Cases:

$M_{4\_STR\_I} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.75 \cdot M_{LL4} = 1504 \cdot kip \cdot ft$

$M_{4\_STR\_I\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.75 \cdot M_{LL4neg} = -1650.2 \cdot kip \cdot ft$

$M_{4\_STR\_III} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.4 \cdot M_{WS4} = 562.5 \cdot kip \cdot ft$

$M_{4\_STR\_III\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.4 \cdot M_{WS4} = -703.1 \cdot kip \cdot ft$

$M_{4\_STR\_V} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.35 \cdot M_{LL4} + 0.4 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 1288.8 \cdot kip \cdot ft$

$M_{4\_STR\_V\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.35 \cdot M_{LL4neg} + 0.4 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -1433.7 \cdot kip \cdot ft$

$M_{4\_SRV\_I} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.0 \cdot M_{LL4} + 0.3 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 977.5 \cdot kip \cdot ft$

$M_{4\_SRV\_I\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.0 \cdot M_{LL4neg} + 0.3 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -1090.6 \cdot kip \cdot ft$

$M_{4\_SRV\_II} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.3 \cdot M_{LL4} = 1138.9 \cdot kip \cdot ft$

$M_{4\_SRV\_II\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.3 \cdot M_{LL4neg} = -1252.9 \cdot kip \cdot ft$

11. FLEXURAL STRENGTH

The flexural resistance shall be determined as specified in LRFD Design Article 6.10.6.2. Determine Stringer Plastic Moment Capacity First.

LFRD Appendix D6 Plastic Moment

Find location of PNA:

Forces:

$P_{rt} := A_{rt} \cdot F_s = 81.4 \cdot kip$ $\quad P_s := 0.85 \cdot f_c \cdot b_{eff} \cdot t_{slab} = 1190 \cdot kip$ $\quad P_w := F_y \cdot D_w \cdot t_w = 863.5 \cdot kip$

$P_{rb} := A_{rb} \cdot F_s = 115.5 \cdot kip$ $\quad P_c := F_y \cdot b_{tf} \cdot t_{tf} = 425.5 \cdot kip$ $\quad P_t := F_y \cdot b_{bf} \cdot t_{bf} = 425.5 \cdot kip$

$$PNA_{pos} := \begin{cases} \text{"case 1"} & \text{if } (P_t + P_w) \ge (P_c + P_s + P_{rt} + P_{rb}) \\ \text{otherwise} & \begin{cases} \text{"case 2"} & \text{if } \left[(P_t + P_w + P_c) \ge (P_s + P_{rt} + P_{rb})\right] \\ \text{otherwise} & \begin{cases} \text{"case 3"} & \text{if } \left[(P_t + P_w + P_c) \ge \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} & \begin{cases} \text{"case 4"} & \text{if } \left[(P_t + P_w + P_c + P_{rb}) \ge \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} & \begin{cases} \text{"case 5"} & \text{if } \left[(P_t + P_w + P_c + P_{rb}) \ge \left(\frac{c_{rt}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} & \begin{cases} \text{"case 6"} & \text{if } (P_t + P_w + P_c + P_{rb} + P_{rt}) \ge \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \\ \text{"case 7"} & \text{if } (P_t + P_w + P_c + P_{rb} + P_{rt}) \le \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \text{ otherwise} \end{cases} \end{cases} \end{cases} \end{cases} \end{cases} \end{cases}$$

$PNA_{pos} = \text{"case 2"}$

$$PNA_{neg} := \begin{cases} \text{"case 1"} & \text{if } (P_c + P_w) \ge (P_t + P_{rt} + P_{rb}) \\ \text{"case 2"} & \text{if } \left[(P_t + P_w + P_c) \ge (P_{rt} + P_{rb})\right] \text{ otherwise} \end{cases}$$

$PNA_{neg} = \text{"case 1"}$

Calculate Y, Dp, and Mp: $D := D_w$ $t_s := t_{slab}$ $t_h := 0$ $C_{rt} := c_{rt}$ $C_{rb} := c_{rb}$

Case I : Plastic Nuetral Axis in the Steel Web

$$Y_1 := \frac{D}{2} \cdot \left(\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right)$$

$$D_{P1} := t_s + t_h + t_{tf} + Y_1$$

$$M_{P1} := \frac{P_w}{2D} \cdot \left[{Y_1}^2 + (D - Y_1)^2\right] + \left[P_s \cdot \left(Y_1 + \frac{t_s}{2} + t_{tf} + t_h\right) + P_{rt} \cdot (t_s - C_{rt} + t_{tf} + Y_1 + t_h) + P_{rb} \cdot (t_s - C_{rb} + t_{tf} + Y_1 + t_h) \ldots \atop + P_c \cdot \left(Y_1 + \frac{t_{tf}}{2}\right) + P_t \cdot \left(D - Y_1 + \frac{t_{bf}}{2}\right)\right]$$

$$Y_{1neg} := \left(\frac{D}{2}\right) \cdot \left[1 + \frac{(P_c - P_t - P_{rt} - P_{rb})}{P_w}\right]$$

$$D_{p1neg} := t_s + t_h + t_{tf} + Y_{1neg}$$

$$D_{CP1neg} := \left(\frac{D}{2 \cdot P_w}\right) \cdot (P_t + P_w + P_{rb} + P_{rt} - P_c)$$

$$M_{p1neg} := \left[\left(\frac{P_w}{2 \cdot D}\right)\cdot\left[{Y_{1neg}}^2 + \left(D_w - Y_{1neg}\right)^2\right] + P_{rt}\cdot\left(t_s - C_{rt} + t_{tf} + Y_{1neg} + t_h\right) + P_{rb}\cdot\left(t_s - C_{rb} + t_{tf} + Y_{1neg} + t_h\right) \ldots \atop + P_t\cdot\left(D - Y_{1neg} + \frac{t_{bf}}{2}\right) + P_c\cdot\left(Y_{1neg} + \frac{t_{tf}}{2}\right)\right]$$

<u>Case II:</u> Plastic Nuetral Axis in the Steel Top Flange

$$Y_2 := \frac{t_{tf}}{2}\cdot\left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right) \qquad D_{P2} := t_s + t_h + Y_2$$

$$M_{P2} := \frac{P_c}{2t_{tf}}\cdot\left[{Y_2}^2 + \left(t_{tf} - Y_2\right)^2\right] + \left[P_s\cdot\left(Y_2 + \frac{t_s}{2} + t_h\right) + P_{rt}\cdot\left(t_s - C_{rt} + t_h + Y_2\right) + P_{rb}\cdot\left(t_s - C_{rb} + t_h + Y_2\right) \ldots \atop + P_w\cdot\left(\frac{D}{2} + t_{tf} - Y_2\right) + P_t\cdot\left(D - Y_2 + \frac{t_{bf}}{2} + t_{tf}\right)\right]$$

$$Y_{2neg} := \left(\frac{t_{tf}}{2}\right)\cdot\left[1 + \frac{\left(P_w + P_c - P_{rt} - P_{rb}\right)}{P_t}\right] \qquad D_{P2neg} := t_s + t_h + Y_{2neg} \qquad D_{CP2neg} := D$$

$$M_{p2neg} := \left(\frac{P_t}{2 \cdot t_{tf}}\right)\cdot\left[{Y_{2neg}}^2 + \left(t_{tf} - Y_{2neg}\right)^2\right] + \left[P_{rt}\cdot\left(t_s - C_{rt} + t_h + Y_{2neg}\right) + P_{rb}\cdot\left(t_s - C_{rb} + t_h + Y_{2neg}\right) \ldots \atop + P_w\cdot\left(t_{tf} - Y_{2neg} + \frac{D}{2}\right) + P_c\cdot\left(\left|t_s + t_h - Y_{2neg} + \frac{t_{tf}}{2}\right|\right)\right]$$

<u>Case III:</u> Plastic Nuetral Axis in the Concrete Deck Below the Bottom Reinforcing

$$Y_3 := t_s\cdot\left(\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right) \qquad D_{P3} := Y_3$$

$$M_{P3} := \frac{P_s}{2t_s}\cdot\left({Y_3}^2\right) + \left[P_{rt}\cdot\left(Y_3 - C_{rt}\right) + P_{rb}\cdot\left(C_{rb} - Y_3\right) + P_c\cdot\left(\frac{t_{tf}}{2} + t_s + t_h - Y_3\right) + P_w\cdot\left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_3\right) \ldots \atop + P_t\cdot\left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_3\right)\right]$$

<u>Case IV:</u> Plastic Nuetral Axis in the Concrete Deck in the bottom reinforcing layer

$$Y_4 := C_{rb} \qquad D_{P4} := Y_4$$

$$M_{P4} := \frac{P_s}{2t_s}\cdot\left({Y_4}^2\right) + \left[P_{rt}\cdot\left(Y_4 - C_{rt}\right) + P_c\cdot\left(\frac{t_{tf}}{2} + t_h + t_s - Y_4\right) + P_w\cdot\left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_4\right) \ldots \atop + P_t\cdot\left(D + \frac{t_{bf}}{2} + t_{tf} + t_h + t_s - Y_4\right)\right]$$

<u>Case V:</u> Plastic Nuetral Axis in the Concrete Deck between top and bot reinforcing layers

$$Y_5 := t_s\cdot\left(\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right) \qquad D_{P5} := Y_5$$

$$M_{P5} := \frac{P_s}{2t_s}\cdot\left({Y_5}^2\right) + \left[P_{rt}\cdot\left(Y_5 - C_{rt}\right) + P_{rb}\cdot\left[\left(t_s - C_{rb}\right) - Y_5\right] + P_c\cdot\left(\frac{t_{tf}}{2} + t_s + t_h - Y_5\right) + P_w\cdot\left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_5\right) \ldots \atop + P_t\cdot\left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_5\right)\right]$$

$$Y_{pos} := \begin{cases} Y_1 & \text{if } PNA_{pos} = \text{"case 1"} \\ Y_2 & \text{if } PNA_{pos} = \text{"case 2"} \\ Y_3 & \text{if } PNA_{pos} = \text{"case 3"} \\ Y_4 & \text{if } PNA_{pos} = \text{"case 4"} \\ Y_5 & \text{if } PNA_{pos} = \text{"case 5"} \end{cases} \qquad D_{Ppos} := \begin{cases} D_{P1} & \text{if } PNA_{pos} = \text{"case 1"} \\ D_{P2} & \text{if } PNA_{pos} = \text{"case 2"} \\ D_{P3} & \text{if } PNA_{pos} = \text{"case 3"} \\ D_{P4} & \text{if } PNA_{pos} = \text{"case 4"} \\ D_{P5} & \text{if } PNA_{pos} = \text{"case 5"} \end{cases} \qquad M_{Ppos} := \begin{cases} M_{P1} & \text{if } PNA_{pos} = \text{"case 1"} \\ M_{P2} & \text{if } PNA_{pos} = \text{"case 2"} \\ M_{P3} & \text{if } PNA_{pos} = \text{"case 3"} \\ M_{P4} & \text{if } PNA_{pos} = \text{"case 4"} \\ M_{P5} & \text{if } PNA_{pos} = \text{"case 5"} \end{cases}$$

$$Y_{pos} = 0.3\cdot in \qquad D_{Ppos} = 8.3\cdot in \qquad M_{Ppos} = 2793\cdot kip\cdot ft$$

Dp = distance from the top of slab of composite section to the neutral axis at the plastic moment (neglect positive moment reinforcement in the slab).

$$Y_{neg} := \begin{cases} Y_{1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ Y_{2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases} \qquad D_{Pneg} := \begin{cases} D_{p1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ D_{P2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases} \qquad M_{Pneg} := \begin{cases} M_{p1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ M_{p2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases}$$

$$Y_{neg} = 12.1\cdot in \qquad D_{Pneg} = 20.9\cdot in \qquad M_{Pneg} = 23955\cdot kip\cdot in$$

Depth of web in compression at the plastic moment [D6.3.2]:

$A_t := b_{bf}\cdot t_{bf} \qquad A_c := b_{tf}\cdot t_{tf}$

$$D_{cppos} := \frac{D}{2}\left(\frac{F_y\cdot A_t - F_y\cdot A_c - 0.85\cdot f_c\cdot A_{slab} - F_s\cdot A_r}{F_y\cdot A_w} + 1\right)$$

$$D_{cppos} := \begin{cases} (0in) & \text{if } PNA_{pos} \neq \text{"case 1"} \\ (0in) & \text{if } (D_{cppos} < 0) \\ D_{cppos} & \text{if } PNA_{pos} = \text{"case 1"} \end{cases} \qquad D_{cpneg} := \begin{cases} D_{CP1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ D_{CP2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases}$$

$$D_{cppos} = 0\cdot in \qquad D_{cpneg} = 19.3\cdot in$$

Positive Flexural Compression Check:

From LRFD Article 6.10.2

Check for compactness:

Web Proportions: $\frac{D_w}{t_w} \le 150 = 1$

Web slenderness Limit: $2\cdot\frac{D_{cppos}}{t_w} \le 3.76\cdot\sqrt{\frac{E_s}{F_y}} = 1$ S 6.10.6.2.2

Therefore Section is considered compact and shall satisfy the requirements of Article 6.10.7.1.

$$M_n := \begin{cases} M_{Ppos} & \text{if } D_{Ppos} \le 0.1\cdot D_t \\ M_{Ppos}\cdot\left(1.07 - 0.7\cdot\frac{D_{Ppos}}{D_t}\right) & \text{otherwise} \end{cases} \qquad M_n = 2592.3\cdot kip\cdot ft$$

Negative Moment Capacity Check (Appendix A6):

Web Slenderness: $D_t = 40.9\cdot in \qquad D_{cneg} := D_t - y_{cr} - t_{bf} = 25.1\cdot in$

$$\frac{2\cdot D_{cneg}}{t_w} < 5.7\cdot\sqrt{\frac{E_s}{F_y}} = 1$$ S Appendix A6 (for skew less than 20 deg).

Moment ignoring concrete:

$M_{yt} := F_y \cdot S_{botcr} = 12036.4 \cdot kip \cdot in$ $\quad M_{yc} := F_s \cdot S_{topcr} = 32194.6 \cdot kip \cdot in$

$M_y := \min(M_{yc}, M_{yt}) = 12036.4 \cdot kip \cdot in$

Web Compactness:

Check for Permanent Deformations (6.10.4.2):

$$D_n := \max(t_{slab} + t_{tf} + D_w - y_c, y_c - t_{slab} - t_{tf}) = 27 \cdot in$$

$$Gov := \text{if}(y_c - t_{slab} - t_{tf}, y_c - c_{rt}, D_n) = 9.7 \cdot in$$

$$f_n := \left|M_{4\_SRV\_II\_neg}\right| \cdot \frac{Gov}{I_z} = 10.4 \cdot ksi$$ Steel stress on side of Dn

$$\rho := \min\left(1.0, \frac{F_y}{f_n}\right) = 1 \qquad \beta := 2 \cdot D_n \cdot \frac{t_w}{A_{tf}} = 3.5 \qquad R_h := \frac{\left[12 + \beta \cdot \left(3\rho - \rho^3\right)\right]}{(12 + 2 \cdot \beta)} = 1$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_s}{F_y}}$$

$$\lambda_{PWdcp} := \min\left[\lambda_{rw} \cdot \frac{D_{cpneg}}{D_{cneg}}, \frac{\sqrt{\frac{E_s}{F_y}}}{\left(0.54 \cdot \frac{M_{Pneg}}{R_h \cdot M_y} - 0.09\right)^2}\right] = 24.8$$

$$2 \cdot \frac{D_{cpneg}}{t_w} \le \lambda_{PWdcp} = 0$$

Web Plastification: $R_{pc} := \frac{M_{Pneg}}{M_{yc}} = 0.7 \qquad R_{pt} := \frac{M_{Pneg}}{M_{yt}} = 2$

Flexure Factor: $\phi_f := 1.0$

Tensile Limit: $M_{r\_neg\_t} := \phi_f \cdot R_{pt} \cdot M_{yt} = 1996.3 \cdot kip \cdot ft$

Compressive Limit:

Local Buckling Resistance:

$$\lambda_f := \frac{b_{bf}}{2 \cdot t_{bf}} = 7.8 \qquad \lambda_{rf} := 0.95 \cdot \sqrt{0.76 \cdot \frac{E_s}{F_y}} = 19.9$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} = 9.2 \qquad F_{yresid} := \max\left(\min\left(0.7 \cdot F_y, R_h \cdot F_y \cdot \frac{S_{topcr}}{S_{botcr}}, F_y\right), 0.5 \cdot F_y\right) = 35.0 \cdot ksi$$

$$M_{ncLB} := \begin{cases} (R_{pc} \cdot M_{yc}) & \text{if } \lambda_f \le \lambda_{pf} \\ \left[R_{pc} \cdot M_{yc} \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{topcr}}{R_{pc} \cdot M_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]\right] & \text{otherwise} \end{cases} \qquad M_{ncLB} = 1996.3 \cdot kip \cdot ft$$

Lateral Torsional Buckling Resistance:

$$L_b := \frac{(L_{str})}{2 \cdot 3} = 13.2 \cdot ft$$ Inflection point assumed to be at 1/6 span

$$r_t := \frac{b_{bf}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_{cneg} \cdot t_w}{b_{bf} \cdot t_{bf}}\right)}} = 2.7 \cdot in$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_s}{F_y}} = 64.4 \cdot in \qquad h := D + t_{bf} = 32.1 \cdot in \qquad C_b := 1.0$$

$$J_b := \frac{D \cdot t_w^{\,3}}{3} + \frac{b_{bf} \cdot t_{bf}^{\,3}}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{bf}}{b_{bf}}\right) + \frac{b_{tf} \cdot t_{tf}^{\,3}}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{tf}}{b_{tf}}\right) = 4.7 \cdot in^4$$

$$L_r := 1.95 \cdot r_t \cdot \frac{E_s}{F_{yresid}} \cdot \sqrt{\frac{J_b}{S_{botcr} \cdot h}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{F_{yresid}}{E_s} \cdot \frac{S_{botcr} \cdot h}{J_b}\right)^2}} = 266.6 \cdot in$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J_b}{S_{botcr} \cdot h} \cdot \left(\frac{L_b}{r_t}\right)^2} = 87.5 \cdot ksi$$

$$M_{ncLTB} := \begin{cases} \left(R_{pc} \cdot M_{yc}\right) & \text{if } L_b \le L_p \\ \min\left[C_b \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{botcr}}{R_{pc} \cdot M_{yc}}\right) \cdot \frac{\left(L_b - L_p\right)}{\left(L_r - L_p\right)}\right] \cdot R_{pc} \cdot M_{yc}, R_{pc} \cdot M_{yc}\right] & \text{if } L_p < L_b \le L_r \\ \min\left(F_{cr} \cdot S_{botcr}, R_{pc} \cdot M_{yc}\right) & \text{if } L_b > L_r \end{cases}$$

$$M_{ncLTB} = 1390.9 \cdot kip \cdot ft$$

$$M_{r\_neg\_c} := \phi_f \cdot \min\left(M_{ncLB}, M_{ncLTB}\right) = 1390.9 \cdot kip \cdot ft$$

Governing negative moment capacity: $M_{r\_neg} := \min\left(M_{r\_neg\_t}, M_{r\_neg\_c}\right) = 1390.9 \cdot kip \cdot ft$

12. FLEXURAL STRENGTH CHECKS

Phase 1: First, check the stress due to the dead load on the steel section only. (LRFD 6.10.3 - Constructability Requirements

Reduction factor for construction $\phi_{const} := 0.9$

Load Combination for construction $1.25 \cdot M_{DC}$

Max Moment applied, Phase 1: (at midspan)

$$M_{int\_P1} := 1.25\, M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 521.7 \cdot kip \cdot ft \quad \text{(Interior)}$$

$$M_{ext\_P1} := 1.25\, M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 532.5 \cdot kip \cdot ft \quad \text{(Exterior)}$$

Maximum Stress, Phase 1:

$$f_{int\_P1} := \frac{M_{int\_P1} \cdot y_{steel}}{I_{zsteel}} = 17.7 \cdot ksi \quad \text{(Interior)}$$

$$f_{ext\_P1} := \frac{M_{ext\_P1} \cdot y_{steel}}{I_{zsteel}} = 18.1 \cdot ksi \quad \text{(Exterior)}$$

Stress limits:

$$f_{P1\_max} := \phi_{const} \cdot F_y$$

$$f_{int\_P1} \le f_{P1\_max} = 1 \qquad f_{ext\_P1} \le f_{P1\_max} = 1$$

Phase 2: Second, check the stress due to dead load on the composite section (with barriers added)

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.25 \cdot M_{DC}$

Max Moment applied, Phase 2: (at midspan) $M_{2\_STR\_I} = 746.5 \cdot kip \cdot ft$

Capacity for positive flexure: $M_n = 2592.3 \cdot kip \cdot ft$

Check Moment: $M_{2\_STR\_I} \le \phi_{const} \cdot M_n = 1$

Phase 3: Next, check the flexural stress on the stringer during transport and picking, to ensure no cracking.

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.5 \cdot M_{DC}$ when dynamic construction loads are involved (Section 10).

Loads and stresses on stringer during transport and picking: $M_{3\_STR\_I\_neg} = 73 \cdot kip \cdot ft$

Concrete rupture stress $f_r := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.5 \cdot ksi$

Concrete stress during construction not to exceed:

$f_{cmax} := \phi_{const} \cdot f_r = 0.5 \cdot ksi$

$$f_{cconst} := \frac{M_{3\_STR\_I\_neg} \cdot y_c}{I_z \cdot n} = 0.1 \cdot ksi$$

$f_{cconst} \le f_{cmax} = 1$

Phase 4: Check flexural capacity under dead load and live load for fully installed continuous composite girders.

Strength I Load Combination $\phi_f := 1.0$

$M_{4\_STR\_I} = 1504 \cdot kip \cdot ft$ $M_{4\_STR\_I\_neg} = -1650.2 \cdot kip \cdot ft$

$M_{4\_STR\_I} \le \phi_f \cdot M_n = 1$ $\left|M_{4\_STR\_I\_neg}\right| \le M_{r\_neg} = 0$

Strength III Load Combination

$M_{4\_STR\_III} = 562.5 \cdot kip \cdot ft$ $M_{4\_STR\_III\_neg} = -703.1 \cdot kip \cdot ft$

$M_{4\_STR\_III} \le \phi_f \cdot M_n = 1$ $\left|M_{4\_STR\_III\_neg}\right| \le M_{r\_neg} = 1$

Strength V Load Combination

$M_{4\_STR\_V} = 1288.8 \cdot kip \cdot ft$ $M_{4\_STR\_V\_neg} = -1433.7 \cdot kip \cdot ft$

$M_{4\_STR\_V} \le \phi_f \cdot M_n = 1$ $\left|M_{4\_STR\_V\_neg}\right| \le M_{r\_neg} = 0$

13. FLEXURAL SERVICE CHECKS

Check service load combinations for the fully continuous beam with live load (Phase 4):

under Service II for stress limits - $M_{4\_SRV\_II} = 1138.9 \cdot kip \cdot ft$

$M_{4\_SRV\_II\_neg} = -1252.9 \cdot kip \cdot ft$

under Service I for cracking - $M_{4\_SRV\_I\_neg} = -1090.6 \cdot kip \cdot ft$

Ignore positive moment for Service I as there is no tension in the concrete in this case.

Service Load Stress Limits:

Top Flange: $f_{tfmax} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$

Bottom Flange: $f_{bfmax} := f_{tfmax} = 47.5 \cdot ksi$

Concrete (Negative bending only): $f_r = 0.5 \cdot ksi$

Service Load Stresses, Positive Moment:

Top Flange: $f_{SRVII\_tf} := M_{4\_SRV\_II} \cdot \frac{(y_c - t_{slab})}{I_z} = 5 \cdot ksi$

$f_{SRVII\_tf} \le f_{tfmax} = 1$

Bottom Flange: $f_{bfs2} := M_{4\_SRV\_II} \cdot \frac{(t_{slab} + t_{tf} + D_w + t_{bf} - y_c)}{I_z} = 27.2 \cdot ksi$

$f_l := 0 \qquad f_{bfs2} + \frac{f_l}{2} \le f_{bfmax} = 1$

Service Load Stresses, Negative Moment:

Top (Concrete): $f_{con.neg} := \frac{M_{4\_SRV\_I\_neg} \cdot y_{cneg}}{n \cdot I_{zneg}} = -2 \cdot ksi$ Using Service I Loading

$|f_{con.neg}| \le |f_r| = 0$

Bottom Flange: $f_{bfs2.neg} := \frac{M_{4\_SRV\_I\_neg} \cdot (t_{slab} + t_{tf} + D_w + t_{bf} - y_{cneg})}{I_{zneg}} = -45.7 \cdot ksi$

$f_{bfs2.neg} \le f_{bfmax} = 1$

Check LL Deflection:

$\Delta_{DT} := 1.104 \cdot in$ from independent Analysis - includes 100% design truck (w/impact), or 25% design truck (w/impact) + 100% lane load

$DF_\delta := \frac{3}{12} = 0.3$ Deflection distribution factor = (no. lanes)/(no. stringers)

$\frac{L_{str}}{\Delta_{DT} \cdot DF_\delta} = 3456.5$ Equivalent X, where L/X = Deflection*Distribution Factor

$\frac{L_{str}}{\Delta_{DT} \cdot DF_\delta} \ge 800 = 1$

14. SHEAR STRENGTH

Shear Capacity based on AASHTO LRFD 6.10.9

Nominal resistance of unstiffened web:

$F_y = 50.0 \cdot ksi \qquad D_w = 31.4 \cdot in \qquad t_w = 0.6 \cdot in \qquad \phi_v := 1.0 \qquad k := 5$

$V_p := 0.58 \cdot F_y \cdot D_w \cdot t_w = 500.8 \cdot kip$

$$C_1 := \begin{cases} 1.0 & \text{if } \dfrac{D_w}{t_w} \le 1.12\cdot\sqrt{\dfrac{E_s\cdot k}{F_y}} \\ \left[\dfrac{1.57}{\left(\dfrac{D_w}{t_w}\right)^2}\cdot\left(\dfrac{E_s\cdot k}{F_y}\right)\right] & \text{if } \dfrac{D_w}{t_w} > 1.40\cdot\sqrt{\dfrac{E_s\cdot k}{F_y}} \\ \left(\dfrac{1.12}{\dfrac{D_w}{t_w}}\cdot\sqrt{\dfrac{E_s\cdot k}{F_y}}\right) & \text{otherwise} \end{cases} \qquad C_1 = 1$$

$$V_n := C_1\cdot V_p = 500.8\cdot kip$$

$$V_u \le \phi_v\cdot V_n = 1$$

15. FATIGUE LIMIT STATES:

Fatigue check shall follow LRFD Article 6.10.5. Moments used for fatigue calculations were found using an outside finite element analysis program.

First check Fatigue I (infinite life); then find maximum single lane ADTT for Fatigue II if needed.

Fatigue Stress Limits:

$\Delta F_{TH\_1} := 16\cdot ksi$ Category B: non-coated weathering steel

$\Delta F_{TH\_2} := 12\cdot ksi$ Category C': Base metal at toe of transverse stiffener fillet welds

$\Delta F_{TH\_3} := 10\cdot ksi$ Category C: Base metal at shear connectors

Fatigue Moment Ranges at Detail Locations (from analysis):

$M_{FAT\_B} := 301\cdot kip\cdot ft$ $M_{FAT\_CP} := 285.7\cdot kip\cdot ft$ $M_{FAT\_C} := 207.1 kip\cdot ft$

$\gamma_{FATI} := 1.5$ $\gamma_{FATII} := 0.75$ $n_{fat} := \begin{cases} 2 & \text{if } L_{str} \le 40\cdot ft \\ 1.0 & \text{otherwise} \end{cases}$

Constants to use for detail checks:

$ADTT_{SL\_INF\_B} := 860$ $A_{FAT\_B} := 120\cdot 10^8$

$ADTT_{SL\_INF\_CP} := 660$ $A_{FAT\_CP} := 44\cdot 10^8$

$ADTT_{SL\_INF\_C} := 1290$ $A_{FAT\_C} := 44\cdot 10^8$

Category B Check: Stress at Bottom Flange, Fatigue I

$$f_{FATI\_B} := \frac{\gamma_{FATI}\cdot M_{FAT\_B}\cdot\left(t_{slab} + t_{tf} + D_w + t_{bf} - y_c\right)}{I_z} = 10.8\cdot ksi$$

$$f_{FATI\_B} \le \Delta F_{TH\_1} = 1$$

$$f_{FATII\_B} := \frac{\gamma_{FATII}}{\gamma_{FATI}}\cdot f_{FATI\_B} = 5.4\cdot ksi$$

$$ADDT_{SL\_B\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_B}}{n_{fat}} & \text{if } f_{FATI\_B} \le \Delta F_{TH\_1} \\[2ex] \dfrac{A_{FAT\_B} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_B}^{\ 3}} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_B\_MAX} = 860$$

Category C' Check: Stress at base of transverse stiffener (top of bottom flange)

$$f_{FATI\_CP} := \gamma_{FATI} \cdot M_{FAT\_CP} \cdot \frac{\left(t_{slab} + t_{tf} + D_w - y_c\right)}{I_z} = 10 \cdot ksi$$

$$f_{FATI\_CP} \le \Delta F_{TH\_2} = 1$$

$$f_{FATII\_CP} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_CP} = 5 \cdot ksi$$

$$ADTT_{SL\_CP\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_CP}}{n_{fat}} & \text{if } f_{FATI\_CP} \le \Delta F_{TH\_2} \\[2ex] \dfrac{A_{FAT\_CP} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_CP}^{\ 3}} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_CP\_MAX} = 660$$

Category C Check: Stress at base of shear connectors (top of top flange)

$$f_{FATI\_C} := \gamma_{FATI} \cdot M_{FAT\_C} \cdot \frac{\left(y_c - t_{slab}\right)}{I_z} = 1.4 \cdot ksi$$

$$f_{FATI\_C} \le \Delta F_{TH\_3} = 1$$

$$f_{FATII\_C} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_C} = 0.7 \cdot ksi$$

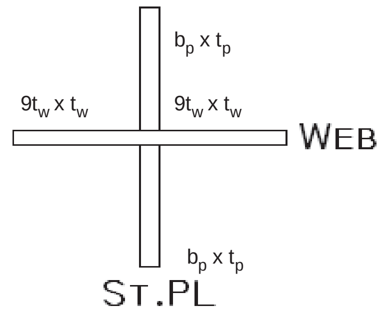
$$ADTT_{SL\_C\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_C}}{n_{fat}} & \text{if } f_{FATI\_C} \le \Delta F_{TH\_3} \\[2ex] \dfrac{A_{FAT\_C} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_C}^{\ 3}} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_C\_MAX} = 1290$$

FATIGUE CHECK: $ADTT_{SL\_MAX} := \min\left(ADTT_{SL\_B\_MAX}, ADTT_{SL\_CP\_MAX}, ADTT_{SL\_C\_MAX}\right)$

Ensure that single lane ADTT is less than $ADTT_{SL\_MAX} = 660$

If not, then the beam requires redesign.

16. BEARING STIFFENERS



Using LRFD Article 6.10.11 for stiffeners:

$t_p := \frac{5}{8}in$ $\quad b_p := 5in$ $\quad \phi_b := 1.0$ $\quad t_{p\_weld} := \left(\frac{5}{16}\right)in$

*Check min weld size

Projecting Width Slenderness Check:

$$b_p \le 0.48t_p \cdot \sqrt{\frac{E_s}{F_y}} = 1$$

Stiffener Bearing Resistance:

$A_{pn} := 2 \cdot (b_p - t_{p\_weld}) \cdot t_p$ $\qquad A_{pn} = 5.9 \cdot in^2$

$R_{sb\_n} := 1.4 \cdot A_{pn} \cdot F_y$ $\qquad R_{sb\_n} = 410.2 \cdot kip$

$R_{sb\_r} := \phi_b \cdot R_{sb\_n}$ $\qquad R_{sb\_r} = 410.2 \cdot kip$

$R_{DC} := 26.721kip$ $\quad R_{DW} := 2.62kip$ $\quad R_{LL} := 53.943kip$

$\phi_{DC\_STR\_I} := 1.25$ $\quad \phi_{DW\_STR\_I} := 1.5$ $\quad \phi_{LL\_STR\_I} := 1.75$

$R_u := \phi_{DC\_STR\_I} \cdot R_{DC} + \phi_{DW\_STR\_I} \cdot R_{DW} + \phi_{LL\_STR\_I} \cdot R_{LL}$ $\qquad R_u = 131.7 \cdot kip$

$R_u \le R_{sb\_r} = 1$

Weld Check:

$throat := t_{p\_weld} \cdot \frac{\sqrt{2}}{2}$ $\qquad throat = 0.2 \cdot in$

$L_{weld} := D_w - 2 \cdot 3in$ $\qquad L_{weld} = 25.4 \cdot in$

$A_{eff\_weld} := throat \cdot L_{weld}$ $\qquad A_{eff\_weld} = 5.6 \cdot in^2$

$F_{exx} := 70ksi$ $\qquad \phi_{e2} := 0.8$

$R_{r\_weld} := 0.6 \cdot \phi_{e2} \cdot F_{exx}$ $\qquad R_{r\_weld} = 33.6 \cdot ksi$

$R_{u\_weld} := \frac{R_u}{4 \cdot A_{eff\_weld}}$ $\qquad R_{u\_weld} = 5.9 \cdot ksi$

$R_{u\_weld} \le R_{u\_weld} = 1$

Axial Resistance of Bearing Stiffeners: $\qquad \phi_c := 0.9$

$A_{eff} := (2 \cdot 9 \cdot t_w + t_p) \cdot t_w + 2 \cdot b_p \cdot t_p$ $\qquad A_{eff} = 12 \cdot in^2$

$L_{eff} := 0.75 \cdot D_w$ $\qquad L_{eff} = 23.6 \cdot in$

$I_{xp} := \frac{2 \cdot 9 \cdot t_w \cdot t_w^3}{12} + \frac{t_p \cdot (2 \cdot b_p + t_w)^3}{12}$ $\qquad I_{xp} = 61.3 \cdot in^4$

$I_{yp} := \frac{t_w \cdot (t_p + 2 \cdot 9 \cdot t_w)^3}{12} + \frac{2b_p \cdot t_p^3}{12}$ $\qquad I_{yp} = 53.6 \cdot in^4$

$r_p := \sqrt{\frac{\min(I_{xp}, I_{yp})}{A_{eff}}}$ $\qquad r_p = 2.1 \cdot in$

$Q := 1$ $\quad$ for bearing stiffeners $\qquad K_p := 0.75$

$P_o := Q \cdot F_y \cdot A_{eff} = 601.9 \cdot kip$

$$P_e := \frac{\pi^2 E_s \cdot A_{eff}}{\left(K_p \cdot \frac{L_{eff}}{r_p}\right)^2} = 49214.3 \cdot kip$$

$$P_n := \begin{cases} \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o & \text{if } \left(\frac{P_e}{P_o}\right) \geq 0.44 \\ 0.877 \cdot P_e & \text{otherwise} \end{cases} \qquad P_n = 598.9 \cdot kip$$

$P_r := \phi_c \cdot P_n \qquad P_r = 539 \cdot kip \qquad R_u \leq P_r = 1$

17. SHEAR CONNECTORS:

Shear Connector design to follow LRFD 6.10.10.

Stud Properties:

$d_s := \frac{7}{8} \cdot in$ Diameter $\qquad h_s := 6in$ Height of Stud $\qquad \frac{h_s}{d_s} \geq 4 = 1$

$c_s := t_{slab} - h_s \qquad c_s \geq 2in = 1$

$s_s := 3.5in$ Spacing $\qquad s_s \geq 4d_s = 1$

$n_s := 3$ Studs per row $\qquad \frac{\left[b_{tf} - s_s \cdot (n_s - 1) - d_s\right]}{2} \geq 1.0in = 1$

$A_{sc} := \pi \cdot \left(\frac{d_s}{2}\right)^2 \qquad A_{sc} = 0.6 \cdot in^2$

$F_u := 60ksi$

Fatigue Resistance:

$Z_r := 5.5 \cdot d_s^{\,2} \cdot \frac{kip}{in^2} \qquad Z_r = 4.2 \cdot kip \qquad Q_{slab} := A_{slab} \cdot (y_c - y_{slab}) \qquad Q_{slab} = 364.9 \cdot in^3$

$V_f := 47.0kip$

$V_{fat} := \frac{V_f \cdot Q_{slab}}{I_z} = 1.2 \cdot \frac{kip}{in}$

$p_s := \frac{n_s \cdot Z_r}{V_{fat}} = 10.3 \cdot in \qquad 6 \cdot d_s \leq p_s \leq 24in = 1$

Strength Resistance:

$\phi_{sc} := 0.85$

$f_c = 5 \cdot ksi$

$E_c := 33000 \cdot 0.15^{1.5} \cdot \sqrt{f_c ksi} = 4286.8 \cdot ksi$

$Q_n := \min\left(0.5 \cdot A_{sc} \cdot \sqrt{f_c \cdot E_c}, A_{sc} \cdot F_u\right) \qquad Q_n = 36.1 \cdot kip$

$Q_r := \phi_{sc} \cdot Q_n \qquad Q_r = 30.7 \cdot kip$

$P_{simple} := \min\left(0.85 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right) \qquad P_{simple} = 1190 \cdot kip$

$P_{cont} := P_{simple} + \min\left(0.45 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right) \qquad P_{cont} = 1820 \cdot kip$

$n_{lines} := \frac{P_{cont}}{Q_r \cdot n_s} \qquad n_{lines} = 19.8$

Find required stud spacing along the girder (varies as applied shear varies)

$i := 0..23$

$$x := \begin{pmatrix} 0.00 \\ 1.414 \\ 4.947 \\ 8.480 \\ 12.013 \\ 15.546 \\ 19.079 \\ 22.612 \\ 26.145 \\ 29.678 \\ 33.210 \\ 33.917 \\ 34.624 \\ 36.037 \\ 36.743 \\ 40.276 \\ 43.809 \\ 47.342 \\ 50.875 \\ 54.408 \\ 57.941 \\ 61.474 \\ 65.007 \\ 67.833 \end{pmatrix} \cdot ft \quad V_{fi} := \begin{pmatrix} 61.5 \\ 59.2 \\ 56.8 \\ 54.4 \\ 52.0 \\ 49.5 \\ 47.1 \\ 44.7 \\ 42.7 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 42.3 \\ 44.2 \\ 46.6 \\ 49.1 \\ 51.5 \\ 53.9 \\ 56.3 \\ 58.7 \\ 61.5 \end{pmatrix} \cdot kip$$

$V_{fati} := \dfrac{V_{fi} \cdot Q_{slab}}{I_z} =$

	0
0	1.6
1	1.5
2	1.5
3	1.4
4	1.4
5	1.3
6	1.2
7	1.2
8	1.1
9	1.1
10	1.1
11	1.1
12	1.1
13	1.1
14	1.1
15	...

$\cdot \dfrac{kip}{in}$

$P_{max} := \dfrac{n_s \cdot Z_r}{V_{fati}} =$

	0
0	7.8
1	8.2
2	8.5
3	8.9
4	9.3
5	9.8
6	10.2
7	10.8
8	11.3
9	11.9
10	11.9
11	11.9
12	11.9
13	11.9
14	11.9
15	...

$\cdot in$

$\min(P_{max}) = 7.8 \cdot in$

$\max(P_{max}) = 11.9 \cdot in$

18. SLAB PROPERTIES

This section details the geometric and material properties of the deck. Because the equivalent strip method is used in accordance with AASHTO LRFD Section 4, different loads are used for positive and negative bending.

Unit Weight Concrete $w_c = 150 \cdot pcf$

Deck Thickness for Design $t_{deck} := 8.0in$ $\quad t_{deck} \geq 7in = 1$

Deck Thickness for Loads $t_d = 10.5 \cdot in$

Rebar yield strength $F_s = 60 \cdot ksi$ $\quad$ Strength of concrete $f_c = 5 \cdot ksi$

Concrete clear cover $\quad$ Bottom $\quad$ Top

$c_b := 1.0in$ $\quad c_b \geq 1.0in = 1$ $\quad c_t := 2.5in$ $\quad c_t \geq 2.5in = 1$

Transverse reinforcement

Bottom Reinforcing $\phi_{tb} := \frac{6}{8}in$

Bottom Spacing $s_{tb} := 8in$

$s_{tb} \geq 1.5\phi_{tb} \wedge 1.5in = 1$

$s_{tb} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$A_{stb} := \frac{12in}{s_{tb}} \cdot \pi \cdot \left(\frac{\phi_{tb}}{2}\right)^2 = 0.7 \cdot in^2$

Top Reinforcing $\phi_{tt} := \frac{5}{8}in$

Top Spacing $s_{tt} := 8in$

$s_{tt} \geq 1.5\phi_{tt} \wedge 1.5in = 1$

$s_{tt} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$A_{stt} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 = 0.5 \cdot in^2$

Design depth of Bar $d_{tb} := t_{deck} - \left(c_b + \frac{\phi_{tb}}{2}\right) = 6.6 \cdot in$ $\quad d_{tt} := t_{deck} - \left(c_t + \frac{\phi_{tt}}{2}\right) = 5.2 \cdot in$

Girder Spacing $spacing_{int\_max} := 2ft + 11in$

$spacing_{ext} = 3\,ft$

Equivalent Strip, +M $w_{posM} := \left(26 + 6.6 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $\quad w_{posM} = 45.3 \cdot in$

Equivalent Strip, -M $w_{negM} := \left(48 + 3.0 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $\quad w_{negM} = 56.8 \cdot in$

Once the strip widths are determined, the dead loads can be calculated.

19. PERMANENT LOADS

This section calculates the dead loads on the slab. These are used later for analysis to determine the design moments.

Weight of deck, +M	$w_{deck\_pos} := w_c \cdot t_d \cdot w_{posM}$	$w_{deck\_pos} = 494.9 \cdot plf$
Weight of deck, -M	$w_{deck\_neg} := w_c \cdot t_d \cdot w_{negM}$	$w_{deck\_neg} = 620.7 \cdot plf$
Unit weight of barrier	$w_b := 433.5plf$	
Barrier point load, +M	$P_{b\_pos} := w_b \cdot w_{posM}$	$P_{b\_pos} = 1.63 \cdot kip$
Barrier point load, -M	$P_{b\_neg} := w_b \cdot w_{negM}$	$P_{b\_neg} = 2.05 \cdot kip$

20. LIVE LOADS

This section calculates the live loads on the slab. These loads are analyzed in a separate program with the permanent loads to determine the design moments.

Truck wheel load	$P_{wheel} := 16kip$		
Impact Factor	$IM := 1.33$		
Multiple presence factors	$MP_1 := 1.2$	$MP_2 := 1.0$	$MP_3 := 0.85$
Wheel Loads	$P_1 := IM \cdot MP_1 \cdot P_{wheel}$	$P_2 := IM \cdot MP_2 \cdot P_{wheel}$	$P_3 := IM \cdot MP_3 \cdot P_{wheel}$
	$P_1 = 25.54 \cdot kip$	$P_2 = 21.3 \cdot kip$	$P_3 = 18.09 \cdot kip$

21. LOAD RESULTS

The separate MathCAD design aides (available in Appendix of the final report) was used to analyze the deck as an 11-span continuous beam without cantilevered overhangs on either end, with supports stationed at girder locations. The dead and live loads were applied separately. The results are represented here as input values, highlighted.

Design Moments

$M_{pos\_deck} := 0.4 \cdot kip \cdot ft$ $\quad M_{pos\_LL} := 15.3 \cdot kip \cdot ft$ $\quad M_{pos} := (1.25 \cdot M_{pos\_deck} + 1.75 \cdot M_{pos\_LL})$

$M_{pos} = 27.3 \cdot kip \cdot ft$ $\quad M_{pos\_dist} := \frac{M_{pos}}{w_{posM}}$ $\quad M_{pos\_dist} = 7.23 \cdot \frac{kip \cdot ft}{ft}$

$M_{neg\_deck} := -0.6 \cdot kip \cdot ft$ $\quad M_{neg\_LL} := -7.8 \cdot kip \cdot ft$ $\quad M_{neg} := (1.25 \cdot M_{neg\_deck} + 1.75 \cdot M_{neg\_LL})$

$M_{neg} = -14.4 \cdot kip \cdot ft$ $\quad M_{neg\_dist} := \frac{M_{neg}}{w_{negM}}$ $\quad M_{neg\_dist} = -3.04 \cdot \frac{kip \cdot ft}{ft}$

22. FLEXURAL STRENGTH CAPACITY CHECK:

Consider a 1'-0" strip: $\phi_b := 0.9$ $\quad b := 12in$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4ksi \\ 0.85 - 0.05 \cdot \left(\frac{f_c}{ksi} - 4\right) & \text{otherwise} \end{cases} \qquad \beta_1 = 0.8$$

Bottom:

$c_{tb} := \frac{A_{stb} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1 \cdot in$

$a_{tb} := \beta_1 \cdot c_{tb} = 0.8 \cdot in$

$M_{ntb} := \frac{A_{stb} \cdot F_s}{b} \cdot \left(d_{tb} - \frac{a_{tb}}{2}\right) = 20.7 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtb} := \phi_b \cdot M_{ntb} = 18.6 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtb} \ge |M_{pos\_dist}| = 1$

Top:

$c_{tt} := \frac{A_{stt} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.7 \cdot in$

$a_{tt} := \beta_1 \cdot c_{tt} = 0.5 \cdot in$

$M_{ntt} := \frac{A_{stt} \cdot F_s}{ft} \cdot \left(d_{tt} - \frac{a_{tt}}{2}\right) = 11.3 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtt} := \phi_b \cdot M_{ntt} = 10.2 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtt} \ge |M_{neg\_dist}| = 1$

23. LONGITUDINAL DECK REINFORCEMENT DESIGN:

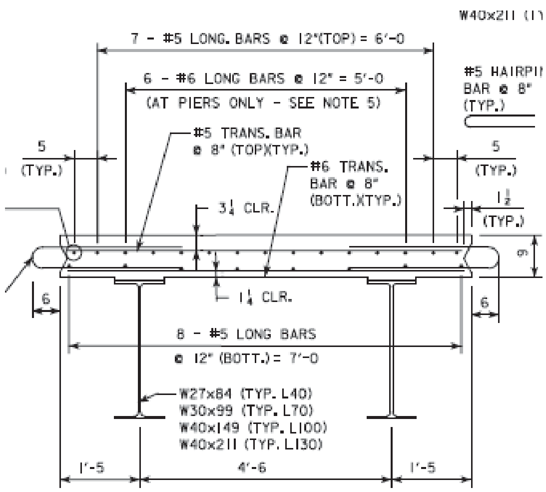
Longitudinal reinforcement $\quad \phi_{lb} := \frac{5}{8} in$ $\quad s_{lb} := 12in$ $\quad \phi_{lt} := \frac{5}{8} in$ $\quad s_{lt} := 12in$

$A_{slb} := \frac{12in}{s_{lb}} \cdot \pi \cdot \left(\frac{\phi_{lb}}{2}\right)^2 = 0.3 \cdot in^2$ $\quad A_{slt} := \frac{12in}{s_{lt}} \cdot \pi \cdot \left(\frac{\phi_{lt}}{2}\right)^2 = 0.3 \cdot in^2$

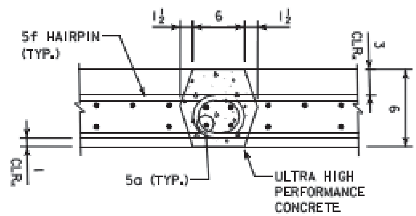
Distribution Reinforcement (AASHTO 9.7.3.2)

$$A_{\%dist} := \frac{\min\left(\frac{220}{\sqrt{\frac{spacing_{int\_max}}{ft}}}, 67\right)}{100} = 67 \cdot \%$$

$A_{dist} := A_{\%dist} \cdot (A_{stb}) = 0.4 \cdot in^2$ $\quad A_{slb} + A_{slt} \ge A_{dist} = 1$



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

24. DESIGN CHECKS

This section will conduct design checks on the reinforcing according to various sections in AASHTO LRFD.

CHECK MINIMUM REINFORCEMENT (AASHTO LRFD 5.7.3.3.2):

Modulus of Rupture $f_r := 0.37 \cdot \sqrt{f_c \cdot ksi} = 0.8 \cdot ksi$ $\qquad E_c = 4286.8 \cdot ksi$

Section Modulus $S_{nc} := \frac{b \cdot t_{deck}^2}{6} = 128 \cdot in^3$ $\qquad E_s = 29000 \cdot ksi$

$A_{deck} := t_{deck} \cdot b = 96 \cdot in^2$

$$y_{bar\_tb} := \frac{A_{deck} \cdot \frac{t_{deck}}{2} + (n - 1) \cdot A_{stb} \cdot d_{tb}}{A_{deck} + (n - 1) \cdot A_{stb}} = 4.1 \cdot in$$

$$y_{bar\_tt} := \frac{A_{deck} \cdot \frac{t_{deck}}{2} + (n - 1) \cdot A_{stt} \cdot d_{tt}}{A_{deck} + (n - 1) \cdot A_{stt}} = 4 \cdot in$$

$$I_{tb} := \frac{b \cdot t_{deck}^{3}}{12} + A_{deck} \cdot \left(\frac{t_{deck}}{2} - y_{bar\_tb}\right)^2 + (n - 1) \cdot A_{stb} \cdot \left(d_{tb} - y_{bar\_tb}\right)^2 = 538.3 \cdot in^4$$

$$I_{tt} := \frac{b \cdot t_{deck}^{3}}{12} + A_{deck} \cdot \left(\frac{t_{deck}}{2} - y_{bar\_tt}\right)^2 + (n - 1) \cdot A_{stt} \cdot \left(d_{tt} - y_{bar\_tt}\right)^2 = 515.8 \cdot in^4$$

$$S_{c\_tb} := \frac{I_{tb}}{t_{deck} - y_{bar\_tb}} = 138.2 \cdot in^3 \qquad S_{c\_tt} := \frac{I_{tt}}{t_{deck} - y_{bar\_tt}} = 130 \cdot in^3$$

Unfactored Dead Load

$$M_{dnc\_pos\_t} := 1.25 \frac{kip \cdot ft}{ft} \qquad M_{dnc\_neg\_t} := -0.542 \frac{kip \cdot ft}{ft}$$

Cracking Moment — S 5.7.3.3.2

$$M_{cr\_tb} := \max\left[\frac{S_{c\_tb} \cdot f_r}{ft} - \left|M_{dnc\_pos\_t}\right| \cdot \left(\frac{S_{c\_tb}}{S_{nc}} - 1\right), \frac{S_{c\_tb} \cdot f_r}{ft}\right] = 9.5 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{cr\_tt} := \max\left[\frac{S_{c\_tt} \cdot f_r}{ft} - \left|M_{dnc\_neg\_t}\right| \cdot \left(\frac{S_{c\_tt}}{S_{nc}} - 1\right), \frac{S_{c\_tt} \cdot f_r}{ft}\right] = 9 \cdot \frac{kip \cdot ft}{ft}$$

Minimum Factored Flexural Resistance

$$M_{r\_min\_tb} := \min\left(1.2 \cdot M_{cr\_tb}, 1.33 \cdot \left|M_{pos\_dist}\right|\right) = 9.6 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtb} \geq M_{r\_min\_tb} = 1$$

$$M_{r\_min\_tt} := \min\left(1.2 \cdot M_{cr\_tt}, 1.33 \cdot \left|M_{neg\_dist}\right|\right) = 4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt} \geq M_{r\_min\_tt} = 1$$

CHECK CRACK CONTROL (AASHTO LRFD 5.7.3.4):

$$\gamma_{eb} := 1.0 \qquad \gamma_{et} := 0.75$$

$$M_{SL\_pos} := 29.64 kip \cdot ft \qquad M_{SL\_neg} := 29.64 kip \cdot ft$$

$$M_{SL\_pos\_dist} := \frac{M_{SL\_pos}}{w_{posM}} = 7.9 \cdot \frac{kip \cdot ft}{ft} \qquad M_{SL\_neg\_dist} := \frac{M_{SL\_neg}}{w_{negM}} = 6.3 \cdot \frac{kip \cdot ft}{ft}$$

$$f_{ssb} := \frac{M_{SL\_pos\_dist} \cdot b \cdot n}{\frac{I_{tb}}{d_{tb} - y_{bar\_tb}}} = 3.1 \cdot ksi \qquad f_{sst} := \frac{M_{SL\_neg\_dist} \cdot b \cdot n}{\frac{I_{tt}}{d_{tt} - y_{bar\_tt}}} = 1.2 \cdot ksi$$

$$d_{cb} := c_b + \frac{\phi_{tb}}{2} = 1.4 \cdot in \qquad d_{ct} := c_t + \frac{\phi_{tt}}{2} = 2.8 \cdot in$$

$$\beta_{sb} := 1 + \frac{d_{cb}}{0.7 \cdot \left(t_{deck} - d_{cb}\right)} = 1.3 \qquad \beta_{st} := 1 + \frac{d_{ct}}{0.7 \cdot \left(t_{deck} - d_{ct}\right)} = 1.8$$

$$s_b := \frac{700 \cdot \gamma_{eb} \cdot kip}{\beta_{sb} \cdot f_{ssb} \cdot in} - 2 \cdot d_{cb} = 171.9 \cdot in \qquad s_t := \frac{700 \cdot \gamma_{et} \cdot kip}{\beta_{st} \cdot f_{sst} \cdot in} - 2 \cdot d_{ct} = 245.5 \cdot in$$

$$s_{tb} \leq s_b = 1 \qquad s_{tt} \leq s_t = 1$$

SHRINKAGE AND TEMPERATURE REINFORCING (AASHTO LRFD 5.10.8):

$$A_{st} := \begin{cases} \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} & \text{if } 0.11in^2 \le \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} \le 0.60in^2 \\ 0.11in^2 & \text{if } \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} < 0.11in^2 \\ 0.60in^2 & \text{if } \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} > 0.60in^2 \end{cases} = 0.1 \cdot in^2$$

$A_{stb} \ge A_{st} = 1$ $\qquad$ $A_{stt} \ge A_{st} = 1$

$A_{slb} \ge A_{st} = 1$ $\qquad$ $A_{slt} \ge A_{st} = 1$

SHEAR RESISTANCE (AASHTO LRFD 5.8.3.3):

$\phi := 0.9$ $\qquad$ $\beta := 2$ $\qquad$ $\theta := 45deg$ $\qquad$ $b = 1\,ft$

$$d_{v\_tb} := \max\left(0.72 \cdot t_{deck}, d_{tb} - \frac{a_{tb}}{2}, 0.9 \cdot d_{tb}\right) = 6.2 \cdot in$$

$$d_{v\_tt} := \max\left(0.72 \cdot t_{deck}, d_{tt} - \frac{a_{tt}}{2}, 0.9 \cdot d_{tt}\right) = 5.8 \cdot in$$

$$d_v := \min\left(d_{v\_tb}, d_{v\_tt}\right) = 5.8 \cdot in$$

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_v = 9.8 \cdot kip$$

$V_s := 0kip$ Shear capacity of reinforcing steel

$V_{ps} := 0kip$ Shear capacity of prestressing steel

$$V_{ns} := \min\left(V_c + V_s + V_{ps}, 0.25 \cdot f_c \cdot b \cdot d_v + V_{ps}\right) = 9.8 \cdot kip$$

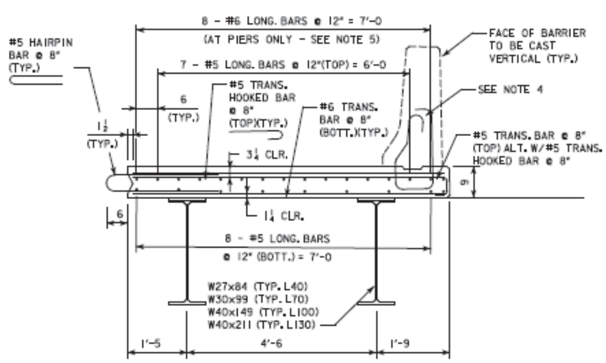
$V_r := \phi \cdot V_{ns} = 8.8 \cdot kip$ Total factored resistance

$V_{us} := 8.38kip$ Total factored load $\qquad$ $V_r \ge V_{us} = 1$

DEVELOPMENT AND SPLICE LENGTHS (AASHTO LRFD 5.11):

Development and splice length design follows standard calculations in AASHTO LRFD 5.11, or as dictated by the State DOT Design Manual.

25. DECK OVERHANG DESIGN (AASHTO LRFD A.13.4):



EXTERIOR MODULE
REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)

Deck Properties:

Deck Overhang Length $L_o := 1ft + 9in$

Parapet Properties:

Note: Parapet properties are per unit length. Compression reinforcement is ignored.

Cross Sectional Area $A_p := 2.84ft^2$ Height of Parapet $H_{par} := 2ft + 10in$

Parapet Weight $W_{par} := w_c \cdot A_p = 426 \cdot plf$

Width at base $w_{base} := 1ft + 5in$ Average width of wall $w_{wall} := \dfrac{13in + 9.5in}{2} = 11.3 \cdot in$

Height of top portion of parapet $h_1 := 2ft$ Width at top of parapet $width_1 := 9.5 \cdot in = 9.5 \cdot in$

Height of middle portion of parapet $h_2 := 7in$ Width at middle transition of parapet $width_2 := 12 \cdot in = 12 \cdot in$

Height of lower portion of parapet $h_3 := 3in$ Width at base of parapet $width_3 := 1ft + 5 \cdot in = 17 \cdot in$

$b_1 := width_1$ $b_2 := width_2 - width_1$ $b_3 := width_3 - width_2$

Parapet Center of Gravity

$$CG_p := \frac{\left(h_1 + h_2 + h_3\right) \cdot \frac{b_1^2}{2} + \frac{1}{2} \cdot h_1 \cdot b_2 \cdot \left(b_1 + \frac{b_2}{3}\right) \ldots + \left(h_2 + h_3\right) \cdot \left(b_2 + b_3\right) \cdot \left(b_1 + \frac{b_2 + b_3}{2}\right) - \frac{1}{2} \cdot h_2 \cdot b_3 \cdot \left(b_1 + b_2 + \frac{2b_3}{3}\right)}{\left(h_1 + h_2 + h_3\right) \cdot b_1 + \frac{1}{2} \cdot h_1 \cdot b_2 + \left(h_2 + h_3\right) \cdot \left(b_2 + b_3\right) - \frac{1}{2} \cdot h_2 \cdot b_3} = 6.3 \cdot in$$

Parapet Reinforcement	Vertically Aligned Bars in Wall	Horizontal Bars
Rebar spacing:	$s_{pa} := 12in$	$n_{pl} := 5$
Rebar Diameter:	$\phi_{pa} := \frac{5}{8} in$	$\phi_{pl} := \frac{5}{8} in$
Rebar Area:	$A_{st\_p} := \pi \cdot \left(\frac{\phi_{pa}}{2}\right)^2 \cdot \frac{b}{s_{pa}} = 0.3 \cdot in^2$	$A_{sl\_p} := \pi \cdot \left(\frac{\phi_{pl}}{2}\right)^2 = 0.3 \cdot in^2$
Cover:	$c_{st} := 3in$	$c_{sl} := 2in + \phi_{pa} = 2.6 \cdot in$
Effective Depth:	$d_{st} := w_{base} - c_{st} - \frac{\phi_{pa}}{2} = 13.7 \cdot in$	$d_{sl} := w_{wall} - c_{sl} - \frac{\phi_{pl}}{2} = 8.3 \cdot in$

Parapet Moment Resistance About Horizontal Axis: $\phi_{ext} := 1.0$

Depth of Equivalent Stress Block: $a_h := \dfrac{A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot b} = 0.4 \cdot in$ S 5.7.3.1.2-4 S 5.7.3.2.3

Moment Capacity of Upper Segment of Barrier (about longitudinal axis):

Average width of section $w_1 := \dfrac{width_1 + width_2}{2} = 10.7 \cdot in$

Cover $c_{st1} := 2in$

Depth $d_{h1} := w_1 - c_{st1} - \dfrac{\phi_{pa}}{2} = 8.4 \cdot in$

Factored Moment Resistance $\phi M_{nh1} := \dfrac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h1} - \frac{a_h}{2}\right)}{b} = 12.7 \cdot \dfrac{kip \cdot ft}{ft}$

Moment Capacity of Middle Segment of Barrier (about longitudinal axis):

Average width of section $w_2 := \frac{width_2 + width_3}{2} = 14.5 \cdot in$

Cover $c_{st2} := 3in$

Depth $d_{h2} := w_2 - c_{st2} - \frac{\phi_{pa}}{2} = 11.2 \cdot in$

Factored Moment Resistance $\phi M_{nh2} := \frac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h2} - \frac{a_h}{2}\right)}{b} = 16.9 \cdot \frac{kip \cdot ft}{ft}$

Parapet Base Moment Resistance (about longitudinal axis):

development in tension $c_{st3} := 3in$ $\quad cover_{base\_vert} := c_{st3} + \frac{\phi_{pa}}{2} = 3.3 \cdot in$

$$m_{inc\_ta} := \begin{cases} 1.5 & \text{if } c_{st3} < 3 \cdot \phi_{pa} \vee s_{pa} - \phi_{pa} < 6 \cdot \phi_{pa} \\ 1.2 & \text{otherwise} \end{cases} = 1.2$$

$$m_{dec\_ta} := \begin{cases} 0.8 & \text{if } s_{pa} \ge 6in \\ 1.0 & \text{otherwise} \end{cases} = 0.8$$

$$l_{db\_ta} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{st\_p} \cdot \frac{F_s}{kip}}{\sqrt{\frac{f_c}{ksi}}}, 0.4 \cdot \phi_{pa} \cdot \frac{F_s}{ksi}\right) & \text{if } \phi_{pa} \le \frac{11}{8} in \\ \dfrac{2.70in \cdot \frac{F_s}{ksi}}{\sqrt{\frac{f_c}{ksi}}} & \text{if } \phi_{pa} = \frac{14}{8} in \\ \dfrac{3.50in \cdot \frac{F_s}{ksi}}{\sqrt{\frac{f_c}{ksi}}} & \text{if } \phi_{pa} = \frac{18}{8} in \end{cases}$$

$l_{dt\_ta} := l_{db\_ta} \cdot m_{inc\_ta} \cdot m_{dec\_ta} = 14.4 \cdot in$

hooked bar developed in tension $l_{hb\_ta} := \frac{38 \cdot \phi_{pa}}{\sqrt{\frac{f_c}{ksi}}} = 10.6 \cdot in$ $\quad m_{inc} = 1.2$

$l_{dh\_ta} := \max\left(6in, 8 \cdot \phi_{pa}, m_{inc} \cdot l_{hb\_ta}\right) = 12.7 \cdot in$

lap splice in tension $l_{lst\_ta} := \max\left(12in, 1.3 \cdot l_{dt\_ta}\right) = 18.7 \cdot in$

$benefit := l_{dt\_ta} - l_{dh\_ta} = 1.7 \cdot in$

$l_{dev\_a} := \left(7 + \frac{13}{16}\right) in$

$F_{dev} := \frac{benefit + l_{dev\_a}}{l_{dt\_ta}} = 0.7$

$F_d := 0.75$

Distance from NA to Compressive Face $c_{t\_b} := \frac{F_d \cdot A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.3 \cdot in$ $\quad$ S 5.7.3.1.2-4

Depth of Equivalent Stress Block $a_t := \beta_1 \cdot c_{t\_b} = 0.3 \cdot in$ S 5.7.3.2.3

Nominal Moment Resistance $M_{nt} := F_d \cdot A_{st\_p} \cdot F_s \cdot \left(d_{st} - \frac{a_t}{2}\right) = 15.6 \cdot kip \cdot ft$ S 5.7.3.2.2-1

Factored Moment Resistance $M_{cb} := \phi_{ext} \cdot \frac{M_{nt}}{ft} = 15.6 \cdot \frac{kip \cdot ft}{ft}$ S 5.7.3.2

Average Moment Capacity of Barrier (about longitudinal axis):

Factored Moment Resistance about Horizontal Axis $M_c := \frac{\phi M_{nh1} \cdot h_1 + \phi M_{nh2} \cdot h_2 + M_{cb} \cdot h_3}{h_1 + h_2 + h_3} = 13.8 \cdot \frac{kip \cdot ft}{ft}$

Parapet Moment Resistance (about vertical axis):

Height of Transverse Reinforcement in Parapet

$y_1 := 5in$

$y_2 := 11.5in$

$y_3 := 18in$

$y_4 := 24.5in$

$y_5 := 31in$

Width of Parapet at Transverse Reinforcement

$x_1 := width_3 - \frac{(y_1 - h_3) \cdot b_3}{h_2} = 15.6 \cdot in$

$x_2 := b_1 + b_2 - \frac{(y_2 - h_3 - h_2) \cdot b_2}{h_1} = 11.8 \cdot in$

$x_3 := b_1 + b_2 - \frac{(y_3 - h_3 - h_2) \cdot b_2}{h_1} = 11.2 \cdot in$

$x_4 := b_1 + b_2 - \frac{(y_4 - h_3 - h_2) \cdot b_2}{h_1} = 10.5 \cdot in$

$x_5 := b_1 + b_2 - \frac{(y_5 - h_3 - h_2) \cdot b_2}{h_1} = 9.8 \cdot in$

Depth of Equivalent Stress Block $a := \frac{n_{pl} \cdot A_{sl\_p} \cdot F_s}{0.85 \cdot f_c \cdot H_{par}} = 0.6 \cdot in$

Concrete Cover in Parapet

$cover_r := 2in$ $cover_{rear} := cover_r + \phi_{pa} + \frac{\phi_{pl}}{2} = 2.9 \cdot in$

$cover_{base} := c_{st3} + \phi_{pa} + \frac{\phi_{pl}}{2} = 3.9 \cdot in$

$cover_f := 2in$ $cover_{front} := 2in + \phi_{pa} + \frac{\phi_{pl}}{2}$

$cover_t := \frac{x_5}{2} = 4.9 \cdot in$ $cover_{top} := cover_t = 4.9 \cdot in$

Design depth

$d_{1i} := x_1 - cover_{base} = 11.6 \cdot in$ $d_{1o} := x_1 - cover_{rear} = 12.6 \cdot in$

$d_{2i} := x_2 - cover_{front} = 8.9 \cdot in$ $d_{2o} := x_2 - cover_{rear} = 8.9 \cdot in$

$d_{3i} := x_3 - cover_{front} = 8.2 \cdot in$ $d_{3o} := x_3 - cover_{rear} = 8.2 \cdot in$

$d_{4i} := x_4 - cover_{front} = 7.6 \cdot in$ $d_{4o} := x_4 - cover_{rear} = 7.6 \cdot in$

$d_{5i} := x_5 - cover_{top} = 4.9 \cdot in$ $d_{5o} := x_5 - cover_{top} = 4.9 \cdot in$

Nominal Moment Resistance - Tension on Inside Face

$\phi Mn_{1i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1i} - \frac{a}{2}\right) = 208.3 \cdot kip \cdot in$

$\phi Mn_{2i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2i} - \frac{a}{2}\right) = 158.1 \cdot kip \cdot in$

$\phi Mn_{3i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3i} - \frac{a}{2}\right) = 145.6 \cdot kip \cdot in$

$\phi Mn_{4i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4i} - \frac{a}{2}\right) = 133.2 \cdot kip \cdot in$

$$\phi Mn_{5i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5i} - \frac{a}{2} \right) = 84.5 \cdot kip \cdot in$$

$$M_{wi} := \phi Mn_{1i} + \phi Mn_{2i} + \phi Mn_{3i} + \phi Mn_{4i} + \phi Mn_{5i} = 60.8 \cdot kip \cdot ft$$

Nominal Moment Resistance - Tension on Outside Face

$$\phi Mn_{1o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1o} - \frac{a}{2} \right) = 18.9 \cdot kip \cdot ft$$

$$\phi Mn_{2o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2o} - \frac{a}{2} \right) = 13.2 \cdot kip \cdot ft$$

$$\phi Mn_{3o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3o} - \frac{a}{2} \right) = 12.1 \cdot kip \cdot ft$$

$$\phi Mn_{4o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4o} - \frac{a}{2} \right) = 11.1 \cdot kip \cdot ft$$

$$\phi Mn_{5o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5o} - \frac{a}{2} \right) = 7 \cdot kip \cdot ft$$

$$M_{wo} := \phi Mn_{1o} + \phi Mn_{2o} + \phi Mn_{3o} + \phi Mn_{4o} + \phi Mn_{5o} = 62.3 \cdot kip \cdot ft$$

Vertical Nominal Moment Resistance of Parapet

$$M_w := \frac{2 \cdot M_{wi} + M_{wo}}{3} = 61.3 \cdot kip \cdot ft$$

Parapet Design Factors:

Crash Level

CL := "TL-4"

Transverse Design Force

$$F_t := \begin{array}{|l} 13.5kip \text{ if } CL = \text{"TL-1"} \\ 27.0kip \text{ if } CL = \text{"TL-2"} \\ 54.0kip \text{ if } CL = \text{"TL-3"} \\ 54.0kip \text{ if } CL = \text{"TL-4"} \\ 124.0kip \text{ if } CL = \text{"TL-5"} \\ 175.0kip \text{ otherwise} \end{array} = 54 \cdot kip \qquad L_t := \begin{array}{|l} 4.0ft \text{ if } CL = \text{"TL-1"} \\ 4.0ft \text{ if } CL = \text{"TL-2"} \\ 4.0ft \text{ if } CL = \text{"TL-3"} \\ 3.5ft \text{ if } CL = \text{"TL-4"} \\ 8.0ft \text{ if } CL = \text{"TL-5"} \\ 8.0ft \text{ otherwise} \end{array} = 3.5 \cdot ft$$

Longitudinal Design Force

$$F_l := \begin{array}{|l} 4.5kip \text{ if } CL = \text{"TL-1"} \\ 9.0kip \text{ if } CL = \text{"TL-2"} \\ 18.0kip \text{ if } CL = \text{"TL-3"} \\ 18.0kip \text{ if } CL = \text{"TL-4"} \\ 41.0kip \text{ if } CL = \text{"TL-5"} \\ 58.0kip \text{ otherwise} \end{array} = 18 \cdot kip \qquad L_l := \begin{array}{|l} 4.0ft \text{ if } CL = \text{"TL-1"} \\ 4.0ft \text{ if } CL = \text{"TL-2"} \\ 4.0ft \text{ if } CL = \text{"TL-3"} \\ 3.5ft \text{ if } CL = \text{"TL-4"} \\ 8.0ft \text{ if } CL = \text{"TL-5"} \\ 8.0ft \text{ otherwise} \end{array} = 3.5 \cdot ft$$

Vertical Design Force (Down)

$$F_v := \begin{array}{|l} 4.5kip \text{ if } CL = \text{"TL-1"} \\ 4.5kip \text{ if } CL = \text{"TL-2"} \\ 4.5kip \text{ if } CL = \text{"TL-3"} \\ 18.0kip \text{ if } CL = \text{"TL-4"} \\ 80.0kip \text{ if } CL = \text{"TL-5"} \\ 80.0kip \text{ otherwise} \end{array} = 18 \cdot kip \qquad L_v := \begin{array}{|l} 18.0ft \text{ if } CL = \text{"TL-1"} \\ 18.0ft \text{ if } CL = \text{"TL-2"} \\ 18.0ft \text{ if } CL = \text{"TL-3"} \\ 18.0ft \text{ if } CL = \text{"TL-4"} \\ 40.0ft \text{ if } CL = \text{"TL-5"} \\ 40.0ft \text{ otherwise} \end{array} = 18 \cdot ft$$

Critical Length of Yield Line Failure Pattern:

$$M_b := 0kip \cdot ft$$

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \cdot H_{par} \cdot (M_b + M_w)}{M_c}} = 11.9 \cdot ft$$ S A13.3.1-2

$$R_w := \frac{2}{2 \cdot L_c - L_t} \cdot \left(8 \cdot M_b + 8 \cdot M_w + \frac{M_c \cdot L_c^2}{H_{par}}\right) = 116.2 \cdot kip$$ S A13.3.1-1

$$T := \frac{R_w \cdot b}{L_c + 2 \cdot H_{par}} = 6.6 \cdot kip$$ S A13.4.2-1

The parapet design must consider three design cases. Design Case 1 is for longitudinal and transverse collision loads under Extreme Event Load Combination II. Design Case 2 represents vertical collision loads under Extreme Event Load Combination II; however, this case does not govern for decks with concrete parapets or barriers. Design Case 3 is for dead and live load under Strength Load Combination I; however, the parapet will not carry wheel loads and therefore this case does not govern. Design Case 1 is the only case that requires a check.

Design Case 1: Longitudinal and Transverse Collision Loads, Extreme Event Load Combination II

DC - 1A: Inside face of parapet

$\phi_{ext} = 1$ $\gamma_{DC} := 1.0$ $\gamma_{DW} := 1.0$ $\gamma_{LL} := 0.5$ S A13.4.1
S Table 3.4.1-1

$l_{lip} := 2in$ $w_{base} = 17 \cdot in$

$A_{deck\_1A} := t_{deck} \cdot (l_{lip} + w_{base}) = 152 \cdot in^2$ $A_p = 2.8 \cdot ft^2$

$W_{deck\_1A} := w_c \cdot A_{deck\_1A} = 0.2 \cdot klf$ $W_{par} = 0.4 \cdot klf$

$$M_{DCdeck\_1A} := \gamma_{DC} \cdot W_{deck\_1A} \cdot \frac{l_{lip} + w_{base}}{2} = 0.1 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1A} := \gamma_{DC} \cdot W_{par} \cdot (l_{lip} + CG_p) = 0.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1A} := M_{cb} + M_{DCdeck\_1A} + M_{DCpar\_1A} = 16 \cdot \frac{kip \cdot ft}{ft}$$

$\phi_{tt\_add} := \frac{5}{8} in$ $s_{tt\_add} := 8in$

$$A_{stt\_p} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 + \frac{12in}{s_{tt\_add}} \cdot \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.9 \cdot in^2$$

$$d_{tt\_add} := t_{deck} - \left(c_t + \frac{\phi_{tt\_add}}{2}\right) = 5.2 \cdot in$$

$c_{tt\_p} := \frac{A_{stt\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.4 \cdot in$ $a_{tt\_p} := \beta_1 \cdot c_{tt\_p} = 1.1 \cdot in$

$$M_{ntt\_p} := \frac{A_{stt\_p} \cdot F_s}{ft} \cdot \left(d_{tt\_add} - \frac{a_{tt\_p}}{2}\right) = 21.4 \cdot \frac{kip \cdot ft}{ft}$$

$M_{rtt\_p} := \phi_b \cdot M_{ntt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft}$ $M_{rtt\_p} \geq M_{total\_1A} = 1$

$A_{sT} := A_{stt} + A_{stb} = 1.1 \cdot in^2$

$\phi P_n := \phi_{ext} \cdot A_{sT} \cdot F_s = 67.4 \cdot kip$ $\phi P_n \geq T = 1$

$M_{u\_1A} := M_{rtt\_p} \cdot \left(1 - \frac{T}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft}$ $M_{u\_1A} \geq M_{total\_1A} = 1$

DC - 1B: Design Section in Overhang

Notes: Distribution length is assumed to increase based on a 30 degree angle from the face of parapet.
Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
Future wearing surface is neglected as contribution is negligible.

$A_{deck\_1B} := t_{deck} \cdot L_o = 168 \cdot in^2$ $\qquad A_p = 2.8 \cdot ft^2$

$W_{deck\_1B} := w_c \cdot A_{deck\_1B} = 0.2 \cdot klf$ $\qquad W_{par} = 0.4 \cdot klf$

$$M_{DCdeck\_1B} := \gamma_{DC} \cdot W_{deck\_1B} \cdot \frac{L_o}{2} = 0.2 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1B} := \gamma_{DC} \cdot W_{par} \cdot \left(L_o - l_{lip} - CG_p\right) = 0.5 \cdot \frac{kip \cdot ft}{ft}$$

$L_{spread\_B} := L_o - l_{lip} - width_3 = 2 \cdot in$ $\qquad spread := 30deg$

$w_{spread\_B} := L_{spread\_B} \cdot \tan(spread) = 1.2 \cdot in$

$$M_{cb\_1B} := \frac{M_{cb} \cdot L_c}{L_c + 2 \cdot w_{spread\_B}} = 15.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1B} := M_{cb\_1B} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 15.9 \cdot \frac{kip \cdot ft}{ft}$$

$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft}$ $\qquad M_{rtt\_p} \geq M_{total\_1B} = 1$

$\phi P_n = 67.4 \cdot kip$

$P_u := \frac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_B}} = 6.5 \cdot kip$ $\qquad \phi P_n \geq P_u = 1$

$M_{u\_1B} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft}$ $\qquad M_{u\_1B} \geq M_{total\_1B} = 1$

DC - 1C: Design Section in First Span

Assumptions: Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
Future wearing surface is neglected as contribution is negligible.

$$M_{par\_G1} := M_{DCpar\_1B} = 0.5 \cdot \frac{kip \cdot ft}{ft}$$

$M_{par\_G2} := -0.137 \frac{kip \cdot ft}{ft}$ $\qquad$ (From model output)

$$M_1 := M_{cb} = 15.6 \cdot \frac{kip \cdot ft}{ft}$$

$$M_2 := M_1 \cdot \frac{M_{par\_G2}}{M_{par\_G1}} = -4.7 \cdot \frac{kip \cdot ft}{ft}$$

$b_f := 10.5in$

$$M_{c\_M2M1} := M_1 + \frac{\frac{1}{4} \cdot b_f \cdot \left(-M_1 + M_2\right)}{spacing_{int\_max}} = 14.1 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{spread\_C} := L_o - l_{lip} - w_{base} + \frac{b_f}{4} = 4.6 \cdot in$$

$$w_{spread\_C} := L_{spread\_C} \cdot \tan(spread) = 2.7 \cdot in$$

$$M_{cb\_1C} := \frac{M_{c\_M2M1} \cdot L_c}{L_c + 2 \cdot w_{spread\_C}} = 13.6 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1C} := M_{cb\_1C} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 14.2 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1C} = 1$$

$$\phi P_n = 67.4 \cdot kip$$

$$P_{uC} := \frac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_C}} = 6.4 \cdot kip \qquad \phi P_n \geq P_{uC} = 1$$

$$M_{u\_1C} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{u\_1B} \geq M_{total\_1B} = 1$$

Compute Overhang Reinforcement Cut-off Length Requirement

Maximum crash load moment at theoretical cut-ff point:

$$M_{c\_max} := M_{rtt} = 10.2 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{Mc\_max} := \frac{M_2 - M_{rtt}}{M_2 - M_1} \cdot spacing_{int\_max} = 2.1 \cdot ft$$

$$L_{spread\_D} := L_o - l_{lip} - w_{base} + L_{Mc\_max} = 27.7 \cdot in$$

$$w_{spread\_D} := L_{spread\_D} \cdot \tan(spread) = 16 \cdot in$$

$$M_{cb\_max} := \frac{M_{c\_max} \cdot L_c}{L_c + 2 \cdot w_{spread\_D}} = 8.3 \cdot \frac{kip \cdot ft}{ft}$$

$$extension := \max\left(d_{tt\_add}, 12 \cdot \phi_{tt\_add}, 0.0625 \cdot spacing_{int\_max}\right) = 7.5 \cdot in$$

$$cutt\_off := L_{Mc\_max} + extension = 33.2 \cdot in$$

$$A_{tt\_add} := \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.3 \cdot in^2$$

$$m_{thick\_tt\_add} := \begin{vmatrix} 1.4 \ \text{ if } \ t_{deck} - c_t \geq 12in \\ 1.0 \ \text{ otherwise} \end{vmatrix} = 1$$

$$m_{epoxy\_tt\_add} := \begin{vmatrix} 1.5 \ \text{ if } \ c_t < 3 \cdot \phi_{tt\_add} \vee \frac{s_{tt\_add}}{2} - \phi_{tt\_add} < 6 \cdot \phi_{tt\_add} \\ 1.2 \ \text{ otherwise} \end{vmatrix} = 1.5$$

$$m_{inc\_tt\_add} := \min\left(m_{thick\_tt\_add} \cdot m_{epoxy\_tt\_add}, 1.7\right) = 1.5$$

$$m_{dec\_tt\_add} := \begin{vmatrix} 0.8 \ \text{ if } \ \frac{s_{tt\_add}}{2} \geq 6in \\ 1.0 \ \text{ otherwise} \end{vmatrix} = 1$$

$$l_{db\_tt\_add} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{tt\_add} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{tt\_add} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{tt\_add} \le \dfrac{11}{8} in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{14}{8} in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{18}{8} in \end{cases} \qquad l_{db\_tt\_add} = 15 \cdot in$$

$l_{dt\_tt\_add} := l_{db\_tt\_add} \cdot m_{inc\_tt\_add} \cdot m_{dec\_tt\_add} = 22.5 \cdot in$

$Cuttoff_{point} := L_{Mc\_max} + l_{dt\_tt\_add} - spacing_{int\_max} = 13.2 \cdot in$ extension past second interior girder

Check for Cracking in Overhang under Service Limit State:

Does not govern - no live load on overhang.

25. COMPRESSION SPLICE:

See sheet S7 for drawing.

Ensure compression splice and connection can handle the compressive force in the force couple due to the negative moment over the pier.

Live load negative moment over pier: $M_{LLPier} := 541.8 \cdot kip \cdot ft$

Factored LL moment: $M_{UPier} := 1.75 \cdot M_{LLPier} = 948.1 \cdot kip \cdot ft$

The compression splice is comprised of a splice plate on the underside of the bottom flange, and built-up angles on either side of the web, connecting to the bottom flange as well.

Calculate Bottom Flange Stress:

Composite moment of inertia: $I_z = 13940.9 \cdot in^4$

Distance to center of bottom flange from composite section centroid: $y_{bf} := \dfrac{t_{bf}}{2} + D_w + t_{tf} + t_{slab} - y_c = 27.4 \cdot in$

Stress in bottom flange: $f_{bf} := M_{UPier} \cdot \dfrac{y_{bf}}{I_z} = 22.4 \cdot ksi$

Calculate Bottom Flange Force:

Design Stress: $F_{bf} := \max\left(\dfrac{f_{bf} + F_y}{2}, 0.75 \cdot F_y\right) = 37.5 \cdot ksi$

Effective Flange Area: $A_{ef} := b_{bf} \cdot t_{bf} = 8.5 \cdot in^2$

Force in Flange: $C_{nf} := F_{bf} \cdot A_{ef} = 319.1 \cdot kip$

Calculate Bottom Flange Stress, Ignoring Concrete:

Moment of inertia: $I_{zsteel} = 5815.1 \cdot in^4$

Distance to center of bottom flange: $y_{bfsteel} := \dfrac{t_{bf}}{2} + D_w + t_{tf} - y_{steel} = 16.1 \cdot in$

Stress in bottom flange: $f_{bfsteel} := M_{UPier} \cdot \frac{y_{bfsteel}}{I_{zsteel}} = 31.4 \cdot ksi$

Bottom Flange Force for design:

Design Stress: $F_{cf} := \max\left(\frac{f_{bfsteel} + F_y}{2}, 0.75 \cdot F_y\right) = 40.7 \cdot ksi$

Design Force: $C_n := \max(F_{bf}, F_{cf}) \cdot A_{ef} = 346.5 \cdot kip$

Compression Splice Plate Dimensions:

Bottom Splice Plate:	$b_{bsp} := b_{bf} = 11.5 \cdot in$	$t_{bsp} := 0.75in$	$A_{bsp} := b_{bsp} \cdot t_{bsp} = 8.6 \cdot in^2$
Built-Up Angle Splice Plate Horizontal Leg:	$b_{asph} := 4.25in$	$t_{asph} := 0.75in$	$A_{asph} := 2 \cdot b_{asph} \cdot t_{asph} = 6.4 \cdot in^2$
Built-Up Angle Splice Plate Vertical Leg:	$b_{aspv} := 7.75in$	$t_{aspv} := 0.75in$	$A_{aspv} := 2 \cdot b_{aspv} \cdot t_{aspv} = 11.6 \cdot in^2$

Total Area: $A_{csp} := A_{bsp} + A_{asph} + A_{aspv} = 26.6 \cdot in^2$

Average Stress: $f_{cs} := \frac{C_n}{A_{csp}} = 13 \cdot ksi$

Proportion Load into each plate based on area:

$$C_{bsp} := \frac{C_n \cdot A_{bsp}}{A_{csp}} = 112.3 \cdot kip \qquad C_{asph} := \frac{C_n \cdot A_{asph}}{A_{csp}} = 83 \cdot kip \qquad C_{aspv} := \frac{C_n \cdot A_{aspv}}{A_{csp}} = 151.3 \cdot kip$$

Check Plates Compression Capacity:

Bottom Splice Plate: $k_{cps} := 0.75$ for bolted connection

$l_{cps} := 9in$

$$r_{bsp} := \sqrt{\frac{\min\left(\frac{b_{bsp} \cdot t_{bsp}^{\,3}}{12}, \frac{t_{bsp} \cdot b_{bsp}^{\,3}}{12}\right)}{A_{bsp}}} = 0.2 \cdot in$$

$$P_{ebsp} := \frac{\pi^2 \cdot E_s \cdot A_{bsp}}{\left(\frac{k_{cps} \cdot l_{cps}}{r_{bsp}}\right)^2} = 2539.7 \cdot kip$$

$$Q_{bsp} := \begin{cases} 1.0 & \text{if } \frac{b_{bsp}}{t_{bsp}} \le 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76 \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right) \cdot \sqrt{\frac{F_y}{E_s}}\right] & \text{if } 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \le \frac{b_{bsp}}{t_{bsp}} \le 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right)^2} & \text{otherwise} \end{cases} = 0.856$$

$P_{obsp} := Q_{bsp} \cdot F_y \cdot A_{bsp} = 369.2 \cdot kip$

$$P_{nbsp} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{obsp}}{P_{ebsp}}\right)}\right]\cdot P_{obsp}\right] & \text{if } \dfrac{P_{ebsp}}{P_{obsp}} \ge 0.44 \\ (0.877\cdot P_{ebsp}) & \text{otherwise} \end{cases} = 347.4\cdot kip$$

$$P_{nbsp\_allow} := 0.9\cdot P_{nbsp} = 312.7\cdot kip \qquad Check := \begin{cases} \text{"NG"} & \text{if } C_{bsp} \ge P_{nbsp\_allow} \\ \text{"OK"} & \text{if } P_{nbsp\_allow} \ge C_{bsp} \end{cases} = \text{"OK"}$$

Horizontal Angle Leg: $k_{cps} = 0.75$ for bolted connection

$l_{cps} = 9\cdot in$

$$r_{asph} := \sqrt{\frac{\min\left(\dfrac{b_{asph}\cdot t_{asph}^{\ 3}}{12}, \dfrac{t_{asph}\cdot b_{asph}^{\ 3}}{12}\right)}{A_{asph}}} = 0.153\cdot in$$

$$P_{easph} := \frac{\pi^2\cdot E_s\cdot A_{asph}}{\left(\dfrac{k_{cps}\cdot l_{cps}}{r_{asph}}\right)^2} = 938.6\cdot kip$$

$$Q_{asph} := \begin{cases} 1.0 & \text{if } \dfrac{b_{asph}}{t_{asph}} \le 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\cdot\left(\dfrac{b_{asph}}{t_{asph}}\right)\cdot\sqrt{\dfrac{F_y}{E_s}}\right] & \text{if } 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{asph}}{t_{asph}} \le 0.91\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\cdot E_s}{F_y\cdot\left(\dfrac{b_{asph}}{t_{asph}}\right)^2} & \text{otherwise} \end{cases} = 1$$

$$P_{oasph} := Q_{asph}\cdot F_y\cdot A_{asph} = 318.7\cdot kip$$

$$P_{nasph} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{oasph}}{P_{easph}}\right)}\right]\cdot P_{oasph}\right] & \text{if } \dfrac{P_{easph}}{P_{oasph}} \ge 0.44 \\ (0.877\cdot P_{easph}) & \text{otherwise} \end{cases} = 276.5\cdot kip$$

$$P_{nasph\_allow} := 0.9\cdot P_{nasph} = 248.9\cdot kip \qquad Check2 := \begin{cases} \text{"NG"} & \text{if } C_{asph} \ge P_{nasph\_allow} \\ \text{"OK"} & \text{if } P_{nasph\_allow} \ge C_{asph} \end{cases} = \text{"OK'}$$

Vertical Angle Leg: $k_{cps} = 0.75$ for bolted connection

$l_{cps} = 9\cdot in$

$$r_{aspv} := \sqrt{\frac{\min\left(\dfrac{b_{aspv}\cdot t_{aspv}^{\ 3}}{12}, \dfrac{t_{aspv}\cdot b_{aspv}^{\ 3}}{12}\right)}{A_{aspv}}} = 0.153\cdot in$$

$$P_{easpv} := \frac{\pi^2\cdot E_s\cdot A_{aspv}}{\left(\dfrac{k_{cps}\cdot l_{cps}}{r_{aspv}}\right)^2} = 1711.6\cdot kip$$

$$Q_{aspv} := \begin{cases} 1.0 & \text{if } \dfrac{b_{aspv}}{t_{aspv}} \le 0.45 \cdot \sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76 \cdot \left(\dfrac{b_{aspv}}{t_{aspv}}\right) \cdot \sqrt{\dfrac{F_y}{E_s}}\right] & \text{if } 0.45 \cdot \sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{aspv}}{t_{aspv}} \le 0.91 \cdot \sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53 \cdot E_s}{F_y \cdot \left(\dfrac{b_{aspv}}{t_{aspv}}\right)^2} & \text{otherwise} \end{cases} = 1$$

$$P_{oaspv} := Q_{aspv} \cdot F_y \cdot A_{aspv} = 581.2 \cdot kip$$

$$P_{naspv} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{oaspv}}{P_{easpv}}\right)}\right] \cdot P_{oaspv}\right] & \text{if } \dfrac{P_{easpv}}{P_{oaspv}} \ge 0.44 \\ (0.877 \cdot P_{easpv}) & \text{otherwise} \end{cases} = 504.2 \cdot kip$$

$$P_{naspv\_allow} := 0.9 \cdot P_{naspv} = 453.8 \cdot kip \qquad Check3 := \begin{cases} \text{"NG"} & \text{if } C_{aspv} \ge P_{naspv\_allow} \\ \text{"OK"} & \text{if } P_{naspv\_allow} \ge C_{aspv} \end{cases} = \text{"OK'}$$

Additional Checks: Design Bolted Connections of the splice plates to the girders, checking for shear, bearing, and slip critical connections.

26. CLOSURE POUR DESIGN:

See sheet S2 for drawing of closure pour.

Check the closure pour according to the negative bending capacity of the section.

Use the minimum reinforcing properties for design, to be conservative.

$A_{steel} = 34.3 \cdot in^2$ $\qquad A_{rt} = 1.4 \cdot in^2$ $\qquad A_{rb} = 1.9 \cdot in^2$

$cg_{steel} := t_{slab} + y_{steel} = 24.4 \cdot in$ $\qquad cg_{rt} := 3in + 1.5 \cdot \frac{5}{8} in = 3.9 \cdot in$ $\qquad cg_{rb} := t_{slab} - \left(1in + 1.5 \cdot \frac{5}{8} in\right) = 6.1 \cdot in$

Overall CG: $\quad A_{neg} := A_{steel} + A_{rt} + A_{rb} = 37.6 \cdot in^2$ $\qquad cg_{neg} := \dfrac{A_{steel} \cdot cg_{steel} + A_{rt} \cdot cg_{rt} + A_{rb} \cdot cg_{rb}}{A_{neg}} = 22.8 \cdot in$

Moment of Inertia: $\quad I_{zstl} := 3990in^4$

$$I_{neg} := I_{zstl} + A_{steel} \cdot (cg_{steel} - cg_{neg})^2 + A_{rt} \cdot (cg_{rt} - cg_{neg})^2 + A_{rb} \cdot (cg_{rb} - cg_{neg})^2 = 5104 \cdot in^4$$

Section Moduli:

$$S_{top\_neg} := \frac{I_{neg}}{cg_{neg} - cg_{rt}} = 271.2 \cdot in^3 \qquad r_{neg} := \sqrt{\frac{I_{neg}}{A_{neg}}} = 11.7 \cdot in$$

$$S_{bot\_neg} := \frac{I_{neg}}{(t_{slab} + t_{tf} + D_w + t_{bf} - cg_{neg})} = 281.7 \cdot in^3$$

Concrete Properties: $\quad f_c = 5 \cdot ksi$ $\qquad$ Steel Properties: $\quad F_y = 50 \cdot ksi$ $\qquad L_{bneg} := 13.42 ft$

$E_c = 4286.8 \cdot ksi$ $\qquad E_s = 29000 \cdot ksi$

$F_{yr} := 0.7 \cdot F_y = 35 \cdot ksi$

Negative Flexural Capacity:

Slenderness ratio for compressive flange: $\lambda_{fneg} := \dfrac{b_{bf}}{2 \cdot t_{bf}} = 7.8$

Limiting ratio for compactness: $\lambda_{pfneg} := 0.38 \cdot \sqrt{\dfrac{E_s}{F_y}} = 9.2$

Limiting ratio for noncompact $\lambda_{rfneg} := 0.56 \cdot \sqrt{\dfrac{E_s}{F_{yr}}} = 16.1$

Hybrid Factor: $R_h = 1$

$D_{cneg2} := \dfrac{D_w}{2} = 15.7 \cdot in$ $\qquad a_{wc} := \dfrac{2 \cdot D_{cneg2} \cdot t_w}{b_{bf} \cdot t_{bf}} = 2$

$$R_b := \begin{cases} 1.0 \ \text{ if } \ 2 \cdot \dfrac{D_{cneg2}}{t_w} \le 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}} \\ \min\left[1.0, 1 - \dfrac{a_{wc}}{1200 + 300 \cdot a_{wc}} \cdot \left(2 \cdot \dfrac{D_{cneg2}}{t_w} - 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}}\right)\right] \ \text{ otherwi} \end{cases}$$

$R_b = 1$

Flange compression resistance:

$$F_{nc1} := \begin{cases} R_b \cdot R_h \cdot F_y \ \text{ if } \ \lambda_{fneg} \le \lambda_{pfneg} \\ \left[\left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(\lambda_{fneg} - \lambda_{pfneg})}{(\lambda_{rfneg} - \lambda_{pfneg})}\right] \cdot R_b \cdot R_h \cdot F_y\right] \ \text{ otherwise} \end{cases}$$

$F_{nc1} = 50 \cdot ksi$

Lateral Torsional Buckling Resistance: $r_{tneg} := \dfrac{b_{bf}}{\sqrt{12 \cdot \left(1 + \dfrac{D_{cneg2} \cdot t_w}{3 \cdot b_{bf} \cdot t_{bf}}\right)}} = 2.9 \cdot in$

$L_{pneg} := 1.0 \cdot r_{tneg} \cdot \sqrt{\dfrac{E_s}{F_y}} = 69.1 \cdot in$

$L_{rneg} := \pi \cdot r_{tneg} \cdot \sqrt{\dfrac{E_s}{F_{yr}}} = 259.5 \cdot in$

$C_b = 1$

$$F_{nc2} := \begin{cases} R_b \cdot R_h \cdot F_y \ \text{ if } \ L_{bneg} \le L_{pneg} \\ \min\left[C_b \cdot \left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(L_{bneg} - L_{pneg})}{(L_{rneg} - L_{pneg})}\right] \cdot R_b \cdot R_h \cdot F_y, R_b \cdot R_h \cdot \right. \end{cases}$$

$F_{nc2} = 42.8 \cdot ksi$

Compressive Resistance: $F_{nc} := \min(F_{nc1}, F_{nc2}) = 42.8 \cdot ksi$

Tensile Flexural Resistance: $F_{nt} := R_h \cdot F_y = 50 \cdot ksi$ For Strength

$F_{nt\_Serv} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$ For Service

Ultimate Moment Resistance: $M_{n\_neg} := \min\left(F_{nt} \cdot S_{top\_neg}, F_{nc} \cdot S_{bot\_neg}\right) = 1003.6 \cdot kip \cdot ft$

$M_{UPier} = 948.1 \cdot kip \cdot ft$ from external FE analysis

$Check4 := M_{n\_neg} \geq M_{UPier} = 1$

For additional design, one may calculate the force couple at the section over the pier to find the force in the UHPC closure joint. This force can be used to design any additional reinforcement used in the joint.

Summary of changes from SHRP2:

- Adapted the AASHTO LRFD Bridge Design Specifications, 6th Edition (2012) and GDOT Standards
- MathCAD Design Aides provided in Appendix of the final report
  - Design loadings calucation (moment, shear, and reaction) for girders
  - Design loadings calucation for deck
- List of variable definitions added
- Enhanced the descriptions for all design steps
- Expansion of detail regarding girder sizing
- New cross-section drawings
- Load combination explanations
- 12 ft travel lanes, 6 ft shoulders and 2% slope from crown to comply with GDOT standards

File Name: Steel Girder-60 ft.xmcd

CONCRETE DECKED STEEL GIRDER DESIGN FOR ABC

The following example details the design of a steel girder bridge accompanied by precast concrete deck panels. This particular example was created in accordance with Accelerated Bridge Construction (ABC) principles. The example shown here is presented for a Georgia Department of Transportation research endeavour into ABC technology, and is intended to simplify the design procedure of ABC style bridges. This example was taken from the SHRP 2 Manual (S2-R04-RR-2), and modified by a Georgia Southern University research team working for the Georgia Department of Transportation.

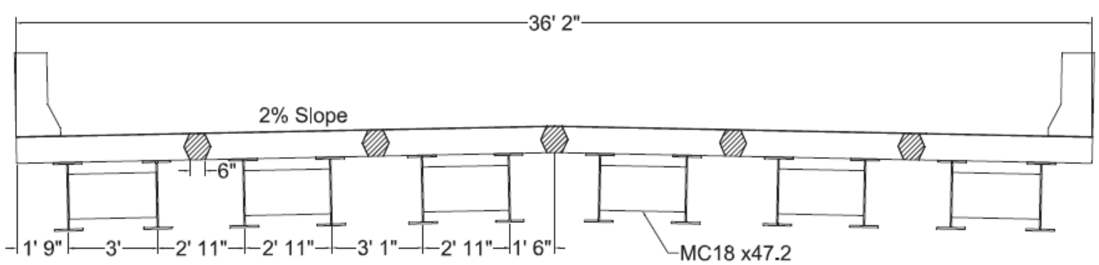
Note: These calculations do not consider every aspect of the bridge design process, and should not be condsidered exhaustive.

Note: All user inputs are highlighted in yellow for easy identification.

AASHTO LRFD Bridge Design Specifications (Sixth Edition with 2012 interims) was used to formulate this example. Located throughout this example are direct references to the AASHTO LRFD Bridge Design Specifications, which are found to the right side of their affiliated calculation.

Before beginning this example, a structural modeling program was used to analyze the superstructure. Although the calculations are not shown, the outputs are used for the design moments, shears and reactions in the example.

