SPECIAL FOCUS: BRIDGE INNOVATIONS

Simple for Dead Load–Continuous for Live Load Steel Bridge Systems

ATOROD AZIZINAMINI

ABSTRACT

This paper presents the development of design, fabrication, erection and construction procedures for a new steel bridge system referred to as simple for dead and continuous for live loads (SDCL). The economy of the SDCL system depends on several factors, including the use of economical and efficient details over the interior supports. Extensive experimental, numerical and analytical work was conducted to develop economical details and efficient ways of using the SDCL steel bridge system for straight and minimally skewed bridges in which the skew angle is less than 10°. Several bridges were then designed using the research results, and their performance from the time of fabrication to construction and their long-term behavior were studied. This knowledge was then used to provide detailed information on design, fabrication, erection and construction procedures for steel bridges using the SDCL system. This paper provides a summary of the entire work, including design recommendations and economical details to be employed in conjunction with the SDCL steel bridge system.

Keywords: steel bridges, steel girders, SDCL, simple for dead load–continuous for live load.

INTRODUCTION

The simple for dead and continuous for live load (SDCL) system is providing steel bridges with new horizons and opportunities for developing economical bridge systems, especially in cases for which accelerating the construction process is a priority. The system has many advantages over conventional methods of constructing straight and minimally skewed steel bridges in which the skew angle is less than 10°, including lower initial and life-cycle costs, easier inspection and reduced maintenance. The key to economic application of the system lies in selecting appropriate connection details over interior supports to provide live load continuity. Due to the lack of guidelines, unnecessarily complicated details are often used in practice. The SDCL steel bridge system also provides an attractive alternative for application in highly seismic areas, but additional research is necessary.

The objective of this paper is to provide complete information for design, fabrication and construction of straight and slightly skewed steel bridges using the SDCL concept. The information presented is based on more than 10 years of comprehensive research, parts of which are published in the form of final research reports (Azizinamini, Lampe and Yakel, 2003; Azizinamini, Yakel and Farimani, 2005; Azizinamini et al., 2005). The investigation included the development of an economical concept for the SDCL system for use in both conventional and Accelerated Bridge Construction (ABC), development of design provisions and field application and long-term monitoring of SDCL systems that are open to traffic. Additional results of this comprehensive research effort are summarized and published in the research dissertations of six graduate students (Lampe, 2001; Mossahebi, 2004; Otte, 2006; Farimani, 2006; Kowalski, 2007; Javidi, 2009). This paper provides a summary of the overall work that was carried out, with references to more detailed information on various aspects of the research study.

The following briefly describes four significant aspects of the research study:

Experimental Investigation. Full-scale tests were carried out to develop practical details for joining girders over the interior supports for live load continuity using conventional construction practices (Lampe et al., 2014).

Force Transfer Mechanism. Comprehensive numerical and analytical studies were carried out to provide an understanding of the force transfer mechanism for the details that were developed to connect the steel girders in the SDCL system over the interior supports (Farimani et al., 2014). A detailed design approach was then developed that provided an excellent comparison with the test results. This detailed design method was then used to develop a more practical and simplified design approach that gives conservative results and is suitable for use in a design office. The simplified design steps for one recommended connection detail are provided in this paper.

Monitoring of In-Service SDCL Bridges. Several bridges were constructed using the recommended details developed in these research projects. The behavior of

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these bridges during construction and their long-term performance were monitored (Yakel and Azizinamini, 2014). Monitoring of the in-service behavior of SDCL steel bridges ranged from two to more than five years.

SDCL System for Accelerated Bridge Construction. Additional work was carried out to customize the SDCL steel bridge systems for accelerated bridge construction (Javidi, Yakel, and Azizinamini, 2014). Work included developing suitable details over the interior supports; conducting experimental, numerical and analytical studies; field application using the ABC philosophy of construction; and short- and long-term monitoring of an in-service SDCL bridge system constructed using ABC principles.

BACKGROUND

The latter half of the 20th century saw many changes in the design of bridges, one of the most significant of which resulted from the introduction of alternative construction materials. Prestressed concrete has become increasingly popular since its introduction in the 1950s (Dunker and Rabbat, 1992). The increase coincided with a decrease in the use of steel in short- to medium-span bridges. Several factors contributed to this trend, among them costly details such as bolted splices. The contemporary trend in steel bridge design and construction is to eliminate unnecessary details that contribute both to cost and decreased service life. Following a series of discussions with designers, steel fabricators and contractors in the late 1990s and early 2000s, it was concluded that the development of a steel bridge system for multi-span bridges, with each span having maximum lengths of about 150 ft, would benefit the bridge industry the most. Review of National Bridge Inventory (NBI) data indicates that about 90% of the bridges in service have a maximum single-span length of less than or equal to 150 ft (Lampe et al., 2014). Two factors were identified to help improve the economy of steel bridges: eliminating bolted splices and simplifying the construction sequences. It was concluded that what was needed was a system in which the girders would act as simple spans during deck casting and then act as continuous girders for live load.

The idea of using a simple-span girder for dead load and then making the girder continuous for live load was originally developed in the 1960s for precast, prestressed concrete girders to prevent leakage through the deck joints in simple beam spans (Freyermuth, 1969). However, there are some major differences between the application of this system to prestressed concrete girder bridges and to steel girder bridges.

Figure 1 shows a conventional two-span continuous steel bridge girder. The construction sequence consists of erecting the middle section and then connecting the two end sections using either bolted or welded field splices. This type of construction usually requires two cranes on site with possible traffic interruptions.

Another possibility is to place two simple-span girders between the abutments and pier, cast the deck slab and provide the continuity for live load and superimposed dead loads only (e.g., barriers and the future wearing surface). Figure 2 shows a schematic of such a system.

