

ACCELERATED BRIDGE CONSTRUCTION  
UNIVERSITY TRANSPORTATION CENTER

ABC-UTC GUIDE FOR:

# REBAR HINGE POCKET CONNECTIONS, HYBRID GROUTED DUCT CONNECTIONS, *AND* SDCL STEEL GIRDER CONNECTIONS

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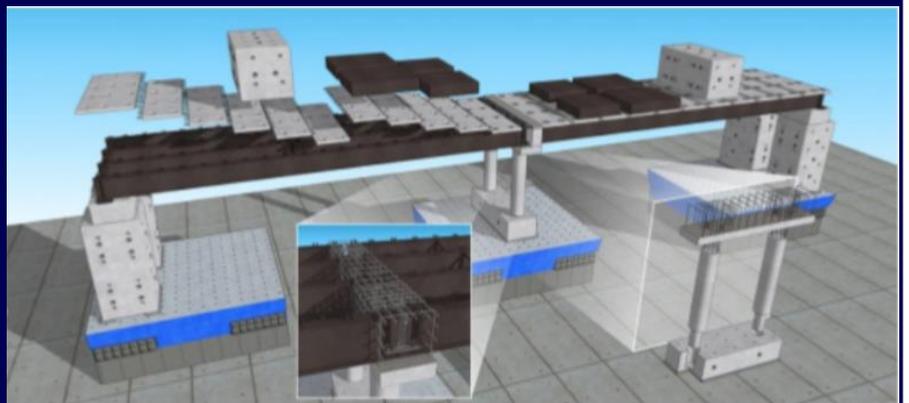
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## TABLE OF CONTENT

<b>ABSTRACT</b> .....	<b>2</b>
<b>1. Introduction</b> .....	<b>3</b>
1.1. <i>Background</i> .....	3
1.2. <i>Intended Users</i> .....	4
1.3. <i>Rebar Hinge Pocket Connections</i> .....	7
1.4. <i>Minimum Area of Rebar Hinge Section</i> .....	8
1.5. <i>Minimum Transverse Steel</i> .....	9
1.6. <i>Shear Design</i> .....	9
1.7. <i>Hinge Throat Thickness</i> .....	10
1.8. <i>Pocket Minimum Depth</i> .....	11
1.9. <i>Pocket and Socket Details</i> .....	12
<b>2. Column To Hybrid Cap Beam Grouted Duct Connections</b> .....	<b>14</b>
2.1. <i>Joint Design</i> .....	14
2.2. <i>Minimum Anchorage Length for Column Longitudinal Bars</i> .....	14
2.3. <i>Precast Cap Beam Design</i> .....	16
2.4. <i>Details of Grouted Ducts</i> .....	16
2.5. <i>Interface Load Transfer Strength</i> .....	16
<b>3. Simple for Dead-Load and Continuous if Liveload Steel Girder Connections</b> .....	<b>17</b>
3.1. <i>Cap Beam Design</i> .....	19
3.2. <i>Deck Live Load Continuity</i> .....	21
3.3. <i>Steel Blocks</i> .....	21
3.4. <i>End Stiffeners</i> .....	22
3.5. <i>Tie Bars</i> .....	23
3.6. <i>Sheer Connectors on the Bottom Flange</i> .....	24



## ABSTRACT

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This ABC-UTC Guide provides step-by-step design specifications that were developed for three connections intended to be used for accelerated bridge construction. These connections include rebar hinge pocket connections, hybrid grouted duct connections, and SDCL steel girder connections. The document also presents a short background information and a concise summary of the main research study conducted toward development of this Guide. The information will be of interest to highway officials, bridge construction, safety, design, and research engineers.

## ACKNOWLEDGMENTS

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## 1. INTRODUCTION

### 1.1. BACKGROUND

Bridge cast-in-place construction often leads to traffic delays, subjects highway workers and the traveling public to increased probability of accidents, and may affect the regional economy of the residents. By utilizing prefabricated bridge elements, accelerated bridge construction (ABC) shortens onsite construction time. Accordingly, ABC saves time and money for the traveling public and enhances the work-zone safety. Due to the fact that prefabricated components are built offsite and under controlled environmental conditions, ABC provides the opportunity to use novel materials and to increase the quality and durability of the components. ABC can also reduce the total duration of projects as prefabrication of bridge components can be performed simultaneously.

Connections between prefabricated elements (hereby referred to as ABC connections) play a crucial role in adequate performance of ABC bridges under moderate and strong earthquakes. ABC connections have to be practical and efficiently constructible and at the same time provide clear load path under vertical and lateral loading. When used for connecting columns to the adjoining members, ABC connections must allow for the energy dissipation in the column while maintaining the capacity and the integrity of the structural system.