BRIDGE GEOMETRY:



Design member parameters:

Deck Width:	$w_{deck} := 36ft + 2in$	C. to C. Piers:	$Length := 60ft$
Roadway Width:	$w_{roadway} := 33ft$	C. to C. Bearings	$L_{span} := 57ft + 10in$
Skew Angle:	$Skew := 0deg$	Bridge Length:	$L_{total} := 3 \cdot Length = 180\,ft$
Deck Thickness	$t_d := 10.5in$	Stringer	W30x90
Haunch Thickness	$t_h := 2in$	Stringer Weight	$w_{s1} := 90plf$
Haunch Width	$w_h := 10.5in$	Stringer Length	$L_{str} := Length - 6 \cdot in = 59.5\,ft$
Girder Spacing	$spacing_{int} := 2ft + 11in$	Average spacing of adjacent beams. This value is used so that effective deck width is not overestimated.	
	$spacing_{ext} := 3ft$		

TABLE OF CONTENTS:

List of Variable Definitions

A = plan area of ice floe (ft^2); depth of temperature gradient (in.) (C3.9.2.3) (3.12.3)
AEP = apparent earth pressure for anchored walls (ksf) (3.4.1)
AF = annual frequency of bridge element collapse (number/yr.) (C3.14.4)
AS = peak seismic ground acceleration coefficient modified by short-period site factor (3.10.4.2)
B = notional slope of backfill (degrees) (3.11.5.8.1)
B′ = equivalent footing width (ft) (3.11.6.3)
Be = width of excavation (ft) (3.11.5.7.2b)
BM = beam (width) for barge, barge tows, and ship vessels (ft) (C3.14.5.1)
Bp = width of bridge pier (ft) (3.14.5.3)
BR = vehicular braking force; base rate of vessel aberrancy (3.3.2) (3.14.5.2.3)
b = braking force coefficient; width of a discrete vertical wall element (ft) (C3.6.4) (3.11.5.6)
bf = width of applied load or footing (ft) (3.11.6.3)
C = coefficient to compute centrifugal forces; constant for terrain conditions in relation to wind approach (3.6.3) (C3.8.1.1)
CD = drag coefficient (s^2 lbs./ft^4) (3.7.3.1)
CH = hydrodynamic mass coefficient (3.14.7)
CL = lateral drag coefficient (C3.7.3.1)
Csm = elastic seismic response coefficient for the m^{th} mode of vibration (3.10.4.2)
c = soil cohesion (ksf) (3.11.5.4)
cf = distance from back of a wall face to the front of an applied load or footing (ft) (3.11.6.3)
D = depth of embedment for a permanent nongravity cantilever wall with discrete vertical wall elements (ft) (3.11.5.6)
DE = minimum depth of earth cover (ft) (3.6.2.2)
Do = calculated embedment depth to provide equilibrium for nongravity cantilevered with continuous vertical elements by the simplified method (ft) (3.11.5.6)
D1 = effective width of applied load at any depth (ft) (3.11.6.3)
d = depth of potential base failure surface below base of excavation (ft); horizontal distance from the back of a wall face to the centerline of an applied load (ft) (3.11.5.7.2b) (3.11.6.3)
dc = total thickness of cohesive soil layers in the top 100 ft (3.10.3.1)
ds = total thickness of cohesionless soil layers in the top 100 ft (3.10.3.1)
E = Young's modulus (ksf) (C3.9.5)
EB = deformation energy (kip-ft) (C3.14.11)
e′ = eccentricity of load on footing (ft) (3.11.6.3)
F1 = lateral force due to earth pressure (kip/ft) (3.11.6.3)
F2 = lateral force due to traffic surcharge (kip/ft) (3.11.6.3)
f = constant applied in calculating the coefficient C used to compute centrifugal forces, taken equal to 4/3 for load combinations other than fatigue and 1.0 for fatigue (3.6.3)
f′c = specified compressive strength of concrete for use in design (ksi) (3.5.1)
g = gravitational acceleration (ft/s^2) (3.6.3)
H = ultimate bridge element strength (kip); final height of retaining wall (ft); total excavation depth (ft); resistance of bridge component to a horizontal force (kip) (C3.11.1) (3.11.5.7.1) (3.14.5.4)
Hp = ultimate bridge pier resistance (kip) (3.14.5.4)
Hs = ultimate bridge superstructure resistance (kip) (3.14.5.4)
H1 = distance from ground surface to uppermost ground anchor (ft) (3.11.5.7.1)
Hn+1 = distance from base of excavation to lowermost ground anchor (ft) (3.11.5.7.1)
h = notional height of earth pressure diagram (ft) (3.11.5.7)
heq = equivalent height of soil for vehicular load (ft) (3.11.6.4)
IM = dynamic load allowance (C3.6.1.2.5)
k = coefficient of lateral earth pressure; number of cohesive soil layers in the top 100 ft (3.11.6.2) (3.10.3.1)
ka = coefficient of active lateral earth pressure (3.11.5.1)
ko = coefficient of at rest lateral earth pressure (3.11.5.1)
kp = coefficient of passive lateral earth pressure (3.11.5.1)
ks = coefficient of earth pressure due to surcharge (3.11.6.1)
L = perimeter of pier (ft); length of soil reinforcing elements in an MSE wall (ft); length of footing (ft);

expansion length (in.) (3.9.5) (3.11.5.8) (3.11.6.3) (3.12.2.3)
ℓ = characteristic length (ft); center-to-center spacing of vertical wall elements (ft) (C3.9.5) (3.11.5.6)
m = multiple presence factor; number of cohesionless soil layers in the top 100 ft (3.6.1.1.2) (3.10.3.1)
N = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile (3.10.3.1)
Nch = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for cohesive soil layers in the upper 100 ft of the soil profile and us for cohesive soil layers (PI > 20) in the top 100 ft (us method) (3.10.3.1)
Nchi = blowcount for a cohesionless soil layer (not to exceed 100 blows/ft in the above expression) (3.10.3.1)
Ni = Standard Penetration Test blow count of a layer (not to exceed 100 blows/ft in the above expression). Note that when using Method B, N values are for cohesionless soils and cohesive soil and rock layers within the upper 100 ft Where refusal is met for a rock layer, Nishould be taken as 100 blows/ft (3.10.3.1)
Ns = stability number (3.11.5.6)
OCR = overconsolidation ratio (3.11.5.2)
P = maximum vertical force for single ice wedge (kip); load resulting from vessel impact (kip); concentrated wheel load (kip); live load intensity; point load (kip) (C3.9.5) (3.14.5.4) (C3.6.1.2.5) (C3.11.6.2) (3.11.6.1)
Pa = force resultant per unit width of wall (kip/ft) (3.11.5.8.1)
PC = probability of bridge collapse (3.14.5)
PD = design wind pressure (ksf) (3.8.1.2.1)
PGA = peak seismic ground acceleration coefficient on rock (Site Class B) (3.10.2.1) (3.10.4.2)
PH = lateral force due to superstructure or other concentrated lateral loads (kip/ft) (3.11.6.3)
Ph = horizontal component of resultant earth pressure on wall (kip/ft) (3.11.5.5)
PI = plasticity index (ASTM D4318) (3.10.3.1)
Pp = passive earth pressure (kip/ft) (3.11.5.4)
Pv = vertical component of resultant earth pressure on wall (kip/ft); load per linear foot of strip footing (kip/ft) (3.11.5.5) (3.11.6.3)
P′v = load on isolated rectangular footing or point load (kip) (3.11.6.3)
p = effective ice crushing strength (ksf); stream pressure (ksf); basic earth pressure (psf); fraction of truck traffic in a single lane; load intensity (ksf) (3.9.2.2) (3.7.3.1) (3.11.5.1) (3.6.1.4.2) (3.11.6.1)
pa = apparent earth pressure (ksf); maximum ordinate of pressure diagram (ksf) (3.11.5.3) (3.11.5.7.1)
pp = passive earth pressure (ksf) (3.11.5.4)
Q = total factored load; load intensity for infinitely long line loading (kip/ft) (3.4.1) (3.11.6.2)
Qi = force effects (3.4.1)
q = surcharge pressure (ksf) (3.11.6.3)
qs = uniform surcharge pressure (ksf) (3.11.6.1)
R = radius of curvature (ft); radius of circular pier (ft); seismic response modification factor; reduction factor of lateral passive earth pressure; radial distance from point of load application to a point on the wall (ft); reaction force to be resisted by subgrade below base of excavation (kip/ft) (3.6.3) (3.9.5) (3.10.7.1) (3.11.5.4) (3.11.6.1) (3.11.5.7.1)
Sm = shear strength of rock mass (ksf) (3.11.5.6)
Su = undrained shear strength of cohesive soil (ksf) (3.11.5.6)
Sub = undrained strength of soil below excavation base (ksf) (3.11.5.7.2b)
Sv = vertical spacing of reinforcements (ft) (3.11.5.8.1)
us = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile (3.10.3.1)
sui = undrained shear strength for a cohesive soil layer (not to exceed 5.0 ksf in the above expression) (3.10.3.1)
S1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock (Site Class B) (3.10.2.1) (3.10.4.2)
T = mean daily air temperature (°F) (C3.9.2.2)
TF = period of fundamental mode of vibration of bridge (s) (3.10.2.2)
Thi = horizontal load in anchor i (kip/ft) (3.11.5.7.1)
Tm = period of vibration for mth mode (s) (3.10.4.2)
Tmax = applied load to reinforcement in a mechanically stabilized earth wall (kip/ft) (3.11.5.8.2)
TMaxDesign= maximum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
TMinDesign = minimum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
TS = corner period at which acceleration response spectrum changes from being independent of period to being inversely proportional to period (s) (3.10.4.2)
T0 = reference period used to define shape of acceleration response spectrum (s) (3.10.4.2)

t = thickness of ice (ft); thickness of deck (in.) (3.9.2.2) (3.12.3)
V = design velocity of water (ft/s); design impact speed of vessel (ft/s) (3.7.3.1) (3.14.6)
VB = base wind velocity taken as 100 mph (3.8.1.1)
VDZ = design wind velocity at design Elevation Z (mph) (3.8.1.1)
VMIN = minimum design impact velocity taken not less than the yearly mean current velocity for the bridge location (ft/s) (3.14.6)
V0 = friction velocity, a meteorological wind characteristic for various upwind surface characteristics (mph) (3.8.1.1)
V30 = wind speed at 30.0 ft above low ground or water level (mph) (3.8.1.1)
v = highway design speed (ft/s) (3.6.3)
s v = average shear wave velocity for the upper 100 ft of the soil profile (3.10.3.1)
W = displacement weight of vessel (tonne) (C3.14.5.1)
w = width of clear roadway (ft); width of clear pedestrian and/or bicycle bridge (ft); width of pier at level of ice action (ft); specific weight of water (kcf); moisture content (ASTM D2216) (3.6.1.1.1) (3.6.1.6) (3.9.2.2) (C3.7.3.1) (3.10.3.1)
X = horizontal distance from back of wall to point of load application (ft); distance to bridge element from the centerline of vessel transit path (ft) (3.11.6.2) (3.14.6)
X1 = distance from the back of the wall to the start of the line load (ft) (3.11.6.2)
X2 = length of the line load (ft) (3.11.6.2)
Z = structure height above low ground or water level > 30.0 ft (ft); depth below surface of soil (ft); depth from the ground surface to a point on the wall under consideration (ft); vertical distance from point of load application to the elevation of a point on the wall under consideration (ft) (3.8.1.1) (3.11.6.3) (3.11.6.2)
Z0 = friction length of upstream fetch, a meteorological wind characteristic (ft) (3.8.1.1)
Z2 = depth where effective width intersects back of wall face (ft) (3.11.6.3)
z = depth below surface of backfill (ft) (3.11.5.1)
α = constant for terrain conditions in relation to wind approach; coefficient for local ice condition; inclination of pier nose with respect to a vertical axis (degrees); inclination of back of wall with respect to a vertical axis (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing nearest to the wall (rad); coefficient of thermal expansion (in./in./°F) (C3.8.1.1) (C3.9.2.2) (3.9.2.2) (C3.11.5.3) (3.11.6.2) (3.12.2.3)
β = safety index; nose angle in a horizontal plane used to calculate transverse ice forces (degrees); slope of backfill surface behind retaining wall; {+ for slope up from wall; − for slope down from wall} (degrees) (C3.4.1) (3.9.2.4.1) (3.11.5.3)

β′ = slope of ground surface in front of wall {+ for slope up from wall; − for slope down from wall} (degrees) (3.11.5.6)
γ = load factors; unit weight of materials (kcf); unit weight of water (kcf); unit weight of soil (kcf) (C3.4.1) (3.5.1) (C3.9.5) (3.11.5.1)
γs = unit weight of soil (kcf) (3.11.5.1)
γ′s = effective soil unit weight (kcf) (3.11.5.6)
γEQ = load factor for live load applied simultaneously with seismic loads (3.4.1)
γeq = equivalent-fluid unit weight of soil (kcf) (3.11.5.5)
γi = load factor (3.4.1)
γp = load factor for permanent loading (3.4.1)
γSE = load factor for settlement (3.4.1)
γTG = load factor for temperature gradient (3.4.1)
Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft) (C3.11.1) (3.11.5.5)
Δp = constant horizontal earth pressure due to uniform surcharge (ksf) (3.11.6.1)
Δph = constant horizontal pressure distribution on wall resulting from various types of surcharge loading (ksf) (3.11.6.2)
ΔT = design thermal movement range (in.) (3.12.2.3)
ΔσH = horizontal stress due to surcharge load (ksf) (3.11.6.3)
Δσv = vertical stress due to surcharge load (ksf) (3.11.6.3)
δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing furthest from the wall (rad) (C3.9.5) (3.11.5.3) (3.11.6.2)
ηi = load modifier specified in Article 1.3.2; wall face batter (3.4.1) (3.11.5.9)

θ = angle of back of wall to the horizontal (degrees); angle of channel turn or bend (degrees); angle between direction of stream flow and the longitudinal axis of pier (degrees) (3.11.5.3) (3.14.5.2.3) (3.7.3.2)
θ_f = friction angle between ice floe and pier (degrees) (3.9.2.4.1)
σ = standard deviation of normal distribution (3.14.5.3)
σ_T = tensile strength of ice (ksf) (C3.9.5)
v = Poisson's Ratio (dim.) (3.11.6.2)
φ = resistance factors (C3.4.1)
φ_f = angle of internal friction (degrees) (3.11.5.4)

φ'_f = effective angle of internal friction (degrees) (3.11.5.2)
φ_r = internal friction angle of reinforced fill (degrees) (3.11.6.3)
φ'_s = angle of internal friction of retained soil (degrees) (3.11.5.6)

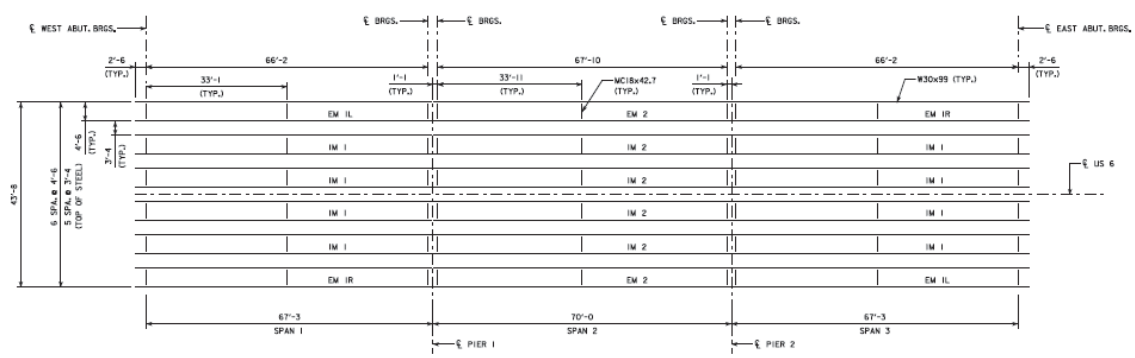
• Permanent Loads
CR = force effects due to creep
DD = downdrag force
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

• Transient Loads

EQ = earthquake load
FR = friction load
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = force effect due to settlement
TG = force effect due to temperature gradient
TU = force effect due to uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

1. INTRODUCTION

AASHTO LRFD principles were used in the design of this superstructure. The example is designed for a bridge with three even spans, and has no skew. The bridge has two 12-foot wide lanes and two 6-foot wide shoulders, for a total roadway width of 36' from curb to curb. The bridge deck is precast reinforced concrete with overhangs at the outermost girders. The longitudinal girders are placed as simply supported modules, and made continuous with connection plates and cast-in-place deck joints. The design of the continuity at the deck joint is addressed in final sections of this example.



FRAMING PLAN

The cross-section consists of six modules. The interior modules are identical and consist of two steel girders and a 6'-0" precast composite deck slab. Exterior modules include two steel girders and a 6'-1" precast composite deck slab, with F-shape barriers. Grade 50 steel is used throughout, and the deck concrete has a compressive strength of 5,000 psi.

The closure pour joints between the modules use Ultra High Performance Concrete with a strength of 21,000 psi.

Steel girder design steps, including constructability checks, fatigue design for infinite fatigue lift (unless otherwise noted), and bearing stiffener design comprise the majority of the example. Diaphragm and deck design procedures are present, but not detailed.

Tips for reading this Design Example:

This calculation was prepared with Mathcad version 14. Mathcad was used in this instance to provide a clear representation of formulas, and their execution. Design software other than Mathcad is recommended for a speedier and more accurate design.

Mathcad is not a design software. Mathcad executes user mathematical and simple logic commands.

Example 1: User inputs are noted with dark shaded boxes. Shading of boxes allows the user to easily find the location of a desired variable. Given that equations are written in mathcad in the same fashion as they are on paper, except that they are interactive, shading input cells allows the user to quicly locate inputs amongst other data on screen. Units are user inputs.

Height of Structure: $H_{structure} := 25ft$

Example 2: Equations are typed directly into the workspace. Mathcad then reads the operators and executes the calculations.

Panels are 2.5' $N_{panels} := \frac{H_{structure}}{2.5ft}$ $N_{panels} = 10$

Example 3: If Statements are an important operator that allow for the user to dictate a future value with given parameters. They are marked by a solid bar and operate with the use of program specific logic commands.

Operator offers discount per volume of panels

$$Discount := \left| \begin{array}{l} .75 \ \text{ if } \ N_{panels} \geq 6 \\ .55 \ \text{ if } \ N_{panels} \geq 10 \\ 1 \ \text{ otherwise} \end{array} \right. \qquad Discount = 0.6$$

Example 4: True or False Verification Statements are an important operator that allow for the user to verify a system criteria that has been manually input. They are marked by lighter shading to make a distinction between the user inputs. True or false statements check a single or pairs of variables and return a Zero or One.

Owner to proceed if discounts on retail below 60% $Discount \leq .55 = 1$

2. DESIGN PHILOSOPHY

The superstructure of the bridge in this example consists of modules, which are two rolled steel girders supporting a bridge deck panel along their length. The girders are assumed to be simply supported under the weight of the deck panels. In each module, one girder is assumed to support half the weight of its respective deck panel.

The barrier wall is added to exterior modules once the deck and girders are joined. When working with the barrier dead load, the weight is assumed to be evenly distributed between the two modules. Under the additional barrier dead load, the girders are again assumed to be simply supported.

Concerning transportation of modules, it is assumed that the deck has reached 28-day concrete strength, and the deck is fully composite with the girders. The self-weight of the module during lifting and placement is assumed as evenly distributed to four pick points (two per girder).

The modules are placed such that there is a bearing on each end and are again simply supported. The continuous span

configuration, which includes two bearings per pier on either side of the UHPC joints, is analyzed for positive and negative bending and shear (using simple or refined methods). The negative bending moment above the pier is used to find the force couple for continuity design, between the compression plates at the bottom of the girders and the closure joint in the deck.

The deck design utilizes the equivalent strip method.

3. DESIGN CRITERIA

The first step for any bridge design is to establish the design criteria. The following is a summary of the primary design criteria for this design example:

Governing Specifications: AASTHO LRFD Bridge Design Specifications (6th Edition with 2012 interims)

Design Methodology: Load and Resistance Factor Design (LRFD)

Live Load Requirements: HL-93 S S3.6

Section Constraints:

$W_{mod.max} := 200 \cdot kip$ Upper limit on the weight of the modules, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

4. MATERIAL PROPERTIES

Structural Steel Yield Strength:	$F_y := 50ksi$		STable 6.4.1-1
Structural Steel Tensile Strength:	$F_u := 65ksi$		STable 6.4.1-1
Concrete 28-day Compressive Strength:	$f_c := 5ksi$	$f_{c\_uhpc} := 21ksi$	S5.4.2.1
Reinforcement Strength:	$F_s := 60ksi$		S5.4.3 & S6.10.3.7
Steel Density:	$w_s := 490pcf$		STable 3.5.1-1
Concrete Density:	$w_c := 150pcf$		STable 3.5.1-1
Modulus of Elasticity - Steel:	$E_s := 29000ksi$		
Modulus of Elasticity - Concrete:	$E_c := 33000 \cdot \left(\frac{w_c}{1000pcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 4286.8 \cdot ksi$		
Modular Ratio:	$n := ceil\left(\frac{E_s}{E_c}\right) = 7$		
Future Wearing Surface Density:	$W_{fws} := 140pcf$		STable 3.5.1-1
Future Wearing Surface Thickness:	$t_{fws} := 2.5in$	(Assumed)	

5. LOAD COMBINATIONS

The following load combinations will be used in this design example, in accordance with Table 3.4.1-1.

Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.

Strength III—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

Strength V—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

Service I—Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.

Service II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

Fatigue I—Fatigue and fracture load combination related to infinite load-induced fatigue life.

Strength I = 1.25DC + 1.5DW + 1.75(LL+IM), where IM = 33%

Strength III = 1.25DC + 1.5DW + 1.40WS

Strength V = 1.25DC + 1.5DW + 1.35(LL+IM) + 0.40WS + 1.0WL, where IM = 33%

Service I = 1.0DC + 1.0DW + 1.0(LL+IM) + 0.3WS + 1.0WL, where IM = 33%

Service II = 1.0DC + 1.0DW + 1.3(LL+IM), where IM = 33%

Fatigue I = 1.5(LL+IM), where IM = 15%

6. BEAM SECTION

Determining the proper girder depth and dimensions is a vital part of any bridge design process. The size of the girder is a major factor in the cost of the bridge. From Table 2.5.2.6.3-1, the suggested minimum overall depth of the composite I-section in a continuous span is equal to 0.032L.

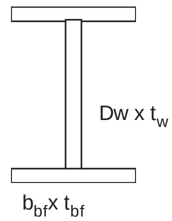
Thus we have, (.032*60ft) = 1.92’ = 23.04” (this is a minimum and may be altered to satisfy criteria)

The following girder dimensions were taken from the AISC Steel Construction Manual (14th Edition).

Determine Beam Section Properties:

Girder W30x90 b_{tf}x t_{tf}

Top Flange	$b_{tf} := 10.4in$	$t_{tf} := 0.61in$
Bottom Flange	$b_{bf} := 10.4in$	$t_{bf} := 0.61in$
Web	$D_w := 31.4in$	$t_w := 0.47in$
Girder Depth	$d_{gird} := 29.5in$	



Check Flange Proportion Requieements Met: S 6.10.2.2

$$\frac{b_{tf}}{2 \cdot t_{tf}} \le 12.0 = 1 \qquad \frac{b_{bf}}{2 \cdot t_{bf}} \le 12.0 = 1$$

$$b_{tf} \ge \frac{D_w}{6} = 1 \qquad b_{bf} \ge \frac{D_w}{6} = 1$$

$$t_{tf} \ge 1.1 \cdot t_w = 1 \qquad t_{bf} \ge 1.1 \cdot t_w = 1$$

$$0.1 \le \frac{\dfrac{t_{bf}^{\,3} \cdot b_{bf}}{12}}{\dfrac{t_{tf}^{\,3} \cdot b_{tf}}{12}} \le 10 = 1 \qquad \frac{\dfrac{t_{bf} \cdot b_{bf}}{12}}{\dfrac{t_{tf} \cdot b_{tf}}{12}} \ge 0.3 = 1$$

Properties for use when analyzing under beam self weight (steel only):

$$A_{tf} := b_{tf} \cdot t_{tf} \qquad A_{bf} := b_{bf} \cdot t_{bf} \qquad A_w := D_w \cdot t_w$$

$$A_{steel} := A_{bf} + A_{tf} + A_w \qquad A_{steel} = 27.4 \cdot in^2$$

Total steel area.

$$y_{steel} := \frac{A_{tf} \cdot \dfrac{t_{tf}}{2} + A_{bf} \cdot \left(\dfrac{t_{bf}}{2} + D_w + t_{tf}\right) + A_w \cdot \left(\dfrac{D_w}{2} + t_{tf}\right)}{A_{steel}} \qquad y_{steel} = 16.3 \cdot in$$

Steel centroid from top.

Calculate Iz:

Moment of inertia about Z axis.

$$I_{zsteel} := \frac{t_w \cdot D_w^{\,3}}{12} + \frac{b_{tf} \cdot t_{tf}^{\,3}}{12} + \frac{b_{bf} \cdot t_{bf}^{\,3}}{12} + A_w \cdot \left(\frac{D_w}{2} + t_{tf} - y_{steel}\right)^2 + A_{tf} \cdot \left(y_{steel} - \frac{t_{tf}}{2}\right)^2 + A_{bf} \cdot \left(D_w + \frac{t_{bf}}{2} + t_{tf} - y_{steel}\right)^2$$

Calculate Iy:

$$I_{ysteel} := \frac{D_w \cdot t_w^{\,3} + t_{tf} \cdot b_{tf}^{\,3} + t_{bf} \cdot b_{bf}^{\,3}}{12}$$

Moment of inertia about Y axis.

Calculate Ix:

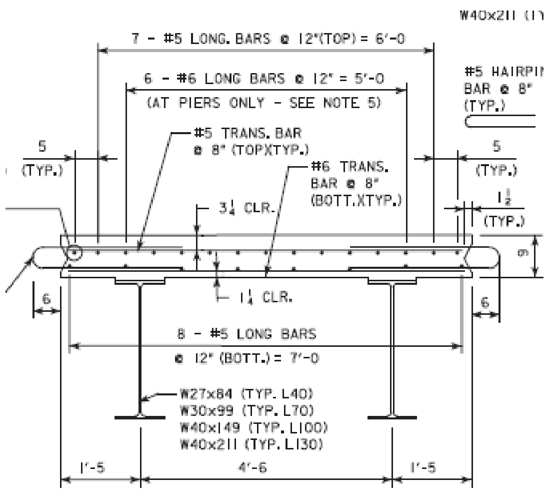
$$I_{xsteel} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\,3} + b_{bf} \cdot t_{bf}^{\,3} + D_w \cdot t_w^{\,3}\right)$$

Moment of inertia about X axis.

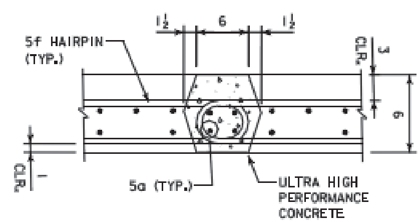
$$I_{zsteel} = 4463.118 \cdot in^4 \qquad I_{ysteel} = 114.633 \cdot in^4 \qquad I_{xsteel} = 2.7 \cdot in^4 \qquad A_{steel} = 27.4 \cdot in^2$$

Composite Section Properties (Uncracked Section - used for barrier dead load and live load positive bending):

Determine composite slab and reinforcing properties



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

Slab thickness assumes some sacrificial thickness; use: $t_{slab} := 8\text{in}$

$D_t := \left(t_{slab} + t_{tf} + D_w + t_{bf}\right) = 40.6\cdot\text{in}$ Total section depth

$b_{eff} := spacing_{int} \quad b_{eff} = 35\cdot\text{in}$ Effective width. S 4.6.2.6.1 LRFD

$b_{tr} := \dfrac{b_{eff}}{n}$ Transformed slab width as steel.

$I_{zslab} := b_{tr}\cdot\dfrac{t_{slab}^{\,3}}{12}$ Transformed slab moment of inertia about z axis as steel.

$A_{slab} := b_{tr}\cdot t_{slab}$ Transformed slab area as steel.

Slab reinforcement: (Use #5 @ 8" top, and #6 @ 8" bottom; additional bar for continuous segments of #6 @ 12")

Typical Cross Section

$A_{rt} := 0.465\,\dfrac{\text{in}^2}{\text{ft}}\cdot b_{eff} = 1.4\cdot\text{in}^2$

Cross Section Over Support

$A_{rb} := 0.66\,\dfrac{\text{in}^2}{\text{ft}}\cdot b_{eff} = 1.9\cdot\text{in}^2 \quad A_{rtadd} := 0.44\cdot\dfrac{\text{in}^2}{\text{ft}}\cdot b_{eff} = 1.3\cdot\text{in}^2$

$A_r := A_{rt} + A_{rb} = 3.3 \cdot in^2$

$c_{rt} := 2.5in + 0.625in + \left(\frac{5}{16}\right)in = 3.4 \cdot in$

$c_r := \frac{\left(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb}\right)}{A_r} = 4.9 \cdot in$

$A_{rneg} := A_r + A_{rtadd} = 4.6 \cdot in^2$

$c_{rb} := t_{slab} - 1.75in - \left(\frac{6}{16}\right)in = 5.9 \cdot in$ ref from top of slab

$c_{rneg} := \frac{\left(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb} + A_{rtadd} \cdot c_{rt}\right)}{A_{rneg}} = 4.5 \cdot in$

Find composite section centroid:

$A_x := A_{steel} + \frac{A_r \cdot (n-1)}{n} + A_{slab}$ $\qquad y_{slab} := \frac{t_{slab}}{2}$

$$y_{st} := \frac{A_{tf} \cdot \left(\frac{t_{tf}}{2} + t_{slab}\right) + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf} + t_{slab}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf} + t_{slab}\right)}{A_{steel}}$$

Centroid of steel from top of slab.

$$y_c := \frac{y_{st} \cdot A_{steel} + \frac{c_r \cdot A_r \cdot (n-1)}{n} + A_{slab} \cdot y_{slab}}{A_x} \qquad y_c = 12 \cdot in$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Iz for composite section:

$$I_z := I_{zsteel} + A_{steel} \cdot \left(y_{st} - y_c\right)^2 + I_{zslab} + A_{slab} \cdot \left(y_{slab} - y_c\right)^2 + \frac{A_r \cdot (n-1)}{n} \cdot \left(c_r - y_c\right)^2$$

Transformed moment of inertia about the z axis.

Calculate Transformed Iy for composite section:

$t_{tr} := \frac{t_{slab}}{n}$ Transformed slab thickness.

$I_{yslab} := \frac{t_{tr} \cdot b_{eff}^{\;3}}{12}$ Transformed moment of inertia about y axis of slab.

$I_y := I_{ysteel} + I_{yslab}$ Transformed moment of inertia about the y axis (ignoring reinforcement).

Calculate Transformed Ix for composite section:

$$I_x := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\;3} + b_{bf} \cdot t_{bf}^{\;3} + D_w \cdot t_w^{\;3} + b_{tr} \cdot t_{slab}^{\;3}\right)$$

Transformed moment of inertia about the x axis.

Results: $A_x = 70.3 \cdot in^2$ $\quad I_y = 4198 \cdot in^4$ $\quad I_z = 11538.5 \cdot in^4$ $\quad I_x = 856 \cdot in^4$

Composite Section Properties (Uncracked Section - used for live load negative bending):

Find composite section area and centroid:

$A_{xneg} := A_{steel} + \frac{A_{rneg} \cdot (n-1)}{n} + A_{slab}$

$$y_{cneg} := \frac{y_{steel} \cdot A_{steel} + \frac{c_{rneg} \cdot A_{rneg} \cdot (n-1)}{n} + A_{slab} \cdot y_{slab}}{A_{xneg}} \qquad y_{cneg} = 8.8 \cdot in$$

Centroid of transformed composite section from top of slab.

Calculate Transformed Izneg for composite negative moment section:

$$I_{zneg} := I_{zsteel} + A_{steel} \cdot (y_{steel} - y_{cneg})^2 + I_{zslab} + A_{slab} \cdot (y_{slab} - y_{cneg})^2 + \frac{A_{rneg} \cdot (n-1)}{n} \cdot (c_{rneg} - y_{cneg})^2$$

Transformed moment of inertia about the z axis.

$$I_{zneg} = 7219.4 \cdot in^4$$

Composite Section Properties (Cracked Section - used for live load negative bending):

Find cracked section area and centroid:

$$A_{cr} := A_{steel} + A_{rneg} = 32 \cdot in^2$$

$$y_{cr} := \frac{(A_{steel} \cdot y_{steel} + A_{rneg} \cdot c_{rneg})}{A_{cr}} = 14.6 \cdot in \qquad y_{crb} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} = 26 \cdot in$$

Find cracked section moments of inertia and section moduli:

$$I_{zcr} := I_{zsteel} + A_{steel} \cdot (y_{steel} - y_{cr})^2 + A_r \cdot (c_r - y_{cr})^2 \qquad I_{zcr} = 4853.6 \cdot in^4$$

$$I_{ycr} := I_{ysteel} \qquad I_{ycr} = 114.6 \cdot in^4$$

$$I_{xcr} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\;3} + b_{bf} \cdot t_{tf}^{\;3} + D_w \cdot t_w^{\;3}\right) \qquad I_{xcr} = 2.7 \cdot in^4$$

$$d_{topcr} := y_{cr} - c_{rt} \qquad d_{topcr} = 11.2 \cdot in$$

$$d_{botcr} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} \qquad d_{botcr} = 26 \cdot in$$

$$S_{topcr} := \frac{I_{zcr}}{d_{topcr}} \qquad S_{topcr} = 434 \cdot in^3$$

$$S_{botcr} := \frac{I_{zcr}}{d_{botcr}} \qquad S_{botcr} = 186.7 \cdot in^3$$

7. PERMANENT LOADS

Phase 1: Steel girders are simply supported, and support their self-weight plus the weight of the slab. Steel girders in each module for this example are separated by three diaphragms - one at each bearing location, and one at midspan. Other module span configurations may require an increase or decrease in the number of diaphragms.

$$W_{deck\_int} := w_c \cdot spacing_{int} \cdot t_d \qquad W_{deck\_int} = 382.8 \cdot plf$$

$$W_{deck\_ext} := w_c \cdot spacing_{ext} \cdot t_d \qquad W_{deck\_ext} = 393.8 \cdot plf$$

$$W_{haunch} := w_c \cdot w_h \cdot t_h \qquad W_{haunch} = 21.9 \cdot plf$$

$$W_{stringer} := w_{s1} \qquad W_{stringer} = 90 \cdot plf$$

Diaphragms:	MC18x42.7	Thickness Conn. Plate	$t_{conn} := \frac{5}{8} in$
Diaphragm Weight	$w_{s2} := 42.7plf$	Width Conn. Plate	$w_{conn} := 5in$
Diaphragm Length	$L_{diaph} := 4ft + 2.5in$	Height Conn. Plate	$h_{conn} := 28.5in$

$$W_{diaphragm} := w_{s2} \cdot \frac{L_{diaph}}{2} \qquad W_{diaphragm} = 89.8 \cdot lbf$$

$W_{conn} := 2 \cdot w_s \cdot t_{conn} \cdot w_{conn} \cdot h_{conn}$ $\qquad W_{conn} = 50.5 \cdot lbf$

$W_{DCpoint} := \left(W_{diaphragm} + W_{conn}\right) \cdot 1.05$ $\qquad W_{DCpoint} = 147.4 \cdot lbf$

Equivalent distributed load from DC point loads: $\qquad w_{DCpt\_equiv} := \frac{3 \cdot W_{DCpoint}}{L_{str}} = 7.4 \cdot plf$

Interior Uniform Dead Load, Phase 1: $\qquad W_{DCuniform1\_int} := W_{deck\_int} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 502.1 \cdot plf$

Exterior Uniform Dead Load, Phase 1: $\qquad W_{DCuniform1\_ext} := W_{deck\_ext} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 513.1 \cdot plf$

Moments due to Phase 1 DL: $\qquad M_{DC1\_int}(x) := \frac{W_{DCuniform1\_int} \cdot x}{2} \cdot \left(L_{str} - x\right) \qquad M_{DC1\_ext}(x) := \frac{W_{DCuniform1\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shear due to Phase 1 DL: $\qquad V_{DC1\_int}(x) := W_{DCuniform1\_int} \cdot \left(\frac{L_{str}}{2} - x\right) \qquad V_{DC1\_ext}(x) := W_{DCuniform1\_ext} \cdot \left(\frac{L_{str}}{2} - x\right)$

Phase 2: Steel girders are simply supported and composite with the deck slab, and support their self-weight plus the weight of the slab in addition to barriers on exterior modules. Barriers are assumed to be evenly distributed between the two exterior module girders.

Barrier Area $\qquad A_{barrier} := 2.89ft^2$

Barrier Weight $\qquad W_{barrier} := \frac{\left(w_c \cdot A_{barrier}\right)}{2} \qquad W_{barrier} = 216.8 \cdot plf$

Interior Dead Load, Phase 2: $\qquad W_{DCuniform\_int} := W_{DCuniform1\_int} = 502.1 \cdot plf$

Exterior Dead Load, Phase 2: $\qquad W_{DCuniform\_ext} := W_{DCuniform1\_ext} + W_{barrier} = 729.8 \cdot plf$

Moments due to Phase 2 DL: $\qquad M_{DC2\_int}(x) := \frac{W_{DCuniform\_int} \cdot x}{2} \cdot \left(L_{str} - x\right) \qquad M_{DC2\_ext}(x) := \frac{W_{DCuniform\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shear due to Phase 2 DL: $\qquad V_{DC2\_int}(x) := W_{DCuniform\_int} \cdot \left(\frac{L_{str}}{2} - x\right) \qquad V_{DC2\_ext}(x) := W_{DCuniform\_ext} \cdot \left(\frac{L_{str}}{2} - x\right)$

Phase 3: Girders are composite and have been made continuous. Utilities and future wearing surface are applied.

Unit Weight Overlay $\qquad w_{ws} := 30psf$

$W_{ws\_int} := w_{ws} \cdot spacing_{int}$ $\qquad W_{ws\_int} = 87.5 \cdot plf$

$W_{ws\_ext} := w_{ws} \cdot \left(spacing_{ext} - 1 \cdot ft - 7in\right)$ $\qquad W_{ws\_ext} = 42.5 \cdot plf$

Unit Weight Utilities $\qquad W_u := 15plf$

$W_{DWuniform\_int} := W_{ws\_int} + W_u$ $\qquad W_{DWuniform\_int} = 102.5 \cdot plf$

$W_{DWuniform\_ext} := W_{ws\_ext} + W_u$ $\qquad W_{DWuniform\_ext} = 57.5 \cdot plf$

Moments due to DW: $\qquad M_{DW\_int}(x) := \frac{W_{DWuniform\_int} \cdot x}{2} \cdot \left(L_{str} - x\right) \qquad M_{DW\_ext}(x) := \frac{W_{DWuniform\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shears due to DW: $\qquad V_{DW\_int}(x) := W_{DWuniform\_int} \cdot \left(\frac{L_{str}}{2} - x\right) \qquad V_{DW\_ext}(x) := W_{DWuniform\_ext} \cdot \left(\frac{L_{str}}{2} - x\right)$

8. PRECAST LIFTING WEIGHTS AND FORCES

This section addresses the construction loads for lifting the module into place. The module is lifted from four points, at some distance, D_{lift} from each end of each girder.

Distance from end of lifting point: $D_{lift} := 8.75ft$

Assume weight uniformly distributed along girder, with 30% Dynamic Dead Load Allowance:

Dynamic Dead Load Allowance: $DLIM := 30\%$

Interior Module:

Total Interior Module Weight: $W_{int} := \left(L_{str} \cdot W_{DCuniform\_int} + 3 \cdot W_{DCpoint}\right) \cdot 2 \cdot (1 + DLIM) = 78.8 \cdot kip$

Vertical force at lifting point: $F_{lift\_int} := \dfrac{W_{int}}{4} = 19.7 \cdot kip$

Equivalent distributed load: $w_{int\_IM} := \dfrac{W_{int}}{\left(2 \cdot L_{str}\right)} = 662.4 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_int} := -w_{int\_IM} \cdot \dfrac{\left(D_{lift}^{\;2}\right)}{2}$ $M_{lift\_neg\_max\_int} = -25.4 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting:
$$M_{lift\_pos\_max\_int} := \left| \begin{array}{l} 0 \quad \text{if} \quad \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} < 0 \\ \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} \end{array} \right.$$

$M_{lift\_pos\_max\_int} = 120.7 \cdot kip \cdot ft$

Exterior Module:

Total Exterior Module Weight: $W_{ext} := \left(L_{str} \cdot W_{DCuniform\_ext} + 3 \cdot W_{DCpoint} + W_{barrier} \cdot L_{str}\right) \cdot 2 \cdot (1 + DLIM) = 147.6 \cdot kip$

Vertical force at lifting point: $F_{lift\_ext} := \dfrac{W_{ext}}{4} = 36.9 \cdot kip$

Equivalent distributed load: $w_{ext\_IM} := \dfrac{W_{ext}}{2 \cdot L_{str}} = 1240.2 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_ext} := -w_{ext\_IM} \cdot \dfrac{D_{lift}^{\;2}}{2}$ $M_{lift\_neg\_max\_ext} = -47.5 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting: $M_{lift\_pos\_max\_ext} := \begin{cases} 0 \text{ if } \dfrac{w_{ext\_IM} \cdot (L_{str} - 2 \cdot D_{lift})^2}{8} + M_{lift\_neg\_max\_ext} < 0 \\ \dfrac{w_{ext\_IM} \cdot (L_{str} - 2 \cdot D_{lift})^2}{8} + M_{lift\_neg\_max\_ext} \end{cases}$

$M_{lift\_pos\_max\_ext} = 226 \cdot kip \cdot ft$

Max Shear during lifting: $V_{lift} := \max(w_{ext\_IM} \cdot D_{lift}, F_{lift\_ext} - w_{ext\_IM} \cdot D_{lift}) = 26 \cdot kip$

9. LIVE LOAD DISTRIBUTION FACTORS

These factors represent the distribution of live load from the deck to the girders in accordance with AASHTO Section 4, and assumes the deck is fully continuous across the joints.

Girder Section Modulus: $I_{zsteel} = 4463.1 \cdot in^4$

Girder Area: $A_{steel} = 27.4 \cdot in^2$

Girder Depth: $d_{gird} = 29.5 \cdot in$

Distance between centroid of deck and centroid of beam: $e_g := \dfrac{t_d}{2} + t_h + \dfrac{d_{gird}}{2} = 22 \cdot in$

Modular Ratio: $n = 7$

Multiple Presence Factors: $MP_1 := 1.2$ $\qquad MP_2 := 1.0$ $\qquad$ S3.6.1.1.2-1

Interior Stringers for Moment:

One Lane Loaded: $K_g := n \cdot (I_{zsteel} + A_{steel} \cdot e_g^2) = 124228.9 \cdot in^4$ $\qquad$ S4.6.2.2.1-1

$$g_{int\_1m} := \left[0.06 + \left(\frac{spacing_{int}}{14ft}\right)^{0.4} \cdot \left(\frac{spacing_{int}}{L_{span}}\right)^{0.3} \cdot \left(\frac{K_g}{L_{span} \cdot t_d^3}\right)^{0.1}\right] = 0.241$$

Two Lanes Loaded: $$g_{int\_2m} := \left[0.075 + \left(\frac{spacing_{int}}{9.5ft}\right)^{0.6} \cdot \left(\frac{spacing_{int}}{L_{span}}\right)^{0.2} \cdot \left(\frac{K_g}{L_{span} \cdot t_d^3}\right)^{0.1}\right] = 0.3$$

Governing Factor: $g_{int\_m} := \max(g_{int\_1m}, g_{int\_2m}) = 0.3$

Interior Stringers for Sheai

One Lane Loaded: $g_{int\_1v} := \left(0.36 + \dfrac{spacing_{int}}{25ft}\right) = 0.477$

Two Lanes Loaded: $$g_{int\_2v} := \left[0.2 + \frac{spacing_{int}}{12ft} + -\left(\frac{spacing_{int}}{35ft}\right)^2\right] = 0.436$$

Governing Factor: $g_{int\_v} := \max(g_{int\_1v}, g_{int\_2v}) = 0.477$

Exterior Stringers for Moment:

One Lane Loaded: Use Lever Rule. Wheel is 2' from barrier; barrier is 2" beyond exterior stringer.

$d_e := 2in$

$L_{spa} := 4.5ft \qquad r := L_{spa} + d_e - 2ft = 2.7 \cdot ft$

$g_{ext\_1m} := MP_1 \cdot \frac{0.5r}{L_{spa}} = 0.356$

Two Lanes Loaded: $e_{2m} := 0.77 + \frac{d_e}{9.1ft} = 0.7883$

$g_{ext\_2m} := e_{2m} \cdot g_{int\_2m} = 0.236$

Governing Factor: $g_{ext\_m} := \max(g_{ext\_1m}, g_{ext\_2m}) = 0.356$

Exterior Stringers for Shear:

One Lane Loaded: Use Lever Rule.

$g_{ext\_1v} := g_{ext\_1m} = 0.356$

Two Lanes Loaded: $e_{2v} := 0.6 + \frac{d_e}{10ft} = 0.62$

$g_{ext\_2v} := e_{2v} \cdot g_{int\_2v} = 0.269$

Governing Factor: $g_{ext\_v} := \max(g_{ext\_1v}, g_{ext\_2v}) = 0.356$

FACTOR TO USE FOR SHEAR: $g_v := \max(g_{int\_v}, g_{ext\_v}) = 0.477$

FACTOR TO USE FOR MOMENT: $g_m := \max(g_{int\_m}, g_{ext\_m}) = 0.356$

10. LOAD RESULTS

Case 1: Dead Load on Steel Only (calculated in Section 7). Negative moments are zero and are not considered. Because the girder is simply supported, the maximum moment is at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder

$M_{DC1int} := M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 222.2 \cdot kip \cdot ft \qquad M_{DW1int} := 0 \cdot kip \cdot ft \qquad M_{LL1int} := 0kip \cdot ft$

$V_{DC1int} := V_{DC1\_int}(0) = 14.9 \cdot kip \qquad V_{DW1int} := 0 \cdot kip \qquad V_{LL1int} := 0 \cdot kip$

Exterior Girder

$M_{DC1ext} := M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 227 \cdot kip \cdot ft \qquad M_{DW1ext} := 0 \cdot kip \cdot ft \qquad M_{LL1ext} := 0 \cdot kip \cdot ft$

$V_{DC1ext} := V_{DC1\_ext}(0) = 15.3 \cdot kip \qquad V_{DW1ext} := 0 \cdot kip \qquad V_{LL1ext} := 0 \cdot kip \cdot ft$

Load Cases:

$M_{1\_STR\_I} := \max(1.25 \cdot M_{DC1int} + 1.5 \cdot M_{DW1int} + 1.75 \cdot M_{LL1int}, 1.25 \cdot M_{DC1ext} + 1.5 \cdot M_{DW1ext} + 1.75 \cdot M_{LL1ext}) = 283.8 \cdot kip \cdot$

$V_{1\_STR\_I} := \max(1.25 \cdot V_{DC1int} + 1.5 \cdot V_{DW1int} + 1.75 \cdot V_{LL1int}, 1.25 \cdot V_{DC1ext} + 1.5 \cdot V_{DW1ext} + 1.75 \cdot V_{LL1ext}) = 19.1 \cdot kip$

Case 2: Dead Load on Composite Section (calculated in Section 7). Negative moments are zero and are not considered. Again, the maximum moment occur at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder

$M_{DC2int} := M_{DC2\_int}\left(\frac{L_{str}}{2}\right) = 222.2 \cdot kip \cdot ft \qquad M_{DW2int} := 0 \cdot kip \cdot ft \qquad M_{LL2int} := 0 \cdot kip \cdot ft$

$V_{DC2int} := V_{DC2\_int}(0) = 14.9 \cdot kip \qquad V_{DW2int} := 0 \cdot kip \qquad V_{LL2int} := 0 \cdot kip$

Exterior Girder

$M_{DC2ext} := M_{DC2\_ext}\left(\frac{L_{str}}{2}\right) = 323 \cdot kip \cdot ft \qquad M_{DW2ext} := 0 \cdot kip \cdot ft \qquad M_{LL2ext} := 0 \cdot kip \cdot ft$

$V_{DC2ext} := V_{DC2\_ext}(0) = 21.7 \cdot kip \qquad V_{DW2ext} := 0 \cdot kip \qquad V_{LL2ext} := 0 \cdot kip$

Load Cases:

$M_{2\_STR\_I} := \max(1.25 \cdot M_{DC2int} + 1.5 \cdot M_{DW2int} + 1.75 \cdot M_{LL2int}, 1.25 \cdot M_{DC2ext} + 1.5 \cdot M_{DW2ext} + 1.75 \cdot M_{LL2ext}) = 403.7 \cdot kip \cdot$

$$V_{2\_STR\_I} := \max\left(1.25 \cdot V_{DC2int} + 1.5 \cdot V_{DW2int} + 1.75 \cdot V_{LL2int}, 1.25 \cdot V_{DC2ext} + 1.5 \cdot V_{DW2ext} + 1.75 \cdot V_{LL2ext}\right) = 27.1 \cdot kip$$

Case 3: Composite girders are lifted into place from lifting points located distance D_{lift} from the girder edges. Maximum moments and shears were calculated in Section 8.

Interior Girder

$M_{DC3int} := M_{lift\_pos\_max\_int} = 120.7 \cdot kip \cdot ft$ $\quad M_{DW3int} := 0 \cdot kip \cdot ft$ $\quad M_{LL3int} := 0 \cdot kip \cdot ft$

$M_{DC3int\_neg} := \left|M_{lift\_neg\_max\_int}\right| = 25.4 \cdot kip \cdot ft$ $\quad M_{DW3int\_neg} := 0 \cdot kip \cdot ft$ $\quad M_{LL3int\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3int} := V_{lift} = 26 \cdot kip$ $\quad V_{DW3int} := 0 \cdot kip$ $\quad V_{LL3int} := 0 \cdot kip$

Exterior Girder

$M_{DC3ext} := M_{lift\_pos\_max\_ext} = 226 \cdot kip \cdot ft$ $\quad M_{DW3ext} := 0 \cdot kip \cdot ft$ $\quad M_{LL3ext} := 0 \cdot kip \cdot ft$

$M_{DC3ext\_neg} := \left|M_{lift\_neg\_max\_ext}\right| = 47.5 \cdot kip \cdot ft$ $\quad M_{DW3ext\_neg} := 0 \cdot kip \cdot ft$ $\quad M_{LL3ext\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3ext} := V_{lift} = 26 \cdot kip$ $\quad V_{DW3ext} := 0 \cdot kip$ $\quad V_{LL3ext} := 0 \cdot kip$

Load Cases:

$$M_{3\_STR\_I} := \max\left(1.5 \cdot M_{DC3int} + 1.5 \cdot M_{DW3int}, 1.5 \cdot M_{DC3ext} + 1.5 \cdot M_{DW3ext}\right) = 339 \cdot kip \cdot ft$$

$$M_{3\_STR\_I\_neg} := \max\left(1.5 \cdot M_{DC3int\_neg} + 1.5 \cdot M_{DW3int\_neg}, 1.5 \cdot M_{DC3ext\_neg} + 1.5 \cdot M_{DW3ext\_neg}\right) = 71.2 \cdot kip \cdot ft$$

$$V_{3\_STR\_I} := \max\left(1.5 \cdot V_{DC3int} + 1.5 \cdot V_{DW3int}, 1.5 \cdot V_{DC3ext} + 1.5 \cdot V_{DW3ext}\right) = 39.1 \cdot kip$$

Case 4: Composite girders made continuous. Utilities and future wearing surface are applied, and live load. Maximum moment and shear results are from a finite element analysis not included in this design example. The live load value includes the lane fraction calculated in Section 9, and impact.

Governing Loads:

$M_{DC4} := 210.2 kip \cdot ft$ $\quad M_{DW4} := 29.5 \cdot kip \cdot ft$ $\quad M_{LL4} := 350.16 \cdot kip \cdot ft$

$M_{WS4} := 0 kip \cdot ft$ $\quad M_{W4} := 0 kip \cdot ft$

$M_{DC4neg} := -262.7 \cdot kip \cdot ft$ $\quad M_{DW4neg} := -36.9 \cdot kip \cdot ft$ $\quad M_{LL4neg} := -310.79 kip \cdot ft$

$M_{WS4neg} := 0 \cdot kip \cdot ft$ $\quad M_{WL4neg} := 0 \cdot kip \cdot ft$

$V_{DC} := 26.3 kip$ $\quad V_{DW} := 3.7 kip$ $\quad V_{LL} := 100.6 kip$

$$V_u := 1.25 \cdot V_{DC} + 1.5 \cdot V_{DW} + 1.75 \cdot V_{LL} \cdot g_v = 122.3 \cdot kip$$

Load Cases:

$$M_{4\_STR\_I} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.75 \cdot M_{LL4} = 919.8 \cdot kip \cdot ft$$

$$M_{4\_STR\_I\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.75 \cdot M_{LL4neg} = -927.6 \cdot kip \cdot ft$$

$$M_{4\_STR\_III} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.4 \cdot M_{WS4} = 307 \cdot kip \cdot ft$$

$$M_{4\_STR\_III\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.4 \cdot M_{WS4} = -383.7 \cdot kip \cdot ft$$

$$M_{4\_STR\_V} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.35 \cdot M_{LL4} + 0.4 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 779.7 \cdot kip \cdot ft$$

$$M_{4\_STR\_V\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.35 \cdot M_{LL4neg} + 0.4 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -803.3 \cdot kip \cdot ft$$

$$M_{4\_SRV\_I} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.0 \cdot M_{LL4} + 0.3 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 589.9 \cdot kip \cdot ft$$

$$M_{4\_SRV\_I\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.0 \cdot M_{LL4neg} + 0.3 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -610.4 \cdot kip \cdot ft$$

$$M_{4\_SRV\_II} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.3 \cdot M_{LL4} = 694.9 \cdot kip \cdot ft$$

$$M_{4\_SRV\_II\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.3 \cdot M_{LL4neg} = -703.6 \cdot kip \cdot ft$$

11. FLEXURAL STRENGTH

The flexural resistance shall be determined as specified in LRFD Design Article 6.10.6.2. Determine Stringer Plastic Moment Capacity First.

LFRD Appendix D6 Plastic Moment

Find location of PNA:

Forces:

$P_{rt} := A_{rt} \cdot F_s = 81.4 \cdot kip$ $\quad P_s := 0.85 \cdot f_c \cdot b_{eff} \cdot t_{slab} = 1190 \cdot kip$ $\quad P_w := F_y \cdot D_w \cdot t_w = 737.9 \cdot kip$

$P_{rb} := A_{rb} \cdot F_s = 115.5 \cdot kip$ $\quad P_c := F_y \cdot b_{tf} \cdot t_{tf} = 317.2 \cdot kip$ $\quad P_t := F_y \cdot b_{bf} \cdot t_{bf} = 317.2 \cdot kip$

$$PNA_{pos} := \begin{array}{|l} \text{"case 1"} \quad \text{if } (P_t + P_w) \geq (P_c + P_s + P_{rt} + P_{rb}) \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"case 2"} \quad \text{if } [(P_t + P_w + P_c) \geq (P_s + P_{rt} + P_{rb})] \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"case 3"} \quad \text{if } \left[(P_t + P_w + P_c) \geq \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"case 4"} \quad \text{if } \left[(P_t + P_w + P_c + P_{rb}) \geq \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"case 5"} \quad \text{if } \left[(P_t + P_w + P_c + P_{rb}) \geq \left(\frac{c_{rt}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"case 6"} \quad \text{if } (P_t + P_w + P_c + P_{rb} + P_{rt}) \geq \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \\ \text{"case 7"} \quad \text{if } (P_t + P_w + P_c + P_{rb} + P_{rt}) \leq \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \quad \text{otherwise} \end{array} \end{array} \end{array} \end{array} \end{array} \end{array}$$

$PNA_{pos} = \text{"case 3"}$

$$PNA_{neg} := \begin{array}{|l} \text{"case 1"} \quad \text{if } (P_c + P_w) \geq (P_t + P_{rt} + P_{rb}) \\ \text{"case 2"} \quad \text{if } [(P_t + P_w + P_c) \geq (P_{rt} + P_{rb})] \quad \text{otherwise} \end{array}$$

$PNA_{neg} = \text{"case 1"}$

<u>Calculate Y, Dp, and Mp:</u> $D := D_w$ $\quad t_s := t_{slab}$ $\quad t_h := 0$ $\quad C_{rt} := c_{rt}$ $\quad C_{rb} := c_{rb}$

<u>Case I :</u> Plastic Nuetral Axis in the Steel Web

$$Y_1 := \frac{D}{2} \cdot \left(\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right)$$

$$D_{P1} := t_s + t_h + t_{tf} + Y_1$$

$$M_{P1} := \frac{P_w}{2D} \cdot \left[{Y_1}^2 + (D - Y_1)^2\right] + \left[\begin{array}{l} P_s \cdot \left(Y_1 + \frac{t_s}{2} + t_{tf} + t_h\right) + P_{rt} \cdot (t_s - C_{rt} + t_{tf} + Y_1 + t_h) + P_{rb} \cdot (t_s - C_{rb} + t_{tf} + Y_1 + t_h) \ldots \\ + P_c \cdot \left(Y_1 + \frac{t_{tf}}{2}\right) + P_t \cdot \left(D - Y_1 + \frac{t_{bf}}{2}\right) \end{array}\right]$$

$$Y_{1neg} := \left(\frac{D}{2}\right) \cdot \left[1 + \frac{(P_c - P_t - P_{rt} - P_{rb})}{P_w}\right]$$

$$D_{p1neg} := t_s + t_h + t_{tf} + Y_{1neg}$$

$$D_{CP1neg} := \left(\frac{D}{2 \cdot P_w}\right) \cdot (P_t + P_w + P_{rb} + P_{rt} - P_c)$$

$$M_{p1neg} := \left[\left(\frac{P_w}{2 \cdot D}\right) \cdot \left[Y_{1neg}{}^2 + \left(D_w - Y_{1neg}\right)^2\right] + P_{rt} \cdot \left(t_s - C_{rt} + t_{tf} + Y_{1neg} + t_h\right) + P_{rb} \cdot \left(t_s - C_{rb} + t_{tf} + Y_{1neg} + t_h\right) \ldots \atop + P_t \cdot \left(D - Y_{1neg} + \frac{t_{bf}}{2}\right) + P_c \cdot \left(Y_{1neg} + \frac{t_{tf}}{2}\right)\right]$$

Case II: Plastic Nuetral Axis in the Steel Top Flange

$$Y_2 := \frac{t_{tf}}{2} \cdot \left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right) \qquad D_{P2} := t_s + t_h + Y_2$$

$$M_{P2} := \frac{P_c}{2t_{tf}} \cdot \left[Y_2{}^2 + \left(t_{tf} - Y_2\right)^2\right] + \left[P_s \cdot \left(Y_2 + \frac{t_s}{2} + t_h\right) + P_{rt} \cdot \left(t_s - C_{rt} + t_h + Y_2\right) + P_{rb} \cdot \left(t_s - C_{rb} + t_h + Y_2\right) \ldots \atop + P_w \cdot \left(\frac{D}{2} + t_{tf} - Y_2\right) + P_t \cdot \left(D - Y_2 + \frac{t_{bf}}{2} + t_{tf}\right)\right]$$

$$Y_{2neg} := \left(\frac{t_{tf}}{2}\right) \cdot \left[1 + \frac{\left(P_w + P_c - P_{rt} - P_{rb}\right)}{P_t}\right] \qquad D_{P2neg} := t_s + t_h + Y_{2neg} \qquad D_{CP2neg} := D$$

$$M_{p2neg} := \left(\frac{P_t}{2 \cdot t_{tf}}\right) \cdot \left[Y_{2neg}{}^2 + \left(t_{tf} - Y_{2neg}\right)^2\right] + \left[P_{rt} \cdot \left(t_s - C_{rt} + t_h + Y_{2neg}\right) + P_{rb} \cdot \left(t_s - C_{rb} + t_h + Y_{2neg}\right) \ldots \atop + P_w \cdot \left(t_{tf} - Y_{2neg} + \frac{D}{2}\right) + P_c \cdot \left(\left|t_s + t_h - Y_{2neg} + \frac{t_{tf}}{2}\right|\right)\right]$$

Case III: Plastic Nuetral Axis in the Concrete Deck Below the Bottom Reinforcing

$$Y_3 := t_s \cdot \left(\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right) \qquad D_{P3} := Y_3$$

$$M_{P3} := \frac{P_s}{2t_s} \cdot \left(Y_3{}^2\right) + \left[P_{rt} \cdot \left(Y_3 - C_{rt}\right) + P_{rb} \cdot \left(C_{rb} - Y_3\right) + P_c \cdot \left(\frac{t_{tf}}{2} + t_s + t_h - Y_3\right) + P_w \cdot \left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_3\right) \ldots \atop + P_t \cdot \left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_3\right)\right]$$

Case IV: Plastic Nuetral Axis in the Concrete Deck in the bottom reinforcing layer

$$Y_4 := C_{rb} \qquad D_{P4} := Y_4$$

$$M_{P4} := \frac{P_s}{2t_s} \cdot \left(Y_4{}^2\right) + \left[P_{rt} \cdot \left(Y_4 - C_{rt}\right) + P_c \cdot \left(\frac{t_{tf}}{2} + t_h + t_s - Y_4\right) + P_w \cdot \left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_4\right) \ldots \atop + P_t \cdot \left(D + \frac{t_{bf}}{2} + t_{tf} + t_h + t_s - Y_4\right)\right]$$

Case V: Plastic Nuetral Axis in the Concrete Deck between top and bot reinforcing layers

$$Y_5 := t_s \cdot \left(\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right) \qquad D_{P5} := Y_5$$

$$M_{P5} := \frac{P_s}{2t_s} \cdot \left(Y_5{}^2\right) + \left[P_{rt} \cdot \left(Y_5 - C_{rt}\right) + P_{rb} \cdot \left[\left(t_s - C_{rb}\right) - Y_5\right] + P_c \cdot \left(\frac{t_{tf}}{2} + t_s + t_h - Y_5\right) + P_w \cdot \left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_5\right) \ldots \atop + P_t \cdot \left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_5\right)\right]$$

$$Y_{pos} := \begin{cases} Y_1 & \text{if } PNA_{pos} = \text{"case 1"} \\ Y_2 & \text{if } PNA_{pos} = \text{"case 2"} \\ Y_3 & \text{if } PNA_{pos} = \text{"case 3"} \\ Y_4 & \text{if } PNA_{pos} = \text{"case 4"} \\ Y_5 & \text{if } PNA_{pos} = \text{"case 5"} \end{cases} \qquad D_{Ppos} := \begin{cases} D_{P1} & \text{if } PNA_{pos} = \text{"case 1"} \\ D_{P2} & \text{if } PNA_{pos} = \text{"case 2"} \\ D_{P3} & \text{if } PNA_{pos} = \text{"case 3"} \\ D_{P4} & \text{if } PNA_{pos} = \text{"case 4"} \\ D_{P5} & \text{if } PNA_{pos} = \text{"case 5"} \end{cases} \qquad M_{Ppos} := \begin{cases} M_{P1} & \text{if } PNA_{pos} = \text{"case 1"} \\ M_{P2} & \text{if } PNA_{pos} = \text{"case 2"} \\ M_{P3} & \text{if } PNA_{pos} = \text{"case 3"} \\ M_{P4} & \text{if } PNA_{pos} = \text{"case 4"} \\ M_{P5} & \text{if } PNA_{pos} = \text{"case 5"} \end{cases}$$

$Y_{pos} = 7.9 \cdot in$ $\qquad D_{Ppos} = 7.9 \cdot in$ $\qquad M_{Ppos} = 2274.2 \cdot kip \cdot ft$

Dp = distance from the top of slab of composite section to the neutral axis at the plastic moment (neglect positive moment reinforcement in the slab).

$$Y_{neg} := \begin{cases} Y_{1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ Y_{2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases} \qquad D_{Pneg} := \begin{cases} D_{p1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ D_{P2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases} \qquad M_{Pneg} := \begin{cases} M_{p1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ M_{p2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases}$$

$Y_{neg} = 11.5 \cdot in$ $\qquad D_{Pneg} = 20.1 \cdot in$ $\qquad M_{Pneg} = 19361.5 \cdot kip \cdot in$

Depth of web in compression at the plastic moment [D6.3.2]:

$A_t := b_{bf} \cdot t_{bf}$ $\qquad A_c := b_{tf} \cdot t_{tf}$

$$D_{cppos} := \frac{D}{2}\left(\frac{F_y \cdot A_t - F_y \cdot A_c - 0.85 \cdot f_c \cdot A_{slab} - F_s \cdot A_r}{F_y \cdot A_w} + 1\right)$$

$$D_{cppos} := \begin{cases} (0in) & \text{if } PNA_{pos} \neq \text{"case 1"} \\ (0in) & \text{if } (D_{cppos} < 0) \\ D_{cppos} & \text{if } PNA_{pos} = \text{"case 1"} \end{cases} \qquad D_{cpneg} := \begin{cases} D_{CP1neg} & \text{if } PNA_{neg} = \text{"case 1"} \\ D_{CP2neg} & \text{if } PNA_{neg} = \text{"case 2"} \end{cases}$$

$D_{cpneg} = 19.9 \cdot in$

$D_{cppos} = 0 \cdot in$

Positive Flexural Compression Check:

From LRFD Article 6.10.2

Check for compactness:

Web Proportions:

$$\frac{D_w}{t_w} \leq 150 = 1$$

Web slenderness Limit:

$$2 \cdot \frac{D_{cppos}}{t_w} \leq 3.76 \cdot \sqrt{\frac{E_s}{F_y}} = 1$$

S 6.10.6.2.2

Therefore Section is considered compact and shall satisfy the requirements of Article 6.10.7.1.

$$M_n := \begin{cases} M_{Ppos} & \text{if } D_{Ppos} \leq 0.1 \cdot D_t \\ M_{Ppos} \cdot \left(1.07 - 0.7 \cdot \frac{D_{Ppos}}{D_t}\right) & \text{otherwise} \end{cases} \qquad M_n = 2123.7 \cdot kip \cdot ft$$

Negative Moment Capacity Check (Appendix A6):

Web Slenderness: $D_t = 40.6 \cdot in$ $\qquad D_{cneg} := D_t - y_{cr} - t_{bf} = 25.4 \cdot in$

$$\frac{2 \cdot D_{cneg}}{t_w} < 5.7 \cdot \sqrt{\frac{E_s}{F_y}} = 1$$

S Appendix A6 (for skew less than 20 deg).

Moment ignoring concrete:

$M_{yt} := F_y \cdot S_{botcr} = 9334.1 \cdot kip \cdot in$ $\qquad M_{yc} := F_s \cdot S_{topcr} = 26039.6 \cdot kip \cdot in$

$M_y := \min(M_{yc}, M_{yt}) = 9334.1 \cdot kip \cdot in$

Web Compactness:

Check for Permanent Deformations (6.10.4.2):

$$D_n := \max(t_{slab} + t_{tf} + D_w - y_c, y_c - t_{slab} - t_{tf}) = 28 \cdot in$$

$$Gov := if(y_c - t_{slab} - t_{tf}, y_c - c_{rt}, D_n) = 8.5 \cdot in$$

$$f_n := \left|M_{4\_SRV\_II\_neg}\right| \cdot \frac{Gov}{I_z} = 6.2 \cdot ksi$$ Steel stress on side of Dn

$$\rho := \min\left(1.0, \frac{F_y}{f_n}\right) = 1 \qquad \beta := 2 \cdot D_n \cdot \frac{t_w}{A_{tf}} = 4.2 \qquad R_h := \frac{\left[12 + \beta \cdot \left(3\rho - \rho^3\right)\right]}{(12 + 2 \cdot \beta)} = 1$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_s}{F_y}}$$

$$\lambda_{PWdcp} := \min\left[\lambda_{rw} \cdot \frac{D_{cpneg}}{D_{cneg}}, \frac{\sqrt{\frac{E_s}{F_y}}}{\left(0.54 \cdot \frac{M_{Pneg}}{R_h \cdot M_y} - 0.09\right)^2}\right] = 22.7$$

$$2 \cdot \frac{D_{cpneg}}{t_w} \le \lambda_{PWdcp} = 0$$

Web Plastification: $R_{pc} := \frac{M_{Pneg}}{M_{yc}} = 0.7 \qquad R_{pt} := \frac{M_{Pneg}}{M_{yt}} = 2.1$

Flexure Factor: $\phi_f := 1.0$

Tensile Limit: $M_{r\_neg\_t} := \phi_f \cdot R_{pt} \cdot M_{yt} = 1613.5 \cdot kip \cdot ft$

Compressive Limit:

Local Buckling Resistance:

$$\lambda_f := \frac{b_{bf}}{2 \cdot t_{bf}} = 8.5 \qquad \lambda_{rf} := 0.95 \cdot \sqrt{0.76 \cdot \frac{E_s}{F_y}} = 19.9$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} = 9.2 \qquad F_{yresid} := \max\left(\min\left(0.7 \cdot F_y, R_h \cdot F_y \cdot \frac{S_{topcr}}{S_{botcr}}, F_y\right), 0.5 \cdot F_y\right) = 35.0 \cdot ksi$$

$$M_{ncLB} := \begin{cases} (R_{pc} \cdot M_{yc}) & \text{if } \lambda_f \le \lambda_{pf} \\ \left[R_{pc} \cdot M_{yc} \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{topcr}}{R_{pc} \cdot M_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]\right] & \text{otherwise} \end{cases} \qquad M_{ncLB} = 1613.5 \cdot kip \cdot ft$$

Lateral Torsional Buckling Resistance:

$$L_b := \frac{(L_{str})}{2 \cdot 3} = 9.9 \cdot ft$$ Inflection point assumed to be at 1/6 span

$$r_t := \frac{b_{bf}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_{cneg} \cdot t_w}{b_{bf} \cdot t_{bf}}\right)}} = 2.4 \cdot in$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_s}{F_y}} = 56.7 \cdot in \qquad h := D + t_{bf} = 32 \cdot in \qquad C_b := 1.0$$

$$J_b := \frac{D \cdot t_w^3}{3} + \frac{b_{bf} \cdot t_{bf}^3}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{bf}}{b_{bf}}\right) + \frac{b_{tf} \cdot t_{tf}^3}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{tf}}{b_{tf}}\right) = 2.6 \cdot in^4$$

$$L_r := 1.95 \cdot r_t \cdot \frac{E_s}{F_{yresid}} \cdot \sqrt{\frac{J_b}{S_{botcr} \cdot h}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{F_{yresid}}{E_s} \cdot \frac{S_{botcr} \cdot h}{J_b}\right)^2}} = 228.3 \cdot in$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J_b}{S_{botcr} \cdot h} \cdot \left(\frac{L_b}{r_t}\right)^2} = 116.7 \cdot ksi$$

$$M_{ncLTB} := \begin{cases} \left(R_{pc} \cdot M_{yc}\right) & \text{if } L_b \le L_p \\ \min\left[C_b \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{botcr}}{R_{pc} \cdot M_{yc}}\right) \cdot \frac{\left(L_b - L_p\right)}{\left(L_r - L_p\right)}\right] \cdot R_{pc} \cdot M_{yc}, R_{pc} \cdot M_{yc}\right] & \text{if } L_p < L_b \le L_r \\ \min\left(F_{cr} \cdot S_{botcr}, R_{pc} \cdot M_{yc}\right) & \text{if } L_b > L_r \end{cases}$$

$$M_{ncLTB} = 1225.3 \cdot kip \cdot ft$$

$$M_{r\_neg\_c} := \phi_f \cdot \min\left(M_{ncLB}, M_{ncLTB}\right) = 1225.3 \cdot kip \cdot ft$$

Governing negative moment capacity: $M_{r\_neg} := \min\left(M_{r\_neg\_t}, M_{r\_neg\_c}\right) = 1225.3 \cdot kip \cdot ft$

12. FLEXURAL STRENGTH CHECKS

Phase 1: First, check the stress due to the dead load on the steel section only. (LRFD 6.10.3 - Constructability Requirements

Reduction factor for construction $\phi_{const} := 0.9$

Load Combination for construction $1.25 \cdot M_{DC}$

Max Moment applied, Phase 1: (at midspan)

$$M_{int\_P1} := 1.25\, M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 277.8 \cdot kip \cdot ft \quad \text{(Interior)}$$

$$M_{ext\_P1} := 1.25\, M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 283.8 \cdot kip \cdot ft \quad \text{(Exterior)}$$

Maximum Stress, Phase 1:

$$f_{int\_P1} := \frac{M_{int\_P1} \cdot y_{steel}}{I_{zsteel}} = 12.2 \cdot ksi \quad \text{(Interior)}$$

$$f_{ext\_P1} := \frac{M_{ext\_P1} \cdot y_{steel}}{I_{zsteel}} = 12.4 \cdot ksi \quad \text{(Exterior)}$$

Stress limits: $f_{P1\_max} := \phi_{const} \cdot F_y$

$$f_{int\_P1} \le f_{P1\_max} = 1 \qquad f_{ext\_P1} \le f_{P1\_max} = 1$$

Phase 2: Second, check the stress due to dead load on the composite section (with barriers added)

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.25 \cdot M_{DC}$

Max Moment applied, Phase 2: (at midspan) $M_{2\_STR\_I} = 403.7 \cdot kip \cdot ft$

Capacity for positive flexure: $M_n = 2123.7 \cdot kip \cdot ft$

Check Moment: $M_{2\_STR\_I} \le \phi_{const} \cdot M_n = 1$

Phase 3: Next, check the flexural stress on the stringer during transport and picking, to ensure no cracking.

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.5 \cdot M_{DC}$ when dynamic construction loads are involved (Section 10).

Loads and stresses on stringer during transport and picking: $M_{3\_STR\_I\_neg} = 71.2 \cdot kip \cdot ft$

Concrete rupture stress $f_r := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.5 \cdot ksi$

Concrete stress during construction not to exceed:

$f_{cmax} := \phi_{const} \cdot f_r = 0.5 \cdot ksi$

$$f_{cconst} := \frac{M_{3\_STR\_I\_neg} \cdot y_c}{I_z \cdot n} = 0.1 \cdot ksi$$

$f_{cconst} \le f_{cmax} = 1$

Phase 4: Check flexural capacity under dead load and live load for fully installed continuous composite girders.

Strength I Load Combination $\phi_f := 1.0$

$M_{4\_STR\_I} = 919.8 \cdot kip \cdot ft$ $M_{4\_STR\_I\_neg} = -927.6 \cdot kip \cdot ft$

$M_{4\_STR\_I} \le \phi_f \cdot M_n = 1$ $\left|M_{4\_STR\_I\_neg}\right| \le M_{r\_neg} = 1$

Strength III Load Combination

$M_{4\_STR\_III} = 307 \cdot kip \cdot ft$ $M_{4\_STR\_III\_neg} = -383.7 \cdot kip \cdot ft$

$M_{4\_STR\_III} \le \phi_f \cdot M_n = 1$ $\left|M_{4\_STR\_III\_neg}\right| \le M_{r\_neg} = 1$

Strength V Load Combination

$M_{4\_STR\_V} = 779.7 \cdot kip \cdot ft$ $M_{4\_STR\_V\_neg} = -803.3 \cdot kip \cdot ft$

$M_{4\_STR\_V} \le \phi_f \cdot M_n = 1$ $\left|M_{4\_STR\_V\_neg}\right| \le M_{r\_neg} = 1$

13. FLEXURAL SERVICE CHECKS

Check service load combinations for the fully continuous beam with live load (Phase 4):

under Service II for stress limits - $M_{4\_SRV\_II} = 694.9 \cdot kip \cdot ft$

$M_{4\_SRV\_II\_neg} = -703.6 \cdot kip \cdot ft$

under Service I for cracking - $M_{4\_SRV\_I\_neg} = -610.4 \cdot kip \cdot ft$

Ignore positive moment for Service I as there is no tension in the concrete in this case.

Service Load Stress Limits:

Top Flange: $f_{tfmax} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$

Bottom Flange: $f_{bfmax} := f_{tfmax} = 47.5 \cdot ksi$

Concrete (Negative bending only): $f_r = 0.5 \cdot ksi$

Service Load Stresses, Positive Moment:

Top Flange: $f_{SRVII\_tf} := M_{4\_SRV\_II} \cdot \frac{(y_c - t_{slab})}{I_z} = 2.9 \cdot ksi$

$f_{SRVII\_tf} \le f_{tfmax} = 1$

Bottom Flange: $f_{bfs2} := M_{4\_SRV\_II} \cdot \frac{(t_{slab} + t_{tf} + D_w + t_{bf} - y_c)}{I_z} = 20.7 \cdot ksi$

$f_l := 0$ $\qquad f_{bfs2} + \frac{f_l}{2} \le f_{bfmax} = 1$

Service Load Stresses, Negative Moment:

Top (Concrete):

$f_{con.neg} := \frac{M_{4\_SRV\_I\_neg} \cdot y_{cneg}}{n \cdot I_{zneg}} = -1.3 \cdot ksi$ Using Service I Loading

$|f_{con.neg}| \le |f_r| = 0$

Bottom Flange: $f_{bfs2.neg} := \frac{M_{4\_SRV\_I\_neg} \cdot (t_{slab} + t_{tf} + D_w + t_{bf} - y_{cneg})}{I_{zneg}} = -32.3 \cdot ksi$

$f_{bfs2.neg} \le f_{bfmax} = 1$

Check LL Deflection:

$\Delta_{DT} := 1.104 \cdot in$ from independent Analysis - includes 100% design truck (w/impact), or 25% design truck (w/impact) + 100% lane load

$DF_\delta := \frac{3}{12} = 0.3$ Deflection distribution factor = (no. lanes)/(no. stringers)

$\frac{L_{str}}{\Delta_{DT} \cdot DF_\delta} = 2587$ Equivalent X, where L/X = Deflection*Distribution Factor

$\frac{L_{str}}{\Delta_{DT} \cdot DF_\delta} \ge 800 = 1$

14. SHEAR STRENGTH

Shear Capacity based on AASHTO LRFD 6.10.9

Nominal resistance of unstiffened web:

$F_y = 50.0 \cdot ksi$ $\qquad D_w = 31.4 \cdot in$ $\qquad t_w = 0.5 \cdot in$ $\qquad \phi_v := 1.0$ $\qquad k := 5$

$V_p := 0.58 \cdot F_y \cdot D_w \cdot t_w = 428.0 \cdot kip$

$$C_1 := \begin{cases} 1.0 & \text{if } \dfrac{D_w}{t_w} \le 1.12 \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}} \\ \left[\dfrac{1.57}{\left(\dfrac{D_w}{t_w}\right)^2} \cdot \left(\dfrac{E_s \cdot k}{F_y}\right)\right] & \text{if } \dfrac{D_w}{t_w} > 1.40 \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}} \\ \left(\dfrac{1.12}{\dfrac{D_w}{t_w}} \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}}\right) & \text{otherwise} \end{cases} \qquad C_1 = 0.903$$

$$V_n := C_1 \cdot V_p = 386.4 \cdot \text{kip}$$

$$V_u \le \phi_v \cdot V_n = 1$$

15. FATIGUE LIMIT STATES:

Fatigue check shall follow LRFD Article 6.10.5. Moments used for fatigue calculations were found using an outside finite element analysis program.

First check Fatigue I (infinite life); then find maximum single lane ADTT for Fatigue II if needed.

Fatigue Stress Limits:

$\Delta F_{TH\_1} := 16 \cdot \text{ksi}$ Category B: non-coated weathering steel

$\Delta F_{TH\_2} := 12 \cdot \text{ksi}$ Category C': Base metal at toe of transverse stiffener fillet welds

$\Delta F_{TH\_3} := 10 \cdot \text{ksi}$ Category C: Base metal at shear connectors

Fatigue Moment Ranges at Detail Locations (from analysis):

$M_{FAT\_B} := 301 \cdot \text{kip} \cdot \text{ft}$ $\quad M_{FAT\_CP} := 285.7 \cdot \text{kip} \cdot \text{ft}$ $\quad M_{FAT\_C} := 207.1 \text{kip} \cdot \text{ft}$

$\gamma_{FATI} := 1.5$ $\quad \gamma_{FATII} := 0.75$ $\quad n_{fat} := \begin{cases} 2 & \text{if } L_{str} \le 40 \cdot \text{ft} \\ 1.0 & \text{otherwise} \end{cases}$

Constants to use for detail checks:

$ADTT_{SL\_INF\_B} := 860$ $\quad A_{FAT\_B} := 120 \cdot 10^8$

$ADTT_{SL\_INF\_CP} := 660$ $\quad A_{FAT\_CP} := 44 \cdot 10^8$

$ADTT_{SL\_INF\_C} := 1290$ $\quad A_{FAT\_C} := 44 \cdot 10^8$

Category B Check: Stress at Bottom Flange, Fatigue I

$$f_{FATI\_B} := \frac{\gamma_{FATI} \cdot M_{FAT\_B} \cdot \left(t_{slab} + t_{tf} + D_w + t_{bf} - y_c\right)}{I_z} = 13.5 \cdot \text{ksi}$$

$$f_{FATI\_B} \le \Delta F_{TH\_1} = 1$$

$$f_{FATII\_B} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_B} = 6.7 \cdot \text{ksi}$$

$$ADDT_{SL\_B\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_B}}{n_{fat}} & \text{if } f_{FATI\_B} \le \Delta F_{TH\_1} \\ \dfrac{A_{FAT\_B} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_B}{}^3} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_B\_MAX} = 860$$

Category C' Check: Stress at base of transverse stiffener (top of bottom flange)

$$f_{FATI\_CP} := \gamma_{FATI} \cdot M_{FAT\_CP} \cdot \frac{\left(t_{slab} + t_{tf} + D_w - y_c\right)}{I_z} = 12.5 \cdot ksi$$

$$f_{FATI\_CP} \le \Delta F_{TH\_2} = 0$$

$$f_{FATII\_CP} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_CP} = 6.2 \cdot ksi$$

$$ADTT_{SL\_CP\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_CP}}{n_{fat}} & \text{if } f_{FATI\_CP} \le \Delta F_{TH\_2} \\ \dfrac{A_{FAT\_CP} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_CP}{}^3} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_CP\_MAX} = 659$$

Category C Check: Stress at base of shear connectors (top of top flange)

$$f_{FATI\_C} := \gamma_{FATI} \cdot M_{FAT\_C} \cdot \frac{\left(y_c - t_{slab}\right)}{I_z} = 1.3 \cdot ksi$$

$$f_{FATI\_C} \le \Delta F_{TH\_3} = 1$$

$$f_{FATII\_C} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_C} = 0.6 \cdot ksi$$

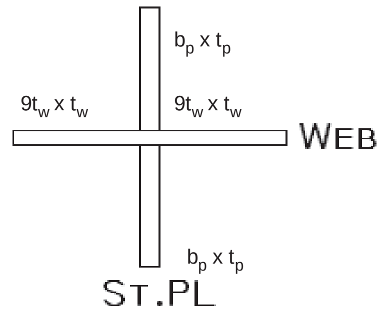
$$ADTT_{SL\_C\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_C}}{n_{fat}} & \text{if } f_{FATI\_C} \le \Delta F_{TH\_3} \\ \dfrac{A_{FAT\_C} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_C}{}^3} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_C\_MAX} = 1290$$

FATIGUE CHECK: $ADTT_{SL\_MAX} := \min\left(ADTT_{SL\_B\_MAX}, ADTT_{SL\_CP\_MAX}, ADTT_{SL\_C\_MAX}\right)$

Ensure that single lane ADTT is less than $ADTT_{SL\_MAX} = 659$

If not, then the beam requires redesign.

16. BEARING STIFFENERS



Using LRFD Article 6.10.11 for stiffeners:

$t_p := \frac{5}{8}in$ $\quad b_p := 5in$ $\quad \phi_b := 1.0$ $\quad t_{p\_weld} := \left(\frac{5}{16}\right)in$

*Check min weld size

Projecting Width Slenderness Check:

$$b_p \le 0.48t_p \cdot \sqrt{\frac{E_s}{F_y}} = 1$$

Stiffener Bearing Resistance:

$A_{pn} := 2 \cdot (b_p - t_{p\_weld}) \cdot t_p$ $\qquad A_{pn} = 5.9 \cdot in^2$

$R_{sb\_n} := 1.4 \cdot A_{pn} \cdot F_y$ $\qquad R_{sb\_n} = 410.2 \cdot kip$

$R_{sb\_r} := \phi_b \cdot R_{sb\_n}$ $\qquad R_{sb\_r} = 410.2 \cdot kip$

$R_{DC} := 26.721kip$ $\quad R_{DW} := 2.62kip$ $\quad R_{LL} := 53.943kip$

$\phi_{DC\_STR\_I} := 1.25$ $\quad \phi_{DW\_STR\_I} := 1.5$ $\quad \phi_{LL\_STR\_I} := 1.75$

$R_u := \phi_{DC\_STR\_I} \cdot R_{DC} + \phi_{DW\_STR\_I} \cdot R_{DW} + \phi_{LL\_STR\_I} \cdot R_{LL}$ $\qquad R_u = 131.7 \cdot kip$

$R_u \le R_{sb\_r} = 1$

Weld Check:

$throat := t_{p\_weld} \cdot \frac{\sqrt{2}}{2}$ $\qquad throat = 0.2 \cdot in$

$L_{weld} := D_w - 2 \cdot 3in$ $\qquad L_{weld} = 25.4 \cdot in$

$A_{eff\_weld} := throat \cdot L_{weld}$ $\qquad A_{eff\_weld} = 5.6 \cdot in^2$

$F_{exx} := 70ksi$ $\qquad \phi_{e2} := 0.8$

$R_{r\_weld} := 0.6 \cdot \phi_{e2} \cdot F_{exx}$ $\qquad R_{r\_weld} = 33.6 \cdot ksi$

$R_{u\_weld} := \frac{R_u}{4 \cdot A_{eff\_weld}}$ $\qquad R_{u\_weld} = 5.9 \cdot ksi$

$R_{u\_weld} \le R_{u\_weld} = 1$

Axial Resistance of Bearing Stiffeners: $\quad \phi_c := 0.9$

$A_{eff} := (2 \cdot 9 \cdot t_w + t_p) \cdot t_w + 2 \cdot b_p \cdot t_p$ $\qquad A_{eff} = 10.5 \cdot in^2$

$L_{eff} := 0.75 \cdot D_w$ $\qquad L_{eff} = 23.6 \cdot in$

$I_{xp} := \frac{2 \cdot 9 \cdot t_w \cdot t_w^3}{12} + \frac{t_p \cdot (2 \cdot b_p + t_w)^3}{12}$ $\qquad I_{xp} = 59.9 \cdot in^4$

$I_{yp} := \frac{t_w \cdot (t_p + 2 \cdot 9 \cdot t_w)^3}{12} + \frac{2b_p \cdot t_p^3}{12}$ $\qquad I_{yp} = 29.6 \cdot in^4$

$r_p := \sqrt{\frac{\min(I_{xp}, I_{yp})}{A_{eff}}}$ $\qquad r_p = 1.7 \cdot in$

$Q := 1$ for bearing stiffeners $\qquad K_p := 0.75$

$P_o := Q \cdot F_y \cdot A_{eff} = 526 \cdot kip$

$$P_e := \frac{\pi^2 E_s \cdot A_{eff}}{\left(K_p \cdot \frac{L_{eff}}{r_p}\right)^2} = 27132.1 \cdot kip$$

$$P_n := \begin{cases} \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o & \text{if } \left(\frac{P_e}{P_o}\right) \geq 0.44 \\ 0.877 \cdot P_e & \text{otherwise} \end{cases} \qquad P_n = 521.7 \cdot kip$$

$P_r := \phi_c \cdot P_n$ $\quad P_r = 469.6 \cdot kip$ $\quad R_u \leq P_r = 1$

17. SHEAR CONNECTORS:

Shear Connector design to follow LRFD 6.10.10.

Stud Properties:

$d_s := \frac{7}{8} \cdot in$ Diameter $\quad h_s := 6in$ Height of Stud $\quad \frac{h_s}{d_s} \geq 4 = 1$

$c_s := t_{slab} - h_s$ $\quad c_s \geq 2in = 1$

$s_s := 3.5in$ Spacing $\quad s_s \geq 4d_s = 1$

$n_s := 3$ Studs per row $\quad \frac{\left[b_{tf} - s_s \cdot (n_s - 1) - d_s\right]}{2} \geq 1.0in = 1$

$A_{sc} := \pi \cdot \left(\frac{d_s}{2}\right)^2$ $\quad A_{sc} = 0.6 \cdot in^2$

$F_u := 60ksi$

Fatigue Resistance:

$Z_r := 5.5 \cdot d_s^{\,2} \cdot \frac{kip}{in^2}$ $\quad Z_r = 4.2 \cdot kip$ $\quad Q_{slab} := A_{slab} \cdot (y_c - y_{slab})$ $\quad Q_{slab} = 318.7 \cdot in^3$

$V_f := 47.0kip$

$$V_{fat} := \frac{V_f \cdot Q_{slab}}{I_z} = 1.3 \cdot \frac{kip}{in}$$

$p_s := \frac{n_s \cdot Z_r}{V_{fat}} = 9.7 \cdot in$ $\quad 6 \cdot d_s \leq p_s \leq 24in = 1$

Strength Resistance:

$\phi_{sc} := 0.85$

$f_c = 5 \cdot ksi$

$E_c := 33000 \cdot 0.15^{1.5} \cdot \sqrt{f_c\,ksi} = 4286.8 \cdot ksi$

$Q_n := \min\left(0.5 \cdot A_{sc} \cdot \sqrt{f_c \cdot E_c}, A_{sc} \cdot F_u\right)$ $\quad Q_n = 36.1 \cdot kip$

$Q_r := \phi_{sc} \cdot Q_n$ $\quad Q_r = 30.7 \cdot kip$

$P_{simple} := \min\left(0.85 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right)$ $\quad P_{simple} = 1190 \cdot kip$

$P_{cont} := P_{simple} + \min\left(0.45 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right)$ $\quad P_{cont} = 1820 \cdot kip$

$n_{lines} := \frac{P_{cont}}{Q_r \cdot n_s}$ $\quad n_{lines} = 19.8$

Find required stud spacing along the girder (varies as applied shear varies)

$i := 0 .. 23$

$$x := \begin{pmatrix} 0.00 \\ 1.414 \\ 4.947 \\ 8.480 \\ 12.013 \\ 15.546 \\ 19.079 \\ 22.612 \\ 26.145 \\ 29.678 \\ 33.210 \\ 33.917 \\ 34.624 \\ 36.037 \\ 36.743 \\ 40.276 \\ 43.809 \\ 47.342 \\ 50.875 \\ 54.408 \\ 57.941 \\ 61.474 \\ 65.007 \\ 67.833 \end{pmatrix} \cdot ft \quad V_{fi} := \begin{pmatrix} 61.5 \\ 59.2 \\ 56.8 \\ 54.4 \\ 52.0 \\ 49.5 \\ 47.1 \\ 44.7 \\ 42.7 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 42.3 \\ 44.2 \\ 46.6 \\ 49.1 \\ 51.5 \\ 53.9 \\ 56.3 \\ 58.7 \\ 61.5 \end{pmatrix} \cdot kip$$

$$V_{fati} := \frac{V_{fi} \cdot Q_{slab}}{I_z} = \cdot \frac{kip}{in}$$

	0
0	1.7
1	1.6
2	1.6
3	1.5
4	1.4
5	1.4
6	1.3
7	1.2
8	1.2
9	1.1
10	1.1
11	1.1
12	1.1
13	1.1
14	1.1
15	...

$$P_{max} := \frac{n_s \cdot Z_r}{V_{fati}} = \cdot in$$

	0
0	7.4
1	7.7
2	8.1
3	8.4
4	8.8
5	9.2
6	9.7
7	10.2
8	10.7
9	11.3
10	11.3
11	11.3
12	11.3
13	11.3
14	11.3
15	...

$\min(P_{max}) = 7.4 \cdot in$

$\max(P_{max}) = 11.3 \cdot in$

18. SLAB PROPERTIES

This section details the geometric and material properties of the deck. Because the equivalent strip method is used in accordance with AASHTO LRFD Section 4, different loads are used for positive and negative bending.

Unit Weight Concrete $w_c = 150 \cdot pcf$

Deck Thickness for Design $t_{deck} := 8.0in$ $\quad t_{deck} \geq 7in = 1$

Deck Thickness for Loads $t_d = 10.5 \cdot in$

Rebar yield strength $F_s = 60 \cdot ksi$ $\quad$ Strength of concrete $f_c = 5 \cdot ksi$

Concrete clear cover $\quad$ Bottom $\quad$ Top

$c_b := 1.0in$ $\quad c_b \geq 1.0in = 1$ $\quad c_t := 2.5in$ $\quad c_t \geq 2.5in = 1$

Transverse reinforcement	Bottom Reinforcing	$\phi_{tb} := \frac{6}{8} in$	Top Reinforcing	$\phi_{tt} := \frac{5}{8} in$
	Bottom Spacing	$s_{tb} := 8in$	Top Spacing	$s_{tt} := 8in$

$s_{tb} \geq 1.5\phi_{tb} \wedge 1.5in = 1$ $\quad$ $s_{tt} \geq 1.5\phi_{tt} \wedge 1.5in = 1$

$s_{tb} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$ $\quad$ $s_{tt} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$$A_{stb} := \frac{12in}{s_{tb}} \cdot \pi \cdot \left(\frac{\phi_{tb}}{2}\right)^2 = 0.7 \cdot in^2 \qquad A_{stt} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 = 0.5 \cdot in^2$$

Design depth of Bar $\quad$ $d_{tb} := t_{deck} - \left(c_b + \frac{\phi_{tb}}{2}\right) = 6.6 \cdot in$ $\quad$ $d_{tt} := t_{deck} - \left(c_t + \frac{\phi_{tt}}{2}\right) = 5.2 \cdot in$

Girder Spacing $\quad$ $spacing_{int\_max} := 2ft + 11in$

$spacing_{ext} = 3\,ft$

Equivalent Strip, +M $\quad$ $w_{posM} := \left(26 + 6.6 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $\quad$ $w_{posM} = 45.3 \cdot in$

Equivalent Strip, -M $\quad$ $w_{negM} := \left(48 + 3.0 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $\quad$ $w_{negM} = 56.8 \cdot in$

Once the strip widths are determined, the dead loads can be calculated.

19. PERMANENT LOADS

This section calculates the dead loads on the slab. These are used later for analysis to determine the design moments.

Weight of deck, +M	$w_{deck\_pos} := w_c \cdot t_d \cdot w_{posM}$	$w_{deck\_pos} = 494.9 \cdot plf$
Weight of deck, -M	$w_{deck\_neg} := w_c \cdot t_d \cdot w_{negM}$	$w_{deck\_neg} = 620.7 \cdot plf$
Unit weight of barrier	$w_b := 433.5plf$	
Barrier point load, +M	$P_{b\_pos} := w_b \cdot w_{posM}$	$P_{b\_pos} = 1.63 \cdot kip$
Barrier point load, -M	$P_{b\_neg} := w_b \cdot w_{negM}$	$P_{b\_neg} = 2.05 \cdot kip$

20. LIVE LOADS

This section calculates the live loads on the slab. These loads are analyzed in a separate program with the permanent loads to determine the design moments.

Truck wheel load	$P_{wheel} := 16kip$		
Impact Factor	$IM := 1.33$		
Multiple presence factors	$MP_1 := 1.2$	$MP_2 := 1.0$	$MP_3 := 0.85$
Wheel Loads	$P_1 := IM \cdot MP_1 \cdot P_{wheel}$	$P_2 := IM \cdot MP_2 \cdot P_{wheel}$	$P_3 := IM \cdot MP_3 \cdot P_{wheel}$
	$P_1 = 25.54 \cdot kip$	$P_2 = 21.3 \cdot kip$	$P_3 = 18.09 \cdot kip$

21. LOAD RESULTS

The separate MathCAD design aides (available in Appendix of the final report) was used to analyze the deck as an 11-span continuous beam without cantilevered overhangs on either end, with supports stationed at girder locations. The dead and live loads were applied separately. The results are represented here as input values, highlighted.

Design Moments

$M_{pos\_deck} := 0.4 \cdot kip \cdot ft$ $\quad M_{pos\_LL} := 15.3 \cdot kip \cdot ft$ $\quad M_{pos} := (1.25 \cdot M_{pos\_deck} + 1.75 \cdot M_{pos\_LL})$

$M_{pos} = 27.3 \cdot kip \cdot ft$ $\quad M_{pos\_dist} := \dfrac{M_{pos}}{w_{posM}}$ $\quad M_{pos\_dist} = 7.23 \cdot \dfrac{kip \cdot ft}{ft}$

$M_{neg\_deck} := -0.6 \cdot kip \cdot ft$ $\quad M_{neg\_LL} := -7.8 \cdot kip \cdot ft$ $\quad M_{neg} := (1.25 \cdot M_{neg\_deck} + 1.75 \cdot M_{neg\_LL})$

$M_{neg} = -14.4 \cdot kip \cdot ft$ $\quad M_{neg\_dist} := \dfrac{M_{neg}}{w_{negM}}$ $\quad M_{neg\_dist} = -3.04 \cdot \dfrac{kip \cdot ft}{ft}$

22. FLEXURAL STRENGTH CAPACITY CHECK:

Consider a 1'-0" strip: $\phi_b := 0.9$ $\quad b := 12in$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4ksi \\ 0.85 - 0.05 \cdot \left(\dfrac{f_c}{ksi} - 4\right) & \text{otherwise} \end{cases} \qquad \beta_1 = 0.8$$

Bottom:

$c_{tb} := \dfrac{A_{stb} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1 \cdot in$

$a_{tb} := \beta_1 \cdot c_{tb} = 0.8 \cdot in$

$M_{ntb} := \dfrac{A_{stb} \cdot F_s}{b} \cdot \left(d_{tb} - \dfrac{a_{tb}}{2}\right) = 20.7 \cdot \dfrac{kip \cdot ft}{ft}$

$M_{rtb} := \phi_b \cdot M_{ntb} = 18.6 \cdot \dfrac{kip \cdot ft}{ft}$

$M_{rtb} \ge |M_{pos\_dist}| = 1$

Top:

$c_{tt} := \dfrac{A_{stt} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.7 \cdot in$

$a_{tt} := \beta_1 \cdot c_{tt} = 0.5 \cdot in$

$M_{ntt} := \dfrac{A_{stt} \cdot F_s}{ft} \cdot \left(d_{tt} - \dfrac{a_{tt}}{2}\right) = 11.3 \cdot \dfrac{kip \cdot ft}{ft}$

$M_{rtt} := \phi_b \cdot M_{ntt} = 10.2 \cdot \dfrac{kip \cdot ft}{ft}$

$M_{rtt} \ge |M_{neg\_dist}| = 1$

23. LONGITUDINAL DECK REINFORCEMENT DESIGN:

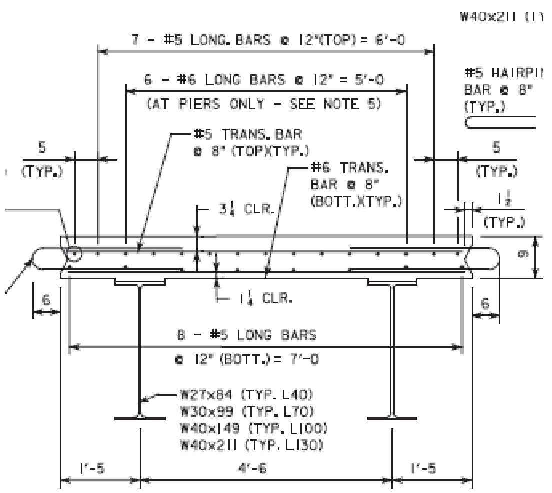
Longitudinal reinforcement $\quad \phi_{lb} := \dfrac{5}{8} in$ $\quad s_{lb} := 12in$ $\quad \phi_{lt} := \dfrac{5}{8} in$ $\quad s_{lt} := 12in$

$A_{slb} := \dfrac{12in}{s_{lb}} \cdot \pi \cdot \left(\dfrac{\phi_{lb}}{2}\right)^2 = 0.3 \cdot in^2$ $\quad A_{slt} := \dfrac{12in}{s_{lt}} \cdot \pi \left(\dfrac{\phi_{lt}}{2}\right)^2 = 0.3 \cdot in^2$

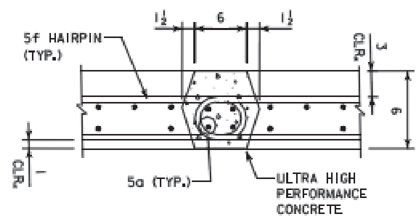
Distribution Reinforcement (AASHTO 9.7.3.2)

$$A_{\%dist} := \frac{\min\left(\dfrac{220}{\sqrt{\dfrac{spacing_{int\_max}}{ft}}}, 67\right)}{100} = 67 \cdot \%$$

$A_{dist} := A_{\%dist} \cdot (A_{stb}) = 0.4 \cdot in^2$ $\quad A_{slb} + A_{slt} \ge A_{dist} = 1$



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

24. DESIGN CHECKS

This section will conduct design checks on the reinforcing according to various sections in AASHTO LRFD.