In the case of both prestressed and steel girder bridges, continuity for live and superimposed dead load is typically accomplished by placing reinforcing bars over the pier and casting concrete diaphragms over the pier. In both cases, the bottom portion of the concrete diaphragms in the vicinity of the girders is subjected to a compressive force transferred from adjacent girders. These compressive forces are generated by negative moments produced by traffic loads and superimposed dead loads. In the case of prestressed girders, the bottom flanges of the girders generally have large areas and are able to distribute the compressive force and prevent crushing of the concrete. However, in the case of steel girder bridges, the bottom flanges of the girder have smaller areas and a higher modulus of elasticity than the flanges of

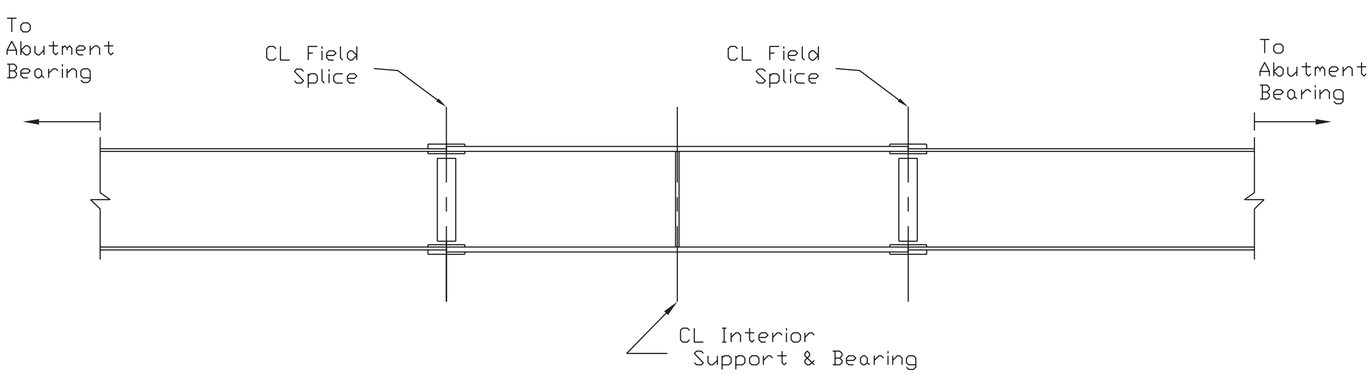


Fig. 1. Conventional two-span continuous steel bridge girder.

concrete girders. These two factors combined can result in crushing of the concrete in the diaphragm in the vicinity of the steel girder bottom flanges when the load is transferred.

In the initial stages of the development of the system, a series of preliminary finite element analyses was conducted (Lampe et al., 2014), which indicated that the level of these compressive stresses is large. For instance, for a two-span bridge with a span length of 100 ft and girder spacing of 10 ft, the resulting compressive stress in the bottom of the concrete diaphragm due to traffic loads could exceed the compressive strength of the concrete by a factor of 4 or higher (assuming 4000-psi concrete is placed in the diaphragms). Therefore, in the case of steel bridges utilizing the SDCL concept, there is a need to develop a detail that could eliminate the possibility of crushing of the concrete in the concrete diaphragm immediately adjacent to the bottom flanges of the girders.

OVERALL STRUCTURAL CHARACTERISTICS

Figures 3 and 4 compare the maximum positive and negative moments, respectively, for two-span steel bridges with equal length spans, analyzed assuming conventional and SDCL systems. In the conventional case, the girders were assumed continuous for both dead and live loads. In the SDCL system, the girders were analyzed as simple spans for the dead weight of the girders and wet concrete deck and as continuous spans for superimposed dead and live loads. Three two-span bridges were considered. The span lengths for these three bridges were 100, 120 and 150 ft. The girder spacing in all cases was 10 ft (Lampe et al., 2014). The solid line in each graph corresponds to the conventional case, whereas the dashed line corresponds to the SDCL system. As indicated in Figure 3, the maximum positive moment in the SDCL system is larger than in the conventional case. This is expected because the girders resist the noncomposite dead loads as a simple span. On the other hand, as shown in Figure 4, the maximum negative moment for the SDCL system is decreased. This behavior is a result of the fact that the noncomposite dead load does not produce any negative moment at the pier.

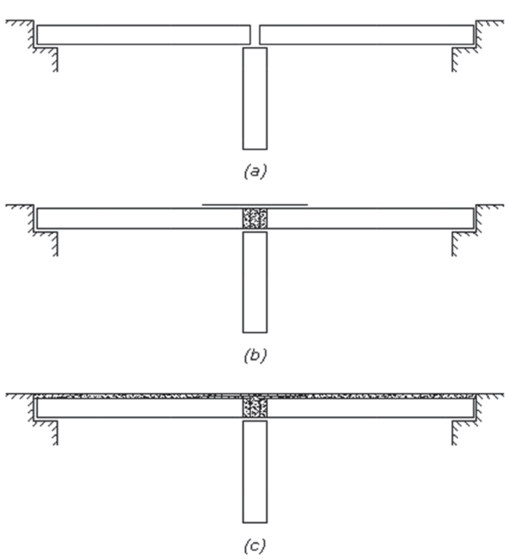


Fig. 2. Construction sequence for SDCL bridge systems.

