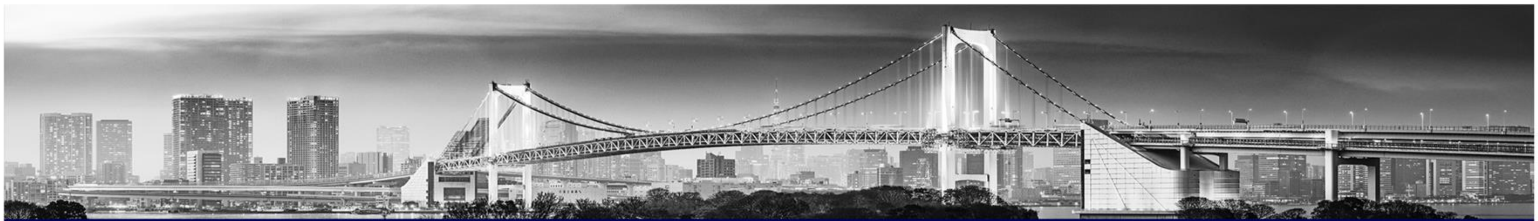
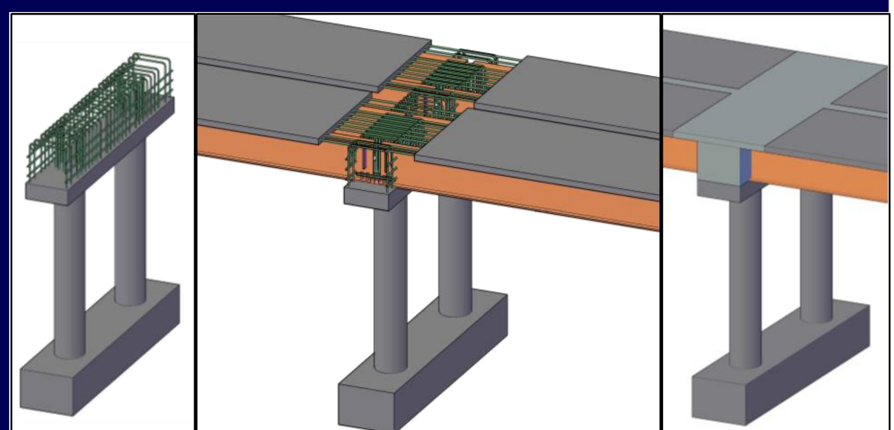


ABC-UTC GUIDE FOR:

SUPERSTRUCTURE TO PIER CONNECTION IN SDCL STEEL BRIDGE SYSTEMS



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ABSTRACT

This report summarizes the work activities undertaken in the study and presents the results of those activities toward development of this ABC-UTC Guide for Superstructure to Pier Connection in Simple for Dead-load Continuous for Live-load (SDCL) Steel Bridge Systems. The information will be of interest to highway officials, bridge construction, safety, design, and research engineers. The document includes summary and background on the research project led to the development of this guide.

ACKNOWLEDGEMENTS

The research study resulting in development of this guideline was supported by the US Department of Transportation through the Accelerated Bridge Construction University Transportation Center (ABC-UTC).

1. INTRODUCTION

Simple for Dead load and Continuous for Live load (SDCL) steel bridge system has been used in conventional and accelerated construction methods of building bridges, mainly in non-seismic areas. The SDCL system is providing new opportunities for developing economical multi-span steel bridge systems. The SDCL system is especially well suited for multi-span bridges with each span having a maximum length of about 75 m. A complete summary of research, application, and performance of the SDCL steel bridge system, as applied to non-seismic areas, using conventional and Accelerated Bridge Construction (ABC) methods of construction, is provided elsewhere (Lampe et al., 2014; Azizinamini, 2014; Farimani et al., 2014; Yakel and Azizinamini, 2014; Javidi et al., 2014).

Application of an SDCL steel bridge system has many advantages including eliminating field splices, eliminating expansion joints, reduced negative moment over the pier, and minimized traffic interruption. Further, encasing the ends of the girder in concrete protects the girder ends and results in enhanced service life and lower inspection and maintenance costs as compared to conventional steel bridge systems.

Conventional steel bridge system construction with field splices may demand closure for intersecting traffic. However, using an integrated deck-girder system (modular approach) with SDCL connection will reduce the impact on feature intersected. Furthermore, for SDCL bridge system with deck-girder modules, the cast-in-place activity is limited to the concrete diaphragm, as opposed to conventional steel bridge construction where the entire deck may be cast-in-place. This results in reducing the on-site construction time.

Figure 1 schematically shows the ABC application of SDCL using modular approach. In this approach the contributory width of the deck is cast on top of the girders and shipped to the final site. These modular units are placed over the pier and abutments and then joined together using longitudinal closure joints and transverse joint (diaphragm over the pier). The key design item is the type of detail that should be used to join the modules over the pier.