Several researchers (Matsumoto et al. 2001; Restrepo et al. 2011; Tazarv and Saiidi 2014; Motaref et al. 2011; Mehroorouh, et al. 2016; Mehraein and Saiidi 2016) have developed and investigated a variety of ABC connections and prefabricated elements in the past decade. These connections include but are not limited to grouted duct connections, pocket and socket connections, mechanical bar splices, simple for dead continuous for live (SDCL) connections of various configurations, and connections for partial or full precast deck panels. The primary intent of these studies was to assess the local behavior of ABC connections, formulate preliminary design guidelines, and build a certain level of confidence in utilizing ABC techniques.

While providing invaluable information on the local behavior of ABC connections, component tests do not provide confidence in the performance of the bridge systems when subjected to bi-directional loading. Therefore, to understand the holistic seismic behavior of ABC bridges, Comprehensive analytical and experimental investigations of a large-scale two-span steel girder bridge model incorporating six ABC connection types subjected to bi-directional horizontal earthquake motions were conducted.

Findings of the aforementioned research study as well as recent related studies were utilized to develop design guidelines and detailing recommendations for three of the six connections incorporated in the bridge model. The objective was to facilitate field deployment of ABC connections. The connections included rebar hinge pocket connections, hybrid grouted duct connections, and SDCL steel girder connections. Design provisions for SDCL steel girder connections were developed by Amir Sadeghnejad and Dr. Atorod Azizinamini at Florida International University.



## 1.2. INTENDED USERS

The design procedures and detailing recommendations for the three ABC connections presented in this document will be of interest to highway officials, bridge construction, safety, design, and research engineers.

### DESIGN PROVISIONS

#### NOTATIONS

- $a$  = Depth of the concrete compressive stress block at critical section (in.)
- $A_{sc}$  = Area of a shear connector (in.<sup>2</sup>)
- $A_{sd}$  = Area of steel deck reinforcement in effective width of the deck (in.<sup>2</sup>)
- $A_{sp}$  = Area of one hinge hoop or spiral (in.<sup>2</sup>)
- $A_{st}$  = Area of tie bars (in.<sup>2</sup>)
- $B_c$  = Column largest cross-sectional dimension (in.)
- $b_{eff}$  = Effective width of the deck (in.)
- $c$  = Cohesion factor
- $c_{cd}$  = Structural concrete cover for deck longitudinal reinforcement (in.)
- $c_c$  = Clear concrete cover (in.)
- $d_b$  = Hinge bar diameter (in.)
- $d_{bl}$  = Diameter of longitudinal column reinforcement (in.)
- $D_{s1}$  = Depth of the precast cap beam (in.)
- $E_c$  = Modulus of elasticity of the deck concrete (ksi)
- $f'_c$  = Nominal compressive strength of concrete (ksi)
- $f'_{cg}$  = Nominal compressive strength of grout (cube strength) (ksi)
- $f_y$  = Hinge bar yield strength (ksi)
- $f_{ye}$  = Expected yield stress of longitudinal column reinforcement (ksi)
- $f_{yh}$  = Nominal yield stress of the hinge reinforcement (ksi)



- $f_{yp}$  = Steel pipe yield stress (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.)
- K1 = Fraction of concrete strength available to resist interface shear (ksi)
- K2 = Limiting interface shear resistance (ksi)
- $l_{ac}$  = Anchored length of column longitudinal bars beyond the ducts (in.)
- $l_d$  = Tension development length of the rebar hinge longitudinal bars (in.)
- $l_{dd}$  = Development length of deck longitudinal bars (in.)
- $l_{dt}$  = Development length of the tie bars (in.)
- $L_p$  = Plastic hinge length (in.)
- $l_t$  = Length of tie bars (in.)
- $M_u^-$  = Superstructure demand negative moment at the face of cap beam (kip-in)
- $M_u^+$  = Superstructure demand positive moment at the face of cap beam (kip-in)
- $n$  = Number of shear connectors on the bottom flange
- P = Applied axial load, under the combined action of the vertical load and the maximum lateral load (kips)
- $P_u$  = Design axial load (kips)
- $Q_n$  = Nominal shear resistance of a single stud shear connector (kips)
- $Q_r$  = Factored shear resistance of one shear connector (kips)



- $S_h$  = Spacing of transverse hoops or spirals in equivalent CIP joint
- $t$  = Height of the hinge throat (in.)
- $t_b$  = Thickness of the steel blocks (in.)
- $t_p$  = Pipe thickness (in.)
- $t_s$  = Thickness of the deck (in)
- $T_s$  = Total tension force in rebar hinge longitudinal bars (kips)
- $V_n$  = Nominal shear capacity of the rebar hinge section (kips)
- $V_u$  = Shear demand at the hinge (kips)
- $w_b$  = Width of the steel block (in.)
- $w_c$  = Width of the cap beam (in.)
- $w_f$  = Width of the girder's bottom flange (in.)
- $\mu$  = Shear friction factor
- $\theta$  = Angle between the horizontal axis of the bent cap and the pipe helical corrugation or lock seam (deg)
- $\theta_n$  = Hinge ultimate rotation
- $\theta_e$  = Hinge elastic rotation
- $\theta_p$  = Hinge plastic rotation
- $\theta_{e\text{clos}}$  = Hinge rotation corresponding to the hinge throat closure
- $\phi$  = Resistance factor
- $\phi_{sc}$  = Resistance factor for the shear connectors