RECOMMENDED CONNECTION DETAIL FOR CONVENTIONAL CONSTRUCTION METHODS

A possible connection detail to provide live load continuity over the pier in an SDCL steel girder system for the case of conventional methods of construction is to use a concrete diaphragm over the piers to connect the girders. Figure 5 shows the recommended connection detail over the pier. The following are the various elements of the connection detail:

- Concrete diaphragm, which should be cast at least a few days prior to casting the deck.
- Steel slab reinforcement to provide live load continuity. This reinforcement is placed before casting the deck.
- End plates welded to the ends of each girder.
- Steel blocks welded toward the bottom of each end plate.
- Elastomeric pads where the steel girders seat.
- A small gap is needed between the bottom of the steel flanges and the top surface of the cap beams. The gap should be filled with soft material, such as expanded polystyrene, to allow the rotation of the entire detail once the concrete diaphragm is cast and cured.
- Dowel reinforcement could also be placed to connect the cap beam to the concrete diaphragm in order to prevent longitudinal movement of the superstructure at the connection point.

The detail shown is applicable to both I- and box-girder bridges. Figures 6 and 7 show photos of the recommended detail used in I- and box-girder bridges, respectively. In both cases, a steel block is provided near the bottom flanges. In the case of the box girder, an additional steel plate is welded to the top flanges, as shown in Figure 7, to prevent pull-out of the tension flanges of the girder from the concrete diaphragm.

Figure 8 shows a portion of the recommended connection detail over the pier. Minimal steel reinforcement is needed within the concrete diaphragm. For the sake of clarity, not all of the reinforcement is shown in Figure 8. Figure 9 shows an example of the steel reinforcement that could be provided inside the concrete diaphragm.

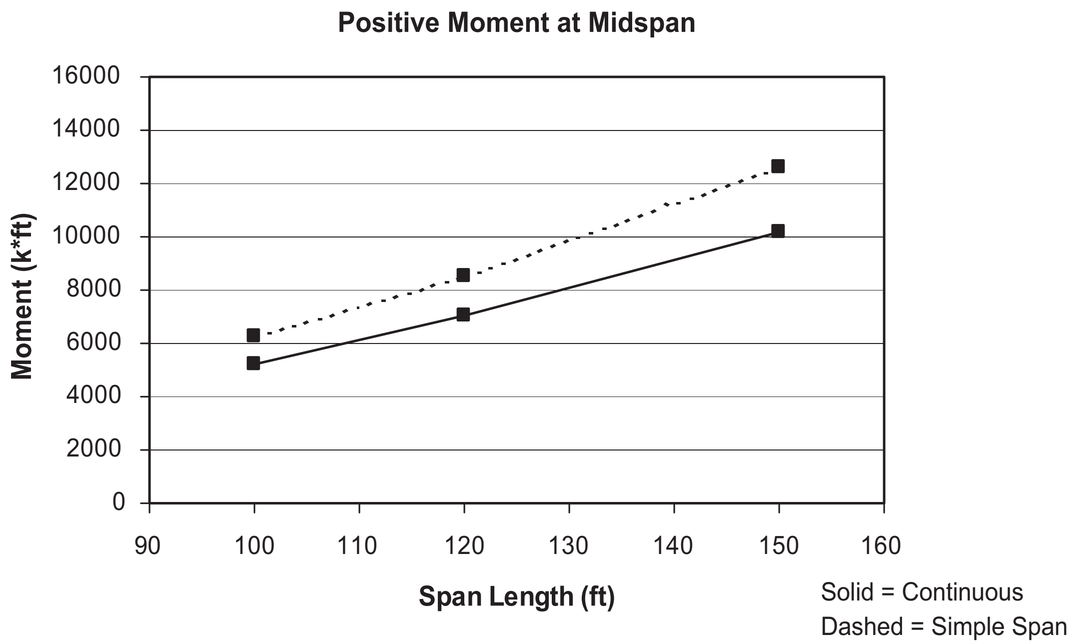


Fig. 3 Maximum positive moment comparison.

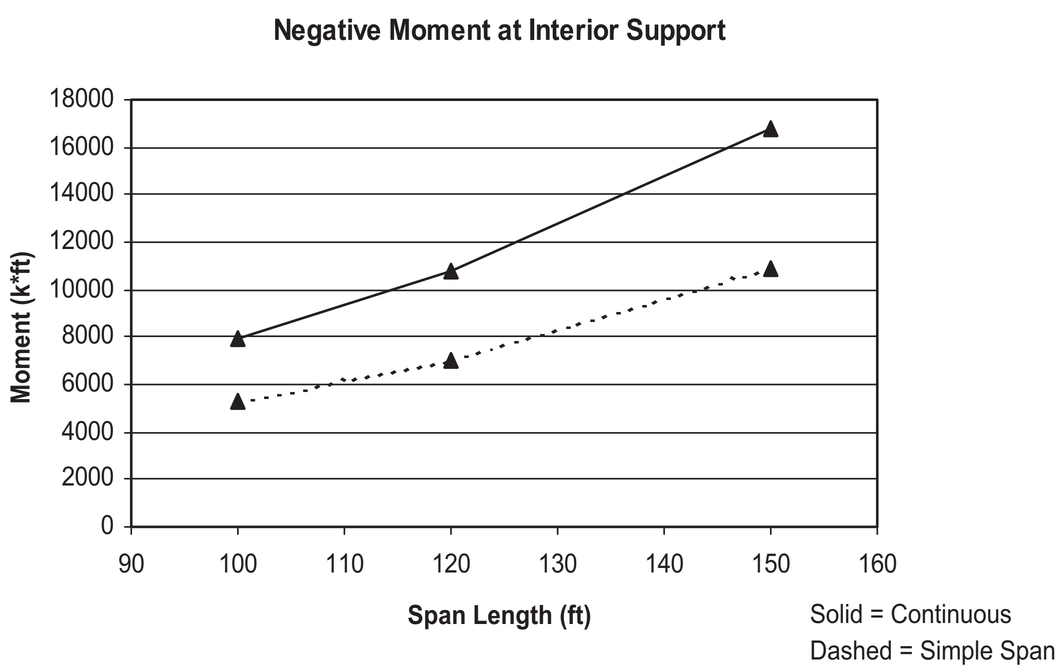


Fig. 4. Maximum negative moment comparison.

