

TR202505 – Simple for Dead, Continuous for Live Load Steel Superstructures



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16. Abstract This report documents research conducted to support the development of design guidance and standard details for Simple for Dead Load, Continuous for Live Load (SDCL) steel superstructures in Missouri. The research evaluated the dead- and live-load behavior of SDCL steel bridges through a literature review, field testing of Bridge A9513, finite element modeling, degree-of-continuity (DOC) evaluations, and development of MoDOT Engineering Policy Guide provisions. The test bridge developed measurable continuity during deck placement, with construction-stage DOC values ranging from approximately 23% to 42%, indicating that it did not behave as a purely simple-span system during deck casting. After completion, the bridge developed a live-load DOC of approximately 92%. Parametric analyses showed that the proposed deck reinforcement and compression block details remained well below yield limits under factored superimposed dead and HL-93 live loading while providing substantial live-load continuity across multiple bridge geometries. Recommendations include using staged analysis, treating the intermediate bent as the critical force-transfer region, considering pinned diaphragms and bond breaker plates to reduce dead-load continuity during deck placement, and refining MoDOT's SDCL guidance as additional field data become available.			
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List of Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ABC	Accelerated Bridge Construction
AISC	American Institute of Steel Construction
ASTM	American Society for Testing and Materials
CAD	Computer-Aided Design
CDOT	Colorado Department of Transportation
DIC	Digital Image Correlation
DOC	Degree of Continuity
DOT	Department of Transportation
EPG	Engineering Policy Guide
FDOT	Florida Department of Transportation
FE	Finite Element
FHWA	Federal Highway Administration
FIU	Florida International University
HL-93	AASHTO LRFD Design Vehicular Live Load
H&S	Horner & Shifrin
IB	Intermediate Bent
ksi	Kilopound per square inch
LC	Load Case
LRFD	Load and Resistance Factor Design
LVDT	Linear Variable Differential Transformer
MassDOT	Massachusetts Department of Transportation
MoDOT	Missouri Department of Transportation
NaBRO	National Bridge Research Organization
NCHRP	National Cooperative Highway Research Program
NSBA	National Steel Bridge Alliance
NYSDOT	New York State Department of Transportation
ODOT	Oregon Department of Transportation
PBU	Prefabricated Bridge Unit
SDCL	Simple for Dead, Continuous for Live Load
SMC	Simple-Made-Continuous
SPOT	String Potentiometer
ST	Strain Transducer
TRID	Transport Research International Documentation
TxDOT	Texas Department of Transportation
UHPC	Ultra-High-Performance Concrete
UTC	University Transportation Center
VDOT	Virginia Department of Transportation

Executive Summary

This report documents the results of research conducted to support the development of design guidance and standard details for Simple for Dead Load, Continuous for Live Load (SDCL) steel superstructures for use in Missouri. SDCL steel bridges are intended to behave primarily as simple spans during construction-stage dead loading and as continuous structures for superimposed dead load and live load after the concrete deck and intermediate diaphragm region have hardened. This approach can reduce fabrication and construction complexity by eliminating or reducing bolted field splices, while still providing the structural and serviceability benefits of continuity in the completed bridge.

The overall objective of the research was to evaluate the behavior of SDCL steel bridges and provide technical support for the proposed inclusion of SDCL steel superstructure guidance in MoDOT's Engineering Policy Guide (EPG). The research included a literature review, field testing of Bridge A9513 during deck placement and controlled live-load testing, development and calibration of three-dimensional finite element models, degree-of-continuity calculations, parametric analyses of representative bridge configurations, and development of proposed design recommendations and standard details.

Summary of Research

A literature review was conducted to evaluate previous research, current design practices, and available transportation-agency guidance related to SDCL steel bridges. The review showed that SDCL steel bridges have been studied and implemented by several agencies, but formal implementation-ready design provisions remain limited. Oregon was the only state identified with a dedicated published SDCL detail for concrete intermediate diaphragm connections. Missouri has documented SDCL implementation through bridge replacement programs, while several other states either mention SDCL or related simple-made-continuous concepts, provide related continuity guidance, or address similar details through retrofit or prefabricated bridge applications.

Bridge A9513 on Route PP over the Middle Fork Little Chariton River in Chariton County, Missouri, was selected as the test bridge for this study. The bridge was instrumented during construction to measure strains and deflections during the concrete deck pour. This construction-stage testing was used to evaluate whether the bridge behaved as a simple-span system during deck placement, as intended by the SDCL concept. After the bridge was completed, controlled live-load testing was performed using loaded dump trucks placed at selected locations to produce positive and negative moment effects near the instrumented intermediate bent.

Three-dimensional finite element models were developed to interpret the measured bridge response. Separate dead-load and live-load models were created because the structural conditions during deck placement and after completion were different. The dead-load model represented the non-composite construction stage, including wet concrete loading, temperature effects, reduced early-age diaphragm stiffness, support conditions, and contact behavior at the intermediate bents. The live-load model represented the completed bridge

after the deck and diaphragm had cured and continuity had developed at the intermediate supports.

The calibrated models were used to evaluate the degree of continuity (DOC) developed during construction-stage dead loading and live loading. The construction-stage DOC was evaluated using deflection-based comparisons between calibrated, free, and fixed bridge models. The live-load DOC was evaluated using curvature- and deflection-based comparisons for selected truck positions. These calculations provided a quantitative measure of how closely the bridge response approached the simple-span and fully continuous limits.

A parametric study was then conducted to evaluate SDCL behavior beyond the specific geometry of Bridge A9513. Four bridge configurations were considered: a Base Bridge, a Skew Angle Bridge, a Plate Girder Bridge, and an Unbalanced Span Bridge. The Base Bridge corresponds to Bridge A9513 and uses rolled-shaped girders. The remaining configurations were generated by varying specific geometric or structural features of this baseline configuration. Each configuration was analyzed under factored superimposed dead loads and five factored AASHTO HL-93 truck loads and lane live-load cases, with fixed and pinned diaphragm configurations and with and without a bond breaker plate. This resulted in 96 parametric finite element models. The analyses were used to evaluate deck reinforcement stresses, compression block stresses, force transfer at the intermediate bents, and live-load DOC for different bridge geometries and diaphragm details.

The main outcome of this research was the development of proposed SDCL steel bridge design guidelines and standard details for inclusion in MoDOT's EPG. The proposed guidance defines the SDCL design concept, identifies limits of applicability, and outlines the staged-analysis approach needed to represent simple-span behavior during girder and deck dead load, continuous behavior for superimposed dead loads, and live load. The guidance also provides design procedures for the intermediate bent continuity connection, including deck reinforcement and compression block design, and includes standard details for the concrete diaphragm, bond breaker plate, pinned and fixed diaphragm configurations, bearings, shear connectors, bracing, girder haunches, and camber or dead-load deflection considerations. These guidelines are intended to provide MoDOT with a practical framework for consistent design, detailing, review, and implementation of SDCL steel superstructures.

Summary of Conclusions

The literature review showed that the primary technical issue in SDCL steel bridge design is the local force-transfer mechanism at the intermediate support. Previous experimental and numerical studies found that negative flexural resistance is provided mainly by the longitudinal slab reinforcement in tension, the bottom connection plate or steel block in compression, and the compressed concrete core in the diaphragm. The literature also showed that the assumption of simple-span behavior during deck placement is reasonable but not exact because partially cast diaphragms and reinforcement can provide some continuity before the deck hardens.

The field testing of Bridge A9513 confirmed that measurable continuity developed during construction-stage dead loading. The bridge did not behave as a purely simple-span system

during the deck pour. The deflection-based dead-load DOC was approximately 42% for the center span and approximately 23% for each of the end spans. The high DOC value on the center span was attributed primarily to the relatively deep partially cast diaphragm at the intermediate bent as well as the lack of a bond breaker plate. These findings indicate that construction-stage continuity can be significant and should not be assumed negligible for all SDCL diaphragm details.

The bridge developed a high level of continuity under live loading. The live-load DOC was approximately 92% on average, based on curvature- and deflection-based estimates. The live-load results also showed that the calibrated finite element model could reproduce the measured bridge response for multiple truck positions without recalibration, indicating that the model captured the global stiffness, load distribution, and continuity behavior of the completed bridge.

The parametric analysis showed that the proposed SDCL continuity details provided substantial live-load continuity for the range of bridge geometries considered. For the Base Bridge, Plate Girder Bridge, and Unbalanced Span Bridge, fixed diaphragm configurations produced DOC values of approximately 91% to 95%, while pinned diaphragm configurations produced DOC values of approximately 83% to 88%. The Skew Angle Bridge produced lower DOC values of approximately 79% to 82% and was less sensitive to diaphragm configuration, indicating that skew can have a stronger influence on effective live-load continuity than diaphragm restraint for the configuration considered.

In the parametric models, the continuity connection demands remained well below the available reinforcement and compression block capacities when the bridges were loaded with factored AASHTO live-load combinations. The maximum average deck reinforcement load factored stresses were less than 50% of the 60 ksi (kilopounds per square inch) reinforcement yield strength, and the maximum average compression block stresses were less than 12% of the 50 ksi compression block steel yield strength. These results indicate that the deck reinforcement and compression block details evaluated in this study were adequately proportioned for the factored dead-load and live-load demands considered.

The bond breaker plate had only a minor influence on live-load continuity in the completed bridge models. This result is expected because live-load continuity after bridge completion is developed primarily through the force couple between the deck reinforcement in tension and the compression blocks in compression. The bond breaker plate is therefore more important for controlling construction-stage diaphragm restraint and cracking than for changing the completed live-load response.

Summary of Recommendations

The research team recommends that MoDOT develop SDCL steel bridge guidance as a distinct topic within the EPG. SDCL steel bridges have a staged construction sequence and force-transfer mechanism that differ from conventional simple-span and continuous steel girder bridges. Therefore, the guidance should clearly define the intended simple-for-dead-load and continuous-for-live-load behavior, the applicable bridge types, staged-analysis requirements, and standard details for the intermediate support region.

The research team recommends that staged analysis be required for SDCL steel bridges. Girder self-weight and deck dead load should be evaluated using the construction-stage simple-span condition before continuity is established. Superimposed dead load and live load should be evaluated using the completed continuous system after the deck and diaphragm concrete have hardened.

The results also indicate that low construction-stage DOC values should not be assumed for all SDCL diaphragm configurations. The expected construction-stage continuity should be related to the specific diaphragm detail, including the presence or absence of a bond breaker plate, the degree of diaphragm restraint, and the height of the diaphragm cast before deck placement. For bridges without a bond breaker plate or with relatively deep partially cast diaphragms, a higher construction-stage DOC, on the order of 40%, may be more representative based on the results of this study. For bridges with pinned diaphragms and a bond breaker plate, a construction-stage DOC of approximately 25% may be assumed. For design using conventional software, it is conservative to ignore these effects since the stress quickly reverse at the time of casting the deck and minimally offset the stresses at controlling points. It would also have minimal impact on deflections during casting of the deck.

Pinned diaphragm configurations and bond breaker plates are recommended when the intent is to reduce unintended dead-load continuity during deck placement. However, the results indicate that these details can still allow the completed SDCL system to develop substantial live-load continuity after the deck and diaphragm region have hardened.

Finally, the research team recommends that future MoDOT SDCL guidance emphasizes the intermediate support region as the critical design detail. The deck reinforcement, compression block, girder end plate, diaphragm, construction joint, bond breaker plate, bearings, and support restraint all contribute to the continuity mechanism. The proposed standard details and design recommendations developed in this study provide a practical basis for implementation, but additional field monitoring of SDCL bridges with different diaphragm details, skew angles, and span arrangements would help refine the guidance as MoDOT gains additional experience with this bridge type.

Chapter 1. Introduction

Simple for Dead Load, Continuous for Live Load (SDCL) steel superstructures provide an alternative design approach for multi-span steel bridges. In this system, the girders are designed to behave primarily as simple spans during construction-stage dead loading, while continuity is developed over the intermediate supports after the deck and diaphragm concrete have hardened. This approach can reduce fabrication complexity by eliminating or reducing the need for bolted field splices, while still providing the benefits of live-load continuity in the completed structure. For Missouri bridges, SDCL steel superstructures may offer cost and constructability advantages in situations where conventional continuous steel superstructures are less economical or less competitive with concrete alternatives. However, these potential benefits cannot be fully realized without clear design guidance, standard details, and practical recommendations for implementation within MoDOT practice.

The primary motivation for this research was to support the development of guidance and standard details for SDCL steel superstructures for inclusion in Section 751.14 of MoDOT's Engineering Policy Guide (EPG). The needed guidance includes recommended span configurations, design procedures for cases not explicitly addressed by the AASHTO LRFD Bridge Design Specifications, standard details for intermediate and end bents, and practical design aids for evaluating SDCL steel girders.

As part of this study, Bridge A9513 on Route PP over the Middle Fork Little Chariton River in Chariton County, Missouri, was selected as the test bridge. The bridge was evaluated experimentally during construction and after completion to assess the development of continuity in the SDCL system. Construction-stage dead-load testing was performed during the concrete deck pour, when the bridge was expected to behave primarily as a simple-span system. Live-load testing was then conducted after the deck and diaphragm had cured, when the completed bridge was expected to behave as a continuous structure. The experimental program included strain and displacement measurements using strain transducers, linear variable differential transformers, and a string potentiometer.

The experimental data were used to support the development and calibration of three-dimensional finite element models of the test bridge. These models were used to interpret the measured bridge response, evaluate the degree of continuity during construction and under live loading, and investigate the influence of support and diaphragm behavior on the measured response. After calibration, additional finite element models were developed for a parametric study to evaluate the behavior of several SDCL bridge configurations, diaphragm details, bond breaker plate configurations, and HL-93 truck and lane live load cases. The current report includes experimental testing, finite element model development and calibration, degree-of-continuity calculations, parametric analyses, and design recommendations.

1.1 Objectives

The Missouri Highway and Transportation Commission contracted Horner & Shifrin to research and study Simple for Dead Load, Continuous for Live Load steel superstructures. The ultimate goal of this research is to support the development of guidance for this type of steel superstructure in MoDOT's Engineering Policy Guide. Horner & Shifrin contracted with the University of Missouri–Columbia to develop finite element models of the test bridge, instrument the test bridge, collect data during construction, conduct live-load testing, calibrate the finite element models using the measured field response, and develop additional parametric models to evaluate the influence of selected bridge and connection parameters.

The specific objectives of this study were to:

- Evaluate the construction-stage and live-load behavior of an SDCL steel bridge using field instrumentation and controlled load testing;
- Develop and calibrate finite element models capable of representing the measured response of the test bridge during construction-stage dead loading and live loading;
- Quantify the degree of continuity developed by the SDCL system during deck placement and in the completed bridge condition;
- Evaluate the effects of diaphragm type, bond breaker plate configuration, bridge geometry, and live-load placement on force transfer at the intermediate supports;

Develop design guidance, standard details, and practical recommendations for implementing SDCL steel superstructures in MoDOT practice.

1.2 Research Tasks

The research was completed through a series of coordinated tasks. First, a literature review was conducted to summarize prior research, current design practices, and available guidance from other transportation agencies related to SDCL steel bridge systems. This task provided background on the development of SDCL details, the expected load-transfer mechanisms at intermediate supports, and modeling approaches used in previous analytical and experimental studies.

Second, an experimental testing program was conducted on Bridge A9513. The bridge was instrumented during construction to measure strains and deflections during deck placement. A separate live-load test was performed after completion of the bridge using loaded dump trucks positioned at selected locations near an intermediate bent. These tests provided field data for evaluating construction-stage behavior, completed-bridge response, and the development of continuity in the SDCL system.

Third, three-dimensional finite element models of the test bridge were developed. Separate models were created for the dead-load and live-load stages to reflect the different structural conditions present during construction and after completion. The models included steel girders, concrete deck, diaphragms, end plates, compression blocks, bent caps, piles, barriers, contact

interfaces, support conditions, and relevant construction-stage loading effects. The models were calibrated using measured strains and deflections from the field tests.

Fourth, the calibrated models were used to evaluate the degree of continuity of the test bridge. The measured and predicted bridge responses were compared with idealized simple-span and fully continuous limits to assess how much continuity developed during deck placement and under live loading.

Fifth, a parametric analysis was conducted using additional finite element models. The parametric study considered multiple bridge configurations, including the base bridge, a skewed bridge, a plate girder bridge, and an unbalanced span bridge. For each configuration, different diaphragm types, bond breaker plate configurations, and HL-93 load cases were evaluated to investigate force transfer through the deck reinforcement and compression block region at the intermediate supports.

Finally, the results from the literature review, field testing, finite element calibration, degree-of-continuity assessment, and parametric analysis were synthesized to develop design guidelines for SDCL steel superstructures for inclusion in Section 751.14 of MoDOT's EPG.

1.3 Report Overview

The report is organized as follows. Chapter 2 presents background information on SDCL steel bridge systems, including a literature review of previous experimental, analytical, and numerical studies. Chapter 3 describes the experimental testing program conducted on Bridge A9513, including the bridge design, instrumentation layout, construction-stage dead-load testing, live-load testing, and measured field response. Chapter 4 presents the finite element modeling approach used for the test bridge, including model assumptions, material properties, contact definitions, support conditions, model calibration, and degree-of-continuity calculations. Chapter 5 presents the parametric analysis results, including the bridge configurations, diaphragm details, bond breaker plate configurations, superimposed dead load and HL-93 live-load cases, and resulting force demands in the deck reinforcement and compression block region. Chapter 6 provides design recommendations based on the findings of the study. Chapter 7 summarizes the main conclusions of the research and identifies the principal findings relevant to implementation of SDCL steel superstructures in MoDOT practice.

Chapter 2. Background

2.1 Overview

The modern use of SDCL steel bridges developed from efforts to make short- and medium-span steel bridges more economical and easier to construct. In conventional continuous steel bridges, girders are typically fabricated and erected with field splices or other continuity details that allow the structure to behave continuously throughout construction and service. In contrast, the SDCL concept allows steel girders to be erected as simple spans for girder self-weight and deck placement, while continuity is developed later through the hardened deck, intermediate diaphragm, slab reinforcement, and bottom compression connection. This approach reduces the need for bolted field splices and simplifies erection while still allowing the completed bridge to benefit from live-load continuity.

SDCL steel bridges are being considered in Missouri because they may provide a practical alternative for routine multi-span bridge replacements. For many short- and medium-span bridges, steel superstructures can be less competitive when fabrication, field splices, erection complexity, and long-term maintenance are considered. SDCL details can improve the competitiveness of steel by simplifying girder fabrication, reducing or eliminating intermediate deck joints, improving durability, and reducing inspection and maintenance concerns associated with bolted field splices and expansion joints. These advantages are particularly relevant for bridge replacement programs where economy, constructability, standardization, and long-term performance are important.

Missouri has already implemented SDCL steel bridge concepts in recent bridge replacement projects, including the FARM Bridge Program [1] and the Northwest Bridge Bundle [2]. These projects demonstrate that SDCL steel bridges are no longer only a research concept, but an emerging practical alternative for MoDOT bridge applications. However, broader implementation requires more than successful project-level use. It also requires consistent design assumptions, standard details, and clear guidance for evaluating staged behavior, diaphragm restraint, continuity development, and force transfer at intermediate supports.

A key challenge is that SDCL bridges do not behave exactly like either simple-span or fully continuous steel bridges. During deck placement, the bridge is intended to behave primarily as a simple-span system, but partial diaphragm casting, reinforcement, and level of restraint can still introduce unintended continuity. After the deck and diaphragm have hardened, the completed bridge must develop sufficient continuity through a local force-transfer mechanism involving the deck reinforcement in tension and the compression block or bottom connection region in compression. Because this behavior depends strongly on construction sequence and local detailing, conventional continuous-girder design assumptions may not fully capture the response of the system.

Although previous research has established the basic SDCL force-transfer mechanism, there remains limited field data documenting how these bridges behave during construction and

under live loading. In addition, limited information is available on how diaphragm type, bond breaker plates, skew, span arrangement, and girder type influence the degree of continuity and the demands in the deck reinforcement and compression block region. This lack of experimental data, calibrated numerical models, and MoDOT-specific design guidance provides the motivation for the present research. The study was therefore conducted to evaluate the behavior of an instrumented SDCL bridge in Missouri, develop calibrated finite element models, extend the results through parametric analyses, and support the development of practical SDCL guidance for inclusion in MoDOT's Engineering Policy Guide.

2.2 Use of SDCL Steel Bridges in the U.S.

Horner & Shifrin and the University of Missouri-Columbia reviewed guidance and research materials from all 50 states to determine the extent to which state transportation agencies have adopted, studied, or addressed SDCL steel bridge systems. The main findings from this review are summarized as follows:

- Oregon has dedicated details for SDCL concrete intermediate diaphragm connections.
- New York and Massachusetts have details for making bridges continuous over piers, although these are not specifically framed as SDCL steel bridge provisions.
- California, Colorado, Delaware, New Jersey, Texas, and West Virginia mention SDCL or related simple-made-continuous concepts but generally do not provide dedicated SDCL details compared to Oregon.
- Nebraska, New Mexico, Pennsylvania, and Tennessee do not include explicit SDCL details in their current guidelines, but research articles, reports, or case studies have documented SDCL-related applications in those states.
- Florida and Virginia include guidance that restrict or does not allow multiple simple spans with continuous decks in their structural guidelines.

The following sections provide a more detailed overview of the outcomes from this literature survey. As discussed next, SDCL steel bridges have been studied and implemented by several transportation agencies, but the available information is not organized around a single national standard. The core research basis comes from Nebraska, where the concept was developed for steel spans erected as simple spans for non-composite dead-load effects and made continuous for live load and superimposed dead load after completion of the deck and pier-region connection. Colorado later extended this work toward economical steel detailing and diaphragm alternatives, while more recent ABC-UTC and TxDOT research has expanded the concept toward prefabricated construction, UHPC connections, accelerated bridge construction, and seismic applications. Overall, the survey shows a broad implementation landscape, with some states providing dedicated details, some recognizing or encouraging the concept, some addressing related continuity or retrofit applications, and others restricting similar multi-span simple steel configurations [1], [3]–[20].

2.2.1 U.S. Department of Transportation Practices

This section summarizes DOT practices and related state-level research on SDCL steel bridges. The reviewed sources include explicit SDCL implementation details, DOT-sponsored research

programs, owner manual recognition, retrofit provisions, and prefabricated-unit continuity guidance.

Missouri

Missouri is one of the strongest current implementation examples of SDCL steel bridge systems. MoDOT documented use of SDCL steel design methodology in the FARM Bridge Program [1] for bridge replacements at 25 locations, Northwest Bridge Bundle [2] with 25 locations, at least 5 built using design-bid-build, and several currently in design. The documented expected benefits included lower initial cost, reduced life-cycle cost, easier inspection, reduced maintenance, reduced uplift in unbalanced spans, reduced negative moments at intermediate bents, simplified fabrication, elimination of bolted field splices, and shallower beam depths.

Oregon

Oregon has reported some owner-standardized SDCL detailing. ODOT explicitly recognizes SDCL systems in its Bridge Design Manual and provides DET3620, a dedicated standard detail sheet for SDCL details. The value of the Oregon material is that it translates the SDCL concept into an owner-usable detail with a concrete diaphragm and link-slab-style continuity region [12], [13]. The SDCL concrete diaphragm details shown in DET3620 are similar to the details used in Missouri on the FARM Bridge Program [1].

Nebraska

Nebraska is the foundational state in modern steel SDCL research practice. Early Nebraska Department of Roads and NaBRO reports identified SDCL as a way to make steel bridges more competitive with prestressed concrete for short- to medium-span rural bridges. The 2001 and 2003 documents developed the primary motivation for the concept: eliminating bolted field splices and expansion joints where possible, simplifying fabrication and erection, and designing steel girders for simple-span dead-load effects while relying on the completed deck and pier region to provide continuity for subsequent loading [3], [4].

The 2005 two-volume Nebraska work remains the most important dedicated transportation-agency research source on steel SDCL bridges. Volume 1 synthesized analytical studies and design recommendations, while Volume 2 documented the experimental program. These reports treated SDCL not as a generic continuous-bridge assumption, but as a connection-design problem in which the intermediate support region must be strong, stiff, constructible, and compatible with staged behavior [5], [6].

The Nebraska research also showed that span arrangement matters. Unbalanced spans can produce bottom-flange tension, force reversals, or local pier-region demands that are not captured by assuming ordinary continuous-composite behavior in all stages [5], [6].

Colorado

Colorado contributed to the literature through CDOT-sponsored research and Mountain Plains Consortium reports focused on innovative and economical steel bridge alternatives. The Colorado work documented simple-made-continuous steel bridges as a strategy for improving

the competitiveness of steel bridges by reducing bolted field splices and improving construction economy [7], [8], [21].

The most distinctive Colorado contribution is the evaluation of steel diaphragm continuity schemes. Much of the earlier SDCL literature assumed concrete diaphragms or concrete-filled pier regions, while the Colorado studies explored how steel diaphragm details might provide continuity while improving fabrication and construction sequencing [7], [8].

The current CDOT bridge manual recognizes simple-made-continuous steel structures primarily through references to industry practice and previous research rather than through a highly prescriptive standard detail [8], [22].

Texas

Texas represents the most current DOT-sponsored research trajectory identified in this review. The 2025 TxDOT report developed and evaluated a prefabricated steel accelerated bridge construction unit intended to behave as simply supported for dead load and continuous for live load. Unlike earlier research that focused primarily on feasibility, the Texas work emphasizes a connection that is relatively easy to fabricate, quick to assemble, safe to construct, cost-effective, durable, and compatible with prefabrication [11].

The TxDOT work is important because it reframes SDCL as part of the current ABC and prefabricated bridge-unit landscape. In this context, the continuity detail must satisfy structural demands while also accommodating construction tolerances, closure operations, inspection access, and rapid assembly. TxDOT bridge design guidance also acknowledges SDCL steel bridge design, placing Texas among the states with both manual recognition and recent implementation-oriented research [11], [18], [19].

Florida

Florida appears in the reviewed literature in two different ways. First, FDOT/FIU research on UHPC slab-beam bridge joint connections includes a simple-for-dead, continuous-for-live connection. Although the report is not a conventional steel-girder SDCL design guide, it is relevant because it treats the continuity region as a material-and-detailing problem and demonstrates how UHPC may improve joint durability, development length, and prefabricated connection behavior [23].

Second, Florida's owner guidance includes restrictive language for units composed of multiple simple-span steel girders with continuous decks. This does not negate the relevance of the FDOT/FIU research, but it shows that owner acceptance of SDCL-like systems depends strongly on the clarity of the force path and confidence in the detailing, load rating, fatigue performance, and service behavior of the completed bridge [24].

California

California's steel plate girder design practice mentions SDCL steel bridge systems, but the reviewed material indicates that the concept is treated as a case-by-case option rather than as

a broadly standardized detail. This places California in the category of states that recognize the idea but do not provide the same owner-level SDCL detail found in Oregon [14].

Delaware

Delaware's Bridge Design Manual recommends simple-made-continuous multi-span steel bridge units as a structure-type selection concept. The Delaware material is important because it treats simple-made-continuous steel bridges as a desirable configuration at the bridge-selection stage, even though the reviewed source does not provide a detailed SDCL pier connection comparable to ODOT DET3620 [15].

New Jersey

New Jersey's design manual encourages project-by-project consideration of SDCL systems to reduce joints over bridge piers. This aligns with one of the central motivations for SDCL bridges: improving durability and lowering maintenance by reducing or eliminating deck joints at intermediate supports. The New Jersey material is not a full design procedure, but it is a useful example of an owner position that encourages the concept while leaving detailed design responsibility to the project engineer [16].

Tennessee

Tennessee's current steel girder guidance includes a rolled-girder made-continuous option. Although this is not the same as a full SDCL steel superstructure standard detail, it belongs in the broader family of simple-span steel bridges made continuous through detailing at the pier [25].

West Virginia

West Virginia's structure directives do not appear to provide a dedicated SDCL detail in the reviewed sources, but they reinforce the broader owner emphasis on economic comparison of bridge alternatives. This matters because the argument for SDCL is often economic and constructability-driven. The main purpose of the concept is to reduce splices, reduce joints, simplify erection, and improve life-cycle performance [26].

New Mexico

The literature survey identified New Mexico as a state where the reviewed guidelines did not clearly include SDCL details. The New Mexico Bridge Procedures and Design Guide remains part of the source base because it was included in the expanded state review and helps document that explicit SDCL provisions are not uniformly available across state DOT guidance [27].

New York

New York's contribution is strongest on retrofit guidance. The NYSDOT bridge manual distinguishes full continuity from continuous-for-live-load retrofit and emphasizes fatigue, deck replacement sequence, construction staging, and the validity of assumptions after retrofit. This is highly relevant because many agencies maintain inventories of simple-span steel bridges for which continuity retrofits may be attractive during deck replacement [17], [28].

Massachusetts

Massachusetts contributes related prefabricated bridge unit continuity details. The MassDOT LRFD Bridge Manual and associated details address continuity over piers for prefabricated bridge units, including negative-moment reinforcement, diaphragm details, and flexural compression transfer. These documents are not direct steel SDCL standards in the same sense as ODOT DET3620, but they address the same fundamental continuity-couple problem [29]–[31].

Pennsylvania

Pennsylvania did not surface in the literature survey as a state with clear SDCL provisions in the design manual, but the I-476 bridge over the Schuylkill River retrofit case study is relevant to the broader simple-span-made-continuous literature. The case study documents a retrofit scheme for making existing simple steel spans continuous [32], [33].

Virginia

Virginia provides one of the clearest restrictive signals in the reviewed state guidance. The VDOT structural-steel guidance does not allow multi-span bridges composed of simple steel spans. This position is important because it indicates that SDCL-like configurations are not universally accepted as default practice [20].

2.2.2 Federal, National, and Agency-Adjacent Practices

The Federal Highway Administration (FHWA) prefabricated bridge connection report is not an SDCL-only document, but it places SDCL-related details within the broader national movement toward prefabricated bridge elements and systems. In ABC practice, the connection often governs the success of the system. Therefore, SDCL steel bridges fit naturally within the national connection-detail literature [34].

ABC-UTC research broadens the SDCL literature beyond conventional non-seismic bridge replacement. The 2016 Phase I work examined the extension of the steel SDCL system to seismic areas and identified the importance of bottom-flange continuity, dowel bars, tie bars, ductility, and integral system behavior under seismic demands. The 2019 shake-table study then moved the literature toward dynamic system validation for bridge systems with ABC connections, including a two-span steel girder system with simple-for-dead, continuous-for-live behavior and a UHPC-filled pier joint [9], [10].

NCHRP and NSBA/AISC resources document national practice and practitioner-level implementation of SDCL steel bridges. These sources are not substitutes for DOT design provisions, but they help connect state implementation to broader industry experience and to the recurring bridge-owner goals of eliminating joints, simplifying erection, improving durability, and improving the competitiveness of steel bridges [35]–[38].

The 2024 ABC-UTC UHPC connection record and 2025 design-resource material indicate that SDCL remains active in the research and training ecosystem. The current trend is toward details

that are not only structurally adequate, but also compatible with prefabrication, durable, inspectable, and suitable for owner adoption [38], [39].

2.2.3 Summary of State DOT Adoption of SDCL Steel Bridges

The adoption of SDCL steel bridges across state DOTs varies broadly across the United States. At one end, Oregon has explicit SDCL manual language and a dedicated detail sheet. Missouri has documented implementation through the FARM Bridge Design Build project and Northwest Bridge Bundle Design Build project. Nebraska, Colorado, and Texas provide the strongest DOT-sponsored research lineage, with Nebraska supplying the foundational basis, Colorado adding alternative steel detail research, and Texas providing a current prefabricated steel ABC implementation study [1], [3], [5], [8], [11]–[13].

A second group of states recognizes SDCL steel systems without providing a full owner-standard detail. California, Delaware, New Jersey, Tennessee, and Texas include language that acknowledges or encourages the concept in steel-girder or structure-type guidance. These states show that the concept has reached owner manuals, but they also demonstrate that manual recognition alone does not equal implementation-ready guidance [14]–[16], [18], [19], [25].

A third group contributes related continuity practice rather than direct new-construction SDCL provisions. New York provides retrofit guidance that distinguishes full continuity from continuous-for-live-load retrofit and emphasizes fatigue, construction sequence, and load-rating assumptions. Massachusetts provides prefabricated bridge unit continuity details that are relevant to the continuity-couple mechanism. Pennsylvania contributes a retrofit case study. These sources are valuable because they focus attention on staged behavior, existing steel members, negative-moment continuity, and force transfer over piers [17], [28]–[31], [33].

A fourth group shows caution or restriction. Florida restricts units composed of multiple simple-span steel girders with continuous decks, even though FDOT/FIU research contributes related UHPC continuity work. Virginia does not allow multi-span bridges composed of steel simple spans. These restrictions should not be interpreted as proof that SDCL is invalid. Rather, they show that agencies are reluctant to permit configurations where the distinction between simple-span construction, continuous deck behavior, and verified live-load continuity is not clearly specified [20], [23], [24].

Table 2-1 provides a compact summary of the findings from the literature survey, which determines the extent to which state transportation agencies have adopted, studied, or addressed SDCL systems throughout the country.

Table 2-1. Summary of state DOT adoption and of SDCL steel bridge systems

State / agency	Observed position	Relevance to SDCL practice	Key refs.
Oregon	Explicit SDCL manual language and detail sheet	Dedicated concrete diaphragm/link-slab style details	[12], [13]
Missouri	FARM Bridge Design Build	25 locations documented in innovation showcase	[1]
	Northwest Bridge Bundle Design Build	25 locations	
	Design-Bid Build	5 Locations	
Nebraska	Foundational research state	NaBRO/Nebraska Department of Roads reports form primary technical basis	[3]–[6]
Colorado	Research and manual recognition	CDOT-sponsored research on SMC steel bridges and steel diaphragm details	[7], [8], [21], [22]
Texas	Manual recognition plus recent research	2025 TxDOT report on prefabricated steel ABC unit continuous for live load	[11], [18], [19]
California	Concept mentioned	Case-by-case SDCL discussion in steel plate girder practice	[14]
Delaware	Concept recommended for selection	Manual recommends simple-made-continuous multi-span steel bridge units	[15]
New Jersey	Concept encouraged project-by-project	Manual encourages SDCL to reduce joints over piers	[16]
New York	Retrofit guidance	Manual distinguishes full continuity and continuous-for-live-load retrofits	[17], [28]
Massachusetts	Related prefabricated unit continuity details	Negative-moment and diaphragm details for PBUs	[29]–[31]
Pennsylvania	Retrofit case-study relevance	Manual itself did not surface SDCL; I-476 retrofit case study is relevant	[32], [33]
Florida	Restrictive manual language; related UHPC research	Manual restricts continuous decks over multiple simple-span steel girders; FDOT UHPC research includes SDCL connection	[23], [24]
Virginia	Restrictive manual language	Manual does not allow multi-span steel simple spans	[20]
Tennessee	Made-continuous option in current steel girder guidance	Rolled-girder made-continuous option in SDG-6	[25]
West Virginia	General economic recognition	Structure directives advise considering span-by-span economics	[26]

2.3 Numerical and Experimental Studies

The primary source of previous work in SDCL has been numerical modeling with some experimental studies. Numerical modeling has been central to the development, validation, and refinement of SDCL steel bridge systems. Because the defining feature of SDCL bridges is the delayed development of continuity at interior supports, finite element modeling has been used primarily to evaluate force transfer in the diaphragm region, quantify the degree of continuity during construction, assess ultimate connection capacity, and support simplified design provisions. The literature shows a progression from detailed nonlinear models of

laboratory connection specimens, to field-validated bridge models, and finally to models for seismic and ultrahigh-performance concrete (UHPC) connection details [40]–[43].

The foundational numerical studies on SDCL steel bridges were conducted as part of a research program led by the University of Nebraska-Lincoln. Farimani et al. [42] developed nonlinear finite element models of connection details that had previously been evaluated experimentally [4], [5], [41], including bottom-flange-continuous, bare-end, end-plate, and modular connection types (see Figure 2-1). These models were used to interpret internal mechanisms that could not be fully inferred from instrumentation alone. The finite element models included the steel girders, concrete slab, concrete diaphragm, bearing pads, reinforcement, stiffeners, shear studs, and connection plates. In the broader research program, ANSYS and ABAQUS models were used with nonlinear steel and concrete constitutive models, including multilinear steel hardening calibrated from coupon tests and concrete damage/cracking formulations capable of representing crushing, cracking, and stress redistribution in the diaphragm region [5], [41], [42].

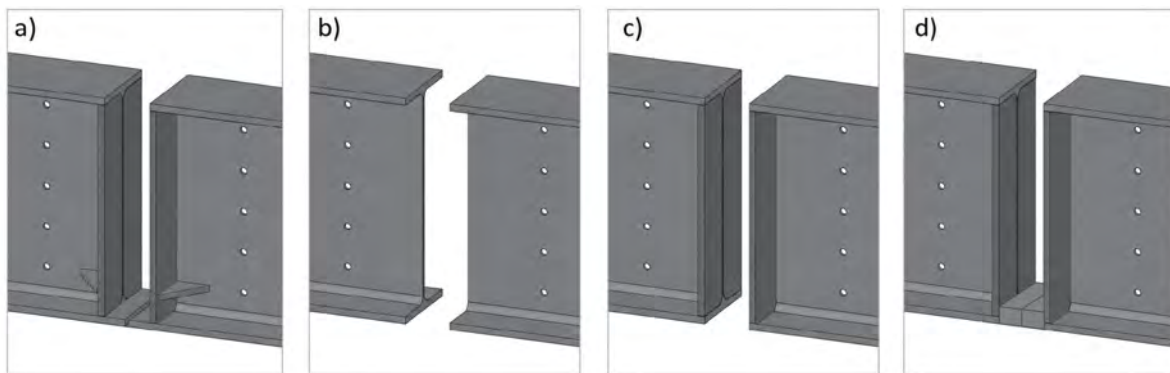


Figure 2-1. SDCL connection types: (a) type 1 (flange-continuous), (b) type 2 (bare-end), (c) type 3 (end-plate), and (d) type 4 (modular) (adapted from [42]).

A major contribution of these models was the identification of the dominant components that resist negative moments at the pier. The numerical results showed that the main contributors to ultimate negative flexural resistance are the slab reinforcement in tension, the bottom connection plate or steel block in compression, and the compressed concrete core in the diaphragm [42]. Other components, such as concrete tension, diaphragm hoops, and concrete outside the core region, made smaller contributions and were neglected in simplified design calculations. The models also showed that plane-section behavior is not valid inside the concrete diaphragm, although strain distributions become more nearly linear away from the pier centerline. This finding is important for practical design because it supports the use of idealized stress distributions while recognizing the highly nonlinear local behavior near the pier.

The numerical work also clarified the importance of the bottom-flange compression-transfer mechanism. Connection details that provide a direct steel-to-steel compression path through welded bottom flanges, end plates, or steel blocks tend to produce more desirable and ductile behavior. Details without a clear compression-transfer mechanism rely more heavily on concrete bearing and are more susceptible to concrete crushing in the diaphragm. Therefore,

SDCL connection performance cannot be evaluated only from global moment demand; the local load path through the diaphragm, reinforcement, girder-end detail, and bearing region must also be represented accurately [5], [42].

The main outcomes from the Nebraska research program were analysis and design recommendations for SDCL steel bridge systems. Among the connection types tested, details that included a bottom-flange compression-transfer mechanism were found to provide the largest load capacities [40], [42]. Of the four connection types discussed previously, the modular Type 4 connection was recommended for ABC applications of SDCL bridge systems because it combines favorable structural performance with improved constructability (see Figure 2-2) [40].

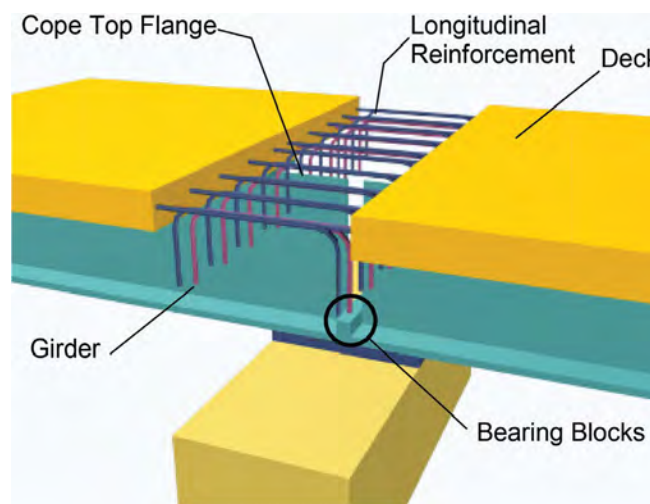


Figure 2-2. SDCL continuity connection detail recommended for ABC applications [40].

Field studies expanded the role of numerical modeling from connection-level behavior to system-level bridge response. Yakel and Azizinamini [43] monitored three demonstration SDCL bridges during construction and service and compared measured strains and deflections with predicted simple-span and fully continuous responses. These studies showed that the assumption of simple-span behavior during deck casting is reasonable but not exact. The partially cast diaphragm and reinforcement can provide some continuity before the deck hardens, which affects strains, deflections, camber predictions, and the interpretation of field data. Azizinamini [40] suggests assuming 25% continuity in the dead load configuration.

Experimental work also considered the use of a bond breaker plate. This is a vertical plate made of thin sheet metal cast in the concrete diaphragm to control cracking in the diaphragm during pouring of the deck (see Figure 2-3). Azizinamini [40] recommends use of the plate to control cracking and limit the development of dead load continuity.



Figure 2-3 (a) Bond breaker plate, (b) uncontrolled cracking in diaphragm [40] .

A related line of research studied SDCL steel bridge systems with steel diaphragms instead of concrete diaphragms. Johnson and Atadero [44] used ABAQUS modeling and full-scale testing to study an alternative detail in which continuity is developed through slab reinforcement in tension and steel diaphragm or sole-plate mechanisms in compression. Although this system differs from the conventional concrete-diaphragm SDCL detail, it provides useful modeling guidance: finite element results are highly sensitive to material characterization, contact and constraint definitions, and validation against full-scale experiments. The study also highlighted potential constructability and inspection advantages of steel-diaphragm systems because the girder ends remain exposed.

More recent studies extended SDCL modeling to seismic applications. Taghinezhadbilondy et al. [9] and Sadeghnejad et al. [45] used detailed nonlinear finite element analysis to develop and verify connection details suitable for high seismic regions (see Figure 2-4). The new SDCL connection design for seismic areas was a modification of the connection Type 4 discussed previously, with the main difference being the presence of tie bars and shear studs on the compression flanges. These new elements in the connection are meant to provide strength and ductility to the connection during a seismic event where positive bending moments can occur at the pier locations. In designing the new connection, representative loading configurations were used to examine gravity, vertical seismic, and longitudinal reversal demands. These models helped identify critical force demands and the roles of tie bars, dowel bars, steel blocks, end stiffeners, and live-load continuity reinforcement. The seismic studies indicated that SDCL connections in high seismic regions should be treated as capacity-protected elements: the superstructure and connection region should remain essentially elastic while inelastic demand is directed to designated plastic hinge regions in the columns [9], [45].

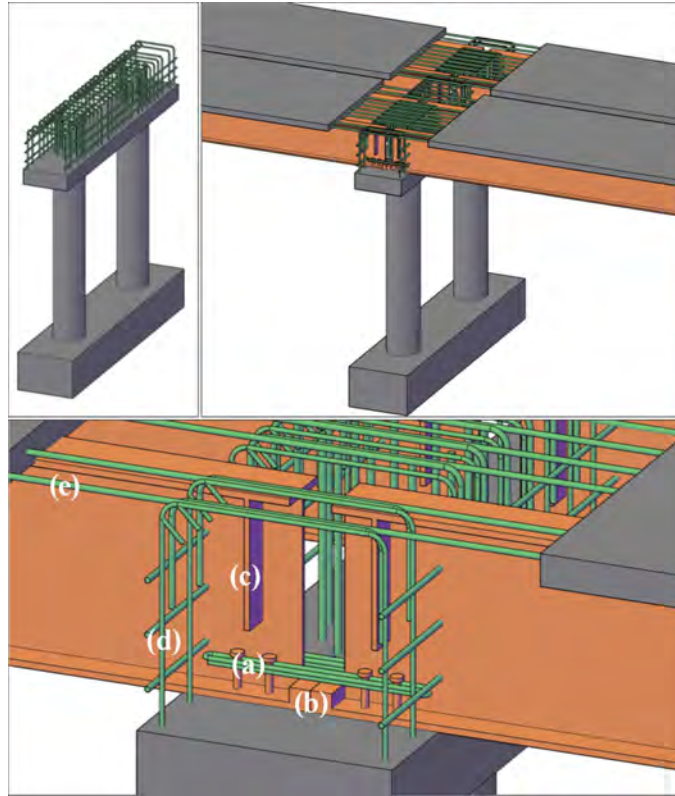


Figure 2-4. SDCL connection detail for seismic areas [45].

The most recent numerical developments involve UHPC diaphragms for SDCL bridges. Khodayari et al. [46] proposed replacing portions of the normal-strength concrete diaphragm with UHPC to simplify connection details, improve durability, and increase construction tolerances. Their ANSYS models used coupled damage-plasticity microplane material formulations for normal-strength concrete and UHPC. After validation against previous SDCL connection experiments and UHPC-encased member tests, the models were used in parametric studies of girder-end gap, seat height, and simplified reinforcement layouts. The results suggested that UHPC can simplify the girder-end detail, reduce reinforcement congestion, and potentially eliminate bottom-flange continuity for selected nonseismic applications [46].

Overall, the literature indicates that finite element modeling of SDCL steel bridges is most useful when treated as a calibrated interpretive tool rather than a stand-alone design method. Detailed nonlinear models are needed to understand local force transfer, concrete cracking and crushing, steel yielding, contact behavior, and construction-stage continuity. However, most studies ultimately use these models to support simplified design equations, detailing recommendations, or interpretation of field data. For MoDOT applications, the most relevant modeling considerations are partial diaphragm continuity during deck casting, realistic girder-diaphragm contact assumptions, bottom-flange compression transfer, validation against field strain and deflection data, and sensitivity studies that quantify how connection stiffness affects degree of continuity, camber, moment redistribution, and local demands at intermediate supports.

2.4 Summary

The use of SDCL steel bridges in other states and in previous research was investigated to identify design provisions, implementation practices, and technical findings relevant to future MoDOT guidance. The review showed that SDCL steel bridges have been studied and implemented by several transportation agencies, but formal design provisions remain limited. Oregon was the only state identified with a dedicated published SDCL detail for concrete intermediate diaphragm connections. Several other states either mention SDCL or related simple-made-continuous concepts, provide guidance for continuity over piers, or address related retrofit and prefabricated-unit details. However, most states do not provide implementation-ready SDCL design provisions, and some states restrict similar multi-span simple steel configurations with continuous decks.

Previous research has focused mainly on numerical studies because the strain distribution and force-transfer mechanism in the diaphragm region are complex, although limited experimental and field studies are also available. The literature found that the main contributors to ultimate negative flexural resistance are the slab reinforcement in tension, the bottom connection plate or steel block in compression, and the compressed concrete core in the diaphragm. These findings show that SDCL design should focus on the local continuity mechanism at the intermediate support, not only on global negative moment demand. The literature also indicates that the assumption of simple-span behavior during deck casting is reasonable but not exact because the partially cast diaphragm and reinforcement can provide some continuity before the deck hardens. The main outcomes from previous experimental and numerical studies have been recommended SDCL details at the intermediate diaphragm regions. The recommended SDCL steel bridge detail at the diaphragm regions is shown in Figure 2-2. This detail includes a steel end block to transfer compressive forces at the bottom of the connection together with longitudinal reinforcement in the deck to carry tensile forces resulting from negative moments at the intermediate pier locations.

Chapter 3. Experimental Testing

Bridge A9513 on Rt. PP over Middle Fork Little Chariton River in Chariton County was tested to verify the FE model because it was scheduled for replacement as an SDCL steel bridge. Experimental testing was divided into two stages: dead load testing and live load testing. Dead load testing was conducted during the concrete deck pour, at which time the bridge behaved theoretically as a simply supported system. Live load testing was performed after the completion of primary construction work and curing of the deck, at which point the bridge acted as a continuous structure. Live load testing was completed by positioning dump trucks at various locations around one of the concrete diaphragms to assess the fixity of the joint. Since the bridge is symmetrical, Spans 2-3 and 3-4 were tested, as shown in Figure 3-1. The testing instrumentation included Linear Variable Differential Transformers (LVDTs), Strain Transducers (STs), and a string potentiometer (SPOT) or string pot.

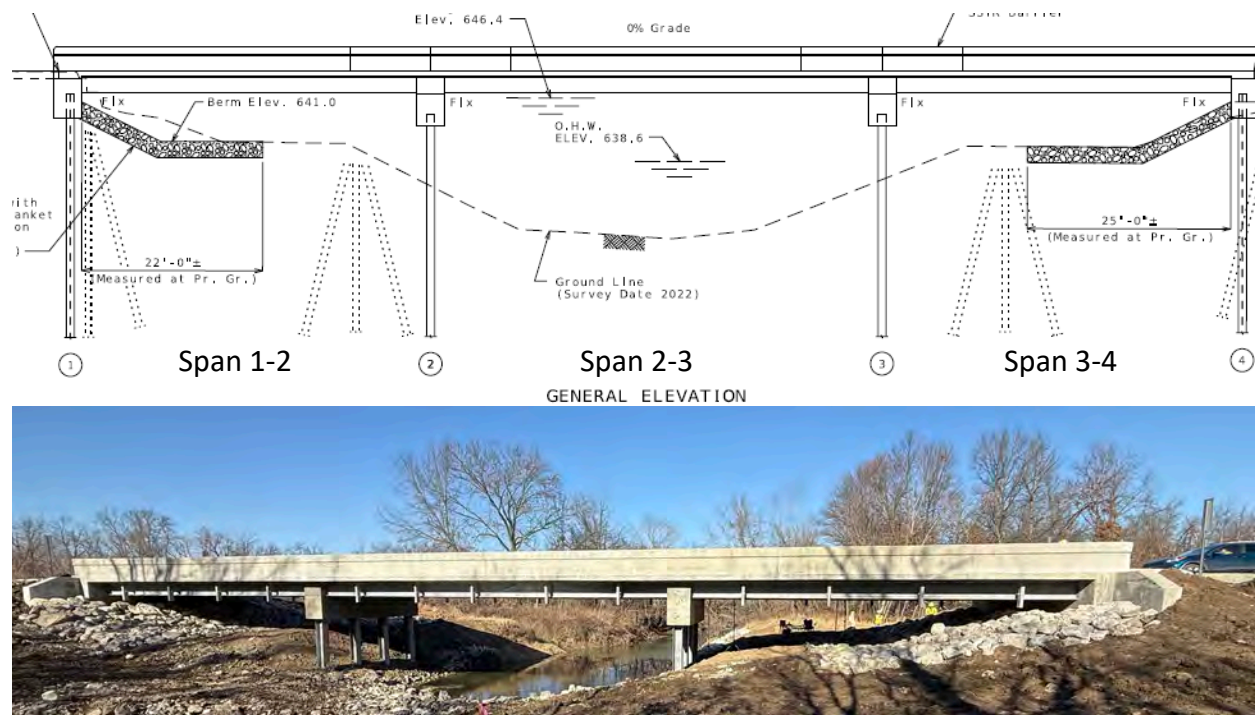


Figure 3-1. Elevation schematic and photo of bridge A9513, span 2-3 and 3-4 were tested.

3.1 Bridge Design

Bridge A9513 was designed using the SDCL concept. This type of bridge design is used to improve continuity across spans in a multi-span bridge. In this design approach, a concrete diaphragm is constructed at intermediate bents to provide continuity throughout a simple span bridge without using bolted field splices. The system relies on strain compatibility between the concrete deck, steel girders, and continuity reinforcement to transfer forces at the supports. Figure 3-2 illustrates the construction sequence for an intermediate bent connection. After the wide flange girders are set on the bearing pads (see Figure 3-3), a concrete diaphragm is partially poured between the girders to begin establishing continuity (see Figure 3-4). The

remainder of the diaphragm is filled during the deck placement. The continuity reinforcement and dowel bars are not shown in the diagram.

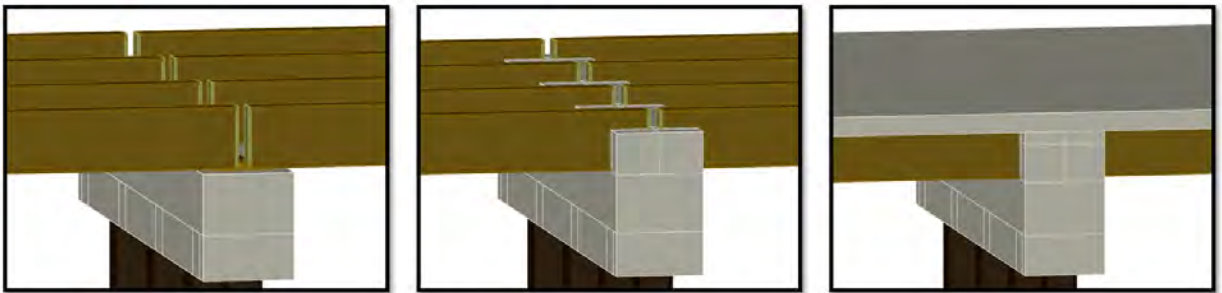


Figure 3-2. SDCL intermediate diaphragm construction sequence: Girders set on bearing pads (left), partial diaphragm pour (middle), and final diaphragm/deck placement (right).



Figure 3-3 Steel girders on bent cap before first diaphragm pour.



Figure 3-4. Concrete diaphragm at bent no. 3 before the concrete deck pour.

3.2 Experimental Testing

Experimental testing was conducted to evaluate the structural behavior of Bridge A9513 during both the construction stage and the completed bridge condition. Two testing programs were performed. The first program measured the bridge response during deck placement to assess construction-stage behavior under dead load. The second program consisted of controlled live-load testing with loaded dump trucks to evaluate the structural response after continuity had developed. The instrumentation, testing procedures, and loading configurations used in each program are described in the following sections.

3.2.1 Dead Load Testing

The dead load testing took place on the day the concrete deck was poured. The instrumentation for the dead load test included six strain transducers (STs) to measure strain in the steel girders, two linear variable displacement transducers (LVDTs), and a string pot (SPOT) to measure deflections. The accuracy of the STs was about $10\ \mu\epsilon$, and the accuracy of the LVDTs was about 0.005 in.

The instrumentation locations for the dead load test are shown in Figure 3-5 and Figure 3-6. The STs were placed beneath the top and bottom flanges of an interior girder on both sides of the concrete diaphragm at bent no. 3, as well as at midspan of span 3-4. The placement of the STs on the girder is shown in Figure 3-7. One LVDT was positioned at midspan of span 3-4 while the other was placed 62 in. to the right of intermediate bent no.3. The LVDT near bent no. 3 was installed as close to the concrete diaphragm as the slope of the hill and temporary shoring allowed. Both LVDTs were placed on an interior girder for dead load testing. The string pot was installed at the midspan of Span 2-3, as shown in Figure 3-9. Since Span 2-3 crosses the Middle Fork Little Chariton River at a height of about 14 ft over the creek, an LVDT could not be

installed at that location. The string pot was also not able to be directly under the girder and was placed about 4ft out-of-plane of the girder. Data from the string pot was corrected for the angle of measurement.

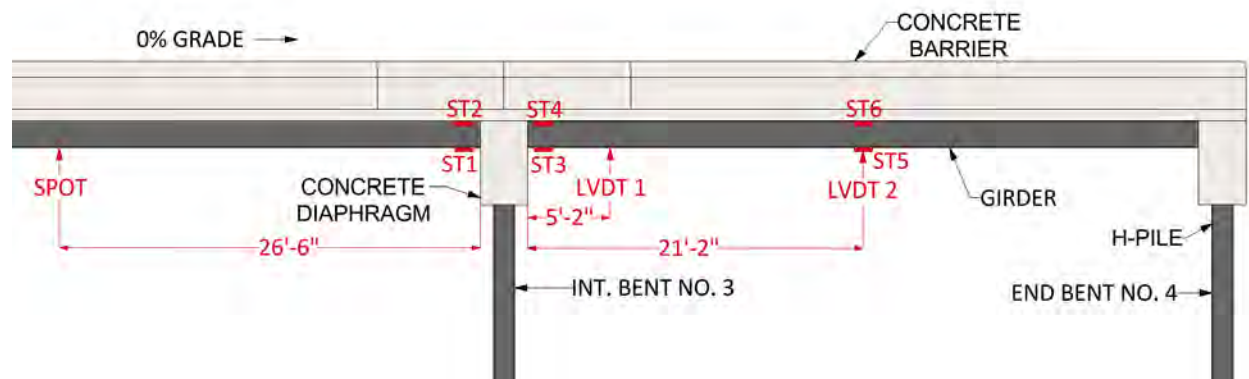


Figure 3-5. Elevation view of instrumentation locations for dead load testing.

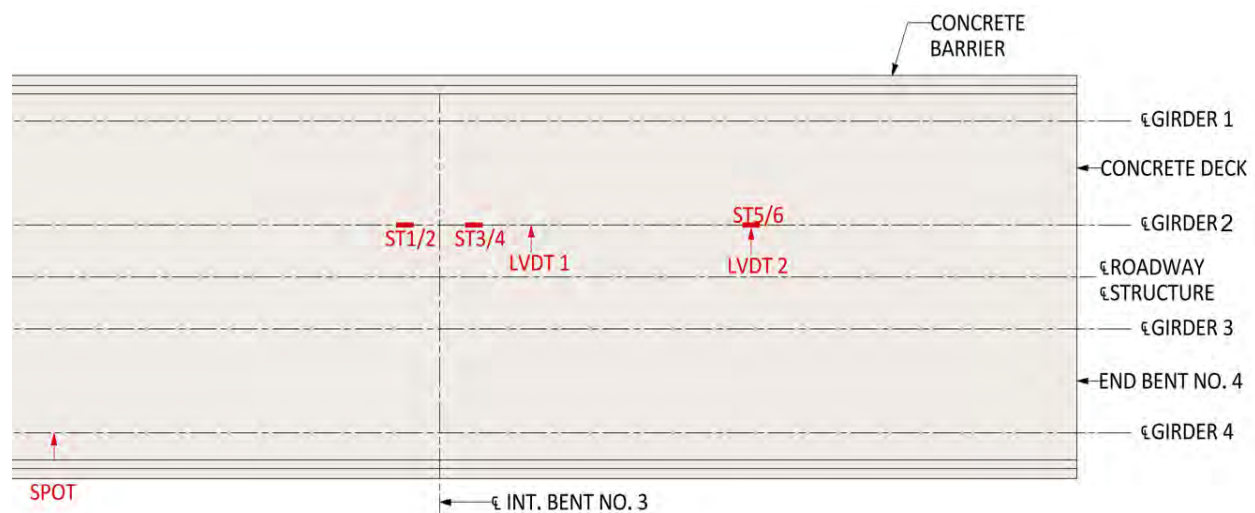


Figure 3-6. Plan view of instrumentation locations on the substructure for dead load testing.



Figure 3-7. Strain transducer placement on girders.



Figure 3-8. Picture of ST and LVDT placement for dead load testing.



Figure 3-9. String pot placement at midspan for dead load testing.

Before the concrete pour began, a baseline measurement was recorded on all devices, and several concrete cylinders were made from the deck material. The concrete cylinders were later tested for compressive strength, and the results were used for the deck strength in the FE model. The deck pour began on end bent no. 4 and worked across the bridge (Figure 3-10). Measurements were recorded during the concrete pour for spans 3-4 and 2-3. Measurements were not collected during the pour of Span 1-2 as its weight was not expected to significantly affect the strain or displacement of the concrete diaphragm at intermediate bent no. 3.

On the day of the dead load testing, the ambient temperature was approximately 40°F while the concrete temperature was near 70°F. This temperature difference was investigated in the FE model, as it may have influenced the deformation of the steel members.



Figure 3-10 Deck pour starting at bent 4.

3.2.2 Live Load Testing

Instrumentation for the live load test consisted of eight STs, four LVDTs, and one string pot. Instruments were placed in some of the same locations on the substructure as the dead load test, shown above. The live load testing also included two more LVDTs positioned on the exterior girder at midspan of span 3-4, and 9 in. right of intermediate bent no. 3. The location of the LVDTs can be seen in Figure 3-11 and Figure 3-12. The two additional STs were installed on the concrete deck for the live load test, as shown in Figure 3-13 and Figure 3-14. The new STs were positioned above the webs of an exterior and an interior girder.

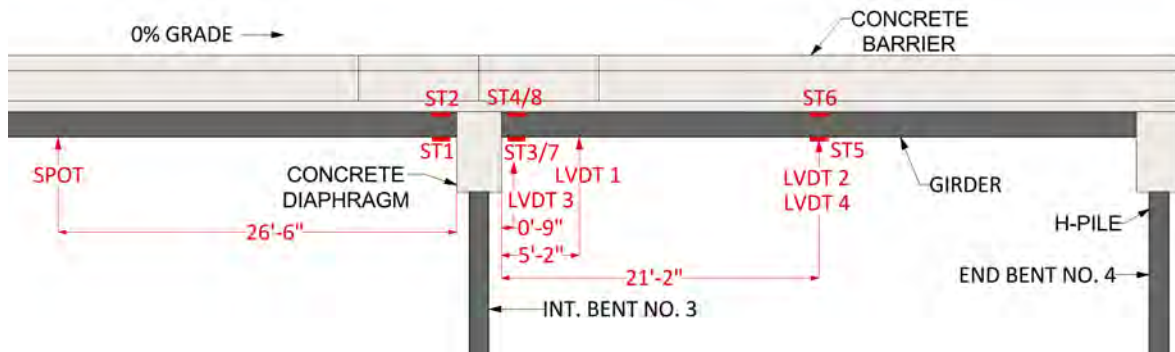


Figure 3-11. LVDT locations for live load testing.

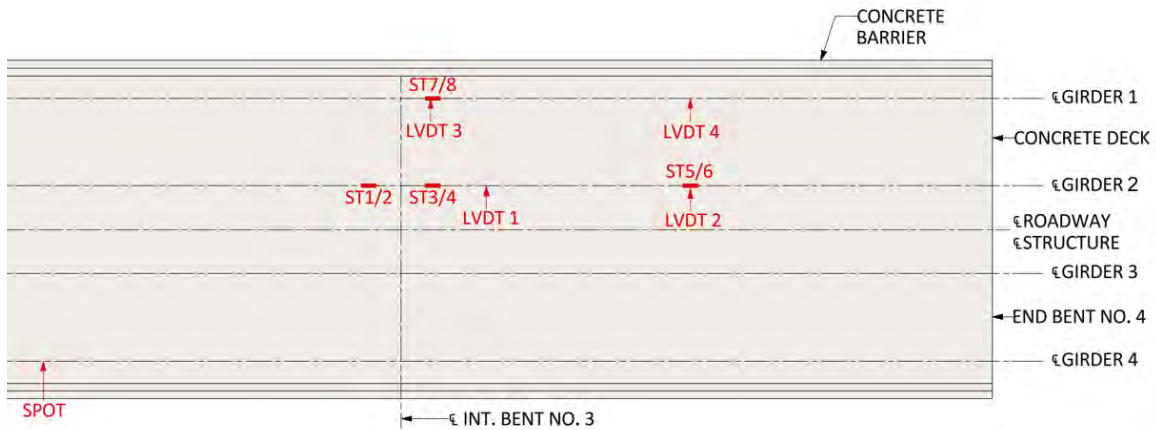


Figure 3-12. Plan view of instrumentation locations on the substructure for live load testing.

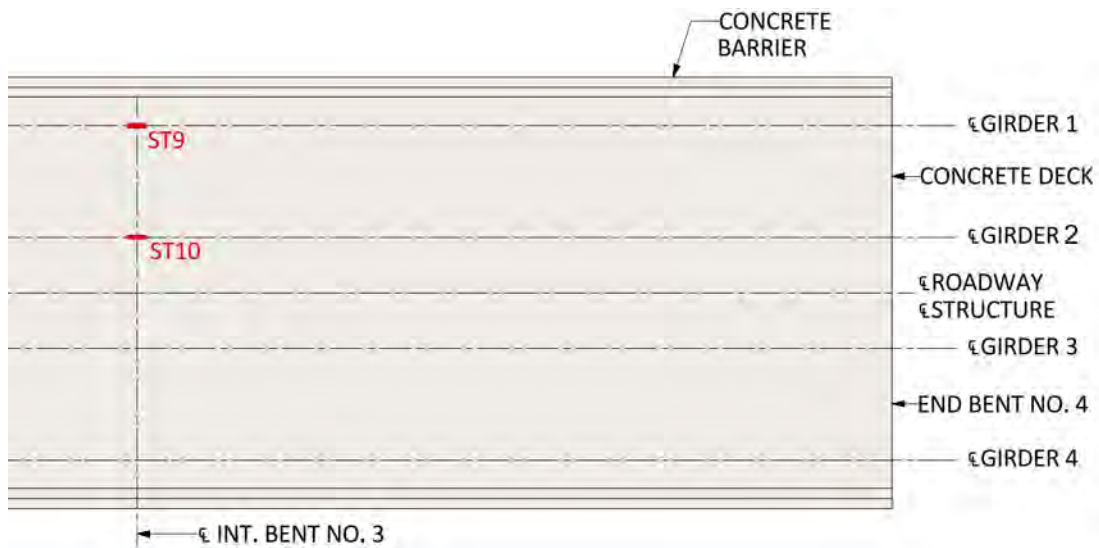


Figure 3-13. Plan view of ST locations on the superstructure for live load testing.



Figure 3-14. ST placement on bridge deck for live load testing.

The live load testing was conducted using two dump trucks. The dump trucks were positioned at six locations to measure bridge strain and deflection under live load and provide sufficient data to validate the FE model. Table 3-1 shows the axle load distribution for the two dump trucks and Figure 3-15 illustrates the truck dimensions. The axle weight and dimensions were used to accurately model the dump truck live load in the FE model.

Table 3-1. Truck weight per axle.

	Truck 1	Truck 2
Loaded Front Axle	21,500 lb	16,840 lb
Loaded Rear Axle	52,800 lb	57,180 lb

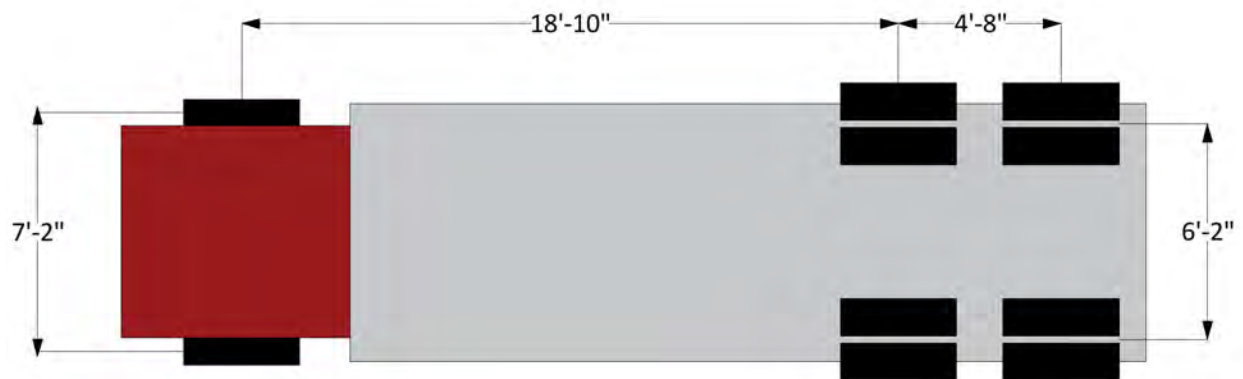


Figure 3-15. Truck dimensions.

The six truck locations used during the live load testing include the following: (1) One truck at midspan of span 3-4, (2) Both trucks at midspan of span 3-4, (3) One truck at midspan of span 3-4 and the other truck at midspan of Span 2-3, (4) Both trucks at midspan of Span 2-3 in different lanes, (5) One truck at midspan of span 2-3, (6) Both trucks in the same lane of Span 2-3. The six truck locations are shown in Figure 3-16 and Figure 3-17 demonstrates truck placement during testing. Trucks were positioned at midspan to produce large deflections and strain responses, and to evaluate the fixity of the concrete diaphragm under maximum moment conditions. The trucks were only positioned on Spans 2-3 and 3-4 due to the instrumentation placement around intermediate bent no. 3.

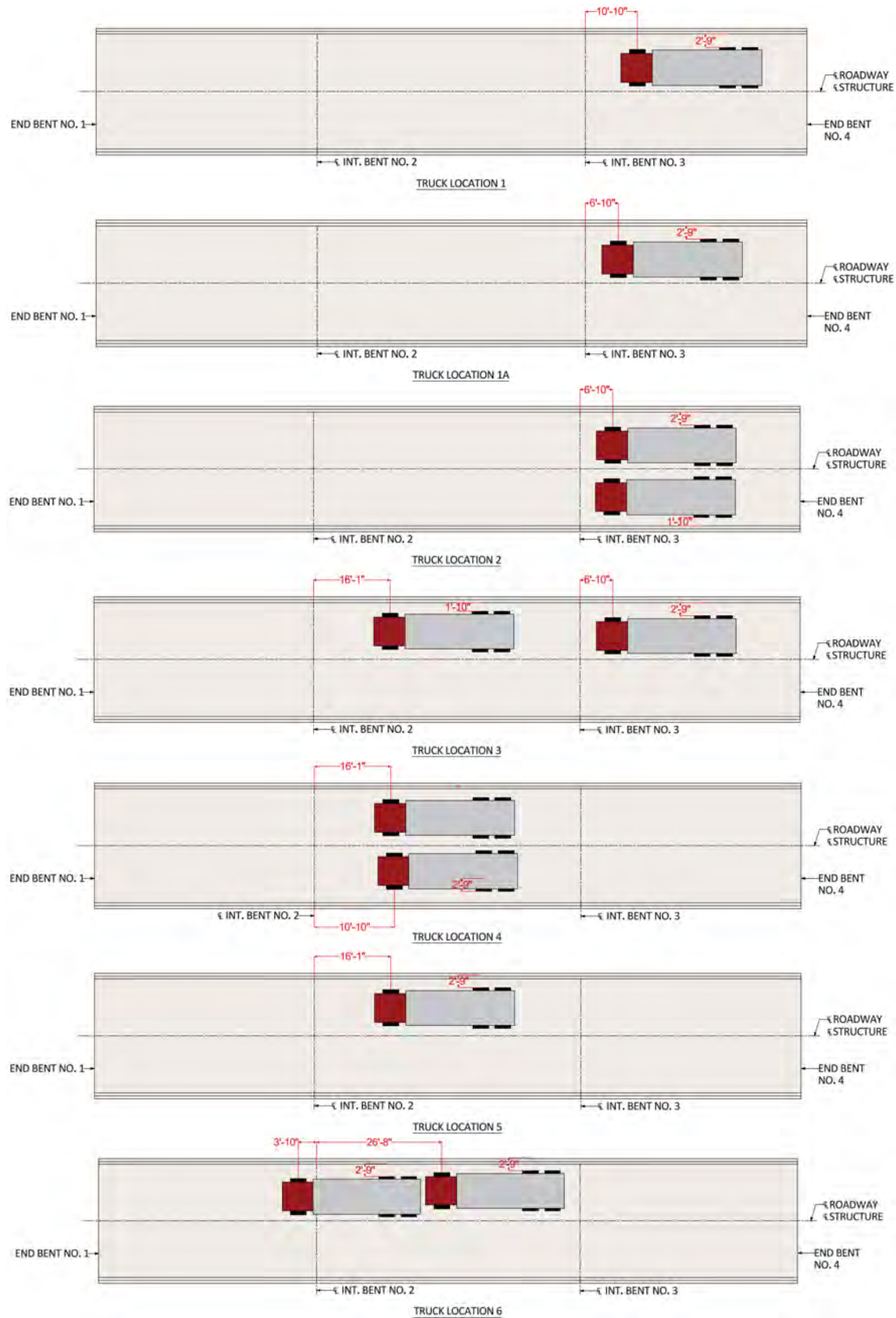


Figure 3-16. Truck locations for live load testing.



Figure 3-17. Truck loading during live load testing.

A baseline measurement was made before the trucks moved onto the bridge. Then strain and displacement measurements were recorded for each truck position shown above. The exact location of each truck was also noted throughout testing for the FE model.

3.3 Experimental Results

This section presents the experimental results obtained from the dead-load testing performed during deck placement and the live-load testing conducted using controlled truck loading. The measured strains and deflections are used to characterize the behavior of Bridge A9513 under dead and live loading and to establish the experimental data used for FE model calibration.

3.3.1 Dead load test results

Data for dead load testing was recorded continuously from the start of the pour at about 11:16 am until about the completion of Span 2-3 at about 1:30pm. Figure 3-18 shows the displacement results with time. Key points of the pour progress are highlighted in the figure. The steplike pattern in the deflection vs. time data is due to pauses during the pour as there was waiting while more concrete was delivered. During the pour of span 3-4 the center Span 2-3 moves upward with a maximum deflection of about 0.1 in. This shows some continuity between the spans as the load in span 3-4 is causing deflections in Span 2-3. The deflections of span 3-4 reach their maximum downward deflection around the completion of the pour for that span with a deflection of about 0.13 in near the support (LVDT 1) and 0.35 in at the midspan (LVDT 2). These deflections decrease as Span 2-3 is poured. Span 2-3 reaches its maximum deflection of about 0.82 in. when that span pour is completed.

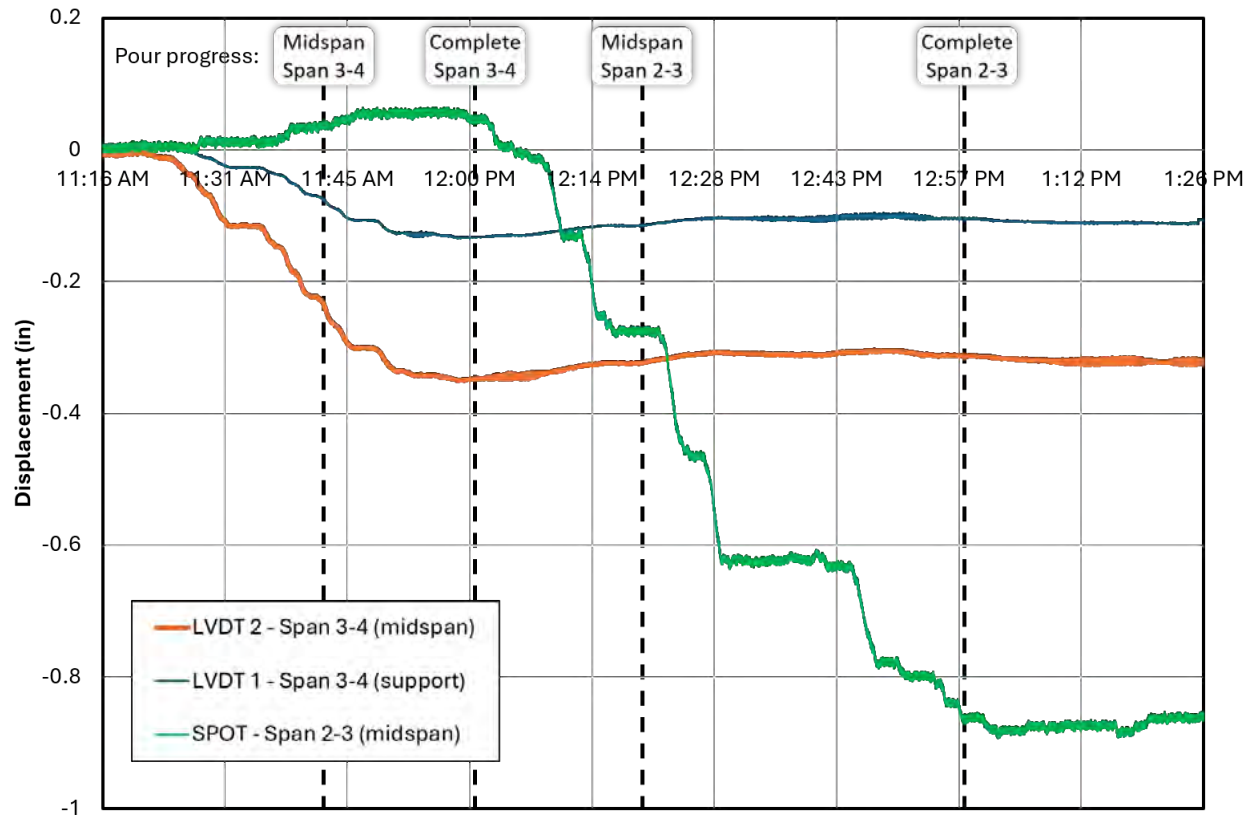


Figure 3-18. Displacement results from dead load testing.

The results of the strain data are shown in Figure 3-19. In the figure tension is positive and compression is negative. At the completion of span 3-4 the greatest strains are at the midspan of span 3-4 (ST6 and ST5) indicating positive bending at midspan as expected. ST3 and ST1 show compressive strain throughout the pour as would be expected from the negative moment developed near the support. The value of the strain (around $-90 \mu\epsilon$ at completion of span 3-4 in ST3) shows the result of the continuity near the support and likely the compression (about 3 ksi) that would be present in the steel end blocks. ST4 and ST2 at the top flanges show initially tension strain that transitions to compression strain during the pour. The strain is less as the top flange was not cast in the concrete diaphragm before pouring. In addition, the temperature of the concrete was about 70 degrees F while the girders were near 40 degrees on the day of the pour. The warm concrete on the cold steel may have affected the strain readings at the top flanges.

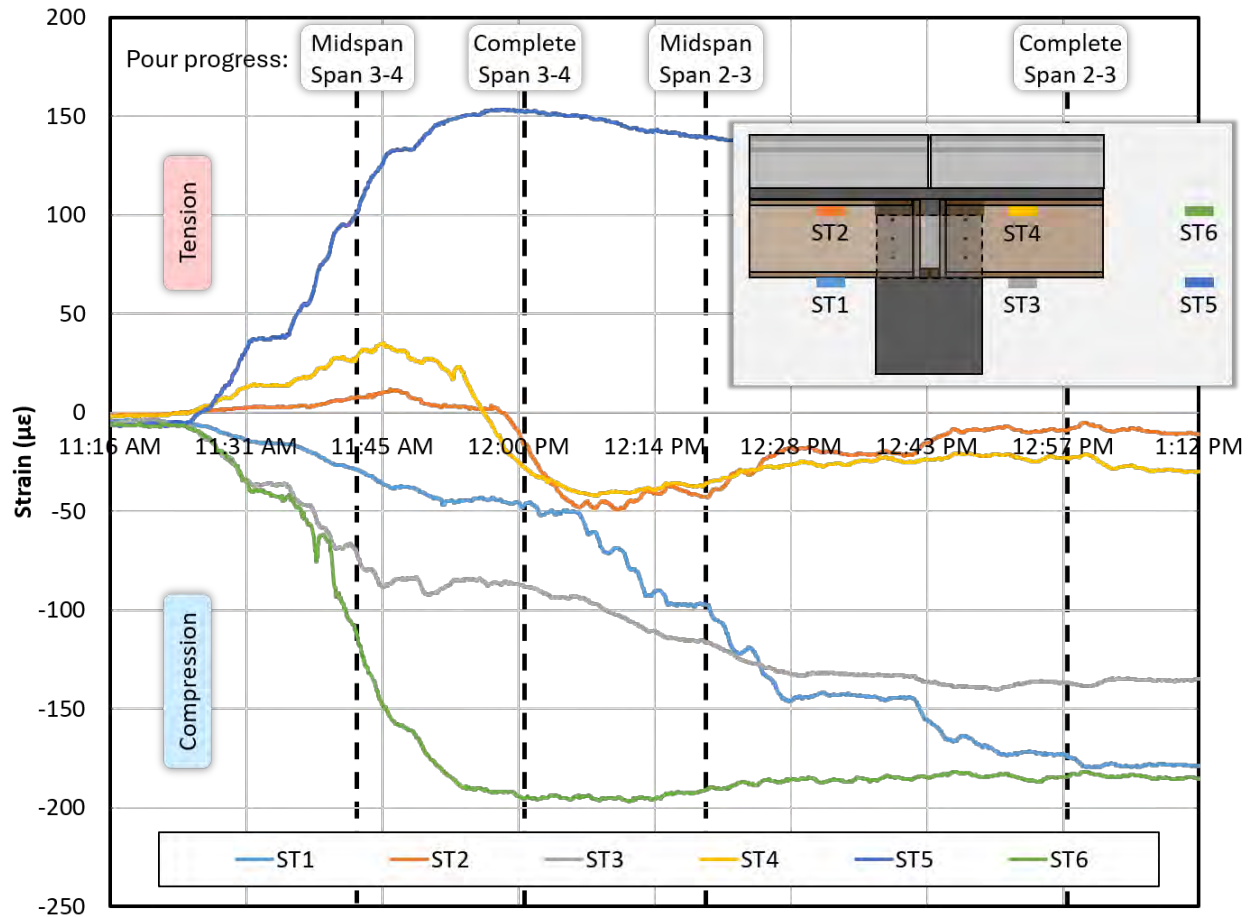


Figure 3-19. Strain results during dead load testing.

3.3.2 Live Load Test Results

The strain results from live load testing are given in Table 3-2, whereas the displacement results are shown in Table 3-3. Positive strains indicate tension and positive displacements indicate upward movement. Additional figures are drawn and compared to the FE model results in Chapter 4. Overall, the results of the live load testing show the expected strains and displacements as the trucks moved across the bridge. The largest strains were measured in ST7 which was located at the bottom of the exterior girder near the support with the largest strain ($-116 \mu\epsilon$) occurring in truck position 3 when trucks were in both spans, as expected as this would be when the greatest negative moment would occur. The largest displacement (0.17 in. downward) occurred at the midspan of Span 2-3 at truck position 4 when both trucks were centered on that span, as expected. At the same truck position (i.e., truck position 4) LVDTs 2 and 4 (at the midspan of Span 3-4) moved upward about 0.03 in. demonstrating the continuity across the bent.

Table 3-2. Strain results from live load testing

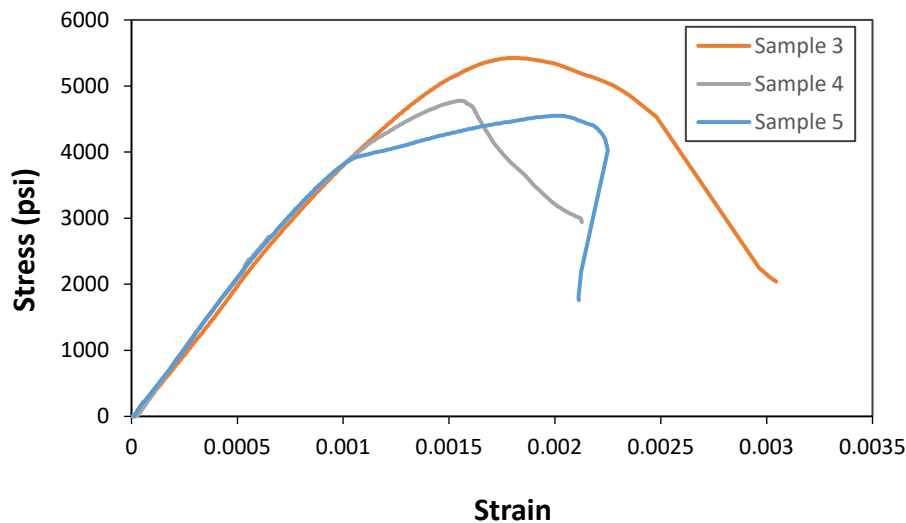
Truck Position	Strain ($\mu\epsilon$)									
	ST1	ST2	ST3	ST4	ST5	ST6	ST7	ST8	ST9	ST10
1	-28.9	-3.4	-16.8	-5.8	47.3	11.5	-47.5	-6.6	6.7	22.3
1a	-31.9	-3.7	-16.3	-7.5	69.7	14.8	-48.2	-9.5	5.3	20.4
2	-54.1	-5.0	-18.4	-11.5	115.9	30.6	-43.3	-10.4	1.9	22.6
3	-97.7	-10.7	-21.9	-13.2	60.9	13.1	-116.4	-23.1	18.2	30.4
4	-96.7	-12.2	-20.0	-8.6	-22.3	-3.1	-98.1	-15.2	10.4	28.7
5	-66.8	-7.9	-7.8	-4.5	-12.2	-1.6	-64.4	-10.4	33.6	82.9
6	-73.9	-11.9	-11.8	-8.4	-17.5	-4.3	-80.5	-15.8	234.0	50.4

Table 3-3. Displacement results from live load testing

Truck Position	Displacement (in)				
	LVDT1	LVDT2	LVDT3	LVDT4	SPOT
1	-0.029	-0.073	-0.017	-0.073	0.003
1a	-0.032	-0.081	-0.020	-0.079	0.012
2	-0.056	-0.137	-0.026	-0.107	0.012
3	-0.026	-0.071	-0.024	-0.070	-0.029
4	0.0199	0.0341	0.0015	0.0312	-0.1739
5	0.0092	0.0169	-0.0034	0.0159	-0.0447
6	0.0091	0.0202	-0.0076	0.0183	-0.0692

3.4 Concrete Material Test Results

Concrete cylinders were cast during the dead load testing and tested following ASTM C39 and ASTM C469 the day after the live load test (52 days). The results, shown in Figure 3-20, show a compressive strength of about 4850 psi and elastic modulus 4250 ksi of at the time of live load testing.