CHECK MINIMUM REINFORCEMENT (AASHTO LRFD 5.7.3.3.2):

Modulus of Rupture $f_r := 0.37\cdot\sqrt{f_c\cdot ksi} = 0.8\cdot ksi$

$E_c = 4286.8\cdot ksi$

$E_s = 29000\cdot ksi$

Section Modulus $S_{nc} := \dfrac{b\cdot t_{deck}^{\,2}}{6} = 128\cdot in^3$

$A_{deck} := t_{deck}\cdot b = 96\cdot in^2$

$$y_{bar\_tb} := \frac{A_{deck}\cdot\frac{t_{deck}}{2} + (n-1)\cdot A_{stb}\cdot d_{tb}}{A_{deck} + (n-1)\cdot A_{stb}} = 4.1\cdot in$$

$$y_{bar\_tt} := \frac{A_{deck}\cdot\frac{t_{deck}}{2} + (n-1)\cdot A_{stt}\cdot d_{tt}}{A_{deck} + (n-1)\cdot A_{stt}} = 4\cdot in$$

$$I_{tb} := \frac{b\cdot t_{deck}^{\ 3}}{12} + A_{deck}\cdot\left(\frac{t_{deck}}{2} - y_{bar\_tb}\right)^2 + (n-1)\cdot A_{stb}\cdot\left(d_{tb} - y_{bar\_tb}\right)^2 = 538.3\cdot in^4$$

$$I_{tt} := \frac{b\cdot t_{deck}^{\ 3}}{12} + A_{deck}\cdot\left(\frac{t_{deck}}{2} - y_{bar\_tt}\right)^2 + (n-1)\cdot A_{stt}\cdot\left(d_{tt} - y_{bar\_tt}\right)^2 = 515.8\cdot in^4$$

$$S_{c\_tb} := \frac{I_{tb}}{t_{deck} - y_{bar\_tb}} = 138.2\cdot in^3 \qquad S_{c\_tt} := \frac{I_{tt}}{t_{deck} - y_{bar\_tt}} = 130\cdot in^3$$

Unfactored Dead Load

$$M_{dnc\_pos\_t} := 1.25\frac{kip\cdot ft}{ft} \qquad M_{dnc\_neg\_t} := -0.542\frac{kip\cdot ft}{ft}$$

Cracking Moment — S 5.7.3.3.2

$$M_{cr\_tb} := \max\left[\frac{S_{c\_tb}\cdot f_r}{ft} - \left|M_{dnc\_pos\_t}\right|\cdot\left(\frac{S_{c\_tb}}{S_{nc}} - 1\right), \frac{S_{c\_tb}\cdot f_r}{ft}\right] = 9.5\cdot\frac{kip\cdot ft}{ft}$$

$$M_{cr\_tt} := \max\left[\frac{S_{c\_tt}\cdot f_r}{ft} - \left|M_{dnc\_neg\_t}\right|\cdot\left(\frac{S_{c\_tt}}{S_{nc}} - 1\right), \frac{S_{c\_tt}\cdot f_r}{ft}\right] = 9\cdot\frac{kip\cdot ft}{ft}$$

Minimum Factored Flexural Resistance

$$M_{r\_min\_tb} := \min\left(1.2\cdot M_{cr\_tb}, 1.33\cdot\left|M_{pos\_dist}\right|\right) = 9.6\cdot\frac{kip\cdot ft}{ft} \qquad M_{rtb} \geq M_{r\_min\_tb} = 1$$

$$M_{r\_min\_tt} := \min\left(1.2\cdot M_{cr\_tt}, 1.33\cdot\left|M_{neg\_dist}\right|\right) = 4\cdot\frac{kip\cdot ft}{ft} \qquad M_{rtt} \geq M_{r\_min\_tt} = 1$$

CHECK CRACK CONTROL (AASHTO LRFD 5.7.3.4):

$$\gamma_{eb} := 1.0 \qquad \gamma_{et} := 0.75$$

$$M_{SL\_pos} := 29.64 kip\cdot ft \qquad M_{SL\_neg} := 29.64 kip\cdot ft$$

$$M_{SL\_pos\_dist} := \frac{M_{SL\_pos}}{w_{posM}} = 7.9\cdot\frac{kip\cdot ft}{ft} \qquad M_{SL\_neg\_dist} := \frac{M_{SL\_neg}}{w_{negM}} = 6.3\cdot\frac{kip\cdot ft}{ft}$$

$$f_{ssb} := \frac{M_{SL\_pos\_dist}\cdot b\cdot n}{\frac{I_{tb}}{d_{tb} - y_{bar\_tb}}} = 3.1\cdot ksi \qquad f_{sst} := \frac{M_{SL\_neg\_dist}\cdot b\cdot n}{\frac{I_{tt}}{d_{tt} - y_{bar\_tt}}} = 1.2\cdot ksi$$

$$d_{cb} := c_b + \frac{\phi_{tb}}{2} = 1.4\cdot in \qquad d_{ct} := c_t + \frac{\phi_{tt}}{2} = 2.8\cdot in$$

$$\beta_{sb} := 1 + \frac{d_{cb}}{0.7\cdot\left(t_{deck} - d_{cb}\right)} = 1.3 \qquad \beta_{st} := 1 + \frac{d_{ct}}{0.7\cdot\left(t_{deck} - d_{ct}\right)} = 1.8$$

$$s_b := \frac{700\cdot\gamma_{eb}\cdot kip}{\beta_{sb}\cdot f_{ssb}\cdot in} - 2\cdot d_{cb} = 171.9\cdot in \qquad s_t := \frac{700\cdot\gamma_{et}\cdot kip}{\beta_{st}\cdot f_{sst}\cdot in} - 2\cdot d_{ct} = 245.5\cdot in$$

$$s_{tb} \leq s_b = 1 \qquad s_{tt} \leq s_t = 1$$

SHRINKAGE AND TEMPERATURE REINFORCING (AASHTO LRFD 5.10.8):

$$A_{st} := \begin{cases} \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot (b + t_{deck}) \cdot F_s} \cdot \dfrac{kip}{in} & \text{if } 0.11in^2 \le \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot (b + t_{deck}) \cdot F_s} \cdot \dfrac{kip}{in} \le 0.60in^2 \\ 0.11in^2 & \text{if } \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot (b + t_{deck}) \cdot F_s} \cdot \dfrac{kip}{in} < 0.11in^2 \\ 0.60in^2 & \text{if } \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot (b + t_{deck}) \cdot F_s} \cdot \dfrac{kip}{in} > 0.60in^2 \end{cases} = 0.1 \cdot in^2$$

$A_{stb} \ge A_{st} = 1$ $\quad$ $A_{stt} \ge A_{st} = 1$

$A_{slb} \ge A_{st} = 1$ $\quad$ $A_{slt} \ge A_{st} = 1$

SHEAR RESISTANCE (AASHTO LRFD 5.8.3.3):

$\phi := 0.9$ $\quad$ $\beta := 2$ $\quad$ $\theta := 45deg$ $\quad$ $b = 1\,ft$

$$d_{v\_tb} := \max\left(0.72 \cdot t_{deck}, d_{tb} - \frac{a_{tb}}{2}, 0.9 \cdot d_{tb}\right) = 6.2 \cdot in$$

$$d_{v\_tt} := \max\left(0.72 \cdot t_{deck}, d_{tt} - \frac{a_{tt}}{2}, 0.9 \cdot d_{tt}\right) = 5.8 \cdot in$$

$$d_v := \min(d_{v\_tb}, d_{v\_tt}) = 5.8 \cdot in$$

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_v = 9.8 \cdot kip$$

$V_s := 0kip$ Shear capacity of reinforcing steel

$V_{ps} := 0kip$ Shear capacity of prestressing steel

$$V_{ns} := \min(V_c + V_s + V_{ps}, 0.25 \cdot f_c \cdot b \cdot d_v + V_{ps}) = 9.8 \cdot kip$$

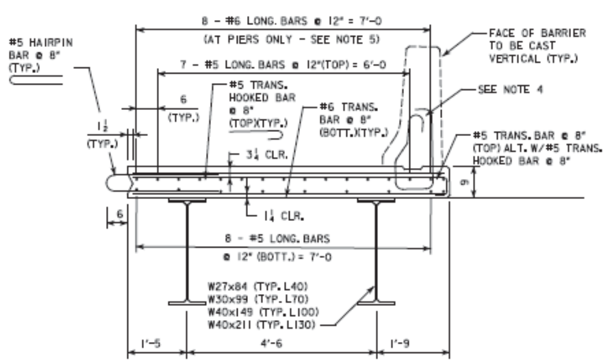
$V_r := \phi \cdot V_{ns} = 8.8 \cdot kip$ Total factored resistance

$V_{us} := 8.38kip$ Total factored load $\quad$ $V_r \ge V_{us} = 1$

DEVELOPMENT AND SPLICE LENGTHS (AASHTO LRFD 5.11):

Development and splice length design follows standard calculations in AASHTO LRFD 5.11, or as dictated by the State DOT Design Manual.

25. DECK OVERHANG DESIGN (AASHTO LRFD A.13.4):



EXTERIOR MODULE
REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)

Deck Properties:

Deck Overhang Length $L_o := 1ft + 9in$

Parapet Properties:

Note: Parapet properties are per unit length. Compression reinforcement is ignored.

Cross Sectional Area	$A_p := 2.84ft^2$	Height of Parapet	$H_{par} := 2ft + 10in$
Parapet Weight	$W_{par} := w_c \cdot A_p = 426 \cdot plf$		
Width at base	$w_{base} := 1ft + 5in$	Average width of wall	$w_{wall} := \frac{13in + 9.5in}{2} = 11.3 \cdot in$
Height of top portion of parapet	$h_1 := 2ft$	Width at top of parapet	$width_1 := 9.5 \cdot in = 9.5 \cdot in$
Height of middle portion of parapet	$h_2 := 7in$	Width at middle transition of parapet	$width_2 := 12 \cdot in = 12 \cdot in$
Height of lower portion of parapet	$h_3 := 3in$	Width at base of parapet	$width_3 := 1ft + 5 \cdot in = 17 \cdot in$

$b_1 := width_1$ $b_2 := width_2 - width_1$ $b_3 := width_3 - width_2$

Parapet Center of Gravity

$$CG_p := \frac{(h_1 + h_2 + h_3) \cdot \frac{b_1^2}{2} + \frac{1}{2} \cdot h_1 \cdot b_2 \cdot \left(b_1 + \frac{b_2}{3}\right) \ldots + (h_2 + h_3) \cdot (b_2 + b_3) \cdot \left(b_1 + \frac{b_2 + b_3}{2}\right) - \frac{1}{2} \cdot h_2 \cdot b_3 \cdot \left(b_1 + b_2 + \frac{2b_3}{3}\right)}{(h_1 + h_2 + h_3) \cdot b_1 + \frac{1}{2} \cdot h_1 \cdot b_2 + (h_2 + h_3) \cdot (b_2 + b_3) - \frac{1}{2} \cdot h_2 \cdot b_3} = 6.3 \cdot in$$

Parapet Reinforcement	Vertically Aligned Bars in Wall	Horizontal Bars
Rebar spacing:	$s_{pa} := 12in$	$n_{pl} := 5$
Rebar Diameter:	$\phi_{pa} := \frac{5}{8}in$	$\phi_{pl} := \frac{5}{8}in$
Rebar Area:	$A_{st\_p} := \pi \cdot \left(\frac{\phi_{pa}}{2}\right)^2 \cdot \frac{b}{s_{pa}} = 0.3 \cdot in^2$	$A_{sl\_p} := \pi \cdot \left(\frac{\phi_{pl}}{2}\right)^2 = 0.3 \cdot in^2$
Cover:	$c_{st} := 3in$	$c_{sl} := 2in + \phi_{pa} = 2.6 \cdot in$
Effective Depth:	$d_{st} := w_{base} - c_{st} - \frac{\phi_{pa}}{2} = 13.7 \cdot in$	$d_{sl} := w_{wall} - c_{sl} - \frac{\phi_{pl}}{2} = 8.3 \cdot in$
Parapet Moment Resistance About Horizontal Axis:	$\phi_{ext} := 1.0$	
Depth of Equivalent Stress Block:	$a_h := \frac{A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot b} = 0.4 \cdot in$	S 5.7.3.1.2-4 S 5.7.3.2.3

Moment Capacity of Upper Segment of Barrier (about longitudinal axis):

Average width of section $w_1 := \frac{width_1 + width_2}{2} = 10.7 \cdot in$

Cover $c_{st1} := 2in$

Depth $d_{h1} := w_1 - c_{st1} - \frac{\phi_{pa}}{2} = 8.4 \cdot in$

Factored Moment Resistance $\phi M_{nh1} := \frac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h1} - \frac{a_h}{2}\right)}{b} = 12.7 \cdot \frac{kip \cdot ft}{ft}$

Moment Capacity of Middle Segment of Barrier (about longitudinal axis):

Average width of section $w_2 := \dfrac{width_2 + width_3}{2} = 14.5 \cdot in$

Cover $c_{st2} := 3in$

Depth $d_{h2} := w_2 - c_{st2} - \dfrac{\phi_{pa}}{2} = 11.2 \cdot in$

Factored Moment Resistance $\phi M_{nh2} := \dfrac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h2} - \dfrac{a_h}{2}\right)}{b} = 16.9 \cdot \dfrac{kip \cdot ft}{ft}$

Parapet Base Moment Resistance (about longitudinal axis):

development in tension $c_{st3} := 3in$ $\qquad cover_{base\_vert} := c_{st3} + \dfrac{\phi_{pa}}{2} = 3.3 \cdot in$

$$m_{inc\_ta} := \begin{cases} 1.5 & \text{if } c_{st3} < 3 \cdot \phi_{pa} \vee s_{pa} - \phi_{pa} < 6 \cdot \phi_{pa} \\ 1.2 & \text{otherwise} \end{cases} = 1.2$$

$$m_{dec\_ta} := \begin{cases} 0.8 & \text{if } s_{pa} \ge 6in \\ 1.0 & \text{otherwise} \end{cases} = 0.8$$

$$l_{db\_ta} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{st\_p} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{pa} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{pa} \le \dfrac{11}{8} in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{pa} = \dfrac{14}{8} in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{pa} = \dfrac{18}{8} in \end{cases}$$

$l_{dt\_ta} := l_{db\_ta} \cdot m_{inc\_ta} \cdot m_{dec\_ta} = 14.4 \cdot in$

hooked bar developed in tension $l_{hb\_ta} := \dfrac{38 \cdot \phi_{pa}}{\sqrt{\dfrac{f_c}{ksi}}} = 10.6 \cdot in$ $\qquad m_{inc} = 1.2$

$l_{dh\_ta} := \max\left(6in, 8 \cdot \phi_{pa}, m_{inc} \cdot l_{hb\_ta}\right) = 12.7 \cdot in$

lap splice in tension $l_{lst\_ta} := \max\left(12in, 1.3 \cdot l_{dt\_ta}\right) = 18.7 \cdot in$

$benefit := l_{dt\_ta} - l_{dh\_ta} = 1.7 \cdot in$

$l_{dev\_a} := \left(7 + \dfrac{13}{16}\right) in$

$F_{dev} := \dfrac{benefit + l_{dev\_a}}{l_{dt\_ta}} = 0.7$

$F_d := 0.75$

Distance from NA to Compressive Face $c_{t\_b} := \dfrac{F_d \cdot A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.3 \cdot in$ $\qquad$ S 5.7.3.1.2-4

Depth of Equivalent Stress Block	$a_t := \beta_1 \cdot c_{t\_b} = 0.3 \cdot in$	S 5.7.3.2.3
Nominal Moment Resistance	$M_{nt} := F_d \cdot A_{st\_p} \cdot F_s \cdot \left(d_{st} - \frac{a_t}{2}\right) = 15.6 \cdot kip \cdot ft$	S 5.7.3.2.2-1
Factored Moment Resistance	$M_{cb} := \phi_{ext} \cdot \frac{M_{nt}}{ft} = 15.6 \cdot \frac{kip \cdot ft}{ft}$	S 5.7.3.2

Average Moment Capacity of Barrier (about longitudinal axis):

Factored Moment Resistance about Horizontal Axis

$$M_c := \frac{\phi M_{nh1} \cdot h_1 + \phi M_{nh2} \cdot h_2 + M_{cb} \cdot h_3}{h_1 + h_2 + h_3} = 13.8 \cdot \frac{kip \cdot ft}{ft}$$

Parapet Moment Resistance (about vertical axis):

Height of Transverse Reinforcement in Parapet; Width of Parapet at Transverse Reinforcement

$y_1 := 5in$ $\qquad x_1 := width_3 - \frac{(y_1 - h_3) \cdot b_3}{h_2} = 15.6 \cdot in$

$y_2 := 11.5in$ $\qquad x_2 := b_1 + b_2 - \frac{(y_2 - h_3 - h_2) \cdot b_2}{h_1} = 11.8 \cdot in$

$y_3 := 18in$ $\qquad x_3 := b_1 + b_2 - \frac{(y_3 - h_3 - h_2) \cdot b_2}{h_1} = 11.2 \cdot in$

$y_4 := 24.5in$ $\qquad x_4 := b_1 + b_2 - \frac{(y_4 - h_3 - h_2) \cdot b_2}{h_1} = 10.5 \cdot in$

$y_5 := 31in$ $\qquad x_5 := b_1 + b_2 - \frac{(y_5 - h_3 - h_2) \cdot b_2}{h_1} = 9.8 \cdot in$

Depth of Equivalent Stress Block $\qquad a := \frac{n_{pl} \cdot A_{sl\_p} \cdot F_s}{0.85 \cdot f_c \cdot H_{par}} = 0.6 \cdot in$

Concrete Cover in Parapet

$cover_r := 2in$ $\qquad cover_{rear} := cover_r + \phi_{pa} + \frac{\phi_{pl}}{2} = 2.9 \cdot in$

$cover_{base} := c_{st3} + \phi_{pa} + \frac{\phi_{pl}}{2} = 3.9 \cdot in$

$cover_f := 2in$ $\qquad cover_{front} := 2in + \phi_{pa} + \frac{\phi_{pl}}{2}$

$cover_t := \frac{x_5}{2} = 4.9 \cdot in$ $\qquad cover_{top} := cover_t = 4.9 \cdot in$

Design depth

$d_{1i} := x_1 - cover_{base} = 11.6 \cdot in$ $\qquad d_{1o} := x_1 - cover_{rear} = 12.6 \cdot in$

$d_{2i} := x_2 - cover_{front} = 8.9 \cdot in$ $\qquad d_{2o} := x_2 - cover_{rear} = 8.9 \cdot in$

$d_{3i} := x_3 - cover_{front} = 8.2 \cdot in$ $\qquad d_{3o} := x_3 - cover_{rear} = 8.2 \cdot in$

$d_{4i} := x_4 - cover_{front} = 7.6 \cdot in$ $\qquad d_{4o} := x_4 - cover_{rear} = 7.6 \cdot in$

$d_{5i} := x_5 - cover_{top} = 4.9 \cdot in$ $\qquad d_{5o} := x_5 - cover_{top} = 4.9 \cdot in$

Nominal Moment Resistance - Tension on Inside Face

$$\phi Mn_{1i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1i} - \frac{a}{2}\right) = 208.3 \cdot kip \cdot in$$

$$\phi Mn_{2i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2i} - \frac{a}{2}\right) = 158.1 \cdot kip \cdot in$$

$$\phi Mn_{3i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3i} - \frac{a}{2}\right) = 145.6 \cdot kip \cdot in$$

$$\phi Mn_{4i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4i} - \frac{a}{2}\right) = 133.2 \cdot kip \cdot in$$

$$\phi Mn_{5i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5i} - \frac{a}{2}\right) = 84.5 \cdot kip \cdot in$$

$$M_{wi} := \phi Mn_{1i} + \phi Mn_{2i} + \phi Mn_{3i} + \phi Mn_{4i} + \phi Mn_{5i} = 60.8 \cdot kip \cdot ft$$

Nominal Moment Resistance - Tension on Outside Face

$$\phi Mn_{1o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1o} - \frac{a}{2}\right) = 18.9 \cdot kip \cdot ft$$

$$\phi Mn_{2o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2o} - \frac{a}{2}\right) = 13.2 \cdot kip \cdot ft$$

$$\phi Mn_{3o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3o} - \frac{a}{2}\right) = 12.1 \cdot kip \cdot ft$$

$$\phi Mn_{4o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4o} - \frac{a}{2}\right) = 11.1 \cdot kip \cdot ft$$

$$\phi Mn_{5o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5o} - \frac{a}{2}\right) = 7 \cdot kip \cdot ft$$

$$M_{wo} := \phi Mn_{1o} + \phi Mn_{2o} + \phi Mn_{3o} + \phi Mn_{4o} + \phi Mn_{5o} = 62.3 \cdot kip \cdot ft$$

Vertical Nominal Moment Resistance of Parapet

$$M_w := \frac{2 \cdot M_{wi} + M_{wo}}{3} = 61.3 \cdot kip \cdot ft$$

Parapet Design Factors:

Crash Level $CL := \text{"TL-4"}$

Transverse Design Force

$$F_t := \begin{array}{|l} 13.5kip \text{ if } CL = \text{"TL-1"} \\ 27.0kip \text{ if } CL = \text{"TL-2"} \\ 54.0kip \text{ if } CL = \text{"TL-3"} \\ 54.0kip \text{ if } CL = \text{"TL-4"} \\ 124.0kip \text{ if } CL = \text{"TL-5"} \\ 175.0kip \text{ otherwise} \end{array} = 54 \cdot kip \qquad L_t := \begin{array}{|l} 4.0ft \text{ if } CL = \text{"TL-1"} \\ 4.0ft \text{ if } CL = \text{"TL-2"} \\ 4.0ft \text{ if } CL = \text{"TL-3"} \\ 3.5ft \text{ if } CL = \text{"TL-4"} \\ 8.0ft \text{ if } CL = \text{"TL-5"} \\ 8.0ft \text{ otherwise} \end{array} = 3.5 \cdot ft$$

Longitudinal Design Force

$$F_l := \begin{array}{|l} 4.5kip \text{ if } CL = \text{"TL-1"} \\ 9.0kip \text{ if } CL = \text{"TL-2"} \\ 18.0kip \text{ if } CL = \text{"TL-3"} \\ 18.0kip \text{ if } CL = \text{"TL-4"} \\ 41.0kip \text{ if } CL = \text{"TL-5"} \\ 58.0kip \text{ otherwise} \end{array} = 18 \cdot kip \qquad L_l := \begin{array}{|l} 4.0ft \text{ if } CL = \text{"TL-1"} \\ 4.0ft \text{ if } CL = \text{"TL-2"} \\ 4.0ft \text{ if } CL = \text{"TL-3"} \\ 3.5ft \text{ if } CL = \text{"TL-4"} \\ 8.0ft \text{ if } CL = \text{"TL-5"} \\ 8.0ft \text{ otherwise} \end{array} = 3.5 \cdot ft$$

Vertical Design Force (Down)

$$F_v := \begin{array}{|l} 4.5kip \text{ if } CL = \text{"TL-1"} \\ 4.5kip \text{ if } CL = \text{"TL-2"} \\ 4.5kip \text{ if } CL = \text{"TL-3"} \\ 18.0kip \text{ if } CL = \text{"TL-4"} \\ 80.0kip \text{ if } CL = \text{"TL-5"} \\ 80.0kip \text{ otherwise} \end{array} = 18 \cdot kip \qquad L_v := \begin{array}{|l} 18.0ft \text{ if } CL = \text{"TL-1"} \\ 18.0ft \text{ if } CL = \text{"TL-2"} \\ 18.0ft \text{ if } CL = \text{"TL-3"} \\ 18.0ft \text{ if } CL = \text{"TL-4"} \\ 40.0ft \text{ if } CL = \text{"TL-5"} \\ 40.0ft \text{ otherwise} \end{array} = 18 \cdot ft$$

Critical Length of Yield Line Failure Pattern:

$$M_b := 0kip \cdot ft$$

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \cdot H_{par} \cdot (M_b + M_w)}{M_c}} = 11.9 \cdot ft$$ S A13.3.1-2

$$R_w := \frac{2}{2 \cdot L_c - L_t} \cdot \left(8 \cdot M_b + 8 \cdot M_w + \frac{M_c \cdot L_c^2}{H_{par}}\right) = 116.2 \cdot kip$$ S A13.3.1-1

$$T := \frac{R_w \cdot b}{L_c + 2 \cdot H_{par}} = 6.6 \cdot kip$$ S A13.4.2-1

The parapet design must consider three design cases. Design Case 1 is for longitudinal and transverse collision loads under Extreme Event Load Combination II. Design Case 2 represents vertical collision loads under Extreme Event Load Combination II; however, this case does not govern for decks with concrete parapets or barriers. Design Case 3 is for dead and live load under Strength Load Combination I; however, the parapet will not carry wheel loads and therefore this case does not govern. Design Case 1 is the only case that requires a check.

Design Case 1: Longitudinal and Transverse Collision Loads, Extreme Event Load Combination II

DC - 1A: Inside face of parapet

$\phi_{ext} = 1$ $\gamma_{DC} := 1.0$ $\gamma_{DW} := 1.0$ $\gamma_{LL} := 0.5$ S A13.4.1
S Table 3.4.1-1

$l_{lip} := 2in$ $w_{base} = 17 \cdot in$

$A_{deck\_1A} := t_{deck} \cdot (l_{lip} + w_{base}) = 152 \cdot in^2$ $A_p = 2.8 \cdot ft^2$

$W_{deck\_1A} := w_c \cdot A_{deck\_1A} = 0.2 \cdot klf$ $W_{par} = 0.4 \cdot klf$

$$M_{DCdeck\_1A} := \gamma_{DC} \cdot W_{deck\_1A} \cdot \frac{l_{lip} + w_{base}}{2} = 0.1 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1A} := \gamma_{DC} \cdot W_{par} \cdot (l_{lip} + CG_p) = 0.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1A} := M_{cb} + M_{DCdeck\_1A} + M_{DCpar\_1A} = 16 \cdot \frac{kip \cdot ft}{ft}$$

$\phi_{tt\_add} := \frac{5}{8} in$ $s_{tt\_add} := 8in$

$$A_{stt\_p} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 + \frac{12in}{s_{tt\_add}} \cdot \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.9 \cdot in^2$$

$$d_{tt\_add} := t_{deck} - \left(c_t + \frac{\phi_{tt\_add}}{2}\right) = 5.2 \cdot in$$

$c_{tt\_p} := \frac{A_{stt\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.4 \cdot in$ $a_{tt\_p} := \beta_1 \cdot c_{tt\_p} = 1.1 \cdot in$

$$M_{ntt\_p} := \frac{A_{stt\_p} \cdot F_s}{ft} \cdot \left(d_{tt\_add} - \frac{a_{tt\_p}}{2}\right) = 21.4 \cdot \frac{kip \cdot ft}{ft}$$

$M_{rtt\_p} := \phi_b \cdot M_{ntt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft}$ $M_{rtt\_p} \geq M_{total\_1A} = 1$

$A_{sT} := A_{stt} + A_{stb} = 1.1 \cdot in^2$

$\phi P_n := \phi_{ext} \cdot A_{sT} \cdot F_s = 67.4 \cdot kip$ $\phi P_n \geq T = 1$

$M_{u\_1A} := M_{rtt\_p} \cdot \left(1 - \frac{T}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft}$ $M_{u\_1A} \geq M_{total\_1A} = 1$

DC - 1B: Design Section in Overhang

Notes: Distribution length is assumed to increase based on a 30 degree angle from the face of parapet.
Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
Future wearing surface is neglected as contribution is negligible.

$$A_{deck\_1B} := t_{deck} \cdot L_o = 168 \cdot in^2 \qquad A_p = 2.8 \cdot ft^2$$

$$W_{deck\_1B} := w_c \cdot A_{deck\_1B} = 0.2 \cdot klf \qquad W_{par} = 0.4 \cdot klf$$

$$M_{DCdeck\_1B} := \gamma_{DC} \cdot W_{deck\_1B} \cdot \frac{L_o}{2} = 0.2 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1B} := \gamma_{DC} \cdot W_{par} \cdot \left(L_o - l_{lip} - CG_p\right) = 0.5 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{spread\_B} := L_o - l_{lip} - width_3 = 2 \cdot in \qquad spread := 30deg$$

$$w_{spread\_B} := L_{spread\_B} \cdot \tan(spread) = 1.2 \cdot in$$

$$M_{cb\_1B} := \frac{M_{cb} \cdot L_c}{L_c + 2 \cdot w_{spread\_B}} = 15.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1B} := M_{cb\_1B} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 15.9 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1B} = 1$$

$$\phi P_n = 67.4 \cdot kip$$

$$P_u := \frac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_B}} = 6.5 \cdot kip \qquad \phi P_n \geq P_u = 1$$

$$M_{u\_1B} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{u\_1B} \geq M_{total\_1B} = 1$$

DC - 1C: Design Section in First Span

Assumptions: Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
Future wearing surface is neglected as contribution is negligible.

$$M_{par\_G1} := M_{DCpar\_1B} = 0.5 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{par\_G2} := -0.137 \frac{kip \cdot ft}{ft}$$ (From model output)

$$M_1 := M_{cb} = 15.6 \cdot \frac{kip \cdot ft}{ft}$$

$$M_2 := M_1 \cdot \frac{M_{par\_G2}}{M_{par\_G1}} = -4.7 \cdot \frac{kip \cdot ft}{ft}$$

$$b_f := 10.5in$$

$$M_{c\_M2M1} := M_1 + \frac{\frac{1}{4} \cdot b_f \cdot \left(-M_1 + M_2\right)}{spacing_{int\_max}} = 14.1 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{spread\_C} := L_o - l_{lip} - w_{base} + \frac{b_f}{4} = 4.6 \cdot in$$

$$w_{spread\_C} := L_{spread\_C} \cdot \tan(spread) = 2.7 \cdot in$$

$$M_{cb\_1C} := \frac{M_{c\_M2M1} \cdot L_c}{L_c + 2 \cdot w_{spread\_C}} = 13.6 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1C} := M_{cb\_1C} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 14.2 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt\_p} \ge M_{total\_1C} = 1$$

$$\phi P_n = 67.4 \cdot kip$$

$$P_{uC} := \frac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_C}} = 6.4 \cdot kip \qquad \phi P_n \ge P_{uC} = 1$$

$$M_{u\_1C} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{u\_1B} \ge M_{total\_1B} = 1$$

Compute Overhang Reinforcement Cut-off Length Requirement

Maximum crash load moment at theoretical cut-ff point:

$$M_{c\_max} := M_{rtt} = 10.2 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{Mc\_max} := \frac{M_2 - M_{rtt}}{M_2 - M_1} \cdot spacing_{int\_max} = 2.1 \cdot ft$$

$$L_{spread\_D} := L_o - l_{lip} - w_{base} + L_{Mc\_max} = 27.7 \cdot in$$

$$w_{spread\_D} := L_{spread\_D} \cdot \tan(spread) = 16 \cdot in$$

$$M_{cb\_max} := \frac{M_{c\_max} \cdot L_c}{L_c + 2 \cdot w_{spread\_D}} = 8.3 \cdot \frac{kip \cdot ft}{ft}$$

$$extension := \max\left(d_{tt\_add}, 12 \cdot \phi_{tt\_add}, 0.0625 \cdot spacing_{int\_max}\right) = 7.5 \cdot in$$

$$cutt\_off := L_{Mc\_max} + extension = 33.2 \cdot in$$

$$A_{tt\_add} := \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.3 \cdot in^2$$

$$m_{thick\_tt\_add} := \begin{vmatrix} 1.4 & \text{if } t_{deck} - c_t \ge 12in \\ 1.0 & \text{otherwise} \end{vmatrix} = 1$$

$$m_{epoxy\_tt\_add} := \begin{vmatrix} 1.5 & \text{if } c_t < 3 \cdot \phi_{tt\_add} \vee \frac{s_{tt\_add}}{2} - \phi_{tt\_add} < 6 \cdot \phi_{tt\_add} \\ 1.2 & \text{otherwise} \end{vmatrix} = 1.5$$

$$m_{inc\_tt\_add} := \min\left(m_{thick\_tt\_add} \cdot m_{epoxy\_tt\_add}, 1.7\right) = 1.5$$

$$m_{dec\_tt\_add} := \begin{vmatrix} 0.8 & \text{if } \frac{s_{tt\_add}}{2} \ge 6in \\ 1.0 & \text{otherwise} \end{vmatrix} = 1$$

$$l_{db\_tt\_add} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{tt\_add} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{tt\_add} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{tt\_add} \le \dfrac{11}{8} in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{14}{8} in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{18}{8} in \end{cases} \qquad l_{db\_tt\_add} = 15 \cdot in$$

$l_{dt\_tt\_add} := l_{db\_tt\_add} \cdot m_{inc\_tt\_add} \cdot m_{dec\_tt\_add} = 22.5 \cdot in$

$Cuttoff_{point} := L_{Mc\_max} + l_{dt\_tt\_add} - spacing_{int\_max} = 13.2 \cdot in$ extension past second interior girder

Check for Cracking in Overhang under Service Limit State:

Does not govern - no live load on overhang.

25. COMPRESSION SPLICE:

See sheet S7 for drawing.

Ensure compression splice and connection can handle the compressive force in the force couple due to the negative moment over the pier.

Live load negative moment over pier: $M_{LLPier} := 541.8 \cdot kip \cdot ft$

Factored LL moment: $M_{UPier} := 1.75 \cdot M_{LLPier} = 948.1 \cdot kip \cdot ft$

The compression splice is comprised of a splice plate on the underside of the bottom flange, and built-up angles on either side of the web, connecting to the bottom flange as well.

Calculate Bottom Flange Stress:

Composite moment of inertia: $I_z = 11538.5 \cdot in^4$

Distance to center of bottom flange from composite section centroid: $y_{bf} := \dfrac{t_{bf}}{2} + D_w + t_{tf} + t_{slab} - y_c = 28.3 \cdot in$

Stress in bottom flange: $f_{bf} := M_{UPier} \cdot \dfrac{y_{bf}}{I_z} = 28 \cdot ksi$

Calculate Bottom Flange Force:

Design Stress: $F_{bf} := \max\left(\dfrac{f_{bf} + F_y}{2}, 0.75 \cdot F_y\right) = 39 \cdot ksi$

Effective Flange Area: $A_{ef} := b_{bf} \cdot t_{bf} = 6.3 \cdot in^2$

Force in Flange: $C_{nf} := F_{bf} \cdot A_{ef} = 247.3 \cdot kip$

Calculate Bottom Flange Stress, Ignoring Concrete:

Moment of inertia: $I_{zsteel} = 4463.1 \cdot in^4$

Distance to center of bottom flange: $y_{bfsteel} := \dfrac{t_{bf}}{2} + D_w + t_{tf} - y_{steel} = 16 \cdot in$

Stress in bottom flange: $f_{bfsteel} := M_{UPier} \cdot \frac{y_{bfsteel}}{I_{zsteel}} = 40.8 \cdot ksi$

Bottom Flange Force for design:

Design Stress: $F_{cf} := \max\left(\frac{f_{bfsteel} + F_y}{2}, 0.75 \cdot F_y\right) = 45.4 \cdot ksi$

Design Force: $C_n := \max(F_{bf}, F_{cf}) \cdot A_{ef} = 288 \cdot kip$

Compression Splice Plate Dimensions:

Bottom Splice Plate:	$b_{bsp} := b_{bf} = 10.4 \cdot in$	$t_{bsp} := 0.75in$	$A_{bsp} := b_{bsp} \cdot t_{bsp} = 7.8 \cdot in^2$
Built-Up Angle Splice Plate Horizontal Leg:	$b_{asph} := 4.25in$	$t_{asph} := 0.75in$	$A_{asph} := 2 \cdot b_{asph} \cdot t_{asph} = 6.4 \cdot in^2$
Built-Up Angle Splice Plate Vertical Leg:	$b_{aspv} := 7.75in$	$t_{aspv} := 0.75in$	$A_{aspv} := 2 \cdot b_{aspv} \cdot t_{aspv} = 11.6 \cdot in^2$

Total Area: $A_{csp} := A_{bsp} + A_{asph} + A_{aspv} = 25.8 \cdot in^2$

Average Stress: $f_{cs} := \frac{C_n}{A_{csp}} = 11.2 \cdot ksi$

Proportion Load into each plate based on area:

$$C_{bsp} := \frac{C_n \cdot A_{bsp}}{A_{csp}} = 87.1 \cdot kip \qquad C_{asph} := \frac{C_n \cdot A_{asph}}{A_{csp}} = 71.2 \cdot kip \qquad C_{aspv} := \frac{C_n \cdot A_{aspv}}{A_{csp}} = 129.8 \cdot kip$$

Check Plates Compression Capacity:

Bottom Splice Plate: $k_{cps} := 0.75$ for bolted connection

$l_{cps} := 9in$

$$r_{bsp} := \sqrt{\frac{\min\left(\frac{b_{bsp} \cdot t_{bsp}^3}{12}, \frac{t_{bsp} \cdot b_{bsp}^3}{12}\right)}{A_{bsp}}} = 0.2 \cdot in$$

$$P_{ebsp} := \frac{\pi^2 \cdot E_s \cdot A_{bsp}}{\left(\frac{k_{cps} \cdot l_{cps}}{r_{bsp}}\right)^2} = 2296.8 \cdot kip$$

$$Q_{bsp} := \begin{cases} 1.0 & \text{if } \frac{b_{bsp}}{t_{bsp}} \le 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76 \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right) \cdot \sqrt{\frac{F_y}{E_s}}\right] & \text{if } 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \le \frac{b_{bsp}}{t_{bsp}} \le 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right)^2} & \text{otherwise} \end{cases} = 0.902$$

$P_{obsp} := Q_{bsp} \cdot F_y \cdot A_{bsp} = 351.9 \cdot kip$

$$P_{nbsp} := \left| \begin{array}{l} \left[\left[0.658^{\left(\frac{P_{obsp}}{P_{ebsp}}\right)}\right] \cdot P_{obsp}\right] \text{ if } \frac{P_{ebsp}}{P_{obsp}} \geq 0.44 \\ (0.877 \cdot P_{ebsp}) \text{ otherwise} \end{array} \right. = 330.1 \cdot kip$$

$$P_{nbsp\_allow} := 0.9 \cdot P_{nbsp} = 297.1 \cdot kip \qquad Check := \left| \begin{array}{l} \text{"NG" if } C_{bsp} \geq P_{nbsp\_allow} \\ \text{"OK" if } P_{nbsp\_allow} \geq C_{bsp} \end{array} \right. = \text{"OK"}$$

Horizontal Angle Leg: $k_{cps} = 0.75$ for bolted connection

$l_{cps} = 9 \cdot in$

$$r_{asph} := \sqrt{\frac{\min\left(\frac{b_{asph} \cdot t_{asph}^3}{12}, \frac{t_{asph} \cdot b_{asph}^3}{12}\right)}{A_{asph}}} = 0.153 \cdot in$$

$$P_{easph} := \frac{\pi^2 \cdot E_s \cdot A_{asph}}{\left(\frac{k_{cps} \cdot l_{cps}}{r_{asph}}\right)^2} = 938.6 \cdot kip$$

$$Q_{asph} := \left| \begin{array}{l} 1.0 \text{ if } \frac{b_{asph}}{t_{asph}} \leq 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76 \cdot \left(\frac{b_{asph}}{t_{asph}}\right) \cdot \sqrt{\frac{F_y}{E_s}}\right] \text{ if } 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \leq \frac{b_{asph}}{t_{asph}} \leq 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b_{asph}}{t_{asph}}\right)^2} \text{ otherwise} \end{array} \right. = 1$$

$$P_{oasph} := Q_{asph} \cdot F_y \cdot A_{asph} = 318.7 \cdot kip$$

$$P_{nasph} := \left| \begin{array}{l} \left[\left[0.658^{\left(\frac{P_{oasph}}{P_{easph}}\right)}\right] \cdot P_{oasph}\right] \text{ if } \frac{P_{easph}}{P_{oasph}} \geq 0.44 \\ (0.877 \cdot P_{easph}) \text{ otherwise} \end{array} \right. = 276.5 \cdot kip$$

$$P_{nasph\_allow} := 0.9 \cdot P_{nasph} = 248.9 \cdot kip \qquad Check2 := \left| \begin{array}{l} \text{"NG" if } C_{asph} \geq P_{nasph\_allow} \\ \text{"OK" if } P_{nasph\_allow} \geq C_{asph} \end{array} \right. = \text{"OK'}$$

Vertical Angle Leg: $k_{cps} = 0.75$ for bolted connection

$l_{cps} = 9 \cdot in$

$$r_{aspv} := \sqrt{\frac{\min\left(\frac{b_{aspv} \cdot t_{aspv}^3}{12}, \frac{t_{aspv} \cdot b_{aspv}^3}{12}\right)}{A_{aspv}}} = 0.153 \cdot in$$

$$P_{easpv} := \frac{\pi^2 \cdot E_s \cdot A_{aspv}}{\left(\frac{k_{cps} \cdot l_{cps}}{r_{aspv}}\right)^2} = 1711.6 \cdot kip$$

$$Q_{aspv} := \begin{cases} 1.0 & \text{if } \dfrac{b_{aspv}}{t_{aspv}} \le 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\cdot\left(\dfrac{b_{aspv}}{t_{aspv}}\right)\cdot\sqrt{\dfrac{F_y}{E_s}}\right] & \text{if } 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{aspv}}{t_{aspv}} \le 0.91\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\cdot E_s}{F_y\cdot\left(\dfrac{b_{aspv}}{t_{aspv}}\right)^2} & \text{otherwise} \end{cases} = 1$$

$$P_{oaspv} := Q_{aspv}\cdot F_y\cdot A_{aspv} = 581.2\cdot kip$$

$$P_{naspv} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{oaspv}}{P_{easpv}}\right)}\right]\cdot P_{oaspv}\right] & \text{if } \dfrac{P_{easpv}}{P_{oaspv}} \ge 0.44 \\ \left(0.877\cdot P_{easpv}\right) & \text{otherwise} \end{cases} = 504.2\cdot kip$$

$$P_{naspv\_allow} := 0.9\cdot P_{naspv} = 453.8\cdot kip \qquad Check3 := \begin{cases} \text{"NG"} & \text{if } C_{aspv} \ge P_{naspv\_allow} \\ \text{"OK"} & \text{if } P_{naspv\_allow} \ge C_{aspv} \end{cases} = \text{"OK'}$$

Additional Checks: Design Bolted Connections of the splice plates to the girders, checking for shear, bearing, and slip critical connections.

26. CLOSURE POUR DESIGN:

See sheet S2 for drawing of closure pour.

Check the closure pour according to the negative bending capacity of the section.

Use the minimum reinforcing properties for design, to be conservative.

$A_{steel} = 27.4\cdot in^2 \qquad A_{rt} = 1.4\cdot in^2 \qquad A_{rb} = 1.9\cdot in^2$

$cg_{steel} := t_{slab} + y_{steel} = 24.3\cdot in \qquad cg_{rt} := 3in + 1.5\cdot\frac{5}{8}in = 3.9\cdot in \qquad cg_{rb} := t_{slab} - \left(1in + 1.5\cdot\frac{5}{8}in\right) = 6.1\cdot in$

Overall CG: $A_{neg} := A_{steel} + A_{rt} + A_{rb} = 30.7\cdot in^2 \qquad cg_{neg} := \dfrac{A_{steel}\cdot cg_{steel} + A_{rt}\cdot cg_{rt} + A_{rb}\cdot cg_{rb}}{A_{neg}} = 22.3\cdot in$

Moment of Inertia: $I_{zstl} := 3990in^4$

$$I_{neg} := I_{zstl} + A_{steel}\cdot\left(cg_{steel} - cg_{neg}\right)^2 + A_{rt}\cdot\left(cg_{rt} - cg_{neg}\right)^2 + A_{rb}\cdot\left(cg_{rb} - cg_{neg}\right)^2 = 5065.7\cdot in^4$$

Section Moduli:

$$S_{top\_neg} := \frac{I_{neg}}{cg_{neg} - cg_{rt}} = 276.4\cdot in^3 \qquad r_{neg} := \sqrt{\frac{I_{neg}}{A_{neg}}} = 12.8\cdot in$$

$$S_{bot\_neg} := \frac{I_{neg}}{\left(t_{slab} + t_{tf} + D_w + t_{bf} - cg_{neg}\right)} = 276\cdot in^3$$

Concrete Properties: $f_c = 5\cdot ksi$ $\qquad E_c = 4286.8\cdot ksi$

Steel Properties: $F_y = 50\cdot ksi$ $\qquad E_s = 29000\cdot ksi$ $\qquad L_{bneg} := 13.42ft$

$F_{yr} := 0.7 \cdot F_y = 35 \cdot ksi$

Negative Flexural Capacity:

Slenderness ratio for compressive flange: $\lambda_{fneg} := \dfrac{b_{bf}}{2 \cdot t_{bf}} = 8.5$

Limiting ratio for compactness: $\lambda_{pfneg} := 0.38 \cdot \sqrt{\dfrac{E_s}{F_y}} = 9.2$

Limiting ratio for noncompact $\lambda_{rfneg} := 0.56 \cdot \sqrt{\dfrac{E_s}{F_{yr}}} = 16.1$

Hybrid Factor: $R_h = 1$

$D_{cneg2} := \dfrac{D_w}{2} = 15.7 \cdot in$ $\quad a_{wc} := \dfrac{2 \cdot D_{cneg2} \cdot t_w}{b_{bf} \cdot t_{bf}} = 2.3$

$$R_b := \begin{vmatrix} 1.0 \text{ if } 2 \cdot \dfrac{D_{cneg2}}{t_w} \le 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}} \\ \min\left[1.0, 1 - \dfrac{a_{wc}}{1200 + 300 \cdot a_{wc}} \cdot \left(2 \cdot \dfrac{D_{cneg2}}{t_w} - 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}}\right)\right] \text{ otherwi} \end{vmatrix}$$

$R_b = 1$

Flange compression resistance:

$$F_{nc1} := \begin{vmatrix} R_b \cdot R_h \cdot F_y \text{ if } \lambda_{fneg} \le \lambda_{pfneg} \\ \left[\left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(\lambda_{fneg} - \lambda_{pfneg})}{(\lambda_{rfneg} - \lambda_{pfneg})}\right] \cdot R_b \cdot R_h \cdot F_y\right] \text{ otherwise} \end{vmatrix}$$

$F_{nc1} = 50 \cdot ksi$

Lateral Torsional Buckling Resistance: $r_{tneg} := \dfrac{b_{bf}}{\sqrt{12 \cdot \left(1 + \dfrac{D_{cneg2} \cdot t_w}{3 \cdot b_{bf} \cdot t_{bf}}\right)}} = 2.5 \cdot in$

$L_{pneg} := 1.0 \cdot r_{tneg} \cdot \sqrt{\dfrac{E_s}{F_y}} = 61.4 \cdot in$

$L_{rneg} := \pi \cdot r_{tneg} \cdot \sqrt{\dfrac{E_s}{F_{yr}}} = 230.5 \cdot in$

$C_b = 1$

$$F_{nc2} := \begin{vmatrix} R_b \cdot R_h \cdot F_y \text{ if } L_{bneg} \le L_{pneg} \\ \min\left[C_b \cdot \left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(L_{bneg} - L_{pneg})}{(L_{rneg} - L_{pneg})}\right] \cdot R_b \cdot R_h \cdot F_y, R_b \cdot R_h \cdot \right. \end{vmatrix}$$

$F_{nc2} = 41.2 \cdot ksi$

Compressive Resistance: $F_{nc} := \min(F_{nc1}, F_{nc2}) = 41.2 \cdot ksi$

Tensile Flexural Resistance: $F_{nt} := R_h \cdot F_y = 50 \cdot ksi$ For Strength

$F_{nt\_Serv} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$ For Service

Ultimate Moment Resistance: $M_{n\_neg} := \min\left(F_{nt} \cdot S_{top\_neg}, F_{nc} \cdot S_{bot\_neg}\right) = 946.7 \cdot kip \cdot ft$

$M_{UPier} = 948.1 \cdot kip \cdot ft$ from external FE analysis

$Check4 := M_{n\_neg} \geq M_{UPier} = 0$

For additional design, one may calculate the force couple at the section over the pier to find the force in the UHPC closure joint. This force can be used to design any additional reinforcement used in the joint.

Summary of changes from SHRP2:

- Adapted the AASHTO LRFD Bridge Design Specifications, 6th Edition (2012) and GDOT Standards
- MathCAD Design Aides provided in Appendix of the final report
  - Design loadings calucation (moment, shear, and reaction) for girders
  - Design loadings calucation for deck
- List of variable definitions added
- Enhanced the descriptions for all design steps
- Expansion of detail regarding girder sizing
- New cross-section drawings
- Load combination explanations
- 12 ft travel lanes, 6 ft shoulders and 2% slope from crown to comply with GDOT standards

File Name: Steel Girder-40 ft.xmcd

CONCRETE DECKED STEEL GIRDER DESIGN FOR ABC

The following example details the design of a steel girder bridge accompanied by precast concrete deck panels. This particular example was created in accordance with Accelerated Bridge Construction (ABC) principles. The example shown here is presented for a Georgia Department of Transportation research endeavour into ABC technology, and is intended to simplify the design procedure of ABC style bridges. This example was taken from the SHRP 2 Manual (S2-R04-RR-2), and modified by a Georgia Southern University research team working for the Georgia Department of Transportation.

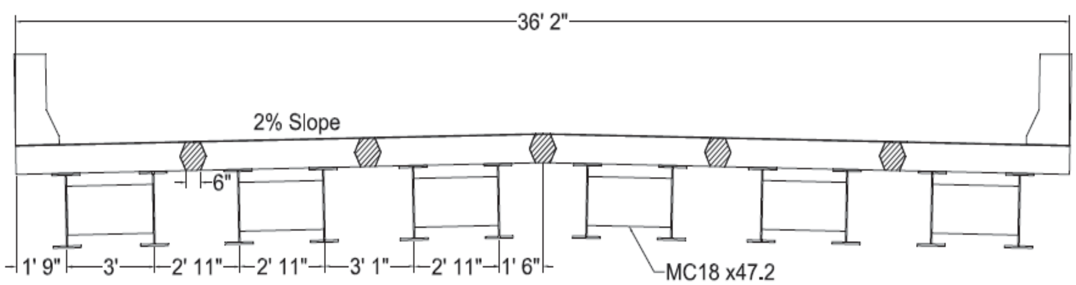
Note: These calculations do not consider every aspect of the bridge design process, and should not be condsidered exhaustive.

Note: All user inputs are highlighted in yellow for easy identification.

AASHTO LRFD Bridge Design Specifications (Sixth Edition with 2012 interims) was used to formulate this example. Located throughout this example are direct references to the AASHTO LRFD Bridge Design Specifications, which are found to the right side of their affiliated calculation.

Before beginning this example, a structural modeling program was used to analyze the superstructure. Although the calculations are not shown, the outputs are used for the design moments, shears and reactions in the example.

BRIDGE GEOMETRY:



Design member parameters:

Deck Width:	$w_{deck} := 36ft + 2in$	C. to C. Piers:	$Length := 40ft$
Roadway Width:	$w_{roadway} := 33ft$	C. to C. Bearings	$L_{span} := 37ft + 10in$
Skew Angle:	$Skew := 0deg$	Bridge Length:	$L_{total} := 3 \cdot Length = 120\,ft$
Deck Thickness	$t_d := 10.5in$	Stringer	W27x84
Haunch Thickness	$t_h := 2in$	Stringer Weight	$w_{s1} := 84plf$
Haunch Width	$w_h := 10.5in$	Stringer Length	$L_{str} := Length - 6 \cdot in = 39.5\,ft$
Girder Spacing	$spacing_{int} := 2ft + 11in$	Average spacing of adjacent beams. This value is used so that effective deck width is not overestimated.	
	$spacing_{ext} := 3ft$		

TABLE OF CONTENTS:

List of Variable Definitions

A = plan area of ice floe (ft^2); depth of temperature gradient (in.) (C3.9.2.3) (3.12.3)
AEP = apparent earth pressure for anchored walls (ksf) (3.4.1)
AF = annual frequency of bridge element collapse (number/yr.) (C3.14.4)
AS = peak seismic ground acceleration coefficient modified by short-period site factor (3.10.4.2)
B = notional slope of backfill (degrees) (3.11.5.8.1)
B′ = equivalent footing width (ft) (3.11.6.3)
Be = width of excavation (ft) (3.11.5.7.2b)
BM = beam (width) for barge, barge tows, and ship vessels (ft) (C3.14.5.1)
Bp = width of bridge pier (ft) (3.14.5.3)
BR = vehicular braking force; base rate of vessel aberrancy (3.3.2) (3.14.5.2.3)
b = braking force coefficient; width of a discrete vertical wall element (ft) (C3.6.4) (3.11.5.6)
bf = width of applied load or footing (ft) (3.11.6.3)
C = coefficient to compute centrifugal forces; constant for terrain conditions in relation to wind approach (3.6.3) (C3.8.1.1)
CD = drag coefficient (s^2 lbs./ft^4) (3.7.3.1)
CH = hydrodynamic mass coefficient (3.14.7)
CL = lateral drag coefficient (C3.7.3.1)
Csm = elastic seismic response coefficient for the m^{th} mode of vibration (3.10.4.2)
c = soil cohesion (ksf) (3.11.5.4)
cf = distance from back of a wall face to the front of an applied load or footing (ft) (3.11.6.3)
D = depth of embedment for a permanent nongravity cantilever wall with discrete vertical wall elements (ft) (3.11.5.6)
DE = minimum depth of earth cover (ft) (3.6.2.2)
Do = calculated embedment depth to provide equilibrium for nongravity cantilevered with continuous vertical elements by the simplified method (ft) (3.11.5.6)
D1 = effective width of applied load at any depth (ft) (3.11.6.3)
d = depth of potential base failure surface below base of excavation (ft); horizontal distance from the back of a wall face to the centerline of an applied load (ft) (3.11.5.7.2b) (3.11.6.3)
dc = total thickness of cohesive soil layers in the top 100 ft (3.10.3.1)
ds = total thickness of cohesionless soil layers in the top 100 ft (3.10.3.1)
E = Young's modulus (ksf) (C3.9.5)
EB = deformation energy (kip-ft) (C3.14.11)
e′ = eccentricity of load on footing (ft) (3.11.6.3)
F1 = lateral force due to earth pressure (kip/ft) (3.11.6.3)
F2 = lateral force due to traffic surcharge (kip/ft) (3.11.6.3)
f = constant applied in calculating the coefficient C used to compute centrifugal forces, taken equal to 4/3 for load combinations other than fatigue and 1.0 for fatigue (3.6.3)
f′c = specified compressive strength of concrete for use in design (ksi) (3.5.1)
g = gravitational acceleration (ft/s^2) (3.6.3)
H = ultimate bridge element strength (kip); final height of retaining wall (ft); total excavation depth (ft); resistance of bridge component to a horizontal force (kip) (C3.11.1) (3.11.5.7.1) (3.14.5.4)
Hp = ultimate bridge pier resistance (kip) (3.14.5.4)
Hs = ultimate bridge superstructure resistance (kip) (3.14.5.4)
H1 = distance from ground surface to uppermost ground anchor (ft) (3.11.5.7.1)
Hn+1 = distance from base of excavation to lowermost ground anchor (ft) (3.11.5.7.1)
h = notional height of earth pressure diagram (ft) (3.11.5.7)
heq = equivalent height of soil for vehicular load (ft) (3.11.6.4)
IM = dynamic load allowance (C3.6.1.2.5)
k = coefficient of lateral earth pressure; number of cohesive soil layers in the top 100 ft (3.11.6.2) (3.10.3.1)
ka = coefficient of active lateral earth pressure (3.11.5.1)
ko = coefficient of at rest lateral earth pressure (3.11.5.1)
kp = coefficient of passive lateral earth pressure (3.11.5.1)
ks = coefficient of earth pressure due to surcharge (3.11.6.1)
L = perimeter of pier (ft); length of soil reinforcing elements in an MSE wall (ft); length of footing (ft);

expansion length (in.) (3.9.5) (3.11.5.8) (3.11.6.3) (3.12.2.3)
ℓ = characteristic length (ft); center-to-center spacing of vertical wall elements (ft) (C3.9.5) (3.11.5.6)
m = multiple presence factor; number of cohesionless soil layers in the top 100 ft (3.6.1.1.2) (3.10.3.1)
N = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile (3.10.3.1)
Nch = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for cohesive soil layers in the upper 100 ft of the soil profile and us for cohesive soil layers (PI > 20) in the top 100 ft (us method) (3.10.3.1)
Nchi = blowcount for a cohesionless soil layer (not to exceed 100 blows/ft in the above expression) (3.10.3.1)
Ni = Standard Penetration Test blow count of a layer (not to exceed 100 blows/ft in the above expression). Note that when using Method B, N values are for cohesionless soils and cohesive soil and rock layers within the upper 100 ft Where refusal is met for a rock layer, Nishould be taken as 100 blows/ft (3.10.3.1)
Ns = stability number (3.11.5.6)
OCR = overconsolidation ratio (3.11.5.2)
P = maximum vertical force for single ice wedge (kip); load resulting from vessel impact (kip); concentrated wheel load (kip); live load intensity; point load (kip) (C3.9.5) (3.14.5.4) (C3.6.1.2.5) (C3.11.6.2) (3.11.6.1)
Pa = force resultant per unit width of wall (kip/ft) (3.11.5.8.1)
PC = probability of bridge collapse (3.14.5)
PD = design wind pressure (ksf) (3.8.1.2.1)
PGA = peak seismic ground acceleration coefficient on rock (Site Class B) (3.10.2.1) (3.10.4.2)
PH = lateral force due to superstructure or other concentrated lateral loads (kip/ft) (3.11.6.3)
Ph = horizontal component of resultant earth pressure on wall (kip/ft) (3.11.5.5)
PI = plasticity index (ASTM D4318) (3.10.3.1)
Pp = passive earth pressure (kip/ft) (3.11.5.4)
Pv = vertical component of resultant earth pressure on wall (kip/ft); load per linear foot of strip footing (kip/ft) (3.11.5.5) (3.11.6.3)
P′v = load on isolated rectangular footing or point load (kip) (3.11.6.3)
p = effective ice crushing strength (ksf); stream pressure (ksf); basic earth pressure (psf); fraction of truck traffic in a single lane; load intensity (ksf) (3.9.2.2) (3.7.3.1) (3.11.5.1) (3.6.1.4.2) (3.11.6.1)
pa = apparent earth pressure (ksf); maximum ordinate of pressure diagram (ksf) (3.11.5.3) (3.11.5.7.1)
pp = passive earth pressure (ksf) (3.11.5.4)
Q = total factored load; load intensity for infinitely long line loading (kip/ft) (3.4.1) (3.11.6.2)
Qi = force effects (3.4.1)
q = surcharge pressure (ksf) (3.11.6.3)
qs = uniform surcharge pressure (ksf) (3.11.6.1)
R = radius of curvature (ft); radius of circular pier (ft); seismic response modification factor; reduction factor of lateral passive earth pressure; radial distance from point of load application to a point on the wall (ft); reaction force to be resisted by subgrade below base of excavation (kip/ft) (3.6.3) (3.9.5) (3.10.7.1) (3.11.5.4) (3.11.6.1) (3.11.5.7.1)
Sm = shear strength of rock mass (ksf) (3.11.5.6)
Su = undrained shear strength of cohesive soil (ksf) (3.11.5.6)
Sub = undrained strength of soil below excavation base (ksf) (3.11.5.7.2b)
Sv = vertical spacing of reinforcements (ft) (3.11.5.8.1)
us = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile (3.10.3.1)
sui = undrained shear strength for a cohesive soil layer (not to exceed 5.0 ksf in the above expression) (3.10.3.1)
S1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock (Site Class B) (3.10.2.1) (3.10.4.2)
T = mean daily air temperature (°F) (C3.9.2.2)
TF = period of fundamental mode of vibration of bridge (s) (3.10.2.2)
Thi = horizontal load in anchor i (kip/ft) (3.11.5.7.1)
Tm = period of vibration for mth mode (s) (3.10.4.2)
Tmax = applied load to reinforcement in a mechanically stabilized earth wall (kip/ft) (3.11.5.8.2)
TMaxDesign= maximum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
TMinDesign = minimum design temperature used for thermal movement effects (°F) (3.12.2.1) (3.12.2.2) (3.12.2.3)
TS = corner period at which acceleration response spectrum changes from being independent of period to being inversely proportional to period (s) (3.10.4.2)
T0 = reference period used to define shape of acceleration response spectrum (s) (3.10.4.2)

t = thickness of ice (ft); thickness of deck (in.) (3.9.2.2) (3.12.3)

V = design velocity of water (ft/s); design impact speed of vessel (ft/s) (3.7.3.1) (3.14.6)

VB = base wind velocity taken as 100 mph (3.8.1.1)

VDZ = design wind velocity at design Elevation Z (mph) (3.8.1.1)

VMIN = minimum design impact velocity taken not less than the yearly mean current velocity for the bridge location (ft/s) (3.14.6)

V0 = friction velocity, a meteorological wind characteristic for various upwind surface characteristics (mph) (3.8.1.1)

V30 = wind speed at 30.0 ft above low ground or water level (mph) (3.8.1.1)

v = highway design speed (ft/s) (3.6.3)

s v = average shear wave velocity for the upper 100 ft of the soil profile (3.10.3.1)

W = displacement weight of vessel (tonne) (C3.14.5.1)

w = width of clear roadway (ft); width of clear pedestrian and/or bicycle bridge (ft); width of pier at level of ice action (ft); specific weight of water (kcf); moisture content (ASTM D2216) (3.6.1.1.1) (3.6.1.6) (3.9.2.2) (C3.7.3.1) (3.10.3.1)

X = horizontal distance from back of wall to point of load application (ft); distance to bridge element from the centerline of vessel transit path (ft) (3.11.6.2) (3.14.6)

X1 = distance from the back of the wall to the start of the line load (ft) (3.11.6.2)

X2 = length of the line load (ft) (3.11.6.2)

Z = structure height above low ground or water level > 30.0 ft (ft); depth below surface of soil (ft); depth from the ground surface to a point on the wall under consideration (ft); vertical distance from point of load application to the elevation of a point on the wall under consideration (ft) (3.8.1.1) (3.11.6.3) (3.11.6.2)

Z0 = friction length of upstream fetch, a meteorological wind characteristic (ft) (3.8.1.1)

Z2 = depth where effective width intersects back of wall face (ft) (3.11.6.3)

z = depth below surface of backfill (ft) (3.11.5.1)

α = constant for terrain conditions in relation to wind approach; coefficient for local ice condition; inclination of pier nose with respect to a vertical axis (degrees); inclination of back of wall with respect to a vertical axis (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing nearest to the wall (rad); coefficient of thermal expansion (in./in./°F) (C3.8.1.1) (C3.9.2.2) (3.9.2.2) (C3.11.5.3) (3.11.6.2) (3.12.2.3)

β = safety index; nose angle in a horizontal plane used to calculate transverse ice forces (degrees); slope of backfill surface behind retaining wall; {+ for slope up from wall; − for slope down from wall} (degrees) (C3.4.1) (3.9.2.4.1) (3.11.5.3)

β′ = slope of ground surface in front of wall {+ for slope up from wall; − for slope down from wall} (degrees) (3.11.5.6)

γ = load factors; unit weight of materials (kcf); unit weight of water (kcf); unit weight of soil (kcf) (C3.4.1) (3.5.1) (C3.9.5) (3.11.5.1)

γs = unit weight of soil (kcf) (3.11.5.1)

γ′s = effective soil unit weight (kcf) (3.11.5.6)

γEQ = load factor for live load applied simultaneously with seismic loads (3.4.1)

γeq = equivalent-fluid unit weight of soil (kcf) (3.11.5.5)

γi = load factor (3.4.1)

γp = load factor for permanent loading (3.4.1)

γSE = load factor for settlement (3.4.1)

γTG = load factor for temperature gradient (3.4.1)

Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft) (C3.11.1) (3.11.5.5)

Δp = constant horizontal earth pressure due to uniform surcharge (ksf) (3.11.6.1)

Δph = constant horizontal pressure distribution on wall resulting from various types of surcharge loading (ksf) (3.11.6.2)

ΔT = design thermal movement range (in.) (3.12.2.3)

ΔσH = horizontal stress due to surcharge load (ksf) (3.11.6.3)

Δσv = vertical stress due to surcharge load (ksf) (3.11.6.3)

δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing furthest from the wall (rad) (C3.9.5) (3.11.5.3) (3.11.6.2)

ηi = load modifier specified in Article 1.3.2; wall face batter (3.4.1) (3.11.5.9)

θ = angle of back of wall to the horizontal (degrees); angle of channel turn or bend (degrees); angle between direction of stream flow and the longitudinal axis of pier (degrees) (3.11.5.3) (3.14.5.2.3) (3.7.3.2)
θ_f = friction angle between ice floe and pier (degrees) (3.9.2.4.1)
σ = standard deviation of normal distribution (3.14.5.3)
σ_T = tensile strength of ice (ksf) (C3.9.5)
ν = Poisson's Ratio (dim.) (3.11.6.2)
φ = resistance factors (C3.4.1)
φ_f = angle of internal friction (degrees) (3.11.5.4)

φ'_f = effective angle of internal friction (degrees) (3.11.5.2)
φ_r = internal friction angle of reinforced fill (degrees) (3.11.6.3)
φ'_s = angle of internal friction of retained soil (degrees) (3.11.5.6)

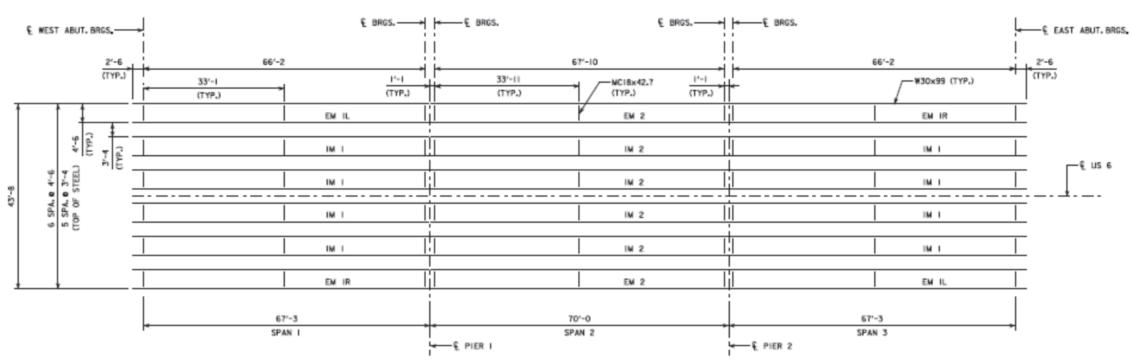
• Permanent Loads
CR = force effects due to creep
DD = downdrag force
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

• Transient Loads

EQ = earthquake load
FR = friction load
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = force effect due to settlement
TG = force effect due to temperature gradient
TU = force effect due to uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

1. INTRODUCTION

AASHTO LRFD principles were used in the design of this superstructure. The example is designed for a bridge with three even spans, and has no skew. The bridge has two 12-foot wide lanes and two 6-foot wide shoulders, for a total roadway width of 36' from curb to curb. The bridge deck is precast reinforced concrete with overhangs at the outermost girders. The longitudinal girders are placed as simply supported modules, and made continuous with connection plates and cast-in-place deck joints. The design of the continuity at the deck joint is addressed in final sections of this example.



FRAMING PLAN

The cross-section consists of six modules. The interior modules are identical and consist of two steel girders and a 6'-0" precast composite deck slab. Exterior modules include two steel girders and a 6'-1" precast composite deck slab, with F-shape barriers. Grade 50 steel is used throughout, and the deck concrete has a compressive strength of 5,000 psi.

The closure pour joints between the modules use Ultra High Performance Concrete with a strength of 21,000 psi.

Steel girder design steps, including constructability checks, fatigue design for infinite fatigue lift (unless otherwise noted), and bearing stiffener design comprise the majority of the example. Diaphragm and deck design procedures are present, but not detailed.

Tips for reading this Design Example:

This calculation was prepared with Mathcad version 14. Mathcad was used in this instance to provide a clear representation of formulas, and their execution. Design software other than Mathcad is recommended for a speedier and more accurate design.

Mathcad is not a design software. Mathcad executes user mathematical and simple logic commands.

Example 1: User inputs are noted with dark shaded boxes. Shading of boxes allows the user to easily find the location of a desired variable. Given that equations are written in mathcad in the same fashion as they are on paper, except that they are interactive, shading input cells allows the user to quicly locate inputs amongst other data on screen. Units are user inputs.

Height of Structure: $H_{structure} := 25ft$

Example 2: Equations are typed directly into the workspace. Mathcad then reads the operators and executes the calculations.

Panels are 2.5' $N_{panels} := \frac{H_{structure}}{2.5ft}$ $N_{panels} = 10$

Example 3: If Statements are an important operator that allow for the user to dictate a future value with given parameters. They are marked by a solid bar and operate with the use of program specific logic commands.

Operator offers discount per volume of panels

$$Discount := \left| \begin{array}{l} .75 \text{ if } N_{panels} \geq 6 \\ .55 \text{ if } N_{panels} \geq 10 \\ 1 \text{ otherwise} \end{array} \right. \qquad Discount = 0.6$$

Example 4: True or False Verification Statements are an important operator that allow for the user to verify a system criteria that has been manually input. They are marked by lighter shading to make a distinction between the user inputs. True or false statements check a single or pairs of variables and return a Zero or One.

Owner to proceed if discounts on retail below 60% $Discount \leq .55 = 1$

2. DESIGN PHILOSOPHY

The superstructure of the bridge in this example consists of modules, which are two rolled steel girders supporting a bridge deck panel along their length. The girders are assumed to be simply supported under the weight of the deck panels. In each module, one girder is assumed to support half the weight of its respective deck panel.

The barrier wall is added to exterior modules once the deck and girders are joined. When working with the barrier dead load, the weight is assumed to be evenly distributed between the two modules. Under the additional barrier dead load, the girders are again assumed to be simply supported.

Concerning transportation of modules, it is assumed that the deck has reached 28-day concrete strength, and the deck is fully composite with the girders. The self-weight of the module during lifting and placement is assumed as evenly distributed to four pick points (two per girder).

The modules are placed such that there is a bearing on each end and are again simply supported. The continuous span

configuration, which includes two bearings per pier on either side of the UHPC joints, is analyzed for positive and negative bending and shear (using simple or refined methods). The negative bending moment above the pier is used to find the force couple for continuity design, between the compression plates at the bottom of the girders and the closure joint in the deck.

The deck design utilizes the equivalent strip method.

3. DESIGN CRITERIA

The first step for any bridge design is to establish the design criteria. The following is a summary of the primary design criteria for this design example:

Governing Specifications: AASTHO LRFD Bridge Design Specifications (6th Edition with 2012 interims)

Design Methodology: Load and Resistance Factor Design (LRFD)

Live Load Requirements: HL-93 S S3.6

Section Constraints:

$W_{mod.max} := 200 \cdot kip$ Upper limit on the weight of the modules, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

4. MATERIAL PROPERTIES

Structural Steel Yield Strength:	$F_y := 50ksi$		STable 6.4.1-1
Structural Steel Tensile Strength:	$F_u := 65ksi$		STable 6.4.1-1
Concrete 28-day Compressive Strength:	$f_c := 5ksi$	$f_{c\_uhpc} := 21ksi$	S5.4.2.1
Reinforcement Strength:	$F_s := 60ksi$		S5.4.3 & S6.10.3.7
Steel Density:	$w_s := 490pcf$		STable 3.5.1-1
Concrete Density:	$w_c := 150pcf$		STable 3.5.1-1
Modulus of Elasticity - Steel:	$E_s := 29000ksi$		
Modulus of Elasticity - Concrete:	$E_c := 33000 \cdot \left(\frac{w_c}{1000pcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 4286.8 \cdot ksi$		
Modular Ratio:	$n := ceil\left(\frac{E_s}{E_c}\right) = 7$		
Future Wearing Surface Density:	$W_{fws} := 140pcf$		STable 3.5.1-1
Future Wearing Surface Thickness:	$t_{fws} := 2.5in$	(Assumed)	

5. LOAD COMBINATIONS

The following load combinations will be used in this design example, in accordance with Table 3.4.1-1.

Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.

Strength III—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

Strength V—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

Service I—Load combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values. Also related to deflection control in buried metal structures, tunnel liner plate, and thermoplastic pipe, to control crack width in reinforced concrete structures, and for transverse analysis relating to tension in concrete segmental girders. This load combination should also be used for the investigation of slope stability.

Service II—Load combination intended to control yielding of steel structures and slip of slip-critical connections due to vehicular live load.

Fatigue I—Fatigue and fracture load combination related to infinite load-induced fatigue life.

Strength I = 1.25DC + 1.5DW + 1.75(LL+IM), where IM = 33%

Strength III = 1.25DC + 1.5DW + 1.40WS

Strength V = 1.25DC + 1.5DW + 1.35(LL+IM) + 0.40WS + 1.0WL, where IM = 33%

Service I = 1.0DC + 1.0DW + 1.0(LL+IM) + 0.3WS + 1.0WL, where IM = 33%

Service II = 1.0DC + 1.0DW + 1.3(LL+IM), where IM = 33%

Fatigue I = 1.5(LL+IM), where IM = 15%

6. BEAM SECTION

Determining the proper girder depth and dimensions is a vital part of any bridge design process. The size of the girder is a major factor in the cost of the bridge. From Table 2.5.2.6.3-1, the suggested minimum overall depth of the composite I-section in a continuous span is equal to 0.032L.

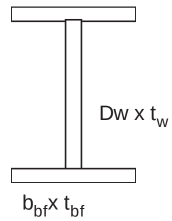
Thus we have, (.032*40ft) = 1.28’ = 15.36” (this is a minimum and can be adjusted to meet criteria)

The following girder dimensions were taken from the AISC Steel Construction Manual (14th Edition).

Determine Beam Section Properties:

Girder W27x84 b_{tf}x t_{tf}

Top Flange	$b_{tf} := 10.0in$	$t_{tf} := 0.64in$
Bottom Flange	$b_{bf} := 10.0in$	$t_{bf} := 0.64in$
Web	$D_w := 25.4in$	$t_w := 0.46in$
Girder Depth	$d_{gird} := 26.7in$	



Check Flange Proportion Requierements Met: S 6.10.2.2

$$\frac{b_{tf}}{2 \cdot t_{tf}} \le 12.0 = 1 \qquad \frac{b_{bf}}{2 \cdot t_{bf}} \le 12.0 = 1$$

$$b_{tf} \ge \frac{D_w}{6} = 1 \qquad b_{bf} \ge \frac{D_w}{6} = 1$$

$$t_{tf} \ge 1.1 \cdot t_w = 1 \qquad t_{bf} \ge 1.1 \cdot t_w = 1$$

$$0.1 \le \frac{\frac{t_{bf}^{3} \cdot b_{bf}}{12}}{\frac{t_{tf}^{3} \cdot b_{tf}}{12}} \le 10 = 1 \qquad \frac{\frac{t_{bf} \cdot b_{bf}}{12}}{\frac{t_{tf} \cdot b_{tf}}{12}} \ge 0.3 = 1$$

Properties for use when analyzing under beam self weight (steel only):

$$A_{tf} := b_{tf} \cdot t_{tf} \qquad A_{bf} := b_{bf} \cdot t_{bf} \qquad A_w := D_w \cdot t_w$$

$$A_{steel} := A_{bf} + A_{tf} + A_w \qquad A_{steel} = 24.5 \cdot in^2$$

Total steel area.

$$y_{steel} := \frac{A_{tf} \cdot \frac{t_{tf}}{2} + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf}\right)}{A_{steel}} \qquad y_{steel} = 13.3 \cdot in$$

Steel centroid from top.

Calculate Iz:

Moment of inertia about Z axis.

$$I_{zsteel} := \frac{t_w \cdot D_w^{3}}{12} + \frac{b_{tf} \cdot t_{tf}^{3}}{12} + \frac{b_{bf} \cdot t_{bf}^{3}}{12} + A_w \cdot \left(\frac{D_w}{2} + t_{tf} - y_{steel}\right)^2 + A_{tf} \cdot \left(y_{steel} - \frac{t_{tf}}{2}\right)^2 + A_{bf} \cdot \left(D_w + \frac{t_{bf}}{2} + t_{tf} - y_{steel}\right)^2$$

Calculate Iy:

$$I_{ysteel} := \frac{D_w \cdot t_w^{3} + t_{tf} \cdot b_{tf}^{3} + t_{bf} \cdot b_{bf}^{3}}{12}$$

Moment of inertia about Y axis.

Calculate Ix:

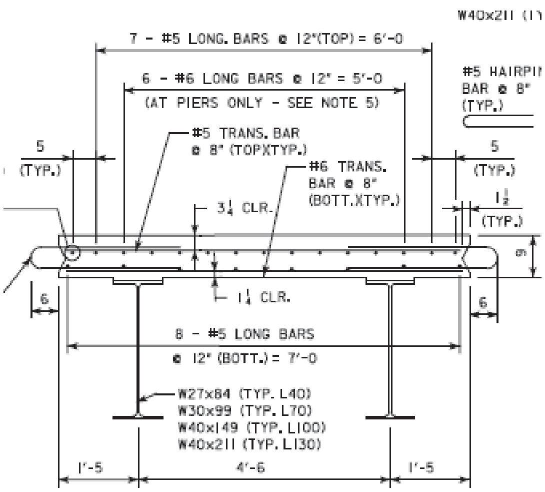
$$I_{xsteel} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{3} + b_{bf} \cdot t_{bf}^{3} + D_w \cdot t_w^{3}\right)$$

Moment of inertia about X axis.

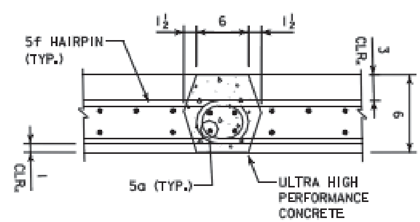
$$I_{zsteel} = 2798.469 \cdot in^4 \qquad I_{ysteel} = 106.873 \cdot in^4 \qquad I_{xsteel} = 2.6 \cdot in^4 \qquad A_{steel} = 24.5 \cdot in^2$$

Composite Section Properties (Uncracked Section - used for barrier dead load and live load positive bending):

Determine composite slab and reinforcing properties



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

Slab thickness assumes some sacrificial thickness; use: $t_{slab} := 8in$

$D_t := \left(t_{slab} + t_{tf} + D_w + t_{bf}\right) = 34.7\cdot in$ Total section depth

$b_{eff} := spacing_{int} \quad b_{eff} = 35\cdot in$ Effective width. S 4.6.2.6.1 LRFD

$b_{tr} := \dfrac{b_{eff}}{n}$ Transformed slab width as steel.

$I_{zslab} := b_{tr}\cdot\dfrac{t_{slab}^{\;3}}{12}$ Transformed slab moment of inertia about z axis as steel.

$A_{slab} := b_{tr}\cdot t_{slab}$ Transformed slab area as steel.

Slab reinforcement: (Use #5 @ 8" top, and #6 @ 8" bottom; additional bar for continuous segments of #6 @ 12")

Typical Cross Section

$A_{rt} := 0.465\,\dfrac{in^2}{ft}\cdot b_{eff} = 1.4\cdot in^2$

Cross Section Over Support

$A_{rb} := 0.66\,\dfrac{in^2}{ft}\cdot b_{eff} = 1.9\cdot in^2 \quad A_{rtadd} := 0.44\cdot\dfrac{in^2}{ft}\cdot b_{eff} = 1.3\cdot in^2$

$A_r := A_{rt} + A_{rb} = 3.3 \cdot in^2$

$A_{rneg} := A_r + A_{rtadd} = 4.6 \cdot in^2$

$c_{rt} := 2.5in + 0.625in + \left(\frac{5}{16}\right)in = 3.4 \cdot in$

$c_{rb} := t_{slab} - 1.75in - \left(\frac{6}{16}\right)in = 5.9 \cdot in$ ref from top of slab

$c_r := \frac{\left(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb}\right)}{A_r} = 4.9 \cdot in$

$c_{rneg} := \frac{\left(A_{rt} \cdot c_{rt} + A_{rb} \cdot c_{rb} + A_{rtadd} \cdot c_{rt}\right)}{A_{rneg}} = 4.5 \cdot in$

Find composite section centroid:

$A_x := A_{steel} + \frac{A_r \cdot (n - 1)}{n} + A_{slab}$ $\qquad y_{slab} := \frac{t_{slab}}{2}$

$y_{st} := \frac{A_{tf} \cdot \left(\frac{t_{tf}}{2} + t_{slab}\right) + A_{bf} \cdot \left(\frac{t_{bf}}{2} + D_w + t_{tf} + t_{slab}\right) + A_w \cdot \left(\frac{D_w}{2} + t_{tf} + t_{slab}\right)}{A_{steel}}$

Centroid of steel from top of slab.

$y_c := \frac{y_{st} \cdot A_{steel} + \frac{c_r \cdot A_r \cdot (n - 1)}{n} + A_{slab} \cdot y_{slab}}{A_x}$ $\qquad y_c = 10.3 \cdot in$

Centroid of transformed composite section from top of slab.

Calculate Transformed Iz for composite section:

$I_z := I_{zsteel} + A_{steel} \cdot \left(y_{st} - y_c\right)^2 + I_{zslab} + A_{slab} \cdot \left(y_{slab} - y_c\right)^2 + \frac{A_r \cdot (n - 1)}{n} \cdot \left(c_r - y_c\right)^2$

Transformed moment of inertia about the z axis.

Calculate Transformed Iy for composite section:

$t_{tr} := \frac{t_{slab}}{n}$ Transformed slab thickness.

$I_{yslab} := \frac{t_{tr} \cdot b_{eff}^{\,3}}{12}$ Transformed moment of inertia about y axis of slab.

$I_y := I_{ysteel} + I_{yslab}$ Transformed moment of inertia about the y axis (ignoring reinforcement).

Calculate Transformed Ix for composite section:

$I_x := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\,3} + b_{bf} \cdot t_{bf}^{\,3} + D_w \cdot t_w^{\,3} + b_{tr} \cdot t_{slab}^{\,3}\right)$

Transformed moment of inertia about the x axis.

Results: $A_x = 67.3 \cdot in^2$ $\quad I_y = 4190.2 \cdot in^4$ $\quad I_z = 7666.4 \cdot in^4$ $\quad I_x = 855.9 \cdot in^4$

Composite Section Properties (Uncracked Section - used for live load negative bending):

Find composite section area and centroid:

$A_{xneg} := A_{steel} + \frac{A_{rneg} \cdot (n - 1)}{n} + A_{slab}$

$y_{cneg} := \frac{y_{steel} \cdot A_{steel} + \frac{c_{rneg} \cdot A_{rneg} \cdot (n - 1)}{n} + A_{slab} \cdot y_{slab}}{A_{xneg}}$ $\qquad y_{cneg} = 7.4 \cdot in$

Centroid of transformed composite section from top of slab.

Calculate Transformed Izneg for composite negative moment section:

$$I_{zneg} := I_{zsteel} + A_{steel} \cdot (y_{steel} - y_{cneg})^2 + I_{zslab} + A_{slab} \cdot (y_{slab} - y_{cneg})^2 + \frac{A_{rneg} \cdot (n-1)}{n} \cdot (c_{rneg} - y_{cneg})^2$$

Transformed moment of inertia about the z axis.

$$I_{zneg} = 4371.7 \cdot in^4$$

Composite Section Properties (Cracked Section - used for live load negative bending):

Find cracked section area and centroid:

$$A_{cr} := A_{steel} + A_{rneg} = 29 \cdot in^2$$

$$y_{cr} := \frac{(A_{steel} \cdot y_{steel} + A_{rneg} \cdot c_{rneg})}{A_{cr}} = 11.9 \cdot in \qquad y_{crb} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} = 22.7 \cdot in$$

Find cracked section moments of inertia and section moduli:

$$I_{zcr} := I_{zsteel} + A_{steel} \cdot (y_{steel} - y_{cr})^2 + A_r \cdot (c_r - y_{cr})^2 \qquad I_{zcr} = 3010.5 \cdot in^4$$

$$I_{ycr} := I_{ysteel} \qquad I_{ycr} = 106.9 \cdot in^4$$

$$I_{xcr} := \frac{1}{3} \cdot \left(b_{tf} \cdot t_{tf}^{\ 3} + b_{bf} \cdot t_{tf}^{\ 3} + D_w \cdot t_w^{\ 3}\right) \qquad I_{xcr} = 2.6 \cdot in^4$$

$$d_{topcr} := y_{cr} - c_{rt} \qquad d_{topcr} = 8.5 \cdot in$$

$$d_{botcr} := t_{slab} + t_{tf} + D_w + t_{bf} - y_{cr} \qquad d_{botcr} = 22.7 \cdot in$$

$$S_{topcr} := \frac{I_{zcr}}{d_{topcr}} \qquad S_{topcr} = 353.8 \cdot in^3$$

$$S_{botcr} := \frac{I_{zcr}}{d_{botcr}} \qquad S_{botcr} = 132.4 \cdot in^3$$

7. PERMANENT LOADS

Phase 1: Steel girders are simply supported, and support their self-weight plus the weight of the slab. Steel girders in each module for this example are separated by three diaphragms - one at each bearing location, and one at midspan. Other module span configurations may require an increase or decrease in the number of diaphragms.

$$W_{deck\_int} := w_c \cdot spacing_{int} \cdot t_d \qquad W_{deck\_int} = 382.8 \cdot plf$$

$$W_{deck\_ext} := w_c \cdot spacing_{ext} \cdot t_d \qquad W_{deck\_ext} = 393.8 \cdot plf$$

$$W_{haunch} := w_c \cdot w_h \cdot t_h \qquad W_{haunch} = 21.9 \cdot plf$$

$$W_{stringer} := w_{s1} \qquad W_{stringer} = 84 \cdot plf$$

Diaphragms:	MC18x42.7	Thickness Conn. Plate	$t_{conn} := \frac{5}{8} in$
Diaphragm Weight	$w_{s2} := 42.7plf$	Width Conn. Plate	$w_{conn} := 5in$
Diaphragm Length	$L_{diaph} := 4ft + 2.5in$	Height Conn. Plate	$h_{conn} := 28.5in$

$$W_{diaphragm} := w_{s2} \cdot \frac{L_{diaph}}{2} \qquad W_{diaphragm} = 89.8 \cdot lbf$$

$W_{conn} := 2 \cdot w_s \cdot t_{conn} \cdot w_{conn} \cdot h_{conn}$ $\quad W_{conn} = 50.5 \cdot lbf$

$W_{DCpoint} := \left(W_{diaphragm} + W_{conn}\right) \cdot 1.05$ $\quad W_{DCpoint} = 147.4 \cdot lbf$

Equivalent distributed load from DC point loads: $w_{DCpt\_equiv} := \dfrac{3 \cdot W_{DCpoint}}{L_{str}} = 11.2 \cdot plf$

Interior Uniform Dead Load, Phase 1: $W_{DCuniform1\_int} := W_{deck\_int} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 499.9 \cdot plf$

Exterior Uniform Dead Load, Phase 1: $W_{DCuniform1\_ext} := W_{deck\_ext} + W_{haunch} + W_{stringer} + w_{DCpt\_equiv} = 510.8 \cdot plf$

Moments due to Phase 1 DL: $M_{DC1\_int}(x) := \dfrac{W_{DCuniform1\_int} \cdot x}{2} \cdot \left(L_{str} - x\right)$ $\quad M_{DC1\_ext}(x) := \dfrac{W_{DCuniform1\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shear due to Phase 1 DL: $V_{DC1\_int}(x) := W_{DCuniform1\_int} \cdot \left(\dfrac{L_{str}}{2} - x\right)$ $\quad V_{DC1\_ext}(x) := W_{DCuniform1\_ext} \cdot \left(\dfrac{L_{str}}{2} - x\right)$

Phase 2: Steel girders are simply supported and composite with the deck slab, and support their self-weight plus the weight of the slab in addition to barriers on exterior modules. Barriers are assumed to be evenly distributed between the two exterior module girders.

Barrier Area $\quad A_{barrier} := 2.89 ft^2$

Barrier Weight $\quad W_{barrier} := \dfrac{\left(w_c \cdot A_{barrier}\right)}{2}$ $\quad W_{barrier} = 216.8 \cdot plf$

Interior Dead Load, Phase 2: $\quad W_{DCuniform\_int} := W_{DCuniform1\_int} = 499.9 \cdot plf$

Exterior Dead Load, Phase 2: $\quad W_{DCuniform\_ext} := W_{DCuniform1\_ext} + W_{barrier} = 727.6 \cdot plf$

Moments due to Phase 2 DL: $M_{DC2\_int}(x) := \dfrac{W_{DCuniform\_int} \cdot x}{2} \cdot \left(L_{str} - x\right)$ $\quad M_{DC2\_ext}(x) := \dfrac{W_{DCuniform\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shear due to Phase 2 DL: $V_{DC2\_int}(x) := W_{DCuniform\_int} \cdot \left(\dfrac{L_{str}}{2} - x\right)$ $\quad V_{DC2\_ext}(x) := W_{DCuniform\_ext} \cdot \left(\dfrac{L_{str}}{2} - x\right)$

Phase 3: Girders are composite and have been made continuous. Utilities and future wearing surface are applied.

Unit Weight Overlay $\quad w_{ws} := 30 psf$

$W_{ws\_int} := w_{ws} \cdot spacing_{int}$ $\quad W_{ws\_int} = 87.5 \cdot plf$

$W_{ws\_ext} := w_{ws} \cdot \left(spacing_{ext} - 1 \cdot ft - 7 in\right)$ $\quad W_{ws\_ext} = 42.5 \cdot plf$

Unit Weight Utilities $\quad W_u := 15 plf$

$W_{DWuniform\_int} := W_{ws\_int} + W_u$ $\quad W_{DWuniform\_int} = 102.5 \cdot plf$

$W_{DWuniform\_ext} := W_{ws\_ext} + W_u$ $\quad W_{DWuniform\_ext} = 57.5 \cdot plf$

Moments due to DW: $M_{DW\_int}(x) := \dfrac{W_{DWuniform\_int} \cdot x}{2} \cdot \left(L_{str} - x\right)$ $\quad M_{DW\_ext}(x) := \dfrac{W_{DWuniform\_ext} \cdot x}{2} \cdot \left(L_{str} - x\right)$

Shears due to DW: $V_{DW\_int}(x) := W_{DWuniform\_int} \cdot \left(\dfrac{L_{str}}{2} - x\right)$ $\quad V_{DW\_ext}(x) := W_{DWuniform\_ext} \cdot \left(\dfrac{L_{str}}{2} - x\right)$

8. PRECAST LIFTING WEIGHTS AND FORCES

This section addresses the construction loads for lifting the module into place. The module is lifted from four points, at some distance, D_{lift} from each end of each girder.

Distance from end of lifting point: $D_{lift} := 8.75ft$

Assume weight uniformly distributed along girder, with 30% Dynamic Dead Load Allowance:

Dynamic Dead Load Allowance: $DLIM := 30\%$

Interior Module:

Total Interior Module Weight: $W_{int} := \left(L_{str} \cdot W_{DCuniform\_int} + 3 \cdot W_{DCpoint}\right) \cdot 2 \cdot (1 + DLIM) = 52.5 \cdot kip$

Vertical force at lifting point: $F_{lift\_int} := \dfrac{W_{int}}{4} = 13.1 \cdot kip$

Equivalent distributed load: $w_{int\_IM} := \dfrac{W_{int}}{\left(2 \cdot L_{str}\right)} = 664.4 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_int} := -w_{int\_IM} \cdot \dfrac{\left(D_{lift}^{\ 2}\right)}{2}$ $\quad M_{lift\_neg\_max\_int} = -25.4 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting:

$$M_{lift\_pos\_max\_int} := \left| \begin{array}{l} 0 \ \text{ if } \ \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} < 0 \\ \dfrac{w_{int\_IM} \cdot \left(L_{str} - 2 \cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_int} \end{array} \right.$$

$M_{lift\_pos\_max\_int} = 14.8 \cdot kip \cdot ft$

Exterior Module:

Total Exterior Module Weight: $W_{ext} := \left(L_{str} \cdot W_{DCuniform\_ext} + 3 \cdot W_{DCpoint} + W_{barrier} \cdot L_{str}\right) \cdot 2 \cdot (1 + DLIM) = 98.1 \cdot kip$

Vertical force at lifting point: $F_{lift\_ext} := \dfrac{W_{ext}}{4} = 24.5 \cdot kip$

Equivalent distributed load: $w_{ext\_IM} := \dfrac{W_{ext}}{2 \cdot L_{str}} = 1242.2 \cdot plf$

Min (Neg.) Moment during lifting: $M_{lift\_neg\_max\_ext} := -w_{ext\_IM} \cdot \dfrac{D_{lift}^{\ 2}}{2}$ $\quad M_{lift\_neg\_max\_ext} = -47.6 \cdot kip \cdot ft$

Max (Pos.) Moment during lifting: $M_{lift\_pos\_max\_ext} := \begin{cases} 0 \text{ if } \dfrac{w_{ext\_IM}\cdot\left(L_{str} - 2\cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} < 0 \\ \dfrac{w_{ext\_IM}\cdot\left(L_{str} - 2\cdot D_{lift}\right)^2}{8} + M_{lift\_neg\_max\_ext} \end{cases}$

$M_{lift\_pos\_max\_ext} = 27.6\cdot kip\cdot ft$

Max Shear during lifting: $V_{lift} := \max\left(w_{ext\_IM}\cdot D_{lift}, F_{lift\_ext} - w_{ext\_IM}\cdot D_{lift}\right) = 13.7\cdot kip$

9. LIVE LOAD DISTRIBUTION FACTORS

These factors represent the distribution of live load from the deck to the girders in accordance with AASHTO Section 4, and assumes the deck is fully continuous across the joints.

Girder Section Modulus: $I_{zsteel} = 2798.5\cdot in^4$

Girder Area: $A_{steel} = 24.5\cdot in^2$

Girder Depth: $d_{gird} = 26.7\cdot in$

Distance between centroid of deck and centroid of beam: $e_g := \dfrac{t_d}{2} + t_h + \dfrac{d_{gird}}{2} = 20.6\cdot in$

Modular Ratio: $n = 7$

Multiple Presence Factors: $MP_1 := 1.2$ $\quad MP_2 := 1.0$ S3.6.1.1.2-1

Interior Stringers for Moment:

One Lane Loaded: $K_g := n\cdot\left(I_{zsteel} + A_{steel}\cdot e_g^{\,2}\right) = 92319.5\cdot in^4$ S4.6.2.2.1-1

$$g_{int\_1m} := \left[0.06 + \left(\frac{spacing_{int}}{14ft}\right)^{0.4}\cdot\left(\frac{spacing_{int}}{L_{span}}\right)^{0.3}\cdot\left(\frac{K_g}{L_{span}\cdot t_d^{\,3}}\right)^{0.1}\right] = 0.268$$

Two Lanes Loaded: $$g_{int\_2m} := \left[0.075 + \left(\frac{spacing_{int}}{9.5ft}\right)^{0.6}\cdot\left(\frac{spacing_{int}}{L_{span}}\right)^{0.2}\cdot\left(\frac{K_g}{L_{span}\cdot t_d^{\,3}}\right)^{0.1}\right] = 0.323$$

Governing Factor: $g_{int\_m} := \max\left(g_{int\_1m}, g_{int\_2m}\right) = 0.323$

Interior Stringers for Sheai

One Lane Loaded: $g_{int\_1v} := \left(0.36 + \dfrac{spacing_{int}}{25ft}\right) = 0.477$

Two Lanes Loaded: $$g_{int\_2v} := \left[0.2 + \frac{spacing_{int}}{12ft} + -\left(\frac{spacing_{int}}{35ft}\right)^2\right] = 0.436$$

Governing Factor: $g_{int\_v} := \max\left(g_{int\_1v}, g_{int\_2v}\right) = 0.477$

Exterior Stringers for Moment:

One Lane Loaded: Use Lever Rule. Wheel is 2' from barrier; barrier is 2" beyond exterior stringer.

$d_e := 2in$

$L_{spa} := 4.5ft \qquad r := L_{spa} + d_e - 2ft = 2.7 \cdot ft$

$g_{ext\_1m} := MP_1 \cdot \frac{0.5r}{L_{spa}} = 0.356$

Two Lanes Loaded: $e_{2m} := 0.77 + \frac{d_e}{9.1ft} = 0.7883$

$g_{ext\_2m} := e_{2m} \cdot g_{int\_2m} = 0.254$

Governing Factor: $g_{ext\_m} := \max(g_{ext\_1m}, g_{ext\_2m}) = 0.356$

Exterior Stringers for Shear:

One Lane Loaded: Use Lever Rule.

$g_{ext\_1v} := g_{ext\_1m} = 0.356$

Two Lanes Loaded: $e_{2v} := 0.6 + \frac{d_e}{10ft} = 0.62$

$g_{ext\_2v} := e_{2v} \cdot g_{int\_2v} = 0.269$

Governing Factor: $g_{ext\_v} := \max(g_{ext\_1v}, g_{ext\_2v}) = 0.356$

FACTOR TO USE FOR SHEAR: $g_v := \max(g_{int\_v}, g_{ext\_v}) = 0.477$

FACTOR TO USE FOR MOMENT: $g_m := \max(g_{int\_m}, g_{ext\_m}) = 0.356$

10. LOAD RESULTS

Case 1: Dead Load on Steel Only (calculated in Section 7). Negative moments are zero and are not considered. Because the girder is simply supported, the maximum moment is at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder

$M_{DC1int} := M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 97.5 \cdot kip \cdot ft \qquad M_{DW1int} := 0 \cdot kip \cdot ft \qquad M_{LL1int} := 0kip \cdot ft$

$V_{DC1int} := V_{DC1\_int}(0) = 9.9 \cdot kip \qquad V_{DW1int} := 0 \cdot kip \qquad V_{LL1int} := 0 \cdot kip$

Exterior Girder

$M_{DC1ext} := M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 99.6 \cdot kip \cdot ft \qquad M_{DW1ext} := 0 \cdot kip \cdot ft \qquad M_{LL1ext} := 0 \cdot kip \cdot ft$

$V_{DC1ext} := V_{DC1\_ext}(0) = 10.1 \cdot kip \qquad V_{DW1ext} := 0 \cdot kip \qquad V_{LL1ext} := 0 \cdot kip \cdot ft$

Load Cases:

$M_{1\_STR\_I} := \max(1.25 \cdot M_{DC1int} + 1.5 \cdot M_{DW1int} + 1.75 \cdot M_{LL1int}, 1.25 \cdot M_{DC1ext} + 1.5 \cdot M_{DW1ext} + 1.75 \cdot M_{LL1ext}) = 124.5 \cdot kip \cdot$

$V_{1\_STR\_I} := \max(1.25 \cdot V_{DC1int} + 1.5 \cdot V_{DW1int} + 1.75 \cdot V_{LL1int}, 1.25 \cdot V_{DC1ext} + 1.5 \cdot V_{DW1ext} + 1.75 \cdot V_{LL1ext}) = 12.6 \cdot kip$

Case 2: Dead Load on Composite Section (calculated in Section 7). Negative moments are zero and are not considered. Again, the maximum moment occur at x = Lstr/2 and the maximum shear is at x = 0.

Interior Girder

$M_{DC2int} := M_{DC2\_int}\left(\frac{L_{str}}{2}\right) = 97.5 \cdot kip \cdot ft \qquad M_{DW2int} := 0 \cdot kip \cdot ft \qquad M_{LL2int} := 0 \cdot kip \cdot ft$

$V_{DC2int} := V_{DC2\_int}(0) = 9.9 \cdot kip \qquad V_{DW2int} := 0 \cdot kip \qquad V_{LL2int} := 0 \cdot kip$

Exterior Girder

$M_{DC2ext} := M_{DC2\_ext}\left(\frac{L_{str}}{2}\right) = 141.9 \cdot kip \cdot ft \qquad M_{DW2ext} := 0 \cdot kip \cdot ft \qquad M_{LL2ext} := 0 \cdot kip \cdot ft$

$V_{DC2ext} := V_{DC2\_ext}(0) = 14.4 \cdot kip \qquad V_{DW2ext} := 0 \cdot kip \qquad V_{LL2ext} := 0 \cdot kip$

Load Cases:

$M_{2\_STR\_I} := \max(1.25 \cdot M_{DC2int} + 1.5 \cdot M_{DW2int} + 1.75 \cdot M_{LL2int}, 1.25 \cdot M_{DC2ext} + 1.5 \cdot M_{DW2ext} + 1.75 \cdot M_{LL2ext}) = 177.4 \cdot kip \cdot$

$$V_{2\_STR\_I} := \max\left(1.25 \cdot V_{DC2int} + 1.5 \cdot V_{DW2int} + 1.75 \cdot V_{LL2int}, 1.25 \cdot V_{DC2ext} + 1.5 \cdot V_{DW2ext} + 1.75 \cdot V_{LL2ext}\right) = 18 \cdot kip$$

Case 3: Composite girders are lifted into place from lifting points located distance D_{lift} from the girder edges. Maximum moments and shears were calculated in Section 8.

Interior Girder

$M_{DC3int} := M_{lift\_pos\_max\_int} = 14.8 \cdot kip \cdot ft$ $\quad M_{DW3int} := 0 \cdot kip \cdot ft$ $\quad M_{LL3int} := 0 \cdot kip \cdot ft$

$M_{DC3int\_neg} := \left|M_{lift\_neg\_max\_int}\right| = 25.4 \cdot kip \cdot ft$ $\quad M_{DW3int\_neg} := 0 \cdot kip \cdot ft$ $\quad M_{LL3int\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3int} := V_{lift} = 13.7 \cdot kip$ $\quad V_{DW3int} := 0 \cdot kip$ $\quad V_{LL3int} := 0 \cdot kip$

Exterior Girder

$M_{DC3ext} := M_{lift\_pos\_max\_ext} = 27.6 \cdot kip \cdot ft$ $\quad M_{DW3ext} := 0 \cdot kip \cdot ft$ $\quad M_{LL3ext} := 0 \cdot kip \cdot ft$

$M_{DC3ext\_neg} := \left|M_{lift\_neg\_max\_ext}\right| = 47.6 \cdot kip \cdot ft$ $\quad M_{DW3ext\_neg} := 0 \cdot kip \cdot ft$ $\quad M_{LL3ext\_neg} := 0 \cdot kip \cdot ft$

$V_{DC3ext} := V_{lift} = 13.7 \cdot kip$ $\quad V_{DW3ext} := 0 \cdot kip$ $\quad V_{LL3ext} := 0 \cdot kip$

Load Cases:

$$M_{3\_STR\_I} := \max\left(1.5 \cdot M_{DC3int} + 1.5 \cdot M_{DW3int}, 1.5 \cdot M_{DC3ext} + 1.5 \cdot M_{DW3ext}\right) = 41.4 \cdot kip \cdot ft$$

$$M_{3\_STR\_I\_neg} := \max\left(1.5 \cdot M_{DC3int\_neg} + 1.5 \cdot M_{DW3int\_neg}, 1.5 \cdot M_{DC3ext\_neg} + 1.5 \cdot M_{DW3ext\_neg}\right) = 71.3 \cdot kip \cdot ft$$

$$V_{3\_STR\_I} := \max\left(1.5 \cdot V_{DC3int} + 1.5 \cdot V_{DW3int}, 1.5 \cdot V_{DC3ext} + 1.5 \cdot V_{DW3ext}\right) = 20.5 \cdot kip$$

Case 4: Composite girders made continuous. Utilities and future wearing surface are applied, and live load. Maximum moment and shear results are from a finite element analysis not included in this design example. The live load value includes the lane fraction calculated in Section 9, and impact.