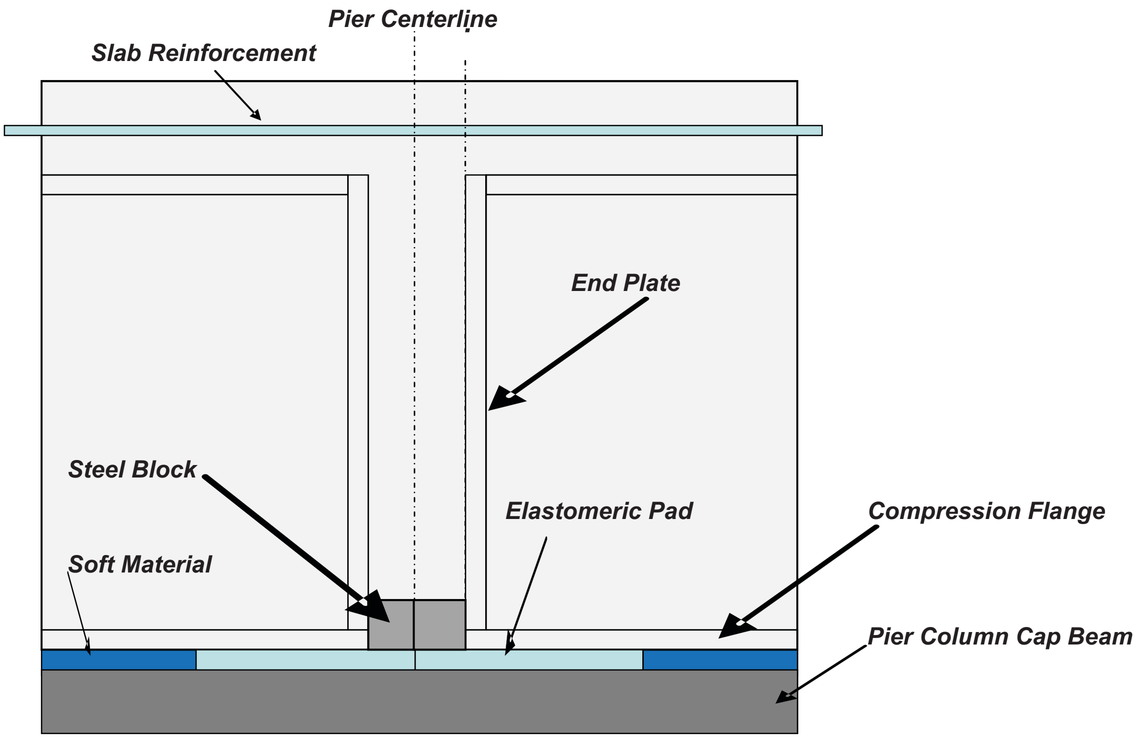


Fig. 5. Concrete diaphragm detail over the pier for conventional methods of construction.

Fig. 6. Recommended detail on an I girder.

Fig. 7. Recommended detail on a box girder.