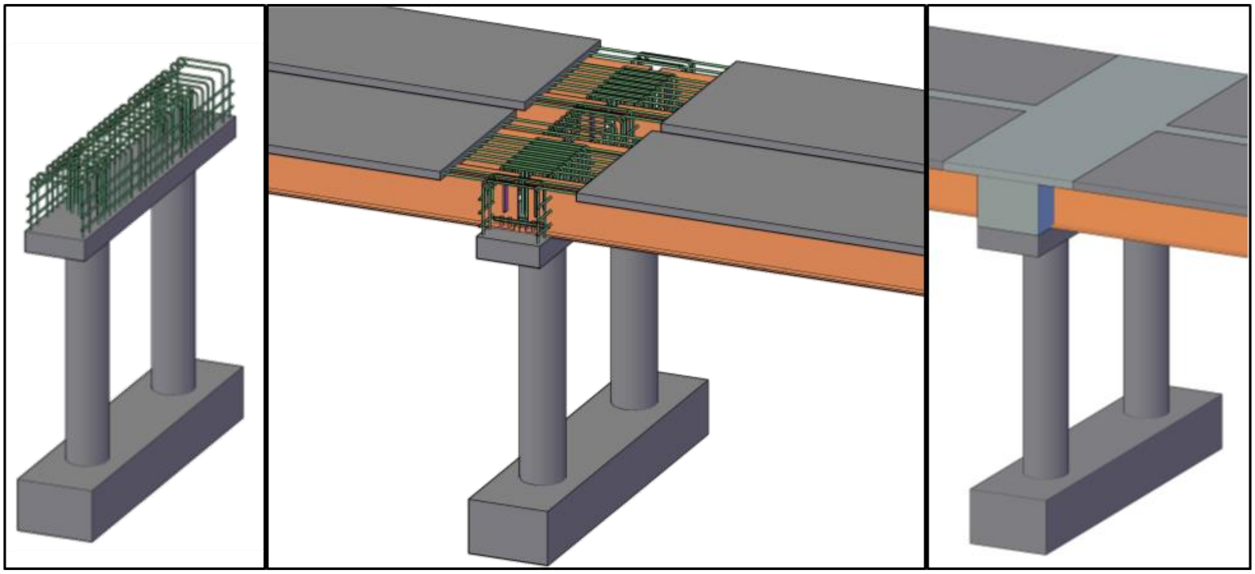

Figure 1 Construction sequence for ABC application of SDCL.

The design philosophy for bridges in seismic areas is to predefine locations for damage to take place and design them for adequate levels of ductility. In this design approach, the superstructure elements are to remain elastic during an entire seismic event. These elements are called capacity-protected elements. The inelasticity is forced to form in the predefined locations, which have sufficient ductility. These damage locations in bridges are located at the ends of columns (forming plastic hinges). In the SDCL steel bridge system, the integral connection of the superstructure and substructure causes the damage location to be at the end of the column near the cap beam.

Developing design provisions for any connection involves identifying failure modes associated with the connection. In this regard, the behavior of the connection was investigated by using detailed non-linear finite element analysis and subjecting the connection to three loading configurations as described below. The numerical model consisted of middle pier and length of the superstructure on either side of the pier to about the point of inflection (zero moment under dead load). The ends of girders were connected over the pier using a concrete diaphragm and the selected connection. The cantilever ends of the girders in the numerical model were subjected to three loading configurations as follows:

A) Push-down loading, simulating the gravity loadings, to approximately comprehend the types of forces that connection elements would experience under gravity loads;

B) Push-up loading, simulating the vertical component of the seismic loads, to approximately comprehend the types of forces and failure modes that connection elements would experience under vertical components of ground motions during seismic events; and

C) Reversal loading, simulating the loadings associated with the longitudinal component (parallel to traffic flow) of the seismic loads, to approximately comprehend the types of forces and failure modes that connection elements would experience under horizontal components of ground motions during seismic events.

The connection selected to join the ends of the girder over the pier is shown in Figure 2. It should be noted that superstructure, including the selected detail and the concrete diaphragm are capacity protected and must remain elastic during entire seismic events.

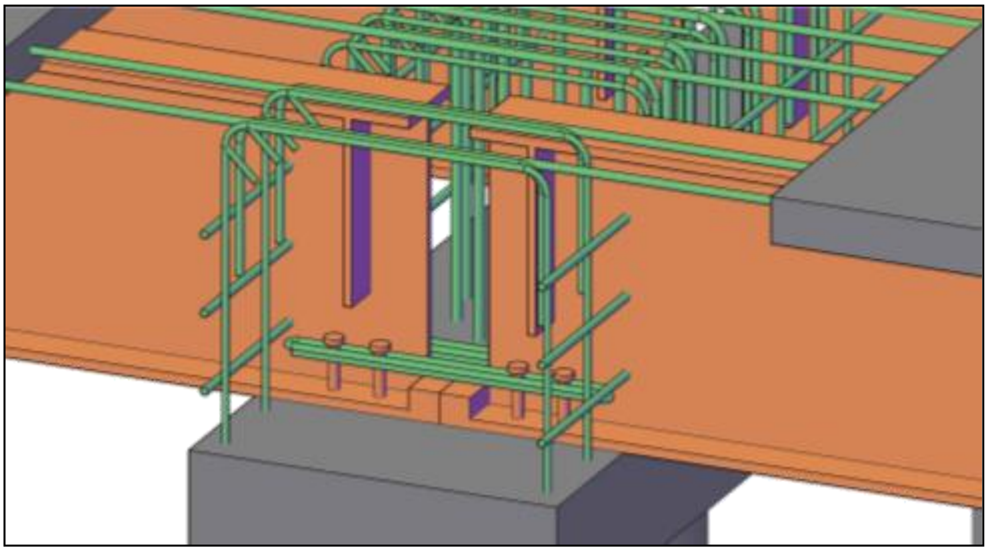

Figure 2 Schematic view of SDCL connection for seismic areas.

Readers are referred elsewhere (Taghinezhadbilondy, 2016, Taghinezhadbilondy et al., 2018) for a detailed description of the numerical work leading to the development of the connection and identifying the function of each connection element during a major seismic event. The following section provides a brief description of the different elements of the proposed connection and their function in an SDCL steel bridge system under high seismic activities.

- ***Tie bars and shear studs on the compression flange:*** This part of the proposed detail is the main difference between the details for non-seismic and seismic application of SDCL. These ties are to accommodate possible tension forces between the girders' bottom flanges. The tension may occur under positive moments, in the pier area, resulting from high vertical seismic excitations. The area of tie bars should be designed to resist a positive moment induced by 25% of the dead load acting upward.
- ***Steel blocks at the end of the compression flanges:*** These blocks are used to transfer the compression forces between girders' bottom flange. This compression force is generated by superimposed dead and live load negative moment. The width of the block is equal to the width of the bottom flange, and the height of it is suggested by the previous work to be one-sixth of the height of the girder. The block size should be checked for negative moment generated from governing live load combination, the resulting moment arm is the distance between deck tension reinforcement and the center of steel blocks. The blocks are welded to the end of the compression flanges.