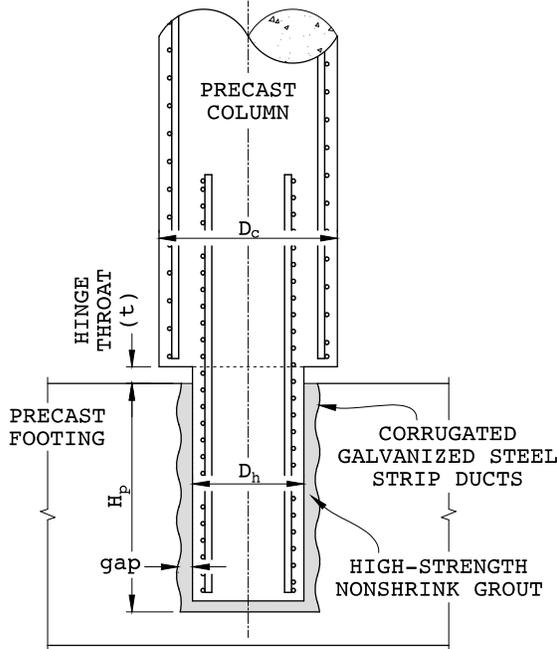
### 1.3—Rebar Hinge Pocket Connections

#### C1.0

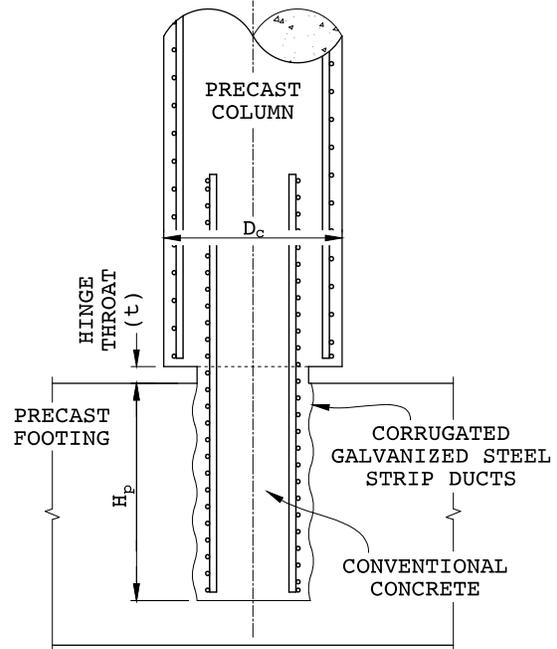
Rebar hinge is the most commonly used column hinge in the United States that can be used either at the top or bottom of reinforced concrete columns. Design of the rebar hinges has not been codified; however, Cheng et al. (2010) developed a step by step design guideline for rebar hinges based on extensive experimental and analytical studies.

Rebar hinge pocket or socket (in which the hinge element is precast or consists of a rebar cage alone, respectively) connection is a viable alternative connection for accelerated bridge construction (ABC), which combines rebar hinge details with those of the pocket connection. A hinge element integrated with a precast column is extended into a pocket left in the footing. The hinge element may be precast or consist of a reinforcing cage that extends from the column into a footing opening. The former is shown in Figure 1.1-1. The latter would consist of only the hinge reinforcement cage as shown in Figure 1.1-2 (Culmo et al. 2017). Only a few experimental studies have incorporated rebar hinge pocket connections as part of a precast bent or bridge system (Mehrsoroush et al. 2017, Mohebbi et al. 2017, Shoushtari et al. 2019).

Design and detailing guidelines for rebar hinge pocket and socket connections are presented herein based on previous research.



**Figure 1.1-1—Column to Footing Rebar Hinge Pocket Connection**



**Figure 1.1-2—Column to Footing Rebar Hinge Socket Connection**

**1.4—Minimum Area of Rebar Hinge Section**

The gross area of the rebar hinge section shall be at least:

$$A_g \geq \frac{P_u}{0.2f'_c} \quad (1.1-1)$$

$P_u$  = Design axial load (kips)

$f'_c$  = Concrete compressive strength (ksi)

**C1.1**

Eq. 1.1-1 was recommended by Cheng et al. (2010). It is intended to avoid compressive failure at the hinge.

**1.5—Minimum Transverse Steel**

The volumetric ratio of the transverse reinforcement in a rebar hinge section shall be determined based on moment-

**C1.2**

Experimental studies by Cheng et al. (2010) showed that using a target curvature ductility of 10 ensures ductile



curvature analysis of the hinge for a minimum curvature ductility of 10.

Transverse steel can be in the form of spiral or hoops and shall be extended  $l_d$  into the column and adjoining member, where:

$l_d$  = Tension development length of the rebar hinge longitudinal bars in accordance to Article 5-11-2-1 (AASHTO, 2012).

behavior of the hinge specimen. The Mortensen-Saiidi method (Mortensen and Saiidi 2002) is a non-iterative performance-based method that was developed to design confinement reinforcement in concrete columns for a specified performance level.