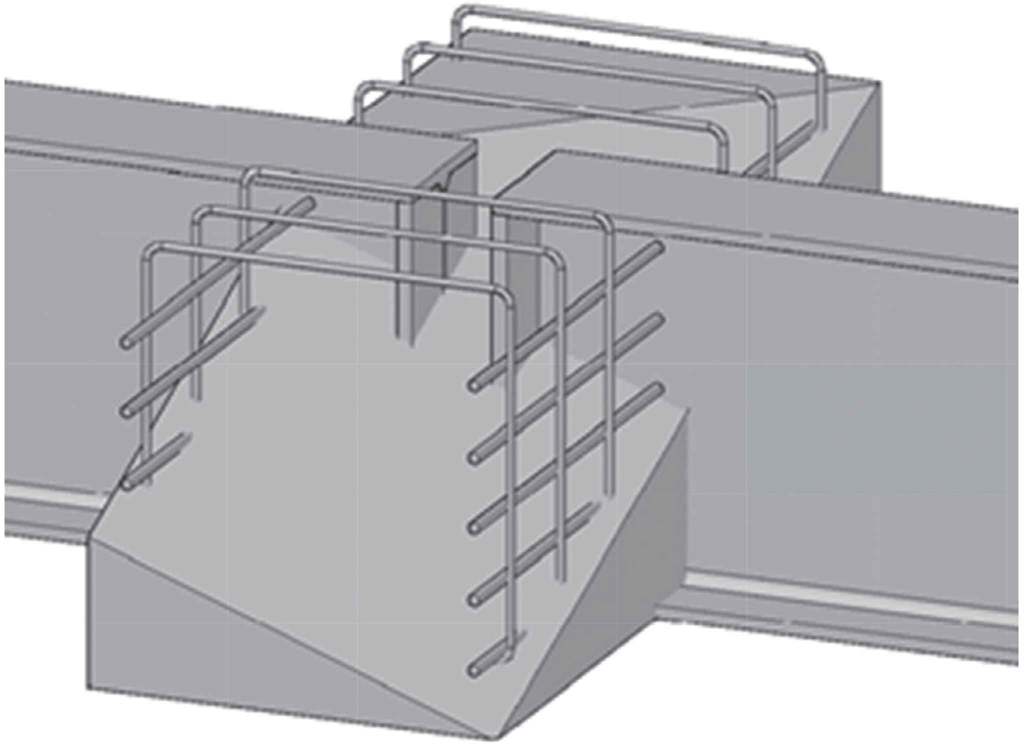


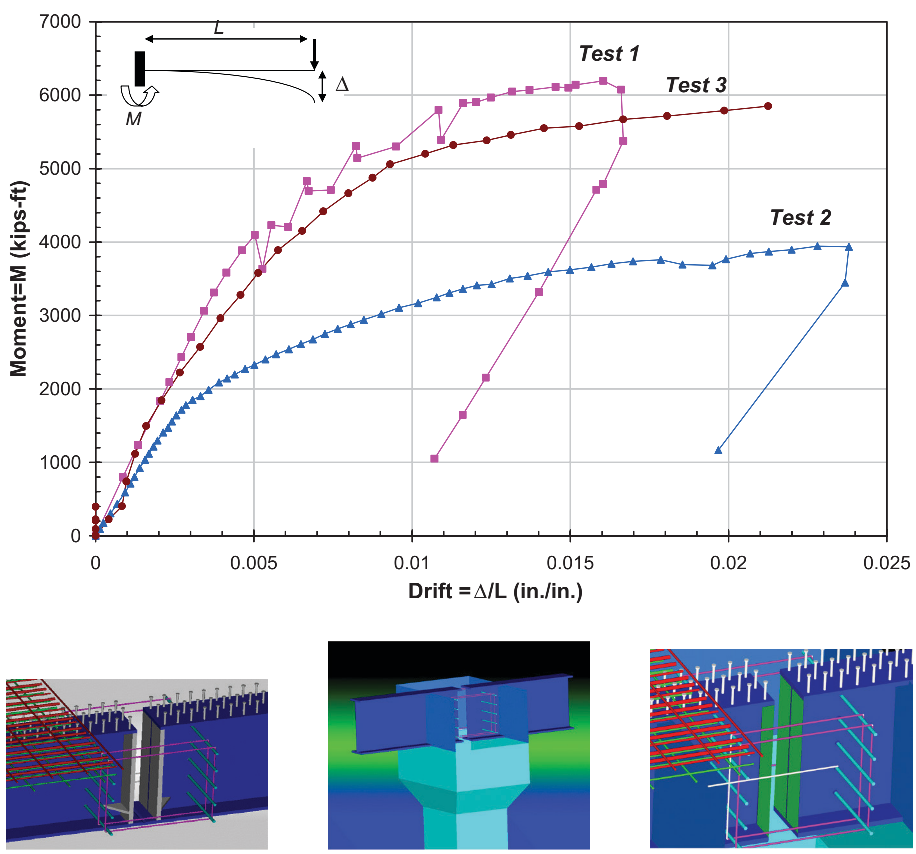
Fig. 8. A partial schematic of the steel reinforcement in the concrete diaphragm.

Fig. 9. Example of the required minimal reinforcement in the concrete diaphragm.

For the recommended connection detail shown in Figure 5, the bottom flanges will be subject to a compression force under traffic live loads. Where the number of spans exceeds two, bottom flanges could also be subject to a tensile force. However, the level of this tensile force is generally small. In cases where the tension force becomes significant, it might be necessary to positively attach the bottom portion of the connection shown in Figure 5 to enhance the tension capacity of the connection.

Research results show that for the recommended detail, the compressive force created by the live loads is capable of crushing the concrete in the vicinity of the compression flanges of the steel girder. To demonstrate this aspect of the connection detail, Figure 10 shows results of three full-scale tests. Each test utilized different connection details. Figure 10 shows the details used in each test along with a resulting moment versus deflection curve for each test specimen. The only difference among the three test specimens was the details used at the end of the steel girders. In test 2, the ends of the girder were simply embedded in the concrete diaphragm, which resulted in the lowest capacity. Welding end plates to the end of the girders, as was the case in test 3, resulted in a larger capacity. Connecting the bottom flanges and welding end plates to the ends of girders, as was the case for test 1, proved to achieve the best performance. Note that for clarity, curved segments associated with unloading and subsequent reloading of the specimen have been removed from the plots in Figure 10. This has resulted in the appearance of a jagged response, particularly for the case of test 1. The reasons for unloading varied, but were typically to deal with fixture and load system issues. Additional details can be found in the referenced research reports.

The main reason for the significant capacity reduction observed in the case of test 2, as compared to tests 1 and 3, is that the bottom flanges crushed the concrete adjacent to the bottom flanges and prevented the development of adequate compression capacity in the connection. With the moment capacity of the connection being equal to total compressive force times the moment arm (i.e., the distance between the resultant compressive force located near the bottom flanges and the resultant tensile force located near the tension reinforcement), a reduction in compression resistance near the



Test 1: Connection type 1

Test 2: Connection type 2

Test 3: Connection type 3

Fig. 10. Effect of end details on moment-deflection performance.

bottom flange translates into a lower moment capacity. Therefore, preventing the crushing of the concrete near the bottom flanges enhances the moment capacity of the connection.

The principal consideration in selecting the connection detail for the SDCL system when a concrete diaphragm is used is the transfer of the compressive force from the compression flange of one steel girder to the compression flange of the adjacent girder without going through the concrete diaphragm. An example of a practical solution used in several bridges is shown in Figure 11. As mentioned previously, in the case of multi-span bridges and in cases in which bottom flanges could be subjected to significant tensile forces, positively attaching the bottom portion of the recommended connection will be needed (see Figure 11).