- ***Dowel bars:*** These reinforcements, similar to available detail for integral cap beam, are designed for the torsion and shear in cap beam. Torsion and shear in the bent-cap occur under longitudinal (along-traffic) excitations, and load transfer from girders to columns. These bars are also the main mechanism to resist the forces developed as a result of moment reversal during the longitudinal component of the ground motion (parallel to traffic flow). Results of numerical studies indicated that the design of dowel bars could be based on established Caltrans (2013) design provisions for capacity-protected elements (Taghinezhadbilondy, 2016).
- ***Live load continuity reinforcement:*** These reinforcements are placed to provide the continuity for live load and superimposed dead loads. The live load continuity deck reinforcement is incorporated in the deck design. In ABC application of SDCL, the deck reinforcement needs to be developed in the diaphragm. One approach is by hooking them inside the concrete diaphragm.

The global and local behavior of the numerical model of the seismic detail under push-down loading was similar to non-seismic detail. Under push-up loading, finite element results showed that continuity of bottom flange increases ductility and capacity of the connection. Since the bottom flange was not continuous, tie bars helped the system to increase the ultimate moment capacity. Under reversal loading, dowel bars were the most critical elements of the connection. The results demonstrated that tie bars over the bottom flange were unable to provide additional moment capacity for the system under moment reversal loading configuration. However, increasing the volume ratio of dowel bars can increase the moment capacity and prevent premature failure of the system under moment reversal, associated with along the traffic component of the ground motion.

Based on the abovementioned details, component level testing was performed at Florida International University (FIU). The main objective of the project was the design and verification testing of a component level specimen using SDCL for seismic areas. If designed properly, the failure should not occur within the connection itself. The test specimen was instrumented to measure the levels of strains in various elements of the detail and verify if capacity-protected elements remained in elastic region, while the plastic hinge formed at the end of the column. Test results verified that the suggested detail can meet the intents of capacity-protected design philosophy. The plastic hinge formed at the end of the column, as designed. The capacity-protected elements remained in elastic region, while the column provided a displacement ductility level of 6 before failing. Failure was by fracture of longitudinal reinforcement in the column and within the plastic hinge regions (Sadeghnejad et al. 2019).

2. NOTATIONS

a	=	Depth of the concrete compressive stress block at critical section (in.)
A_{sc}	=	Area of a shear connector (in.^{2})
A_{sd}	=	Area of steel deck reinforcement in effective width of the deck (in.^{2})
A_{st}	=	Area of tie bars (in.^{2})
b_{eff}	=	Effective width of the deck (in.)
c	=	Structural concrete cover for deck longitudinal reinforcement (in.)
c_c	=	Clear concrete cover (in.)
f_c'	=	Nominal compressive strength of concrete (ksi)
E_c	=	Modulus of elasticity of the deck concrete (ksi)
F_u	=	Specified minimum tensile strength of a stud shear connector (ksi)
F_{yb}	=	Nominal yield stress of steel blocks (ksi)
F_{yd}	=	Nominal yield stress of deck longitudinal reinforcing bars (ksi)
F_{yt}	=	Nominal yield stress of the tie bars (ksi)
h_b	=	Height of steel blocks (in.)
h_c	=	Height of diaphragm (cast-in-place portion of cap beam) (in.)
h_t	=	Distance of tie bars from the precast portion of the cap beam (in.)
l_{dd}	=	Development length of deck longitudinal bars (in.)
l_{dt}	=	Development length of the tie bars (in.)
l_t	=	Length of tie bars (in.)
M_u^-	=	Demand negative moment over the pier (kip-in)
M_u^+	=	Demand positive moment over the pier (kip-in)
n	=	Number of shear connectors on the bottom flange

Q_n = Nominal shear resistance of a single stud shear connector (kips)

Q_r = Factored shear resistance of one shear connector (kips)

t_b = Thickness of the steel blocks (in.)

t_s = Thickness of the deck (in)

w_b = Width of the steel block (in.)

w_c = Width of the cap beam (in.)

w_f = Width of the girder's bottom flange (in.)

ϕ = Resistance factor

ϕ_{sc} = Resistance factor for the shear connectors

3. DESIGN GUIDE

1.0 Simple for Dead-load and Continuous for Live-load (SDCL) steel girder connection

C1.0

The provisions in this section apply to the design and detailing of connection detail over the middle supports for Simple for Dead Continuous for Live (SDCL) steel bridge systems. The SDCL bridge system is constructed as simply supported under superstructure dead load and continuous under superimposed dead load and live load. The continuity is attained through a connection detail at pier cap beam which accommodates force transfer. The connection eliminates the field splice and expansion joint requirement and provides a viable option for Accelerated Bridge Construction (ABC) of steel bridges.

SDCL connection details for non-seismic zones has been extensively investigated numerically and experimentally and their design and field performances have been monitored (Azizinamini 2014, Farimani et al. 2014, Javidi et al. 2014, Lampe et al. 2014, Yakel et al. 2014). The suggested detail for extending the application of SDCL steel bridge system to moderate to high seismic areas is very similar to non-seismic details with some modification (Taghinezhadbilondy 2016, Taghinezhadbilondy et al. 2018, Sadeghnejad et al. 2019).

The design provisions presented are limited to cap beams on pier with no skew. Connection details for curved steel girders are not considered.