For hinge section, the core concrete is essentially confined by the transverse reinforcement in both the hinge and the column because of the relatively small depth of the hinge throat. The hinge cover concrete is confined by the column transverse steel for the same reason. Therefore, an effective confined lateral pressure, and transverse steel ratio should be used in determining the confined concrete properties in the moment-curvature analysis of the hinge section (Cheng et al. 2010).

**1.6—Shear Design**

The plastic shear demand at the hinge shall satisfy Eq. 1.3-1.

$$V_u \leq \phi_s V_n \quad (1.3-1)$$

Where:

$V_u$  = Shear demand at the hinge (kips), determined based on Article 8.6.1 AASHTO (2014)

$V_n$  = Nominal shear capacity of rebar hinge section (kips)

$\phi_s$  = 0.9 for shear in reinforced

**C1.3**

The amount of longitudinal steel is determined from shear design procedure.

The design procedure is iterative and may require revision of the hinge area or longitudinal steel.

Under lateral loading, the flexural moment at the hinge section causes flexural crack. Therefore, conventional shear friction theory (ACI 318 2008; AASHTO 2012) that assumes a clamping force at the entire interface is not applicable (Cheng et al. 2010). Experimental studies have shown that cyclic loads reduce roughness in the hinge and the aggregate interlock in the compression zone of the hinge (Cheng et



concrete

Nominal shear capacity of a two-way hinge section shall be taken as:

$$V_n = \mu(P + T_s) \quad (1.3-2)$$

where:

$P$  = Applied axial load, under the combined action of the vertical load and the maximum lateral load (kips)

$T_s$  = Total tension force in rebar hinge longitudinal bars (kips)

$\mu$  = 0.45, shear friction factor

al. 2010). Therefore, a reduced shear friction factor is recommended in Eq. 1.3-2, compared to the corresponding factor in AASHTO (2012), which is  $\mu=0.6$ , for concrete cast against hardened concrete which is not intentionally roughened.

**1.7—Hinge Throat Thickness**

The height of the hinge throat,  $t$ , as shown in Figures 1.1-1 and 1.1-2, shall satisfy the following criteria:

$$\theta_n < \theta_{close} \quad (1.4-1)$$

where:

$$\theta_n = \theta_e + \theta_p \quad (1.4-2)$$

$$\theta_e = t \phi_y \quad (1.4-3)$$

$$\theta_p = L_p (\phi_y - \phi_y) \quad (1.4-4)$$

$$L_p = t + 0.15 f_y d_b \quad (1.4-5)$$

$$\theta_{close} = \sin^{-1} (t / 0.5 B_c) \quad (1.4-6)$$

**C1.4**

The purpose of hinge throat is to allow for hinge rotation and avoid closure of the gap that could damage the edge of the column and increase the hinge moment. Sufficient height of the hinge throat ensures that hinge closure is prevented (Cheng et al. 2010).



Where:

- $L_p$  = Plastic hinge length (in.)
- $t$  = Height of the hinge throat (in.)
- $f_y$  = Hinge bar yield strength (ksi)
- $\phi_y$  = Hinge section effective yield curvature
- $\phi_u$  = Hinge section ultimate curvature
- $d_b$  = Hinge bar diameter (in.)
- $\theta_n$  = Hinge ultimate rotation
- $\theta_e$  = Hinge elastic rotation
- $\theta_p$  = Hinge plastic rotation
- $\theta_{close}$  = Hinge rotation corresponding to the hinge throat closure
- $B_c$  = Column largest cross-sectional dimension (in.)

**1.8—Pocket Minimum Depth**

When the hinge element is precast, the depth of a rebar hinge pocket,  $H_p$ , as shown in Figures 1.1-1, shall be at least:

$$H_p \geq l_d + cc + gap \quad (1.5-2)$$

where:

- $H_p$  = Rebar hinge pocket or socket depth (in.)
- $l_d$  = Required tension development length of the hinge longitudinal bars into the adjoining members in accordance to Article 5-11-2-1 (AASHTO, 2012) (in.)
- $cc$  = Concrete cover over hinge

**C1.5**

Providing concrete cover over the reinforcement at the end of the hinge specimen is not necessary, as filler material between the specimen and pocket provides adequate protection against corrosion. However, concrete cover, when provided, shall be considered in Eq. 1.5-2.



reinforcement (Article 5-12-3 ,AASHTO 2012) (in.)

*gap* = The gap between the precast hinge element and pocket base (Article 1.7, Figure 1.1-1) (in.)

When the hinge element consists of only extended hinge rebar cage (Figure 1.1-2), *gap* shall be taken as zero.

### 1.9—Pocket and Socket Details

Pockets and sockets shall be constructed with helical, lock-seam, corrugated steel pipes, conforming to ASTM A706. The pipe thickness ( $t_p$ ) shall be greater than 0.06 in.