**Figure 3-20. Concrete compressive stress strain results.**

3.5 Summary

The field testing of the bridge under dead (concrete self-weight) and live load went well and produced expected results. During the dead load testing, displacement measurements showed an upward movement in Span 2-3 during pour of Span 3-4, indicating some continuity in the dead load condition. A compressive strain of around $-90\ \mu\epsilon$ at completion of span 3-4 in ST3 shows continuity near the support and likely the compression (about 3 ksi) that would be present in the steel end blocks. The maximum deflection of Span 2-3 was 0.86 in., which was in line with what was expected.

The live load testing also demonstrated continuity with a maximum downward deflection in Span 2-3 of 0.17 in. and upward deflection in Span 3-4 of approximately 0.03 in. when the trucks were centered in Span 2-3. The highest strain readings were about $116\ \mu\epsilon$, or 5% of yield. The small amount of deflection and strain under a heavy truck load also demonstrates the efficiency of the design.

Chapter 4. Finite Element Modeling

This chapter describes the FE models developed to evaluate the behavior of Bridge A9513 discussed in the previous chapter. The purpose of numerical modeling is to provide a reliable tool for interpreting experimental measurements, evaluating the degree of continuity developed at the intermediate bents, and establishing a numerical testbed suitable for conducting parametric analyses that support the design recommendation for MoDOT's Engineering Policy Guide (EPG).

Two FE models were developed in ANSYS Mechanical using bridge geometry, support conditions, construction sequence, and loading configurations associated with the experimental program. The first model, referred to here as the dead-load model, represented the construction-stage dead-load condition during deck placement, when the steel girders carried the wet concrete deck load before composite action fully developed. The second model, referred to here as the live-load model, represented the completed bridge under live-load testing, after the deck and continuity diaphragms had hardened and the bridge was expected to exhibit continuous behavior under live loading.

The remainder of this chapter discusses the specific geometry of Bridge A9513 as well as details of the dead-load and live-load models alongside the specific calibration procedures employed for each model. The chapter then discusses the dead-load and live-load model results and finalizes with a discussion of the degree of continuity that Bridge A9513 experienced during construction and after completion.

4.1 Description of the Test Bridge and 3D Computer-Aided Design (CAD) Modeling

Bridge A9513 is a three-span, non-skewed steel girder bridge with span lengths of 45 ft, 56 ft, and 45 ft, as shown in Figure 4-1. The bridge superstructure consists of four steel girders supporting a reinforced concrete deck. The girder spacing is 6 ft 10 in. on center across the bridge width, as shown in Figure 4-2. The end spans use W24×84 girders, while the middle span uses W24×94 girders. The girder layout and girder-end details are shown in Figure 4-3. Continuity between adjacent spans is provided through concrete diaphragms at the intermediate bents, along with girder-end connection details that allow the bridge to behave as simple for non-composite dead load and continuous for live load. The slab section and diaphragm details near the intermediate bents are shown in Figure 4-4.

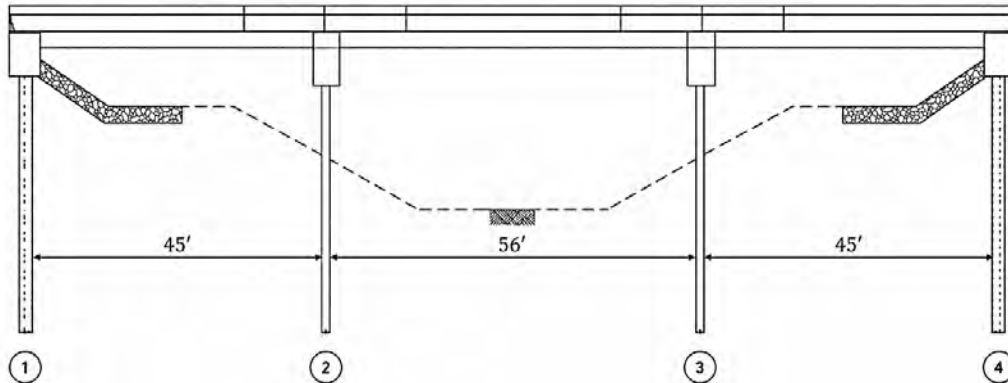


Figure 4-1. General elevation of Bridge A9513 showing 45 ft–56 ft–45 ft span arrangement.

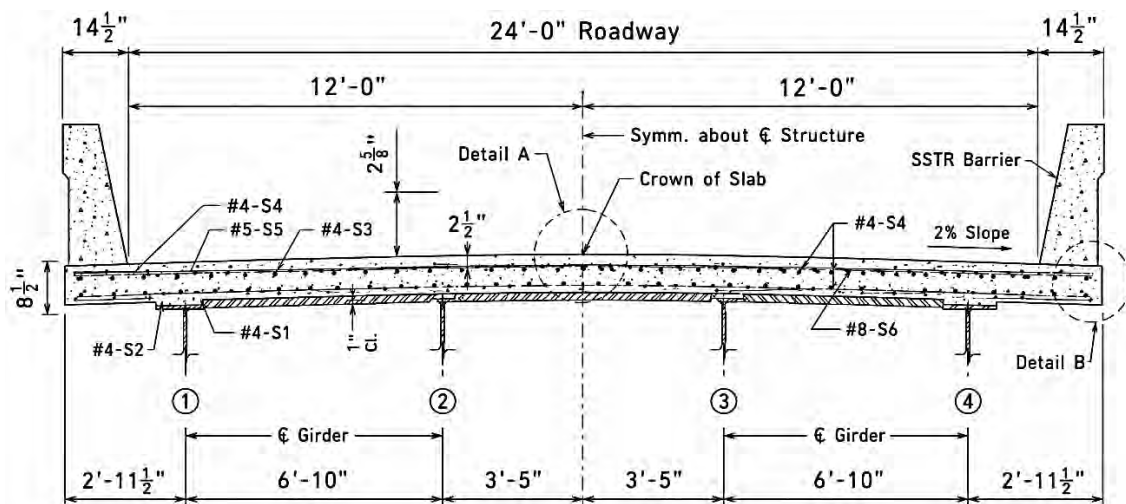


Figure 4-2. Bridge cross section showing four girders and girder spacing.

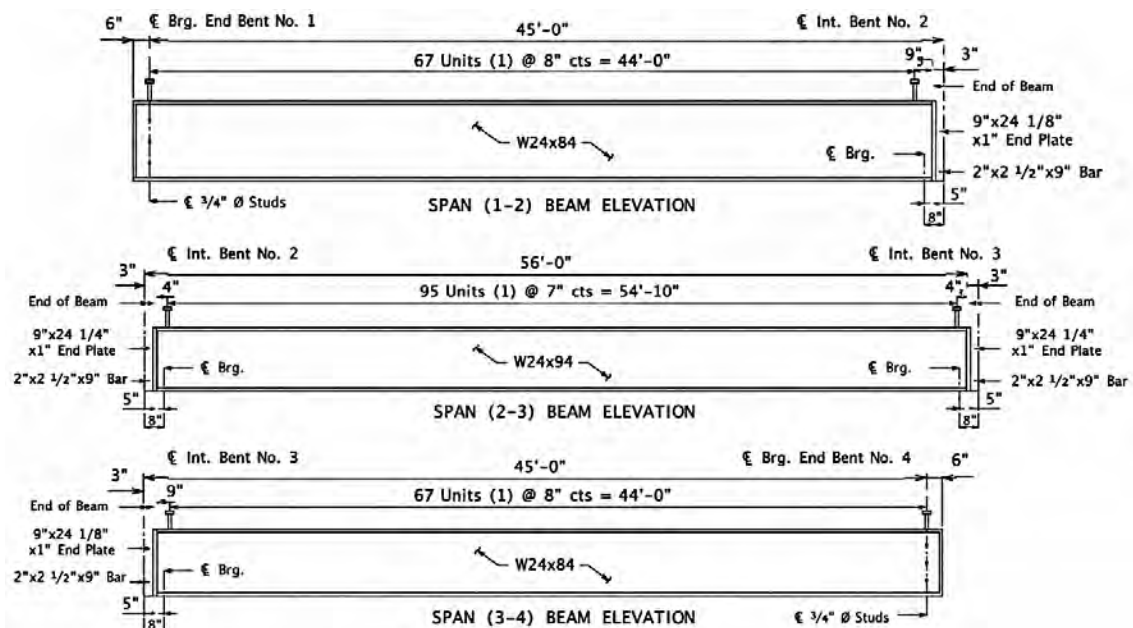


Figure 4-3. Girders and girder-end details at intermediate bents.

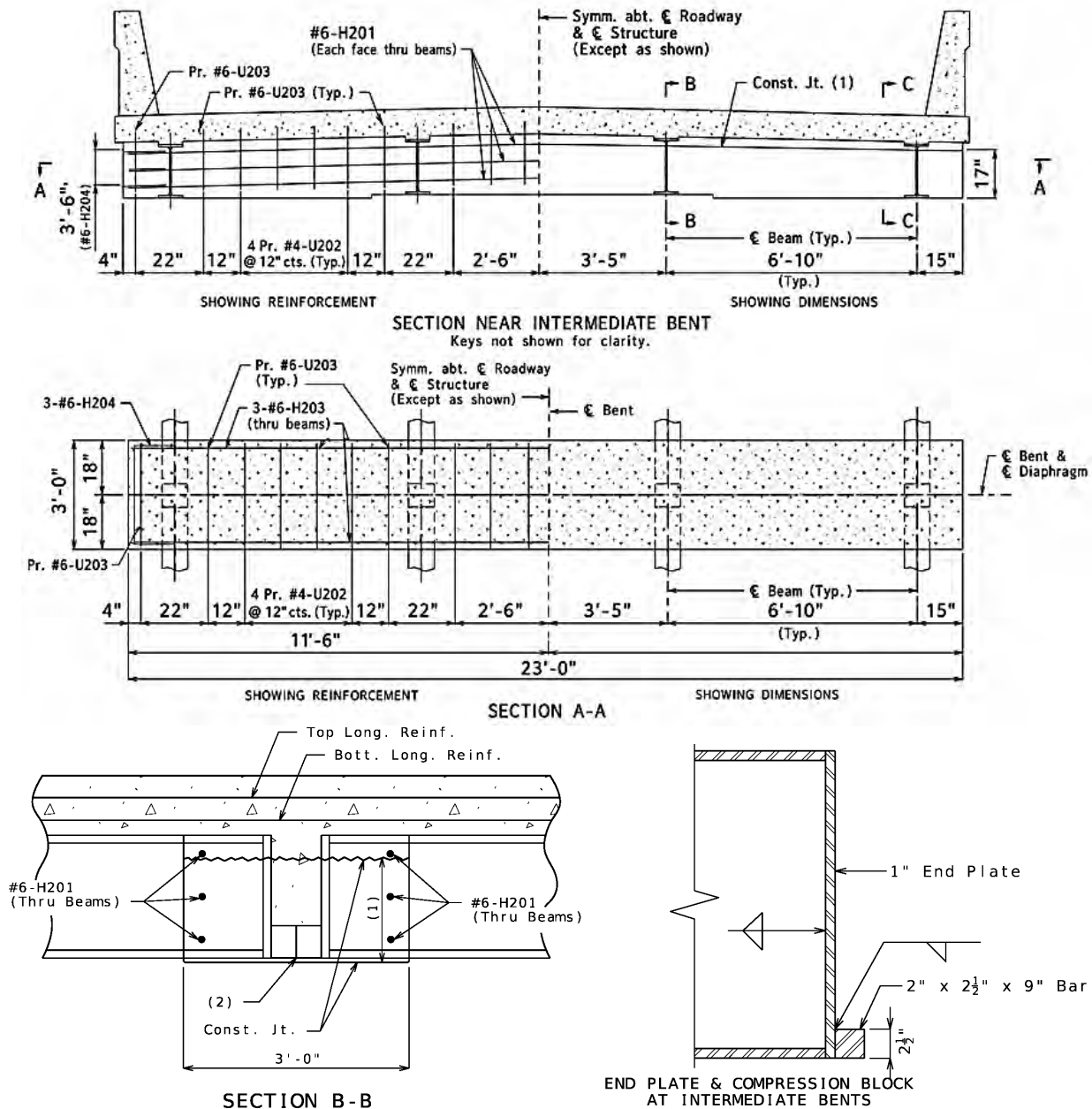


Figure 4-4. Slab section near intermediate bent (top), diaphragm detail (middle), and SDCL connection detail above girders (bottom).

The finite element models were developed from the construction drawings shown in Figure 4-1 through Figure 4-4 (additional construction drawing details are provided in Appendix A). These drawings were used to create three-dimensional CAD models of the bridge, including the steel girders, reinforced concrete deck, concrete barriers, intermediate diaphragms, girder-end details, bent caps, and pile-supported foundations. Several geometric simplifications were introduced to facilitate meshing and improve computational efficiency. Secondary features, small fillets, and minor geometric irregularities were omitted from the CAD models. In addition, the deck was modeled without the 2% cross slope shown in Figure 4-2. This simplification was

considered acceptable because the cross slope was not expected to significantly affect the primary response mechanisms of interest, including force transfer at the intermediate bents and the degree of continuity developed by the bridge. The completed, simplified CAD model used to construct the finite element models is shown in Figure 4-5.

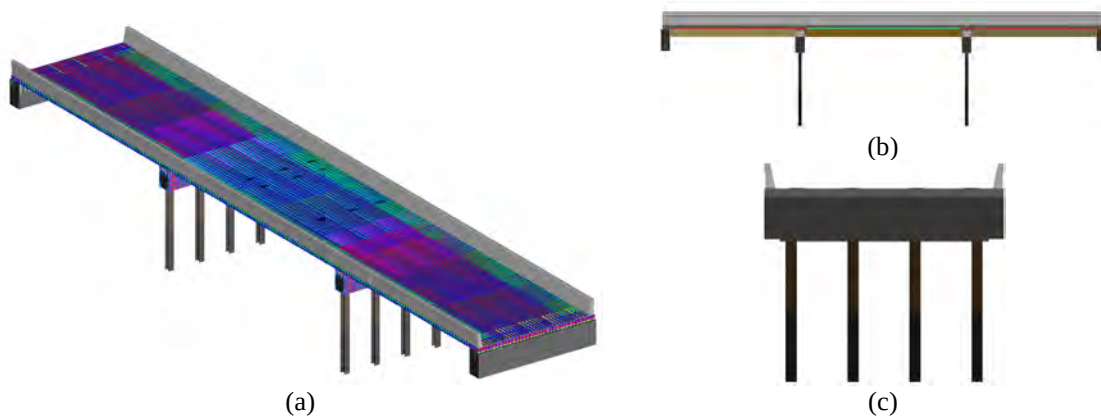


Figure 4-5. Details of the 3D CAD drawings used to construct the ANSYS models of the test bridge: (a) three-dimensional view, (b) side view, and (c) front view.

In the FE models, particular attention was given to the regions near the intermediate bents because these locations govern moment transfer and continuity behavior in the SDCL system. Details of the components modeled at the intermediate bent locations are summarized in Figure 4-6. The main components included compression blocks, end plates, concrete diaphragm, concrete cap, and H-piles. These components were modeled explicitly to capture the force-transfer mechanisms at the interior bents with sufficient accuracy.

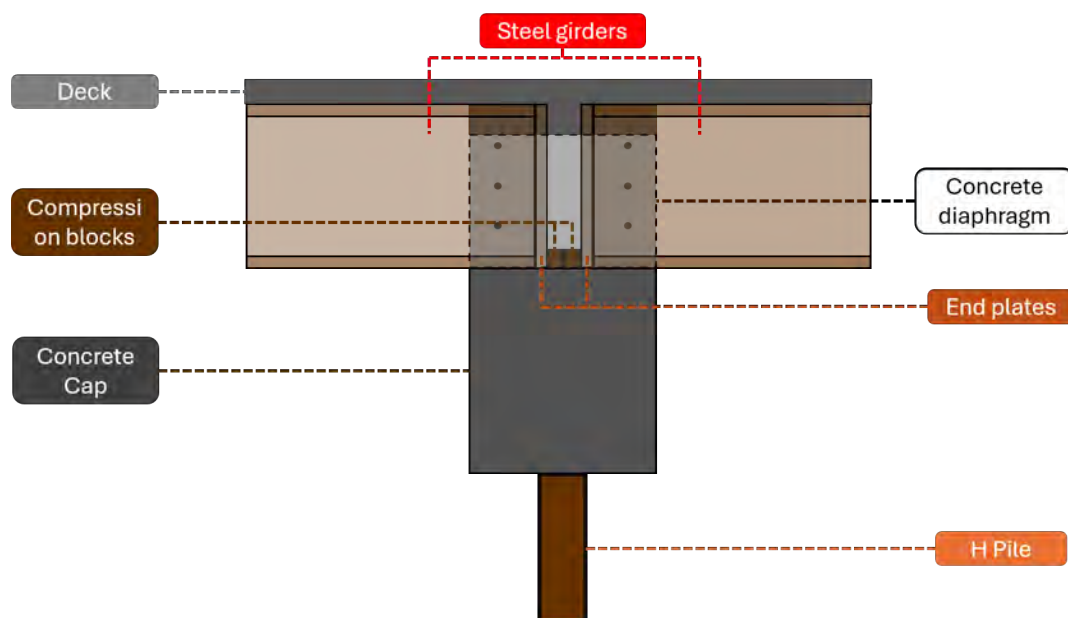


Figure 4-6. Intermediate bent detail summarizing the main components considered in the FE model of the test bridge.

4.2 Finite Element Modeling Approach, Scope, and Assumptions

This section describes the finite element modeling approach used for the dead-load and live-load models. It summarizes the main modeling assumptions, model geometries, material properties, contact definitions used to represent interactions between components, as well as calibration strategies and key limitations of each model.

4.2.1 General Finite Element Modeling Assumptions

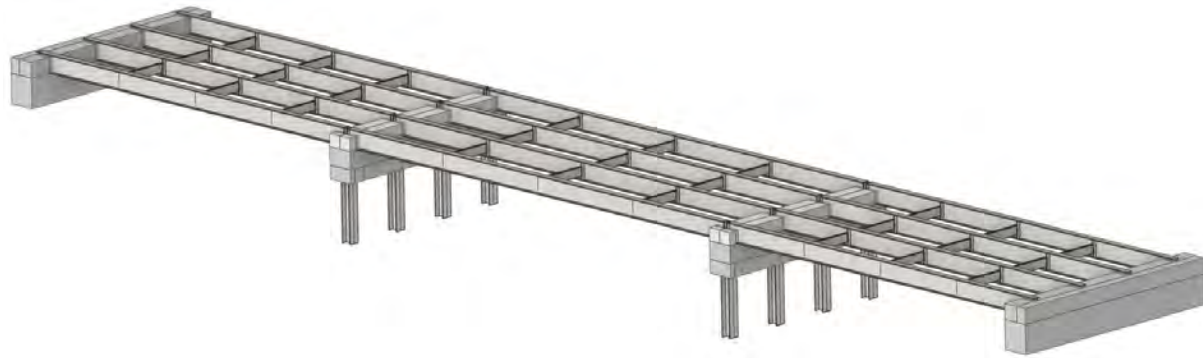
The modeling approach was selected to balance accuracy and practicality. The objective was not to reproduce every local detail of the bridge, but rather to represent the structural features that most strongly influence the measured strains, curvatures, deflections, load distribution, and moment continuity. Particular attention was given to the regions near the intermediate bents, where force-transfer mechanisms and local stiffness conditions have a direct influence on the global bridge response. Secondary geometric features that were not expected to significantly affect the overall behavior were simplified to facilitate model development and meshing.

All materials in the models were assumed to behave as linear elastic. This assumption was considered appropriate because the goal was to simulate the bridge response during dead-load and live-load testing, for which damage was not expected to occur. The models were calibrated to match the experimental responses from these two tests and then used to evaluate whether the bridge behaved primarily as a simply supported system during construction and whether continuity developed after completion.

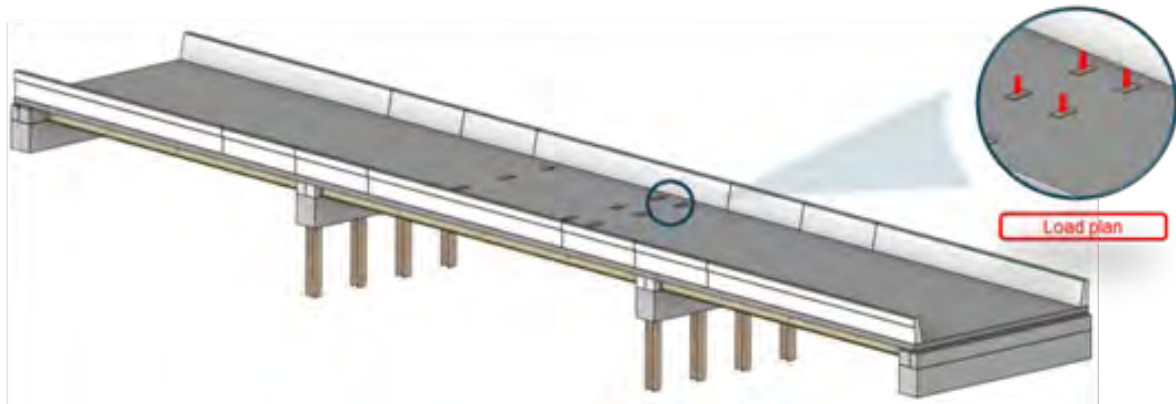
Considering these assumptions, the results from the models are most appropriate for interpreting global response quantities such as strains, curvatures, deflections, load distribution, support behavior, and degree of continuity. The models are not intended to serve as detailed local strength or damage models of the connection region, as they were not used to predict local cracking, crushing, yielding, or ultimate failure of the diaphragm or connection components. This distinction is important when interpreting the numerical results.

4.2.2 Dead-Load and Live-Load Model Geometries

The dead-load and live-load models were developed in ANSYS Mechanical. General views of the dead-load and live-load models are shown in Figure 4-7. The dead-load model did not consider the deck or concrete barriers, whereas the live-load model included all components discussed before.



(a)



(b)

Figure 4-7. General view of the ANSYS models: (a) dead-load model and (b) live-load model.

Figure 4-8 provides a detailed view of the main components included in the live-load model near the intermediate bents. The top-left figure shows the bent cap region, including the cap reinforcement and shear key. The top-right figure provides a closer view of the intermediate bent region, showing the girder-end details, compression blocks, end plates, bearing pads, and deck reinforcement around the diaphragm. The bottom figure shows a more global perspective of the model, including adjacent spans, concrete barriers, deck, girders, bent cap, and H-piles. As shown in the bottom figure, the expansion joints in the barrier curbs were modeled explicitly. Together, these views illustrate the level of geometric detail used in the live-load model to represent the components controlling force transfer, support restraint, and continuity behavior at the intermediate bents.

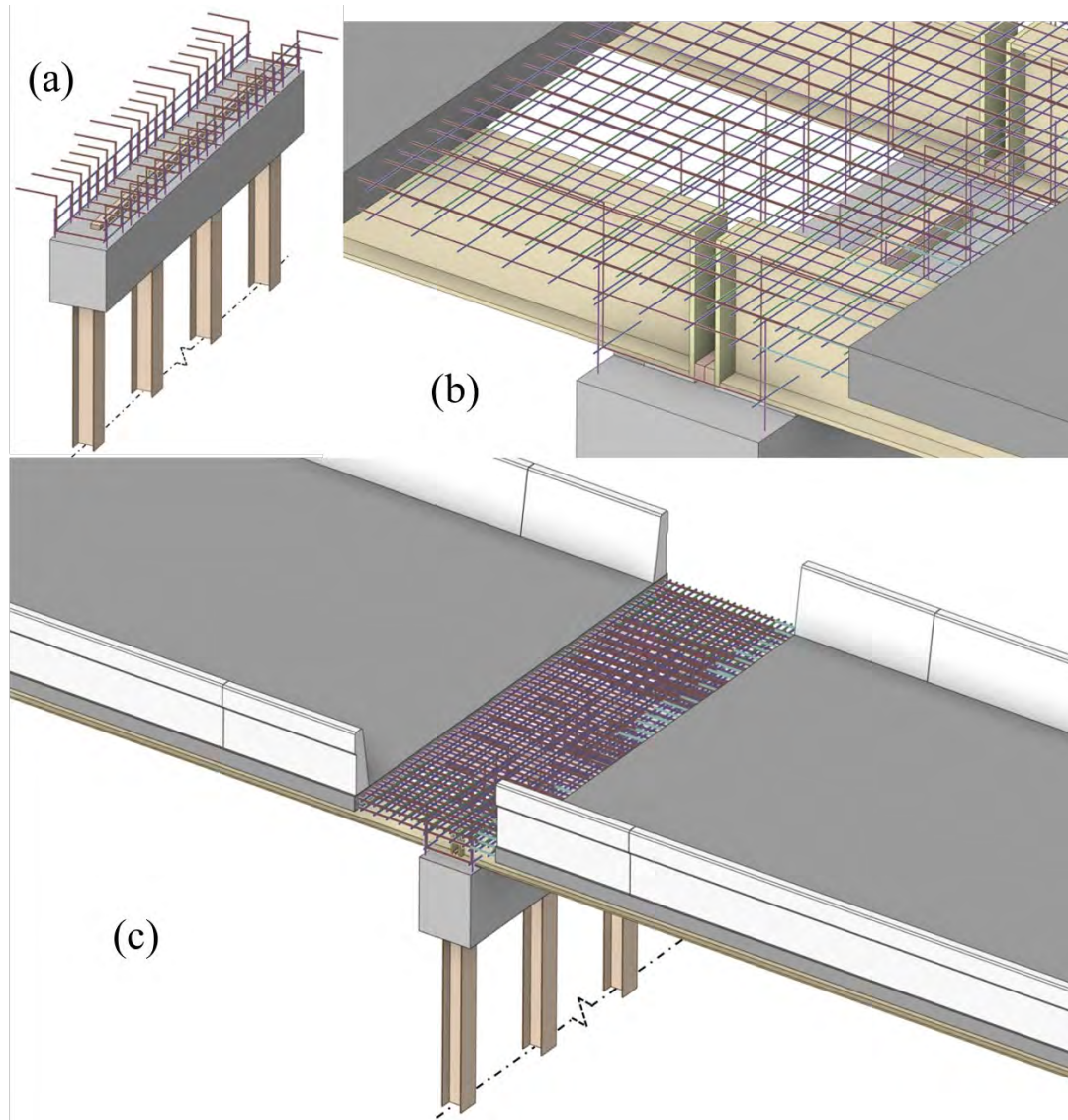


Figure 4-8. Detailed view of the live-load model: (a) intermediate bent reinforcement, (b) girder placement and deck reinforcement, and (c) overall view of the intermediate bent.

4.2.3 Material Properties

As discussed before, all materials were modeled as linear elastic. Table 4-1 summarizes the material properties used in the FE models. Material strengths and elastic properties were selected based on the bridge design specifications and construction records. Structural steel components, including the girders, diaphragms, end plates, compression blocks, and HP piles, were modeled using ASTM A709 Grade 50 steel. Reinforcing steel was modeled using ASTM A615 Grade 60 properties. Separate concrete properties were assigned to the superstructure and substructure to reflect the different concrete classes used in construction. The compressive concrete strength for the superstructure was 5.6 ksi, which resulted in a Young's modulus of approximately 4265 ksi, closely matching the Young's modulus of 4250 ksi measured experimentally in Section 3.4. The neoprene bearing pads and tire loading surfaces were

assigned appropriate elastic properties to represent their stiffness characteristics and influence on load transfer.

Table 4-1. Material properties used for the different components of the test bridge model.

Material	Specification	Property	Value (ksi)
Structural Steel (Girders, Diaphragms, Plates, etc.)	ASTM A709 Grade 50	Yield Strength, f_y	50
		Modulus of Elasticity, E	29,000
Structural Steel HP Piles	ASTM A709 Grade 50	Yield Strength, f_y	50
		Modulus of Elasticity, E	29,000
Reinforcing Steel	ASTM A615 Grade 60	Yield Strength, f_y	60
		Modulus of Elasticity, E	29,000
Class B-1 Concrete (Superstructure)	MoDOT Class B-1	Compressive Strength, f'_c	5.6
Class B Concrete (Substructure)	MoDOT Class B	Compressive Strength, f'_c	3.0
Neoprene Bearing Pad	—	Modulus of Elasticity, E	7.2
Tire Loading Area	—	Modulus of Elasticity, E	1.45

4.2.4 Contact Definitions and Support Conditions

Contact definitions were used to represent the expected load transfer between structural components. Interfaces where full composite action or effective connection was expected were modeled as bonded. These interfaces included the girder-to-deck surfaces, cap beam-to-diaphragm surfaces, and barrier-to-deck surfaces. Other interfaces near the diaphragm and construction joints were modeled to permit limited relative movement while still transferring compressive and shear forces. This approach allowed the bridge to develop continuity through the diaphragm region without artificially forcing all components to act as a fully monolithic system. Specific details regarding the contact definitions used in the dead-load and live-load models are provided later in this chapter.

The intermediate bents were modeled with pile-supported foundations. The upper portion of the piles was represented as part of the bent system, while the lower portion was supported by elastic foundation springs to approximate soil flexibility. The end bents were restrained to represent the support conditions indicated in the construction drawings. This simplified foundation representation is aimed to capture the influence of support flexibility on the bridge-level response while avoiding the complexity of detailed nonlinear soil-pile interaction. Specific details on how the foundations were modeled for the dead-load and live-load models are discussed later in this chapter.

4.2.5 Model Calibration Strategy and Limitations

Both the dead-load and live-load finite element models were calibrated using the experimental results presented in the previous chapter. The purpose of the calibration process was to improve the reliability of the models by adjusting various model parameters within reasonable

physical limits until matching the results from both the dead-load and live-load tests. As discussed later in this chapter, separate calibration strategies were used for the dead-load and live-load models because each model represented a different stage of bridge behavior and was influenced by different sources of uncertainty.

Some differences between measured and predicted responses are unavoidable. Potential sources of discrepancy include temperature effects, construction tolerances, uncertainty in material properties, local cracking, truck placement tolerances, boundary-condition uncertainty, foundation flexibility, and measurement noise. These sources of uncertainty should be considered when interpreting point-by-point comparisons between numerical and experimental results. Consequently, the calibrated models should be viewed as tools for understanding the global bridge behavior and continuity mechanisms, rather than as exact representations of every local response or connection detail.

4.3 Dead-Load Model

The dead-load model was developed to simulate the bridge during deck placement. The rest of this section provides details of the dead-load model and the calibration process employed to obtain the numerical results discussed later in this chapter.

4.3.1 Contact Definitions at Intermediate Bents

Because the partially cast diaphragms could introduce restraint at the intermediate bents, they were explicitly represented in the dead-load model. Particular attention was given to the contact definitions in these regions because the interfaces between the girders, diaphragm, compression blocks, bent cap, and supporting piles controlled the force-transfer mechanisms during deck placement.

The contact conditions used at the intermediate bents in the dead-load model are shown in Figure 4-9. Bonded contacts were used for interfaces that were expected to behave as fully connected components. These included the welded steel end-plate assembly at the girder ends (A), the connection between the steel compression blocks and the end plates (E), the interface between the concrete diaphragm and the concrete bent cap (C), and the connection between the bent cap and the supporting H-piles (F). These bonded contacts prevent relative motion between the connected components and allow full force transfer across the corresponding interfaces.

The interfaces between the partially embedded steel girders and the cast-in-place concrete diaphragm were modeled using frictional contact with a coefficient of friction of 0.95 (B). This definition allows load transfer through bearing and friction while still permitting limited relative movement if the contact stresses exceed the frictional resistance. In contrast, the interface between the opposing compression blocks was modeled using frictionless contact (D). This assumption allows the compression blocks to transfer compressive forces through direct contact while allowing sliding or separation.

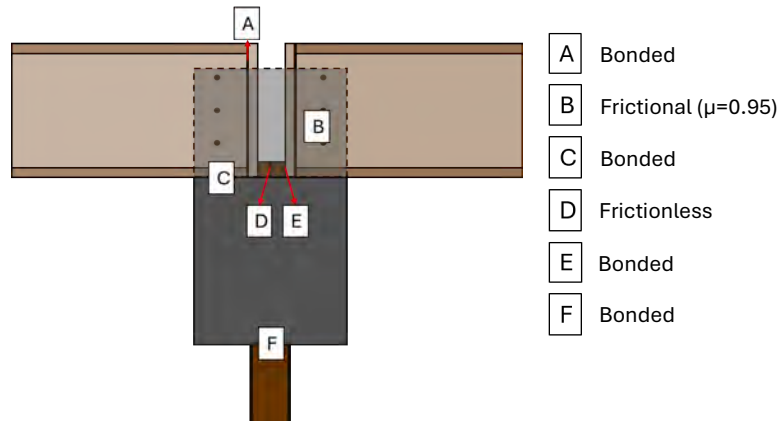


Figure 4-9. Contact definitions at intermediate bents for the dead-load model.

4.3.2 Support Conditions and Foundation Modeling

The support conditions and elastic foundation definition used in the dead-load model are summarized in Figure 4-10. The end bents were fixed to the ground, as this resembled what was observed in the construction drawings. At the intermediate bents, the and H-piles were modeled explicitly to represent the support flexibility provided by the pile-supported foundation system.

To account for soil–structure interaction in a simplified manner, an elastic foundation was assigned along the lower portion of the intermediate-bent piles. The upper portion of each pile, approximately 8 ft below the bent cap, was modeled without elastic support to represent the exposed pile length. Below this region, the piles were supported by an elastic foundation with a stiffness of 100 pci over a depth of approximately 37 ft. The base of each pile was restrained to provide overall stability of the foundation model. The same boundary conditions and elastic foundation definition were used for the live-load models.

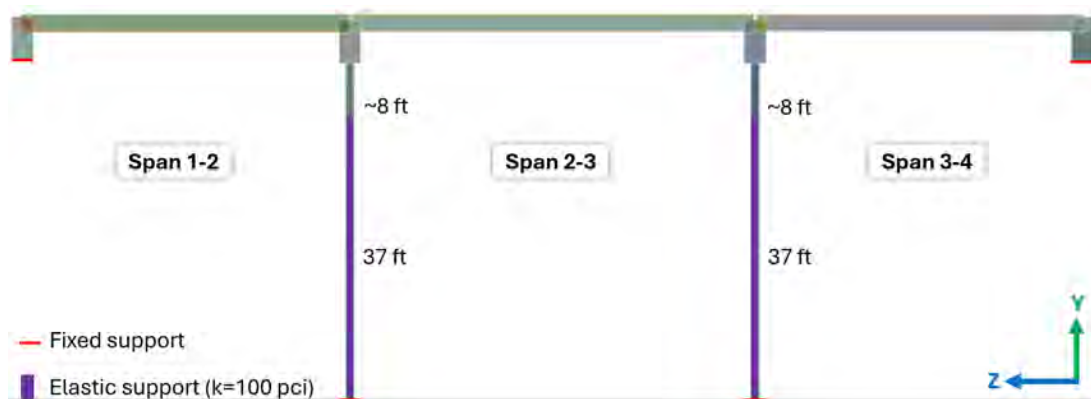


Figure 4-10. Boundary conditions and intermediate bent support representation used for the finite element models.

4.3.3 Dead Load Modeling

The weight of the fresh concrete deck was applied to the girders as equivalent pressure loads on the top flanges of the girders, as shown in Figure 4-11. Different pressure values were

assigned to interior and exterior girders based on tributary deck width. The applied pressure on interior girders was approximately 6.7 psi, while the applied pressure on exterior girders was approximately 4.8 psi. Only spans 2-3 and 3-4 were loaded to match the experimental program.

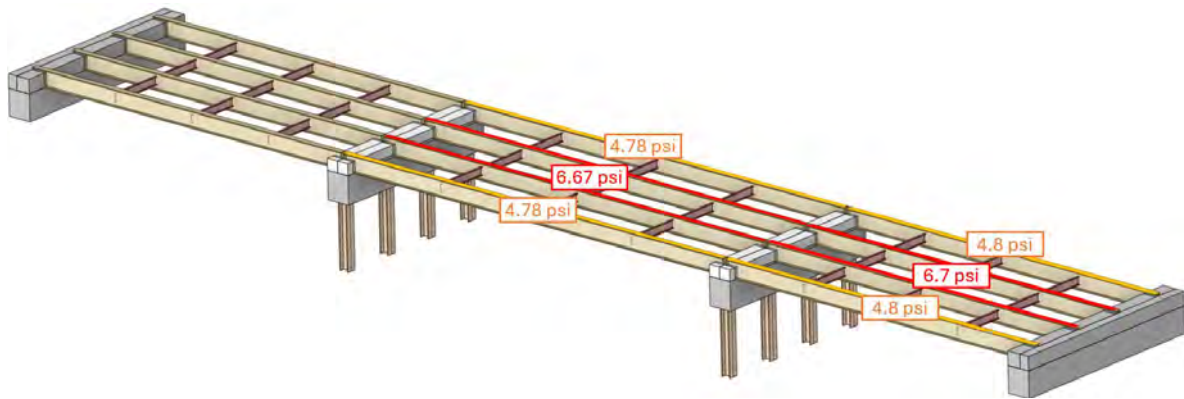


Figure 4-11. Dead-load model showing equivalent wet concrete pressure loads on girders.

4.3.4 Modeling of Temperature Effects During Concrete Pouring

Temperature effects were also considered in the dead-load model because it was suspected that the strains measured during deck placement were influenced not only by mechanical loading, but also by thermal effects from the warmer wet concrete. On the day of the deck pour, the ambient air temperature was approximately 40°F, while the temperature of the wet concrete was approximately 70°F. This temperature difference may have induced thermal strains at the top flanges of the steel girders.

As a result, the measured strains during deck pour were assumed to represent the combined effects of dead-load-induced mechanical strain and thermally induced strain. To account for this behavior, a temperature boundary condition was applied to the top flange of the steel girders in the dead-load model, as shown in Figure 4-12. The magnitude of this temperature load was treated as a calibration parameter. During calibration, different temperature values were applied to the girder top flanges, and the resulting total strains from the numerical model were compared directly with the field measurements.

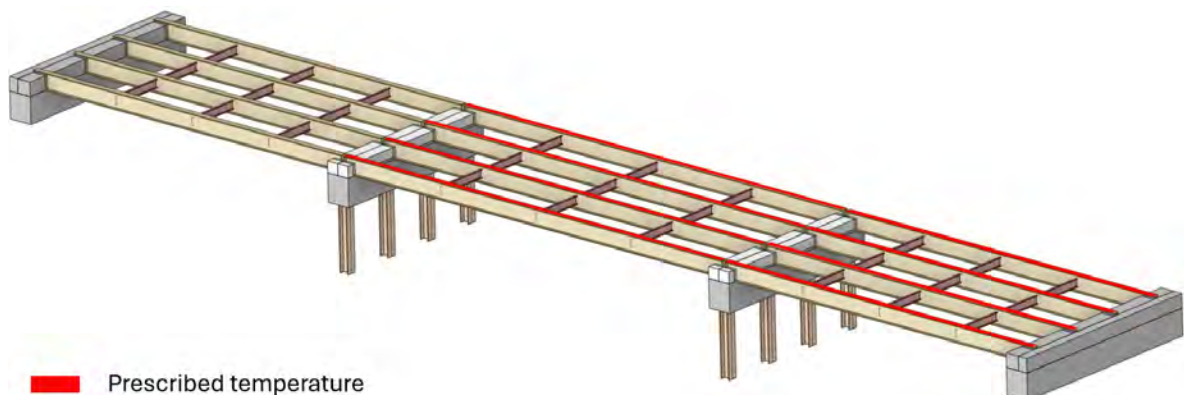


Figure 4-12. Dead-load model showing temperature boundary conditions applied on the top surface of the girders.

4.3.5 Modeling of Diaphragm Concrete

Another important consideration in the dead-load model was the condition of the partial-depth concrete diaphragms during deck placement. At the time of the deck pour, the diaphragm concrete had not yet reached its full design stiffness. In addition, preliminary dead-load analyses suggested that cracking may have developed in the diaphragm region during wet concrete placement. This was indicated by relatively large tensile strains in the upper portion of the diaphragm, which exceeded $100\ \mu\epsilon$ (the typical cracking strain of concrete).

To represent the reduced stiffness associated with early-age concrete behavior and possible cracking, the highlighted diaphragm regions near the girder ends in Figure 4-13 were assigned a reduced Young's modulus. This approach was not intended to explicitly model crack initiation or propagation. Rather, it was used as a practical means of accounting for the loss of effective stiffness in the diaphragm region during deck placement. Together with the top flange temperature, this reduced modulus of elasticity was treated as another calibration parameter.

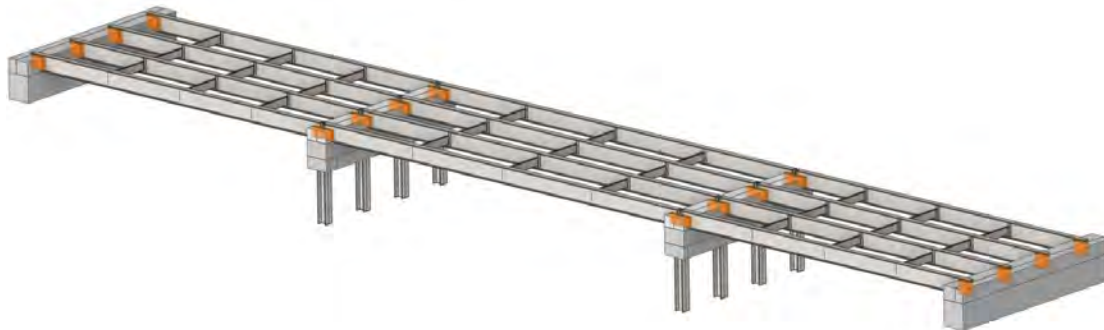


Figure 4-13. Regions within the partially-cast diaphragm where the concrete elastic modulus was reduced to account for possible concrete cracking.

4.3.6 Calibration Process

The dead-load model was calibrated using measurements collected during the dead-load testing. The calibration considered several parameters that could influence the construction-stage response, including the boundary conditions, contact behavior among the deck, girders, concrete diaphragms, compression blocks, and end plates, the embedded length of the pile foundations, and the elastic foundation stiffness used to represent the supporting soil.

The calibration also considered the effects of girder temperature and diaphragm stiffness discussed previously. The girder top-flange temperature was varied from 40°F ($\Delta T = 0^{\circ}\text{F}$ relative to ambient temperature), to 70°F ($\Delta T = 30^{\circ}\text{F}$ relative to ambient temperature). The elastic modulus in the highlighted regions of Figure 4-13 was varied between 5% and 10% of the fully cured concrete modulus.

The final calibrated dead-load model used the contact definitions shown in Figure 4-9, the support boundary conditions and pile embedment length shown in Figure 4-10, a top-flange temperature value of 54°F (i.e., $\Delta T = 14^{\circ}\text{F}$), and a diaphragm stiffness near the girder ends equal to 5% of the design concrete stiffness. This combination produced the closest agreement

between the numerical predictions and the measured strains and deflections obtained during deck placement. The results from the calibrated dead-load model are discussed in detail next.

4.4 Results From the Calibrated Dead-Load Model

The comparison between strains predicted by the calibrated model and those measured experimentally is summarized in Table 4-2. Overall, the model captured the measured response reasonably well at the bottom-flange and midspan locations. At midspan 3-4, the predicted and measured strains at ST5 were 125 $\mu\epsilon$ and 124 $\mu\epsilon$, respectively, while those at ST6 were approximately -180 $\mu\epsilon$ and -184 $\mu\epsilon$, respectively. These results indicate that the model agreed well with the measured bending response at midspan. The bottom-flange gages near the intermediate bent also showed reasonable agreement. At ST1, the predicted and measured strains were -168 $\mu\epsilon$ and -178 $\mu\epsilon$, respectively, while those at ST3 were -169 $\mu\epsilon$ and -136 $\mu\epsilon$.

The largest discrepancies were observed at the top-flange gages, ST2 and ST4, located near the intermediate bent. At these locations, the calibrated model predicted strains of approximately 92 $\mu\epsilon$ and 85 $\mu\epsilon$, whereas the measured strains were -8 $\mu\epsilon$ and -28 $\mu\epsilon$. These discrepancies were expected, given that ST2 and ST4 were in localized regions of high strain gradients that are sensitive to boundary effects near the intermediate bents. ST2 and ST4 were located close to the face of the partially cast diaphragm, where the top portion of the girder was in contact with the wet concrete. As a result, the measured response at these locations may have included thermal strain from the wet concrete, local restraint effects from the diaphragm, early-age concrete stiffness and cracking effects, and possible local strain gradients that were not fully represented in the global finite element model. Because these measurements likely reflected a combination of mechanical, thermal, and local construction-stage effects, they were considered less reliable as primary calibration targets. Consequently, the calibration focused primarily on matching the bottom-flange and midspan strain measurements.

Table 4-2. Predicted and measured construction-stage strains.

Strain	Location	Predicted ($\mu\epsilon$)	Measured ($\mu\epsilon$)	Difference ($\mu\epsilon$)
ST1	Bottom flange, near IB3 ^a	-168	-178	-10
ST2	Top flange, near IB3 ^a	91.7	-8	-99.7
ST3	Bottom flange, near IB3 ^a	-169	-136	33
ST4	Top flange, near IB3 ^a	84.6	-28	-112.6
ST5	Bottom flange, midspan	125	124	-1
ST6	Top flange, midspan	-180	-184	-4

^a IB3 refers to intermediate bent No. 3.

The comparison between the deflections predicted by the calibrated dead-load model and those measured during deck placement is summarized in Table 4-3. Overall, the model reproduced the measured construction-stage deflections with good accuracy. At LVDT 1, located in Span 3-4 near intermediate bent No. 3, the predicted and measured deflections were -0.14 in. and -0.11 in., respectively, resulting in a difference of 0.03 in. At LVDT 2, located at midspan of Span 3-4, the predicted deflection was -0.33 in. compared with a measured

deflection of -0.32 in., resulting in a difference of only 0.01 in. The largest deflection was measured at the SPOT location in Span 2-3, where the model predicted a deflection of -0.86 in. compared with a measured value of -0.82 in. The difference at this location was 0.04 in. These small differences indicate that the calibrated finite element model captured the global stiffness and deformation behavior of the bridge during deck placement with reasonable accuracy.

Table 4-3. Predicted and measured construction-stage deflections.

LVDT	Location	Predicted (in)	Measured (in)	Difference (in)
LVDT 1	Span 3-4, near IB3 ^a	-0.14	-0.11	0.03
LVDT 2	Span 3-4, midspan	-0.33	-0.32	0.01
SPOT	Span 2-3, midspan	-0.86	-0.82	0.04

^a IB3 refers to the intermediate bent No. 3.

In addition to the point-by-point comparisons summarized in Table 4-2 and Table 4-3, the calibrated dead-load model was evaluated by comparing the predicted strain and deflection profiles along the bridge. The bottom-flange strain profile and the deflection profiles are shown in Figure 4-14 and Figure 4-15, respectively. These comparisons provide a broader assessment of whether the model captured the overall construction-stage response, rather than only the response at individual instrumentation locations.

Overall, the model reproduced the general shape and magnitude of the measured strains and deflections. The strain profile shown in Figure 4-14 indicates that the model captured the variation in bottom-flange strain across the instrumented spans, including the change in response near intermediate bent No. 3. Similarly, the deflection profiles shown in Figure 4-15 show that the model captured the overall deformed shape during deck placement, including the larger downward deflection in Span 2-3 and the smaller deflection in Span 3-4.

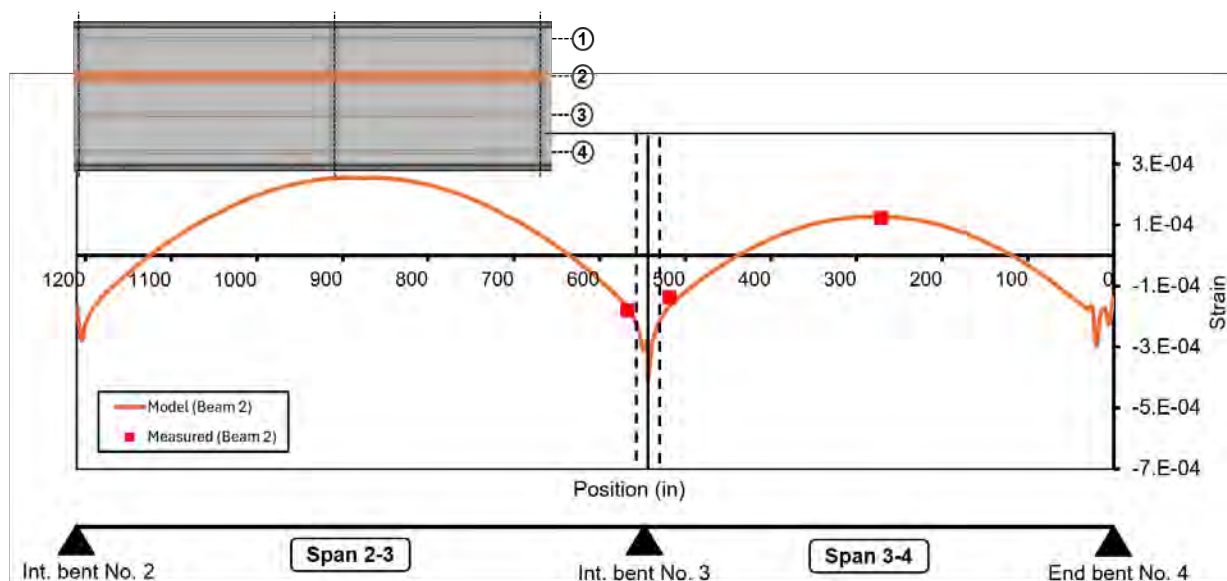


Figure 4-14. Predicted and measured strain profile during deck placement.

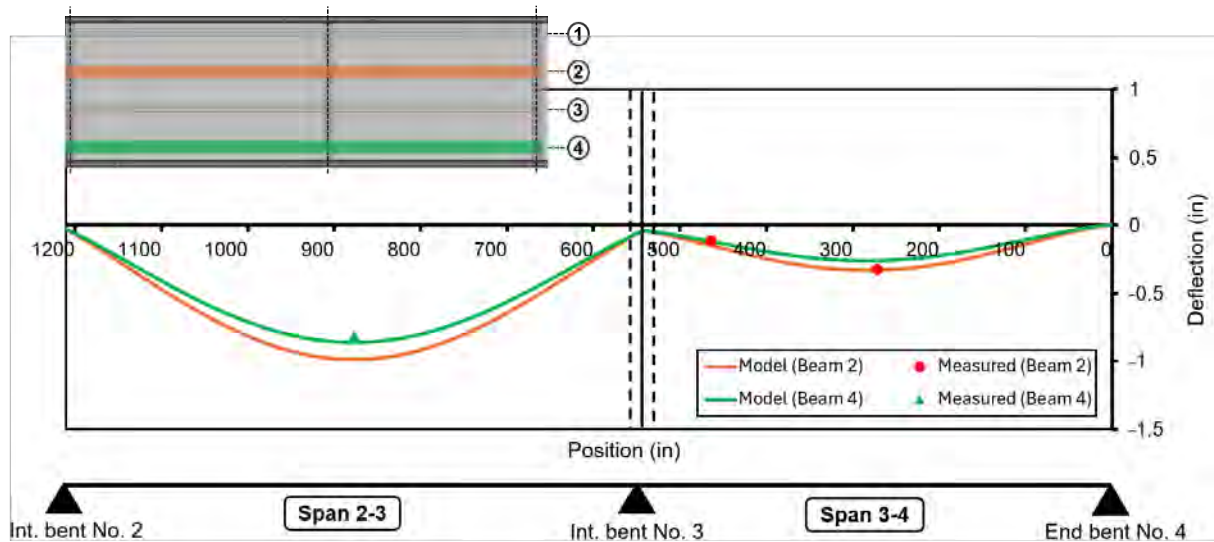


Figure 4-15. Predicted and measured deflection profiles during deck placement.

Although some local differences remained between the predicted and measured responses, these discrepancies are expected. Potential sources of uncertainty include the concrete placement sequence, temperature variation, foundation flexibility, support conditions, local stiffness variations, and the timing of data collection. Therefore, the agreement shown in Figure 4-14 and Figure 4-15 indicates that the calibrated dead-load model provided a reasonable representation of the global bridge response during deck placement.

4.5 Live-Load Model

The live-load model was developed to simulate the completed Bridge A9513 under the controlled truck-loading configurations used during the live-load testing. The rest of this section provides details of the live-load model and the calibration process employed to obtain the numerical results discussed later in this chapter.

4.5.1 Contact Definitions at Intermediate Bents

The contact definitions used to represent the completed continuity connection at the intermediate bents are shown in Figure 4-16. Several of the contact definitions were consistent with those used in the dead-load model. Bonded contacts were assigned to interfaces where full force transfer and no relative motion were expected. These included the welded connection between the steel girder and end plate (A), the interface between the concrete diaphragm and bent cap (C), the welded connection between the steel end plate and compression block (E), and the pile-to-cap connection (F). Bonded contact was also used to represent the composite interface between the concrete deck and steel girders (G), as well as the connection between the concrete deck and concrete barriers (I).

Frictional contact with a friction coefficient of 0.95 was assigned to the interface between the partially embedded steel girders and the concrete diaphragm (B). This contact definition allowed force transfer through bearing and friction while permitting limited relative movement

if the contact stresses exceeded the frictional resistance. A similar contact definition was assigned to the construction joint between the concrete diaphragm and the concrete deck (H), representing the interface between concrete components cast at different times. The interface between the opposing compression blocks was modeled using frictionless contact (D), which allows the blocks to transfer compressive forces through direct bearing while allowing sliding or separation. Finally, no contact definition was applied to the expansion joints along the barriers to prevent unintended compressive stresses to develop across the open joint interfaces.

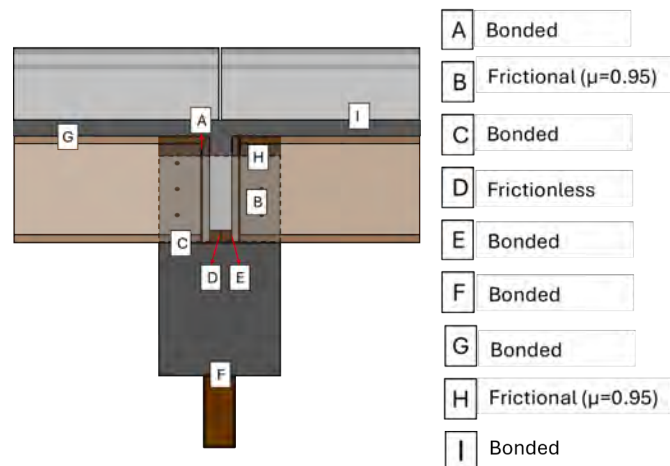


Figure 4-16. Contact definitions at intermediate bents for the live-load model.

4.5.2 Truck Loading

The truck loading used in the live-load model was defined to match the live-load testing program discussed in Chapter 3. Six truck positions were analyzed, as summarized in Figure 4-17. The selected positions were intended to reproduce the field-testing conditions and to generate different combinations of positive moment, negative moment, and transverse load distribution. As a result, the live-load model could be evaluated under load cases that activated both the span regions and the continuity regions near the intermediate bents.

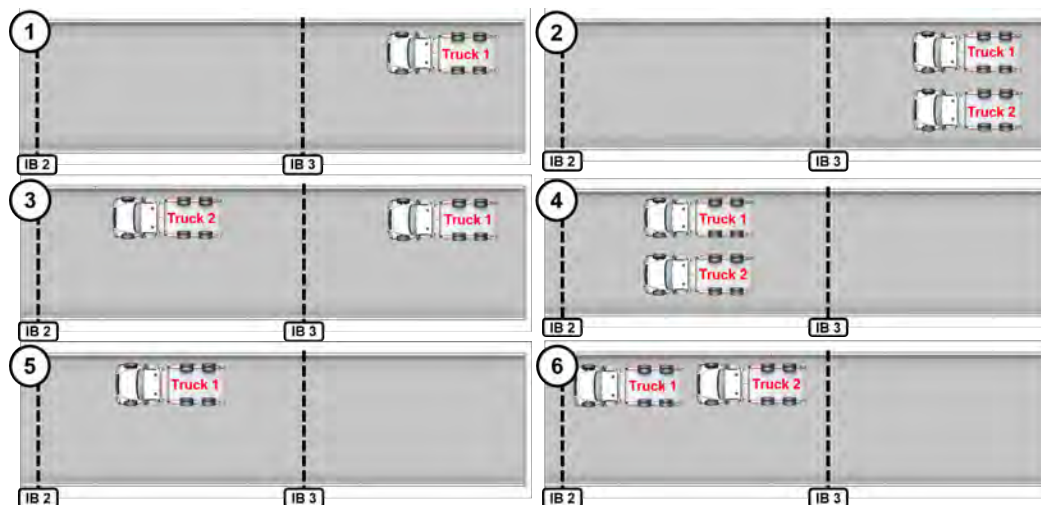


Figure 4-17. Truck positions used in live-load testing and model calibration.

4.5.3 Calibration Process

The live-load model was calibrated by comparing the predicted bridge response with the measurements obtained during the controlled truck-load testing program. The calibration process was similar in concept to that used for the dead-load model, but the temperature effects associated with deck placement were not included as those are only relevant in the dead-load model. Instead, the calibration focused on modeling parameters that influenced the stiffness, continuity, and load-transfer behavior of the completed bridge.

One of the primary calibration parameters was the effective length of the H-piles at the intermediate bents, which control the vertical flexibility of the pile-supported foundation system. This parameter was calibrated using the field-measured deflection from the LVDT located near the intermediate bent No. 3. An initial pile length, L_0 , was estimated from the construction drawings, and the finite element model was then used to compute the corresponding support displacement, δ_0 . This value was compared with the displacement, δ_{field} , from LVDT 3 (see Figure 3-12) measured during the live-load test. Because the materials were linear elastic, a pile length correction was estimated using the axial deformation relationship for an elastic column. The effective pile length was updated as $L_{eff} = L_0 + \Delta L$, in which $\Delta L = (\delta_{field} - \delta_0)EA/P$, E is the Young's modulus of the H-piles, A is the cross-sectional area of the H-piles, and P is the axial force in the H-piles obtained from the finite element model. The updated pile length was then used in a new model, and the process was repeated until $\delta_0 \approx \delta_{field}$.

In addition to the pile lengths, the contact behavior at the intermediate bents was refined during calibration. In particular, the friction coefficients assigned to selected interfaces between the steel girders, concrete diaphragm, compression blocks, end plates, and adjacent concrete components were varied within reasonable physical limits. These contact parameters were adjusted to improve agreement between the predicted and measured curvatures and deflections at the different instrumentation locations. Curvatures were computed from the measured strain differences between the top and bottom flanges, while deflections were obtained from displacement measurements collected during the truck tests.

The calibration was performed across the truck positions used in the field-testing program to ensure that the model captured the bridge response under different loading configurations. Rather than matching a single measurement or load case, the calibration sought to reproduce the overall trends in girder curvatures and vertical deflections for all truck positions. This comprehensive calibration process provided a live-load model capable of representing the global behavior of the completed bridge. The calibration process resulted in the contact definitions shown in Figure 4-16, which correspond to the same contact definitions adopted in the dead-load model. The results from the calibrated live-load model are discussed next.

4.6 Results From the Calibrated Live-Load Model

Table 4-4 summarizes the comparison between the curvatures predicted by the calibrated live-load model and those obtained from the field measurements for selected truck positions. The

table includes representative results for Truck Positions 2, 3, and 4, while the complete set of curvature comparisons is provided in Appendix B. Overall, the model reproduced the measured curvatures with good agreement across the selected locations, capturing both the sign and relative magnitude of the response at the support and midspan sections.

The agreement was good at the support locations, where the predicted and measured curvatures were generally close. For example, for Truck Position 3, the predicted and measured curvatures at Beam 2 near the intermediate bent were $-3.97 \times 10^{-6} \text{ in}^{-1}$ and $-3.85 \times 10^{-6} \text{ in}^{-1}$, respectively. Some larger differences were observed at the midspan location for Truck Position 2, where the model predicted a curvature of $5.18 \times 10^{-6} \text{ in}^{-1}$ compared with a measured value of $3.78 \times 10^{-6} \text{ in}^{-1}$. Nevertheless, the comparisons indicate that the calibrated model captured the overall curvature response of the bridge under live-load testing with reasonable accuracy.

Table 4-4. Summary of predicted and measured curvatures during live-load testing.

Truck position	Location	Predicted κ ($\times 10^{-6} \text{ in}^{-1}$)	Measured κ ($\times 10^{-6} \text{ in}^{-1}$)	Difference ($\times 10^{-6} \text{ in}^{-1}$)
2	Beam 1 (near support) ^a	-0.9	-1.4	0.5
	Beam 2 (midspan) ^b	5.18	3.78	1.4
	Beam 2 (near support) ^c	-1.86	-2.17	0.31
3	Beam 1 (near support)	-3.6	-4.13	0.5
	Beam 2 (midspan)	2.58	2.11	0.47
	Beam 2 (near support)	-3.97	-3.85	-0.12
4	Beam 1 (near support)	-3.56	-3.67	0.11
	Beam 2 (midspan)	-1.16	-0.85	-0.31
	Beam 2 (near support)	-4.28	-3.75	-0.53

^a Curvatures at this location were computed based on readings from ST7 and ST8

^b Curvatures at this location were computed based on readings from ST5 and ST6

^c Curvatures at this location were computed based on readings from ST1 and ST2

Table 4-5 summarizes the comparison between the deflections predicted by the calibrated live-load model and those measured during field testing for selected truck positions. The table includes results for Truck Positions 2, 3, and 4, while the remaining comparisons are provided in Appendix B. Overall, the model reproduced the measured deflections with good agreement across the different LVDT and SPOT measurement locations. The differences between predicted and measured values were generally small, with most discrepancies below 0.02 in. and an average difference between model predictions and measured values of approximately 0.007 in. for Truck Positions 2, 3, and 4, and of approximately 0.006 in. across all truck positions which is close to the measurement accuracy of the instrumentation.

For Truck Positions 2 and 3, the model captured both the magnitude and distribution of the measured deflections across the bridge. For example, for Truck Position 2, the predicted and measured deflections at LVDT 2 were -0.131 in. and -0.137 in. , respectively. Similarly, for Truck Position 3, the predicted and measured deflections at the SPOT location were both -0.029 in. For Truck Position 4, the model also reproduced the measured response well, including the larger downward deflection at the SPOT location, where the predicted and measured values

were -0.195 in. and -0.174 in., respectively. These comparisons indicate that the calibrated live-load model captured the global stiffness and deformation behavior of the completed bridge with reasonable accuracy.

Table 4-5. Summary of predicted and measured deflections during live-load testing.

Truck position	LVDT	Location ^a	Predicted (in)	Measured (in)	Difference (in)
2	LVDT1	Span 3-4, BL2 near IB3	-0.059	-0.056	-0.003
	LVDT2	Span 3-4, BL2 midspan	-0.131	-0.137	0.006
	LVDT3	Span 3-4, BL1 near IB3	-0.026	-0.026	0.000
	LVDT4	Span 3-4, BL 1 midspan	-0.084	-0.107	0.023
	SPOT	Span 2-3, BL4 midspan	0.022	0.012	0.010
3	LVDT1	Span 3-4, BL2 near IB3	-0.031	-0.026	-0.005
	LVDT2	Span 3-4, BL2 midspan	-0.066	-0.071	0.005
	LVDT3	Span 3-4, BL1 near IB3	-0.036	-0.024	-0.012
	LVDT4	Span 3-4, BL 1 midspan	-0.054	-0.070	0.016
	SPOT	Span 2-3, BL4 midspan	-0.029	-0.029	0.000
4	LVDT1	Span 3-4, BL2 near IB3	0.018	0.020	-0.002
	LVDT2	Span 3-4, BL2 midspan	0.036	0.034	0.002
	LVDT3	Span 3-4, BL1 near IB3	-0.007	0.001	-0.008
	LVDT4	Span 3-4, BL 1 midspan	0.030	0.031	-0.001
	SPOT	Span 2-3, BL4 midspan	-0.195	-0.174	-0.021

^a BL refers to beam line and IB refers to intermediate bent

In addition to the point-by-point curvature comparisons summarized in Table 4-4, Figure 4-18 through Figure 4-20 show the predicted curvature profiles along the bridge for Truck Positions 2, 3, and 4. These figures provide a broader assessment of the live-load model by showing whether the calibrated model captured the overall curvature distribution, including the positive-moment regions within the spans and the negative-moment regions near the intermediate bents.

Overall, the model reproduced the main trends observed in the measured curvature response. For the selected truck positions, the predicted curvature profiles captured the location and relative magnitude of the peak responses, as well as the change in curvature sign near the intermediate bents. The agreement was generally good at the instrumented locations, indicating that the model reasonably represented the flexural response and continuity developed in the completed bridge under live loading.

Some local differences were observed between the predicted and measured curvature values, particularly near regions with steep curvature gradients close to the intermediate bents and near the applied truck loads. These differences are expected because the curvature response is sensitive to factors such as truck placement, strain-gage location, local stiffness variations, and the contact behavior within the diaphragm region. Nevertheless, the comparisons shown in Figure 4-18 through Figure 4-20 indicate that the calibrated live-load model captured the overall curvature behavior of the bridge with reasonable accuracy.

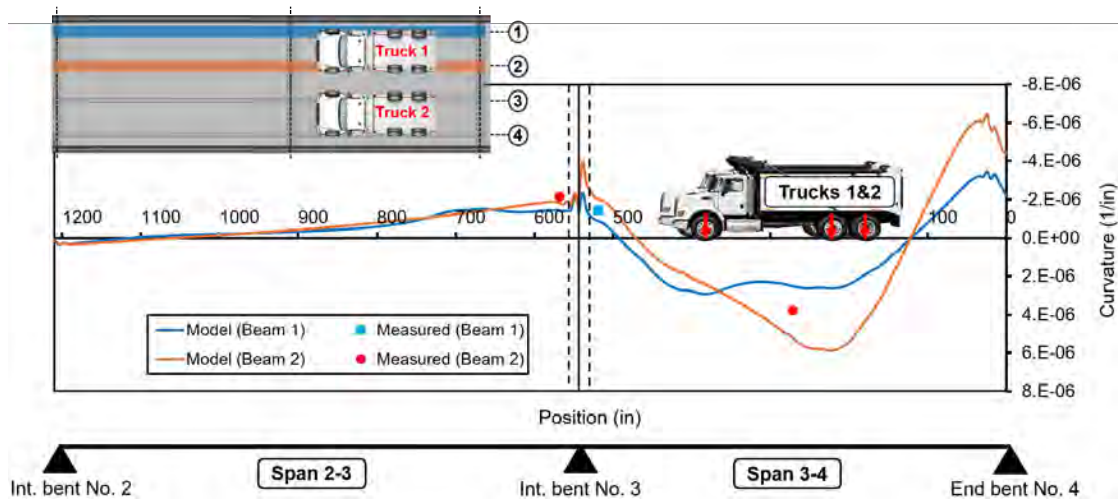


Figure 4-18. Predicted and measured curvature profiles for Truck Position 2.

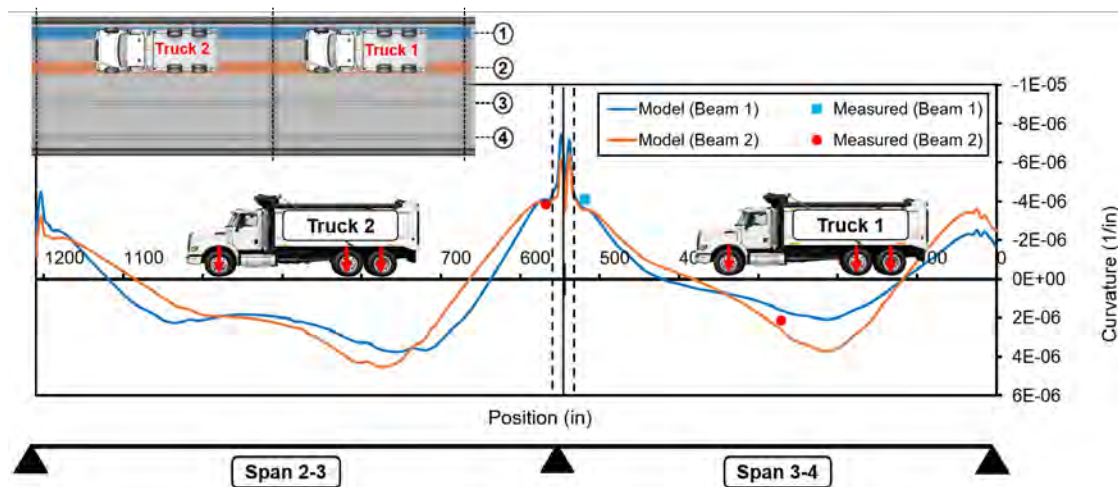


Figure 4-19. Predicted and measured curvature profiles for Truck Position 3.

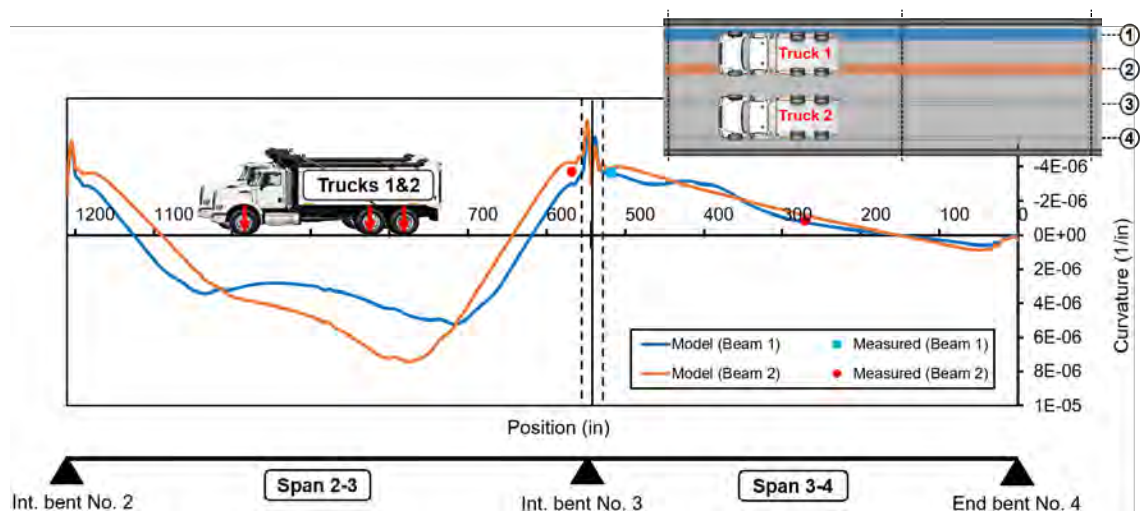


Figure 4-20. Predicted and measured curvature profiles for Truck Position 4.

In addition to the point-by-point deflection comparisons summarized in Table 4-5, Figure 4-21 through Figure 4-23 show the predicted and measured deflection profiles for Truck Positions 2, 3, and 4. These figures provide a broader assessment of the calibrated live-load model by comparing the overall deformed shape of the bridge, rather than only the response at individual measurement locations.

Overall, the model captures the main trends in the measured deflection response for the selected truck positions. For Truck Position 2, the model reproduced the downward deflection in Span 3-4 and the smaller upward deflection in Span 2-3. Importantly, the model predicted the downward deflections near the intermediate supports, which was only possible due to the effective H-pile foundation length calibration procedure discussed above. For Truck Position 3, the model captured the larger deflections in Span 2-3 as well as the smaller deflections in Span 3-4. For Truck Position 4, the model reproduced the overall deflected shape, including the larger downward response in Span 2-3 and the upward response observed in Span 3-4.

As expected, some differences were observed between the predicted and measured deflections. These discrepancies may be attributed to several sources of uncertainty, including environmental conditions, truck placement accuracy, material-property variability, boundary-condition idealizations, foundation flexibility, and measurement noise. However, it is important to note that a single calibrated model was able to capture the response of six different live-load tests, corresponding to Truck Positions 1 through 6, without recalibration for each loading case. Across all truck positions, the average difference between predicted and measured deflections was approximately 0.006 in., indicating that the model provided a consistent and accurate representation of the global deformation behavior of the completed bridge.

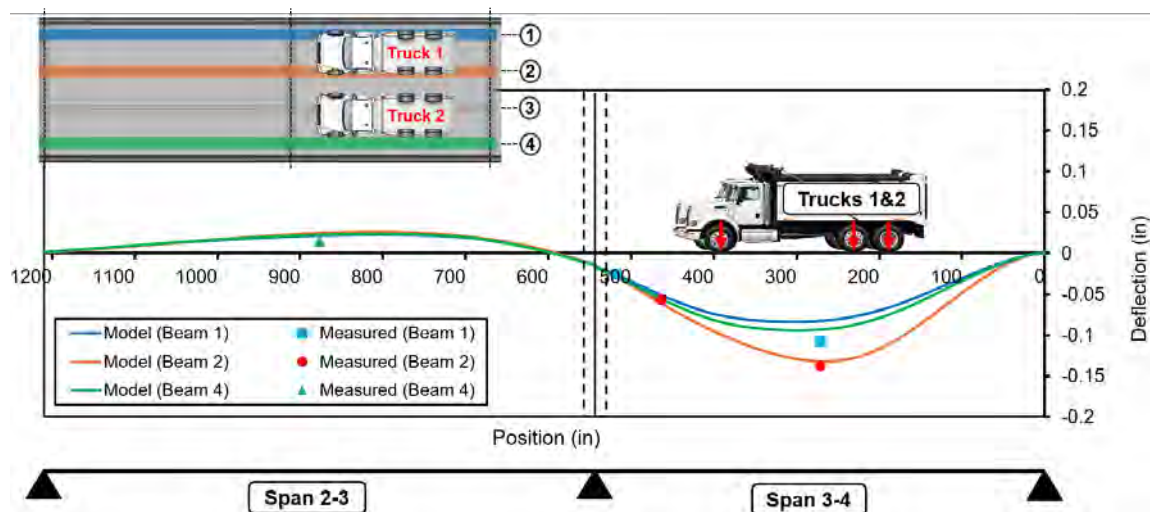


Figure 4-21. Predicted and measured deflection profiles for Truck Position 2.

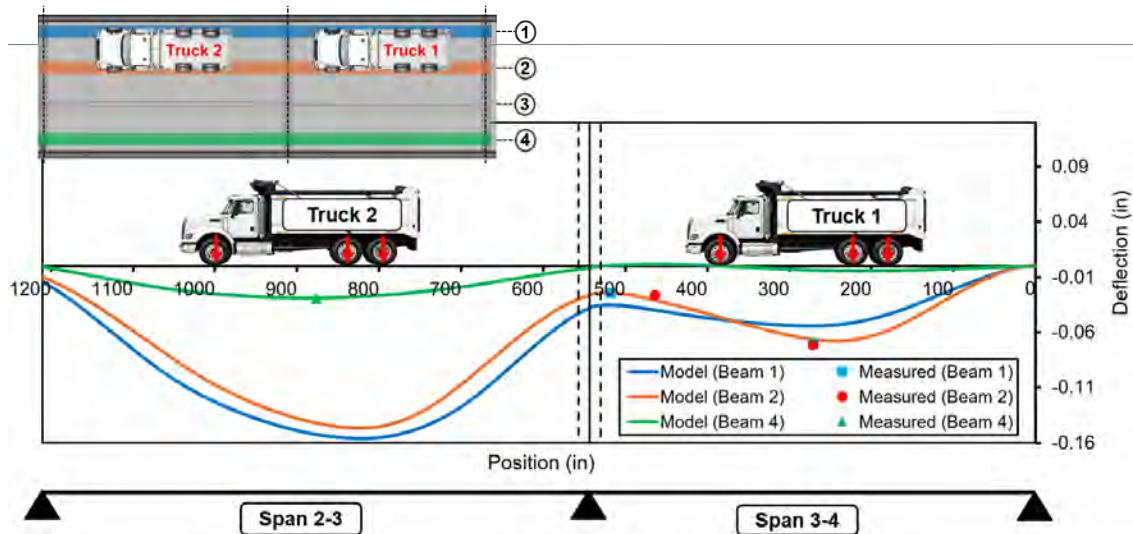


Figure 4-22. Predicted and measured deflection profiles for Truck Position 3.

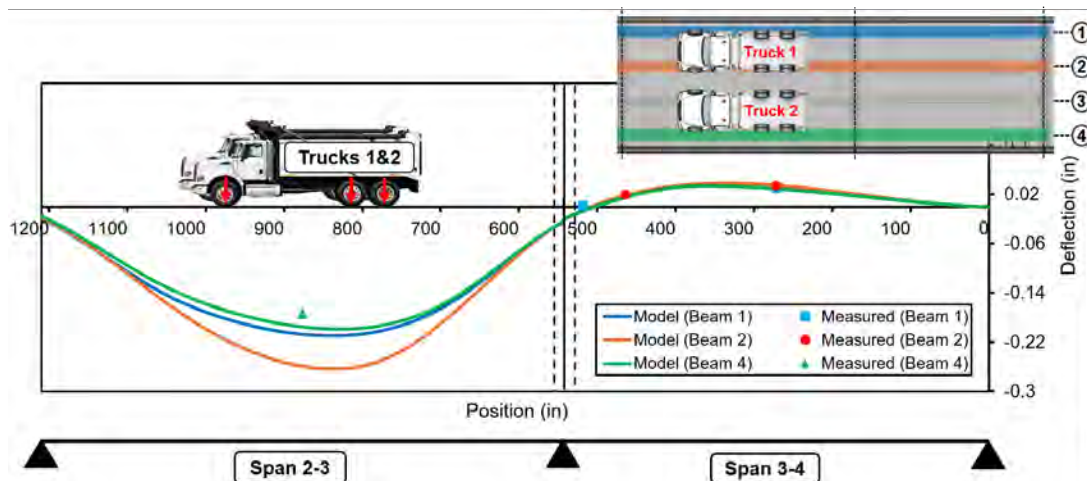


Figure 4-23. Predicted and measured deflection profiles for Truck Position 4.

4.7 Model Calibration Summary

The calibration results indicate that the dead-load and live-load models provided a reasonable representation of the measured bridge behavior. The dead-load model captured the construction-stage response of the steel girders under wet concrete loading, particularly at the bottom-flange and midspan instrumentation locations. The live-load model reproduced the main response trends observed during controlled truck testing, including positive moment response in the loaded spans, negative moment response near the intermediate bents, and increased response under two-truck loading.

Table 4-6 provides a summary of the calibration quality of the dead-load and live-load models. For the dead-load model, the calibration quality was evaluated mostly using girder strains and deflections. In terms of strains, the best agreement was obtained at the bottom-flange near the intermediate bent (ST1 and ST3) and at midspan measurement locations (ST5 and ST6), which

were considered the most reliable indicators of the mechanical response during deck placement. Larger discrepancies were observed at the top-flange gages near the intermediate bent (ST2 and ST4), likely because these measurements were influenced by thermal effects from the wet concrete, local effects from the partially cast diaphragm, possible diaphragm cracking effects, among others. However, the deflection profiles predicted by the model agreed well with the experimental measurements, with average deflection differences of 0.027 in.

For the live-load model, the calibration quality was evaluated using both curvature and deflection comparisons for the truck positions used during field testing. The model captured the overall curvature trends, including the sign and relative magnitude of the response at support and midspan locations. It also reproduced the measured deflection profiles with good agreement across the different truck positions, with an average deflection difference of approximately 0.006 in. across all live-load cases. The fact that a single calibrated model was able to reproduce the response from multiple truck configurations without recalibration indicates that the model captured the global stiffness, load distribution, and continuity behavior of the completed bridge with reasonable accuracy.

Table 4-6. Summary of the calibration quality of the dead-load and live-load models.

Model	Main response quantities evaluated	Calibration quality	Main observations	Limitations / interpretation
Dead-load model	Girder strains, and deflection profiles during deck placement	Good for bottom-flange and midspan strains; fair for top-flange strains; and good for deflection profiles	The model captured the main construction-stage response of the steel girders under wet concrete loading. Agreement was good at midspan strain gages (ST5 and ST6), as well as at bottom-flange gages near the intermediate bent (ST1 and ST3).	Top-flange gages ST2 and ST4 showed large differences between predicted and measured strain. These readings were likely affected by temperature from wet concrete during deck placement, local effects from the partially cast diaphragm, possible diaphragm cracking, among others.
Live-load model	Curvature and deflection profiles for multiple truck positions	Good for overall live-load trends and continuity assessment	The model reproduced the main live-load response trends, including positive moment behavior in loaded spans, negative moment response near intermediate bents, and larger response under two-truck loading. Representative Truck Positions 2, 3, and 4 showed reasonable agreement with measured curvature and deflection profiles.	Differences between predicted and measured values are expected because of truck positioning tolerances, boundary condition uncertainty, foundation flexibility, local construction details, and measurement noise. The model is appropriate for bridge-level interpretation rather than exact point-by-point prediction.

4.8 Degree of Continuity

A major objective of the finite element modeling was to evaluate the degree of continuity (DOC) that Bridge A9513 developed at the intermediate bents. The DOC provides a practical measure of how closely the bridge approaches simple-span behavior during construction and continuous behavior under live loading. A DOC close to zero indicates behavior closer to the simple-span limit, while a DOC close to one indicates behavior closer to the fully continuous limit. This distinction is important because the desired behavior of an SDCL bridge changes between construction and service. During deck placement, the bridge is intended to behave primarily as a simple-span system so that the steel girders do not attract large negative moments from the noncomposite dead load. After the deck and continuity diaphragms have hardened, the completed bridge is expected to develop continuity under live loading.

Physically, the DOC can be interpreted as a measure of moment continuity developed at the intermediate support. In an ideal simple-span system, no moment is transferred across the intermediate bent. In a fully continuous system, moment is transferred from one span to the adjacent span through the continuity connection. Therefore, the DOC provides an estimate of how much of the fully continuous response is developed by the actual bridge. For example, if the DOC were calculated directly from the negative moment at the intermediate bent, the simple-span moment at that location would be approximately zero, and the DOC could be interpreted approximately as the ratio between the moment developed in the calibrated model and the moment developed in the fully continuous model (i.e., $DOC = M_{calibrated}/M_{fixed}$). In this sense, a DOC of 0.40 would indicate that the bridge develops approximately 40% of the moment continuity associated with the fully continuous condition for the selected load case.

Because bending moments are directly related to curvature in the elastic range, curvature provides a useful response quantity for evaluating moment continuity. A curvature-based DOC reflects how much the bending response of the calibrated model has shifted from the simple-span limit toward the fully continuous limit. However, continuity also affects the overall deflected shape of the bridge. This is particularly important during construction because predicted dead-load deflections are used to establish girder camber during fabrication. For this reason, a deflection-based DOC was also evaluated. The deflection-based metric provides a global measure of how much the bridge stiffness and deflected shape are affected by continuity.

The curvature-based and deflection-based DOC values are not expected to be identical. Curvature is related to local bending deformation and is therefore more directly connected to moment continuity. Deflection, on the other hand, includes the accumulated effect of bending over the span, support flexibility, foundation movement, and possible rigid-body motion of the superstructure. Some of these displacement components can change the measured or predicted deflection without producing a proportional change in girder strain or curvature. Therefore, the two DOC metrics provide complementary information, with the curvature-based metric being more directly related to moment transfer, while the deflection-based DOC being more directly related to global stiffness and construction deflection demands.

The procedure used to determine the DOC is illustrated in Figure 4-24. For each loading stage, the response of the calibrated finite element model was compared with that from two idealized models. The first idealized model represents the simple-span (or free) condition, where the girders are allowed to rotate freely at the intermediate diaphragms. The second idealized model represents the fully continuous (or fixed) condition, where the girder ends are fully bonded to the diaphragms. The DOC was then computed by interpolating the calibrated response between these two limiting cases. For the curvature-based metric, the DOC was calculated using curvature responses at a specified location from the free, calibrated, and fixed models. For the deflection-based metric, the same interpolation was applied using midspan deflections at any given span.

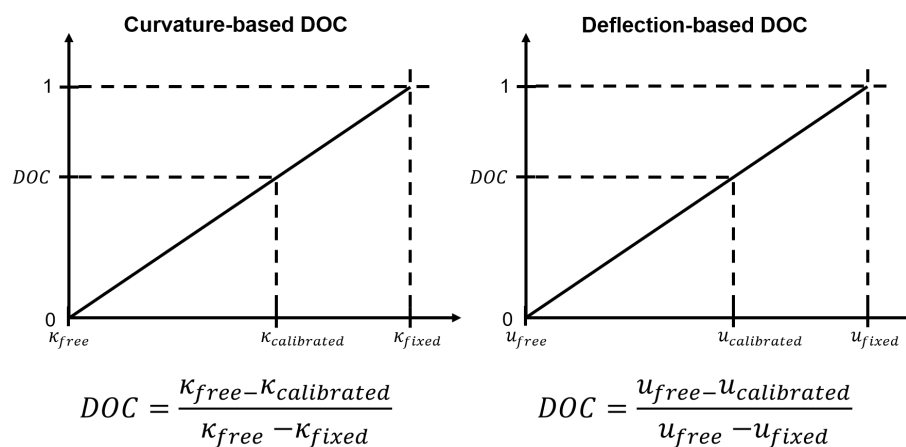


Figure 4-24. Definition of degree of continuity using free, calibrated, and fixed models.