Governing Loads:

$M_{DC4} := 93.4 \cdot kip \cdot ft$ $\quad M_{DW4} := 13.1 \cdot kip \cdot ft$ $\quad M_{LL4} := 201.6 \cdot kip \cdot ft$

$M_{WS4} := 0 kip \cdot ft$ $\quad M_{W4} := 0 kip \cdot ft$

$M_{DC4neg} := -116.8 \cdot kip \cdot ft$ $\quad M_{DW4neg} := -16.4 \cdot kip \cdot ft$ $\quad M_{LL4neg} := -134.7 kip \cdot ft$

$M_{WS4neg} := 0 \cdot kip \cdot ft$ $\quad M_{WL4neg} := 0 \cdot kip \cdot ft$

$V_{DC} := 17.5 kip$ $\quad V_{DW} := 2.5 kip$ $\quad V_{LL} := 87.6 kip$

$$V_u := 1.25 \cdot V_{DC} + 1.5 \cdot V_{DW} + 1.75 \cdot V_{LL} \cdot g_v = 98.7 \cdot kip$$

Load Cases:

$$M_{4\_STR\_I} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.75 \cdot M_{LL4} = 489.2 \cdot kip \cdot ft$$

$$M_{4\_STR\_I\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.75 \cdot M_{LL4neg} = -406.3 \cdot kip \cdot ft$$

$$M_{4\_STR\_III} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.4 \cdot M_{WS4} = 136.4 \cdot kip \cdot ft$$

$$M_{4\_STR\_III\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.4 \cdot M_{WS4} = -170.6 \cdot kip \cdot ft$$

$$M_{4\_STR\_V} := 1.25 \cdot M_{DC4} + 1.5 \cdot M_{DW4} + 1.35 \cdot M_{LL4} + 0.4 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 408.6 \cdot kip \cdot ft$$

$$M_{4\_STR\_V\_neg} := 1.25 \cdot M_{DC4neg} + 1.5 \cdot M_{DW4neg} + 1.35 \cdot M_{LL4neg} + 0.4 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -352.4 \cdot kip \cdot ft$$

$$M_{4\_SRV\_I} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.0 \cdot M_{LL4} + 0.3 \cdot M_{WS4} + 1.0 \cdot M_{W4} = 308.1 \cdot kip \cdot ft$$

$$M_{4\_SRV\_I\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.0 \cdot M_{LL4neg} + 0.3 \cdot M_{WS4neg} + 1.0 \cdot M_{WL4neg} = -267.9 \cdot kip \cdot ft$$

$$M_{4\_SRV\_II} := 1.0 \cdot M_{DC4} + 1.0 \cdot M_{DW4} + 1.3 \cdot M_{LL4} = 368.6 \cdot kip \cdot ft$$

$$M_{4\_SRV\_II\_neg} := 1.0 \cdot M_{DC4neg} + 1.0 \cdot M_{DW4neg} + 1.3 \cdot M_{LL4neg} = -308.3 \cdot kip \cdot ft$$

11. FLEXURAL STRENGTH

The flexural resistance shall be determined as specified in LRFD Design Article 6.10.6.2. Determine Stringer Plastic Moment Capacity First.

LFRD Appendix D6 Plastic Moment

Find location of PNA:

Forces:

$P_{rt} := A_{rt} \cdot F_s = 81.4 \cdot kip$ $\quad P_s := 0.85 \cdot f_c \cdot b_{eff} \cdot t_{slab} = 1190 \cdot kip$ $\quad P_w := F_y \cdot D_w \cdot t_w = 584.2 \cdot kip$

$P_{rb} := A_{rb} \cdot F_s = 115.5 \cdot kip$ $\quad P_c := F_y \cdot b_{tf} \cdot t_{tf} = 320 \cdot kip$ $\quad P_t := F_y \cdot b_{bf} \cdot t_{bf} = 320 \cdot kip$

$$PNA_{pos} := \begin{cases} \text{"case 1"} & \text{if } (P_t + P_w) \ge (P_c + P_s + P_{rt} + P_{rb}) \\ \text{otherwise} & \begin{cases} \text{"case 2"} & \text{if } [(P_t + P_w + P_c) \ge (P_s + P_{rt} + P_{rb})] \\ \text{otherwise} & \begin{cases} \text{"case 3"} & \text{if } \left[(P_t + P_w + P_c) \ge \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt} + P_{rb}\right)\right] \\ \text{otherwise} & \begin{cases} \text{"case 4"} & \text{if } \left[(P_t + P_w + P_c + P_{rb}) \ge \left(\frac{c_{rb}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} & \begin{cases} \text{"case 5"} & \text{if } \left[(P_t + P_w + P_c + P_{rb}) \ge \left(\frac{c_{rt}}{t_{slab}} \cdot P_s + P_{rt}\right)\right] \\ \text{otherwise} & \begin{cases} \text{"case 6"} & \text{if } (P_t + P_w + P_c + P_{rb} + P_{rt}) \ge \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \\ \text{"case 7"} & \text{if } (P_t + P_w + P_c + P_{rb} + P_{rt}) \le \left(\frac{c_{rt}}{t_{slab}} \cdot P_s\right) \text{ otherwise} \end{cases} \end{cases} \end{cases} \end{cases} \end{cases} \end{cases}$$

$PNA_{pos} = \text{"case 3"}$

$$PNA_{neg} := \begin{cases} \text{"case 1"} & \text{if } (P_c + P_w) \ge (P_t + P_{rt} + P_{rb}) \\ \text{"case 2"} & \text{if } [(P_t + P_w + P_c) \ge (P_{rt} + P_{rb})] \text{ otherwise} \end{cases}$$

$PNA_{neg} = \text{"case 1"}$

Calculate Y, Dp, and Mp: $D := D_w$ $t_s := t_{slab}$ $t_h := 0$ $C_{rt} := c_{rt}$ $C_{rb} := c_{rb}$

Case I : Plastic Nuetral Axis in the Steel Web

$$Y_1 := \frac{D}{2} \cdot \left(\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right)$$

$$D_{P1} := t_s + t_h + t_{tf} + Y_1$$

$$M_{P1} := \frac{P_w}{2D} \cdot \left[{Y_1}^2 + (D - Y_1)^2\right] + \left[P_s \cdot \left(Y_1 + \frac{t_s}{2} + t_{tf} + t_h\right) + P_{rt} \cdot (t_s - C_{rt} + t_{tf} + Y_1 + t_h) + P_{rb} \cdot (t_s - C_{rb} + t_{tf} + Y_1 + t_h) \ldots \\ + P_c \cdot \left(Y_1 + \frac{t_{tf}}{2}\right) + P_t \cdot \left(D - Y_1 + \frac{t_{bf}}{2}\right)\right]$$

$$Y_{1neg} := \left(\frac{D}{2}\right) \cdot \left[1 + \frac{(P_c - P_t - P_{rt} - P_{rb})}{P_w}\right]$$

$$D_{p1neg} := t_s + t_h + t_{tf} + Y_{1neg}$$

$$D_{CP1neg} := \left(\frac{D}{2 \cdot P_w}\right) \cdot (P_t + P_w + P_{rb} + P_{rt} - P_c)$$

$$M_{p1neg} := \left[\left(\frac{P_w}{2 \cdot D}\right) \cdot \left[Y_{1neg}^{\;2} + \left(D_w - Y_{1neg}\right)^2\right] + P_{rt} \cdot \left(t_s - C_{rt} + t_{tf} + Y_{1neg} + t_h\right) + P_{rb} \cdot \left(t_s - C_{rb} + t_{tf} + Y_{1neg} + t_h\right) \ldots \\ + P_t \cdot \left(D - Y_{1neg} + \frac{t_{bf}}{2}\right) + P_c \cdot \left(Y_{1neg} + \frac{t_{tf}}{2}\right)\right]$$

Case II: **Plastic Nuetral Axis in the Steel Top Flange**

$$Y_2 := \frac{t_{tf}}{2} \cdot \left(\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right) \qquad D_{P2} := t_s + t_h + Y_2$$

$$M_{P2} := \frac{P_c}{2t_{tf}} \cdot \left[Y_2^{\;2} + \left(t_{tf} - Y_2\right)^2\right] + \left[P_s \cdot \left(Y_2 + \frac{t_s}{2} + t_h\right) + P_{rt} \cdot \left(t_s - C_{rt} + t_h + Y_2\right) + P_{rb} \cdot \left(t_s - C_{rb} + t_h + Y_2\right) \ldots \\ + P_w \cdot \left(\frac{D}{2} + t_{tf} - Y_2\right) + P_t \cdot \left(D - Y_2 + \frac{t_{bf}}{2} + t_{tf}\right)\right]$$

$$Y_{2neg} := \left(\frac{t_{tf}}{2}\right) \cdot \left[1 + \frac{\left(P_w + P_c - P_{rt} - P_{rb}\right)}{P_t}\right] \qquad D_{P2neg} := t_s + t_h + Y_{2neg} \qquad D_{CP2neg} := D$$

$$M_{p2neg} := \left(\frac{P_t}{2 \cdot t_{tf}}\right) \cdot \left[Y_{2neg}^{\;2} + \left(t_{tf} - Y_{2neg}\right)^2\right] + \left[P_{rt} \cdot \left(t_s - C_{rt} + t_h + Y_{2neg}\right) + P_{rb} \cdot \left(t_s - C_{rb} + t_h + Y_{2neg}\right) \ldots \\ + P_w \cdot \left(t_{tf} - Y_{2neg} + \frac{D}{2}\right) + P_c \cdot \left(\left|t_s + t_h - Y_{2neg} + \frac{t_{tf}}{2}\right|\right)\right]$$

Case III: **Plastic Nuetral Axis in the Concrete Deck Below the Bottom Reinforcing**

$$Y_3 := t_s \cdot \left(\frac{P_c + P_w + P_t - P_{rt} - P_{rb}}{P_s}\right) \qquad D_{P3} := Y_3$$

$$M_{P3} := \frac{P_s}{2t_s} \cdot \left(Y_3^{\;2}\right) + \left[P_{rt} \cdot \left(Y_3 - C_{rt}\right) + P_{rb} \cdot \left(C_{rb} - Y_3\right) + P_c \cdot \left(\frac{t_{tf}}{2} + t_s + t_h - Y_3\right) + P_w \cdot \left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_3\right) \ldots \\ + P_t \cdot \left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_3\right)\right]$$

Case IV: **Plastic Nuetral Axis in the Concrete Deck in the bottom reinforcing layer**

$$Y_4 := C_{rb} \qquad D_{P4} := Y_4$$

$$M_{P4} := \frac{P_s}{2t_s} \cdot \left(Y_4^{\;2}\right) + \left[P_{rt} \cdot \left(Y_4 - C_{rt}\right) + P_c \cdot \left(\frac{t_{tf}}{2} + t_h + t_s - Y_4\right) + P_w \cdot \left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_4\right) \ldots \\ + P_t \cdot \left(D + \frac{t_{bf}}{2} + t_{tf} + t_h + t_s - Y_4\right)\right]$$

Case V: **Plastic Nuetral Axis in the Concrete Deck between top and bot reinforcing layers**

$$Y_5 := t_s \cdot \left(\frac{P_{rb} + P_c + P_w + P_t - P_{rt}}{P_s}\right) \qquad D_{P5} := Y_5$$

$$M_{P5} := \frac{P_s}{2t_s} \cdot \left(Y_5^{\;2}\right) + \left[P_{rt} \cdot \left(Y_5 - C_{rt}\right) + P_{rb} \cdot \left[\left(t_s - C_{rb}\right) - Y_5\right] + P_c \cdot \left(\frac{t_{tf}}{2} + t_s + t_h - Y_5\right) + P_w \cdot \left(\frac{D}{2} + t_{tf} + t_h + t_s - Y_5\right) \ldots \\ + P_t \cdot \left(D + \frac{t_{bf}}{2} + t_{tf} + t_s + t_h - Y_5\right)\right]$$

$$Y_{pos} := \begin{vmatrix} Y_1 \text{ if } PNA_{pos} = \text{"case 1"} \\ Y_2 \text{ if } PNA_{pos} = \text{"case 2"} \\ Y_3 \text{ if } PNA_{pos} = \text{"case 3"} \\ Y_4 \text{ if } PNA_{pos} = \text{"case 4"} \\ Y_5 \text{ if } PNA_{pos} = \text{"case 5"} \end{vmatrix} \quad D_{Ppos} := \begin{vmatrix} D_{P1} \text{ if } PNA_{pos} = \text{"case 1"} \\ D_{P2} \text{ if } PNA_{pos} = \text{"case 2"} \\ D_{P3} \text{ if } PNA_{pos} = \text{"case 3"} \\ D_{P4} \text{ if } PNA_{pos} = \text{"case 4"} \\ D_{P5} \text{ if } PNA_{pos} = \text{"case 5"} \end{vmatrix} \quad M_{Ppos} := \begin{vmatrix} M_{P1} \text{ if } PNA_{pos} = \text{"case 1"} \\ M_{P2} \text{ if } PNA_{pos} = \text{"case 2"} \\ M_{P3} \text{ if } PNA_{pos} = \text{"case 3"} \\ M_{P4} \text{ if } PNA_{pos} = \text{"case 4"} \\ M_{P5} \text{ if } PNA_{pos} = \text{"case 5"} \end{vmatrix}$$

$Y_{pos} = 6.9 \cdot in$ $\quad D_{Ppos} = 6.9 \cdot in$ $\quad M_{Ppos} = 1781.7 \cdot kip \cdot ft$

Dp = distance from the top of slab of composite section to the neutral axis at the plastic moment (neglect positive moment reinforcement in the slab).

$$Y_{neg} := \begin{vmatrix} Y_{1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ Y_{2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{vmatrix} \quad D_{Pneg} := \begin{vmatrix} D_{p1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ D_{P2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{vmatrix} \quad M_{Pneg} := \begin{vmatrix} M_{p1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ M_{p2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{vmatrix}$$

$Y_{neg} = 8.4 \cdot in$ $\quad D_{Pneg} = 17.1 \cdot in$ $\quad M_{Pneg} = 14864.2 \cdot kip \cdot in$

Depth of web in compression at the plastic moment [D6.3.2]:

$A_t := b_{bf} \cdot t_{bf} \qquad A_c := b_{tf} \cdot t_{tf}$

$$D_{cppos} := \frac{D}{2}\left(\frac{F_y \cdot A_t - F_y \cdot A_c - 0.85 \cdot f_c \cdot A_{slab} - F_s \cdot A_r}{F_y \cdot A_w} + 1\right)$$

$$D_{cppos} := \begin{vmatrix} (0in) \text{ if } PNA_{pos} \neq \text{"case 1"} \\ (0in) \text{ if } (D_{cppos} < 0) \\ D_{cppos} \text{ if } PNA_{pos} = \text{"case 1"} \end{vmatrix} \qquad D_{cpneg} := \begin{vmatrix} D_{CP1neg} \text{ if } PNA_{neg} = \text{"case 1"} \\ D_{CP2neg} \text{ if } PNA_{neg} = \text{"case 2"} \end{vmatrix}$$

$D_{cpneg} = 17 \cdot in$

$D_{cppos} = 0 \cdot in$

Positive Flexural Compression Check:

From LRFD Article 6.10.2

Check for compactness:

Web Proportions: $\frac{D_w}{t_w} \le 150 = 1$

Web slenderness Limit: $2 \cdot \frac{D_{cppos}}{t_w} \le 3.76 \cdot \sqrt{\frac{E_s}{F_y}} = 1$ S 6.10.6.2.2

Therefore Section is considered compact and shall satisfy the requirements of Article 6.10.7.1.

$$M_n := \begin{vmatrix} M_{Ppos} \text{ if } D_{Ppos} \le 0.1 \cdot D_t \\ M_{Ppos} \cdot \left(1.07 - 0.7 \cdot \frac{D_{Ppos}}{D_t}\right) \text{ otherwise} \end{vmatrix} \qquad M_n = 1658 \cdot kip \cdot ft$$

Negative Moment Capacity Check (Appendix A6):

Web Slenderness: $D_t = 34.7 \cdot in \qquad D_{cneg} := D_t - y_{cr} - t_{bf} = 22.1 \cdot in$

$$\frac{2 \cdot D_{cneg}}{t_w} < 5.7 \cdot \sqrt{\frac{E_s}{F_y}} = 1$$

S Appendix A6 (for skew less than 20 deg).

Moment ignoring concrete:

$M_{yt} := F_y \cdot S_{botcr} = 6620.9 \cdot kip \cdot in$ $\qquad$ $M_{yc} := F_s \cdot S_{topcr} = 21230.4 \cdot kip \cdot in$

$M_y := \min(M_{yc}, M_{yt}) = 6620.9 \cdot kip \cdot in$

Web Compactness:

Check for Permanent Deformations (6.10.4.2):

$$D_n := \max(t_{slab} + t_{tf} + D_w - y_c, y_c - t_{slab} - t_{tf}) = 23.7 \cdot in$$

$$Gov := if(y_c - t_{slab} - t_{tf}, y_c - c_{rt}, D_n) = 6.9 \cdot in$$

$$f_n := \left|M_{4\_SRV\_II\_neg}\right| \cdot \frac{Gov}{I_z} = 3.3 \cdot ksi$$ Steel stress on side of Dn

$$\rho := \min\left(1.0, \frac{F_y}{f_n}\right) = 1 \qquad \beta := 2 \cdot D_n \cdot \frac{t_w}{A_{tf}} = 3.4 \qquad R_h := \frac{\left[12 + \beta \cdot \left(3\rho - \rho^3\right)\right]}{(12 + 2 \cdot \beta)} = 1$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_s}{F_y}}$$

$$\lambda_{PWdcp} := \min\left[\lambda_{rw} \cdot \frac{D_{cpneg}}{D_{cneg}}, \frac{\sqrt{\frac{E_s}{F_y}}}{\left(0.54 \cdot \frac{M_{Pneg}}{R_h \cdot M_y} - 0.09\right)^2}\right] = 19.1$$

$$2 \cdot \frac{D_{cpneg}}{t_w} \le \lambda_{PWdcp} = 0$$

Web Plastification: $\quad R_{pc} := \frac{M_{Pneg}}{M_{yc}} = 0.7 \qquad R_{pt} := \frac{M_{Pneg}}{M_{yt}} = 2.2$

Flexure Factor: $\quad \phi_f := 1.0$

Tensile Limit: $\quad M_{r\_neg\_t} := \phi_f \cdot R_{pt} \cdot M_{yt} = 1238.7 \cdot kip \cdot ft$

Compressive Limit:

Local Buckling Resistance:

$$\lambda_f := \frac{b_{bf}}{2 \cdot t_{bf}} = 7.8 \qquad \lambda_{rf} := 0.95 \cdot \sqrt{0.76 \cdot \frac{E_s}{F_y}} = 19.9$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} = 9.2 \qquad F_{yresid} := \max\left(\min\left(0.7 \cdot F_y, R_h \cdot F_y \cdot \frac{S_{topcr}}{S_{botcr}}, F_y\right), 0.5 \cdot F_y\right) = 35.0 \cdot ksi$$

$$M_{ncLB} := \begin{cases} (R_{pc} \cdot M_{yc}) & \text{if } \lambda_f \le \lambda_{pf} \\ \left[R_{pc} \cdot M_{yc} \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{topcr}}{R_{pc} \cdot M_{yc}}\right)\left(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]\right] & \text{otherwise} \end{cases} \qquad M_{ncLB} = 1238.7 \cdot kip \cdot ft$$

Lateral Torsional Buckling Resistance:

$$L_b := \frac{(L_{str})}{2 \cdot 3} = 6.6 \cdot ft$$ Inflection point assumed to be at 1/6 span

$$r_t := \frac{b_{bf}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_{cneg} \cdot t_w}{b_{bf} \cdot t_{bf}}\right)}} = 2.3 \cdot in$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_s}{F_y}} = 56.2 \cdot in \qquad h := D + t_{bf} = 26 \cdot in \qquad C_b := 1.0$$

$$J_b := \frac{D \cdot t_w^{\ 3}}{3} + \frac{b_{bf} \cdot t_{bf}^{\ 3}}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{bf}}{b_{bf}}\right) + \frac{b_{tf} \cdot t_{tf}^{\ 3}}{3} \cdot \left(1 - 0.63 \cdot \frac{t_{tf}}{b_{tf}}\right) = 2.5 \cdot in^4$$

$$L_r := 1.95 \cdot r_t \cdot \frac{E_s}{F_{yresid}} \cdot \sqrt{\frac{J_b}{S_{botcr} \cdot h}} \cdot \sqrt{1 + \sqrt{1 + 6.76 \cdot \left(\frac{F_{yresid}}{E_s} \cdot \frac{S_{botcr} \cdot h}{J_b}\right)^2}} = 236.9 \cdot in$$

$$F_{cr} := \frac{C_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J_b}{S_{botcr} \cdot h} \cdot \left(\frac{L_b}{r_t}\right)^2} = 257.9 \cdot ksi$$

$$M_{ncLTB} := \begin{cases} \left(R_{pc} \cdot M_{yc}\right) & \text{if } L_b \le L_p \\ \min\left[C_b \cdot \left[1 - \left(1 - \frac{F_{yresid} \cdot S_{botcr}}{R_{pc} \cdot M_{yc}}\right) \cdot \frac{\left(L_b - L_p\right)}{\left(L_r - L_p\right)}\right] \cdot R_{pc} \cdot M_{yc}, R_{pc} \cdot M_{yc}\right] & \text{if } L_p < L_b \le L_r \\ \min\left(F_{cr} \cdot S_{botcr}, R_{pc} \cdot M_{yc}\right) & \text{if } L_b > L_r \end{cases}$$

$$M_{ncLTB} = 1131.2 \cdot kip \cdot ft$$

$$M_{r\_neg\_c} := \phi_f \cdot \min\left(M_{ncLB}, M_{ncLTB}\right) = 1131.2 \cdot kip \cdot ft$$

Governing negative moment capacity: $M_{r\_neg} := \min\left(M_{r\_neg\_t}, M_{r\_neg\_c}\right) = 1131.2 \cdot kip \cdot ft$

12. FLEXURAL STRENGTH CHECKS

Phase 1: First, check the stress due to the dead load on the steel section only. (LRFD 6.10.3 - Constructability Requirements

Reduction factor for construction $\phi_{const} := 0.9$

Load Combination for construction $1.25 \cdot M_{DC}$

Max Moment applied, Phase 1: (at midspan)

$$M_{int\_P1} := 1.25\, M_{DC1\_int}\left(\frac{L_{str}}{2}\right) = 121.9 \cdot kip \cdot ft \quad \text{(Interior)}$$

$$M_{ext\_P1} := 1.25\, M_{DC1\_ext}\left(\frac{L_{str}}{2}\right) = 124.5 \cdot kip \cdot ft \quad \text{(Exterior)}$$

Maximum Stress, Phase 1:

$$f_{int\_P1} := \frac{M_{int\_P1} \cdot y_{steel}}{I_{zsteel}} = 7 \cdot ksi \quad \text{(Interior)}$$

$$f_{ext\_P1} := \frac{M_{ext\_P1} \cdot y_{steel}}{I_{zsteel}} = 7.1 \cdot ksi \quad \text{(Exterior)}$$

Stress limits:

$$f_{P1\_max} := \phi_{const} \cdot F_y$$

$$f_{int\_P1} \le f_{P1\_max} = 1 \qquad f_{ext\_P1} \le f_{P1\_max} = 1$$

Phase 2: Second, check the stress due to dead load on the composite section (with barriers added)

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.25 \cdot M_{DC}$

Max Moment applied, Phase 2: (at midspan) $M_{2\_STR\_I} = 177.4 \cdot kip \cdot ft$

Capacity for positive flexure: $M_n = 1658 \cdot kip \cdot ft$

Check Moment: $M_{2\_STR\_I} \le \phi_{const} \cdot M_n = 1$

Phase 3: Next, check the flexural stress on the stringer during transport and picking, to ensure no cracking.

Reduction factor for construction $\phi_{const} = 0.9$

Load Combination for construction $1.5 \cdot M_{DC}$ when dynamic construction loads are involved (Section 10).

Loads and stresses on stringer during transport and picking: $M_{3\_STR\_I\_neg} = 71.3 \cdot kip \cdot ft$

Concrete rupture stress $f_r := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.5 \cdot ksi$

Concrete stress during construction not to exceed:

$$f_{cmax} := \phi_{const} \cdot f_r = 0.5 \cdot ksi$$

$$f_{cconst} := \frac{M_{3\_STR\_I\_neg} \cdot y_c}{I_z \cdot n} = 0.2 \cdot ksi$$

$$f_{cconst} \le f_{cmax} = 1$$

Phase 4: Check flexural capacity under dead load and live load for fully installed continuous composite girders.

Strength I Load Combination $\phi_f := 1.0$

$M_{4\_STR\_I} = 489.2 \cdot kip \cdot ft$ $\qquad M_{4\_STR\_I\_neg} = -406.3 \cdot kip \cdot ft$

$M_{4\_STR\_I} \le \phi_f \cdot M_n = 1$ $\qquad \left|M_{4\_STR\_I\_neg}\right| \le M_{r\_neg} = 1$

Strength III Load Combination

$M_{4\_STR\_III} = 136.4 \cdot kip \cdot ft$ $\qquad M_{4\_STR\_III\_neg} = -170.6 \cdot kip \cdot ft$

$M_{4\_STR\_III} \le \phi_f \cdot M_n = 1$ $\qquad \left|M_{4\_STR\_III\_neg}\right| \le M_{r\_neg} = 1$

Strength V Load Combination

$M_{4\_STR\_V} = 408.6 \cdot kip \cdot ft$ $\qquad M_{4\_STR\_V\_neg} = -352.4 \cdot kip \cdot ft$

$M_{4\_STR\_V} \le \phi_f \cdot M_n = 1$ $\qquad \left|M_{4\_STR\_V\_neg}\right| \le M_{r\_neg} = 1$

13. FLEXURAL SERVICE CHECKS

Check service load combinations for the fully continuous beam with live load (Phase 4):

under Service II for stress limits - $M_{4\_SRV\_II} = 368.6 \cdot kip \cdot ft$

$M_{4\_SRV\_II\_neg} = -308.3 \cdot kip \cdot ft$

under Service I for cracking - $M_{4\_SRV\_I\_neg} = -267.9 \cdot kip \cdot ft$

Ignore positive moment for Service I as there is no tension in the concrete in this case.

Service Load Stress Limits:

Top Flange: $f_{tfmax} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$

Bottom Flange: $f_{bfmax} := f_{tfmax} = 47.5 \cdot ksi$

Concrete (Negative bending only): $f_r = 0.5 \cdot ksi$

Service Load Stresses, Positive Moment:

Top Flange: $f_{SRVII\_tf} := M_{4\_SRV\_II} \cdot \frac{(y_c - t_{slab})}{I_z} = 1.4 \cdot ksi$

$f_{SRVII\_tf} \le f_{tfmax} = 1$

Bottom Flange: $f_{bfs2} := M_{4\_SRV\_II} \cdot \frac{(t_{slab} + t_{tf} + D_w + t_{bf} - y_c)}{I_z} = 14 \cdot ksi$

$f_l := 0 \qquad f_{bfs2} + \frac{f_l}{2} \le f_{bfmax} = 1$

Service Load Stresses, Negative Moment:

Top (Concrete):

$f_{con.neg} := \frac{M_{4\_SRV\_I\_neg} \cdot y_{cneg}}{n \cdot I_{zneg}} = -0.8 \cdot ksi$ Using Service I Loading

$|f_{con.neg}| \le |f_r| = 0$

Bottom Flange: $f_{bfs2.neg} := \frac{M_{4\_SRV\_I\_neg} \cdot (t_{slab} + t_{tf} + D_w + t_{bf} - y_{cneg})}{I_{zneg}} = -20.1 \cdot ksi$

$f_{bfs2.neg} \le f_{bfmax} = 1$

Check LL Deflection:

$\Delta_{DT} := 1.104 \cdot in$ from independent Analysis - includes 100% design truck (w/impact), or 25% design truck (w/impact) + 100% lane load

$DF_\delta := \frac{3}{12} = 0.3$ Deflection distribution factor = (no. lanes)/(no. stringers)

$\frac{L_{str}}{\Delta_{DT} \cdot DF_\delta} = 1717.4$ Equivalent X, where L/X = Deflection*Distribution Factor

$\frac{L_{str}}{\Delta_{DT} \cdot DF_\delta} \ge 800 = 1$

14. SHEAR STRENGTH

Shear Capacity based on AASHTO LRFD 6.10.9

Nominal resistance of unstiffened web:

$F_y = 50.0 \cdot ksi \qquad D_w = 25.4 \cdot in \qquad t_w = 0.5 \cdot in \qquad \phi_v := 1.0 \qquad k := 5$

$V_p := 0.58 \cdot F_y \cdot D_w \cdot t_w = 338.8 \cdot kip$

$$C_1 := \begin{cases} 1.0 & \text{if } \dfrac{D_w}{t_w} \le 1.12 \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}} \\ \left[\dfrac{1.57}{\left(\dfrac{D_w}{t_w}\right)^2} \cdot \left(\dfrac{E_s \cdot k}{F_y}\right)\right] & \text{if } \dfrac{D_w}{t_w} > 1.40 \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}} \\ \left(\dfrac{1.12}{\dfrac{D_w}{t_w}} \cdot \sqrt{\dfrac{E_s \cdot k}{F_y}}\right) & \text{otherwise} \end{cases} \qquad C_1 = 1$$

$$V_n := C_1 \cdot V_p = 338.8 \cdot kip$$

$$V_u \le \phi_v \cdot V_n = 1$$

15. FATIGUE LIMIT STATES:

Fatigue check shall follow LRFD Article 6.10.5. Moments used for fatigue calculations were found using an outside finite element analysis program.

First check Fatigue I (infinite life); then find maximum single lane ADTT for Fatigue II if needed.

Fatigue Stress Limits:

$\Delta F_{TH\_1} := 16 \cdot ksi$ Category B: non-coated weathering steel

$\Delta F_{TH\_2} := 12 \cdot ksi$ Category C': Base metal at toe of transverse stiffener fillet welds

$\Delta F_{TH\_3} := 10 \cdot ksi$ Category C: Base metal at shear connectors

Fatigue Moment Ranges at Detail Locations (from analysis):

$M_{FAT\_B} := 301 \cdot kip \cdot ft$ $\quad M_{FAT\_CP} := 285.7 \cdot kip \cdot ft$ $\quad M_{FAT\_C} := 207.1 kip \cdot ft$

$\gamma_{FATI} := 1.5$ $\quad \gamma_{FATII} := 0.75$ $\quad n_{fat} := \begin{cases} 2 & \text{if } L_{str} \le 40 \cdot ft \\ 1.0 & \text{otherwise} \end{cases}$

Constants to use for detail checks:

$ADTT_{SL\_INF\_B} := 860$ $\quad A_{FAT\_B} := 120 \cdot 10^8$

$ADTT_{SL\_INF\_CP} := 660$ $\quad A_{FAT\_CP} := 44 \cdot 10^8$

$ADTT_{SL\_INF\_C} := 1290$ $\quad A_{FAT\_C} := 44 \cdot 10^8$

Category B Check: Stress at Bottom Flange, Fatigue I

$$f_{FATI\_B} := \frac{\gamma_{FATI} \cdot M_{FAT\_B} \cdot \left(t_{slab} + t_{tf} + D_w + t_{bf} - y_c\right)}{I_z} = 17.2 \cdot ksi$$

$$f_{FATI\_B} \le \Delta F_{TH\_1} = 0$$

$$f_{FATII\_B} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_B} = 8.6 \cdot ksi$$

$$ADTT_{SL\_B\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_B}}{n_{fat}} & \text{if } f_{FATI\_B} \le \Delta F_{TH\_1} \\ \dfrac{A_{FAT\_B} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_B}{}^3} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_B\_MAX} = 345$$

Category C' Check: Stress at base of transverse stiffener (top of bottom flange)

$$f_{FATI\_CP} := \gamma_{FATI} \cdot M_{FAT\_CP} \cdot \frac{\left(t_{slab} + t_{tf} + D_w - y_c\right)}{I_z} = 15.9 \cdot ksi$$

$$f_{FATI\_CP} \le \Delta F_{TH\_2} = 0$$

$$f_{FATII\_CP} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_CP} = 7.9 \cdot ksi$$

$$ADTT_{SL\_CP\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_CP}}{n_{fat}} & \text{if } f_{FATI\_CP} \le \Delta F_{TH\_2} \\ \dfrac{A_{FAT\_CP} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_CP}{}^3} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_CP\_MAX} = 160$$

Category C Check: Stress at base of shear connectors (top of top flange)

$$f_{FATI\_C} := \gamma_{FATI} \cdot M_{FAT\_C} \cdot \frac{\left(y_c - t_{slab}\right)}{I_z} = 1.1 \cdot ksi$$

$$f_{FATI\_C} \le \Delta F_{TH\_3} = 1$$

$$f_{FATII\_C} := \frac{\gamma_{FATII}}{\gamma_{FATI}} \cdot f_{FATI\_C} = 0.6 \cdot ksi$$

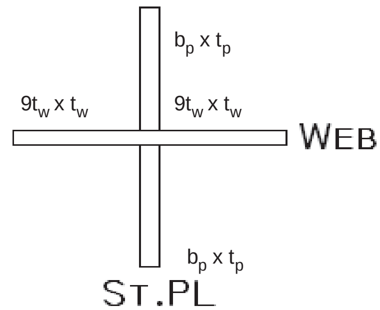
$$ADTT_{SL\_C\_MAX} := \begin{cases} \dfrac{ADTT_{SL\_INF\_C}}{n_{fat}} & \text{if } f_{FATI\_C} \le \Delta F_{TH\_3} \\ \dfrac{A_{FAT\_C} \cdot ksi^3}{365 \cdot 75 \cdot n_{fat} \cdot f_{FATII\_C}{}^3} & \text{otherwise} \end{cases} \qquad ADTT_{SL\_C\_MAX} = 645$$

FATIGUE CHECK: $ADTT_{SL\_MAX} := \min\left(ADTT_{SL\_B\_MAX}, ADTT_{SL\_CP\_MAX}, ADTT_{SL\_C\_MAX}\right)$

Ensure that single lane ADTT is less than $ADTT_{SL\_MAX} = 160$

If not, then the beam requires redesign.

16. BEARING STIFFENERS



Using LRFD Article 6.10.11 for stiffeners:

$t_p := \frac{5}{8}in$ $\quad b_p := 5in$ $\quad \phi_b := 1.0$ $\quad t_{p\_weld} := \left(\frac{5}{16}\right)in$

*Check min weld size

Projecting Width Slenderness Check:

$b_p \le 0.48t_p \cdot \sqrt{\frac{E_s}{F_y}} = 1$

Stiffener Bearing Resistance:

$A_{pn} := 2 \cdot (b_p - t_{p\_weld}) \cdot t_p$ $\quad A_{pn} = 5.9 \cdot in^2$

$R_{sb\_n} := 1.4 \cdot A_{pn} \cdot F_y$ $\quad R_{sb\_n} = 410.2 \cdot kip$

$R_{sb\_r} := \phi_b \cdot R_{sb\_n}$ $\quad R_{sb\_r} = 410.2 \cdot kip$

$R_{DC} := 26.721kip$ $\quad R_{DW} := 2.62kip$ $\quad R_{LL} := 53.943kip$

$\phi_{DC\_STR\_I} := 1.25$ $\quad \phi_{DW\_STR\_I} := 1.5$ $\quad \phi_{LL\_STR\_I} := 1.75$

$R_u := \phi_{DC\_STR\_I} \cdot R_{DC} + \phi_{DW\_STR\_I} \cdot R_{DW} + \phi_{LL\_STR\_I} \cdot R_{LL}$ $\quad R_u = 131.7 \cdot kip$

$R_u \le R_{sb\_r} = 1$

Weld Check:

$throat := t_{p\_weld} \cdot \frac{\sqrt{2}}{2}$ $\quad throat = 0.2 \cdot in$

$L_{weld} := D_w - 2 \cdot 3in$ $\quad L_{weld} = 19.4 \cdot in$

$A_{eff\_weld} := throat \cdot L_{weld}$ $\quad A_{eff\_weld} = 4.3 \cdot in^2$

$F_{exx} := 70ksi$ $\quad \phi_{e2} := 0.8$

$R_{r\_weld} := 0.6 \cdot \phi_{e2} \cdot F_{exx}$ $\quad R_{r\_weld} = 33.6 \cdot ksi$

$R_{u\_weld} := \frac{R_u}{4 \cdot A_{eff\_weld}}$ $\quad R_{u\_weld} = 7.7 \cdot ksi$

$R_{u\_weld} \le R_{u\_weld} = 1$

Axial Resistance of Bearing Stiffeners: $\quad \phi_c := 0.9$

$A_{eff} := (2 \cdot 9 \cdot t_w + t_p) \cdot t_w + 2 \cdot b_p \cdot t_p$ $\quad A_{eff} = 10.3 \cdot in^2$

$L_{eff} := 0.75 \cdot D_w$ $\quad L_{eff} = 19.1 \cdot in$

$I_{xp} := \frac{2 \cdot 9 \cdot t_w \cdot t_w^3}{12} + \frac{t_p \cdot (2 \cdot b_p + t_w)^3}{12}$ $\quad I_{xp} = 59.7 \cdot in^4$

$I_{yp} := \frac{t_w \cdot (t_p + 2 \cdot 9 \cdot t_w)^3}{12} + \frac{2b_p \cdot t_p^3}{12}$ $\quad I_{yp} = 27.3 \cdot in^4$

$r_p := \sqrt{\frac{\min(I_{xp}, I_{yp})}{A_{eff}}}$ $\quad r_p = 1.6 \cdot in$

$Q := 1$ for bearing stiffeners $\quad K_p := 0.75$

$P_o := Q \cdot F_y \cdot A_{eff} = 517.3 \cdot kip$

$$P_e := \frac{\pi^2 E_s \cdot A_{eff}}{\left(K_p \cdot \frac{L_{eff}}{r_p}\right)^2} = 38239.8 \cdot kip$$

$$P_n := \begin{cases} \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o & \text{if } \left(\frac{P_e}{P_o}\right) \geq 0.44 \\ 0.877 \cdot P_e & \text{otherwise} \end{cases} \qquad P_n = 514.4 \cdot kip$$

$P_r := \phi_c \cdot P_n$ $\quad P_r = 463 \cdot kip$ $\quad R_u \leq P_r = 1$

17. SHEAR CONNECTORS:

Shear Connector design to follow LRFD 6.10.10.

Stud Properties:

$d_s := \frac{7}{8} \cdot in$ Diameter $\quad h_s := 6in$ Height of Stud $\quad \frac{h_s}{d_s} \geq 4 = 1$

$c_s := t_{slab} - h_s$ $\quad c_s \geq 2in = 1$

$s_s := 3.5in$ Spacing $\quad s_s \geq 4d_s = 1$

$n_s := 3$ Studs per row $\quad \frac{\left[b_{tf} - s_s \cdot (n_s - 1) - d_s\right]}{2} \geq 1.0in = 1$

$A_{sc} := \pi \cdot \left(\frac{d_s}{2}\right)^2$ $\quad A_{sc} = 0.6 \cdot in^2$

$F_u := 60ksi$

Fatigue Resistance:

$Z_r := 5.5 \cdot d_s^{\ 2} \cdot \frac{kip}{in^2}$ $\quad Z_r = 4.2 \cdot kip$ $\quad Q_{slab} := A_{slab} \cdot (y_c - y_{slab})$ $\quad Q_{slab} = 253.8 \cdot in^3$

$V_f := 47.0kip$

$V_{fat} := \frac{V_f \cdot Q_{slab}}{I_z} = 1.6 \cdot \frac{kip}{in}$

$p_s := \frac{n_s \cdot Z_r}{V_{fat}} = 8.1 \cdot in$ $\quad 6 \cdot d_s \leq p_s \leq 24in = 1$

Strength Resistance:

$\phi_{sc} := 0.85$

$f_c = 5 \cdot ksi$

$E_c := 33000 \cdot 0.15^{1.5} \cdot \sqrt{f_c ksi} = 4286.8 \cdot ksi$

$Q_n := \min\left(0.5 \cdot A_{sc} \cdot \sqrt{f_c \cdot E_c}, A_{sc} \cdot F_u\right)$ $\quad Q_n = 36.1 \cdot kip$

$Q_r := \phi_{sc} \cdot Q_n$ $\quad Q_r = 30.7 \cdot kip$

$P_{simple} := \min\left(0.85 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right)$ $\quad P_{simple} = 1190 \cdot kip$

$P_{cont} := P_{simple} + \min\left(0.45 \cdot f_c \cdot b_{eff} \cdot t_s, F_y \cdot A_{steel}\right)$ $\quad P_{cont} = 1820 \cdot kip$

$n_{lines} := \frac{P_{cont}}{Q_r \cdot n_s}$ $\quad n_{lines} = 19.8$

Find required stud spacing along the girder (varies as applied shear varies)

$i := 0 .. 23$

$$x := \begin{pmatrix} 0.00 \\ 1.414 \\ 4.947 \\ 8.480 \\ 12.013 \\ 15.546 \\ 19.079 \\ 22.612 \\ 26.145 \\ 29.678 \\ 33.210 \\ 33.917 \\ 34.624 \\ 36.037 \\ 36.743 \\ 40.276 \\ 43.809 \\ 47.342 \\ 50.875 \\ 54.408 \\ 57.941 \\ 61.474 \\ 65.007 \\ 67.833 \end{pmatrix} \cdot ft \quad V_{fi} := \begin{pmatrix} 61.5 \\ 59.2 \\ 56.8 \\ 54.4 \\ 52.0 \\ 49.5 \\ 47.1 \\ 44.7 \\ 42.7 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 40.6 \\ 42.3 \\ 44.2 \\ 46.6 \\ 49.1 \\ 51.5 \\ 53.9 \\ 56.3 \\ 58.7 \\ 61.5 \end{pmatrix} \cdot kip$$

$V_{fat_i} := \dfrac{V_{fi} \cdot Q_{slab}}{I_z} =$

	0
0	2
1	2
2	1.9
3	1.8
4	1.7
5	1.6
6	1.6
7	1.5
8	1.4
9	1.3
10	1.3
11	1.3
12	1.3
13	1.3
14	1.3
15	...

$\cdot \dfrac{kip}{in}$

$P_{max} := \dfrac{n_s \cdot Z_r}{V_{fat}} =$

	0
0	6.2
1	6.4
2	6.7
3	7
4	7.3
5	7.7
6	8.1
7	8.5
8	8.9
9	9.4
10	9.4
11	9.4
12	9.4
13	9.4
14	9.4
15	...

$\cdot in$

$\min(P_{max}) = 6.2 \cdot in$

$\max(P_{max}) = 9.4 \cdot in$

18. SLAB PROPERTIES

This section details the geometric and material properties of the deck. Because the equivalent strip method is used in accordance with AASHTO LRFD Section 4, different loads are used for positive and negative bending.

Unit Weight Concrete $w_c = 150 \cdot pcf$

Deck Thickness for Design $t_{deck} := 8.0in$ $\quad t_{deck} \geq 7in = 1$

Deck Thickness for Loads $t_d = 10.5 \cdot in$

Rebar yield strength $F_s = 60 \cdot ksi$ $\quad$ Strength of concrete $f_c = 5 \cdot ksi$

Concrete clear cover

Bottom

$c_b := 1.0in$ $\quad c_b \geq 1.0in = 1$

Top

$c_t := 2.5in$ $\quad c_t \geq 2.5in = 1$

Transverse reinforcement

Bottom Reinforcing $\phi_{tb} := \frac{6}{8}in$

Bottom Spacing $s_{tb} := 8in$

$s_{tb} \geq 1.5\phi_{tb} \wedge 1.5in = 1$

$s_{tb} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$A_{stb} := \frac{12in}{s_{tb}} \cdot \pi \cdot \left(\frac{\phi_{tb}}{2}\right)^2 = 0.7 \cdot in^2$

Top Reinforcing $\phi_{tt} := \frac{5}{8}in$

Top Spacing $s_{tt} := 8in$

$s_{tt} \geq 1.5\phi_{tt} \wedge 1.5in = 1$

$s_{tt} \leq 1.5 \cdot t_{deck} \wedge 18in = 1$

$A_{stt} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 = 0.5 \cdot in^2$

Design depth of Bar $d_{tb} := t_{deck} - \left(c_b + \frac{\phi_{tb}}{2}\right) = 6.6 \cdot in$ $\quad d_{tt} := t_{deck} - \left(c_t + \frac{\phi_{tt}}{2}\right) = 5.2 \cdot in$

Girder Spacing $spacing_{int\_max} := 2ft + 11in$

$spacing_{ext} = 3\,ft$

Equivalent Strip, +M $w_{posM} := \left(26 + 6.6 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $\quad w_{posM} = 45.3 \cdot in$

Equivalent Strip, -M $w_{negM} := \left(48 + 3.0 \cdot \frac{spacing_{int\_max}}{ft}\right) \cdot in$ $\quad w_{negM} = 56.8 \cdot in$

Once the strip widths are determined, the dead loads can be calculated.

19. PERMANENT LOADS

This section calculates the dead loads on the slab. These are used later for analysis to determine the design moments.

Weight of deck, +M $w_{deck\_pos} := w_c \cdot t_d \cdot w_{posM}$ $\quad w_{deck\_pos} = 494.9 \cdot plf$

Weight of deck, -M $w_{deck\_neg} := w_c \cdot t_d \cdot w_{negM}$ $\quad w_{deck\_neg} = 620.7 \cdot plf$

Unit weight of barrier $w_b := 433.5plf$

Barrier point load, +M $P_{b\_pos} := w_b \cdot w_{posM}$ $\quad P_{b\_pos} = 1.63 \cdot kip$

Barrier point load, -M $P_{b\_neg} := w_b \cdot w_{negM}$ $\quad P_{b\_neg} = 2.05 \cdot kip$

20. LIVE LOADS

This section calculates the live loads on the slab. These loads are analyzed in a separate program with the permanent loads to determine the design moments.

Truck wheel load $P_{wheel} := 16kip$

Impact Factor $IM := 1.33$

Multiple presence factors $MP_1 := 1.2$ $\quad MP_2 := 1.0$ $\quad MP_3 := 0.85$

Wheel Loads $P_1 := IM \cdot MP_1 \cdot P_{wheel}$ $\quad P_2 := IM \cdot MP_2 \cdot P_{wheel}$ $\quad P_3 := IM \cdot MP_3 \cdot P_{wheel}$

$P_1 = 25.54 \cdot kip$ $\quad P_2 = 21.3 \cdot kip$ $\quad P_3 = 18.09 \cdot kip$

21. LOAD RESULTS

The separate MathCAD design aides (available in Appendix of the final report) was used to analyze the deck as an 11-span continuous beam without cantilevered overhangs on either end, with supports stationed at girder locations. The dead and live loads were applied separately. The results are represented here as input values, highlighted.

Design Moments

$M_{pos\_deck} := 0.4 \cdot kip \cdot ft$ $\quad M_{pos\_LL} := 15.3 \cdot kip \cdot ft$ $\quad M_{pos} := (1.25 \cdot M_{pos\_deck} + 1.75 \cdot M_{pos\_LL})$

$M_{pos} = 27.3 \cdot kip \cdot ft$ $\quad M_{pos\_dist} := \frac{M_{pos}}{w_{posM}}$ $\quad M_{pos\_dist} = 7.23 \cdot \frac{kip \cdot ft}{ft}$

$M_{neg\_deck} := -0.6 \cdot kip \cdot ft$ $\quad M_{neg\_LL} := -7.8 \cdot kip \cdot ft$ $\quad M_{neg} := (1.25 \cdot M_{neg\_deck} + 1.75 \cdot M_{neg\_LL})$

$M_{neg} = -14.4 \cdot kip \cdot ft$ $\quad M_{neg\_dist} := \frac{M_{neg}}{w_{negM}}$ $\quad M_{neg\_dist} = -3.04 \cdot \frac{kip \cdot ft}{ft}$

22. FLEXURAL STRENGTH CAPACITY CHECK:

Consider a 1'-0" strip: $\phi_b := 0.9$ $\quad b := 12in$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \le 4ksi \\ 0.85 - 0.05 \cdot \left(\frac{f_c}{ksi} - 4\right) & \text{otherwise} \end{cases} \qquad \beta_1 = 0.8$$

Bottom:

$c_{tb} := \frac{A_{stb} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1 \cdot in$

$a_{tb} := \beta_1 \cdot c_{tb} = 0.8 \cdot in$

$M_{ntb} := \frac{A_{stb} \cdot F_s}{b} \cdot \left(d_{tb} - \frac{a_{tb}}{2}\right) = 20.7 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtb} := \phi_b \cdot M_{ntb} = 18.6 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtb} \ge |M_{pos\_dist}| = 1$

Top:

$c_{tt} := \frac{A_{stt} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.7 \cdot in$

$a_{tt} := \beta_1 \cdot c_{tt} = 0.5 \cdot in$

$M_{ntt} := \frac{A_{stt} \cdot F_s}{ft} \cdot \left(d_{tt} - \frac{a_{tt}}{2}\right) = 11.3 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtt} := \phi_b \cdot M_{ntt} = 10.2 \cdot \frac{kip \cdot ft}{ft}$

$M_{rtt} \ge |M_{neg\_dist}| = 1$

23. LONGITUDINAL DECK REINFORCEMENT DESIGN:

Longitudinal reinforcement

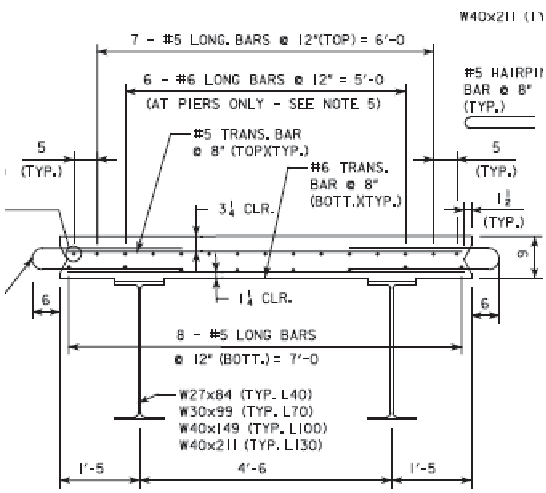
$\phi_{lb} := \frac{5}{8} in$ $\quad s_{lb} := 12in$ $\qquad \phi_{lt} := \frac{5}{8} in$ $\quad s_{lt} := 12in$

$A_{slb} := \frac{12in}{s_{lb}} \cdot \pi \cdot \left(\frac{\phi_{lb}}{2}\right)^2 = 0.3 \cdot in^2$ $\qquad A_{slt} := \frac{12in}{s_{lt}} \cdot \pi \left(\frac{\phi_{lt}}{2}\right)^2 = 0.3 \cdot in^2$

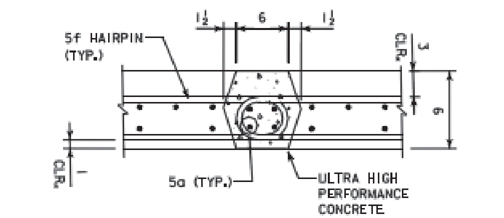
Distribution Reinforcement (AASHTO 9.7.3.2)

$$A_{\%dist} := \frac{\min\left(\frac{220}{\sqrt{\frac{spacing_{int\_max}}{ft}}}, 67\right)}{100} = 67 \cdot \%$$

$A_{dist} := A_{\%dist} \cdot (A_{stb}) = 0.4 \cdot in^2$ $\qquad A_{slb} + A_{slt} \ge A_{dist} = 1$



INTERIOR MODULE
REINFORCING DETAIL



LONGITUDINAL CLOSURE POUR DETAIL
(TRANSVERSE REINFORCEMENT NOT SHOWN FOR CLARITY)

24. DESIGN CHECKS

This section will conduct design checks on the reinforcing according to various sections in AASHTO LRFD.

CHECK MINIMUM REINFORCEMENT (AASHTO LRFD 5.7.3.3.2):

Modulus of Rupture $f_r := 0.37 \cdot \sqrt{f_c \cdot ksi} = 0.8 \cdot ksi$

$E_c = 4286.8 \cdot ksi$

Section Modulus $S_{nc} := \dfrac{b \cdot t_{deck}^{2}}{6} = 128 \cdot in^{3}$

$E_s = 29000 \cdot ksi$

$A_{deck} := t_{deck} \cdot b = 96 \cdot in^{2}$

$$y_{bar\_tb} := \frac{A_{deck} \cdot \frac{t_{deck}}{2} + (n-1) \cdot A_{stb} \cdot d_{tb}}{A_{deck} + (n-1) \cdot A_{stb}} = 4.1 \cdot in$$

$$y_{bar\_tt} := \frac{A_{deck} \cdot \frac{t_{deck}}{2} + (n-1) \cdot A_{stt} \cdot d_{tt}}{A_{deck} + (n-1) \cdot A_{stt}} = 4 \cdot in$$

$$I_{tb} := \frac{b \cdot t_{deck}^{3}}{12} + A_{deck} \cdot \left(\frac{t_{deck}}{2} - y_{bar\_tb}\right)^2 + (n-1) \cdot A_{stb} \cdot \left(d_{tb} - y_{bar\_tb}\right)^2 = 538.3 \cdot in^4$$

$$I_{tt} := \frac{b \cdot t_{deck}^{3}}{12} + A_{deck} \cdot \left(\frac{t_{deck}}{2} - y_{bar\_tt}\right)^2 + (n-1) \cdot A_{stt} \cdot \left(d_{tt} - y_{bar\_tt}\right)^2 = 515.8 \cdot in^4$$

$$S_{c\_tb} := \frac{I_{tb}}{t_{deck} - y_{bar\_tb}} = 138.2 \cdot in^3 \qquad S_{c\_tt} := \frac{I_{tt}}{t_{deck} - y_{bar\_tt}} = 130 \cdot in^3$$

Unfactored Dead Load

$$M_{dnc\_pos\_t} := 1.25 \frac{kip \cdot ft}{ft} \qquad M_{dnc\_neg\_t} := -0.542 \frac{kip \cdot ft}{ft}$$

Cracking Moment

$$M_{cr\_tb} := \max\left[\frac{S_{c\_tb} \cdot f_r}{ft} - \left|M_{dnc\_pos\_t}\right| \cdot \left(\frac{S_{c\_tb}}{S_{nc}} - 1\right), \frac{S_{c\_tb} \cdot f_r}{ft}\right] = 9.5 \cdot \frac{kip \cdot ft}{ft}$$ S 5.7.3.3.2

$$M_{cr\_tt} := \max\left[\frac{S_{c\_tt} \cdot f_r}{ft} - \left|M_{dnc\_neg\_t}\right| \cdot \left(\frac{S_{c\_tt}}{S_{nc}} - 1\right), \frac{S_{c\_tt} \cdot f_r}{ft}\right] = 9 \cdot \frac{kip \cdot ft}{ft}$$

Minimum Factored Flexural Resistance

$$M_{r\_min\_tb} := \min\left(1.2 \cdot M_{cr\_tb}, 1.33 \cdot \left|M_{pos\_dist}\right|\right) = 9.6 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtb} \ge M_{r\_min\_tb} = 1$$

$$M_{r\_min\_tt} := \min\left(1.2 \cdot M_{cr\_tt}, 1.33 \cdot \left|M_{neg\_dist}\right|\right) = 4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt} \ge M_{r\_min\_tt} = 1$$

CHECK CRACK CONTROL (AASHTO LRFD 5.7.3.4):

$$\gamma_{eb} := 1.0 \qquad \gamma_{et} := 0.75$$

$$M_{SL\_pos} := 29.64 kip \cdot ft \qquad M_{SL\_neg} := 29.64 kip \cdot ft$$

$$M_{SL\_pos\_dist} := \frac{M_{SL\_pos}}{w_{posM}} = 7.9 \cdot \frac{kip \cdot ft}{ft} \qquad M_{SL\_neg\_dist} := \frac{M_{SL\_neg}}{w_{negM}} = 6.3 \cdot \frac{kip \cdot ft}{ft}$$

$$f_{ssb} := \frac{M_{SL\_pos\_dist} \cdot b \cdot n}{\frac{I_{tb}}{d_{tb} - y_{bar\_tb}}} = 3.1 \cdot ksi \qquad f_{sst} := \frac{M_{SL\_neg\_dist} \cdot b \cdot n}{\frac{I_{tt}}{d_{tt} - y_{bar\_tt}}} = 1.2 \cdot ksi$$

$$d_{cb} := c_b + \frac{\phi_{tb}}{2} = 1.4 \cdot in \qquad d_{ct} := c_t + \frac{\phi_{tt}}{2} = 2.8 \cdot in$$

$$\beta_{sb} := 1 + \frac{d_{cb}}{0.7 \cdot \left(t_{deck} - d_{cb}\right)} = 1.3 \qquad \beta_{st} := 1 + \frac{d_{ct}}{0.7 \cdot \left(t_{deck} - d_{ct}\right)} = 1.8$$

$$s_b := \frac{700 \cdot \gamma_{eb} \cdot kip}{\beta_{sb} \cdot f_{ssb} \cdot in} - 2 \cdot d_{cb} = 171.9 \cdot in \qquad s_t := \frac{700 \cdot \gamma_{et} \cdot kip}{\beta_{st} \cdot f_{sst} \cdot in} - 2 \cdot d_{ct} = 245.5 \cdot in$$

$$s_{tb} \le s_b = 1 \qquad s_{tt} \le s_t = 1$$

SHRINKAGE AND TEMPERATURE REINFORCING (AASHTO LRFD 5.10.8):

$$A_{st} := \begin{vmatrix} \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} & \text{if } 0.11in^2 \le \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} \le 0.60in^2 \\ 0.11in^2 & \text{if } \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} < 0.11in^2 \\ 0.60in^2 & \text{if } \dfrac{1.30 \cdot b \cdot t_{deck}}{2 \cdot \left(b + t_{deck}\right) \cdot F_s} \cdot \dfrac{kip}{in} > 0.60in^2 \end{vmatrix} = 0.1 \cdot in^2$$

$A_{stb} \ge A_{st} = 1$ $\qquad$ $A_{stt} \ge A_{st} = 1$

$A_{slb} \ge A_{st} = 1$ $\qquad$ $A_{slt} \ge A_{st} = 1$

SHEAR RESISTANCE (AASHTO LRFD 5.8.3.3):

$\phi := 0.9$ $\qquad$ $\beta := 2$ $\qquad$ $\theta := 45deg$ $\qquad$ $b = 1\,ft$

$$d_{v\_tb} := \max\left(0.72 \cdot t_{deck}, d_{tb} - \frac{a_{tb}}{2}, 0.9 \cdot d_{tb}\right) = 6.2 \cdot in$$

$$d_{v\_tt} := \max\left(0.72 \cdot t_{deck}, d_{tt} - \frac{a_{tt}}{2}, 0.9 \cdot d_{tt}\right) = 5.8 \cdot in$$

$$d_v := \min\left(d_{v\_tb}, d_{v\_tt}\right) = 5.8 \cdot in$$

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_v = 9.8 \cdot kip$$

$V_s := 0kip$ Shear capacity of reinforcing steel

$V_{ps} := 0kip$ Shear capacity of prestressing steel

$$V_{ns} := \min\left(V_c + V_s + V_{ps}, 0.25 \cdot f_c \cdot b \cdot d_v + V_{ps}\right) = 9.8 \cdot kip$$

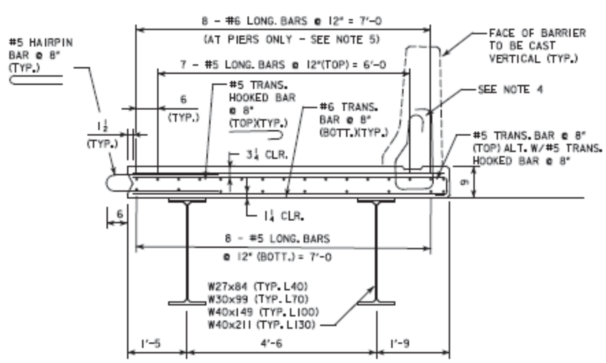
$V_r := \phi \cdot V_{ns} = 8.8 \cdot kip$ Total factored resistance

$V_{us} := 8.38kip$ Total factored load $\qquad$ $V_r \ge V_{us} = 1$

DEVELOPMENT AND SPLICE LENGTHS (AASHTO LRFD 5.11):

Development and splice length design follows standard calculations in AASHTO LRFD 5.11, or as dictated by the State DOT Design Manual.

25. DECK OVERHANG DESIGN (AASHTO LRFD A.13.4):



EXTERIOR MODULE
REINFORCING DETAIL
(SHEAR STUDS OMITTED FOR CLARITY)
(L70 SHOWN; L40, L100 AND L130 SIMILAR)

Deck Properties:

Deck Overhang Length $L_o := 1ft + 9in$

Parapet Properties:

Note: Parapet properties are per unit length. Compression reinforcement is ignored.

Cross Sectional Area	$A_p := 2.84ft^2$	Height of Parapet	$H_{par} := 2ft + 10in$
Parapet Weight	$W_{par} := w_c \cdot A_p = 426 \cdot plf$		
Width at base	$w_{base} := 1ft + 5in$	Average width of wall	$w_{wall} := \dfrac{13in + 9.5in}{2} = 11.3 \cdot in$
Height of top portion of parapet	$h_1 := 2ft$	Width at top of parapet	$width_1 := 9.5 \cdot in = 9.5 \cdot in$
Height of middle portion of parapet	$h_2 := 7in$	Width at middle transition of parapet	$width_2 := 12 \cdot in = 12 \cdot in$
Height of lower portion of parapet	$h_3 := 3in$	Width at base of parapet	$width_3 := 1ft + 5 \cdot in = 17 \cdot in$

$b_1 := width_1$ $b_2 := width_2 - width_1$ $b_3 := width_3 - width_2$

Parapet Center of Gravity

$$CG_p := \frac{\left(h_1 + h_2 + h_3\right) \cdot \frac{b_1^2}{2} + \frac{1}{2} \cdot h_1 \cdot b_2 \cdot \left(b_1 + \frac{b_2}{3}\right) \ldots + \left(h_2 + h_3\right) \cdot \left(b_2 + b_3\right) \cdot \left(b_1 + \frac{b_2 + b_3}{2}\right) - \frac{1}{2} \cdot h_2 \cdot b_3 \cdot \left(b_1 + b_2 + \frac{2b_3}{3}\right)}{\left(h_1 + h_2 + h_3\right) \cdot b_1 + \frac{1}{2} \cdot h_1 \cdot b_2 + \left(h_2 + h_3\right) \cdot \left(b_2 + b_3\right) - \frac{1}{2} \cdot h_2 \cdot b_3} = 6.3 \cdot in$$

Parapet Reinforcement	Vertically Aligned Bars in Wall	Horizontal Bars
Rebar spacing:	$s_{pa} := 12in$	$n_{pl} := 5$
Rebar Diameter:	$\phi_{pa} := \frac{5}{8} in$	$\phi_{pl} := \frac{5}{8} in$
Rebar Area:	$A_{st\_p} := \pi \cdot \left(\frac{\phi_{pa}}{2}\right)^2 \cdot \frac{b}{s_{pa}} = 0.3 \cdot in^2$	$A_{sl\_p} := \pi \cdot \left(\frac{\phi_{pl}}{2}\right)^2 = 0.3 \cdot in^2$
Cover:	$c_{st} := 3in$	$c_{sl} := 2in + \phi_{pa} = 2.6 \cdot in$
Effective Depth:	$d_{st} := w_{base} - c_{st} - \frac{\phi_{pa}}{2} = 13.7 \cdot in$	$d_{sl} := w_{wall} - c_{sl} - \frac{\phi_{pl}}{2} = 8.3 \cdot in$
Parapet Moment Resistance About Horizontal Axis:	$\phi_{ext} := 1.0$	
Depth of Equivalent Stress Block:	$a_h := \dfrac{A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot b} = 0.4 \cdot in$	S 5.7.3.1.2-4 S 5.7.3.2.3

Moment Capacity of Upper Segment of Barrier (about longitudinal axis):

Average width of section $w_1 := \dfrac{width_1 + width_2}{2} = 10.7 \cdot in$

Cover $c_{st1} := 2in$

Depth $d_{h1} := w_1 - c_{st1} - \dfrac{\phi_{pa}}{2} = 8.4 \cdot in$

Factored Moment Resistance

$$\phi M_{nh1} := \frac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h1} - \frac{a_h}{2}\right)}{b} = 12.7 \cdot \frac{kip \cdot ft}{ft}$$

Moment Capacity of Middle Segment of Barrier (about longitudinal axis):

Average width of section $w_2 := \dfrac{width_2 + width_3}{2} = 14.5 \cdot in$

Cover $c_{st2} := 3in$

Depth $d_{h2} := w_2 - c_{st2} - \dfrac{\phi_{pa}}{2} = 11.2 \cdot in$

Factored Moment Resistance $\phi M_{nh2} := \dfrac{\phi_{ext} \cdot A_{st\_p} \cdot F_s \cdot \left(d_{h2} - \dfrac{a_h}{2}\right)}{b} = 16.9 \cdot \dfrac{kip \cdot ft}{ft}$

Parapet Base Moment Resistance (about longitudinal axis):

development in tension $c_{st3} := 3in$ $\qquad cover_{base\_vert} := c_{st3} + \dfrac{\phi_{pa}}{2} = 3.3 \cdot in$

$$m_{inc\_ta} := \begin{cases} 1.5 & \text{if } c_{st3} < 3 \cdot \phi_{pa} \vee s_{pa} - \phi_{pa} < 6 \cdot \phi_{pa} \\ 1.2 & \text{otherwise} \end{cases} = 1.2$$

$$m_{dec\_ta} := \begin{cases} 0.8 & \text{if } s_{pa} \ge 6in \\ 1.0 & \text{otherwise} \end{cases} = 0.8$$

$$l_{db\_ta} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{st\_p} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{pa} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{pa} \le \dfrac{11}{8} in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{pa} = \dfrac{14}{8} in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{pa} = \dfrac{18}{8} in \end{cases}$$

$l_{dt\_ta} := l_{db\_ta} \cdot m_{inc\_ta} \cdot m_{dec\_ta} = 14.4 \cdot in$

hooked bar developed in tension $l_{hb\_ta} := \dfrac{38 \cdot \phi_{pa}}{\sqrt{\dfrac{f_c}{ksi}}} = 10.6 \cdot in$ $\qquad m_{inc} = 1.2$

$l_{dh\_ta} := \max(6in, 8 \cdot \phi_{pa}, m_{inc} \cdot l_{hb\_ta}) = 12.7 \cdot in$

lap splice in tension $l_{lst\_ta} := \max(12in, 1.3 \cdot l_{dt\_ta}) = 18.7 \cdot in$

$benefit := l_{dt\_ta} - l_{dh\_ta} = 1.7 \cdot in$

$l_{dev\_a} := \left(7 + \dfrac{13}{16}\right) in$

$F_{dev} := \dfrac{benefit + l_{dev\_a}}{l_{dt\_ta}} = 0.7$

$F_d := 0.75$

Distance from NA to Compressive Face $c_{t\_b} := \dfrac{F_d \cdot A_{st\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.3 \cdot in$ $\qquad$ S 5.7.3.1.2-4

Depth of Equivalent Stress Block $a_t := \beta_1 \cdot c_{t\_b} = 0.3 \cdot in$ S 5.7.3.2.3

Nominal Moment Resistance $M_{nt} := F_d \cdot A_{st\_p} \cdot F_s \cdot \left(d_{st} - \frac{a_t}{2}\right) = 15.6 \cdot kip \cdot ft$ S 5.7.3.2.2-1

Factored Moment Resistance $M_{cb} := \phi_{ext} \cdot \frac{M_{nt}}{ft} = 15.6 \cdot \frac{kip \cdot ft}{ft}$ S 5.7.3.2

Average Moment Capacity of Barrier (about longitudinal axis):

Factored Moment Resistance about Horizontal Axis $M_c := \frac{\phi M_{nh1} \cdot h_1 + \phi M_{nh2} \cdot h_2 + M_{cb} \cdot h_3}{h_1 + h_2 + h_3} = 13.8 \cdot \frac{kip \cdot ft}{ft}$

Parapet Moment Resistance (about vertical axis):

Height of Transverse Reinforcement in Parapet

Width of Parapet at Transverse Reinforcement

$y_1 := 5in$ $\quad x_1 := width_3 - \frac{(y_1 - h_3) \cdot b_3}{h_2} = 15.6 \cdot in$

$y_2 := 11.5in$ $\quad x_2 := b_1 + b_2 - \frac{(y_2 - h_3 - h_2) \cdot b_2}{h_1} = 11.8 \cdot in$

$y_3 := 18in$ $\quad x_3 := b_1 + b_2 - \frac{(y_3 - h_3 - h_2) \cdot b_2}{h_1} = 11.2 \cdot in$

$y_4 := 24.5in$ $\quad x_4 := b_1 + b_2 - \frac{(y_4 - h_3 - h_2) \cdot b_2}{h_1} = 10.5 \cdot in$

$y_5 := 31in$ $\quad x_5 := b_1 + b_2 - \frac{(y_5 - h_3 - h_2) \cdot b_2}{h_1} = 9.8 \cdot in$

Depth of Equivalent Stress Block $a := \frac{n_{pl} \cdot A_{sl\_p} \cdot F_s}{0.85 \cdot f_c \cdot H_{par}} = 0.6 \cdot in$

Concrete Cover in Parapet $cover_r := 2in$ $\quad cover_{rear} := cover_r + \phi_{pa} + \frac{\phi_{pl}}{2} = 2.9 \cdot in$

$cover_{base} := c_{st3} + \phi_{pa} + \frac{\phi_{pl}}{2} = 3.9 \cdot in$

$cover_f := 2in$ $\quad cover_{front} := 2in + \phi_{pa} + \frac{\phi_{pl}}{2}$

$cover_t := \frac{x_5}{2} = 4.9 \cdot in$ $\quad cover_{top} := cover_t = 4.9 \cdot in$

Design depth

$d_{1i} := x_1 - cover_{base} = 11.6 \cdot in$ $\quad d_{1o} := x_1 - cover_{rear} = 12.6 \cdot in$

$d_{2i} := x_2 - cover_{front} = 8.9 \cdot in$ $\quad d_{2o} := x_2 - cover_{rear} = 8.9 \cdot in$

$d_{3i} := x_3 - cover_{front} = 8.2 \cdot in$ $\quad d_{3o} := x_3 - cover_{rear} = 8.2 \cdot in$

$d_{4i} := x_4 - cover_{front} = 7.6 \cdot in$ $\quad d_{4o} := x_4 - cover_{rear} = 7.6 \cdot in$

$d_{5i} := x_5 - cover_{top} = 4.9 \cdot in$ $\quad d_{5o} := x_5 - cover_{top} = 4.9 \cdot in$

Nominal Moment Resistance - Tension on Inside Face

$\phi Mn_{1i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1i} - \frac{a}{2}\right) = 208.3 \cdot kip \cdot in$

$\phi Mn_{2i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2i} - \frac{a}{2}\right) = 158.1 \cdot kip \cdot in$

$\phi Mn_{3i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3i} - \frac{a}{2}\right) = 145.6 \cdot kip \cdot in$

$\phi Mn_{4i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4i} - \frac{a}{2}\right) = 133.2 \cdot kip \cdot in$

$$\phi Mn_{5i} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5i} - \frac{a}{2}\right) = 84.5 \cdot kip \cdot in$$

$$M_{wi} := \phi Mn_{1i} + \phi Mn_{2i} + \phi Mn_{3i} + \phi Mn_{4i} + \phi Mn_{5i} = 60.8 \cdot kip \cdot ft$$

Nominal Moment Resistance - Tension on Outside Face

$$\phi Mn_{1o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{1o} - \frac{a}{2}\right) = 18.9 \cdot kip \cdot ft$$

$$\phi Mn_{2o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{2o} - \frac{a}{2}\right) = 13.2 \cdot kip \cdot ft$$

$$\phi Mn_{3o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{3o} - \frac{a}{2}\right) = 12.1 \cdot kip \cdot ft$$

$$\phi Mn_{4o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{4o} - \frac{a}{2}\right) = 11.1 \cdot kip \cdot ft$$

$$\phi Mn_{5o} := \phi_{ext} \cdot A_{sl\_p} \cdot F_s \cdot \left(d_{5o} - \frac{a}{2}\right) = 7 \cdot kip \cdot ft$$

$$M_{wo} := \phi Mn_{1o} + \phi Mn_{2o} + \phi Mn_{3o} + \phi Mn_{4o} + \phi Mn_{5o} = 62.3 \cdot kip \cdot ft$$

Vertical Nominal Moment Resistance of Parapet

$$M_w := \frac{2 \cdot M_{wi} + M_{wo}}{3} = 61.3 \cdot kip \cdot ft$$

Parapet Design Factors:

Crash Level $CL := \text{"TL-4"}$

Transverse Design Force

$$F_t := \left| \begin{array}{l} 13.5kip \text{ if } CL = \text{"TL-1"} \\ 27.0kip \text{ if } CL = \text{"TL-2"} \\ 54.0kip \text{ if } CL = \text{"TL-3"} \\ 54.0kip \text{ if } CL = \text{"TL-4"} \\ 124.0kip \text{ if } CL = \text{"TL-5"} \\ 175.0kip \text{ otherwise} \end{array} \right. = 54 \cdot kip \qquad L_t := \left| \begin{array}{l} 4.0ft \text{ if } CL = \text{"TL-1"} \\ 4.0ft \text{ if } CL = \text{"TL-2"} \\ 4.0ft \text{ if } CL = \text{"TL-3"} \\ 3.5ft \text{ if } CL = \text{"TL-4"} \\ 8.0ft \text{ if } CL = \text{"TL-5"} \\ 8.0ft \text{ otherwise} \end{array} \right. = 3.5 \cdot ft$$

Longitudinal Design Force

$$F_l := \left| \begin{array}{l} 4.5kip \text{ if } CL = \text{"TL-1"} \\ 9.0kip \text{ if } CL = \text{"TL-2"} \\ 18.0kip \text{ if } CL = \text{"TL-3"} \\ 18.0kip \text{ if } CL = \text{"TL-4"} \\ 41.0kip \text{ if } CL = \text{"TL-5"} \\ 58.0kip \text{ otherwise} \end{array} \right. = 18 \cdot kip \qquad L_l := \left| \begin{array}{l} 4.0ft \text{ if } CL = \text{"TL-1"} \\ 4.0ft \text{ if } CL = \text{"TL-2"} \\ 4.0ft \text{ if } CL = \text{"TL-3"} \\ 3.5ft \text{ if } CL = \text{"TL-4"} \\ 8.0ft \text{ if } CL = \text{"TL-5"} \\ 8.0ft \text{ otherwise} \end{array} \right. = 3.5 \cdot ft$$

Vertical Design Force (Down)

$$F_v := \left| \begin{array}{l} 4.5kip \text{ if } CL = \text{"TL-1"} \\ 4.5kip \text{ if } CL = \text{"TL-2"} \\ 4.5kip \text{ if } CL = \text{"TL-3"} \\ 18.0kip \text{ if } CL = \text{"TL-4"} \\ 80.0kip \text{ if } CL = \text{"TL-5"} \\ 80.0kip \text{ otherwise} \end{array} \right. = 18 \cdot kip \qquad L_v := \left| \begin{array}{l} 18.0ft \text{ if } CL = \text{"TL-1"} \\ 18.0ft \text{ if } CL = \text{"TL-2"} \\ 18.0ft \text{ if } CL = \text{"TL-3"} \\ 18.0ft \text{ if } CL = \text{"TL-4"} \\ 40.0ft \text{ if } CL = \text{"TL-5"} \\ 40.0ft \text{ otherwise} \end{array} \right. = 18 \cdot ft$$

Critical Length of Yield Line Failure Pattern:

$$M_b := 0kip \cdot ft$$

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \cdot H_{par} \cdot (M_b + M_w)}{M_c}} = 11.9 \cdot ft$$ S A13.3.1-2

$$R_w := \frac{2}{2 \cdot L_c - L_t} \cdot \left(8 \cdot M_b + 8 \cdot M_w + \frac{M_c \cdot L_c^2}{H_{par}}\right) = 116.2 \cdot kip$$ S A13.3.1-1

$$T := \frac{R_w \cdot b}{L_c + 2 \cdot H_{par}} = 6.6 \cdot kip$$ S A13.4.2-1

The parapet design must consider three design cases. Design Case 1 is for longitudinal and transverse collision loads under Extreme Event Load Combination II. Design Case 2 represents vertical collision loads under Extreme Event Load Combination II; however, this case does not govern for decks with concrete parapets or barriers. Design Case 3 is for dead and live load under Strength Load Combination I; however, the parapet will not carry wheel loads and therefore this case does not govern. Design Case 1 is the only case that requires a check.

Design Case 1: Longitudinal and Transverse Collision Loads, Extreme Event Load Combination II

DC - 1A: Inside face of parapet

$\phi_{ext} = 1$ $\gamma_{DC} := 1.0$ $\gamma_{DW} := 1.0$ $\gamma_{LL} := 0.5$ S A13.4.1
S Table 3.4.1-1

$l_{lip} := 2in$ $w_{base} = 17 \cdot in$

$A_{deck\_1A} := t_{deck} \cdot (l_{lip} + w_{base}) = 152 \cdot in^2$ $A_p = 2.8 \cdot ft^2$

$W_{deck\_1A} := w_c \cdot A_{deck\_1A} = 0.2 \cdot klf$ $W_{par} = 0.4 \cdot klf$

$$M_{DCdeck\_1A} := \gamma_{DC} \cdot W_{deck\_1A} \cdot \frac{l_{lip} + w_{base}}{2} = 0.1 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1A} := \gamma_{DC} \cdot W_{par} \cdot (l_{lip} + CG_p) = 0.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1A} := M_{cb} + M_{DCdeck\_1A} + M_{DCpar\_1A} = 16 \cdot \frac{kip \cdot ft}{ft}$$

$\phi_{tt\_add} := \frac{5}{8} in$ $s_{tt\_add} := 8in$

$$A_{stt\_p} := \frac{12in}{s_{tt}} \cdot \pi \cdot \left(\frac{\phi_{tt}}{2}\right)^2 + \frac{12in}{s_{tt\_add}} \cdot \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.9 \cdot in^2$$

$$d_{tt\_add} := t_{deck} - \left(c_t + \frac{\phi_{tt\_add}}{2}\right) = 5.2 \cdot in$$

$c_{tt\_p} := \frac{A_{stt\_p} \cdot F_s}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.4 \cdot in$ $a_{tt\_p} := \beta_1 \cdot c_{tt\_p} = 1.1 \cdot in$

$$M_{ntt\_p} := \frac{A_{stt\_p} \cdot F_s}{ft} \cdot \left(d_{tt\_add} - \frac{a_{tt\_p}}{2}\right) = 21.4 \cdot \frac{kip \cdot ft}{ft}$$

$M_{rtt\_p} := \phi_b \cdot M_{ntt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft}$ $M_{rtt\_p} \geq M_{total\_1A} = 1$

$A_{sT} := A_{stt} + A_{stb} = 1.1 \cdot in^2$

$\phi P_n := \phi_{ext} \cdot A_{sT} \cdot F_s = 67.4 \cdot kip$ $\phi P_n \geq T = 1$

$M_{u\_1A} := M_{rtt\_p} \cdot \left(1 - \frac{T}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft}$ $M_{u\_1A} \geq M_{total\_1A} = 1$

DC - 1B: Design Section in Overhang

Notes: Distribution length is assumed to increase based on a 30 degree angle from the face of parapet.
Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
Future wearing surface is neglected as contribution is negligible.

$A_{deck\_1B} := t_{deck} \cdot L_o = 168 \cdot in^2$ $\qquad A_p = 2.8 \cdot ft^2$

$W_{deck\_1B} := w_c \cdot A_{deck\_1B} = 0.2 \cdot klf$ $\qquad W_{par} = 0.4 \cdot klf$

$$M_{DCdeck\_1B} := \gamma_{DC} \cdot W_{deck\_1B} \cdot \frac{L_o}{2} = 0.2 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{DCpar\_1B} := \gamma_{DC} \cdot W_{par} \cdot \left(L_o - l_{lip} - CG_p\right) = 0.5 \cdot \frac{kip \cdot ft}{ft}$$

$L_{spread\_B} := L_o - l_{lip} - width_3 = 2 \cdot in$ $\qquad spread := 30deg$

$w_{spread\_B} := L_{spread\_B} \cdot \tan(spread) = 1.2 \cdot in$

$$M_{cb\_1B} := \frac{M_{cb} \cdot L_c}{L_c + 2 \cdot w_{spread\_B}} = 15.3 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1B} := M_{cb\_1B} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 15.9 \cdot \frac{kip \cdot ft}{ft}$$

$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft}$ $\qquad M_{rtt\_p} \geq M_{total\_1B} = 1$

$\phi P_n = 67.4 \cdot kip$

$P_u := \dfrac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_B}} = 6.5 \cdot kip$ $\qquad \phi P_n \geq P_u = 1$

$M_{u\_1B} := M_{rtt\_p} \cdot \left(1 - \dfrac{P_u}{\phi P_n}\right) = 17.4 \cdot \dfrac{kip \cdot ft}{ft}$ $\qquad M_{u\_1B} \geq M_{total\_1B} = 1$

DC - 1C: Design Section in First Span

Assumptions: Moment of collision loads is distributed over the length Lc + 30 degree spread from face of parapet to location of overhang design section.
Axial force of collision loads is distributed over the length Lc + 2Hpar + 30 degree spread from face of parapet to location of overhang design section.
Future wearing surface is neglected as contribution is negligible.

$$M_{par\_G1} := M_{DCpar\_1B} = 0.5 \cdot \frac{kip \cdot ft}{ft}$$

$M_{par\_G2} := -0.137 \dfrac{kip \cdot ft}{ft}$ $\qquad$ (From model output)

$$M_1 := M_{cb} = 15.6 \cdot \frac{kip \cdot ft}{ft}$$

$$M_2 := M_1 \cdot \frac{M_{par\_G2}}{M_{par\_G1}} = -4.7 \cdot \frac{kip \cdot ft}{ft}$$

$b_f := 10.5in$

$$M_{c\_M2M1} := M_1 + \frac{\frac{1}{4} \cdot b_f \cdot \left(-M_1 + M_2\right)}{spacing_{int\_max}} = 14.1 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{spread\_C} := L_o - l_{lip} - w_{base} + \frac{b_f}{4} = 4.6 \cdot in$$

$$w_{spread\_C} := L_{spread\_C} \cdot \tan(spread) = 2.7 \cdot in$$

$$M_{cb\_1C} := \frac{M_{c\_M2M1} \cdot L_c}{L_c + 2 \cdot w_{spread\_C}} = 13.6 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{total\_1C} := M_{cb\_1C} + M_{DCdeck\_1B} + M_{DCpar\_1B} = 14.2 \cdot \frac{kip \cdot ft}{ft}$$

$$M_{rtt\_p} = 19.2 \cdot \frac{kip \cdot ft}{ft} \qquad M_{rtt\_p} \geq M_{total\_1C} = 1$$

$$\phi P_n = 67.4 \cdot kip$$

$$P_{uC} := \frac{T \cdot \left(L_c + 2 \cdot H_{par}\right)}{L_c + 2 \cdot H_{par} + 2 \cdot w_{spread\_C}} = 6.4 \cdot kip \qquad \phi P_n \geq P_{uC} = 1$$

$$M_{u\_1C} := M_{rtt\_p} \cdot \left(1 - \frac{P_u}{\phi P_n}\right) = 17.4 \cdot \frac{kip \cdot ft}{ft} \qquad M_{u\_1B} \geq M_{total\_1B} = 1$$

Compute Overhang Reinforcement Cut-off Length Requirement

Maximum crash load moment at theoretical cut-ff point:

$$M_{c\_max} := M_{rtt} = 10.2 \cdot \frac{kip \cdot ft}{ft}$$

$$L_{Mc\_max} := \frac{M_2 - M_{rtt}}{M_2 - M_1} \cdot spacing_{int\_max} = 2.1 \cdot ft$$

$$L_{spread\_D} := L_o - l_{lip} - w_{base} + L_{Mc\_max} = 27.7 \cdot in$$

$$w_{spread\_D} := L_{spread\_D} \cdot \tan(spread) = 16 \cdot in$$

$$M_{cb\_max} := \frac{M_{c\_max} \cdot L_c}{L_c + 2 \cdot w_{spread\_D}} = 8.3 \cdot \frac{kip \cdot ft}{ft}$$

$$extension := \max\left(d_{tt\_add}, 12 \cdot \phi_{tt\_add}, 0.0625 \cdot spacing_{int\_max}\right) = 7.5 \cdot in$$

$$cutt\_off := L_{Mc\_max} + extension = 33.2 \cdot in$$

$$A_{tt\_add} := \pi \cdot \left(\frac{\phi_{tt\_add}}{2}\right)^2 = 0.3 \cdot in^2$$

$$m_{thick\_tt\_add} := \begin{cases} 1.4 & \text{if } t_{deck} - c_t \geq 12in \\ 1.0 & \text{otherwise} \end{cases} = 1$$

$$m_{epoxy\_tt\_add} := \begin{cases} 1.5 & \text{if } c_t < 3 \cdot \phi_{tt\_add} \vee \frac{s_{tt\_add}}{2} - \phi_{tt\_add} < 6 \cdot \phi_{tt\_add} \\ 1.2 & \text{otherwise} \end{cases} = 1.5$$

$$m_{inc\_tt\_add} := \min\left(m_{thick\_tt\_add} \cdot m_{epoxy\_tt\_add}, 1.7\right) = 1.5$$

$$m_{dec\_tt\_add} := \begin{cases} 0.8 & \text{if } \frac{s_{tt\_add}}{2} \geq 6in \\ 1.0 & \text{otherwise} \end{cases} = 1$$

$$l_{db\_tt\_add} := \begin{cases} \max\left(\dfrac{1.25in \cdot A_{tt\_add} \cdot \dfrac{F_s}{kip}}{\sqrt{\dfrac{f_c}{ksi}}}, 0.4 \cdot \phi_{tt\_add} \cdot \dfrac{F_s}{ksi}\right) & \text{if } \phi_{tt\_add} \le \dfrac{11}{8} in \\ \dfrac{2.70in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{14}{8} in \\ \dfrac{3.50in \cdot \dfrac{F_s}{ksi}}{\sqrt{\dfrac{f_c}{ksi}}} & \text{if } \phi_{tt\_add} = \dfrac{18}{8} in \end{cases} \qquad l_{db\_tt\_add} = 15 \cdot in$$

$l_{dt\_tt\_add} := l_{db\_tt\_add} \cdot m_{inc\_tt\_add} \cdot m_{dec\_tt\_add} = 22.5 \cdot in$

$Cuttoff_{point} := L_{Mc\_max} + l_{dt\_tt\_add} - spacing_{int\_max} = 13.2 \cdot in$ extension past second interior girder

Check for Cracking in Overhang under Service Limit State:

Does not govern - no live load on overhang.

25. COMPRESSION SPLICE:

See sheet S7 for drawing.

Ensure compression splice and connection can handle the compressive force in the force couple due to the negative moment over the pier.

Live load negative moment over pier: $M_{LLPier} := 541.8 \cdot kip \cdot ft$

Factored LL moment: $M_{UPier} := 1.75 \cdot M_{LLPier} = 948.1 \cdot kip \cdot ft$

The compression splice is comprised of a splice plate on the underside of the bottom flange, and built-up angles on either side of the web, connecting to the bottom flange as well.

Calculate Bottom Flange Stress:

Composite moment of inertia: $I_z = 7666.4 \cdot in^4$

Distance to center of bottom flange from composite section centroid: $y_{bf} := \dfrac{t_{bf}}{2} + D_w + t_{tf} + t_{slab} - y_c = 24 \cdot in$

Stress in bottom flange: $f_{bf} := M_{UPier} \cdot \dfrac{y_{bf}}{I_z} = 35.6 \cdot ksi$

Calculate Bottom Flange Force:

Design Stress: $F_{bf} := \max\left(\dfrac{f_{bf} + F_y}{2}, 0.75 \cdot F_y\right) = 42.8 \cdot ksi$

Effective Flange Area: $A_{ef} := b_{bf} \cdot t_{bf} = 6.4 \cdot in^2$

Force in Flange: $C_{nf} := F_{bf} \cdot A_{ef} = 274.1 \cdot kip$

Calculate Bottom Flange Stress, Ignoring Concrete:

Moment of inertia: $I_{zsteel} = 2798.5 \cdot in^4$

Distance to center of bottom flange: $y_{bfsteel} := \dfrac{t_{bf}}{2} + D_w + t_{tf} - y_{steel} = 13 \cdot in$

Stress in bottom flange: $f_{bfsteel} := M_{UPier} \cdot \frac{y_{bfsteel}}{I_{zsteel}} = 52.9 \cdot ksi$

Bottom Flange Force for design:

Design Stress: $F_{cf} := \max\left(\frac{f_{bfsteel} + F_y}{2}, 0.75 \cdot F_y\right) = 51.5 \cdot ksi$

Design Force: $C_n := \max(F_{bf}, F_{cf}) \cdot A_{ef} = 329.4 \cdot kip$

Compression Splice Plate Dimensions:

Bottom Splice Plate:	$b_{bsp} := b_{bf} = 10 \cdot in$	$t_{bsp} := 0.75in$	$A_{bsp} := b_{bsp} \cdot t_{bsp} = 7.5 \cdot in^2$
Built-Up Angle Splice Plate Horizontal Leg:	$b_{asph} := 4.25in$	$t_{asph} := 0.75in$	$A_{asph} := 2 \cdot b_{asph} \cdot t_{asph} = 6.4 \cdot in^2$
Built-Up Angle Splice Plate Vertical Leg:	$b_{aspv} := 7.75in$	$t_{aspv} := 0.75in$	$A_{aspv} := 2 \cdot b_{aspv} \cdot t_{aspv} = 11.6 \cdot in^2$

Total Area: $A_{csp} := A_{bsp} + A_{asph} + A_{aspv} = 25.5 \cdot in^2$

Average Stress: $f_{cs} := \frac{C_n}{A_{csp}} = 12.9 \cdot ksi$

Proportion Load into each plate based on area:

$$C_{bsp} := \frac{C_n \cdot A_{bsp}}{A_{csp}} = 96.9 \cdot kip \qquad C_{asph} := \frac{C_n \cdot A_{asph}}{A_{csp}} = 82.3 \cdot kip \qquad C_{aspv} := \frac{C_n \cdot A_{aspv}}{A_{csp}} = 150.2 \cdot kip$$

Check Plates Compression Capacity:

Bottom Splice Plate: $k_{cps} := 0.75$ for bolted connection

$l_{cps} := 9in$

$$r_{bsp} := \sqrt{\frac{\min\left(\frac{b_{bsp} \cdot t_{bsp}^3}{12}, \frac{t_{bsp} \cdot b_{bsp}^3}{12}\right)}{A_{bsp}}} = 0.2 \cdot in$$

$$P_{ebsp} := \frac{\pi^2 \cdot E_s \cdot A_{bsp}}{\left(\frac{k_{cps} \cdot l_{cps}}{r_{bsp}}\right)^2} = 2208.5 \cdot kip$$

$$Q_{bsp} := \begin{cases} 1.0 & \text{if } \frac{b_{bsp}}{t_{bsp}} \le 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ \left[1.34 - 0.76 \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right) \cdot \sqrt{\frac{F_y}{E_s}}\right] & \text{if } 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \le \frac{b_{bsp}}{t_{bsp}} \le 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b_{bsp}}{t_{bsp}}\right)^2} & \text{otherwise} \end{cases} = 0.919$$

$P_{obsp} := Q_{bsp} \cdot F_y \cdot A_{bsp} = 344.7 \cdot kip$

$$P_{nbsp} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{obsp}}{P_{ebsp}}\right)}\right]\cdot P_{obsp}\right] & \text{if } \dfrac{P_{ebsp}}{P_{obsp}} \ge 0.44 \\ (0.877\cdot P_{ebsp}) & \text{otherwise} \end{cases} = 322.9\cdot kip$$

$$P_{nbsp\_allow} := 0.9\cdot P_{nbsp} = 290.6\cdot kip \qquad Check := \begin{cases} \text{"NG"} & \text{if } C_{bsp} \ge P_{nbsp\_allow} \\ \text{"OK"} & \text{if } P_{nbsp\_allow} \ge C_{bsp} \end{cases} = \text{"OK"}$$

Horizontal Angle Leg: $k_{cps} = 0.75$ for bolted connection

$$l_{cps} = 9\cdot in$$

$$r_{asph} := \sqrt{\frac{\min\left(\dfrac{b_{asph}\cdot t_{asph}^{3}}{12}, \dfrac{t_{asph}\cdot b_{asph}^{3}}{12}\right)}{A_{asph}}} = 0.153\cdot in$$

$$P_{easph} := \frac{\pi^2\cdot E_s\cdot A_{asph}}{\left(\dfrac{k_{cps}\cdot l_{cps}}{r_{asph}}\right)^2} = 938.6\cdot kip$$

$$Q_{asph} := \begin{cases} 1.0 & \text{if } \dfrac{b_{asph}}{t_{asph}} \le 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\cdot\left(\dfrac{b_{asph}}{t_{asph}}\right)\cdot\sqrt{\dfrac{F_y}{E_s}}\right] & \text{if } 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{asph}}{t_{asph}} \le 0.91\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\cdot E_s}{F_y\cdot\left(\dfrac{b_{asph}}{t_{asph}}\right)^2} & \text{otherwise} \end{cases} = 1$$

$$P_{oasph} := Q_{asph}\cdot F_y\cdot A_{asph} = 318.7\cdot kip$$

$$P_{nasph} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{oasph}}{P_{easph}}\right)}\right]\cdot P_{oasph}\right] & \text{if } \dfrac{P_{easph}}{P_{oasph}} \ge 0.44 \\ (0.877\cdot P_{easph}) & \text{otherwise} \end{cases} = 276.5\cdot kip$$

$$P_{nasph\_allow} := 0.9\cdot P_{nasph} = 248.9\cdot kip \qquad Check2 := \begin{cases} \text{"NG"} & \text{if } C_{asph} \ge P_{nasph\_allow} \\ \text{"OK"} & \text{if } P_{nasph\_allow} \ge C_{asph} \end{cases} = \text{"OK'}$$

Vertical Angle Leg: $k_{cps} = 0.75$ for bolted connection

$$l_{cps} = 9\cdot in$$

$$r_{aspv} := \sqrt{\frac{\min\left(\dfrac{b_{aspv}\cdot t_{aspv}^{3}}{12}, \dfrac{t_{aspv}\cdot b_{aspv}^{3}}{12}\right)}{A_{aspv}}} = 0.153\cdot in$$

$$P_{easpv} := \frac{\pi^2\cdot E_s\cdot A_{aspv}}{\left(\dfrac{k_{cps}\cdot l_{cps}}{r_{aspv}}\right)^2} = 1711.6\cdot kip$$

$$Q_{aspv} := \begin{cases} 1.0 & \text{if } \dfrac{b_{aspv}}{t_{aspv}} \le 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \left[1.34 - 0.76\cdot\left(\dfrac{b_{aspv}}{t_{aspv}}\right)\cdot\sqrt{\dfrac{F_y}{E_s}}\right] & \text{if } 0.45\cdot\sqrt{\dfrac{E_s}{F_y}} \le \dfrac{b_{aspv}}{t_{aspv}} \le 0.91\cdot\sqrt{\dfrac{E_s}{F_y}} \\ \dfrac{0.53\cdot E_s}{F_y\cdot\left(\dfrac{b_{aspv}}{t_{aspv}}\right)^2} & \text{otherwise} \end{cases} = 1$$

$$P_{oaspv} := Q_{aspv}\cdot F_y\cdot A_{aspv} = 581.2\cdot kip$$

$$P_{naspv} := \begin{cases} \left[\left[0.658^{\left(\frac{P_{oaspv}}{P_{easpv}}\right)}\right]\cdot P_{oaspv}\right] & \text{if } \dfrac{P_{easpv}}{P_{oaspv}} \ge 0.44 \\ (0.877\cdot P_{easpv}) & \text{otherwise} \end{cases} = 504.2\cdot kip$$

$$P_{naspv\_allow} := 0.9\cdot P_{naspv} = 453.8\cdot kip \qquad Check3 := \begin{cases} \text{"NG"} & \text{if } C_{aspv} \ge P_{naspv\_allow} \\ \text{"OK"} & \text{if } P_{naspv\_allow} \ge C_{aspv} \end{cases} = \text{"OK'}$$

Additional Checks: Design Bolted Connections of the splice plates to the girders, checking for shear, bearing, and slip critical connections.

26. CLOSURE POUR DESIGN:

See sheet S2 for drawing of closure pour.

Check the closure pour according to the negative bending capacity of the section.

Use the minimum reinforcing properties for design, to be conservative.

$$A_{steel} = 24.5\cdot in^2 \qquad A_{rt} = 1.4\cdot in^2 \qquad A_{rb} = 1.9\cdot in^2$$

$$cg_{steel} := t_{slab} + y_{steel} = 21.3\cdot in \qquad cg_{rt} := 3in + 1.5\cdot\frac{5}{8}in = 3.9\cdot in \qquad cg_{rb} := t_{slab} - \left(1in + 1.5\cdot\frac{5}{8}in\right) = 6.1\cdot in$$

Overall CG: $A_{neg} := A_{steel} + A_{rt} + A_{rb} = 27.8\cdot in^2$ $\qquad cg_{neg} := \dfrac{A_{steel}\cdot cg_{steel} + A_{rt}\cdot cg_{rt} + A_{rb}\cdot cg_{rb}}{A_{neg}} = 19.4\cdot in$

Moment of Inertia: $I_{zstl} := 3990in^4$

$$I_{neg} := I_{zstl} + A_{steel}\cdot(cg_{steel} - cg_{neg})^2 + A_{rt}\cdot(cg_{rt} - cg_{neg})^2 + A_{rb}\cdot(cg_{rb} - cg_{neg})^2 = 4748.8\cdot in^4$$

Section Moduli: $S_{top\_neg} := \dfrac{I_{neg}}{cg_{neg} - cg_{rt}} = 306.5\cdot in^3$ $\qquad r_{neg} := \sqrt{\dfrac{I_{neg}}{A_{neg}}} = 13.1\cdot in$

$$S_{bot\_neg} := \frac{I_{neg}}{(t_{slab} + t_{tf} + D_w + t_{bf} - cg_{neg})} = 311.4\cdot in^3$$

Concrete Properties: $f_c = 5\cdot ksi$ $\qquad$ Steel Properties: $F_y = 50\cdot ksi$ $\qquad L_{bneg} := 13.42ft$

$E_c = 4286.8\cdot ksi$ $\qquad E_s = 29000\cdot ksi$

$$F_{yr} := 0.7 \cdot F_y = 35 \cdot ksi$$

Negative Flexural Capacity:

Slenderness ratio for compressive flange: $\lambda_{fneg} := \dfrac{b_{bf}}{2 \cdot t_{bf}} = 7.8$

Limiting ratio for compactness: $\lambda_{pfneg} := 0.38 \cdot \sqrt{\dfrac{E_s}{F_y}} = 9.2$

Limiting ratio for noncompact $\lambda_{rfneg} := 0.56 \cdot \sqrt{\dfrac{E_s}{F_{yr}}} = 16.1$

Hybrid Factor: $R_h = 1$

$$D_{cneg2} := \frac{D_w}{2} = 12.7 \cdot in \qquad a_{wc} := \frac{2 \cdot D_{cneg2} \cdot t_w}{b_{bf} \cdot t_{bf}} = 1.8$$

$$R_b := \begin{cases} 1.0 & \text{if } 2 \cdot \dfrac{D_{cneg2}}{t_w} \le 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}} \\ \min\left[1.0, 1 - \dfrac{a_{wc}}{1200 + 300 \cdot a_{wc}} \cdot \left(2 \cdot \dfrac{D_{cneg2}}{t_w} - 5.7 \cdot \sqrt{\dfrac{E_s}{F_y}}\right)\right] & \text{otherwi} \end{cases}$$

$$R_b = 1$$

Flange compression resistance:

$$F_{nc1} := \begin{cases} R_b \cdot R_h \cdot F_y & \text{if } \lambda_{fneg} \le \lambda_{pfneg} \\ \left[\left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(\lambda_{fneg} - \lambda_{pfneg})}{(\lambda_{rfneg} - \lambda_{pfneg})}\right] \cdot R_b \cdot R_h \cdot F_y\right] & \text{otherwise} \end{cases}$$

$$F_{nc1} = 50 \cdot ksi$$

Lateral Torsional Buckling Resistance:

$$r_{tneg} := \frac{b_{bf}}{\sqrt{12 \cdot \left(1 + \dfrac{D_{cneg2} \cdot t_w}{3 \cdot b_{bf} \cdot t_{bf}}\right)}} = 2.5 \cdot in$$

$$L_{pneg} := 1.0 \cdot r_{tneg} \cdot \sqrt{\frac{E_s}{F_y}} = 60.9 \cdot in$$

$$L_{rneg} := \pi \cdot r_{tneg} \cdot \sqrt{\frac{E_s}{F_{yr}}} = 228.6 \cdot in$$

$$C_b = 1$$

$$F_{nc2} := \begin{cases} R_b \cdot R_h \cdot F_y & \text{if } L_{bneg} \le L_{pneg} \\ \min\left[C_b \cdot \left[1 - \left(1 - \dfrac{F_{yr}}{R_h \cdot F_y}\right) \cdot \dfrac{(L_{bneg} - L_{pneg})}{(L_{rneg} - L_{pneg})}\right] \cdot R_b \cdot R_h \cdot F_y, R_b \cdot R_h \cdot \right. \end{cases}$$

$$F_{nc2} = 41 \cdot ksi$$

Compressive Resistance: $F_{nc} := \min(F_{nc1}, F_{nc2}) = 41 \cdot ksi$

Tensile Flexural Resistance: $F_{nt} := R_h \cdot F_y = 50 \cdot ksi$ For Strength

$F_{nt\_Serv} := 0.95 \cdot R_h \cdot F_y = 47.5 \cdot ksi$ For Service

Ultimate Moment Resistance: $M_{n\_neg} := \min\left(F_{nt} \cdot S_{top\_neg}, F_{nc} \cdot S_{bot\_neg}\right) = 1065.1 \cdot kip \cdot ft$

$M_{UPier} = 948.1 \cdot kip \cdot ft$ from external FE analysis

$Check4 := M_{n\_neg} \geq M_{UPier} = 1$

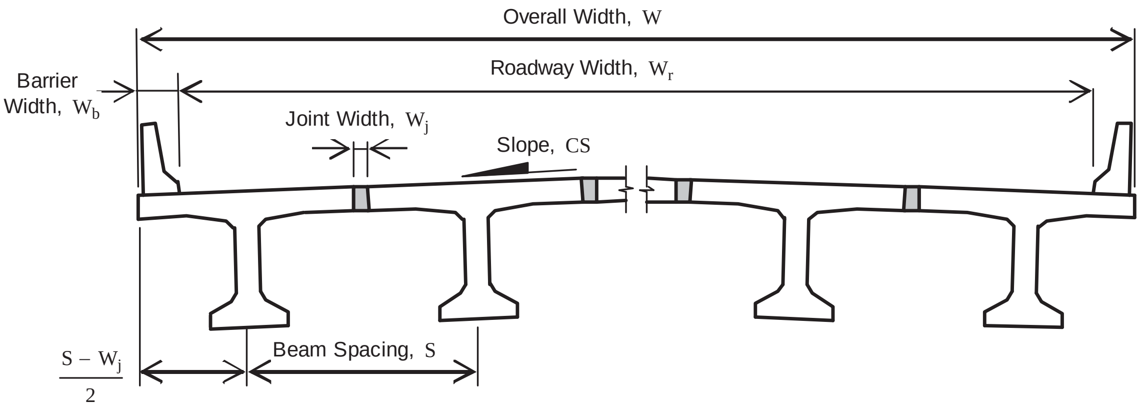
For additional design, one may calculate the force couple at the section over the pier to find the force in the UHPC closure joint. This force can be used to design any additional reinforcement used in the joint.