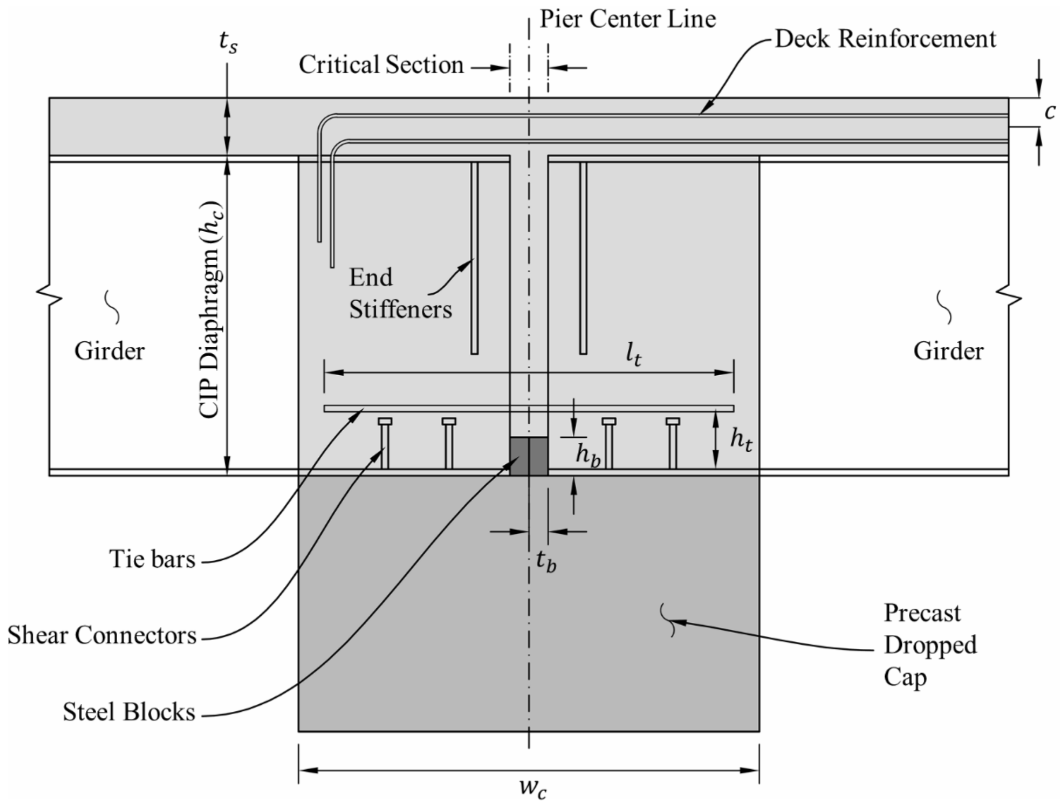

Figure 1.1.1 SDCL Connection.

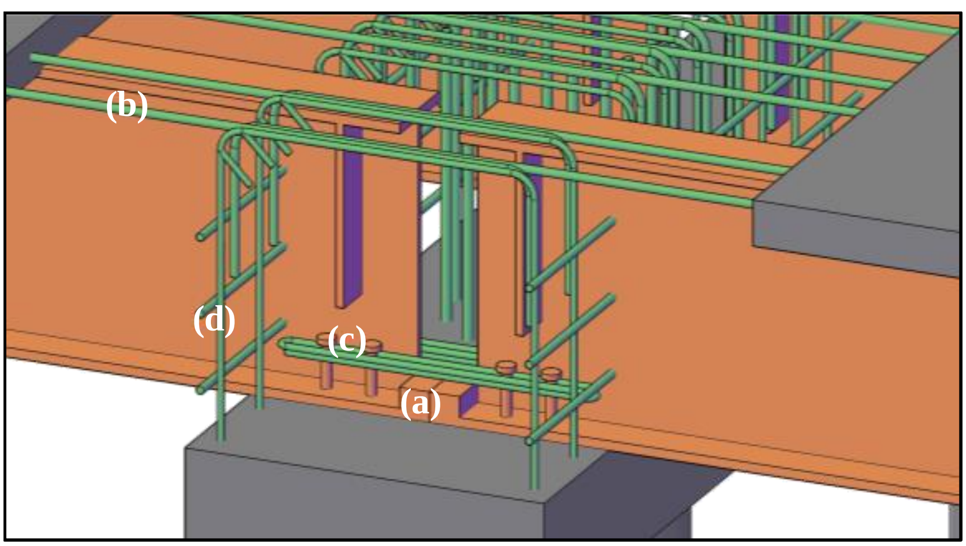

Figure 1.1.2 3D schematic view of SDCL Connection.

1.1 Cap Beam Design

C1.1

Cap beam and column joint should be designed according to Section 8 of AASHTO-LRFD Seismic (2011), Section 5 of AASHTO-LRFD (2014), and Section 7 of Caltrans (2013).

The cap beam in an SDCL system consists of a precast dropped cap and a cast-in-place portion (which creates an integral connection). The combined section contributes to the load carrying capacity of the member and shall be designed accordingly.

For ABC application of connection detail described in this Guide, a dropped cap beam is first placed over the precast columns. The next step is to place the girders with pre-topped deck, supported over cap beam. The last step is to cast the concrete diaphragm and complete the connection.

Figures 1.1.1 and 1.1.2 show schematic of the reinforcements that need to be included in the concrete diaphragm.

Major elements of the connection and their contribution to the load carrying capacity of the SDCL seismic connection, as described in this guide are as follows:

- Tension deck reinforcement and steel blocks as shown in Figure 1.1.2 (a and b) provide tension and compression force mechanism to form a couple that resists the negative moment produced by the live load.
- The tie bars, shown in Figure 1.1.2 (c), resist the tension from the vertical component of the ground acceleration.
- Vertical legs of the closed loop stirrups, shown in Figure 1.1.2 (d), resist the moment reversal during seismic events.

References (Taghinezhadbilondy 2016, Taghinezhadbilondy et al. 2018, Sadeghnejad et al. 2019) provide more detail information on different components of the connection detail and their contribution in resisting different loads applied during a seismic event.

The used design philosophy ensures that concrete diaphragm and superstructure remain elastic, therefore cap beam and connection are capacity protected elements.

Research has shown that the detailing of cap beam satisfying AASHTO and Caltrans joint design requirements is adequate for the SDCL detail (Taghinezhadbilondy 2016, Taghinezhadbilondy et al. 2018).

The dimension of the cap beam along the length of the bridge shall satisfy the following equations:

$$w_c \geq 2(l_{dd} + t_b + c_c)$$
$$\geq 2(l_{dt} + t_b + c_c)$$

Where:

$w_c =$ Width of cap beam (in.)