4.8.1 Degree of Continuity During Construction-Stage Dead Loading

For the dead-load stage, the DOC was computed using the weight of the wet concrete deck as the applied loading. The calibrated dead-load model was compared with idealized free and fixed models subjected to the same wet concrete deck loading. This comparison was used to determine the extent to which the partially cast diaphragm and girder-end details introduced restraint at the intermediate bents during construction.

The comparison between the free, calibrated, and fixed dead-load models is shown in Figure 4-25. The behavior of the calibrated model falls between that of the free and fixed models, indicating that Bridge A9513 did not behave as a purely simple-span system during deck placement. Instead, measurable restraint developed at the intermediate bents due to the presence of the partially cast diaphragm. Therefore, the DOC values for the dead-load condition are expected to be greater than zero.

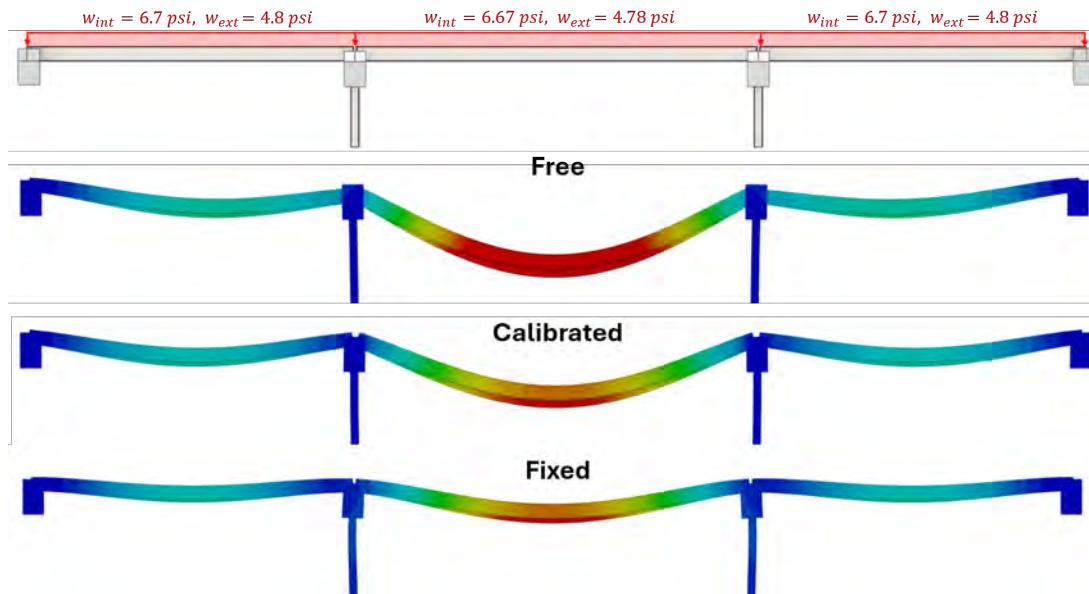


Figure 4-25. Comparison of free, calibrated, and fixed model responses under wet concrete deck loading.

As discussed previously, two degree-of-continuity metrics were defined: one based on curvature and one based on deflection. For the dead-load model, only the deflection-based degree of continuity was reported. This decision was made based on the calibration results because the curvature-based metric depends on the difference between top- and bottom-flange strains, and the top-flange strain measurements during deck placement were affected by thermal effects from the wet concrete (see Section 4.4). In contrast, the dead-load model reproduced the measured deflections with reasonable accuracy. Therefore, the deflection-based metric was considered a more reliable indicator of the construction-stage degree of continuity, and only deflection-based values are presented in this section.

The deflection-based DOC estimates for the dead-load model are shown in Figure 4-26. These results were obtained based on midspan deflections for beam line 2. The calculated DOC values were approximately 42% based on the midspan deflection response in Span 2-3 and 23% based on the midspan deflection response in Span 3-4. The difference in the DOC values is due to the difference in the end restraints of the center span vs. the end spans. Notice that the DOC for Span 2-3 is greater than the 25% DOC as suggested in Azizinamini [40]. The absence of the bond breaker plate in Bridge A9513 may be a reason and will be investigated later in this section.

Although the bridge was far from being fully continuous during deck placement, the measured response clearly indicates a degree of restraint greater than expected for a purely simple-span system. This agrees with the raw experimental data that showed upward movement in Span 2-3 when concrete was placed in Span 3-4. The relatively high DOC values may be attributed to the depth of the partially cast diaphragm. For the test bridge, the first diaphragm pour extended approximately 17 in. in depth, while the steel girder depth was approximately 24 in. Therefore, the partially cast diaphragm had a depth equal to approximately 70% of the girder depth. This

created a relatively large concrete section around the girder ends and may have provided considerable stiffness within the continuity region. These findings suggest that the depth of the partially cast diaphragm before the deck placement can significantly influence construction-stage behavior and should be considered when evaluating the dead-load response of SDCL bridges.

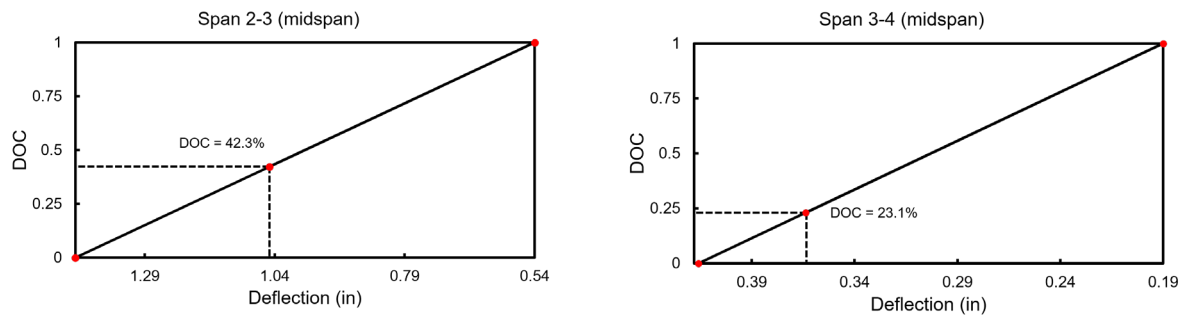


Figure 4-26. Deflection-based degree of continuity for the dead-load model.

To further investigate the construction-stage DOC, additional dead-load models were developed with fixed and pinned diaphragm configurations and with and without a bond breaker plate, as shown in Figure 4-27 and Figure 4-28. Specific details explaining how the bond breaker plate and pinned diaphragm configurations were modeled are provided in the next Chapter. These additional models were used to evaluate whether the diaphragm restraint condition and bond breaker plate would affect the partial continuity that developed during deck placement.

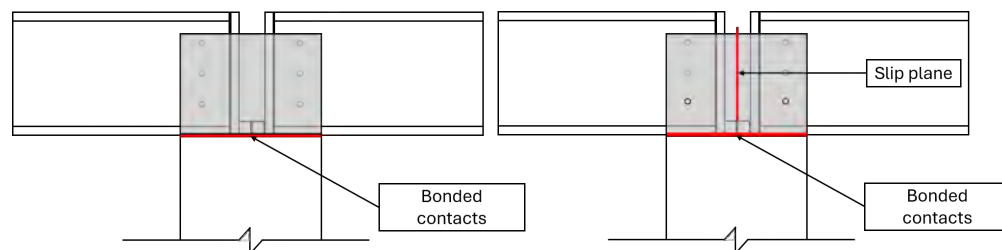


Figure 4-27. Fixed diaphragm modeled with bonded contacts and no bond breaker plate (left), and with bonded contacts and a bond breaker plate (right).

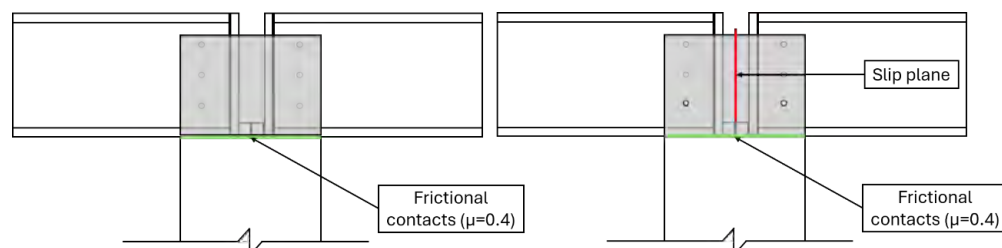


Figure 4-28. Pinned diaphragm modeled with frictional contacts and no bond breaker plate (left), and with frictional contacts and a bond breaker plate (right).

Using the deflection-based DOC measure, the construction-stage continuity was calculated for the pinned and fixed diaphragm configurations, and the results are shown in Table 4-7. When the bond breaker plate was included, the average DOC decreased from about 42% to 41% for the fixed diaphragm and from about 44% to 32% for the pinned diaphragm. These results indicate that the bond breaker plate had a greater effect when the diaphragm was pinned. Compared with the fixed diaphragm configuration without a bond breaker plate, the pinned diaphragm with a bond breaker plate reduced the DOC by approximately 10% for Span 2-3 and by approximately 8% for Span 3-4. Overall, the results suggest that the bond breaker plate does not have a large effect (<10%) on the overall continuity and the choice to include the plate may be based on other design considerations such as crack control. In addition, the pinned diaphragm configuration with a bond breaker plate is the most effective for reducing unintended continuity during construction-stage dead loading, however the choice of pinned or fixed diaphragm may be controlled by the column/pile design considerations.

Table 4-7. Deflection-based dead load DOC values for fixed and pinned diaphragm configurations with and without bond breaker plate.

	Fixed diaphragm		Pinned diaphragm	
	No plate	Plate	No plate	Plate
Span 2-3	42.3	40.9	43.6	32
Span 3-4	23.1	20.7	20.5	17.7
Average	32.7	30.8	32.1	24.9

Some of the DOC values reported in Table 4-7 are greater than the 25% construction-stage DOC recommended by Azizinamini [40]. However, this value appears to be an approximate design recommendation rather than a value measured from a specific bridge test. In addition, the recommendation in [40] is associated with diaphragm details that include a bond breaker plate and a diaphragm cast to approximately 1/2 to 2/3 of its final depth before deck placement. Bridge A9513 did not include a bond breaker plate, and its diaphragm was cast to approximately 70% of its final girder depth before deck placement. These differences likely increased the stiffness of the intermediate bent region and allowed greater restraint to develop during the deck pour. Additional restraint may also have been provided by the fixed diaphragm-cap connection and the reinforcement passing through the girder webs and diaphragm region (see Figure 3-3). Therefore, the results suggest that assuming 25% construction-stage DOC may be unconservative for SDCL bridges with relatively deep partially cast diaphragms, fixed diaphragm-cap connections, or no bond breaker plate.

In a related study, Yakel and Azizinamini [43] evaluated the construction-stage DOC in three SDCL steel bridges in Nebraska. In their study, the diaphragms were cast and cured before deck placement, and the authors reported that this produced measurable continuity during the deck pour. For one of the tested bridges, which also used I-girders, they reported moment-based DOC values of the order of 55% on average based on midspan stress measures. These values are slightly higher than the construction-stage DOC values obtained in the present study and support the conclusion that substantial continuity can develop during deck placement when the girder ends are encased in a previously cast partial diaphragm.

Based on these findings, the research team recommends that low construction-stage DOC values should not be assumed to apply to all diaphragm configurations. Instead, the expected construction-stage continuity should be related to the specific diaphragm detail, including the presence or absence of a bond breaker plate, the degree of diaphragm restraint (fixed vs. pinned), and the height of the diaphragm cast before deck placement. Based on the results from this investigation, the team recommends that bridges without a bond breaker plate or with relatively deep partially cast diaphragm, a construction-stage DOC on the order of 40% may be representative. For bridges with pinned diaphragms and a bond breaker plate, a construction-stage DOC of approximately 25% may be assumed. For design using conventional software, it is conservative to ignore these effects since the stress quickly reverse at the time of casting the deck and minimally offset the stresses at controlling points. It would also have minimal impact on deflections during casting of the deck. These values should be subject to verification as MoDOT refines their SDCL design guidance in the future.

Another factor not specifically addressed in this study is the control of cracking with the bond breaker plate. The tensile stresses developed in the models all indicate that cracking of the diaphragm is likely when the concrete deck is poured. The addition of the bond breaker plate will help to control that cracking and limit reflecting to the exposed surface of the concrete.

4.8.2 Degree of Continuity Under Live Loading

For the live-load stage, the DOC was computed using the calibrated live-load model and representative truck loading positions. Unlike the dead-load stage, the live-load model represented the completed bridge after the deck and continuity diaphragms had hardened. Therefore, the bridge was expected to develop a substantially higher level of continuity at the intermediate bents.

The DOC calculations were performed for representative live-load cases (Truck Positions 3 and 4). These truck positions were selected because they produced meaningful response in both span and support regions and were useful for distinguishing between simple-span and continuous behavior. The comparison among the calibrated model, simple-span limit, and fully continuous limit for Truck Positions 3 and 4 is shown in Figure 4-29 and Figure 4-30, respectively. This comparison demonstrates that the completed bridge response is much closer to the continuous limit than to the simple-span limit under the selected live-load configurations, as expected.

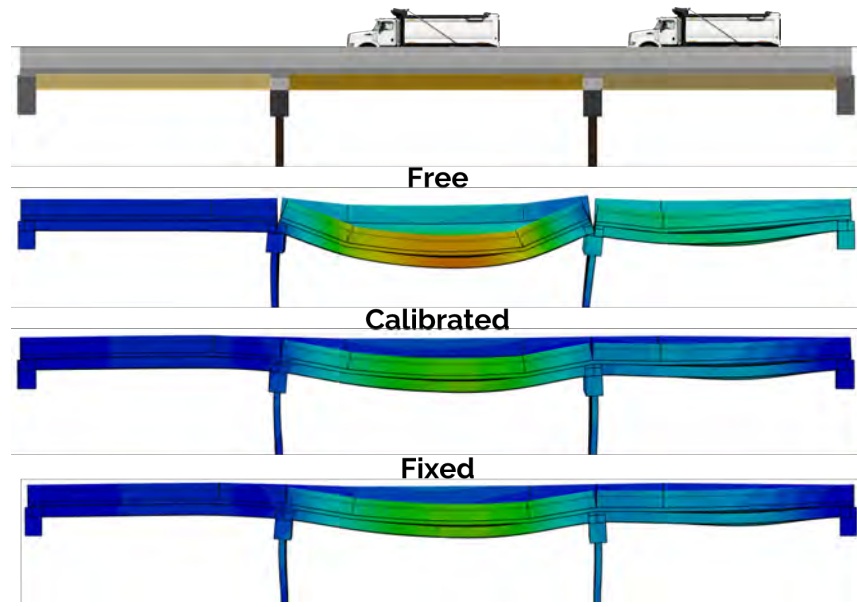


Figure 4-29. Comparison of calibrated model response with simple-span and fully continuous limits for Truck Position 3.

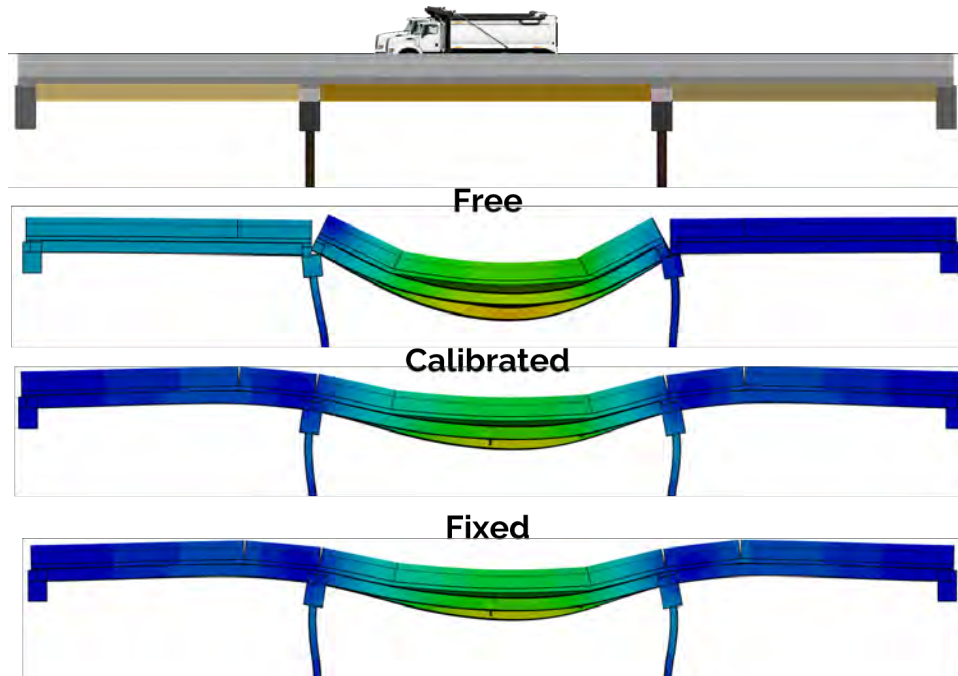


Figure 4-30. Comparison of calibrated model response with simple-span and fully continuous limits for Truck Position 4.

Because the live-load model accurately captures both curvature and deflection values (see Section 4.6), both DOC metrics are reported herein. The curvature-based and deflection-based DOC estimates for Truck Position 3 are shown in Figure 4-31 and Figure 4-32, respectively. These DOC values were computed using response quantities measured at the midspan of Spans 1 and 2 and for girder line 2. The curvature-based DOC values averaged approximately 93%

while the displacement-based DOC values averaged approximately 91%. These values, which are close to 100%, support the qualitative observations from Figure 4-29, where the calibrated model response approximated that of the fixed model.

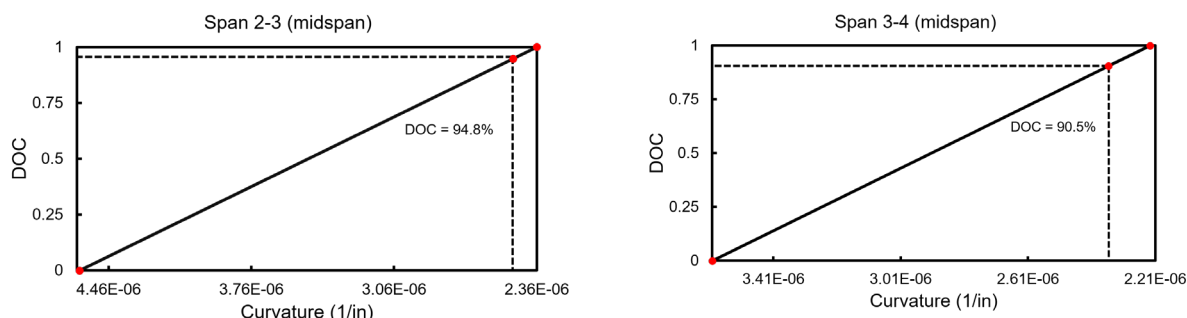


Figure 4-31. Curvature-based degree of continuity for Truck Position 3.

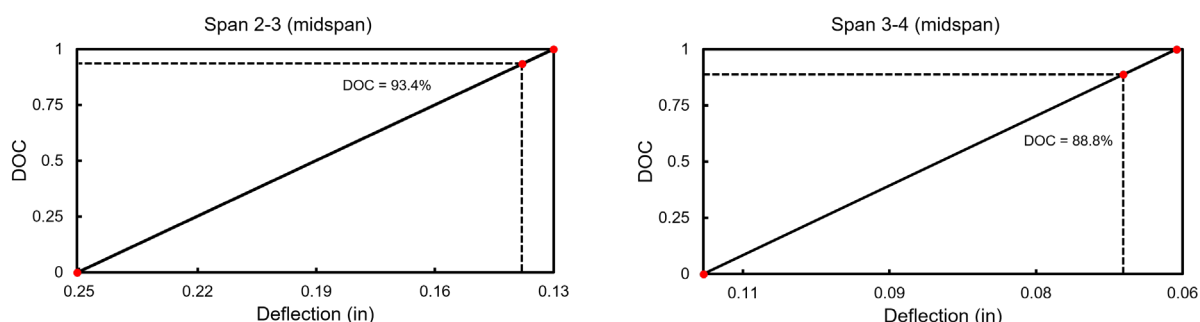


Figure 4-32. Deflection-based degree of continuity for Truck Position 3.

Consistent with the results for Truck position 3, the DOC estimates for Truck position 4 also indicated a high degree of continuity. The curvature- and deflection-based DOC estimates for this loading case are shown in Figure 4-33. Because Span 2-3 was the only loaded span for Truck position 4 (see Figure 4-30), the DOC was calculated using only response quantities measured at the midspan of Span 2-3. The curvature-based DOC was 92.3%, while the deflection-based DOC was 93.1%, corresponding to an average DOC of approximately 93%. These values are consistent with those obtained for Truck position 3 and further support the qualitative observations from Figure 4-30.

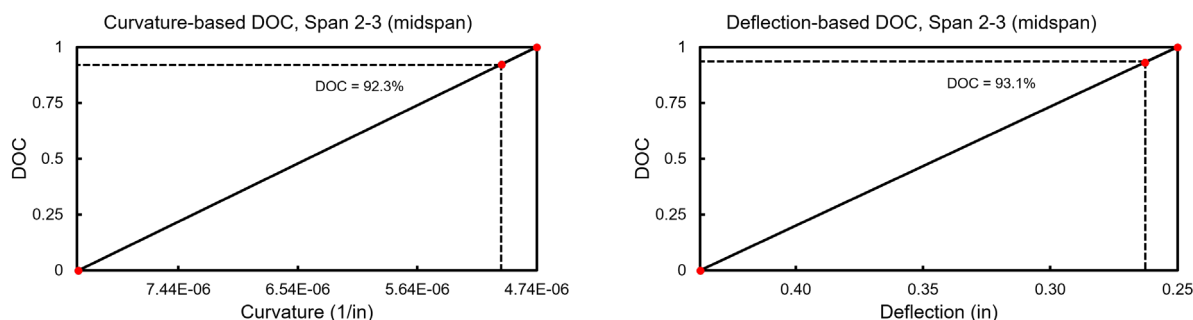


Figure 4-33. Curvature- and deflection-based degree of continuity for Truck Position 4.

The live-load DOC results are summarized in Table 4-8. Based on the results from Truck Positions 3 and 4, the average live-load DOC was approximately 92%. The curvature-based and deflection-based estimates were consistent, with an overall difference of less than approximately 4%. This close agreement indicates that the continuity estimate was not strongly dependent on the response quantity used in the calculation. The results confirm that Bridge A9513 developed substantial live-load continuity at the intermediate bents, consistent with the intended SDCL behavior.

Table 4-8. Degree-of-continuity summary for Truck Positions 3 and 4.

Truck position	Location	Method	
		Curvature	Deflection
3	Span 2-3	94.8	93.4
	Span 3-4	90.5	88.8
4	Span 2-3	92.3	93.1

4.9 Summary

Two finite element models were developed for Bridge A9513 to evaluate its behavior during deck pour and under live-load testing. The first, referred to here as the dead-load model, represented the construction-stage condition during deck placement, when the steel girders carried the wet concrete deck load primarily as non-composite members. The second, referred to here as the live-load model, represented the completed bridge, after the deck and continuity diaphragms had hardened and the bridge was expected to develop continuity across the intermediate bents.

The calibrated models captured the measured bridge response under both construction-stage and live-load conditions. The dead-load model reproduced the measured bottom-flange strains and deflections recorded during deck placement, while the live-load model accurately represented the curvatures and deflections measured during controlled truck testing. These results indicate that the selected material properties, contact definitions, support conditions, and continuity details provided a realistic representation of the bridge behavior.

The dead-load analyses showed that the bridge developed measurable restraint during deck placement. Deflection-based calculations indicated a dead-load DOC of approximately 42% for Span 2-3 and of approximately 23% for Span 3-4, suggesting that the bridge did not behave as a purely simple-span system during deck placement. The relatively high DOC in Span 2-3 was attributed to the depth of the partially cast diaphragm, which was approximately 70% of the steel girder depth.

In contrast, the live-load analyses showed that Bridge A9513 developed a high level of continuity after completion. The average degree of continuity calculated from the live-load analyses was approximately 92%. Deflection-based and curvature-based estimates produced similar results, with differences generally less than 4%. This close agreement suggests that

continuity was reflected consistently in both the local structural response and the overall stiffness of the bridge system.

Together, the dead-load and live-load results demonstrate the staged behavior intended in the SDCL concept, while also showing that partial continuity can develop during construction when a relatively deep partial diaphragm is present. The calibrated models therefore provide a useful numerical framework for interpreting the field response of Bridge A9513 and for extending the study beyond the specific bridge tested.

Although Bridge A9513 provides direct evidence for one non-skewed, three-span configuration, MoDOT design recommendations must cover a broader range of bridge geometries and design conditions. Therefore, the validated modeling framework is used in the next chapter to study the effects of span arrangement, skew angle, support flexibility, diaphragm stiffness, connection stiffness, and AASHTO design loading configurations. The results from this parametric study are intended to support the development of practical design recommendations for MoDOT's EPG.

Chapter 5. Parametric Analysis

The validated modeling framework discussed in the previous chapter was extended to evaluate how selected bridge parameters influence continuity, internal force distribution, and force transfer at the intermediate bents for SDCL bridge systems. Four bridge configurations were considered, as summarized in Table 5-1: (1) a Base Bridge, which used the same geometry as Bridge A9513; (2) a Skew Angle Bridge, which had span lengths similar to Bridge A9513 but with the intermediate and end bents skewed at 45°; (3) a Plate Girder Bridge, which had longer spans than Bridge A9513 while maintaining similar interior-to-exterior span length ratios; and (4) an Unbalanced Spans Bridge, which had a geometry similar to Bridge A9513 but with the interior span length approximately doubled relative to the base bridge.

Each bridge configuration was analyzed under five live-load cases from the AASHTO LRFD Bridge Design Specifications, 10th Edition [47], along with one superimposed dead-load case representing the future wearing surface. For each load case, two diaphragm types, pinned and fixed, were evaluated with and without a bond breaker plate within the diaphragm region. This resulted in 24 finite element models per bridge configuration and 96 finite element models in total, as summarized in Table 5-1. The factored live-load and superimposed dead-load responses were analyzed separately and then combined by superposition to evaluate the force demands in the continuity connection.

Table 5-1. Summary of parametric analysis models.

<i>Bridge Configuration</i>	<i>Live-Load Cases</i>	<i>Superimposed Dead-Load Cases</i>	<i>Pinned Diaphragm</i>	<i>Fixed Diaphragm</i>	<i>Plate</i>	<i>No Plate</i>	<i>Total Models</i>
<i>Base Bridge</i>	5	1	✓		✓		6
	5	1	✓			✓	6
	5	1		✓	✓		6
	5	1		✓		✓	6
<i>Skew Angle Bridge</i>	5	1	✓		✓		6
	5	1	✓			✓	6
	5	1		✓	✓		6
	5	1		✓		✓	6
<i>Plate Girder Bridge</i>	5	1	✓		✓		6
	5	1	✓			✓	6
	5	1		✓	✓		6
	5	1		✓		✓	6
<i>Unbalanced Spans Bridge</i>	5	1	✓		✓		6
	5	1	✓			✓	6
	5	1		✓	✓		6
	5	1		✓		✓	6
<i>Total</i>							96

The results from the parametric models were used to quantify the average tensile stresses developed in the deck reinforcement and the compressive stresses transferred through the steel compression blocks at the intermediate bents. These stress demands were then compared with the available reinforcement and compression block capacities to assess whether the continuity connection was adequately proportioned. If the connection demands remain below the corresponding capacities for the range of models considered in this study, the results provide evidence that similar SDCL bridge configurations may be expected to achieve adequate continuity-connection performance under comparable design conditions.

The models that included a bond breaker plate used the detail shown in Figure 5-1. The bond breaker plate was located within the diaphragm region and extended vertically from the top of the compression blocks to 1 in. above the construction joint. The depth of the first diaphragm pour was assumed to be one-half of the total superstructure depth, which was measured from the top of the slab to the bottom of the bottom flange. In the finite element models, the bond breaker plate was represented through contact definitions rather than as a separate structural component. Frictional contact was assigned along the diaphragm interfaces formed during the first diaphragm pour and along the portions of the diaphragm interface above the construction joint that come into contact with the bond breaker plate after the second diaphragm pour. This modeling approach allowed the diaphragm to transfer compression and frictional forces while eliminating unintended tensile forces across the bond breaker plate region.

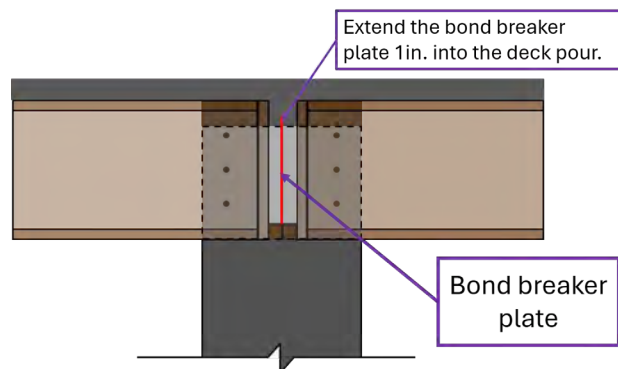


Figure 5-1. Schematic representation of the bond breaker plate detail at the intermediate concrete diaphragm.

5.1 Diaphragm Types and Reinforcement Layout

Two diaphragm types, pinned and fixed, were considered in this study to evaluate the influence of support restraint on bridge behavior. In the finite element models, the distinction between these diaphragm types was represented through two main features, namely the contact definition between the concrete bent cap and the concrete diaphragm, and the reinforcement details used to transfer forces across the diaphragm-cap connection.

The first diaphragm type corresponds to a pinned diaphragm. In practice, this condition is achieved by applying a debonding agent, such as bituminous pile paint or a layer of roofing felt,

at the top surface of the concrete bent cap, before the first part of the concrete diaphragm is poured. This detail reduces restraint at the support while still allowing vertical and lateral load transfer through the shear key and dowel bars. In the finite element models, the diaphragm-cap interface was represented using frictional contact with a coefficient of friction equal to 0.4, as shown in Figure 5-2.

In addition to the surface treatment, the pinned diaphragm configuration includes dowel bars that extend through the shear key along the centerline of the bent cap. These dowel bars provide load transfer while allowing limited rotation and separation between the diaphragm and bent cap. In the finite element models, these dowel bars were modeled explicitly, as discussed later in this section. The specific dowel bar layouts used for the pinned diaphragm configuration with and without bond breaker plate are shown Figure 5-3, Figure 5-4, and Figure 5-5. In the finite element models, the dowel bars were modeled explicitly using beam elements, as shown in Figure 5-6.

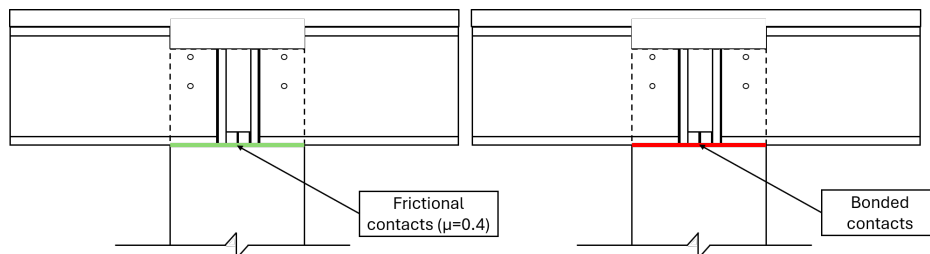


Figure 5-2. Contact definition for pinned (left) and fixed (right) diaphragm, respectively.

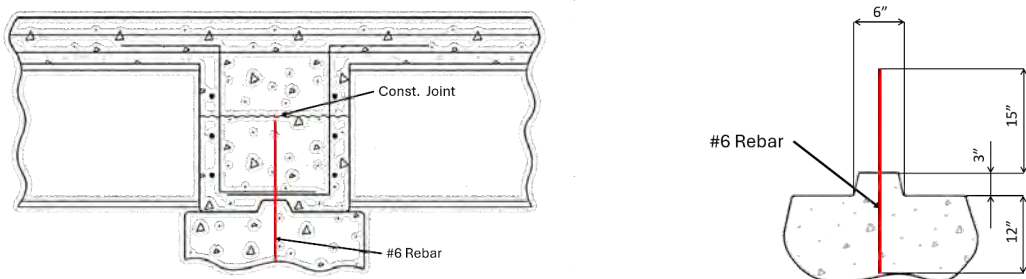


Figure 5-3. Dowel bar detail for pinned diaphragm type without bond breaker plate.

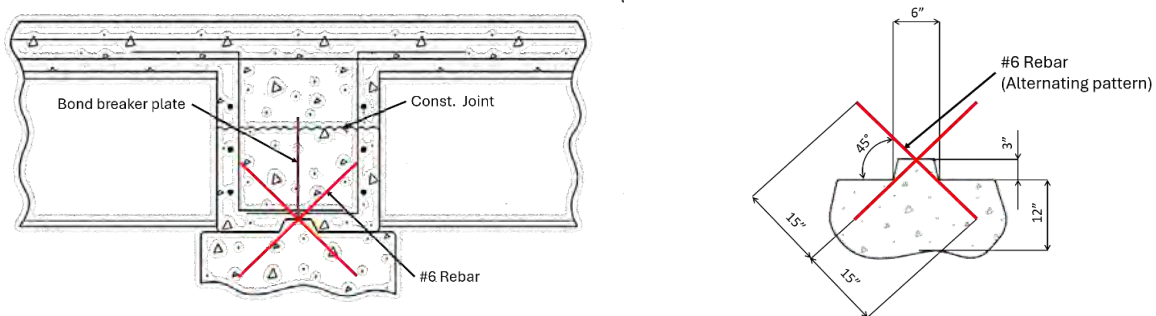


Figure 5-4. Dowel bar detail for fixed diaphragm type with bond breaker plate.

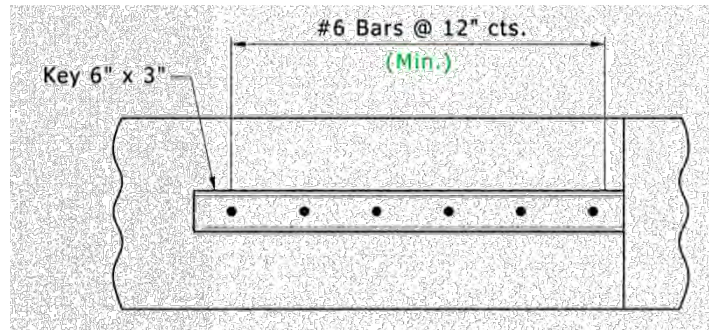


Figure 5-5. Spacing of the vertical dowel bars through shear keys.

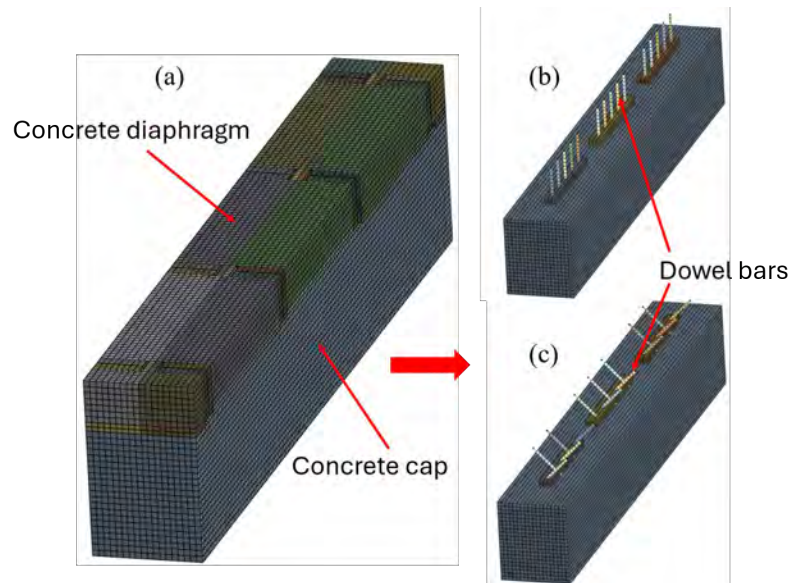


Figure 5-6. Dowel bar modeling for pinned diaphragm models using beam elements: (a) concrete diaphragm and bent cap; (b) vertical dowel bars used for the models without a bond breaker plate; and (c) inclined dowel bars used for the models with a bond breaker plate.

The second diaphragm type corresponds to a fixed diaphragm. For this configuration, no debonding agent or roofing felt is applied at the interface between the bent cap and diaphragm. In practice, the fixed diaphragm configuration includes reinforcement bars, located within the outer faces of the bent cap, which extend into the diaphragm and create a more monolithic connection that restrains rotation and separation. In the finite element models, the diaphragm-cap interface was represented using a bonded contact between the concrete cap and diaphragm surfaces, as shown in Figure 5-2. Because a bonded interface was assumed to provide full force transfer between the diaphragm and bent cap, the reinforcement bars crossing the diaphragm-cap interface were not modeled explicitly.

5.2 Bridge Types and Load Cases

All bridge configurations considered in the parametric analysis were subjected to superimposed dead loads and live loads based on the AASHTO LRFD Bridge Design Specifications, 10th Edition [47]. The superimposed dead load consisted of a uniform future wearing surface load of 35 psf

applied over the bridge deck between the concrete barriers and was designated as DW. Vehicular live loading consisted of a combination of the design truck (HL-93) shown in Figure 5-7 or the design tandem with the design lane load (AASHTO LRFD Section 3.6.1.2.1). Additional live-load cases included 90% of the effect of two design trucks with the design lane load in accordance with Section 3.6.1.3.1 and 100% of the effect of two design tandems with the design lane load, as described in Commentary Section C3.6.1.3.1. Multiple presence factors and dynamic load allowance factors were applied in accordance with Sections 3.6.1.1.2-1 and 3.6.2.1-1, respectively. Because the bridge response was evaluated using detailed finite element models, the live-load distribution factors specified in Section 4.6.2.2 were not applied.

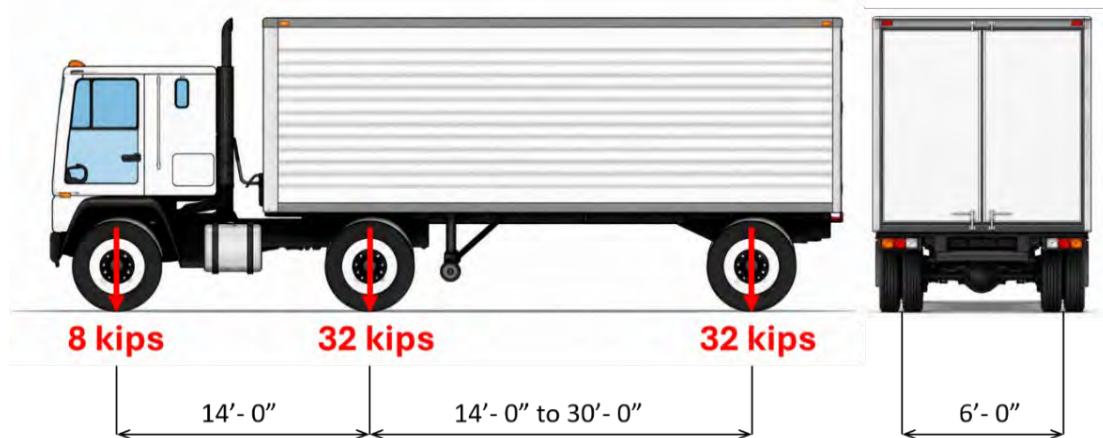


Figure 5-7. AASHTO HL-93 design truck dimensions and axle loads.

5.2.1 Base Bridge Description and Load Cases

The Base Bridge configuration retains the geometry, material properties, support conditions, and continuity connection details of the Bridge A9513 model, as discussed in Chapter 4. The overall bridge dimensions, span lengths, deck geometry, intermediate bent configuration, steel girder sections, pile-supported foundation system, and reinforcement details remain unchanged throughout the analysis. Any changes introduced in the subsequent bridge configurations are limited to the specific geometric parameter under investigation.

The five live-load cases used to evaluate the Base Bridge were developed from the AASHTO LRFD Bridge Design Specifications [47] and are summarized in Figure 5-8. Each load case consisted of the AASHTO HL-93 design truck (Figure 5-7) combined with the design lane load to represent the governing traffic load. Both the design truck and lane load were factored in accordance with the AASHTO LRFD Specifications. For load cases 1, 2, 4, and 5, the factored design truck loads with impact consist of an 18.62-kip front axle, two 74.48-kip rear axles and equal spacing of 14 ft between axles, while the design lane load consisted of a uniformly distributed load of 0.112 kips/ft². For load case 3, the factored design truck loads with impact consist of a 16.76-kip front axle, two 67.02-kip rear axles and equal spacing of 14 ft between axles, while the design lane load consisted of a uniformly distributed load of 0.101 kips/ft². The truck positions and lane loads were selected to produce critical positive and negative moments at the intermediate supports and adjacent spans.

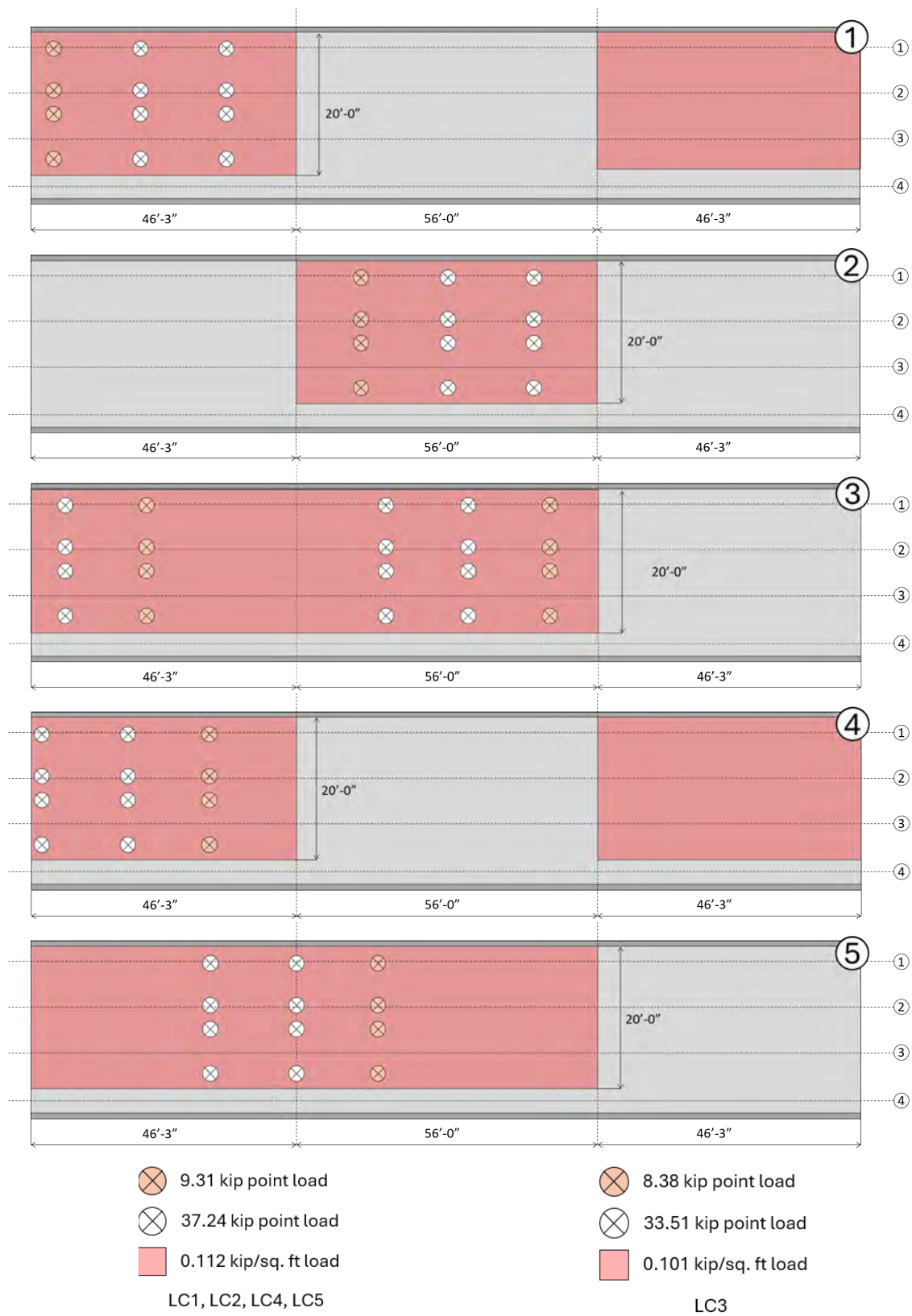


Figure 5-8. Load cases 1-5 for the Base Bridge to produce most critical moments at the intermediate bents.

5.2.2 Skew Angle Bridge Description and Load Cases

The second bridge configuration is a Skew Angle Bridge, which is used to investigate the influence of skew on the structural behavior of SDCL bridges. In this configuration, the intermediate and end bents are oriented at a 45° skew relative to the bridge centerline, while all other bridge properties remained consistent with those of the Base Bridge. As shown in Figure 5-9, the bridge consisted of three spans measuring 45 ft, 56 ft, and 45 ft between bent centerlines, and was supported by four longitudinal steel girders spaced at 6 ft 10 in. on center. The end spans were modeled using W24x84 rolled steel sections, while the middle span used W24x94 sections to accommodate the larger load demand. The figure also shows the intermediate bearings, end plates, connection plates, and shear studs that form the continuity connection at the intermediate supports.

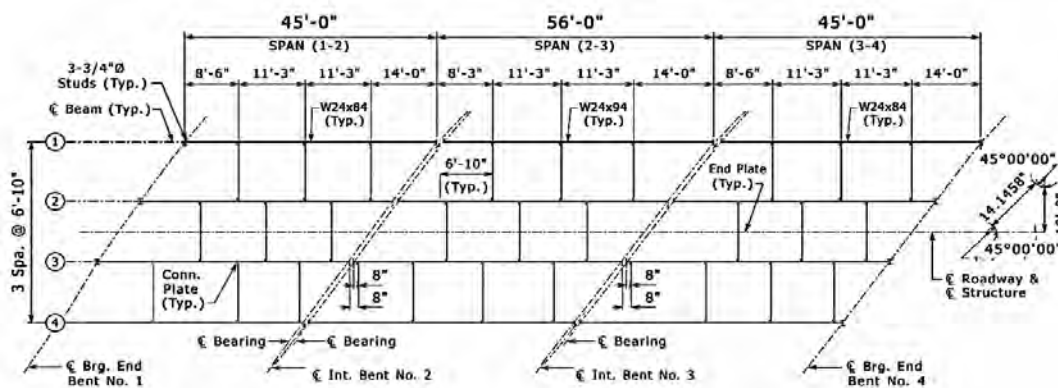


Figure 5-9. Steel plan view and span lengths for the Skew Angle Bridge.

Figure 5-10 shows the transverse cross section of the Skew Angle Bridge. The bridge has a 24-ft-wide roadway with two 12-ft traffic lanes and includes reinforced concrete barriers along both edges. The reinforced concrete deck is 8.5 in. thick. The deck reinforcement follows the same detailing used for the Base Bridge over most of the bridge length. However, additional reinforcement is provided within the end regions of the deck to satisfy the requirements for skewed bridges. Because the bridge skew exceeds 25° , supplemental reinforcement is placed in accordance with AASHTO LRFD Section 9.7.2.5 to control cracking and improve force transfer near the skewed supports.

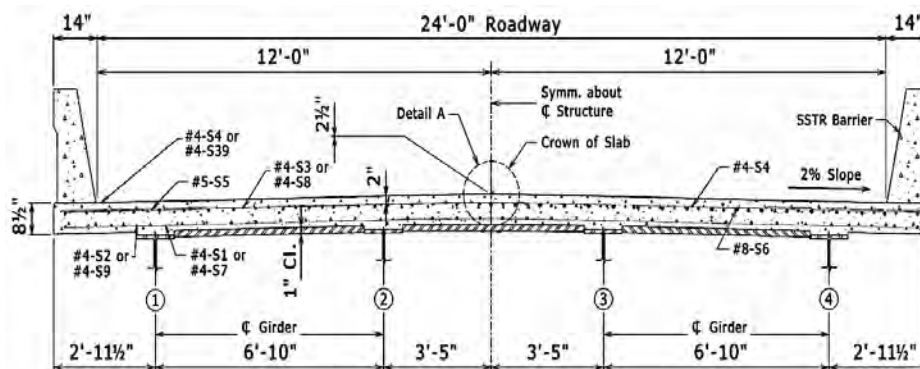


Figure 5-10. Skew Angle Bridge cross section shows four girders and girder spacing.

This bridge was evaluated using the same AASHTO LRFD HL-93 live-load cases applied to the Base Bridge. Figure 5-11 illustrates the truck positions and lane loading used in the analysis.

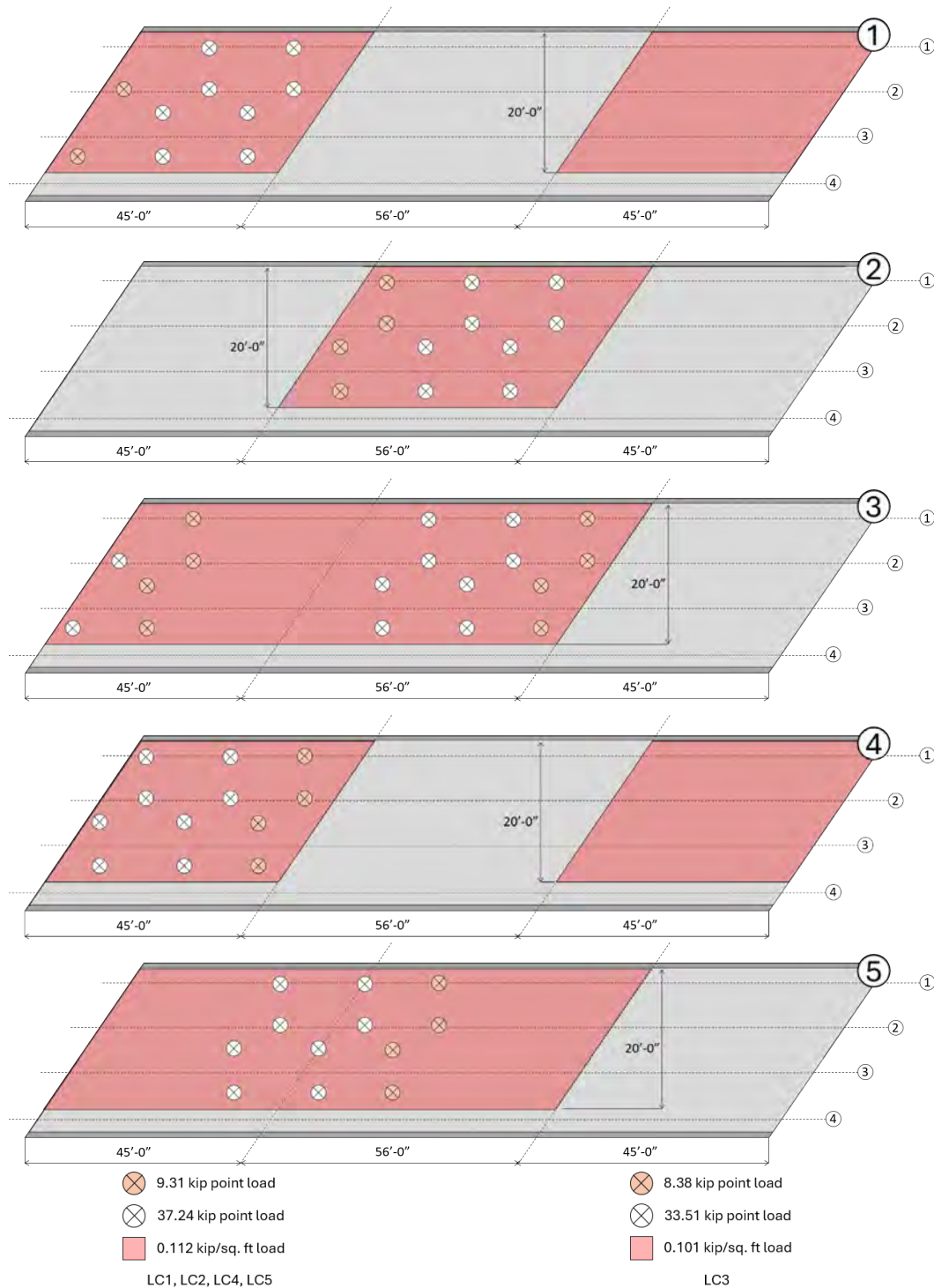


Figure 5-11. Load cases 1-5 for the Skew Angle Bridge to produce the most critical moments at the intermediate bents.

5.2.3 Plate Girder Bridge Description and Load Cases

The third bridge configuration is a Plate Girder Bridge, which is used to evaluate the influence of girder type and increased span length on the structural behavior and continuity performance of SDCL bridges. Unlike the Base Bridge, which uses rolled steel sections, this configuration uses welded steel plate girders to accommodate longer spans and higher flexural demands. As shown in Figure 5-12, the bridge consists of three spans measuring 100 ft, 120 ft, and 100 ft between bent centerlines, and is supported by four longitudinal plate girders spaced at 6 ft 10 in. on center. The bridge also uses two 3-ft-diameter reinforced concrete columns at each intermediate bent, spaced 5 ft from the ends of the beam cap, instead of the H-pile supports used in the Base Bridge. Intermediate diaphragms are provided at regular intervals throughout each span to improve lateral stability, and the figure also shows the end plates, connection plates, and bearings located at the intermediate supports to develop continuity between adjacent spans. All other bridge properties, including the deck geometry, material properties, and continuity details, remained consistent with those of the Base Bridge.

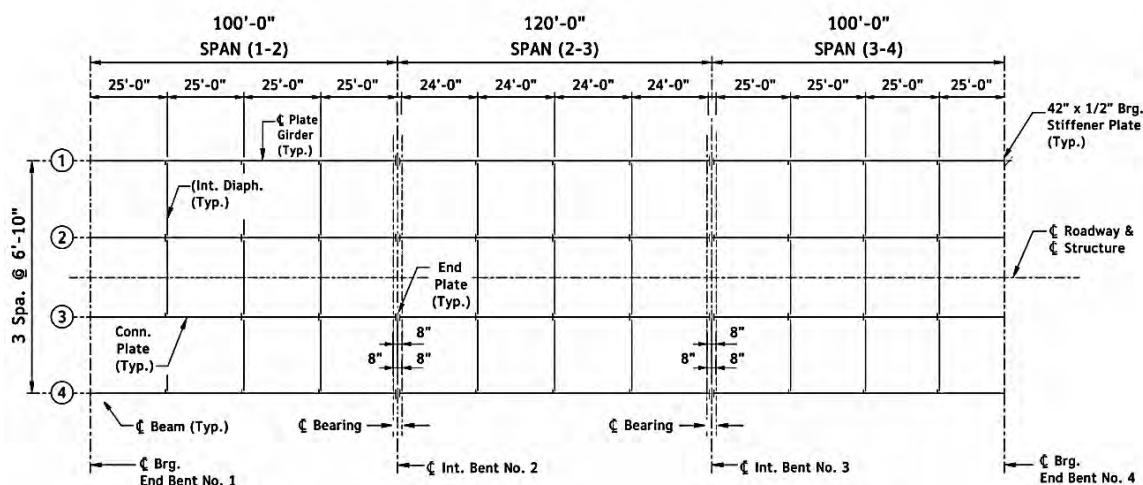


Figure 5-12. Steel plan view and span lengths for the Plate Girder Bridge.

Figure 5-13 shows the cross section of the plate girder bridge. The bridge carries a 24-ft-wide roadway with two 12-ft traffic lanes and reinforced concrete barriers along both edges. The reinforced concrete deck is 8.5 in. thick. The four plate girders are spaced at 6 ft 10 in. on center with overhang dimensions identical to those of the Base Bridge, and reinforcement layout is also shown in the figure.

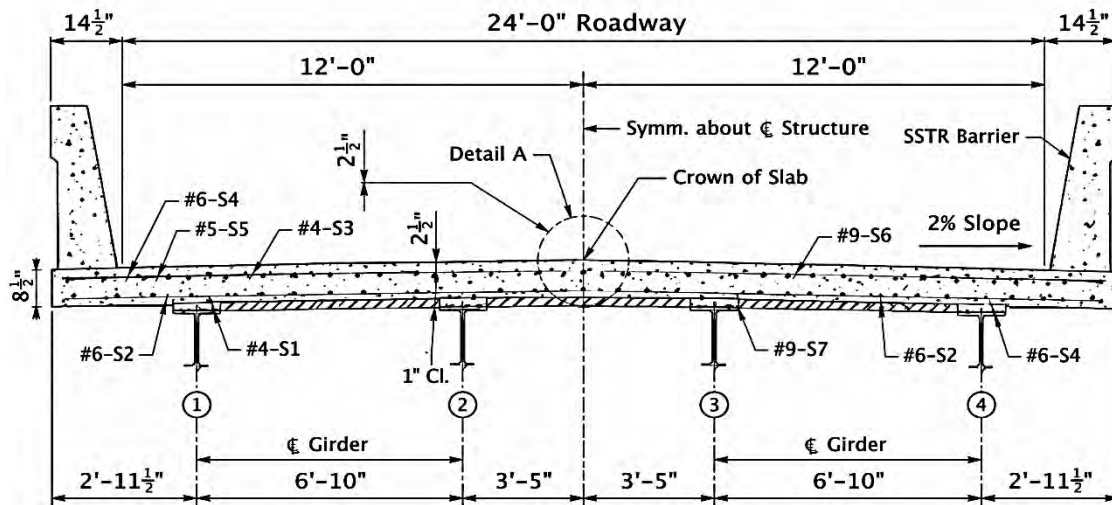


Figure 5-13. Plate Girder Bridge cross section showing four girders and girder spacing.

Figure 5-14 presents the built-up steel girder details for the Plate Girder Bridge. The exterior spans (Spans 1 and 3) use welded plate girders with 16 in. wide by 1 in. thick top and bottom flanges and a 42 in. deep web fabricated from a 1/2 in. thick plate. The center span uses a larger built-up section with 18 in. wide by 7/8" on top, and 1 in. thick flange on the bottom and the same 42 in. deep, 1/2 in. thick web to accommodate the increased span length and flexural demand. The exterior span girders use 16 in. × 44 in. × 1 in. end plates, while the center span girders use 18 in. × 44½ in. × 1 in. end plates. Steel compression blocks are welded to the bottom of each end plate and measure 2 in. × 3 in. × 16 in. for the exterior spans and 2 in. × 3 in. × 18 in. for the center span.

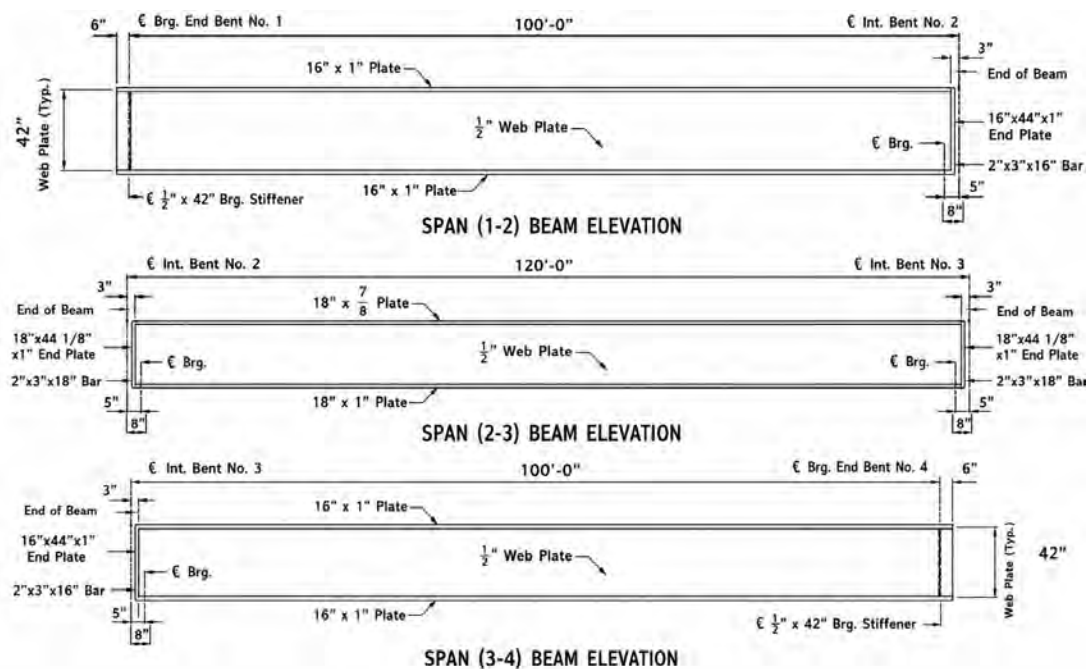


Figure 5-14. Girders and girder-end details at intermediate bents for the Plate Girder Bridge.

This bridge was evaluated using the same five AASHTO LRFD HL-93 live-load cases applied to the Base Bridge. Figure 5-15 illustrates the truck positions and lane loading used in the analysis.

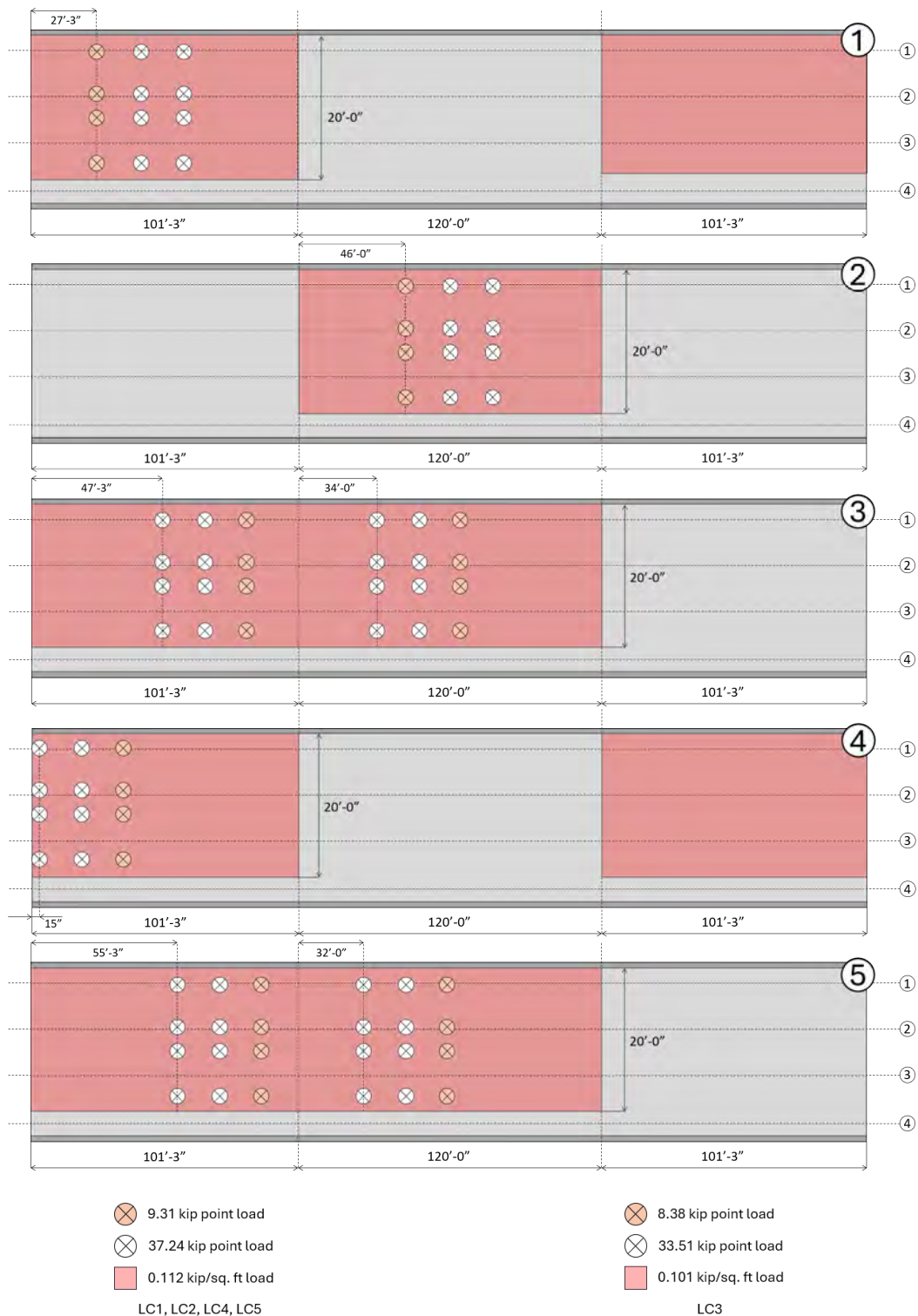


Figure 5-15. Load cases 1-5 for the Plate Girder Bridge to produce most critical moments at the intermediate bents.

5.2.4 Unbalanced Span Bridge Description and Load Cases

The fourth and final bridge configuration is the Unbalanced Spans Bridge, which is used to evaluate the influence of unequal span lengths on the structural behavior and continuity performance of SDCL bridges. Unlike the Base Bridge, this configuration has a longer center span and shorter exterior spans, which changes the distribution of positive moments within the spans and negative moments at the intermediate supports. As shown in Figure 5-16, the bridge consists of three spans measuring 45 ft, 90 ft, and 45 ft between bent centerlines, and is supported by four longitudinal steel girders spaced at 6 ft 10 in. on center. The exterior spans use W33×118 rolled steel sections, while the center span uses heavier W33×263 sections to accommodate the increased span length and flexural demand. Intermediate diaphragms are provided at regular intervals throughout the bridge, and the figure also shows the end plates, connection plates, bearings, and shear studs located at the intermediate supports to provide continuity between adjacent spans.

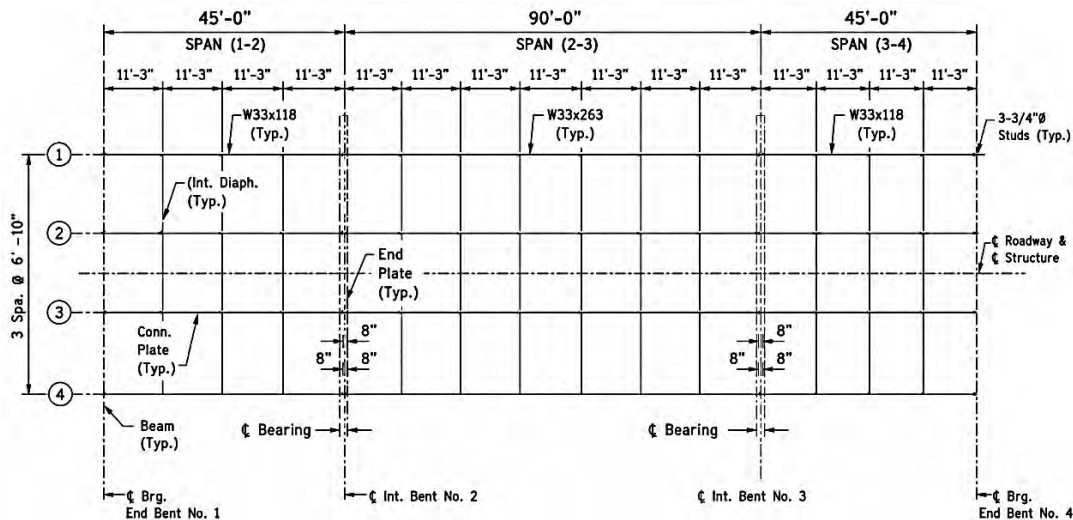


Figure 5-16. Steel plan view and span lengths for the Unbalanced Span Bridge.

The cross section of the unbalanced spans bridge is shown in Figure 5-17. The bridge carries a 24-ft-wide roadway with two 12-ft traffic lanes and reinforced concrete barriers along both edges. The reinforced concrete deck is 8.5 in. thick. The four steel girders are spaced at 6 ft 10 in. on center with overhang dimensions identical to those of the Base Bridge. The figure also illustrates the deck reinforcement layout.

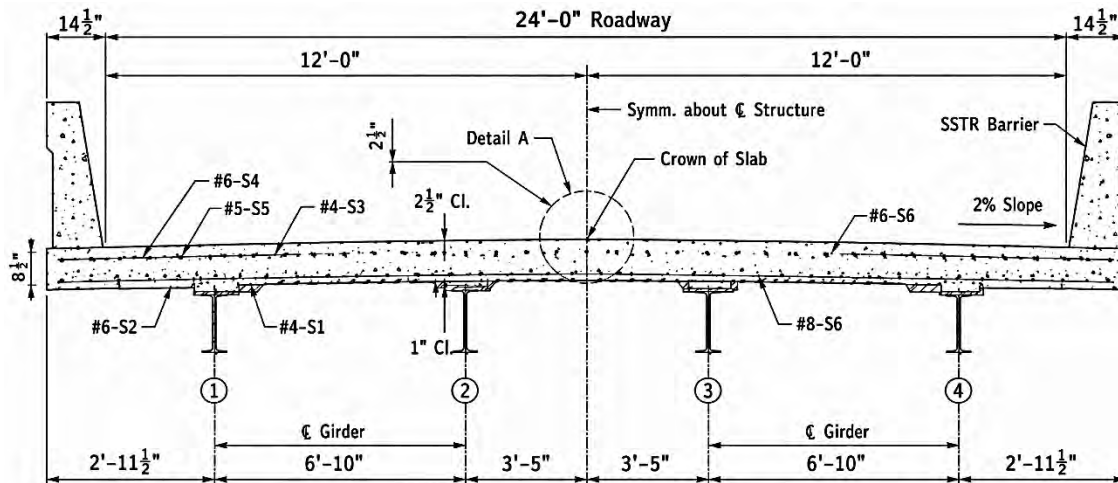


Figure 5-17. Unbalanced Span Bridge cross section showing four girders and girder spacing.

Figure 5-18 shows the steel girder details for the unbalanced spans bridge. The exterior spans use W33×118 rolled steel sections, while the center span uses W33×263 rolled steel sections to accommodate the greater flexural demands of the longer span. The exterior span girders use 11½ in. × 33 in. × 1 in. end plates, while the center span girders use 15¾ in. × 34½ in. × 1 in. end plates. Steel compression blocks are welded to the bottom of each end plate and measure 2 in. × 3 in. × 11½ in. for the exterior spans and 2 in. × 3 in. × 15¾ in. for the center span.

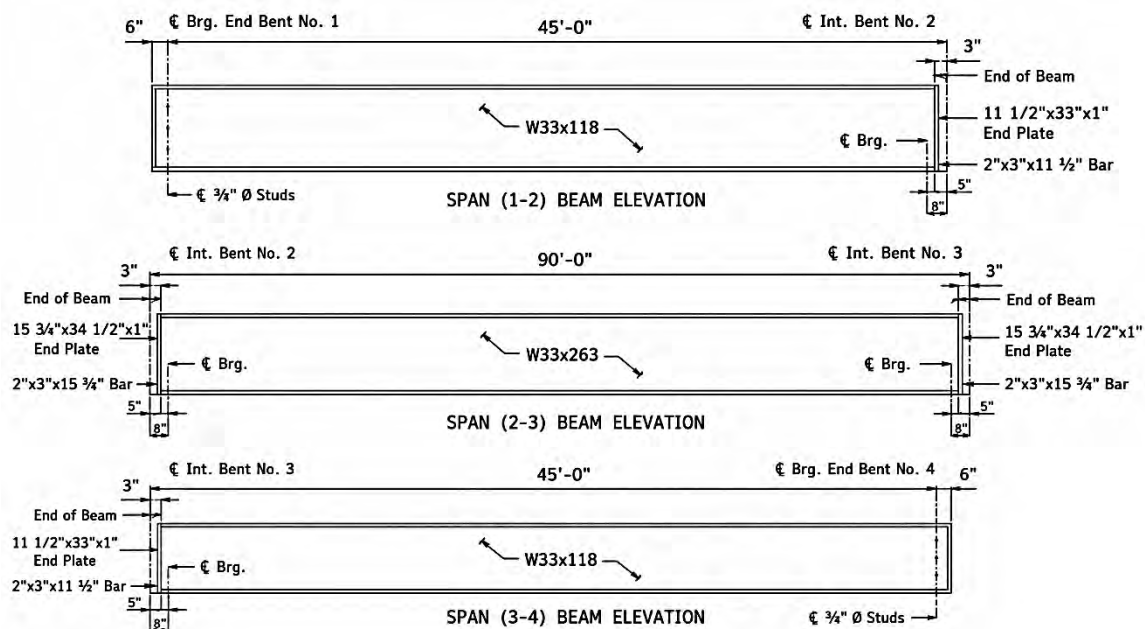


Figure 5-18. Girders and girder-end details at intermediate bents for the Unbalanced Span Bridge.

This bridge was evaluated using the same five AASHTO LRFD HL-93 live-load cases applied to the Base Bridge. Figure 5-19 illustrates the truck positions and lane loading used in the analysis.

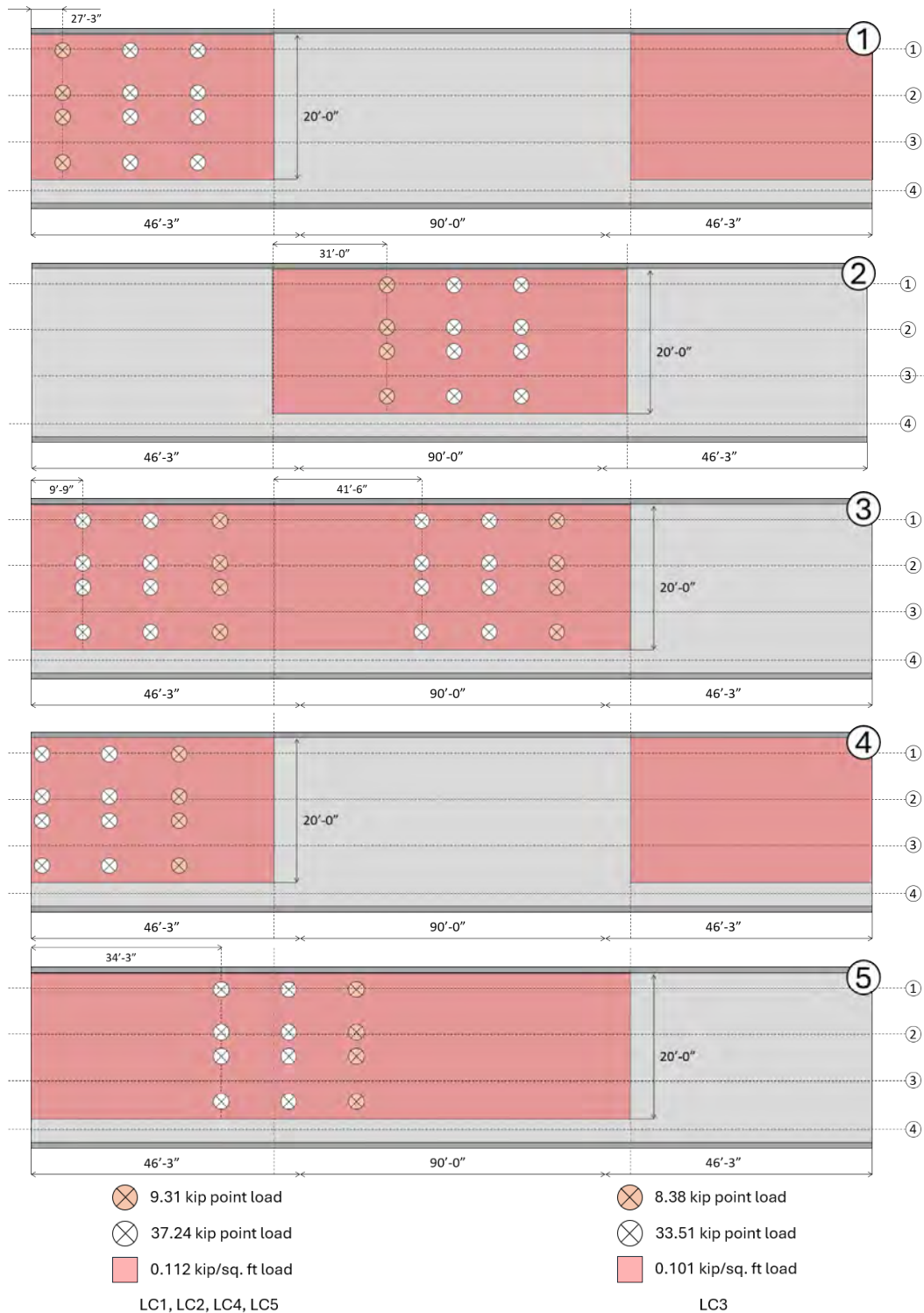


Figure 5-19. Load cases 1-5 for the Unbalanced Span Bridge to produce most critical moments at the intermediate bents.

5.3 Parametric Model Results

The parametric models were analyzed under factored HL-93 truck, lane live loading, and superimposed dead loading using the same linear-elastic modeling framework. Because these analyses were intended to evaluate the performance of the continuity connection, girder self-weight, deck concrete weight, and barrier loads were not included. The factored live-load and superimposed dead-load cases were analyzed separately and then combined by superposition. The corresponding stresses and continuity connection forces were added at the same output locations to obtain the total reported response for each bridge configuration.

The performance of the continuity connection was evaluated using the simplified design approach proposed by Azizinamini et al. [40] for SDCL steel bridges. The recommended design method assumes that the moment resistance of the continuity connection is provided primarily by the couple formed between the deck reinforcement in tension and the steel compression block in compression. Experimental and numerical studies showed that these two components contribute most of the connection capacity, while the contribution of the diaphragm concrete and transverse reinforcement may be conservatively neglected for design purposes.

Accordingly, the nominal flexural capacity of the continuity connection is

$$M_n = A_s f_y \left(d - \frac{H}{2} \right)$$

where

- A_s = area of deck reinforcement crossing the continuity joint
- f_y = reinforcement yield strength
- d = distance from the center of the deck reinforcement to the bottom of the girder
- H = height of the steel compression block

The required deck reinforcement may be determined from

$$A_s = \frac{M_u}{\phi f_y \left(d - \frac{H}{2} \right)}$$

where M_u the required factored moment at the continuity connection and ϕ is the strength reduction factor. To ensure that the compression block remains elastic, Azizinamini et al. [40] recommend verifying the minimum required compression block height using

$$H_{\min} = \frac{1.7 A_s f_y}{b_f F_{ypl}}$$

where b_f is the girder flange width and F_{ypl} is the yield strength of the steel compression block. Because the finite element models directly calculate the tensile force developed in the deck reinforcement region and the compressive force transferred through the steel compression blocks, the parametric analyses compare these force demands with the available reinforcement capacity and the compression block yield capacity.

The tensile force in the deck was obtained by integrating the normal stresses in the deck over the continuity region, as shown in Figure 5-20. Similarly, the average contact stress on the compression block face was calculated from the compressive force developed at that interface,

as shown in Figure 5-21. These quantities were compared with the yield strength of the deck reinforcement and the steel compression blocks to evaluate the adequacy of the continuity connection.

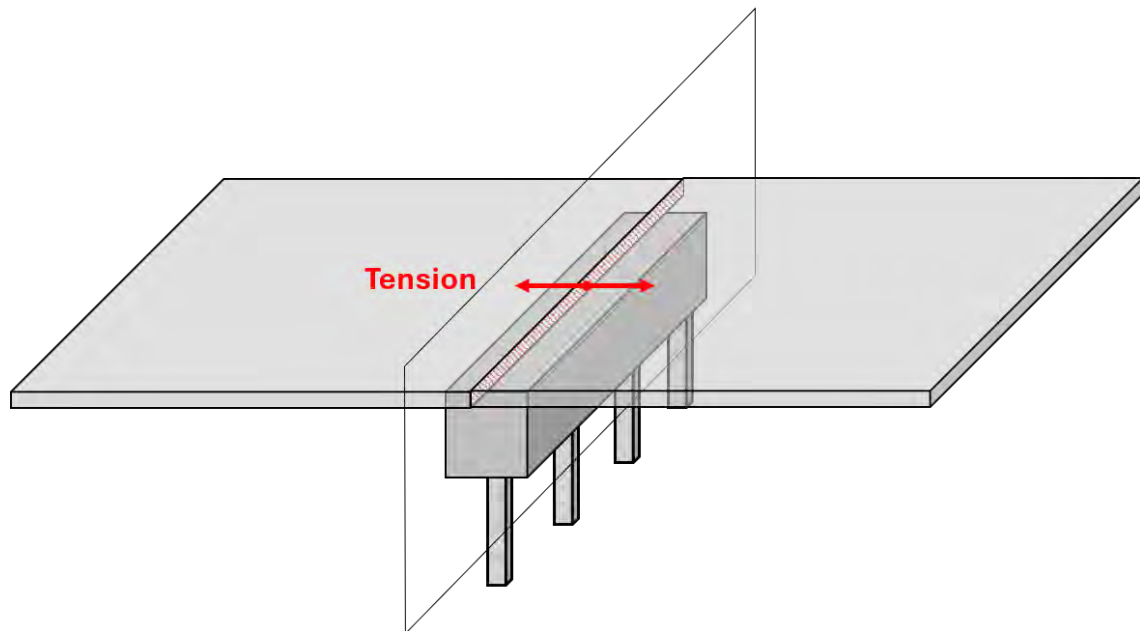


Figure 5-20. Tensile force developed in the deck reinforcement across the intermediate support.

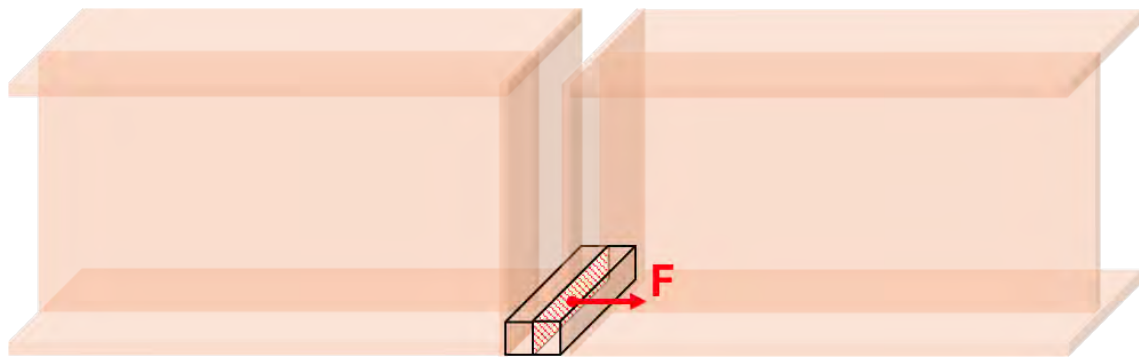


Figure 5-21. Compressive force transfer through the steel compression block at the intermediate support.

5.3.1 Base Bridge

Table 5-2 summarizes the average deck reinforcement stresses at Intermediate Bents 2 and 3, along with the compression block stresses for each diaphragm type, bond breaker plate configuration, and load case. The reported stresses represent the combined response from the factored live loads and superimposed dead loads. The breakdown of stresses due to factored live loads and superimposed dead loads for all bridge configurations is provided in Appendix B. Table 5-3 summarizes the reinforcement provided over the intermediate supports. Each intermediate bent includes 101 reinforcing bars with a total steel area of 39.67 in², and each

compression block, made with ASTM A709 Grade 50 steel, has a contact area of approximately 22.7 in².

Table 5-2. Average deck reinforcement stresses at Intermediate Bents 2 and 3 and average compression block contact stresses for the Base Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. stress, $\sigma_{c,avg}$ (ksi)	$\sigma_{c,avg}/F_{ypl}$ (%)
Fixed diaphragm	No Plate	LC1	9.47	15.8	3.90	6.5	3.04	6.1
		LC2	15.66	26.1	18.05	30.1	4.62	9.2
		LC3	16.97	28.3	14.99	25.0	4.52	9.0
		LC4	5.69	9.5	2.99	5.0	1.89	3.8
		LC5	11.55	19.2	5.18	8.6	3.32	6.6
	Plate	LC1	9.52	15.9	3.90	6.5	3.10	6.2
		LC2	15.71	26.2	18.11	30.2	4.66	9.3
		LC3	17.01	28.4	15.03	25.1	4.56	9.1
		LC4	5.71	9.5	2.99	5.0	1.92	3.8
		LC5	11.56	19.3	5.19	8.7	3.34	6.7
Pinned diaphragm	No Plate	LC1	9.95	16.6	4.29	7.1	2.42	4.9
		LC2	17.26	28.8	19.65	32.7	3.95	7.9
		LC3	17.64	29.4	15.88	26.5	4.76	9.5
		LC4	6.00	10.0	3.24	5.4	1.50	3.0
		LC5	12.25	20.4	5.60	9.3	2.71	5.4
	Plate	LC1	9.98	16.6	4.28	7.1	2.42	4.8
		LC2	17.38	29.0	19.75	32.9	3.96	7.9
		LC3	18.23	30.4	16.41	27.4	3.86	7.7
		LC4	5.91	9.8	3.28	5.5	1.82	3.6
		LC5	12.26	20.4	5.61	9.4	2.73	5.5

Table 5-3. Longitudinal deck reinforcement details for the Base Bridge.