When precast hinge elements are used, high-strength, non-shrink grout shall be used as the pocket filler. The grout shall be sufficiently fluid when rebar hinge specimen is embedded into the pocket. The compressive strength of the filler material sampled and tested according to an appropriate ASTM standard shall be at least 15 percent higher than concrete compressive strength of the footing.

The gap between the pocket and rebar hinge specimen shall be at least 2.0 in. but no more than 4.0 in.

When the hinge element consists of hinge rebar cage alone, concrete with a compressive strength of at least equal to that of the footing shall be used as the socket filler.

### C1.6

The 0.06 in., which was proposed by Restrepo et al. (2011), ensures the constructability of the pipe. Further information about corrugated steel pipe material and thickness can be found in Tazarv and Saiidi (2015), and Restrepo et al. (2011).

The requirement for grout compressive strength exceeding that of the concrete in the footing ensures that no weak link is formed in the connection. The 15-percent overstrength factor is due to the fact that compressive strength of 2.0-in cubes (as recommended by ASTM for grout sampling) are typically more than those obtained from cylinder testing. Further information can be found in Tazarv and Saiidi (2015).

Sufficient gap between the hinge specimen and pocket not only provides adequate construction tolerance, but also ensures that filler material easily flows through the pocket.



## C2.0

A grouted duct connection includes corrugated metal ducts embedded in the adjoining precast members to anchor individual projected column longitudinal reinforcing bars. The ducts are then filled with high-strength non-shrink grout. Several researchers have studied bond behavior and performance of the grouted duct connections (Matsumoto et al. 2001, Pang et al. 2008, Steuck et al. 2009, Restrepo et al. 2011). Experimental studies have shown that grouted duct connections are emulative of cast-in-place construction.

Hybrid cap beams consist of a precast and a cast-in-place segment with the former incorporating grouted ducts. A column to hybrid cap beam grouted duct connection consists of a lower precast cap beam (stage I cap beam or precast dropped cap beam) to support the girders and a cast-in-place portion (stage II cap beam) to integrate the pier and superstructure. Column bars are extended into the corrugated metal ducts that are grouted afterward, but extend beyond the ducts into the CIP segment of the cap beam (Fig.2.1-1).



## 2. COLUMN TO HYBRID CAP BEAM GROUTED DUCT CONNECTIONS

### 2.1—Joint Design

Joint proportioning and joint shear design shall satisfy AASHTO (2014) 8-13.

The full depth of the combined lower and upper parts of the cap beam participates in resisting the joint forces in both the longitudinal and transverse directions.

In the precast part of the cap beam, joint transverse reinforcement shall be placed around the ducts that anchor the column bars.

### 2.2—Minimum Anchorage Length for Column Longitudinal Bars

Anchorage of the column longitudinal bars is provided through bond in a combination of grouted ducts and CIP concrete. The stress that is transferred through bond in the ducts is:

$$f_s = \frac{D_{s1} f'_{cg}}{2d_{bl}} \quad (2.2-1)$$

Where:

$f_s$  = Steel stress transferred through bond in the ducts (ksi)

$D_{s1}$  = Depth of the precast cap beam (in.)

$f'_{cg}$  = Nominal compressive strength

### C2.2

Eq. 2.2-1 is based on research by Matsumoto et al. 2008.



of grout (cube strength) to be taken no greater than 7 ksi (ksi)

$d_{bl}$  = Diameter of longitudinal column reinforcement (in.)

The extension of the bars beyond the ducts shall satisfy the following equation:

$$l_{ac} \geq \frac{2d_{bl}(f_{ye}-f_s)}{f'_c} \quad (2.2-2)$$

$l_{ac}$  = Anchored length of column longitudinal beyond the ducts (in.)

$f_{ye}$  = Expected yield stress of longitudinal column reinforcement (ksi)

$f'_c$  = Concrete compressive strength (ksi)

Grout compressive strength in Eq. 2.2-1 shall be limited to 7000 psi.

### 2.3—Precast Cap Beam Design

Precast cap beam shall be designed for its self-weight and the superstructure. Depth of the precast cap beam shall be sufficient to develop the required strength in column bars for construction loading. Torsional moments due to sequential placement of girders shall be taken into consideration in design of the precast cap beam.



## 2.4—Details of Grouted Ducts

### C2.4

Semi-rigid corrugated metal (steel) ducts specified per ASTM A653 shall be used to anchor column bars.

Semi-rigid corrugated metal ducts provide sufficient anchorage between the column bar, grout, and surrounding concrete (Restrepo et al. 2011).

Matsumoto et al. (2001) provides background and details on grouting of duct connections in terms of grout testing, grout placement, and other grouting issues.