$l_{dd} =$ Development length of deck longitudinal bars (in.) according to Article 5.11.2 of AASHTO-LRFD (2014).

$t_b =$ Thickness of steel blocks (in.) based on Section 1.3.

$c_c =$ Clear cover concrete (in.) according to Article 5.10.1 of AASHTO-LRFD (2014)

$l_{dt} =$ Development length of the tie bars (in.) according to Article 5.11.2 of AASHTO-LRFD (2014).

The dimension of cap beam along the length of the bridge should accommodate the development of the deck reinforcement (Section 1.2) and tie reinforcement (Section 1.5) at critical section.

1.2 Deck Live Load Continuity Reinforcement

C1.2

At the critical section (end of steel girder) the flexural capacity is provided by tension

Deck live load continuity reinforcement shall be designed according to the negative moment required at the end of the girder as follows:

$$A_{sd} \geq \frac{M_u^-}{\phi F_{yd}(h_c + t_s - c - h_b/2)}$$

Where:

$A_{sd} =$ Area of steel deck reinforcement in effective width of the deck (in.^{2})

$M_u^- =$ Demand negative moment over the pier (kip-in) determined according to Section 3 of AASHTO-LRFD (2014) and Section 7.2.2 of Caltrans (2013)

$\phi =$ Flexural resistance factor according to Article 5.5.4.2 of AASHTO-LRFD (2014) for tension-controlled reinforced concrete sections.

$F_{yd} =$ Nominal yield stress of deck longitudinal reinforcing bars (ksi)

$h_c =$ Height of diaphragm (cast-in-place portion of cap beam) (in.)

$t_s =$ Thickness of deck (in.)

$c =$ Height of steel blocks (in.)

$h_b =$ Height of steel blocks (in.)

in deck longitudinal reinforcement and compression in steel blocks.

Azizinamini (2014) defined the desired mode of failure under negative moments for SDCL connection as yielding of deck reinforcement resulting in a tension-controlled critical section.

The maximum negative moment, from either live load combination of AASHTO-LRFD (2014) or 25% of the dead load applied downward on the superstructure to account for vertical ground acceleration as specified in Caltrans (2013), is used.

The longitudinal deck reinforcement shall be fully developed inside diaphragm (cast-in-place potion of cap beam) at critical section.

The development of deck reinforcement can be achieved by 90° hooked bars.

1.3 Steel Blocks

Steel block dimensions shall be proportioned as follows:

$$w_b = w_f$$

$$h_b \geq \frac{1.7 A_{sd} F_{yd}}{w_b F_{yb}}$$

C1.3

Design and proportioning of steel blocks are according to non-seismic SDCL described by Azizinamini (2014) and Farimani et al. (2014).

An iterative process can be used to size the steel block and find the amount of

$$t_b \geq 2 \text{ in.}$$

Where:

$h_b =$ Height of steel blocks (in.)

$A_{sd} =$ Area of steel deck reinforcement in effective width of the deck (in.^{2})

$F_{yd} =$ Nominal yield stress of deck longitudinal reinforcing bars (ksi)

$w_b =$ Width of steel block (in.)

$F_{yb} =$ Nominal yield stress of steel blocks (ksi)

$w_f =$ Width of the bottom flange (in.)

$t_b =$ Thickness of steel block (in.)

deck reinforcing steel required in the connection.

Steel blocks can be welded to the bottom flange and part of the web using full penetration weld.

1.4 End Stiffeners

The end stiffeners shall be designed according to Article 6.10.11 of AASHTO (2014).

C1.4

The use of end stiffeners along with steel blocks help in improving the behavior of connection under negative moment loading (Azizinamini 2014, Farimani et al. 2014). Stiffeners may be required for the bearing of girder seats.

The proportioning of stiffeners should accommodate placement of the tie bars (Section 1.5)

1.5 Tie bars

The required area of tie bars shall be determined from the following equations:

$$A_{st} \geq \frac{M_u^+}{\phi F_{yt}(d_c + d_d - h_t - a/2)}$$

$$a = \frac{A_{st} F_{yt}}{0.85 f_c' b_{eff}}$$

Where:

$A_{st} =$ Area of tie bars (in.^{2})

C1.5

The tie bars should be designed for the positive moment resulting from vertical ground acceleration at the end of the girder. The demand positive moment is determined by applying 25% of the dead load upward to the superstructure to account for vertical ground excitation as specified in Caltrans (2013).

The design of the critical section is based on concrete stress block parameters with the tie bars as tension reinforcement and

$M_u^+ =$ Demand positive moment over the pier (kip-in) determined based on Article 7.2.2 of Caltrans (2013)

$\phi =$ Flexural resistance factor according to Article 5.5.4.2 of AASHTO-LRFD (2014) for tension-controlled reinforced concrete sections.

$F_{yt} =$ Nominal yield stress of the tie bars (ksi)

$h_c =$ Depth of diaphragm (cast-in-place portion of cap beam) (in.)

$t_s =$ Depth of the deck (in.)

$a =$ Depth of the concrete compressive stress block at critical section (in.)

$h_t =$ Height of the position of tie bars (in.)

$f_c' =$ Nominal compressive strength of concrete (ksi)

$b_{eff} =$ Effective width of the deck (in.) determined according to Article 6.10.1.1.1e and Article 4.6.2.6 of AASHTO-LRFD (2014).

effective width of the deck as the width of the compression block.

Length of the tie bars shall be determined using the following equation:

$$l_t \geq 2(l_{dt} + t_b)$$

Where:

$l_t =$ Length of the tie bars (in.)

$t_b =$ Thickness of steel blocks (in.)

$l_{dt} =$ Development length of the tie bars (in.)

The tie bars at the critical section are required to be fully developed.

1.6 Shear connectors on the bottom flange

The total shear resistance of the shear connectors shall be determined from the following equation

C1.6

The shear connectors on the bottom flange transfer the tensile force in the bottom flange to the tie bars.

$$nQ_r = A_{st}F_{yt}$$

$n =$ Number of shear connectors on the bottom flange

$Q_r =$ Factored shear resistance of one shear connector (kips) determined from Article 6.10.10.4.1 of AASHTO (2014).