Size	Number of bars	Area per bar (in ²)	Total area (in ²)
#4 Top	36	0.20	7.20
#8 Top	33	0.79	26.07
#4 Bottom	32	0.20	6.40
Total	101		39.67

The largest average tensile stresses in the deck reinforcement occurred during Load Case 2 for the pinned diaphragm with the bond breaker plate, where the reinforcement at Intermediate Bent 3 reached 19.75 ksi, corresponding to approximately 33% of the 60 ksi reinforcement yield strength. Therefore, the negative moment reinforcement remained well below yielding for all factored live-load and superimposed dead-load cases. The maximum average compression block contact stress was 4.76 ksi and occurred during Load Case 3 for the pinned diaphragm

without the bond breaker plate. This stress is less than 10% of the 50 ksi yield strength of the compression block steel. These results indicate that the reinforcement provided over the intermediate supports and the selected compression block dimensions were adequate to resist the applied demands. The relatively low stress ratios are also consistent with the 3D load-transfer captured by the FE models, in which force is distributed through the deck, diaphragm, girder-end region, and compression block rather than being restricted exclusively by an idealized reinforcement-compression block force couple. In addition, the stresses reported in Table 5-2 represent average stresses, which do not reflect localized stresses that occur because of the uneven load distributions shown in Figure 5-19.

Only small differences were observed between models with and without the bond breaker plate. The plate changed the average reinforcement stress by approximately 0.3% for the fixed diaphragm and 1.2% for the pinned diaphragm. The average compression block stress changed by approximately 1.1% for the fixed diaphragm and 3.6% for the pinned diaphragm, with no consistent increase or decrease across the individual load cases. These results indicate that the bond breaker plate had a negligible influence on the completed bridge reinforcement and compression block stresses. In contrast, the diaphragm condition produced a more noticeable effect. The pinned diaphragm increased the average reinforcement stress from 10.46 ksi to 11.24 ksi, an increase of approximately 7.5%, while the fixed diaphragm developed an average compression block stress of 3.50 ksi compared with 3.01 ksi for the pinned diaphragm, which was approximately 16% greater. These results show that the diaphragm configuration had a greater influence on the continuity connection stress distribution than the bond breaker plate.

5.3.2 Skew Angle Bridge

Table 5-4 summarizes the average deck reinforcement stresses at Intermediate Bents 2 and 3 and the average compression block contact stresses for each diaphragm configuration, bond breaker plate detail, and load case. As before, the stresses reported in the table include the combined effect of the factored live loads and superimposed dead load. The reinforcement provided over the intermediate supports are summarized in Table 5-5. Each compression block had a contact area of approximately 22.7 in². The reinforcement provided at each intermediate bent consisted of 101 reinforcing bars with a total steel area of 39.67 in².

Table 5-4. Average deck reinforcement stresses at Intermediate Bents 2 and 3 and average compression block contact stresses for the Skew Angle Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. stress, $\sigma_{c,avg}$ (ksi)	$\sigma_{c,avg}/F_{ypl}$ (%)
Fixed diaphragm	No Plate	LC1	6.70	11.2	3.21	5.4	1.59	3.2
		LC2	13.95	23.2	15.26	25.4	2.02	4.0
		LC3	14.54	24.2	12.90	21.5	2.09	4.2
		LC4	4.99	8.3	2.68	4.5	1.06	2.1
		LC5	10.45	17.4	5.16	8.6	1.42	2.8
	Plate	LC1	6.82	11.4	3.25	5.4	1.58	3.2
		LC2	14.23	23.7	15.40	25.7	2.00	4.0
		LC3	14.84	24.7	13.02	21.7	2.06	4.1
		LC4	5.10	8.5	2.71	4.5	1.06	2.1
		LC5	10.63	17.7	5.24	8.7	1.45	2.9
Pinned diaphragm	No Plate	LC1	6.37	10.6	3.15	5.2	1.43	2.9
		LC2	13.62	22.7	14.93	24.9	2.51	5.0
		LC3	14.22	23.7	12.59	21.0	2.77	5.6
		LC4	4.55	7.6	2.68	4.5	1.12	2.2
		LC5	10.04	16.7	4.92	8.2	1.88	3.8
	Plate	LC1	6.43	10.7	3.16	5.3	1.50	3.0
		LC2	13.87	23.1	15.24	25.4	2.60	5.2
		LC3	14.44	24.1	12.83	21.4	2.74	5.5
		LC4	4.58	7.6	2.71	4.5	1.17	2.3
		LC5	10.10	16.8	4.98	8.3	1.95	3.9

Table 5-5. Longitudinal deck reinforcement details for the Skew Angle Bridge.

Size	Number of bars	Area per bar (in ²)	Total area (in ²)
#4 Top	36	0.20	7.20
#8 Top	33	0.79	26.07
#4 Bottom	32	0.20	6.40
Total	101		39.67

The Skew Angle Bridge exhibited lower stress levels than the Base Bridge. The largest average reinforcement stress occurred during Load Case 2 for the fixed diaphragm with the bond breaker plate at Intermediate Bent 3, where it reached 15.40 ksi, or approximately 26% of the 60 ksi reinforcement yield strength. Thus, the reinforcement remained well below yielding for all loading conditions. The maximum average compression block stress was 2.77 ksi and occurred during Load Case 3 for the pinned diaphragm without the bond breaker plate, corresponding to approximately 6% of the 50 ksi yield strength of the compression block steel.

Similar to the Base Bridge, the bond breaker plate produced only minor changes in the deck reinforcement and compression block stresses, while the diaphragm condition had a more noticeable influence. The pinned diaphragm produced a slightly lower average reinforcement stress than the fixed diaphragm (8.77 ksi compared with 9.05 ksi), but a higher average compression block stress (1.97 ksi compared with 1.63 ksi). Overall, the skewed support did not significantly alter the force transfer mechanism but reduced the average reinforcement and compression block stresses by approximately 18% and 45%, respectively, compared with the Base Bridge.

5.3.3 Plate Girder Bridge

Table 5-6 provides the average reinforcement stresses at Intermediate Bents 2 and 3 and the average compression block contact stresses for each diaphragm type, bond breaker plate detail, and load case. The reinforcement provided over the intermediate supports are summarized in Table 5-7. Each compression block used in calculation of the average stress had a contact area of approximately 48 in². The reinforcement provided at each intermediate bent consisted of 126 reinforcing bars with a total steel area of 87.92 in².

Table 5-6. Average deck reinforcement stresses at Intermediate Bents 2 and 3 and average compression block contact stresses for the Plate Girder Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. stress, $\sigma_{c,avg}$ (ksi)	$\sigma_{c,avg}/F_{ypl}$ (%)
Fixed diaphragm	No Plate	LC1	8.05	13.4	3.48	5.8	2.84	5.7
		LC2	13.93	23.2	14.89	24.8	3.88	7.8
		LC3	21.03	35.0	9.85	16.4	4.70	9.4
		LC4	4.86	8.1	4.08	6.8	1.57	3.2
		LC5	23.39	39.0	10.40	17.3	5.19	10.4
	Plate	LC1	8.13	13.6	3.53	5.9	2.78	5.6
		LC2	14.18	23.6	15.15	25.3	3.83	7.7
		LC3	21.29	35.5	10.00	16.7	4.71	9.4
		LC4	4.91	8.2	4.12	6.9	1.54	3.1
		LC5	23.65	39.4	10.56	17.6	5.21	10.4
Pinned diaphragm	No Plate	LC1	7.95	13.3	3.75	6.3	2.50	5.0
		LC2	14.40	24.0	15.30	25.5	3.66	7.3
		LC3	21.45	35.8	9.93	16.6	4.69	9.4
		LC4	4.78	8.0	3.97	6.6	1.41	2.8
		LC5	23.91	39.9	10.49	17.5	5.25	10.5
	Plate	LC1	7.86	13.1	3.88	6.5	2.30	4.6
		LC2	14.42	24.0	15.33	25.5	3.43	6.9
		LC3	21.67	36.1	9.92	16.5	4.67	9.3
		LC4	4.74	7.9	3.92	6.5	1.31	2.6
		LC5	24.12	40.2	10.46	17.4	5.18	10.4

Table 5-7. Longitudinal deck reinforcement details for the Plate Girder Bridge.

Size	Number of bars	Area per bar (in ²)	Total area (in ²)
#6 Top	36	0.44	15.84
#9 Top	33	1.00	33.00
#6 Bottom	32	0.44	14.08
#9 Bottom	25	1.00	25.00
Total	126		87.92

The Plate Girder Bridge developed the highest reinforcement stresses among the bridge configurations investigated. The maximum average reinforcement stress occurred during Load Case 5 for the pinned diaphragm with the bond breaker plate at Intermediate Bent 2, where the reinforcement reached 24.12 ksi. Despite the larger bridge size and longer spans, this stress corresponds to about 40% of the 60 ksi reinforcement yield strength. The maximum compression block contact stress was approximately 5.25 ksi, which is about 10.5% of the 50 ksi yield strength of the compression block steel. The provided reinforcement and compression block dimensions remained adequate for the applied factored live-load and dead-load demands. These results indicate that the increased reinforcement area and larger compression block details adopted for the plate girder bridge are consistent with the increased force demands associated with this bridge configuration.

The bond breaker plate produced only minor changes in the deck tensile forces and compression block stresses, consistent with the observations for the previous bridge configurations. The bond breaker plate increased the average reinforcement stress by approximately 1.4% for the fixed diaphragm and 0.3% for the pinned diaphragm. The average compression block stress decreased by approximately 0.6% for the fixed diaphragm and 3.5% for the pinned diaphragm. In contrast, the diaphragm configuration produced a more noticeable, although still relatively small, effect. Averaged across both plate conditions, intermediate bents, and all load cases, the pinned diaphragm developed an average reinforcement stress of 11.61 ksi compared with 11.47 ksi for the fixed diaphragm, an increase of approximately 1.2%. The fixed diaphragm developed an average compression block contact stress of 3.63 ksi compared with 3.44 ksi for the pinned diaphragm, which was approximately 5.4% greater. Overall, the plate girder bridge followed the same force transfer mechanism observed for the Base Bridge and the skew angle bridge. However, the longer spans and larger plate girder sections increased the magnitude of the tensile and compressive forces transferred through the continuity connection, and the negative moment demands developed at the intermediate supports.

5.3.4 Unbalanced Span Bridge

Table 5-8 summarizes the average reinforcement stresses at Intermediate Bents 2 and 3 and the average compression block stresses for each diaphragm type, bond breaker plate detail, and load case. The reinforcement provided over the intermediate supports are summarized in Table 5-9. Each compression block used in calculation of the average stress had a contact area of approximately 22.7 in². The reinforcement provided at each intermediate bent consisted of 101 reinforcing bars with a total steel area of 55.99 in².

Table 5-8. Average deck reinforcement stresses at Intermediate Bents 2 and 3 and average compression block contact stresses for the Unbalanced Span Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. reinf. stress, $\sigma_{t,avg}$ (ksi)	$\sigma_{t,avg}/f_y$ (%)	Avg. stress, $\sigma_{c,avg}$ (ksi)	$\sigma_{c,avg}/F_{ypl}$ (%)
Fixed diaphragm	No Plate	LC1	6.76	11.3	3.17	5.3	2.27	4.5
		LC2	15.59	26.0	17.00	28.3	4.08	8.2
		LC3	18.53	30.9	14.32	23.9	5.30	10.6
		LC4	4.50	7.5	2.66	4.4	1.45	2.9
		LC5	13.46	22.4	7.96	13.3	3.39	6.8
	Plate	LC1	6.76	11.3	3.48	5.8	2.29	4.6
		LC2	15.85	26.4	17.28	28.8	5.19	10.4
		LC3	18.91	31.5	14.53	24.2	5.22	10.4
		LC4	4.57	7.6	2.69	4.5	1.46	2.9
		LC5	13.61	22.7	8.10	13.5	3.33	6.7
Pinned diaphragm	No Plate	LC1	6.73	11.2	3.31	5.5	2.11	4.2
		LC2	16.27	27.1	17.59	29.3	5.01	10.0
		LC3	18.87	31.4	14.63	24.4	5.22	10.4
		LC4	4.50	7.5	2.75	4.6	1.37	2.7
		LC5	13.59	22.7	8.14	13.6	3.29	6.6
	Plate	LC1	6.63	11.1	3.35	5.6	2.11	4.2
		LC2	16.21	27.0	17.46	29.1	4.67	9.4
		LC3	18.87	31.5	14.52	24.2	5.10	10.2
		LC4	4.44	7.4	2.76	4.6	1.35	2.7
		LC5	13.56	22.6	8.09	13.5	3.20	6.4

Table 5-9. Longitudinal deck reinforcement details for the Unbalanced Span Bridge.

Size	Number of bars	Area per bar (in ²)	Total area (in ²)
#6 Top	36	0.44	15.84
#8 Top	33	0.79	26.07
#6 Bottom	32	0.44	14.08
Total	101		55.99

The Unbalanced Span Bridge developed reinforcement stresses between those of the Base Bridge and the Plate Girder Bridge. The maximum average reinforcement stress occurred during Load Case 3 for the fixed diaphragm with the bond breaker plate at Intermediate Bent 2, where the reinforcement reached 18.91 ksi. This value corresponds to 31.5% of the 60 ksi reinforcement yield strength. The maximum average compression block contact stress was 5.30 ksi and occurred during Load Case 3 for the fixed diaphragm without the bond breaker plate. This stress corresponds to 10.6% of the 50 ksi yield strength of the compression block steel. These results indicate that both the deck reinforcement and the compression blocks remain

well below yielding under the applied factored live and superimposed dead loads, indicating that the continuity connection details are appropriately proportioned.

The influence of the bond breaker plate remained relatively small. For the fixed diaphragm, the plate increased the average reinforcement stress by approximately 1.8% and the average compression block stress by 6.1%. For the pinned diaphragm, the plate reduced the average reinforcement and compression block stresses by approximately 0.5% and 3.4%, respectively. The pinned and fixed diaphragm configurations again had similar force levels. Across all load cases, the pinned diaphragm increased the average reinforcement stress from 10.49 ksi to 10.61 ksi, or approximately 1.2%, while the fixed diaphragm produced an average compression block stress of 3.40 ksi compared with 3.34 ksi for the pinned diaphragm with a difference of approximately 1.6%. Overall, the unbalanced span bridge showed the same fundamental force transfer mechanism observed for the previous bridge configurations. Its average reinforcement stress was 10.55 ksi, which was approximately 2.8% lower than the Base Bridge and 8.6% lower than the Plate Girder Bridge. The average compression block contact stress was 3.37 ksi, which was approximately 3.5% greater than the Base Bridge but 4.6% lower than the Plate Girder Bridge. These results indicate that the unequal span arrangement influenced the distribution of stress between the intermediate supports without changing the overall continuity connection behavior.

5.4 Summary of Deck Reinforcement and Compression Block Stresses

Figure 5-22 and Figure 5-23 summarize the maximum average deck reinforcement stresses and compression block contact stresses obtained from the controlling load case for each bridge configuration, diaphragm type, and bond breaker plate condition. As shown in Figure 5-22,, the maximum average deck reinforcement stresses remained well below the 60 ksi reinforcement yield strength for all models. The Plate Girder Bridge produced the largest reinforcement stresses, with maximum values ranging from 23.4 ksi to 24.1 ksi, corresponding to approximately 39% to 40% of yield. The Base Bridge and Unbalanced Span Bridge produced intermediate reinforcement stresses, with maximum values of approximately 18 ksi to 20 ksi, while the Skew Angle Bridge produced the lowest reinforcement stresses, with maximum values of approximately 15 ksi. These results indicate that the negative moment reinforcement remained elastic for all bridge configurations and detailing conditions considered.

Similarly, the compression block stresses remained well below the 50 ksi yield strength of the compression block steel, as shown in Figure 5-23. The largest maximum average compression block stresses occurred in the Plate Girder Bridge and Unbalanced Span Bridge models, with values of approximately 5.1 ksi to 5.3 ksi, or about 10% to 11% of yield. The Base Bridge produced maximum average compression block stresses between 4.0 ksi and 4.8 ksi, while the Skew Angle Bridge produced the lowest values, ranging from approximately 2.1 ksi to 2.8 ksi. Overall, the bond breaker plate and diaphragm condition produced only modest changes in the maximum stress demands. The results indicate that the deck reinforcement and compression block dimensions provided adequate capacity for all factored post-continuity load demands considered in the parametric study.



Figure 5-22. Summary of maximum average deck reinforcement stresses for all parametric models.

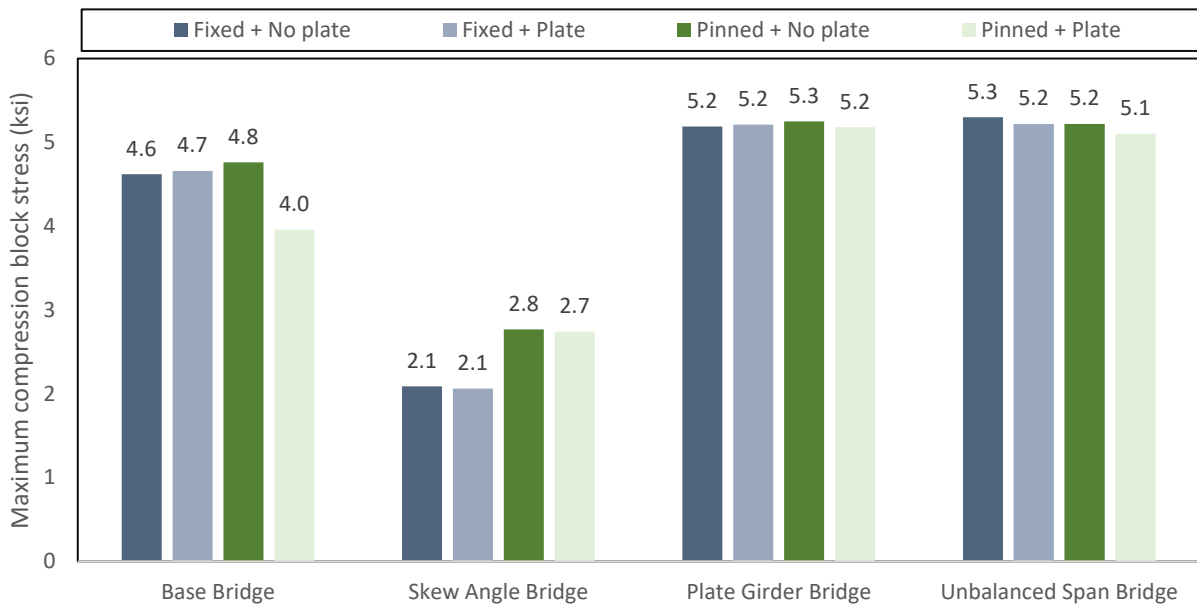


Figure 5-23. Summary of maximum compression block stresses for all parametric models.

5.5 Live-Load DOC of Representative Parametric Models

The live-load DOC of representative parametric bridge models was calculated to evaluate whether the high level of live-load continuity observed in the completed Bridge A9513 is maintained for other bridge geometries and diaphragm configurations. The analysis was also used to assess the influence of the bond breaker plate on the live-load continuity of the completed bridges.

The DOC of the parametric models was obtained with the deflection-based metric using the midspan deflection of Span 2-3 and girder line 2 under Load Case 2. This response quantity was selected because the design truck and lane load are applied entirely within Span 2-3 for this load case (Figure 5-8), which produced the largest positive moments and midspan deflections in that span. Because the DOC is calculated by comparing the free, calibrated and fixed bridge responses, this loading configuration provides the greatest sensitivity to continuity effects.

The calculated DOC values are summarized in Table 5-10 and Figure 5-24, and the corresponding deflected shapes for the fixed diaphragm case without a bond breaker plate are shown in Figure 5-25. In general, the bond breaker plate had a negligible effect on the live-load DOC. This result is expected because, after bridge completion, live-load continuity is developed primarily through the force couple between the deck reinforcement and the compression block at the bottom of the girder ends. Therefore, the bond breaker plate, which mainly controls construction-stage diaphragm restraint and cracking, has limited influence on the completed live-load response.

The diaphragm configuration had a more noticeable influence on the calculated DOC. For the Base Bridge, the DOC was approximately 95% for the fixed diaphragm configuration and 86% for the pinned diaphragm configuration. For the Plate Girder Bridge, the DOC was approximately 92% for the fixed diaphragm configuration and 84% for the pinned diaphragm configuration. For the Unbalanced Span Bridge, the DOC was approximately 93% for the fixed diaphragm configuration and 87% for the pinned diaphragm configuration. The Skew Angle Bridge showed a different trend, with DOC values remaining close to 80% for all diaphragm and bond breaker plate configurations, indicating that skew had a stronger influence on the live-load continuity response than the diaphragm restraint condition for this bridge configuration.

Overall, the results indicate that a pinned diaphragm configuration generally reduced the live-load DOC by approximately 6% to 9% relative to the fixed diaphragm configuration for the Base, Plate Girder, and Unbalanced Span bridges. However, even with pinned diaphragms, the completed bridges still developed a high level of live-load continuity. This contrasts with the construction-stage dead-load results in Section 4.8.1, where the pinned diaphragm and bond breaker plate combination had a more pronounced effect, reducing the average DOC to approximately 25%. Thus, pinned diaphragms and bond breaker plates appear most effective for limiting unintended continuity during deck placement, while still allowing substantial live-load continuity in the completed SDCL system.

Table 5-10. Deflection-based DOC values for the parametric bridge models.

	Fixed diaphragm		Pinned diaphragm	
	No plate	Plate	No plate	Plate
Base Bridge	95.1	95.0	85.7	85.5
Skew Angle Bridge	82.1	80.0	79.6	79.4
Plate Girder Bridge	92.4	91.1	84.9	83.4
Unbalanced Span Bridge	93.1	92.3	87.5	86.9

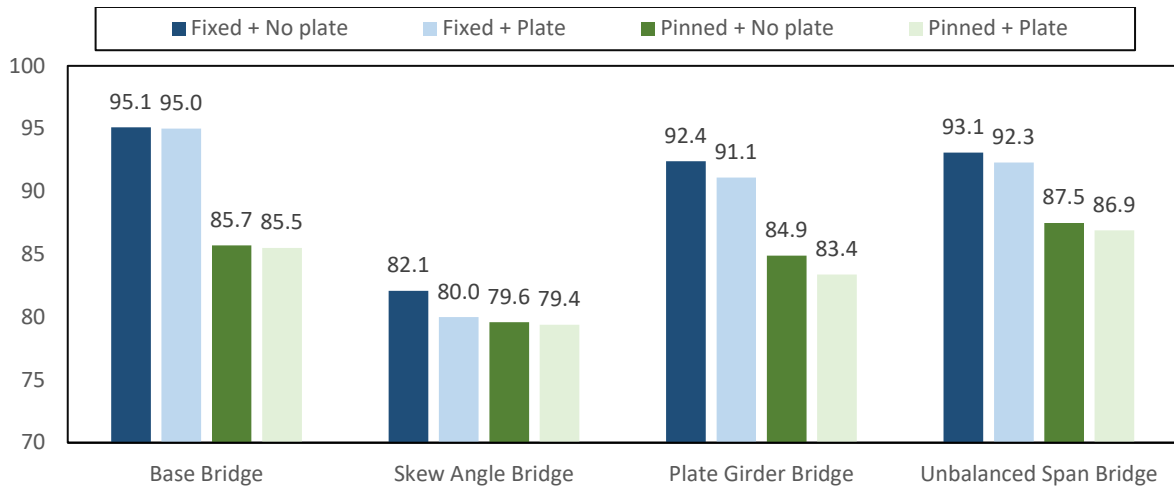


Figure 5-24. Effect of diaphragm restraint and bond breaker plate on deflection-based DOC values for the parametric bridge models.

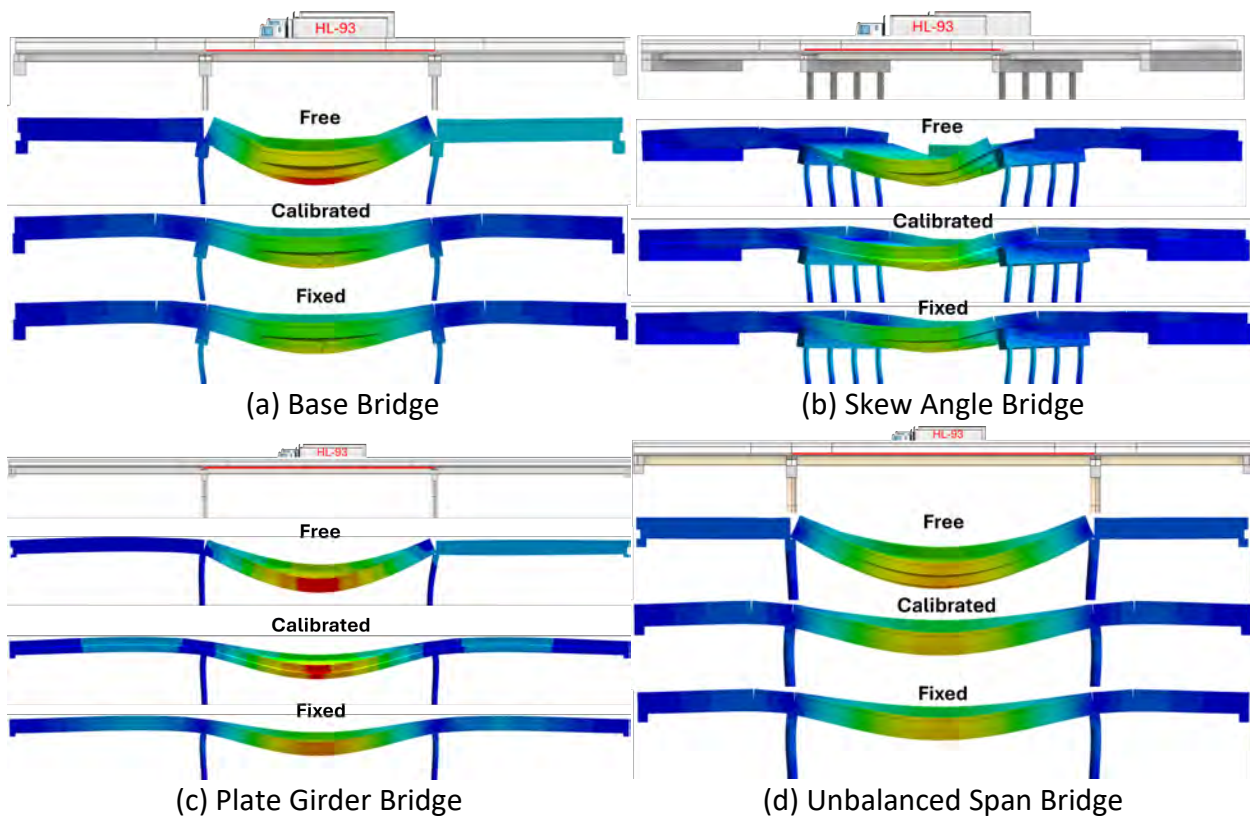


Figure 5-25. Comparison of calibrated model responses with free and fixed models for the parametric bridge models under Load Case 2.

5.6 Summary

This chapter extended the calibrated FE modeling framework developed for Bridge A9513 to evaluate the behavior of SDCL steel bridges with different bridge geometries, diaphragm configurations, and bond breaker plate details. Four representative bridge configurations were

investigated (i.e., the Base Bridge, Skew Angle Bridge, Plate Girder Bridge, and Unbalanced Span Bridge). Each configuration was analyzed under superimposed dead loads and AASHTO HL-93 live-load cases, with pinned and fixed diaphragm configurations and with and without a bond breaker plate. This resulted in a total of 96 parametric FE models.

The parametric analyses showed that the average stresses in the tensile reinforcement in the deck and that in the compression block remained well within the available reinforcement and compression block capacities for all bridge configurations. The maximum average deck reinforcement stresses were approximately 20 ksi for the Base Bridge, 15 ksi for the Skew Angle Bridge, 24 ksi for the Plate Girder Bridge, and 19 ksi for the Unbalanced Span Bridge. These stresses correspond to less than 50% of the 60 ksi reinforcement yield strength. Similarly, the maximum average compression block stresses remained below 6 ksi for all bridge configurations, corresponding to less than 12% of the 50 ksi compression block yield strength. These results indicate that the deck reinforcement and compression block details used in the models were adequate for the factored dead-load and live-load demands considered in this study.

The bond breaker plate had only a minor influence on the live-load force-transfer mechanism. This behavior is expected because, after bridge completion, live-load continuity is developed primarily through the force couple between the deck reinforcement in tension and the steel compression blocks in compression. Therefore, the bond breaker plate mainly affects construction-stage continuity and diaphragm cracking rather than the completed live-load response. The diaphragm configuration had a more noticeable effect, with pinned diaphragm models generally developing slightly larger deck reinforcement stresses, while fixed diaphragm models tended to transfer a larger portion of the demand through the compression blocks.

The deflection-based DOC results showed that the completed SDCL bridge models developed a high level of live-load continuity across the range of configurations considered. For the Base Bridge, Plate Girder Bridge, and Unbalanced Span Bridges, the fixed diaphragm configuration produced DOC values of approximately 92% and 95%, while the pinned diaphragm configuration produced DOC values of approximately 84% to 87%. The Skew Angle Bridge showed lower DOC values of approximately 80% and was less sensitive to diaphragm configuration. These results indicate that skew can have a stronger influence on the effective live-load continuity than the diaphragm restraint condition. However, these conclusions should be interpreted with caution, as they are based on a single skewed bridge configuration.

Overall, the parametric study indicates that the proposed SDCL continuity details can provide substantial live-load continuity for a range of bridge geometries when the deck reinforcement and compression blocks are proportioned as proposed in this study. The results also suggest that pinned diaphragms and bond breaker plates are most beneficial for reducing unintended continuity under dead loading, while still allowing the completed SDCL system to develop high live-load continuity. The findings support the use of the proposed SDCL steel bridge details, while highlighting the need to further investigate the effects of skew, span arrangement, diaphragm restraint, and construction-stage behavior in future MoDOT SDCL research.

Chapter 6. Design Recommendations

Horner & Shifrin led the development of design guidance and standard details for SDCL steel superstructures for incorporation into MoDOT's EPG. Although the original project scope referenced revisions to EPG 751.14, the research team proposes creating a separate EPG Section 751.15 for SDCL for steel superstructures. This organization is intended to treat SDCL steel bridges as a distinct bridge type, similar to the stand-alone EPG sections used for prestressed bridge systems.

This chapter provides a summary of the proposed EPG Section 751.15 for SDCL steel superstructures, and the entire new section is provided in Appendix C. The following sections include proposed policy language, design and detailing provisions, drafting aids, software guidance, and related changes to EPG 751.10, 751.11, and 751.14. Several provisions explicitly refer back to existing EPG 751.14 requirements where conventional steel-bridge guidance remains applicable.

6.1 SDCL Bridge Concept and Intended Benefits

The proposed guidance defines SDCL steel design as an alternative to conventional continuous steel girder bridges and prestressed concrete bridges. The steel girders are erected as simple spans and are then made continuous over the intermediate bents after the deck and concrete diaphragm region are completed. For the proposed MoDOT detail, the girder ends are embedded in a closed concrete diaphragm. Negative-moment continuity is provided through the longitudinal slab reinforcement in tension and steel compression blocks welded to the girder bottom flanges in compression.

The guidance identifies several expected benefits. The most important are the elimination of bolted field splices, simplified girder detailing and fabrication, reduced negative moment demand at intermediate bents during construction, increased girder-end protection from concrete encasement, potential reductions in construction time and crane/shipping requirements, reduced inspection and maintenance associated with field splices and exposed girder ends, and improved competitiveness of steel bridges for short- to moderate-span applications.

6.2 Applicability Limits and General Requirements

The proposed guidance limits routine SDCL use to bridge configurations for which the connection behavior and construction sequence are considered suitable for standard implementation. Use outside these limits may be permitted with approval of the Structural Project Manager (SPM) or Structural Liaison Engineer (SLE). Table 6-1 summarizes the general selection criteria defining when steel SDCL designs may be used.

Table 6-1. Selection criteria for SDCL steel bridges

Topic	Proposed guidance
Seismicity	Routine use for Seismic Design Category A or B. For moderate-to-high seismic applications, the guidance directs designers to consult MoDOT bridge leadership and the ABC-UTC SDCL seismic connection guidance.
Skew	Permitted for skews up to 45 degrees.
Curvature	Curved bridges and curved girders are not covered; all girders are required to be tangent from end to end.
Intermediate supports	Intermediate bents are assumed to be fixed. Plain or laminated elastomeric bearing pads are preferred. Expansion bearings are not permitted because the compression blocks must remain in contact.
Span and structure length	Maximum span length is 120 ft unless approved by the SPM or SLE. Longer spans may be justified by site constraints, elimination of splices over roadways or railroads, or accelerated bridge construction needs. The draft also shows maximum overall structure lengths of 500 ft for integral end bents and 700 ft for non-integral end bents.
Materials	Reinforcing steel is Grade 60. ASTM A615 Grade 60 is specified for SDC A and rehab/redeck applications, while ASTM A706 Grade 60 is specified for SDC B, C, and D.
Girder selection	Galvanizing is recommended for girder sections under 60 ft to increase longevity and reduce maintenance. Weathering steel selection should consider mill order requirements and rolling schedules. If girder size varies at an intermediate bent, the overall girder depth must remain constant, and compression blocks are designed for the smaller girder section.

6.3 Analysis and Design Philosophy

The proposed analysis requirement is the defining feature of the guidance. The selected design software must be able to model the girder weight and deck dead load as simple-span actions before deck hardening and before continuity is established at the intermediate bents. After continuity is established, the model must evaluate DC2, DW, and live loads with the deck reinforcement providing continuity at the intermediate supports. The guidance identifies AASHTOWare BrD/BrR, MDX Software, LEAP Bridge Steel, and Merlin-DASH as software capable of modeling SDCL structures. Other software may be capable of SDCL analysis but have not been evaluated.

The design of the intermediate support connection is based on the force couple developed between tensile reinforcement in the slab and compression blocks attached to the girder bottom flanges. The compression blocks transfer compressive forces from one girder end to the adjacent girder end without relying on the concrete diaphragm alone. The nominal flexural resistance of the connection is based on the area of tensile reinforcement, the reinforcement yield strength, and the moment arm between the reinforcement centroid and the compression-block compression resultant. The proposed design steps are as follows:

Step 1 - Assume compression block height:

The compression block height is assumed to determine an area of tensile reinforcement required to balance the force couple.

$$A_{s,required} = \frac{M_u}{\phi f_y \left(d - \frac{H}{2} \right)}$$

where:

$A_{s, required}$ is the minimum area of tensile reinforcement required, in^2

M_u is the factored moment at the connection location, $kip - in$.

f_y is the yield strength of the reinforcing steel, in^2 .

d is the distance between bottom of the girder and centroid of tensile reinforcement, in .

H is the assumed compression block height starting at 2" and increases at ¼" increments measured parallel to the girder depth, in .

ϕ is the reinforcement resistance factor for tension-controlled concrete sections [AASHTO 5.5.4.2].

Step 2 – Calculate the minimum block height

The compression blocks are designed to carry a force equal to 1.7 times that carried by the tensile reinforcement to ensure its behavior remains elastic.

$$H_{min} = \frac{1.7A_s f_y}{b_f F_{ypl}}$$

where:

H_{min} is the minimum height of the compression block parallel to the girder depth, in .

b_f is the girder bottom flange width, in .

F_{ypl} is the yield strength of the steel compression block, ksi .

Step 3 – Calculate the capacity of the connection

Once the minimum height of the compression block and required area of tensile reinforcement is determined, the nominal flexural capacity,

$$M_n = A_s f_y \left(d - \frac{H}{2} \right)$$

of the connection can be calculated. If the provided area of tensile reinforcement is greater than the required area, the height of the compression block should be determined based on the actual arrangement of tensile reinforcement.

6.4 Compression Block and Girder-End Details

Compression blocks are central to the proposed MoDOT SDCL detail. The suggested compression block detail is shown in Figure 6-1. The suggested compression-block width shall match the girder bottom flange width. The block thickness, measured parallel to the girder length, should be a minimum of 2 in. in 1/4-in. increments. The block height, measured parallel to the girder depth, must be determined by design, as discussed in the previous section. For fabrication simplicity, using a block height equal to the block thickness is preferred where design criteria are satisfied. Adjacent compression blocks do not need full-height contact, but the vertical faces are required to be in partial contact and to overlap at least 0.75 times the block height.

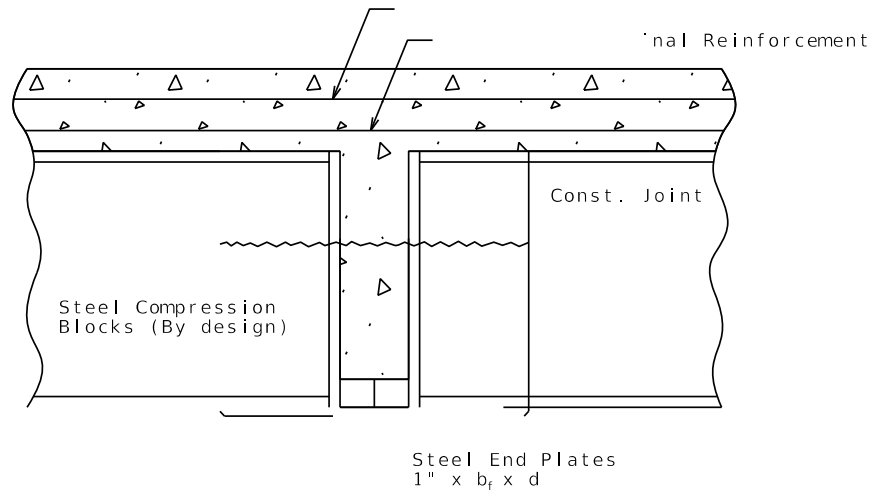


Figure 6-1. Elevation view of the compression block detail.

The proposed EPG guidelines also include end plate and compression block end details for new designs or superstructure replacements, as shown in Figure 6-2.

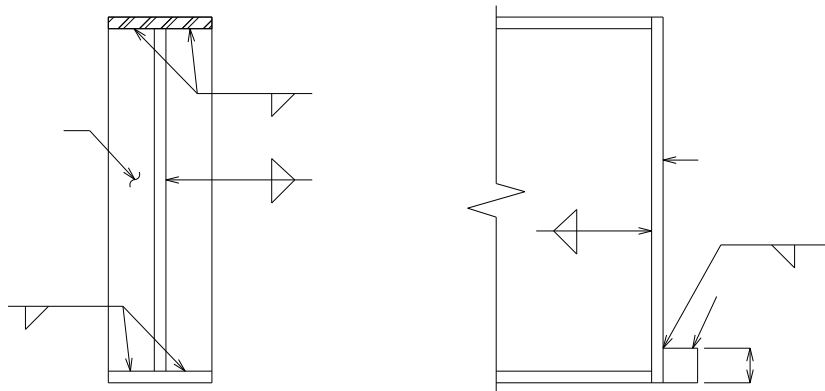


Figure 6-2. End plate and compression block welding details.

For redecking projects that convert existing multi-simple-span bridges to SDCL behavior, the guidance provides tapered compression blocks and shim-plate details, as shown in Figure 6-3 and Figure 6-4. The contractor is required to measure each girder-end gap before fabrication and adjust the tapered block or shim configuration as needed. Field adjustment provisions are provided for both narrower and wider gaps, while maintaining minimum extension requirements beyond the flanges.

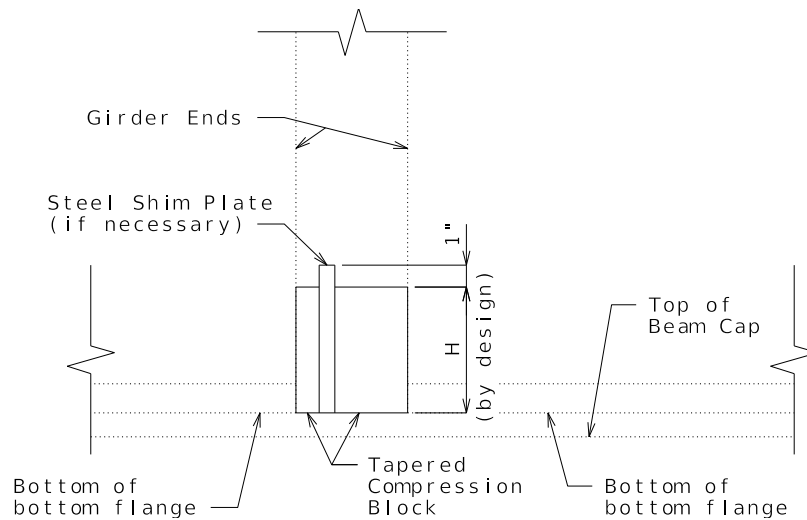


Figure 6-3. Elevation view of tapered compression block and shim plate part.

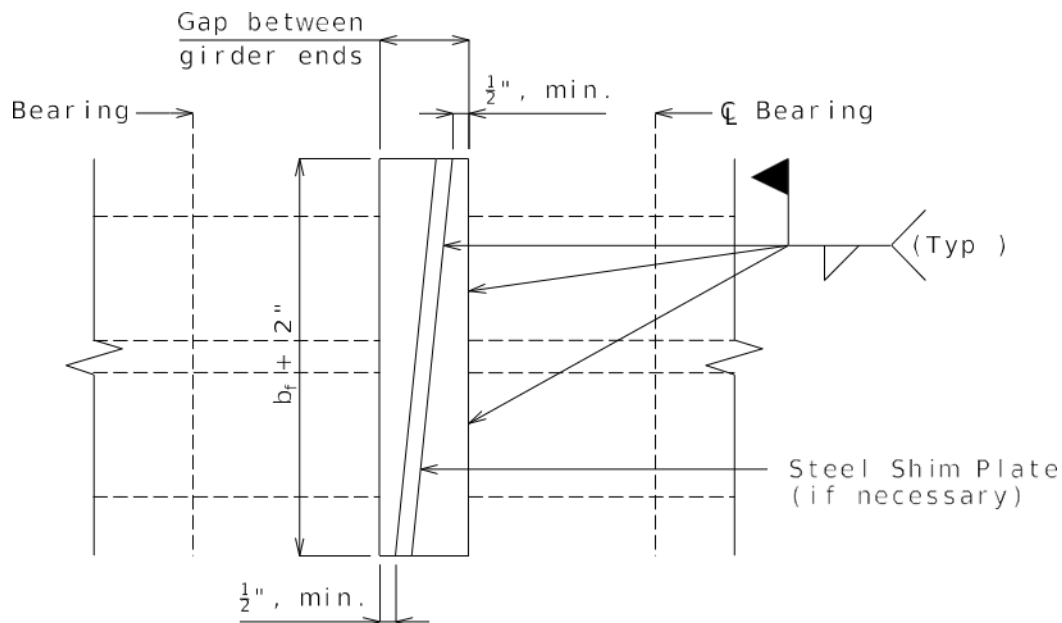


Figure 6-4. Plan view of tapered compression block and shim plate part plan

6.5 Concrete Intermediate Diaphragms and Reinforcement

The proposed SDCL system uses closed concrete intermediate diaphragms cast between girder ends at interior supports. These diaphragms develop continuity for loads applied after the continuity connection is completed. The diaphragm at each bent is cast initially to approximately one-half to two-thirds of its full depth to improve girder stability while reducing unintended dead-load continuity and cracking during deck placement. The diaphragm pour must occur at least 12 hours before the slab and remainder of the diaphragm are poured with the deck placement.

The guidance includes standard details for the stainless-steel bond breaker plate and when to use it. The stainless-steel bond breaker plate is recommended for grade separations where salt exposure would accelerate deterioration or more publicly visible locations to control cracking. The sheet metal plate may be omitted in areas such as stream crossings. There is a higher potential for uncontrolled cracking without the sheet metal plate, however the test Bridge A9513 was constructed without the plate and no visible cracking was observed at the time of live load testing. The sheet metal plate did not significantly reduce the degree of continuity as narrated in Section 4.8, Table 4-7 and Table 4-8.

Reinforcement details for closed concrete intermediate diaphragms are presented in Appendix C. Reinforcement details shown for skewed bents are also applicable to square bents. All U-bar reinforcements use stirrup bends, and epoxy coated. Dowel-bar coating is required to match the coating of the intermediate bent reinforcement. The guidance distinguishes details for diaphragms constructed independent of the bent cap (pinned) from cases where bent-cap reinforcement extends into the diaphragm (fixed).

6.6 Bracing, Bearings, Composite Design, and Miscellaneous Details

The proposed guidance retains much of the existing EPG 751.14 framework for steel superstructures but adds SDCL-specific provisions where the construction sequence or connection detail differs. For W24x and shallower rolled shapes, smaller intermediate diaphragm or cross-frame spacing should be considered to increase stability before deck curing. A required plan note directs the contractor to provide temporary bracing needed for lateral and torsional stability during deck construction and to remove the bracing after the slab reaches 75 percent of its design strength.

At non-integral end bents where slab continuity is broken, steel end diaphragms are required. Integral end bents use concrete diaphragms. Bearing stiffeners are not required at intermediate bents because 1-in.-thick steel end plates are welded to the girder ends and the girder ends are encased in concrete. Shear connectors are required throughout the entire span, designed for fatigue and strength loading for positive and negative moment regions. Shear-connector details generally follow EPG 751.14. Variable haunching may be used in lieu of cambering where appropriate. Wide-flange beams are to be cambered over roadways when non-composite dead-load deflection exceeds 1 in.; variable haunching may be used over creek crossings when dead-load deflection is less than 4 in.; and all plate girders are to be cambered.

6.7 Relationship to Existing EPG Sections and Implementation Status

The proposed Section 751.15 is intended to incorporate applicable portions of EPG 751.14 while removing or modifying provisions that do not apply to SDCL steel bridges, such as field bolted splice guidance for conventional continuous girders, horizontally curved girder provisions, and several conventional diaphragm and stiffener details. The guidance also identifies related changes to EPG 751.10 for deck reinforcement, EPG 751.11 for bearings, and EPG 751.14 for steel superstructure provisions.

6.8 Summary

This chapter summarizes the proposed Section 751.15 for inclusion in MoDOT's EPG. The proposed guidance establishes SDCL steel superstructures as an alternative to conventional continuous steel girder bridges and prestressed concrete bridges for appropriate site conditions. The approach uses steel girders erected as simple spans for girder and deck dead-load effects, followed by development of continuity at the intermediate bents for superimposed dead load and live loads through the completed deck, concrete diaphragm, slab reinforcement, and bottom compression-block connection.

The proposed guidance defines the main limits of applicability for routine SDCL use, including restrictions on seismic design category, skew, girder curvature, intermediate bent fixity, and maximum span length. It also identifies analysis requirements for staged behavior, including the need for design software capable of modeling simple-span behavior for initial dead-load stages and continuous behavior after the deck and pier-region connection are completed. The guidance further provides design procedures for the compression-block connection, where the negative-moment resistance is developed primarily through a force couple between tensile reinforcement in the deck and compression transferred through steel blocks welded to the girder bottom flanges.

The standard details included in the proposed guidance focus on the intermediate bent region, which is the critical component of the SDCL system. These details address compression block dimensions and welding, closed concrete diaphragm geometry, diaphragm reinforcement, recommendation on when to specify the use of the bond breaker or sheet-metal details, dowel bars and keys, reinforcing holes, bearing requirements, intermediate diaphragms and cross frames, shear connectors, girder haunches, and camber or dead-load deflection requirements. The guidance also distinguishes between new construction or superstructure replacement details and redecking applications where existing multi-simple-span steel bridges may be converted to SDCL behavior.

Chapter 7. Summary and Conclusions

The overall objective of this research was to support the development of design guidance and standard details for Simple for Dead Load, Continuous for Live Load (SDCL) steel superstructures for use in MoDOT practice. The research included a literature review, field testing of Bridge A9513 during deck placement and under controlled live loading, development and calibration of finite element models, evaluation of construction-stage and live-load degree of continuity (DOC), parametric analyses of representative SDCL bridge configurations, and development of proposed guidance for inclusion in MoDOT's Engineering Policy Guide (EPG).

The main conclusions, recommendations, and future research needs from this investigation are summarized below.

7.1 Conclusions

7.1.1 Literature Review

The literature review showed that SDCL steel bridges have been studied and implemented by several transportation agencies, but formal implementation-ready design provisions remain limited. Oregon was the only state identified with a dedicated published SDCL detail for concrete intermediate diaphragm connections. Several other states mention SDCL or related simple-made-continuous concepts, provide related continuity or retrofit guidance, or address similar details through prefabricated bridge unit applications. However, most state DOTs do not currently provide detailed SDCL design provisions, and some states restrict similar multi-span simple steel configurations with continuous decks.

Previous experimental and numerical studies showed that SDCL behavior is governed primarily by the local force-transfer mechanism at the intermediate support. The main contributors to negative flexural resistance are the slab reinforcement in tension, the bottom connection plate or steel block in compression, and the compressed concrete core in the diaphragm. The literature also showed that the assumption of simple-span behavior during deck placement is reasonable but not exact because partially cast diaphragms and reinforcement can provide some construction-stage continuity before the deck hardens.

7.1.2 Experimental Testing

Bridge A9513 was instrumented and monitored during construction-stage dead loading and after completion under controlled truck loading. During deck placement, the measured response indicated that the bridge did not behave as a purely simple-span system. Upward movement in the adjacent span during placement of the neighboring span and compressive strains at bottom flanges of the girders near the intermediate bent showed that some restraint developed during construction. The maximum measured midspan deflection during deck placement was approximately 0.86 in., which was consistent with the expected magnitude of the wet concrete loading response.

The live-load test demonstrated that the completed bridge developed substantial continuity at the intermediate bents. When the loaded trucks were positioned in Span 2-3, the bridge showed downward deflection in the loaded span and upward response in the adjacent span, which is consistent with continuous behavior. The measured live-load deflections and strains were relatively small, with a maximum downward deflection of approximately 0.17 in. and maximum strain readings of approximately 116 $\mu\epsilon$. These results indicate that the completed SDCL system behaved efficiently under the controlled truck loading.

7.1.3 Finite Element Modeling

Three-dimensional finite element models were developed for both the dead-load and live-load stages of Bridge A9513. Separate models were required because the bridge configuration, composite action, contact behavior, and support conditions differed between deck placement and the completed bridge condition. The dead-load model represented the construction stage, when the steel girders carried the wet concrete deck primarily as noncomposite members. The live-load model represented the completed bridge after the deck and diaphragm had hardened and continuity had developed at the intermediate bents.

The calibrated models provided a reasonable representation of the measured bridge response. The dead-load model captured the measured bottom-flange strains and deflections during deck placement, while the live-load model reproduced the measured curvatures and deflections from the controlled truck tests. The live-load model was able to reproduce the response from multiple truck positions without recalibration, indicating that the model captured the global stiffness, load distribution, and continuity behavior of the completed bridge.

The DOC calculations showed two important trends. First, Bridge A9513 developed measurable construction-stage continuity during deck placement. The dead-load DOC on Span 2-3 was approximately 42%, which was attributed primarily to the relatively deep partially cast diaphragm at the intermediate bent as well as the lack of a bond breaker plate. Additional FE models of Bridge A9513 showed that the dead-load DOC can be reduced by about 10% if a pinned diaphragm is used together with a bond breaker plate. Second, the completed bridge developed a high level of live-load continuity. The average live-load DOC was approximately 92%, and the curvature-based and deflection-based DOC estimates were in close agreement. These results confirm that the SDCL system can provide substantial continuity after completion, while also showing that unintended construction-stage continuity can be significant when the partially cast diaphragm provides considerable restraint.

7.1.4 Parametric Analysis

The calibrated modeling framework was extended to evaluate representative SDCL bridge configurations beyond Bridge A9513. The parametric study included a Base Bridge, Skew Angle Bridge, Plate Girder Bridge, and Unbalanced Span Bridge. Each configuration was evaluated under superimposed dead loads and AASHTO HL-93 live-load cases with fixed and pinned diaphragm configurations, with and without a bond breaker plate, resulting in 96 FE models.

The parametric analyses showed that the proposed continuity connection details remained well within the available reinforcement and compression block capacities for the bridge

configurations and factored superimposed dead load and live-load cases considered. The maximum average deck reinforcement stresses were approximately 20 ksi for the Base Bridge, 15 ksi for the Skew Angle Bridge, 24 ksi for the Plate Girder Bridge, and 19 ksi for the Unbalanced Span Bridge. These values were less than 50% of the 60 ksi reinforcement yield strength. The maximum average compression block stresses remained below 6 ksi for all configurations, corresponding to less than 12% of the 50 ksi compression block yield strength.

The bond breaker plate had only a minor effect on the completed live-load response. This result is expected because live-load continuity in the completed bridge is developed primarily through the force couple between the deck reinforcement in tension and the compression blocks in compression. The bond breaker plate is therefore more important for controlling construction-stage restraint and diaphragm cracking than for changing the completed live-load force-transfer mechanism.

The diaphragm configuration had a more noticeable influence on live-load DOC. For the Base, Plate Girder, and Unbalanced Span bridges, fixed diaphragm configurations produced DOC values of approximately 92% to 95%, while pinned diaphragm configurations produced DOC values of approximately 85% to 87%. The Skew Angle Bridge produced lower DOC values of approximately 80% and was less sensitive to diaphragm configuration, indicating that skew can have a stronger influence on effective live-load continuity than diaphragm restraint for the configuration considered.

Overall, the parametric study indicates that the proposed SDCL continuity details can provide substantial live-load continuity for a range of bridge geometries when the deck reinforcement and compression blocks are properly proportioned. The results also suggest that pinned diaphragms and bond breaker plates are most beneficial for reducing unintended construction-stage continuity while still allowing the completed SDCL bridge to develop high live-load continuity.

7.1.5 EPG Design Guidance

The research team developed proposed SDCL steel bridge guidance and standard details for inclusion in MoDOT's EPG. The proposed guidance defines SDCL steel superstructures as an alternative to conventional continuous steel girder bridges and prestressed concrete bridges for appropriate site conditions. The proposed system uses steel girders erected as simple spans for girder and deck dead-load effects, followed by development of continuity at the intermediate bents for superimposed dead load and live load through the completed deck, concrete diaphragm, slab reinforcement, and bottom compression-block connection.

The proposed guidance includes applicability limits, staged-analysis requirements, compression block design procedures, girder-end details, concrete diaphragm details, bond breaker plate details, reinforcement requirements, bracing and bearing provisions, shear connector guidance, and camber or haunch recommendations. The guidance also distinguishes between new construction or superstructure replacement projects and redecking projects where existing simple-span steel bridges may be converted to SDCL behavior.

7.2 Recommendations

Based on the findings of this research, the following recommendations are made:

1. **Develop SDCL steel bridge guidance as a distinct EPG topic.** SDCL steel bridges have a construction sequence, analysis philosophy, and continuity mechanism that differ from conventional continuous steel girder bridges. Therefore, the proposed EPG guidance should clearly distinguish SDCL steel superstructures from conventional simple-span and continuous steel systems, as well as from SDCL precast concrete bridges.
2. **Require staged analysis for SDCL steel bridges.** The analysis should represent girder weight and deck dead load as simple-span actions before continuity is established. Loads applied after deck and diaphragm hardening, including superimposed dead load and live load, should be analyzed using the completed continuous system.
3. **For design simplicity and conservative results, ignore construction-stage continuity for all diaphragm details.** The field testing and finite element results showed that construction-stage continuity can develop when the partially cast diaphragm provides substantial restraint. When using conventional software, is conservative to ignore these effects since the stresses quickly reverse at the time of casting the deck and minimally offset the stresses at controlling points. It would also have minimal impact on deflections during casting of the deck. If the designer takes a refined finite element approach, the expected construction-stage DOC can be incorporated to reduce the stresses and deflections at key points related to the diaphragm detail, first-pour height, bond breaker plate configuration, and degree of diaphragm restraint. For first-pour diaphragm height of about 70% of the total diaphragm height, the results suggest assuming a DOC of about 30% for fixed diaphragms with or without a bond breaker plate, and about 25% for pinned diaphragms with a bond breaker plate.
4. **Use pinned diaphragms and bond breaker plates when the intent is to reduce dead-load continuity.** The results showed that this configuration was most effective for reducing construction-stage DOC. However, the completed bridge can still develop substantial live-load continuity through the deck reinforcement and compression block force couple.
5. **Design the intermediate support region as a local force-transfer mechanism.** SDCL design should not be based only on global negative moment demand. The deck reinforcement, compression block, girder-end plate, diaphragm, construction joint, and support restraint all contribute to the force path and should be addressed in design and detailing.
6. **Use compression blocks as a defined bottom compression-transfer mechanism.** The parametric analyses showed that the compression block stresses remained well below yield for the details evaluated. Providing a direct steel-to-steel compression path helps reduce reliance on concrete bearing alone and provides a clear mechanism for live-load continuity.

7. **Consider skew and span arrangement in SDCL design guidance.** The skewed bridge configuration produced lower live-load DOC values than the other configurations and was less sensitive to diaphragm restraint. Unbalanced spans also redistributed demands between intermediate supports. These effects should be considered when establishing limits of applicability or determining when refined analysis is needed.
8. **Use the proposed standard details as a basis for implementation but refine them as additional bridge data becomes available.** The details evaluated in this study performed well under the modeled HL-93 live-load cases, but additional field data would improve confidence in construction-stage assumptions, diaphragm behavior, and long-term performance.

7.3 Future Research

The following future research is recommended to further support implementation of SDCL steel superstructures in Missouri:

1. **Monitor additional SDCL bridges with different diaphragm details.** Future field studies should include bridges with pinned diaphragms, bond breaker plates, and different first-pour diaphragm heights to directly evaluate their influence on construction-stage DOC.
2. **Evaluate the effect of first-pour diaphragm height.** The current test bridge had a relatively deep partially cast diaphragm, which contributed to significant construction-stage continuity. Additional analytical and experimental work should evaluate how diaphragm first-pour height affects restraint, cracking, girder deflections, and camber predictions.
3. **Expand the parametric study to additional skew angles and span arrangements.** The skewed configuration showed lower live-load DOC values, but only one skew angle was evaluated. Additional studies should consider a range of skew angles, span ratios, girder spacings, and support configurations.
4. **Investigate long-term behavior of SDCL details.** Additional monitoring could help evaluate the effects of creep, shrinkage, temperature, repeated live loading, and seasonal variation on diaphragm behavior, deck reinforcement stresses, and continuity development.
5. **Develop load-rating guidance for SDCL steel bridges.** Future work should address how SDCL behavior should be represented in bridge load rating, especially for existing bridges converted to SDCL behavior through redecking or diaphragm modifications.
6. **Evaluate constructability and inspection requirements.** Additional input from designers, fabricators, contractors, inspectors, and MoDOT bridge personnel would help refine standard details so that the proposed SDCL system remains practical for routine design, construction, inspection, and maintenance.
7. **Evaluate girder stability by placing a steel diaphragm just outside of concrete diaphragm and cast concrete diaphragm with concrete deck.** Similar to prestress

girder diaphragm placement to pour 30 minutes before to a maximum of 2 hours before the slab is cast. This would likely eliminate any continuity during deck placement. It would appear that the skew limit would be 20 degrees due to detailing of the diaphragms on a skewed placement.

8. **Evaluate diaphragm cracking if steel diaphragms are placed on end plates for stability and cast diaphragm with deck placement.** Similar to bullet point 7 above, this can apply for accelerated bridge construction, and it is possible to increase skew over 20 degrees but would seem to tend to promote more cracking in the cured diaphragm but would still allow the girders to deflect without a degree of continuity.

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Appendix A: Detailed Construction Drawings of Bridge A9513

Figure A-1 through Figure A-9 show the structural details of the main components of Bridge A9513 used for the finite element modeling.

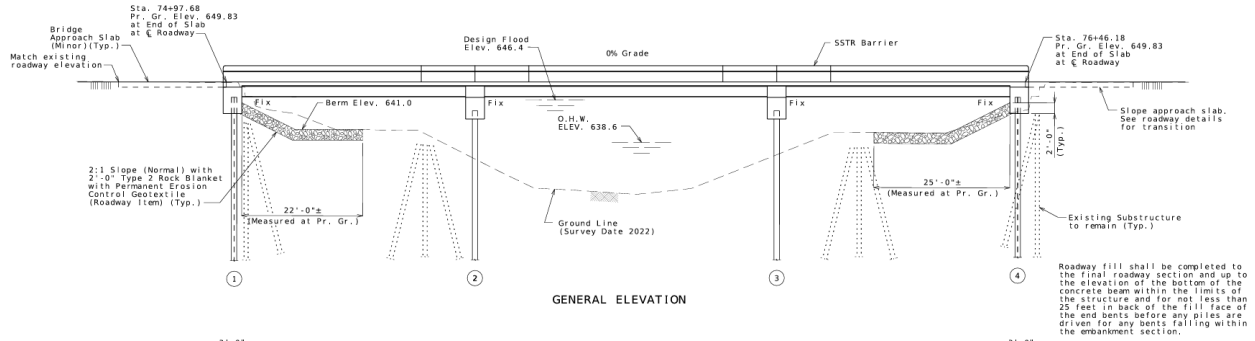


Figure A-1. General elevation view of Bride A9513.

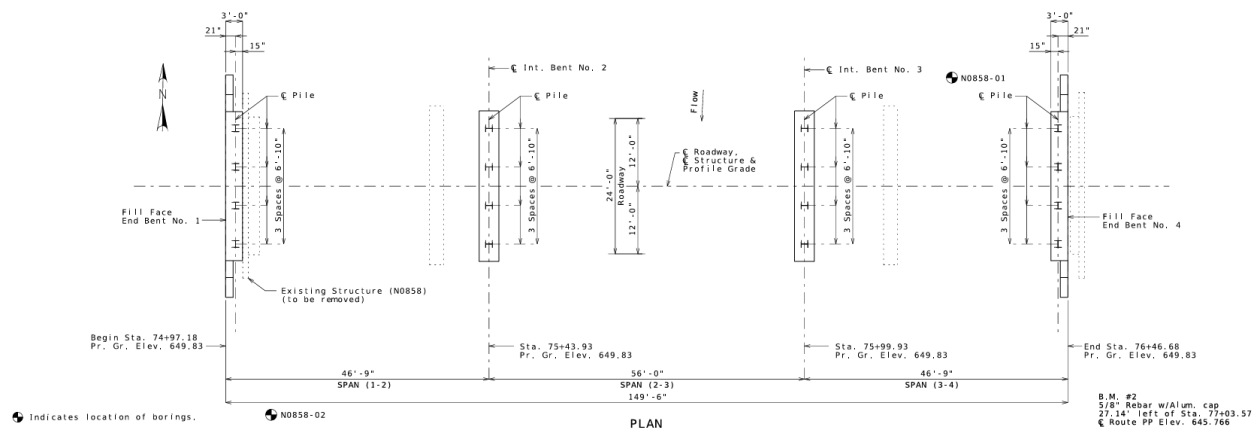


Figure A-2. General plan view of Bride A9513.

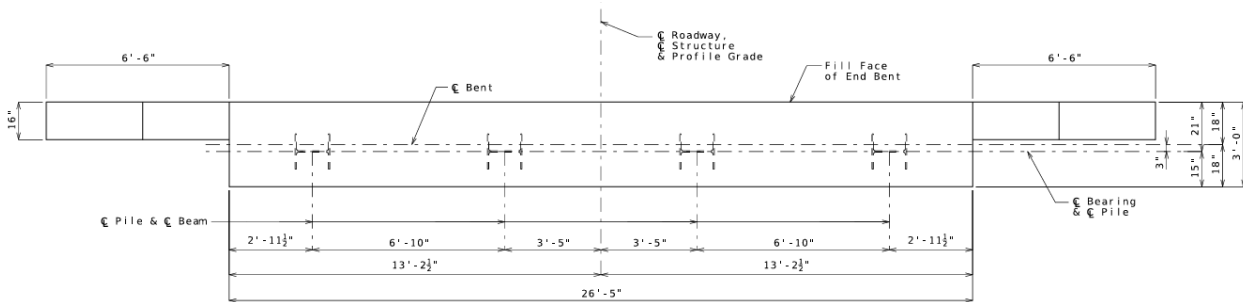


Figure A-3. Plan view of end bent showing dimensions.

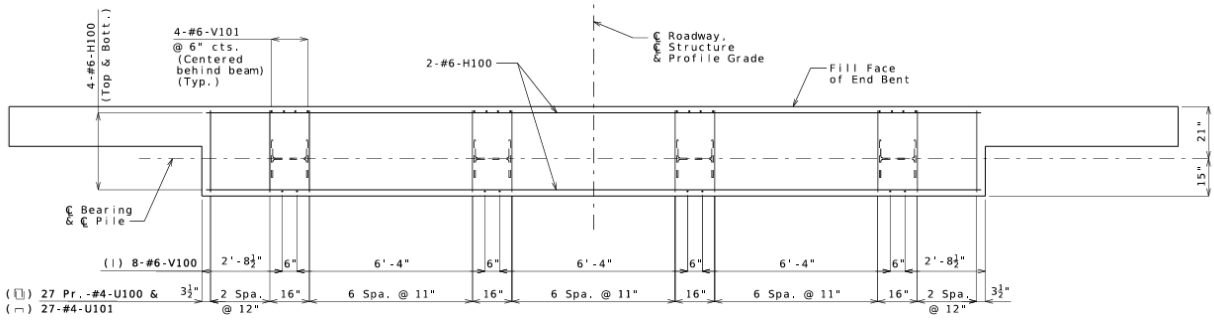


Figure A-4. Plan view of end bent showing reinforcement below steel I-beam.

al

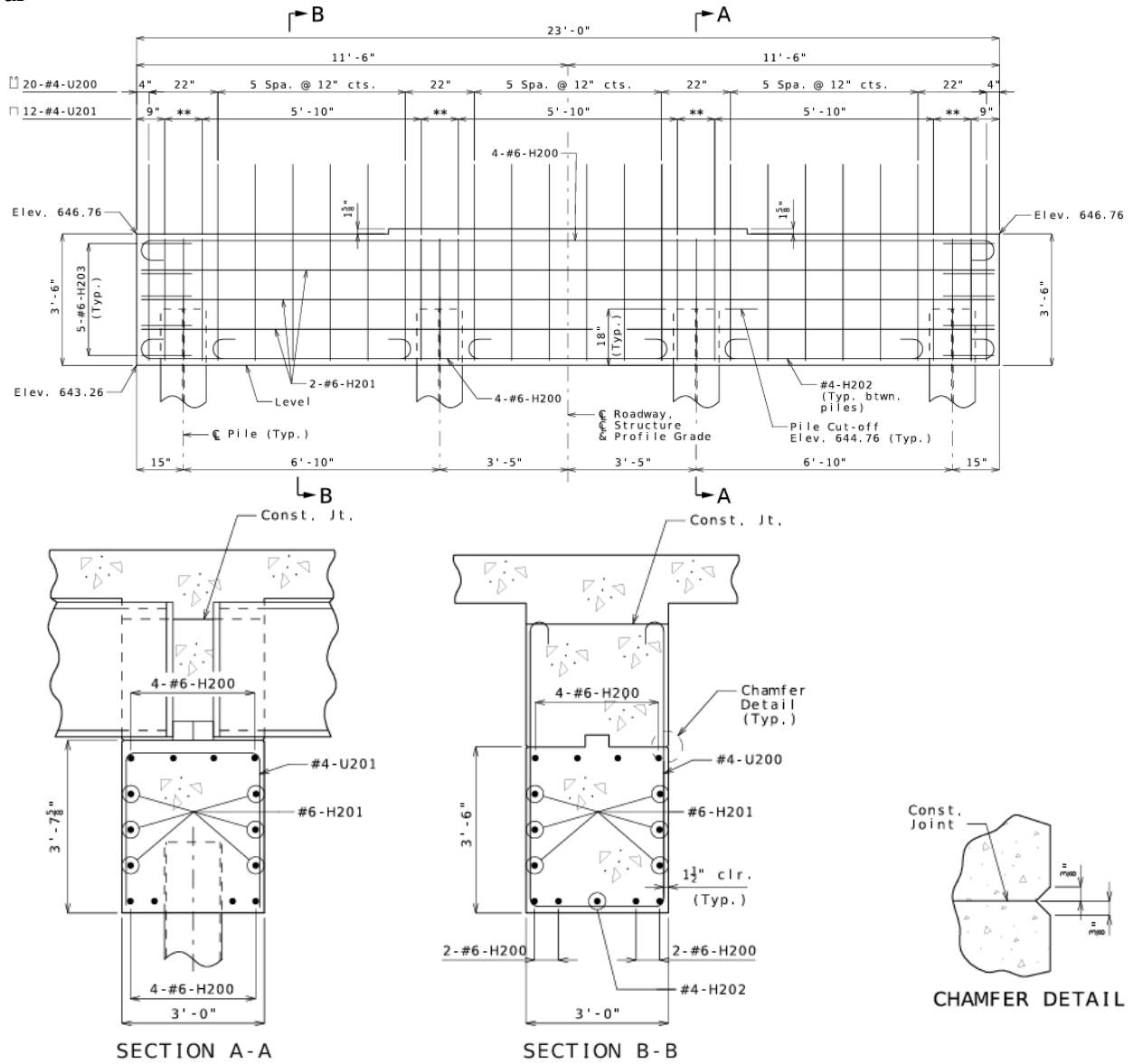


Figure A-5. Elevation view of intermediate bent and bent details.

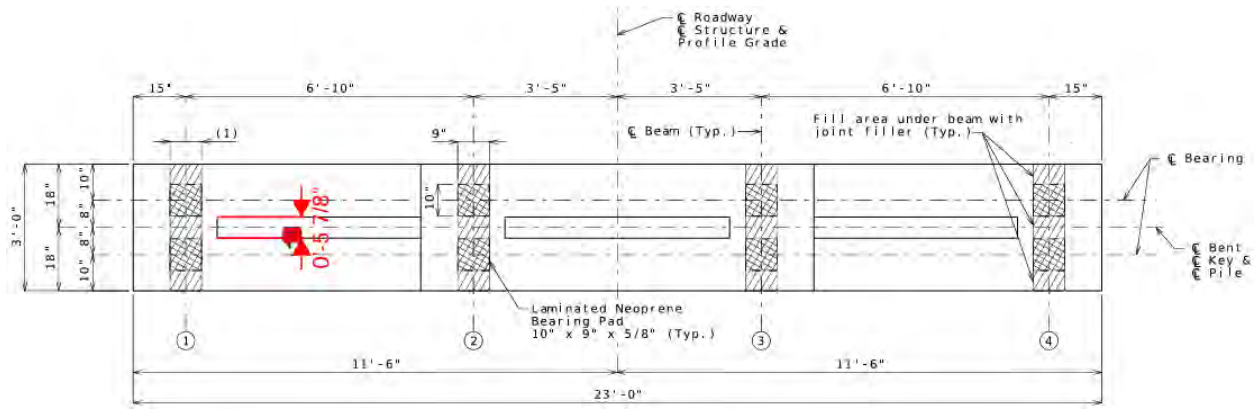


Figure A-6. Plan view of bent cap showing dimensions.

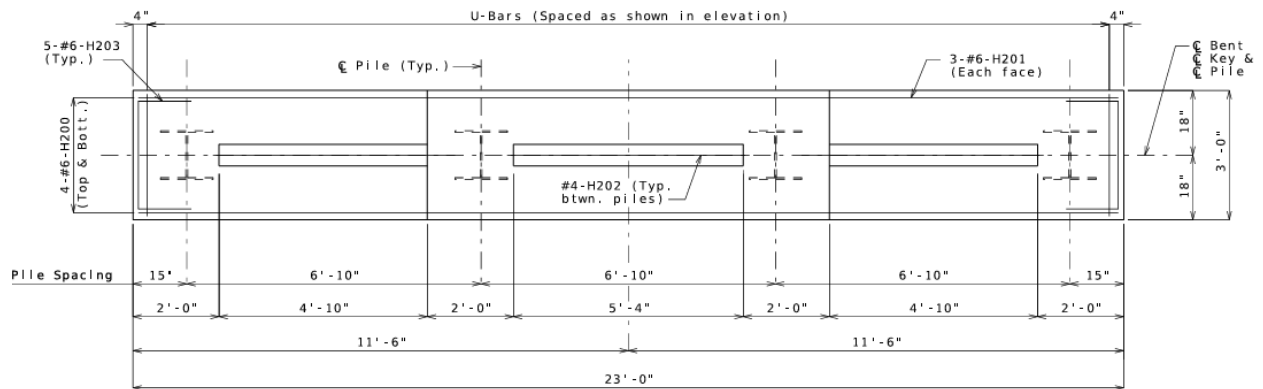


Figure A-7. Plan view of bent cap showing reinforcement.

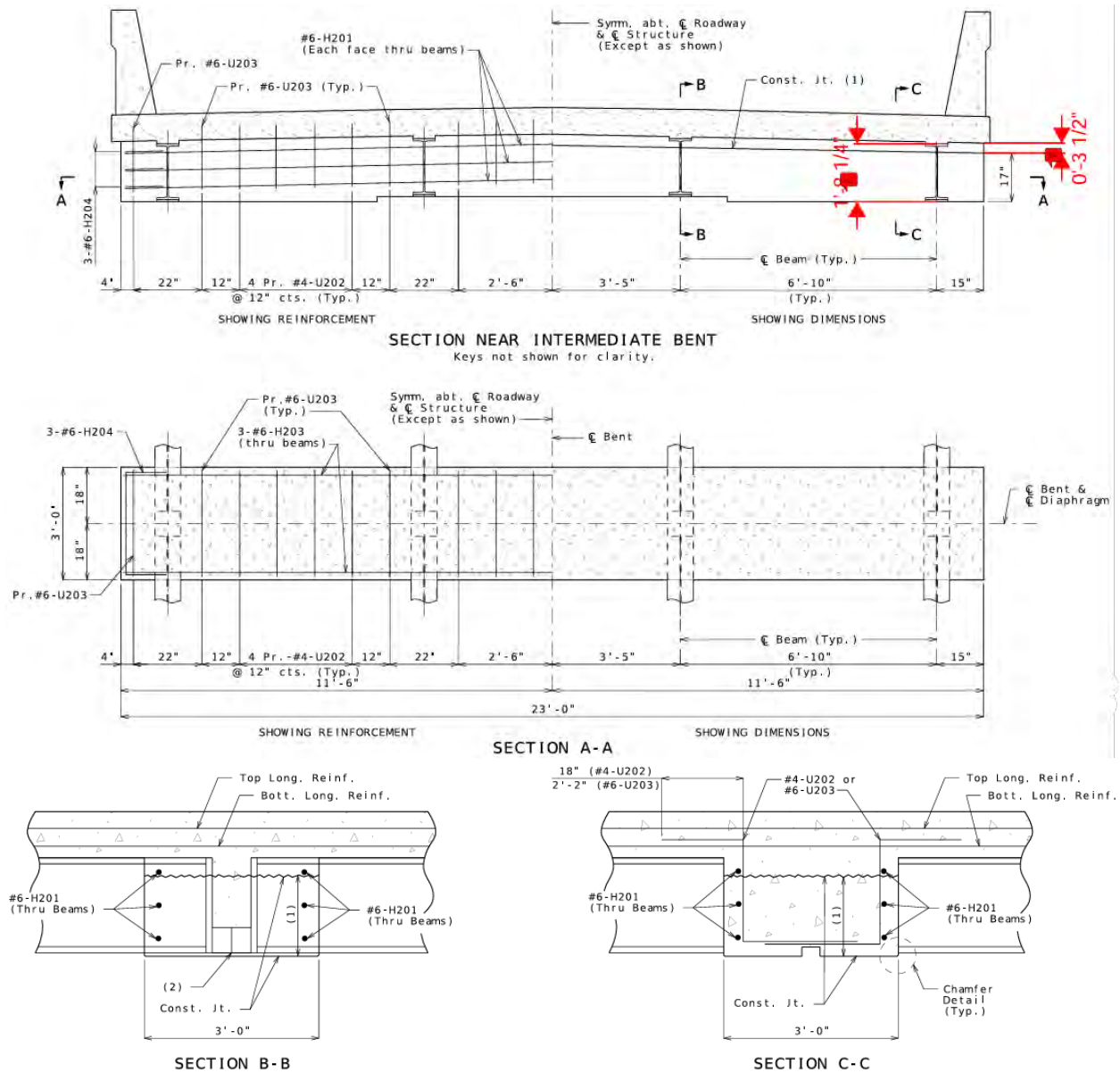


Figure A-8. Diaphragm details near intermediate bents.

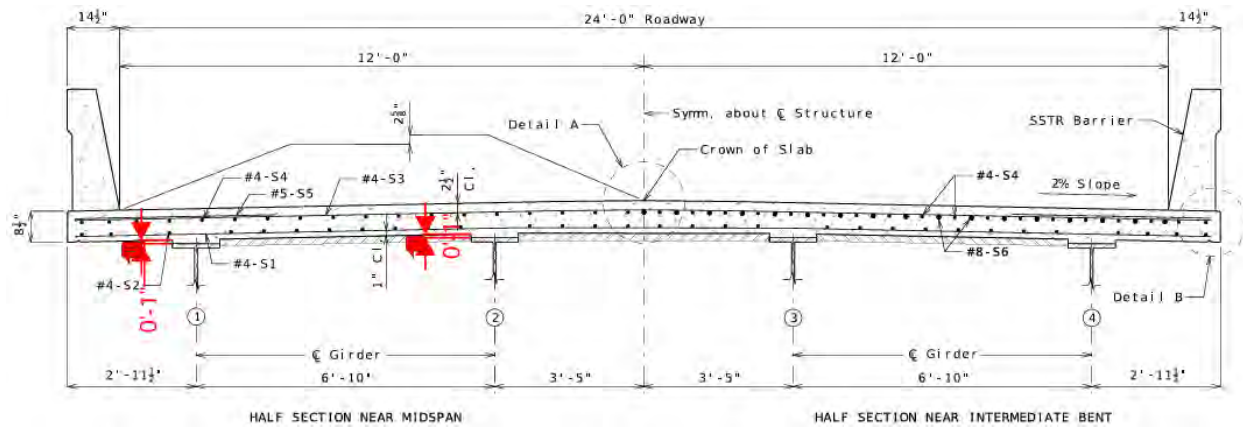


Figure A-9. Elevation view of slab showing dimensions and reinforcement.

Appendix B: Additional Results of the Live-Load Finite Element Models

Predicted vs. Measured Curvature Profiles

Figure B-1 through Figure B-4 and Table B-1 show additional curvature predictions from the calibrated live-load model discussed in Chapter 4. These results complement the results shown in Section 4.6 of the main report.

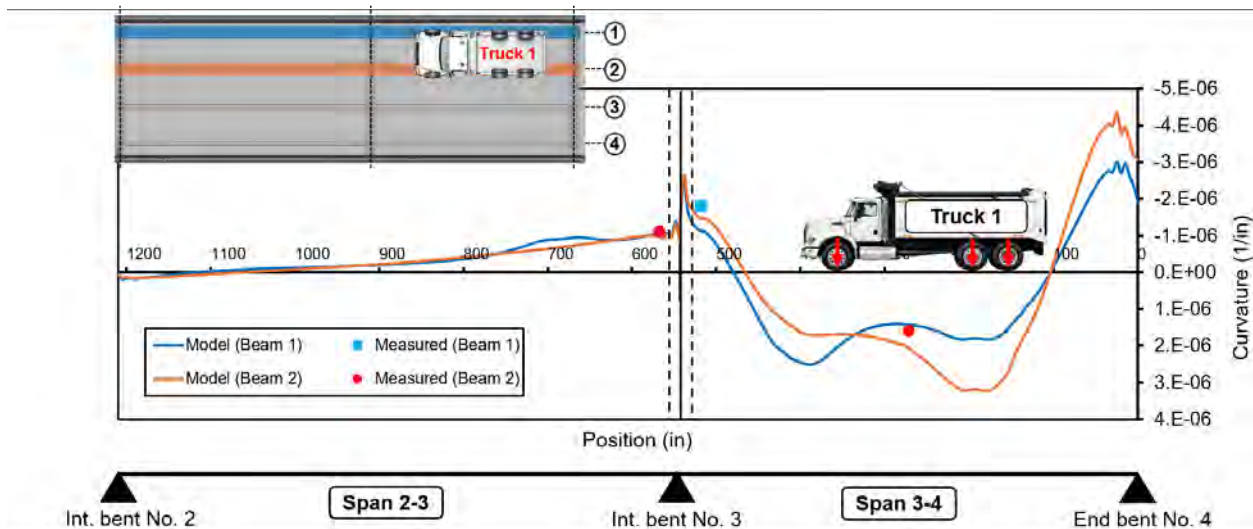


Figure B-1. Predicted and measured curvature profiles for Truck Position 1.

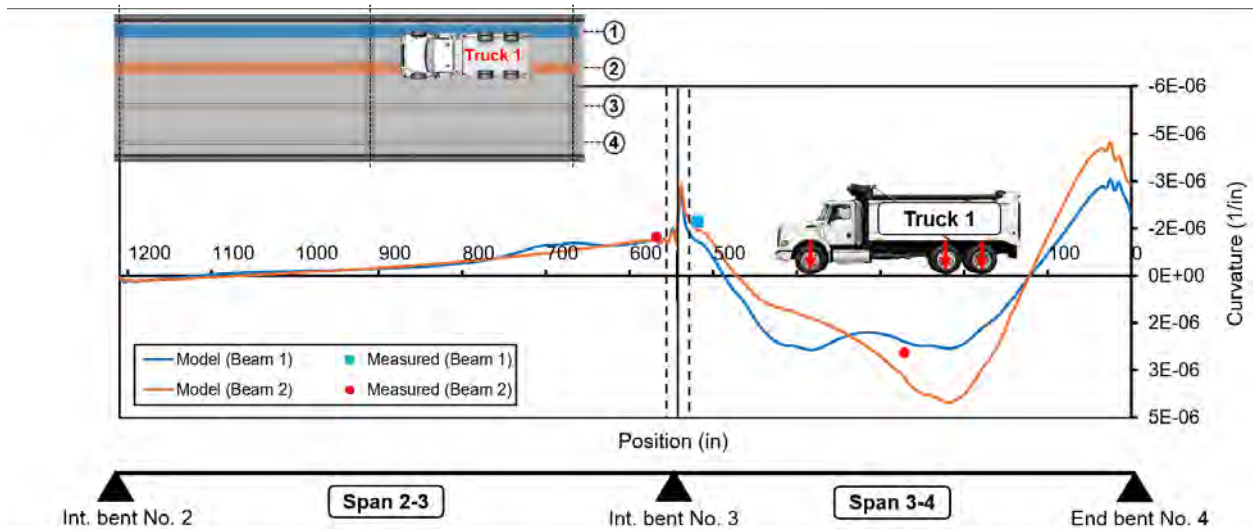


Figure B-2. Predicted and measured curvature profiles for Truck Position 1a.

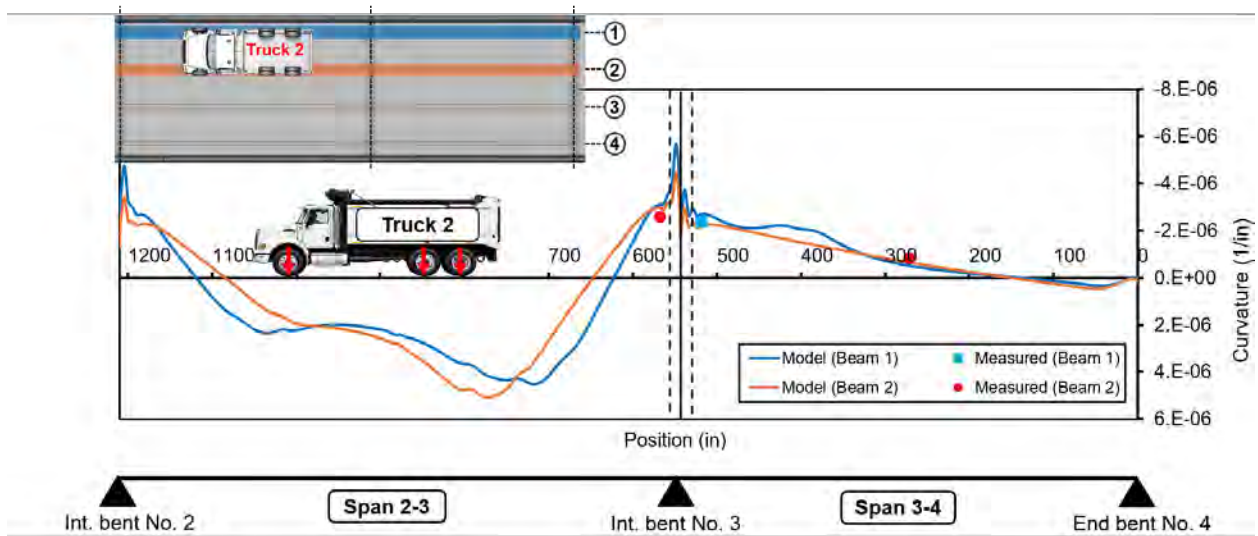


Figure B-3. Predicted and measured curvature profiles for Truck Position 5.