## 2.5—Interface Load Transfer Strength

### C2.5

The load transfer strength at the column-cap beam interface shall be calculated in accordance to AASHTO (2012) 5.8.4.1-3 equation, using the following parameters:

Specified values for  $c$ ,  $\mu$ ,  $K1$  and  $K2$  in equation 5.8.4.1-3 were proposed by Marsh et al. (2011). It was shown that, to account for cyclic loading effects and the potential for significant cracking, the cohesion factor,  $c$ , should be ignored.

$$c = 0$$

$$\mu = 0.6$$

$$K1 = 0.2$$

ksi

$$K2 = 0.8$$

ksi



### 3. SIMPLE FOR DEAD-LOAD AND CONTINUOUS FOR LIVE LOAD (SDCL) STEEL GIRDER CONNECTIONS

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

#### C3.0

The provisions in this section apply to the design and detailing of connection detail over interior piers 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 that accommodates force transfer. The connection eliminates plate girder field splices and expansion joints. This detail provides a viable option for accelerated bridge construction (ABC) of steel plate girder bridges.

SDCL connection details for non-seismic zones has been extensively investigated and their design and field performances have been established (Azizinamini 2014, Farimani et al. 2014, Javidi et al. 2014, Lampe et al. 2014, Yakel et al. 2014). The details of this connection are similar to non-seismic details with some modification (Taghinezhadbilondy 2016, Taghinezhadbilondy et al. 2018, Sadeghnejad et al. 2019).

Currently, the design provisions are limited to straight and non-skew bridges.

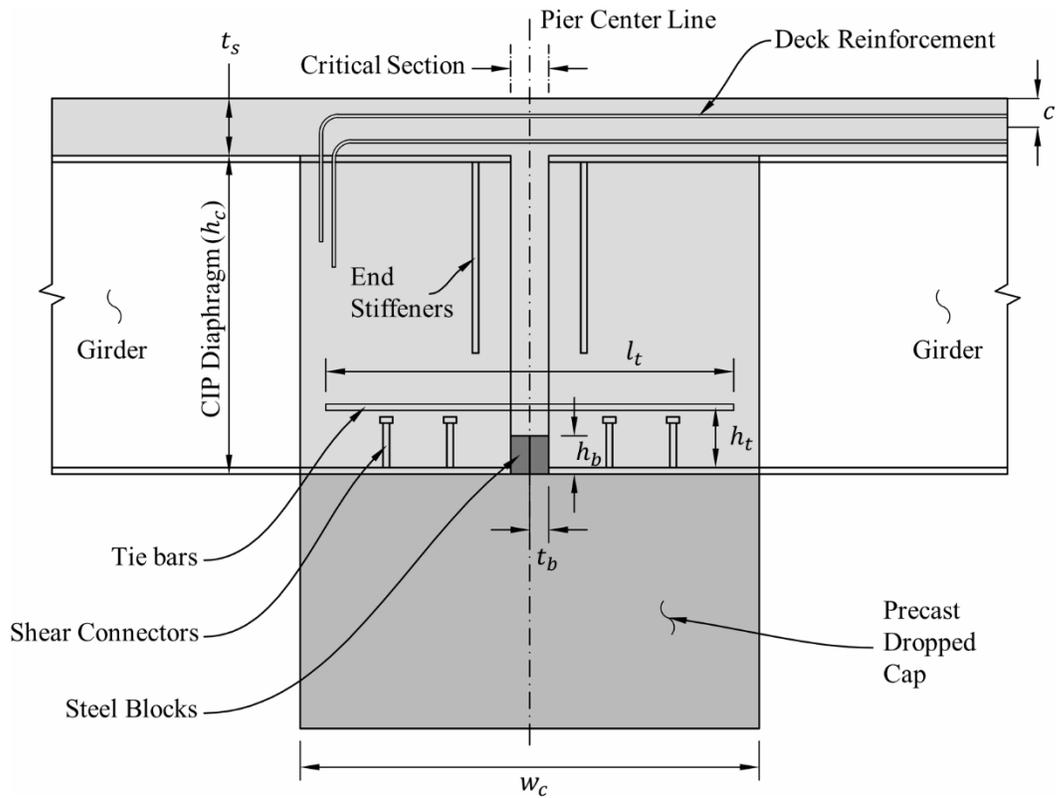


Figure 1.1.1 SDCL Connection.

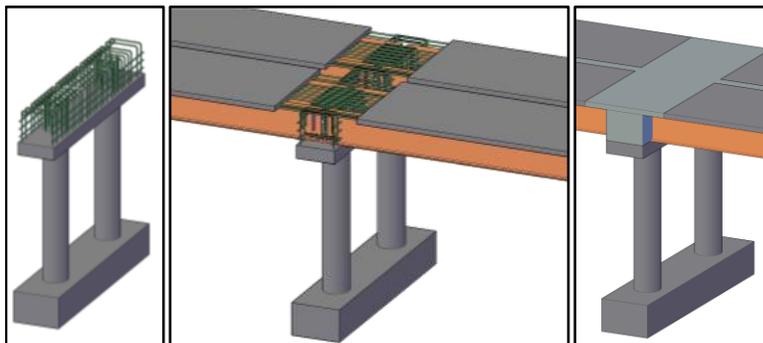


Figure 1.1.2 Construction sequence for ABC application of SDCL.