Figure 11 shows the recommended detail for providing live load continuity over the pier before casting the diaphragm concrete. The recommended detail to transfer the compression force from one girder to the next without going through the concrete diaphragm is to weld the end plate to the end of each girder and then weld a steel block to the end plate near the bottom of the connection, as shown in Figures 5, 6, 7 and 11. These steel blocks are about 2 in. thick by 4 in. high and as wide as the width of the girder flanges. Although the steel blocks need to touch each other, they do not need to be perfectly in contact, as shown in Figure 11. The holes shown in the webs of the steel girders, visible in Figure 11, are provided to accommodate the steel reinforcement needed in the concrete diaphragm (see also Figures 8 and 9).

The use of concrete to connect the two girders provides several advantages. Based on research and field observations, it is recommended that about ½ to ⅔ of the depth of the concrete diaphragm be cast and cured for about seven days before casting the deck and the remainder of the diaphragm. Casting ½ to ⅔ of the depth of the diaphragm provides excellent torsional bracing for the ends of the girders, which is required with respect to flexural capacity and during casting of the deck. This eliminates the need to provide a steel diaphragm at each girder end simply for construction purposes. Indeed, the load stiffener visible in Figure 11 is technically unnecessary but is an artifact from conventional design requested to be left in by the owner. The use of concrete over the support encases the girder ends and protects them against corrosion, enhancing the service life of the girder ends.

DESIGN PROVISIONS FOR CONVENTIONAL CONSTRUCTION METHODS

A combination of experimental and numerical studies was carried out to better comprehend the contribution of the various elements of connection types 1, 2 and 3 shown in Figure 10. Connection type 2 (no end details) was used as a reference point and should be avoided. Research results indicated that connection type 3 could still experience crushing of the concrete in the vicinity of the compression flange and inside the concrete diaphragm. Classes of connection details resembling connection type 1 are recommended for use in practice. The main characteristics of connection type 1 are that the compression flanges of the girder inside the concrete diaphragm are in contact, and the possibility of crushing of the diaphragm concrete is eliminated. Table 1 shows the contribution of each connection element to the overall moment capacity of connection types 1 and 3 as determined by finite element analysis. The moment of each element was computed about the centerline of the bottom plate.

Every element within the concrete diaphragm contributes

Fig. 11. Typical live load continuity detail used in field application; overall view on the left, close-up view on the right.

Table 1. Contribution of Each Resisting Element to Overall Moment Capacity		
Resisting Element	**Connection Type 1**	**Connection Type 3**
Tension reinforcement in slab	60.82%	66.77%
Stirrups in tension	5.09%	5.42%
Concrete in tension	4.35%	6.03%
Stirrups in compression	0.00%	1.58%
Concrete in compression	12.37%	20.20%
Bottom plate in compression	17.37%	NA
Total	100.00%	100.00%

to the overall moment capacity of the connection; however, the contribution of some elements could conservatively be ignored. It is recommended to use connection types that would resemble type 1, which is the focus of the following discussion. The design procedure for connection type 3 is more complex and is provided in Farimani et al. (2014). It is not recommended to use connection type 3 in practice since some level of crushing of the concrete adjacent to the compression flange was observed in the experimental tests.

Closer examination of the information provided in Table 1 indicates that the flexural capacity for connection type 1 is predominantly (approximately 77%) provided by a couple created by the tension reinforcement placed in the deck and the compression steel block located at bottom of the connection. Based on the research findings, it is suggested that simplified design provisions be followed for calculating the flexural capacity of connection type 1. The same procedure can also be used to calculate the area of steel reinforcement needed in the slab to provide live load continuity.

As mentioned previously, based on the values shown in Table 1, the effect of the stirrups and diaphragm concrete can be ignored when calculating the moment capacity of connection type 1. This leaves only the bottom compression block and slab tension reinforcement to provide the moment-resisting capacity. The capacity is then simply the tension capacity of the slab reinforcement multiplied by the moment arm. Because the bottom compression block will be sized so that it will not yield, the resulting moment arm is equal to the distance between the slab tension reinforcement and the center of the bottom compression block. The steel blocks are in contact (bearing) and can resist stresses well beyond the yield stress.

Research has shown that failure of connection detail type 1 coincides with the slab tension reinforcement reaching its ultimate capacity. Research has also shown that all tension reinforcement within the effective width will yield. The resulting equation for calculating the capacity of the connection is therefore:

$$M_n = A_s f_y (d - H/2) \tag{1}$$

where

M_n = nominal moment capacity of the connection, in.-kip
A_s = area of the slab tension reinforcement, in.^{2}
f_y = yield stress of the slab tension reinforcement, ksi
d = distance between the bottom of the girder and the centroid of the slab tension reinforcement, in.
H = height of the steel block, parallel to the girder depth, in.
t_{pl} = thickness of the steel block, parallel to the girder length, in.

The use of the recommended design procedures is demonstrated using a two-span bridge consisting of two 97-ft spans with the following design information:

Required live load moment capacity, M_u(LL)	34,770 in.-kip
Steel beam	W40×249
Depth of the steel beam	43.375 in.
Width of compression flange, b_f	15.75 in.
Bottom of girder to centroid of rebar, d	47.75 in.
Effective width of concrete slab, b_e	92 in.
Slab thickness	8.5 in.
Strength reduction factor, ϕ	0.9

The nomenclature used to describe the size of the steel blocks is as follows:

- The dimension of the steel block parallel to the depth of the girder will be referred to as the height of the steel block and is designated as H.
- The dimension of the steel block parallel to the width of the girder flange will be referred to as the steel block width and is designated as b_f.
- The dimension of the steel block parallel to the length of the girder is referred to as t_{pl}. This dimension could vary and it is suggested to be at least 2 in.