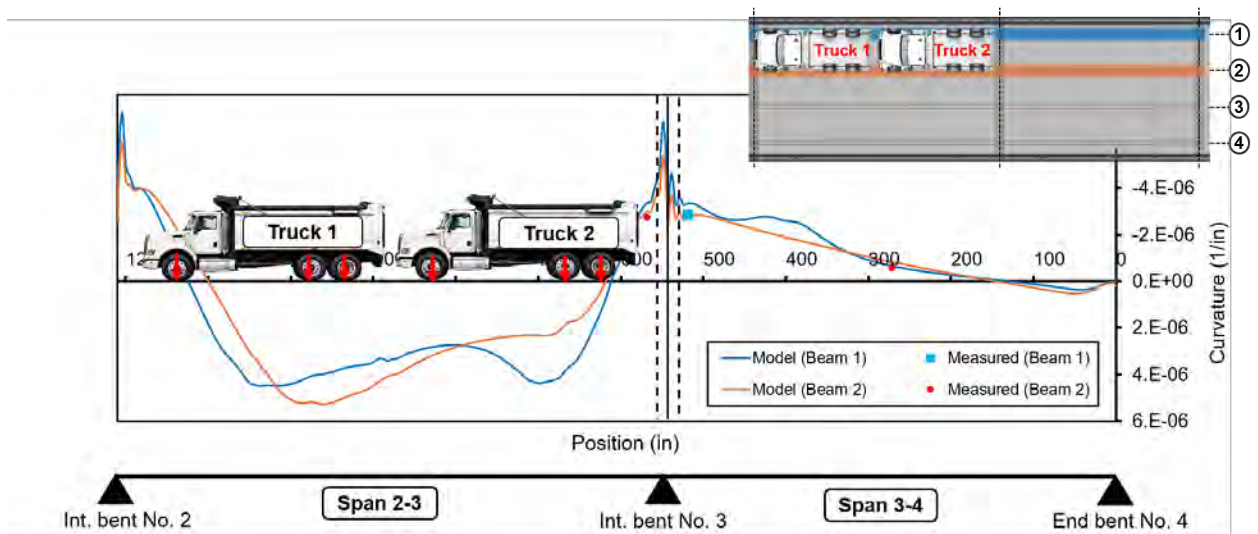


Figure B-4. Predicted and measured curvature profiles for Truck Position 6.

Table B-1. Summary of predicted and measured curvatures during live-load testing

Truck position	Location	Predicted κ ($\times 10^{-6} \text{ in}^{-1}$)	Measured κ ($\times 10^{-6} \text{ in}^{-1}$)	Difference ($\times 10^{-6} \text{ in}^{-1}$)
1	Beam 1 (near support) ^a	-1.12	-1.81	0.69
	Beam 2 (midspan) ^b	2.06	1.57	0.49
	Beam 2 (near support) ^c	-0.97	-1.13	0.16
1a	Beam 1 (near support) ^a	-1.07	-1.71	0.64
	Beam 2 (midspan) ^b	3.2	2.4	0.8
	Beam 2 (near support) ^c	-1.08	-1.25	0.17
2	Beam 1 (near support) ^a	-0.9	-1.4	0.5
	Beam 2 (midspan) ^b	5.18	3.78	1.4
	Beam 2 (near support) ^c	-1.86	-2.17	0.31
3	Beam 1 (near support)	-3.6	-4.13	0.5
	Beam 2 (midspan)	2.58	2.11	0.47
	Beam 2 (near support)	-3.97	-3.85	-0.12
4	Beam 1 (near support)	-3.56	-3.67	0.11
	Beam 2 (midspan)	-1.16	-0.85	-0.31
	Beam 2 (near support)	-4.28	-3.75	-0.53
5	Beam 1 (near support) ^a	-2.71	-2.39	-0.32
	Beam 2 (midspan) ^b	-0.65	-0.85	0.2
	Beam 2 (near support) ^c	-2.97	-2.61	-0.36
6	Beam 1 (near support) ^a	-3.34	-2.87	-0.47
	Beam 2 (midspan) ^b	-0.83	-0.58	-0.25
	Beam 2 (near support) ^c	-3	-2.75	-0.25

^a Curvatures at this location were computed based on readings from ST7 and ST8

^b Curvatures at this location were computed based on readings from ST5 and ST6

^c Curvatures at this location were computed based on readings from ST1 and ST2

Predicted vs. Measured Deflection Profiles

Figure B-5 through Figure B-8 and Table B-2 show additional deflection predictions from the calibrated live-load model discussed in Chapter 4. These results complement the results shown in Section 4.6 of the main report.

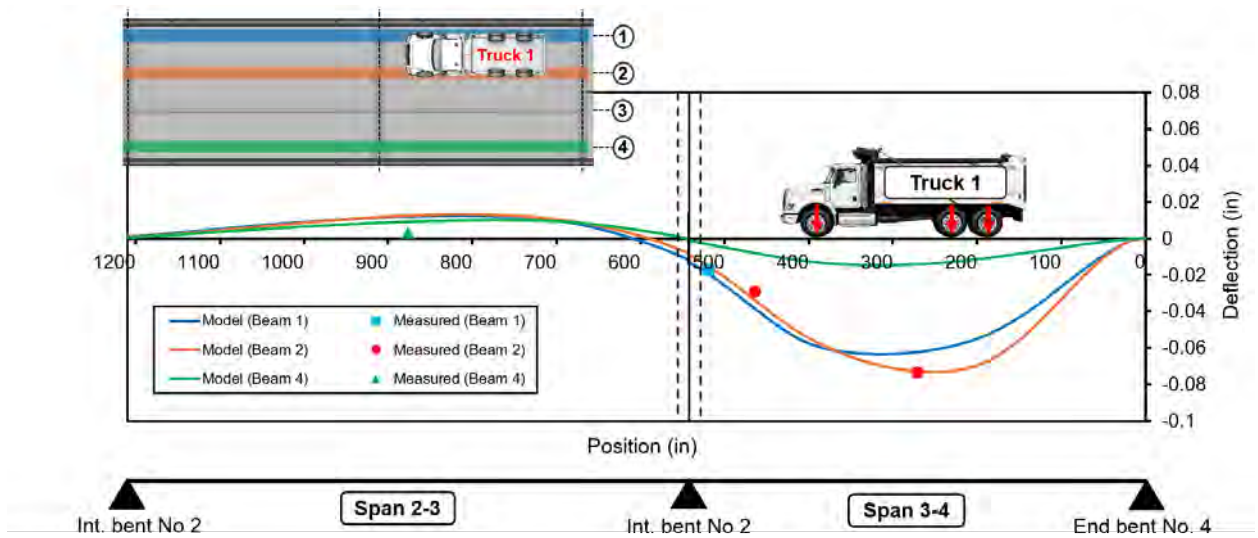


Figure B-5. Predicted and measured deflection profiles for Truck Position 1.

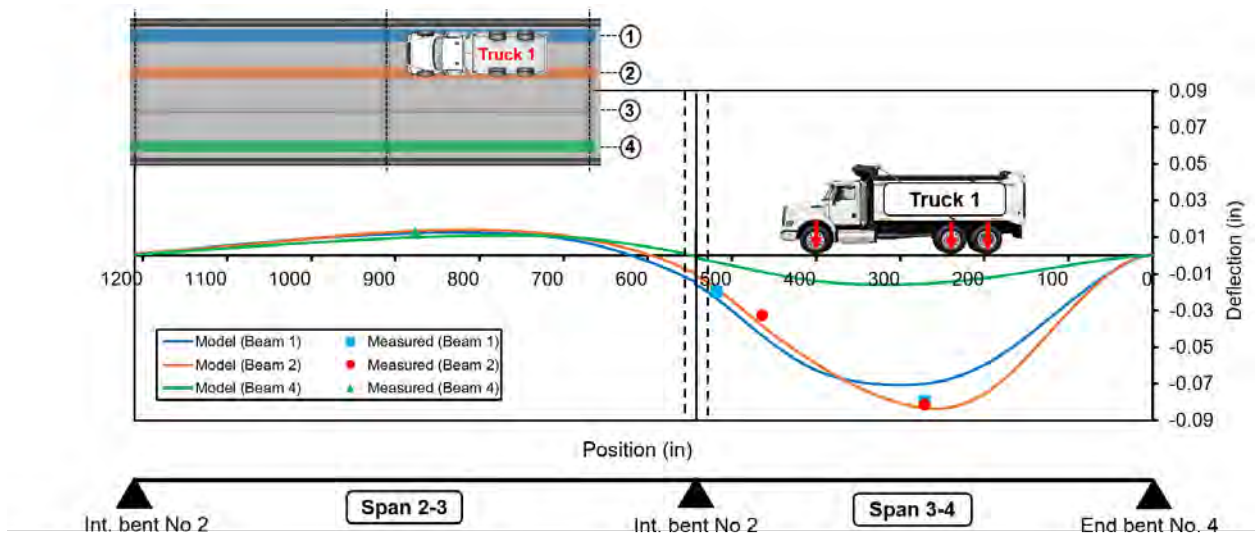


Figure B-6. Predicted and measured deflection profiles for Truck Position 1a.

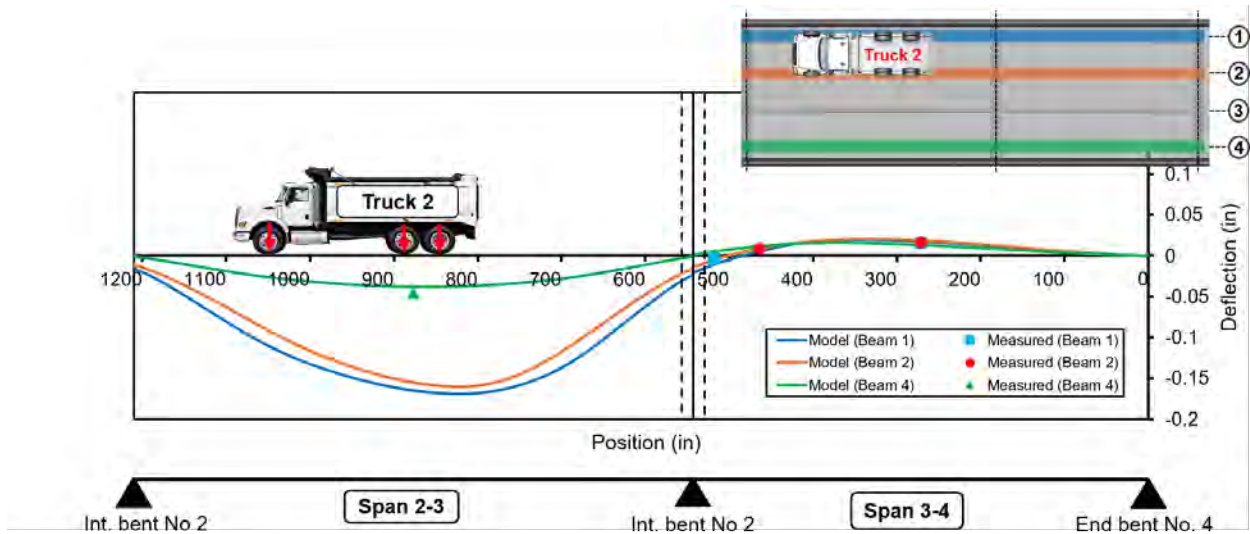


Figure B-7. Predicted and measured deflection profiles for Truck Position 5.

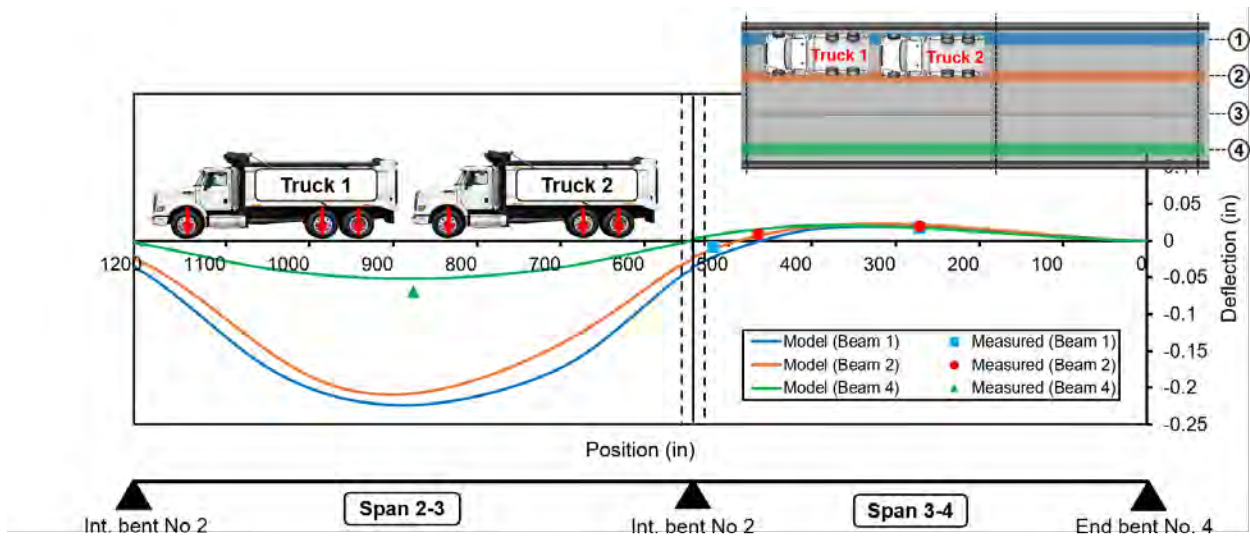


Figure B-8. Predicted and measured deflection profiles for Truck Position 6.

Table B-2. Summary of predicted and measured deflections during live-load testing

Truck position	LVDT	Location ^a	Predicted (in)	Measured (in)	Difference (in)
1	LVDT1	Span 3-4, BL2 near IB3	-0.031	-0.029	0.002
	LVDT2	Span 3-4, BL2 midspan	-0.073	-0.073	-0.001
	LVDT3	Span 3-4, BL1 near IB3	-0.02	-0.017	0.003
	LVDT4	Span 3-4, BL 1 midspan	-0.063	-0.073	-0.01
	SPOT	Span 2-3, BL4 midspan	0.009	0.003	-0.006
1a	LVDT1	Span 3-4, BL2 near IB3	-0.038	-0.032	-0.006
	LVDT2	Span 3-4, BL2 midspan	-0.083	-0.081	-0.002
	LVDT3	Span 3-4, BL1 near IB3	-0.023	-0.020	-0.003
	LVDT4	Span 3-4, BL 1 midspan	-0.070	-0.079	0.009
	SPOT	Span 2-3, BL4 midspan	0.010	0.012	-0.002
2	LVDT1	Span 3-4, BL2 near IB3	-0.059	-0.056	-0.003
	LVDT2	Span 3-4, BL2 midspan	-0.131	-0.137	0.006
	LVDT3	Span 3-4, BL1 near IB3	-0.026	-0.026	0.000
	LVDT4	Span 3-4, BL 1 midspan	-0.084	-0.107	0.023
	SPOT	Span 2-3, BL4 midspan	0.022	0.012	0.010
3	LVDT1	Span 3-4, BL2 near IB3	-0.031	-0.026	-0.005
	LVDT2	Span 3-4, BL2 midspan	-0.066	-0.071	0.005
	LVDT3	Span 3-4, BL1 near IB3	-0.036	-0.024	-0.012
	LVDT4	Span 3-4, BL 1 midspan	-0.054	-0.070	0.016
	SPOT	Span 2-3, BL4 midspan	-0.029	-0.029	0.000
4	LVDT1	Span 3-4, BL2 near IB3	0.018	0.020	-0.002
	LVDT2	Span 3-4, BL2 midspan	0.036	0.034	0.002
	LVDT3	Span 3-4, BL1 near IB3	-0.007	0.001	-0.008
	LVDT4	Span 3-4, BL 1 midspan	0.030	0.031	-0.001
	SPOT	Span 2-3, BL4 midspan	-0.195	-0.174	-0.021
5	LVDT1	Span 3-4, BL2 near IB3	0.008	0.009	-0.001
	LVDT2	Span 3-4, BL2 midspan	0.019	0.017	0.002
	LVDT3	Span 3-4, BL1 near IB3	-0.011	-0.003	-0.008
	LVDT4	Span 3-4, BL 1 midspan	0.018	0.016	0.002
	SPOT	Span 2-3, BL4 midspan	-0.038	-0.045	0.007
6	LVDT1	Span 3-4, BL2 near IB3	0.006	0.009	-0.003
	LVDT2	Span 3-4, BL2 midspan	0.021	0.020	0.001
	LVDT3	Span 3-4, BL1 near IB3	-0.022	-0.008	-0.014
	LVDT4	Span 3-4, BL 1 midspan	0.018	0.018	0.000
	SPOT	Span 2-3, BL4 midspan	-0.052	-0.069	0.017

^a BL refers to beam line and IB refers to intermediate bent

Additional Degree of Continuity Results

Figure B-9 and Figure B-10 report additional degree of continuity estimates beyond those discussed in Chapter 4 (Section 4.8) of the main results.

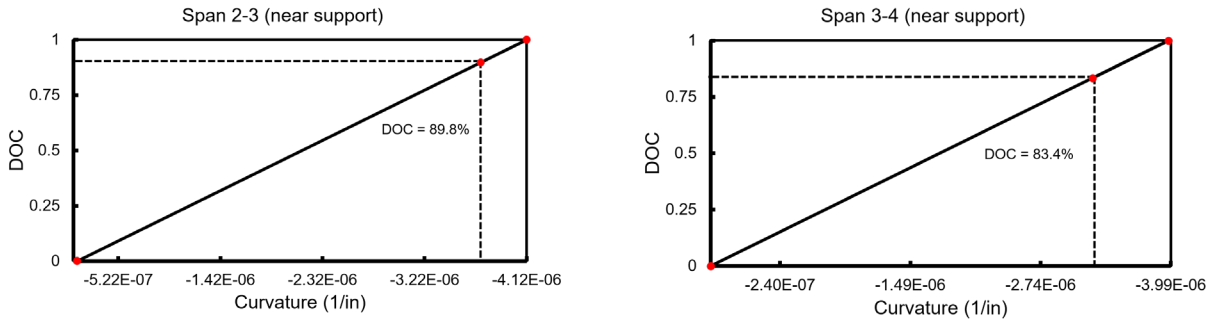


Figure B-9. Curvature-based degree of continuity for Truck Position 3.

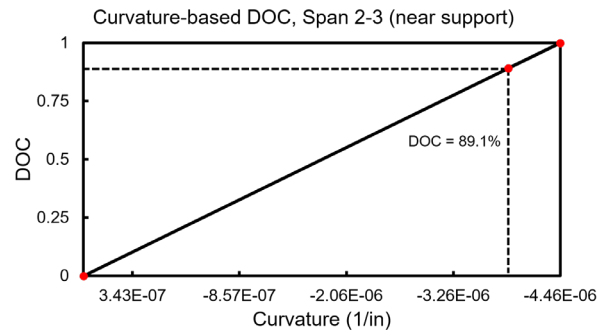


Figure B-10. Curvature-based degree of continuity for Truck Position 4.

Additional Stress Results from the Parametric Models

Table B-3 through Table B-10 show the breakdown of the average deck reinforcement stresses and compression block stresses coming from the factored live loads and superimposed dead loads for the different parametric models considered in Chapter 5 of the main report.

Table B-3. Factored HL-93 and lane load stresses – Base Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate	LC1	7.00	11.67	1.43	2.39	2.44	4.89
		LC2	13.19	21.99	15.59	25.98	4.02	8.04
		LC3	14.50	24.16	12.53	20.88	3.92	7.84
		LC4	3.21	5.36	0.53	0.88	1.29	2.58
		LC5	9.07	15.12	2.72	4.53	2.72	5.44
	Plate	LC1	7.05	11.75	1.43	2.39	2.50	4.99
		LC2	13.23	22.06	15.65	26.08	4.05	8.11
		LC3	14.54	24.23	12.56	20.94	3.95	7.91
		LC4	3.24	5.40	0.53	0.88	1.32	2.63
		LC5	9.08	15.14	2.72	4.54	2.73	5.47
Pinned diaphragm	No Plate	LC1	7.32	12.20	1.68	2.79	1.92	3.84
		LC2	14.63	24.38	17.04	28.39	3.45	6.89
		LC3	15.01	25.02	13.27	22.11	4.26	8.52
		LC4	3.36	5.61	0.63	1.05	1.00	2.00
		LC5	9.62	16.03	2.99	4.98	2.21	4.42
	Plate	LC1	7.35	12.25	1.66	2.77	1.91	3.83
		LC2	14.75	24.58	17.13	28.56	3.46	6.92
		LC3	15.59	25.99	13.80	22.99	3.36	6.71
		LC4	3.27	5.45	0.66	1.11	1.32	2.64
		LC5	9.62	16.03	3.00	4.99	2.22	4.45

Table B-4. Factored superimposed dead load stresses – Base Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate		2.47	4.12	2.46	4.11	0.60	1.20
	Plate		2.47	4.12	2.46	4.11	0.60	1.21
Pinned diaphragm	No Plate		2.63	4.38	2.61	4.35	0.50	1.00
	Plate		2.63	4.39	2.61	4.36	0.50	1.01

Table B-5. Factored HL-93 and lane load stresses – Skew Angle Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate	LC1	4.86	8.10	1.36	2.26	1.14	2.27
		LC2	12.10	20.17	13.41	22.35	1.57	3.14
		LC3	12.69	21.15	11.05	18.41	1.63	3.27
		LC4	3.15	5.25	0.83	1.39	0.61	1.22
		LC5	8.60	14.34	3.31	5.51	0.96	1.93
	Plate	LC1	4.92	8.20	1.35	2.25	1.13	2.26
		LC2	12.33	20.56	13.50	22.50	1.54	3.09
		LC3	12.94	21.57	11.12	18.54	1.61	3.22
		LC4	3.20	5.34	0.81	1.36	0.61	1.22
		LC5	8.73	14.55	3.34	5.57	1.00	2.00
Pinned diaphragm	No Plate	LC1	4.78	7.97	1.39	2.32	0.99	1.98
		LC2	12.04	20.06	13.18	21.96	2.06	4.13
		LC3	12.64	21.07	10.84	18.06	2.33	4.66
		LC4	2.97	4.95	0.93	1.55	0.67	1.35
		LC5	8.46	14.10	3.16	5.27	1.44	2.88
	Plate	LC1	4.86	8.10	1.42	2.37	1.04	2.08
		LC2	12.30	20.50	13.49	22.49	2.14	4.27
		LC3	12.87	21.45	11.08	18.47	2.28	4.56
		LC4	3.01	5.02	0.96	1.60	0.71	1.42
		LC5	8.53	14.22	3.23	5.39	1.49	2.98

Table B-6. Factored superimposed dead load stresses – Skew Angle Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate		1.85	3.08	1.85	3.08	0.45	0.91
	Plate		1.90	3.16	1.90	3.16	0.45	0.90
Pinned diaphragm	No Plate		1.58	2.64	1.75	2.92	0.44	0.89
	Plate		1.57	2.61	1.75	2.91	0.46	0.92

Table B-7. Factored HL-93 and lane load stresses – Plate Girder Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate	LC1	4.86	8.10	0.26	0.43	2.22	4.43
		LC2	10.74	17.89	11.67	19.44	3.25	6.51
		LC3	17.83	29.72	6.63	11.05	4.08	8.16
		LC4	1.67	2.78	0.86	1.43	0.95	1.90
		LC5	20.20	33.67	7.18	11.96	4.57	9.14
	Plate	LC1	4.90	8.17	0.27	0.45	2.15	4.31
		LC2	10.95	18.25	11.89	19.81	3.20	6.41
		LC3	18.06	30.10	6.75	11.24	4.09	8.18
		LC4	1.69	2.81	0.86	1.43	0.92	1.84
		LC5	20.42	34.04	7.30	12.17	4.59	9.17
Pinned diaphragm	No Plate	LC1	4.73	7.88	0.50	0.83	1.87	3.74
		LC2	11.18	18.63	12.04	20.07	3.03	6.06
		LC3	18.22	30.37	6.68	11.13	4.07	8.14
		LC4	1.56	2.60	0.72	1.20	0.79	1.58
		LC5	20.69	34.48	7.23	12.05	4.62	9.25
	Plate	LC1	4.61	7.68	0.60	1.00	1.69	3.38
		LC2	11.17	18.62	12.05	20.08	2.81	5.63
		LC3	18.43	30.71	6.65	11.08	4.06	8.11
		LC4	1.49	2.49	0.65	1.08	0.70	1.40
		LC5	20.87	34.78	7.19	11.98	4.56	9.13

Table B-8. Factored superimposed dead load stresses – Plate Girder Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate		3.19	5.32	3.22	5.37	0.62	1.25
	Plate		3.23	5.38	3.26	5.43	0.62	1.25
Pinned diaphragm	No Plate		3.23	5.38	3.26	5.43	0.62	1.25
	Plate		3.25	5.41	3.28	5.46	0.61	1.23

Table B-9. Factored HL-93 and lane load stresses – Unbalanced Span Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate	LC1	4.27	7.12	0.69	1.14	1.74	3.49
		LC2	13.11	21.84	14.51	24.18	3.56	7.11
		LC3	16.05	26.75	11.84	19.73	4.78	9.55
		LC4	2.01	3.35	0.18	0.30	0.93	1.86
		LC5	10.98	18.30	5.48	9.13	2.86	5.73
	Plate	LC1	4.25	7.08	0.97	1.61	1.77	3.54
		LC2	13.34	22.23	14.77	24.61	4.67	9.34
		LC3	16.40	27.33	12.02	20.03	4.70	9.40
		LC4	2.06	3.43	0.18	0.30	0.94	1.88
		LC5	11.09	18.49	5.59	9.32	2.82	5.63
Pinned diaphragm	No Plate	LC1	4.24	7.07	0.82	1.37	1.58	3.16
		LC2	13.78	22.97	15.10	25.17	4.48	8.97
		LC3	16.38	27.29	12.14	20.23	4.69	9.39
		LC4	2.01	3.36	0.26	0.43	0.84	1.68
		LC5	11.10	18.50	5.65	9.41	2.77	5.54
	Plate	LC1	4.16	6.93	0.86	1.44	1.60	3.20
		LC2	13.73	22.88	14.98	24.96	4.16	8.32
		LC3	16.40	27.33	12.03	20.06	4.59	9.17
		LC4	1.96	3.27	0.28	0.47	0.84	1.68
		LC5	11.08	18.47	5.61	9.35	2.69	5.38

Table B-10. Factored superimposed dead load stresses – Unbalanced Span Bridge.

			Deck IB 2		Deck IB 3		Compression block	
			Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. rein. stress (ksi)	$f_{s,avg}/f_{y,s}$ (%)	Avg. stress (ksi)	$f_{c,avg}/f_{y,pl}$ (%)
Fixed diaphragm	No Plate		3.19	5.32	3.22	5.37	0.62	1.25
	Plate		3.23	5.38	3.26	5.43	0.62	1.25
Pinned diaphragm	No Plate		3.23	5.38	3.26	5.43	0.62	1.25
	Plate		3.25	5.41	3.28	5.46	0.61	1.23

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751.15 SDCL Steel Superstructure

Simple for Dead Load, Continuous for Live Load (SDCL) design for steel superstructures is an alternative design approach to conventional continuous steel girder bridges and prestressed concrete bridges. Similar to prestressed design methodology, girders are placed on substructure units as simple spans and then made continuous over the intermediate bents. For SDCL steel bridges, the ends of the girders are cast into a concrete diaphragm and continuity is achieved through tensile reinforcement in the slab and contact between steel blocks welded to the bottom flanges to transfer compressive forces². This design approach has been used in conventional and accelerated construction methods.

SDCL steel girder design methodology offers several benefits over conventional steel or prestressed design that make it a competitive option:

- eliminates the need for bolted splices that are typical of conventional continuous steel superstructures²,
- simpler girder detailing¹ allowing for lower levels of AISC fabricator certification,
- reduced negative moment demand at int. bents due to simple behavior of girders prior to achieving full continuity²,
- increased stability during construction due to partial casting of girder ends²,
- simplified casting sequences compared to continuous multi-span structures²,
- increased protection at girder ends against rust and corrosion from concrete encasement²,
- potential for reduced construction time from simplified girder detailing, fabrication, and installation,
- reduced movement in seismic scenarios due to lighter superstructures,
- lower substructure vertical load demand,
- potential for shipping/transport savings due to smaller section sizes and smaller required cranes⁵,
- a uniform cross section can be used for an entire span¹,
- and lower inspection and maintenance costs from encased girder ends compared to conventional steel bridge systems⁴.

751.15.1 General

Steel SDCL design may be used if the criteria below are satisfied, or with approval of the SPM or SLE.

- Seismic Design Category (SDC) A or B*
 - Skews less than or equal to 45 degrees
 - Non-curved girders⁴
 - Fixed intermediate bents**
 - Maximum span length of 120 feet. Longer spans require approval of the SPM or SLE.
 - Other considerations can be used to justify longer spans like the elimination of splices over roadways, railroads, accelerated bridge construction, and site constraints.
- * Refer to EPG 751.15.2.3 Seismic Application
- ** Refer to EPG 751.22.3.7 Closed Concrete Intermediate Diaphragms

751.15.1.1 Materials and Selection of Steel

Reinforcing Steel Yield Strength (F_y) and Tensile Strength (F_u)

ASTM A615 Grade 60 (SDC A and all rehab/redeck)	$F_y = 60$ ksi	$F_u = 80$ ksi
ASTM A706 Grade 60 (SDC B, C, and D)	$F_y = 60$ ksi	$F_u = 80$ ksi

[Refer to the *Omissions* section of the document. This section will be similar to 751.14.1.1, except with the removal of field splices, and revisions as shown in the *Omissions* section.]

751.15.1.2 Girder Limits and Preferences

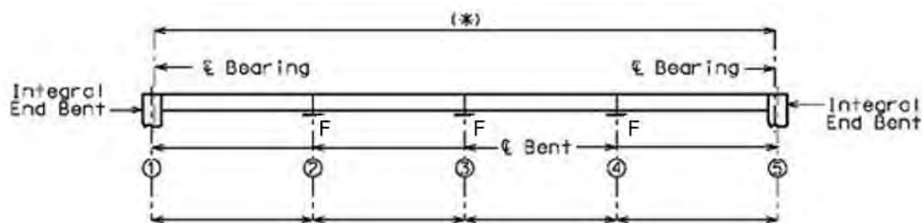
Consideration should be given to the use of galvanization or painting over the use of weathering steel for wide flange shapes when projects depend on the availability and rolling schedules of the girders, particularly smaller rolled shapes. Galvanizing is recommended for girder sections under 60' to increase longevity and reduce maintenance. Rolled girder weights may vary by span as availability allows but it is recommended to use the same rolled girder section size for weathering steel due to minimum mill order requirements and rolling schedules.

Consideration should be given to use of a vibratory screed for deck finishing, provided that the requirements of EPG 751.10.1.15 are satisfied or with approval of the SPM, SLE, or owner's representative for a particular bridge.

If varying the girder size at the intermediate bent, the overall girder depth shall remain constant. The bottom flange widths need not be identical, but design of the compression blocks shall be for the smaller girder section.

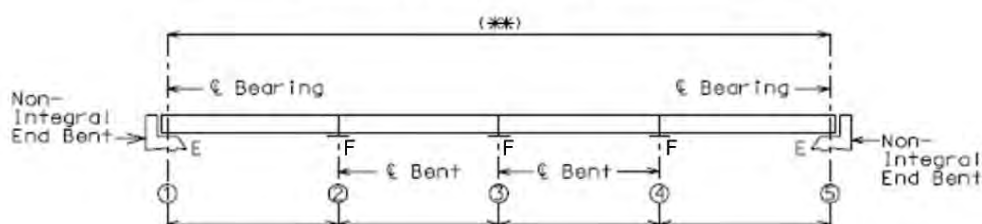
Structure Length

Typical SDCL steel structure – integral and non-integral end bents:



(*) Maximum length from End Bent to End Bent = 500 feet.

Structure with Integral End Bents



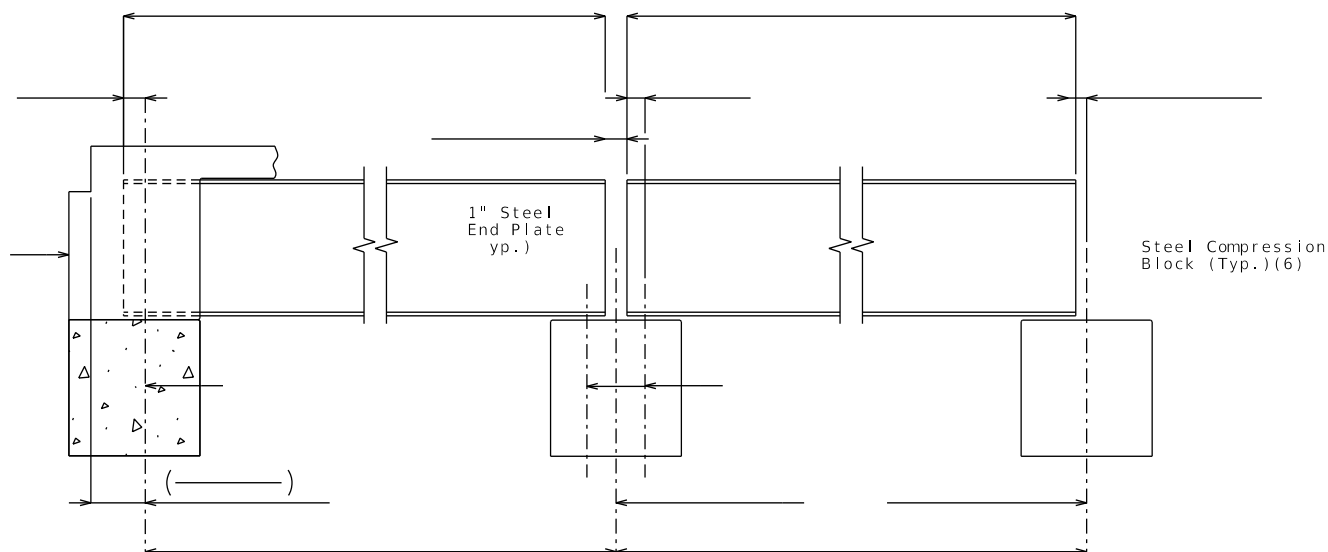
(**) Maximum length from End Bent to End Bent = 700 feet.

Structure with Non-Integral End Bents

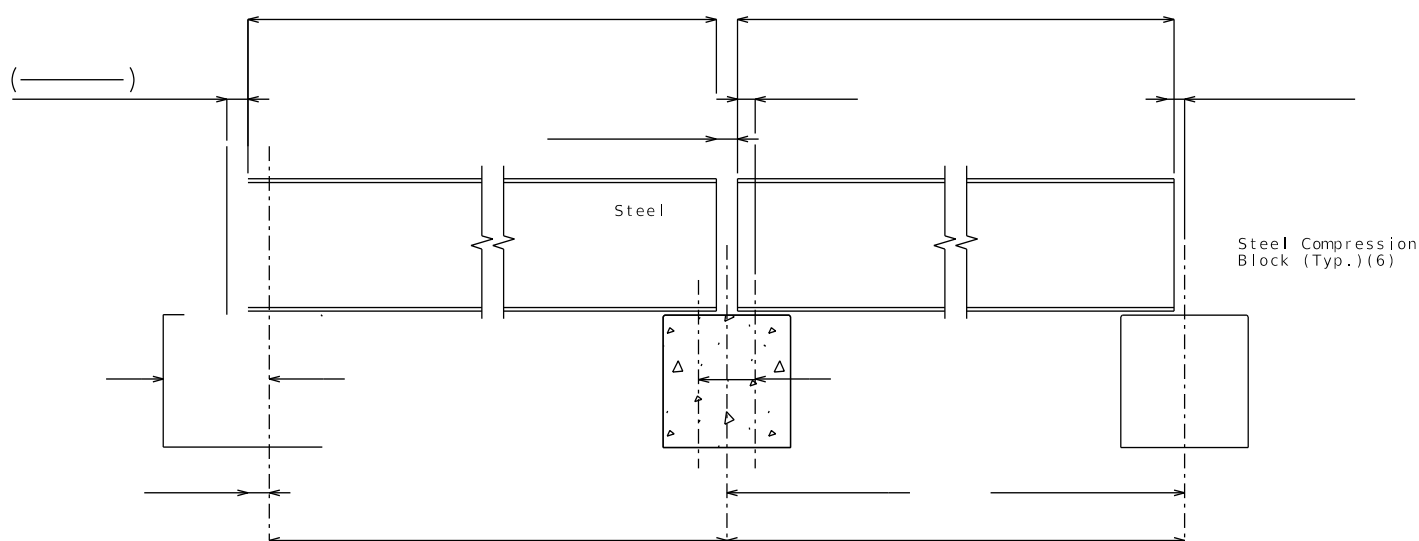
Girder Length and Geometric Layout

Tangent Bridges

Girder lengths of exterior spans (i.e., end spans) and interior spans shall be computed using the requirements shown below:



Layout Length with Integral End Bents



Layout Length with Non-Integral End Bents

- (1) Minimum dimension from edge of bearing pad to end of girder equals one inch.
- (2) Design layout lengths are horizontal lengths. Girder lengths should be adjusted according to grade and shall be specified to the nearest 1/8 inch.
- (3) For large skews, end bent beam caps may need to be larger to provide edge distance.
- (4) Horizontal distance along centerline of girder.
- (5) = 1" (minimum) + 1/2 bearing pad length which equals 5" min. for Steel I-girders.
- (6) Minimum compression block length, parallel to girder length, = 2". Refer to 751.15.2.3.

Curved Bridges

The use of the SDCL methodology has not been evaluated for bridges on curves or using curved girders. All girders shall be tangent from end to end.

[The remainder of this section will be similar to 751.14.1.2 -- refer to the Omissions section of the document for changes to 751.14.1.2 in creation of 751.15.1.2.]

751.15.1.3 Optimizing Girder Spacing on Non-Standard Roadways

[Same as 751.14.1.3.]

751.15.2 Design

751.15.2.1 Limit States and Load Factors

[Same as 751.14.2.1.]

751.15.2.2 Analysis Methods

Design Analysis Software

Design software used for Simple for Dead Load, Continuous for Live Load steel superstructures must have the ability to evaluate the girder under the girder weight and deck dead load acting as simple spans prior to deck hardening and establishing continuity for DC2, DW and live loads at the intermediate bent. This is typically accomplished by utilizing hinges (or other release mechanisms) at the intermediate bent for DC1 loads and then establishing continuity with the reinforcement in the deck at the intermediate bents for the remainder of the applied structure loading.

The following design software is capable of modeling and analyzing SDCL structures⁶:

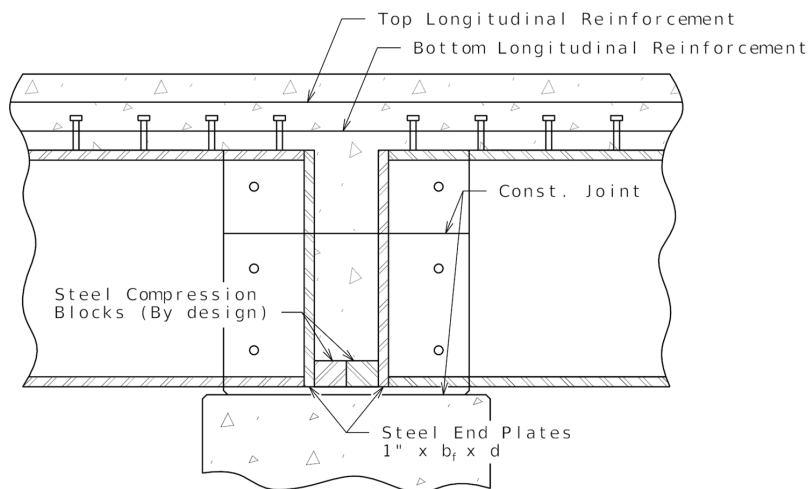
- AASHTOWare BrD/BrR
- MDX Software
- LEAP Bridge Steel
- Merlin-DASH

[Remainder of section same as 751.14.2.2, refer to the *Omissions* section of the document.]

751.15.2.3 Compression Blocks

General

Steel blocks are used to transfer compressive forces from the bottom flange of one steel girder to the bottom flange of an adjacent steel girder without relying on the concrete in the diaphragm².



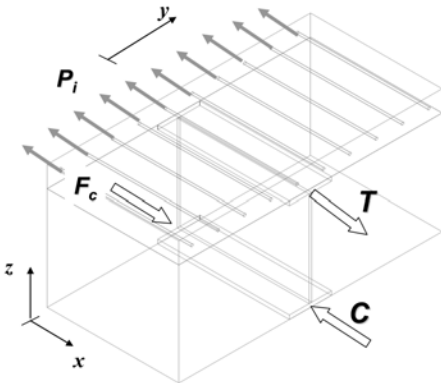
Part Elevation

Dimensions

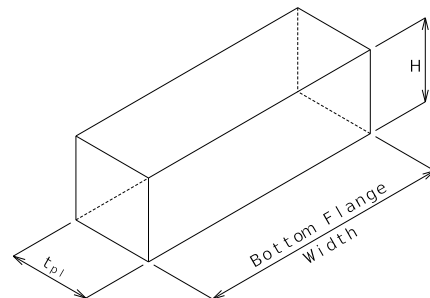
The width of the compression block shall match the bottom flange width of the narrower flange if there is a shape transition.. The thickness of the compression block, measured parallel to the girder length, shall be a minimum of 2 inches² (1/4" increments). The height of the compression block, measured parallel to the girder depth, shall be determined by design. Not required, but preferred for fabrication, is to use a block height equivalent to the block thickness where design criteria are satisfied. See EPG 751.15.4.1 for details.

Connection Capacity

The flexural capacity of the connection is primarily driven by the force couple created by the tensile reinforcement placed in the slab and the compression blocks attached to the bottom flanges².



Force Couple Diagram



Compression Block Dimensions

The flexural capacity is calculated as follows²:

$$M_n = A_s f_y \left(d - \frac{H}{2} \right)$$

Where:

M_n = nominal moment capacity of the connection, k – in.

A_s = area of tensile reinforcement, in.²

f_y = yield strength of reinforcing steel, ksi

d = distance between bottom of the girder and centroid of the tensile reinforcement, in.

H = compression block height measured parallel to the girder depth, in.

t_{pl} = thickness of the compression block measured parallel to the girder length, in.

The area of tensile reinforcement present in the slab shall be determined in accordance with EPG 751.10.1.5 similar to Prestressed Structures.

Compression Block Design Steps

Step 1 – Assume compression block height

The compression block height is assumed to determine an area of tensile reinforcement required to balance the force couple².

$$A_{s,required} = \frac{M_u}{\phi f_y (d - \frac{H}{2})}$$

Where:

$A_{s,required}$ = minimum area of tensile reinforcement required, in.²

M_u = factored ultimate moment demand, k – in

ϕ = resistance factor for tension – controlled concrete sections [AASHTO 5.5.4.2]

Step 2 – Calculate minimum block height

The compression block is designed to carry a force equal to 1.7 times that carried by the tensile reinforcement, or $1.7f_y$, to ensure its behavior remains elastic². Compare H_{min} with the block height, H , used in Step 1 to ensure the assumed block height is adequate.

$$H_{min} = \frac{1.7f_y A_{s,provided}}{b_f F_{ypl}}$$

Where:

H_{min} = minimum height of the compression block parallel to the girder depth, in.

b_f = girder bottom flange width, in.

F_{ypl} = yield strength of the steel compression block, ksi

Step 3 – Calculate capacity of connection

Once the minimum height of the compression block and required area of tensile reinforcement is determined, the nominal flexural capacity, M_n , of the connection can be calculated. If the provided area of tensile reinforcement is greater than the required area, the height of the compression block should be determined based on the actual arrangement of tensile reinforcement.

$$M_n = A_s F_y (d - \frac{H}{2})$$

Seismic Application

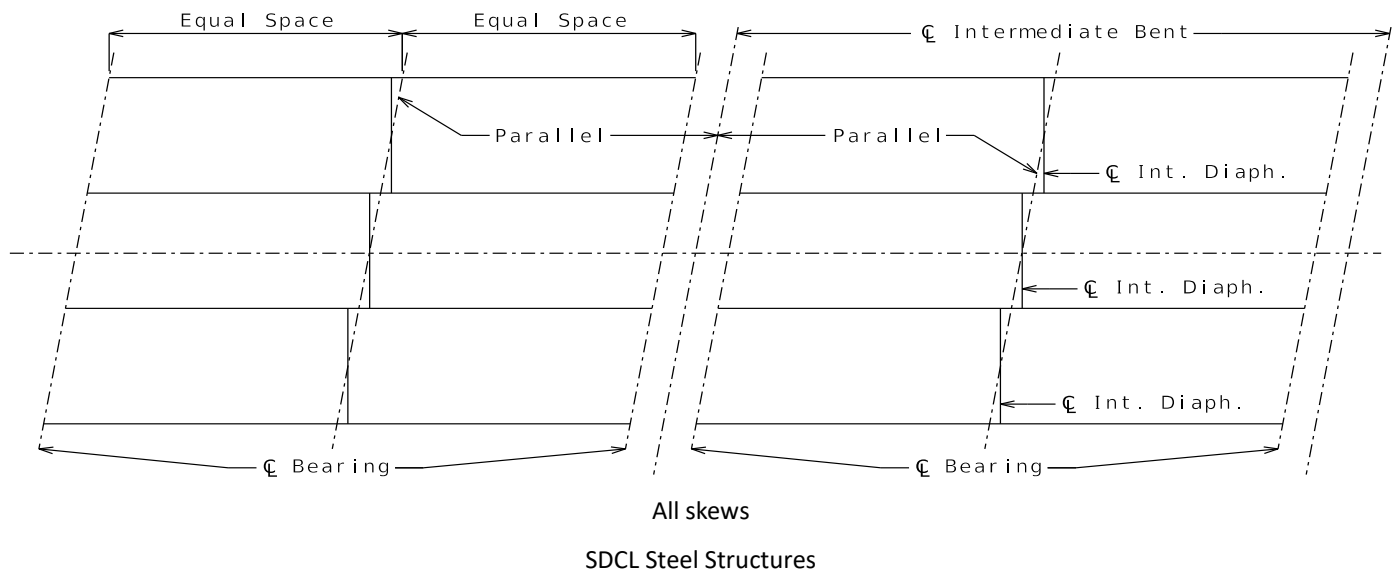
The SDCL connection design and details for non-seismic zones have been investigated both numerically and experimentally and numerous standing SDCL bridges have been monitored for field performance. The suggested SDCL connection design and details for moderate-to-high seismic areas are very similar to non-seismic zones⁶; however, SDCL bridges in highly seismic regions have not been constructed and monitored to the same extent as bridges in non-seismic zones. For SDCL design in moderate-to-high seismic areas,

consult with the State Bridge Engineer, Structural Project Manager, or District. Refer to EPG 751.9 for the Seismic Design Flowchart and see the *ABC-UTC Guide for Superstructure to Pier Connection in SDCL Steel Bridge Systems* for seismic SDCL connection information⁴.

751.15.2.4 Intermediate Diaphragms and Cross Frames

Straight Girder Layout

For W24x and shallower rolled shapes, consideration should be given to providing intermediate diaphragms and cross frames at smaller spacings to increase stability during construction, prior to deck curing.



Diaphragm Layout for SDCL Steel Structures

A note on the plans is required, see EPG 751.50 Standard Detailing Notes for note (H6.20):

"The Contractor shall provide bracing necessary for lateral and torsional stability of the girders to remain plumb during construction of the concrete slab and remove the bracing after the slab has attained 75% design strength. Contractor shall not weld on or drill holes in beam without Engineer's written approval."

Remainder of section same as 751.14.2.3, refer to the *Omissions* section of the document.

751.15.2.5 End Diaphragms

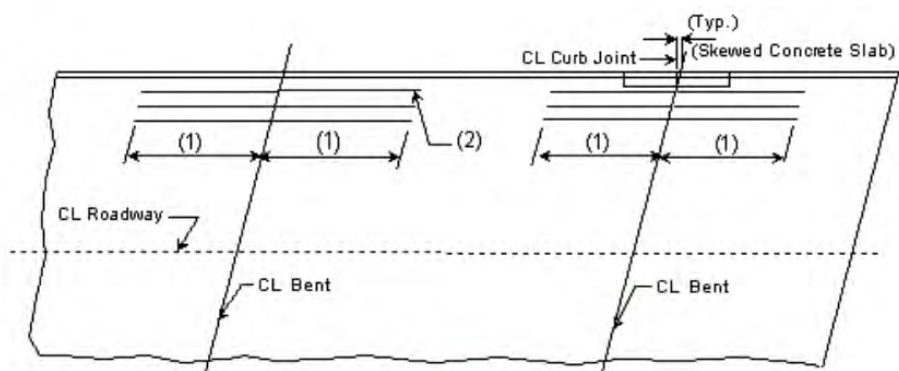
Steel end diaphragms shall be located at all non-integral end bents where continuity of the slab is broken (i.e., expansion joints). Integral end bents shall use a concrete diaphragm. See EPG 751.14.5.3 and 751.14.5.4 for details of end diaphragms for wide flange beams and plate girders, respectively.

Remainder of section same as 751.14.2.4, refer to the *Omissions* section of the document.

751.15.2.6 Other Requirements

Minimum Negative Flexure Reinforcement

Modify EPG 751.10.1.5



Prestressed Structures

- (1) Bar length by **design** and Simple for Dead Load Continuous for Live Load Steel
 - (2) Reinforcement placed between longitudinal temperature reinforcing in top.
- Bar size: #5 bars at 7 1/2" cts. (Min.)
#8 bars at 5" cts. (Max.)

Bearings

Plain and laminated elastomeric pads are preferred for SDCL steel structures; see EPG 751.11. Expansion bearings shall not be used for SDCL steel structures as the ends of the compression blocks need to remain in contact and testing/research have not explored the effects of expansion bearings on the performance of the SDCL connection.

Bearing Stiffeners

Bearing stiffeners are not required for SDCL steel structures: one-inch-thick steel plates are welded to the girder ends at intermediate bents and the ends of the girders are encased in a concrete diaphragm. For details of bearing stiffeners for rolled shapes at non-integral end bents and for plate girders, refer to EPG 751.14.5.1.

Remainder of section same as 751.14.2.6, refer to the *Omissions* section of the document.

751.15.3 Composite Design

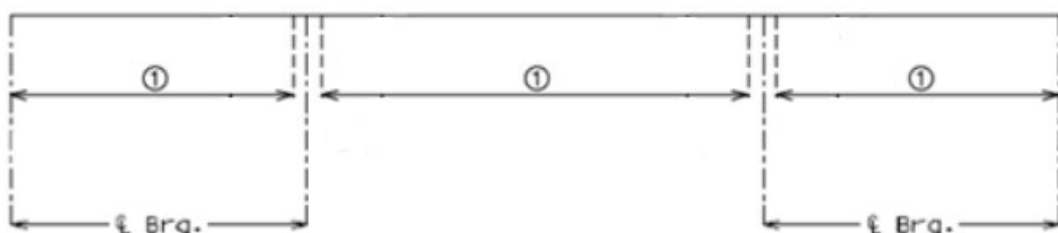
751.15.3.1 General

Same as 751.14.4.1, refer to the *Omissions* section of the document.

751.15.3.2 Shear Connector Limits

Shear Connector Spacing

Shear connectors for SDCL steel structures shall be provided along the entire length of the span. For a typical 3-span SDCL steel bridge, the shear connectors can be approximately arranged as below:



① Shear connector units at pitch as indicated by design, providing total shear connectors satisfy both fatigue stress and ultimate strength.

Remainder of section same as 751.14.4.4, refer to the *Omissions* section of the document.

751.15.3.3 Fatigue Design

Same as 751.14.4.3, refer to the *Omissions* section of the document.

751.15.3.4 Strength Design

Same as 751.14.4.4, refer to the *Omissions* section of the document.

751.15.3.5 Shear Connector Details

Same as 751.14.4.5, refer to the *Omissions* section of the document.

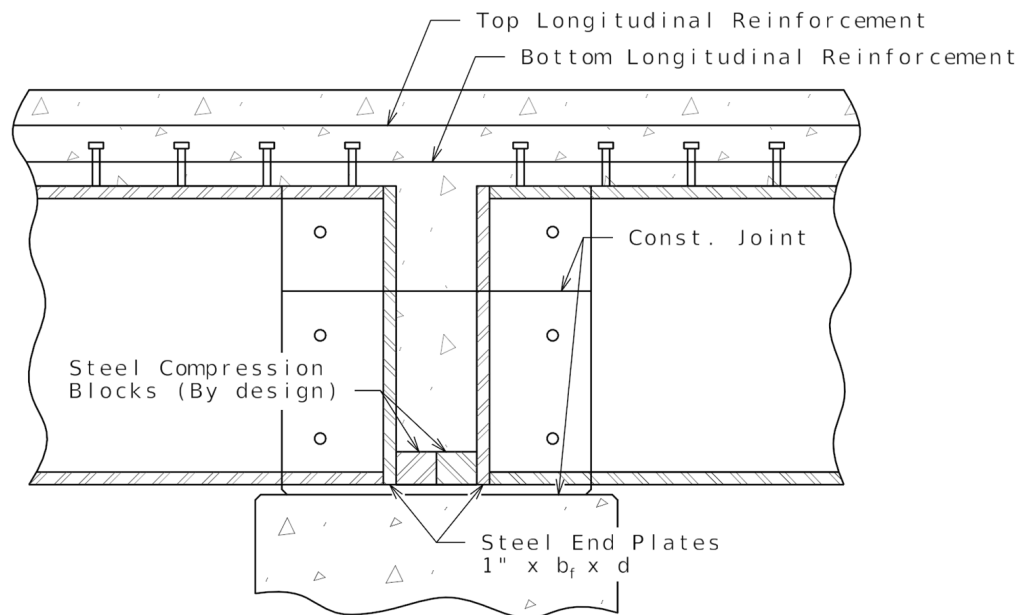
751.15.4 Details

751.15.4.1 Compression Blocks (New Design or Superstructure Replacements)

General

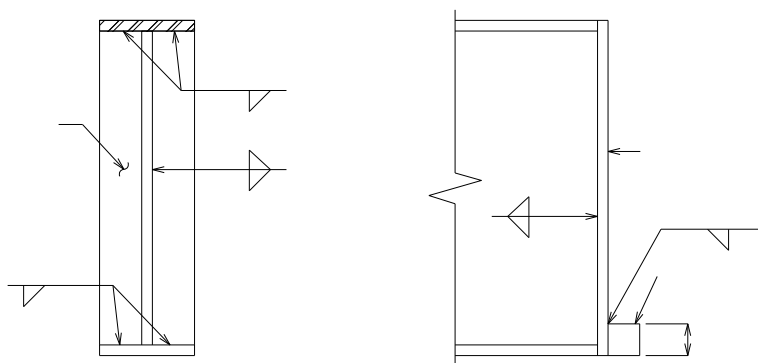
Compression block dimensions shall be determined by design, see EPG 751.15.2.3.

After beam erection, the vertical faces of the compression blocks shall be in partial-to-full contact. Full contact throughout block height is not necessary². Vertical faces of adjacent blocks shall overlap a minimum of $0.75H$.



Part Elevation

Welding



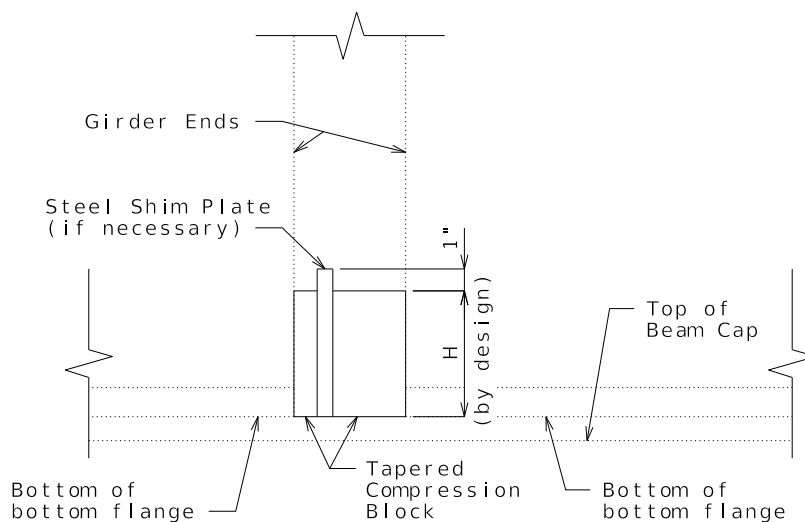
End Plate and Compression Block Welding Details

Where:

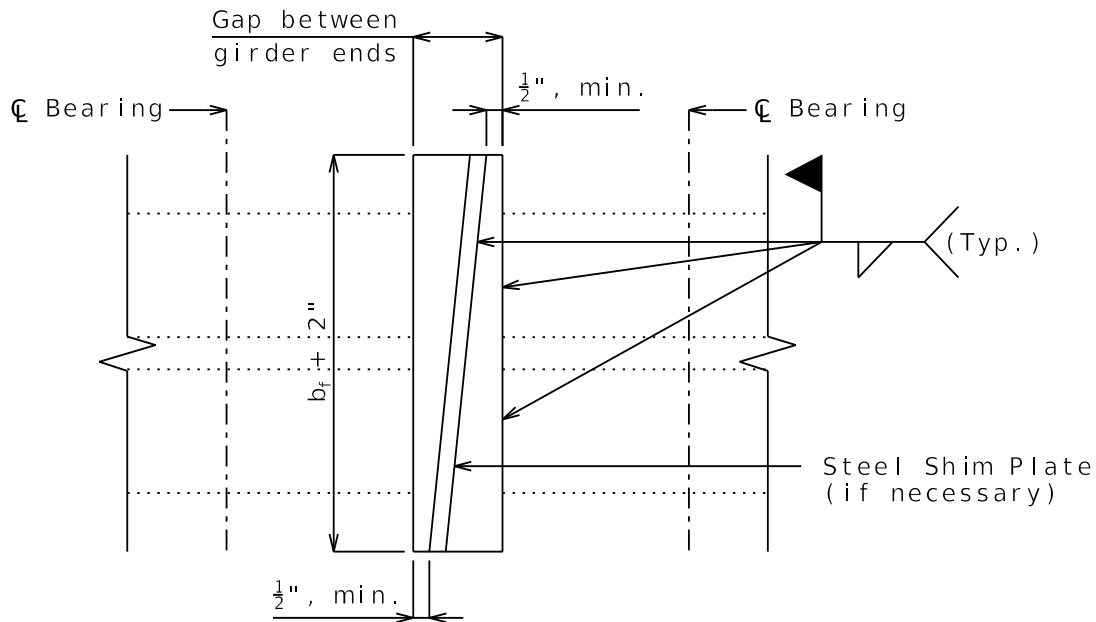
- b_f = girder bottom flange width, in.
- d = girder depth, in.
- H = compression block height measured parallel to the girder depth, in.
- t_{pl} = thickness of the compression block measured parallel to the girder length, in.

Compression Blocks for SDCL Redecking

For redecks involving conversion of a multi-simple span structure to continuous using SDCL, the compression block design shall follow EPG 751.15.2.3 with modifications to the block dimensions as shown using tapered blocks:



Tapered Compression Block and Shim Plate Part Elevation



Tapered Compression Block and Shim Plate Part Plan

The material of the tapered compression blocks and shim plate shall be in accordance with EPG 751.15.1.1. The shim plates shall not be less than 1/4" in thickness for welding.

The tapered block dimensions are based on design requirements and existing plan dimensions at the Intermediate Bent(s). Prior to fabrication, the contractor shall measure each gap at all beams and make necessary adjustments.

Field Adjustment of Blocks for Varying Gap Widths

For narrower gaps, the tapered blocks can be uniformly slid away from each other provided that the narrow end of the tapered block extends past the web by a minimum of 0.4 times the larger bottom flange width. If the design criteria per EPG 751.15.1.1 are met, the contractor may have the option to reduce the thicker end dimension to allow the narrower end of the tapered block to extend as required beyond the web.

For wider gaps, the tapered blocks can be uniformly slid toward each other provided the thicker end of the tapered block extends past the web by a minimum of 0.4 times the larger bottom flange width. If the design criteria per EPG 751.15.1.1 are met, the contractor may have the option of adjusting the tapered block dimensions or using a shim plate of uniform thickness between the tapered blocks.

751.15.4.2 Closed Concrete Intermediate Diaphragms

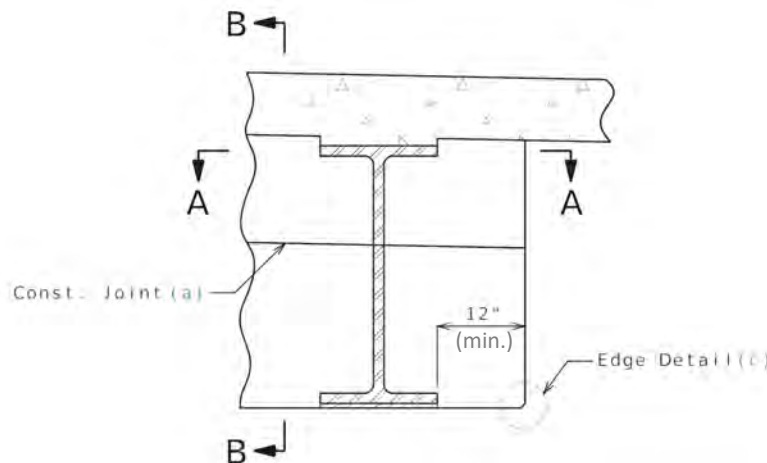
Concrete intermediate diaphragms are cast between the ends of SDCL steel girders at interior supports and make the girders continuous for loads applied after continuity is established.

The diaphragm at bents shall be cast to $\frac{2}{3}$ of its full depth to ensure stability of the girders and reduce unintended moment continuity and cracking during deck pours². The concrete diaphragm shall be poured a minimum of 12 hours before the slab and remainder of the diaphragm is poured.

Reinforcement guidance details for skewed bents are shown below and are also applicable to squared bents. Refer to EPG 751.22.3.7 for details not shown.

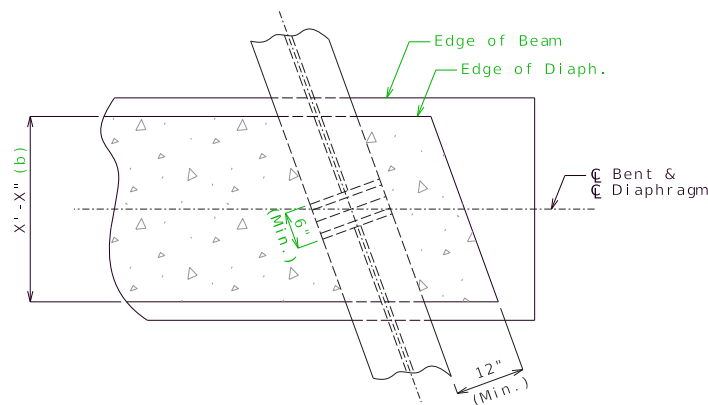
751.15.4.2.1 Dimensions

The following dimensions shall be detailed on the plan sheets for closed concrete intermediate diaphragms.

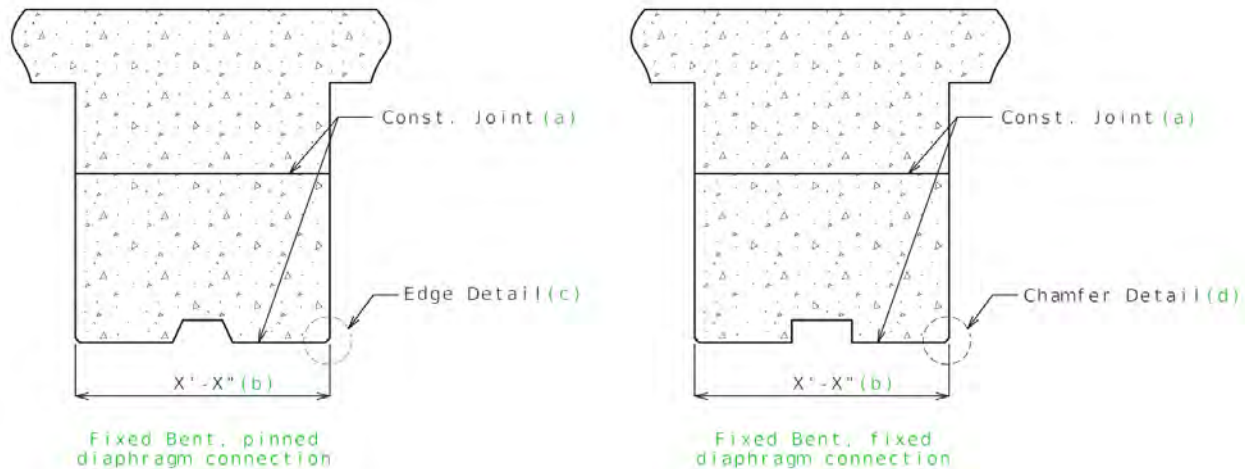


SECTION NEAR DIAPHRAGM

(Normal to centerline of girders)



Section A-A



Section B-B (Normal)

Detailing Guidance:

Green items are guidance and shall not be shown on the plans.

- (a) Refer to EPG 751.15.4.2.2.
- (b) Diaphragm shall be 2'-6" wide unless skew requires a wider diaphragm.
- (c) Refer to EPG 751.22.3.7.3 Edge Detail for Fixed Bents.
- (d) Refer to EPG 751.35.3.4 for Chamfer Detail.

751.15.4.2.2 Reinforcement

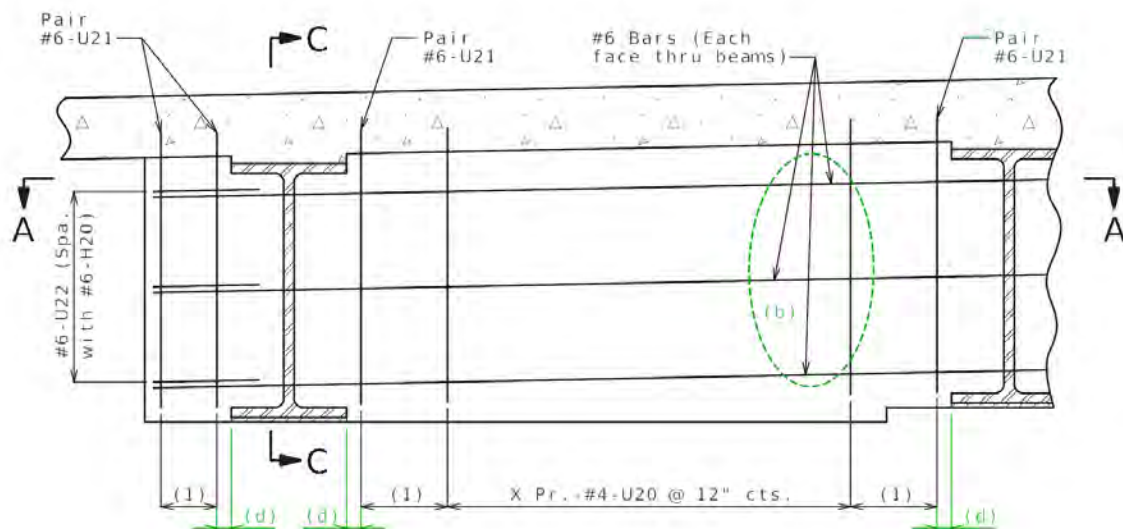
The reinforcement shall be detailed on the plan sheets for closed concrete intermediate diaphragms as shown below except:

- Bar marks revised as required.
- Abbreviations used as required.
- Add "(Typ.)" to dimensions and leader notes as appropriate.

All U bar reinforcement shall use stirrup bends.

All reinforcement in diaphragms shall be epoxy coated. Coating of dowel bars shall match the coating of intermediate bent reinforcement. Intermediate bent reinforcement shall be epoxy coated in the case that reinforcement from the intermediate bent (except dowel bars) extends into the diaphragm.

The details shown are for the same girder heights within continuous girder series and diaphragms not constructed integral with the bent cap (no reinforcement extending from cap into diaphragm). For diaphragms constructed integral with the bent cap (reinforcement extending from cap into diaphragm), refer to EPG 751.35.

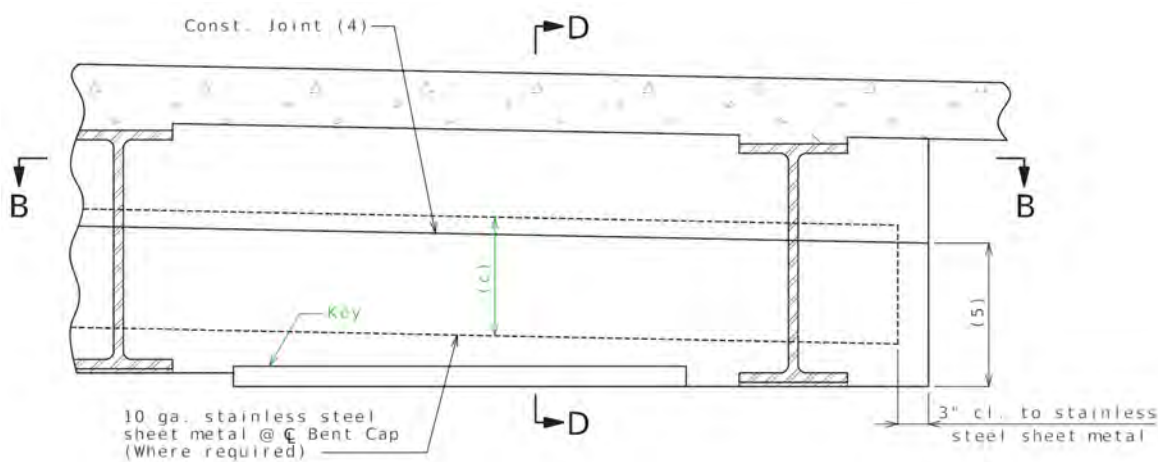


SHOWING REINFORCEMENT

SECTION NEAR DIAPHRAGM

Keys not shown for clarity.

(Normal to centerline of girders)

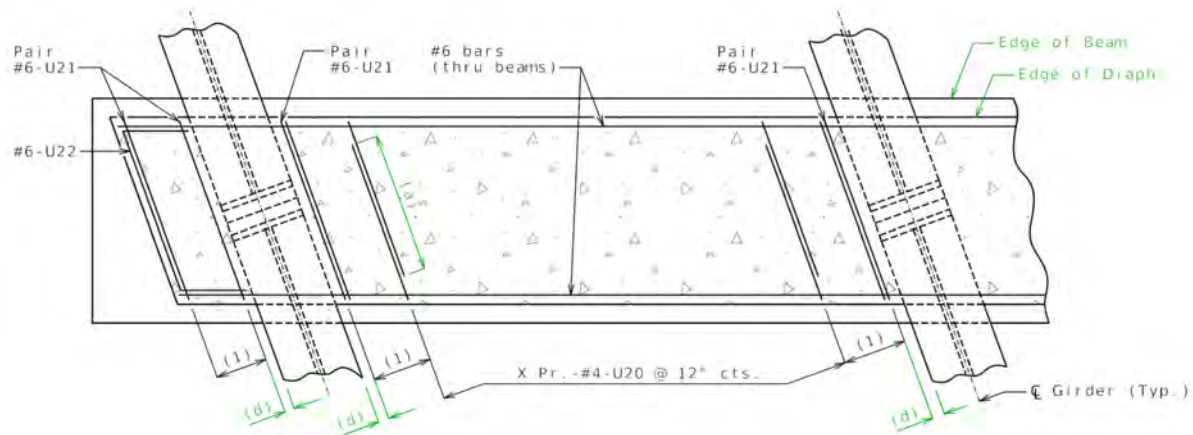


SHOWING SHEET METAL

SECTION NEAR DIAPHRAGM

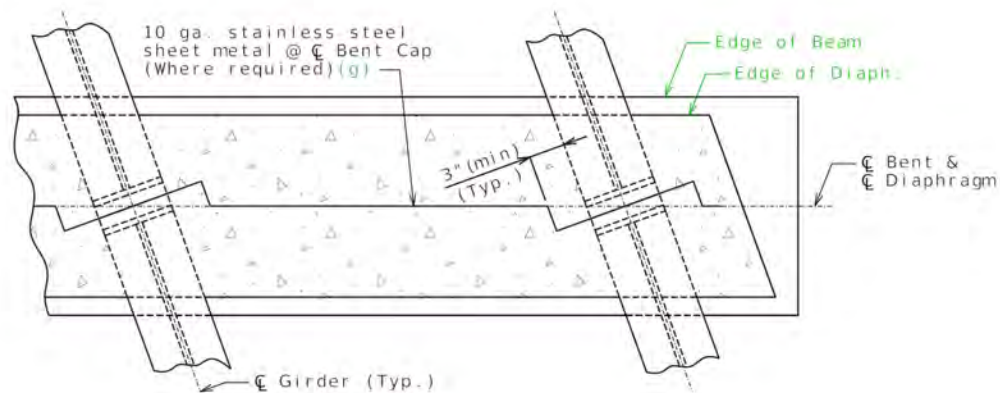
(Normal to centerline of girders)

The sheet metal shall be included in the diaphragm in regions with salt exposure at grade separations or more publicly visible areas to help control cracking. The sheet metal plate may be omitted in areas such as stream crossings that are not visible to the traveling public as the potential for uncontrolled cracking may be higher. Research shows inclusion of the sheet metal does not significantly reduce the degree of continuity in the diaphragm during deck placement.



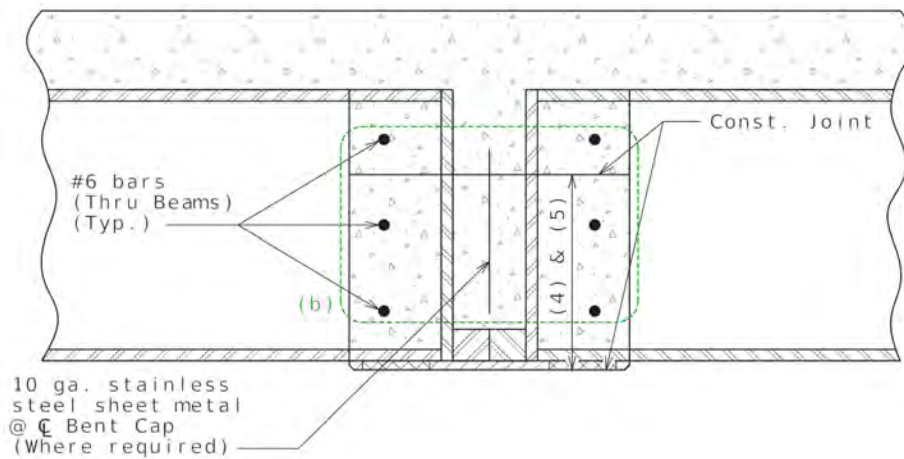
SECTION A-A

(Skewed diaphragm shown, square diaphragm similar)

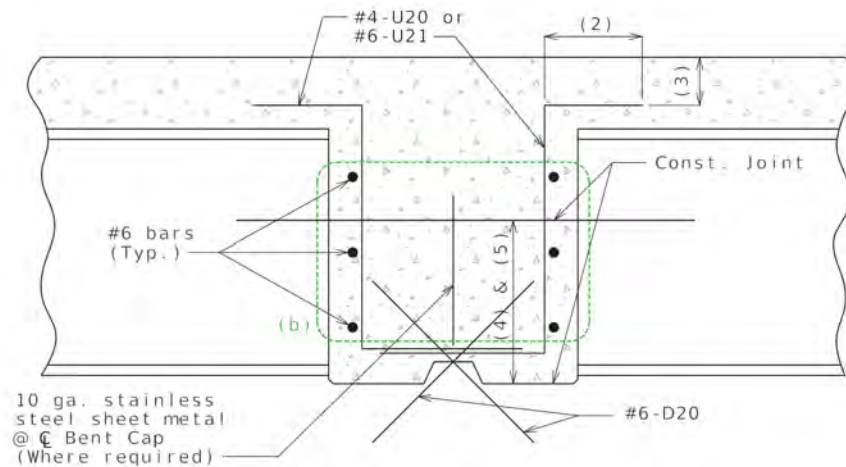


SECTION B-B

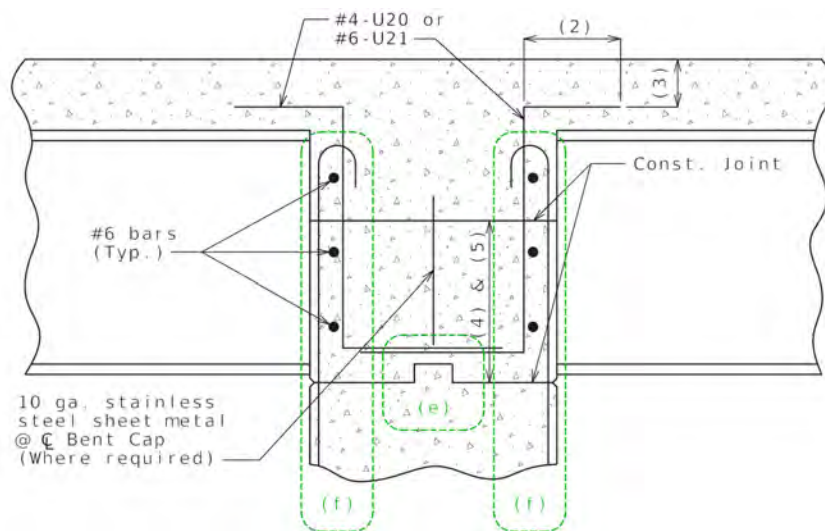
(Skewed diaphragm shown, square diaphragm similar)



SECTION C-C



SECTION D-D



Alternate Section D-D for Fixed Diaphragm Connections

Detailing Guidance:

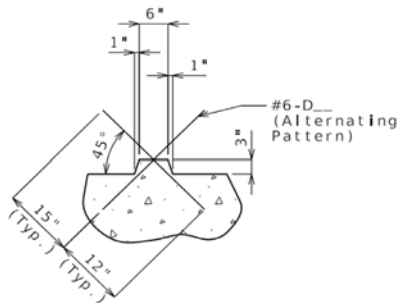
Green items are guidance and shall not be shown on the plans.

- | | |
|--|---|
| <ul style="list-style-type: none"> (a) 19 inches minimum for #4 bars and full available width for #6 bars. (b) Equally spaced, see EPG 751.15.4.3 for the reinforcing hole spacing through the girder web. (c) The 10 ga. stainless steel sheet metal plate, where required, shall extend from the top of the bottom leg of the #6-U21 bar to 1" above the construction joint. (d) Keep 1½" min. between the beams and the #6-U bars. (e) Refer to EPG 751.35.3.4 for Section Thru Key. (f) Refer to EPG 751.35.4 for reinforcement details and detailing guidance. (g) See 751.50 Standard Detailing Notes for note (H1.54) required for the stainless steel sheet metal plates. | <ul style="list-style-type: none"> (1) X"(6" min. & 12" max.) (2) 15" (#4) & 22" (#6) (3) 3½" (#4) & 3" (#6) cl. (4) Varies, match roadway cross slope. (5) Diaphragms at bents shall be poured X"min. (2/3 of total superstructure depth) from top of bent cap after all girders are in place. The remainder of the diaphragm shall be poured concurrently with the slab. |
|--|---|

751.15.4.2.3 Bent Details

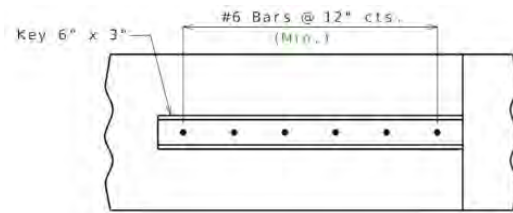
Dowel Bars and Keys

Dowel bars are by design. Where the stainless-steel plate is omitted, a straight bar can be used in lieu of the alternating X-pattern bars shown below in the "Section Thru Key" for pinned diaphragms. For details of dowel bars and keys where the stainless-steel sheet metal plate is omitted, refer to EPG 751.22.3.7.4.



Section Thru Key

(For pinned diaphragms with the stainless steel sheet metal plate)



Plan of Beam Showing Reinforcement

Joint Filler and Bearings

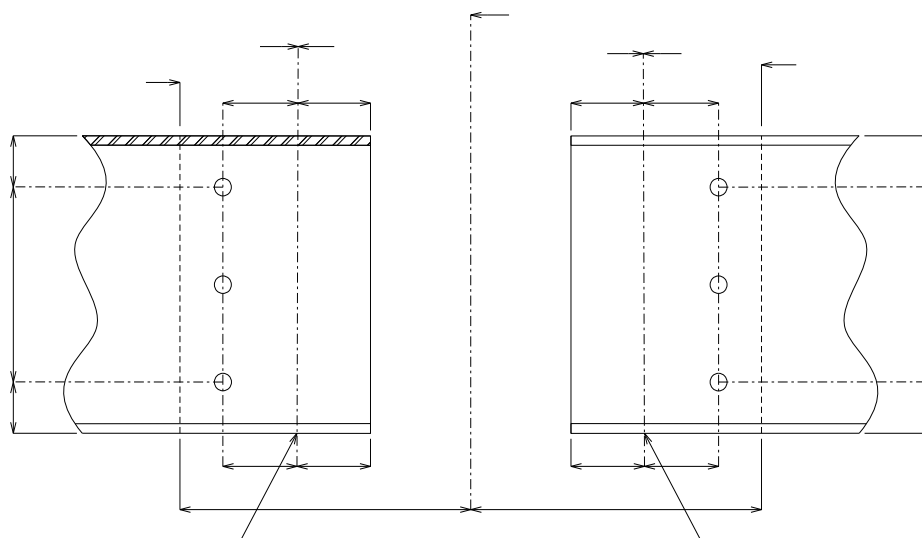
The joint filler required for closed concrete intermediate diaphragms shall be shown in the plan of beam along with the bearings. No expansion bents nor girder chairs shall be used.

- At pinned diaphragm connections:



- (a) "W" is width of bearing. Bearing length and thickness is by design.
- (b) Remove thickness for tapered bearings.
- (c) 3 3/4" minimum. With the approval of the SPM, the diaphragm may be made flush with substructure beams not greater than three feet wide.

751.15.4.3 Reinforcing Holes



Section at End of Beams at Intermediate Bents

(End plates and steel compression blocks omitted for clarity)

- (1) 1" plus half of bearing pad length for bearing pad
- (2) 1 1/16" θ holes for skews thru 20°. For skews > 20°, use slotted holes = 1 1/16" + 2(web thickness) x (tan of the skew angle)
- (3) Line perpendicular to the bottom of the bottom flange at the intersection of CL Brg. and Point of Rotation.

For section at end of beam for integral end bents, refer to EPG 751.35.5.1 Reinforcing Holes.

WF Beam Depth	Stud Spacing *	A	Reinforcement Hole Spacing
12	1 spa. @ 4"	4"	1 space
14	1 spa. @ 6"	4"	1 space
16	2 spa. @ 4"	4"	2 equal spaces
18	2 spa. @ 5 1/2"	4"	2 equal spaces
21	2 spa. @ 4 1/2"	4"	2 equal spaces
24	2 spa. @ 6"	4"	2 equal spaces
27	2 spa. @ 7 1/2"	4 1/2"	2 equal spaces
30	3 spa. @ 6"	4 1/2"	3 equal spaces
33	3 spa. @ 7"	4 1/2"	3 equal spaces
36	4 spa. @ 6"	4 1/2"	3 equal spaces

* only at end of beam at integral end bent

751.15.4.4 Int. Diaphragms and Cross Frames

Intermediate Diaphragm and Cross Frame Details**

Shape	"D" (in)	"N"	Channel, Angle or Bent Plate	"A" (in)	"B" (in)
W12X45 THRU W12X106	6	2	C6 x 8.2	Shapes too shallow to allow for 6x6x3/8 at tension flange. Welding on both flanges required.	
W14X68 THRU W14X145	7	2	C7 x 9.8		
W16X40 THRU W16X89	8	2	C8 x 11.5		
W18X46 THRU W18X106	10	3	C10 x 15.3		
W21X48 THRU W21X93	12	4	C12 x 20.7	3	1/2
W24X68 THRU W24X103	12	4	C12 x 20.7	3	1/2
W27X84 THRU W27X129	15	4	5/16 x 23	4	1 1/2
W30X90 THRU W30X148	18	5	5/16 x 26	4	1 1/2
W33X118 THRU W33X221	21	5	5/16 x 29	4	1 1/2
W36X135 THRU W36X194	21	5	5/16 x 29	4	1 1/2
W40X149 THRU W40X215	21	5	5/16 x 29	4	1 1/2
36" thru 40" Plate Girders	21	5	5/16 x 29	4	1 1/2
42" Plate Girders	27	5	5/16 x 35	4	1 1/2
> 42" Plate Girders	Diagonal Member		L3x3x5/16	4	1 1/2
	Horizontal Member	*** S < 10'-2"	L3.5x3.5x5/16		
		other	L4x4x5/16		

** Table is applicable for diaphragms and cross frames used as bracing members. For criterion used in determining members sizes see the footnotes for the "Bottom Horizontal Members for Web Depths" and "Diagonal Members for Web Depths > 48" tables for End Diaphragms.

*** S = girder spacing along diaphragm skew

Diaphragms and cross frames used as primary members must be checked for torsion effects. (See Analysis of Horizontally Curved Girders)

ATTENTION DETAILER:

Intermediate diaphragm connection plates shall be detailed for Connection "A" for skews thru 20°. Details of Connection "A" are shown in EPG 751.14.5.1 Bearing Stiffeners.

Remainder of section same as 751.14.5.2, refer to the *Omissions* section of the document.