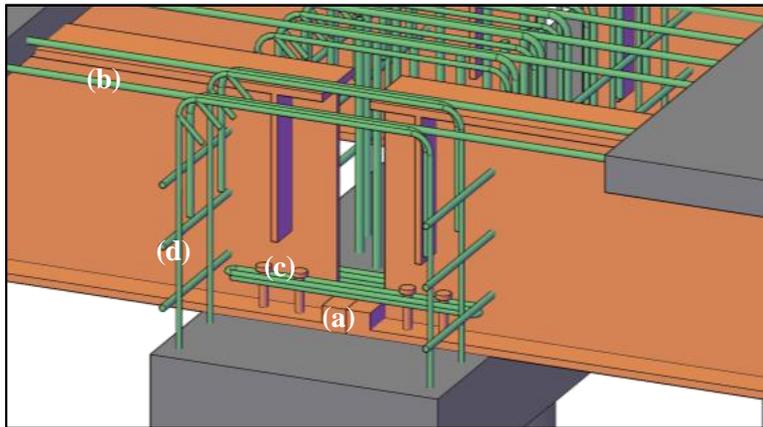


Figure 1.1.3 3D schematic view of SDCL Connection.

### 3.1. CAP BEAM DESIGN

#### 3.1 Cap Beam Design

Cap beam and column joint shall be designed according to Section 8 of AASHTO-Irfd Seismic (2014), Section 5 of AASHTO-LRFD (2012), and Section 7 of Caltrans (2013).

#### C3.1

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

For ABC application, a dropped cap beam is first placed over 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, as shown in Figure 3.1.2.

Figure 3.1.3 shows schematic of the reinforcement that needs 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 3.1.3 (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 3.1.3 (c), resist the tension from the vertical component of the ground acceleration.
- Vertical legs of the closed loop stirrups, shown in Figure 3.1.3 (d), resist the moment reversal during seismic events.

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

The cap beam and connection are capacity protected elements.

Research has shown that 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 (2012).

$t_b$  = Thickness of steel blocks (in.) based on Section 3.3.

$c_c$  = Clear cover concrete (in.) according to Article 5.10.1 of AASHTO-LRFD (2012)

$l_{dt}$  = Development length of the tie bars (in.) according to Article 5.11.2 of AASHTO-LRFD (2012).

The dimension of cap beam along the length of the bridge should accommodate the development of the deck reinforcement (Section 3.2) and tie reinforcement (Section 3.5) at critical sections.



### 3.2. DECK LIVE LOAD CONTINUITY REINFORCEMENT

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

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

Where:

$A_{sd}$  = Area of steel deck reinforcement in effective width of the deck (in.<sup>2</sup>)

$M_u^-$  = Demand negative moment over the pier (kip-in) determined according to Section 3 of AASHTO-LRFD (2012) and Section 7.2.2 of Caltrans (2013)

$\phi$  = Flexural resistance factor according to Article 5.5.4.2 of AASHTO-LRFD (2012) 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.)

$c_{cd}$  = Structural concrete cover for deck longitudinal reinforcement (in.)

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

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

#### C3.2

At the critical section (end of steel girder) the flexural capacity is provided by tension 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 (2012) 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 development of deck reinforcement can be achieved by 90° hooked bars.



### 3.3. STEEL BLOCKS

Steel block dimensions shall be proportioned as follows:

$$w_b = w_f$$

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

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

Where:

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

$A_{sd}$  = Area of steel deck reinforcement in effective width of the deck (in.<sup>2</sup>)

$F_{yd}$  = 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.)

#### C3.3

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 determine the amount of 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.

### 3.4.END STIFFENERS

The end stiffeners shall be designed according to Article 6.10.11 of AASHTO (2012).

#### C3.4

The use of end stiffeners along with steel blocks help improve 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)



### 3.5. 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.<sup>2</sup>)

$M_u^+$  = 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 (2012) 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 (2012).

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

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

#### C3.5

The tie bars should be designed for the positive moment resulting from vertical ground acceleration at the end of the girder. According to Caltrans (2013), the demand positive moment is determined by applying 25% of the dead load upward to the superstructure to account for vertical ground excitation. Recent studies by Shoushtari, et al. (2019) have indicated that a larger fraction of the dead load should be applied in bridges in near-fault zones.

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

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



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.)

### 3.6 Shear Connectors on the Bottom Flange

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

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

$n$  = 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 (2012).

$A_{st}$  = Area of tie bars (in.<sup>2</sup>) according to Section 1.5.