To begin, a steel block with a height of 2 in. will be assumed. After calculating the required area of the slab tension reinforcement, the adequacy of this block will be checked. To calculate the required area of the reinforcement, Equation 1 can be set equal the required strength and solved for A_s:

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

Using Equation 2, the required area of slab tension reinforcement is calculated as shown:

$$\begin{aligned} A_s &= \frac{34{,}770}{0.9(60)(47.75 - 2/2)} \\ &= 13.8 \text{ in.}^2 \end{aligned}$$

In order to ensure that the bottom compression block remains elastic when the ultimate capacity of the connection is reached, the required minimum height of the steel block can be found using Equation 3. Note that the ultimate strength of the tension reinforcement is used. Based on the results of the research, the ultimate strength is assumed to be 1.7 times the yield strength.

$$H_{\min} = \frac{1.7 A_s f_y}{b_f F_{ypl}} \tag{3}$$

where

H_{min} = minimum height of the bottom compression block, parallel to the depth of the girder, in.
F_{ypl} = yield strength of the steel block, ksi
b_f = width of the girder flange, in.

The width of the block is assumed to be equivalent to the width of the girder bottom flange and the area of reinforcement is assumed to be equivalent to the required area calculated with Equation 1. If the actual area of reinforcement is larger based on the arrangement of bars selected, the thickness of the block should be re-evaluated.

$$\begin{aligned} H_{\min} &= \frac{1.7(13.8)(60)}{15.75(50)} \\ &= 1.79 \text{ in.} < 2 \text{ in.} \\ &\therefore \text{ assumed block height acceptable} \end{aligned}$$

Therefore, using steel blocks with dimensions of 2 in. by 2 in. by 15.75 in. is adequate.

FABRICATION AND CONSTRUCTION RECOMMENDATIONS FOR CONVENTIONAL CONSTRUCTION METHODS

For the recommended connection detail shown in Figure 11, test results indicated that minimum size fillet welds could be used to attach the end plates to the girder ends. In addition, the attachment between the steel blocks and the steel end plate could be achieved using minimum size fillet welds. As shown in Figure 11, the steel blocks need to touch each other; however, they do not need to be in full contact. During construction, the longitudinal movement of the girders could create a gap between the steel blocks due to thermal loads. This should not be a concern, however, because concrete paste can fill the space between the steel blocks and create a smooth load transfer path. This recommendation is based on observed test results and field experiences.

One of the critical steps during the construction stage is the casting of the concrete diaphragm prior to the casting of the deck. This involves two opposing requirements. First, the ends of the girders must be prevented from twisting during the casting of the deck to ensure stability of the girders. This is a requirement for using the flexural design provisions of the *LRFD Bridge Design Specifications* (AASHTO, 2012). In this regard, casting the full depth of the concrete diaphragm would be ideal. However, field measurements indicate that full-depth casting of the concrete diaphragm can provide about 50% moment continuity during deck casting. This unintended continuity can result in cracking of the concrete diaphragm during deck casting. This behavior was observed in the construction of Highway N-2 over the I-80 bridge in Nebraska (Yakel and Azizinamini, 2014) for which the concrete diaphragm was completely cast and cured before casting the deck. After removing the formwork, cracks were observed on the face of the concrete diaphragm, as shown in Figure 12. Formation of cracks in the concrete diaphragm resulted in a reduction in the level of continuity moment. The cracks were later filled with epoxy. It should be noted that these cracks did not pose any safety hazard or potential for failure—they can be repaired with epoxy filling—rather, the issue was more of an aesthetic problem and possibly a service-life issue. Nevertheless, departments of transportation generally do not favor the formation of such cracks.

Field experience indicates that the level of unintended continuity is significantly reduced if the concrete diaphragm is cast to about ½ to ⅔ of its full depth before placing a thin steel sheet between the end plates running across the entire width of the concrete diaphragm. Figure 13 shows the suggested detail that was used in construction of a bridge using an SDCL steel bridge system in Montana.

The thin cold-formed steel sheet shown in Figure 13 should be extended to near the bottom flange of the girders and secured while casting the partial-depth concrete diaphragm. Other details or approaches may also be feasible to reduce this unintended continuity. The main objective is to secure the ends of the girder in concrete and provide good torsional bracing while, to the extent possible, allowing the girder ends to rotate when casting the deck without subjecting the concrete diaphragm to tensile forces. The incorporation of cold-formed thin steel sheet, as shown in Figure 13,

Fig. 12. An example of cracking in the concrete diaphragm if it is cast full depth.

Fig. 13. Recommended solution for reducing unintended continuity over the pier.

also controls the direction of cracking, if any, in the concrete diaphragm while casting the deck. It is recommended that about 25% dead load continuity should be assumed, despite precautionary measures that may be taken to avoid the development of unintended dead load continuity during the design process.

Although this discussion focused on I-shaped steel girders, the same conclusions are applicable to box-shaped girders.

DESIGN AND CONSTRUCTION ADVANTAGES USING CONVENTIONAL CONSTRUCTION METHODS

SDCL steel bridge systems present several advantages when used in conjunction with conventional construction methods, including:

1. Eliminating the need for bolted splices.
2. Allowing the use of the same cross-section throughout the girder length. The positive moment along the girder length is increased slightly, while the negative moment in the same span near the interior supports is decreased. Investigated cases that included span lengths of 90 to 150 ft, which are typically found in simple-grade crossings, have shown that the resulting moments allow the use of the same cross-section throughout the girder length. This observation favors the use of rolled beams where possible.
3. Reducing negative moment near the interior pier. This allows for increasing the spacing between the interior pier and first cross frame, which can in turn reduce the number of cross frames. Casting the ends of the girder in concrete creates a boundary condition that is fixed against torsion, which can allow a further increase in the distance between the interior support and first cross frame. However, this additional fixity is usually ignored in routine design.
4. Providing a stable configuration during construction. The recommended detail and construction procedure, as described earlier, simplifies the bridge details while producing a stable configuration during deck casting. This is achieved by rigidly securing the ends of the girder in the partially cast concrete diaphragm.
5. Allowing the sequence of casting for multi-span bridges to be ignored since the deck may be cast from one end of the bridge to the other end in a single cast. This advantage is a reflection of the fact that each span acts as a simply supported span when casting the deck; the number of spans is not a design consideration.
6. Providing protection of the girder ends against possible corrosion by using the recommended detail over the pier, which enhances the service life of the bridge. Eliminating the bolted splices also enhances the service life by reducing the risk of corrosion.