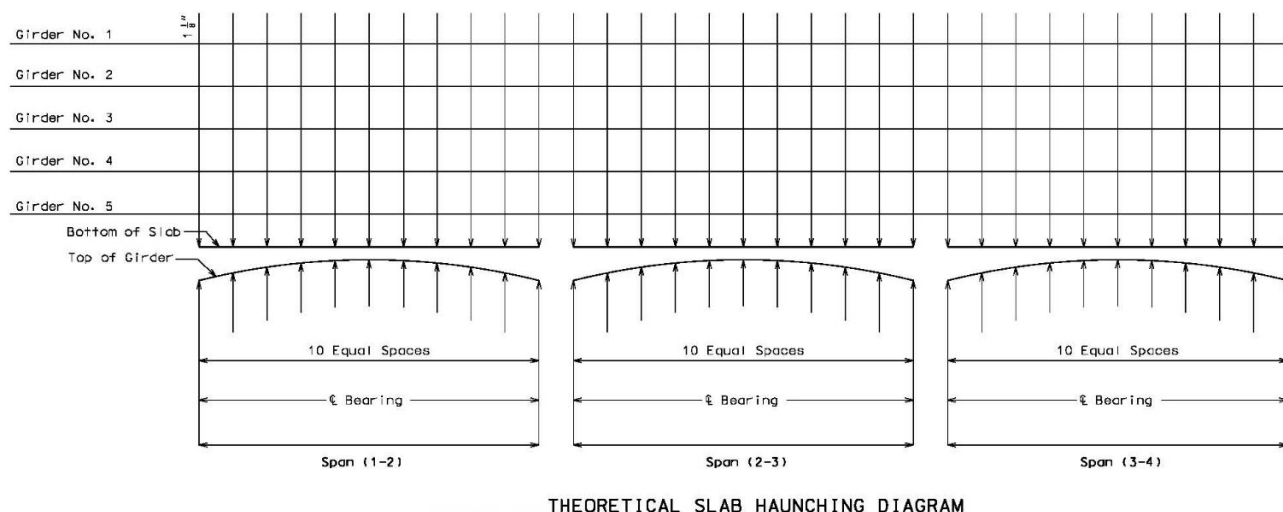
751.15.4.5 Other

For details of: transverse stiffeners, longitudinal stiffeners, lateral bracing, protecting coating requirements, weathering steel, drip bar guidance, and weld design/detail guidance, refer to EPG 751.14.5.

751.15.5 Miscellaneous Details

751.15.5.1 Girder Haunch

When variable haunching is used in lieu of cambering, the following detail shall be modified as needed and shown on the plans. There is no maximum haunch for full depth cast-in-place decks using conventional or SIP forms, but additional hairpin bars are required in accordance with EPG 751.10.1.14 Girder and Beam Haunch Reinforcement.



Remainder of section same as 751.14.6.1.

751.15.5.2 Girder Camber and Dead Load Deflection

Cambering of wide flange beams shall be utilized over roadways with a non-composite dead load deflection greater than 1". The variable haunch can be used over creek crossings for simplification and ease of girder fabrication with less than 4" of dead load deflection.

All plate girders shall be cambered.

Remainder of section same as 751.14.6.2.

Omissions and Changes to EPG 751.14, 751.11, and 751.10

References

- (1) Azizinamini, A., Yakel, A.J. and Farimani, M.R. (2005), *Development of a Steel Bridge System—Simple for Dead Load and Continuous for Live Load, Volume 1: Analysis and Recommendations*, Final Report NDOR 542, Nebraska Department of Roads, Lincoln, NE.
- (2) Azizinamini, A. (2014), "Simple for Dead Load–Continuous for Live Load Steel Bridge Systems," AISC, *Engineering Journal*, Second Quarter.
- (3) Taghinezhadbilondy, Ramin. 2016. "Extending Use of Simple for Dead Load and Continuous for Live Load (SDCL) Steel Bridge System to Seismic Areas." Ph.D. dissertation, Florida International University.
- (4) Sadeghnejad, A.; Mantawy, I.; Azizinamini, A. *ABC-UTC Guideline for Superstructure to Pier Connection in SDCL Steel Bridge Systems*; Technical Report; Florida International University: Miami, FL, USA, 2018
- (5) Wasserman, E. *Transportation Research Record: Journal of the Transportation Research Board, CD 11-S*, Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 197–202.
- (6) Azizinamini, Atorod. Simple for Dead Load - Continuous for Live Load Steel Bridge Systems. IBT/ABC-UTC, 9 Sept. 2025. Webinar.

Omissions and Changes to EPG 751.14, 751.11, and 751.10

751.14 Steel Superstructure

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[751.14.2.2 Analysis Methods](#)

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[751.14.2.4 End Diaphragms](#)

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751.14.1 General

751.14.1.1 Materials and Selection of Steel

General

Unit weight of reinforced concrete, $\gamma_c = 150 \text{ lb/ft}^3$

for modulus of elasticity, $E_c = 145 \text{ lb/ft}^3$

Unit weight of future wearing surface, $\gamma_{ws} = 140 \text{ lb/ft}^3$

Compressive strength of concrete, $f'_c = 4 \text{ ksi}$

Minimum yield strength of reinforcing steel, $f_y = 60 \text{ ksi}$

Unit Weight of Structural Steel, $\frac{\text{lb}}{\text{ft}^3} = 490 \text{ lbs/ft}^3$

~~When calculating the weight of splice, the following simplified weight shall be used.~~

~~Weight per bolt incl. nut (7/8" diameter ASTM F3125 Grade A325 bolt with A563 nut) = 0.95 lb~~

Steel Yield Strength (F_y) and Tensile Strength (F_u)

ASTM A709 Grade 36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$
ASTM A709 Grade 50	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$
ASTM A709 Grade 50W	$F_y = 50 \text{ ksi}$	$F_u = 70 \text{ ksi}$
ASTM A709 Grade HPS 70W*	$F_y = 70 \text{ ksi}$	$F_u = 85 \text{ ksi}$

(* See Project Manager for use of F_y greater than 50 ksi.)

Fasteners

~~Splices – 7/8" diameter ASTM F3125 Grade A325 Bolt $F_{ub} = 120 \text{ ksi}$~~

Diaphragms - 3/4" diameter ASTM F3125 Grade A325 Bolt $F_{ub} = 120 \text{ ksi}$

Selection of Steel

Welded Plate Girders:

Grade 50, Grade 50W, or HPS70W plate. Grade 36 may be used for webs in hybrid sections. Where appropriate for economy, hybrid sections are used for plate girder design, otherwise use homogeneous sections.

Wide Flange Beams:

Grade 50 or Grade 50W.

Diaphragms, cross frames, stiffeners, or other connection elements:

Grade 36, unless Grade 50 or Grade 50W is required by design

Splices:

Information for ASTM Grade 50 or Grade 50W is shown in this section. For Grade 36, designer shall design splice using LRFD procedure discussed in this section.

Shear Connectors:

AASHTO M169 (ASTMA108), $F_y = 50 \text{ ksi}$ $F_u = 60 \text{ ksi}$

751.14.1.2 Girder Limits and Preferences

There are situations to consider when deciding to choose between a wide flange or plate girder design.

- Curved structures or structures requiring severe camber, are more typical for welded plate girders.
- The designer should check the availability of standard wide flange shapes.
- Typically if wide flanges are capable for the design, a wide flange beam design will be more economical than a welded plate girder.

Maximum Section Length

Maximum girder section length shall be limited to meeting:

- Flange shipping and erection limit of $b_i \leq \frac{L}{85}$, where b_i = flange width and L = section length.
- Site accessibility restrictions (consult with District)

~~Additional provision for heat curving (Horizontally Curved Girders):~~

- During fabrication the weight of the girder should not cause a flange stress greater than 55 percent of the flange yield strength ($0.55F_y$) when the web is laid in the horizontal position. Assume the ends are simply supported. If the flange stress exceeds $0.55F_y$ then the plans shall indicate that heat curving of the girder must be performed with the web in the vertical position.

Minimum Plate Length

10 ft. Shop flange splices should be eliminated and extra plate material used when economy indicates and span lengths permit.

Minimum Radius (Horizontally Curved Girders)

Minimum radius for heat-curved girders shall be 150 ft. In addition the minimum radius is limited by the girder section properties per LRFD 6.7.7.

Stiffeners

Longitudinal stiffeners are generally not economical in spans less than 300 ft. Consult with the State Bridge Engineer prior to using longitudinal stiffeners.

The number of different sizes of transverse stiffeners should be reduced to a minimum for stiffener widths up to 8". This is because small plate materials of any thickness and up to 8" in width must be ordered by fabricators as bars in 20 ton lots for each size (A "bar" is generally a plate 8" wide or less). Also, it is recommended that changes in stiffener thickness should be kept to a minimum. Best practices suggest using transverse stiffener thicknesses with the same thickness as the intermediate diaphragm connection plates or bearing stiffeners but no less than required by design to limit the steel fabricator from having to order small quantities of bars or plates.

Web Depth

It is preferred to use web depths in 6" increments. Other increments may be used when required by the Design Layout (See Structural Project Manager). Note that Standard Web Splice Tables shown are provided in 2" web depth increments.

For girders analyzed as horizontally curved (see Analysis of Horizontally Curved Girders), the girder depth shall be limited in relation to the span length as follows:

$$D \geq \frac{L_{as}}{25}$$

Where:

D = steel girder depth

L_{as} = $C \cdot$ (Arc span length)

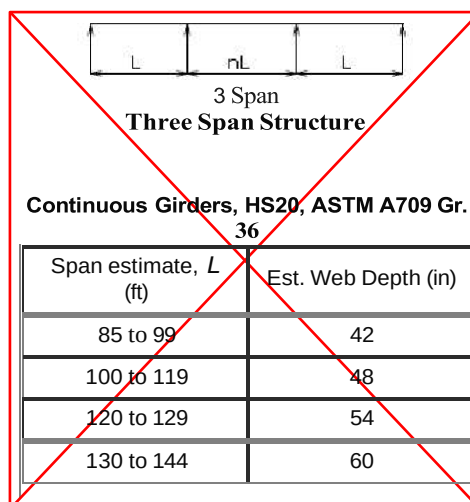
Where:

$C = 1.0$... simple spans:

$C = 0.9$... end continuous spans:

$C = 0.8$... interior continuous spans:

In the absence of LRFD-specific data, the following information is provided. Three spans with ratio $n = 1.3 \pm$ and HS20 live load were considered.



3 Span
Three Span Structure

Continuous Girders, HS20, ASTM A709 Gr.

36

Span estimate, L (ft)	Est. Web Depth (in)
85 to 99	42
100 to 119	48
120 to 129	54
130 to 144	60

145 to 154	66
155 to 169	72
170 to 179	78
180 to 189	84
190 to 200	90

Continuous Girders, H520, ASTM A709 Gr. 50

Span estimate, L (ft)	Web Depth (in)
85 to 104	42
105 to 124	48
125 to 134	54
135 to 144	60
145 to 159	66
160 to 174	72
175 to 179	78
185 to 189	84
195 to 204	90

Web thickness

Minimum web thickness for plate girders = $1/2$ "

Web thickness increment is $1/16$ inch.

Transverse stiffeners may be omitted when indicated by design and economy. A cost comparison should be made based on current average bid prices that may be obtained from the Structural Project Manager for comparable bridges.

It will usually be economical to eliminate transverse stiffeners for shallow webs (36" to 42"). For webs deeper than 42 in., a general rule of thumb is to determine the minimum web thickness without stiffeners; then, use a web thickness $1/16$ inch less.

Flanges

Minimum flange dimensions are $3/4$ " x 12". For shipping and erection purposes, minimum width of both compression and tension flanges shall not be less than $L/85$, where L is the shipping length of the girder section. This limitation is to prevent out-of-plane distortion of the girder.

Maximum flange thickness = 4", except for HPS70W limitations ~~and curved girders with radii less than 1000 feet.~~

Flange thickness increment is $1/8$ inch. Flange width increment is 1 inch.

~~For radii less than 1000 ft. the following dimensions shall not be exceeded:~~

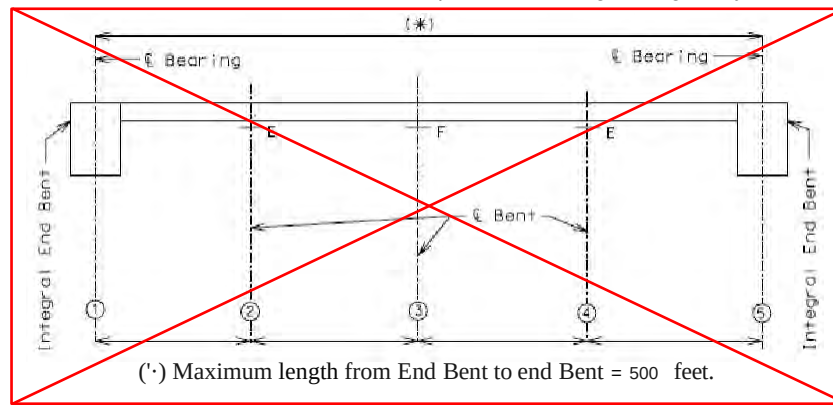
~~Flange Thickness = 3"~~

~~Flange Width = 30"~~

~~Note that flange transitions may be economical if the flange thickness is constant and the flange width is varied since the flanges could be cut from the same plate material. If the flange width is held constant and the flange thickness is varied, then economy could result if welded butt splices were permitted.~~

Structure Length

Typical Continuous Steel Structures - Integral End Bents:



751.14.1.3 Optimizing Girder Spacing on Non-Standard Roadways

Several items shall be considered when determining the girder spacing for non-standard roadways:

General Guidelines

In general, an economical girder arrangement is one that:

Balance M & V Distribution Factors. For efficiency, it is desired for all girders to be designed for the same loads, resulting in the same section for exterior and interior girders.

Practical Constraints

Some practical constraints to consider when optimizing girder spacing:

Number of girders. For consideration of future stage construction, there may be a minimum number of girders that should be considered.

Maximum Form Span. Precast panels and transparent forms have maximum span limits that should not be exceeded. For cases where conventional forms or corrugated steel forms are used, it is generally recognized that 12 ft girder spacing is a maximum practical limit to allow for future redecking.

Vertical Clearance and Deflection Requirements. It is expected that fewer girder lines would result in deeper girders. Vertical clearance or deflection constraints may dictate the use of more girder lines than optimum, or a shallower web than optimum.

Slab Overhang Limits. In order to use distribution factors provided in LRFD Table 4.6.2.2.2 for girder design, the roadway overhang shall not exceed AASHTO's specified limits, otherwise the structure shall be analyzed using a refined method of analysis specified in LRFD 4.6.3.

751.14.2 Design

751.14.2.1 Limit States and Load Factors

In general, each component shall satisfy the following equation:

$$Q = \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r$$

Where:

Q = Total factored force effect

Q_i = Force effect

f_i = Load modifier

γ_i = Load factor

φ = Resistance factor

R_n = Nominal resistance

R_r = Factored resistance

Limit States

The following limit states shall be considered for steel superstructure design:

STRENGTH-I
STRENGTH - III
STRENGTH - IV(*)
STRENGTH-V

SERVICE-II FATIGUE

For constant amplitude fatigue threshold (ϕF)_{TH}, see LRFD Table 6.6.1.2.3-1 for applicable detail. For base metal, except noncoated weathering steel, detail category "A" (ϕF)_{TH} = 24 ksi and for base metal, noncoated weathering steel, detail category "B" (ϕF)_{TH} = 16 ksi.

Combination for loads during construction and construction loads shall be evaluated at the Strength Limit State and Service Limit State in accordance with EPG 751.2.3.2 (EPG Load Combinations) LRFD 3-4.2.

(*) Use for bridges with high DL to LL ratios. Typically, bridges with spans > 200 ft.

See LRFD Table 3-4.1-1 and LRFD 3-4.2 for Loads and Load Factors applied at each given limit state.

Resistance factors

STRENGTH limit states, see LRFD Article 6.5.4.2

For all other limit states, $\phi_i = 1.00$

751.14.2.2 Analysis Methods

MoDOT office practice generally utilizes the following:

The elastic stress at any location on the composite section due to the applied loads shall be the sum of the stresses caused by the loads applied separately at the following three stages:

Non-Composite Stage - The dead load of slab and haunching and beam self weight shall be analyzed at this stage. This stage shall also be used for any construction loading checks. The section properties used are of the steel section only.

Long Term Composite Stage - The dead load of barrier or railing, future wearing surface and any other appurtenances shall be analyzed at this stage. The section properties used are of the composite section of slab and beam assuming an elastic modulus of $3n$ for the slab. Where "n" is the modular ratio.

Short Term Composite Stage - Any live loading shall be analyzed at this stage. The section properties used are of the composite section of slab and beam assuming an elastic modulus of "n" for the slab. Where "n" is the modular ratio.

Composite regions shall be defined as regions where shear connectors are used to connect the steel section to a concrete deck. Simple for Dead Load and Continuous for Live Load (SDCL) structures are designed as composite throughout.

Tension Flanges with Holes

Compact Design is not allowed in composite positive flexure regions where holes are located in the tension flange. Similarly, in negative flexure regions where holes are present in the tension flange the girders shall not be proportioned with compact or noncompact webs in accordance with LRFD Appendix A6. Girders with holes in the tension flange shall further meet the requirements of LRFD 6.10.1.8 Tension Flanges with Holes.

Wind

For girder and intermediate diaphragm and cross-frame design, wind acting on the top half of girder is distributed to deck and wind on the lower half is assumed to be carried by the bottom girder flange.

~~Analysis of Horizontally Curved Girders~~

~~Horizontally curved girders may be analyzed as straight girders if the following criteria are met:~~

- ~~■ Girders are concentric~~
- ~~■ Bent skews $\leq 10^\circ$~~
- ~~■ Similar girder stiffness throughout bridge width~~
- ~~■ $L_{Ras} < 0.06$ for all spans~~

~~Where:~~

~~R = girder radius~~

~~$L_{as} = C * (\text{Arc span length})$~~

~~Where:~~

C = 1.0... simple spans:

C = 0.9... end continuous spans:

C = 0.8... interior continuous spans:

For straight girder analysis:

- Use the arc length as the span length
- Diaphragms and cross-frames treated as bracing members
- Shear connectors are omitted in negative moment regions

For horizontally curved girder analysis:

- Overturning (load-shifting) effects must be considered to contribute to the major-axis bending.
- Diaphragms and cross frames shall be designed as primary members.
- Design diaphragm connections to transfer forces to flanges.
- Shear connectors provided throughout girder length
- Lateral flange bending effects should be considered in the girder, shear connector, and flange splice design.

Note: Lateral flange bending stresses may be superimposed with stresses resulting from major-axis bending.

751.14.2.3 Intermediate Diaphragms and Cross Frames

Cross frames are located at all intermediate bents with continuous slabs.

Diaphragms or cross-frames for rolled beams shall be at least $\frac{1}{2}$ the beam depth.

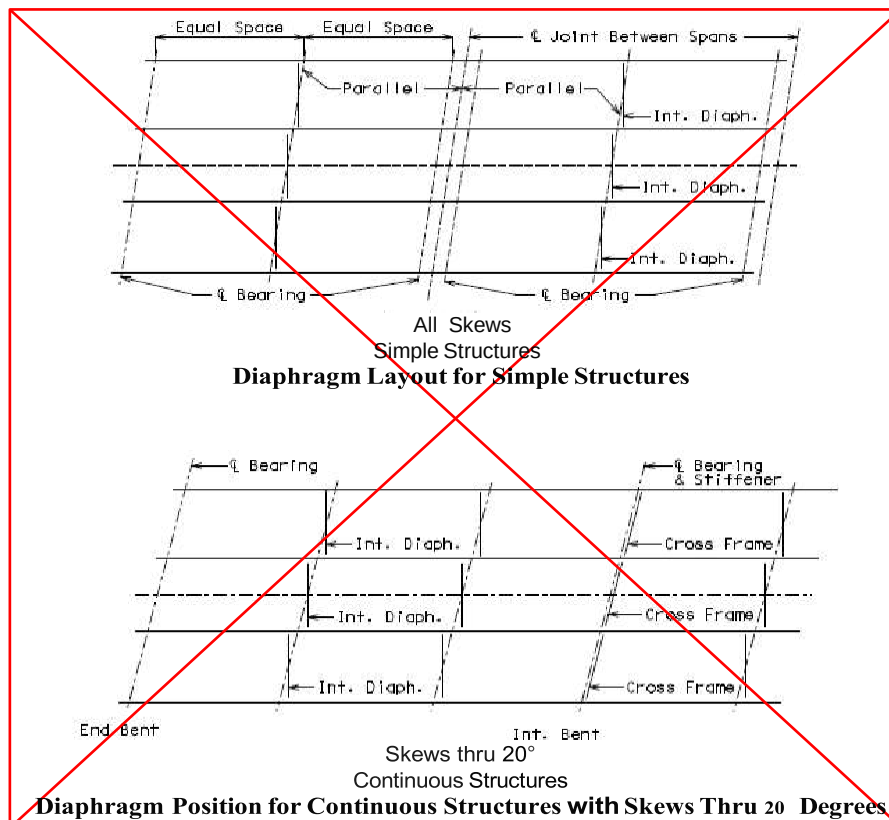
Girders shall be investigated for the effects of diaphragms or cross-frames for all stages of assumed construction procedures and the final condition.

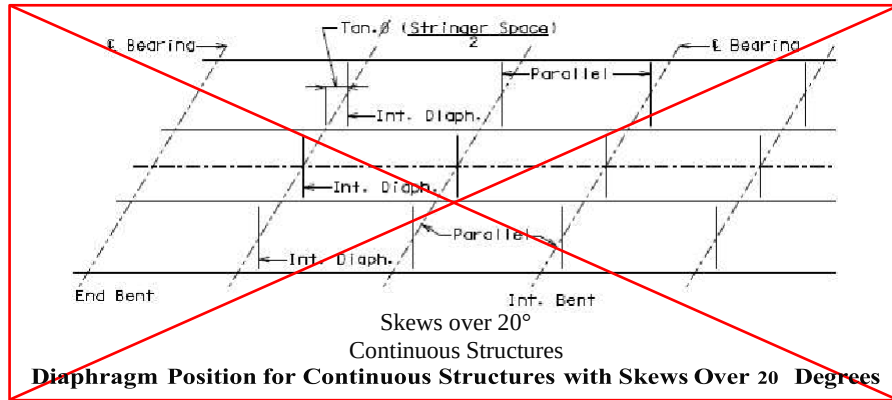
Investigation shall include, but not be limited to:

- Transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings.
- Stability of the bottom flange for all loads when it is in compression
- Stability of the top flange in compression prior to curing of the deck

Straight Girder Layout

Typically the maximum intermediate diaphragm spacing shall be 25'-0", unless determined otherwise for special designs.



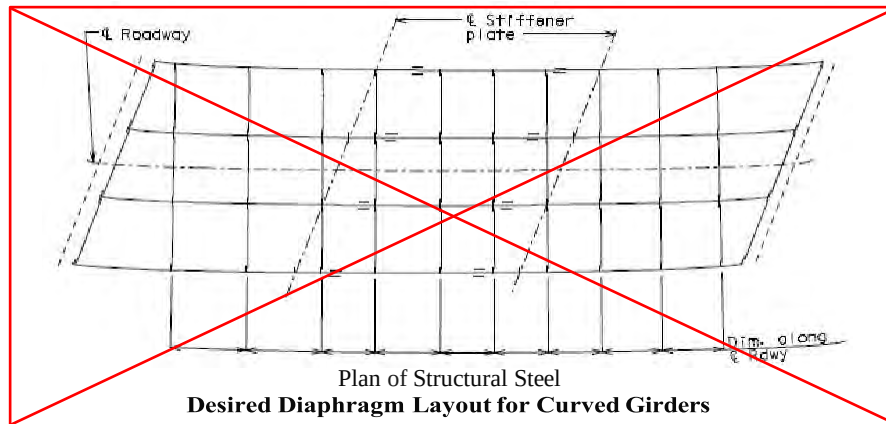


Curved Girder Layout (see Analysis of Horizontally Curved Girders)

Maximum intermediate diaphragm spacing shall be 25'-0", unless a lesser spacing is warranted by design. A lesser spacing may be warranted to satisfy LRFD Eq. 6.7.4.2-1 or to reduce lateral flange bending stresses.

~~Diaphragms shall be positioned radially and in line, except in bridges having extreme skews. Many different diaphragm spacing arrangements are possible. Attach diaphragm to a bearing stiffener where possible. The proposed diaphragm layout shall be reviewed with the Structural Project Manager prior to detailing on the plans.~~

A sketch of the desired diaphragm layout is given below.



751.14.2.4 End Diaphragms

General

Steel end diaphragms shall be located at all non-integral end bents ~~and intermediate bents~~ where continuity of the slab is broken (i.e. expansion joints). Integral end bent shall use a concrete diaphragm

Steel diaphragms for rolled beams shall be at least $\frac{1}{2}$ the beam depth.

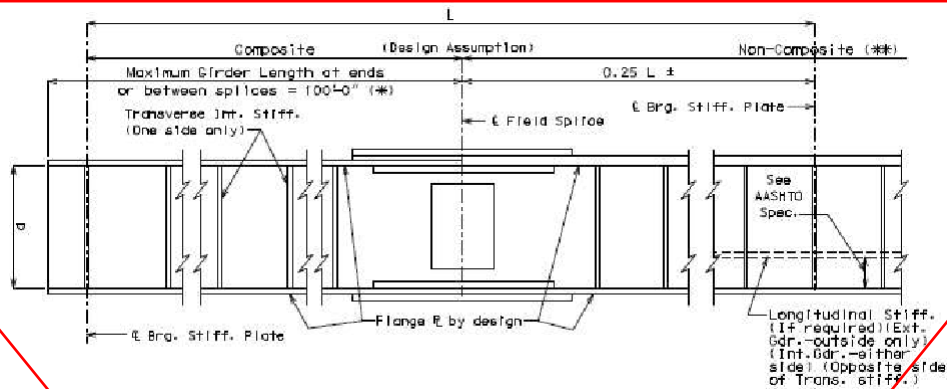
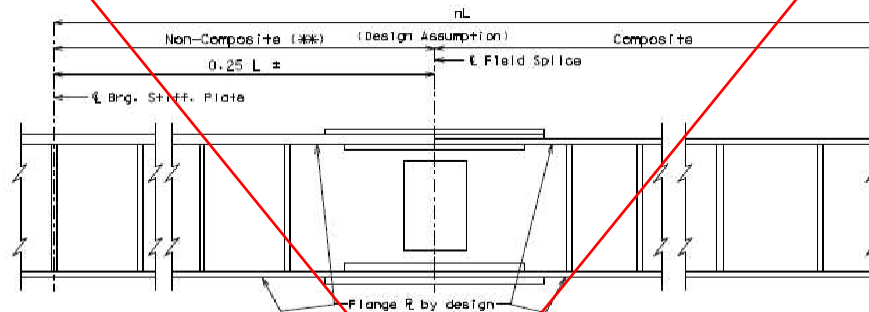
Steel diaphragms shall be investigated for all stages of assumed construction procedures and the final condition. Investigation shall include, but not be limited to:

- Transfer of lateral wind loads from the bottom of the girder to the deck and from the deck to the bearings.
- Stability of the bottom flange for all loads when it is in compression
- Stability of the top flange in compression prior to curing of the deck
- Distribution of vertical dead and live loads applied to the structure. (*)

(*) At the end of the bridge and intermediate points where the continuity of the slab is broken, the edges of the slab shall be supported by diaphragms or other suitable means as specified in LRFD 9.4.4. Top horizontal members shall be designed for vertical live and dead loads.

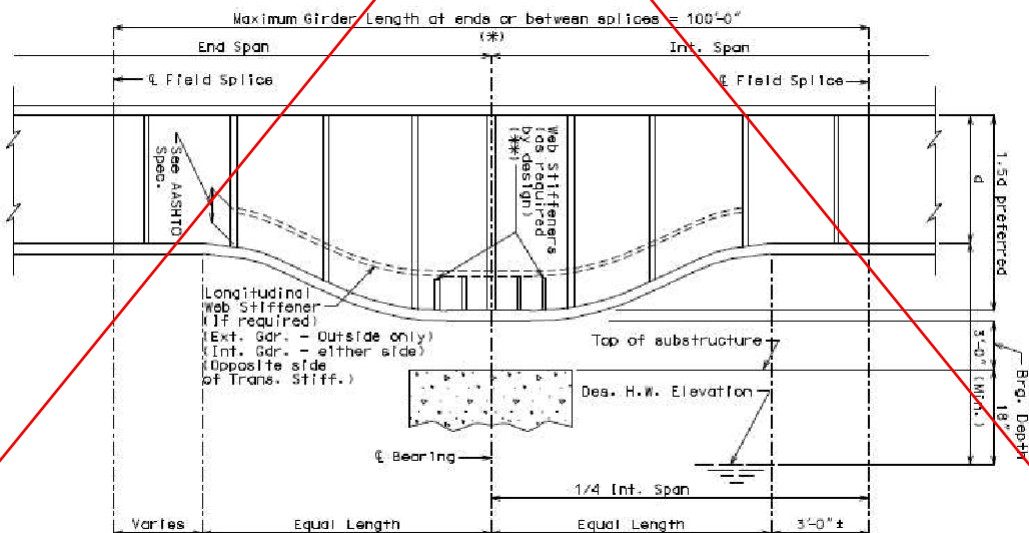
~~751.14.2.5 Part Elevation of Girders~~

~~Part Elevation of Constant Depth Girder~~

**Part Elevation (End Span)****Part Elevation (Interior Span)**

(*) See Structural Project Manager for approval to deviate from the Maximum Girder Length of plate girder sections.
 (**) Composite throughout for horizontally curved girders (see Analysis of Horizontally Curved Girders)

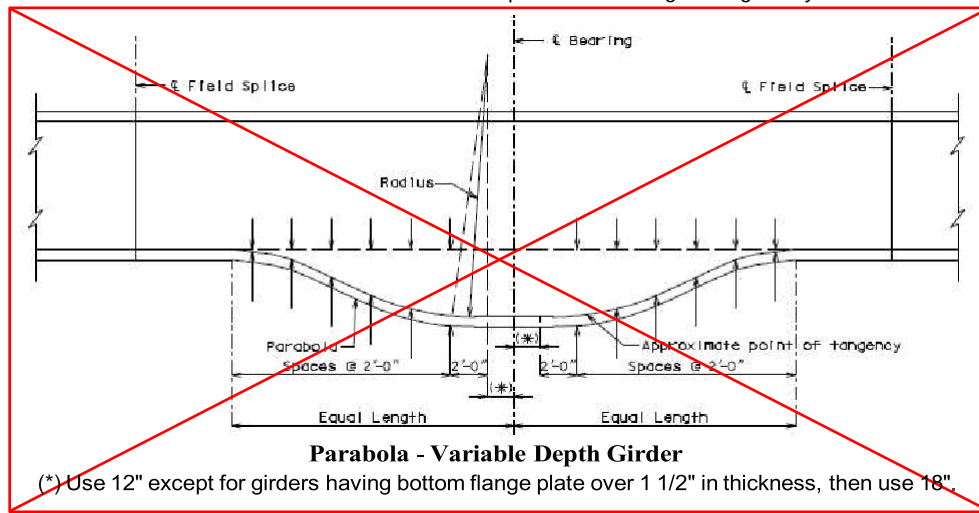
Part Elevation of Variable Depth Girder - Plate Girder Only

**Part Elevation**

Use Variable depth section only as specified on Design Layout Sheet. Consult the Structural Project Manager for consideration of constant depth section in lieu of variable depth section based on design references or structural adequacy.

(*) See Structural Project Manager for approval to deviate from the Maximum Girder Length of plate girder sections.

(**) See Design of Welded Structures by The James F. Lincoln Arc Welding Foundation, page 5.11-19.



751.14.2.6 Other Requirements

Deflection

See EPG 751.2-4.2 for Live Load deflection criteria.

Compute at 1/4 points for bridges with spans < 75 ft, Compute at 1/10 points for spans ≥ 75 ft.

Camber in Horizontally Curved Girders

Rolled Beams are required to be heat curved to develop a horizontal curvature. Plate girders may either be heat curved or the flanges cut to curvature.

Per LRFD 6.7.7.3 heat curving causes inherent residual stresses that result in camber losses during construction and for several months of in-service use. It is not required that these camber losses be accounted for in the camber profile. Industry leaders recognize that the calculation offered by LRFD 6.7.7.3 is theoretical and often inaccurate.

Minimum Negative Flexure Deck Reinforcement

See EPG 751.10.1.5

Bearing Stiffeners

- Bearing stiffener width shall be given in 1/2" increments and shall extend to within a 1/2" of the bottom flange.
- Bearing stiffener thickness shall be given in 1/8" increments.
- If the skewed stiffener option is used, make stiffeners on both sides of web for skews thru 45° the same size as the larger except in cases where overhang would be produced. This does not apply to end bearing stiffeners for skews over 45°.

Diaphragms and Cross-Frames

Diaphragms and cross-frames and their connections shall:

- Meet all applicable limit states for the calculated force effects.
- Follow LRFD 4.6.2.7 for transfer of wind loads
- Meet slenderness requirements of LRFD 6.8.4 or 6.9.3
- Meet connection plate design requirements of LRFD 6.6.1.3.1
- Be designed for stability of top flange in compression during noncomposite stage, and bottom flange for all loads when in compression.

~~Top horizontal members in end diaphragms shall be designed for vertical live load and dead loads.~~

Cross-frames or diaphragms are either considered as bracing members or primary members as discussed in the section "Analysis of Horizontally Curved Girders"

Primary Members

When diaphragms are required to resist torsional forces, such as for horizontally curved girders or straight girders with highly skewed bents, they shall be designed as primary or main members. Primary members require that forces be computed and considered in the design of both the members and their connections to the girder.

Load Rating Check

The final girder sections and steel layout should be adequate for both the design and rating analysis. It is recommended to perform the load rating before finalizing the girder design.

751.14.3 Splice Design

751.14.3.1 Field Bolted Splice Design

General

In continuous spans, splices should be made at or near points of dead load contraflexure excluding future wearing surface. Web and Flange splices in areas of stress reversal shall be investigated for both positive and negative flexure.

In both web and flange splices, there shall not be less than two rows of bolts on each side of the joint. Oversize or slotted holes shall not be used in either the member or the splice plates at bolted splices.

Bolted splices for flexural members shall be designed using slip-critical connections as specified in LRFD 6.13.2.1.1. The connections shall also be proportioned to prevent slip during the erection of the steel and during the casting of the concrete deck.

The tensile stress on the flange should be checked per LRFD 6.10.1.8.

In bolted slip-critical connections subject to shear, the load is transferred between the connected parts by friction up to a certain level of force that is dependent upon the total clamping force on the faying surfaces and the coefficient of friction of the faying surfaces. As loading is increased to a level in excess of the frictional resistance between the faying surfaces, slip occurs, but failure in the sense does not occur. As a result, slip critical connections are able to resist even greater loads by shear and bearing against the connected material. The strength of the connection is not related to the slip load. Slip resistance, shear resistance and bending shall be computed separately. Any potential greater resistance due to combined effect is ignored.

Bolt Design

Slip Resistance

Splices shall be designed as slip critical connections with Class B surface preparation and standard holes. Slip shall be checked against the maximum of the Service-II limit stresses and the Strength-I construction stresses due to slab pouring sequences. All splice bolts shall be 7/8" diameter ASTM F3125 Grade A325 bolts with a minimum pretension of 39 kips upon girder erection.

See SEG 20-02 (<https://www.modot.org/structural-engineering-guidance>) for Galvanized Bolted Connection Design Requirements.

Shear Resistance

The Bolt Shear Resistance shall be adequate to resist loads at the Strength-I limit state. (See Loads in this article.)

If a filler plate not less than 0.25" in thickness is used in a flange splice then the Bolt Shear Resistance shall be reduced by the following factor:

$$R = \frac{(1 + \gamma)}{(1 + 2\gamma)}$$

Where:

$$1' = \frac{A_t}{A_p}$$

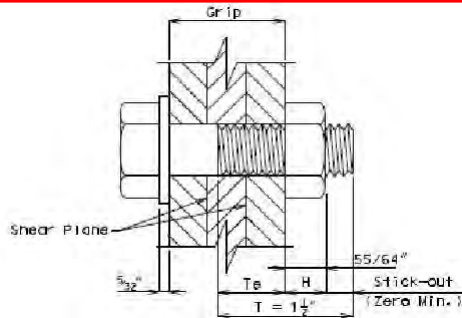
A_t = sum of the area of the fillers on the top and bottom of the connected plate (in²)

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in²)

If the distance between extreme fasteners on one side of a flange splice (joint length) is greater than 50 inches then the Bolt Shear Resistance shall be reduced by 20%. Excluding the threads from the shear plane (when applicable) in order to reduce the overall length of a joint is preferred over reducing the Bolt Shear Resistance for large joint lengths.

Min. splice plate thickness:

- $< \frac{5}{8}$ " $\Rightarrow$ Bolt threads included
 - 8
 $> \frac{3}{4}$ " $\Rightarrow$ Bolt threads excluded
 - 4



Note: The washer may be located under the nut or bolt head.

Critical dimensions for checking thread exclusion in 7/8" diameter ASTM F3125 Grade A325 bolted connections.

Threads may be excluded from the shear plane when the minimum splice plate thickness at the joint is greater than 5/8". Otherwise, the bolt threads shall be assumed to be included in the shear plane.

Thread Embedment, $T_e = T - H$ - washer height - (stick-out)

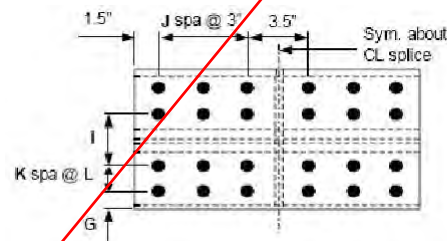
$T_{e,max} = 1.5" - 55/64" - 0" - 0" = 0.641"$ or 3/4" rounded up to the nearest 1/8".

Flange Bolt Patterns

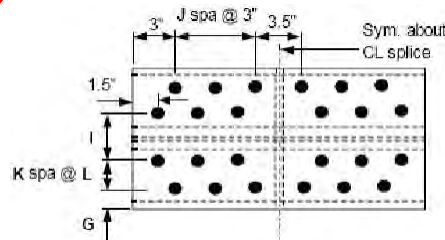
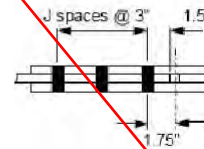
The minimum distance from the center of any fastener in a standard hole to a sheared or thermally cut edge shall be 1 1/2 inches for 7/8" diameter fasteners. The minimum distance between centers of fasteners in standard holes shall be three times the diameter of the fastener, but shall not be less than 3 inches parallel to the line of force for 7/8" diameter fasteners. The minimum edge distance shall be 2 inches for top flange outer splice plates to allow for panel clearance.

Uniform bolt patterns are preferred in all cases except that a staggered pattern may be used for flanges that are 14 and 15 inches wide. A staggered bolt pattern reduces the distance between rows of bolts so panels can be placed along the edges of the plate.

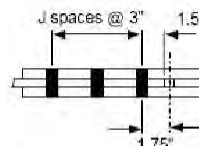
Bolt tightening clearances may become a problem for thick inner flange splice plates or flange bolt patterns that use a narrow bolt gauge, I, across the web. A thorough review is required to ensure that a 2.5" diameter socket with 1/16" clearance can be used. Typically, when bolt tightening clearances are an issue the bolt gauge, I, needs to be increased but the sealing check shall not be violated. If increasing the bolt gauge is not an option the clearance to web bolts, e, shall be increased the minimum extent necessary to maintain a practical uniform web bolt pattern.



Uniform Bolt Pattern



Staggered Bolt Pattern

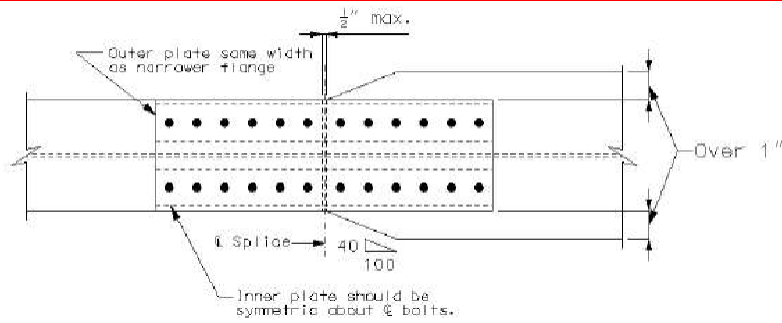


Note: This section is typical for the outer rows of bolts.

Flange Splice Plate Design

Flange Width Transitions

When the width of the flanges being spliced differs by more than 2", the larger flange shall be beveled as shown below.



Flange Splice Dimensions

1. Minimum thickness of splice plate = 3/8"
2. Splice plate thickness increment = 1/8"
3. At the strength limit state, if the gross areas of the outer and inner splice plates vary by less than 10% then the shear force can be distributed by double shear. Otherwise, the shear force must be distributed to each shear plane by the percentage of the gross area of the plate in contact with the shear plane from the total gross area of splice plates. (Note: Double shear can be assumed at the service limit state for all cases)
4. The total gross area of the splice plates shall be greater than the gross area of the smaller flange.
5. Bolt holes are considered to be 1" in diameter for the purpose of determining the net flange or splice area.
6. Inner splice plates shall be symmetric for uniform bolt patterns.

Loads

- **Strength I** - Splice plates shall be designed to resist a minimum design stress equal to the average of the factored elastic flexural stress at the mid-depth of the flange and the yield strength of the flange (F_yf), but shall not be less than $0.75F_yf$.
- **Service II** - Use factored loading to determine design force.
- **Construction** - Use factored loading to determine design force.
- **Fatigue** - Use factored loading to determine design stress range.

Resistance

The nominal flexural resistance (F_n) of the flange at the point of splice at the strength limit state shall be assumed to be equal to the specified minimum yield strength of the flange (F_yf). For hybrid sections, a hybrid factor (R_h) shall be used to increase the stress seen by the flange when the factored elastic flexural stress exceeds F_{yw} at the top or bottom of the web.

Flange splice plates shall be checked for:

- Tensile Resistance @ Strength-I Limit State
 - Yielding of Gross Section
 - Fracture of Net Section
 - Block Shear Rupture of Net Section
- Compressive Resistance @ Strength-I Limit State
- Fatigue Resistance @ Fatigue and Fracture Limit State
- Permanent Deflection under Construction loads

Note: The last two checks may be ignored if the sum of the gross splice plate areas is greater than the gross area of the flange.

Girder flange shall be checked for:

- Tensile Resistance @ Strength-I Limit State
 - Fracture of Net Section
 - Block Shear Rupture of Net Section
- Bearing Resistance @ Strength-I Limit State

Web Splice Plate Design

Web Splice Dimensions

1. Webs shall be spliced symmetrically by plates on each side.
2. These plates shall extend as near as practical for the full depth between flanges.
3. The thickness of the web splice plates shall be 3/8". The total gross area of the splice plates shall be greater than the gross area of the smaller web at splice.

4. Bolt holes are considered to be 1" in diameter for the purpose of determining the net web or splice area.

Loads

Web splice plates and their connections shall be designed for shear, the moment due to the eccentricity of the shear at the point of splice, and the portion of the flexural moment assumed to be resisted by the web at the point of splice.

- **Strength I** - If $V_u > 0.5V_r$, then the design shear (V_{uw}) shall be equal to the average of the factored shear loading (V_u) and the factored shear resistance (V_r) of the smaller web. Otherwise, $V_{uw} = 1.5V_u$. The design moment shall be calculated using the design stresses determined for flange design.
- **Service II** - Use the factored loading to determine the design shear and moment.
- **Construction** - Use the factored loading to determine the design moment and shear.
- **Fatigue** - Use the factored loading to determine the design stress range.

Resistance

The factored shear resistance (V_n) of the web shall be determined as specified in LRFD 6.12.1.2.3. For hybrid sections the web shall resist only the stress not exceeding the yield strength of the web (F_{yw}). A hybrid factor (R_h) shall be used to reduce the stress seen by the web when the factored elastic flexural stress exceeds F_{yw} at the top or bottom of the web.

Web splice plates shall be checked for:

- Shear Resistance @ Strength-I Limit State
 - Yielding of Gross Section
 - Fracture of Net Section
 - Block Shear Rupture of Net Section
- Flexural Resistance @ Strength-I Limit State
- Fatigue Resistance @ Fatigue and Fracture Limit State
- Permanent Deflection under construction loads

Note: The last two checks may be ignored if the sum of the gross splice plate areas is greater than the gross area of the web.

Girder web shall be checked for:

- Bearing Resistance @ Strength-I Limit State

Design Tools

Field Flange Splice Tables

The flange tables were developed assuming a symmetric girder section with the tension flange near yield. The flange splice plates were designed to carry this tension force along with the associated compression force found on the opposing flange. Net fracture of the flange is eliminated by limiting the factored tensile stress, f_t , to the maximums allowed in the table below. When the flange section or steel grade changes at a splice, use the narrower flange. For flanges with the same width, the smaller flange strength may be used from the table as shown below. Note: The tables are not designed for the lateral bending effects induced in horizontally curved bridges. Lateral forces may increase the design bolt force above the value used in the tables.

Which flange should be used to enter in the tables?

A. Grade 50W - 18" x 2" vs. B. Grade 70W - 18" x 1.375"
 $50\text{ksi} \times 18" \times 2" = 1800 \text{ k}$ > $70\text{ksi} \times 18" \times 1.375" = 1733 \text{ k}$

Use Flange B from the table (50W splice plate)

Field Web Splice Tables

The dimensions of the web splices shown in the tables are based on the assumption that the factored shear force (V_u) is equal to the web's unstiffened shear capacity (V_n). Also, the moment required to be resisted by the web (M_u) is assumed to be half of the elastic web moment capacity ($0.5M_{cap}$). If the design shear (V_{uw}) is greater than the unstiffened shear capacity, V_n , (i.e., stiffened webs) then the web splice shall be designed in accordance with LRFD 6.13.6.1.4b. Flange splices shall be checked to ensure that they can resist the entire moment applied to the girder section at the centerline of the splice, or the web and flange splices will have to be designed in accordance with LRFD 6.13.6.1.4b & c. Lateral bending effects from horizontal curvature need not be considered in the design of the web splice.

Maximum allowable gross tensile stress in flange at splice
 per
 LRFD 6.10.1.8. (*)

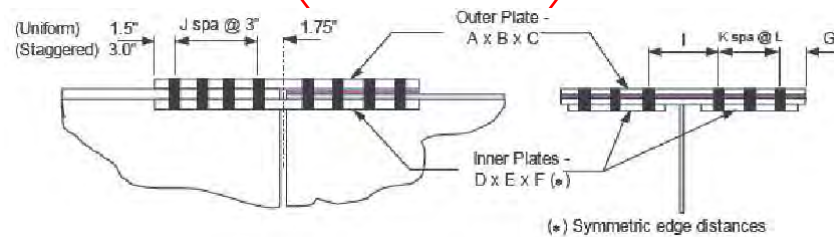
Flange Width	Flange Grade

	50	SOW	70W
12"	45.5	49.0	63.0
13"	46.2	49.7	64.0
14"	46.0	49.6	63.7
15"	46.6	50	64.5
16"	40.9	44.1	56.7
17"	41.7	45.0	57.8
18"	42.5	45.7	58.8
19"	43.1	46.4	59.7
20"	43.7	47.0	60.5
21"	44.2	47.6	61.2
22"	39.7	42.8	55.0
23"	40.4	43.5	55.9
24"	40.9	44.1	56.7

(*) Lower stresses will not fracture the net section when using the Flange Splice tables.
Maximum stresses are dependant on bolt pattern.

751.14.3.2 Plate Girder - Flange Splice Tables

Flange Plate Size: 12" thru 24" (2, 4 or 6 rows of Bolts) ASTM A709, Grade 50 and 50W Flanges, Grade 50 and 50W Splice



Note: For staggered bolt patterns the spacing shown above is for the edge rows of bolts.

Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
12x3/4"	< 1/4"	12	0.500	36.5	4	0.625	36.5	3	6	5	0	0	24	136.7
	other	12	0.500	54.5	4	0.625	54.5	3	6	8	0	0	36	204.2
12 x 7/8"	< 1/4"	12	0.625	42.5	4.5	0.625	42.5	2.75	6.5	6	0	0	28	184.8
	other	12	0.625	60.5	4.5	0.750	60.5	2.75	6.5	9	0	0	40	282.5
12 x 1"	< 1/4"	12	0.625	42.5	4.5	0.750	42.5	2.75	6.5	6	0	0	28	198.3
	other	12	0.625	66.5	4.5	0.750	66.5	2.75	6.5	10	0	0	44	310.5
12x11/8"	< 1/4"	12	0.750	42.5	5	0.750	42.5	2.5	7	6	0	0	28	225.4
	other	12	0.750	60.5	5	0.750	60.5	2.5	7	9	0	0	40	321.1
12x11/4"	< 1/4"	12	0.750	48.5	5	0.750	48.5	2.5	7	7	0	0	32	257.3
	other	12	0.750	66.5	5	0.750	66.5	2.5	7	10	0	0	44	352.9
12x13/8"	< 1/4"	12	0.875	54.5	5	0.875	54.5	2.5	7	8	0	0	36	331.7
	other	12	0.875	66.5	5	1.000	66.5	2.5	7	10	0	0	44	428.4
12x11/2"	< 1/4"	12	0.875	54.5	5	1.000	54.5	2.5	7	8	0	0	36	351.0
	other	12	0.875	66.5	5	1.000	66.5	2.5	7	10	0	0	44	428.4
12x15/8"	< 1/4"	12	1.000	54.5	5	1.125	54.5	2.5	7	8	0	0	36	393.5
	other	12	1.000	66.5	5	1.125	66.5	2.5	7	10	0	0	44	480.2
12x13/4"	< 1/4"	12	1.000	60.5	5	1.125	60.5	2.5	7	9	0	0	40	436.9
	other	12	1.000	72.5	5	1.125	72.5	2.5	7	11	0	0	48	523.6
12x17/8"	< 1/4"	12	1.125	66.5	5	1.250	66.5	2.5	7	10	0	0	44	532.1
	other	12	1.125	72.5	5	1.250	72.5	2.5	7	11	0	0	48	580.1
12 x 2" *	< 1/4"	12	1.250	66.5	5	1.375	66.5	2.5	7	10	0	0	44	583.9
	other	12	1.250	78.5	5	1.375	78.5	2.5	7	12	0	0	52	689.4

12 x 2 1/8" *	< 1/4"	12	1.250	72.5	5	1.375	72.5	2.5	7	11	0	0	48	636.7
	other	12	1.250	84.5	5	1.375	84.5	2.5	7	13	0	0	56	742.1
12 x 2 1/4" *	< 1/4"	12	1.375	78.5	5	1.500	78.5	2.5	7	12	0	0	52	750.6
	other	12	1.375	90.5	5	1.500	90.5	2.5	7	14	0	0	60	865.4
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
13 x 3/4"	< 1/4"	13	0.500	36.5	4	0.750	36.5	3.5	6	5	0	0	24	152.2
	other	13	0.500	54.5	4	0.750	54.5	3.5	6	8	0	0	36	227.4
13 x 7/8"	< 1/4"	13	0.625	42.5	4.5	0.750	42.5	3.25	6.5	6	0	0	28	205.9
	other	13	0.625	66.5	4.5	0.875	66.5	3.25	6.5	10	0	0	44	343.5
13 x 1"	< 1/4"	13	0.625	48.5	4.5	0.875	48.5	3.25	6.5	7	0	0	32	250.4
	other	13	0.625	72.5	4.5	0.875	72.5	3.25	6.5	11	0	0	48	374.5
13 x 1 1/8" *	< 1/4"	13	0.750	48.5	5	0.875	48.5	3	7	7	0	0	32	284.8
	other	13	0.750	66.5	5	0.875	66.5	3	7	10	0	0	44	390.7
13 x 1 1/4" *	< 1/4"	13	0.750	48.5	5	0.875	48.5	3	7	7	0	0	32	284.8
	other	13	0.750	66.5	5	0.875	66.5	3	7	10	0	0	44	390.7
13 x 1 3/8" *	< 1/4"	13	0.750	54.5	5	1.000	54.5	3	7	8	0	0	36	339.4
	other	13	0.750	66.5	5	1.000	66.5	3	7	10	0	0	44	414.2
13 x 1 1/2" *	< 1/4"	13	0.875	54.5	5	1.125	54.5	3	7	8	0	0	36	383.9
	other	13	0.875	72.5	5	1.125	72.5	3	7	11	0	0	48	510.7
13 x 1 5/8" *	< 1/4"	13	1.000	60.5	5	1.250	60.5	3	7	9	0	0	40	475.5
	other	13	1.000	72.5	5	1.250	72.5	3	7	11	0	0	48	569.8
13 x 1 3/4" *	< 1/4"	13	1.000	66.5	5	1.250	66.5	3	7	10	0	0	44	522.7
	other	13	1.000	78.5	5	1.250	78.5	3	7	12	0	0	52	617.0
13 x 1 7/8" *	< 1/4"	13	1.125	72.5	5	1.250	72.5	3	7	11	0	0	48	603.2
	other	13	1.125	84.5	5	1.250	84.5	3	7	13	0	0	56	703.1
13 x 2" *	< 1/4"	13	1.250	78.5	5	1.375	78.5	3	7	12	0	0	52	717.2
	other	13	1.250	90.5	5	1.500	90.5	3	7	14	0	0	60	859.0
13 x 2 1/8" *	< 1/4"	13	1.250	78.5	5	1.500	78.5	3	7	12	0	0	52	745.0
	other	13	1.250	90.5	5	1.500	90.5	3	7	14	0	0	60	859.0
13 x 2 1/4" *	< 1/4"	13	1.375	84.5	5	1.625	84.5	3	7	13	0	0	56	870.9
	other	13	1.375	96.5	5	1.625	96.5	3	7	15	0	0	64	994.6
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Staggered Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
14 x 3/4"	< 1/4"	14	0.500	21.5	5.5	0.625	21.5	2	5	2	1	2.5	24	107.4
	other	14	0.500	33.5	5.5	0.625	33.5	2	5	4	1	2.5	40	169.8
14 x 7/8"	< 1/4"	14	0.500	27.5	5.5	0.750	27.5	2	5	3	1	2.5	32	149.3
	other	14	0.500	39.5	5.5	0.750	39.5	2	5	5	1	2.5	48	216.4
14 x 1"	< 1/4"	14	0.625	27.5	5.5	0.750	27.5	2	5	3	1	2.5	32	163.0
	other	14	0.625	45.5	5.5	0.750	45.5	2	5	6	1	2.5	56	272.5
14 x 1 1/8" *	< 1/4"	14	0.750	27.5	5.5	0.750	27.5	2	5	3	1	2.5	32	176.6
	other	14	0.750	39.5	5.5	0.750	39.5	2	5	5	1	2.5	48	255.6
14 x 1 1/4" *	< 1/4"	14	0.875	33.5	5.5	0.875	33.5	2	5	4	1	2.5	40	245.8
	other	14	0.875	39.5	5.5	1.000	39.5	2	5	5	1	2.5	48	306.0
14 x 1 3/8" *	< 1/4"	14	0.875	33.5	5.5	1.000	33.5	2	5	4	1	2.5	40	258.9
	other	14	0.875	45.5	5.5	1.000	45.5	2	5	6	1	2.5	56	353.2
14 x 1 1/2" *	< 1/4"	14	0.875	33.5	5.5	1.125	33.5	2	5	4	1	2.5	40	271.9
	other	14	0.875	45.5	5.5	1.125	45.5	2	5	6	1	2.5	56	370.9
14 x 1 5/8" *	< 1/4"	14	1.000	39.5	5.5	1.125	39.5	2	5	5	1	2.5	48	341.0
	other	14	1.000	45.5	5.5	1.125	45.5	2	5	6	1	2.5	56	393.5
14 x 1 3/4" *	< 1/4"	14	1.125	39.5	5.5	1.250	39.5	2	5	5	1	2.5	48	376.0
	other	14	1.125	45.5	5.5	1.250	45.5	2	5	6	1	2.5	56	433.8
14 x 1 7/8" *	< 1/4"	14	1.250	45.5	5.5	1.250	45.5	2	5	6	1	2.5	56	456.4
	other	14	1.250	51.5	5.5	1.250	51.5	2	5	7	1	2.5	64	517.2
14 x 2" *	< 1/4"	14	1.250	45.5	5.5	1.375	45.5	2	5	6	1	2.5	56	474.1
	other	14	1.250	51.5	5.5	1.375	51.5	2	5	7	1	2.5	64	537.2

14 x 2 1/8" *	< 1/4"	14	1.375	51.5	5.5	1.375	51.5	2	5	7	1	2.5	64	562.8
	other	14	1.375	57.5	5.5	1.375	57.5	2	5	8	1	2.5	72	628.9
14 x 2 1/4" *	< 1/4"	14	1.500	51.5	5.5	1.500	51.5	2	5	7	1	2.5	64	608.4
	other	14	1.500	63.5	5.5	1.500	63.5	2	5	9	1	2.5	80	751.2
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Staggered Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
15 x 3/4"	< 1/4"	15	0.500	27.5	6.5	0.500	27.5	2	6	3	1	2.5	32	139.6
	other	15	0.500	39.5	6.5	0.500	39.5	2	6	5	1	2.5	48	202.4
15 x 7/8"	< 1/4"	15	0.500	27.5	6.5	0.625	27.5	2	6	3	1	2.5	32	152.2
	other	15	0.500	39.5	6.5	0.625	39.5	2	6	5	1	2.5	48	220.6
15 x 1"	< 1/4"	15	0.625	33.5	6.5	0.625	33.5	2	6	4	1	2.5	40	204.2
	other	15	0.625	45.5	6.5	0.625	45.5	2	6	6	1	2.5	56	279.0
15 x 1 1/8"	< 1/4"	15	0.625	33.5	6.5	0.750	33.5	2	6	4	1	2.5	40	219.7
	other	15	0.625	51.5	6.5	0.750	51.5	2	6	7	1	2.5	64	340.1
15 x 1 1/4" *	< 1/4"	15	0.750	33.5	6.5	0.875	33.5	2	6	4	1	2.5	40	252.9
	other	15	0.750	45.5	6.5	0.875	45.5	2	6	6	1	2.5	56	345.1
15 x 1 3/8" *	< 1/4"	15	0.875	33.5	6.5	0.875	33.5	2	6	4	1	2.5	40	270.7
	other	15	0.875	45.5	6.5	0.875	45.5	2	6	6	1	2.5	56	369.3
15 x 1 1/2" *	< 1/4"	15	0.875	39.5	6.5	1.000	39.5	2	6	5	1	2.5	48	338.2
	other	15	0.875	45.5	6.5	1.000	45.5	2	6	6	1	2.5	56	390.3
15 x 1 5/8" *	< 1/4"	15	1.000	39.5	6.5	1.000	39.5	2	6	5	1	2.5	48	359.2
	other	15	1.000	51.5	6.5	1.000	51.5	2	6	7	1	2.5	64	469.7
15 x 1 3/4" *	< 1/4"	15	1.125	45.5	6.5	1.125	45.5	2	6	6	1	2.5	56	459.6
	other	15	1.125	51.5	6.5	1.125	51.5	2	6	7	1	2.5	64	520.8
15 x 1 7/8" *	< 1/4"	15	1.125	45.5	6.5	1.250	45.5	2	6	6	1	2.5	56	480.6
	other	15	1.125	51.5	6.5	1.250	51.5	2	6	7	1	2.5	64	544.5
15 x 2" *	< 1/4"	15	1.250	51.5	6.5	1.250	51.5	2	6	7	1	2.5	64	571.9
	other	15	1.250	57.5	6.5	1.250	57.5	2	6	8	1	2.5	72	639.1
15 x 2 1/8" *	< 1/4"	15	1.250	51.5	6.5	1.375	51.5	2	6	7	1	2.5	64	595.7
	other	15	1.250	57.5	6.5	1.375	57.5	2	6	8	1	2.5	72	665.6
15 x 2 1/4" *	< 1/4"	15	1.375	51.5	6.5	1.375	51.5	2	6	7	1	2.5	64	623.0
	other	15	1.375	63.5	6.5	1.375	63.5	2	6	9	1	2.5	80	769.2
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
16 x 3/4"	< 1/4"	16	0.500	24.5	7	0.500	24.5	2	6	3	1	3	32	134.6
	other	16	0.500	36.5	7	0.500	36.5	2	6	5	1	3	48	200.9
16 x 7/8"	< 1/4"	16	0.500	24.5	7	0.500	24.5	2	6	3	1	3	32	134.6
	other	16	0.500	42.5	7	0.500	42.5	2	6	6	1	3	56	234.0
16 x 1"	< 1/4"	16	0.625	30.5	7	0.625	30.5	2	6	4	1	3	40	200.2
	other	16	0.625	42.5	7	0.750	42.5	2	6	6	1	3	56	300.3
16 x 1 1/8"	< 1/4"	16	0.625	30.5	7	0.750	30.5	2	6	4	1	3	40	215.3
	other	16	0.625	48.5	7	0.750	48.5	2	6	7	1	3	64	342.7
16 x 1 1/4" *	< 1/4"	16	0.750	30.5	7	0.750	30.5	2	6	4	1	3	40	232.6
	other	16	0.750	42.5	7	0.750	42.5	2	6	6	1	3	56	324.4
16 x 1 3/8" *	< 1/4"	16	0.750	30.5	7	0.875	30.5	2	6	4	1	3	40	247.7
	other	16	0.750	42.5	7	0.875	42.5	2	6	6	1	3	56	345.4
16 x 1 1/2" *	< 1/4"	16	0.875	36.5	7	0.875	36.5	2	6	5	1	3	48	317.3
	other	16	0.875	48.5	7	0.875	48.5	2	6	7	1	3	64	421.8
16 x 1 5/8" *	< 1/4"	16	0.875	36.5	7	1.000	36.5	2	6	5	1	3	48	335.4
	other	16	0.875	48.5	7	1.000	48.5	2	6	7	1	3	64	445.9
16 x 1 3/4" *	< 1/4"	16	1.000	36.5	7	1.125	36.5	2	6	5	1	3	48	374.2
	other	16	1.000	48.5	7	1.125	48.5	2	6	7	1	3	64	497.5
16 x 1 7/8" *	< 1/4"	16	1.125	42.5	7	1.125	42.5	2	6	6	1	3	56	459.9
	other	16	1.125	54.5	7	1.125	54.5	2	6	8	1	3	72	590.0
16 x 2" *	< 1/4"	16	1.125	42.5	7	1.250	42.5	2	6	6	1	3	56	481.0
	other	16	1.125	54.5	7	1.250	54.5	2	6	8	1	3	72	617.0

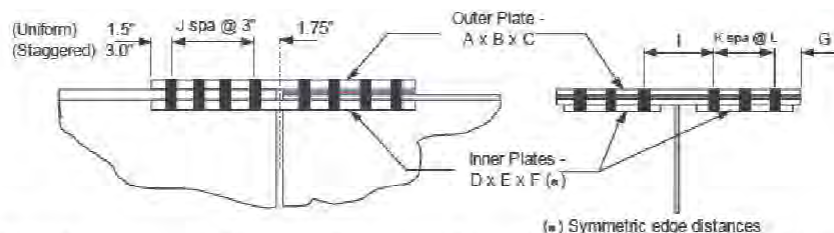
16 x 2 1/8" *	< 1/4"	16	1.250	48.5	7	1.250	48.5	2	6	7	1	3	64	576.5
	other	16	1.250	60.5	7	1.250	60.5	2	6	9	1	3	80	719.3
16 x 2 1/4" *	< 1/4"	16	1.250	48.5	7	1.375	48.5	2	6	7	1	3	64	600.6
	other	16	1.250	60.5	7	1.375	60.5	2	6	9	1	3	80	749.4
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
17 x 3/4"	< 1/4"	17	0.500	24.5	7	0.500	24.5	2.5	6	3	1	3	32	138.1
	other	17	0.500	36.5	7	0.625	36.5	2.5	6	5	1	3	48	224.1
17 x 7/8"	< 1/4"	17	0.500	24.5	7	0.625	24.5	2.5	6	3	1	3	32	150.2
	other	17	0.500	42.5	7	0.625	42.5	2.5	6	6	1	3	56	261.1
17 x 1"	< 1/4"	17	0.625	30.5	7	0.625	30.5	2.25	6.5	4	1	3	40	205.6
	other	17	0.625	48.5	7	0.625	48.5	2.25	6.5	7	1	3	64	327.3
17 x 1 1/8"	< 1/4"	17	0.625	36.5	7	0.750	36.5	2.25	6.5	5	1	3	48	264.2
	other	17	0.625	48.5	7	0.750	48.5	2.25	6.5	7	1	3	64	351.3
17 x 1 1/4" *	< 1/4"	17	0.750	30.5	7	0.875	30.5	2	7	4	1	3	40	254.2
	other	17	0.750	42.5	7	0.875	42.5	2	7	6	1	3	56	354.5
17 x 1 3/8" *	< 1/4"	17	0.875	36.5	7	0.875	36.5	2	7	5	1	3	48	326.3
	other	17	0.875	48.5	7	0.875	48.5	2	7	7	1	3	64	433.8
17 x 1 1/2" *	< 1/4"	17	0.875	36.5	7	1.000	36.5	2	7	5	1	3	48	344.5
	other	17	0.875	48.5	7	1.000	48.5	2	7	7	1	3	64	457.9
17 x 1 5/8" *	< 1/4"	17	1.000	42.5	7	1.000	42.5	2	7	6	1	3	56	426.8
	other	17	1.000	54.5	7	1.000	54.5	2	7	8	1	3	72	547.5
17 x 1 3/4" *	< 1/4"	17	1.125	42.5	7	1.125	42.5	2	7	6	1	3	56	473.5
	other	17	1.125	54.5	7	1.125	54.5	2	7	8	1	3	72	607.4
17 x 1 7/8" *	< 1/4"	17	1.125	48.5	7	1.125	48.5	2	7	7	1	3	64	540.4
	other	17	1.125	54.5	7	1.125	54.5	2	7	8	1	3	72	607.4
17 x 2" *	< 1/4"	17	1.250	48.5	7	1.250	48.5	2	7	7	1	3	64	593.7
	other	17	1.250	60.5	7	1.250	60.5	2	7	9	1	3	80	740.8
17 x 2 1/8" *	< 1/4"	17	1.250	48.5	7	1.375	48.5	2	7	7	1	3	64	617.8
	other	17	1.250	60.5	7	1.375	60.5	2	7	9	1	3	80	770.8
17 x 2 1/4" *	< 1/4"	17	1.375	54.5	7	1.375	54.5	2	7	8	1	3	72	727.1
	other	17	1.375	66.5	7	1.375	66.5	2	7	10	1	3	88	887.4
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
18 x 3/4"	< 1/4"	18	0.500	24.5	8	0.500	24.5	2	6	3	1	4	32	148.5
	other	18	0.500	42.5	8	0.500	42.5	2	6	6	1	4	56	258.1
18 x 7/8"	< 1/4"	18	0.500	30.5	8	0.500	30.5	2	6	4	1	4	40	185.0
	other	18	0.500	48.5	8	0.500	48.5	2	6	7	1	4	64	294.6
18 x 1"	< 1/4"	18	0.500	36.5	8	0.625	36.5	2	6	5	1	4	48	242.3
	other	18	0.500	48.5	8	0.625	48.5	2	6	7	1	4	64	322.1
18 x 1 1/8"	< 1/4"	18	0.625	36.5	8	0.750	36.5	2.25	6.5	5	1	3.5	48	286.2
	other	18	0.625	54.5	8	0.750	54.5	2.25	6.5	8	1	3.5	72	427.7
18 x 1 1/4" *	< 1/4"	18	0.750	36.5	8	0.750	36.5	2.5	7	5	1	3	48	309.5
	other	18	0.750	48.5	8	0.750	48.5	2.5	7	7	1	3	64	411.5
18 x 1 3/8" *	< 1/4"	18	0.750	36.5	8	0.875	36.5	2.5	7	5	1	3	48	330.2
	other	18	0.750	48.5	8	0.875	48.5	2.5	7	7	1	3	64	439.0
18 x 1 1/2" *	< 1/4"	18	0.875	42.5	8	0.875	42.5	2.5	7	6	1	3	56	411.7
	other	18	0.875	54.5	8	0.875	54.5	2.5	7	8	1	3	72	528.2
18 x 1 5/8" *	< 1/4"	18	0.875	42.5	8	1.000	42.5	2.5	7	6	1	3	56	435.8
	other	18	0.875	54.5	8	1.000	54.5	2.5	7	8	1	3	72	559.1
18 x 1 3/4" *	< 1/4"	18	1.000	48.5	8	1.000	48.5	2.5	7	7	1	3	64	528.4
	other	18	1.000	54.5	8	1.000	54.5	2.5	7	8	1	3	72	593.8
18 x 1 7/8" *	< 1/4"	18	1.000	48.5	8	1.125	48.5	2.5	7	7	1	3	64	555.9
	other	18	1.000	54.5	8	1.125	54.5	2.5	7	8	1	3	72	624.8
18 x 2" *	< 1/4"	18	1.125	54.5	8	1.125	54.5	2.5	7	8	1	3	72	659.5
	other	18	1.125	60.5	8	1.125	60.5	2.5	7	9	1	3	80	732.2

18 x 2 1/8" *	< 1/4"	18	1.250	54.5	8	1.250	54.5	2.5	7	8	1	3	72	725.2
	other	18	1.250	66.5	8	1.250	66.5	2.5	7	10	1	3	88	885.0
18 x 2 1/4" *	< 1/4"	18	1.250	54.5	8	1.375	54.5	2.5	7	8	1	3	72	756.1
	other	18	1.250	66.5	8	1.375	66.5	2.5	7	10	1	3	88	922.7
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
19 x 7/8"	< 1/4"	19	0.500	30.5	8.5	0.500	30.5	2	6	4	1	4.5	40	193.7
	other	19	0.500	48.5	8.5	0.500	48.5	2	6	7	1	4.5	64	308.4
19 x 1"	< 1/4"	19	0.500	36.5	8.5	0.625	36.5	2	6	5	1	4.5	48	253.9
	other	19	0.500	54.5	8.5	0.625	54.5	2	6	8	1	4.5	72	379.4
19 x 1 1/8"	< 1/4"	19	0.625	36.5	8.5	0.750	36.5	2.25	6.5	5	1	4	48	300.5
	other	19	0.625	54.5	8.5	0.750	54.5	2.25	6.5	8	1	4	72	449.0
19 x 1 1/4" *	< 1/4"	19	0.750	36.5	8.5	0.750	36.5	2.5	7	5	1	3.5	48	325.1
	other	19	0.750	48.5	8.5	0.750	48.5	2.5	7	7	1	3.5	64	432.1
19 x 1 3/8" *	< 1/4"	19	0.750	36.5	8.5	0.875	36.5	2.5	7	5	1	3.5	48	347.0
	other	19	0.750	48.5	8.5	0.875	48.5	2.5	7	7	1	3.5	64	461.4
19 x 1 1/2" *	< 1/4"	19	0.875	42.5	8.5	0.875	42.5	2.5	7	6	1	3.5	56	432.8
	other	19	0.875	54.5	8.5	0.875	54.5	2.5	7	8	1	3.5	72	555.2
19 x 1 5/8" *	< 1/4"	19	0.875	42.5	8.5	1.000	42.5	2.5	7	6	1	3.5	56	458.4
	other	19	0.875	54.5	8.5	1.000	54.5	2.5	7	8	1	3.5	72	588.1
19 x 1 3/4" *	< 1/4"	19	1.000	48.5	8.5	1.000	48.5	2.5	7	7	1	3.5	64	555.9
	other	19	1.000	60.5	8.5	1.000	60.5	2.5	7	9	1	3.5	80	693.6
19 x 1 7/8" *	< 1/4"	19	1.000	48.5	8.5	1.125	48.5	2.5	7	7	1	3.5	64	585.1
	other	19	1.000	60.5	8.5	1.125	60.5	2.5	7	9	1	3.5	80	730.1
19 x 2" *	< 1/4"	19	1.125	54.5	8.5	1.125	54.5	2.5	7	8	1	3.5	72	694.3
	other	19	1.125	66.5	8.5	1.125	66.5	2.5	7	10	1	3.5	88	847.3
19 x 2 1/8" *	< 1/4"	19	1.125	54.5	8.5	1.250	54.5	2.5	7	8	1	3.5	72	727.1
	other	19	1.125	66.5	8.5	1.250	66.5	2.5	7	10	1	3.5	88	887.4
19 x 2 1/4" *	< 1/4"	19	1.250	60.5	8.5	1.250	60.5	2.5	7	9	1	3.5	80	848.0
	other	19	1.250	72.5	8.5	1.250	72.5	2.5	7	11	1	3.5	96	1016.3
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
20 x 7/8"	< 1/4"	20	0.500	30.5	9	0.500	30.5	2	6	4	1	5	40	202.3
	other	20	0.500	48.5	9	0.500	48.5	2	6	7	1	5	64	322.1
20 x 1"	< 1/4"	20	0.500	36.5	9	0.625	36.5	2	6	5	1	5	48	265.5
	other	20	0.500	54.5	9	0.625	54.5	2	6	8	1	5	72	396.8
20 x 1 1/8"	< 1/4"	20	0.625	42.5	9	0.625	42.5	2.25	6.5	6	1	4.5	56	339.4
	other	20	0.625	60.5	9	0.625	60.5	2.25	6.5	9	1	4.5	80	483.4
20 x 1 1/4" *	< 1/4"	20	0.750	36.5	9	0.750	36.5	2.5	7	5	1	4	48	340.6
	other	20	0.750	54.5	9	0.750	54.5	2.5	7	8	1	4	72	508.8
20 x 1 3/8" *	< 1/4"	20	0.750	42.5	9	0.875	42.5	2.5	7	6	1	4	56	423.8
	other	20	0.750	54.5	9	0.875	54.5	2.5	7	8	1	4	72	543.6
20 x 1 1/2" *	< 1/4"	20	0.875	42.5	9	0.875	42.5	2.5	7	6	1	4	56	453.9
	other	20	0.875	60.5	9	0.875	60.5	2.5	7	9	1	4	80	646.4
20 x 1 5/8" *	< 1/4"	20	0.875	48.5	9	1.000	48.5	2.5	7	7	1	4	64	549.0
	other	20	0.875	54.5	9	1.000	54.5	2.5	7	8	1	4	72	617.0
20 x 1 3/4" *	< 1/4"	20	1.000	48.5	9	1.000	48.5	2.5	7	7	1	4	64	583.4
	other	20	1.000	60.5	9	1.000	60.5	2.5	7	9	1	4	80	727.9
20 x 1 7/8" *	< 1/4"	20	1.000	54.5	9	1.125	54.5	2.5	7	8	1	4	72	690.4
	other	20	1.000	60.5	9	1.125	60.5	2.5	7	9	1	4	80	766.5
20 x 2" *	< 1/4"	20	1.125	60.5	9	1.125	60.5	2.5	7	9	1	4	80	809.4
	other	20	1.125	66.5	9	1.125	66.5	2.5	7	10	1	4	88	889.7
20 x 2 1/8" *	< 1/4"	20	1.125	60.5	9	1.250	60.5	2.5	7	9	1	4	80	848.0
	other	20	1.125	66.5	9	1.250	66.5	2.5	7	10	1	4	88	932.2
20 x 2 1/4" *	< 1/4"	20	1.250	66.5	9	1.250	66.5	2.5	7	10	1	4	88	979.3
	other	20	1.250	78.5	9	1.250	78.5	2.5	7	12	1	4	104	1156.1
* Bolt threads are EXCLUDED from shear plane														

Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						Total # of Bolts	WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)			
21 x 7/8"	< 1/4"	21	0.500	36.5	9.5	0.500	36.5	2	6	5	1	5.5	48	252.6	
	other	21	0.500	54.5	9.5	0.500	54.5	2	6	8	1	5.5	72	377.5	
21 x 1"	< 1/4"	21	0.625	36.5	9.5	0.625	36.5	2.25	6.5	5	1	5	48	304.4	
	other	21	0.500	60.5	9.5	0.625	60.5	2	6	9	1	5.5	80	459.9	
21 x 1 1/8"	< 1/4"	21	0.625	42.5	9.5	0.625	42.5	2.25	6.5	6	1	5	56	354.5	
	other	21	0.625	60.5	9.5	0.750	60.5	2.25	6.5	9	1	5	80	545.6	
21 x 1 1/4" *	< 1/4"	21	0.750	42.5	9.5	0.750	42.5	2.5	7	6	1	4.5	56	414.7	
	other	21	0.750	54.5	9.5	0.750	54.5	2.5	7	8	1	4.5	72	532.0	
21 x 1 3/8" *	< 1/4"	21	0.750	42.5	9.5	0.875	42.5	2.5	7	6	1	4.5	56	443.4	
	other	21	0.750	54.5	9.5	0.875	54.5	2.5	7	8	1	4.5	72	568.7	
21 x 1 1/2" *	< 1/4"	21	0.875	48.5	9.5	0.875	48.5	2.5	7	7	1	4.5	64	542.2	
	other	21	0.875	60.5	9.5	0.875	60.5	2.5	7	9	1	4.5	80	676.4	
21 x 1 5/8" *	< 1/4"	21	0.875	48.5	9.5	1.000	48.5	2.5	7	7	1	4.5	64	574.8	
	other	21	0.875	60.5	9.5	1.000	60.5	2.5	7	9	1	4.5	80	717.2	
21 x 1 3/4" *	< 1/4"	21	1.000	54.5	9.5	1.000	54.5	2.5	7	8	1	4.5	72	686.6	
	other	21	1.000	66.5	9.5	1.000	66.5	2.5	7	10	1	4.5	88	837.9	
21 x 1 7/8" *	< 1/4"	21	1.000	54.5	9.5	1.125	54.5	2.5	7	8	1	4.5	72	723.3	
	other	21	1.000	66.5	9.5	1.125	66.5	2.5	7	10	1	4.5	88	882.7	
21 x 2" *	< 1/4"	21	1.125	60.5	9.5	1.125	60.5	2.5	7	9	1	4.5	80	848.0	
	other	21	1.125	72.5	9.5	1.125	72.5	2.5	7	11	1	4.5	96	1016.3	
21 x 2 1/8" *	< 1/4"	21	1.125	60.5	9.5	1.250	60.5	2.5	7	9	1	4.5	80	888.7	
	other	21	1.125	72.5	9.5	1.250	72.5	2.5	7	11	1	4.5	96	1065.2	
21 x 2 1/4" *	< 1/4"	21	1.250	66.5	9.5	1.250	66.5	2.5	7	10	1	4.5	88	1026.5	
	other	21	1.250	78.5	9.5	1.250	78.5	2.5	7	12	1	4.5	104	1211.8	
* Bolt threads are EXCLUDED from shear plane															
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						Total # of Bolts	WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)			
22 x 1"	< 1/4"	22	0.625	24.5	10	0.625	24.5	2	6	3	2	3	48	228.0	
	other	22	0.625	36.5	10	0.625	36.5	2	6	5	2	3	72	340.1	
22 x 1 1/8"	< 1/4"	22	0.625	30.5	10	0.750	30.5	2	6	4	2	3	60	305.7	
	other	22	0.625	42.5	10	0.750	42.5	2	6	6	2	3	84	426.3	
22 x 1 1/4" *	< 1/4"	22	0.750	24.5	10	0.750	24.5	2	6	3	2	3	48	264.4	
	other	22	0.750	36.5	10	0.750	36.5	2	6	5	2	3	72	394.4	
22 x 1 3/8" *	< 1/4"	22	0.750	30.5	10	0.875	30.5	2	6	4	2	3	60	351.1	
	other	22	0.750	36.5	10	0.875	36.5	2	6	5	2	3	72	420.3	
22 x 1 1/2" *	< 1/4"	22	0.875	30.5	10	1.000	30.5	2	6	4	2	3	60	396.5	
	other	22	0.875	42.5	10	1.000	42.5	2	6	6	2	3	84	552.8	
22 x 1 5/8" *	< 1/4"	22	0.875	30.5	10	1.000	30.5	2	6	4	2	3	60	396.5	
	other	22	0.875	42.5	10	1.000	42.5	2	6	6	2	3	84	552.8	
22 x 1 3/4" *	< 1/4"	22	1.000	36.5	10	1.125	36.5	2	6	5	2	3	72	529.0	
	other	22	1.000	42.5	10	1.125	42.5	2	6	6	2	3	84	616.1	
22 x 1 7/8" *	< 1/4"	22	1.000	36.5	10	1.125	36.5	2	6	5	2	3	72	529.0	
	other	22	1.000	42.5	10	1.125	42.5	2	6	6	2	3	84	616.1	
22 x 2" *	< 1/4"	22	1.125	42.5	10	1.250	42.5	2	6	6	2	3	84	679.4	
	other	22	1.125	48.5	10	1.250	48.5	2	6	7	2	3	96	775.4	
22 x 2 1/8" *	< 1/4"	22	1.125	42.5	10	1.250	42.5	2	6	6	2	3	84	679.4	
	other	22	1.125	48.5	10	1.250	48.5	2	6	7	2	3	96	775.4	
22 x 2 1/4" *	< 1/4"	22	1.250	42.5	10	1.375	42.5	2	6	6	2	3	84	742.6	
	other	22	1.250	54.5	10	1.375	54.5	2	6	8	2	3	108	952.6	
* Bolt threads are EXCLUDED from shear plane															
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						Total # of Bolts	WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)			
23 x 1"	< 1/4"	23	0.500	30.5	10.5	0.625	30.5	2	6	4	2	3.25	60	270.0	
	other	23	0.500	42.5	10.5	0.625	42.5	2	6	6	2	3.25	84	376.6	
23 x 1 1/8"	< 1/4"	23	0.625	30.5	10.5	0.750	30.5	2.25	6.5	4	2	3	60	317.5	

	other	23	0.625	42.5	10.5	0.750	42.5	2.25	6.5	6	2	3	84	442.9
23 x 1 1/4" *	< 1/4"	23	0.750	30.5	10.5	0.750	30.5	2.25	6.5	4	2	3	60	342.4
	other	23	0.750	36.5	10.5	0.750	36.5	2.25	6.5	5	2	3	72	410.0
23 x 1 3/8" *	< 1/4"	23	0.750	30.5	10.5	0.875	30.5	2.25	6.5	4	2	3	60	365.1
	other	23	0.750	42.5	10.5	0.875	42.5	2.25	6.5	6	2	3	84	509.1
23 x 1 1/2" *	< 1/4"	23	0.875	30.5	10.5	0.875	30.5	2.25	6.5	4	2	3	60	390.0
	other	23	0.875	42.5	10.5	0.875	42.5	2.25	6.5	6	2	3	84	543.8
23 x 1 5/8" *	< 1/4"	23	0.875	36.5	10.5	1.000	36.5	2.25	6.5	5	2	3	72	494.0
	other	23	0.875	42.5	10.5	1.000	42.5	2.25	6.5	6	2	3	84	575.4
23 x 1 3/4" *	< 1/4"	23	1.000	36.5	10.5	1.125	36.5	2.25	6.5	5	2	3	72	551.0
	other	23	1.000	42.5	10.5	1.125	42.5	2.25	6.5	6	2	3	84	641.7
23 x 1 7/8" *	< 1/4"	23	1.000	36.5	10.5	1.125	36.5	2.25	6.5	5	2	3	72	551.0
	other	23	1.000	48.5	10.5	1.125	48.5	2.25	6.5	7	2	3	96	732.4
23 x 2" *	< 1/4"	23	1.125	42.5	10.5	1.250	42.5	2.25	6.5	6	2	3	84	708.0
	other	23	1.125	48.5	10.5	1.250	48.5	2.25	6.5	7	2	3	96	808.1
23 x 2 1/8" *	< 1/4"	23	1.125	42.5	10.5	1.250	42.5	2.25	6.5	6	2	3	84	708.0
	other	23	1.125	54.5	10.5	1.250	54.5	2.25	6.5	8	2	3	108	908.2
23 x 2 1/4" *	< 1/4"	23	1.250	48.5	10.5	1.375	48.5	2.25	6.5	7	2	3	96	883.7
	other	23	1.250	54.5	10.5	1.375	54.5	2.25	6.5	8	2	3	108	993.2
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
24 x 1"	< 1/4"	24	0.500	30.5	11	0.625	30.5	2	6	4	2	3.5	60	279.7
	other	24	0.625	42.5	11	0.625	42.5	2.25	6.5	6	2	3.25	84	426.3
24 x 1 1/8"	< 1/4"	24	0.625	30.5	11	0.750	30.5	2.25	6.5	4	2	3.25	60	329.4
	other	24	0.625	48.5	11	0.750	48.5	2.25	6.5	7	2	3.25	96	524.4
24 x 1 1/4" *	< 1/4"	24	0.750	30.5	11	0.750	30.5	2.5	7	4	2	3	60	355.4
	other	24	0.750	42.5	11	0.750	42.5	2.5	7	6	2	3	84	495.6
24 x 1 3/8" *	< 1/4"	24	0.750	30.5	11	0.875	30.5	2.5	7	4	2	3	60	379.2
	other	24	0.750	42.5	11	0.875	42.5	2.5	7	6	2	3	84	528.7
24 x 1 1/2" *	< 1/4"	24	0.875	36.5	11	0.875	36.5	2.5	7	5	2	3	72	485.0
	other	24	0.875	42.5	11	0.875	42.5	2.5	7	6	2	3	84	564.9
24 x 1 5/8" *	< 1/4"	24	0.875	36.5	11	1.000	36.5	2.5	7	5	2	3	72	513.5
	other	24	0.875	42.5	11	1.000	42.5	2.5	7	6	2	3	84	598.0
24 x 1 3/4" *	< 1/4"	24	1.000	36.5	11	1.125	36.5	2.5	7	5	2	3	72	573.0
	other	24	1.000	48.5	11	1.125	48.5	2.5	7	7	2	3	96	761.7
24 x 1 7/8" *	< 1/4"	24	1.000	42.5	11	1.125	42.5	2.5	7	6	2	3	84	667.3
	other	24	1.000	48.5	11	1.125	48.5	2.5	7	7	2	3	96	761.7
24 x 2" *	< 1/4"	24	1.125	42.5	11	1.250	42.5	2.5	7	6	2	3	84	736.6
	other	24	1.125	54.5	11	1.250	54.5	2.5	7	8	2	3	108	944.9
24 x 2 1/8" *	< 1/4"	24	1.125	48.5	11	1.250	48.5	2.5	7	7	2	3	96	840.7
	other	24	1.125	54.5	11	1.250	54.5	2.5	7	8	2	3	108	944.9
24 x 2 1/4" *	< 1/4"	24	1.250	48.5	11	1.375	48.5	2.5	7	7	2	3	96	919.8
	other	24	1.250	60.5	11	1.375	60.5	2.5	7	9	2	3	120	1147.6
* Bolt threads are EXCLUDED from shear plane														

Flange Plate Size: 12" thru 24" (2-6 rows of Bolts) ASTM A709, Grade 70W Flange, Grade 50W Splice



Note: For staggered bolt patterns the spacing shown above is for the edge rows of bolts.

Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
12 x 3/4"	< 1/4"	12	0.750	42.5	5	0.750	42.5	2.5	7	6	0	0	28	225.4
	other	12	0.750	72.5	5	0.750	72.5	2.5	7	9	0	0	40	377.2
12 x 7/8" *	< 1/4"	12	0.750	42.5	5	0.750	42.5	2.5	7	6	0	0	28	225.4
	other	12	0.750	66.5	5	0.875	66.5	2.5	7	10	0	0	44	376.5
12 x 1" *	< 1/4"	12	0.875	48.5	5	0.875	48.5	2.5	7	7	0	0	32	295.1
	other	12	0.875	72.5	5	1.000	72.5	2.5	7	11	0	0	48	467.0
12 x 1 1/8" *	< 1/4"	12	0.875	54.5	5	1.000	54.5	2.5	7	8	0	0	36	351.0
	other	12	0.875	78.5	5	1.000	78.5	2.5	7	12	0	0	52	505.7
12 x 1 1/4" *	< 1/4"	12	1.000	54.5	5	1.125	54.5	2.5	7	8	0	0	36	393.5
	other	12	1.000	78.5	5	1.125	78.5	2.5	7	12	0	0	52	566.9
12 x 1 3/8" *	< 1/4"	12	1.125	60.5	5	1.250	60.5	2.5	7	9	0	0	40	484.0
	other	12	1.125	84.5	5	1.250	84.5	2.5	7	13	0	0	56	676.2
12 x 1 1/2" *	< 1/4"	12	1.250	66.5	5	1.375	66.5	2.5	7	10	0	0	44	583.9
	other	12	1.250	90.5	5	1.375	90.5	2.5	7	14	0	0	60	794.8
12 x 1 5/8" *	< 1/4"	12	1.375	72.5	5	1.500	72.5	2.5	7	11	0	0	48	693.2
	other	12	1.375	90.5	5	1.500	90.5	2.5	7	14	0	0	60	865.4
12 x 1 3/4" *	< 1/4"	12	1.375	78.5	5	1.625	78.5	2.5	7	12	0	0	52	778.4
	other	12	1.375	96.5	5	1.625	96.5	2.5	7	15	0	0	64	957.0
12 x 1 7/8" *	< 1/4"	12	1.500	84.5	5	1.750	84.5	2.5	7	13	0	0	56	903.8
	other	12	1.500	96.5	5	1.750	96.5	2.5	7	15	0	0	64	1032.2
12 x 2" *	< 1/4"	12	1.500	90.5	5	1.750	90.5	2.5	7	14	0	0	60	968.0
	other	12	1.500	96.5	5	1.750	96.5	2.5	7	15	0	0	64	1032.2
12 x 2 1/8" *	< 1/4"	12	1.625	96.5	5	1.875	96.5	2.5	7	15	0	0	64	1107.5
12 x 2 1/4" *	< 1/4"	12	1.750	102.5	5	1.875	102.5	2.5	7	16	0	0	68	1219.9
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
13 x 3/4"	< 1/4"	13	0.625	48.5	4.5	0.875	48.5	3.25	6.5	7	0	0	32	250.4
	other	13	0.625	78.5	4.5	0.875	78.5	3.25	6.5	12	0	0	52	405.6
13 x 7/8" *	< 1/4"	13	0.750	48.5	5	0.750	48.5	3	7	7	0	0	32	267.6
	other	13	0.750	72.5	5	0.875	72.5	3	7	11	0	0	48	425.9
13 x 1" *	< 1/4"	13	0.875	54.5	5	0.875	54.5	3	7	8	0	0	36	345.2
	other	13	0.750	78.5	5	1.000	78.5	3	7	12	0	0	52	489.0
13 x 1 1/8" *	< 1/4"	13	0.875	54.5	5	1.125	54.5	3	7	8	0	0	36	383.9
	other	13	0.875	84.5	5	1.125	84.5	3	7	13	0	0	56	595.3
13 x 1 1/4" *	< 1/4"	13	1.000	60.5	5	1.250	60.5	3	7	9	0	0	40	475.5
	other	13	1.000	90.5	5	1.250	90.5	3	7	14	0	0	60	711.4
13 x 1 3/8" *	< 1/4"	13	1.125	66.5	5	1.375	66.5	3	7	10	0	0	44	576.9
	other	13	1.125	90.5	5	1.375	90.5	3	7	14	0	0	60	785.2
13 x 1 1/2" *	< 1/4"	13	1.250	72.5	5	1.500	72.5	3	7	11	0	0	48	688.1
	other	13	1.250	96.5	5	1.500	96.5	3	7	15	0	0	64	915.9
13 x 1 5/8" *	< 1/4"	13	1.375	78.5	5	1.625	78.5	3	7	12	0	0	52	809.0
	other	13	1.375	102.5	5	1.625	102.5	3	7	16	0	0	68	1056.5
13 x 1 3/4" *	< 1/4"	13	1.500	90.5	5	1.625	90.5	3	7	14	0	0	60	974.4
	other	13	1.500	102.5	5	1.875	102.5	3	7	16	0	0	68	1176.4
13 x 1 7/8" *	< 1/4"	13	1.500	96.5	5	1.625	96.5	3	7	15	0	0	64	1039.1
	other	13	1.500	102.5	5	1.875	102.5	3	7	16	0	0	68	1176.4
13 x 2" *	< 1/4"	13	1.625	102.5	5	1.750	102.5	3	7	16	0	0	68	1187.3
	other	13	1.625	144.5	5	1.750	144.5	3	7	23	0	0	96	1673.9
13 x 2 1/8" *	< 1/4"	13	1.750	102.5	5	2.125	102.5	3	7	16	0	0	68	1343.5
13 x 2 1/4" *	< 1/4"	13	1.750	144.5	5	2.000	144.5	3	7	23	0	0	96	1842.9
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate	Outer Plate			Inner Plates			Staggered Bolt Pattern						WT./FLG. with 7/8"

	Thick	A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	Bolts (lb)
14 x 3/4"	< 1/4"	14	0.625	27.5	5.5	0.750	27.5	2	5	3	1	2.5	32	163.0
	other	14	0.625	45.5	5.5	0.750	45.5	2	5	6	1	2.5	56	272.5
14 x 7/8" *	< 1/4"	14	0.750	33.5	5.5	0.875	33.5	2	5	4	1	2.5	40	229.2
	other	14	0.750	39.5	5.5	0.875	39.5	2	5	5	1	2.5	48	271.0
14 x 1" *	< 1/4"	14	0.875	33.5	5.5	0.875	33.5	2	5	4	1	2.5	40	245.8
	other	14	0.875	45.5	5.5	1.000	45.5	2	5	6	1	2.5	56	353.2
14 x 1 1/8" *	< 1/4"	14	1.000	39.5	5.5	1.000	39.5	2	5	5	1	2.5	48	325.6
	other	14	1.000	51.5	5.5	1.000	51.5	2	5	7	1	2.5	64	425.9
14 x 1 1/4" *	< 1/4"	14	1.000	39.5	5.5	1.125	39.5	2	5	5	1	2.5	48	347.0
	other	14	1.000	51.5	5.5	1.125	51.5	2	5	7	1	2.5	64	446.0
14 x 1 3/8" *	< 1/4"	14	1.125	39.5	5.5	1.250	39.5	2	5	5	1	2.5	48	376.0
	other	14	1.125	57.5	5.5	1.250	57.5	2	5	8	1	2.5	72	549.4
14 x 1 1/2" *	< 1/4"	14	1.250	45.5	5.5	1.375	45.5	2	5	6	1	2.5	56	474.1
	other	14	1.250	57.5	5.5	1.375	57.5	2	5	8	1	2.5	72	600.3
14 x 1 5/8" *	< 1/4"	14	1.375	51.5	5.5	1.375	51.5	2	5	7	1	2.5	64	562.8
	other	14	1.375	63.5	5.5	1.375	63.5	2	5	9	1	2.5	80	695.0
14 x 1 3/4" *	< 1/4"	14	1.500	51.5	5.5	1.500	51.5	2	5	7	1	2.5	64	608.4
	other	14	1.500	63.5	5.5	1.500	63.5	2	5	9	1	2.5	80	751.2
14 x 1 7/8" *	< 1/4"	14	1.625	57.5	5.5	1.625	57.5	2	5	8	1	2.5	72	730.8
	other	14	1.500	63.5	5.5	1.750	63.5	2	5	9	1	2.5	80	800.8
14 x 2" *	< 1/4"	14	1.625	57.5	5.5	1.750	57.5	2	5	8	1	2.5	72	753.2
	other	14	1.625	63.5	5.5	1.750	63.5	2	5	9	1	2.5	80	832.3
14 x 2 1/8" *	< 1/4"	14	1.750	63.5	5.5	1.875	63.5	2	5	9	1	2.5	80	888.5
14 x 2 1/4" *	< 1/4"	14	1.875	69.5	5.5	1.875	69.5	2	5	10	1	2.5	88	1007.4
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Staggered Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
15 x 3/4"	< 1/4"	15	0.625	33.5	6.5	0.625	33.5	2	6	4	1	2.5	40	204.2
	other	15	0.625	51.5	6.5	0.625	51.5	2	6	7	1	2.5	64	316.4
15 x 7/8"	< 1/4"	15	0.625	33.5	6.5	0.750	33.5	2	6	4	1	2.5	40	219.7
	other	15	0.625	51.5	6.5	0.750	51.5	2	6	7	1	2.5	64	340.1
15 x 1" *	< 1/4"	15	0.750	33.5	6.5	0.875	33.5	2	6	4	1	2.5	40	252.9
	other	15	0.750	45.5	6.5	0.875	45.5	2	6	6	1	2.5	56	345.1
15 x 1 1/8" *	< 1/4"	15	0.875	39.5	6.5	1.000	39.5	2	6	5	1	2.5	48	338.2
	other	15	0.875	51.5	6.5	1.000	51.5	2	6	7	1	2.5	64	442.3
15 x 1 1/4" *	< 1/4"	15	1.000	39.5	6.5	1.000	39.5	2	6	5	1	2.5	48	359.2
	other	15	1.000	57.5	6.5	1.000	57.5	2	6	8	1	2.5	72	524.9
15 x 1 3/8" *	< 1/4"	15	1.125	45.5	6.5	1.125	45.5	2	6	6	1	2.5	56	459.6
	other	15	1.125	57.5	6.5	1.125	57.5	2	6	8	1	2.5	72	582.0
15 x 1 1/2" *	< 1/4"	15	1.250	51.5	6.5	1.250	51.5	2	6	7	1	2.5	64	571.9
	other	15	1.250	63.5	6.5	1.250	63.5	2	6	9	1	2.5	80	706.2
15 x 1 5/8" *	< 1/4"	15	1.250	51.5	6.5	1.375	51.5	2	6	7	1	2.5	64	595.7
	other	15	1.250	63.5	6.5	1.375	63.5	2	6	9	1	2.5	80	735.5
15 x 1 3/4" *	< 1/4"	15	1.375	57.5	6.5	1.375	57.5	2	6	8	1	2.5	72	696.1
	other	15	1.375	63.5	6.5	1.375	63.5	2	6	9	1	2.5	80	769.2
15 x 1 7/8" *	< 1/4"	15	1.500	57.5	6.5	1.500	57.5	2	6	8	1	2.5	72	753.2
	other	15	1.500	69.5	6.5	1.500	69.5	2	6	10	1	2.5	88	911.3
15 x 2" *	< 1/4"	15	1.625	63.5	6.5	1.625	63.5	2	6	9	1	2.5	80	895.3
	other	15	1.625	69.5	6.5	1.625	69.5	2	6	10	1	2.5	88	980.3
15 x 2 1/8" *	< 1/4"	15	1.625	63.5	6.5	1.750	63.5	2	6	9	1	2.5	80	924.5
15 x 2 1/4" *	< 1/4"	15	1.750	69.5	6.5	1.750	69.5	2	6	10	1	2.5	88	1049.3
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total #	

														of Bolts	
16 x 3/4"	< 1/4"	16	0.625	30.5	7	0.625	30.5	2	6	4	1	3	40	200.2	
	other	16	0.625	48.5	7	0.625	48.5	2	6	7	1	3	64	318.7	
16 x 7/8"	< 1/4"	16	0.625	36.5	7	0.750	36.5	2	6	5	1	3	48	257.8	
	other	16	0.625	54.5	7	0.750	54.5	2	6	8	1	3	72	385.2	
16 x 1" *	< 1/4"	16	0.750	30.5	7	0.875	30.5	2	6	4	1	3	40	247.7	
	other	16	0.750	48.5	7	0.875	48.5	2	6	7	1	3	64	394.3	
16 x 1 1/8"	< 1/4"	16	0.875	36.5	7	0.875	36.5	2	6	5	1	3	48	317.3	
	other	16	0.875	54.5	7	0.875	54.5	2	6	8	1	3	72	474.1	
16 x 1 1/4"	< 1/4"	16	1.000	42.5	7	1.000	42.5	2	6	6	1	3	56	414.7	
	other	16	1.000	54.5	7	1.000	54.5	2	6	8	1	3	72	532.0	
16 x 1 3/8"	< 1/4"	16	1.000	42.5	7	1.125	42.5	2	6	6	1	3	56	435.8	
	other	16	1.000	54.5	7	1.125	54.5	2	6	8	1	3	72	559.1	
16 x 1 1/2"	< 1/4"	16	1.125	42.5	7	1.250	42.5	2	6	6	1	3	56	481.0	
	other	16	1.125	60.5	7	1.250	60.5	2	6	9	1	3	80	685.0	
16 x 1 5/8"	< 1/4"	16	1.250	48.5	7	1.250	48.5	2	6	7	1	3	64	576.5	
	other	16	1.250	66.5	7	1.250	66.5	2	6	10	1	3	88	790.7	
16 x 1 3/4"	< 1/4"	16	1.375	54.5	7	1.375	54.5	2	6	8	1	3	72	705.9	
	other	16	1.375	66.5	7	1.375	66.5	2	6	10	1	3	88	861.5	
16 x 1 7/8"	< 1/4"	16	1.500	60.5	7	1.500	60.5	2	6	9	1	3	80	848.0	
	other	16	1.500	66.5	7	1.500	66.5	2	6	10	1	3	88	932.2	
16 x 2" *	< 1/4"	16	1.500	60.5	7	1.500	60.5	2	6	9	1	3	80	848.0	
	other	16	1.500	66.5	7	1.500	66.5	2	6	10	1	3	88	932.2	
16 x 2 1/8"	< 1/4"	16	1.625	66.5	7	1.625	66.5	2	6	10	1	3	88	1002.9	
16 x 2 1/4"	< 1/4"	16	1.625	66.5	7	1.750	66.5	2	6	10	1	3	88	1035.9	
* Bolt threads are EXCLUDED from shear plane															
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						Total # of Bolts	WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)			
17 x 3/4"	< 1/4"	17	0.625	30.5	7	0.750	30.5	2.25	6.5	4	1	3	40	220.7	
	other	17	0.625	48.5	7	0.750	48.5	2.25	6.5	7	1	3	64	351.3	
17 x 7/8" *	< 1/4"	17	0.750	30.5	7	0.750	30.5	2	7	4	1	3	40	239.1	
	other	17	0.750	48.5	7	0.750	48.5	2	7	7	1	3	64	380.6	
17 x 1" *	< 1/4"	17	0.875	36.5	7	0.875	36.5	2	7	5	1	3	48	326.3	
	other	17	0.875	48.5	7	1.000	48.5	2	7	7	1	3	64	457.9	
17 x 1 1/8"	< 1/4"	17	0.875	36.5	7	1.000	36.5	2	7	5	1	3	48	344.5	
	other	17	0.875	54.5	7	1.000	54.5	2	7	8	1	3	72	514.6	
17 x 1 1/4"	< 1/4"	17	1.000	42.5	7	1.000	42.5	2	7	6	1	3	56	426.8	
	other	17	1.000	54.5	7	1.125	54.5	2	7	8	1	3	72	574.5	
17 x 1 3/8"	< 1/4"	17	1.000	42.5	7	1.250	42.5	2	7	6	1	3	56	469.0	
	other	17	1.000	60.5	7	1.250	60.5	2	7	9	1	3	80	667.9	
17 x 1 1/2"	< 1/4"	17	1.125	48.5	7	1.375	48.5	2	7	7	1	3	64	588.6	
	other	17	1.125	60.5	7	1.375	60.5	2	7	9	1	3	80	734.3	
17 x 1 5/8"	< 1/4"	17	1.250	54.5	7	1.375	54.5	2	7	8	1	3	72	694.3	
	other	17	1.250	66.5	7	1.375	66.5	2	7	10	1	3	88	847.3	
17 x 1 3/4"	< 1/4"	17	1.375	60.5	7	1.375	60.5	2	7	9	1	3	80	807.3	
	other	17	1.375	72.5	7	1.375	72.5	2	7	11	1	3	96	967.5	
17 x 1 7/8"	< 1/4"	17	1.500	66.5	7	1.500	66.5	2	7	10	1	3	88	960.5	
	other	17	1.500	72.5	7	1.500	72.5	2	7	11	1	3	96	1047.2	
17 x 2" *	< 1/4"	17	1.500	66.5	7	1.625	66.5	2	7	10	1	3	88	993.5	
	other	17	1.500	72.5	7	1.625	72.5	2	7	11	1	3	96	1083.1	
17 x 2 1/8"	< 1/4"	17	1.625	66.5	7	1.750	66.5	2	7	10	1	3	88	1066.5	
17 x 2 1/4"	< 1/4"	17	1.750	72.5	7	1.750	72.5	2	7	11	1	3	96	1206.5	
* Bolt threads are EXCLUDED from shear plane															
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						Total # of Bolts	WT./FLG. with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)			

18 x 3/4"	< 1/4"	18	0.625	36.5	8	0.625	36.5	2.25	6.5	5	1	3.5	48	265.5
	other	18	0.625	54.5	8	0.625	54.5	2.25	6.5	8	1	3.5	72	396.8
18 x 7/8" *	< 1/4"	18	0.750	30.5	8	0.750	30.5	2.5	7	4	1	3	40	258.5
	other	18	0.750	48.5	8	0.750	48.5	2.5	7	7	1	3	64	411.5
18 x 1" *	< 1/4"	18	0.750	36.5	8	0.875	36.5	2.5	7	5	1	3	48	330.2
	other	18	0.750	54.5	8	0.875	54.5	2.5	7	8	1	3	72	493.4
18 x 1 1/8" *	< 1/4"	18	0.875	42.5	8	0.875	42.5	2.5	7	6	1	3	56	411.7
	other	18	0.875	60.5	8	0.875	60.5	2.5	7	9	1	3	80	586.4
18 x 1 1/4" *	< 1/4"	18	1.000	42.5	8	1.000	42.5	2.5	7	6	1	3	56	463.0
	other	18	1.000	60.5	8	1.125	60.5	2.5	7	9	1	3	80	693.6
18 x 1 3/8" *	< 1/4"	18	1.000	48.5	8	1.125	48.5	2.5	7	7	1	3	64	555.9
	other	18	1.000	66.5	8	1.125	66.5	2.5	7	10	1	3	88	762.5
18 x 1 1/2" *	< 1/4"	18	1.125	48.5	8	1.250	48.5	2.5	7	7	1	3	64	674.4
	other	18	1.125	66.5	8	1.250	66.5	2.5	7	10	1	3	88	842.6
18 x 1 5/8" *	< 1/4"	18	1.250	54.5	8	1.250	54.5	2.5	7	8	1	3	72	725.2
	other	18	1.250	72.5	8	1.250	72.5	2.5	7	11	1	3	96	964.9
18 x 1 3/4" *	< 1/4"	18	1.375	60.5	8	1.375	60.5	2.5	7	9	1	3	80	878.0
	other	18	1.375	72.5	8	1.375	72.5	2.5	7	11	1	3	96	1052.3
18 x 1 7/8" *	< 1/4"	18	1.375	60.5	8	1.500	60.5	2.5	7	9	1	3	80	912.3
	other	18	1.375	72.5	8	1.500	72.5	2.5	7	11	1	3	96	1093.4
18 x 2" *	< 1/4"	18	1.500	66.5	8	1.625	66.5	2.5	7	10	1	3	88	1083.0
	other	18	1.500	72.5	8	1.625	72.5	2.5	7	11	1	3	96	1180.8
18 x 2 1/8" *	< 1/4"	18	1.625	72.5	8	1.625	72.5	2.5	7	11	1	3	96	1227.1
18 x 2 1/4" *	< 1/4"	18	1.750	78.5	8	1.750	78.5	2.5	7	12	1	3	104	1423.3

* Bolt threads are **EXCLUDED** from shear plane

Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
19 x 7/8" *	< 1/4"	19	0.750	36.5	8.5	0.750	36.5	2.5	7	5	1	3.5	48	325.1
	other	19	0.750	54.5	8.5	0.750	54.5	2.5	7	8	1	3.5	72	485.7
19 x 1" *	< 1/4"	19	0.750	36.5	8.5	0.875	36.5	2.5	7	5	1	3.5	48	347.0
	other	19	0.750	54.5	8.5	0.875	54.5	2.5	7	8	1	3.5	72	518.5
19 x 1 1/8" *	< 1/4"	19	0.875	42.5	8.5	0.875	42.5	2.5	7	6	1	3.5	56	432.8
	other	19	0.875	60.5	8.5	0.875	60.5	2.5	7	9	1	3.5	80	616.4
19 x 1 1/4" *	< 1/4"	19	1.000	48.5	8.5	1.000	48.5	2.5	7	7	1	3.5	64	555.9
	other	19	1.000	66.5	8.5	1.000	66.5	2.5	7	10	1	3.5	88	762.5
19 x 1 3/8" *	< 1/4"	19	1.000	48.5	8.5	1.125	48.5	2.5	7	7	1	3.5	64	585.1
	other	19	1.000	66.5	8.5	1.125	66.5	2.5	7	10	1	3.5	88	802.5
19 x 1 1/2" *	< 1/4"	19	1.125	54.5	8.5	1.250	54.5	2.5	7	8	1	3.5	72	727.1
	other	19	1.125	72.5	8.5	1.250	72.5	2.5	7	11	1	3.5	96	967.5
19 x 1 5/8" *	< 1/4"	19	1.250	60.5	8.5	1.250	60.5	2.5	7	9	1	3.5	80	848.0
	other	19	1.250	78.5	8.5	1.250	78.5	2.5	7	12	1	3.5	104	1100.5
19 x 1 3/4" *	< 1/4"	19	1.375	66.5	8.5	1.375	66.5	2.5	7	10	1	3.5	88	1017.0
	other	19	1.375	78.5	8.5	1.375	78.5	2.5	7	12	1	3.5	104	1200.7
19 x 1 7/8" *	< 1/4"	19	1.375	66.5	8.5	1.500	66.5	2.5	7	10	1	3.5	88	1057.1
	other	19	1.375	78.5	8.5	1.500	78.5	2.5	7	12	1	3.5	104	1248.0
19 x 2" *	< 1/4"	19	1.500	66.5	8.5	1.625	66.5	2.5	7	10	1	3.5	88	1142.0
	other	19	1.500	78.5	8.5	1.625	78.5	2.5	7	12	1	3.5	104	1348.1
19 x 2 1/8" *	< 1/4"	19	1.625	78.5	8.5	1.625	78.5	2.5	7	12	1	3.5	104	1401.0
19 x 2 1/4" *	< 1/4"	19	1.750	84.5	8.5	1.750	84.5	2.5	7	13	1	3.5	112	1616.0

* Bolt threads are **EXCLUDED** from shear plane

Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
20 x 7/8" *	< 1/4"	20	0.750	36.5	9	0.750	36.5	2.5	7	5	1	4	48	340.6
	other	20	0.750	54.5	9	0.750	54.5	2.5	7	8	1	4	72	508.8
20 x 1" *	< 1/4"	20	0.750	36.5	9	0.875	36.5	2.5	7	5	1	4	48	363.9
	other	20	0.750	60.5	9	0.875	60.5	2.5	7	9	1	4	80	603.5

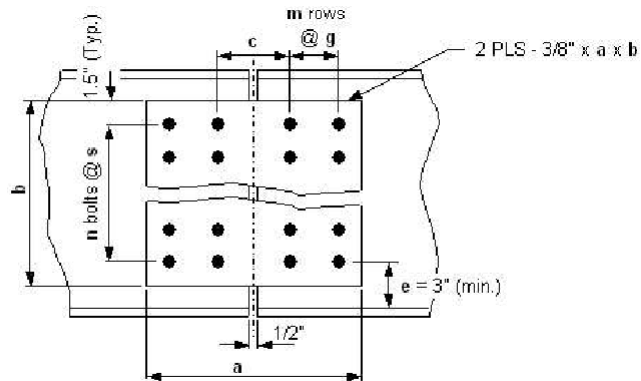
20 x 1 1/8"	< 1/4"	20	0.875	42.5	9	0.875	42.5	2.5	7	6	1	4	56	453.9
*	other	20	0.875	66.5	9	0.875	66.5	2.5	7	10	1	4	88	710.6
20 x 1 1/4"	< 1/4"	20	1.000	48.5	9	1.000	48.5	2.5	7	7	1	4	64	583.4
*	other	20	1.000	72.5	9	1.000	72.5	2.5	7	11	1	4	96	872.4
20 x 1 3/8"	< 1/4"	20	1.000	54.5	9	1.125	54.5	2.5	7	8	1	4	72	690.4
*	other	20	1.000	72.5	9	1.125	72.5	2.5	7	11	1	4	96	918.7
20 x 1 1/2"	< 1/4"	20	1.125	54.5	9	1.250	54.5	2.5	7	8	1	4	72	763.8
*	other	20	1.125	72.5	9	1.250	72.5	2.5	7	11	1	4	96	1016.3
20 x 1 5/8"	< 1/4"	20	1.250	60.5	9	1.250	60.5	2.5	7	9	1	4	80	890.9
*	other	20	1.250	78.5	9	1.250	78.5	2.5	7	12	1	4	104	1156.1
20 x 1 3/4"	< 1/4"	20	1.375	66.5	9	1.375	66.5	2.5	7	10	1	4	88	1068.9
*	other	20	1.375	84.5	9	1.375	84.5	2.5	7	13	1	4	112	1358.4
20 x 1 7/8"	< 1/4"	20	1.375	66.5	9	1.500	66.5	2.5	7	10	1	4	88	1111.3
*	other	20	1.375	78.5	9	1.500	78.5	2.5	7	12	1	4	104	1312.0
20 x 2"	< 1/4"	20	1.500	72.5	9	1.625	72.5	2.5	7	11	1	4	96	1309.3
*	other	20	1.500	78.5	9	1.625	78.5	2.5	7	12	1	4	104	1417.7
20 x 2 1/8"	< 1/4"	20	1.625	78.5	9	1.625	78.5	2.5	7	12	1	4	104	1473.3
*	< 1/4"	20	1.750	84.5	9	1.750	84.5	2.5	7	13	1	4	112	1699.8
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
21 x 7/8"	< 1/4"	21	0.750	36.5	9.5	0.750	36.5	2.5	7	5	1	4.5	48	356.1
*	other	21	0.750	60.5	9.5	0.750	60.5	2.5	7	9	1	4.5	80	590.7
21 x 1"	< 1/4"	21	0.750	42.5	9.5	0.875	42.5	2.5	7	6	1	4.5	56	443.4
*	other	21	0.750	60.5	9.5	0.875	60.5	2.5	7	9	1	4.5	80	631.4
21 x 1 1/8"	< 1/4"	21	0.875	48.5	9.5	0.875	48.5	2.5	7	7	1	4.5	64	542.2
*	other	21	0.875	66.5	9.5	0.875	66.5	2.5	7	10	1	4.5	88	743.6
21 x 1 1/4"	< 1/4"	21	1.000	54.5	9.5	1.000	54.5	2.5	7	8	1	4.5	72	686.6
*	other	21	1.000	72.5	9.5	1.000	72.5	2.5	7	11	1	4.5	96	913.5
21 x 1 3/8"	< 1/4"	21	1.000	54.5	9.5	1.125	54.5	2.5	7	8	1	4.5	72	723.3
*	other	21	1.000	72.5	9.5	1.125	72.5	2.5	7	11	1	4.5	96	962.4
21 x 1 1/2"	< 1/4"	21	1.125	60.5	9.5	1.250	60.5	2.5	7	9	1	4.5	80	888.7
*	other	21	1.125	78.5	9.5	1.250	78.5	2.5	7	12	1	4.5	104	1153.4
21 x 1 5/8"	< 1/4"	21	1.250	66.5	9.5	1.250	66.5	2.5	7	10	1	4.5	88	1026.5
*	other	21	1.250	84.5	9.5	1.250	84.5	2.5	7	13	1	4.5	112	1304.5
21 x 1 3/4"	< 1/4"	21	1.375	72.5	9.5	1.375	72.5	2.5	7	11	1	4.5	96	1221.9
*	other	21	1.375	84.5	9.5	1.375	84.5	2.5	7	13	1	4.5	112	1424.3
21 x 1 7/8"	< 1/4"	21	1.375	72.5	9.5	1.500	72.5	2.5	7	11	1	4.5	96	1270.7
*	other	21	1.375	84.5	9.5	1.500	84.5	2.5	7	13	1	4.5	112	1481.2
21 x 2"	< 1/4"	21	1.500	78.5	9.5	1.625	78.5	2.5	7	12	1	4.5	104	1487.3
*	other	21	1.500	84.5	9.5	1.625	84.5	2.5	7	13	1	4.5	112	1601.0
21 x 2 1/8"	< 1/4"	21	1.625	84.5	9.5	1.625	84.5	2.5	7	13	1	4.5	112	1663.9
*	< 1/4"	21	1.750	90.5	9.5	1.750	90.5	2.5	7	14	1	4.5	120	1910.4
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
22 x 1"	< 1/4"	22	0.750	30.5	10	0.875	30.5	2	6	4	2	3	60	351.1
*	other	22	0.750	42.5	10	0.875	42.5	2	6	6	2	3	84	489.6
22 x 1 1/8"	< 1/4"	22	0.875	30.5	10	0.875	30.5	2	6	4	2	3	60	374.8
*	other	22	0.875	48.5	10	0.875	48.5	2	6	7	2	3	96	596.6
22 x 1 1/4"	< 1/4"	22	1.000	36.5	10	1.000	36.5	2	6	5	2	3	72	503.1
*	other	22	1.000	48.5	10	1.000	48.5	2	6	7	2	3	96	668.8
22 x 1 3/8"	< 1/4"	22	1.000	36.5	10	1.125	36.5	2	6	5	2	3	72	529.0
*	other	22	1.000	48.5	10	1.125	48.5	2	6	7	2	3	96	703.2
22 x 1 1/2"	< 1/4"	22	1.125	42.5	10	1.250	42.5	2	6	6	2	3	84	679.4
*	other	22	1.125	54.5	10	1.250	54.5	2	6	8	2	3	108	871.5

22 x 1 5/8"	< 1/4"	22	1.250	42.5	10	1.250	42.5	2	6	6	2	3	84	712.5
*	other	22	1.250	54.5	10	1.250	54.5	2	6	8	2	3	108	913.9
22 x 1 3/4"	< 1/4"	22	1.250	48.5	10	1.375	48.5	2	6	7	2	3	96	847.6
*	other	22	1.250	54.5	10	1.375	54.5	2	6	8	2	3	108	952.6
22 x 1 7/8"	< 1/4"	22	1.375	48.5	10	1.500	48.5	2	6	7	2	3	96	919.8
*	other	22	1.375	60.5	10	1.500	60.5	2	6	9	2	3	120	1147.6
22 x 2"	< 1/4"	22	1.500	54.5	10	1.625	54.5	2	6	8	2	3	108	1114.9
*	other	22	1.500	60.5	10	1.625	60.5	2	6	9	2	3	120	1237.7
22 x 2 1/8"	< 1/4"	22	1.500	54.5	10	1.625	54.5	2	6	8	2	3	108	1114.9
*	< 1/4"	22	1.625	60.5	10	1.750	60.5	2	6	9	2	3	120	1327.8
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
23 x 1"	< 1/4"	23	0.750	30.5	10.5	0.875	30.5	2.25	6.5	4	2	3	60	365.1
*	other	23	0.750	42.5	10.5	0.875	42.5	2.25	6.5	6	2	3	84	509.1
23 x 1 1/8"	< 1/4"	23	0.875	30.5	10.5	0.875	30.5	2.25	6.5	4	2	3	60	390.0
*	other	23	0.875	48.5	10.5	0.875	48.5	2.25	6.5	7	2	3	96	620.7
23 x 1 1/4"	< 1/4"	23	1.000	36.5	10.5	1.000	36.5	2.25	6.5	5	2	3	72	523.8
*	other	23	1.000	48.5	10.5	1.000	48.5	2.25	6.5	7	2	3	96	696.3
23 x 1 3/8"	< 1/4"	23	1.000	42.5	10.5	1.125	42.5	2.25	6.5	6	2	3	84	641.7
*	other	23	1.000	54.5	10.5	1.125	54.5	2.25	6.5	8	2	3	108	823.2
23 x 1 1/2"	< 1/4"	23	1.125	42.5	10.5	1.250	42.5	2.25	6.5	6	2	3	84	708.0
*	other	23	1.125	54.5	10.5	1.250	54.5	2.25	6.5	8	2	3	108	908.2
23 x 1 5/8"	< 1/4"	23	1.250	48.5	10.5	1.250	48.5	2.25	6.5	7	2	3	96	847.6
*	other	23	1.250	60.5	10.5	1.250	60.5	2.25	6.5	9	2	3	120	1057.6
23 x 1 3/4"	< 1/4"	23	1.250	48.5	10.5	1.375	48.5	2.25	6.5	7	2	3	96	883.7
*	other	23	1.250	60.5	10.5	1.375	60.5	2.25	6.5	9	2	3	120	1102.6
23 x 1 7/8"	< 1/4"	23	1.375	54.5	10.5	1.500	54.5	2.25	6.5	8	2	3	108	1078.2
*	other	23	1.375	60.5	10.5	1.500	60.5	2.25	6.5	9	2	3	120	1197.0
23 x 2"	< 1/4"	23	1.500	54.5	10.5	1.625	54.5	2.25	6.5	8	2	3	108	1163.2
*	other	23	1.500	60.5	10.5	1.625	60.5	2.25	6.5	9	2	3	120	1291.3
23 x 2 1/8"	< 1/4"	23	1.500	60.5	10.5	1.625	60.5	2.25	6.5	9	2	3	120	1291.3
*	< 1/4"	23	1.625	60.5	10.5	1.750	60.5	2.25	6.5	9	2	3	120	1385.7
* Bolt threads are EXCLUDED from shear plane														
Flange	Filler Plate Thick.	Outer Plate			Inner Plates			Uniform Bolt Pattern						WT./FLG with 7/8" Bolts (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J (in)	K (in)	L (in)	Total # of Bolts	
24 x 1"	< 1/4"	24	0.750	30.5	11	0.875	30.5	2.5	7	4	2	3	60	379.2
*	other	24	0.750	48.5	11	0.875	48.5	2.5	7	7	2	3	96	603.5
24 x 1 1/8"	< 1/4"	24	0.875	36.5	11	0.875	36.5	2.5	7	5	2	3	72	485.0
*	other	24	0.875	48.5	11	0.875	48.5	2.5	7	7	2	3	96	644.8
24 x 1 1/4"	< 1/4"	24	1.000	36.5	11	1.000	36.5	2.5	7	5	2	3	72	544.5
*	other	24	1.000	54.5	11	1.000	54.5	2.5	7	8	2	3	108	813.5
24 x 1 3/8"	< 1/4"	24	1.000	42.5	11	1.125	42.5	2.5	7	6	2	3	84	667.3
*	other	24	1.000	54.5	11	1.125	54.5	2.5	7	8	2	3	108	856.0
24 x 1 1/2"	< 1/4"	24	1.125	42.5	11	1.250	42.5	2.5	7	6	2	3	84	736.6
*	other	24	1.125	60.5	11	1.250	60.5	2.5	7	9	2	3	120	1049.0
24 x 1 5/8"	< 1/4"	24	1.250	48.5	11	1.250	48.5	2.5	7	7	2	3	96	882.0
*	other	24	1.250	60.5	11	1.250	60.5	2.5	7	9	2	3	120	1100.5
24 x 1 3/4"	< 1/4"	24	1.250	48.5	11	1.375	48.5	2.5	7	7	2	3	96	919.8
*	other	24	1.250	60.5	11	1.375	60.5	2.5	7	9	2	3	120	1147.6
24 x 1 7/8"	< 1/4"	24	1.375	54.5	11	1.500	54.5	2.5	7	8	2	3	108	1122.6
*	other	24	1.375	60.5	11	1.500	60.5	2.5	7	9	2	3	120	1246.3
24 x 2"	< 1/4"	24	1.500	60.5	11	1.625	60.5	2.5	7	9	2	3	120	1344.9
*	other	24	1.500	66.5	11	1.625	66.5	2.5	7	10	2	3	132	1478.4
24 x 2 1/8"	< 1/4"	24	1.500	60.5	11	1.625	60.5	2.5	7	9	2	3	120	1344.9

24 x 2 1/4"	< 1/4"	24	1.625	66.5	11	1.750	66.5	2.5	7	10	2	3	132	1586.8
* Bolt threads are EXCLUDED from shear plane														

751.14.3.3 Plate Girder - Web Splice Tables

Web Plate Depth: 36" thru 96" (4 or 6 rows of Bolts), ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices



Note: Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Uniformity of web plate thicknesses, which would eliminate the need for the minimum thickness of shim plate, should also be considered.

Web Dimensions		Splice Plates		Bolts						WT./Splice with 7/8" Bolts (lb)
Web Depth (in)	Web Thickness (in)	Plate Width a (in)	Plate Depth b (in)	# of rows (m)	# of bolts per row (n)	Spacing				
						pitch (s) (in)	gage (g) (in)	center (c) (in)	clearance (e) (in)	
36	3/8	14	31	2	9	3.5	3	5	4	126.5
	7/16	13.5	33	2	11	3	3	4.5	3	136.5
38	3/8	14	35	2	9	4	3	5	3	138.4
	7/16	14	34.5	2	10	3.5	3	5	3.25	140.7
40	3/8	14	35	2	9	4	3	5	4	138.4
	7/16	13.5	36	2	12	3	3	4.5	3.5	149.0
	1/2	19.5	35	3	9	4	3	4.5	4	196.4
42	3/8	13.5	38	2	11	3.5	3	4.5	3.5	150.9
	7/16	13.5	38	2	11	3.5	3	4.5	3.5	150.9
	1/2	13.5	39	2	13	3	3	4.5	3	161.4
44	3/8	14	39	2	9	4.5	3	5	4	150.3
	7/16	14	38	2	11	3.5	3	5	4.5	154.9
	1/2	13.5	39	2	13	3	3	4.5	4	161.4
46	3/8	14	43	2	9	5	3	5	3	162.2
	7/16	13.5	43	2	11	4	3	4.5	3	165.3
	1/2	13.5	42	2	14	3	3	4.5	3.5	173.8
	9/16	19	43	3	11	4	3	4	3	236.5
48	3/8	14	43	2	9	5	3	5	4	162.2
	7/16	14	43	2	11	4	3	5	4	169.8
	1/2	13.5	45	2	13	3.5	3	4.5	3	178.6
	9/16	19	43	3	11	4	3	4	4	236.5
Web Dimensions		Splice Plates		Bolts						WT./Splice with 7/8" Bolts (lb)
Web Depth (in)	Web Thickness (in)	Plate Width a (in)	Plate Depth b (in)	# of rows (m)	# of bolts per row (n)	Spacing				
						pitch (s) (in)	gage (g) (in)	center (c) (in)	clearance (e) (in)	
50	3/8	14	43.5	2	10	4.5	3	5	4.75	167.5
	7/16	13.5	47	2	12	4	3	4.5	3	180.5
	1/2	13.5	45	2	15	3	3	4.5	4	186.2
	9/16	19	47	3	12	4	3	4	3	258.3
52	3/8	14	48	2	10	5	3	5	3.5	180.9
	7/16	14	48	2	11	4.5	3	5	3.5	184.7

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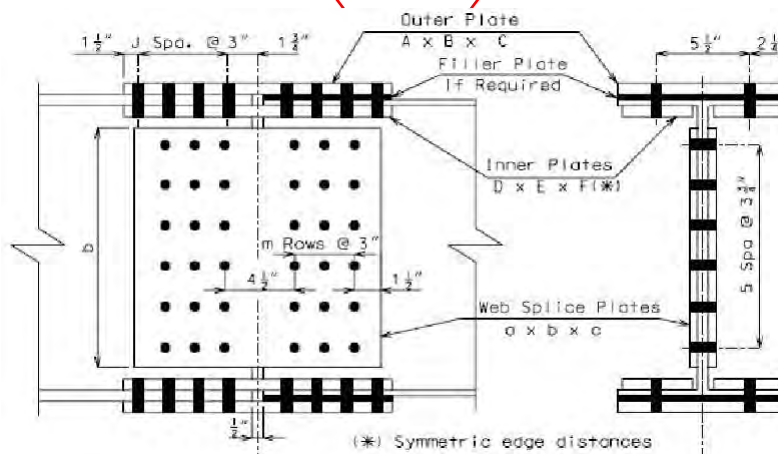
	11/16	19	79	3	20	4	3	4	3	433.2
84	9/16	13	80	2	23	3.5	3	4	3.5	308.6
	5/8	13	78	2	26	3	3	4	4.5	314.5
	11/16	13	81	2	27	3	3	4	3	326.5
86	5/8	13	81	2	27	3	3	4	4	326.5
	11/16	19	79	3	20	4	3	4	5	433.2
88	5/8	13	81	2	27	3	3	4	5	326.5
	11/16	13	84	2	28	3	3	4	3.5	338.6
90	5/8	13	87	2	25	3.5	3	4	3	335.5
	11/16	13	87	2	29	3	3	4	3	350.7
92	5/8	13	84	2	28	3	3	4	5.5	338.6
	11/16	19	88.5	3	20	4.5	3	4	3.25	471.6
94	5/8	13	90.5	2	26	3.5	3	4	3.25	349.0
	11/16	13	90	2	30	3	3	4	3.5	362.8
96	5/8	13	90	2	30	3	3	4	4.5	362.8
	11/16	13	93	2	31	3.5	3	4	3	374.9

751.14.3.4 Wide Flange Splice Tables

Note: In the 15th edition of the AISC Steel Construction Manual (2017) the k1 values for many w-shapes increased. The k1 dimension is the distance between the centerline of web and the toe of the fillet into the flange. The inner plates below were designed before this change and as designed may encroach on the toe of the flange fillet. Before using one of the tabulated wide flange splice designs the inner plate shall be checked to ensure it does not encroach on the toe of the flange fillet.

27" Wide Flange Beams

ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices



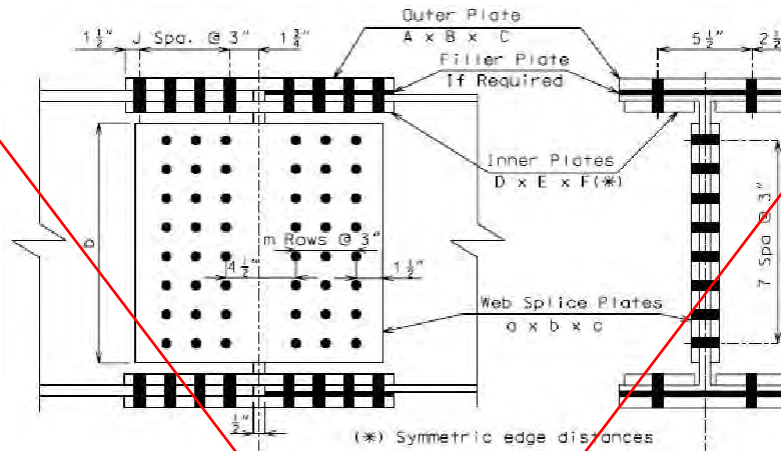
Note: Bottom flange splice plates are the same as the top. Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Use the smallest beam size to determine splice details.

Size of Beam	Filler Plate Thick.	Outer Plate			Inner Plates				Web Splice Plates				Total Number of Bolts	Weight (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	J	a (in)	b (in)	c (in)	m		
W27x84**	< 1/4"	10	0.375	18.5	3.5	0.5	18.5	2	19.5	21.75	0.4375	3	60	238.3
	other	10	0.375	24.5	3.5	0.5	24.5	3	19.5	21.75	0.4375	3	68	270.6
W27x94**	< 1/4"	10	0.5	24.5	3.5	0.625	24.5	3	25.5	21.75	0.5	4	80	363.5
	other	10	0.5	30.5	3.5	0.625	30.5	4	25.5	21.75	0.5	4	88	403.0
W27x102**	< 1/4"	10	0.5	24.5	3.5	0.625	24.5	3	25.5	21.75	0.5625	4	80	383.2
	other	Design by Hand												
W27x114**	< 1/4"	10	0.5	24.5	3.5	0.75	24.5	3	25.5	21.75	0.5625	4	80	395.4
	other	Design by Hand												
W27x129**	< 1/4"	10	0.5625	30.5	3.5	0.875	30.5	4	31.5	21.75	0.6875	5	100	565.4

other	----- Design by Hand -----
** Denotes flange widths less than 12" where P/S Panel placement should be thoroughly investigated.	

30" Wide Flange Beams

ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices

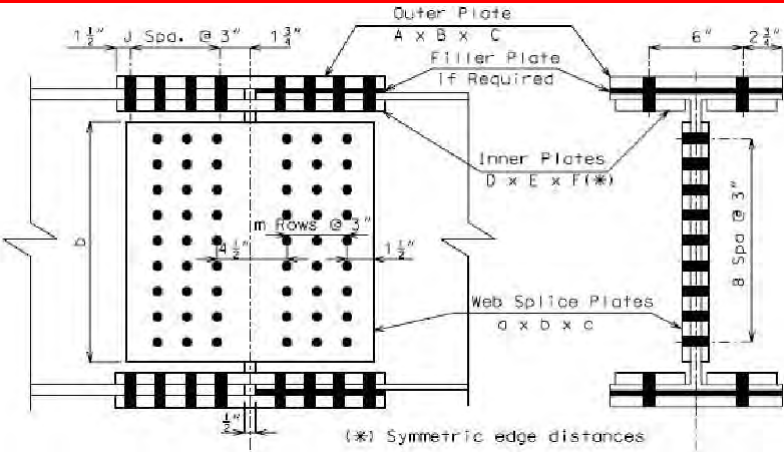


Note: Bottom flange splice plates are the same as the top. Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Use the smallest beam size to determine splice details.

Size of Beam	Filler Plate Thick.	Outer Plate			Inner Plates				Web Splice Plates				Total Number of Bolts	Weight (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	J	a (in)	b (in)	c (in)	m		
W30x90**	< 1/4"	10.5	0.375	18.5	3.5	0.5	18.5	2	19.5	24	0.4375	3	72	262.6
	other	10.5	0.375	30.5	3.5	0.5	30.5	4	19.5	24	0.4375	3	88	328.4
W30x99**	< 1/4"	10.5	0.375	24.5	3.5	0.5	24.5	3	19.5	24	0.5	3	80	312.0
	other	10.5	0.375	30.5	3.5	0.5	30.5	4	19.5	24	0.5	3	88	345.0
W30x108**	< 1/4"	10.5	0.5	24.5	3.5	0.625	24.5	3	19.5	24	0.5	3	80	342.4
	other	10.5	0.5	36.5	3.5	0.625	36.5	5	19.5	24	0.5	3	96	423.1
W30x116**	< 1/4"	10.5	0.5	30.5	3.5	0.75	30.5	4	25.5	24	0.5625	4	104	475.7
	other	10.5	0.5	30.5	3.5	0.75	30.5	4	25.5	24	0.5625	4	104	475.7
W30x124**	< 1/4"	10.5	0.5	30.5	3.5	0.75	30.5	4	25.5	24	0.5625	4	104	475.7
	other	10.5	0.5	30.5	3.5	0.75	30.5	4	25.5	24	0.5625	4	104	475.7
W30x132**	< 1/4"	10.5	0.5	30.5	3.5	0.75	30.5	4	25.5	24	0.625	4	104	497.4
	other	Design by Hand												
W30x148**	< 1/4"	10.5	0.625	36.5	3.5	0.875	36.5	5	25.5	24	0.625	4	112	586.0
	other	Design by Hand												
** Denotes flange widths less than 12" where P/S Panel placement should be thoroughly investigated.														

33" Wide Flange Beams

ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices

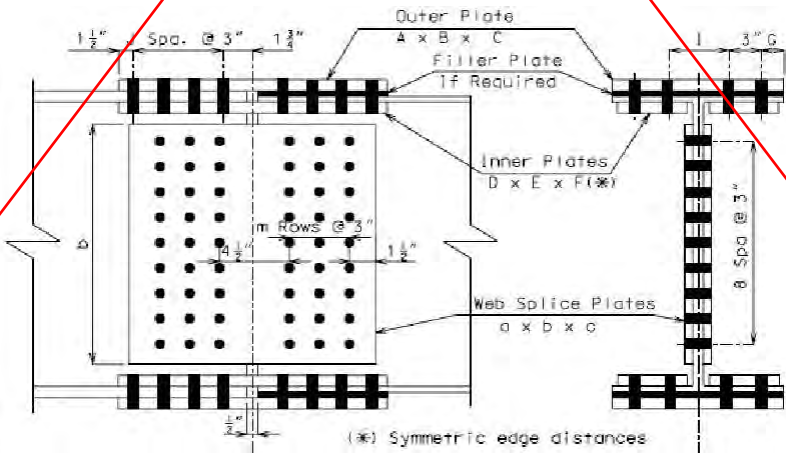


Note: Bottom flange splice plates are the same as the top. Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Use the smallest beam size to determine splice details.

Size of Beam	Filler Plate Thick.	Outer Plate			Inner Plates				Web Splice Plates				Total Number of Bolts	Weight (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	J	a (in)	b (in)	c (in)	m		
W33x118**	< 1/4"	11.5	0.5	30.5	3.5	0.625	30.5	4	19.5	27	0.5	3	94	413.7
	other	11.5	0.5	36.5	3.5	0.625	36.5	5	19.5	27	0.5	3	102	455.8
W33x130**	< 1/4"	11.5	0.5	30.5	3.5	0.75	30.5	4	25.5	27	0.5625	4	112	516.3
	other	11.5	0.5	42.5	3.5	0.75	42.5	6	25.5	27	0.5625	4	128	606.4
W33x141**	< 1/4"	11.5	0.625	36.5	3.5	0.875	36.5	5	25.5	27	0.5625	4	120	609.2
	other	11.5	0.625	42.5	3.5	0.875	42.5	6	25.5	27	0.5625	4	128	662.1
W33x152**	< 1/4"	11.5	0.625	36.5	3.5	0.875	36.5	5	25.5	27	0.625	4	120	633.6
	other	Design by Hand												
W33x169**	< 1/4"	11.5	0.625	36.5	3.5		36.5	5	25.5	27	0.625	4	120	651.7
	other	Design by Hand												
** D denotes flange widths less than 12" where P/S Panel placement should be thoroughly investigated.														

33" Wide Flange Beams

ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices

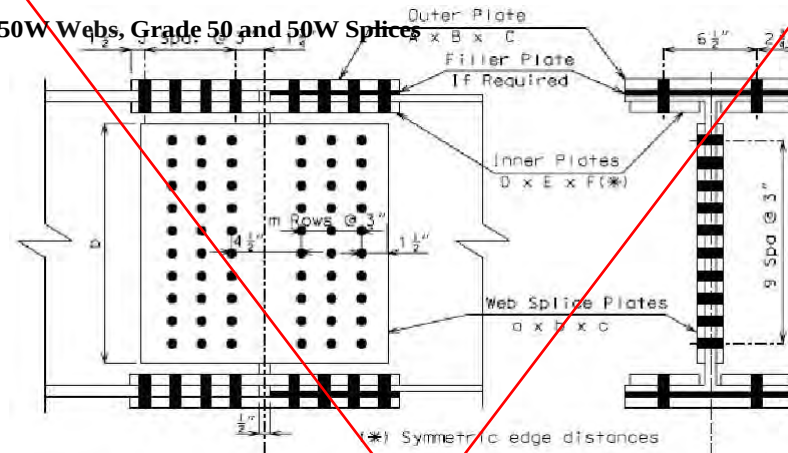


Note: Bottom flange splice plates are the same as the top. Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Use the smallest beam size to determine splice details.

Size of Beam	Filler Plate	Outer Plate			Inner Plates					Web Splice Plates				Total Number	Weight (lb)	
		(A)	(B)	(C)	(D)	(E)	(F)	G (in)	I (in)	J	(la)	(lb)	(lc)			m
W33x201	Thick. < 1/4"	75	0.625	24.5		0.875	24.5					0.875				
		15.			6			2	5.75	3	24.5	27	.6	4	136	669.8
	other	75			6	0.875	30.5							5		
		Design by Hand														
W33x221	other	15.	0.75	30.5				2	5.75	4	30.5	27	0.75		170	897.7
		Design by Hand														

36" Wide Flange Beams

ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices

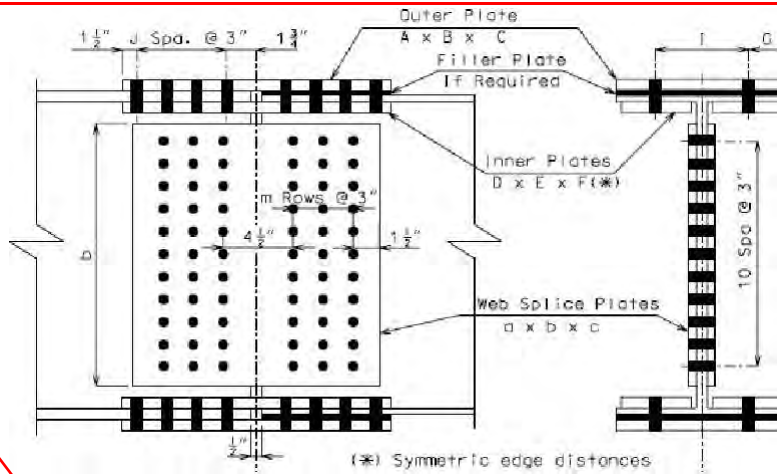


Note: Bottom flange splice plates are the same as the top. Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Use the smallest beam size to determine splice details.

smallest beam size to determine splice details.															
Size of Beam	Filler Plate Thick.	Outer Plate			Inner Plates					Web Splice Plates				Total Number of Bolts	Weight (lb)
		A	B	C	d	e	f	g	h	i	j	k			
		(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)	(in)			
		12	0.625	0.625	4		0.625							3	
W36x135	< 1/4"			30.5	4		30.5			19.5	30	0.5	4	100	477.1
	other			42.5	4	0.625	42.5			19.5	30	0.5	4	116	577.4
W36x150	< 1/4"		0.625	36.5	4	0.75	36.5			25.5	30	0.5625	4	128	645.1
	other			42.5		0.75	42.5			25.5	30	0.5625		136	698.6
W36x160	< 1/4"	12		36.5		0.875	36.5			25.5	30	0.5625	4	128	665.8
	other	Design by Hand													
W36x170	< 1/4"	12		36.5		0.875	36.5	5	25.5	30	0.5625		4	128	665.8
	other	Design by Hand													
W36x182	< 1/4"	12	0.75	42.5	4		42.5	6	25.5	30	0.625			136	810.1
	other	Design by Hand													
W36x194	< 1/4"	12	0.75	42.5	4		42.5	6	25.5	30	0.625			136	810.1
	other	Design by Hand													

40" Wide Flange Beams

ASTM A709, Grade 50 and 50W Webs, Grade 50 and 50W Splices



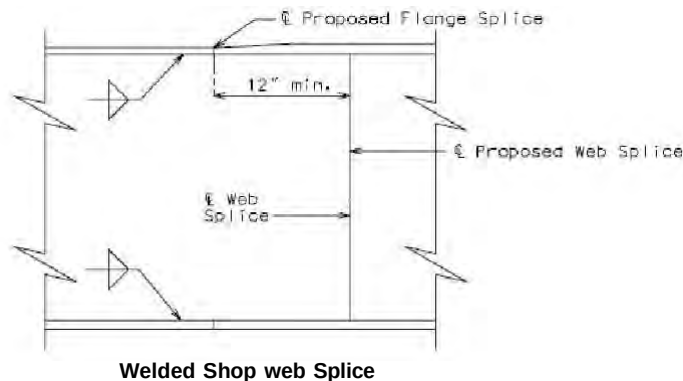
Note: Bottom flange splice plates are the same as the top. Provide shim plates when connecting webs that differ in thickness by 1/8" or more. Use an equal number of shim plates on each side of the web. Use the smallest beam size to determine splice details.

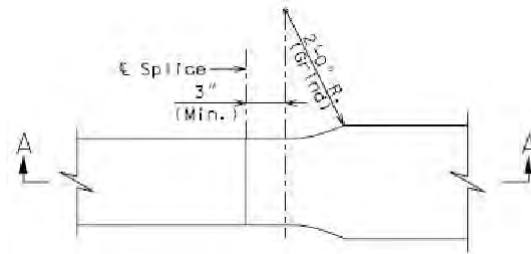
Size of Beam	Filler Plate Thick.	Outer Plate			Inner Plates						Web Splice Plates				Total Number of Bolts	Weight (lb)
		A (in)	B (in)	C (in)	D (in)	E (in)	F (in)	G (in)	I (in)	J	a (in)	b (in)	c (in)	m		
W40x149**	< 1/4"	11.75	0.625	30.5	3.5	0.75	30.5	2.625	6.5	4	19.5	33	0.4375	3	106	478.2
	other	11.75	0.625	42.5	3.5	0.75	42.5	2.625	6.5	6	19.5	33	0.4375	3	122	579.1
W40x167**	< 1/4"	11.75	0.75	36.5	3.5	1	36.5	2.5	6.75	5	25.5	33	0.5	4	136	695.1
	other	11.75	0.75	48.5	3.5	1	48.5	2.5	6.75	7	25.5	33	0.5	4	152	818.0
W40x183**	< 1/4"	11.75	0.75	42.5	3.5	1.125	42.5	2.5	6.75	6	25.5	33	0.5	4	144	777.6
	other	11.75	0.75	42.5	3.5	1.125	42.5	2.5	6.75	6	25.5	33	0.5	4	144	777.6
W40x199***	< 1/4"	15.75	0.625	24.5	5.75	0.75	24.5	2	6.25	3	25.5	33	0.5	4	152	639.6
	other	Design by Hand														
W40x211**	< 1/4"	11.75	0.75	48.5	3.5	1.25	48.5	2.5	6.75	7	25.5	33	0.625	4	152	925.7
	other	Design by Hand														
W40x215***	< 1/4"	15.75	0.625	30.5	5.75	0.875	30.5	2	6.25	4	25.5	33	0.5625	4	168	772.4
	other	Design by Hand														
** Denotes flange widths less than 12" where P/S Panel placement should be thoroughly investigated.																
*** Outer splice plates with widths of 15 3/4" have 4 rows of flange bolts in lieu of 2 rows per drawing.																

751.14.3.5 Shop Welded Splices

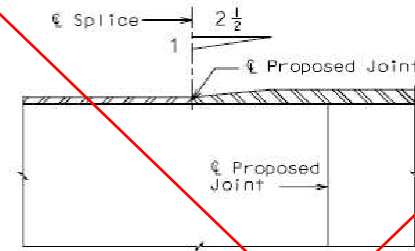
Reference: LRFD Figure 6.13.6.2-1

The following details and note shall be provided on all plate girder bridges.



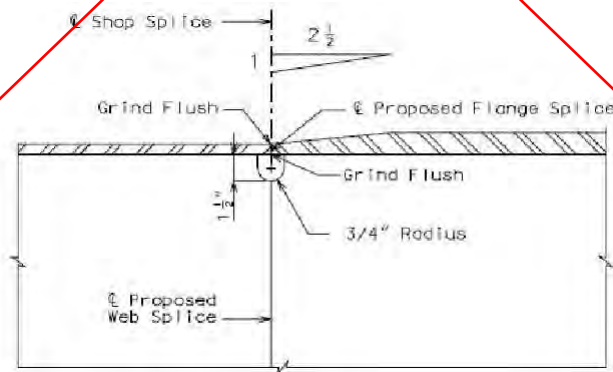
**Plan**

2'-0" Radius Transition

**Section A-A**

Welded Shop Flange Splice

Note: Welded shop web and flange splices may be permitted when detailed on the shop drawings and approved by the engineer. No additional payment will be made for optional welded shop web and flange splices.

**Welded Shop Splice**

Notes: See Structural Project Manager before utilizing this detail.

It is recommended to avoid this detail if possible.

751.14.4 Composite Design

751.14.4.1 General

Overview

The pitch of shear connectors shall be determined to satisfy the fatigue limit state, and the resulting number of shear connectors shall not be less than the number required to satisfy the strength limit state.

Cross-section properties are based on the transformed short-term composite section. Cross-section properties of the composite section shall include concrete slab and steel section. Longitudinal deck reinforcement shall not be considered to be a part of the composite section. Cross-section properties of the non-composite section shall include the steel section only. Negative flexure regions in horizontally curved girders may include the steel reinforcement in the composite section.

The stresses of composite girders and slab shall be computed using the composite cross-section properties and shall be consistent with the properties of the various materials used. The ratio of modulus of elasticity of steel to that of concrete, n , shall be taken to be 8 when using deck concrete with $f'_c = 4$ KSI.

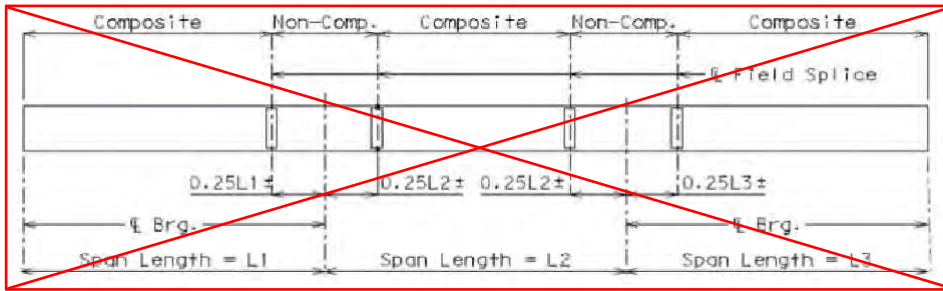
The effective flange widths for calculating composite sections for interior and exterior girders should be determined by LRFD 4.6.2.6 prior to beginning shear connector design. In order to have a uniform and conservative shear connector design for all girders, the smaller effective flange width, between the interior and exterior girders, should only be used to locate the acceptable areas for placement of additional connectors near point of dead load contraflexure. The larger effective flange width should be used in all other calculations.

An effective slab thickness shall be used for composite design by deducting from the actual slab thickness a 1" integral, sacrificial wearing surface.

New Bridges or Replacement of Existing Superstructures

The regions of the bridge subjected to positive flexure are to be designed as composite and the regions subjected to negative flexure are to be designed as non-composite. Simple span composite bridges shall be provided with shear connectors throughout the length of the span. Continuous composite bridges are to be designed with shear connectors provided in the positive flexure regions only, with additional connectors required in the regions of permanent load contraflexure. Horizontally curved girders require shear connectors throughout the girder length.

For the initial girder design, composite/noncomposite regions can be approximately assumed as:



Redeckings

If existing bridge is partially composite, then leave as partially composite unless making full-length composite will reduce or eliminate a load posting subject to Structural Project Manager or Structural Liaison Engineer approval. If the existing bridge is not composite, then make it full-length composite since this will give the maximum capacity of the bridge for the next service life cycle. This guidance applies to either LRFD or LFD redecking designs. Shear connector design should follow same design code used for redecking.

Design Loading

The girder shear force range due to fatigue loading, V_{sR} , should be determined from structural analysis prior to beginning shear connector design. The shear force range is the difference in the minimum and maximum shear force envelopes at any point. For design, use the average shear force range over the effective span (i.e. between contraflexure points). For horizontally curved girders the lateral component of the fatigue force range should be considered.

The stress range in the longitudinal reinforcement due to fatigue loading, f_{sr} , should be determined prior to beginning shear stud design. In lieu of more accurate computation, f_{sr} may be assumed to be 10 ksi.

- Note: This assumption is not provided by AASHTO LRFD, but is held over from AASHTO LFD Specifications.

Other Considerations

~~Shear connectors shall not be placed on the flange splice plates. Maintain at least 3" from end of splice plate to nearest connector for fabrication purposes.~~

The effect of the shear connector on the fatigue resistance of the flange shall be investigated using the provisions of LRFD 6.6.1.2.

Note: All spacing dimensions are measured to center of shear connector

751.14.4.2 Shear Connector Limits

Shear Connector Proportions

Shear connector height, h_s , shall satisfy the following cover and deck penetration provisions:

Minimum shear connector penetration into deck:

- For cast in place deck: 2 in.
- For prestressed panel option: minimum penetration to top of panel

Minimum concrete clear cover over shear connectors = 3 in.

Choose a shear connector height that is acceptable for both SIP and CIP deck options. Use 1 in. increments for selecting shear connector height. Minimum shear connector height = 4 in.

Shear connector diameter, d , shall satisfy:

$$\frac{h}{d} \geq 4$$

Recommended shear connector diameters, d :

- 3/4 in.
- 7/8 in.

Shear Connector Configuration

The following transverse spacing provisions must be satisfied when determining the number of shear connectors per row, n :

Minimum transverse shear connector spacing:

- Preferred minimum: $4d$
- Absolute minimum: 2 1/4 in.

Minimum distance between edge of top flange and nearest shear connector:

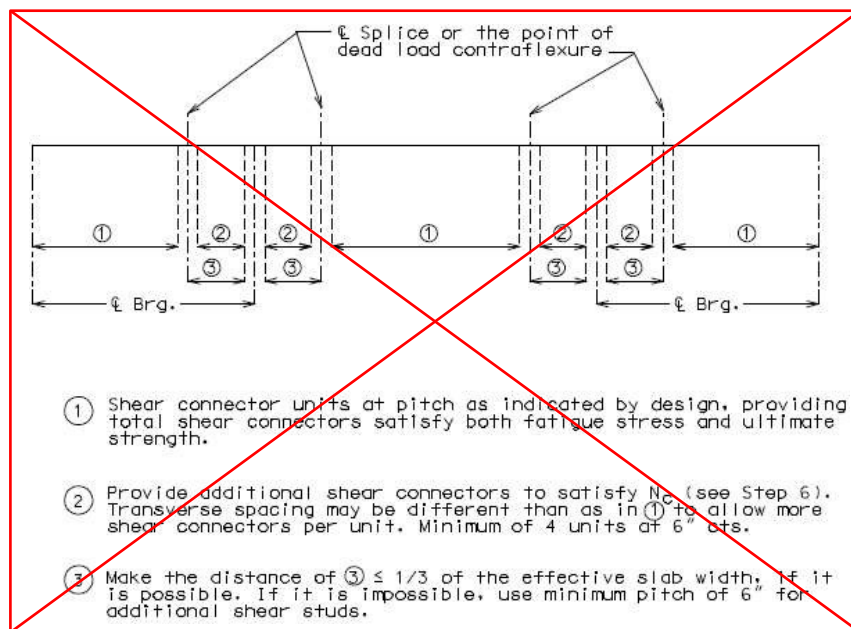
- For cast in place deck: 1 1/2 in.
- For prestressed panel option: 4 1/4 in.

Shear connectors shall be placed by units. A shear connector unit refers to all shear connectors in a given cross-section, spaced transversely across the top flange. Multiple shear connectors per unit may be used if spacing and clearances allow. ~~For horizontally curved girders, when practical, a consistent unit layout and pitch shall be used throughout the span.~~

Shear Connector Spacing

If it becomes impractical to place the number of shear connectors required by ultimate strength in the specified distance (structures with span ratios greater than 1.5); base the number and spacing of shear connectors on the fatigue analysis only.

For a typical 3-span bridge, the shear connector units can be approximately arranged as below:



751.14.4.3 Fatigue Design

Fatigue Resistance

The number of fatigue cycles for design, N , shall be determined as:

$$N = (365)(75)(nc)(ADTTsL)$$

Where:

nc = number of stress range cycles per truck passage

$ADTTsL$ = single lane average daily truck traffic

For longitudinal members not near interior supports*:

$nc = 2.0$ for spans ≤ 40 ft.

$nc = 1.0$ for spans > 40 ft.

- See LRFD Table 6.6.1.2.5-2 for other situations.

In absence of better information, the single lane average daily truck traffic shall be taken as:

$$ADTTsL = (ptr)(ADTT)$$

Where:

$ADTT$ = number of trucks per day in one direction, averaged over the design life

Ptr = fraction of truck traffic in a single lane

Number of Lanes Available to Trucks	Ptr
	1.00
2	0.85
3 or more	0.80

The shear fatigue resistance of an individual shear connector, Z_r , is determined as follows:

$$Z_r = a^{d^2} \geq \frac{5.5 \cdot d^2}{2}$$

Where:

a = shear fatigue factor

$$= 34.5 - 4.28 \log(N)$$

Pitch

Straight Girders

The maximum pitch of shear connectors along the longitudinal axis required to satisfy the fatigue limit state, P_{fat} , is determined as:

$$P_{fat} \leq \frac{nZ_r I}{V_f Q}$$

Where:

n = number of shear connectors per unit

I = moment of inertia of the short-term composite section

V_f = vertical shear force range due to fatigue loading

Q = first moment of the transformed area of the slab about the neutral axis of the short-term composite section

Maximum pitch = 24 in.

Preferred minimum pitch = 6 in.

Absolute minimum pitch = 6d

Use 1 in. increments for pitch.

~~Horizontally Curved Girders (see Analysis of Horizontally Curved Girders)~~

Horizontally curved girders will receive a lateral fatigue component introduced by the diaphragms. The above equation for pitch shall be modified as follows:

~~$$P_{fat} = \frac{nZr}{V_{sr}}$$~~

Where V_{sr} includes the longitudinal (V_{fat}) and radial (F_{fat}) fatigue shear ranges:

~~$$V_{sr} = \sqrt{(V_{fat})^2 + (F_{fat})^2}$$~~

~~$$V_{fat} = \frac{V_f Q}{I}$$~~

For bridge skews $\leq 20^\circ$

~~$$F_{fat} \approx 0$$~~

For bridge skews $> 20^\circ$

~~$$F_{fat} = \text{MAX} \left[\frac{A_{bot} \sigma_{flg} l}{wR}, \frac{F_{rc}}{w} \right]$$~~

Where:

~~A_{bot} = area of bottom flange~~

~~σ_{flg} = range of longitudinal fatigue stress range in bottom flange~~

~~l = distance between brace points~~

~~w = effective length of deck, 24" - end supports, 48" elsewhere~~

~~R = radius of girder; use minimum if radius transitions inside girder panel~~

~~F_{rc} = net range of diaphragm force at the top flange.~~

Note: For the purpose of the above calculations the bridge skew shall be taken as the maximum bent skew along the length of the bridge.

751.14.4.4 Strength Design

Resistance

The total nominal horizontal shear force between the point of maximum positive moment and each adjacent point of zero moment, V_h , is determined as:

~~$$P_p = \text{MIN} \left\{ 0.85 f'_c b t_s, F_{yw} D t_w + F_{yt} b_t t_t + F_{yc} b_f t_f \right\}$$~~

Where:

~~f'_c = 28 day compressive strength of deck concrete~~

~~b = effective flange width~~

~~t_s = effective slab thickness~~

~~F_{yw} = yield strength of web~~

~~D = web depth~~

~~t_w = web thickness~~

~~F_{yt} = yield strength of tension flange~~

~~b_t = width of tension flange~~

~~t_t = thickness of tension flange~~

~~F_{yc} = yield strength of compression flange~~

~~b_f = width of compression flange~~

~~t_f = thickness of compression flange~~

The nominal shear resistance of a single shear connector embedded in a concrete slab, Q_n , is calculated as follows:

~~$$Q_n = 0.5 A_{SC} \sqrt{f'_c E_c} \leq A_{sc} F_u$$~~

Where:

A_{se} = cross sectional area of a shear connector

E_c = modulus of elasticity of deck concrete

F_u = tensile strength of shear connector steel

The factored resistance of a single shear connector, Q_r , shall be determined as:

$$Q_r = \phi_{sc} Q_n$$

Where

ϕ_{sc} = resistance factor for shear connectors = 0.85

Calculated Resistance of Single Shear Connector

Q_r - Factored Resistance of One Shear Connector (deck/ $\phi 2$ 4ksi)	
Stud Diameter	Q_r (k)
3/4"	22.5
7/8"	30.7

The minimum number of shear connectors required for strength in each region, n_{str} , shall be computed:

$$n_{str} = \frac{P_p}{Q_r}$$

Regions are defined as the areas between points of zero moment and adjacent points of maximum positive moment. n_{str} is the minimum number of shear connectors that must be placed in each region to satisfy the strength limit state.

~~Additional Connectors~~

~~The number of additional connectors to be placed in the region of permanent load contraflexure, n_{AC} , shall be:~~

~~$$n_{AC} = \frac{A_r f_{sr}}{Z_r}$$~~

~~Where:~~

~~A_r = total area of deck reinforcement within the effective flange width~~

~~f_{sr} = stress range in the longitudinal reinforcement due to fatigue loading~~

~~Minimum number of additional connectors = 4 units at 6d.~~

~~The additional shear connectors shall be placed within a distance of 1/3 of the minimum effective flange width on each side of the point of permanent load contraflexure.~~

~~If it is not possible to place additional connectors within b/3 from the point of dead load contraflexure, the additional connectors shall be placed as close to the contraflexure point as practical, using the minimum pitch of 6d.~~

~~Horizontally Curved Girders~~

~~Girders that are analyzed for curvature effects require shear connectors to be placed throughout the bridge. The provision for additional shear connector placement does not apply. Horizontally curved girders require separate analyses for regions near the end of the bridge and regions near interior supports. They also require a lateral shear force to be considered as shown below.~~

~~End of Bridge Region:~~

~~$$P = \sqrt{P_p^2 + F_p^2}$$~~

~~Where:~~

~~$$F_p = P_p \frac{L_p}{R} = \text{lateral shear force}$$~~

L_p = arc length between end of girder and location of maximum positive moment

R = minimum radius over arc length, L_p .

Near interior support - Region

$$P = \sqrt{P_T^2 + F_T^2}$$

Where:

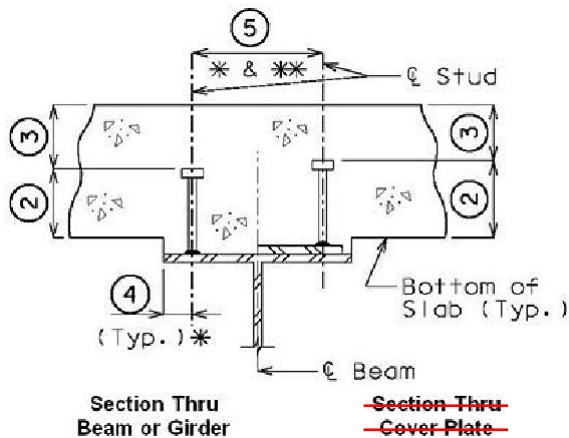
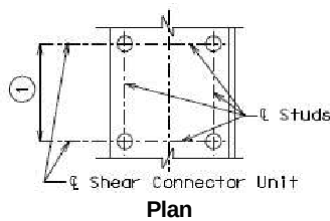
$$Pr = \min \left\{ 0.45f_c/bt, \frac{F_y w D t_w + F_y t_b t_t + F_y c b_1 t_1}{R} \right\} \quad \text{for section over pier}$$

$$Fr = \frac{Pr L_n}{R} = \text{lateral shear force near an interior support}$$

L_n = arc length between point of maximum positive moment and interior support

R = minimum girder radius over length, L_n .

751.14.4.5 Shear Connector Details

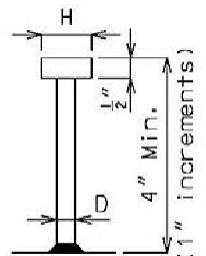


- ① 6" els. preferred min., 6d els. absolute min., 24" els. maximum. (1" increments min.)
- ② = 2" minimum for CIP slab;
- ② = a minimum height equal to the top of panel for P/S panel option.
- ③ 3" min. clear depth of concrete cover over shear connectors.
- ④ 1 1/2" (Min.) CIP slab
4 1/4" (Min.) panel option
- ⑤ 4 x (Stud diameter) preferred minimum, may be reduced if necessary for a more economical design; 2 1/4" absolute minimum.

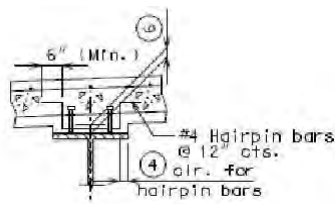
* Dimensions are not for detailing purposes.

** Requires the same dimension throughout the span.

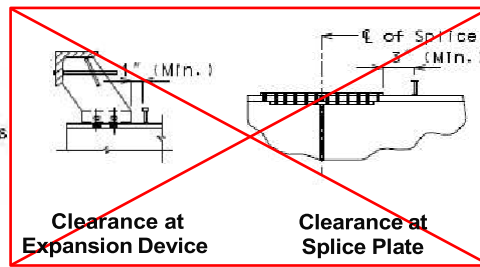
D	H
3/4"	1 1/4"
7/8"	1 3/8"



Minimum Flange Width For Given Rows of Studs (in.) (For decks utilizing prestressed panels)				Weight in Place Per 100 Studs (lbs)				
# of Studs per unit, n	Absolute Min.	Preferred Min.		Stud Dia.	4"	5"	6"	7"
		3/4" dia.	7/8" dia.					
2	10.75	11.50	12.00	3/4"	63	75	88	100
3	13.00	14.50	15.50	7/8"	81	98	115	132
4	15.25	17.50	19.00					

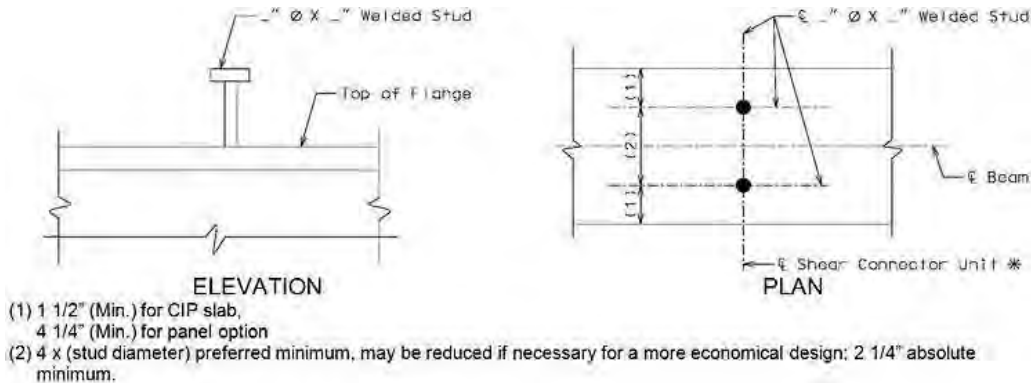


Section Thru Haunch

Clearance at
Expansion DeviceClearance at
Splice Plate

- ⑥ For redecks of existing composite bridges #4 hairpin bars required if depth exceeds 2" for existing 4" shear connectors and 3" for existing 5" shear connectors. For new bridges and redecks of non-composite bridges, studs shall provide a minimum 2" slab embedment avoiding the need of hairpin bars.

Shear Connector Unit



* Two studs per unit shown is for illustration purpose only.

P/C P/S Panel Forms on Steel Structures

Evaluate the viability of the use of P/S panels on tangent and curved structures on a case by case basis and use or include as an option to a CIP slab where deemed appropriate.

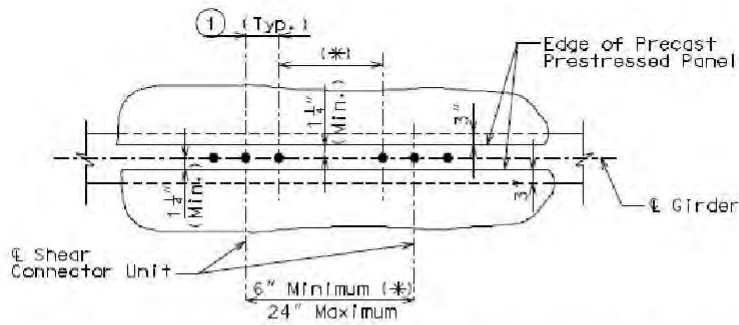
Whenever panels are used, the minimum top flange width shall be 12" for plate girders and 10" for wide flange beams.

Steel girders shall be cambered when using P/S Panels. Minimum joint filler thickness is 1", except over splice plates, in which case use 1/4" minimum. Maximum joint filler thickness is 2", except over splice plates, in which case use 1".

Shear connectors shall have a minimum height equal to the top of panel.

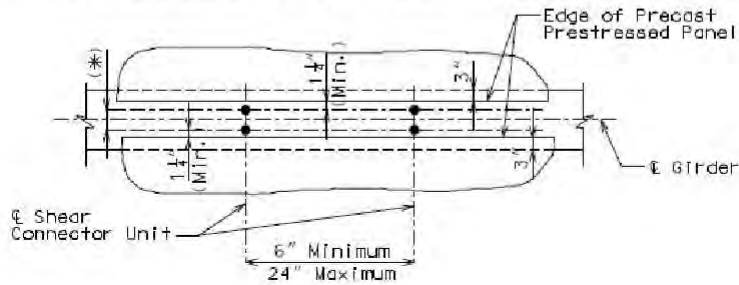
Shear connectors shall be spaced by units and shear connectors in each unit shall be placed along centerline (f) of girder. On wide flange widths, two lines of connectors may be used if spacings and clearances allow.

~~Additional shear connectors, Nae, at point of contraflexure may be placed in units normal to centerline of girder as space allows or in a single row along centerline of girder as shown below:~~



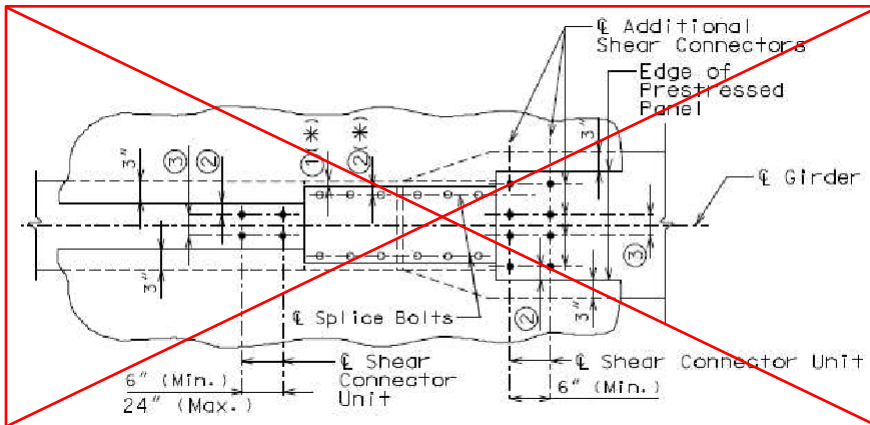
* Adjacent shear connectors shall not be closer than (1) center to center, either longitudinal or transversely.

- (1) 4 x (Stud diameter) preferred minimum, may be reduced if necessary for a more economical design; 2-1/4" absolute minimum.



P/S strands shall extend 3" minimum and 6" maximum past edge of precast prestressed panel and not closer than 1" to the adjacent panels.

Panel end at splices shall be notched to avoid bolt heads as shown below:



- (1) 3/4" min. wide bearing edge for panel at splice, typ.

- (2) 1-1/4" min. (Typ.)

- (3) 4 x (Stud diameter) preferred minimum, may be reduced if necessary for a more economical design; 2-1/4" absolute minimum.

~~* In order to meet (1) and (2) above, it is necessary to have an edge bolt distance of 2" or greater for splice plate.~~

~~Minimum joint filler thickness is 1" except over splice plates in which case use 1/4" minimum. Use 3/4" joint filler width for panels over splice plates. When joint filler is less than 1/2" thick over splice plate, the contractor is required to make the width of joint filler at splice the same width as panel on splice.~~

Maximum difference in top of flange thickness should be checked so that joint filler thickness does not exceed 2".