$A_{st} =$ Area of tie bars (in.^{2}) according to Section 1.5.

$F_{yt} =$ Nominal yield stress of the tie bars (ksi)

4. DESIGN EXAMPLE

The design example 2A from FHWA Steel bridge design handbook (HIF-16-002) (Barth 2015) is used for presenting the connection design for an SDCL bridge system. The girders in the original design example were spliced according to the traditional design method so the girder is continuous for dead loads. The procedures herein are limited to the design of the connection. However, the design of the girder has no significant impact on the presented design process. For completeness, readers may refer to conventional design procedures for steel girders with SDCL condition.

The bridge under consideration is a two-span (90 ft - 90 ft) continuous bridge as shown in Figure 3. The example bridge has four plate girders spaced at 10.0 ft and 3.5 ft overhangs. The roadway width is 34.0 ft and is centered over the girders. The reinforced concrete deck is 8.5-inch thick, including a 0.5-inch integral wearing surface, and has a 2.0-inch haunch thickness.

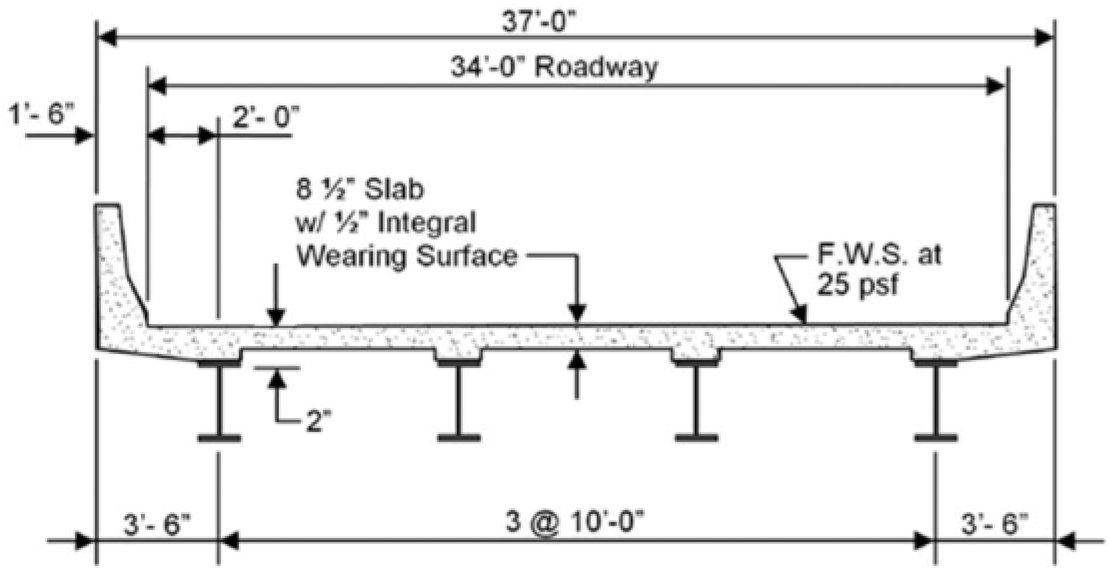

Figure 3 Sketch of bridge cross section (Barth 2015).

The structural steel is ASTM A709, Grade 50W, and the concrete is normal weight with a 28-day compressive strength, f_c', of 4.0 ksi. The concrete slab is reinforced with nominal Grade 60 reinforcing steel.

The calculations presented here are with the assumption of a constant cross-section of steel girder along the length of the bridge. “Section 1” of the original design example was used for this example.

Loads

The following loads were taken from the original design example.

$$DC1 = 1.308\ \mathrm{k/ft}$$

$$DC2 = 0.260\ \mathrm{k/ft}$$

$$DW = 0.213\ \mathrm{k/ft}$$

Live load and dead load moments at the middle pier:

$M_{DC1,SDCL} = 0\ \ \mathrm{kip.ft} = 0\ \mathrm{kip.in}$	Zero moment for DC1 (Simply supported)
$M_{DC1,Cont.} = -1334\ \ \mathrm{kip.ft} = -16008\ \mathrm{kip.in}$	To be used for the vertical component of Seismic Excitation. See Figure 4 at Pier
$M_{DC2} = -265\ \ \mathrm{kip.ft} = -3180\ \mathrm{kip.in}$	See Figure 4 at Pier
$M_{DW} = -217\ \ \mathrm{kip.ft} = -2604\ \mathrm{kip.in}$	See Figure 4 at Pier
$M_{LL}^{-} = -1737\ \ \mathrm{kip.ft} = -20844\ \mathrm{kip.in}$	See Figure 4 at Pier
$M_{LL}^{+} = 0\ \mathrm{kip.ft} = 0\ \mathrm{kip.in}$	See Figure 4 at Pier