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

### C3.6

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



## REFERENCES

- ACI Committee, & International Organization for Standardization. (2008). Building code requirements for structural concrete (ACI 318-08) and commentary. American Concrete Institute.
- American Association of State Highway and Transportation Officials (AASHTO), (2012). "AASHTO LRFD Bridge Design Specifications", 6th edition.
- American Association of State Highway and Transportation Officials (AASHTO). (2014). "Guide Specifications for LRFD Seismic Bridge Design", Washington, D.C.
- Azizinamini, Atorod. 2014. "Simple for dead load-continuous for live load steel bridge systems." *AISC Engineering Journal* 51 (2):59-81.
- Barth, K., 2015. *Steel bridge design handbook design example 2a: Two-span continuous straight composite steel I-girder bridge* (No. FHWA-HIF-16-002-Vol. 21).
- Caltrans. 2013. Seismic Design Criteria Version 1.7.: California Department of Transportation, Sacramento, CA.
- Cheng, Z. Y., Saiidi, M. S., & Sanders, D. (2010). "Seismic Design Method for Reinforced Concrete Two-Way Column Hinges". *ACI Structural Journal*, 107(5).
- Culmo, M. P., Marsh, L., Stanton, J., & Mertz, D. (2017). "Recommended AASHTO Guide Specifications for ABC Design and Construction" (No. NCHRP Project 12-102).
- Farimani, Reza, Saeed Javidi, Derek Kowalski, and Atorod Azizinamini. 2014. "Numerical Analysis and Design Provision Development for the Simple for Dead Load – Continuous for Live Load Steel Bridge System." *AISC Engineering Journal*.
- Javidi, Saeed, Aaron Yakel, and Atorod Azizinamini. 2014. "Experimental Investigation, Application and Monitoring of a Simple for Dead Load – Continuous for Live Load Connection for Accelerated Modular." *AISC Engineering Journal*.
- Lampe, Nick, Nazanin Mossahebi, Aaron Yakel, Reza Farimani, and Atorod Azizinamini. 2014. "Development and experimental testing of connections for the simple for dead load-continuous for live load steel bridge system." *AISC Engineering Journal* 51 (2):83-106.
- Marsh, L.M., Wernli, M., B.E.Garrett, J.F.Stanton, M.O.Eberhard, and M.D.Weinert. (2011). NCHRP Report 698 "Application of Accelerated Bridge Construction Connections in Moderate-to-High Seismic Regions". Washington, D.C.: National Cooperative Highway Research Program, Transportation Research Board.
- Matsumoto, E. E., Mark, C., Waggoner, M. E. K., Vogel, J., & Wolf, L. (2008). "Development of a precast concrete bent-cap system". *PCI journal*.
- Matsumoto, E. E., Waggoner, M. C., Sumen, G., Kreger, M. E., Wood, S. L., & Breen, J. E. (2001). "Development of a Precast Bent Cap System" (No. Report no. RR-1748-2). University of Texas at Austin. Center for Transportation Research.



- Mehrsoroush, A., Saiidi, M., & Ryan, K. (2017). "Development of Earthquake-Resistant Precast Pier Systems for Accelerated Bridge Construction in Nevada". (No. 555-14-803).
- Mohebbi, A., Saiidi, M. S., & Itani, A. M. (2018). "Shake Table Studies and Analysis of a Precast Two-Column Bent with Advanced Materials and Pocket Connections". *Journal of Bridge Engineering*, 23(7), 04018046.
- Mortensen, J., and M. Saiidi, (2002) "A Performance-Based Design Method for Confinement in Circular Columns," Civil Engineering Department, University of Nevada, Reno, Report No. CCEER 02-07.
- Pang, J., K.P.Steuck, L.Cohagen, Stanton, J., and Eberhard, M. (2008). "Rapidly Constructible Large-Bar Precast Bridge-Bent Seismic Connection". Olympia, WA: Washington State Department of Transportation. WA-RD 684.2
- Restrepo, J. I., M. J. Tobolski, and E. E. Matsumoto, (2011). "Development of a precast bent cap system for seismic regions." NCHRP Rep. 681. Washington, DC: National Cooperative Highway Research Program.
- Sadeghnejad, Amir, Ramin Taghinezhadbilondy, and Atorod Azizinamini. 2019. "Seismic Performance of a New Connection Detail in an SDCL Steel Bridge System." *Journal of Bridge Engineering*, in press
- Shoushtari, E., Saiidi, M., and Itani, A., Moustafa, M. (2019) "Design, Construction, and Shake Table Testing of a Two-Span Steel Girder Bridge System with ABC Connections". *ASCE Journal of Bridge Engineering*, in press
- Steuck, K., Stanton, J., and Eberhard, M. (2009). "Anchorage of Large-Diameter Reinforcing Bars in Ducts". *ACI Structural Journal*, Volume 106, No. 4, pp. 506-513.
- 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.
- Taghinezhadbilondy, Ramin, Aaron Yakel, and Atorod Azizinamini. 2018. "Deck-pier connection detail for the simple for dead load and continuous for live load bridge system in seismic regions." *Engineering Structures* 173:76-88. doi: 10.1016/j.engstruct.2018.06.086.
- Tazarv, M., & Saiidi, M. S. (2015). "Design and Construction of Precast Bent Caps with Pocket Connections for High Seismic Regions". Report No. CCEER 15-06, August 2015
- Yakel, Aaron, and Atorod Azizinamini. 2014. "Field Application Case Studies and Long-Term Monitoring of Bridges Utilizing the Simple for Dead — Continuous for Live Bridge System." *AISC Engineering Journal*:155-176.



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