File Name: Prestressed Concrete Girder-80ft.xmcd

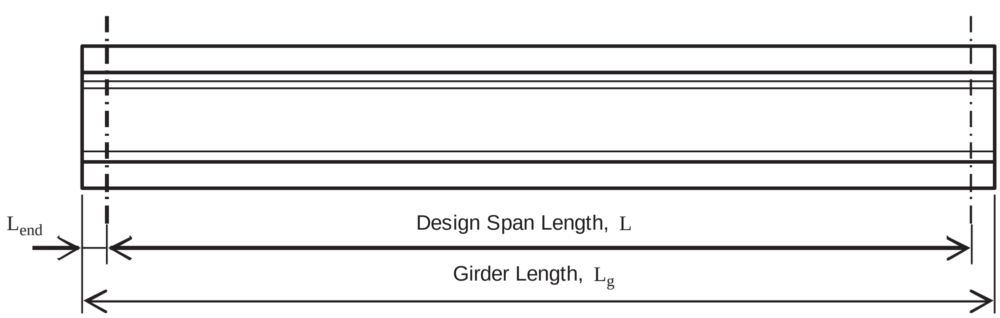
DECKED PRECAST PRESTRESSED CONCRETE GIRDER DESIGN FOR ABC

Unit Definition: $kcf \equiv kip \cdot ft^{-3}$

This example is for the design of a superstructure system that can be used for rapid bridge replacement in an Accelerated Bridge Construction (ABC) application. The following calculations are intended to provide the designer guidance in developing a similar design with regard to design considerationS characteristic of this type of construction, and they shall not be considered fully exhaustive.



TYPICAL SECTION THROUGH SPAN



GIRDER ELEVATION

Bridge Geometry:

$L := 80 \cdot ft$ $\quad L_{end} := 2 \cdot ft$ $\quad skew := 0 \cdot deg$

$W := 63 \cdot ft$ $\quad W_b := 1.5 \cdot ft$

$S_{max} := 8 \cdot ft$ $\quad W_j := 0.5 \cdot ft$

$N_g := ceil\left(\frac{W + W_j}{S_{max}}\right) = 8$ Minimum number of girders in cross-section

$S := \frac{W + W_j}{N_g} = 7.938 \cdot ft$ Girder spacing

ORDER OF CALCULATIONS

1. Introduction
2. Design Philosophy
3. Design Criteria
4. Beam Section
5. Material Properties
6. Permanent Loads
7. Precast Lifting Weight
8. Live Load
9. Prestress Properties
10. Prestress Losses
11. Concrete Stresses
12. Flexural Strength
13. Shear Strength
14. Splitting Resistance
15. Camber and Deflections
16. Negative Moment Flexural Strength

1. INTRODUCTION

The bridge that is designed in this example consists of precast prestressed concrete girders with a top flange equal to the beam spacing, so the top flange will be the riding surface of the designed bridge. The purpose for these girders is to rapidly construct the bridge by providing a precast deck on the girders, which eliminates cast-in-place decks in the field and improves safety.

The concepts used in this example have been taken from on-going research, which focuses on the benefits of decked precast beams and promoting widespread acceptance from transportation and construction industries. The cross-section is adapted from the optimized girder sections recommended by NCHRP Project No. 12-69, Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges. The girder design has not taken into account the option to re-deck due to the final re-decked girder, without additional prestressed, having a shorter life span. Use of stainless steal rebar and the application of a future membrane can get ride of the need to replace the deck. This case is included in “re-deckability”.

The bridge used in this example is a general design of a typical bridge in Georgia. The calculations can be modified for single-span and multiple-span bridges due to the beam design moments are not reduced for continuity at intermediate supports (continuity details are not shown in this example). The cross-section consists of a four-lane roadway with normal crown, with standard shoulder lengths and barrier walls. The precast prestressed concrete girder has been uniformly designed to simplify bearing details. The girder flanges are 9’’ at the tips, imitating a 8’’ slab with a ½’’ allowable wear and another ½’’ for smoothness and profile adjustments.

This example is intended to illustrate design aspect specific to precast prestressed concrete girders used for ABC application. Girders with uncommon cross-sections, high self-weight, or unconventional load application create major concerns and more detailed calculations must be done.

2. DESIGN PHILOSOPHY

The geometry of the section is based on GDOT standards and general bridges across the state of Georgia. Depth variations are dependent on the construction company but must maintain the shapes of the top flange and the bottom bulb.

Concrete strengths can vary but are mostly between 6 ksi and 10 ksi. For the purpose of these calculations the concrete with a 28-day minimum compressive strength of 8 ksi is used. Due to its casting sequence this beam is unable to take advantage of composite sequences along with tension at the bottom of the beam at the service limit state being limited. This is further discussed in section 4 along with end region stresses being critical. Therefore the minimum concrete strength at release must be 80 percent of the 28-day compressive strength, which increases the allowable stresses at the top and bottom of the section. The prestressed steel can also be optimized to minimize stresses at the end region.

The prestressed steel is arranged in a draped, or harped, pattern to maximize the midspan effectiveness while it minimizes the failure at the end of the beam where is concrete is easily overstressed due to the lack of dead load acting on the beam. The strand group is optimized at the midspan by bundling the strands between hold-points, maximizing the stiffness of the strand group. The number and deflection angles are depended on the type of single strands you are using for the girder. In longer span cases the concrete at the end of the girder will be too large and will debond. Without harped strands it is unlikely to reduce stresses to the allowable limit, since harped strands are required this method of stress relief will be used without debonding for long spans.

3. DESIGN CRITERIA

Criteria has been selected to govern the design of these concrete girders while following provisions set by AASHTO, GDOT design specifications, as well as criteria of past projects and current research related to ABC and decked precast sections. A summary of the limiting design values are categorized as section constraints, prestress limits, and concrete limits.

Section Constraints:

$W_{pc.max} := 200 \cdot kip$ — Upper limit on the weight of the entire precast element, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

$S_{max} = 8 \cdot ft$ — Upper limit on girder spacing and, therefore, girder flange width (defined on first page)

Prestress Limits:

$F_{hd.single} := 4 \cdot kip$ — Maximum hold-down force for a single strand

$F_{hd.group} := 48 \cdot kip$ — Maximum hold-down force for the group of harped strands

Stress limits in the prestressing steel immediately prior to prestress and at the service limit state after all losses are as prescribed by AASHTO LRFD.

3. DESIGN CRITERIA (cont'd)

Concrete Limits:

Allowable concrete stresses meet standards set by AASHTO LRFD with one exception that at Service III Limit State, allowable bottom fiber tension when camber leveling forces are to be neglected, regardless of exposure, are to be 0-ksi. Minimum strength of concrete at release is 80 percent of the 28-day minimum compressive strength (f-ksi).

$f_{t.all.ser} := 0 \cdot ksi$ — Allowable bottom fiber tension at the Service III Limit State, when camber leveling forces are to be neglected, regardless of exposure

As previously mentioned, release concrete strength is specified as 80 percent of the minimum 28-day compressive strength to maximize allowable stresses in the end region of beam at release.

$f_{c.rel}(f) := 0.80 \cdot f$ — Minimum strength of concrete at release

Due to various lifting and transportation conditions, stresses in the concrete need to be considered. A “no cracking” approach is used for allowable tension due to reduced lateral stability after cracking. Assuming the girders will be lifted before the 28-day minimum strength is attained, the strength of concrete during lifting and transportation is assumed to be 90 percent of the 28-day minimum compressive strength. A dynamic dead load allowance of 30 percent is used for compression during handling. A factor of safety (FS) of 1.5 is used against cracking during handling.

$DIM := 30\%$ — Dynamic dead load allowance

$f_{c.erec}(f) := 0.90 \cdot f$ — Assumed attained concrete strength during lifting and transportation

$FS_c := 1.5$ — Factor of safety against cracking during lifting transportation

$f_{t.erec}(f) := \dfrac{-0.24 \cdot \sqrt{f \cdot ksi}}{FS_c}$ — Allowable tension in concrete during lifting and transportation to avoid cracking

4. BEAM SECTION

Use trapezoidal areas to define the cross-section. The flange width is defined as the beam spacing less the width of the longitudinal closure joint to reflect pre-erection conditions. Live load can be conservatively applied to this section, as well.

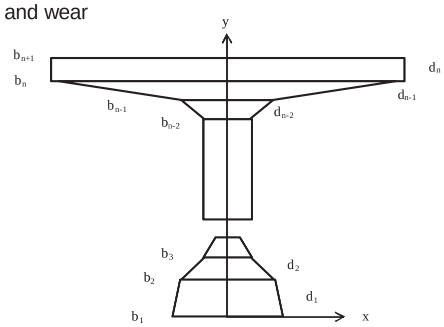
$h := 49.5 \cdot in$ Beam section depth

$t_{flange} := 9 \cdot in$ Flange thickness at tip

$t_{sac} := 1 \cdot in$ Total sacrificial depth for grinding and wear

$b_1 := 26 \cdot in$	$b_2 := 26 \cdot in$	$d_1 := 6 \cdot in$
$b_2 = 26 \cdot in$	$b_3 := 6 \cdot in$	$d_2 := 4.5 \cdot in$
$b_3 = 6 \cdot in$	$b_4 := 6 \cdot in$	
$b_4 = 6 \cdot in$	$b_5 := 10 \cdot in$	$d_4 := 2 \cdot in$
$b_5 = 10 \cdot in$	$b_6 := 42 \cdot in$	$d_5 := 2 \cdot in$
$b_6 = 42 \cdot in$	$b_7 := S - W_j$	$d_6 := 0 \cdot in$
$b_7 = 89.25 \cdot in$	$b_8 := S - W_j$	$d_7 := t_{flange} - t_{sac}$

$d_3 := h - t_{sac} - \sum d$ $\qquad d_3 = 26 \cdot in$



TYPICAL GIRDER SECTION COMPRISED OF n TRAPEZOIDAL REGIONS

Gross Section Properties

$b_f = 89.25 \cdot in$ Precast girder flange width

$A_g = 1166 \cdot in^2$ Cross-sectional area (does not include sacrifical thickness)

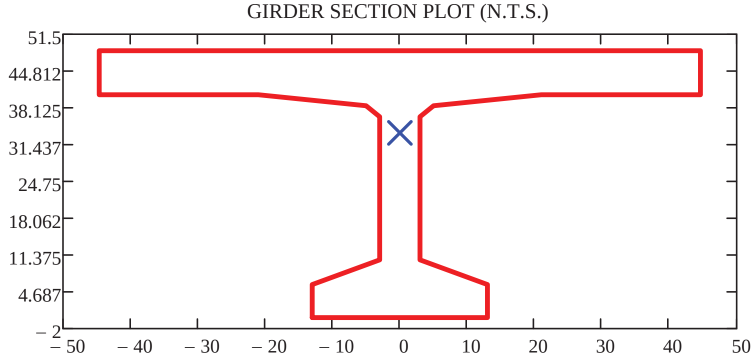
$I_{xg} = 310192 \cdot in^4$ Moment of inertia (does not include sacrificial thickness)

$y_{tg} = 14.938 \cdot in$ $\quad y_{bg} = -33.562 \cdot in$ Top and bottom fiber distances from neutal axis (positive up)

$S_{tg} = 20765.3 \cdot in^3$ $\quad S_{bg} = -9242.4 \cdot in^3$ Top and bottom section moduli

$I_{yg} = 487758 \cdot in^4$ Weak-axis moment of inertia

GIRDER SECTION PLOT (N.T.S.)



5. MATERIAL PROPERTIES

These properties are standard (US units) values with equations that can be found in AASHTO LRFD Bridge Design Specifications.

Concrete:

$f_c := 8 \cdot ksi$	Minimum 28-day compressive strength of concrete
$f_{ci} := f_{c.rel}(f_c) = 6.4 \cdot ksi$	Minimum strength of concrete at release
$\gamma_c := .150 \cdot kcf$	Unit weight of concrete
$K_1 := 1.0$	Correction factor for standard aggregate (5.4.2.4)
$E_{ci} := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_{ci} \cdot ksi} = 4850 \cdot ksi$	Modulus of elasticity at release (5.4.2.4-1)
$E_c := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 5422 \cdot ksi$	Modulus of elasticity (5.4.2.4-1)
$f_{r.cm} := 0.37 \cdot \sqrt{f_c \cdot ksi} = 1.047 \cdot ksi$	Modulus of rupture for cracking moment (5.4.2.6)
$f_{r.cd} := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.679 \cdot ksi$	Modulus of rupture for camber and deflection (5.4.2.6)
$H := 70$	Relative humidity (5.4.2.3)

Prestressing Steel:

$f_{pu} := 270 \cdot ksi$	Ultimate tensile strength
$f_{py} := 0.9 \cdot f_{pu} = 243 \cdot ksi$	Yield strength, low-relaxation strand (Table 5.4.4.1-1)
$f_{pbt.max} := 0.75 \cdot f_{pu} = 202.5 \cdot ksi$	Maximum stress in steel immediately prior to transfer
$f_{pe.max} := 0.80 \cdot f_{py} = 194.4 \cdot ksi$	Maximum stress in steel after all losses
$E_p := 28500 \cdot ksi$	Modulus of elasticity (5.4.4.2)
$d_{ps} := 0.5 \cdot in$	Strand diameter
$A_p := 0.153 \cdot in^2$	Strand area
$N_{ps.max} := 40$	Maximum number of strands in section
$n_{pi} := \frac{E_p}{E_{ci}} = 5.9$	Modular ratio at release
$n_p := \frac{E_p}{E_c} = 5.3$	Modular ratio

Mild Steel:

$f_y := 60 \cdot ksi$	Specified minimum yield strength
$E_s := 29000 \cdot ksi$	Modulus of elasticity (5.4.3.2)

6. PERMANENT LOADS

Permanent loads or dead loads that must be considered are self-weight, diaphragms, barriers, and future wearing surface. The barrier can be cast to the beam before it is taken on sight or attached to the bridge after the joints have reached sufficient strength. Distribution of the barriers weight should be established once you decide when it would be attached to the bridge. For this example the barrier has been cast on the exterior girder in the casting yard, before shipping but after release of prestresses. Due to this the dead load is increased on the exterior girders but it eliminates the time-consuming task that would have been completed in the field.

$BeamLoc := 1$ — Location of beam within the cross-section (0 - Interior, 1 - Exterior)

Load at Release:

$\gamma_{c.DL} := .155 \cdot kcf$ — Concrete density used for weight calculations

$A_{g.DL} := A_g + t_{sac} \cdot (S - W_j) = 1255.25 \cdot in^2$ — Area used for weight calculations, including sacrificial thickness

$w_g := A_{g.DL} \cdot \gamma_{c.DL} = 1.351 \cdot klf$ — Uniform load due to self-weight, including sacrificial thickness

$L_g := L + 2 \cdot L_{end} = 84 \cdot ft$ — Span length at release

$M_{gr}(x) := \frac{w_g \cdot x}{2} \cdot (L_g - x)$ — Moment due to beam self-weight (supported at ends)

$V_{gr}(x) := w_g \cdot \left(\frac{L_g}{2} - x\right)$ — Shear due to beam self-weight (supported at ends)

Load at Erection:

$M_g(x) := \frac{w_g \cdot x}{2} \cdot (L - x)$ — Moment due to beam self-weight

$V_g(x) := w_g \cdot \left(\frac{L}{2} - x\right)$ — Shear due to beam self-weight

$w_{bar} := 0.430 \cdot klf$ — Uniform load due to barrier weight, exterior beams only

$w_{bar} := if(BeamLoc = 1, w_{bar}, 0) = 0.43 \cdot klf$ — Redfine to 0 if interior beam (BeamLoc = 0)

$M_{bar}(x) := \frac{w_{bar} \cdot x}{2} \cdot (L - x)$ — Moment due to beam self-weight

$V_{bar}(x) := w_{bar} \cdot \left(\frac{L}{2} - x\right)$ — Shear due to beam self-weight

6. PERMANENT LOADS (cont'd)

Load at Service:

$p_{fws} := 25 \cdot psf$ — Assumed weight of future wearing surface

$w_{fws} := p_{fws} \cdot S = 0.198 \cdot klf$ — Uniform load due to future wearing surface

$M_{fws}(x) := \frac{w_{fws} \cdot x}{2} \cdot (L - x)$ — Moment due to future wearing surface

$V_{fws}(x) := w_{fws} \cdot \left(\frac{L}{2} - x\right)$ — Shear due to future wearing surface

$w_j := W_j \cdot d_7 \cdot \gamma_{c.DL} = 0.052 \cdot klf$ — Uniform load due to weight of longitudinal closure joint

$M_j(x) := \frac{w_j \cdot x}{2} \cdot (L - x)$ — Moment due to longitudinal closure joint

$V_j(x) := w_j \cdot \left(\frac{L}{2} - x\right)$ — Shear due to longitudinal closure joint

7. PRECAST LIFTING WEIGHT

For Accelerated Bridge Construction the beams are casted in a factory and transported to the job site. When they arrive at the site they must be lifted and put into place. When designing we have to consider the weight of each slab to insure safety and design for possible cracking.

Precast Superstructure

$W_g := (w_g + w_{bar}) \cdot L_g = 149.6 \cdot kip$ — Precast girder, including barrier if necessary

Substructure Precast with Superstructure

$L_{corb} := 1 \cdot ft$ — Length of approach slab corbel

$B_{corb} := b_f$ $\quad b_f = 89.25 \cdot in$ — Width of corbel cast with girder

$D_{corb} := 1.5 \cdot ft$ — Average depth of corbel

$V_{corb} := L_{corb} \cdot B_{corb} \cdot D_{corb} = 11.16 \cdot ft^3$ — Volume of corbel

$L_{ia} := 2.167 \cdot ft$ — Length of integral abutment

$L_{gia} := 1.167 \cdot ft$ — Length of girder embedded in integral abutment

$B_{ia} := S - W_j = 7.438 \cdot ft$ — Width of integral abutment cast with girder

$D_{ia} := h + 4 \cdot in = 53.5 \cdot in$ — Depth of integral abutment

$V_{ia} := V_{corb} + \left[L_{ia} \cdot B_{ia} \cdot D_{ia} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gia}\right] = 80.07 \cdot ft^3$ — Volume of integral abutment cast with girder

$W_{ia} := V_{ia} \cdot \gamma_c = 12 \cdot kip$ — Weight of integral abutment cast with girder

$L_{sa} := 2.167 \cdot ft$ — Length of semi-integral abutment

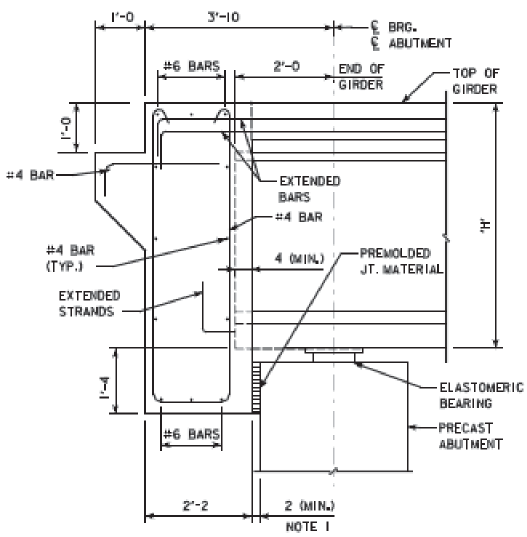
$L_{gsa} := 4 \cdot in$ — Length of girder embedded in semi-integral abutment

$B_{sa} := S - W_j = 7.438 \cdot ft$ — Width of semi-integral abutment cast with girder

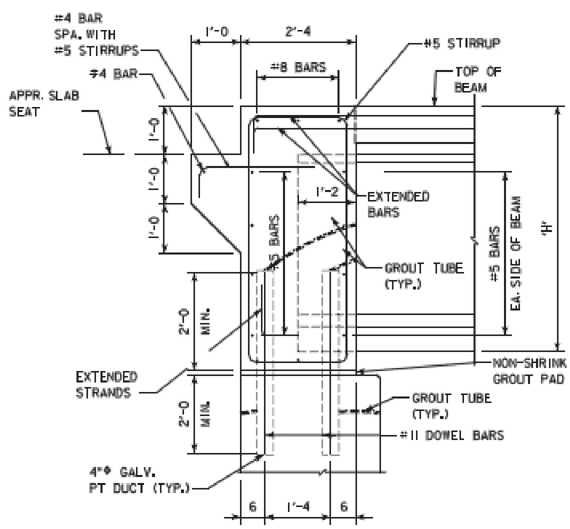
$D_{sa} := h + 16 \cdot in = 65.5 \cdot in$ — Depth of semi-integral abutment

$V_{sa} := V_{corb} + \left[L_{sa} \cdot B_{sa} \cdot D_{sa} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gsa}\right] = 98.29 \cdot ft^3$ — Volume of semi-integral abutment cast with girder

$W_{sa} := V_{sa} \cdot \gamma_c = 15 \cdot kip$ — Weight of semi-integral abutment cast with girder



Semi-Integral Abutment Backwall



Integral Abutment Backwall

8. LIVE LOAD

When considering Live Loads you must refer to the vertical load section HL-93 in the AASHTO manual. If the project you are working on requires the bridge to support construction loads at any stage, these loads must be considered separately and applied. The longitudinal joints are designed for full moment connections so the beams will act as a unit when sufficiently connected. The distribution factors are then computed for cross-section type "j" (defined in AASHTO 4.6.2.2). When calculating the stiffness parameter, the constant- depth region at the top flange is treated like the slab and the remaining area of the beam will be considered a non-composite beam.

Definitions:

I_{bb}	=	moment of inertia of section below the top flange
A_{bb}	=	area of beam section below the top flange
y_{bb}	=	distance of top fiber below the top flange from neutral axis
t_s	=	thickness of slab not including sacrificial thickness

Distribution Factors for Moment:

From Table 4.6.2.2.2b-1 for moment in interior girders,

$I_{bb} = 86022 \cdot in^4$ Moment of inertia of section below the top flange

$A_{bb} = 452 \cdot in^2$ Area of beam section below the top flange

$e_g := h - \left(t_{sac} + \frac{t_s}{2}\right) + y_{bb} = 28.216 \cdot in$ Distance between c.g.'s of beam and flange

$K_g := 1.0 \cdot \left(I_{bb} + A_{bb} \cdot e_g^2\right) = 445885 \cdot in^4$ Longitudinal stiffness parameter (Eqn. 4.6.2.2.1-1)

Verify this girder design is within the range of applicability for Table 4.6.2.2.2b-1.

$$CheckMint := if\left[(S \le 16 \cdot ft) \cdot (S \ge 3.5 \cdot ft) \cdot (t_s \ge 4.5 \cdot in) \cdot (t_s \le 12.0 \cdot in) \cdot (L \ge 20 \cdot ft) \cdot (L \le 240 \cdot ft), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckMint := if\left[(CheckMint = \text{"OK"}) \cdot (N_g \ge 4) \cdot \left(K_g \ge 10000 \cdot in^4\right) \cdot \left(K_g \le 7000000 \cdot in^4\right), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckMint = \text{"OK"}$$

$g_{mint1} := 0.06 + \left(\frac{S}{14 \cdot ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot t_s^3}\right)^{0.1} = 0.455$ Single loaded lane

$g_{mint2} := 0.075 + \left(\frac{S}{9.5 \cdot ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot t_s^3}\right)^{0.1} = 0.635$ Two or more loaded lanes

$g_{mint} := \max(g_{mint1}, g_{mint2}) = 0.635$ Distribution factor for moment at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.2d-1 for moment in exterior girders,

$d_e := \frac{S}{2} - W_b = 29.625 \cdot in$

Distance from centerline of exterior beam to edge of curb or barrier

$CheckMext := if\left[\left(d_e \geq -1 \cdot ft\right) \cdot \left(d_e \leq 5.5 \cdot ft\right) \cdot \left(N_g \geq 4\right), "OK", "No Good"\right] = "OK"$

For a single loaded lane, use the Lever Rule.

$g_{mext1} := \frac{\left(S + 0.5 \cdot b_f - W_b - 5 \cdot ft\right)}{S} = 0.65$

Single loaded lane

$e_m := 0.77 + \frac{d_e}{9.1 \cdot ft} = 1.041$

Correction factor for moment (Table 4.6.2.2.2d-1)

$g_{mext2} := e_m \cdot g_{mint} = 0.661$

Two or more loaded lanes

$g_{mext} := max\left(g_{mext1}, g_{mext2}\right) = 0.661$

Distribution factor for moment at exterior beams

Distribution Factors for Shear:

From Table 4.6.2.2.3a-1 for shear in interior girders,

Verify this girder design is within the range of applicability for Table 4.6.2.2.3a-1.

$CheckVint := if\left[(S \leq 16 \cdot ft) \cdot (S \geq 3.5 \cdot ft) \cdot \left(t_s \geq 4.5 \cdot in\right) \cdot \left(t_s \leq 12.0in\right) \cdot (L \geq 20 \cdot ft) \cdot (L \leq 240 \cdot ft), "OK", "No Good"\right]$

$CheckVint := if\left[(CheckMint = "OK") \cdot \left(N_g \geq 4\right), "OK", "No Good"\right]$

$CheckVint = "OK"$

$g_{vint1} := 0.36 + \left(\frac{S}{25 \cdot ft}\right) = 0.678$

Single loaded lane

$g_{vint2} := 0.2 + \left(\frac{S}{12 \cdot ft}\right) - \left(\frac{S}{35 \cdot ft}\right)^{2.0} = 0.81$

Two or more loaded lanes

$g_{vint} := max\left(g_{vint1}, g_{vint2}\right) = 0.81$

Distribution factor for shear at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.3b-1 for shear in exterior girders,

For a single loaded lane, use the Lever Rule.

$$\text{CheckVext} := \text{if}\left[\left(d_e \geq -1\cdot ft\right)\cdot\left(d_e \leq 5.5\cdot ft\right)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$g_1 := \frac{\left(S + 0.5\cdot b_f - W_b - 5\cdot ft\right)}{S} = 0.65$$ Single loaded lane (same as for moment)

$$e_v := 0.6 + \frac{d_e}{10\cdot ft} = 0.847$$ Correction factor for shear (Table 4.6.2.2.3b-1)

$$g_2 := e_v\cdot g_{vint} = 0.686$$ Two or more loaded lanes

$$g_{vext} := \max\left(g_1, g_2\right) = 0.686$$ Distribution factor for shear at exterior beams

From Table 4.6.2.2.3c-1 for skewed bridges,

$$\theta := skew = 0\cdot deg$$

$$\text{CheckSkew} := \text{if}\left[(\theta \leq 60\cdot deg)\cdot(3.5\cdot ft \leq S \leq 16\cdot ft)\cdot(20\cdot ft \leq L \leq 240\cdot ft)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$c_{skew} := 1.0 + 0.20\cdot\left(\frac{L\cdot {t_s}^3}{K_g}\right)^{0.3}\cdot\tan(\theta) = 1.00$$ Correction factor for skew

8. LIVE LOAD (cont'd)

Design Live Load Moment at Midspan:

$w_{lane} := 0.64 \cdot klf$ — Design lane load

$P_{truck} := 32 \cdot kip$ — Design truck axle load

$IM := 33\%$ — Dynamic load allowance (truck only)

$M_{lane}(x) := \frac{w_{lane} \cdot x}{2} \cdot (L - x)$ — Design lane load moment

$\delta(x) := \frac{x \cdot L - x^2}{L}$ — Influence coefficient for truck moment calculation

$M_{truck}(x) := P_{truck} \cdot \delta(x) \cdot \max\left[\frac{9 \cdot x \cdot (L - x) - 14 \cdot ft \cdot (3 \cdot x + L)}{4 \cdot x \cdot (L - x)}, \frac{9 \cdot (L - x) - 84 \cdot ft}{4 \cdot (L - x)}\right]$ — Design truck moment

$M_{HL93}(x) := M_{lane}(x) + (1 + IM) \cdot M_{truck}(x)$ — HL93 design live load moment per lane

$M_{ll.i}(x) := M_{HL93}(x) \cdot g_{mint}$ — Design live load moment at interior beam

$M_{ll.e}(x) := M_{HL93}(x) \cdot g_{mext}$ — Design live load moment at exterior beam

$M_{ll}(x) := \text{if}\left(BeamLoc = 1, M_{ll.e}(x), M_{ll.i}(x)\right)$ — Design live load moment

Design Live Load Shear:

$V_{lane}(x) := w_{lane} \cdot \left(\frac{L}{2} - x\right)$ — Design lane load shear

$V_{truck}(x) := P_{truck} \cdot \left(\frac{9 \cdot L - 9 \cdot x - 84 \cdot ft}{4 \cdot L}\right)$ — Design truck shear

$V_{HL93}(x) := V_{lane}(x) + (1 + IM) \cdot V_{truck}(x)$ — HL93 design live load shear

$V_{ll.i}(x) := V_{HL93}(x) \cdot g_{vint}$ — Design live load shear at interior beam

$V_{ll.e}(x) := V_{HL93}(x) \cdot g_{vext}$ — Design live load shear at exterior beam

$V_{ll}(x) := \text{if}\left(BeamLoc = 1, V_{ll.e}(x), V_{ll.i}(x)\right)$ — Design live load shear

9. PRESTRESS PROPERTIES

Due to tension at the surface limit state be reduced to account for camber leveling forces, the prestress force required at the midspan is expected to be excessive at the ends when released. Not measuring the reduction of prestress moments. Estimate prestress losses at the midspan to find trial prestress forces, that will occur in the bottom tension fibers, that are less than allowable. Compute immediate losses in the prestressed steel and check released stresses at the end of the beam. Once you satisfy end stresses, estimate total loss of prestress. As long as these losses are not drastically different from the assumed stresses, the prestress layout should be acceptable. Concrete stress at all limit states are in Section 9.

$y_{p.est} := 5 \cdot in$ — Assumed distance from bottom of beam to centroid of prestress at midspan

$y_{cgp.est} := y_{bg} + y_{p.est} = -28.56 \cdot in$ — Eccentricity of prestress from neutral axis, based on assumed location

$\Delta f_{p.est} := 25\%$ — Estimate of total prestress losses at the service limit state

Compute bottom fiber service stresses at midspan using gross section properties.

$X := \frac{L}{2}$ — Distance from support

$M_{dl.ser} := M_g(X) + M_{fws}(X) + M_j(X) + M_{bar}(X) = 1625 \cdot kip \cdot ft$ — Total dead load moment

$f_{b.serIII} := \frac{M_{dl.ser} + 0.8 \cdot M_{ll}(X)}{S_{bg}} = -3.521 \cdot ksi$ — Total bottom fiber service stress

$f_{pj} := f_{pbt.max} = 202.5 \cdot ksi$ — Prestress jacking force

$f_{pe.est} := f_{pj} \cdot \left(1 - \Delta f_{p.est}\right) = 151.9 \cdot ksi$ — Estimate of effective prestress force

$$A_{ps.est} := A_g \cdot \frac{\left(\frac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \frac{A_g \cdot y_{cgp.est}}{S_{bg}}} = 5.873 \cdot in^2$$ — Estimated minimum area of prestressing steel

$N_{ps.est} := ceil\left(\frac{A_{ps.est}}{A_p}\right) = 39$ — Estimated number of strands required

$N_{ps} := 38$ — Number of strands used ($N_{ps.max} = 40$)

The number above is used for the layout strand pattern and to compute the actual location of the strand group. After this is done the required area is computed again. If the estimated location is accurate the number of strands should be equal to the number of strands that we calculated above. The number of strands that was estimated was based on our assumed prestressed losses and gross section properties, which may not accurately reflect the final number of strands required for the design. These stresses for concrete are evaluated in Section 10.
The geometry is assuming a vertical spacing of 2" between straight spans, as well as 2" for harped strands at the end of the beam. Harped strands are bundled at the midspan where the centroid is 5" from the bottom.

9. PRESTRESS PROPERTIES (cont'd)

$$N_h := \begin{cases} 2 & \text{if } N_{ps} \le 12 \\ 4 & \text{if } 12 < N_{ps} \le 24 \\ 6 & \text{if } 24 < N_{ps} \le 30 \\ 6 + (N_{ps} - 30) & \text{if } N_{ps} > 30 \end{cases}$$

$N_h = 14$

Assumes all flange rows are filled prior to filling rows in web above the flange, which maximized efficiency. Use override below to shift strands from flange to web if needed to satisfy end stresses.

$N_{h.add} := 16$

Additional harped strands in web (strands to be moved from flange to web)

$$N_h := \min\left(N_h + N_{h.add}, 16, 2\cdot \text{floor}\left(\frac{N_{ps}}{4}\right)\right)$$

$N_h = 16$

16 strands or half of total strands maximum harped in web

$$y_h := 1\cdot in + (2\cdot in)\cdot\left(1 + \frac{0.5\cdot N_h - 1}{2}\right)$$

$y_h = 10\cdot in$

Centroid of harped strands from bottom, equally spaced

$y_{hb} := 5\cdot in$

Centroid of harped strands from bottom, bundled

$N_s := N_{ps} - N_h$

$N_s = 22$

Number of straight strands in flange

$$y_s := 1\cdot in + \begin{cases} 2\cdot in & \text{if } N_s \le 10 \\ \dfrac{(4\cdot in)\cdot N_s - 20\cdot in}{N_s} & \text{if } 10 < N_s \le 20 \\ \dfrac{(6\cdot in)\cdot N_s - 60\cdot in}{N_s} & \text{if } 20 < N_s \le 24 \\ 3.5\cdot in & \text{otherwise} \end{cases}$$

$y_s = 4.273\cdot in$

Centroid of straight strands from bottom

$$y_p := \frac{N_s\cdot y_s + N_h\cdot y_{hb}}{N_s + N_h} = 4.579\cdot in$$

Centroid of prestress from bottom at midspan

$y_{cgp} := y_{bg} + y_p = -28.98\cdot in$

Eccentricity of prestress from neutral axis

$$A_{ps.req} := A_g\cdot\frac{\left(\dfrac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \dfrac{A_g\cdot y_{cgp}}{S_{bg}}} = 5.806\cdot in^2$$

Estimated minimum area of prestressing steel

$$N_{ps.req} := \text{ceil}\left(\frac{A_{ps.req}}{A_p}\right) = 38$$

Estimated number of strands required

$CheckNps := \text{if}\left[(N_{ps} \le N_{ps.max})\cdot(N_{ps.req} \le N_{ps}), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$A_{ps.h} := N_h\cdot A_p = 2.448\cdot in^2$

Area of prestress in web (harped)

$A_{ps.s} := N_s\cdot A_p = 3.366\cdot in^2$

Area of prestress in flange (straight)

$A_{ps} := A_{ps.h} + A_{ps.s} = 5.814\cdot in^2$

Total area of prestress

9. PRESTRESS PROPERTIES (cont'd)

Compute transformed section properties based on prestress layout.

Transformed Section Properties

Initial Transformed Section (release):

$A_{ti} = 1194.4 \cdot in^2$

$I_{xti} = 333442 \cdot in^4$

$y_{tti} = 15.626 \cdot in \qquad S_{tti} = 21339 \cdot in^3$

$y_{cgpi} = -28.295 \cdot in \qquad S_{cgpi} = -11784 \cdot in^3$

$y_{bti} = -32.874 \cdot in \qquad S_{bti} = -10143 \cdot in^3$

Final Transformed Section (service):

$A_{tf} = 1190.7 \cdot in^2$

$I_{xtf} = 330546 \cdot in^4$

$y_{ttf} = 15.540 \cdot in \qquad S_{ttf} = 21270 \cdot in^3$

$y_{cgpf} = -28.381 \cdot in \qquad S_{cgpf} = -11647 \cdot in^3$

$y_{btf} = -32.960 \cdot in \qquad S_{btf} = -10029 \cdot in^3$

Determine initial prestress force after instantaneous loss due to elastic shortening. Use transformed properties to compute stress in the concrete at the level of prestress.

$P_j := f_{pj} \cdot A_{ps} = 1177.3 \cdot kip$ — Jacking force in prestress, prior to losses

$$f_{cgpi} := P_j \cdot \left(\frac{1}{A_{ti}} + \frac{y_{cgpi}}{S_{cgpi}} \right) + \frac{M_{gr}\left(\frac{L_g}{2}\right)}{S_{cgpi}} = 2.599 \cdot ksi$$

Stress in concrete at the level of prestress after instantaneous losses

$\Delta f_{pES} := n_{pi} \cdot f_{cgpi} = 15.273 \cdot ksi$ — Prestress loss due to elastic shortening (5.9.5.2.3a-1)

$f_{pi} := f_{pj} - \Delta f_{pES} = 187.227 \cdot ksi$ — Initial prestress after instantaneous losses

$P_i := f_{pi} \cdot A_{ps} = 1088.5 \cdot kip$ — Initial prestress force

Determine deflection of harped strands required to satisfy allowable stresses at the end of the beam at release.

$f_{c.all.rel} := 0.6 \cdot f_{ci} = 3.84 \cdot ksi$ — Allowable compression before losses (5.9.4.1.1)

$f_{t.all.rel} := \max\left(-0.0948 \cdot \sqrt{f_{ci} \cdot ksi}, -0.2 \cdot ksi\right) = -0.200 \cdot ksi$ — Allowable tension before losses (Table 5.9.4.1.2-1)

$L_t := 60 \cdot d_{ps} = 2.5 \cdot ft$ — Transfer length (AASHTO 5.11.4.1)

$$y_{cgp.t} := \left(\frac{f_{t.all.rel} - \frac{M_{gr}(L_t)}{S_{tti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{tti} = -23.305 \cdot in$$

Prestress eccentricity required for tension

$$y_{cgp.b} := \left(\frac{f_{c.all.rel} - \frac{M_{gr}(L_t)}{S_{bti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{bti} = -28.806 \cdot in$$

Prestress eccentricity required for compression

9. PRESTRESS PROPERTIES (cont'd)

$y_{cgp.req} := \max(y_{cgp.t}, y_{cgp.b}) = -23.305 \cdot in$

Required prestress eccentricity at end of beam

$$y_{h.brg.req} := \frac{(y_{cgp.req} - y_{bti}) \cdot (N_s + N_h) - y_s \cdot N_s}{N_h} = 16.852 \cdot in$$

Minimum distance to harped prestress centroid from bottom of beam at centerline of bearing

$y_{top.min} := 18 \cdot in$

Minimum distance between uppermost strand and top of beam

$\alpha_{hd} := 0.4$

Hold-down point, fraction of the design span length

$$slope_{max} := if\left(d_{ps} = 0.6 \cdot in, \frac{1}{12}, \frac{1}{8}\right) = 0.125$$

Maximum slope of an individual strand to limit hold-down force to 4 kips/strand

$$y_{h.brg} := h - y_{top.min} - \left(\frac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) = 24.5 \cdot in$$

Set centroid of harped strands as high as possible to minimize release and handling stresses

$y_{h.brg} := \min(y_{h.brg}, y_{hb} + slope_{max} \cdot \alpha_{hd} \cdot L) = 24.5 \cdot in$

Verify that slope requirement is satisfied at uppermost strand

$CheckEndPrestress := if(y_{h.brg} \geq y_{h.brg.req}, \text{"OK"}, \text{"Verify release stresses."}) = \text{"OK"}$

$$y_{p.brg} := \frac{N_s \cdot y_s + N_h \cdot y_{h.brg}}{N_s + N_h} = 12.789 \cdot in$$

Centroid of prestress from bottom at bearing

$$slope_{cgp} := \frac{y_{p.brg} - y_p}{\alpha_{hd} \cdot L} = 0.021$$

Slope of prestress centroid within the harping length

$$y_{px}(x) := \begin{cases} y_p + slope_{cgp} \cdot (L_{end} + \alpha_{hd} \cdot L - x) & \text{if } x \leq L_{end} + \alpha_{hd} \cdot L \\ y_p & \text{otherwise} \end{cases}$$

Distance to center of prestress from the bottom of the beam at any position

10. PRESTRESS LOSSES

Prestressed losses can be evaluated like regular concrete, in short-term and long-term losses. When the beam is a pretension girder there are instantaneous losses when the beam is shortened upon release of the prestress forces. Time-dependent losses happen when the beam is under creep and shrinkage of the beam concrete, creep and shrinkage c the deck concrete, and the relaxation of prestressed steel. These long term effects are separated into two stages that represent significant events in bridge construction. The first stage is the time between transfer of the prestress forces anc placement of the decked beam and the second is the period of time between placement of the deck and the final service load. For decked beams the computation of long-term losses is slightly simplified due to the cross-section not changing between the two stages and the shrinkage term of the deck concrete is eliminated since the deck and beam being cast together. No losses or gains in the steel associated with deck placement after transfer.

AASHTO methods for estimating time-dependent losses:
Approximate Estimate (5.9.5.3)
Refined Estimate (5.9.5.4)

The Approximate method is based on systems with composite decks and is based on the following assumptions: timing ot load application, the cross-section in which the load is applied, and the ratio of dead and live loads to the total load. The conditions for the beams to be fabricated, formed and loaded depend on conditions assumed in the development of the approximate method. The refined method is used to estimate time-dependent losses in the prestressed steel.

Equations 5.9.5.4 are time-dependent and calculate the age-adjustment factors that effect losses using gross section properties.

$t_i := 1$ Time (days) between casting and release of prestress

$t_b := 20$ Time (days) to barrier casting (exterior girder only)

$t_d := 30$ Time (days) to erection of precast section, closure joint pour

$t_f := 20000$ Time (days) to end of service life

Terms and equations used in the loss calculations:

$K_L := 45$ Prestressing steel factor for low-relaxation strands (C5.9.5.4.2c)

$VS := \frac{A_g}{Peri} = 3.857 \cdot in$ Volume-to-surface ratio of the precast section

$k_s := \max\left(1.45 - 0.13 \cdot \frac{VS}{in}, 1.0\right) = 1.00$ Factor for volume-to-surface ratio (5.4.2.3.2-2)

$k_{hc} := 1.56 - 0.008 \cdot H = 1.00$ Humidity factor for creep (5.4.2.3.2-3)

$k_{hs} := 2.00 - 0.014 \cdot H = 1.02$ Humidity factor for shrinkage (5.4.2.3.3-2)

$k_f := \frac{5}{1 + \frac{f_{ci}}{ksi}} = 0.676$ Factor for effect of concrete strength (5.4.2.3.2-4)

10. PRESTRESS LOSSES (cont'd)

$$k_{td}(t) := \frac{t}{61 - 4\cdot\frac{f_{ci}}{ksi} + t}$$

Time development factor (5.4.2.3.2-5)

$$\psi\left(t, t_{init}\right) := 1.9\cdot k_s\cdot k_{hc}\cdot k_f\cdot k_{td}(t)\cdot\left(t_{init}\right)^{-0.118}$$

Creep coefficient (5.4.2.3.2-1)

$$\varepsilon_{sh}(t) := k_s\cdot k_{hs}\cdot k_f\cdot k_{td}(t)\cdot\left(0.48\cdot 10^{-3}\right)$$

Concrete shrinkage strain (5.4.2.3.3-1)

Time from Transfer to Erection:

$$e_{pg} := -\left(y_p + y_{bg}\right) = 28.983\cdot in$$

Eccentricity of prestress force with respect to the neutral axis of the gross non-composite beam, positive below the beam neutral axis

$$f_{cgp} := P_i\cdot\left(\frac{1}{A_g} + \frac{e_{pg}^{\ 2}}{I_{xg}}\right) + \frac{M_g\left(\frac{L}{2}\right)}{I_{xg}}\cdot\left(y_p + y_{bg}\right) = 2.669\cdot ksi$$

Stress in the concrete at the center prestress immediately after transfer

$$f_{pt} := \max\left(f_{pi}, 0.55\cdot f_{py}\right) = 187.227\cdot ksi$$

Stress in strands immediately after transfer (5.9.5.4.2c-1)

$$\psi_{bid} := \psi\left(t_d, t_i\right) = 0.589$$

Creep coefficient at erection due to loading at transfer

$$\psi_{bif} := \psi\left(t_f, t_i\right) = 1.282$$

Creep coefficient at final due to loading at transfer

$$\varepsilon_{bid} := \varepsilon_{sh}\left(t_d - t_i\right) = 1.490 \times 10^{-4}$$

Concrete shrinkage between transfer and erection

$$K_{id} := \frac{1}{1 + n_{pi}\cdot\frac{A_{ps}}{A_g}\cdot\left(1 + \frac{A_g\cdot e_{pg}^{\ 2}}{I_{xg}}\right)\cdot\left(1 + 0.7\cdot\psi_{bif}\right)} = 0.812$$

Age-adjusted transformed section coefficient (5.9.5.4.2a-2)

$$\Delta f_{pSR} := \varepsilon_{bid}\cdot E_p\cdot K_{id} = 3.449\cdot ksi$$

Loss due to beam shrinkage (5.9.5.4.2a-1)

$$\Delta f_{pCR} := n_{pi}\cdot f_{cgp}\cdot\psi_{bid}\cdot K_{id} = 7.504\cdot ksi$$

Loss due to creep (5.9.5.4.2b-1)

$$\Delta f_{pR1} := \left[\frac{f_{pt}}{K_L}\cdot\frac{\log\left(24\cdot t_d\right)}{\log\left(24\cdot t_i\right)}\cdot\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\cdot\left[1 - \frac{3\cdot\left(\Delta f_{pSR} + \Delta f_{pCR}\right)}{f_{pt}}\right]\cdot K_{id} = 1.272\cdot ksi$$

Loss due to relaxation (C5.9.5.4.2c-1

$$\Delta f_{pid} := \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} = 12.224\cdot ksi$$

10. PRESTRESS LOSSES (cont'd)

Time from Erection to Final:

$e_{pc} := e_{pg} = 28.983 \cdot in$ — Eccentricity of prestress force does not change

$A_c := A_g$ $\quad I_c := I_{xg}$ — Section properties remain unchanged

$$\Delta f_{cd} := \frac{M_{fws}\left(\frac{L}{2}\right) + M_j\left(\frac{L}{2}\right)}{S_{cgpf}} + \frac{\Delta f_{pid}}{n_p} = 2.12 \cdot ksi$$

Change in concrete stress at center of prestress due to initial time-dependent losses and superimposed dead load. Deck weight is not included for this design.

$\psi_{bdf} := \psi(t_f, t_d) = 0.858$ — Creep coefficient at final due to loading at erection

$\varepsilon_{bif} := \varepsilon_{sh}(t_f - t_i) = 3.302 \times 10^{-4}$ — Concrete shrinkage between transfer and final

$\varepsilon_{bdf} := \varepsilon_{bif} - \varepsilon_{bid} = 1.813 \times 10^{-4}$ — Concrete shrinkage between erection and final

$$K_{df} := \frac{1}{1 + n_{pi} \cdot \frac{A_{ps}}{A_c} \cdot \left(1 + \frac{A_c \cdot e_{pc}^{\;2}}{I_c}\right) \cdot \left(1 + 0.7 \cdot \psi_{bif}\right)} = 0.812$$

Age-adjusted transformed section coefficient remains unchanged

$\Delta f_{pSD} := \varepsilon_{bdf} \cdot E_p \cdot K_{df} = 4.196 \cdot ksi$ — Loss due to beam shrinkage

$\Delta f_{pCD} := n_{pi} \cdot f_{cgp} \cdot \left(\psi_{bif} - \psi_{bid}\right) \cdot K_{df} + n_p \cdot \Delta f_{cd} \cdot \psi_{bdf} \cdot K_{df} = 16.588 \cdot ksi$ — Loss due to creep

$\Delta f_{pR2} := \Delta f_{pR1} = 1.272 \cdot ksi$ — Loss due to relaxation

$\Delta f_{pSS} := 0$ — Loss due to deck shrinkage

$\Delta f_{pdf} := \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} + \Delta f_{pSS} = 22.056 \cdot ksi$

Prestress Loss Summary

$\Delta f_{pES} = 15.273 \cdot ksi$ $\qquad \frac{\Delta f_{pES}}{f_{pj}} = 7.5 \cdot \%$

$\Delta f_{pLT} := \Delta f_{pid} + \Delta f_{pdf} = 34.28 \cdot ksi$ $\qquad \frac{\Delta f_{pLT}}{f_{pj}} = 16.9 \cdot \%$

$\Delta f_{pTotal} := \Delta f_{pES} + \Delta f_{pLT} = 49.553 \cdot ksi$ $\qquad \frac{\Delta f_{pTotal}}{f_{pj}} = 24.5 \cdot \%$ $\qquad \Delta f_{p.est} = 25 \cdot \%$

$f_{pe} := f_{pj} - \Delta f_{pTotal} = 152.9 \cdot ksi$ — Final effective prestress

$CheckFinalPrestress := if\left(f_{pe} \le f_{pe.max}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$

11. CONCRETE STRESSES

Concrete Stresses at release, during handling and at final service are computed and compared to approximated values for each stage.

Concrete Stresses at Release

When calculating the stresses at release use the overall beam length due to the beam being supported at each end in the casting bed after prestress forces are transformed.

Define locations for which stresses are to be calculated:

$$x_r := L_g \cdot \left(0 \quad \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad 0.1 \quad 0.2 \quad 0.3 \quad \alpha_{hd} \quad 0.5 \right)^T \qquad ir := 1 .. \text{last}(x_r)$$

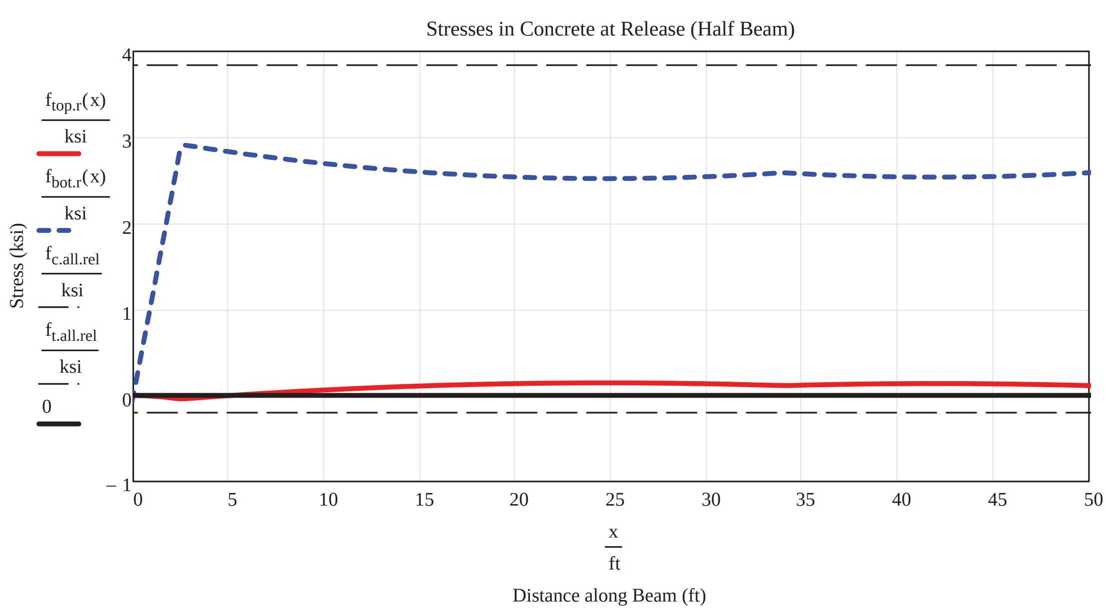
Functions for computing beam stresses:

$$f_{top.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{tti}}\right) + \frac{M_{gr}(x)}{S_{tti}}$$

Top fiber stress at release

$$f_{bot.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{bti}}\right) + \frac{M_{gr}(x)}{S_{bti}}$$

Bottom fiber stress at release



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$f_{t.all.rel} = -0.2 \cdot ksi$ Allowable tension at release

$f_{c.all.rel} = 3.84 \cdot ksi$ Allowable compression at release

$TopRel_{ir} := f_{top.r}\left(x_{r_{ir}}\right) \qquad TopRel^{T} = (0.000 \;\; -0.028 \;\; -0.042 \;\; 0.044 \;\; 0.122 \;\; 0.146 \;\; 0.117 \;\; 0.138) \cdot ksi$

$CheckTopRel := if\left[\left(max(TopRel) \le f_{c.all.rel}\right) \cdot \left(min(TopRel) \ge f_{t.all.rel}\right), "OK", "No\ Good"\right] = "OK"$

$BotRel_{ir} := f_{bot.r}\left(x_{r_{ir}}\right) \qquad BotRel^{T} = (0.000 \;\; 2.322 \;\; 2.918 \;\; 2.736 \;\; 2.572 \;\; 2.521 \;\; 2.583 \;\; 2.538) \cdot ksi$

$CheckBotRel := if\left[\left(max(BotRel) \le f_{c.all.rel}\right) \cdot \left(min(BotRel) \ge f_{t.all.rel}\right), "OK", "No\ Good"\right] = "OK"$

Concrete Stresses During Lifting and Transportation

Lifting and transportation stresses can govern over final stresses due to different support locations, dynamic effects that dead load can cause during movement, bending stresses during lifting and superelevation of the roadway in shipping. End diaphragms on both ends are assumed. For prestressing effects, calculate the effective prestressed force losses between transfer and building.

$a := h = 4.125 \cdot ft$ Maximum distance to lift point from bearing line

$a' := a + L_{end} = 6.125 \cdot ft$ Distance to lift point from end of beam

$P_{dia} := max\left(W_{ia}, W_{sa}\right) = 14.7 \cdot kip$ Approximate abutment weight

$P_m := P_j \cdot \left[1 - \frac{\left(\Delta f_{pES} + \Delta f_{pid}\right)}{f_{pj}}\right] = 1017.5 \cdot kip$ Effective prestress during lifting and shipping

Define locations for which stresses are to be calculated:

$$x_e := L_g \cdot \left(0 \quad min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \frac{a'}{L_g} \quad \alpha_{hd} \quad 0.5\right)^{T} \qquad ie := 1 .. last\left(x_e\right)$$

Compute moment in the girder during lifting with supports at the lift points.

$$M_{lift}(x) := \begin{cases} -\left[\frac{\left(w_g + w_{bar}\right) \cdot x^2}{2} + P_{dia} \cdot x\right] & \text{if } x \le a' \\ M_{gr}(x) - \left[M_{gr}(a') + \frac{\left(w_g + w_{bar}\right) \cdot (a')^2}{2} + P_{dia} \cdot a'\right] & \text{otherwise} \end{cases}$$

11. CONCRETE STRESSES (cont'd)

Functions for computing beam stresses:

$$f_{top.lift}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}$$

Top fiber stress during lifting

$$f_{top.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} \cdot (1 + DIM)$$

Top fiber stress during lifting, impact increasing dead load

$$f_{top.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} \cdot (1 - DIM)$$

Top fiber stress during lifting, impact decreasing dead load

$TopLift1_{ie} := f_{top.lift}\left(x_{e_{ie}}\right)$ $\qquad TopLift1^T = (0.000 \ \ -0.107 \ \ -0.140 \ \ -0.231 \ \ -0.104 \ \ -0.082) \cdot ksi$

$TopLift2_{ie} := f_{top.DIM.inc}\left(x_{e_{ie}}\right)$ $\qquad TopLift2^T = (0.000 \ \ -0.113 \ \ -0.148 \ \ -0.252 \ \ 0.014 \ \ 0.044) \cdot ksi$

$TopLift3_{ie} := f_{top.DIM.dec}\left(x_{e_{ie}}\right)$ $\qquad TopLift3^T = (0.000 \ \ -0.101 \ \ -0.133 \ \ -0.210 \ \ -0.223 \ \ -0.209) \cdot ksi$

$$f_{bot.lift}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}$$

Bottom fiber stress during lifting

$$f_{bot.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} \cdot (1 + DIM)$$

Bottom fiber stress during lifting, impact increasing dead load

$$f_{bot.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} \cdot (1 - DIM)$$

Bottom fiber stress during lifting, impact decreasing dead load

$BotLift1_{ie} := f_{bot.lift}\left(x_{e_{ie}}\right)$ $\qquad BotLift1^T = (0.000 \ \ 2.360 \ \ 2.965 \ \ 3.156 \ \ 2.888 \ \ 2.842) \cdot ksi$

$BotLift2_{ie} := f_{bot.DIM.inc}\left(x_{e_{ie}}\right)$ $\qquad BotLift2^T = (0.000 \ \ 2.372 \ \ 2.980 \ \ 3.201 \ \ 2.638 \ \ 2.574) \cdot ksi$

$BotLift3_{ie} := f_{bot.DIM.dec}\left(x_{e_{ie}}\right)$ $\qquad BotLift3^T = (0.000 \ \ 2.348 \ \ 2.949 \ \ 3.112 \ \ 3.139 \ \ 3.109) \cdot ksi$

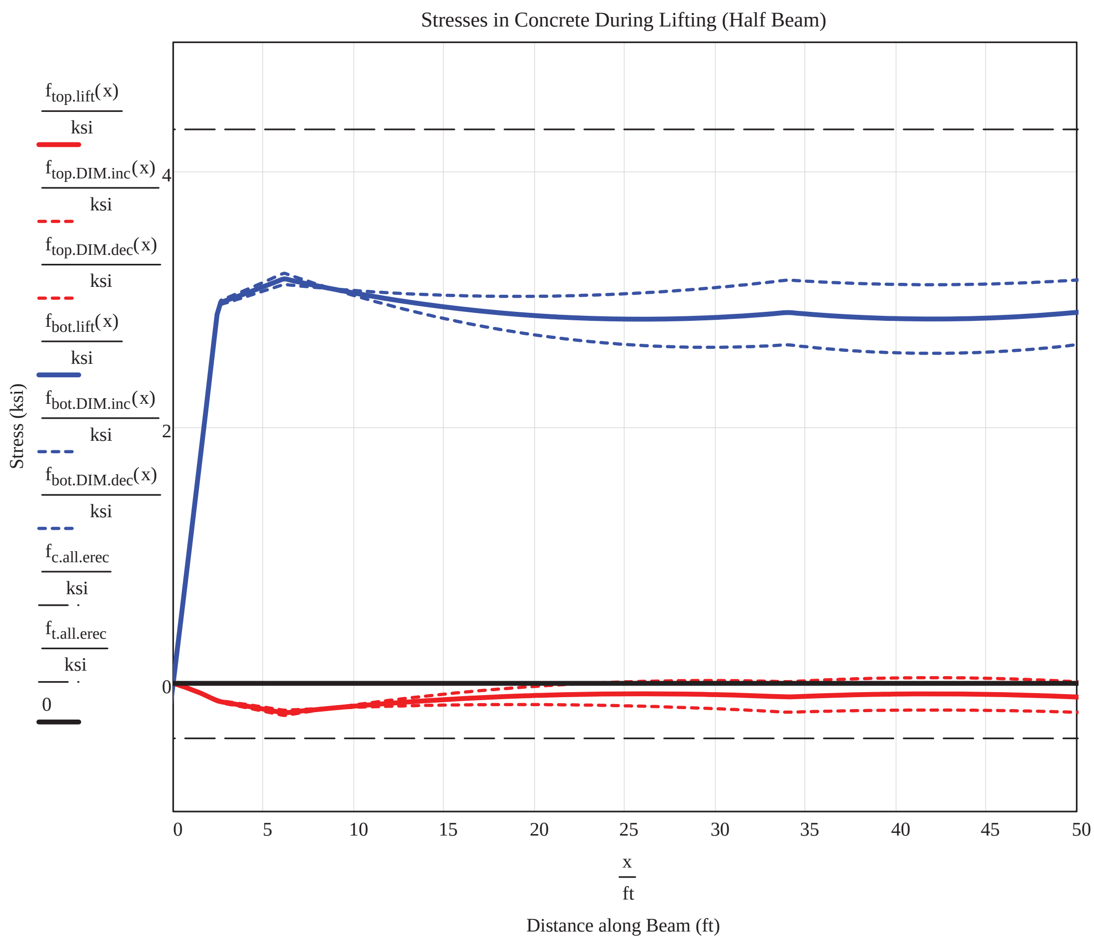
Allowable stresses during handling:

$f_{cm} := f_{c.erec}\left(f_c\right) = 7.2 \cdot ksi$ — Assumed concrete strength when handling operations begin

$f_{c.all.erec} := 0.6 \cdot f_{cm} = 4.32 \cdot ksi$ — Allowable compression during lifting and shipping

$f_{t.all.erec} := f_{t.erec}\left(f_{cm}\right) = -0.429 \cdot ksi$ — Allowable tension during lifting and shipping

11. CONCRETE STRESSES (cont'd)



Compare beam stresses to allowable stresses.

$TopLiftMax_{ie} := \max\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right)$ $\qquad TopLiftMax^T = (0\ \ -0.101\ \ -0.133\ \ -0.21\ \ 0.014\ \ 0.044)\cdot ksi$

$TopLiftMin_{ie} := \min\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right)$ $\qquad TopLiftMin^T = (0\ \ -0.113\ \ -0.148\ \ -0.252\ \ -0.223\ \ -0.209)\cdot ks$

$CheckTopLift := \text{if}\left[\left(\max(TopLiftMax) \le f_{c.all.erec}\right)\cdot\left(\min(TopLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$BotLiftMax_{ie} := \max\left(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie}\right)$ $\qquad BotLiftMax^T = (0\ \ 2.372\ \ 2.98\ \ 3.201\ \ 3.139\ \ 3.109)\cdot ksi$

$BotLiftMin_{ie} := \min\left(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie}\right)$ $\qquad BotLiftMin^T = (0\ \ 2.348\ \ 2.949\ \ 3.112\ \ 2.638\ \ 2.574)\cdot ksi$

$CheckBotLift := \text{if}\left[\left(\max(BotLiftMax) \le f_{c.all.erec}\right)\cdot\left(\min(BotLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

11. CONCRETE STRESSES (cont'd)

Concrete Stresses at Final

Stresses are calculated using design span length. The top flange compression and bottom flange under tension are computed at Service I and Service III states.

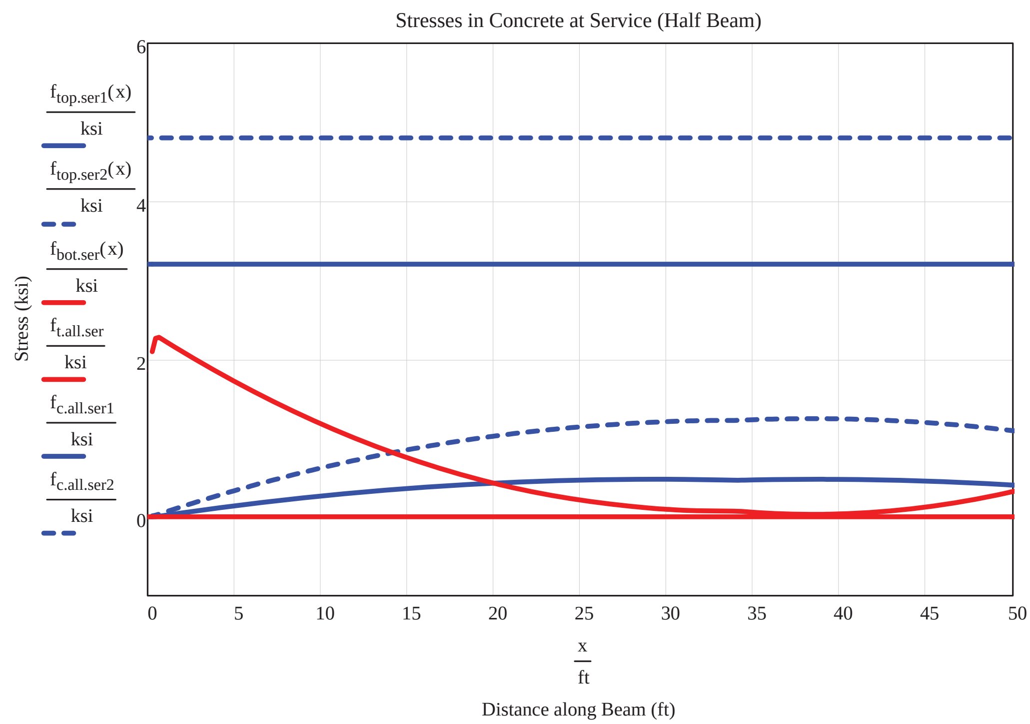
$f_{c.all.ser1} := 0.4 \cdot f_c = 3.2 \cdot ksi$	Allowable compression due to effective prestress and dead load (Table 5.9.4.2.1-1)
$f_{c.all.ser2} := 0.6 \cdot f_c = 4.8 \cdot ksi$	Allowable compression due to effective prestress, permanent load, and transient loads, as well as stresses during shipping and handling (Table 5.9.4.2.1-1)
$f_{t.all.ser} = 0 \cdot ksi$	Allowable tension (computed previously)
$P_e := f_{pe} \cdot A_{ps} = 889.2 \cdot kip$	Effective prestress after all losses

Compute stresses at midspan and compare to allowable values.

$$f_{top.ser1}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g(x + L_{end})}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x)}{S_{ttf}}$$

$$f_{top.ser2}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g(x + L_{end})}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + M_{ll}(x)}{S_{ttf}}$$

$$f_{bot.ser}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_g(x + L_{end})}{S_{bti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + 0.8 \cdot M_{ll}(x)}{S_{btf}}$$



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$$x_s := L\cdot\left(\frac{L_t}{L}\ \ 0.1\ \ 0.15\ \ 0.2\ \ 0.25\ \ 0.3\ \ 0.35\ \ \alpha_{hd}\ \ 0.45\ \ 0.5\right)^T \qquad is := 1 .. \text{last}(x_s)$$

$$TopSer1_{is} := f_{top.ser1}\left(x_{s_{is}}\right) \qquad TopSer1^T = (0.064\ \ 0.216\ \ 0.304\ \ 0.374\ \ 0.425\ \ 0.459\ \ 0.474\ \ 0.471\ \ 0.471\ \ 0.474)\cdot ksi$$

$$TopSer2_{is} := f_{top.ser2}\left(x_{s_{is}}\right) \qquad TopSer2^T = (0.164\ \ 0.508\ \ 0.716\ \ 0.887\ \ 1.021\ \ 1.119\ \ 1.184\ \ 1.218\ \ 1.236\ \ 1.240)\cdot ksi$$

$$CheckCompSerI := \text{if}\left[\left(\max(TopSer1) \le f_{c.all.ser1}\right)\cdot\left(\max(TopSer2) \le f_{c.all.ser2}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotSer_{is} := f_{bot.ser}\left(x_{s_{is}}\right) \qquad BotSer^T = (2.028\ \ 1.381\ \ 0.993\ \ 0.675\ \ 0.426\ \ 0.246\ \ 0.131\ \ 0.075\ \ 0.044\ \ 0.036)\cdot ksi$$

$$CheckTenSerIII := \text{if}\left(\min(BotSer) \ge f_{t.all.ser}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

12. FLEXURAL STRENGTH

Confirm flexural resistance at Strength Limit State. Calculate Factored moment at midspan during Strength I load combination. Compare this to factored resistance in AASHTO LRFD 5.7.3.

$M_{DC}(x) := M_g(x) + M_{bar}(x) + M_j(x)$ — Self weight of components

$M_{DW}(x) := M_{fws}(x)$ — Weight of future wearing surface

$M_{LL}(x) := M_{ll}(x)$ — Live load

$M_{StrI}(x) := 1.25\cdot M_{DC}(x) + 1.5\cdot M_{DW}(x) + 1.75\cdot M_{LL}(x)$ — Factored design moment

For minimum reinforcement check, per 5.7.3.3.2

$f_{cpe} := P_e\cdot\left(\frac{1}{A_g} + \frac{y_{cgp}}{S_{bg}}\right) = 3.551\cdot ksi$ — Concrete compression at extreme fiber due to effective prestress

$M_{cr} := -\left(f_{r.cm} + f_{cpe}\right)\cdot S_{bg} = 3541\cdot kip\cdot ft$ — Cracking moment (5.7.3.3.2-1)

$M_u(x) := \max\left(M_{StrI}(x), \min\left(1.33\cdot M_{StrI}(x), 1.2\cdot M_{cr}\right)\right)$ — Design moment

12. FLEXURAL STRENGTH (cont'd)

Compute factored flexural resistance.

$$\beta_1 := \max\left[0.65, 0.85 - 0.05 \cdot \left(\frac{f_c}{ksi} - 4\right)\right] = 0.65$$

Stress block factor (5.7.2.2)

$$k := 2 \cdot \left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$$

Tendon type factor (5.7.3.1.1-2)

$$d_p(x) := h - y_{px}\left(x + L_{end}\right) \qquad d_p(X) = 44.921 \cdot in$$

Distance from compression fiber to prestress centroid

$$h_f := d_7 = 8 \cdot in$$

Structural flange thickness

$$b_{taper} := \frac{b_6 - b_5}{2} = 16 \cdot in$$

Average width of taper at bottom of flange

$$h_{taper} := d_5 = 2 \cdot in$$

Depth of taper at bottom of flange

$$a(x) := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f_c \cdot b_f + \frac{k}{\beta_1} \cdot A_{ps} \cdot \left(\frac{f_{pu}}{d_p(x)}\right)} \qquad a(X) = 2.524 \cdot in$$

Depth of equivalent stress block for rectangular section

$$c(x) := \frac{a(x)}{\beta_1} \qquad c(X) = 3.883 \cdot in$$

Neutral axis location

$$CheckTC := \text{if}\left[\frac{c(X)}{d_p(X)} \le \left(\frac{.003}{.003 + .005}\right), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$$

Tension-controlled section check (midspan)

$$\varphi_f := \min\left[1.0, \max\left[0.75, 0.583 + 0.25 \cdot \left(\frac{d_p(X)}{c(X)} - 1\right)\right]\right] = 1.00$$

Resistance factor for prestressed concrete (5.5.4.2)

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c(X)}{d_p(X)}\right) = 263.5 \cdot ksi$$

Average stress in the prestressing steel (5.7.3.1.1-1)

$$L_d := \frac{1.6}{ksi} \cdot \left(f_{ps} - \frac{2}{3} \cdot f_{pe}\right) \cdot d_{ps} = 10.767 \cdot ft$$

Bonded strand devlepment length (5.11.4.2-1)

$$f_{px}(x) := \begin{cases} \dfrac{f_{pe} \cdot \left(x + L_{end}\right)}{L_t} & \text{if } x \le L_t - L_{end} \\ f_{pe} + \dfrac{\left(x + L_{end}\right) - L_t}{L_d - L_t} \cdot \left(f_{ps} - f_{pe}\right) & \text{if } L_t - L_{end} < x \le L_d - L_{end} \\ f_{ps} & \text{otherwise} \end{cases}$$

Stress in prestressing steel along the length for bonded strand (5.11.4.2)

$$M_r(x) := \varphi_f \cdot \left[A_{ps} \cdot f_{px}(x) \cdot \left(d_p(x) - \frac{a(x)}{2}\right)\right]$$

Flexure resistance along the length

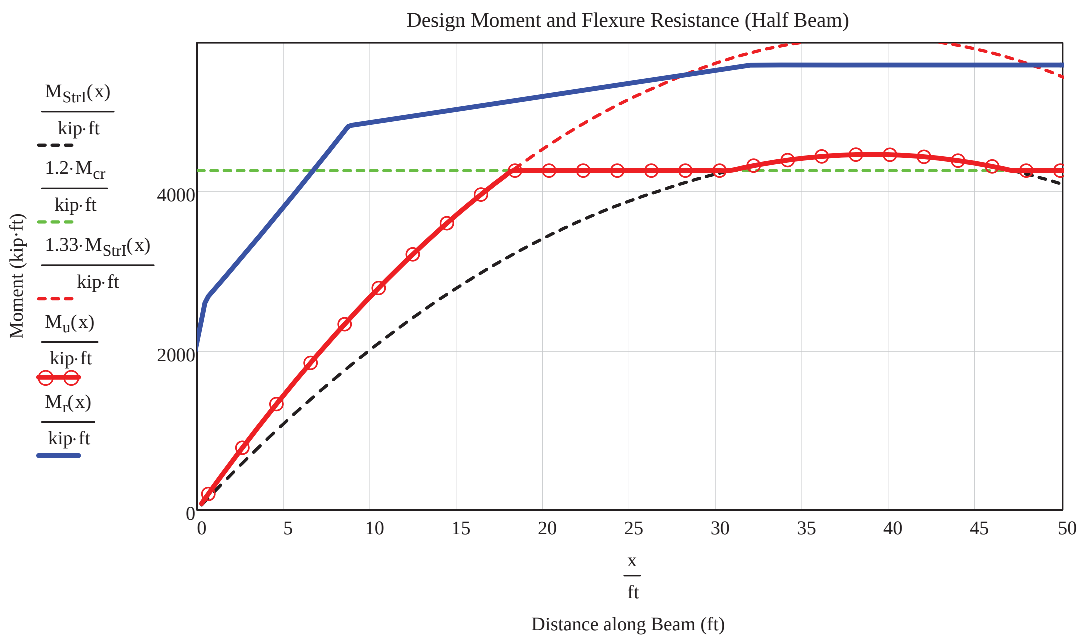
12. **FLEXURAL STRENGTH (cont'd)**

$$x_{mom} := L \cdot \left(0.01 \quad \frac{L_t - L_{end}}{L} \quad \frac{L_d - L_{end}}{L} \quad \alpha_{hd} \quad 0.5 \right)^T \qquad imom := 1 .. last(x_{mom})$$

$$M_{rx_{imom}} := M_r\left(x_{mom_{imom}}\right) \qquad M_{ux_{imom}} := M_u\left(x_{mom_{imom}}\right)$$

$$DC_{mom} := \frac{M_{ux}}{M_{rx}} \qquad max\left(DC_{mom}\right) = 0.798$$ Demand-Capacity ratio for moment

$$CheckMom := if\left(max\left(DC_{mom}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$ Flexure resistance check



13. SHEAR STRENGTH

Shear Resistance

Use Strength I load combination to calculate factored shear at the critical shear section and at tenth points along the span. Compare it to factored resistance in AASHTO LRFD 5.8.

$V_{DC}(x) := V_g(x) + V_{bar}(x) + V_j(x)$ — Self weight of components

$V_{DW}(x) := V_{fws}(x)$ — Weight of future wearing surface

$V_{LL}(x) := V_{ll}(x)$ — Live load

$V_u(x) := 1.25 \cdot V_{DC}(x) + 1.5 \cdot V_{DW}(x) + 1.75 \cdot V_{LL}(x)$ — Factored design shear

$\varphi_v := 0.90$ — Resistance factor for shear in normal weight concrete (AASHTO LRFD 5.5.4.2)

$d_{end} := h - y_{px}(L_{end}) = 36.711 \cdot in$ — Depth to steel centroid at bearing

$d_v := \min(0.9 \cdot d_{end}, 0.72 \cdot h) = 33.039 \cdot in$ — Effective shear depth lower limit at end

$$V_p(x) := \begin{cases} P_e \cdot slope_{cgp} \cdot \dfrac{x + L_{end}}{L_t} & \text{if } x \le L_t - L_{end} \\ P_e \cdot slope_{cgp} & \text{if } L_t - L_{end} < x \le \alpha_{hd} \cdot L \\ 0 & \text{otherwise} \end{cases}$$

Vertical component of effective prestress force

$b_v := b_3 = 6 \cdot in$ — Web thickness

$$v_u(x) := \frac{\left|V_u(x) - \varphi_v \cdot V_p(x)\right|}{\varphi_v \cdot b_v \cdot d_v}$$

Shear stress on concrete (5.8.2.9-1)

$M_{ushr}(x) := \max\left(M_{StrI}(x), \left|V_u(x) - V_p(x)\right| \cdot d_v\right)$ — Factored moment for shear

$f_{po} := 0.7 \cdot f_{pu} = 189 \cdot ksi$ — Stress in prestressing steel due to locked-in strain after casting concrete

$$\varepsilon_s(x) := \max\left(-0.4 \cdot 10^{-3}, \frac{\dfrac{\left|M_u(x)\right|}{d_v} + \left|V_u(x) - V_p(x)\right| - A_{ps} \cdot f_{po}}{E_p \cdot A_{ps}}\right)$$

Steel strain at the centroid of the prestressing steel

$$\beta(x) := \frac{4.8}{1 + 750 \cdot \varepsilon_s(x)}$$

Shear resistance parameter

$\theta(x) := (29 + 3500 \cdot \varepsilon_s(x)) \cdot deg$ — Principal compressive stress angle

$$V_c(x) := 0.0316 \cdot ksi \cdot \beta(x) \cdot \sqrt{\frac{f_c}{ksi}} \cdot b_v \cdot d_v$$

Concrete contribution to total shear resistance

13. SHEAR STRENGTH (cont'd)

$\alpha := 90 \cdot deg$ — Angle of inclination of transverse reinforcement

$A_v := (1.02 \;\; 0.62 \;\; 0.62 \;\; 0.62 \;\; 0.31)^T \cdot in^2$ $\quad s_v := (3 \;\; 6 \;\; 6 \;\; 12 \;\; 12)^T \cdot in$ — Transverse reinforcement area and spacing provided

$x_v := (0 \;\; 0.25 \cdot h \;\; 1.5 \cdot h \;\; 0.3 \cdot L \;\; 0.5 \cdot L \;\; 0.6 \cdot L)^T$ $\quad x_v^T = (0 \;\; 1.031 \;\; 6.187 \;\; 24 \;\; 40 \;\; 48) \cdot ft$

$$A_{vs}(x) := \left| \begin{array}{l} \text{for } i \in 1 .. \text{last}(A_v) \\ \quad out \leftarrow \dfrac{A_{v_i}}{s_{v_i}} \text{ if } x_{v_i} \le x \le x_{v_{i+1}} \\ out \end{array} \right.$$

.

$V_s(x) := A_{vs}(x) \cdot f_y \cdot d_v \cdot (\cot(\theta(x)) + \cot(\alpha)) \cdot \sin(\alpha)$ — Steel contribution to total shear resistance

$V_r(x) := \varphi_v \cdot (V_c(x) + V_s(x) + V_p(x))$ — Factored shear resistance

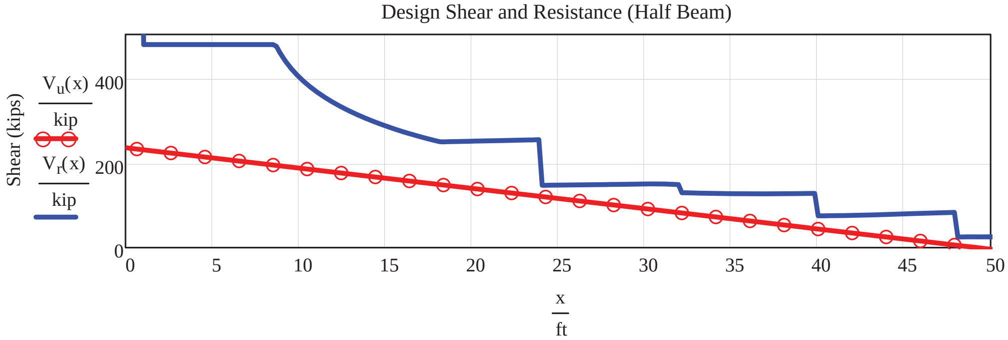
$$x_{shr} := \left| \begin{array}{l} \text{for } i \in 1 .. 100 \\ \quad out_i \leftarrow i \cdot \dfrac{0.5 \cdot L}{100} \\ out \end{array} \right. \qquad ishr := 1 .. \text{last}(x_{shr})$$

$V_{ux_{ishr}} := V_u\left(x_{shr_{ishr}}\right)$ $\quad V_{rx_{ishr}} := V_r\left(x_{shr_{ishr}}\right)$

$DC_{shr} := \dfrac{V_{ux}}{V_{rx}}$ $\quad \max(DC_{shr}) = 0.822$

$CheckShear := \text{if}\left(\max(DC_{shr}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Shear resistance check

Design Shear and Resistance (Half Beam)



Distance along Beam (ft)

13. SHEAR STRENGTH (cont'd)

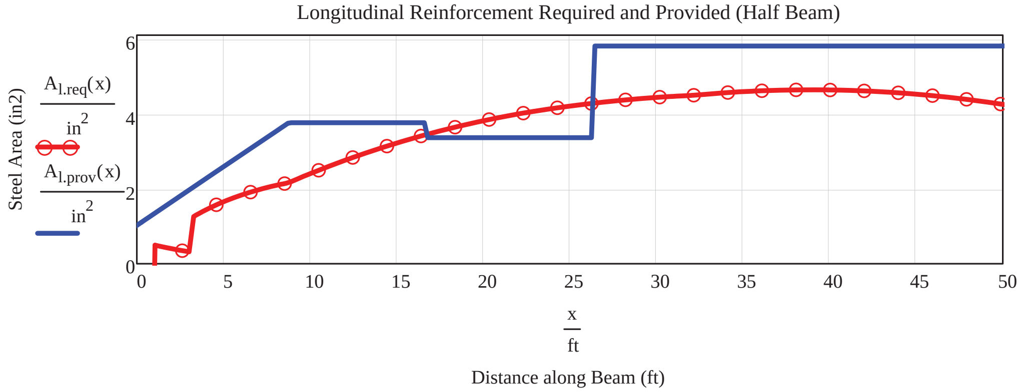
Longitudinal Reinforcement

$$A_{l.req}(x) := \left| \begin{array}{l} a1 \leftarrow \dfrac{M_{StrI}(x)}{\varphi_f \cdot f_{px}(x) \cdot \left(d_p(x) - \dfrac{a(x)}{2}\right)} \\ a2 \leftarrow \dfrac{\left(\dfrac{V_u(x)}{\varphi_v} - 0.5 \cdot V_s(x) - V_p(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ a3 \leftarrow \dfrac{\dfrac{M_{ushr}(x)}{d_v \cdot \varphi_f} + \left(\left|\dfrac{V_u(x)}{\varphi_v} - V_p(x)\right| - 0.5 \cdot V_s(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ \min(a1, a2) \quad \text{if } x \le d_v + 5 \cdot in \\ \min(a1, a3) \quad \text{otherwise} \end{array} \right.$$

Longitudinal reinforcement required for shear (5.8.3.5)

$A_{s.add} := 0.40 \cdot in^2$ $\qquad L_{d.add} := 18.67 \cdot ft$ $\qquad$ Additional longitudinal steel and developed length from end of beam

$$A_{l.prov}(x) := \text{if}\left(x < L_{d.add} - L_{end}, A_{s.add}, 0\right) + \left| \begin{array}{l} A_p \cdot N_s \cdot \dfrac{x + L_{end}}{L_d} \quad \text{if } x \le L_d - L_{end} \\ A_p \cdot N_s \quad \text{if } L_d - L_{end} < x \le \dfrac{y_{h.brg} - 0.5 \cdot h}{slope_{cgp}} + \left(\dfrac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) \cdot \cot\left(slope_{cgp}\right) \\ A_p \cdot \left(N_h + N_s\right) \quad \text{otherwise} \end{array} \right.$$



$A_{l.req_{ishr}} := A_{l.req}\left(x_{shr_{ishr}}\right)$ $\qquad A_{l.prov_{ishr}} := A_{l.prov}\left(x_{shr_{ishr}}\right)$

$DC_{long} := \dfrac{A_{l.req}}{A_{l.prov}}$ $\qquad \max\left(DC_{long}\right) = 1.268$

$CheckLong := \text{if}\left(\max\left(DC_{long}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"No Good"}$ $\qquad$ Longitudinal reinforcement check

14. SPLITTING RESISTANCE

Splitting Resistance

Checking splitting by zone of transverse reinforcement. Defined in Shear Strength section.

$$A_s := \frac{A_{v_1} \cdot x_{v_2}}{s_{v_1}} = 4.208 \cdot in^2$$

$f_s := 20 \cdot ksi$ — Limiting stress in steel for crack control (5.10.10.1)

$P_r := f_s \cdot A_s = 84.2 \cdot kip$ — Splitting resistance provided (5.10.10.1-1)

$P_{r.min} := 0.04 \cdot P_j = 47.1 \cdot kip$ — Minimum splitting resistance required

$CheckSplit := if\left(P_r \geq P_{r.min}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Splitting resistance check

15. CAMBER AND DEFLECTIONS

Calculate Deflections due to different weights, joints, and future wearings.

$$\Delta_{ps} := \frac{-P_i}{E_{ci} \cdot I_{xg}} \cdot \left[\frac{y_{cgp} \cdot L_g^{\,2}}{8} - \frac{\left(y_{bg} + y_{p.brg}\right) \cdot \left(\alpha_{hd} \cdot L + L_{end}\right)^2}{6}\right] = 2.246 \cdot in$$

Deflection due to prestress at release

$$\Delta_{gr} := \frac{-5}{384} \cdot \frac{w_g \cdot L_g^{\,4}}{E_{ci} \cdot I_{xg}} = -1.006 \cdot in$$

Deflection due to self-weight at release

$$\Delta_{bar} := \frac{-5}{384} \cdot \frac{w_{bar} \cdot L_g^{\,4}}{E_c \cdot I_{xg}} = -0.286 \cdot in$$

Deflection due to barrier weight

$$\Delta_j := \frac{-5}{384} \cdot \frac{w_j \cdot L^4}{E_c \cdot I_{xg}} \cdot if\,(BeamLoc = 0, 1, 0.5) = -0.014 \cdot in \qquad 2$$

Deflection due to longitudinal joint

$$\Delta_{fws} := \frac{-5}{384} \cdot \frac{w_{fws} \cdot L^4}{E_c \cdot I_{xg}} \cdot if\left(BeamLoc = 0, 1, \frac{S - W_b}{S}\right) = -0.088 \cdot in$$

Deflection due to future wearing surface

$t_{bar} := 20$ — Age at which barrier is assumed to be cast

$T := \left(t_i \;\; 7 \;\; 14 \;\; 21 \;\; 28 \;\; 60 \;\; 120 \;\; 240 \;\; \infty\right)^T$ — Concrete ages at which camber is computed

15. CAMBER AND DEFLECTIONS (cont'd)

$$\Delta_{cr1}(t) := \psi(t - t_i, t_i)(\Delta_{gr} + \Delta_{ps})$$

$$\Delta_{cr2}(t) := (\psi(t - t_i, t_i) - \psi(t_{bar} - t_i, t_i)) \cdot (\Delta_{gr} + \Delta_{ps}) + \psi(t - t_{bar}, t_{bar}) \cdot \Delta_{bar}$$

$$\Delta_{cr3}(t) := (\psi(t - t_i, t_i) - \psi(t_d - t_i, t_i)) \cdot (\Delta_{gr} + \Delta_{ps}) + (\psi(t - t_{bar}, t_{bar}) - \psi(t_d - t_{bar}, t_{bar})) \cdot \Delta_{bar} \ldots \\ + \psi(t - t_d, t_d) \cdot (\Delta_j)$$

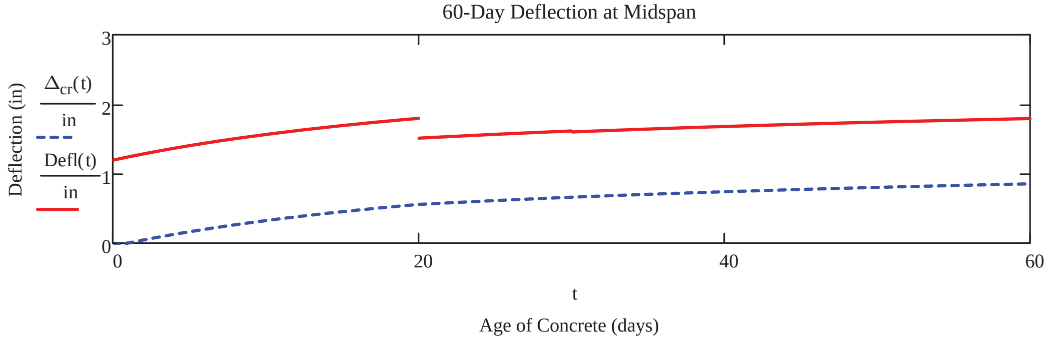
$$\Delta_{cr}(t) := \begin{cases} \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t_d) + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

$$\text{Defl}(t) := \begin{cases} (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t_d) + \Delta_j + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

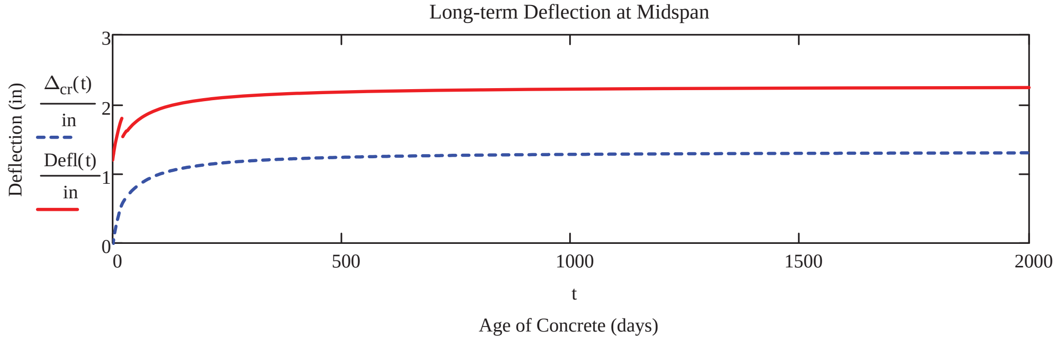
$$C := \begin{array}{|l} \text{for } j \in 1 .. \text{last}(T) \\ \quad out_j \leftarrow \text{Defl}(T_j) \\ out \end{array}$$

$$C^T = (1.24 \quad 1.471 \quad 1.668 \quad 1.522 \quad 1.595 \quad 1.793 \quad 1.968 \quad 2.094 \quad 2.262) \cdot in$$

60-Day Deflection at Midspan



Long-term Deflection at Midspan



16. NEGATIVE MOMENT FLEXURAL STRENGTH

Calculate factored moment that must be resisted across the interior pier and find required steel to be developed in the top flange.

Negative Live Load Moment

Compute the negative moment over the interior support due to the design live load load, in accordance with AASHTO LRFD 3.6.1.3.1.

Live Load Truck and Truck Train Moment Calculations

$\min(M_{truck}) = -1037 \cdot kip \cdot ft$	Maximum negative moment due to a single truck
$\min(M_{train}) = -2038 \cdot kip \cdot ft$	Maximum negative moment due to two trucks in a single lane
$M_{neg.lane} := \frac{-w_{lane} \cdot L^2}{2} = -2048 \cdot kip \cdot ft$	Negative moment due to lane load on adjacent spans
$M_{neg.truck} := M_{neg.lane} + (1 + IM) \cdot \min(M_{truck}) = -3427 \cdot kip \cdot ft$	Live load negative moment for single truck
$M_{neg.train} := 0.9 \cdot \left[M_{neg.lane} + (1 + IM) \cdot \min(M_{train})\right] = -4282 \cdot kip \cdot ft$	Live load negative moment for two trucks in a single lane
$M_{HL93.neg} := \min(M_{neg.truck}, M_{neg.train}) = -4282 \cdot kip \cdot ft$	Design negative live load moment, per design lane
$M_{ll.neg.i} := M_{HL93.neg} \cdot g_{mint} = -2720 \cdot kip \cdot ft$	Design negative live load moment at interior beam
$M_{ll.neg.e} := M_{HL93.neg} \cdot g_{mext} = -2832 \cdot kip \cdot ft$	Design negative live load moment at exterior beam
$M_{LL.neg} := \text{if}(BeamLoc = 1, M_{ll.neg.e}, M_{ll.neg.i}) = -2832 \cdot kip \cdot ft$	Design negative live load moment

Factored Negative Design Moment

Dead load applied to the continuity section at interior supports is limited to the future overlay.

$M_{DW.neg} := \frac{-w_{fws} \cdot L^2}{2} = -635 \cdot kip \cdot ft$	Superimposed dead load resisted by continuity section
$M_{u.neg.StrI} := 1.5 \cdot M_{DW.neg} + 1.75 \cdot M_{LL.neg} = -5908 \cdot kip \cdot ft$	Strength Limit State
$M_{u.neg.StrI} := 1.0 \cdot M_{DW.neg} + 1.0 \cdot M_{LL.neg} = -3467 \cdot kip \cdot ft$	Service Limit State

16. NEGATIVE MOMENT FLEXURAL STRENGTH (cont'd)

Reinforcing Steel Requirement in the Top Flange for Strength

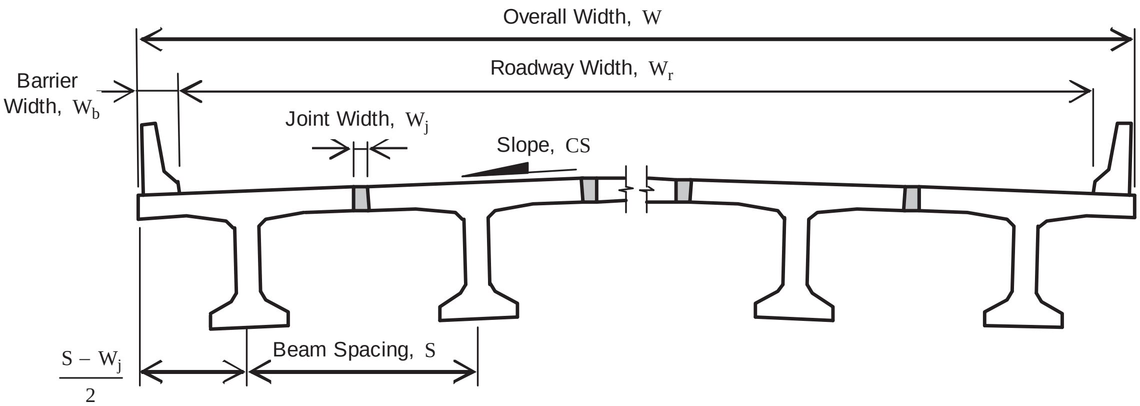
$\varphi_f := 0.90$	Reduction factor for strength in tension-controlled reinforced concrete (5.5.4.2)
$b_c := b_1 = 26\cdot in$	Width of compression block at bottom flange
$d_{nms} := h - t_{sac} - 0.5\cdot(t_{flange} - t_{sac}) = 44.5\cdot in$	Distance to centroid of negative moment steel, taken at mid-depth of top flange
$R_u := \dfrac{\lvert M_{u.neg.StrI}\rvert}{\varphi_f\cdot b_c\cdot d_{nms}^{\,2}} = 0.898\cdot ksi$	Factored load, in terms of stress in concrete at depth of steel, for computing steel requirement
$m := \dfrac{f_y}{0.85\cdot f_c} = 8.824$	Steel-to-concrete strength ratio
$\rho_{req} := \dfrac{1}{m}\cdot\left(1 - \sqrt{1 - \dfrac{2\cdot m\cdot R_u}{f_y}}\right) = 0.0161$	Required negative moment steel ratio
$A_{nms.req} := \rho_{req}\cdot b_c\cdot d_{nms} = 18.638\cdot in^2$	Required negative moment steel in top flange
$A_{s.long.t} := 2.0\cdot in^2$ $\quad A_{s.long.b} := 2.0\cdot in^2$	Full-length longitudinal reinforcement to be made continuous across joint
$A_{bar} := 0.44\cdot in^2$	Additional negative moment reinforcing bar area
$A_{nms.t} := \dfrac{2}{3}\cdot A_{nms.req} - A_{s.long.t} = 10.425\cdot in^2$	Additional reinforcement area required in the top mat (2/3 of total)
$n_{bar.t} := ceil\left(\dfrac{A_{nms.t}}{A_{bar}}\right) = 24$	Additional bars required in the top mat
$A_{nms.b} := \dfrac{1}{3}\cdot A_{nms.req} - A_{s.long.b} = 4.213\cdot in^2$	Additional reinforcement area required in the bottom mat
$n_{bar.b} := ceil\left(\dfrac{A_{nms.b}}{A_{bar}}\right) = 10$	Additional bars required in the top mat
$s_{bar.top} := \dfrac{S - W_j - 6\cdot in}{n_{bar.t} - 1} = 3.62\cdot in$	Spacing of bars in top mat
$A_{s.nms} := (n_{bar.t} + n_{bar.b})\cdot A_{bar} + A_{s.long.t} + A_{s.long.b} = 18.96\cdot in^2$	Total reinforcing steel provided over pier
$a := \dfrac{A_{s.nms}\cdot f_y}{0.85\cdot f_c\cdot b_c} = 6.434\cdot in$	Depth of compression block
$M_{r.neg} := \varphi_f\cdot A_{s.nms}\cdot f_y\cdot\left(d_{nms} - \dfrac{a}{2}\right) = 3522\cdot kip\cdot ft$	Factored flexural resistance at interior pier
$DC_{neg.mom} := \dfrac{\lvert M_{u.neg.StrI}\rvert}{M_{r.neg}} = 0.984$	
CheckNegMom := if$(DC_{neg.mom} \le 1.0$, "OK" , "No Good" $)$ = "OK"	Negative flexure resistance check

File Name: Prestressed Concrete Girder-60ft.xmcd

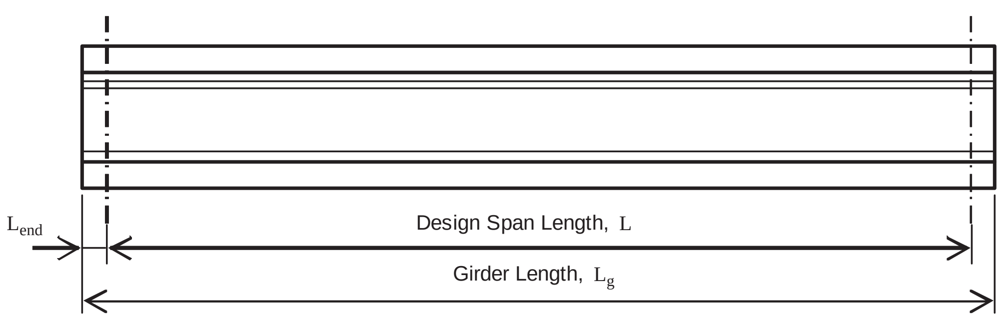
DECKED PRECAST PRESTRESSED CONCRETE GIRDER DESIGN FOR ABC

Unit Definition: $kcf \equiv kip \cdot ft^{-3}$

This example is for the design of a superstructure system that can be used for rapid bridge replacement in an Accelerated Bridge Construction (ABC) application. The following calculations are intended to provide the designer guidance in developing a similar design with regard to design considerationS characteristic of this type of construction, and they shall not be considered fully exhaustive.