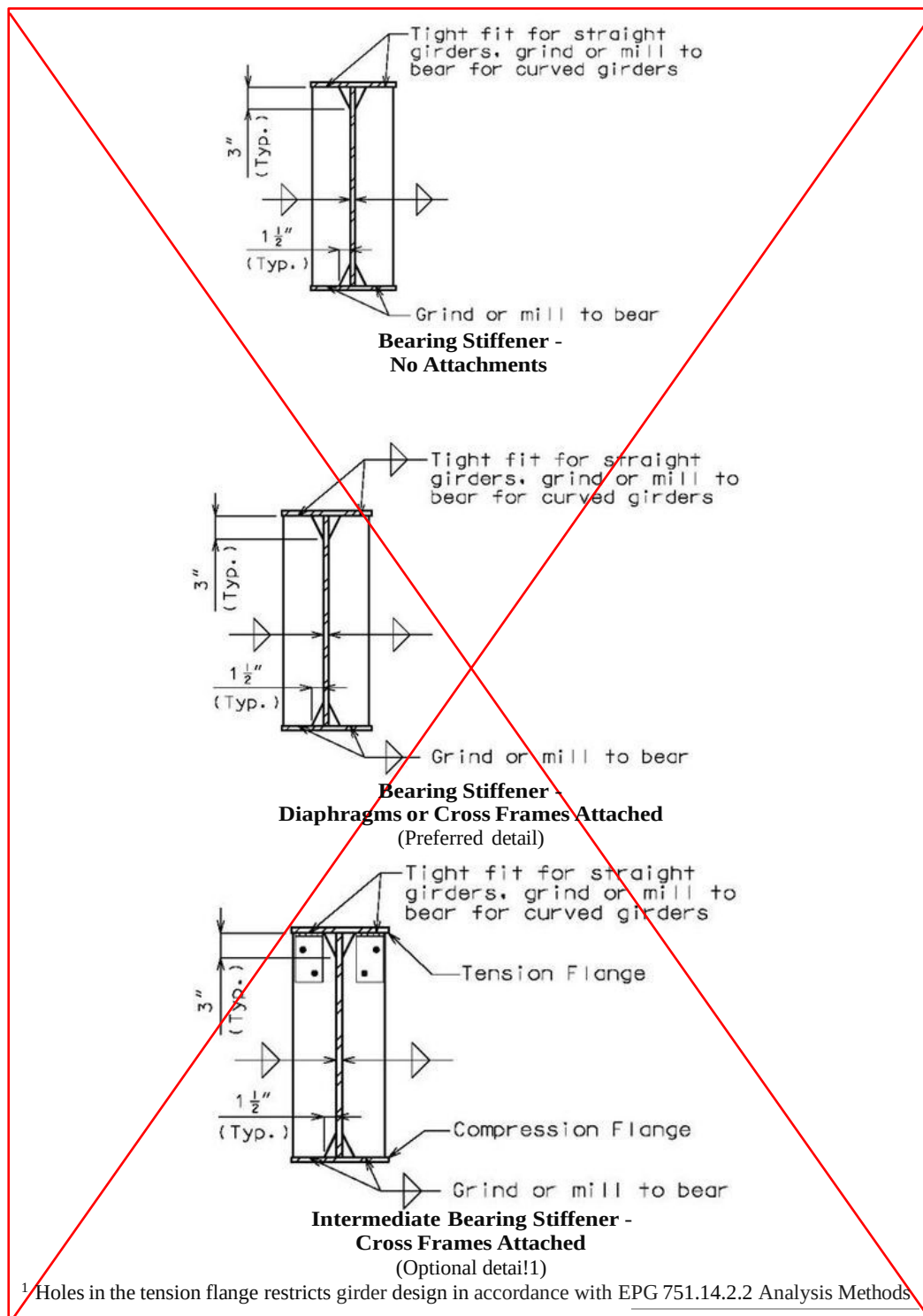
751.14.5 Details

~~751.14.5.1 Bearing Stiffeners~~

~~Refer to EPG 751.14.5.10 for weld design and detail guidance. Refer to EPG 751.14.5.5 Transverse Stiffeners for beveled plate guidance for unequal flange widths.~~

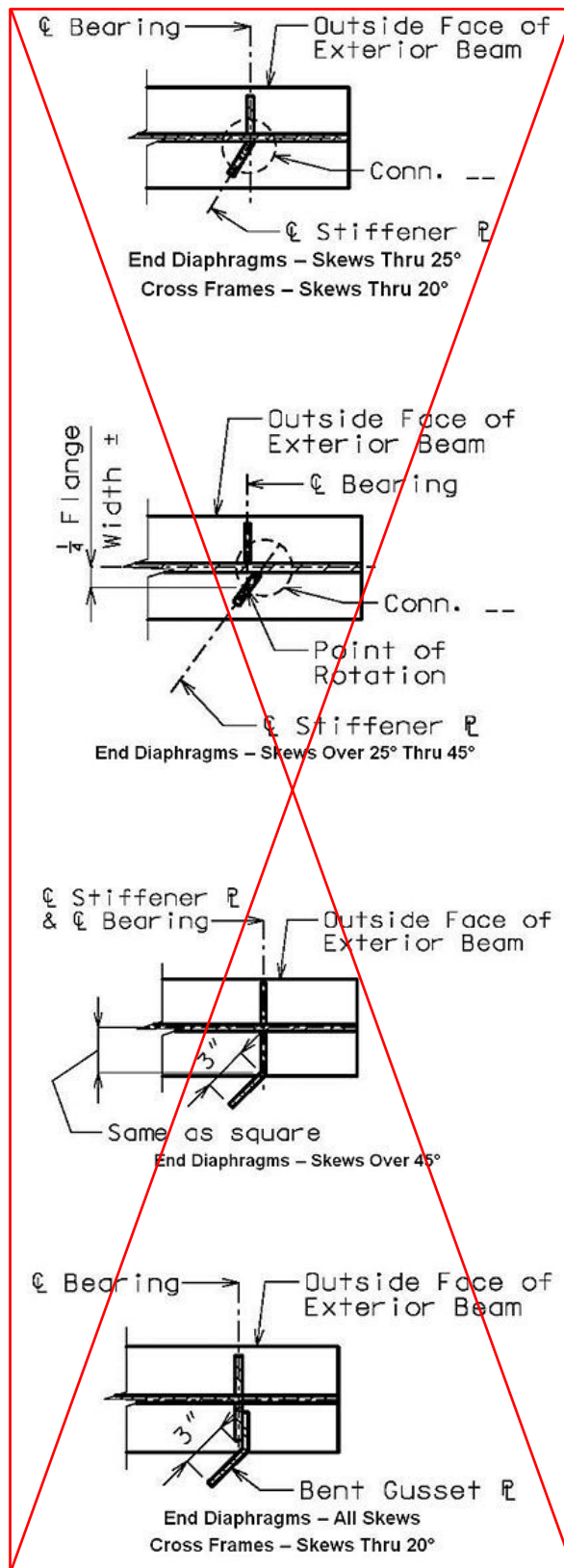
The preferred welded connection to flange is more prone to fatigue than the bolted connection. Girder shall be sized to reduce fatigue stresses to allow for preferred detail. If resizing is impractical the bolted connection shall be used.

Size bearing stiffener plate by design (1/2" thick minimum). See EPG 751.14.2.6 Other Requirements.



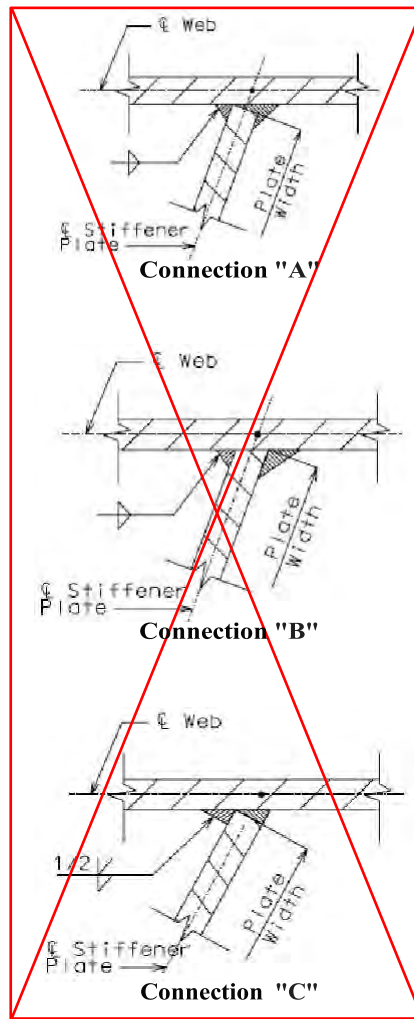
~~Locations~~

~~Refer to EPG 751.14.5.10 for weld design and detail guidance.~~

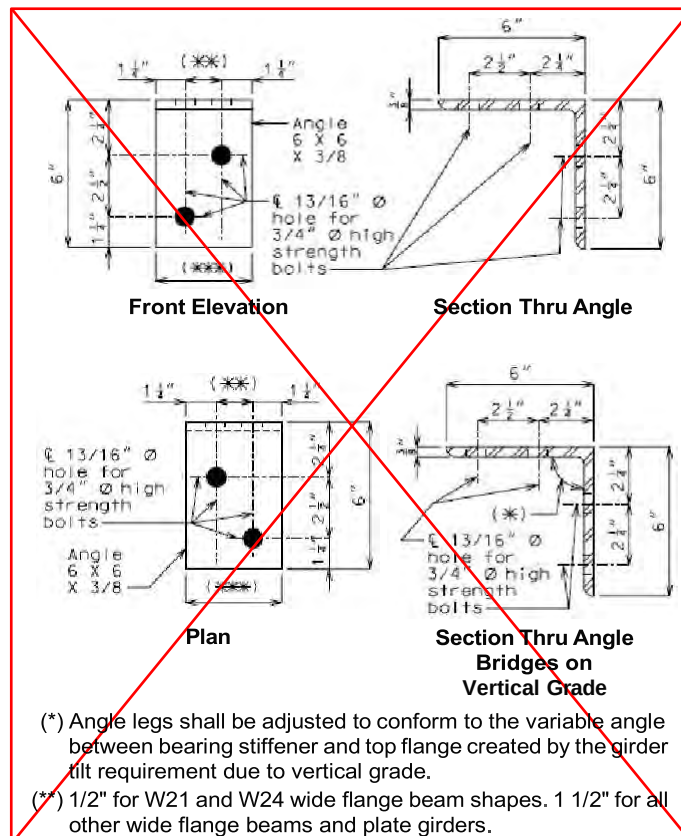


~~Welds for Optional Stiffener Locations~~

~~Refer to EPG 751.14.5.10 for weld design and detail guidance.~~



Connection Angle Details for Optional Stiffener Connection



(**) 3" for W21 and W24 wide flange beam shapes. 4" for all other wide flange beams and plate girders.

Bearing Stiffener Connections For Given Skew And Stiffener Size

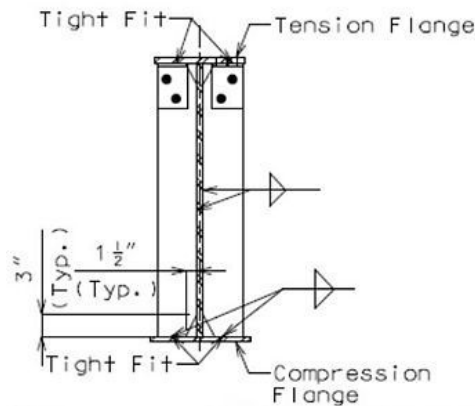
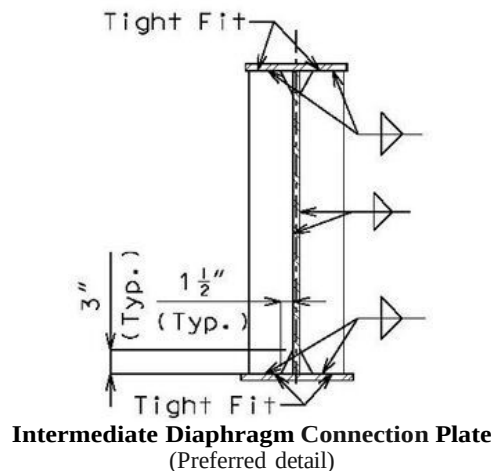
Skew	Stiffener Thickness (in)															
	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	2	2 1/8	2 1/4	2 3/8
0° THRU 5°	A	A	A	A	A	A	A	A	A	A	A	A	A	A	B	B
6° THRU 10°	A	A	A	A	A	B	B	B	B	B	B	B	B	B	B	B
11° THRU 15°	A	A	B	B	B	B	B	B	B	B	B	B	B	B	B	B
16° THRU 20°	A	B	B	B	B	B	B	B	B	B	B	B	B	B	B	B
21° THRU 25°	B	B	B	B	B	B	B	B	B	B	B	B	B	B	B	B
26° THRU 30°	B	B	B	B	B	B	B	B	B	B	B	B	B	B	B	B
31° THRU 35°	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C
36° THRU 40°	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C
41° THRU 45°	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C	C

751.14.5.2 Int. Diaphragms and Cross Frames

Refer to EPG 751.14.5.10 for weld design and detail guidance. Refer to EPG 751.14.5.5 Transverse Stiffeners for beveled plate guidance for unequal flange widths.

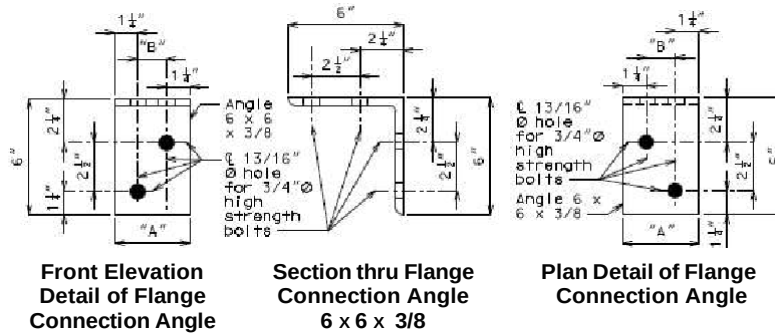
The preferred welded connection to flange is more prone to fatigue than the bolted connection. Girder shall be sized to reduce fatigue stresses to allow for preferred detail. If resizing is impractical, the bolted connection shall be used. When the bolted connection is used, the top and bottom tension or compression flanges shall be shown on the "Elevation of Girder" detail on plans.

Minimum connection plate size is 5 1/2" x 1/2". Width of plate shall be increased as required by 1/2" increments. Thickness of plate shall be increased as required by 1/8" increments.

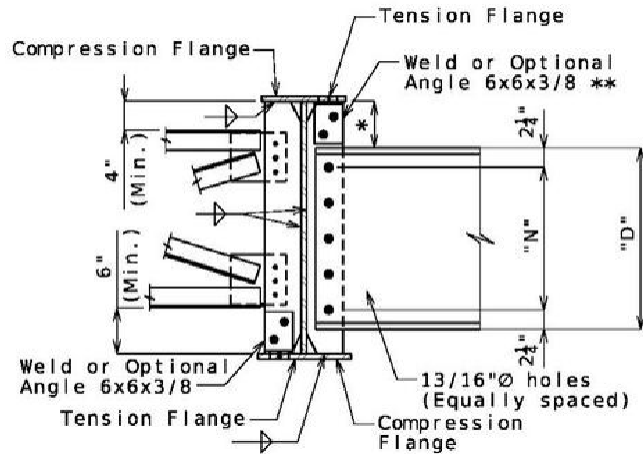


¹ Holes in the tension flange restricts girder design in accordance with EPG 751.14.2.2 Analysis Methods

Connection Angle Details (See table below for "A" and "B" values.)



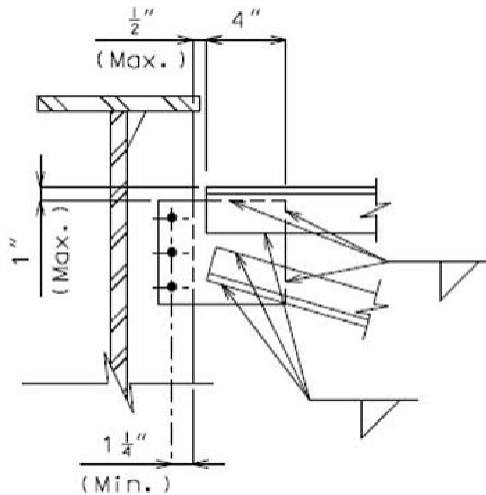
Diaphragm Details



- * When angles are required verify clearance for cross slope.
 6 1/4" (Min.) for wide flanges
 6 1/2" (Min.) for plate girders

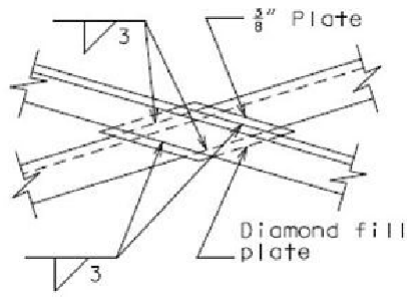
Holes in the tension flange restricts girder design in accordance with EPG 751.14.2.2 Analysis Methods

See table below for "D" and "N" values.

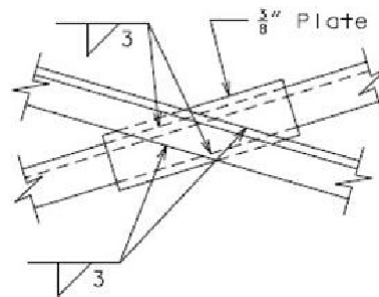


Diaphragm Connection at Stiffener Detail

Note: The above details are assumed adequate for straight girders. Horizontally curved girders require design for torsional effects (see Analysis of Horizontally Curved Girders)



Fill Plate Detail - Preferred



Optional Fill Plate Detail for Contractor

Refer to EPG 751.14.5.10 for weld design and detail guidance.

Intermediate Diaphragm and Cross Frame Details **

Shape	"D" (in)	"N"	Channel, Angle or Bent Plate	"A" (in)	"B" (in)
W21X48 THRU W21X93	12	4	C12 x 20.7	3	1/2
W24X68 THRU W24X103	12	4	C12 x 20.7	3	1/2
W27X84 THRU W27X129	15	4	5/16 x 23	4	1 1/2
W30X90 THRU W30X148	18	5	5/16 x 26	4	1 1/2
W33X118 THRU W33X221	21	5	5/16 x 29	4	1 1/2
W36X135 THRU W36X194	21	5	5/16 x 29	4	1 1/2
W40X149 THRU W40X215	21	5	5/16 x 29	4	1 1/2
36" thru 40" Plate Girders	21	5	5/16 x 29	4	1 1/2
42" Plate Girders	27	5	5/16 x 35	4	1 1/2
> 42" Plate Girders	Diagonal Member		L3x3x5/16	4	1 1/2
	Horizontal Member	*** S < 10'-2"	L3.5x3.5x5/16		
		other	L4x4x5/16		

Table is applicable for diaphragms and cross frames used as bracing members. For criterion used in determining members sizes see the footnotes for the "Bottom Horizontal Members for Web Depths" and "Diagonal Members for Web Depths > 48" tables for End Diaphragms.

S = girder spacing along diaphragm skew

Diaphragms and cross frames used as primary members must be checked for torsion effects. (See Analysis of Horizontally Curved Girders)

ATTENTION DETAILER:

Intermediate diaphragm connection plates shall be detailed for Connection "A" for skews thru 20°. Details of Connection "A" are shown in EPG 751.14.5.1 Bearing Stiffeners.

~~751.14.5.3 End Diaphragms for Wide Flange Beams~~

W21 thru W30	Beam Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0	C15x33.9	C15x33.9	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40
10	C15x33.9	C15x40	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40
20	C15x33.9	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40	W16x45
30	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40	W16x45	W16x45
40	C15x50	W16x36	W16x40	W16x45	W16x45	W16x50	W16x50	W16x57
50	W16x40	W16x45	W16x50	W16x50	W16x57	W16x57	W16x67	W16x67
60	W16x57	W16x67	W16x67	W16x67	W16x67	W16x77	W16x77	W16x89

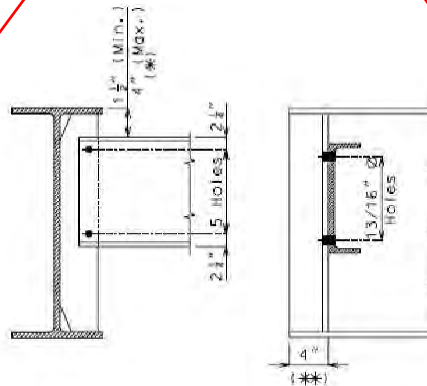
W33	Beam Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0	W16x31	W16x31	W16x31	W16x36	W16x36	W16x36	W16x40	W16x40
10	W16x31	W16x31	W16x31	W16x36	W16x36	W16x36	W16x40	W16x40

20	W16x31	W16x31	W16x36	W16x36	W16x36	W16x40	W16x40	W16x45
30	W16x31	W16x36	W16x36	W16x36	W16x40	W16x40	W16x45	W16x45
40	W16x36	W16x36	W16x40	W16x45	W16x45	W16x50	W16x50	W16x57
50	W16x40	W16x45	W16x50	W16x50	W16x57	W16x57	W16x67	W16x67
60	W16x57	W16x67	W16x67	W16x67	W16x67	W16x77	W16x77	W16x89

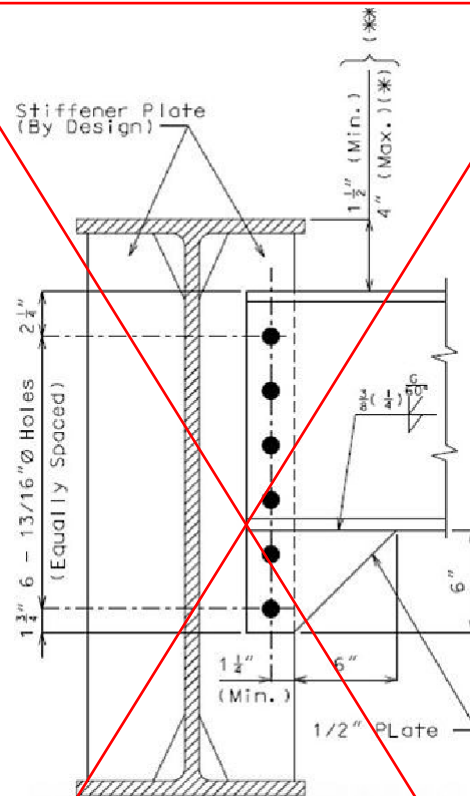
W36	Beam Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0	W18x35	W18x35	W18x35	W18x35	W18x35	W18x35	W18x40	W18x40
10	W18x35	W18x35	W18x35	W18x35	W18x35	W18x35	W18x40	W18x40
20	W18x35	W18x35	W18x35	W18x35	W18x35	W18x40	W18x40	W18x40
30	W18x35	W18x35	W18x35	W18x35	W18x40	W18x40	W18x46	W18x46
40	W18x35	W18x35	W18x40	W18x40	W18x46	W18x46	W18x50	W18x50
50	W18x40	W18x46	W18x46	W18x50	W18x50	W18x55	W18x55	W18x60
60	W18x50	W18x55	W18x60	W18x65	W18x65	W18x71	W18x76	W18x76

W40 thru W44	Beam Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
10	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
20	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
30	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
40	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x48
50	W21x44	W21x44	W21x44	W21x44	W21x48	W21x48	W21x55	W21x55
60	W21x48	W21x50	W21x55	W21x55	W21x62	W21x62	W21x68	W21x73

Note: The values in the table above may be used for both straight and horizontally curved beams.



End Diaphragm Connections for W21 thru W24 Beams



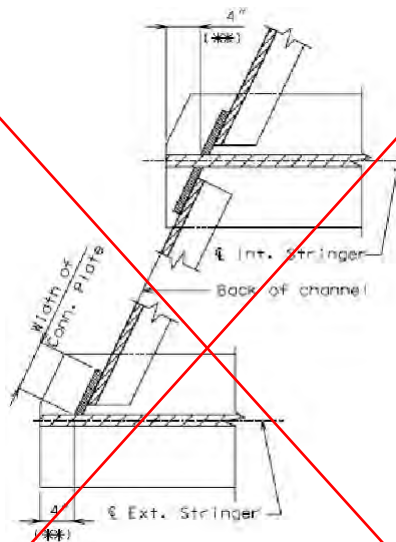
End Diaphragm Connections for W27 thru W40 Beams

Refer to EPG 751.14.5.10 for weld design and detail guidance.

(*) Slope diaphragms when structure is superelevated or when 4" maximum depth is exceeded. See EPG 751.13 Expansion Devices for the details of end diaphragms on a structure.

(**) Modify if necessary to clear the anchor bolts of the flat plate bearing or connection bolts of the expansion device.

~~All Sizes~~



ATTENTION DETAILER:

End diaphragms shall be detailed horizontally, stepped, or sloped as required.

~~751.14.5.4 End Diaphragms for Plate Girders~~

~~Top Horizontal Members for Given Web Depths~~

Webs Thru 40"	Girder Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
10	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
20	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
30	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44
40	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x44	W21x48
50	W21x44	W21x44	W21x44	W21x44	W21x48	W21x48	W21x55	W21x55
60	W21x48	W21x50	W21x55	W21x55	W21x62	W21x62	W21x68	W21x73

Webs 42"	Girder Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0	C15x33.9	C15x33.9	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40
10	C15x33.9	C15x40	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40
20	C15x33.9	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40	W16x45
30	C15x40	C15x50	C15x50	W16x36	W16x40	W16x40	W16x45	W16x45
40	C15x50	W16x36	W16x40	W16x45	W16x45	W16x50	W16x50	W16x57
50	W16x40	W16x45	W16x50	W16x50	W16x57	W16x57	W16x67	W16x67
60	W16x57	W16x67	W16x67	W16x67	W16x67	W16x77	W16x77	W16x89

Bottom Horizontal Members for Web Depths 42"

Webs 42"	Girder Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0 thru 10	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16
20	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16
30	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16	L5x5x5/16
40	L3 1/2x3 1/2x5/16	L4x4x5/16	L4x4x5/16	L4x4x5/16	L5x5x5/16	L5x5x5/16	L5x5x5/16	L5x5x5/16
50	L4x4x5/16	L4x4x5/16	L5x5x5/16	L5x5x5/16	L5x5x5/16	L5x5x5/16	L6x6x3/8	L6x6x3/8
60	L5x5x5/16	L5x5x5/16	L6x6x3/8	L6x6x3/8	L6x6x3/8	L6x6x3/8	WT6x17.5	WT6x17.5

Diagonal Members for Web Depths > 48"

Webs :5 60"	Girder Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
0 thru 50	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16
60	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16

Webs < 112"	Girder Spacing, ft.							
Skew, deg.	7	7.5	8	8.5	9	9.5	10	10.5
40	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16	L3x3x5/16
60	L3x3x5/16	L3x3x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16	L3 1/2x3 1/2x5/16

For girder spacing or skews not listed in table, use nearest tabulated angle size.

Enter the tables with the skew of the diaphragm, not the bridge skew.

The members in Tables 2 and 3 were selected to meet only the slenderness requirement $\frac{K}{T_z} \leq 140$. For diagonal bracing members the following K values were used for equal leg single angle members:

L3x3x5/16	$K = 0.85$
L3 1/2x3 1/2x5/16 and longer leg lengths	$K = 0.75$

For horizontal bracing members the following K values were used for equal leg single angle members:

L3x3x5/16	$K = 1.1$
L3 1/2x3 1/2x5/16	$K = 0.95$
L4x4x5/16	$K = 0.85$

~~L5x5x5/16~~	~~K = 0.80~~
~~L6x6x3/8~~	~~K = 0.76~~

The above **K** values were determined using the following reference article: *"Critical Slenderness of Compression Members with Effective Lengths about Nonprincipal Axes"*, L.A. Lutz (1992)

For WT shapes: **K** = 0.75 (in-plane), 1.0 (out-of-plane)

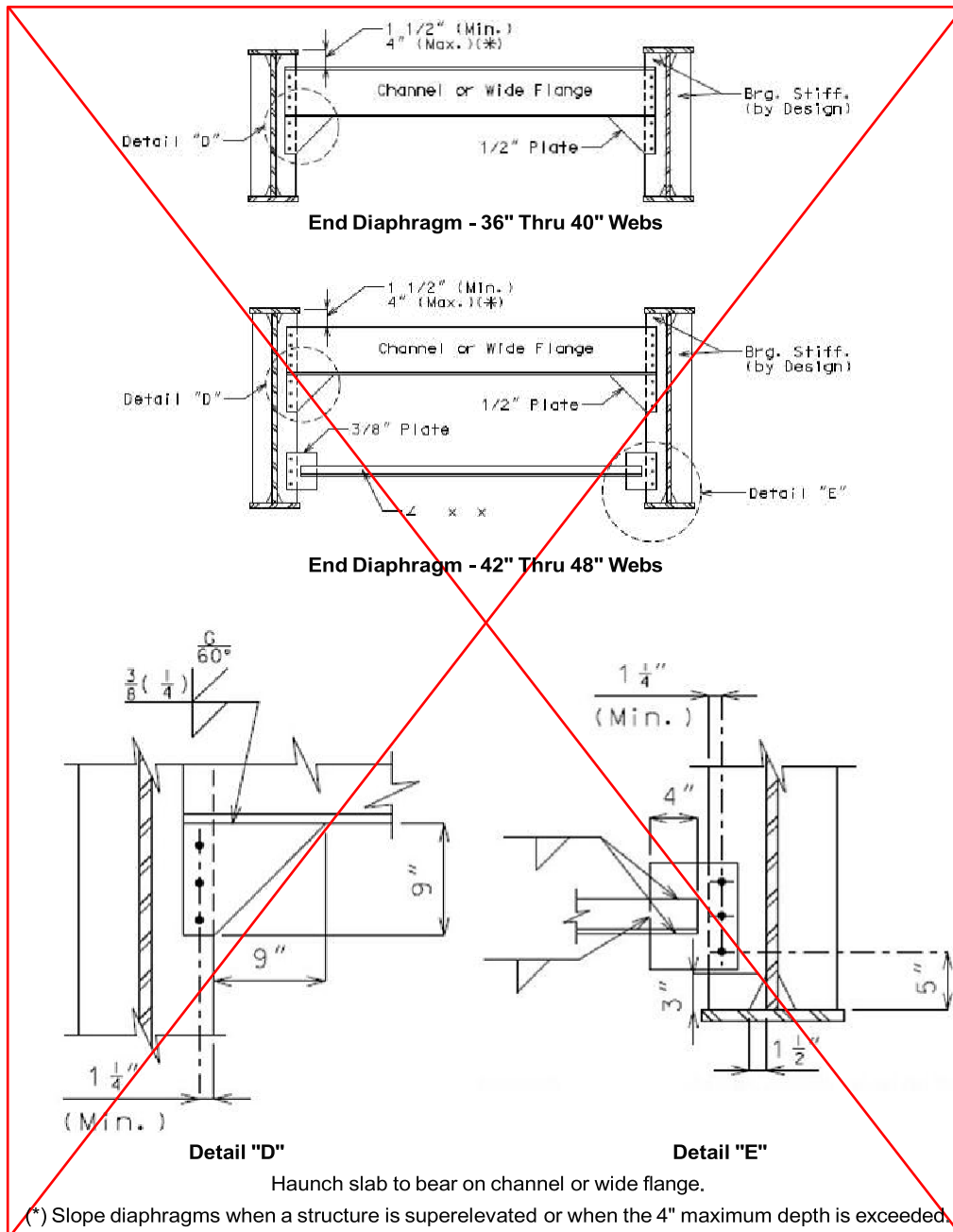
Member lengths, **L**, were measured from the center of bolted connection. The center of bolted connection is assumed to be located 3.5" perpendicular to CL web. Vertically, diagonal members are assumed to extend to 2 3/8" from outer gusset plate edge. In-plane bending was assumed to be fixed-fixed while out-of-plane bending is assumed to be pinned at the bolted connection and fixed at the diagonal member crossing. Diagonal members are assumed to be braced in the middle.

For diaphragms and cross-frames designed as primary members in horizontally curved girders:

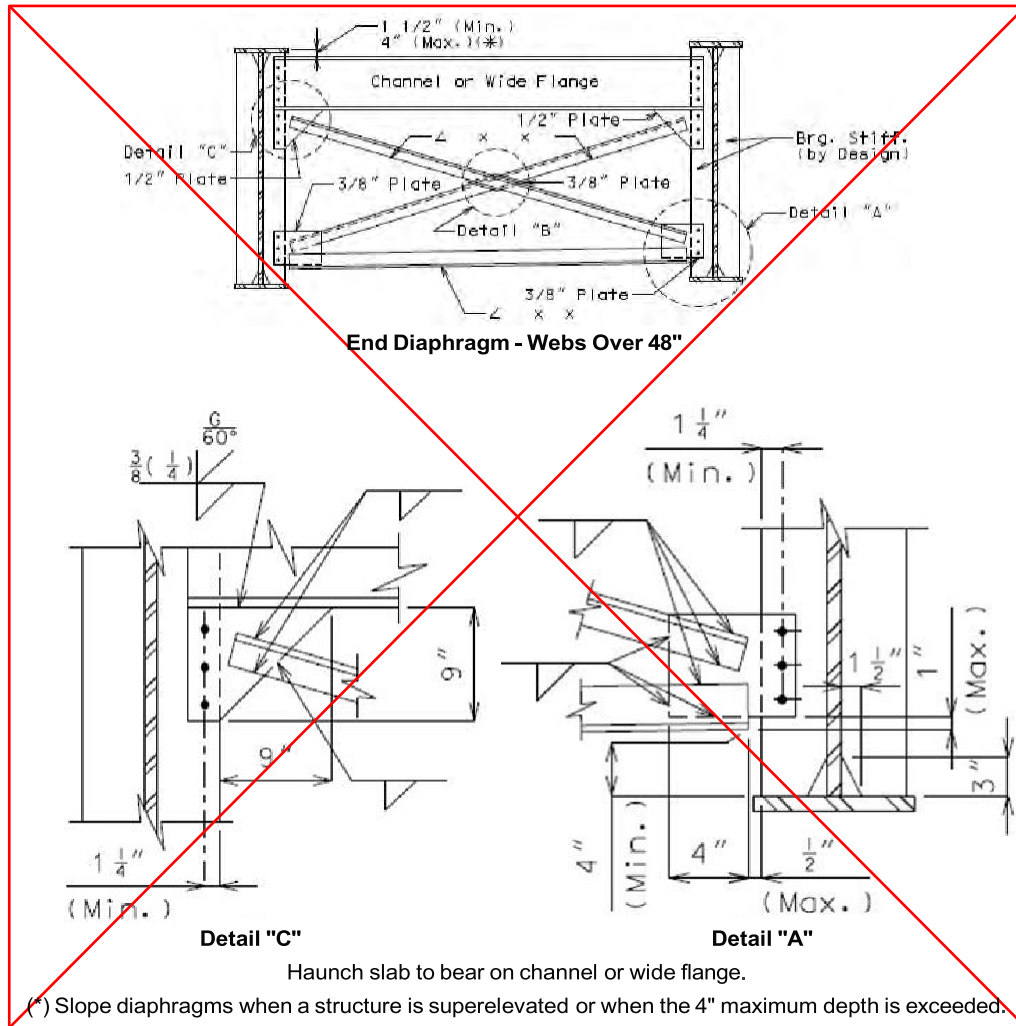
$$\frac{KL}{r} < 120$$

Primary members shall also be designed for torsional effects. (See Analysis of Horizontally Curved Girders)

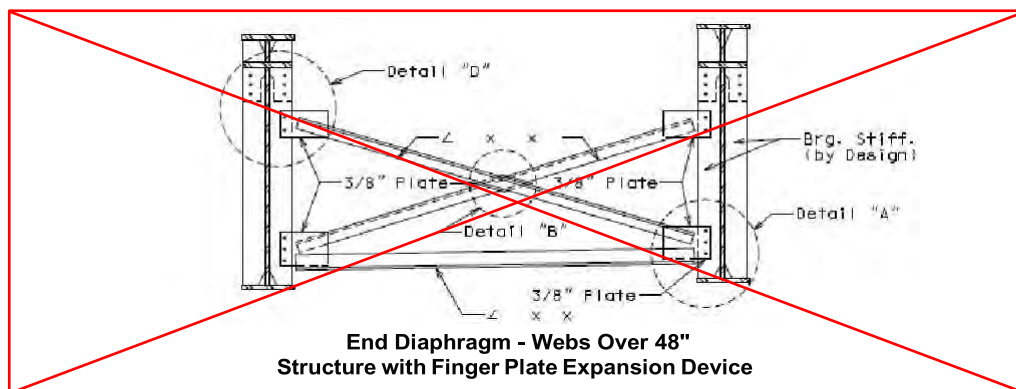
End Diaphragms

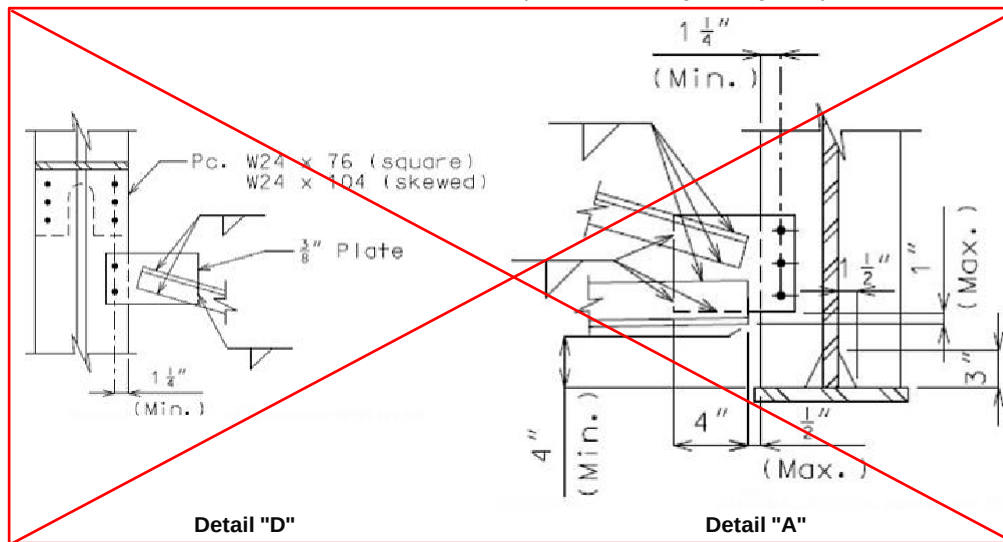


Refer to EPG 751.14.5.10 for weld design and detail guidance.



Refer to EPG 751.14.5.10 for weld design and detail guidance.





Refer to EPG 751.14.5.10 for weld design and detail guidance.

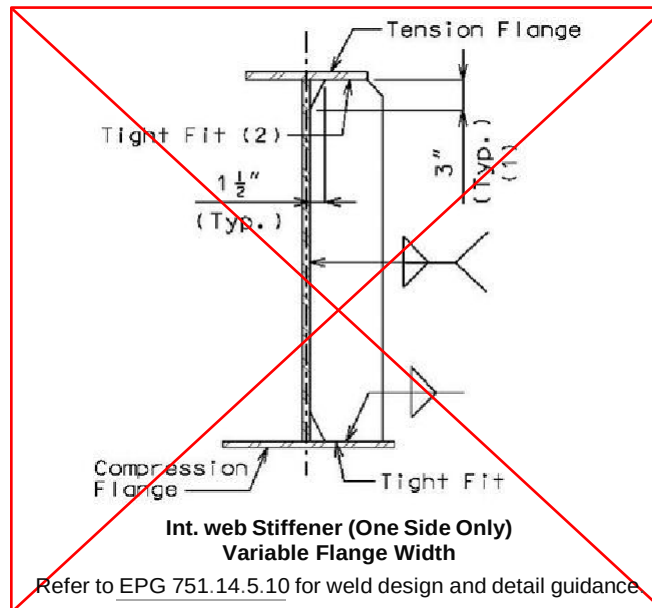
~~751.14.5.5 Transverse Stiffeners~~

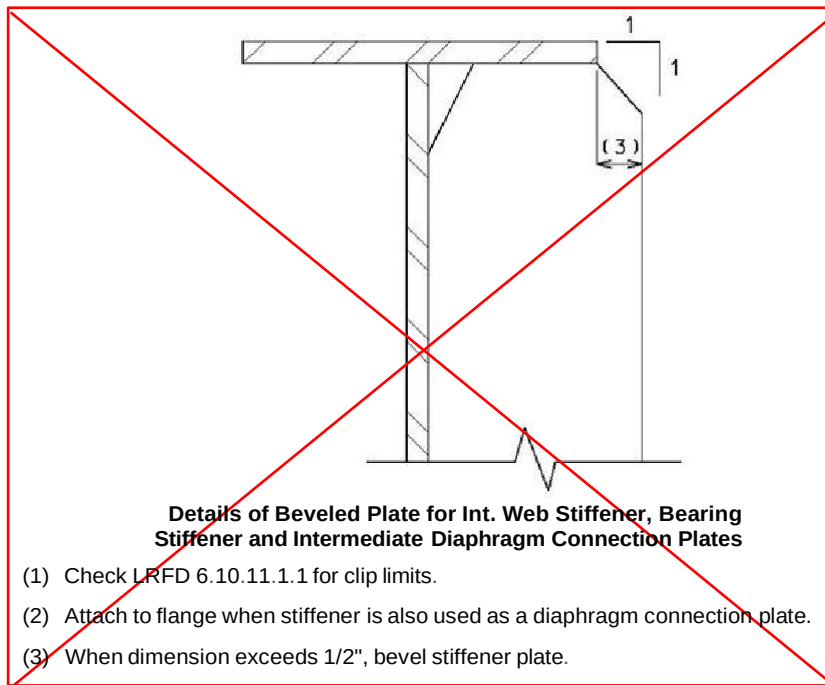
Transverse stiffeners are also referred to as intermediate web stiffeners.

Minimum stiffener size is 5 1/2" x 1/2". Width of plate shall be increased as required by 1/2" increments. Thickness of plate shall be increased as required by 1/8" increments.

Transverse stiffeners shall be used on inside face of exterior girders and in successive alignment along either side of interior girders.

When transverse stiffeners are used, top and bottom tension or compression flanges shall be shown on the "Elevation of Girder" detail on plans.





751.14.5.6 Longitudinal Stiffeners

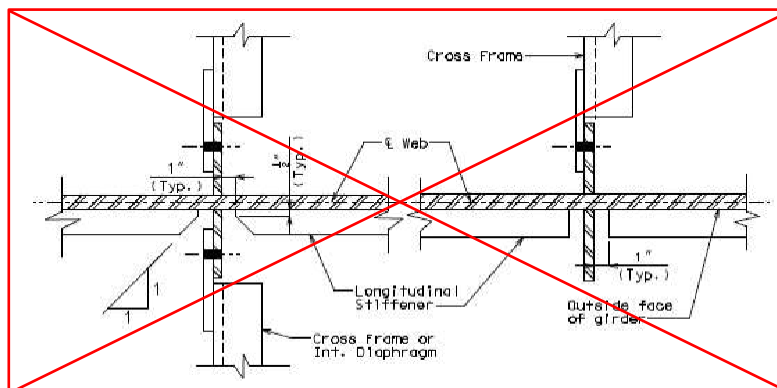
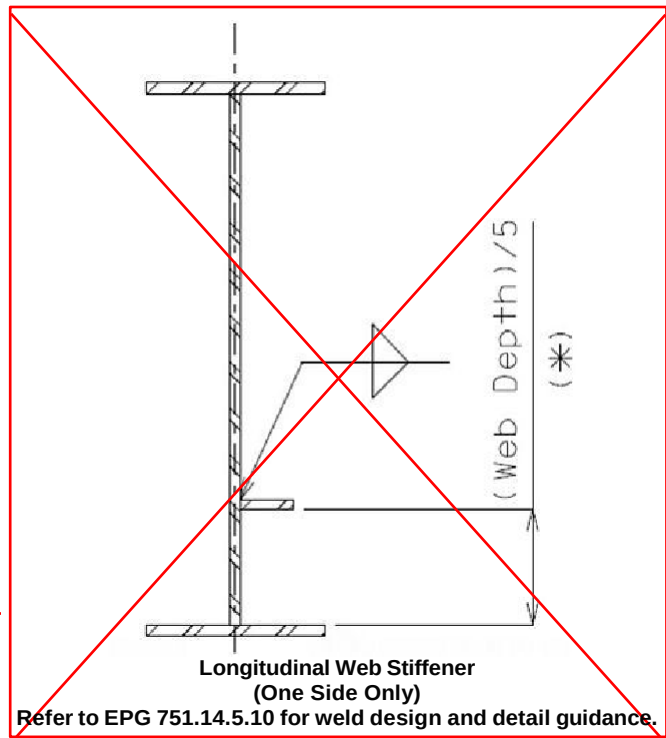
When longitudinal stiffeners are required they shall be used on the outside of exterior girders and in successive alignment along either side of interior girders.

The placement of longitudinal stiffeners may interfere with bolting the diaphragm or cross frame in place. Should this occur, it will be necessary to clip the longitudinal stiffeners as shown below.

Also, when lateral bracing is required, place the longitudinal stiffener on the opposite side of girder.

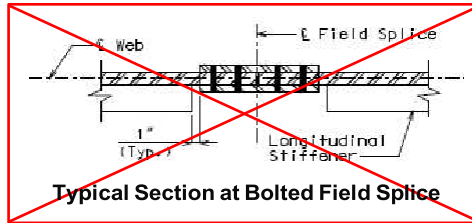
The appropriate details shown below shall be modified as needed and placed on the design plans.

(*) Location shall satisfy requirements of LRFD 6.10.8.3.



Interior Girders
Square or Skewed

Exterior Girders
at Intermediate
Bearing Stiffeners



751.14.5.7 Lateral Bracing

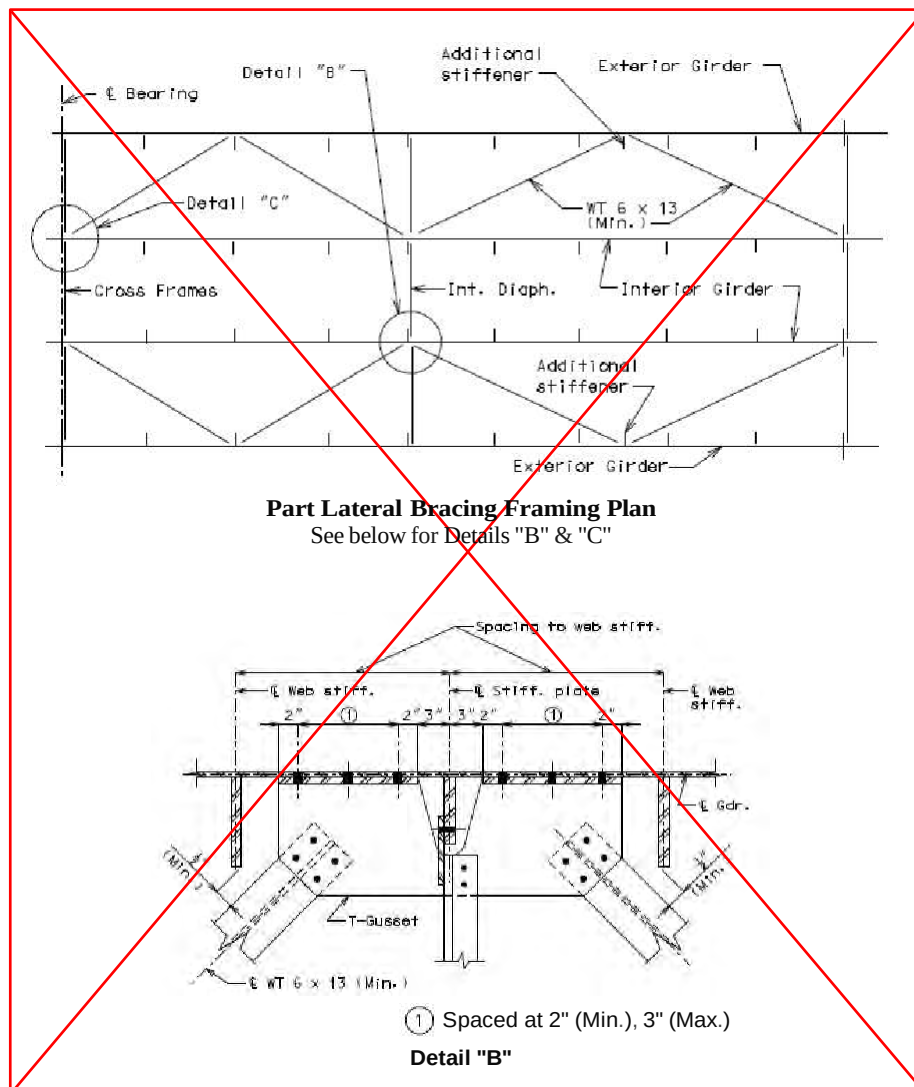
Lateral bracing shall be omitted on all spans < 140 ft. Lateral bracing may be omitted on spans ≥ 140 ft provided the girder meets all applicable Strength limit states.

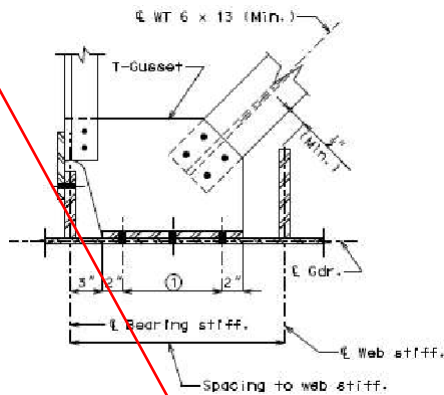
Lateral bracing shall be placed in outer bays.

Members selected shall meet requirements of bracing members with $KL/r : S : 140$.

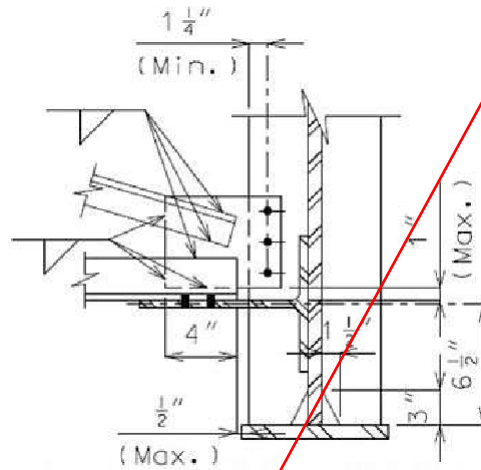
Under no circumstance shall lateral bracing be detailed to go through diaphragms. In order to facilitate erection, structural tees for lateral bracing may be inverted to connect to the top gusset plates.

Whenever practicable, diaphragms and lateral bracing shall connect to bearing stiffeners in bridges having large skews, diaphragms and lateral bracing need not connect to end bearing stiffeners for such structures but should connect to a girder midway between the bearing stiffener and the first transverse web stiffener.





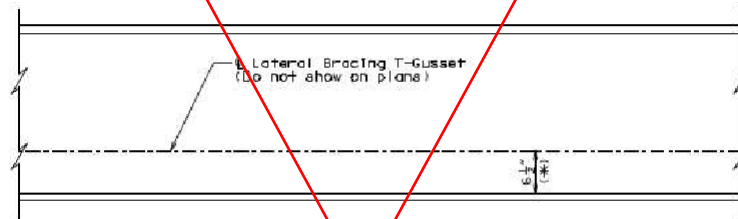
Detail "C"



Detail "A"

Note: Lateral Bracing T-Gusset shall be cut to required depth from W27 x 94, or T-Gusset can be built from plates in shop.

Refer to EPG 751.14.5.10 for weld design and detail guidance.

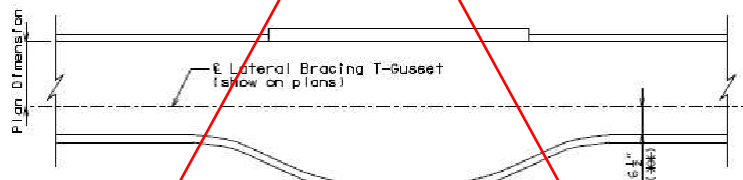


(*) Locate T-Gusset as shown above, dimension at all connections should be constant from top of bottom flange. Note: T-Gusset shall be placed (*) inches

Pl f . t

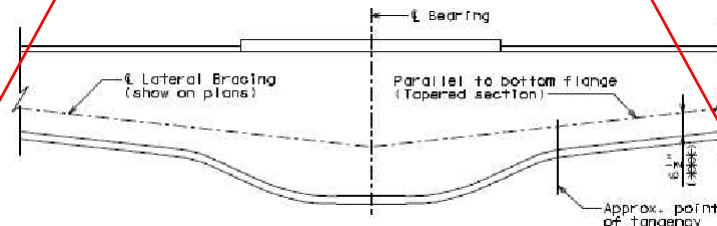
ace o11owing no on Plans: above top of bottom flange.

Constant Depth Girders



(**) Locate T-Gusset as shown above, dimension at all connections should be constant from top of bottom flange in constant web depth portions.

Constant - Variable Depth Girders



(***) Locate T-Gusset as shown above, dimension at all connections thru tapered section should be constant from top of bottom flange (Slope T-Gusset same as bottom flange). Continue same slope through variable depth section of girders.

Tapered - Variable Depth Girders**751.14.5.8 Protective Coating Requirements**

Coating requirements for new steel girder bridge shall be in accordance with Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14) and Sec 1081 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14). See EPG 751.1.2.9.2, EPG 751.6.2.11 and EPG 751.6.2.12 for additional guidance.

System G (three-coat system) may be used for non-weathering steel and weathering steel structures (Sec 1081 (<https://www.modot.org/missouri-standard-specifications-highway-construction>)). System G typically is not preferred when overlapping an existing vinyl coating but may be allowed if the existing coating is determined to be in good condition. System G uses a solvent based finish coat which may cause issues when overlapping an existing solvent-based vinyl coating system (System C) because it may re-wet the existing coating and cause delamination of the base coat. If the existing coating is in good condition as determined by paint pull-off tests the intermediate epoxy coating will provide a reliable barrier between the solvent-based coatings. Consult the structural project manager or structural liaison engineer before using System G near existing vinyl coatings.

System G has replaced calcium sulfonate as the preferred overcoating system. To ensure sufficient bond of the existing coating, adhesion pull-off tests shall be performed in accordance with ASTM D4541. If the adhesion test fails, as determined by the engineer of record, then overcoating shall not be allowed and recoating should be considered.

System H (three-coat system) is typically used when the bond for System G is considered questionable where recoating operations will take place near an existing vinyl coating system (System C). System H uses a waterborne acrylic for the intermediate and finish field coats that does not tend to interfere with the solvent-based vinyl coating.

System I (two-coat system) may be used for non-weathering and weathering steel and should be based on the following guidance:

(a) System I should be considered in areas where the aesthetics of a coating system over the long term is critical. While System G, L and I provide long term protection, System I has excellent gloss retention and UV resistance. System I is a context sensitive design (CSD) solution. CSD follows from project scoping and is subject to the project core team protocols.

(1) Consider for locations where the structure is more visible or the public has leisurely time for more than just a casual glance, for example structures near a ballpark or a pedestrian bridge. Using same rationale, bridges that are tall or have wide girder spacing or a low number of girders where more of the superstructure is visible could also be candidates.

(2) Consider the image consciousness of the surroundings in conjunction with rather than solely the protection of the structure which is equally provided by systems G and I. Maintenance of either System G or I should be considered the same. Reduced maintenance is an expectation for System L.

(b) System I is a polysiloxane finish coat that is normally applied directly over an inorganic or organic zinc primer with no intermediate coating. Since the system is a two-coat system, it may be applied in less time which can influence critical path scheduling and impacts to the driving public. For example, it may be possible for a contractor to get in and out quicker than if they were to use a three-coat system. MoDOT coating policy as described in Standard Specification Section 1081 requires different field coating requirements based on the type of bridge crossing. For roadway grade separations, it is required that interior girders have only a single field coat in order to satisfy that all girders on a roadway grade separation bridge have at least two coatings for protection. In the case of System I, the Standard Specifications require that a System G epoxy intermediate field coat be applied to all interior girders and the interior of fascia girders and that the System I polysiloxane finish coating be applied to the exterior of the fascia girders only. This is based on a system I polysiloxane coating cost being greater than a system G epoxy coating on a per-gallon cost basis. It also requires that the contractor be given the option to substitute the System I finish coat in place of a System G intermediate coat. If CSD determines that the polysiloxane should be applied to all girders, then the general notes for coatings and the quantities on the contract plans will need to reflect the revised coating requirements.

(c) System I is approved for use on state highway projects beginning February 2011. Alternate bidding is encouraged if guideline (a) is not required to be met and with approval of the Structural Project Manager or Structural Liaison Engineer and the project core team.

System L (two-coat system) may be used for non-weathering steel and weathering steel structures. System L requires an inorganic zinc primer so organic zinc shall not be substituted. Testing indicates that System L is expected to outlast System G, H or I coatings when properly applied. Since the system is a two-coat system, it may be applied in less time which can influence critical path scheduling and impacts to the driving public. Similarly, the expectation for reduced maintenance should be considered for areas where access is limited or impactful (e.g., over railroads and interstates).

Inorganic zinc primer shall be used for new steel fabrication with System G, I or L coatings. For recoating operations with System G, H or I, where closure time has severe impacts on cost or safety, organic zinc primer should be considered as a direct replacement for the inorganic zinc required in the specifications. Organic zinc primers require a lower level of surface preparation (SSPC-SP6: commercial blast cleaning vs SSPC-SP10: near white blast cleaning) and are generally easier to apply in the field than inorganic zinc primers. Only organic zinc primers that can provide a Class B slip coefficient are allowed for use in recoating operations.

See EPG 751.50 A4. Protective Coatings (https://epg.modot.org/index.php?title=751.14_Steel_Superstructure) for standard detailing notes and guidance on how they are used.

Epoxy-mastic primers may be used for overcoating lead-based coatings if the existing coating is determined to be in good condition, but this is considered a short-term solution in comparison to System G overcoating. Consult the structural project manager or structural liaison engineer before using epoxy-mastics near existing lead-based coatings.

Galvanized non-weathering structural steel beams, girders, bracing and diaphragms may be used as required or allowed by alternate, on a case-by-case basis, with approval of the Structural Project Manager or Structural Liaison Engineer and the project core team.

When galvanized structural steel is required, place note EPG 751.50 (A4a1.8.2a) on the plans. Do not use notes EPG 751.50 (A4a1.1 -A4a1.7). When galvanized structural steel is bid as an alternate, place notes EPG 751.50 (A4a1.8.1a, A4a1.8.1b, and A4a1.8.1c) on the plans under the applicable coating new steel notes EPG 751.50 (A4a1.1-A4a1.7).

751.14.5.9 Weathering Steel

Use of weathering steel may require special detailing as specified in the referenced documents in EPG 751.1.2.9 Girder Type Selection (https://epg.mdot.org/index.php?title=751.1_Preliminary_Design#751.1.2.9_Girder_Type_Selection).

751.14.5.9.1 Drip Bar Guidance

General

The purpose of a drip bar placed on the bottom flange is to intercept and deflect water and debris. Drip bars should be installed on new, or retrofitting existing weathering steel where a need is discovered.

Drip bars should be utilized for three reasons:

1. To protect girders and beams from sustained and accumulated (heavy) water flow along the web-flange crease that can promote deterioration by wearing action (due to particulate matter and debris) and long term moisture retention along the crease and along the lower web whereby water is drawn up the sides of the lower web by capillary action which is observable in the field as constantly "wetted" sides or permanent discoloration. For a new girder or beam, this can retard the development of or reduce the efficacy of the protective oxide coating.
2. To protect bearing devices from sustained moisture due to accumulating water and debris.
3. To protect substructure from long term staining. (For the short term, temporary protective coating is utilized during weathering steel construction; see EPG 711 Protective Coatings for Exposed Concrete Surfaces.)

Guidance

Drip bars shall not be provided on flat structures (0.0 % grade) unless in the case of long spans where permanently cambered girders or beams can produce a water flow.

Drip bars shall be placed skewed on top of bottom flange and preferably placed squared on bottom of bottom flange.

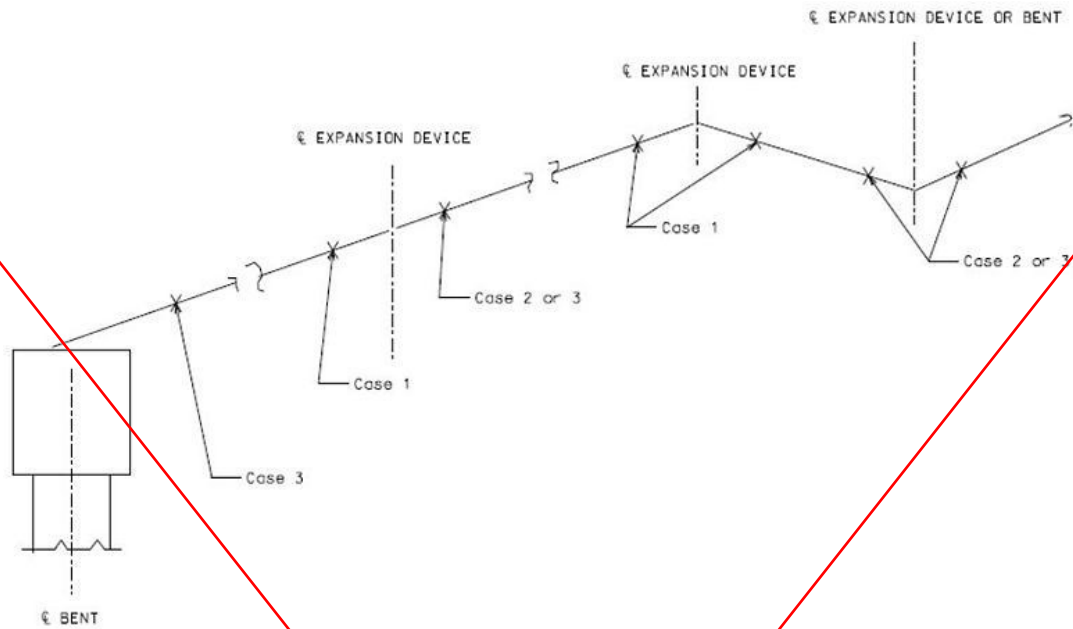
Drip bars shall not be provided directly over railroads, roadways and walkways.

(Case 1) To protect girders and beams: Drip bars should be located at the downgrade side of open or closed expansion devices between 5 ft. from the center of the expansion device and to within 1 ft. from the end of the partial coating limit (to ensure the drip bars are coated) for all girders or beams within a span.

(Case 2) To protect girder and beam ends, bearings and bearing stiffeners: Drip bars should be located at the upgrade side of bearings at the exterior faces of exterior girders or beams at about 4 ft. away from the face of the bent for downhill runs of 100 ft. or longer span girders or beams.

(Case 3) To protect substructure: Drip bars should be located adjacent to the upgrade side of the bent at the exterior faces of exterior girders or beams at about 4 ft. away from the face of the bent for downhill runs of 100 ft. or longer span girders or beams to prevent stain marks on the substructure in either urban or context sensitive designs, for example where the aesthetics of the structure is considered important. For aesthetics reasons drip bars adjacent to substructure may be provided where the structure is over railroad, water or near integral abutments.

Fatigue analysis of girder or beam for welded connection of drip bars shall be required in the event that there will be effective stress cycles in tension and/or stress reversal on the welds. Bolted drip angle details may be considered instead of welded drip bars details as an alternative.

**Notes:**

Identify direction of grade on plans and orient drip bar accordingly to deflect water and debris.

Case 1 - Provide drip bar detail at all girders to block water from expansion device to protect the girders.

Case 2 or 3 - Provide drip bar detail at exterior face of exterior girders to deflect water and debris from 100 ft. or longer span draining to a bent.

751.14.5.9.2 Drip Bar Details

Bridge Standard Drawings - Drip Bar (http://www.modot.org/business/standards_and_drawings2/DRIP.htm)
Drip Bar for Weathered Steel (https://www.modot.org/drip-bar-weathering-steel-drip)

Welded drip bar details are preferred because they are simpler, require less work and produce less damage to new or existing weathering steel flanges. Bolted drip angles are optional and should only be used when welded drip bars cannot be used. Both sets of details are not intended to be used as alternates on a single job.

751.14.5.10 Weld Design and Detail Guidance**General***Fillet Welds*

Weld design and detail guidance is provided to ensure that the correct fillet weld size is used for fabrication and to reduce incidences of inconsistent and/or incorrect fillet weld sizes shown on the plans which can cause problems for both designer and fabricator.

Preferred MoDOT standard practice is to exclude (leave blank) fillet weld sizes from the bridge plans which is to mean that the fillet weld size shall be determined by the fabricator and that they are contractually obligated to use the minimum fillet weld sizes given in Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14) (shown as the minimum size of fillet weld) of the Standard Specifications.

Excluding fillet weld sizes from the bridge plans is correct in most situations, for example routine welded connections. This accepted and preferred practice prevents the designer from performing unnecessary and complicated load estimates and weld strength computations for mundane welded connections while ensuring consistent plan details for ease of fabrication. Standard Drawings and the details shown in EPG 751.14.5 Details show fillet weld symbols without the fillet weld size and should be assumed adequate for typical applications. Minimum fillet weld sizes are based only on material thickness of parts to be adjoined and are not specific to loading condition or magnitude of load, and are proven empirically to work in practice for typical applications. For more information, see itemized guidance following.

However, excluding fillet weld sizes shall not preclude sound engineering practice of ensuring that a minimum fillet weld size given in accordance with Sec 1080 is acceptable to resist maximum factored loads. This is for typical applications only-ALWAYS CHECK to be sure.

Specialty Welds

For purposes of this section only, any weld not a fillet weld is considered a specialty weld and weld symbol should include all pertinent design information and instruction for fabricating weld, i.e. type, size and instructions.

Itemized Guidance

Itemized guidance is provided to be used for all types of welds.

- No. 1 Weld design shall be in accordance with AISC Steel Construction Manual, 13th Ed., Part 8; AASHTO LRFD Bridge Design Specifications, Bridge Welding Code AASHTO/AWS D1.5M/D1.5 Bridge Welding Code, Missouri Standard Specifications Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14) and EPG 751.5.9 Miscellaneous Details.
- No. 2 The minimum fillet weld size for welding connections shall not be less than the size required by design to resist the maximum factored loads in accordance with LRFD 6.13.3 nor the minimum fillet weld size shown in Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14).
- No. 3 Design of welds shall include fatigue investigation and diaphragm moment effects due to bridge skew and curvature effects if applicable. For routine* plate girders and wide flange beams, leave the weld size blank for the stiffener to web weld and use the requirements in the Standard Specifications. For curved plate girders, skews greater than 50° and spans longer than 200 ft., the stiffener to web fillet weld size should be investigated.
* Judgment and experience are required to determine this; guidance is provided and may not be strict enough in characterizing nonroutine (or nontypical) bridges.
- No. 4 Fillet weld size for weld connections shall not be shown on the plans if minimum required fillet weld size by design is less than or equal to fillet weld size shown in Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14). (Fabricators and inspectors will use the minimum fillet weld size given in Sec 1080 unless larger fillet weld size is shown on the bridge plans.)
- No. 5 Seismic design effects shall also be considered for design of welds when required.
- No. 6 Fillet weld size shall be shown on the bridge plans for specialty weld connections, for example, as shown in EPG 751.14.5.3 End Diaphragm Connection for W27 thru W40 beams and EPG 751.14.5.4 End Diaphragm for Plate Girders, Detail "D". Specialty welds as shown shall be checked for conditions given in No. 4.
- No. 7 Working clearance requirements to allow adequate room for welder to make proper welds shall be checked in accordance with AISC Steel Construction Manual, 13th Ed., Part 8; AASHTO Bridge Design LRFD Specifications and Bridge Welding Code AASHTO/AWS D1.5M/D1.5 Bridge Welding Code.
- No. 8 Field weld symbols should be shown with fillet weld size. In general, field welding symbols should be shown complete, i.e. include all pertinent design information and instruction for fabricating weld, i.e. type, size and instructions.
- No. 9 Angle size for steel diaphragms should be shown without inch (") marks on the plans.

Examples of Design and Detail of Bearing Stiffener Fillet Welds

Ex. 1 Routine Bridge - Nonseismic

- Check No. 3 Routine by criteria (given); no weld size check necessary.
- Check No. 4 Leave weld size blank but show weld symbol.
- Check No. 7 Check working clearances. Is there adequate clearance to make the weld?

Ex. 2 Nonroutine Bridge - Nonseismic

- Check No. 3 Nonroutine by criteria (given); weld size check necessary using No. 1 and No. 2; if design fillet weld size $\leq$ minimum fillet weld size shown in Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14), do not show fillet weld size (leave blank) on plans, but do show fillet weld symbol.
- Check No. 4 Insert design fillet weld size if design fillet weld size $>$ minimum fillet weld size shown in Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14).
- Check No. 7 Check working clearances. Is there adequate clearance to make the weld?

Ex. 3 Nonroutine Bridge - Seismic

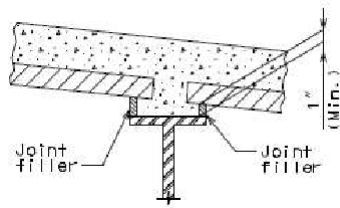
- Check No. 4 Nonroutine by criteria (given); weld size check necessary using No. 1 and No. 2. Seismic; Consult with Structural Project Manager or Structural Liaison Engineer to consider developing loads and designing fillet weld size; If design fillet weld size $<$ minimum fillet weld size shown in Sec 1080 (http://www.modot.org/business/standards_and_specs/SpecbookEPG.pdf#page=14), do not show fillet weld size (leave blank) on plans, but do show fillet weld symbol.
- Check No. 5 Insert design fillet weld size if design fillet weld size $>$ minimum fillet weld size shown in Sec 1080.
- Check No. 7 Check working clearances. Is there adequate clearance to make the weld?

751.14.6 Miscellaneous Details

751.14.6.1 Girder Haunch

Prestressed Panel Section:

Steel girders shall be cambered when using P/S Panels.



Cast-in-Place Section:

Cambering steel girders is preferred over varying the haunch profile. For wide flange beams with nominal depths ≥ 36 in., the cambering method required by the fabricator may not be economical and should be compared to the cost of using variable haunching. When variable haunching is used in lieu of cambering, the maximum haunch limit may be maintained by changing girder slopes at field splices.

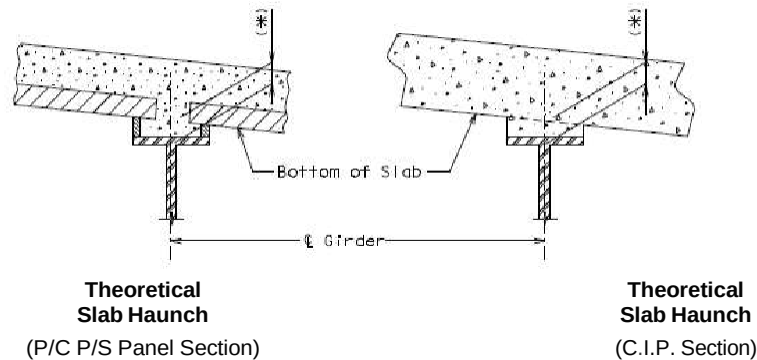
Slab Haunch Limits:



(*) 1/2 in. minimum edge clearance except over splice plates use 1/4 in. min.

751.14.6.2 Girder Camber and Dead Load Deflection**Straight Grade (Dead Load Deflection)**

Compute Dead load Deflection at 1/4 points for bridges with spans less than 75', at 1/10 points for spans 75' and over.



(*) Dimension (bottom of slab to top of web) may vary if girder camber after erection differs from plan camber by more than the % of Dead Load Deflection due

to weight of structural steel. No payment will be made for any adjustment in forming or additional concrete required for variation in haunching.

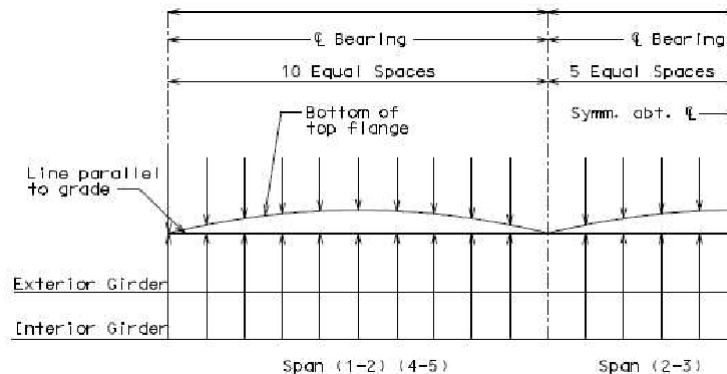


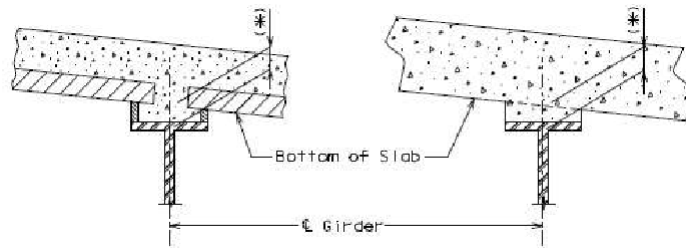
Plate Girder Camber Diagram

Note: Camber included allowance for _____ (**)

_____% of dead load deflection due to weight of structural steel.

(**) Provide proper notes.

Vertical Curves(Dead Load Deflection)



**Theoretical
Slab Haunch**
(P/C P/S Panel Section)

**Theoretical
Slab Haunch**
(C.I.P. Section)

(*) Dimension (bottom of slab to top of web) may vary if girder camber after erection differs from plan camber by more than the % of Dead Load Deflection due to weight of structural steel. No payment will be made for any adjustment in forming or additional concrete required for variation in haunching.

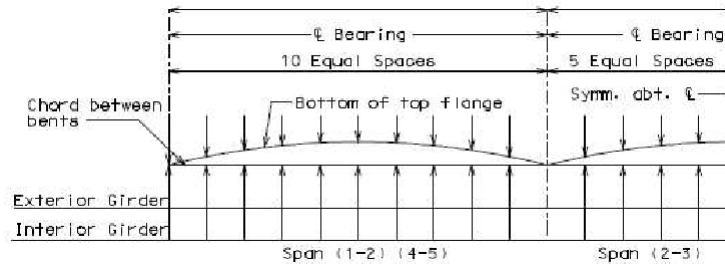


Plate Girder Camber Diagram

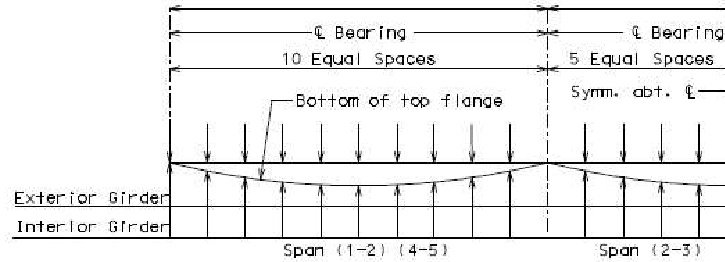
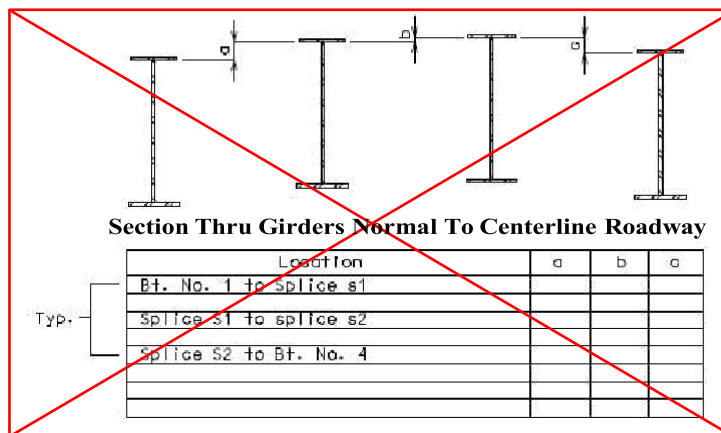


Plate Girder Deflection Diagram

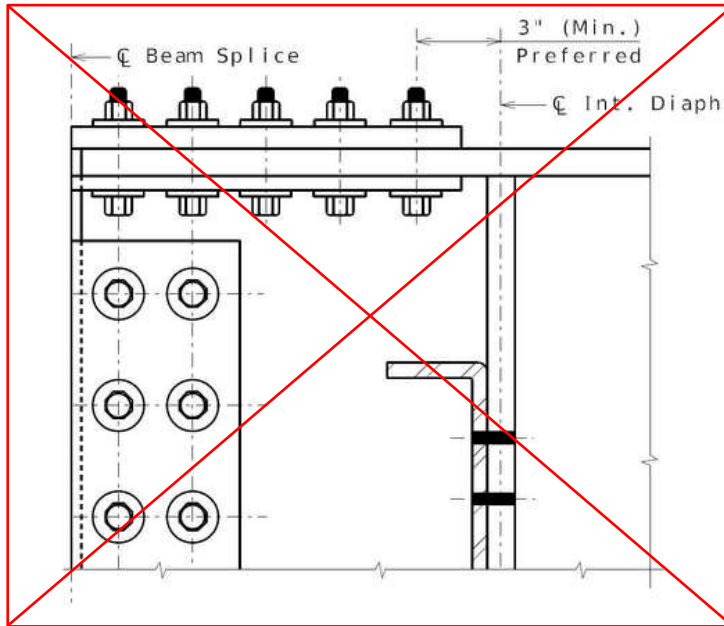
Note: Camber included allowance for _____. (**)
 ____% of dead load deflection due to weight of structural steel.
 (**) Provide proper notes.

~~751.14.6.3 Girder Elevation Variation Sketch~~



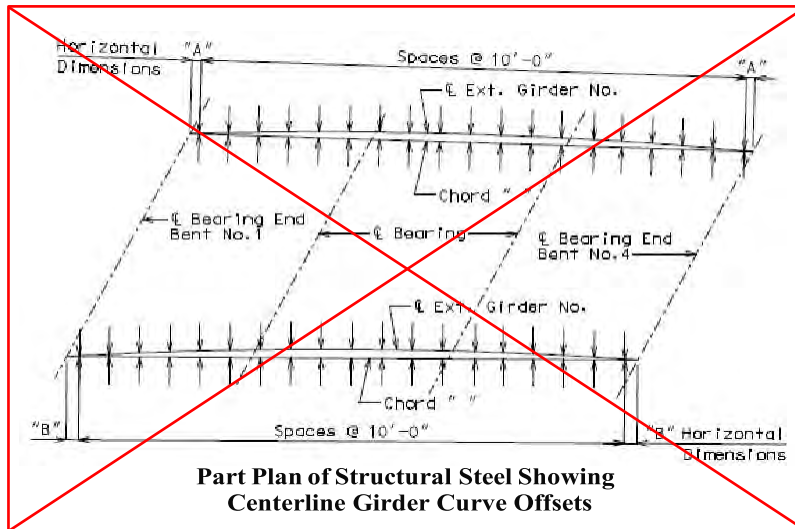
Dimensions showing girder variations shall be placed on the cross section thru slab or, if necessary because of the number and type of variations, the above type of detail and table shall be placed near the structural steel layout.

~~751.14.6.4 Spacing of Intermediate Diaphragm from Splice~~



~~751.14.6.5 Offsets for Curved Plate Girders~~

Plans for structures having horizontally curved plate girders shall have the detail shown below placed near the plan of structural steel.



Note: Typical example for 3 span structure shown.

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751.11 Bearings

751.11.2 Design

751.11.2.1 Elastomeric Bearings

General

Elastomeric Bearing design shall follow AASHTO LRFD "Method A".

The elastomeric expansion bearings and fixed bearings for ^{continuous} steel girders consist of a sole plate and elastomeric bearing pad. Elastomeric bearings at integral end bents and fixed bearings ~~for prestressed~~ ~~girders~~ at intermediate bents ^{with a concrete diaphragm} consist of elastomeric bearing pad without a sole plate.

The sole plates are flat or beveled to match the profile grade of the roadway surface along the centerline of the girder. If the profile grade is equal to or less than 0.01 or the total drop across the bearing is equal to or less than 1/8 inch, then a flat sole plate may be used and if necessary, elastomer thickness increased to address the bearing rotation due to the profile grade. Sole plates are used to anchor girders, ensure uniform distribution of the compressive stress and strain over bearing area and reduce dead load bearing rotations.

At fixed bearings, girder chairs may be considered as an alternate if roadway slope, rotation or bearing pressure is requiring tall or large bearing pads.

The elastomer bearing pad shall be 60 durometer hardness and reinforced with 1/8 inch steel shim plates when laminated pads are required by design.

When rectangular bearings are used, increased rotation bearing capacity can be achieved by orientating the pad with the shorter dimension of the pad parallel with the span of the girder.

Plain elastomeric bearing pads should be utilized where vertical loads, translations and rotations are relatively small. For integral end bents, use 1/2" fixed plain pads when taper due to girder slope or grade is less than 1/8", or use a laminated bearing pad when taper equals or exceeds 1/8" due to girder slope or grade. In the rare circumstance when intermediate bents are made integral by extending the beam cap stirrups into the diaphragm, consideration can be given to utilizing 1/2" plain pads under similar conditions of slope.

Laminated elastomeric bearing pads should be utilized where there is greater need for vertical loading, translational and rotational capacity. For non-integral end bents, non-integral intermediate bents and for integral end bents when taper exceeds 1/8" due to girder slope or grade use laminated elastomeric bearing pads where the pad thickness and number of laminates is based on design that should account for larger vertical loads, translation, rotation and meet slope of girder and grade requirements.

Size Limitations

Use the values in the standard bearing pad tables if possible.

Bearing pad length shall be 8" minimum for MoDOT Standard Prestressed (P/S) I-Girders, Adjacent P/S Box Beams and Steel I-Girders.

Bearing pad length shall be 5" minimum for P/S NU Girders and P/S Spread Box Beams. Not preferred but for consideration of some lower bound limits as used by the Nebraska Department of Roads (NDOR) that developed the NU I-Girder, for P/S NU Girders only, and based on NDOR guidelines, a 4" minimum bearing pad length and/or 2 ft. minimum bearing pad width can be used with Structural Project Manager or Structural Liaison Engineer approval.

Plain Fixed for P/S I-Girder:

$W, \text{ width of bearing} \leq \text{Bottom flange width} - 1.5"$

Plain Fixed for steel girders:

$\text{Bottom flange width} - 2" \leq W \leq \text{Bottom flange width}$

Laminated Fixed for P/S I-Girder:

$W = \text{Bottom Flange width} - 1.5"$

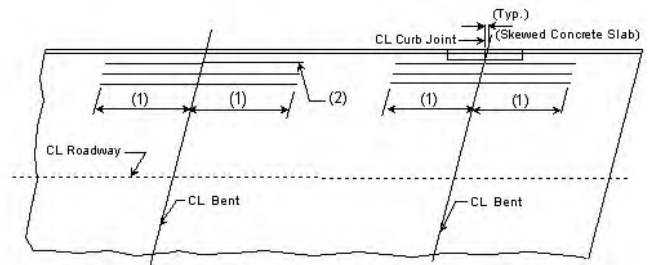
Laminated Expansion Bearing Pads for PS I-Girders and Laminated Fixed and Expansion Pads for Steel Girders:

$9" \leq W \leq \text{Bottom flange width} + 7"$

751.10.1.5 Interior Section Design

Negative Moment Steel over Intermediate Supports

Dimension negative moment steel over intermediate supports as shown.



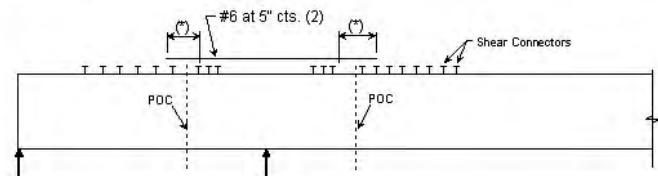
Prestressed Structures

- (1) Bar length by design
 - (2) Reinforcement placed between longitudinal temperature reinforcing in top.
- Bar size: #5 bars at 7 1/2" cts. (Min.)
#8 bars at 5" cts. (Max.)

ad Steel

Cgr
Steel Structures:

- (1) Extend into positive moment region beyond "Anchor" Stud shear connectors at least $40 \times \text{bar diameter} \times 1.5$ (Epoxy Coated Factor) (*) as shown below.
- (2) Use #6 bars at 5-inch centers between longitudinal temperature reinforcing in top.



POC = DC Contra-Flexure Point

(*) $40 \times \text{bar diameter} \times 1.5 = 40 \times 0.75" \times 1.5 = 45"$ for #6 epoxy coated bars.

Simple for Dead Load, Continuous for Live Load Steel and

Locations of termination of reinforcement steel in the deck slab for Prestressed Structures shall be checked for the following criteria and adjusted as necessary:

No greater than 50 percent of the bar count shall be terminated at any section.

Adjacent bars shall not be terminated in the same section.

Flexural reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

The effective depth of the member

15 times the nominal diameter of bar

1/20 of the clear span (centerline to centerline of pier)

Continuing reinforcement shall extend not less than the development length, l_d , beyond the point where reinforcement is no longer required to resist flexure.

At least one third of the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than:

The effective depth of the member

12 times the nominal diameter of bar

0.0625 times the clear span (centerline to centerline of pier)

751.50 Standard Detailing Notes

Stainless Steel Sheet Metal Plate (Use for skewed SDCL structures)

(H1.54)

The stainless steel sheet metal plates shall be bent in the field to clear beams.