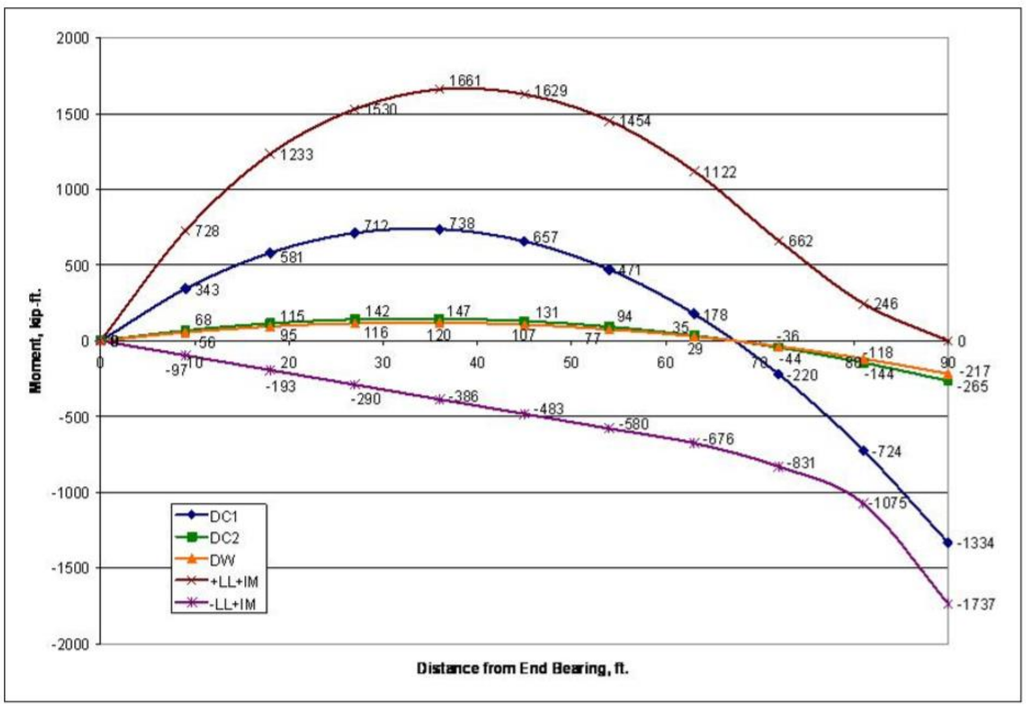

Figure 4 Dead and live load for the case of spliced girder (Barth 2015).

$M_{DC1,SDCL}$ is the resulting moment from DC1 loading in an SDCL condition and $M_{DC1,Cont.}$ is the moment in continuous girder condition. Therefore, after the concrete diaphragm hardening and having a continuous condition, the resulting positive and negative moment from vertical accelerations can be calculated using $M_{DC1,Cont.}$. The resulting earthquake loading assuming $PGA \geq 0.6g$ according to section 7.2.2 of Caltrans (2013) will be:

$$M_{EQ} = \pm 0.25\left(M_{DC1,Cont.} + M_{DC2} + M_{DW}\right) = \pm 5448 \text{ kip. in}$$

Load Combinations

Strength I:

$$M^{-}_{u,Strength\_I} = 1.25M_{DC1,SDCL} + 1.25M_{DC2} + 1.5M_{DW} + 1.75M_{LL} = -44358 \text{ kip. in}$$

$$M^{+}_{u,Strength\_I} = 0 \text{ kip. in}$$

Extreme I:

$$M^{-}_{u,Extreme\_I} = 1.25M_{DC1,SDCL} + 1.25M_{DC2} + 1.5M_{DW} + 0.5M^{-}_{LL} + 1.0M^{-}_{EQ} = -23751 \text{ kip. in}$$

$$M^{+}_{u,Strength\_I} = 0.5M^{+}_{LL} + 1.0M^{+}_{EQ} = 5448 \text{ kip. in}$$

Critical Load Combination

$$M^{-}_{u} = \max\left(M^{-}_{u,Strength\_I}, M^{-}_{u,Extreme\_I}\right) = -44358 \text{ kip. in}$$

$$M^{+}_{u} = \max\left(M^{+}_{u,Strength\_I}, M^{+}_{u,Extreme\_I}\right) = 5448 \text{ kip. in}$$

Cap Beam Design

The combined section of the cap beam should be designed and detailed according to Section 8 of AASHTO (2011) and Section 5 of AASHTO (2014). The precast dropped cap should be designed for construction loadings (including the weight of superstructure and wet concrete of diaphragm) considering the girders/modules placement sequence.

Deck Live Load Continuity Reinforcement

A steel block with a height of $h_b = 3$ in. is assumed. After calculating the required deck live load continuity reinforcement, the adequacy of this height will be checked.

$$A_{sd,req} = \frac{M^{-}_{u}}{\phi F_{yd}(h_c + t_s - c - h_b/2)} = \frac{44358 \text{ kip. in.}}{0.9(60)(46 \text{ in.} + 8 \text{ in.} - 4 \text{ in.} - 3 \text{ in.}/2)} = 16.94 \text{ in.}^2$$

$$h_c = d_{girder} + d_{haunch} = 44 \text{ in.} + 2 \text{ in.} = 46 \text{ in.}$$

$$t_s = 8 \text{ in.}$$

$$c = 4 \text{ in.}$$

The ratio of total deck reinforcement for exterior girder, which has a smaller effective width of the deck, $b_{eff} = 102 \text{ in.}$, will be as following:

$$\rho_{req} = \frac{A_{sd,req}}{t_s b_{eff}} = 0.0206 \text{ (\#7 @ 6 in. Top and Bottom)}$$

Steel Blocks

The width of the steel block was assumed to be the same as the bottom flange and thickness was assumed as 2 in. The assumed height of block should be checked according to section 1.3. In case the required height is more than the assumption, the previous step, for calculating the deck reinforcement, should be repeated with a higher steel block height.

$$w_b = w_f = 16 \text{ in.}$$

$$t_b = 2 \text{ in.}$$

$$h_b = 3 \text{ in.} \geq \frac{1.7 A_{sd} F_{yd}}{w_b F_{yb}} = 2.16 \text{ in.} \qquad \text{(O.K.)}$$

Tie bars

The placement of the tie bars was assumed to be 1 in. above the steel blocks, $h_t = 4 \text{ in.}$ Area of 2 in.^{2} was assumed as the first guess for tie bars. Then the requirement of section 1.5 was checked.