TYPICAL SECTION THROUGH SPAN



GIRDER ELEVATION

Bridge Geometry:

$L := 60 \cdot ft$ $\quad L_{end} := 2 \cdot ft$ $\quad skew := 0 \cdot deg$

$W := 63 \cdot ft$ $\quad W_b := 1.5 \cdot ft$

$S_{max} := 8 \cdot ft$ $\quad W_j := 0.5 \cdot ft$

$N_g := ceil\left(\frac{W + W_j}{S_{max}}\right) = 8$ Minimum number of girders in cross-section

$S := \frac{W + W_j}{N_g} = 7.938 \cdot ft$ Girder spacing

ORDER OF CALCULATIONS

1. Introduction
2. Design Philosophy
3. Design Criteria
4. Beam Section
5. Material Properties
6. Permanent Loads
7. Precast Lifting Weight
8. Live Load
9. Prestress Properties
10. Prestress Losses
11. Concrete Stresses
12. Flexural Strength
13. Shear Strength
14. Splitting Resistance
15. Camber and Deflections
16. Negative Moment Flexural Strength

1. INTRODUCTION

The bridge that is designed in this example consists of precast prestressed concrete girders with a top flange equal to the beam spacing, so the top flange will be the riding surface of the designed bridge. The purpose for these girders is to rapidly construct the bridge by providing a precast deck on the girders, which eliminates cast-in-place decks in the field and improves safety.

The concepts used in this example have been taken from on-going research, which focuses on the benefits of decked precast beams and promoting widespread acceptance from transportation and construction industries. The cross-section is adapted from the optimized girder sections recommended by NCHRP Project No. 12-69, Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges. The girder design has not taken into account the option to re-deck due to the final re-decked girder, without additional prestressed, having a shorter life span. Use of stainless steal rebar and the application of a future membrane can get ride of the need to replace the deck. This case is included in “re-deckability”.

The bridge used in this example is a general design of a typical bridge in Georgia. The calculations can be modified for single-span and multiple-span bridges due to the beam design moments are not reduced for continuity at intermediate supports (continuity details are not shown in this example). The cross-section consists of a four-lane roadway with normal crown, with standard shoulder lengths and barrier walls. The precast prestressed concrete girder has been uniformly designed to simplify bearing details. The girder flanges are 9’’ at the tips, imitating a 8’’ slab with a ½’’ allowable wear and another ½’’ for smoothness and profile adjustments.

This example is intended to illustrate design aspect specific to precast prestressed concrete girders used for ABC application. Girders with uncommon cross-sections, high self-weight, or unconventional load application create major concerns and more detailed calculations must be done.

2. DESIGN PHILOSOPHY

The geometry of the section is based on GDOT standards and general bridges across the state of Georgia. Depth variations are dependent on the construction company but must maintain the shapes of the top flange and the bottom bulb.

Concrete strengths can vary but are mostly between 6 ksi and 10 ksi. For the purpose of these calculations the concrete with a 28-day minimum compressive strength of 8 ksi is used. Due to its casting sequence this beam is unable to take advantage of composite sequences along with tension at the bottom of the beam at the service limit state being limited. This is further discussed in section 4 along with end region stresses being critical. Therefore the minimum concrete strength at release must be 80 percent of the 28-day compressive strength, which increases the allowable stresses at the top and bottom of the section. The prestressed steel can also be optimized to minimize stresses at the end region.

The prestressed steel is arranged in a draped, or harped, pattern to maximize the midspan effectiveness while it minimizes the failure at the end of the beam where is concrete is easily overstressed due to the lack of dead load acting on the beam. The strand group is optimized at the midspan by bundling the strands between hold-points, maximizing the stiffness of the strand group. The number and deflection angles are depended on the type of single strands you are using for the girder. In longer span cases the concrete at the end of the girder will be too large and will debond. Without harped strands it is unlikely to reduce stresses to the allowable limit, since harped strands are required this method of stress relief will be used without debonding for long spans.

3. DESIGN CRITERIA

Criteria has been selected to govern the design of these concrete girders while following provisions set by AASHTO, GDOT design specifications, as well as criteria of past projects and current research related to ABC and decked precast sections. A summary of the limiting design values are categorized as section constraints, prestress limits, and concrete limits.

Section Constraints:

$W_{pc.max} := 200 \cdot kip$ — Upper limit on the weight of the entire precast element, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

$S_{max} = 8 \cdot ft$ — Upper limit on girder spacing and, therefore, girder flange width (defined on first page)

Prestress Limits:

$F_{hd.single} := 4 \cdot kip$ — Maximum hold-down force for a single strand

$F_{hd.group} := 48 \cdot kip$ — Maximum hold-down force for the group of harped strands

Stress limits in the prestressing steel immediately prior to prestress and at the service limit state after all losses are as prescribed by AASHTO LRFD.

3. DESIGN CRITERIA (cont'd)

Concrete Limits:

Allowable concrete stresses meet standards set by AASHTO LRFD with one exception that at Service III Limit State, allowable bottom fiber tension when camber leveling forces are to be neglected, regardless of exposure, are to be 0-ksi. Minimum strength of concrete at release is 80 percent of the 28-day minimum compressive strength (f-ksi).

$f_{t.all.ser} := 0 \cdot ksi$	Allowable bottom fiber tension at the Service III Limit State, when camber leveling forces are to be neglected, regardless of exposure

As previously mentioned, release concrete strength is specified as 80 percent of the minimum 28-day compressive strength to maximize allowable stresses in the end region of beam at release.

$f_{c.rel}(f) := 0.80 \cdot f$	Minimum strength of concrete at release

Due to various lifting and transportation conditions, stresses in the concrete need to be considered. A "no cracking" approach is used for allowable tension due to reduced lateral stability after cracking. Assuming the girders will be lifted before the 28-day minimum strength is attained, the strength of concrete during lifting and transportation is assumed to be 90 percent of the 28-day minimum compressive strength. A dynamic dead load allowance of 30 percent is used for compression during handling. A factor of safety (FS) of 1.5 is used against cracking during handling.

$DIM := 30\%$	Dynamic dead load allowance
$f_{c.erec}(f) := 0.90 \cdot f$	Assumed attained concrete strength during lifting and transportation
$FS_c := 1.5$	Factor of safety against cracking during lifting transportation
$f_{t.erec}(f) := \dfrac{-0.24 \cdot \sqrt{f \cdot ksi}}{FS_c}$	Allowable tension in concrete during lifting and transportation to avoid cracking

4. BEAM SECTION

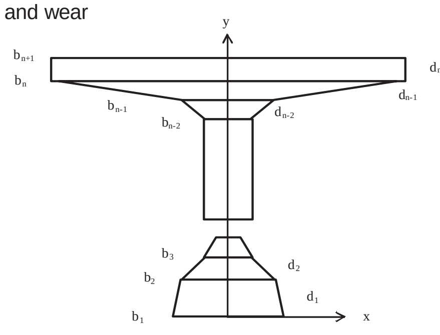
Use trapezoidal areas to define the cross-section. The flange width is defined as the beam spacing less the width of the longitudinal closure joint to reflect pre-erection conditions. Live load can be conservatively applied to this section, as well.

$h := 45 \cdot in$ Beam section depth

$t_{flange} := 9 \cdot in$ Flange thickness at tip

$t_{sac} := 1 \cdot in$ Total sacrificial depth for grinding and wear

$b_1 := 26 \cdot in$	$b_2 := 26 \cdot in$	$d_1 := 6 \cdot in$
$b_2 = 26 \cdot in$	$b_3 := 6 \cdot in$	$d_2 := 4.5 \cdot in$
$b_3 = 6 \cdot in$	$b_4 := 6 \cdot in$	
$b_4 = 6 \cdot in$	$b_5 := 10 \cdot in$	$d_4 := 2 \cdot in$
$b_5 = 10 \cdot in$	$b_6 := 42 \cdot in$	$d_5 := 2 \cdot in$
$b_6 = 42 \cdot in$	$b_7 := S - W_j$	$d_6 := 0 \cdot in$
$b_7 = 89.25 \cdot in$	$b_8 := S - W_j$	$d_7 := t_{flange} - t_{sac}$
$d_3 := h - t_{sac} - \sum d$		$d_3 = 21.5 \cdot in$



TYPICAL GIRDER SECTION COMPRISED
OF n TRAPEZOIDAL REGIONS

Gross Section Properties

$b_f = 89.25 \cdot in$ Precast girder flange width

$A_g = 1139 \cdot in^2$ Cross-sectional area (does not include sacrifical thickness)

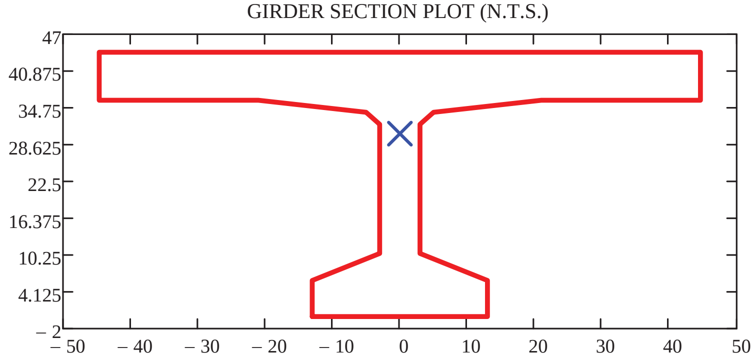
$I_{xg} = 241240 \cdot in^4$ Moment of inertia (does not include sacrificial thickness)

$y_{tg} = 13.544 \cdot in$ $y_{bg} = -30.456 \cdot in$ Top and bottom fiber distances from neutal axis (positive up)

$S_{tg} = 17811.7 \cdot in^3$ $S_{bg} = -7920.9 \cdot in^3$ Top and bottom section moduli

$I_{yg} = 487677 \cdot in^4$ Weak-axis moment of inertia

GIRDER SECTION PLOT (N.T.S.)



5. MATERIAL PROPERTIES

These properties are standard (US units) values with equations that can be found in AASHTO LRFD Bridge Design Specifications.

Concrete:

$f_c := 8 \cdot ksi$	Minimum 28-day compressive strength of concrete
$f_{ci} := f_{c.rel}(f_c) = 6.4 \cdot ksi$	Minimum strength of concrete at release
$\gamma_c := .150 \cdot kcf$	Unit weight of concrete
$K_1 := 1.0$	Correction factor for standard aggregate (5.4.2.4)
$E_{ci} := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_{ci} \cdot ksi} = 4850 \cdot ksi$	Modulus of elasticity at release (5.4.2.4-1)
$E_c := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 5422 \cdot ksi$	Modulus of elasticity (5.4.2.4-1)
$f_{r.cm} := 0.37 \cdot \sqrt{f_c \cdot ksi} = 1.047 \cdot ksi$	Modulus of rupture for cracking moment (5.4.2.6)
$f_{r.cd} := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.679 \cdot ksi$	Modulus of rupture for camber and deflection (5.4.2.6)
$H := 70$	Relative humidity (5.4.2.3)

Prestressing Steel:

$f_{pu} := 270 \cdot ksi$	Ultimate tensile strength
$f_{py} := 0.9 \cdot f_{pu} = 243 \cdot ksi$	Yield strength, low-relaxation strand (Table 5.4.4.1-1)
$f_{pbt.max} := 0.75 \cdot f_{pu} = 202.5 \cdot ksi$	Maximum stress in steel immediately prior to transfer
$f_{pe.max} := 0.80 \cdot f_{py} = 194.4 \cdot ksi$	Maximum stress in steel after all losses
$E_p := 28500 \cdot ksi$	Modulus of elasticity (5.4.4.2)
$d_{ps} := 0.5 \cdot in$	Strand diameter
$A_p := 0.153 \cdot in^2$	Strand area
$N_{ps.max} := 40$	Maximum number of strands in section
$n_{pi} := \frac{E_p}{E_{ci}} = 5.9$	Modular ratio at release
$n_p := \frac{E_p}{E_c} = 5.3$	Modular ratio

Mild Steel:

$f_y := 60 \cdot ksi$ Specified minimum yield strength

$E_s := 29000 \cdot ksi$ Modulus of elasticity (5.4.3.2)

6. PERMANENT LOADS

Permanent loads or dead loads that must be considered are self-weight, diaphragms, barriers, and future wearing surface. The barrier can be cast to the beam before it is taken on sight or attached to the bridge after the joints have reached sufficient strength. Distribution of the barriers weight should be established once you decide when it would be attached to the bridge. For this example the barrier has been cast on the exterior girder in the casting yard, before shipping but after release of prestresses. Due to this the dead load is increased on the exterior girders but it eliminates the time-consuming task that would have been completed in the field.

$\mathrm{BeamLoc} := 1$ — Location of beam within the cross-section (0 - Interior, 1 - Exterior)

Load at Release:

$\gamma_{c.DL} := .155 \cdot \mathrm{kcf}$ — Concrete density used for weight calculations

$A_{g.DL} := A_g + t_{sac} \cdot \left(S - W_j\right) = 1228.25 \cdot \mathrm{in}^2$ — Area used for weight calculations, including sacrificial thickness

$w_g := A_{g.DL} \cdot \gamma_{c.DL} = 1.322 \cdot \mathrm{klf}$ — Uniform load due to self-weight, including sacrificial thickness

$L_g := L + 2 \cdot L_{end} = 64 \cdot \mathrm{ft}$ — Span length at release

$M_{gr}(x) := \dfrac{w_g \cdot x}{2} \cdot \left(L_g - x\right)$ — Moment due to beam self-weight (supported at ends)

$V_{gr}(x) := w_g \cdot \left(\dfrac{L_g}{2} - x\right)$ — Shear due to beam self-weight (supported at ends)

Load at Erection:

$M_g(x) := \dfrac{w_g \cdot x}{2} \cdot (L - x)$ — Moment due to beam self-weight

$V_g(x) := w_g \cdot \left(\dfrac{L}{2} - x\right)$ — Shear due to beam self-weight

$w_{bar} := 0.430 \cdot \mathrm{klf}$ — Uniform load due to barrier weight, exterior beams only

$w_{bar} := \mathrm{if}\left(\mathrm{BeamLoc} = 1, w_{bar}, 0\right) = 0.43 \cdot \mathrm{klf}$ — Redfine to 0 if interior beam (BeamLoc = 0)

$M_{bar}(x) := \dfrac{w_{bar} \cdot x}{2} \cdot (L - x)$ — Moment due to beam self-weight

$V_{bar}(x) := w_{bar} \cdot \left(\dfrac{L}{2} - x\right)$ — Shear due to beam self-weight

6. PERMANENT LOADS (cont'd)

Load at Service:

$p_{fws} := 25 \cdot psf$ — Assumed weight of future wearing surface

$w_{fws} := p_{fws} \cdot S = 0.198 \cdot klf$ — Uniform load due to future wearing surface

$M_{fws}(x) := \frac{w_{fws} \cdot x}{2} \cdot (L - x)$ — Moment due to future wearing surface

$V_{fws}(x) := w_{fws} \cdot \left(\frac{L}{2} - x\right)$ — Shear due to future wearing surface

$w_j := W_j \cdot d_7 \cdot \gamma_{c.DL} = 0.052 \cdot klf$ — Uniform load due to weight of longitudinal closure joint

$M_j(x) := \frac{w_j \cdot x}{2} \cdot (L - x)$ — Moment due to longitudinal closure joint

$V_j(x) := w_j \cdot \left(\frac{L}{2} - x\right)$ — Shear due to longitudinal closure joint

7. PRECAST LIFTING WEIGHT

For Accelerated Bridge Construction the beams are casted in a factory and transported to the job site. When they arrive at the site they must be lifted and put into place. When designing we have to consider the weight of each slab to insure safety and design for possible cracking.

Precast Superstructure

$W_g := (w_g + w_{bar}) \cdot L_g = 112.1 \cdot kip$ — Precast girder, including barrier if necessary

Substructure Precast with Superstructure

$L_{corb} := 1 \cdot ft$ — Length of approach slab corbel

$B_{corb} := b_f$ $\quad b_f = 89.25 \cdot in$ — Width of corbel cast with girder

$D_{corb} := 1.5 \cdot ft$ — Average depth of corbel

$V_{corb} := L_{corb} \cdot B_{corb} \cdot D_{corb} = 11.16 \cdot ft^3$ — Volume of corbel

$L_{ia} := 2.167 \cdot ft$ — Length of integral abutment

$L_{gia} := 1.167 \cdot ft$ — Length of girder embedded in integral abutment

$B_{ia} := S - W_j = 7.438 \cdot ft$ — Width of integral abutment cast with girder

$D_{ia} := h + 4 \cdot in = 49 \cdot in$ — Depth of integral abutment

$V_{ia} := V_{corb} + \left[L_{ia} \cdot B_{ia} \cdot D_{ia} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gia}\right] = 74.25 \cdot ft^3$ — Volume of integral abutment cast with girder

$W_{ia} := V_{ia} \cdot \gamma_c = 11 \cdot kip$ — Weight of integral abutment cast with girder

$L_{sa} := 2.167 \cdot ft$ — Length of semi-integral abutment

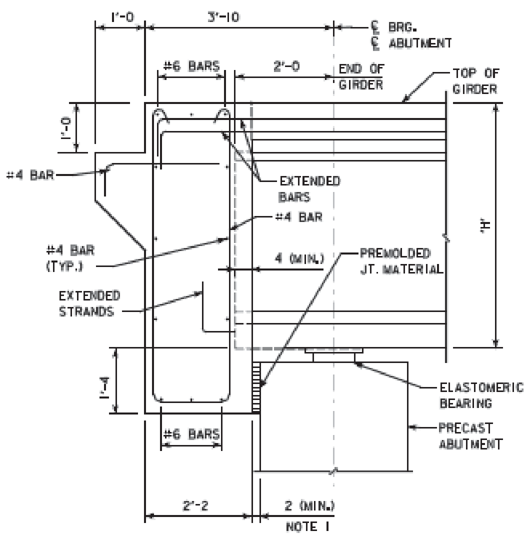
$L_{gsa} := 4 \cdot in$ — Length of girder embedded in semi-integral abutment

$B_{sa} := S - W_j = 7.438 \cdot ft$ — Width of semi-integral abutment cast with girder

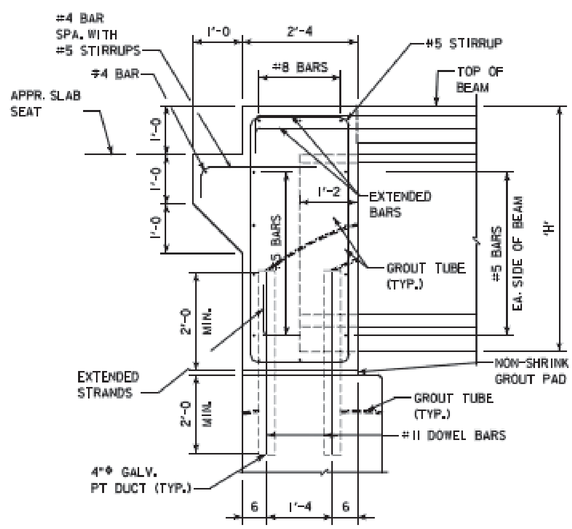
$D_{sa} := h + 16 \cdot in = 61 \cdot in$ — Depth of semi-integral abutment

$V_{sa} := V_{corb} + \left[L_{sa} \cdot B_{sa} \cdot D_{sa} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gsa}\right] = 92.31 \cdot ft^3$ — Volume of semi-integral abutment cast with girder

$W_{sa} := V_{sa} \cdot \gamma_c = 14 \cdot kip$ — Weight of semi-integral abutment cast with girder



Semi-Integral Abutment Backwall



Integral Abutment Backwall

8. LIVE LOAD

When considering Live Loads you must refer to the vertical load section HL-93 in the AASHTO manual. If the project you are working on requires the bridge to support construction loads at any stage, these loads must be considered separately and applied. The longitudinal joints are designed for full moment connections so the beams will act as a unit when sufficiently connected. The distribution factors are then computed for cross-section type "j" (defined in AASHTO 4.6.2.2). When calculating the stiffness parameter, the constant- depth region at the top flange is treated like the slab and the remaining area of the beam will be considered a non-composite beam.

Definitions:

I_{bb}	=	moment of inertia of section below the top flange
A_{bb}	=	area of beam section below the top flange
y_{bb}	=	distance of top fiber below the top flange from neutral axis
t_s	=	thickness of slab not including sacrificial thickness

Distribution Factors for Moment:

From Table 4.6.2.2.2b-1 for moment in interior girders,

$I_{bb} = 63137 \cdot in^4$ — Moment of inertia of section below the top flange

$A_{bb} = 425 \cdot in^2$ — Area of beam section below the top flange

$e_g := h - \left(t_{sac} + \dfrac{t_s}{2}\right) + y_{bb} = 25.578 \cdot in$ — Distance between c.g.'s of beam and flange

$K_g := 1.0 \cdot \left(I_{bb} + A_{bb} \cdot e_g^{\,2}\right) = 341179 \cdot in^4$ — Longitudinal stiffness parameter (Eqn. 4.6.2.2.1-1)

Verify this girder design is within the range of applicability for Table 4.6.2.2.2b-1.

$$\text{CheckMint} := \text{if}\left[(S \le 16 \cdot ft) \cdot (S \ge 3.5 \cdot ft) \cdot (t_s \ge 4.5 \cdot in) \cdot (t_s \le 12.0 \cdot in) \cdot (L \ge 20 \cdot ft) \cdot (L \le 240 \cdot ft), \text{"OK"}, \text{"No Good"}\right]$$

$$\text{CheckMint} := \text{if}\left[(\text{CheckMint} = \text{"OK"}) \cdot (N_g \ge 4) \cdot \left(K_g \ge 10000 \cdot in^4\right) \cdot \left(K_g \le 7000000 \cdot in^4\right), \text{"OK"}, \text{"No Good"}\right]$$

$$\text{CheckMint} = \text{"OK"}$$

$g_{mint1} := 0.06 + \left(\dfrac{S}{14 \cdot ft}\right)^{0.4} \cdot \left(\dfrac{S}{L}\right)^{0.3} \cdot \left(\dfrac{K_g}{L \cdot t_s^{\,3}}\right)^{0.1} = 0.491$ — Single loaded lane

$g_{mint2} := 0.075 + \left(\dfrac{S}{9.5 \cdot ft}\right)^{0.6} \cdot \left(\dfrac{S}{L}\right)^{0.2} \cdot \left(\dfrac{K_g}{L \cdot t_s^{\,3}}\right)^{0.1} = 0.669$ — Two or more loaded lanes

$g_{mint} := \max\left(g_{mint1}, g_{mint2}\right) = 0.669$ — Distribution factor for moment at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.2d-1 for moment in exterior girders,

$$d_e := \frac{S}{2} - W_b = 29.625 \cdot in$$

Distance from centerline of exterior beam to edge of curb or barrier

$$CheckMext := if\left[\left(d_e \geq -1 \cdot ft\right) \cdot \left(d_e \leq 5.5 \cdot ft\right) \cdot \left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

For a single loaded lane, use the Lever Rule.

$$g_{mext1} := \frac{\left(S + 0.5 \cdot b_f - W_b - 5 \cdot ft\right)}{S} = 0.65$$

Single loaded lane

$$e_m := 0.77 + \frac{d_e}{9.1 \cdot ft} = 1.041$$

Correction factor for moment (Table 4.6.2.2.2d-1)

$$g_{mext2} := e_m \cdot g_{mint} = 0.697$$

Two or more loaded lanes

$$g_{mext} := max\left(g_{mext1}, g_{mext2}\right) = 0.697$$

Distribution factor for moment at exterior beams

Distribution Factors for Shear:

From Table 4.6.2.2.3a-1 for shear in interior girders,

Verify this girder design is within the range of applicability for Table 4.6.2.2.3a-1.

$$CheckVint := if\left[(S \leq 16 \cdot ft) \cdot (S \geq 3.5 \cdot ft) \cdot \left(t_s \geq 4.5 \cdot in\right) \cdot \left(t_s \leq 12.0in\right) \cdot (L \geq 20 \cdot ft) \cdot (L \leq 240 \cdot ft), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckVint := if\left[(CheckMint = \text{"OK"}) \cdot \left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckVint = \text{"OK"}$$

$$g_{vint1} := 0.36 + \left(\frac{S}{25 \cdot ft}\right) = 0.678$$

Single loaded lane

$$g_{vint2} := 0.2 + \left(\frac{S}{12 \cdot ft}\right) - \left(\frac{S}{35 \cdot ft}\right)^{2.0} = 0.81$$

Two or more loaded lanes

$$g_{vint} := max\left(g_{vint1}, g_{vint2}\right) = 0.81$$

Distribution factor for shear at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.3b-1 for shear in exterior girders,

For a single loaded lane, use the Lever Rule.

$$\text{CheckVext} := \text{if}\left[\left(d_e \geq -1\cdot ft\right)\cdot\left(d_e \leq 5.5\cdot ft\right)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$g_1 := \frac{\left(S + 0.5\cdot b_f - W_b - 5\cdot ft\right)}{S} = 0.65$$ Single loaded lane (same as for moment)

$$e_v := 0.6 + \frac{d_e}{10\cdot ft} = 0.847$$ Correction factor for shear (Table 4.6.2.2.3b-1)

$$g_2 := e_v\cdot g_{vint} = 0.686$$ Two or more loaded lanes

$$g_{vext} := \max\left(g_1, g_2\right) = 0.686$$ Distribution factor for shear at exterior beams

From Table 4.6.2.2.3c-1 for skewed bridges,

$$\theta := \text{skew} = 0\cdot deg$$

$$\text{CheckSkew} := \text{if}\left[(\theta \leq 60\cdot deg)\cdot(3.5\cdot ft \leq S \leq 16\cdot ft)\cdot(20\cdot ft \leq L \leq 240\cdot ft)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$c_{skew} := 1.0 + 0.20\cdot\left(\frac{L\cdot {t_s}^3}{K_g}\right)^{0.3}\cdot\tan(\theta) = 1.00$$ Correction factor for skew

8. LIVE LOAD (cont'd)

Design Live Load Moment at Midspan:

$w_{lane} := 0.64 \cdot klf$ — Design lane load

$P_{truck} := 32 \cdot kip$ — Design truck axle load

$IM := 33\%$ — Dynamic load allowance (truck only)

$M_{lane}(x) := \dfrac{w_{lane} \cdot x}{2} \cdot (L - x)$ — Design lane load moment

$\delta(x) := \dfrac{x \cdot L - x^2}{L}$ — Influence coefficient for truck moment calculation

$M_{truck}(x) := P_{truck} \cdot \delta(x) \cdot \max\left[\dfrac{9 \cdot x \cdot (L - x) - 14 \cdot ft \cdot (3 \cdot x + L)}{4 \cdot x \cdot (L - x)}, \dfrac{9 \cdot (L - x) - 84 \cdot ft}{4 \cdot (L - x)}\right]$ — Design truck moment

$M_{HL93}(x) := M_{lane}(x) + (1 + IM) \cdot M_{truck}(x)$ — HL93 design live load moment per lane

$M_{ll.i}(x) := M_{HL93}(x) \cdot g_{mint}$ — Design live load moment at interior beam

$M_{ll.e}(x) := M_{HL93}(x) \cdot g_{mext}$ — Design live load moment at exterior beam

$M_{ll}(x) := \text{if}\left(BeamLoc = 1, M_{ll.e}(x), M_{ll.i}(x)\right)$ — Design live load moment

Design Live Load Shear:

$V_{lane}(x) := w_{lane} \cdot \left(\dfrac{L}{2} - x\right)$ — Design lane load shear

$V_{truck}(x) := P_{truck} \cdot \left(\dfrac{9 \cdot L - 9 \cdot x - 84 \cdot ft}{4 \cdot L}\right)$ — Design truck shear

$V_{HL93}(x) := V_{lane}(x) + (1 + IM) \cdot V_{truck}(x)$ — HL93 design live load shear

$V_{ll.i}(x) := V_{HL93}(x) \cdot g_{vint}$ — Design live load shear at interior beam

$V_{ll.e}(x) := V_{HL93}(x) \cdot g_{vext}$ — Design live load shear at exterior beam

$V_{ll}(x) := \text{if}\left(BeamLoc = 1, V_{ll.e}(x), V_{ll.i}(x)\right)$ — Design live load shear

9. PRESTRESS PROPERTIES

Due to tension at the surface limit state be reduced to account for camber leveling forces, the prestress force required at the midspan is expected to be excessive at the ends when released. Not measuring the reduction of prestress moments. Estimate prestress losses at the midspan to find trial prestress forces, that will occur in the bottom tension fibers, that are less than allowable. Compute immediate losses in the prestressed steel and check released stresses at the end of the beam. Once you satisfy end stresses, estimate total loss of prestress. As long as these losses are not drastically different from the assumed stresses, the prestress layout should be acceptable. Concrete stress at all limit states are in Section 9.

$y_{p.est} := 5 \cdot in$ — Assumed distance from bottom of beam to centroid of prestress at midspan

$y_{cgp.est} := y_{bg} + y_{p.est} = -25.46 \cdot in$ — Eccentricity of prestress from neutral axis, based on assumed location

$\Delta f_{p.est} := 25\%$ — Estimate of total prestress losses at the service limit state

Compute bottom fiber service stresses at midspan using gross section properties.

$X := \frac{L}{2}$ — Distance from support

$M_{dl.ser} := M_g(X) + M_{fws}(X) + M_j(X) + M_{bar}(X) = 901 \cdot kip \cdot ft$ — Total dead load moment

$f_{b.serIII} := \frac{M_{dl.ser} + 0.8 \cdot M_{ll}(X)}{S_{bg}} = -2.507 \cdot ksi$ — Total bottom fiber service stress

$f_{pj} := f_{pbt.max} = 202.5 \cdot ksi$ — Prestress jacking force

$f_{pe.est} := f_{pj} \cdot \left(1 - \Delta f_{p.est}\right) = 151.9 \cdot ksi$ — Estimate of effective prestress force

$A_{ps.est} := A_g \cdot \frac{\left(\frac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \frac{A_g \cdot y_{cgp.est}}{S_{bg}}} = 4.035 \cdot in^2$ — Estimated minimum area of prestressing steel

$N_{ps.est} := \mathrm{ceil}\left(\frac{A_{ps.est}}{A_p}\right) = 27$ — Estimated number of strands required

$N_{ps} := 38$ — Number of strands used ($N_{ps.max} = 40$)

The number above is used for the layout strand pattern and to compute the actual location of the strand group. After this is done the required area is computed again. If the estimated location is accurate the number of strands should be equal to the number of strands that we calculated above. The number of strands that was estimated was based on our assumed prestressed losses and gross section properties, which may not accurately reflect the final number of strands required for the design. These stresses for concrete are evaluated in Section 10.
The geometry is assuming a vertical spacing of 2” between straight spans, as well as 2” for harped strands at the end of the beam. Harped strands are bundled at the midspan where the centroid is 5” from the bottom.

9. **PRESTRESS PROPERTIES (cont'd)**

$$N_h := \begin{cases} 2 & \text{if } N_{ps} \le 12 \\ 4 & \text{if } 12 < N_{ps} \le 24 \\ 6 & \text{if } 24 < N_{ps} \le 30 \\ 6 + (N_{ps} - 30) & \text{if } N_{ps} > 30 \end{cases} \qquad N_h = 14$$

Assumes all flange rows are filled prior to filling rows in web above the flange, which maximized efficiency. Use override below to shift strands from flange to web if needed to satisfy end stresses.

$N_{h.add} := 16$ — Additional harped strands in web (strands to be moved from flange to web)

$N_h := \min\left(N_h + N_{h.add}, 16, 2\cdot \text{floor}\left(\frac{N_{ps}}{4}\right)\right) \qquad N_h = 16$ — 16 strands or half of total strands maximum harped in web

$y_h := 1\cdot in + (2\cdot in)\cdot\left(1 + \frac{0.5\cdot N_h - 1}{2}\right) \qquad y_h = 10\cdot in$ — Centroid of harped strands from bottom, equally spaced

$y_{hb} := 5\cdot in$ — Centroid of harped strands from bottom, bundled

$N_s := N_{ps} - N_h \qquad N_s = 22$ — Number of straight strands in flange

$$y_s := 1\cdot in + \begin{cases} 2\cdot in & \text{if } N_s \le 10 \\ \dfrac{(4\cdot in)\cdot N_s - 20\cdot in}{N_s} & \text{if } 10 < N_s \le 20 \\ \dfrac{(6\cdot in)\cdot N_s - 60\cdot in}{N_s} & \text{if } 20 < N_s \le 24 \\ 3.5\cdot in & \text{otherwise} \end{cases} \qquad y_s = 4.273\cdot in$$

Centroid of straight strands from bottom

$y_p := \dfrac{N_s\cdot y_s + N_h\cdot y_{hb}}{N_s + N_h} = 4.579\cdot in$ — Centroid of prestress from bottom at midspan

$y_{cgp} := y_{bg} + y_p = -25.88\cdot in$ — Eccentricity of prestress from neutral axis

$$A_{ps.req} := A_g\cdot\frac{\left(\dfrac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \dfrac{A_g\cdot y_{cgp}}{S_{bg}}} = 3.983\cdot in^2$$

Estimated minimum area of prestressing steel

$N_{ps.req} := \text{ceil}\left(\dfrac{A_{ps.req}}{A_p}\right) = 27$ — Estimated number of strands required

$\text{CheckNps} := \text{if}\left[(N_{ps} \le N_{ps.max})\cdot(N_{ps.req} \le N_{ps}), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$A_{ps.h} := N_h\cdot A_p = 2.448\cdot in^2$ — Area of prestress in web (harped)

$A_{ps.s} := N_s\cdot A_p = 3.366\cdot in^2$ — Area of prestress in flange (straight)

$A_{ps} := A_{ps.h} + A_{ps.s} = 5.814\cdot in^2$ — Total area of prestress

9. PRESTRESS PROPERTIES (cont'd)

Compute transformed section properties based on prestress layout.

Transformed Section Properties

Initial Transformed Section (release):

$A_{ti} = 1167.4 \cdot in^2$

$I_{xti} = 259764 \cdot in^4$

$y_{tti} = 14.172 \cdot in \qquad S_{tti} = 18329 \cdot in^3$

$y_{cgpi} = -25.249 \cdot in \qquad S_{cgpi} = -10288 \cdot in^3$

$y_{bti} = -29.828 \cdot in \qquad S_{bti} = -8709 \cdot in^3$

Final Transformed Section (service):

$A_{tf} = 1163.7 \cdot in^2$

$I_{xtf} = 257457 \cdot in^4$

$y_{ttf} = 14.094 \cdot in \qquad S_{ttf} = 18267 \cdot in^3$

$y_{cgpf} = -25.327 \cdot in \qquad S_{cgpf} = -10165 \cdot in^3$

$y_{btf} = -29.906 \cdot in \qquad S_{btf} = -8609 \cdot in^3$

Determine initial prestress force after instantaneous loss due to elastic shortening. Use transformed properties to compute stress in the concrete at the level of prestress.

$P_j := f_{pj} \cdot A_{ps} = 1177.3 \cdot kip$ — Jacking force in prestress, prior to losses

$$f_{cgpi} := P_j \cdot \left(\frac{1}{A_{ti}} + \frac{y_{cgpi}}{S_{cgpi}} \right) + \frac{M_{gr}\left(\frac{L_g}{2}\right)}{S_{cgpi}} = 3.108 \cdot ksi$$

Stress in concrete at the level of prestress after instantaneous losses

$\Delta f_{pES} := n_{pi} \cdot f_{cgpi} = 18.266 \cdot ksi$ — Prestress loss due to elastic shortening (5.9.5.2.3a-1)

$f_{pi} := f_{pj} - \Delta f_{pES} = 184.234 \cdot ksi$ — Initial prestress after instantaneous losses

$P_i := f_{pi} \cdot A_{ps} = 1071.1 \cdot kip$ — Initial prestress force

Determine deflection of harped strands required to satisfy allowable stresses at the end of the beam at release.

$f_{c.all.rel} := 0.6 \cdot f_{ci} = 3.84 \cdot ksi$ — Allowable compression before losses (5.9.4.1.1)

$f_{t.all.rel} := \max\left(-0.0948 \cdot \sqrt{f_{ci} \cdot ksi}, -0.2 \cdot ksi\right) = -0.200 \cdot ksi$ — Allowable tension before losses (Table 5.9.4.1.2-1)

$L_t := 60 \cdot d_{ps} = 2.5 \cdot ft$ — Transfer length (AASHTO 5.11.4.1)

$$y_{cgp.t} := \left(\frac{f_{t.all.rel} - \frac{M_{gr}(L_t)}{S_{tti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{tti} = -20.262 \cdot in$$

Prestress eccentricity required for tension

$$y_{cgp.b} := \left(\frac{f_{c.all.rel} - \frac{M_{gr}(L_t)}{S_{bti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{bti} = -24.899 \cdot in$$

Prestress eccentricity required for compression

9. PRESTRESS PROPERTIES (cont'd)

$y_{cgp.req} := \max\left(y_{cgp.t}, y_{cgp.b}\right) = -20.262 \cdot in$ — Required prestress eccentricity at end of beam

$y_{h.brg.req} := \dfrac{\left(y_{cgp.req} - y_{bti}\right) \cdot \left(N_s + N_h\right) - y_s \cdot N_s}{N_h} = 16.843 \cdot in$ — Minimum distance to harped prestress centroid from bottom of beam at centerline of bearing

$y_{top.min} := 18 \cdot in$ — Minimum distance between uppermost strand and top of beam

$\alpha_{hd} := 0.4$ — Hold-down point, fraction of the design span length

$slope_{max} := \text{if}\left(d_{ps} = 0.6 \cdot in, \dfrac{1}{12}, \dfrac{1}{8}\right) = 0.125$ — Maximum slope of an individual strand to limit hold-down force to 4 kips/strand

$y_{h.brg} := h - y_{top.min} - \left(\dfrac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) = 20 \cdot in$ — Set centroid of harped strands as high as possible to minimize release and handling stresses

$y_{h.brg} := \min\left(y_{h.brg}, y_{hb} + slope_{max} \cdot \alpha_{hd} \cdot L\right) = 20 \cdot in$ — Verify that slope requirement is satisfied at uppermost strand

$CheckEndPrestress := \text{if}\left(y_{h.brg} \geq y_{h.brg.req}, \text{"OK"}, \text{"Verify release stresses."}\right) = \text{"OK"}$

$y_{p.brg} := \dfrac{N_s \cdot y_s + N_h \cdot y_{h.brg}}{N_s + N_h} = 10.895 \cdot in$ — Centroid of prestress from bottom at bearing

$slope_{cgp} := \dfrac{y_{p.brg} - y_p}{\alpha_{hd} \cdot L} = 0.022$ — Slope of prestress centroid within the harping length

$y_{px}(x) := \begin{cases} y_p + slope_{cgp} \cdot \left(L_{end} + \alpha_{hd} \cdot L - x\right) & \text{if } x \leq L_{end} + \alpha_{hd} \cdot L \\ y_p & \text{otherwise} \end{cases}$ — Distance to center of prestress from the bottom of the beam at any position

10. PRESTRESS LOSSES

Prestressed losses can be evaluated like regular concrete, in short-term and long-term losses. When the beam is a pretension girder there are instantaneous losses when the beam is shortened upon release of the prestress forces. Time-dependent losses happen when the beam is under creep and shrinkage of the beam concrete, creep and shrinkage c the deck concrete, and the relaxation of prestressed steel. These long term effects are separated into two stages that represent significant events in bridge construction. The first stage is the time between transfer of the prestress forces anc placement of the decked beam and the second is the period of time between placement of the deck and the final service load. For decked beams the computation of long-term losses is slightly simplified due to the cross-section not changing between the two stages and the shrinkage term of the deck concrete is eliminated since the deck and beam being cast together. No losses or gains in the steel associated with deck placement after transfer.

AASHTO methods for estimating time-dependent losses:
Approximate Estimate (5.9.5.3)
Refined Estimate (5.9.5.4)

The Approximate method is based on systems with composite decks and is based on the following assumptions: timing of load application, the cross-section in which the load is applied, and the ratio of dead and live loads to the total load. The conditions for the beams to be fabricated, formed and loaded depend on conditions assumed in the development of the approximate method. The refined method is used to estimate time-dependent losses in the prestressed steel.

Equations 5.9.5.4 are time-dependent and calculate the age-adjustment factors that effect losses using gross section properties.

$t_i := 1$ Time (days) between casting and release of prestress

$t_b := 20$ Time (days) to barrier casting (exterior girder only)

$t_d := 30$ Time (days) to erection of precast section, closure joint pour

$t_f := 20000$ Time (days) to end of service life

Terms and equations used in the loss calculations:

$K_L := 45$ Prestressing steel factor for low-relaxation strands (C5.9.5.4.2c)

$VS := \frac{A_g}{Peri} = 3.883 \cdot in$ Volume-to-surface ratio of the precast section

$k_s := \max\left(1.45 - 0.13 \cdot \frac{VS}{in}, 1.0\right) = 1.00$ Factor for volume-to-surface ratio (5.4.2.3.2-2)

$k_{hc} := 1.56 - 0.008 \cdot H = 1.00$ Humidity factor for creep (5.4.2.3.2-3)

$k_{hs} := 2.00 - 0.014 \cdot H = 1.02$ Humidity factor for shrinkage (5.4.2.3.3-2)

$k_f := \frac{5}{1 + \frac{f_{ci}}{ksi}} = 0.676$ Factor for effect of concrete strength (5.4.2.3.2-4)

10. PRESTRESS LOSSES (cont'd)

$$k_{td}(t) := \frac{t}{61 - 4\cdot\frac{f_{ci}}{ksi} + t}$$

Time development factor (5.4.2.3.2-5)

$$\psi(t, t_{init}) := 1.9\cdot k_s\cdot k_{hc}\cdot k_f\cdot k_{td}(t)\cdot(t_{init})^{-0.118}$$

Creep coefficient (5.4.2.3.2-1)

$$\varepsilon_{sh}(t) := k_s\cdot k_{hs}\cdot k_f\cdot k_{td}(t)\cdot\left(0.48\cdot 10^{-3}\right)$$

Concrete shrinkage strain (5.4.2.3.3-1)

Time from Transfer to Erection:

$$e_{pg} := -(y_p + y_{bg}) = 25.877\cdot in$$

Eccentricity of prestress force with respect to the neutral axis of the gross non-composite beam, positive below the beam neutral axis

$$f_{cgp} := P_i\cdot\left(\frac{1}{A_g} + \frac{e_{pg}^{\,2}}{I_{xg}}\right) + \frac{M_g\left(\frac{L}{2}\right)}{I_{xg}}\cdot(y_p + y_{bg}) = 3.148\cdot ksi$$

Stress in the concrete at the center prestress immediately after transfer

$$f_{pt} := \max(f_{pi}, 0.55\cdot f_{py}) = 184.234\cdot ksi$$

Stress in strands immediately after transfer (5.9.5.4.2c-1)

$$\psi_{bid} := \psi(t_d, t_i) = 0.589$$

Creep coefficient at erection due to loading at transfer

$$\psi_{bif} := \psi(t_f, t_i) = 1.282$$

Creep coefficient at final due to loading at transfer

$$\varepsilon_{bid} := \varepsilon_{sh}(t_d - t_i) = 1.490 \times 10^{-4}$$

Concrete shrinkage between transfer and erection

$$K_{id} := \frac{1}{1 + n_{pi}\cdot\frac{A_{ps}}{A_g}\cdot\left(1 + \frac{A_g\cdot e_{pg}^{\,2}}{I_{xg}}\right)\cdot(1 + 0.7\cdot\psi_{bif})} = 0.809$$

Age-adjusted transformed section coefficient (5.9.5.4.2a-2)

$$\Delta f_{pSR} := \varepsilon_{bid}\cdot E_p\cdot K_{id} = 3.433\cdot ksi$$

Loss due to beam shrinkage (5.9.5.4.2a-1)

$$\Delta f_{pCR} := n_{pi}\cdot f_{cgp}\cdot\psi_{bid}\cdot K_{id} = 8.807\cdot ksi$$

Loss due to creep (5.9.5.4.2b-1)

$$\Delta f_{pR1} := \left[\frac{f_{pt}}{K_L}\cdot\frac{\log(24\cdot t_d)}{\log(24\cdot t_i)}\cdot\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\cdot\left[1 - \frac{3\cdot(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}}\right]\cdot K_{id} = 1.142\cdot ksi$$

Loss due to relaxation (C5.9.5.4.2c-1

$$\Delta f_{pid} := \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} = 13.382\cdot ksi$$

10. PRESTRESS LOSSES (cont'd)

Time from Erection to Final:

$e_{pc} := e_{pg} = 25.877 \cdot in$ — Eccentricity of prestress force does not change

$A_c := A_g \qquad I_c := I_{xg}$ — Section properties remain unchanged

$$\Delta f_{cd} := \frac{M_{fws}\left(\frac{L}{2}\right) + M_j\left(\frac{L}{2}\right)}{S_{cgpf}} + \frac{\Delta f_{pid}}{n_p} = 2.413 \cdot ksi$$

Change in concrete stress at center of prestress due to initial time-dependent losses and superimposed dead load. Deck weight is not included for this design.

$\psi_{bdf} := \psi\left(t_f, t_d\right) = 0.858$ — Creep coefficient at final due to loading at erection

$\varepsilon_{bif} := \varepsilon_{sh}\left(t_f - t_i\right) = 3.302 \times 10^{-4}$ — Concrete shrinkage between transfer and final

$\varepsilon_{bdf} := \varepsilon_{bif} - \varepsilon_{bid} = 1.813 \times 10^{-4}$ — Concrete shrinkage between erection and final

$$K_{df} := \frac{1}{1 + n_{pi} \cdot \frac{A_{ps}}{A_c} \cdot \left(1 + \frac{A_c \cdot e_{pc}^{\,2}}{I_c}\right) \cdot \left(1 + 0.7 \cdot \psi_{bif}\right)} = 0.809$$

Age-adjusted transformed section coefficient remains unchanged

$\Delta f_{pSD} := \varepsilon_{bdf} \cdot E_p \cdot K_{df} = 4.177 \cdot ksi$ — Loss due to beam shrinkage

$\Delta f_{pCD} := n_{pi} \cdot f_{cgp} \cdot \left(\psi_{bif} - \psi_{bid}\right) \cdot K_{df} + n_p \cdot \Delta f_{cd} \cdot \psi_{bdf} \cdot K_{df} = 19.157 \cdot ksi$ — Loss due to creep

$\Delta f_{pR2} := \Delta f_{pR1} = 1.142 \cdot ksi$ — Loss due to relaxation

$\Delta f_{pSS} := 0$ — Loss due to deck shrinkage

$\Delta f_{pdf} := \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} + \Delta f_{pSS} = 24.476 \cdot ksi$

Prestress Loss Summary

$\Delta f_{pES} = 18.266 \cdot ksi \qquad \frac{\Delta f_{pES}}{f_{pj}} = 9 \cdot \%$

$\Delta f_{pLT} := \Delta f_{pid} + \Delta f_{pdf} = 37.858 \cdot ksi \qquad \frac{\Delta f_{pLT}}{f_{pj}} = 18.7 \cdot \%$

$\Delta f_{pTotal} := \Delta f_{pES} + \Delta f_{pLT} = 56.124 \cdot ksi \qquad \frac{\Delta f_{pTotal}}{f_{pj}} = 27.7 \cdot \% \qquad \Delta f_{p.est} = 25 \cdot \%$

$f_{pe} := f_{pj} - \Delta f_{pTotal} = 146.4 \cdot ksi$ — Final effective prestress

$\text{CheckFinalPrestress} := \text{if}\left(f_{pe} \le f_{pe.max}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$

11. CONCRETE STRESSES

Concrete Stresses at release, during handling and at final service are computed and compared to approximated values for each stage.

Concrete Stresses at Release

When calculating the stresses at release use the overall beam length due to the beam being supported at each end in the casting bed after prestress forces are transformed.

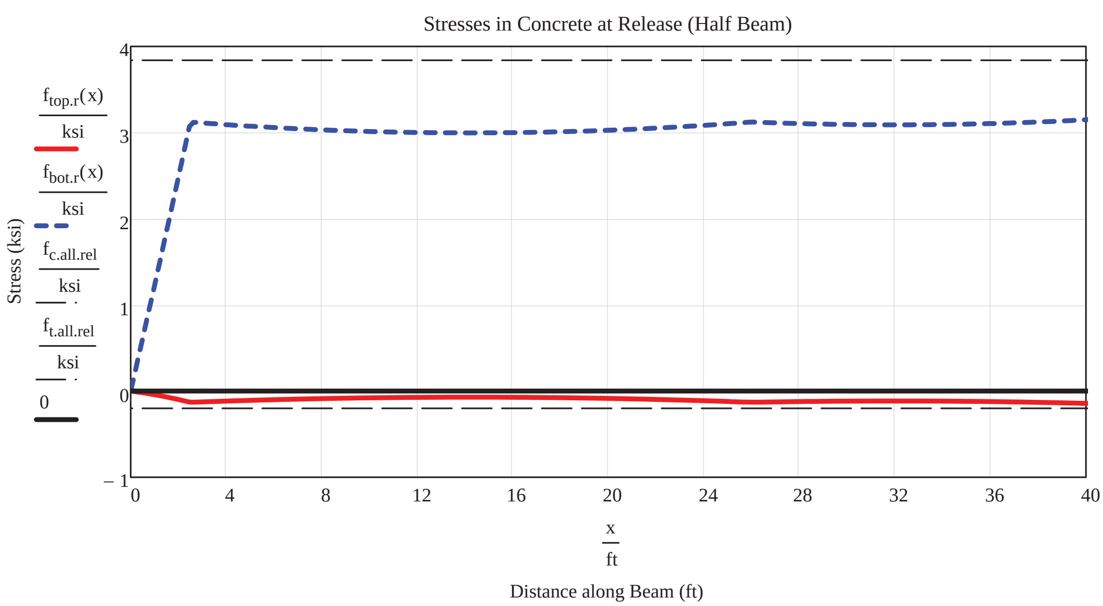
Define locations for which stresses are to be calculated:

$$x_r := L_g \cdot \left(0 \quad \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad 0.1 \quad 0.2 \quad 0.3 \quad \alpha_{hd} \quad 0.5 \right)^T \qquad ir := 1 .. \text{last}(x_r)$$

Functions for computing beam stresses:

$$f_{top.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{tti}}\right) + \frac{M_{gr}(x)}{S_{tti}}$$ Top fiber stress at release

$$f_{bot.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{bti}}\right) + \frac{M_{gr}(x)}{S_{bti}}$$ Bottom fiber stress at release



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$f_{t.all.rel} = -0.2 \cdot ksi$ Allowable tension at release

$f_{c.all.rel} = 3.84 \cdot ksi$ Allowable compression at release

$TopRel_{ir} := f_{top.r}\left(x_{r_{ir}}\right)$ $TopRel^{T} = (0.000\ \ -0.097\ \ -0.130\ \ -0.097\ \ -0.071\ \ -0.081\ \ -0.126\ \ -0.115)\cdot ksi$

$CheckTopRel := if\left[\left(max(TopRel) \le f_{c.all.rel}\right)\cdot\left(min(TopRel) \ge f_{t.all.rel}\right), "OK", "No Good"\right] = "OK"$

$BotRel_{ir} := f_{bot.r}\left(x_{r_{ir}}\right)$ $BotRel^{T} = (0.000\ \ 2.484\ \ 3.122\ \ 3.053\ \ 2.999\ \ 3.019\ \ 3.115\ \ 3.090)\cdot ksi$

$CheckBotRel := if\left[\left(max(BotRel) \le f_{c.all.rel}\right)\cdot\left(min(BotRel) \ge f_{t.all.rel}\right), "OK", "No Good"\right] = "OK"$

Concrete Stresses During Lifting and Transportation

Lifting and transportation stresses can govern over final stresses due to different support locations, dynamic effects that dead load can cause during movement, bending stresses during lifting and superelevation of the roadway in shipping. End diaphragms on both ends are assumed. For prestressing effects, calculate the effective prestressed force losses between transfer and building.

$a := h = 3.75 \cdot ft$ Maximum distance to lift point from bearing line

$a' := a + L_{end} = 5.75 \cdot ft$ Distance to lift point from end of beam

$P_{dia} := max\left(W_{ia}, W_{sa}\right) = 13.8 \cdot kip$ Approximate abutment weight

$P_m := P_j \cdot \left[1 - \frac{\left(\Delta f_{pES} + \Delta f_{pid}\right)}{f_{pj}}\right] = 993.3 \cdot kip$ Effective prestress during lifting and shipping

Define locations for which stresses are to be calculated:

$$x_e := L_g \cdot \left(0\ \ min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right)\ \ max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right)\ \ \frac{a'}{L_g}\ \ \alpha_{hd}\ \ 0.5\right)^{T} \qquad ie := 1 .. last\left(x_e\right)$$

Compute moment in the girder during lifting with supports at the lift points.

$$M_{lift}(x) := \begin{cases} -\left[\frac{\left(w_g + w_{bar}\right)\cdot x^2}{2} + P_{dia}\cdot x\right] & \text{if } x \le a' \\ M_{gr}(x) - \left[M_{gr}(a') + \frac{\left(w_g + w_{bar}\right)\cdot (a')^2}{2} + P_{dia}\cdot a'\right] & \text{otherwise} \end{cases}$$

11. CONCRETE STRESSES (cont'd)

Functions for computing beam stresses:

$$f_{top.lift}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}$$

Top fiber stress during lifting

$$f_{top.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} \cdot (1 + DIM)$$

Top fiber stress during lifting, impact increasing dead load

$$f_{top.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}} \cdot (1 - DIM)$$

Top fiber stress during lifting, impact decreasing dead load

$TopLift1_{ie} := f_{top.lift}\left(x_{e_{ie}}\right)$ $\qquad TopLift1^T = (0.000 \ \ -0.165 \ \ -0.214 \ \ -0.305 \ \ -0.308 \ \ -0.296) \cdot ksi$

$TopLift2_{ie} := f_{top.DIM.inc}\left(x_{e_{ie}}\right)$ $\qquad TopLift2^T = (0.000 \ \ -0.171 \ \ -0.222 \ \ -0.327 \ \ -0.245 \ \ -0.227) \cdot ksi$

$TopLift3_{ie} := f_{top.DIM.dec}\left(x_{e_{ie}}\right)$ $\qquad TopLift3^T = (0.000 \ \ -0.159 \ \ -0.206 \ \ -0.284 \ \ -0.371 \ \ -0.364) \cdot ksi$

$$f_{bot.lift}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}$$

Bottom fiber stress during lifting

$$f_{bot.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} \cdot (1 + DIM)$$

Bottom fiber stress during lifting, impact increasing dead load

$$f_{bot.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_m \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}} \cdot (1 - DIM)$$

Bottom fiber stress during lifting, impact decreasing dead load

$BotLift1_{ie} := f_{bot.lift}\left(x_{e_{ie}}\right)$ $\qquad BotLift1^T = (0.000 \ \ 2.481 \ \ 3.118 \ \ 3.312 \ \ 3.318 \ \ 3.292) \cdot ksi$

$BotLift2_{ie} := f_{bot.DIM.inc}\left(x_{e_{ie}}\right)$ $\qquad BotLift2^T = (0.000 \ \ 2.494 \ \ 3.135 \ \ 3.358 \ \ 3.184 \ \ 3.147) \cdot ksi$

$BotLift3_{ie} := f_{bot.DIM.dec}\left(x_{e_{ie}}\right)$ $\qquad BotLift3^T = (0.000 \ \ 2.468 \ \ 3.101 \ \ 3.267 \ \ 3.452 \ \ 3.437) \cdot ksi$

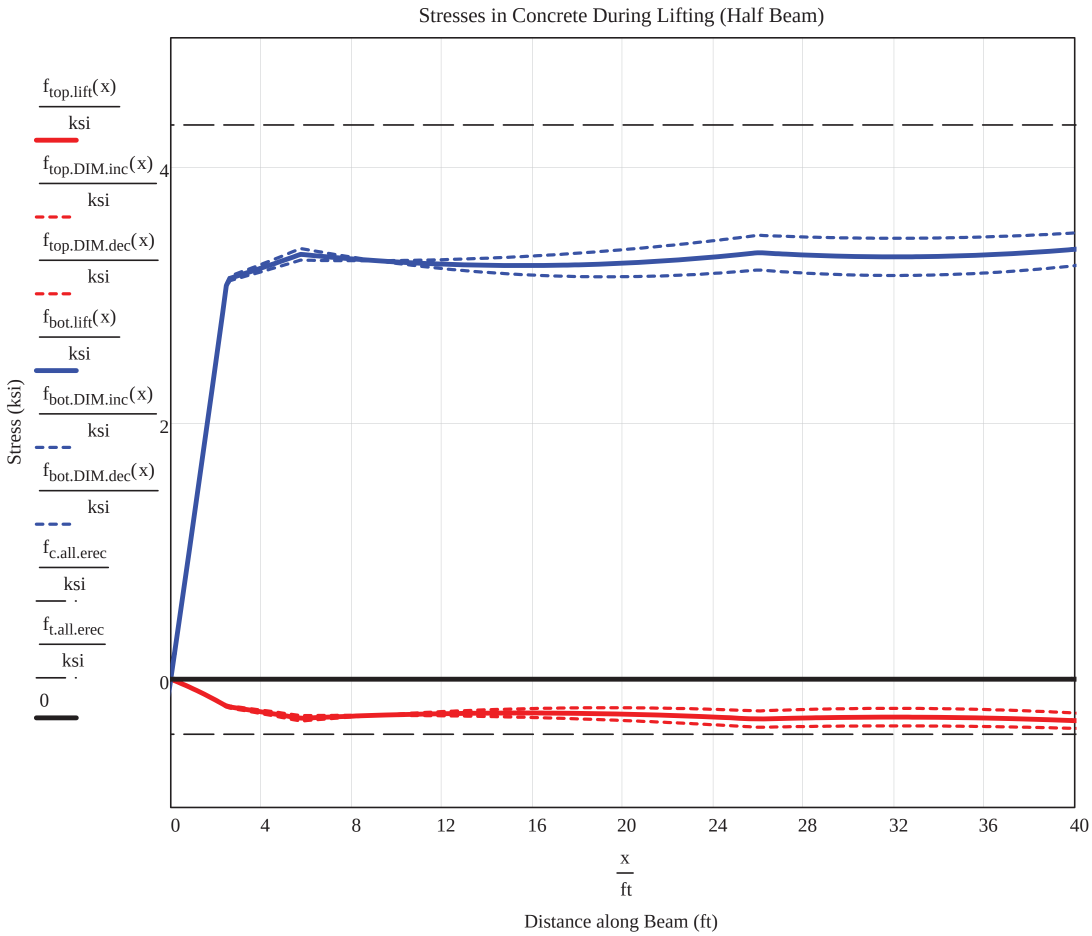
Allowable stresses during handling:

$f_{cm} := f_{c.erec}\left(f_c\right) = 7.2 \cdot ksi$ — Assumed concrete strength when handling operations begin

$f_{c.all.erec} := 0.6 \cdot f_{cm} = 4.32 \cdot ksi$ — Allowable compression during lifting and shipping

$f_{t.all.erec} := f_{t.erec}\left(f_{cm}\right) = -0.429 \cdot ksi$ — Allowable tension during lifting and shipping

11. CONCRETE STRESSES (cont'd)



Compare beam stresses to allowable stresses.

$TopLiftMax_{ie} := \max\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right)$ $TopLiftMax^{T} = (0\ \ -0.159\ \ -0.206\ \ -0.284\ \ -0.245\ \ -0.227\ )\cdot ks$

$TopLiftMin_{ie} := \min\left(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie}\right)$ $TopLiftMin^{T} = (0\ \ -0.171\ \ -0.222\ \ -0.327\ \ -0.371\ \ -0.364\ )\cdot ks$

$CheckTopLift := if\left[\left(\max(TopLiftMax) \le f_{c.all.erec}\right)\cdot\left(\min(TopLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$BotLiftMax_{ie} := \max\left(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie}\right)$ $BotLiftMax^{T} = (0\ \ 2.494\ \ 3.135\ \ 3.358\ \ 3.452\ \ 3.437\ )\cdot ksi$

$BotLiftMin_{ie} := \min\left(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie}\right)$ $BotLiftMin^{T} = (0\ \ 2.468\ \ 3.101\ \ 3.267\ \ 3.184\ \ 3.147\ )\cdot ksi$

$CheckBotLift := if\left[\left(\max(BotLiftMax) \le f_{c.all.erec}\right)\cdot\left(\min(BotLiftMin) \ge f_{t.all.erec}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

11. CONCRETE STRESSES (cont'd)

Concrete Stresses at Final

Stresses are calculated using design span length. The top flange compression and bottom flange under tension are computed at Service I and Service III states.

$f_{c.all.ser1} := 0.4 \cdot f_c = 3.2 \cdot ksi$ Allowable compression due to effective prestress and dead load (Table 5.9.4.2.1-1)

$f_{c.all.ser2} := 0.6 \cdot f_c = 4.8 \cdot ksi$ Allowable compression due to effective prestress, permanent load, and transient loads, as well as stresses during shipping and handling (Table 5.9.4.2.1-1)

$f_{t.all.ser} = 0 \cdot ksi$ Allowable tension (computed previously)

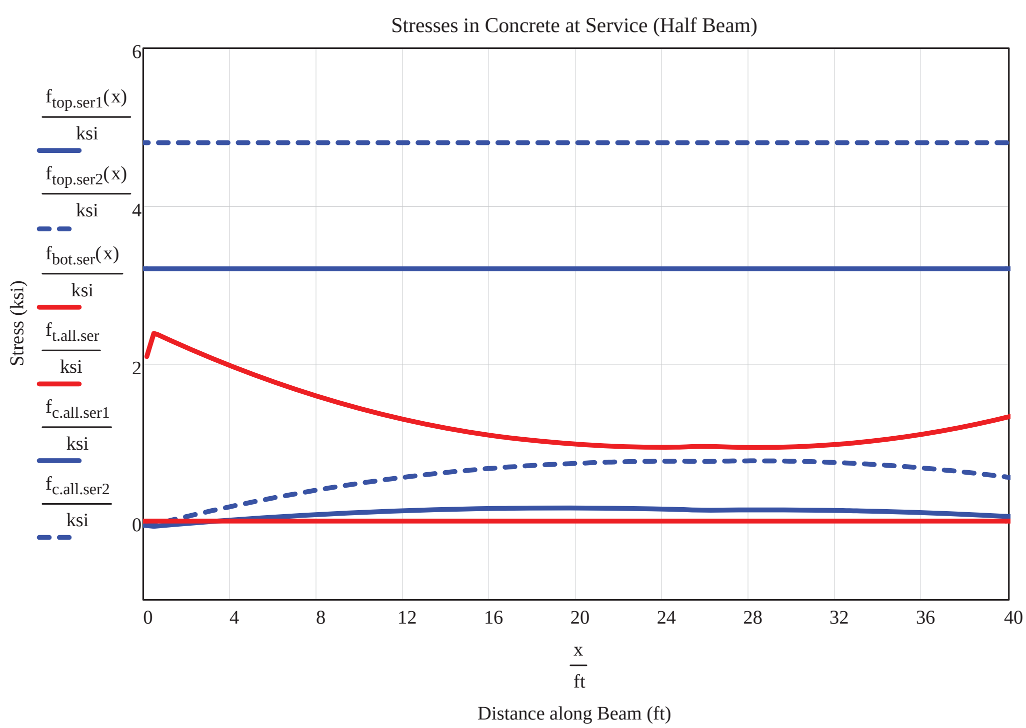
$P_e := f_{pe} \cdot A_{ps} = 851.0 \cdot kip$ Effective prestress after all losses

Compute stresses at midspan and compare to allowable values.

$$f_{top.ser1}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g\left(x + L_{end}\right)}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x)}{S_{ttf}}$$

$$f_{top.ser2}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g\left(x + L_{end}\right)}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + M_{ll}(x)}{S_{ttf}}$$

$$f_{bot.ser}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_g\left(x + L_{end}\right)}{S_{bti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + 0.8 \cdot M_{ll}(x)}{S_{btf}}$$



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$$x_s := L\cdot\left(\frac{L_t}{L}\ \ 0.1\ \ 0.15\ \ 0.2\ \ 0.25\ \ 0.3\ \ 0.35\ \ \alpha_{hd}\ \ 0.45\ \ 0.5\right)^T \qquad is := 1\,..\,\mathrm{last}(x_s)$$

$$TopSer1_{is} := f_{top.ser1}\left(x_{s_{is}}\right) \qquad TopSer1^T = (-0.020\ \ 0.049\ \ 0.096\ \ 0.130\ \ 0.153\ \ 0.165\ \ 0.164\ \ 0.151\ \ 0.139\ \ 0.140)\cdot ksi$$

$$TopSer2_{is} := f_{top.ser2}\left(x_{s_{is}}\right) \qquad TopSer2^T = (0.089\ \ 0.292\ \ 0.437\ \ 0.554\ \ 0.643\ \ 0.705\ \ 0.743\ \ 0.759\ \ 0.761\ \ 0.759)\cdot ksi$$

$$CheckCompSerI := if\left[\left(\max(TopSer1) \le f_{c.all.ser1}\right)\cdot\left(\max(TopSer2) \le f_{c.all.ser2}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotSer_{is} := f_{bot.ser}\left(x_{s_{is}}\right) \qquad BotSer^T = (2.143\ \ 1.769\ \ 1.505\ \ 1.292\ \ 1.131\ \ 1.022\ \ 0.959\ \ 0.936\ \ 0.939\ \ 0.941)\cdot ksi$$

$$CheckTenSerIII := if\left(\min(BotSer) \ge f_{t.all.ser}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

12. FLEXURAL STRENGTH

Confirm flexural resistance at Strength Limit State. Calculate Factored moment at midspan during Strength I load combination. Compare this to factored resistance in AASHTO LRFD 5.7.3.

$M_{DC}(x) := M_g(x) + M_{bar}(x) + M_j(x)$ Self weight of components

$M_{DW}(x) := M_{fws}(x)$ Weight of future wearing surface

$M_{LL}(x) := M_{ll}(x)$ Live load

$M_{StrI}(x) := 1.25\cdot M_{DC}(x) + 1.5\cdot M_{DW}(x) + 1.75\cdot M_{LL}(x)$ Factored design moment

For minimum reinforcement check, per 5.7.3.3.2

$f_{cpe} := P_e\cdot\left(\frac{1}{A_g} + \frac{y_{cgp}}{S_{bg}}\right) = 3.527\cdot ksi$ Concrete compression at extreme fiber due to effective prestress

$M_{cr} := -\left(f_{r.cm} + f_{cpe}\right)\cdot S_{bg} = 3019\cdot kip\cdot ft$ Cracking moment (5.7.3.3.2-1)

$M_u(x) := \max\left(M_{StrI}(x), \min\left(1.33\cdot M_{StrI}(x), 1.2\cdot M_{cr}\right)\right)$ Design moment

12. FLEXURAL STRENGTH (cont'd)

Compute factored flexural resistance.

$$\beta_1 := \max\left[0.65, 0.85 - 0.05\cdot\left(\frac{f_c}{ksi} - 4\right)\right] = 0.65$$ Stress block factor (5.7.2.2)

$$k := 2\cdot\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$$ Tendon type factor (5.7.3.1.1-2)

$$d_p(x) := h - y_{px}\left(x + L_{end}\right) \qquad d_p(X) = 40.421\cdot in$$ Distance from compression fiber to prestress centroid

$$h_f := d_7 = 8\cdot in$$ Structural flange thickness

$$b_{taper} := \frac{b_6 - b_5}{2} = 16\cdot in$$ Average width of taper at bottom of flange

$$h_{taper} := d_5 = 2\cdot in$$ Depth of taper at bottom of flange

$$a(x) := \frac{A_{ps}\cdot f_{pu}}{0.85\cdot f_c\cdot b_f + \frac{k}{\beta_1}\cdot A_{ps}\cdot\left(\frac{f_{pu}}{d_p(x)}\right)} \qquad a(X) = 2.517\cdot in$$ Depth of equivalent stress block for rectangular section

$$c(x) := \frac{a(x)}{\beta_1} \qquad c(X) = 3.873\cdot in$$ Neutral axis location

$$CheckTC := \mathrm{if}\left[\frac{c(X)}{d_p(X)} \le \left(\frac{.003}{.003 + .005}\right), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$$ Tension-controlled section check (midspan)

$$\varphi_f := \min\left[1.0, \max\left[0.75, 0.583 + 0.25\cdot\left(\frac{d_p(X)}{c(X)} - 1\right)\right]\right] = 1.00$$ Resistance factor for prestressed concrete (5.5.4.2)

$$f_{ps} := f_{pu}\cdot\left(1 - k\cdot\frac{c(X)}{d_p(X)}\right) = 262.8\cdot ksi$$ Average stress in the prestressing steel (5.7.3.1.1-1)

$$L_d := \frac{1.6}{ksi}\cdot\left(f_{ps} - \frac{2}{3}\cdot f_{pe}\right)\cdot d_{ps} = 11.012\cdot ft$$ Bonded strand devlepment length (5.11.4.2-1)

$$f_{px}(x) := \begin{cases} \frac{f_{pe}\cdot\left(x + L_{end}\right)}{L_t} & \text{if } x \le L_t - L_{end} \\ f_{pe} + \frac{\left(x + L_{end}\right) - L_t}{L_d - L_t}\cdot\left(f_{ps} - f_{pe}\right) & \text{if } L_t - L_{end} < x \le L_d - L_{end} \\ f_{ps} & \text{otherwise} \end{cases}$$ Stress in prestressing steel along the length for bonded strand (5.11.4.2)

$$M_r(x) := \varphi_f\cdot\left[A_{ps}\cdot f_{px}(x)\cdot\left(d_p(x) - \frac{a(x)}{2}\right)\right]$$ Flexure resistance along the length

12. FLEXURAL STRENGTH (cont'd)

$$x_{mom} := L \cdot \left(0.01 \quad \frac{L_t - L_{end}}{L} \quad \frac{L_d - L_{end}}{L} \quad \alpha_{hd} \quad 0.5 \right)^T \qquad imom := 1 \, .. \, \text{last}\left(x_{mom}\right)$$

$$M_{rx_{imom}} := M_r\left(x_{mom_{imom}}\right) \qquad M_{ux_{imom}} := M_u\left(x_{mom_{imom}}\right)$$

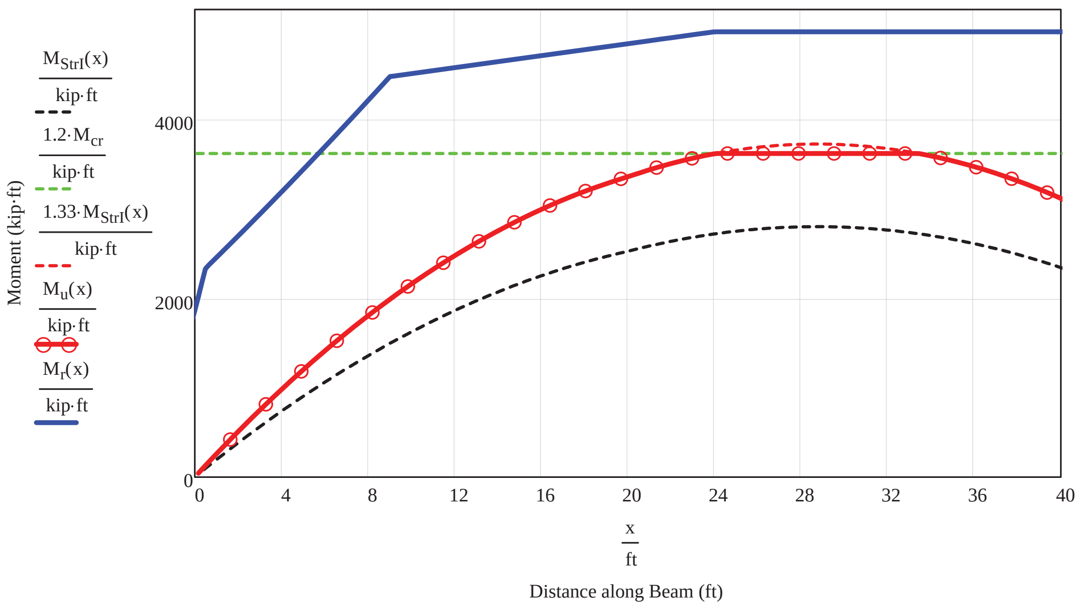
$$DC_{mom} := \frac{M_{ux}}{M_{rx}} \qquad \max\left(DC_{mom}\right) = 0.727$$

Demand-Capacity ratio for moment

$$CheckMom := \text{if}\left(\max\left(DC_{mom}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

Flexure resistance check

Design Moment and Flexure Resistance (Half Beam)



13. SHEAR STRENGTH

Shear Resistance

Use Strength I load combination to calculate factored shear at the critical shear section and at tenth points along the span. Compare it to factored resistance in AASHTO LRFD 5.8.

$V_{DC}(x) := V_g(x) + V_{bar}(x) + V_j(x)$ — Self weight of components

$V_{DW}(x) := V_{fws}(x)$ — Weight of future wearing surface

$V_{LL}(x) := V_{ll}(x)$ — Live load

$V_u(x) := 1.25 \cdot V_{DC}(x) + 1.5 \cdot V_{DW}(x) + 1.75 \cdot V_{LL}(x)$ — Factored design shear

$\varphi_v := 0.90$ — Resistance factor for shear in normal weight concrete (AASHTO LRFD 5.5.4.2)

$d_{end} := h - y_{px}(L_{end}) = 34.105 \cdot in$ — Depth to steel centroid at bearing

$d_v := \min(0.9 \cdot d_{end}, 0.72 \cdot h) = 30.695 \cdot in$ — Effective shear depth lower limit at end

$$V_p(x) := \begin{cases} P_e \cdot slope_{cgp} \cdot \dfrac{x + L_{end}}{L_t} & \text{if } x \le L_t - L_{end} \\ P_e \cdot slope_{cgp} & \text{if } L_t - L_{end} < x \le \alpha_{hd} \cdot L \\ 0 & \text{otherwise} \end{cases}$$

Vertical component of effective prestress force

$b_v := b_3 = 6 \cdot in$ — Web thickness

$$v_u(x) := \frac{\left|V_u(x) - \varphi_v \cdot V_p(x)\right|}{\varphi_v \cdot b_v \cdot d_v}$$

Shear stress on concrete (5.8.2.9-1)

$M_{ushr}(x) := \max\left(M_{StrI}(x), \left|V_u(x) - V_p(x)\right| \cdot d_v\right)$ — Factored moment for shear

$f_{po} := 0.7 \cdot f_{pu} = 189 \cdot ksi$ — Stress in prestressing steel due to locked-in strain after casting concrete

$$\varepsilon_s(x) := \max\left(-0.4 \cdot 10^{-3}, \frac{\dfrac{\left|M_u(x)\right|}{d_v} + \left|V_u(x) - V_p(x)\right| - A_{ps} \cdot f_{po}}{E_p \cdot A_{ps}}\right)$$

Steel strain at the centroid of the prestressing steel

$$\beta(x) := \frac{4.8}{1 + 750 \cdot \varepsilon_s(x)}$$

Shear resistance parameter

$\theta(x) := \left(29 + 3500 \cdot \varepsilon_s(x)\right) \cdot deg$ — Principal compressive stress angle

$$V_c(x) := 0.0316 \cdot ksi \cdot \beta(x) \cdot \sqrt{\frac{f_c}{ksi}} \cdot b_v \cdot d_v$$

Concrete contribution to total shear resistance

13. SHEAR STRENGTH (cont'd)

$\alpha := 90 \cdot deg$ — Angle of inclination of transverse reinforcement

$A_v := (1.02\ \ 0.62\ \ 0.62\ \ 0.62\ \ 0.31)^T \cdot in^2$ $\quad s_v := (3\ \ 6\ \ 6\ \ 12\ \ 12)^T \cdot in$ — Transverse reinforcement area and spacing provided

$x_v := (0\ \ 0.25 \cdot h\ \ 1.5 \cdot h\ \ 0.3 \cdot L\ \ 0.5 \cdot L\ \ 0.6 \cdot L)^T$ $\quad x_v^T = (0\ \ 0.938\ \ 5.625\ \ 18\ \ 30\ \ 36) \cdot ft$

$$A_{vs}(x) := \left| \begin{array}{l} \text{for } i \in 1 .. \text{last}(A_v) \\ \quad out \leftarrow \dfrac{A_{v_i}}{s_{v_i}} \ \text{ if } x_{v_i} \le x \le x_{v_{i+1}} \\ out \end{array} \right.$$

$V_s(x) := A_{vs}(x) \cdot f_y \cdot d_v \cdot (\cot(\theta(x)) + \cot(\alpha)) \cdot \sin(\alpha)$ — Steel contribution to total shear resistance

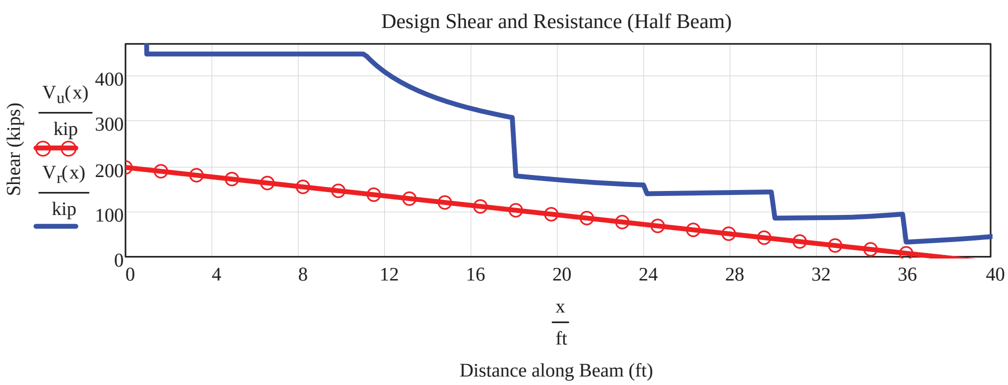
$V_r(x) := \varphi_v \cdot (V_c(x) + V_s(x) + V_p(x))$ — Factored shear resistance

$$x_{shr} := \left| \begin{array}{l} \text{for } i \in 1 .. 100 \\ \quad out_i \leftarrow i \cdot \dfrac{0.5 \cdot L}{100} \\ out \end{array} \right. \qquad ishr := 1 .. \text{last}(x_{shr})$$

$V_{ux_{ishr}} := V_u\left(x_{shr_{ishr}}\right) \qquad V_{rx_{ishr}} := V_r\left(x_{shr_{ishr}}\right)$

$DC_{shr} := \dfrac{V_{ux}}{V_{rx}} \qquad \max(DC_{shr}) = 0.574$

$CheckShear := \text{if}\left(\max(DC_{shr}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Shear resistance check



13. SHEAR STRENGTH (cont'd)

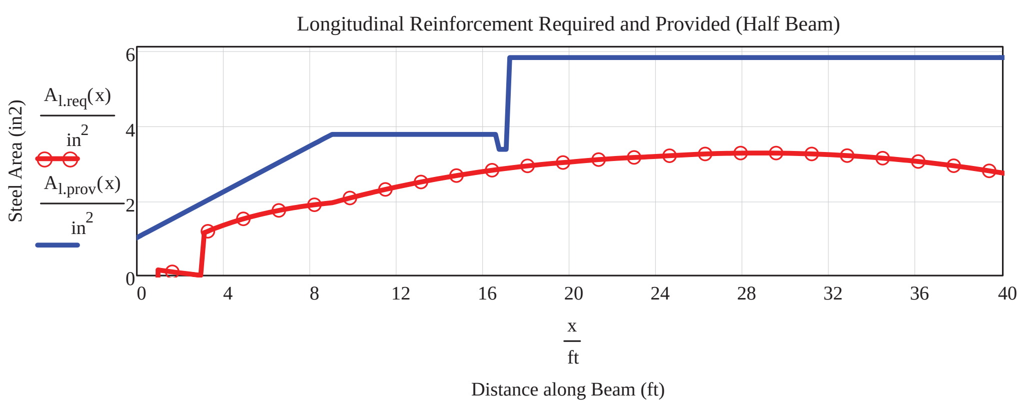
Longitudinal Reinforcement

$$A_{l.req}(x) := \left| \begin{array}{l} a1 \leftarrow \dfrac{M_{StrI}(x)}{\varphi_f \cdot f_{px}(x) \cdot \left(d_p(x) - \dfrac{a(x)}{2}\right)} \\ a2 \leftarrow \dfrac{\left(\dfrac{V_u(x)}{\varphi_v} - 0.5 \cdot V_s(x) - V_p(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ a3 \leftarrow \dfrac{\dfrac{M_{ushr}(x)}{d_v \cdot \varphi_f} + \left(\left|\dfrac{V_u(x)}{\varphi_v} - V_p(x)\right| - 0.5 \cdot V_s(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ \min(a1, a2) \quad \text{if } x \le d_v + 5 \cdot in \\ \min(a1, a3) \quad \text{otherwise} \end{array} \right.$$

Longitudinal reinforcement required for shear (5.8.3.5)

$A_{s.add} := 0.40 \cdot in^2$ $\quad L_{d.add} := 18.67 \cdot ft$ Additional longitudinal steel and developed length from end of beam

$$A_{l.prov}(x) := \text{if}\left(x < L_{d.add} - L_{end}, A_{s.add}, 0\right) + \left| \begin{array}{l} A_p \cdot N_s \cdot \dfrac{x + L_{end}}{L_d} \quad \text{if } x \le L_d - L_{end} \\ A_p \cdot N_s \quad \text{if } L_d - L_{end} < x \le \dfrac{y_{h.brg} - 0.5 \cdot h}{slope_{cgp}} + \left(\dfrac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) \cdot \cot\left(slope_{cgp}\right) \\ A_p \cdot \left(N_h + N_s\right) \quad \text{otherwise} \end{array} \right.$$



$A_{l.req_{ishr}} := A_{l.req}\left(x_{shr_{ishr}}\right)$ $\qquad A_{l.prov_{ishr}} := A_{l.prov}\left(x_{shr_{ishr}}\right)$

$DC_{long} := \dfrac{A_{l.req}}{A_{l.prov}}$ $\qquad \max\left(DC_{long}\right) = 0.844$

$CheckLong := \text{if}\left(\max\left(DC_{long}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ Longitudinal reinforcement check

14. SPLITTING RESISTANCE

Splitting Resistance

Checking splitting by zone of transverse reinforcement. Defined in Shear Strength section.

$$A_s := \frac{A_{v_1} \cdot x_{v_2}}{s_{v_1}} = 3.825 \cdot in^2$$

$f_s := 20 \cdot ksi$ — Limiting stress in steel for crack control (5.10.10.1)

$P_r := f_s \cdot A_s = 76.5 \cdot kip$ — Splitting resistance provided (5.10.10.1-1)

$P_{r.min} := 0.04 \cdot P_j = 47.1 \cdot kip$ — Minimum splitting resistance required

$\text{CheckSplit} := \text{if}\left(P_r \geq P_{r.min}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Splitting resistance check

15. CAMBER AND DEFLECTIONS

Calculate Deflections due to different weights, joints, and future wearings.

$$\Delta_{ps} := \frac{-P_i}{E_{ci} \cdot I_{xg}} \cdot \left[\frac{y_{cgp} \cdot L_g^{\,2}}{8} - \frac{\left(y_{bg} + y_{p.brg}\right) \cdot \left(\alpha_{hd} \cdot L + L_{end}\right)^2}{6}\right] = 1.456 \cdot in$$

Deflection due to prestress at release

$$\Delta_{gr} := \frac{-5}{384} \cdot \frac{w_g \cdot L_g^{\,4}}{E_{ci} \cdot I_{xg}} = -0.427 \cdot in$$

Deflection due to self-weight at release

$$\Delta_{bar} := \frac{-5}{384} \cdot \frac{w_{bar} \cdot L_g^{\,4}}{E_c \cdot I_{xg}} = -0.124 \cdot in$$

Deflection due to barrier weight

$$\Delta_j := \frac{-5}{384} \cdot \frac{w_j \cdot L^4}{E_c \cdot I_{xg}} \cdot \text{if}\,(\text{BeamLoc} = 0, 1, 0.5) = -0.006 \cdot in \qquad 2$$

Deflection due to longitudinal joint

$$\Delta_{fws} := \frac{-5}{384} \cdot \frac{w_{fws} \cdot L^4}{E_c \cdot I_{xg}} \cdot \text{if}\left(\text{BeamLoc} = 0, 1, \frac{S - W_b}{S}\right) = -0.036 \cdot in$$

Deflection due to future wearing surface

$t_{bar} := 20$ — Age at which barrier is assumed to be cast

$T := \left(t_i \;\; 7 \;\; 14 \;\; 21 \;\; 28 \;\; 60 \;\; 120 \;\; 240 \;\; \infty\right)^T$ — Concrete ages at which camber is computed

15. CAMBER AND DEFLECTIONS (cont'd)

$$\Delta_{cr1}(t) := \psi(t - t_i, t_i)(\Delta_{gr} + \Delta_{ps})$$

$$\Delta_{cr2}(t) := (\psi(t - t_i, t_i) - \psi(t_{bar} - t_i, t_i)) \cdot (\Delta_{gr} + \Delta_{ps}) + \psi(t - t_{bar}, t_{bar}) \cdot \Delta_{bar}$$

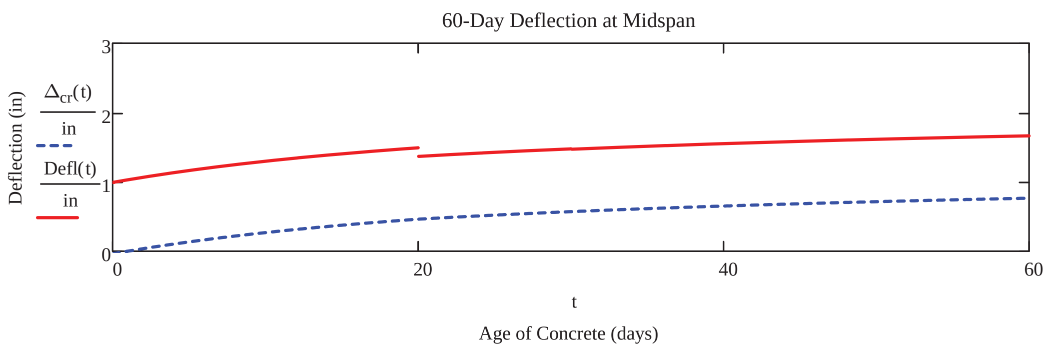
$$\Delta_{cr3}(t) := (\psi(t - t_i, t_i) - \psi(t_d - t_i, t_i)) \cdot (\Delta_{gr} + \Delta_{ps}) + (\psi(t - t_{bar}, t_{bar}) - \psi(t_d - t_{bar}, t_{bar})) \cdot \Delta_{bar} \ldots \\ + \psi(t - t_d, t_d) \cdot (\Delta_j)$$

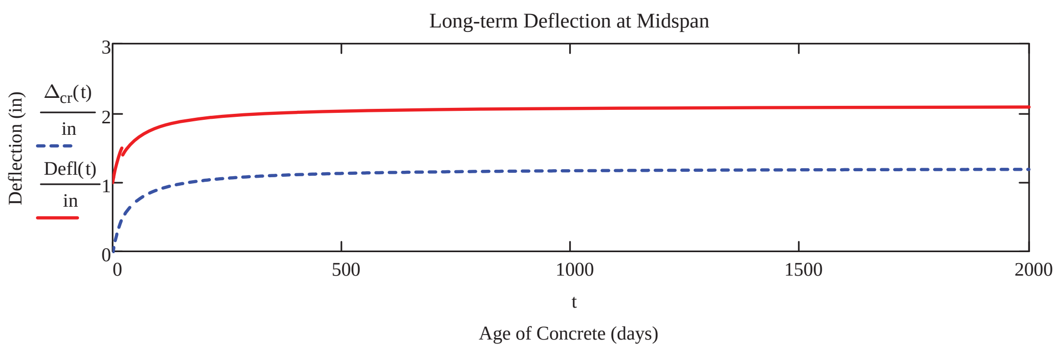
$$\Delta_{cr}(t) := \begin{cases} \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t_d) + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

$$\mathrm{Defl}(t) := \begin{cases} (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t_d) + \Delta_j + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

$$C := \left| \begin{array}{l} \text{for } j \in 1..\mathrm{last}(T) \\ \quad out_j \leftarrow \mathrm{Defl}(T_j) \\ out \end{array} \right.$$

$$C^T = (1.03 \;\; 1.221 \;\; 1.385 \;\; 1.38 \;\; 1.457 \;\; 1.664 \;\; 1.832 \;\; 1.95 \;\; 2.105) \cdot in$$





16. NEGATIVE MOMENT FLEXURAL STRENGTH

Calculate factored moment that must be resisted across the interior pier and find required steel to be developed in the top flange.

Negative Live Load Moment

Compute the negative moment over the interior support due to the design live load load, in accordance with AASHTO LRFD 3.6.1.3.1.

Live Load Truck and Truck Train Moment Calculations

$\min(M_{truck}) = -738 \cdot kip \cdot ft$	Maximum negative moment due to a single truck
$\min(M_{train}) = -1186 \cdot kip \cdot ft$	Maximum negative moment due to two trucks in a single lane
$M_{neg.lane} := \dfrac{-w_{lane} \cdot L^2}{2} = -1152 \cdot kip \cdot ft$	Negative moment due to lane load on adjacent spans
$M_{neg.truck} := M_{neg.lane} + (1 + IM) \cdot \min(M_{truck}) = -2134 \cdot kip \cdot ft$	Live load negative moment for single truck
$M_{neg.train} := 0.9 \cdot \left[M_{neg.lane} + (1 + IM) \cdot \min(M_{train})\right] = -2456 \cdot kip \cdot ft$	Live load negative moment for two trucks in a single lane
$M_{HL93.neg} := \min(M_{neg.truck}, M_{neg.train}) = -2456 \cdot kip \cdot ft$	Design negative live load moment, per design lane
$M_{ll.neg.i} := M_{HL93.neg} \cdot g_{mint} = -1644 \cdot kip \cdot ft$	Design negative live load moment at interior beam
$M_{ll.neg.e} := M_{HL93.neg} \cdot g_{mext} = -1712 \cdot kip \cdot ft$	Design negative live load moment at exterior beam
$M_{LL.neg} := \mathrm{if}(BeamLoc = 1, M_{ll.neg.e}, M_{ll.neg.i}) = -1712 \cdot kip \cdot ft$	Design negative live load moment

Factored Negative Design Moment

Dead load applied to the continuity section at interior supports is limited to the future overlay.

$M_{DW.neg} := \dfrac{-w_{fws} \cdot L^2}{2} = -357 \cdot kip \cdot ft$	Superimposed dead load resisted by continuity section
$M_{u.neg.StrI} := 1.5 \cdot M_{DW.neg} + 1.75 \cdot M_{LL.neg} = -3532 \cdot kip \cdot ft$	Strength Limit State
$M_{u.neg.StrI} := 1.0 \cdot M_{DW.neg} + 1.0 \cdot M_{LL.neg} = -2069 \cdot kip \cdot ft$	Service Limit State

16. NEGATIVE MOMENT FLEXURAL STRENGTH (cont'd)

Reinforcing Steel Requirement in the Top Flange for Strength

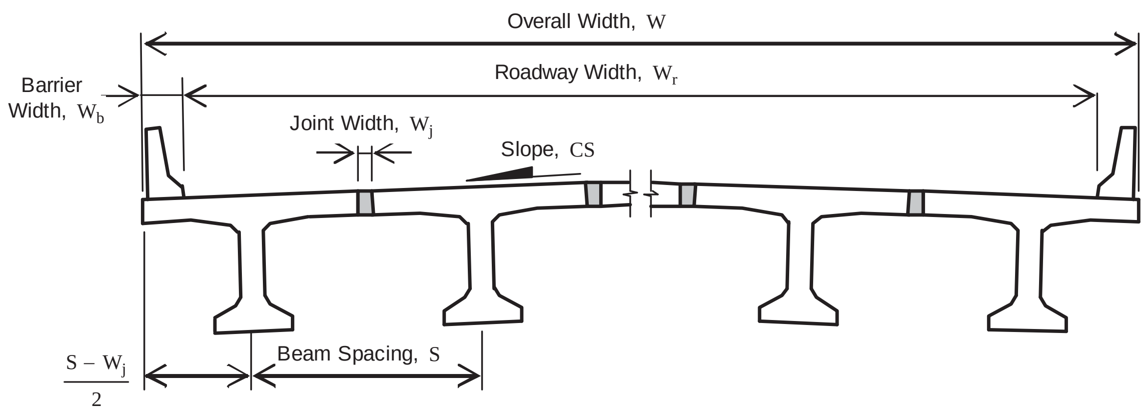
$\varphi_f := 0.90$	Reduction factor for strength in tension-controlled reinforced concrete (5.5.4.2)
$b_c := b_1 = 26 \cdot in$	Width of compression block at bottom flange
$d_{nms} := h - t_{sac} - 0.5 \cdot (t_{flange} - t_{sac}) = 40 \cdot in$	Distance to centroid of negative moment steel, taken at mid-depth of top flange
$R_u := \dfrac{\lvert M_{u.neg.StrI} \rvert}{\varphi_f \cdot b_c \cdot d_{nms}^2} = 0.663 \cdot ksi$	Factored load, in terms of stress in concrete at depth of steel, for computing steel requirement
$m := \dfrac{f_y}{0.85 \cdot f_c} = 8.824$	Steel-to-concrete strength ratio
$\rho_{req} := \dfrac{1}{m} \cdot \left(1 - \sqrt{1 - \dfrac{2 \cdot m \cdot R_u}{f_y}}\right) = 0.0117$	Required negative moment steel ratio
$A_{nms.req} := \rho_{req} \cdot b_c \cdot d_{nms} = 12.12 \cdot in^2$	Required negative moment steel in top flange
$A_{s.long.t} := 2.0 \cdot in^2 \qquad A_{s.long.b} := 2.0 \cdot in^2$	Full-length longitudinal reinforcement to be made continuous across joint
$A_{bar} := 0.44 \cdot in^2$	Additional negative moment reinforcing bar area
$A_{nms.t} := \dfrac{2}{3} \cdot A_{nms.req} - A_{s.long.t} = 6.08 \cdot in^2$	Additional reinforcement area required in the top mat (2/3 of total)
$n_{bar.t} := \text{ceil}\left(\dfrac{A_{nms.t}}{A_{bar}}\right) = 14$	Additional bars required in the top mat
$A_{nms.b} := \dfrac{1}{3} \cdot A_{nms.req} - A_{s.long.b} = 2.04 \cdot in^2$	Additional reinforcement area required in the bottom mat
$n_{bar.b} := \text{ceil}\left(\dfrac{A_{nms.b}}{A_{bar}}\right) = 5$	Additional bars required in the top mat
$s_{bar.top} := \dfrac{S - W_j - 6 \cdot in}{n_{bar.t} - 1} = 6.404 \cdot in$	Spacing of bars in top mat
$A_{s.nms} := (n_{bar.t} + n_{bar.b}) \cdot A_{bar} + A_{s.long.t} + A_{s.long.b} = 12.36 \cdot in^2$	Total reinforcing steel provided over pier
$a := \dfrac{A_{s.nms} \cdot f_y}{0.85 \cdot f_c \cdot b_c} = 4.195 \cdot in$	Depth of compression block
$M_{r.neg} := \varphi_f \cdot A_{s.nms} \cdot f_y \cdot \left(d_{nms} - \dfrac{a}{2}\right) = 2108 \cdot kip \cdot ft$	Factored flexural resistance at interior pier
$DC_{neg.mom} := \dfrac{\lvert M_{u.neg.StrI} \rvert}{M_{r.neg}} = 0.982$	
$\text{CheckNegMom} := \text{if}(DC_{neg.mom} \le 1.0, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$	Negative flexure resistance check

File Name: Prestressed Concrete Girder-40ft.xmcd

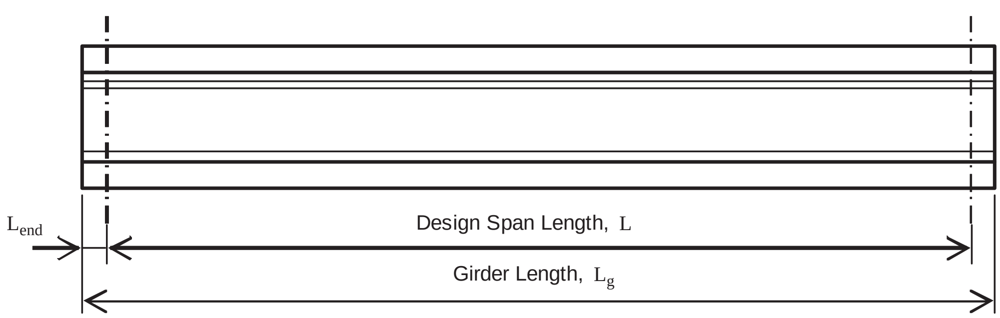
DECKED PRECAST PRESTRESSED CONCRETE GIRDER DESIGN FOR ABC

Unit Definition: $kcf \equiv kip \cdot ft^{-3}$

This example is for the design of a superstructure system that can be used for rapid bridge replacement in an Accelerated Bridge Construction (ABC) application. The following calculations are intended to provide the designer guidance in developing a similar design with regard to design considerationS characteristic of this type of construction, and they shall not be considered fully exhaustive.



TYPICAL SECTION THROUGH SPAN



GIRDER ELEVATION

Bridge Geometry:

$L := 40 \cdot ft$ $\qquad L_{end} := 2 \cdot ft$ $\qquad skew := 0 \cdot deg$

$W := 63 \cdot ft$ $\qquad W_b := 1.5 \cdot ft$

$S_{max} := 8 \cdot ft$ $\qquad W_j := 0.5 \cdot ft$

$N_g := ceil\left(\frac{W + W_j}{S_{max}}\right) = 8$ Minimum number of girders in cross-section

$S := \frac{W + W_j}{N_g} = 7.938 \cdot ft$ Girder spacing

ORDER OF CALCULATIONS

1. Introduction
2. Design Philosophy
3. Design Criteria
4. Beam Section
5. Material Properties
6. Permanent Loads
7. Precast Lifting Weight
8. Live Load
9. Prestress Properties
10. Prestress Losses
11. Concrete Stresses
12. Flexural Strength
13. Shear Strength
14. Splitting Resistance
15. Camber and Deflections
16. Negative Moment Flexural Strength

1. INTRODUCTION

The bridge that is designed in this example consists of precast prestressed concrete girders with a top flange equal to the beam spacing, so the top flange will be the riding surface of the designed bridge. The purpose for these girders is to rapidly construct the bridge by providing a precast deck on the girders, which eliminates cast-in-place decks in the field and improves safety.

The concepts used in this example have been taken from on-going research, which focuses on the benefits of decked precast beams and promoting widespread acceptance from transportation and construction industries. The cross-section is adapted from the optimized girder sections recommended by NCHRP Project No. 12-69, Design and Construction Guidelines for Long-Span Decked Precast, Prestressed Concrete Girder Bridges. The girder design has not taken into account the option to re-deck due to the final re-decked girder, without additional prestressed, having a shorter life span. Use of stainless steal rebar and the application of a future membrane can get ride of the need to replace the deck. This case is included in “re-deckability”.

The bridge used in this example is a general design of a typical bridge in Georgia. The calculations can be modified for single-span and multiple-span bridges due to the beam design moments are not reduced for continuity at intermediate supports (continuity details are not shown in this example). The cross-section consists of a four-lane roadway with normal crown, with standard shoulder lengths and barrier walls. The precast prestressed concrete girder has been uniformly designed to simplify bearing details. The girder flanges are 9’’ at the tips, imitating a 8’’ slab with a ½’’ allowable wear and another ½’’ for smoothness and profile adjustments.

This example is intended to illustrate design aspect specific to precast prestressed concrete girders used for ABC application. Girders with uncommon cross-sections, high self-weight, or unconventional load application create major concerns and more detailed calculations must be done.

2. DESIGN PHILOSOPHY

The geometry of the section is based on GDOT standards and general bridges across the state of Georgia. Depth variations are dependent on the construction company but must maintain the shapes of the top flange and the bottom bulb.

Concrete strengths can vary but are mostly between 6 ksi and 10 ksi. For the purpose of these calculations the concrete with a 28-day minimum compressive strength of 8 ksi is used. Due to its casting sequence this beam is unable to take advantage of composite sequences along with tension at the bottom of the beam at the service limit state being limited. This is further discussed in section 4 along with end region stresses being critical. Therefore the minimum concrete strength at release must be 80 percent of the 28-day compressive strength, which increases the allowable stresses at the top and bottom of the section. The prestressed steel can also be optimized to minimize stresses at the end region.

The prestressed steel is arranged in a draped, or harped, pattern to maximize the midspan effectiveness while it minimizes the failure at the end of the beam where is concrete is easily overstressed due to the lack of dead load acting on the beam. The strand group is optimized at the midspan by bundling the strands between hold-points, maximizing the stiffness of the strand group. The number and deflection angles are depended on the type of single strands you are using for the girder. In longer span cases the concrete at the end of the girder will be too large and will debond. Without harped strands it is unlikely to reduce stresses to the allowable limit, since harped strands are required this method of stress relief will be used without debonding for long spans.

3. DESIGN CRITERIA

Criteria has been selected to govern the design of these concrete girders while following provisions set by AASHTO, GDOT design specifications, as well as criteria of past projects and current research related to ABC and decked precast sections. A summary of the limiting design values are categorized as section constraints, prestress limits, and concrete limits.

Section Constraints:

$W_{pc.max} := 200 \cdot kip$ — Upper limit on the weight of the entire precast element, based on common lifting and transport capabilities without significantly increasing time and/or cost due to unconventional equipment or permits

$S_{max} = 8 \cdot ft$ — Upper limit on girder spacing and, therefore, girder flange width (defined on first page)

Prestress Limits:

$F_{hd.single} := 4 \cdot kip$ — Maximum hold-down force for a single strand

$F_{hd.group} := 48 \cdot kip$ — Maximum hold-down force for the group of harped strands

Stress limits in the prestressing steel immediately prior to prestress and at the service limit state after all losses are as prescribed by AASHTO LRFD.

3. DESIGN CRITERIA (cont'd)

Concrete Limits:

Allowable concrete stresses meet standards set by AASHTO LRFD with one exception that at Service III Limit State, allowable bottom fiber tension when camber leveling forces are to be neglected, regardless of exposure, are to be 0-ksi. Minimum strength of concrete at release is 80 percent of the 28-day minimum compressive strength (f-ksi).

$f_{t.all.ser} := 0 \cdot ksi$	Allowable bottom fiber tension at the Service III Limit State, when camber leveling forces are to be neglected, regardless of exposure

As previously mentioned, release concrete strength is specified as 80 percent of the minimum 28-day compressive strength to maximize allowable stresses in the end region of beam at release.

$f_{c.rel}(f) := 0.80 \cdot f$	Minimum strength of concrete at release

Due to various lifting and transportation conditions, stresses in the concrete need to be considered. A “no cracking” approach is used for allowable tension due to reduced lateral stability after cracking. Assuming the girders will be lifted before the 28-day minimum strength is attained, the strength of concrete during lifting and transportation is assumed to be 90 percent of the 28-day minimum compressive strength. A dynamic dead load allowance of 30 percent is used for compression during handling. A factor of safety (FS) of 1.5 is used against cracking during handling.

$DIM := 30\%$	Dynamic dead load allowance
$f_{c.erec}(f) := 0.90 \cdot f$	Assumed attained concrete strength during lifting and transportation
$FS_c := 1.5$	Factor of safety against cracking during lifting transportation
$f_{t.erec}(f) := \dfrac{-0.24 \cdot \sqrt{f \cdot ksi}}{FS_c}$	Allowable tension in concrete during lifting and transportation to avoid cracking

4. BEAM SECTION

Use trapezoidal areas to define the cross-section. The flange width is defined as the beam spacing less the width of the longitudinal closure joint to reflect pre-erection conditions. Live load can be conservatively applied to this section, as well.

$h := 40.5 \cdot in$ Beam section depth

$t_{flange} := 9 \cdot in$ Flange thickness at tip

$t_{sac} := 1 \cdot in$ Total sacrificial depth for grinding and wear

$b_1 := 26 \cdot in$ $b_2 := 26 \cdot in$ $d_1 := 6 \cdot in$

$b_2 = 26 \cdot in$ $b_3 := 6 \cdot in$ $d_2 := 4.5 \cdot in$

$b_3 = 6 \cdot in$ $b_4 := 6 \cdot in$

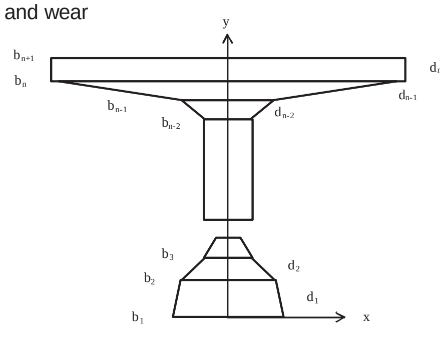
$b_4 = 6 \cdot in$ $b_5 := 10 \cdot in$ $d_4 := 2 \cdot in$

$b_5 = 10 \cdot in$ $b_6 := 42 \cdot in$ $d_5 := 2 \cdot in$

$b_6 = 42 \cdot in$ $b_7 := S - W_j$ $d_6 := 0 \cdot in$

$b_7 = 89.25 \cdot in$ $b_8 := S - W_j$ $d_7 := t_{flange} - t_{sac}$

$d_3 := h - t_{sac} - \sum d$ $d_3 = 17 \cdot in$



TYPICAL GIRDER SECTION COMPRISED OF n TRAPEZOIDAL REGIONS

Gross Section Properties

$b_f = 89.25 \cdot in$ Precast girder flange width

$A_g = 1112 \cdot in^2$ Cross-sectional area (does not include sacrifical thickness)

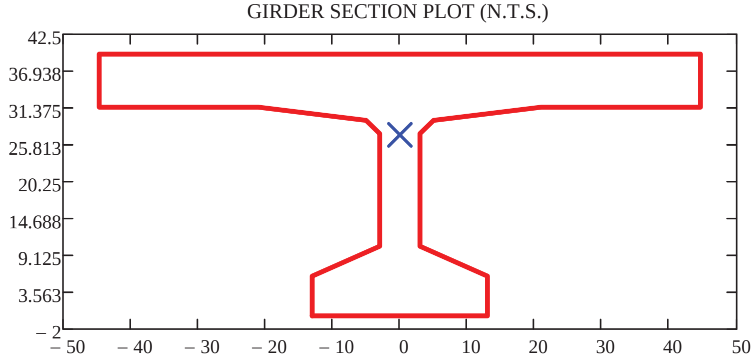
$I_{xg} = 182071 \cdot in^4$ Moment of inertia (does not include sacrificial thickness)

$y_{tg} = 12.191 \cdot in$ $y_{bg} = -27.309 \cdot in$ Top and bottom fiber distances from neutal axis (positive up)

$S_{tg} = 14934.5 \cdot in^3$ $S_{bg} = -6667.1 \cdot in^3$ Top and bottom section moduli

$I_{yg} = 487596 \cdot in^4$ Weak-axis moment of inertia

GIRDER SECTION PLOT (N.T.S.)



5. MATERIAL PROPERTIES

These properties are standard (US units) values with equations that can be found in AASHTO LRFD Bridge Design Specifications.

Concrete:

$f_c := 8 \cdot ksi$	Minimum 28-day compressive strength of concrete
$f_{ci} := f_{c.rel}(f_c) = 6.4 \cdot ksi$	Minimum strength of concrete at release
$\gamma_c := .150 \cdot kcf$	Unit weight of concrete
$K_1 := 1.0$	Correction factor for standard aggregate (5.4.2.4)
$E_{ci} := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_{ci} \cdot ksi} = 4850 \cdot ksi$	Modulus of elasticity at release (5.4.2.4-1)
$E_c := 33000 \cdot K_1 \cdot \left(\frac{\gamma_c}{kcf}\right)^{1.5} \cdot \sqrt{f_c \cdot ksi} = 5422 \cdot ksi$	Modulus of elasticity (5.4.2.4-1)
$f_{r.cm} := 0.37 \cdot \sqrt{f_c \cdot ksi} = 1.047 \cdot ksi$	Modulus of rupture for cracking moment (5.4.2.6)
$f_{r.cd} := 0.24 \cdot \sqrt{f_c \cdot ksi} = 0.679 \cdot ksi$	Modulus of rupture for camber and deflection (5.4.2.6)
$H := 70$	Relative humidity (5.4.2.3)

Prestressing Steel:

$f_{pu} := 270 \cdot ksi$	Ultimate tensile strength
$f_{py} := 0.9 \cdot f_{pu} = 243 \cdot ksi$	Yield strength, low-relaxation strand (Table 5.4.4.1-1)
$f_{pbt.max} := 0.75 \cdot f_{pu} = 202.5 \cdot ksi$	Maximum stress in steel immediately prior to transfer
$f_{pe.max} := 0.80 \cdot f_{py} = 194.4 \cdot ksi$	Maximum stress in steel after all losses
$E_p := 28500 \cdot ksi$	Modulus of elasticity (5.4.4.2)
$d_{ps} := 0.5 \cdot in$	Strand diameter
$A_p := 0.153 \cdot in^2$	Strand area
$N_{ps.max} := 40$	Maximum number of strands in section
$n_{pi} := \frac{E_p}{E_{ci}} = 5.9$	Modular ratio at release
$n_p := \frac{E_p}{E_c} = 5.3$	Modular ratio

Mild Steel:

$f_y := 60 \cdot ksi$ Specified minimum yield strength

$E_s := 29000 \cdot ksi$ Modulus of elasticity (5.4.3.2)

6. PERMANENT LOADS

Permanent loads or dead loads that must be considered are self-weight, diaphragms, barriers, and future wearing surface. The barrier can be cast to the beam before it is taken on sight or attached to the bridge after the joints have reached sufficient strength. Distribution of the barriers weight should be established once you decide when it would be attached to the bridge. For this example the barrier has been cast on the exterior girder in the casting yard, before shipping but after release of prestresses. Due to this the dead load is increased on the exterior girders but it eliminates the time-consuming task that would have been completed in the field.

$BeamLoc := 1$ — Location of beam within the cross-section (0 - Interior, 1 - Exterior)

Load at Release:

$\gamma_{c.DL} := .155\cdot kcf$ — Concrete density used for weight calculations

$A_{g.DL} := A_g + t_{sac}\cdot(S - W_j) = 1201.25\cdot in^2$ — Area used for weight calculations, including sacrificial thickness

$w_g := A_{g.DL}\cdot\gamma_{c.DL} = 1.293\cdot klf$ — Uniform load due to self-weight, including sacrificial thickness

$L_g := L + 2\cdot L_{end} = 44\cdot ft$ — Span length at release

$M_{gr}(x) := \dfrac{w_g\cdot x}{2}\cdot(L_g - x)$ — Moment due to beam self-weight (supported at ends)

$V_{gr}(x) := w_g\cdot\left(\dfrac{L_g}{2} - x\right)$ — Shear due to beam self-weight (supported at ends)

Load at Erection:

$M_g(x) := \dfrac{w_g\cdot x}{2}\cdot(L - x)$ — Moment due to beam self-weight

$V_g(x) := w_g\cdot\left(\dfrac{L}{2} - x\right)$ — Shear due to beam self-weight

$w_{bar} := 0.430\cdot klf$ — Uniform load due to barrier weight, exterior beams only

$w_{bar} := if(BeamLoc = 1, w_{bar}, 0) = 0.43\cdot klf$ — Redfine to 0 if interior beam (BeamLoc = 0)

$M_{bar}(x) := \dfrac{w_{bar}\cdot x}{2}\cdot(L - x)$ — Moment due to beam self-weight

$V_{bar}(x) := w_{bar}\cdot\left(\dfrac{L}{2} - x\right)$ — Shear due to beam self-weight

6. PERMANENT LOADS (cont'd)

Load at Service:

$p_{fws} := 25 \cdot psf$ Assumed weight of future wearing surface

$w_{fws} := p_{fws} \cdot S = 0.198 \cdot klf$ Uniform load due to future wearing surface

$M_{fws}(x) := \frac{w_{fws} \cdot x}{2} \cdot (L - x)$ Moment due to future wearing surface

$V_{fws}(x) := w_{fws} \cdot \left(\frac{L}{2} - x\right)$ Shear due to future wearing surface

$w_j := W_j \cdot d_7 \cdot \gamma_{c.DL} = 0.052 \cdot klf$ Uniform load due to weight of longitudinal closure joint

$M_j(x) := \frac{w_j \cdot x}{2} \cdot (L - x)$ Moment due to longitudinal closure joint

$V_j(x) := w_j \cdot \left(\frac{L}{2} - x\right)$ Shear due to longitudinal closure joint

7. PRECAST LIFTING WEIGHT

For Accelerated Bridge Construction the beams are casted in a factory and transported to the job site. When they arrive at the site they must be lifted and put into place. When designing we have to consider the weight of each slab to insure safety and design for possible cracking.

Precast Superstructure

$W_g := (w_g + w_{bar}) \cdot L_g = 75.8 \cdot kip$ — Precast girder, including barrier if necessary

Substructure Precast with Superstructure

$L_{corb} := 1 \cdot ft$ — Length of approach slab corbel

$B_{corb} := b_f$ $\quad b_f = 89.25 \cdot in$ — Width of corbel cast with girder

$D_{corb} := 1.5 \cdot ft$ — Average depth of corbel

$V_{corb} := L_{corb} \cdot B_{corb} \cdot D_{corb} = 11.16 \cdot ft^3$ — Volume of corbel

$L_{ia} := 2.167 \cdot ft$ — Length of integral abutment

$L_{gia} := 1.167 \cdot ft$ — Length of girder embedded in integral abutment

$B_{ia} := S - W_j = 7.438 \cdot ft$ — Width of integral abutment cast with girder

$D_{ia} := h + 4 \cdot in = 44.5 \cdot in$ — Depth of integral abutment

$V_{ia} := V_{corb} + \left[L_{ia} \cdot B_{ia} \cdot D_{ia} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gia}\right] = 68.42 \cdot ft^3$ — Volume of integral abutment cast with girder

$W_{ia} := V_{ia} \cdot \gamma_c = 10 \cdot kip$ — Weight of integral abutment cast with girder

$L_{sa} := 2.167 \cdot ft$ — Length of semi-integral abutment

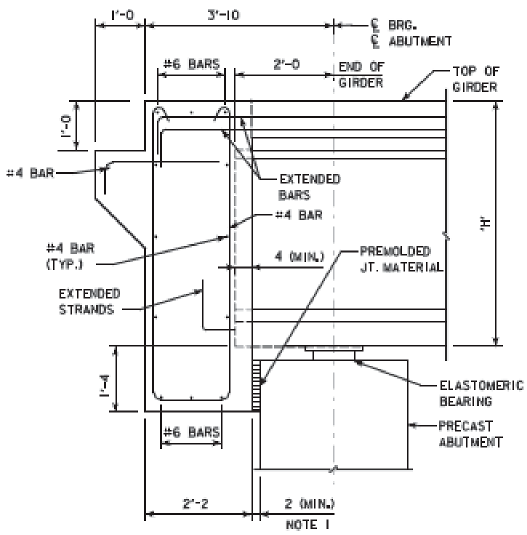
$L_{gsa} := 4 \cdot in$ — Length of girder embedded in semi-integral abutment

$B_{sa} := S - W_j = 7.438 \cdot ft$ — Width of semi-integral abutment cast with girder

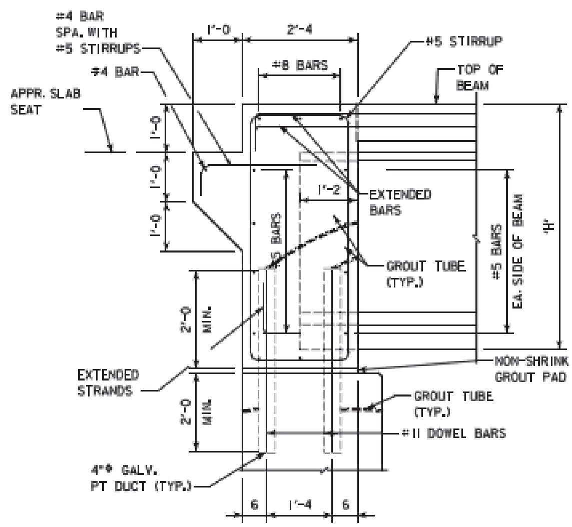
$D_{sa} := h + 16 \cdot in = 56.5 \cdot in$ — Depth of semi-integral abutment

$V_{sa} := V_{corb} + \left[L_{sa} \cdot B_{sa} \cdot D_{sa} - \left(A_g - t_{flange} \cdot b_f\right) \cdot L_{gsa}\right] = 86.33 \cdot ft^3$ — Volume of semi-integral abutment cast with girder

$W_{sa} := V_{sa} \cdot \gamma_c = 13 \cdot kip$ — Weight of semi-integral abutment cast with girder



Semi-Integral Abutment Backwall



Integral Abutment Backwall

8. LIVE LOAD

When considering Live Loads you must refer to the vertical load section HL-93 in the AASHTO manual. If the project you are working on requires the bridge to support construction loads at any stage, these loads must be considered separately and applied. The longitudinal joints are designed for full moment connections so the beams will act as a unit when sufficiently connected. The distribution factors are then computed for cross-section type "j" (defined in AASHTO 4.6.2.2). When calculating the stiffness parameter, the constant- depth region at the top flange is treated like the slab and the remaining area of the beam will be considered a non-composite beam.

Definitions:

I_{bb}	=	moment of inertia of section below the top flange
A_{bb}	=	area of beam section below the top flange
y_{bb}	=	distance of top fiber below the top flange from neutral axis
t_s	=	thickness of slab not including sacrificial thickness

Distribution Factors for Moment:

From Table 4.6.2.2.2b-1 for moment in interior girders,

$$I_{bb} = 44410 \cdot in^4$$
Moment of inertia of section below the top flange

$$A_{bb} = 398 \cdot in^2$$
Area of beam section below the top flange

$$e_g := h - \left(t_{sac} + \frac{t_s}{2}\right) + y_{bb} = 22.886 \cdot in$$
Distance between c.g.'s of beam and flange

$$K_g := 1.0 \cdot \left(I_{bb} + A_{bb} \cdot e_g^{\,2}\right) = 252876 \cdot in^4$$
Longitudinal stiffness parameter (Eqn. 4.6.2.2.1-1)

Verify this girder design is within the range of applicability for Table 4.6.2.2.2b-1.

$$CheckMint := if\left[(S \le 16 \cdot ft) \cdot (S \ge 3.5 \cdot ft) \cdot (t_s \ge 4.5 \cdot in) \cdot (t_s \le 12.0 \cdot in) \cdot (L \ge 20 \cdot ft) \cdot (L \le 240 \cdot ft), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckMint := if\left[(CheckMint = \text{"OK"}) \cdot (N_g \ge 4) \cdot \left(K_g \ge 10000 \cdot in^4\right) \cdot \left(K_g \le 7000000 \cdot in^4\right), \text{"OK"}, \text{"No Good"}\right]$$

$$CheckMint = \text{"OK"}$$

$$g_{mint1} := 0.06 + \left(\frac{S}{14 \cdot ft}\right)^{0.4} \cdot \left(\frac{S}{L}\right)^{0.3} \cdot \left(\frac{K_g}{L \cdot t_s^{\,3}}\right)^{0.1} = 0.552$$
Single loaded lane

$$g_{mint2} := 0.075 + \left(\frac{S}{9.5 \cdot ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot t_s^{\,3}}\right)^{0.1} = 0.727$$
Two or more loaded lanes

$$g_{mint} := \max\left(g_{mint1}, g_{mint2}\right) = 0.727$$
Distribution factor for moment at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.2d-1 for moment in exterior girders,

$$d_e := \frac{S}{2} - W_b = 29.625 \cdot in$$

Distance from centerline of exterior beam to edge of curb or barrier

$$CheckMext := if\left[\left(d_e \geq -1 \cdot ft\right) \cdot \left(d_e \leq 5.5 \cdot ft\right) \cdot \left(N_g \geq 4\right), "OK", "No Good"\right] = "OK"$$

For a single loaded lane, use the Lever Rule.

$$g_{mext1} := \frac{\left(S + 0.5 \cdot b_f - W_b - 5 \cdot ft\right)}{S} = 0.65$$

Single loaded lane

$$e_m := 0.77 + \frac{d_e}{9.1 \cdot ft} = 1.041$$

Correction factor for moment (Table 4.6.2.2.2d-1)

$$g_{mext2} := e_m \cdot g_{mint} = 0.757$$

Two or more loaded lanes

$$g_{mext} := \max\left(g_{mext1}, g_{mext2}\right) = 0.757$$

Distribution factor for moment at exterior beams

Distribution Factors for Shear:

From Table 4.6.2.2.3a-1 for shear in interior girders,

Verify this girder design is within the range of applicability for Table 4.6.2.2.3a-1.

$$CheckVint := if\left[(S \leq 16 \cdot ft) \cdot (S \geq 3.5 \cdot ft) \cdot \left(t_s \geq 4.5 \cdot in\right) \cdot \left(t_s \leq 12.0in\right) \cdot (L \geq 20 \cdot ft) \cdot (L \leq 240 \cdot ft), "OK", "No Good"\right]$$

$$CheckVint := if\left[(CheckMint = "OK") \cdot \left(N_g \geq 4\right), "OK", "No Good"\right]$$

$$CheckVint = "OK"$$

$$g_{vint1} := 0.36 + \left(\frac{S}{25 \cdot ft}\right) = 0.678$$

Single loaded lane

$$g_{vint2} := 0.2 + \left(\frac{S}{12 \cdot ft}\right) - \left(\frac{S}{35 \cdot ft}\right)^{2.0} = 0.81$$

Two or more loaded lanes

$$g_{vint} := \max\left(g_{vint1}, g_{vint2}\right) = 0.81$$

Distribution factor for shear at interior beams

8. LIVE LOAD (cont'd)

From Table 4.6.2.2.3b-1 for shear in exterior girders,

For a single loaded lane, use the Lever Rule.

$$\text{CheckVext} := \text{if}\left[\left(d_e \geq -1\cdot ft\right)\cdot\left(d_e \leq 5.5\cdot ft\right)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$g_1 := \frac{\left(S + 0.5\cdot b_f - W_b - 5\cdot ft\right)}{S} = 0.65$$ Single loaded lane (same as for moment)

$$e_v := 0.6 + \frac{d_e}{10\cdot ft} = 0.847$$ Correction factor for shear (Table 4.6.2.2.3b-1)

$$g_2 := e_v\cdot g_{vint} = 0.686$$ Two or more loaded lanes

$$g_{vext} := \max\left(g_1, g_2\right) = 0.686$$ Distribution factor for shear at exterior beams

From Table 4.6.2.2.3c-1 for skewed bridges,

$$\theta := skew = 0\cdot deg$$

$$\text{CheckSkew} := \text{if}\left[(\theta \leq 60\cdot deg)\cdot(3.5\cdot ft \leq S \leq 16\cdot ft)\cdot(20\cdot ft \leq L \leq 240\cdot ft)\cdot\left(N_g \geq 4\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$c_{skew} := 1.0 + 0.20\cdot\left(\frac{L\cdot t_s^{\,3}}{K_g}\right)^{0.3}\cdot\tan(\theta) = 1.00$$ Correction factor for skew

8. LIVE LOAD (cont'd)

Design Live Load Moment at Midspan:

$w_{lane} := 0.64 \cdot klf$ — Design lane load

$P_{truck} := 32 \cdot kip$ — Design truck axle load

$IM := 33\%$ — Dynamic load allowance (truck only)

$M_{lane}(x) := \frac{w_{lane} \cdot x}{2} \cdot (L - x)$ — Design lane load moment

$\delta(x) := \frac{x \cdot L - x^2}{L}$ — Influence coefficient for truck moment calculation

$M_{truck}(x) := P_{truck} \cdot \delta(x) \cdot \max\left[\frac{9 \cdot x \cdot (L - x) - 14 \cdot ft \cdot (3 \cdot x + L)}{4 \cdot x \cdot (L - x)}, \frac{9 \cdot (L - x) - 84 \cdot ft}{4 \cdot (L - x)}\right]$ — Design truck moment

$M_{HL93}(x) := M_{lane}(x) + (1 + IM) \cdot M_{truck}(x)$ — HL93 design live load moment per lane

$M_{ll.i}(x) := M_{HL93}(x) \cdot g_{mint}$ — Design live load moment at interior beam

$M_{ll.e}(x) := M_{HL93}(x) \cdot g_{mext}$ — Design live load moment at exterior beam

$M_{ll}(x) := \text{if}\left(BeamLoc = 1, M_{ll.e}(x), M_{ll.i}(x)\right)$ — Design live load moment

Design Live Load Shear:

$V_{lane}(x) := w_{lane} \cdot \left(\frac{L}{2} - x\right)$ — Design lane load shear

$V_{truck}(x) := P_{truck} \cdot \left(\frac{9 \cdot L - 9 \cdot x - 84 \cdot ft}{4 \cdot L}\right)$ — Design truck shear

$V_{HL93}(x) := V_{lane}(x) + (1 + IM) \cdot V_{truck}(x)$ — HL93 design live load shear

$V_{ll.i}(x) := V_{HL93}(x) \cdot g_{vint}$ — Design live load shear at interior beam

$V_{ll.e}(x) := V_{HL93}(x) \cdot g_{vext}$ — Design live load shear at exterior beam

$V_{ll}(x) := \text{if}\left(BeamLoc = 1, V_{ll.e}(x), V_{ll.i}(x)\right)$ — Design live load shear

9. PRESTRESS PROPERTIES

Due to tension at the surface limit state be reduced to account for camber leveling forces, the prestress force required at the midspan is expected to be excessive at the ends when released. Not measuring the reduction of prestress moments. Estimate prestress losses at the midspan to find trial prestress forces, that will occur in the bottom tension fibers, that are less than allowable. Compute immediate losses in the prestressed steel and check released stresses at the end of the beam. Once you satisfy end stresses, estimate total loss of prestress. As long as these losses are not drastically different from the assumed stresses, the prestress layout should be acceptable. Concrete stress at all limit states are in Section 9.

$y_{p.est} := 5 \cdot in$	Assumed distance from bottom of beam to centroid of prestress at midspan
$y_{cgp.est} := y_{bg} + y_{p.est} = -22.31 \cdot in$	Eccentricity of prestress from neutral axis, based on assumed location
$\Delta f_{p.est} := 25\%$	Estimate of total prestress losses at the service limit state

Compute bottom fiber service stresses at midspan using gross section properties.

$X := \frac{L}{2}$	Distance from support
$M_{dl.ser} := M_g(X) + M_{fws}(X) + M_j(X) + M_{bar}(X) = 395 \cdot kip \cdot ft$	Total dead load moment
$f_{b.serIII} := \frac{M_{dl.ser} + 0.8 \cdot M_{ll}(X)}{S_{bg}} = -1.487 \cdot ksi$	Total bottom fiber service stress
$f_{pj} := f_{pbt.max} = 202.5 \cdot ksi$	Prestress jacking force
$f_{pe.est} := f_{pj} \cdot \left(1 - \Delta f_{p.est}\right) = 151.9 \cdot ksi$	Estimate of effective prestress force
$A_{ps.est} := A_g \cdot \frac{\left(\frac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \frac{A_g \cdot y_{cgp.est}}{S_{bg}}} = 2.307 \cdot in^2$	Estimated minimum area of prestressing steel
$N_{ps.est} := ceil\left(\frac{A_{ps.est}}{A_p}\right) = 16$	Estimated number of strands required
$N_{ps} := 38$	Number of strands used ($N_{ps.max} = 40$)

The number above is used for the layout strand pattern and to compute the actual location of the strand group. After this is done the required area is computed again. If the estimated location is accurate the number of strands should be equal to the number of strands that we calculated above. The number of strands that was estimated was based on our assumed prestressed losses and gross section properties, which may not accurately reflect the final number of strands required for the design. These stresses for concrete are evaluated in Section 10.
The geometry is assuming a vertical spacing of 2” between straight spans, as well as 2” for harped strands at the end of the beam. Harped strands are bundled at the midspan where the centroid is 5” from the bottom.

9. PRESTRESS PROPERTIES (cont'd)

$$N_h := \begin{cases} 2 & \text{if } N_{ps} \le 12 \\ 4 & \text{if } 12 < N_{ps} \le 24 \\ 6 & \text{if } 24 < N_{ps} \le 30 \\ 6 + (N_{ps} - 30) & \text{if } N_{ps} > 30 \end{cases}$$

$N_h = 14$

Assumes all flange rows are filled prior to filling rows in web above the flange, which maximized efficiency. Use override below to shift strands from flange to web if needed to satisfy end stresses.

$N_{h.add} := 16$

Additional harped strands in web (strands to be moved from flange to web)

$N_h := \min\left(N_h + N_{h.add}, 16, 2\cdot \text{floor}\left(\frac{N_{ps}}{4}\right)\right)$ $N_h = 16$

16 strands or half of total strands maximum harped in web

$y_h := 1\cdot in + (2\cdot in)\cdot\left(1 + \frac{0.5\cdot N_h - 1}{2}\right)$ $y_h = 10\cdot in$

Centroid of harped strands from bottom, equally spaced

$y_{hb} := 5\cdot in$

Centroid of harped strands from bottom, bundled

$N_s := N_{ps} - N_h$ $N_s = 22$

Number of straight strands in flange

$$y_s := 1\cdot in + \begin{cases} 2\cdot in & \text{if } N_s \le 10 \\ \dfrac{(4\cdot in)\cdot N_s - 20\cdot in}{N_s} & \text{if } 10 < N_s \le 20 \\ \dfrac{(6\cdot in)\cdot N_s - 60\cdot in}{N_s} & \text{if } 20 < N_s \le 24 \\ 3.5\cdot in & \text{otherwise} \end{cases}$$

$y_s = 4.273\cdot in$

Centroid of straight strands from bottom

$y_p := \frac{N_s\cdot y_s + N_h\cdot y_{hb}}{N_s + N_h} = 4.579\cdot in$

Centroid of prestress from bottom at midspan

$y_{cgp} := y_{bg} + y_p = -22.73\cdot in$

Eccentricity of prestress from neutral axis

$$A_{ps.req} := A_g\cdot\frac{\left(\frac{-f_{b.serIII} + f_{t.all.ser}}{f_{pe.est}}\right)}{1 + \frac{A_g\cdot y_{cgp}}{S_{bg}}} = 2.273\cdot in^2$$

Estimated minimum area of prestressing steel

$N_{ps.req} := \text{ceil}\left(\frac{A_{ps.req}}{A_p}\right) = 15$

Estimated number of strands required

$CheckNps := \text{if}\left[(N_{ps} \le N_{ps.max})\cdot(N_{ps.req} \le N_{ps}), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

$A_{ps.h} := N_h\cdot A_p = 2.448\cdot in^2$

Area of prestress in web (harped)

$A_{ps.s} := N_s\cdot A_p = 3.366\cdot in^2$

Area of prestress in flange (straight)

$A_{ps} := A_{ps.h} + A_{ps.s} = 5.814\cdot in^2$

Total area of prestress

9. PRESTRESS PROPERTIES (cont'd)

Compute transformed section properties based on prestress layout.

Transformed Section Properties

Initial Transformed Section (release):

$A_{ti} = 1140.4 \cdot in^2$

$I_{xti} = 196354 \cdot in^4$

$y_{tti} = 12.756 \cdot in$ $\quad S_{tti} = 15393 \cdot in^3$

$y_{cgpi} = -22.165 \cdot in$ $\quad S_{cgpi} = -8859 \cdot in^3$

$y_{bti} = -26.744 \cdot in$ $\quad S_{bti} = -7342 \cdot in^3$

Final Transformed Section (service):

$A_{tf} = 1136.7 \cdot in^2$

$I_{xtf} = 194576 \cdot in^4$

$y_{ttf} = 12.686 \cdot in$ $\quad S_{ttf} = 15338 \cdot in^3$

$y_{cgpf} = -22.235 \cdot in$ $\quad S_{cgpf} = -8751 \cdot in^3$

$y_{btf} = -26.814 \cdot in$ $\quad S_{btf} = -7257 \cdot in^3$

Determine initial prestress force after instantaneous loss due to elastic shortening. Use transformed properties to compute stress in the concrete at the level of prestress.

$P_i := f_{pi} \cdot A_{ps} = 1177.3 \cdot kip$ — Jacking force in prestress, prior to losses

$$f_{cgpi} := P_j \cdot \left(\frac{1}{A_{ti}} + \frac{y_{cgpi}}{S_{cgpi}} \right) + \frac{M_{gr}\left(\frac{L_g}{2}\right)}{S_{cgpi}} = 3.554 \cdot ksi$$

Stress in concrete at the level of prestress after instantaneous losses

$\Delta f_{pES} := n_{pi} \cdot f_{cgpi} = 20.886 \cdot ksi$ — Prestress loss due to elastic shortening (5.9.5.2.3a-1)

$f_{pi} := f_{pj} - \Delta f_{pES} = 181.614 \cdot ksi$ — Initial prestress after instantaneous losses

$P_i := f_{pi} \cdot A_{ps} = 1055.9 \cdot kip$ — Initial prestress force

Determine deflection of harped strands required to satisfy allowable stresses at the end of the beam at release.

$f_{c.all.rel} := 0.6 \cdot f_{ci} = 3.84 \cdot ksi$ — Allowable compression before losses (5.9.4.1.1)

$f_{t.all.rel} := \max\left(-0.0948 \cdot \sqrt{f_{ci} \cdot ksi}, -0.2 \cdot ksi\right) = -0.200 \cdot ksi$ — Allowable tension before losses (Table 5.9.4.1.2-1)

$L_t := 60 \cdot d_{ps} = 2.5 \cdot ft$ — Transfer length (AASHTO 5.11.4.1)

$$y_{cgp.t} := \left(\frac{f_{t.all.rel} - \frac{M_{gr}(L_t)}{S_{tti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{tti} = -17.176 \cdot in$$

Prestress eccentricity required for tension

$$y_{cgp.b} := \left(\frac{f_{c.all.rel} - \frac{M_{gr}(L_t)}{S_{bti}}}{P_i} - \frac{1}{A_{ti}} \right) \cdot S_{bti} = -21.025 \cdot in$$

Prestress eccentricity required for compression

9. PRESTRESS PROPERTIES (cont'd)

$y_{cgp.req} := \max(y_{cgp.t}, y_{cgp.b}) = -17.176 \cdot in$

Required prestress eccentricity at end of beam

$$y_{h.brg.req} := \frac{(y_{cgp.req} - y_{bti}) \cdot (N_s + N_h) - y_s \cdot N_s}{N_h} = 16.848 \cdot in$$

Minimum distance to harped prestress centroid from bottom of beam at centerline of bearing

$y_{top.min} := 18 \cdot in$

Minimum distance between uppermost strand and top of beam

$\alpha_{hd} := 0.4$

Hold-down point, fraction of the design span length

$$slope_{max} := \text{if}\left(d_{ps} = 0.6 \cdot in, \frac{1}{12}, \frac{1}{8}\right) = 0.125$$

Maximum slope of an individual strand to limit hold-down force to 4 kips/strand

$$y_{h.brg} := h - y_{top.min} - \left(\frac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) = 15.5 \cdot in$$

Set centroid of harped strands as high as possible to minimize release and handling stresses

$y_{h.brg} := \min(y_{h.brg}, y_{hb} + slope_{max} \cdot \alpha_{hd} \cdot L) = 15.5 \cdot in$

Verify that slope requirement is satisfied at uppermost strand

$CheckEndPrestress := \text{if}(y_{h.brg} \geq y_{h.brg.req}, \text{"OK"}, \text{"Verify release stresses."}) = \text{"Verify release stresses."}$

$$y_{p.brg} := \frac{N_s \cdot y_s + N_h \cdot y_{h.brg}}{N_s + N_h} = 9 \cdot in$$

Centroid of prestress from bottom at bearing

$$slope_{cgp} := \frac{y_{p.brg} - y_p}{\alpha_{hd} \cdot L} = 0.023$$

Slope of prestress centroid within the harping length

$$y_{px}(x) := \begin{cases} y_p + slope_{cgp} \cdot (L_{end} + \alpha_{hd} \cdot L - x) & \text{if } x \leq L_{end} + \alpha_{hd} \cdot L \\ y_p & \text{otherwise} \end{cases}$$

Distance to center of prestress from the bottom of the beam at any position

10. PRESTRESS LOSSES

Prestressed losses can be evaluated like regular concrete, in short-term and long-term losses. When the beam is a pretension girder there are instantaneous losses when the beam is shortened upon release of the prestress forces. Time-dependent losses happen when the beam is under creep and shrinkage of the beam concrete, creep and shrinkage c the deck concrete, and the relaxation of prestressed steel. These long term effects are separated into two stages that represent significant events in bridge construction. The first stage is the time between transfer of the prestress forces anc placement of the decked beam and the second is the period of time between placement of the deck and the final service load. For decked beams the computation of long-term losses is slightly simplified due to the cross-section not changing between the two stages and the shrinkage term of the deck concrete is eliminated since the deck and beam being cast together. No losses or gains in the steel associated with deck placement after transfer.

AASHTO methods for estimating time-dependent losses:
Approximate Estimate (5.9.5.3)
Refined Estimate (5.9.5.4)

The Approximate method is based on systems with composite decks and is based on the following assumptions: timing of load application, the cross-section in which the load is applied, and the ratio of dead and live loads to the total load. The conditions for the beams to be fabricated, formed and loaded depend on conditions assumed in the development of the approximate method. The refined method is used to estimate time-dependent losses in the prestressed steel.

Equations 5.9.5.4 are time-dependent and calculate the age-adjustment factors that effect losses using gross section properties.

$t_i := 1$ — Time (days) between casting and release of prestress

$t_b := 20$ — Time (days) to barrier casting (exterior girder only)

$t_d := 30$ — Time (days) to erection of precast section, closure joint pour

$t_f := 20000$ — Time (days) to end of service life

Terms and equations used in the loss calculations:

$K_L := 45$ — Prestressing steel factor for low-relaxation strands (C5.9.5.4.2c)

$VS := \dfrac{A_g}{Peri} = 3.911 \cdot in$ — Volume-to-surface ratio of the precast section

$k_s := \max\left(1.45 - 0.13 \cdot \dfrac{VS}{in}, 1.0\right) = 1.00$ — Factor for volume-to-surface ratio (5.4.2.3.2-2)

$k_{hc} := 1.56 - 0.008 \cdot H = 1.00$ — Humidity factor for creep (5.4.2.3.2-3)

$k_{hs} := 2.00 - 0.014 \cdot H = 1.02$ — Humidity factor for shrinkage (5.4.2.3.3-2)

$k_f := \dfrac{5}{1 + \dfrac{f_{ci}}{ksi}} = 0.676$ — Factor for effect of concrete strength (5.4.2.3.2-4)

10. PRESTRESS LOSSES (cont'd)

$$k_{td}(t) := \frac{t}{61 - 4\cdot\frac{f_{ci}}{ksi} + t}$$

Time development factor (5.4.2.3.2-5)

$$\psi\left(t, t_{init}\right) := 1.9\cdot k_s\cdot k_{hc}\cdot k_f\cdot k_{td}(t)\cdot\left(t_{init}\right)^{-0.118}$$

Creep coefficient (5.4.2.3.2-1)

$$\varepsilon_{sh}(t) := k_s\cdot k_{hs}\cdot k_f\cdot k_{td}(t)\cdot\left(0.48\cdot 10^{-3}\right)$$

Concrete shrinkage strain (5.4.2.3.3-1)

Time from Transfer to Erection:

$$e_{pg} := -\left(y_p + y_{bg}\right) = 22.73\cdot in$$

Eccentricity of prestress force with respect to the neutral axis of the gross non-composite beam, positive below the beam neutral axis

$$f_{cgp} := P_i\cdot\left(\frac{1}{A_g} + \frac{e_{pg}^{\ 2}}{I_{xg}}\right) + \frac{M_g\left(\frac{L}{2}\right)}{I_{xg}}\cdot\left(y_p + y_{bg}\right) = 3.558\cdot ksi$$

Stress in the concrete at the center prestress immediately after transfer

$$f_{pt} := \max\left(f_{pi}, 0.55\cdot f_{py}\right) = 181.614\cdot ksi$$

Stress in strands immediately after transfer (5.9.5.4.2c-1)

$$\psi_{bid} := \psi\left(t_d, t_i\right) = 0.589$$

Creep coefficient at erection due to loading at transfer

$$\psi_{bif} := \psi\left(t_f, t_i\right) = 1.282$$

Creep coefficient at final due to loading at transfer

$$\varepsilon_{bid} := \varepsilon_{sh}\left(t_d - t_i\right) = 1.490 \times 10^{-4}$$

Concrete shrinkage between transfer and erection

$$K_{id} := \frac{1}{1 + n_{pi}\cdot\frac{A_{ps}}{A_g}\cdot\left(1 + \frac{A_g\cdot e_{pg}^{\ 2}}{I_{xg}}\right)\cdot\left(1 + 0.7\cdot\psi_{bif}\right)} = 0.805$$

Age-adjusted transformed section coefficient (5.9.5.4.2a-2)

$$\Delta f_{pSR} := \varepsilon_{bid}\cdot E_p\cdot K_{id} = 3.418\cdot ksi$$

Loss due to beam shrinkage (5.9.5.4.2a-1)

$$\Delta f_{pCR} := n_{pi}\cdot f_{cgp}\cdot\psi_{bid}\cdot K_{id} = 9.913\cdot ksi$$

Loss due to creep (5.9.5.4.2b-1)

$$\Delta f_{pR1} := \left[\frac{f_{pt}}{K_L}\cdot\frac{\log\left(24\cdot t_d\right)}{\log\left(24\cdot t_i\right)}\cdot\left(\frac{f_{pt}}{f_{py}} - 0.55\right)\right]\cdot\left[1 - \frac{3\cdot\left(\Delta f_{pSR} + \Delta f_{pCR}\right)}{f_{pt}}\right]\cdot K_{id} = 1.035\cdot ksi$$

Loss due to relaxation (C5.9.5.4.2c-1

$$\Delta f_{pid} := \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1} = 14.366\cdot ksi$$

10. PRESTRESS LOSSES (cont'd)

Time from Erection to Final:

$e_{pc} := e_{pg} = 22.73 \cdot in$ — Eccentricity of prestress force does not change

$A_c := A_g \qquad I_c := I_{xg}$ — Section properties remain unchanged

$$\Delta f_{cd} := \frac{M_{fws}\left(\frac{L}{2}\right) + M_j\left(\frac{L}{2}\right)}{S_{cgpf}} + \frac{\Delta f_{pid}}{n_p} = 2.665 \cdot ksi$$

Change in concrete stress at center of prestress due to initial time-dependent losses and superimposed dead load. Deck weight is not included for this design.

$\psi_{bdf} := \psi(t_f, t_d) = 0.858$ — Creep coefficient at final due to loading at erection

$\varepsilon_{bif} := \varepsilon_{sh}(t_f - t_i) = 3.302 \times 10^{-4}$ — Concrete shrinkage between transfer and final

$\varepsilon_{bdf} := \varepsilon_{bif} - \varepsilon_{bid} = 1.813 \times 10^{-4}$ — Concrete shrinkage between erection and final

$$K_{df} := \frac{1}{1 + n_{pi} \cdot \frac{A_{ps}}{A_c} \cdot \left(1 + \frac{A_c \cdot e_{pc}^{\,2}}{I_c}\right) \cdot \left(1 + 0.7 \cdot \psi_{bif}\right)} = 0.805$$

Age-adjusted transformed section coefficient remains unchanged

$\Delta f_{pSD} := \varepsilon_{bdf} \cdot E_p \cdot K_{df} = 4.159 \cdot ksi$ — Loss due to beam shrinkage

$\Delta f_{pCD} := n_{pi} \cdot f_{cgp} \cdot (\psi_{bif} - \psi_{bid}) \cdot K_{df} + n_p \cdot \Delta f_{cd} \cdot \psi_{bdf} \cdot K_{df} = 21.331 \cdot ksi$ — Loss due to creep

$\Delta f_{pR2} := \Delta f_{pR1} = 1.035 \cdot ksi$ — Loss due to relaxation

$\Delta f_{pSS} := 0$ — Loss due to deck shrinkage

$\Delta f_{pdf} := \Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} + \Delta f_{pSS} = 26.525 \cdot ksi$

Prestress Loss Summary

$\Delta f_{pES} = 20.886 \cdot ksi \qquad \frac{\Delta f_{pES}}{f_{pj}} = 10.3 \cdot \%$

$\Delta f_{pLT} := \Delta f_{pid} + \Delta f_{pdf} = 40.891 \cdot ksi \qquad \frac{\Delta f_{pLT}}{f_{pj}} = 20.2 \cdot \%$

$\Delta f_{pTotal} := \Delta f_{pES} + \Delta f_{pLT} = 61.777 \cdot ksi \qquad \frac{\Delta f_{pTotal}}{f_{pj}} = 30.5 \cdot \% \qquad \Delta f_{p.est} = 25 \cdot \%$

$f_{pe} := f_{pj} - \Delta f_{pTotal} = 140.7 \cdot ksi$ — Final effective prestress

$\text{CheckFinalPrestress} := \text{if}\left(f_{pe} \le f_{pe.max}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$

11. CONCRETE STRESSES

Concrete Stresses at release, during handling and at final service are computed and compared to approximated values for each stage.

Concrete Stresses at Release

When calculating the stresses at release use the overall beam length due to the beam being supported at each end in the casting bed after prestress forces are transformed.

Define locations for which stresses are to be calculated:

$$x_r := L_g \cdot \left(0 \;\; \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \;\; \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \;\; 0.1 \;\; 0.2 \;\; 0.3 \;\; \alpha_{hd} \;\; 0.5 \right)^T \qquad ir := 1 .. \mathrm{last}(x_r)$$

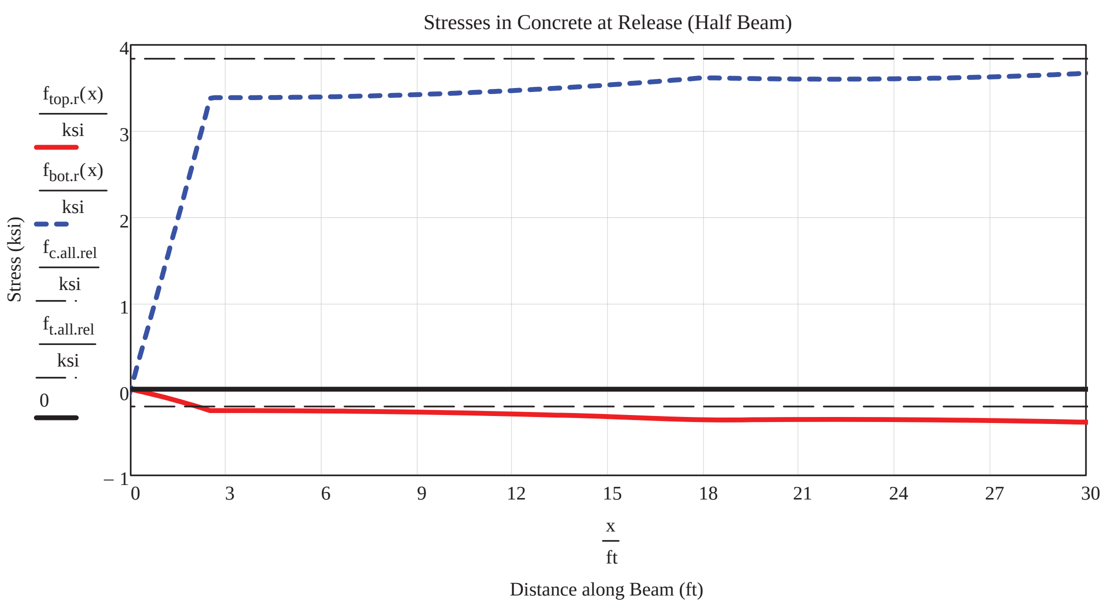
Functions for computing beam stresses:

$$f_{top.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{tti}}\right) + \frac{M_{gr}(x)}{S_{tti}}$$

Top fiber stress at release

$$f_{bot.r}(x) := \min\left(\frac{x}{L_t}, 1\right) \cdot P_i \cdot \left(\frac{1}{A_{ti}} + \frac{y_{bti} + y_{px}(x)}{S_{bti}}\right) + \frac{M_{gr}(x)}{S_{bti}}$$

Bottom fiber stress at release



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$f_{t.all.rel} = -0.2 \cdot ksi$ Allowable tension at release

$f_{c.all.rel} = 3.84 \cdot ksi$ Allowable compression at release

$TopRel_{ir} := f_{top.r}\left(x_{r_{ir}}\right)$ $TopRel^T = (0.000 \;\; -0.191 \;\; -0.248 \;\; -0.249 \;\; -0.264 \;\; -0.299 \;\; -0.353 \;\; -0.351) \cdot ksi$

$CheckTopRel := if\left[\left(\max(TopRel) \le f_{c.all.rel}\right) \cdot \left(\min(TopRel) \ge f_{t.all.rel}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"No Good"}$

$BotRel_{ir} := f_{bot.r}\left(x_{r_{ir}}\right)$ $BotRel^T = (0.000 \;\; 2.693 \;\; 3.388 \;\; 3.389 \;\; 3.421 \;\; 3.493 \;\; 3.607 \;\; 3.602) \cdot ksi$

$CheckBotRel := if\left[\left(\max(BotRel) \le f_{c.all.rel}\right) \cdot \left(\min(BotRel) \ge f_{t.all.rel}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

Concrete Stresses During Lifting and Transportation

Lifting and transportation stresses can govern over final stresses due to different support locations, dynamic effects that dead load can cause during movement, bending stresses during lifting and superelevation of the roadway in shipping. End diaphragms on both ends are assumed. For prestressing effects, calculate the effective prestressed force losses between transfer and building.

$a := h = 3.375 \cdot ft$ Maximum distance to lift point from bearing line

$a' := a + L_{end} = 5.375 \cdot ft$ Distance to lift point from end of beam

$P_{dia} := \max\left(W_{ia}, W_{sa}\right) = 12.9 \cdot kip$ Approximate abutment weight

$P_m := P_j \cdot \left[1 - \frac{\left(\Delta f_{pES} + \Delta f_{pid}\right)}{f_{pj}}\right] = 972.4 \cdot kip$ Effective prestress during lifting and shipping

Define locations for which stresses are to be calculated:

$$x_e := L_g \cdot \left(0 \quad \min\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \max\left(\frac{L_t}{L_g}, \frac{L_{end}}{L_g}\right) \quad \frac{a'}{L_g} \quad \alpha_{hd} \quad 0.5\right)^T \qquad ie := 1 .. last\left(x_e\right)$$

Compute moment in the girder during lifting with supports at the lift points.

$$M_{lift}(x) := \begin{cases} -\left[\frac{\left(w_g + w_{bar}\right) \cdot x^2}{2} + P_{dia} \cdot x\right] & \text{if } x \le a' \\ M_{gr}(x) - \left[M_{gr}(a') + \frac{\left(w_g + w_{bar}\right) \cdot (a')^2}{2} + P_{dia} \cdot a'\right] & \text{otherwise} \end{cases}$$

11. CONCRETE STRESSES (cont'd)

Functions for computing beam stresses:

$$f_{top.lift}(x) := \min\left(\frac{x}{L_t}, 1\right)\cdot P_m\cdot\left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}$$

Top fiber stress during lifting

$$f_{top.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right)\cdot P_m\cdot\left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}\cdot(1 + DIM)$$

Top fiber stress during lifting, impact increasing dead load

$$f_{top.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right)\cdot P_m\cdot\left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_{lift}(x)}{S_{ttf}}\cdot(1 - DIM)$$

Top fiber stress during lifting, impact decreasing dead load

$TopLift1_{ie} := f_{top.lift}\left(x_{e_{ie}}\right)$ $\qquad TopLift1^T = (0.000 \ \ -0.242 \ \ -0.312 \ \ -0.407 \ \ -0.491 \ \ -0.488)\cdot ksi$

$TopLift2_{ie} := f_{top.DIM.inc}\left(x_{e_{ie}}\right)$ $\qquad TopLift2^T = (0.000 \ \ -0.249 \ \ -0.321 \ \ -0.429 \ \ -0.474 \ \ -0.469)\cdot ksi$

$TopLift3_{ie} := f_{top.DIM.dec}\left(x_{e_{ie}}\right)$ $\qquad TopLift3^T = (0.000 \ \ -0.235 \ \ -0.303 \ \ -0.385 \ \ -0.508 \ \ -0.508)\cdot ksi$

$$f_{bot.lift}(x) := \min\left(\frac{x}{L_t}, 1\right)\cdot P_m\cdot\left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}$$

Bottom fiber stress during lifting

$$f_{bot.DIM.inc}(x) := \min\left(\frac{x}{L_t}, 1\right)\cdot P_m\cdot\left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}\cdot(1 + DIM)$$

Bottom fiber stress during lifting, impact increasing dead load

$$f_{bot.DIM.dec}(x) := \min\left(\frac{x}{L_t}, 1\right)\cdot P_m\cdot\left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_{lift}(x)}{S_{btf}}\cdot(1 - DIM)$$

Bottom fiber stress during lifting, impact decreasing dead load

$BotLift1_{ie} := f_{bot.lift}\left(x_{e_{ie}}\right)$ $\qquad BotLift1^T = (0.000 \ \ 2.643 \ \ 3.323 \ \ 3.524 \ \ 3.702 \ \ 3.696)\cdot ksi$

$BotLift2_{ie} := f_{bot.DIM.inc}\left(x_{e_{ie}}\right)$ $\qquad BotLift2^T = (0.000 \ \ 2.657 \ \ 3.342 \ \ 3.571 \ \ 3.666 \ \ 3.654)\cdot ksi$

$BotLift3_{ie} := f_{bot.DIM.dec}\left(x_{e_{ie}}\right)$ $\qquad BotLift3^T = (0.000 \ \ 2.628 \ \ 3.305 \ \ 3.477 \ \ 3.737 \ \ 3.737)\cdot ksi$

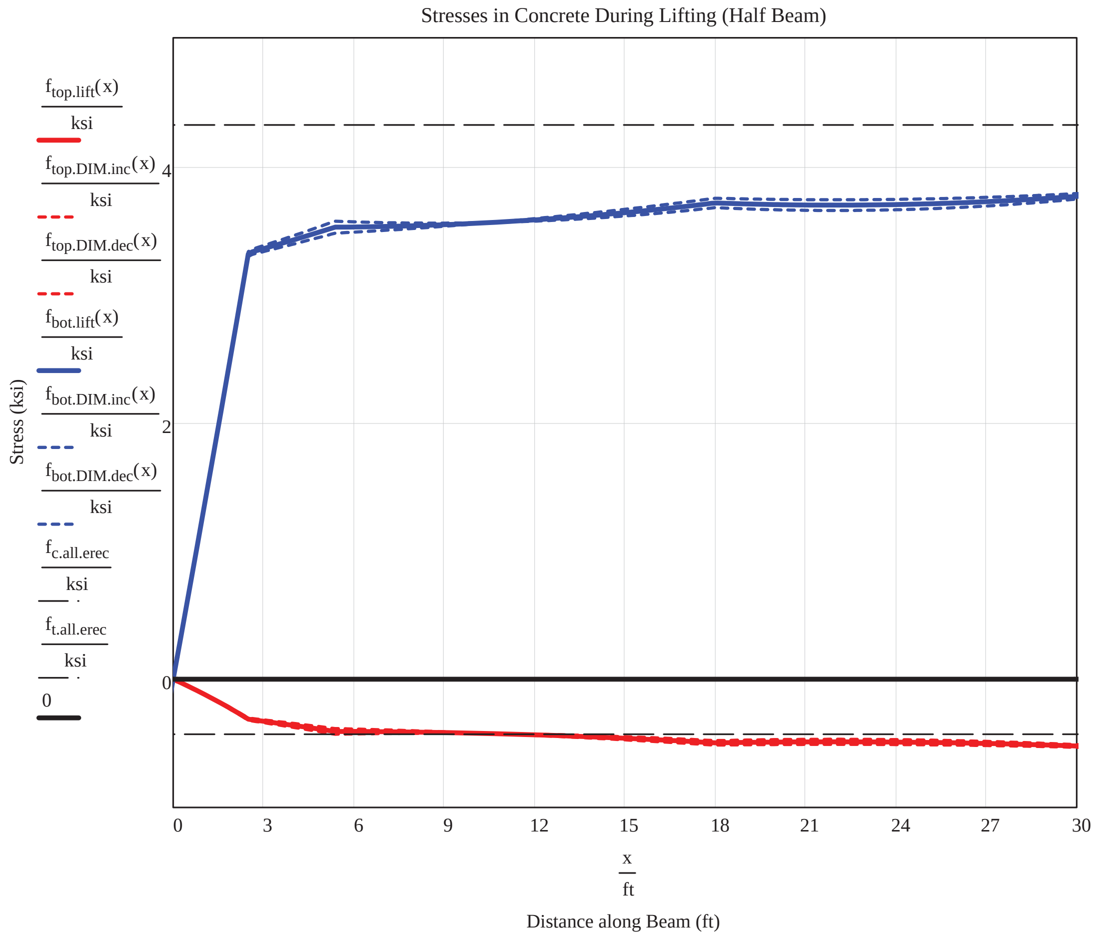
Allowable stresses during handling:

$f_{cm} := f_{c.erec}(f_c) = 7.2\cdot ksi$ — Assumed concrete strength when handling operations begin

$f_{c.all.erec} := 0.6\cdot f_{cm} = 4.32\cdot ksi$ — Allowable compression during lifting and shipping

$f_{t.all.erec} := f_{t.erec}(f_{cm}) = -0.429\cdot ksi$ — Allowable tension during lifting and shipping

11. **CONCRETE STRESSES (cont'd)**



Compare beam stresses to allowable stresses.

$TopLiftMax_{ie} := max(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie})$ $\quad TopLiftMax^T = (0\ \ -0.235\ \ -0.303\ \ -0.385\ \ -0.474\ \ -0.469)\cdot ks$

$TopLiftMin_{ie} := min(TopLift1_{ie}, TopLift2_{ie}, TopLift3_{ie})$ $\quad TopLiftMin^T = (0\ \ -0.249\ \ -0.321\ \ -0.429\ \ -0.508\ \ -0.508)\cdot ks$

$CheckTopLift := if\left[(max(TopLiftMax) \le f_{c.all.erec})\cdot(min(TopLiftMin) \ge f_{t.all.erec}), \text{"OK"}, \text{"No Good"}\right] = \text{"No Goc}$

$BotLiftMax_{ie} := max(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie})$ $\quad BotLiftMax^T = (0\ \ 2.657\ \ 3.342\ \ 3.571\ \ 3.737\ \ 3.737)\cdot ksi$

$BotLiftMin_{ie} := min(BotLift1_{ie}, BotLift2_{ie}, BotLift3_{ie})$ $\quad BotLiftMin^T = (0\ \ 2.628\ \ 3.305\ \ 3.477\ \ 3.666\ \ 3.654)\cdot ksi$

$CheckBotLift := if\left[(max(BotLiftMax) \le f_{c.all.erec})\cdot(min(BotLiftMin) \ge f_{t.all.erec}), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$

11. CONCRETE STRESSES (cont'd)

Concrete Stresses at Final

Stresses are calculated using design span length. The top flange compression and bottom flange under tension are computed at Service I and Service III states.

$f_{c.all.ser1} := 0.4 \cdot f_c = 3.2 \cdot ksi$ — Allowable compression due to effective prestress and dead load (Table 5.9.4.2.1-1)

$f_{c.all.ser2} := 0.6 \cdot f_c = 4.8 \cdot ksi$ — Allowable compression due to effective prestress, permanent load, and transient loads, as well as stresses during shipping and handling (Table 5.9.4.2.1-1)

$f_{t.all.ser} = 0 \cdot ksi$ — Allowable tension (computed previously)

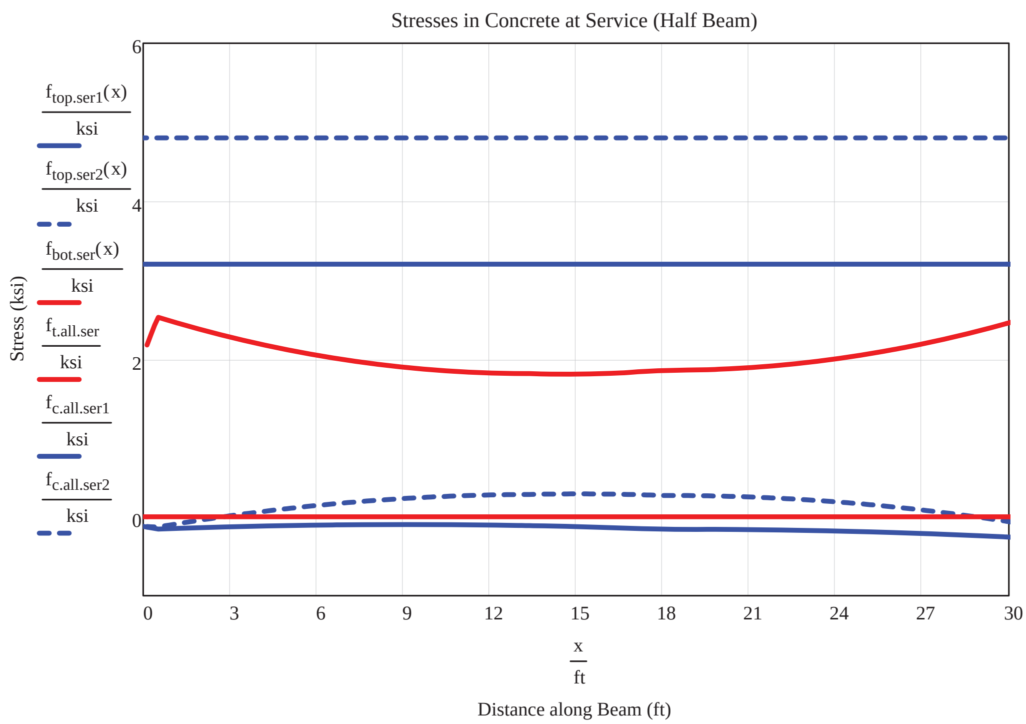
$P_e := f_{pe} \cdot A_{ps} = 818.2 \cdot kip$ — Effective prestress after all losses

Compute stresses at midspan and compare to allowable values.

$$f_{top.ser1}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g(x + L_{end})}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x)}{S_{ttf}}$$

$$f_{top.ser2}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{ttf}}\right) + \frac{M_g(x + L_{end})}{S_{tti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + M_{ll}(x)}{S_{ttf}}$$

$$f_{bot.ser}(x) := \min\left(\frac{L_{end} + x}{L_t}, 1\right) \cdot P_e \cdot \left(\frac{1}{A_{tf}} + \frac{y_{btf} + y_{px}(x)}{S_{btf}}\right) + \frac{M_g(x + L_{end})}{S_{bti}} + \frac{M_{bar}(x) + M_{fws}(x) + M_j(x) + 0.8 \cdot M_{ll}(x)}{S_{btf}}$$



11. CONCRETE STRESSES (cont'd)

Compare beam stresses to allowable stresses.

$$x_s := L\cdot\left(\frac{L_t}{L}\ \ 0.1\ \ 0.15\ \ 0.2\ \ 0.25\ \ 0.3\ \ 0.35\ \ \alpha_{hd}\ \ 0.45\ \ 0.5\right)^T \qquad is := 1..\text{last}(x_s)$$

$$TopSer1_{is} := f_{top.ser1}\left(x_{s_{is}}\right) \quad TopSer1^T = (-0.132\ \ -0.119\ \ -0.106\ \ -0.100\ \ -0.099\ \ -0.105\ \ -0.117\ \ -0.135\ \ -0.159\ \ -0$$

$$TopSer2_{is} := f_{top.ser2}\left(x_{s_{is}}\right) \quad TopSer2^T = (-0.015\ \ 0.060\ \ 0.142\ \ 0.206\ \ 0.250\ \ 0.276\ \ 0.287\ \ 0.288\ \ 0.270\ \ 0.262)\cdot ksi$$

$$CheckCompSerI := \text{if}\left[\left(\max(TopSer1) \le f_{c.all.ser1}\right)\cdot\left(\max(TopSer2) \le f_{c.all.ser2}\right), \text{"OK"}, \text{"No Good"}\right] = \text{"OK"}$$

$$BotSer_{is} := f_{bot.ser}\left(x_{s_{is}}\right) \qquad BotSer^T = (2.324\ \ 2.192\ \ 2.048\ \ 1.938\ \ 1.862\ \ 1.822\ \ 1.809\ \ 1.815\ \ 1.856\ \ 1.870)\cdot ksi$$

$$CheckTenSerIII := \text{if}\left(\min(BotSer) \ge f_{t.all.ser}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

12. FLEXURAL STRENGTH

Confirm flexural resistance at Strength Limit State. Calculate Factored moment at midspan during Strength I load combination. Compare this to factored resistance in AASHTO LRFD 5.7.3.

$M_{DC}(x) := M_g(x) + M_{bar}(x) + M_j(x)$ — Self weight of components

$M_{DW}(x) := M_{fws}(x)$ — Weight of future wearing surface

$M_{LL}(x) := M_{ll}(x)$ — Live load

$M_{StrI}(x) := 1.25\cdot M_{DC}(x) + 1.5\cdot M_{DW}(x) + 1.75\cdot M_{LL}(x)$ — Factored design moment

For minimum reinforcement check, per 5.7.3.3.2

$f_{cpe} := P_e\cdot\left(\frac{1}{A_g} + \frac{y_{cgp}}{S_{bg}}\right) = 3.525\cdot ksi$ — Concrete compression at extreme fiber due to effective prestress

$M_{cr} := -\left(f_{r.cm} + f_{cpe}\right)\cdot S_{bg} = 2540\cdot kip\cdot ft$ — Cracking moment (5.7.3.3.2-1)

$M_u(x) := \max\left(M_{StrI}(x), \min\left(1.33\cdot M_{StrI}(x), 1.2\cdot M_{cr}\right)\right)$ — Design moment

12. FLEXURAL STRENGTH (cont'd)

Compute factored flexural resistance.

$$\beta_1 := \max\left[0.65, 0.85 - 0.05\cdot\left(\frac{f_c}{ksi} - 4\right)\right] = 0.65$$ Stress block factor (5.7.2.2)

$$k := 2\cdot\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = 0.28$$ Tendon type factor (5.7.3.1.1-2)

$$d_p(x) := h - y_{px}(x + L_{end}) \qquad d_p(X) = 35.921\cdot in$$ Distance from compression fiber to prestress centroid

$$h_f := d_7 = 8\cdot in$$ Structural flange thickness

$$b_{taper} := \frac{b_6 - b_5}{2} = 16\cdot in$$ Average width of taper at bottom of flange

$$h_{taper} := d_5 = 2\cdot in$$ Depth of taper at bottom of flange

$$a(x) := \frac{A_{ps}\cdot f_{pu}}{0.85\cdot f_c\cdot b_f + \frac{k}{\beta_1}\cdot A_{ps}\cdot\left(\frac{f_{pu}}{d_p(x)}\right)} \qquad a(X) = 2.509\cdot in$$ Depth of equivalent stress block for rectangular section

$$c(x) := \frac{a(x)}{\beta_1} \qquad c(X) = 3.86\cdot in$$ Neutral axis location

$$CheckTC := if\left[\frac{c(X)}{d_p(X)} \le \left(\frac{.003}{.003 + .005}\right), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$$ Tension-controlled section check (midspan)

$$\varphi_f := \min\left[1.0, \max\left[0.75, 0.583 + 0.25\cdot\left(\frac{d_p(X)}{c(X)} - 1\right)\right]\right] = 1.00$$ Resistance factor for prestressed concrete (5.5.4.2)

$$f_{ps} := f_{pu}\cdot\left(1 - k\cdot\frac{c(X)}{d_p(X)}\right) = 261.9\cdot ksi$$ Average stress in the prestressing steel (5.7.3.1.1-1)

$$L_d := \frac{1.6}{ksi}\cdot\left(f_{ps} - \frac{2}{3}\cdot f_{pe}\right)\cdot d_{ps} = 11.204\cdot ft$$ Bonded strand devlepment length (5.11.4.2-1)

$$f_{px}(x) := \begin{cases} \frac{f_{pe}\cdot(x + L_{end})}{L_t} & \text{if } x \le L_t - L_{end} \\ f_{pe} + \frac{(x + L_{end}) - L_t}{L_d - L_t}\cdot(f_{ps} - f_{pe}) & \text{if } L_t - L_{end} < x \le L_d - L_{end} \\ f_{ps} & \text{otherwise} \end{cases}$$ Stress in prestressing steel along the length for bonded strand (5.11.4.2)

$$M_r(x) := \varphi_f\cdot\left[A_{ps}\cdot f_{px}(x)\cdot\left(d_p(x) - \frac{a(x)}{2}\right)\right]$$ Flexure resistance along the length

12. **FLEXURAL STRENGTH (cont'd)**

$$x_{mom} := L \cdot \left(0.01 \quad \frac{L_t - L_{end}}{L} \quad \frac{L_d - L_{end}}{L} \quad \alpha_{hd} \quad 0.5 \right)^T \qquad imom := 1 .. \mathrm{last}(x_{mom})$$

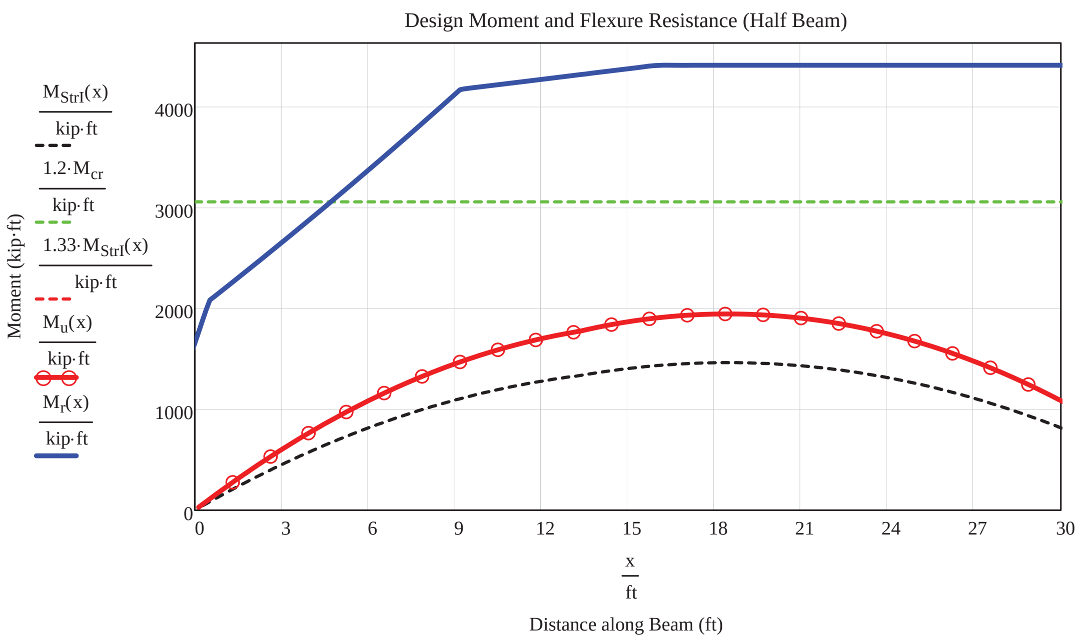
$$M_{rx_{imom}} := M_r\left(x_{mom_{imom}}\right) \qquad M_{ux_{imom}} := M_u\left(x_{mom_{imom}}\right)$$

$$DC_{mom} := \frac{M_{ux}}{M_{rx}} \qquad \max(DC_{mom}) = 0.438$$

Demand-Capacity ratio for moment

$$CheckMom := \mathrm{if}\left(\max(DC_{mom}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$$

Flexure resistance check



13. SHEAR STRENGTH

Shear Resistance

Use Strength I load combination to calculate factored shear at the critical shear section and at tenth points along the span. Compare it to factored resistance in AASHTO LRFD 5.8.

$V_{DC}(x) := V_g(x) + V_{bar}(x) + V_j(x)$ — Self weight of components

$V_{DW}(x) := V_{fws}(x)$ — Weight of future wearing surface

$V_{LL}(x) := V_{ll}(x)$ — Live load

$V_u(x) := 1.25 \cdot V_{DC}(x) + 1.5 \cdot V_{DW}(x) + 1.75 \cdot V_{LL}(x)$ — Factored design shear

$\varphi_v := 0.90$ — Resistance factor for shear in normal weight concrete (AASHTO LRFD 5.5.4.2)

$d_{end} := h - y_{px}(L_{end}) = 31.5 \cdot in$ — Depth to steel centroid at bearing

$d_v := \min(0.9 \cdot d_{end}, 0.72 \cdot h) = 28.35 \cdot in$ — Effective shear depth lower limit at end

$$V_p(x) := \begin{cases} P_e \cdot slope_{cgp} \cdot \dfrac{x + L_{end}}{L_t} & \text{if } x \le L_t - L_{end} \\ P_e \cdot slope_{cgp} & \text{if } L_t - L_{end} < x \le \alpha_{hd} \cdot L \\ 0 & \text{otherwise} \end{cases}$$

Vertical component of effective prestress force

$b_v := b_3 = 6 \cdot in$ — Web thickness

$$v_u(x) := \frac{\left|V_u(x) - \varphi_v \cdot V_p(x)\right|}{\varphi_v \cdot b_v \cdot d_v}$$

Shear stress on concrete (5.8.2.9-1)

$M_{ushr}(x) := \max\left(M_{StrI}(x), \left|V_u(x) - V_p(x)\right| \cdot d_v\right)$ — Factored moment for shear

$f_{po} := 0.7 \cdot f_{pu} = 189 \cdot ksi$ — Stress in prestressing steel due to locked-in strain after casting concrete

$$\varepsilon_s(x) := \max\left(-0.4 \cdot 10^{-3}, \frac{\dfrac{\left|M_u(x)\right|}{d_v} + \left|V_u(x) - V_p(x)\right| - A_{ps} \cdot f_{po}}{E_p \cdot A_{ps}}\right)$$

Steel strain at the centroid of the prestressing steel

$$\beta(x) := \frac{4.8}{1 + 750 \cdot \varepsilon_s(x)}$$

Shear resistance parameter

$\theta(x) := (29 + 3500 \cdot \varepsilon_s(x)) \cdot deg$ — Principal compressive stress angle

$$V_c(x) := 0.0316 \cdot ksi \cdot \beta(x) \cdot \sqrt{\frac{f_c}{ksi}} \cdot b_v \cdot d_v$$

Concrete contribution to total shear resistance

13. SHEAR STRENGTH (cont'd)

$\alpha := 90 \cdot deg$ — Angle of inclination of transverse reinforcement

$A_v := (1.02 \;\; 0.62 \;\; 0.62 \;\; 0.62 \;\; 0.31)^T \cdot in^2$ $\quad s_v := (3 \;\; 6 \;\; 6 \;\; 12 \;\; 12)^T \cdot in$ — Transverse reinforcement area and spacing provided

$x_v := (0 \;\; 0.25 \cdot h \;\; 1.5 \cdot h \;\; 0.3 \cdot L \;\; 0.5 \cdot L \;\; 0.6 \cdot L)^T$ $\quad x_v^T = (0 \;\; 0.844 \;\; 5.063 \;\; 12 \;\; 20 \;\; 24) \cdot ft$

$$A_{vs}(x) := \left| \begin{array}{l} \text{for } i \in 1..\text{last}(A_v) \\ \quad out \leftarrow \dfrac{A_{v_i}}{s_{v_i}} \text{ if } x_{v_i} \le x \le x_{v_{i+1}} \\ out \end{array} \right.$$

.

$V_s(x) := A_{vs}(x) \cdot f_y \cdot d_v \cdot (\cot(\theta(x)) + \cot(\alpha)) \cdot \sin(\alpha)$ — Steel contribution to total shear resistance

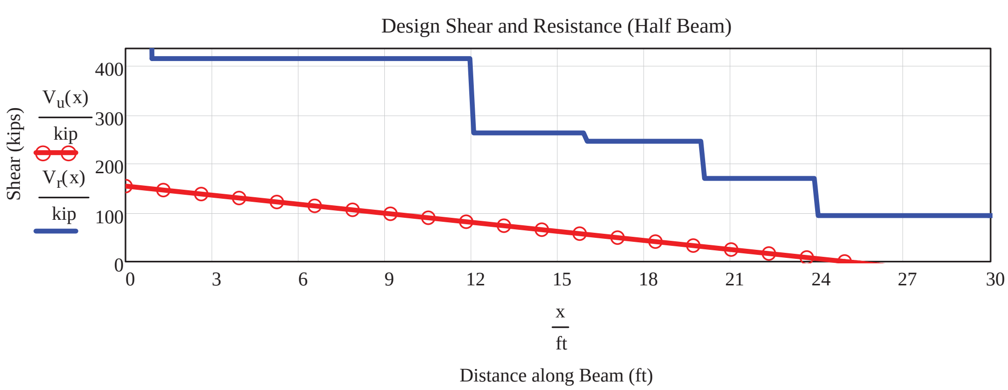
$V_r(x) := \varphi_v \cdot (V_c(x) + V_s(x) + V_p(x))$ — Factored shear resistance

$$x_{shr} := \left| \begin{array}{l} \text{for } i \in 1..100 \\ \quad out_i \leftarrow i \cdot \dfrac{0.5 \cdot L}{100} \\ out \end{array} \right. \qquad ishr := 1..\text{last}(x_{shr})$$

$V_{ux_{ishr}} := V_u\left(x_{shr_{ishr}}\right) \qquad V_{rx_{ishr}} := V_r\left(x_{shr_{ishr}}\right)$

$DC_{shr} := \dfrac{V_{ux}}{V_{rx}} \qquad \max(DC_{shr}) = 0.357$

$CheckShear := \text{if}\left(\max(DC_{shr}) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Shear resistance check



13. SHEAR STRENGTH (cont'd)

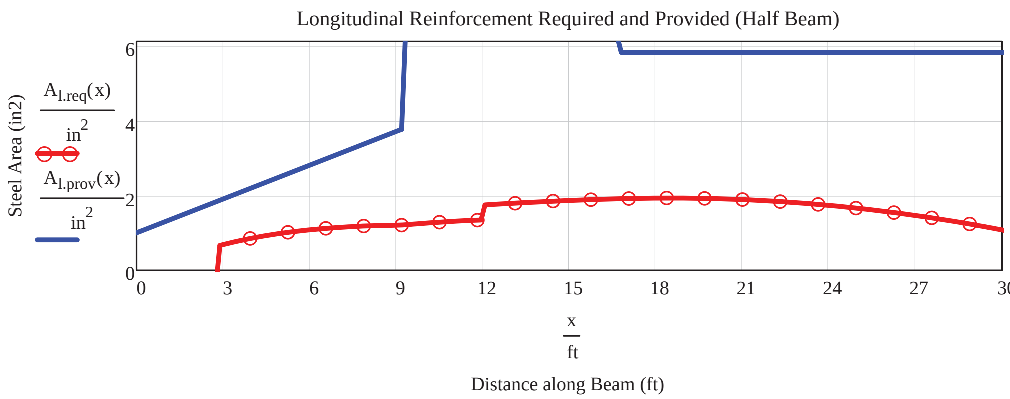
Longitudinal Reinforcement

$$A_{l.req}(x) := \begin{array}{|l} a1 \leftarrow \dfrac{M_{StrI}(x)}{\varphi_f \cdot f_{px}(x) \cdot \left(d_p(x) - \dfrac{a(x)}{2}\right)} \\ a2 \leftarrow \dfrac{\left(\dfrac{V_u(x)}{\varphi_v} - 0.5 \cdot V_s(x) - V_p(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ a3 \leftarrow \dfrac{\dfrac{M_{ushr}(x)}{d_v \cdot \varphi_f} + \left(\left|\dfrac{V_u(x)}{\varphi_v} - V_p(x)\right| - 0.5 \cdot V_s(x)\right) \cdot \cot(\theta(x))}{f_{px}(x)} \\ \min(a1, a2) \quad \text{if } x \le d_v + 5 \cdot in \\ \min(a1, a3) \quad \text{otherwise} \end{array}$$

Longitudinal reinforcement required for shear (5.8.3.5)

$A_{s.add} := 0.40 \cdot in^2$ $\quad L_{d.add} := 18.67 \cdot ft$ $\quad$ Additional longitudinal steel and developed length from end of beam

$$A_{l.prov}(x) := \text{if}\left(x < L_{d.add} - L_{end}, A_{s.add}, 0\right) + \begin{array}{|l} A_p \cdot N_s \cdot \dfrac{x + L_{end}}{L_d} \quad \text{if } x \le L_d - L_{end} \\ A_p \cdot N_s \quad \text{if } L_d - L_{end} < x \le \dfrac{y_{h.brg} - 0.5 \cdot h}{slope_{cgp}} + \left(\dfrac{0.5 \cdot N_h - 1}{2}\right) \cdot (2 \cdot in) \cdot \cot\left(slope_{cgp}\right) \\ A_p \cdot \left(N_h + N_s\right) \quad \text{otherwise} \end{array}$$



$A_{l.req_{ishr}} := A_{l.req}\left(x_{shr_{ishr}}\right)$ $\qquad A_{l.prov_{ishr}} := A_{l.prov}\left(x_{shr_{ishr}}\right)$

$DC_{long} := \dfrac{A_{l.req}}{A_{l.prov}}$ $\qquad \max\left(DC_{long}\right) = 0.395$

$CheckLong := \text{if}\left(\max\left(DC_{long}\right) \le 1.0, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ $\qquad$ Longitudinal reinforcement check

14. SPLITTING RESISTANCE

Splitting Resistance

Checking splitting by zone of transverse reinforcement. Defined in Shear Strength section.

$$A_s := \frac{A_{v_1} \cdot x_{v_2}}{s_{v_1}} = 3.443 \cdot in^2$$

$f_s := 20 \cdot ksi$ — Limiting stress in steel for crack control (5.10.10.1)

$P_r := f_s \cdot A_s = 68.9 \cdot kip$ — Splitting resistance provided (5.10.10.1-1)

$P_{r.min} := 0.04 \cdot P_j = 47.1 \cdot kip$ — Minimum splitting resistance required

$CheckSplit := if\left(P_r \geq P_{r.min}, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$ — Splitting resistance check

15. CAMBER AND DEFLECTIONS

Calculate Deflections due to different weights, joints, and future wearings.

$$\Delta_{ps} := \frac{-P_i}{E_{ci} \cdot I_{xg}} \cdot \left[\frac{y_{cgp} \cdot L_g^{\,2}}{8} - \frac{\left(y_{bg} + y_{p.brg}\right) \cdot \left(\alpha_{hd} \cdot L + L_{end}\right)^2}{6}\right] = 0.777 \cdot in$$

Deflection due to prestress at release

$$\Delta_{gr} := \frac{-5}{384} \cdot \frac{w_g \cdot L_g^{\,4}}{E_{ci} \cdot I_{xg}} = -0.123 \cdot in$$

Deflection due to self-weight at release

$$\Delta_{bar} := \frac{-5}{384} \cdot \frac{w_{bar} \cdot L_g^{\,4}}{E_c \cdot I_{xg}} = -0.037 \cdot in$$

Deflection due to barrier weight

$$\Delta_j := \frac{-5}{384} \cdot \frac{w_j \cdot L^4}{E_c \cdot I_{xg}} \cdot if\,(BeamLoc = 0, 1, 0.5) = -0.002 \cdot in \qquad 2$$

Deflection due to longitudinal joint

$$\Delta_{fws} := \frac{-5}{384} \cdot \frac{w_{fws} \cdot L^4}{E_c \cdot I_{xg}} \cdot if\left(BeamLoc = 0, 1, \frac{S - W_b}{S}\right) = -0.009 \cdot in$$

Deflection due to future wearing surface

$t_{bar} := 20$ — Age at which barrier is assumed to be cast

$T := \left(t_i \;\; 7 \;\; 14 \;\; 21 \;\; 28 \;\; 60 \;\; 120 \;\; 240 \;\; \infty\right)^T$ — Concrete ages at which camber is computed

15. CAMBER AND DEFLECTIONS (cont'd)

$$\Delta_{cr1}(t) := \psi(t - t_i, t_i)(\Delta_{gr} + \Delta_{ps})$$

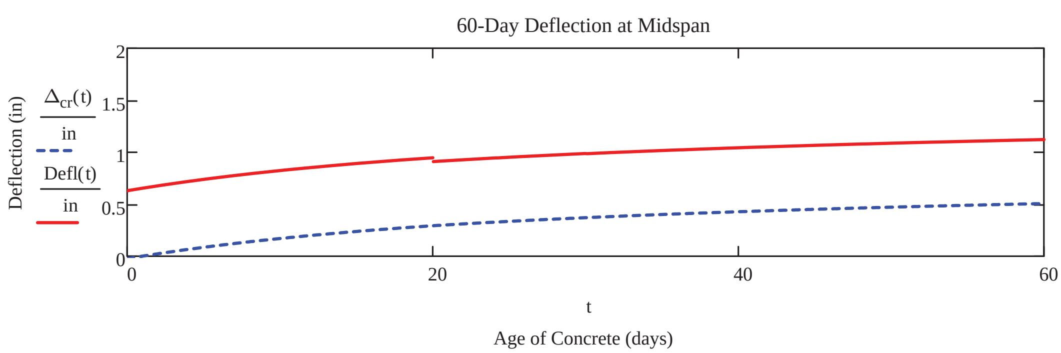
$$\Delta_{cr2}(t) := (\psi(t - t_i, t_i) - \psi(t_{bar} - t_i, t_i))\cdot(\Delta_{gr} + \Delta_{ps}) + \psi(t - t_{bar}, t_{bar})\cdot\Delta_{bar}$$

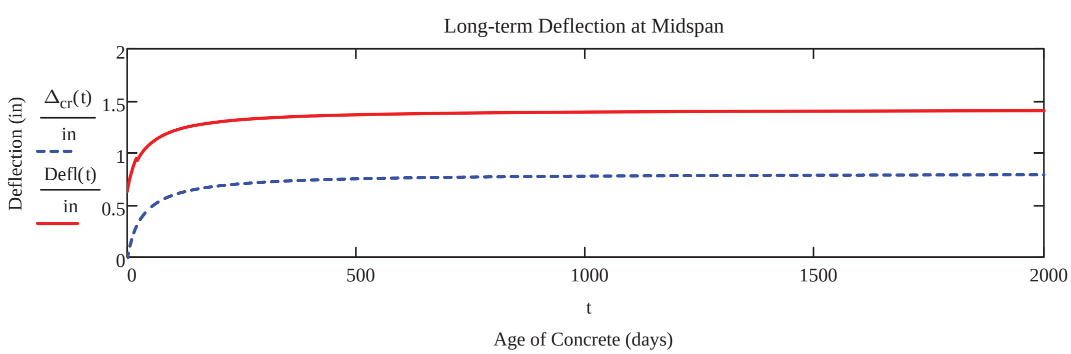
$$\Delta_{cr3}(t) := (\psi(t - t_i, t_i) - \psi(t_d - t_i, t_i))\cdot(\Delta_{gr} + \Delta_{ps}) + (\psi(t - t_{bar}, t_{bar}) - \psi(t_d - t_{bar}, t_{bar}))\cdot\Delta_{bar} \ldots + \psi(t - t_d, t_d)\cdot(\Delta_j)$$

$$\Delta_{cr}(t) := \begin{cases} \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ \Delta_{cr1}(t_{bar}) + \Delta_{cr2}(t_d) + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

$$\text{Defl}(t) := \begin{cases} (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t) & \text{if } t \le t_{bar} \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t) & \text{if } t_{bar} < t \le t_d \\ (\Delta_{gr} + \Delta_{ps}) + \Delta_{cr1}(t_{bar}) + \Delta_{bar} + \Delta_{cr2}(t_d) + \Delta_j + \Delta_{cr3}(t) & \text{if } t > t_d \end{cases}$$

$$C := \begin{array}{|l} \text{for } j \in 1..\text{last}(T) \\ \quad out_j \leftarrow \text{Defl}(T_j) \\ out \end{array} \qquad C^T = (0.653 \;\; 0.775 \;\; 0.879 \;\; 0.919 \;\; 0.974 \;\; 1.121 \;\; 1.236 \;\; 1.316 \;\; 1.42)\cdot in$$





16. NEGATIVE MOMENT FLEXURAL STRENGTH

Calculate factored moment that must be resisted across the interior pier and find required steel to be developed in the top flange.

Negative Live Load Moment

Compute the negative moment over the interior support due to the design live load load, in accordance with AASHTO LRFD 3.6.1.3.1.

Live Load Truck and Truck Train Moment Calculations

$\min(M_{truck}) = -530 \cdot kip \cdot ft$	Maximum negative moment due to a single truck
$\min(M_{train}) = -450 \cdot kip \cdot ft$	Maximum negative moment due to two trucks in a single lane
$M_{neg.lane} := \dfrac{-w_{lane} \cdot L^2}{2} = -512 \cdot kip \cdot ft$	Negative moment due to lane load on adjacent spans
$M_{neg.truck} := M_{neg.lane} + (1 + IM) \cdot \min(M_{truck}) = -1217 \cdot kip \cdot ft$	Live load negative moment for single truck
$M_{neg.train} := 0.9 \cdot \left[M_{neg.lane} + (1 + IM) \cdot \min(M_{train})\right] = -999 \cdot kip \cdot ft$	Live load negative moment for two trucks in a single lane
$M_{HL93.neg} := \min(M_{neg.truck}, M_{neg.train}) = -1217 \cdot kip \cdot ft$	Design negative live load moment, per design lane
$M_{ll.neg.i} := M_{HL93.neg} \cdot g_{mint} = -884 \cdot kip \cdot ft$	Design negative live load moment at interior beam
$M_{ll.neg.e} := M_{HL93.neg} \cdot g_{mext} = -921 \cdot kip \cdot ft$	Design negative live load moment at exterior beam
$M_{LL.neg} := \text{if}(BeamLoc = 1, M_{ll.neg.e}, M_{ll.neg.i}) = -921 \cdot kip \cdot ft$	Design negative live load moment

Factored Negative Design Moment

Dead load applied to the continuity section at interior supports is limited to the future overlay.

$M_{DW.neg} := \dfrac{-w_{fws} \cdot L^2}{2} = -159 \cdot kip \cdot ft$	Superimposed dead load resisted by continuity section
$M_{u.neg.StrI} := 1.5 \cdot M_{DW.neg} + 1.75 \cdot M_{LL.neg} = -1850 \cdot kip \cdot ft$	Strength Limit State
$M_{u.neg.StrI} := 1.0 \cdot M_{DW.neg} + 1.0 \cdot M_{LL.neg} = -1080 \cdot kip \cdot ft$	Service Limit State

16. NEGATIVE MOMENT FLEXURAL STRENGTH (cont'd)

Reinforcing Steel Requirement in the Top Flange for Strength

$\varphi_f := 0.90$ — Reduction factor for strength in tension-controlled reinforced concrete (5.5.4.2)

$b_c := b_1 = 26 \cdot in$ — Width of compression block at bottom flange

$d_{nms} := h - t_{sac} - 0.5 \cdot (t_{flange} - t_{sac}) = 35.5 \cdot in$ — Distance to centroid of negative moment steel, taken at mid-depth of top flange

$R_u := \dfrac{|M_{u.neg.StrI}|}{\varphi_f \cdot b_c \cdot d_{nms}^2} = 0.439 \cdot ksi$ — Factored load, in terms of stress in concrete at depth of steel, for computing steel requirement

$m := \dfrac{f_y}{0.85 \cdot f_c} = 8.824$ — Steel-to-concrete strength ratio

$\rho_{req} := \dfrac{1}{m} \cdot \left(1 - \sqrt{1 - \dfrac{2 \cdot m \cdot R_u}{f_y}}\right) = 0.0076$ — Required negative moment steel ratio

$A_{nms.req} := \rho_{req} \cdot b_c \cdot d_{nms} = 6.992 \cdot in^2$ — Required negative moment steel in top flange

$A_{s.long.t} := 2.0 \cdot in^2$ $\quad A_{s.long.b} := 2.0 \cdot in^2$ — Full-length longitudinal reinforcement to be made continuous across joint

$A_{bar} := 0.44 \cdot in^2$ — Additional negative moment reinforcing bar area

$A_{nms.t} := \dfrac{2}{3} \cdot A_{nms.req} - A_{s.long.t} = 2.661 \cdot in^2$ — Additional reinforcement area required in the top mat (2/3 of total)

$n_{bar.t} := ceil\left(\dfrac{A_{nms.t}}{A_{bar}}\right) = 7$ — Additional bars required in the top mat

$A_{nms.b} := \dfrac{1}{3} \cdot A_{nms.req} - A_{s.long.b} = 0.331 \cdot in^2$ — Additional reinforcement area required in the bottom mat

$n_{bar.b} := ceil\left(\dfrac{A_{nms.b}}{A_{bar}}\right) = 1$ — Additional bars required in the top mat

$s_{bar.top} := \dfrac{S - W_j - 6 \cdot in}{n_{bar.t} - 1} = 13.875 \cdot in$ — Spacing of bars in top mat

$A_{s.nms} := (n_{bar.t} + n_{bar.b}) \cdot A_{bar} + A_{s.long.t} + A_{s.long.b} = 7.52 \cdot in^2$ — Total reinforcing steel provided over pier

$a := \dfrac{A_{s.nms} \cdot f_y}{0.85 \cdot f_c \cdot b_c} = 2.552 \cdot in$ — Depth of compression block

$M_{r.neg} := \varphi_f \cdot A_{s.nms} \cdot f_y \cdot \left(d_{nms} - \dfrac{a}{2}\right) = 1158 \cdot kip \cdot ft$ — Factored flexural resistance at interior pier

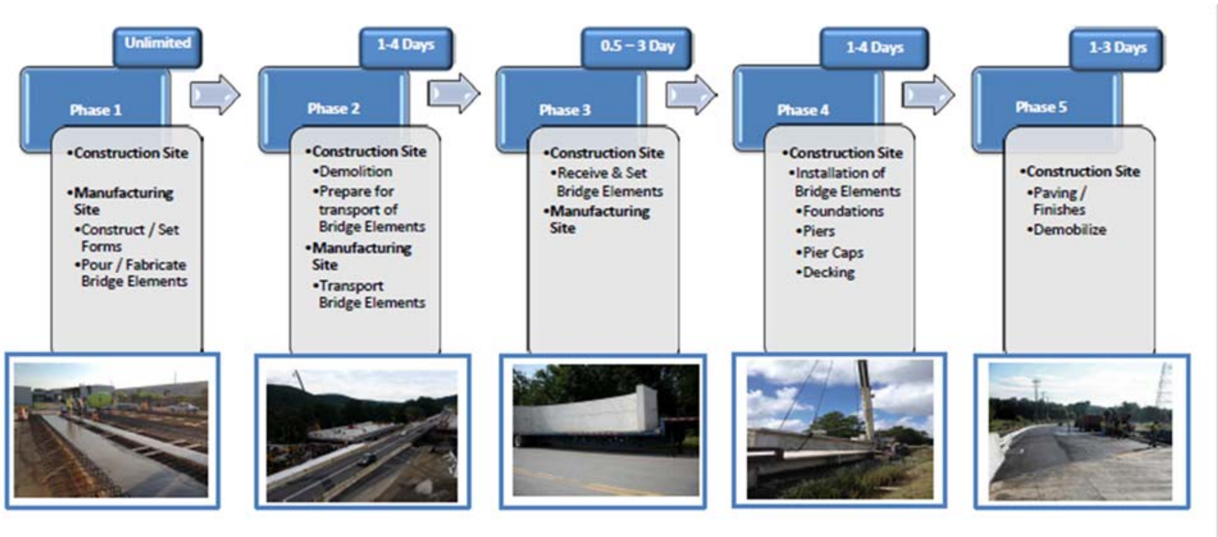
$DC_{neg.mom} := \dfrac{|M_{u.neg.StrI}|}{M_{r.neg}} = 0.932$

$CheckNegMom := if(DC_{neg.mom} \le 1.0, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$ — Negative flexure resistance check

APPENDIX C

ABC CONSTRUCTION PRACTICE FLOWCHARTS

APPENDIX C - ABC CONSTRUCTION PRACTICE FLOWCHARTS



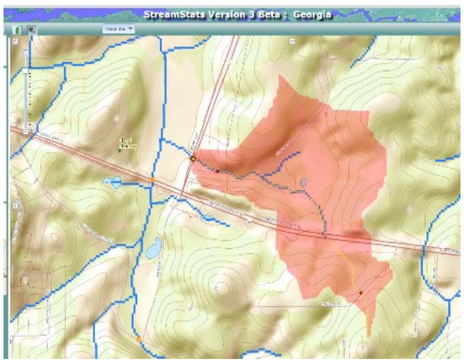
APPENDIX D

RISK ANALYSIS EXAMPLES AND INTERACTIVE FLOWCHART

APPENDIX D - RISK ANALYSIS EXAMPLES AND INTERACTIVE FLOWCHART

Example Problem 1:

As part of project involving the reconstruction of a portion of a state owned roadway, officials from Floyd County are considering replacing the three (3) 42” multi-barrel corrugated steel pipe culverts with larger size culverts. Rather than manually delineating the watershed, they use the USGS StreamStats application which reveals that the drainage area is 0.21miles^2 (134 acres), 3.5% of which is impervious (see Figure 1).



Label	Value	Units	Definition
DRNAREA	0.21	square miles	Area that drains to a point on a stream
PCTREG2	0	percent	Percentage of drainage area located in Region 2
PCTREG3	0	percent	Percentage of drainage area located in Region 3
PCTREG4	0	percent	Percentage of drainage area located in Region 4
PCTREG5	0	percent	Percentage of drainage area located in Region 5
CSL10_85	205	feet per mi	Change in elevation divided by length between points 10 and 85 percent of distance along main channel to basin divide - main channel method not known
LC11IMP	3.46	percent	Percentage of impervious area determined from NLCD 2011 impervious dataset
LC11DEV	19.6	percent	Percentage of developed (urban) land from NLCD 2011 classes 21-24

Figure 1. Output from USGS StreamStats Application

Using the USGS regression equations shown in Table 1 for a 50 year return period within Region 1, the peak flow is calculated to be:

$$Q_p = 661(DA)^{0.600} = 661\ (.21)^{0.600} = 259.\ 14\ ft^3/s$$

Computing the waterway area for V = 3 ft/s and 5 ft/s yields:

$$V = 3\text{ ft/s:}\quad A = \frac{Q_p}{V} = \frac{259.14\ ft^3/s}{3\ ft/s} = 86.38\ ft^2$$

$$V = 5\text{ ft/s:}\quad A = \frac{Q_p}{V} = \frac{259.14\ ft^3/s}{5\ ft/s} = 51.83\ ft^2$$

Based on the area information shown in Table 5, a single corrugated pipe culvert would have a size of 126” (for V = 3 ft/s) or 102” (for V = 5 ft/s). Using the more conservative size (126”), Table 7 indicates that the county could replace the existing culverts with three (3) 78” or four (4) 66” multi-barrel culverts.

Example Problem 2:

County officials from the same county (Floyd) are examining another state roadway project in which a single 4' x 4' concrete box culvert might have to be replaced. The location of the culvert is in an urbanized area with a total area of 0.125 miles^2 (80 acres). A site investigation reveals that the area is predominately flat, with 30% (24 acres) single family residential, 25% (20 acres) apartment homes, 35% (28 acres) lawns (clay soil), and the remaining 10% (8 acres) woodlands and forests.