$$a = \frac{A_{st} F_{yt}}{0.85 f_c' b_{eff}} = 0.34 \text{ in.}$$

$$A_{st,req} = \frac{M_u^+}{\phi F_{yt}(d_c + d_d - h_t - a/2)} = \frac{5448 \text{ kip. in.}}{0.9(60)(46 \text{ in.} + 8 \text{ in.} - 4 \text{ in.} - 0.34 \text{ in.}/2)}$$

$$A_{st,req} = 2.025 \text{ in.}^2 > 2 \text{ in.}^2 \qquad \text{(Not O.K.)}$$

Second guess: $A_{st} = 2.1 \text{ in.}^2$

$$a = \frac{A_{st} F_{yt}}{0.85 f_c' b_{eff}} = 0.36 \text{ in.}$$

$$A_{st,req} = \frac{M_u^+}{\phi F_{yt}(d_c + d_d - h_t - a/2)} = \frac{5448 \text{ kip. in.}}{0.9(60)(46 \text{ in.} + 8 \text{ in.} - 4 \text{ in.} - 0.36 \text{ in.}/2)}$$

$$A_{st,req} = 2.025 \text{ in.}^2 < 2.1 \text{ in.}^2 \qquad \text{(O.K.)}$$

$$\text{(4 \#7 bars placed 2 one each side of girders)}$$

Shear connectors on the bottom flange

Stud shear connectors with a height of 4 in. and diameter of 0.75 in. was assumed to be used. The factored shear resistance of a single stud shear connector is calculated based on Article 6.10.10.4.1 of AASHTO (2014) as follows.

$$Q_r = \phi_{sc} Q_n$$

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u$$

Where:

$Q_n =$ Nominal shear resistance of a single stud shear connector (kips)

$\phi_{sc} =$ Resistance factor for shear connectors as specified in Article 6.5.4.2 of AASHTO (2014)

$A_{sc} =$ Area of a stud shear connector (in.^{2})

$E_c =$ Modulus of elasticity of the deck concrete determined as specified in Article 5.4.2.4 of AASHTO (2014) (ksi)

$F_u =$ Specified minimum tensile strength of a stud shear connector determined as specified in Article 6.4.4 of AASHTO (2014) (ksi)

The shear resistance of a stud and number of studs required can be calculated as:

$$Q_n = 0.5(0.44 \text{ in.})\sqrt{(4 \text{ ksi})(3834 \text{ ksi})} \leq (0.44 \text{ in.})(60 \text{ ksi}) = 26.4 \text{ kips}$$

$$Q_r = 0.85\ (26.4 \text{ kips}) = 22.44 \text{ kips}$$

$$n_{req} = \frac{A_{st} F_{yt}}{Q_r} = 5.6$$

$$n = 6$$

Total of 6 studs are required and can be placed equally (3) on both sides of the girder's bottom flange.

The connection designed in this example is schematically shown in Figure 5.

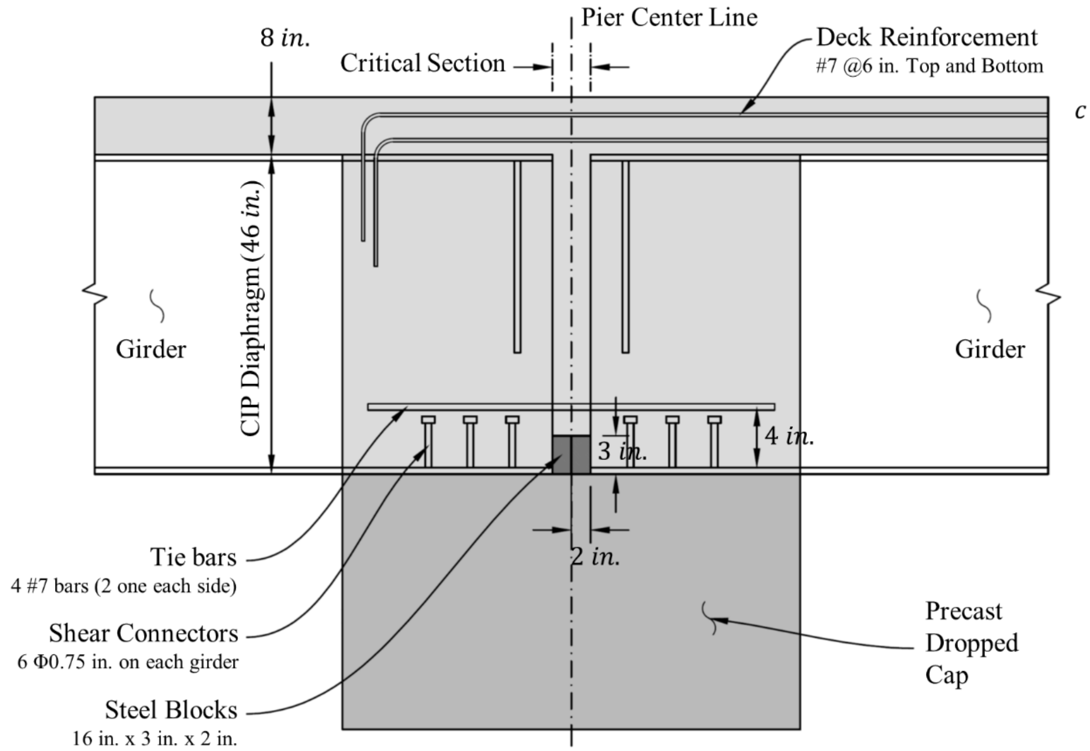

Figure 5 Designed connection detail over the intermediate pier.

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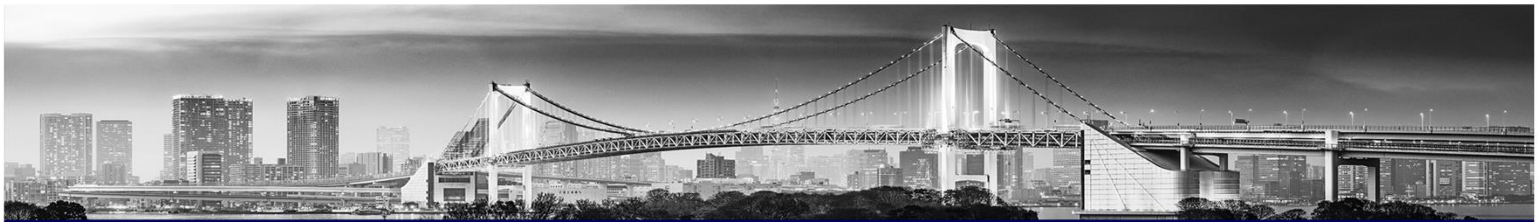

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