Applying the Rational Method, the runoff coefficients for a 50 year return period are:

- Single family residential: (0.30 * 1.2) = 0.36
- Apartment homes: (0.50 * 1.2) = 0.60
- Lawns (clay soil): (0.17 * 1.2) = 0.20
- Woodlands and forests: (0.10 * 1.2) = 0.12

The weighted runoff coefficient is computed as follows:

$$C_{weighted} = \frac{(0.36)(24) + (0.60)(20) + (0.20)(28) + (0.12)(8)}{80} = 0.34$$

Since the project location is near the City of Rome, the rainfall intensity data can be taken directly from Table 4:

$$I = 3.12 \text{ in/hr}$$

Using the runoff coefficient, rainfall intensity, and area information, the peak flow is computed as:

$$Q_p = CIA = (0.34) * (3.12 \text{ in/hr}) * (80 \text{ acres}) = 84.86 \text{ ft}^3/\text{s}$$

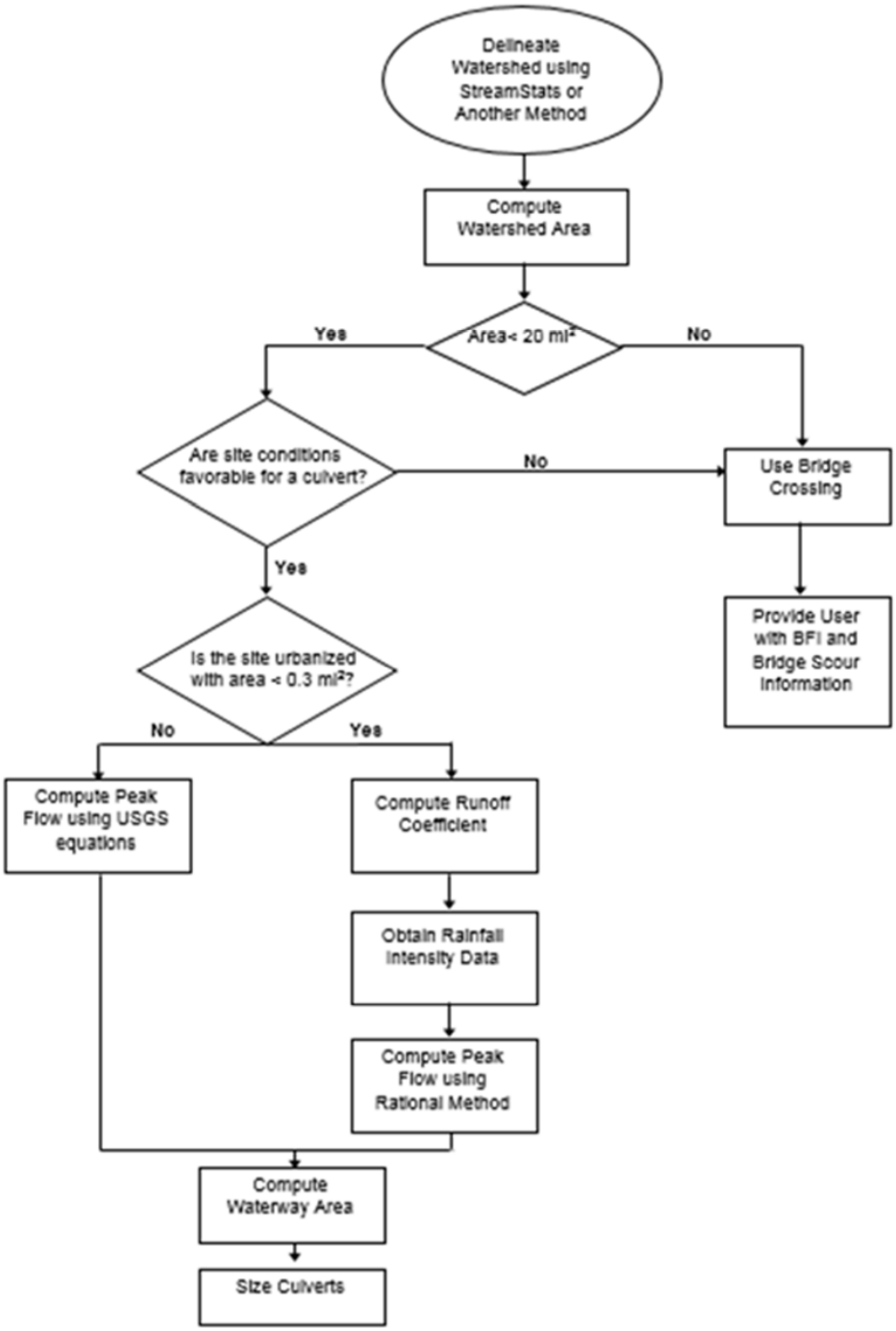
Computing the waterway area for V = 3 ft/s and 5 ft/s yields:

$$\text{V = 3 ft/s:} \quad A = \frac{Q_p}{V} = \frac{84.86\ ft^3/s}{3\ ft/s} = 28.29\ ft^2$$

$$\text{V = 5 ft/s:} \quad A = \frac{Q_p}{V} = \frac{84.86\ ft^3/s}{5\ ft/s} = 16.97\ ft^2$$

Based on the area information shown in Table 6, a single box culvert would have a size of 6' x 5' (for V = 3 ft/s) or 6' x 3' (for V = 5 ft/s). Using the more conservative size, the county could replace the existing 4' x 4' culvert with a 6' x 5' culvert.

Risk Analysis Flowchart:



APPENDIX E

CONCEPTUAL COST ESTIMATES EXAMPLES

APPENDIX E - CONCEPTUAL COST ESTIMATES EXAMPLES

Case study 1 summary - Interstate Bridge Replacement over Local Road in Urban Environment (FHWA-Every Day Counts)

Construction Type	Labor ($,%, Hrs)	Material ($,%)	Subcontractors ($,%)	Equipment ($,%, Hrs)	Other ($,%)	Total ($)
Prefabricated	$1,889,726 35.38% 10,4449.281 Hours	$1,887,855 35.35%	$789,209 14.78%	$679,331 12.72% 3,261.644 Hours	$94,564 1.77%	$5,340,685
Conventional	$2,119,985 38.82% 21,670.339 Hours	$1,210,878 17.60%	$0.00 0%	$418,030 6.08% 2,561.883 Hours	$3,129,659 45.50%	$6,878,552

Details for the breakdown of direct cost components, quantity takeoffs and total cost/units are shown in exhibits E1 and E2. Users may refer also to the notes provided to each case study for additional information regarding durations and pricing.

Case study 2 summary - Route 10 over Passaic River Superstructure Replacement estimate report. This particular example was obtained from an experienced contractor. The cost estimate report is a detailed report of break down costs obtained by construction items assigned individually and their total units for total cost calculations on the bridge replacement job. Therefore, the job estimate report is presented as a guidance for cost estimators and it is presented in exhibit E3 of this appendix.

The responses of the special survey are attached to this Appendix E, as exhibit E4. They can be used as further guidance to potential users and to complement the decision-making matrix with inclusive items affecting directly the conceptual cost estimates. They were a total of eighteen responses collected and they are all attached to this report.

The followings are the special survey questions on conceptual cost estimates factors and cost components deployed to all state DOTs:

1) Based on your experience with ABC, are prefabricated bridges more costly than conventional bridges?
2) Have you ever used another contracting method besides Design-Bid-Build for ABC? If yes, under what circumstances?
3) What ABC elements have been more costly than conventional bridge construction?
4) Have any of your contractors had cost concerns when using ABC?
5) Did federal and/or state requirements affect the overall ABC cost/duration of the project? If yes, how?
6) What, if any, environmental factors/policies affected your ABC projects? How did these affect the overall cost?

Cost Estimating Spreadsheet Report

Prefabricated Alt. CS 1

Every Day Counts Case Study 1

Project name Prefabricated Alt. CS 1
Fla.

Labor rate table Labor 2011

Equipment rate table Equip 2011

Notes
1. Pricing is 2011 $.
2. Construction Schedule of 12 mos.
3. Rates reflect majority of work performed days with the exception of actual installation of bridges.
4. 2 Bridges.
5. Labor Cost/Unit - This reflects the cost of Labor to put one unit of measure of work in place. This is comprised of a typical crew with associated productivity required to perform the activity. Labor is priced up to include base rate, fringes, taxes, insurance, etc.
6. Material Cost/Unit - This cost represents the final installed cost for all the materials associated with the item of work.
7. Sub Contract Cost/Unit - this represents the Sub Contract all in Cost to put one unit of work in place. This cost includes all costs necessary to reflect a total installed cost (includes Labor, equipment and material).
8. Equipment Cost/Unit - This represents the cost of Construction Equipment necessary to put one unit of work in place. This cost includes equipment ownership and maintenance and operational costs.
9. Other Cost/Unit - This cost represents all other costs necessary to perform the item of work and not covered by the abovementioned costs.
10. Total cost/unit - This is the summation of all unit costs to arrive at a total cost per unit to put one item of work in place.
11. Total Amount - Total cost to put the quantity of units specified in place.

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
Additional Cost Additional Cost SPMT's								
*** unassigned ***								
01-54-23.00 Temporary Scaffolding And Platforms								
01-54-23.70 Scaffolding								
Scaffolding, steel tubular, heavy duty shoring for elev slab forms, floor area, rent/month of complete system, to 14'-8" H	300.00 csf	-	51.42	-	-	-	51.42	15,425
Scaffolding								*15,425*
Temporary Scaffolding And Platforms								*15,425*
02-43-00.00 Structure Moving								
02-43-13.13 Bridge Relocation								
Remove Existing Bridges Out & Install New Bridges SPMT	1.00 totl	-	-	526,139.46	-		526,139.46	526,139
Remove Existing Bridges Out & Install New Bridges SPMT 2nd Bridge	1.00 totl	-	-	263,069.73	-		263,069.73	263,070
Bridge Relocation								*789,209*
Structure Moving								*789,209*
32-12-16.00 Asphalt Paving								
32-12-16.13 Plant-Mix Asphalt Paving								
Allow for additional Paving etc.	1,000.00 ton	6.23	62.18	-	3.46	-	71.86	71,861
Plant-Mix Asphalt Paving								*71,861*
Asphalt Paving								*71,861*
32-34-00.00 Fabricated Bridges								
32-34-10.10 Bridges, Highway								
Temporary Concrete	100.00 cy	270.95	198.50	-	19.01	-	488.46	48,846
Temporary Concrete Remove	100.00 cy			-		149.47	149.47	14,947
Bridges, Highway								*63,793*
Fabricated Bridges								*63,793*
34-71-13.00 Vehicle Barriers								
34-71-13.17 Security Vehicle Barriers								
Jersey Barriers	64.00 ea	79.32	119.58	-	31.50	-	230.40	14,745

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
34-71-13.17 Security Vehicle Barriers								
Jersey Barriers Pier Expansion	200.00 ea	79.32	119.58	-	31.50	-	230.40	46,080
Detour	2.00 Day	-	-	-	-	3,826.47	3,826.47	7,653
Security Vehicle Barriers								*68,478*
Vehicle Barriers								*68,478*
*** unassigned ***								**1,008,766**
Additional Cost								***1,008,766***
Additional Cost SPMT's								
General Conditions General Conditions								
*** unassigned ***								
01-31-00.00 Project Management And Coordination								
01-31-13.20 Field Personnel								
Field Personnel, clerk, average	42.00 week	436.46	-	-	-	-	436.46	18,331
Field engineer, average	47.00 week	1,345.24	-	-	-	-	1,345.24	63,226
Field Personnel, project manager, average	47.00 week	2,212.18	-	-	-	-	2,212.18	103,972
Field Personnel, superintendent, average	42.00 week	2,032.81	-	-	-	-	2,032.81	85,378
Field Personnel								*270,908*
Project Management And Coordination								*270,908*
01-32-33.00 Photographic Documentation								
01-32-33.50 Photographs								
Construction Photographs	42.00 set	-	567.99	-	-	-	567.99	23,856
Photographs								*23,856*
Photographic Documentation								*23,856*
01-45-00.00 Quality Control								
01-45-23.50 Testing								
Field Testing	2.00 prjc	-	-	-	-	35,981.96	35,981.96	71,964
Testing								*71,964*
Quality Control								*71,964*
01-52-13.00 Field Offices And Sheds								
01-52-13.20 Office And Storage Space								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
01-52-13.20 Office And Storage Space								
Office Trailer, furnished, rent per month, 32' x 8', excl. hookups	12.00 ea	-	288.18	-	-	-	288.18	3,458
Storage Boxes, rent per month, 20' x 8'	24.00 ea	-	87.89	-	-	-	87.89	2,109
Office And Storage Space								*5,568*
01-52-13.40 Field Office Expense								
Field Office Expense, office equipment rental, average Field	12.00 mo	-	179.37	-	-	-	179.37	2,152
Office Expense, office supplies, average	12.00 mo	-	113.60	-	-	-	113.60	1,363
Field Office Expense, telephone bill; avg. bill/month, incl. long dist.	12.00 mo	-	251.11	-	-	-	251.11	3,013
Field Office Expense, field office lights & HVAC	12.00 mo	-	131.53	-	-	-	131.53	1,578
Field Office Expense								*8,107*
Field Offices And Sheds								*13,675*
01-56-26.00 Temporary Fencing								
01-56-26.50 Temporary Fencing								
Temporary Fencing, chain link, 6' high, 11 ga	2,000.00 lf	3.17	8.37	-	-	-	11.54	23,077
Temporary Fencing								*23,077*
Temporary Fencing								*23,077*
01-58-00.00 Project Identification								
01-58-13.50 Signs								
Project Signs	50.00 sf	-	21.40	-	-	-	21.40	1,070
Signs								*1,070*
Project Identification								*1,070*
01-71-23.00 Field Engineering								
01-71-23.13 Construction Layout								
Survey Crew	30.00 day	1,898.46	-	-	90.63	-	1,989.09	59,673
Construction Layout								*59,673*
Field Engineering								*59,673*
*** unassigned ***								**464,223**
General Conditions								***464,223***
General Conditions								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
Permanent Walls Permanent Walls								
400-2-10 Concrete Class II								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Approach Slab	84.40 cy	142.64	173.39	-	20.87	-	336.90	28,434
Concrete In Place								*28,434*
Cast-In-Place Concrete								*28,434*
400-2-10 Concrete Class II								**28,434**
415-1-9 Reinforcing Steel								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								
Reinforcing Steel Approach Slabs	21,112.00 lb	0.37	0.06	-	0.08	-	0.51	10,676
Reinforcing In Place								*10,676*
Reinforcing Steel Accessories								*10,676*
415-1-9 Reinforcing Steel								**10,676**
*** unassigned ***								
03-37-13.00 Shotcrete								
03-37-13.60 Shotcrete (Wet-Mix)								
Shotcrete	5,456.00 sf	26.04	3.68	-	14.76	-	44.49	242,711
Shotcrete (Wet-Mix)								*242,711*
Shotcrete								*242,711*
31-32-36.00 Soil Nailing								
31-32-36.16 Grouted Soil Nailing								
Gouted soil nailing,drill hole,install # 8 nail,grout,diffclt,grade 75,20 min setup per hole&80'/hr drilling	220.00 ea	255.14	639.74	-	243.13	-	1,138.01	250,361
Grouted Soil Nailing								*250,361*
Soil Nailing								*250,361*
*** unassigned ***								**493,072**
Permanent Walls								***532,182***
Permanent Walls								
Substructure-End Substructure-End Bents								
400-4-5 Concrete Class IV (Substructure)								
03-30-00.00 Cast-In-Place Concrete								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
03-30-53.40 Concrete In Place								
Concrete Class IV	78.20 cy	285.29	197.30	-	41.73	-	524.32	41,002
Concrete In Place								*41,002*
Cast-In-Place Concrete								*41,002*
400-4-5 Concrete Class IV (Substructure)								**41,002**
415-1-5 Reinforcing Steel Substructure								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								
Reinforcing Steel	10,550.00 lb	0.49	0.50	-	0.11	-	1.09	11,508
Reinforcing In Place								*11,508*
Reinforcing Steel Accessories								*11,508*
415-1-5 Reinforcing Steel Substructure								**11,508**
455-133-2 Sheetpile Wall (Temporary)								
31-41-16.00 Sheet Piling								
31-41-16.10 Sheet Piling Systems								
Sheet piling, steel, Temporary	3,680.00 sf	4.83	14.29	-	4.33	-	23.45	86,293
Sheet Piling Systems								*86,293*
Sheet Piling								*86,293*
455-133-2 Sheetpile Wall (Temporary)								**86,293**
455-143-3 Test Piles								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed Concrete Piles, Test Piles	380.00 vlf	29.69	37.67	-	18.54	-	85.90	32,641
Prestressed Concrete Piles								*32,641*
Driven Piles								*32,641*
455-143-3 Test Piles								**32,641**
455-34-3 Concrete Piling Prestressed								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed Concrete Piles	640.00 vlf	10.80	37.67	-	6.74	-	55.21	35,331
Prestressed Concrete Piles								*35,331*

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
Driven Piles								*35,331*
455-34-3 Concrete Piling Prestressed								**35,331**
Substructure-End								***206,775***
Substructure-End								
Bents								
Substructure-Piers Substructure-Piers								
400-4-25 Concrete Class V Superstructure								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class V	94.60 cy	228.23	179.37	-	33.38	-	440.98	41,717
Concrete In Place								*41,717*
Cast-In-Place Concrete								*41,717*
400-4-25 Concrete Class V Superstructure								**41,717**
400-4-5 Concrete Class IV (Substructure)								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class IV	39.20 cy	285.29	197.30	-	41.73	-	524.32	20,553
Concrete In Place								*20,553*
Cast-In-Place Concrete								*20,553*
400-4-5 Concrete Class IV (Substructure)								**20,553**
415-1-5 Reinforcing Steel Substructure								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								
Reinforcing Steel	20,042.00 lb	0.49	0.50	-	0.11	-	1.09	21,861
Reinforcing In Place								*21,861*
Reinforcing Steel Accessories								*21,861*
415-1-5 Reinforcing Steel Substructure								**21,861**
455-143-5 Test Piles								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed concrete piles, 24" square, Test Pile	210.00 vlf	29.69	65.77	-	18.54	-	114.00	23,940
Prestressed Concrete Piles								*23,940*

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
Driven Piles								*23,940*
455-143-5 Test Piles								**23,940**
455-34-5 Concrete Piling Prestressed								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed concrete piles, 24" square,	1,800.00 vlf	10.80	65.77	-	6.74	-	83.31	149,951
Prestressed Concrete Piles								*149,951*
Driven Piles								*149,951*
455-34-5 Concrete Piling Prestressed								**149,951**
Substructure-Piers								***258,022***
Substructure-Piers								
Superstructure Superstructure								
110-3 Structure Removal of Existing								
02-41-16.00 Structure Demolition								
02-41-16.33 Bridge Demolition								
Bridge demolition	21,048.00 sf	12.72	-	-	8.26	-	20.98	441,610
Bridge Demolition								*441,610*
Structure Demolition								*441,610*
110-3 Structure Removal of Existing								**441,610**
400-147 Composite Neoprene Pads								
05-05-23.00 Metal Fastenings								
05-05-23.80 Vibration & Bearing Pads								
Bearing Pads	10.60 cf	78.49	717.46	-	-	-	795.95	8,437
Vibration & Bearing Pads								*8,437*
Metal Fastenings								*8,437*
400-147 Composite Neoprene Pads								**8,437**
400-2-4 Concrete Class II								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class II	866.20 cy	142.65	173.39	-	20.87	-	336.90	291,820
Concrete In Place								*291,820*
Cast-In-Place Concrete								*291,820*

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
400-2-4 Concrete Class II								**291,820**
400-5-25 Concrete Class V Superstructure								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class V	162.60 cy	228.23	179.37	-	33.38	-	440.98	71,703
Concrete In Place								*71,703*
Cast-In-Place Concrete								*71,703*
400-5-25 Concrete Class V Superstructure								**71,703**
400-9 Bridge Floor Grooving								
32-13-13.00 Concrete Paving								
32-13-13.23 Concrete Paving Surface Treatment								
Concrete Grooving	3,536.00 sy	2.31	-	-	5.20	-	7.52	26,581
Concrete Paving Surface Treatment								*26,581*
Concrete Paving								*26,581*
400-9 Bridge Floor Grooving								**26,581**
415-1-4 Reinforcing Steel								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								
Reinforcing Steel Superstructure	201,944.00 lb	0.37	0.06	-	0.08	-	0.51	102,118
Reinforcing In Place								*102,118*
Reinforcing Steel Accessories								*102,118*
415-1-4 Reinforcing Steel								**102,118**
458-1-12 Bridge Deck Expansion Joint								
32-34-00.00 Fabricated Bridges								
32-34-10.10 Bridges, Highway								
Bridge Deck Expansion Joint	274.00 lf	105.39	326.45	-	3.69	-	435.53	119,335
Bridges, Highway								*119,335*
Fabricated Bridges								*119,335*
458-1-12 Bridge Deck Expansion Joint								**119,335**
460-2-1 Structural Steel								
05-12-23.00 Structural Steel For Bridges								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
05-12-23.77 Structural Steel Projects								
Structural Steel	450.00 ton	1,271.58	1,913.23	-	515.29	-	3,700.10	1,665,046
Structural Steel Projects								*1,665,046*
Structural Steel For Bridges								*1,665,046*
460-2-1 Structural Steel								**1,665,046**
461-113-7 Multi Rotational Bearing Assembly								
05-05-23.00 Metal Fastenings								
05-05-23.80 Vibration & Bearing Pads								
Multirotational Bearing (1750 Kip)	2.00 ea	594.61	9,566.17	-	-	-	10,160.78	20,322
Vibration & Bearing Pads								*20,322*
Metal Fastenings								*20,322*
461-113-7 Multi Rotational Bearing Assembly								**20,322**
461-114-5 Multi Rotational Bearing Assembly								
05-05-23.00 Metal Fastenings								
05-05-23.80 Vibration & Bearing Pads								
Multirotational Bearing (1200 Kip)	2.00 ea	392.44	5,978.86	-	-	-	6,371.30	12,743
Vibration & Bearing Pads								*12,743*
Metal Fastenings								*12,743*
461-114-5 Multi Rotational Bearing Assembly								**12,743**
462-2-11 Post Tensioning Tendons								
03-23-00.00 Stressing Tendons								
03-23-05.50 Prestressing Steel								
Post Tensioning Strands	11,364.00 lb	1.90	0.61	-	0.02	-	2.53	28,728
Prestressing Steel								*28,728*
Stressing Tendons								*28,728*
462-2-11 Post Tensioning Tendons								**28,728**
521-5-1 Concrete Traffic Railing								
34-71-13.00 Vehicle Barriers								
34-71-13.26 Vehicle Guide Rails								
Concrete Traffic Railing Barrier Bridge	1,240.00 lf	12.88	49.03	-	4.44	-	66.35	82,275
Vehicle Guide Rails								*82,275*

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	TotalAmount
Vehicle Barriers								*82,275*
521-5-1 Concrete Traffic Railing								82,275
Superstructure								*2,870,718*
Superstructure								

Estimate Totals

Description	Amount	Net Amount	Totals	Hours	Rate	Cost Basis	Percent of Total	
Labor	1,889,726	1,889,726		10,449.281 ch			35.38%	
Material	1,887,855	1,887,855					35.35%	
Subcontract	789,209	789,209					14.78%	
Equipment	679,331	679,331		3,261.644 ch			12.72%	
Other	94,564	94,564					1.77%	
	5,340,685		**5,340,685**				**100.00**	**100.00%**
Total			***5,340,685***					

Every Day Counts - Case Study 1

Project name	Conventional Alt. CS 1 Fla.
Labor rate table	Labor 2011
Equipment rate table	Equip 2011
Notes	1. Pricing is 2011 $. 2. Construction Schedule of 28 mos. 3. Rates reflect majority of work performed evenings to minimize disruption. 4. 2 Bridges. 5. Labor Cost/Unit - This reflects the cost of Labor to put one unit of measure of work in place. This is comprised of a typical crew with associated productivity required to perform the activity. Labor is priced up to include base rate, fringes, taxes, insurance, etc. 6. Material Cost/Unit - This cost represents the final installed cost for all the materials associated with the item of work. 7. Sub Contract Cost/Unit - this represents the Sub Contract all in Cost to put one unit of work in place. This cost includes all costs necessary to reflect a total installed cost (incls. Labor, equipment and material). 8. Equipment Cost/Unit - This represents the cost of Construction Equipment necessary to put one unit of work in place. This cost includes equipment ownership and maintenance and operational costs. 9. Other Cost/Unit - This cost represents all other costs necessary to perform the item of work and not covered by the abovementioned costs. 10. Total cost/unit - This is the summation of all unit costs to arrive at a total cost per unit to put one item of work in place. 11. Total Amount - Total cost to put the quantity of units specified in place.

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
Additional Cost Additional Cost Detour								
*** unassigned ***								
01-54-36.00 Equipment Mobilization								
01-54-36.50 Mobilization Or Demob.								
Transport Bridge to Site	4.00 ea	517.79	-	-	817.10	-	1,334.89	5,340
Transport Bridge back to FDOT	4.00 ea	517.79	-	-	817.11	-	1,334.90	5,340
Mobilization Or Demob.								*10,679*
Equipment Mobilization								*10,679*
01-55-00.00 Vehicular Access And Parking								
01-55-23.50 Roads And Sidewalks								
Temporary, roads	2,000.00 lf			-		956.62	956.62	1,913,234
Temporary, roads Remove	2,000.00 lf			-		119.58	119.58	239,154
Roads And Sidewalks								*2,152,389*
Vehicular Access And Parking								*2,152,389*
32-34-00.00 Fabricated Bridges								
32-34-10.10 Bridges, Highway								
Fabricated highway bridges, Install	330.00 lf	50.64	0.00	-	13.97	-	64.61	21,320
Fabricated highway bridges, Remove	330.00 lf	50.64		-	13.97	-	64.61	21,322
Fabricated highway bridges, concrete in place, abutment	350.00 cy	293.61	198.50	-	19.01	-	511.12	178,891
Fabricated highway bridges, concrete in place, abutment Remove	350.00 cy		0.00	-		149.47	149.47	52,315
Bridges, Highway								*273,848*
Fabricated Bridges								*273,848*
34-71-13.00 Vehicle Barriers								
34-71-13.17 Security Vehicle Barriers								
Maintain Detour	500.00 day	-	-	-	-	89.68	89.68	44,841
Security Vehicle Barriers								*44,841*
Vehicle Barriers								*44,841*
*** unassigned ***								**2,481,758**
Additional Cost Additional Cost Detour								***2,481,758***

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
General Conditions General Conditions								
*** unassigned ***								
01-31-00.00 Project Management And Coordination								
01-31-13.20 Field Personnel								
Field Personnel, clerk, average	117.00 week	472.96	-	-	-	-	472.96	55,336
Field engineer, average	121.00 week	1,457.74	-	-	-	-	1,457.74	176,387
Field Personnel, project manager, average	121.00 week	2,397.18	-	-	-	-	2,397.18	290,059
Field Personnel, superintendent, average	117.00 week	2,202.81	-	-	-	-	2,202.81	257,729
Field Personnel								*779,511*
Project Management And Coordination								*779,511*
01-32-33.00 Photographic Documentation								
01-32-33.50 Photographs								
Construction Photographs	120.00 set	-	567.99	-	-	-	567.99	68,159
Photographs								*68,159*
Photographic Documentation								*68,159*
01-45-00.00 Quality Control								
01-45-23.50 Testing								
Field Testing	2.00 prjc	-	-	-	-	35,981.96	35,981.96	71,964
Testing								*71,964*
Quality Control								*71,964*
01-52-13.00 Field Offices And Sheds								
01-52-13.20 Office And Storage Space								
Office Trailer, furnished, rent per month, 32' x 8', excl. hookups	28.00 ea	-	288.18	-	-	-	288.18	8,069
Storage Boxes, rent per month, 20' x 8'	56.00 ea	-	87.89	-	-	-	87.89	4,922
Office And Storage Space								*12,991*
01-52-13.40 Field Office Expense								
Field Office Expense, office equipment rental, average	28.00 mo	-	179.37	-	-	-	179.37	5,022
Field Office Expense, office supplies, average	28.00 mo	-	113.60	-	-	-	113.60	3,181

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
01-52-13.40 Field Office Expense								
Field Office Expense, telephone bill; avg. bill/month, incl. long dist.	28.00 mo	-	251.11	-	-	-	251.11	7,031
Field Office Expense, field office lights & HVAC	28.00 mo	-	131.54	-	-	-	131.54	3,683
Field Office Expense								*18,917*
Field Offices And Sheds								*31,908*
01-56-26.00 Temporary Fencing								
01-56-26.50 Temporary Fencing								
Temporary Fencing, chain link, 6' high, 11 ga	2,000.00 lf	3.43	8.37	-	-	-	11.80	23,607
Temporary Fencing								*23,607*
Temporary Fencing								*23,607*
01-58-00.00 Project Identification								
01-58-13.50 Signs								
Project Signs	50.00 sf	-	21.40	-	-	-	21.40	1,070
Signs								*1,070*
Project Identification								*1,070*
01-71-23.00 Field Engineering								
01-71-23.13 Construction Layout								
Survey Crew	90.00 day	2,057.22	-	-	90.63	-	2,147.85	193,306
Construction Layout								*193,306*
Field Engineering								*193,306*
*** unassigned ***								**1,169,525**
General Conditions								***1,169,525***
General Conditions								
Permanent Walls Permanent Walls								
400-2-10 Concrete Class II								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Approach Slab	262.60 cy	154.57	173.39	-	20.87	-	348.83	91,602
Concrete In Place								*91,602*
Cast-In-Place Concrete								*91,602*
400-2-10 Concrete Class II								**91,602**
415-1-9 Reinforcing Steel								
03-21-05.00 Reinforcing Steel Accessories								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
03-21-10.60 Reinforcing In Place								
Reinforcing Steel Approach Slabs	53,832.00 lb	0.40	0.06	-	0.08	-	0.54	28,866
Reinforcing In Place								*28,866*
Reinforcing Steel Accessories								*28,866*
415-1-9 Reinforcing Steel								**28,866**
521-8-1 Concret Traffic Railing								
34-71-13.00 Vehicle Barriers								
34-71-13.26 Vehicle Guide Rails								
Concrete Traffic Railing Barrier Retaining Wall	100.00 lf	13.96	49.03	-	4.44	-	67.43	6,743
Vehicle Guide Rails								*6,743*
Vehicle Barriers								*6,743*
521-8-1 Concret Traffic Railing								**6,743**
548-12 Retainig Wall System								
32-32-23.00 Retaining Walls								
32-32-23.13 Retaining Walls								
Retaining Wall System	13,826.00 sf	8.28	14.05	-	4.24	-	26.58	367,434
Retaining Walls								*367,434*
Retaining Walls								*367,434*
548-12 Retainig Wall System								**367,434**
Permanent Walls								***494,644***
Permanent Walls								
Substructure-End Substructure-End Bents								
400-4-5 Concrete Class IV (Substructure)								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class IV	144.00 cy	309.15	197.30	-	41.73	-	548.18	78,938
Concrete In Place								*78,938*
Cast-In-Place Concrete								*78,938*
400-4-5 Concrete Class IV (Substructure)								**78,938**
415-1-5 Reinforcing Steel Substructure								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
03-21-10.60 Reinforcing In Place								
Reinforcing Steel	19,440.00 lb	0.53	0.50	-	0.11	-	1.13	21,996
Reinforcing In Place								*21,996*
Reinforcing Steel Accessories								*21,996*
415-1-5 Reinforcing Steel Substructure								**21,996**
455-143-5 Test Piles								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed concrete piles, 24" square, Test Pile	380.00 vlf	32.18	65.77	-	18.54	-	116.48	44,263
Prestressed Concrete Piles								*44,263*
Driven Piles								*44,263*
455-143-5 Test Piles								**44,263**
455-34-5 Concrete Piling Prestressed								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed concrete piles, 24" square,	2,240.00 vlf	11.70	65.77	-	6.74	-	84.21	188,628
Prestressed Concrete Piles								*188,628*
Driven Piles								*188,628*
455-34-5 Concrete Piling Prestressed								**188,628**
Substructure-End								***333,825***
Substructure-End Bents								
Substructure-Piers Substructure-Piers								
400-4-5 Concrete Class IV (Substructure)								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class IV	283.40 cy	309.15	197.30	-	41.73	-	548.18	155,354
Concrete In Place								*155,354*
Cast-In-Place Concrete								*155,354*
400-4-5 Concrete Class IV (Substructure)								**155,354**

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
415-1-5 Reinforcing Steel Substructure								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								
Reinforcing Steel	42,510.00 lb	0.53	0.50	-	0.11	-	1.13	48,100
Reinforcing In Place								*48,100*
Reinforcing Steel Accessories								*48,100*
415-1-5 Reinforcing Steel Substructure								**48,100**
455-143-5 Test Piles								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed concrete piles, 24" square, Test Pile	170.00 vlf	32.18	65.77	-	18.54	-	116.48	19,802
Prestressed Concrete Piles								*19,802*
Driven Piles								*19,802*
455-143-5 Test Piles								**19,802**
455-34-5 Concrete Piling Prestressed								
31-62-00.00 Driven Piles								
31-62-13.23 Prestressed Concrete Piles								
Prestressed concrete piles, 24" square,	1,960.00 vlf	11.70	65.77	-	6.74	-	84.21	165,050
Prestressed Concrete Piles								*165,050*
Driven Piles								*165,050*
455-34-5 Concrete Piling Prestressed								**165,050**
Substructure-Piers Substructure-Piers								***388,305***
Superstructure Superstructure								
110-3 Structure Removal of Existing								
02-41-16.00 Structure Demolition								
02-41-16.33 Bridge Demolition								
Bridge demolition	21,048.00 sf	15.31	-	-	9.18	-	24.49	515,552
Bridge Demolition								*515,552*
Structure Demolition								*515,552*

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
110-3 Structure Removal of Existing								**515,552**
400-147 Composite Neoprene Pads								
05-05-23.00 Metal Fastenings								
05-05-23.80 Vibration & Bearing Pads								
Bearing Pads	37.80 cf	85.05	717.46	-	-	-	802.51	30,335
Vibration & Bearing Pads								*30,335*
Metal Fastenings								*30,335*
400-147 Composite Neoprene Pads								**30,335**
400-2-4 Concrete Class II								
03-30-00.00 Cast-In-Place Concrete								
03-30-53.40 Concrete In Place								
Concrete Class II	905.20 cy	154.57	173.39	-	20.87	-	348.83	315,757
Concrete In Place								*315,757*
Cast-In-Place Concrete								*315,757*
400-2-4 Concrete Class II								**315,757**
400-9 Bridge Floor Grooving								
32-13-13.00 Concrete Paving								
32-13-13.23 Concrete Paving Surface Treatment								
Concrete grooving	4,158.00 sy	2.51	-	-	5.20	-	7.71	32,061
Concrete Paving Surface Treatment								*32,061*
Concrete Paving								*32,061*
400-9 Bridge Floor Grooving								**32,061**
415-1-4 Reinforcing Steel								
03-21-05.00 Reinforcing Steel Accessories								
03-21-10.60 Reinforcing In Place								
Reinforcing Steel Superstructure	185,566.00 lb	0.40	0.06	-	0.08	-	0.54	99,503
Reinforcing In Place								*99,503*
Reinforcing Steel Accessories								*99,503*
415-1-4 Reinforcing Steel								**99,503**
450-2-54 Prestressed Beams								
32-34-00.00 Fabricated Bridges								
32-34-10.10 Bridges, Highway								

Spreadsheet Level	Takeoff Quantity	Labor Cost/Unit	Material Cost/Unit	Sub Cost/Unit	Equip Cost/Unit	Other Cost/Unit	Total Cost/Unit	Total Amount
32-34-10.10 Bridges, Highway								
Fabricated highway bridges, precast, prestressed concrete, I beams	4,224.00 lf	-	-	-	-	191.32	191.32	808,150
Bridges, Highway								*808,150*
Fabricated Bridges								*808,150*
450-2-54 Prestressed Beams								**808,150**
458-1-12 Bridge Deck Expansion Joint								
32-34-00.00 Fabricated Bridges								
32-34-10.10 Bridges, Highway								
Bridge Deck Expansion Joint	274.00 lf	114.21	326.45	-	3.69	-	444.34	121,750
Bridges, Highway								*121,750*
Fabricated Bridges								*121,750*
458-1-12 Bridge Deck Expansion Joint								**121,750**
521-5-1 Concrete Traffic Railing								
34-71-13.00 Vehicle Barriers								
34-71-13.26 Vehicle Guide Rails								
Concrete Traffic Railing Barrier Bridge	1,296.00 lf	13.96	49.03	-	4.44	-	67.43	87,387
Vehicle Guide Rails								*87,387*
Vehicle Barriers								*87,387*
521-5-1 Concrete Traffic Railing ***Superstructure***								**87,387**
Superstructure								***2,010,495***

Estimate Totals

Description	Amount Cuts/Adds	Net Amount	Totals	Hours	Rate	Cost Basis	Percent of Total	
Labor	2,119,985	2,119,985		21,670.339 ch			30.82%	
Material	1,210,878	1,210,878					17.60%	
Subcontract								
Equipment	418,030	418,030		2,561.883 ch			6.08%	
Other	3,129,659	3,129,659					45.50%	
	6,878,552		**6,878,552**				**100.00**	**100.00%**
Total			***6,878,552***					

New Jersey Department of Transportation

DATE : 11/15/2013
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JOB ESTIMATE REPORT

JOB NUMBER : 960694 SPEC YEAR: 07
DESCRIPTION: N.J. ROUTE 10 OVER PASSAIC RIVER SUPERSTRUCTURE REPLACEMENT

ITEMS FOR JOB 960694

LINE	ITEM	ALT	UNITS	DESCRIPTION	QUANTITY	PRICE	AMOUNT
0001	151006M		DOLL	PERFORMANCE BOND AND PAYMENT BOND	1.000	27000.00	27000.00
0002	152004P		DOLL	OWNER'S AND CONTRACTOR'S PROTECTIVE LIAB	1.000	10000.00	10000.00
0003	152015P		DOLL	POLLUTION LIABILITY INSURANCE	1.000	10000.00	10000.00
0004	153003P		LS	PROGRESS SCHEDULE	1.000	10000.00	10000.00
0005	153006P		U	PROGRESS SCHEDULE UPDATE	2.000	500.00	1000.00
0006	153012P		HOUR	TRAINEES	300.000	1.00	300.00
0007	154003P		LS	MOBILIZATION	1.000	380000.00	380000.00
0008	155009M		U	FIELD OFFICE TYPE C SET UP	1.000	25000.00	25000.00
0009	155027M		MO	FIELD OFFICE TYPE C MAINTENANCE	10.000	3400.00	34000.00
0010	157004M		DOLL	CONSTRUCTION LAYOUT	1.000	45000.00	45000.00
0011	158009M		LF	HEAVY DUTY SILT FENCE, ORANGE	1944.000	7.00	13608.00
0012	158030M		U	INLET FILTER TYPE 2, 2' X 4'	27.000	124.00	3348.00
0013	158033M		U	INLET FILTER TYPE 2, 4' X 4'	8.000	165.00	1320.00
0014	158045M		LF	FLOATING TURBIDITY BARRIER, TYPE 2	312.000	20.00	6240.00
0015	158055M		U	SEDIMENT CONTROL BAG	1.000	700.00	700.00
0016	158063P		LS	CONCRETE WASHOUT SYSTEM	1.000	1000.00	1000.00
0017	158072M		U	OIL ONLY EMERGENCY SPILL KIT, TYPE 1	2.000	800.00	1600.00
0018	159003M		U	BREAKAWAY BARRICADE	20.000	65.00	1300.00
0019	159006M		U	DRUM	140.000	45.00	6300.00
0020	159009M		U	TRAFFIC CONE	100.000	12.00	1200.00
0021	159012M		SF	CONSTRUCTION SIGNS	1750.000	11.00	19250.00
0022	159015M		U	CONSTRUCTION IDENTIFICATION SIGN, 4' X 8	2.000	1200.00	2400.00
0023	159021P		LF	CONSTRUCTION BARRIER CURB	1500.000	50.00	75000.00
0024	159027M		U	FLASHING ARROW BOARD, 4' X 8'	2.000	2500.00	5000.00
0025	159030M		U	PORTABLE VARIABLE MESSAGE SIGN	8.000	10000.00	80000.00
0026	159051M		U	TEMPORARY CRASH CUSHION, INERTIAL BARRIE	1.000	11000.00	11000.00
0027	159108M		U	TRAFFIC CONTROL TRUCK WITH MOUNTED CRASH	2.000	12500.00	25000.00
0028	159114M		LF	REMOVABLE BLACK LINE MASKING TAPE, 6"	16000.000	2.00	32000.00
0029	159120M		LF	TEMPORARY PAVEMENT MARKING TAPE, 4"	900.000	1.00	900.00
0030	159126M		LF	TEMPORARY TRAFFIC STRIPES, 4"	36000.000	0.25	9000.00
0031	159132M		SF	TEMPORARY PAVEMENT MARKINGS	650.000	3.00	1950.00
0032	159141M		HOUR	TRAFFIC DIRECTOR, FLAGGER	120.000	95.00	11400.00
0033	160004M		DOLL	FUEL PRICE ADJUSTMENT	1.000	1400.00	1400.00
0034	160007M		DOLL	ASPHALT PRICE ADJUSTMENT	1.000	5700.00	5700.00
0035	161003P		LS	FINAL CLEANUP	1.000	7500.00	7500.00
0036	201003P		LS	CLEARING SITE	1.000	40000.00	40000.00
0037	202006M		CY	EXCAVATION, TEST PIT	10.000	265.00	2650.00
0038	202009P		CY	EXCAVATION, UNCLASSIFIED	755.000	30.00	22650.00
0039	202021P		SY	REMOVAL OF PAVEMENT	1156.000	28.00	32368.00

JOB ESTIMATE REPORT

0040	203054M	CY	FLOWABLE CONCRETE FILL	5.000	28.00	140.00
0041	301006P	CY	SUBBASE	337.000	40.00	13480.00
0042	302036P	SY	DENSE-GRADED AGGREGATE BASE COURSE, 6" T	1160.000	13.00	15080.00
0043	304006P	SY	CONCRETE BASE COURSE, 9" THICK	97.000	108.00	10476.00
0044	401009P	SY	HMA MILLING, 3" OR LESS	18927.000	3.00	56781.00
0045	401021M	SY	HOT MIX ASPHALT PAVEMENT REPAIR	100.000	38.00	3800.00
0046	401027M	LF	POLYMERIZED JOINT ADHESIVE	16500.000	0.25	4125.00
0047	401030M	GAL	TACK COAT	2049.000	1.50	3073.50
0048	401036M	GAL	PRIME COAT	373.000	2.50	932.50
0049	401048M	T	HOT MIX ASPHALT 9.5 M 76 SURFACE COURSE	1200.000	110.00	132000.00
0050	401060M	T	HOT MIX ASPHALT 12.5 M 76 SURFACE COURSE	1197.000	100.00	119700.00
0051	401078M	T	HOT MIX ASPHALT 12.5 M 76 INTERMEDIATE C	188.000	95.00	17860.00
0052	401099M	T	HOT MIX ASPHALT 25 M 64 BASE COURSE	437.000	135.00	58995.00
0053	401108M	U	CORE SAMPLES, HOT MIX ASPHALT	5.000	150.00	750.00
0054	453006M	SY	FULL DEPTH CONCRETE PAVEMENT REPAIR, HMA	420.000	220.00	92400.00
0055	501009P	LS	TEMPORARY COFFERDAM	1.000	5000.00	5000.00
0056	601122P	LF	15" REINFORCED CONCRETE PIPE	102.000	74.00	7548.00
0057	601249P	LF	6" HIGH DENSITY POLYETHYLENE PIPE	45.000	40.00	1800.00
0058	601404P	LF	SUBBASE OUTLET DRAIN	106.000	40.00	4240.00
0059	602012M	U	INLET, TYPE B	7.000	2750.00	19250.00
0060	602099M	U	RESET EXISTING CASTING	37.000	490.00	18130.00
0061	602210M	U	BICYCLE SAFE GRATE	2.000	430.00	860.00
0062	602213M	U	CURB PIECE	11.000	360.00	3960.00
0063	602290M	U	INLET, NON-STANDARD SEE CONSTRUCTION DETAILS	3.000	5100.00	15300.00
0064	603103P	CY	RIPRAP STONE SCOUR PROTECTION (D50=12")	22.000	115.00	2530.00
0065	605209P	LF	ORNAMENTAL FENCE	95.000	55.00	5225.00
0066	605212P	LF	RESET FENCE	85.000	35.00	2975.00
0067	606024P	SY	CONCRETE SIDEWALK, REINFORCED, 6" THICK	50.000	65.00	3250.00
0068	606039P	SY	HOT MIX ASPHALT DRIVEWAY, 6" THICK	106.000	58.00	6148.00
0069	606075P	SY	CONCRETE ISLAND, 4" THICK	24.000	105.00	2520.00
0070	606084P	SY	DETECTABLE WARNING SURFACE	4.000	290.00	1160.00
0071	607018P	LF	9" X 16" CONCRETE VERTICAL CURB	767.000	25.00	19175.00
0072	607030P	LF	12" X 13" CONCRETE SLOPING CURB	112.000	25.00	2800.00
0073	607039P	LF	24" X 35" CONCRETE BARRIER CURB, DOWELLE	1279.000	450.00	575550.00
0074	607076P	LF	BELGIAN BLOCK CURB	420.000	35.00	14700.00
0075	608003P	SY	NONVEGETATIVE SURFACE, HOT MIX ASPHALT	233.000	26.00	6058.00
0076	609003M	LF	BEAM GUIDE RAIL	232.000	15.00	3480.00
0077	609024M	U	FLARED GUIDE RAIL TERMINAL	2.000	2000.00	4000.00
0078	609039M	U	BEAM GUIDE RAIL ANCHORAGE	3.000	750.00	2250.00
0079	609075M	LF	REMOVAL OF BEAM GUIDE RAIL	268.000	2.00	536.00
0080	610003M	LF	TRAFFIC STRIPES, 4"	16785.000	0.30	5035.50
0081	610006M	LF	TRAFFIC STRIPES, 6"	1070.000	0.70	749.00
0082	610009M	SF	TRAFFIC MARKINGS	932.000	9.00	8388.00
0083	610012M	U	RPM, MONO-DIRECTIONAL, WHITE LENS	72.000	27.00	1944.00

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0084	610018M	U	RPM, MONO-DIRECTIONAL, AMBER LENS	72.000	27.00	1944.00
0085	610024M	U	REMOVAL OF RPM	144.000	22.00	3168.00
0086	610036M	LF	REMOVAL OF TRAFFIC STRIPES	11100.000	0.50	5550.00
0087	610039M	SF	REMOVAL OF TRAFFIC MARKINGS	30.000	3.50	105.00
0088	611348M	U	CRASH CUSHION, LOW MAINTENANCE, COMPRESS	2.000	21000.00	42000.00
0089	612003P	SF	REGULATORY AND WARNING SIGN	70.000	36.00	2520.00
0090	612030P	SF	OVERHEAD STREET NAME SIGNS	6.000	70.00	420.00
0091	651255M	U	RESET WATER VALVE BOX	8.000	200.00	1600.00
0092	654007P	LS	ELECTRICAL UTILITY RELOCATION, _________ JCP&L	1.000	15000.00	15000.00
0093	702033P	LF	TRAFFIC SIGNAL CABLE, 10 CONDUCTOR	446.000	4.00	1784.00
0094	702036M	U	TRAFFIC SIGNAL HEAD	4.000	2000.00	8000.00
0095	702039M	U	PEDESTRIAN SIGNAL HEAD	2.000	1000.00	2000.00
0096	702042M	U	PUSH BUTTON	2.000	200.00	400.00
0097	702054M	LS	TEMPORARY TRAFFIC SIGNAL SYSTEM, LOCATIO 1	1.000	60000.00	60000.00
0098	702054M	LS	TEMPORARY TRAFFIC SIGNAL SYSTEM, LOCATIO 2	1.000	70000.00	70000.00
0099	702054M	LS	TEMPORARY TRAFFIC SIGNAL SYSTEM, LOCATIO 3	1.000	150000.00	150000.00
0100	702057M	LS	INTERIM TRAFFIC SIGNAL SYSTEM, LOCATION 1	1.000	11000.00	11000.00
0101	804006P	SY	TOPSOILING, 4" THICK	423.000	5.00	2115.00
0102	806006P	SY	FERTILIZING AND SEEDING, TYPE A-3	423.000	5.00	2115.00
0103	809003M	SY	STRAW MULCHING	423.000	4.00	1692.00
0104	809018M	SY	WOOD MULCHING	80.000	6.00	480.00
0105	811069M	U	EVERGREEN SHRUB, 36-42" HIGH, B&B	9.000	90.00	810.00
0106	811075M	U	EVERGREEN SHRUB, 24-30" HIGH, B&B	21.000	80.00	1680.00
0107	201006P	LS	CLEARING SITE, BRIDGE (___) STAGE 1, 1402-153	1.000	55000.00	55000.00
0108	201006P	LS	CLEARING SITE, BRIDGE (___) STAGE 2, 1402-153	1.000	55000.00	55000.00
0109	201039P	LS	TEMPORARY SHIELDING	1.000	56000.00	56000.00
0110	203009P	CY	I-9 SOIL AGGREGATE	20.000	75.00	1500.00
0111	504006P	LB	REINFORCEMENT STEEL, EPOXY-COATED	26030.000	1.50	39045.00
0112	504024P	CY	CONCRETE ABUTMENT WALL	10.000	1200.00	12000.00
0113	504030P	CY	CONCRETE PIER SHAFT	9.000	1000.00	9000.00
0114	504032P	CY	CONCRETE DIAPHRAGM, HPC	49.000	2150.00	105350.00
0115	504036P	SY	EPOXY WATERPROOFING	49.000	50.00	2450.00
0116	505064P	SF	PREFAB PRESTRESS CONC SUPER UNIT	6886.000	65.00	447590.00
0117	506003P	LS	STRUCTURAL STEEL	1.000	3000.00	3000.00
0118	506006P	U	REINFORCED ELASTOMERIC BEARING ASSEMBLY	84.000	2000.00	168000.00
0119	507015P	LF	STRIP SEAL EXPANSION JOINT ASSEMBLY	114.000	300.00	34200.00
0120	507023P	CY	CONCRETE BRIDGE APPROACH, HES	52.000	750.00	39000.00
0121	507025P	CY	CONCRETE BRIDGE DECK, HES	77.000	750.00	57750.00
0122	507033P	CY	CONCRETE BRIDGE SIDEWALK, HPC	42.000	600.00	25200.00
0123	507039P	LF	CONCRETE BRIDGE PARAPET, HPC	224.000	225.00	50400.00
0124	507048M	LF	24" BY 32" CONCRETE BARRIER CURB, BRIDGE	107.000	135.00	14445.00
0125	507066P	SY	PRECAST CONCRETE BRIDGE APPROACH	296.000	600.00	177600.00
0126	509003P	LF	BRIDGE RAILING (1 RAIL, ALUMINUM)	205.000	100.00	20500.00
0127	520003P	U	PERMANENT GROUND ANCHOR	18.000	13200.00	237600.00

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0128	520006P	U	GROUND ANCHOR PERFORMANCE LOAD TEST	4.000	1500.00	6000.00
0129	555003M	SF	SUBSTRUCTURE CONCRETE REPAIR	115.000	240.00	27600.00
0130	701021P	LF	3" RIGID METALLIC CONDUIT	560.000	35.00	19600.00

ITEM TOTAL	4286451.50
INFLATED ITEM TOTAL	4286451.50

TOTALS FOR JOB 960694

ESTIMATED COST:	4286451.50
CONTINGENCY PERCENT (0.0):	0.00
ESTIMATED TOTAL:	4286451.50

Exhibit E4- ABC Conceptual Cost Estimate Survey Responses

State	Initial ABC Questions					
	In your experience, did you conclude that conventional or prefabricated bridges were more cost efficient?	**When would it be advised to apply a different method than Design-Bid-Build contracting method?**	**Which portion of ABC did you find to be the most costly through your estimation research?**	**What were the contractors biggest concerns when it came to ABC construction costs?**	**How did the federal and state requirements affect the cost and duration of the project?**	**What environmental factors play a significant role in ABC construction and how do they affect the overall cost of projects?**
Louisiana	This is a very general question, would depend on the scope of the project and the bridge type used	If early contractor input is warranted based on a high level of risk such as MOT issues on high ADT routes and/or the acceleration of large projects or programs.	MOT and traffic management plans to meet the ABC requirements, accelerated schedules that increase costs.	Risks to meet accelerated project schedules.	FHWA now requires a traffic management plant or TMP based on the project scope and can affect the contractor’s schedule.	MOT concerns with local communities.
Utah	Costs are defined based on the goals of the project.	ABC is evaluated for all projects within all contracting methods.	ABC costs are dependent on the project.	Risks to meet accelerated project schedules.	FHWA now requires a traffic management plant or TMP based on the project scope and can affect the contractor’s schedule.	Environmental constraints make ABC a viable option for projects - limiting onsite construction time.

State	Initial ABC Questions					
	In your experience, did you conclude that conventional or prefabricated bridges were more cost efficient?	**When would it be advised to apply a different method than Design-Bid-Build contracting method?**	**Which portion of ABC did you find to be the most costly through your estimation research?**	**What were the contractors biggest concerns when it came to ABC construction costs?**	**How did the federal and state requirements affect the cost and duration of the project?**	**What environmental factors play a significant role in ABC construction and how do they affect the overall cost of projects?**
Hawaii	If the contractor has adequate time with no restrictions on opening the bridge to traffic, conventional construction techniques may be more cost effective. Prefabrication of girders has been proven to be cost effective because it can eliminate forming and shoring. However, prefabricating bridges and other components of bridges to minimize construction time and inconveniences to the traveling public is not necessarily cost effective. The major benefit is that the bridge can be opened to traffic faster thereby inconveniencing the public less.	In emergencies and other situations where a highway needs to be opened to traffic very quickly as a result of a catastrophic event or complete closure of a highway or bridge for construction, etc.	Not available.	Finding qualified precasting fabricators is always a problem in Hawaii. We have only one fabricator that is certified and only for specific products.	No significant effect	Permits and environmental clearances play a major part in any project. In some cases, use of ABC techniques such as prefabrication of girders can eliminating permits required if constructing within a stream by completely spanning over the affected area saving some cost in design and construction.
New Jersey #1	Conventional bridges are more cost efficient, because you have more contractors bidding on the jobs, you do not need large cranes, or special materials for the closure pours.	NJDOT uses ABC construction, A+B Bidding and Incentive/Disincentive provisions only when the anticipated traffic impacts are significant and cannot be staged or detoured permanently.	Production and Erection of Pre-Cast Structure was most costly on my project.	It was critical for the contractors' to have the best estimate, be awarded & monitor the job closely, effective planning and timely execution of ABC structural members.	The federal/state government encourages using ABC technique where it is cost-effective and benefits to road users are significant. NJDOT's priority is minimizing traffic impacts and construction duration at a reasonable additional cost.	Seasonal restrictions to construct ABC bridge must be clearly defined in contract documents and at times will cost a little bit more when the construction window is narrow.

State	Initial ABC Questions					
	In your experience, did you conclude that conventional or prefabricated bridges were more cost efficient?	**When would it be advised to apply a different method than Design-Bid-Build contracting method?**	**Which portion of ABC did you find to be the most costly through your estimation research?**	**What were the contractors biggest concerns when it came to ABC construction costs?**	**How did the federal and state requirements affect the cost and duration of the project?**	**What environmental factors play a significant role in ABC construction and how do they affect the overall cost of projects?**
New Jersey #2	On our project, I believe the ABC bridges were very competitive in terms of cost to conventional construction. We replaced three single-span bridges in three separate weekends. Costs for the Maintenance and Protection of Traffic over a long duration were avoided. Traffic impacts to motorists were during a weekend period that only impacted discretionary traffic and not the high volume workday traffic. The design of certain ABC or precast composite structures/bridges can actually be less costly than a conventional bridge because the precast manufacturer completes much of the design.	We used A+B Bidding (Incentive/Disincentive) on the Route 1 Freeway ABC projects because it allowed the Department to obtain bids that considered the construction duration/schedule. On each of the three bridges, the contractor beat the deadline and opened the three bridges ahead of schedule, which was important for the heavily travel and congested Route 1 Corridor.	The cost of the precast structure was the most costly part of the ABC effort on my project.	The only major concern of the contractor on my ABC project was the penalty or disincentive of not meeting the deadline in opening the roadway back up to traffic in time.	On my project, the federal and state requirements were typically the same whether utilizing conventional construction or ABC construction. My team did not see any significant difference.	ABC construction can be a benefit in reducing the duration of impact on environmental factors. More and more of the environmental regulations tend to impact the construction season and the time available for construction. ABC construction can be done in a fraction of the time of conventional construction. This means ABC construction can be completed, in a narrow environmental window, without the need for additional mobilizations from the contractor. Completion of construction in a short time can also reduce the risk or delay claims when the project is constructed over more than one construction season.
California	When considering just structure costs conventional methods tend to be less expensive unless the site is a long distance from the nearest batch plant, in which case the precast alternative can be less expensive. However, one must consider costs of the overall project to properly evaluate the most cost efficient alternative.	Projects that present challenges in the area of constructability, staging, and constrained work windows would benefit from the innovation and practical feedback delivered by the Contract Manager/General Contractor (CM/GC) method.	In the case of the 3 main ABC methods (precast, slide-in, and large bridge moves) we have found, based on limited experience in state and through evaluation national data, that large bridge moves are the most expensive, followed by slide-in, then precast.	The California bridge construction industry is built upon cast-in-place concrete bridges. Contractors have invested heavily over the years in training their labor force and purchasing forms and false work for this type of construction.	We do not have information on this topic.	Wetland mitigation plays a large role in ABC construction. Wetlands impacted by conventional construction must be mitigated by up to a 10:1 ratio (for each square foot of wetland impacted, 10 square feet must be developed elsewhere) and then the new wetlands must be monitored for years after the project is complete.

State	Initial ABC Questions					
	In your experience, did you conclude that conventional or prefabricated bridges were more cost efficient?	**When would it be advised to apply a different method than Design-Bid-Build contracting method?**	**Which portion of ABC did you find to be the most costly through your estimation research?**	**What were the contractors biggest concerns when it came to ABC construction costs?**	**How did the federal and state requirements affect the cost and duration of the project?**	**What environmental factors play a significant role in ABC construction and how do they affect the overall cost of projects?**
Maryland	Maryland does not have much cost data comparing the two. Our limited information does conclude that prefabricated elements tend to be more expensive. The decision to use them, however, is based on many other factors such as time and durability. ***(Maryland's typical ABC project is the use of prestressed slab to replace small rural bridges. The typical cost for this type of bridge is $225 per square foot.	When design time is limited, design-bid –build offers an advantage of being able to be under construction and designing at the same time.	Don't have enough information to respond to this.	Time constraints is the biggest concern. ABC methods are often used with extremely time constrained projects, which often include large monetary penalties for not meeting deadlines. ABC methods are often new to contractors and they are unsure how to handle the risk in the bidding process.	Some ABC methods / technologies are proprietary so it is difficult to get exactly what you want since sole sourcing is not allowed. This is an area where design / build can be advantageous.	Maryland is often cautious to try new methods of construction and new materials when there is uncertainty of how it will perform long term under environmental influences.
Iowa	The direct cost of prefabricated (ABC) bridges tend to be more expensive but they become more cost efficient if you consider the indirect cost (cost incurred by the traveling public).	Tight overall schedule (design and construction) would make Design-Build method more favorable if allowed by state laws.	N/A	Very short schedule requires contractors to anticipate high dollar liquidated damages in their bids. Also, having subcontractors (precasters) take significant amount of their work.	Not a factor.	Environmental issues can be minimized by ABC methods, so they do not play a significant role. However, this is always on a case by case basis.

State	Initial ABC Questions					
	In your experience, did you conclude that conventional or prefabricated bridges were more cost efficient?	**When would it be advised to apply a different method than Design-Bid-Build contracting method?**	**Which portion of ABC did you find to be the most costly through your estimation research?**	**What were the contractors biggest concerns when it came to ABC construction costs?**	**How did the federal and state requirements affect the cost and duration of the project?**	**What environmental factors play a significant role in ABC construction and how do they affect the overall cost of projects?**
Florida	This depends on many factors. The size of the bridge is a major factor. The larger the bridge the more repetition and thus the most cost efficient it is to use prefabricated elements. It is almost impossible to be cost effective with unique prefabricated elements on small structures. Using standardized prefabricated elements across the state on numerous projects does aid greatly in making prefabricated elements more cost effective on small projects.	In Florida, almost all projects cost more using ABC methods. I believe you will find this is true of projects across many states. The real cost savings is in the reduction in what is called user delay costs. The costs of sitting in traffic and moving slower. This is frequently more costs than he actual construction costs increases for using ABC. ABC is best utilized when there is an overall cost savings approach realized, not just construction dollars.	In Florida, almost all projects cost more using ABC methods. I believe you will find this is true of projects across many states. The real cost savings is in the reduction in what is called user delay costs. The costs of sitting in traffic and moving slower. This is frequently more costs than he actual construction costs increases for using ABC.	RISK. Many ABC projects contain incentive/disincentive clause and they usually bid assuming the incentive will be realized. If something delays the project the risk in loosing the incentive and pay disincentives is a real issue.	This varies on every project and there is no good single answer.	Many times with ABC methods, the environmental exposure is decreased either with more efficient construction methods or shorter periods of disturbance. For large waterborne projects, the cost is usually less than using conventional methods.

State	Revised Questions					
	Based on your experience with ABC, are prefabricated bridges more costly than conventional bridges?	Have you ever used another contracting method besides Design-Bid-Build for ABC? If yes, under what circumstances?	What ABC elements have been more costly than conventional bridge construction?	Have any of your contractors had cost concerns when using ABC?	Did federal and/or state requirements affect the overall ABC cost/duration of the project? If yes, how?	What, if any, environmental factors/policies affected your ABC projects? How did these affect the overall cost?
Michigan	Yes. A little, but hard to place hard values on. Probably about 10 more per project.	Yes, CMGC	Elements we have done a precast abutment walls, pier columns, pier caps, decked beams. I can not say that any of these was significantly more costly than others.	If the project progress schedule would make cast in place construction feasible, the contractor will value engineer the project to remove PBES.	No	No
South Dakota	Not a lot of experience with ABC but we would say yes.	No	Pre-cast deck units and pre-cast sleeper slabs for us	No	No	None on our limited projects
Minnesota	Yes	Yes. CMGC and Design Build. We evaluate the characteristics of each project to determine the contract administration method.	Full depth deck panels. Precast substructure units, inverted tee superstructures.	Yes, have asked to do the work using conventional methods.	No	Use of precast products in lieu of cast-in-place concrete over water is generally faster and less likely to cause environmental issues, but is more expensive.
Illinois	Our first projects are in plan development so we don't have cost history yet.	No, not yet.	No cost data yet.	No concerns known yet.	No	Not yet

State	Revised Questions					
	Based on your experience with ABC, are prefabricated bridges more costly than conventional bridges?	**Have you ever used another contracting method besides Design-Bid-Build for ABC? If yes, under what circumstances?**	**What ABC elements have been more costly than conventional bridge construction?**	**Have any of your contractors had cost concerns when using ABC?**	**Did federal and/or state requirements affect the overall ABC cost/duration of the project? If yes, how?**	**What, if any, environmental factors/policies affected your ABC projects? How did these affect the overall cost?**
Missouri	Yes	Yes. We did a single contract design-build project to replace 554 bridges across the state. One of the main goals was to build the replacement bridges quickly.	We haven't identified individual elements. We just see an increase in all the pay items.	We don't have enough experience to answer this one.	No federal requirements. We have had a few isolated incidents where we were willing to pay a higher price for faster bridge construction to limit the number of days of head-to-head traffic on interstate.	N/A
North Dakota	We have no direct experience with ABC construction.	N/A	N/A	N/A	N/A	N/A
Kansas	Yes, our attempt to let an ABC bridge project using Prefabricated Bridge Elements cost more than $1 million more than a conventional bridge.	No. KDOT is prohibited by Kansas law from using Design-Build or CM/GC. (There was on special exception made for a large interchange project in Kansas City to use Design-Build.)	Precast columns; precast pier caps; precast abutment grade beams; precast deck sections.	We attempted one ABC bridge project and let it twice. Both times, the cost of the ABC bridge was more than the cost of the conventional + local detour. The contractors were "concerned" that we did not go ahead and award the bids. We rejected both bids and are in the process of redesigning the bridge to use conventional construction.	The one ABC project was attempted was financed in a conventional manner with a mix of sate and federal funds, without any special grants. KDOT chose a schedule based on local concerns and a reasonable traffic closure (30 days) for the ABC methods employed.	No environmental factors weighed on the ABC project we attempted.

State	Revised Questions					
	Based on your experience with ABC, are prefabricated bridges more costly than conventional bridges?	**Have you ever used another contracting method besides Design-Bid-Build for ABC? If yes, under what circumstances?**	**What ABC elements have been more costly than conventional bridge construction?**	**Have any of your contractors had cost concerns when using ABC?**	**Did federal and/or state requirements affect the overall ABC cost/duration of the project? If yes, how?**	**What, if any, environmental factors/policies affected your ABC projects? How did these affect the overall cost?**
Pennsylvania	Based on typical unit bridge construction costs we typically spend about $250/SF but a prefabricated ABC bridge is around $450/sf.	A. Recently completed 581 project in Harrisburg, we bid that superstructure replacement with ABC as a Design Build. This project was recently presented via the FIU WebEx. B. A couple of years ago a contractor submitted a value engineering proposal to change a stage construction bridge to an all precast ABC bridge.	Primarily the precast pieces for full height abutments. These full height abutment pieces are relatively heavy and thus require large cranes. The cost to rent a large crane to set the pieces is a significant cost factor.	Contractor's has various concerns with ABC projects. The primary issue is the very tight timeframes to complete the project and the risk of liquated damages if the project is not completed per the contract schedule.	One issue is the various rules in determining/calculating the "liquidated damages" if the contract completion date is exceeded. The rules in calculating liquidated damages are such that for low volume roads the cost for liquidated damages is minimal thus contractors ignore the use of ABC and go conventional by simply including in the contract bid the costs for liquidated damages. Thus, the rules for calculating liquidated damages need to be revised.	Not aware of any environmental issues.

APPENDIX F

ABC DECISION-MAKING TOOLS

APPENDIX F - ABC DECISION-MAKING TOOLS

% Weight	Category	Decision-Making-Item	Possible Points	Points Allocated		Scoring Guidance
17%	Disruptions (on/under Bridge)	Railroad on Bridge?	8		0	No
					4	Yes: Little Traffic
					8	Yes: Heavy Traffic
		Railroad Under Bridge?	3		0	No
					1	Yes: Minor Railroad Track
					3	Yes: Major Railroad Track
		Over Navigation Channel that needs to remain open?	6		0	No
					3	Yes: Minor amount of traffic
					6	Yes: Considerable amount of traffic
8%	Urgency	Emergency Replacement?	8		0	No
					4	Yes: Minor Roadway
					8	Yes: Major Roadway
23%	User Costs and Delays	ADT and/or ADTT	6		0	No Traffic Impacts
		(Combined Construction Year ADT on and under bridge)			1	< 10,000
					2	10,000 to 25,000
					3	25,000 to 50,000
					4	50,000 to 75,000

					5	75,000 to 100,000
					6	100,000 or more
		Required Lane Closure/Detours?	6		0	Delay 0-5 minutes
		(Length of Delay to Traveling Public)			1	5-10 minutes
					2	15-25 minutes
					3	25-35 minutes
					4	35-45 minutes
					5	45-55 minutes
					6	55 or more minutes
		Are only Short Term Closures Allowable?	5		0	Available alternatives for staged construction
					3	Available alternatives for staged construction, undesirable
					5	No available alternatives
		Impact to Economy?	6		0	Little to no impact
		(Local Business Access, impact to manufacturing, etc.)			3	Moderate impact
					6	Considerable Impact
14%	Construction Time	Impacts Critical Path of the Total Project?	6		0	Little to no impact
					3	Moderate impact
					6	Considerable Impact

		Restricted Construction Time	**8**		**0**	**No restrictions**
		(Environmental schedules, Economic Impact - e.g. local business access, Holiday schedules, special events, etc.)			**3**	**Minor restrictions**
					6	**Moderate restrictions**
					8	**Considerable restrictions**
5%	**Environment**	**Does ABC mitigate a critical environmental impact or sensitive environmental issue?**	**5**		**0**	**No**
					2	**Minor**
					3	**Several Minor**
					4	**Considerable**
					5	**Several Considerable**
3%	**Cost**	**Compare Comprehensive Construction Costs**	**3**		**0**	**25% or higher than conventional**
		(Compare conventional vs. prefabrication)			**1**	**1% to 25% higher than conventional**
					2	**Equal to conventional**
					3	**Lower than conventional**
18%	**Risk Management**	**Does ABC Allow Management of a Particular Risk?**	**6**		**0-6**	**Determine if risks can be managed through ABC that aren't discussed in other topics**
		Safety (Worker Concerns)	**6**		**0**	**TMP type 1**
					3	**TMP type 2**
					6	**TMP type 3-4**

		Safety (Traveling Public Concerns)	6		0	TMP type 1
					3	TMP type 2
					6	TMP type 3-4
12%	Other	Economy of Scale	5		0	1 span
		(Repetition of components in a bridge or bridges in a project)			1	2 spans
		(Total spans = sum of all spans on all bridges on the project)			2	3 spans
					3	4 spans
					4	5 spans
					5	6 or more spans
		Weather Limitations for Conventional Construction?	2		0	No
					1	Moderate
					2	Considerable
		Use of Typical Standard Details (Complexity)	5		0	No
					3	Some
					5	All details
			Sum of Points:	max.	100	Possible Points

Decision-Making Item	Scoring Guidance Description
Railroad on Bridge?	This is a measure of how railroad traffic on the bridge will be affected by the project. If a major railroad line runs over the bridge that requires minimum closures and a shoo fly (a temporary railroad bridge bypass) cannot be used, provide a high score here. If a railroad line that is rarely used runs over the bridge, consider providing a mid-range or low score here. If there is no railroad on the bridge, assign a value of zero here.
Railroad under Bridge?	This is a measure of how railroad traffic under the bridge will be affected by the project. If a major railroad line runs under the bridge that would disrupt construction progress significantly, provide a high score here. If a railroad track runs under the structure, but it is used rarely enough that it will not disrupt construction progress significantly, provide a low score here. Consider if the railroad traffic is able to be suspended long enough to move a new bridge into place. If there is not a large enough window to move a new bridge into place, SPMT could be eliminated as an alternative for this project. For this case, PBES may be a more applicable alternative. If there is no railroad under the bridge, assign a value of zero here.
Over Navigation Channel that needs to remain open?	This is a measure of how a navigation channel under a bridge will be affected by the project. If a navigation channel is highly traveled and needs to remain open for shipments, provide a high score here. If a navigation channel is rarely traveled and there are not requirements for it to remain open at certain time periods, provide a low score here. If there is no navigation channel under the bridge, assign a value of zero here.
Emergency Replacement?	This is a measure of the urgency of the bridge replacement. A more urgent replacement supports the use of accelerated bridge construction methods, since demolition and construction can be progressing concurrently. Depending on the particular project, accelerated bridge construction methods can also allow multiple components of the bridge to be constructed concurrently. If the bridge replacement is extremely urgent and the bridge can be replaced quicker by using accelerated construction methods, provide a high score here.

ADT and/or ADTT (Construction Year)	This is a measure of the total amount of traffic crossing the bridge site. A higher ADT value at a site will help support the use of accelerated bridge construction methods. Use a construction year ADT value equal to the sum of the traffic on the structure and under the structure. For cases where there is a very high ADT on the bridge and very low or no ADT under the bridge, consider using a "slide" method (on rollers or Polytetrafluorethylene (PTFE)/Elastomeric pads) or SPMT's, which can be very cost effective ABC techniques for this situation. For structures with a higher-than-average percentage of truck traffic, consider providing a higher score than indicated solely by the ADT values in the table.
Required Lane Closures/Detours?	This is a measure of the delay time imposed on the traveling public. If conventional construction methods will provide significant delays to the traveling public, provide a high score here. If conventional construction methods will provide minimal delays to the traveling public, provide a low score here. Use the delay times provided in the table as guidance for scoring.
Are only Short Term Closures Allowable?	This is a measure of what other alternatives are available besides accelerated bridge construction. If staged construction is not an alternative at a particular site, the only alternative may be to completely shut down the bridge for an SPMT move, and therefore a high score should be provided here. If there is a good alternative available for staged construction that works at the site, a low score should be provided here.
Impact to Economy	This is a measure of the impact to the local businesses around the project location. Consider how the construction staging, road closures, etc. will impact local businesses (public access, employee access, etc.) A high impact to the economy equates to a high score here. A low impact to the economy equates to a low score here.
Impacts Critical Path of Total Project?	This is a measure of how the construction schedule of the structure impacts the construction schedule of the entire project. If the construction of the structure impacts the critical path of the entire project, and utilizing ABC methods provides shorter overall project duration, provide

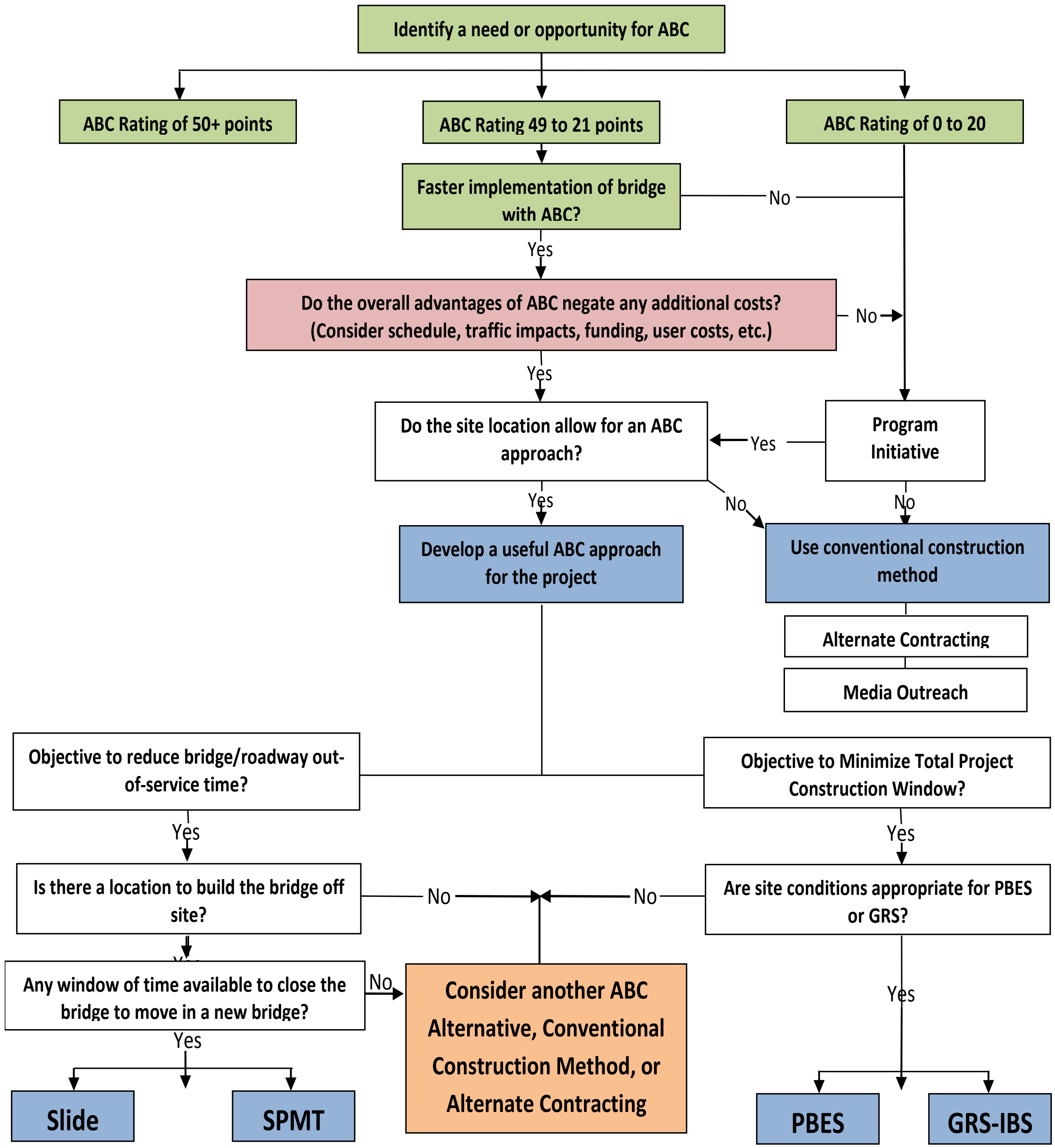
	a high score here. If other project factors are more critical for the overall project schedule and utilizing ABC methods will not affect the overall project duration, provide a low score here.
Restricted Construction Time	This is a measure of how the construction schedule is impacted by environmental and community concerns or requirements. Items to consider are local business access windows, holiday schedules and traffic, special event traffic, etc. If there are significant restrictions on construction schedule, provide a high score here. If there are little to no restrictions on the construction schedule, provide a low score here.

Does ABC mitigate a critical environmental impact or sensitive environmental issue?	This is a measure of how using accelerated bridge construction methods can help mitigate impacts to the environment surrounding the project. Since accelerated methods allow a shorter on-site construction time, the impacts to the environment can be reduced. If the reduced on-site construction time provided by accelerated bridge construction methods mitigates a significant or critical environmental concern or issue, provide a high score here. If there are no environmental concerns that can be mitigated with accelerated construction methods, provide a low score here.
Compare Comprehensive Construction Costs	This is a measure of the complete comprehensive cost difference between conventional construction methods versus using an accelerated bridge construction method. Some costs will increase with the use of accelerated construction methods, such as the cost of the SPMT equipment and the learning curve that will be incorporated into using new technologies. However, some costs will decrease with the use of accelerated construction methods, such as the reduced cost for traffic control, equipment rentals, inspector wages, etc. Many of the reduced costs are a direct result of completing the project in less time. Use the cost comparisons in the table as guidance for scoring here.

Does ABC allow management of a particular risk?	This is an opportunity to add any project-specific items or unique issues that have risk associated with them that are not incorporated into another section in this text. Consider how ABC may or may not manage those particular risks.
Safety (Worker Concerns)	This is a measure of the relative safety of the construction workers between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of workers in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here.
Safety (Traveling Public Concerns)	This is a measure of the relative safety of the traveling public between conventional construction methods and accelerated construction methods. The reduced on-site construction time from using accelerated bridge construction methods reduces the exposure time of the traveling public in a construction zone, thus increasing safety. If a significant increase in safety can be seen by utilizing accelerated construction methods, provide a high score here. If utilizing accelerated construction methods does not provide additional safety, provide a low score here.
Economy of Scale	This is a measure of how much repetition is used for elements on the project, which can help keep costs down. Repetition can be used on both substructure and superstructure elements. To measure the economy of scale, sum the total number of spans that will be constructed on the project. For example, if there are 2 bridges on the project that each have 2 spans, the total number of spans on the project is equal to 4. Use the notes in the table for scoring guidance here.
Weather Limitations for Conventional Construction?	This is a measure of the restrictions that the local weather causes for on-site construction progress. Accelerated bridge construction methods may allow a large portion of the construction to be done in a controlled facility, which helps reduce delays caused by inclement weather (rain, snow, etc.). Depending on the location and the season,

	faster construction progress could be obtained by minimizing the on-site construction time.
Use of Typical Standard Details (Complexity)	This is a measure of the efficiency that can be gained by using standard details that have already been developed and approved. If standard details are used, some errors in the field can be prevented. If new details are going to be created for a project, the contractors will be less familiar with the details and problems may arise during construction that were not considered in the design phase. Use the notes in the table for scoring guidance here.

F1 – Decision Making Flowchart (adapted WisDOT, 2015)



APPENDIX G

ABC TOOLKIT TEMPLATE

APPENDIX G - ABC TOOLKIT TEMPLATE

The summary of ABC toolkit components below can be used as a template of the web-based toolkit.

ABC Components	Contents	Related Chapters from the Final Report
Decision-Making Tool	• Decision-making matrix • Decision-making flowchart	CHAPTER 3 APPENDIX F
Design	• Design concepts • Pre-design examples • Design aides	CHAPTER 4 APPENDIX B APPENDIX H
Construction	• Construction guidelines • Construction flowcharts	CHAPTER 7 APPENDIX C
Risk Analysis	• Risk analysis guidelines • Interactive flowcharts	CHAPTER 5 APPENDIX D
Cost Estimates	• Cost estimates guidelines • Examples of cost estimates	CHAPTER 6 APPENDIX E

APPENDIX H

DESIGN AIDES Using Mathcad

APPENDIX H - DESIGN AIDES Using Mathcad

An analysis of the superstructure can be performed using structural modeling software or computational aide to calculate the design moments, shears, and reactions. However, the SHRP2 design examples just provided the results from finite element analyses. It is desirable that this toolkit can be used without any additional computational supports from other sources. Therefore, this study provided the Mathcad examples to evaluate the maximum design loadings using qBridge software, a Mathcad program developed by Professor Emeritus, Noyan Turkkan at the University of Montana, Canada. Dr. Turkkan granted a permission to use this software for this project. The Authors gratefully acknowledge his permission and support.

H1-Mathcad Examples to calculate the design loadings for concrete decked steel girder examples

1. Design loadings for girders (Design Step 10. Load Results. Case 4)

1) 80ft span
- Design Loads-Steel Girder-80ft-MDC4.xmcd
- Design Loads-Steel Girder-80ft-MDW4.xmcd
- Design Loads-Steel Girder-80ft-MLL4.xmcd
- Design Loads-Steel Girder-80ft-MLL4_neg.xmcd

2) 60ft span
- Design Loads-Steel Girder-60ft-MDC4.xmcd
- Design Loads-Steel Girder-60ft-MDW4.xmcd
- Design Loads-Steel Girder-60ft-MLL4.xmcd
- Design Loads-Steel Girder-60ft-MLL4_neg.xmcd

3) 40ft span
- Design Loads-Steel Girder-40ft-MDC4.xmcd
- Design Loads-Steel Girder-40ft-MDW4.xmcd
- Design Loads-Steel Girder-40ft-MLL4.xmcd
- Design Loads-Steel Girder-40ft-MLL4_neg.xmcd

2. Design loadings for deck (Design Step 21. Load Results)
- Design Loads_Deck_MDC.xmcd
- Design Loads_Deck_MLL.xmcd

Note:
The electronic files of these Mathcad examples are provided through an external hard drive or email.

H2- AASHTO HL-93 Loading

The AASHTO HL-93 loading is a hypothetical live load model proposed by AASHTO for analysis of high bridges. Reason for proposing this live load model is to prescribe a set of loads such that it produces an extreme load effect approximately same as that produced by the exclusion vehicles. Exclusion vehicles were the vehicles above the legal limit but due to grand fathering provision in the state they were allowed to operate routinely.

It has 3 basic live loads for bridges called HL-93 Loading, where H stands for highway and L stands for Loading, developed in 1993.

1. Design Truck
2. Design Tandem
3. Design Lane

1. Design Truck

It is commonly called as HS-20 44 (where H stands for highway, S for semi-trailer, 20 TON weight of the tractor (1st two axles) and was proposed in 1994).

2. Design Tandem

It consists of two axles weighing 25 kips (110 KN) each spaced at 4 ft (1.2 m).

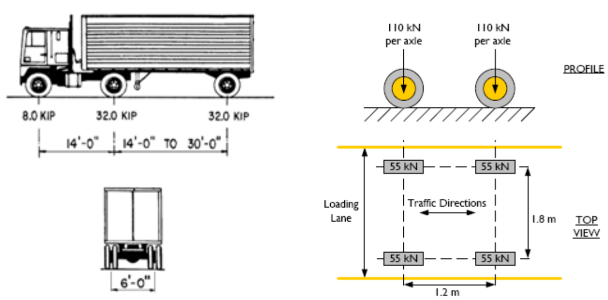


Figure H1. HS-20 truck and tandem loadings

3. Design Lane

It consists of uniformly distributed load of 0.64 kip/ft (9.3 N/mm) and assumed to occupy 10 ft (3 m) transversely.

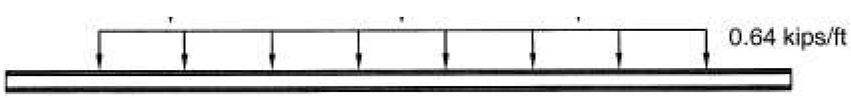


Figure H2. Lane loading

Design loading is the maximum of the three cases:

1. Design Tandem + Design Lane: referred as HL-93M
2. Design Truck + Design Lane: referred as HL-93K
3. 90% of (2 Design Trucks + Design Lane): referred as HL-93S

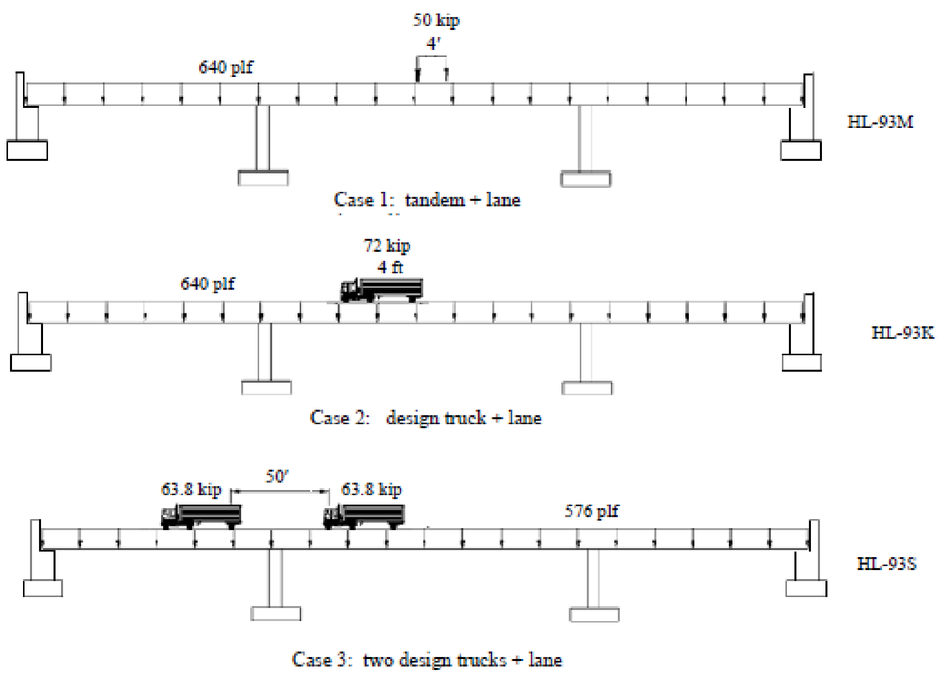


Figure H3. HL-93 load cases.

File Name: Design Load-Steel Girder-80ft-MLL4.xmcd

qBridge

Quick analysis of bridge structures

- User defined moving truck
- Multiple span continuous bridge beams, EI constant
- Moment and shear (absolute value) envelopes
- Support reactions
- Lane load

- This program is written using Mathcad 14.0 M020.
- This program was developed by Professor Emeritus, Noyan Turkkan at the University of Montana. He granted a permission to use this Software for this project. The Authors gratefully acknowledge his permission and support.

USAGE : $\begin{pmatrix} M \\ R \end{pmatrix}$:= qBridge(T,B)

T : Truck definition
B : Bridge definition
M : Vector (4 x 1) - xcoordinates, Vmax, Mmin and Mmax
R : Vector (2 x 1) - Support reactions : Rmin and Rmax

Truck definition : a matrix of nAxles x 2
First column: Axle x-coordinates in ascending order (not axle spacing), beginning with 0 (zero).
Second column: axle weights
Note: there must be at least two (2) axles.

Bridge definition : a matrix of nSpans x 3
First column: span lengths
Second column: uniform load (lane load) on the corresponding span
Third column: number of divisions on a span. Critical values of shears and moments are computed on each division point along the beam.

INPUT

$$HS20 := \begin{pmatrix} 0 & 8\cdot 1.33 \\ 14 & 32\cdot 1.33 \\ 28 & 32\cdot 1.33 \end{pmatrix} \qquad B := \begin{pmatrix} 80 & 0.64 & 10 \\ 80 & 0.64 & 10 \\ 80 & 0.64 & 10 \end{pmatrix}$$

$$TD := \begin{pmatrix} 0 & 25\cdot 1.33 \\ 4 & 25\cdot 1.33 \end{pmatrix}$$

Notes: Truck will move in one direction only. Reverse the truck geometry to simulate moving in the other direction. Consistent units must be used (for example : Kips and feets or kN and m).
When using two or more spans, lane loads should be applied in a checkerboard fashion in order to obtain maximum (or minimum) moments, shears and reactions.
IM (Impact Factor)= 33%
Design values= LLDF*Maximum Values
Where, LLDF= live load distribution factor, and Maximum Values calculated from this MathCAD program

qBridge routines

Example : 3-span bridge beam with HL-93 loading (HS-20 and Tandem)
2 LC shown here. Loa

$$\begin{pmatrix} M1 \\ R1 \end{pmatrix} := \mathrm{qBridge}(HS20, B) \qquad \begin{pmatrix} M2 \\ R2 \end{pmatrix} := \mathrm{qBridge}(TD, B)$$

Loading case 1 envelopes

$x := M1^{\langle 1 \rangle}$ $\quad Vmax1 := M1^{\langle 2 \rangle}$ $\quad Mmin1 := M1^{\langle 3 \rangle}$ $\quad Mmax1 := M1^{\langle 4 \rangle}$

$Rmin1 := R1^{\langle 1 \rangle}$ $\quad Rmax1 := R1^{\langle 2 \rangle}$

Loading case 2 envelopes

$Vmax2 := M2^{\langle 2 \rangle}$ $\quad Mmin2 := M2^{\langle 3 \rangle}$ $\quad Mmax2 := M2^{\langle 4 \rangle}$

$Rmin2 := R2^{\langle 1 \rangle}$ $\quad Rmax2 := R2^{\langle 2 \rangle}$

$n := \mathrm{rows}(x)$ $\quad m := \mathrm{rows}(Rmin1)$

$i := 1 .. n$ $\quad j := 1 .. m$

Resulting envelopes

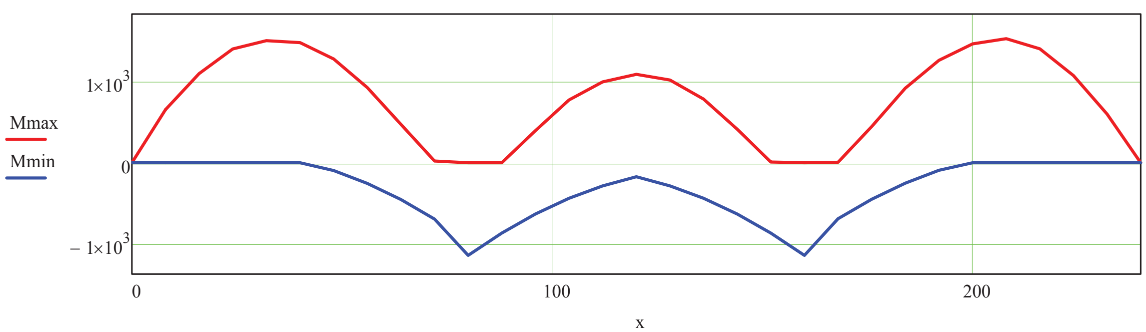
$Vmax_i := \max(Vmax1_i, Vmax2_i)$ $\quad Mmin_i := \min(Mmin1_i, Mmin2_i)$ $\quad Mmax_i := \max(Mmax1_i, Mmax2_i)$

$Rmin_j := \min(Rmin1_j, Rmin2_j)$ $\quad Rmax_j := \max(Rmax1_j, Rmax2_j)$

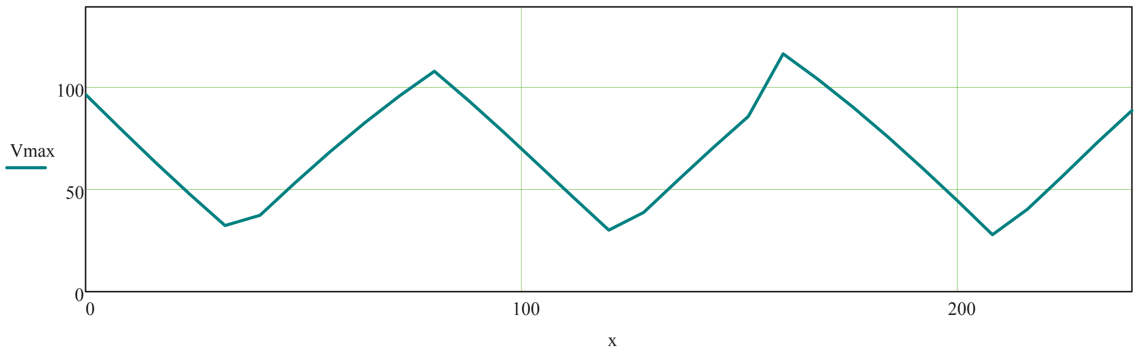
$xh := x_n$ $\quad yv1 := 1.2 \cdot \mathrm{round}(\max(Mmax))$ $\quad yv3 := 1.2 \cdot \mathrm{round}(\max(Vmax))$

$yv2 := 1.2 \cdot \mathrm{round}(\min(Mmin))$

Moment envelopes



Shear envelopes (absolute values)



x =

	1
1	0
2	8
3	16
4	24
5	32
6	40
7	48
8	56
9	64
10	72
11	80
12	88

Mmin =

	1
1	0
2	0
3	0
4	0
5	0
6	0
7	-96
8	-255.4
9	-455.7
10	-697
11	-1146.6
12	-870.2

Mmax =

	1
1	0
2	656.9
3	1102.7
4	1407.7
5	1511.2
6	1486.8
7	1284.6
8	930.6
9	475
10	21.4
11	0
12	0

Vmax =

	1
1	96.3
2	79.6
3	63.2
4	47.4
5	32.3
6	37.3
7	53.1
8	68.2
9	82.4
10	95.6
11	107.7
12	93.1

12	[illegible]
13	96
14	104
15	112
16	...

12	[illegible]
13	-634.7
14	-440.2
15	-286.6
16	...

12	[illegible]
13	396.7
14	776.5
15	1001.3
16	...

12	[illegible]
13	77.7
14	61.8
15	45.8
16	...

Rmin =

	1
1	0
2	0
3	0
4	0

Rmax =

	1
1	96.3
2	150.4
3	150.5
4	88.5

File Name: Design Loads_Deck_MLL.xmcd

qBridge

This program is written using Mathcad 14.0 M020

Quick analysis of bridge structures

- User defined moving truck
- Multiple span continuous bridge beams, EI constant
- Moment and shear (absolute value) envelopes
- Support reactions
- Lane load

USAGE : $\begin{pmatrix} M \\ R \end{pmatrix}$:= qBridge(T,B)

T : Truck definition
B : Bridge definition
M : Vector (4 x 1) - xcoordinates, Vmax, Mmin and Mmax
R : Vector (2 x 1) - Support reactions : Rmin and Rmax

Truck definition : a matrix of nAxles x 2
First column: Axle x-coordinates in ascending order (not axle spacing), beginning with 0 (zero).
Second column: axle weights
Note: there must be at least two (2) axles.

INPUT

$$HS20 := \begin{pmatrix} 0 & 25.54 \\ 6 & 25.54 \\ 12 & 0 \end{pmatrix}$$

$$TD := \begin{pmatrix} 0 & 0 \\ 4 & 0 \end{pmatrix}$$

$$B := \begin{pmatrix} 3 & 0 & 4 \\ 2+\frac{11}{12} & 0 & 4 \\ 2+\frac{11}{12} & 0 & 4 \\ 3+\frac{1}{12} & 0 & 4 \\ 2+\frac{11}{12} & 0 & 4 \\ 3 & 0 & 4 \\ 2+\frac{11}{12} & 0 & 4 \\ 3+\frac{1}{12} & 0 & 4 \\ 2+\frac{11}{12} & 0 & 4 \\ 2+\frac{11}{12} & 0 & 4 \\ 3 & 0 & 4 \end{pmatrix}$$

qBridge routines

Example : 2-span bridge beam with HL-93 loading (HS-20 and Tandem)
2 LC shown here.

$$\begin{pmatrix} M1 \\ R1 \end{pmatrix} := \mathrm{qBridge}(HS20, B) \qquad \begin{pmatrix} M2 \\ R2 \end{pmatrix} := \mathrm{qBridge}(TD, B)$$

Loading case 1 envelopes

$x := M1^{\langle 1 \rangle}$ $\quad$ $Vmax1 := M1^{\langle 2 \rangle}$ $\quad$ $Mmin1 := M1^{\langle 3 \rangle}$ $\quad$ $Mmax1 := M1^{\langle 4 \rangle}$

$Rmin1 := R1^{\langle 1 \rangle}$ $\quad$ $Rmax1 := R1^{\langle 2 \rangle}$

Loading case 2 envelopes

$Vmax2 := M2^{\langle 2 \rangle}$ $\quad$ $Mmin2 := M2^{\langle 3 \rangle}$ $\quad$ $Mmax2 := M2^{\langle 4 \rangle}$

$Rmin2 := R2^{\langle 1 \rangle}$ $\quad$ $Rmax2 := R2^{\langle 2 \rangle}$

$n := \mathrm{rows}(x)$ $\quad$ $m := \mathrm{rows}(Rmin1)$

$i := 1 .. n$ $\quad$ $j := 1 .. m$

Resulting envelopes

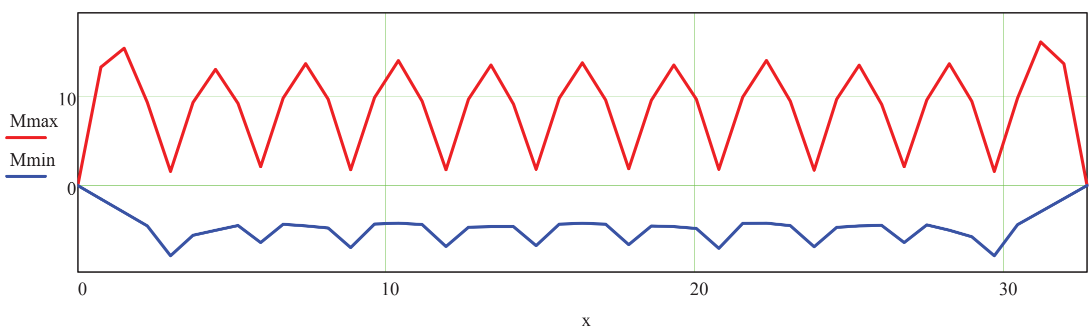
$Vmax_i := max(Vmax1_i, Vmax2_i)$ $Mmin_i := min(Mmin1_i, Mmin2_i)$ $Mmax_i := max(Mmax1_i, Mmax2_i)$

$Rmin_j := min(Rmin1_j, Rmin2_j)$ $Rmax_j := max(Rmax1_j, Rmax2_j)$

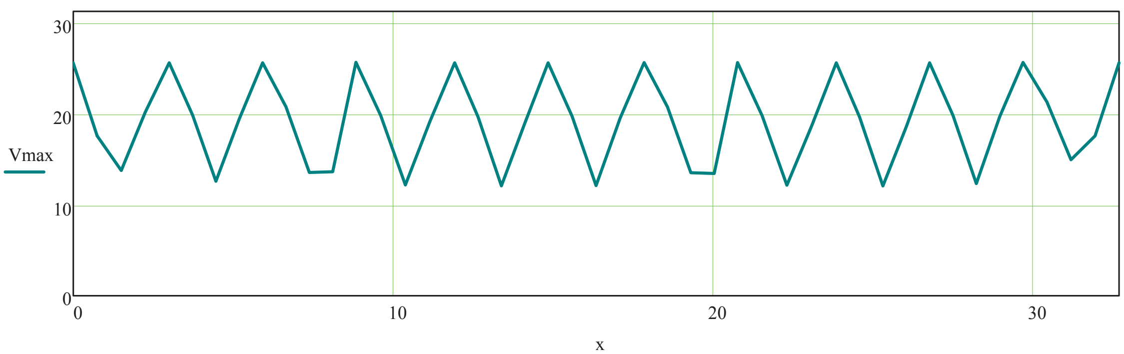
$xh := x_n$ $yv1 := 1.2 \cdot round(max(Mmax))$ $yv3 := 1.2 \cdot round(max(Vmax))$

$yv2 := 1.2 \cdot round(min(Mmin))$

Moment envelopes



Shear envelopes (absolute values)



x =

	1
1	0
2	0.8
3	1.5
4	2.3
5	3
6	3.7
7	4.5
8	5.2
9	5.9
10	6.6
11	7.4
12	8.1

Mmin =

	1
1	0
2	-1.5
3	-3
4	-4.5
5	-7.8
6	-5.5
7	-5
8	-4.4
9	-6.3
10	-4.3
11	-4.5
12	-4.7

Mmax =

	1
1	0
2	13.1
3	15.3
4	9.2
5	1.6
6	9.2
7	12.9
8	9.1
9	2.1
10	9.7
11	13.5
12	9.6

Vmax =

	1
1	25.5
2	17.5
3	13.7
4	20.1
5	25.5
6	19.8
7	12.6
8	19.4
9	25.5
10	20.7
11	13.5
12	13.6

13	8.8
14	9.6
15	10.4
16	...

13	-6.9
14	-4.3
15	-4.2
16	...

13	1.7
14	9.8
15	13.9
16	...

13	25.6
14	19.8
15	12.1
16	...

Rmin =

	1
1	-2
2	-3.2
3	-4.3
4	-3.4
5	-3.6
6	-3.7
7	-3.8
8	-3.6
9	-3.3
10	-4.3
11	-3.2
12	-1.9

Rmax =

	1
1	25.5
2	25.8
3	25.5
4	25.8
5	25.5
6	25.5
7	25.7
8	25.7
9	25.5
10	25.5
11	25.8
12	25.5

Bridge definition : a matrix of nSpans x 3
First column: span lengths
Second column: uniform load (lane load) on the corresponding span
Third column: number of divisions on a span. Critical values of shears and moments are computed on each division point along the beam.

Notes: Truck will move in one direction only. Reverse the truck geometry to simulate moving in the other direction.
Consistent units must be used (for example : Kips and feets or kN and m).
When using two or more spans, lane loads should be applied in a checkerboard fashion in order to obtain maximum (or minimum) moments, shears and reactions.

APPENDIX I

IMPLEMENTATION PLAN

APPENDIX I - IMPLEMENTATION PLAN

The primary implementation of this toolkit is to share its contents with GDOT, LGs, and other local bridge professionals through the technology transfer activities. The proposed toolkit including currently available ABC-related information websites will be used as educational tools accordingly.
Specific technology transfer activities were done as below.

- Presentation at the 2015 National ABC Conference, December 7-8, 2015, in Miami, FL
- Presentation at the SHRP2 R04 Peer-to-Peer Workshop, November 18, Atlanta, GA

In the future, using feedback obtained from GDOT, ABC Conference, and SHRP2 R04 Peer-to-Peer Workshop, the research team can develop specific implementation tasks such as outreach and training, including workshops, webinars, peer exchanges and demonstration projects for local governments in GA.