DESIGN, FABRICATION AND CONSTRUCTION RECOMMENDATIONS FOR ACCELERATED CONSTRUCTION

Accelerated Bridge Construction (ABC) is a response to public demand for avoiding traffic interruption. The SDCL steel bridge system provides an excellent opportunity for accelerating the construction process. This objective can be achieved in several ways. One approach is to use the adjacent girder concept as described later. The concept can be implemented using either I-shaped girders or box girders. However, the following discussion focuses on using steel box girders in applying the SDCL system to ABC. The application of the SDCL system to ABC is further explained using as an example the bridge over I-80 at 262nd Street in Nebraska.

The adjacent girder concept utilizes prefabricated units consisting of an individual steel box girder pre-topped by a portion of deck slab, as shown in Figure 14. In this discussion, each steel section with a pre-topped deck will be

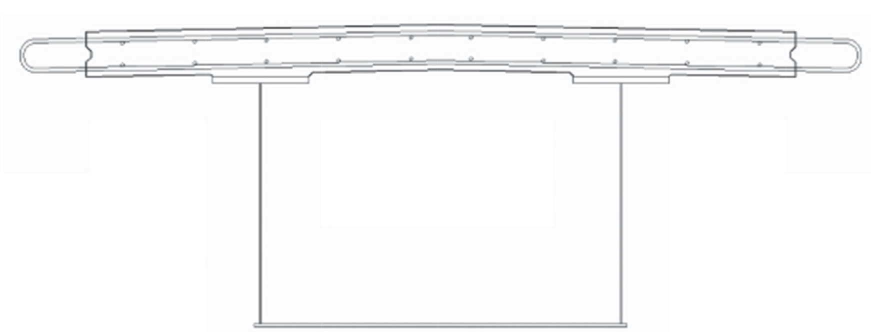


Fig. 14. Single (interior) box girder and deck unit.

referred to as a unit. With this approach, the units are prefabricated and then shipped to the job site. The portion of the deck shown in Figure 14 is cast at the fabrication shop or temporary staging location. Once on site, the individual units are set into place on the supports adjacent to one another (see Figure 15). A longitudinal deck closure strip between the individual units is then cast, joining them together. At the same time, the concrete diaphragm over the interior pier is cast. The interior concrete diaphragm connects the spans and provides continuity between them for subsequent loading (e.g., live load). Step-by-step details of the procedure are described as follows.

Construction Sequence

Step 1. Steel Box-Girder Fabrication. The first step is to fabricate the steel box girders. One option is to have webs perpendicular to the flanges. This allows fabricators to use standard I-girder techniques and jigs. Often, webs of box girders are sloped, which can be costly to fabricate and can require special modifications to equipment.

Step 2. Cast the Deck onto the Girders. This operation can be performed in a precast concrete facility or some other temporary location. Two available options for formwork are conventional form jacks or a shored deck bed. The chosen system depends on fabricator preference and the nature of the job.

Figure 16 shows the fabrication of pre-topped units for the bridge over I-80 at 262nd Street. A primary advantage of ABC is that most of the construction activities can be accomplished off site, away from traffic zones, eliminating the risk associated with exposing workers to work zone hazards.

Given the inherent stability of the steel box section, conventional form jacks can be used for the deck forming. The steel girder is placed on temporary supports, and construction is carried out using conventional methods. Depending on the fabricator's preference, the segments can be constructed individually or all at once.

Other options include placing the steel girder on a flat bed and fully supporting the unit while casting the deck.

Step 3. Placement of the Units on Supports. After casting the deck, the individual pre-topped units can be picked up using regular cranes and placed on the supports, as shown in Figure 17. The main advantage of steel box girders is their significantly lower weight compared to concrete box girders, especially for longer span lengths. Use of lightweight concrete can further reduce the total weight of the pre-topped units if necessary.

Figure 18 shows the photo of the bridge over I-80 at 262nd Street after placement of all the pre-topped units side by side.

Another major advantage of using steel box girders in the adjacent girder system is the ability to match the elevation of different units and achieve the overall geometry. In the case of concrete girders, creep and shrinkage deflections cause different pre-topped units to experience different displacements, and it is almost impossible to match the elevation of adjacent units. Further, in the case of prestressed concrete girders, attaching them over the pier for live load continuity can result in cracking in the concrete diaphragm because of creep and shrinkage-related deformation of the girders, which is not an issue for steel girders. These issues can result in serious field challenges and job delays, especially for longer span lengths.

Step 4. Detail for the Interior Supports and Longitudinal Closure Pour. The detail recommended for ABC application of SDCL steel bridge systems is shown in Figure 19 and is similar to that used for conventional methods of construction.

The longitudinal reinforcement extending from the pre-topped deck can be developed over shorter distances by hooking the bars inside the concrete diaphragm. This particular detail was investigated extensively—including a full-scale test—by Javidi, Yakel, and Azizinamini (2014) and was found to provide good performance. It is recommended that development of the extended longitudinal reinforcement using straight splices be avoided. Developing the longitudinal reinforcement extending from pre-topped girders is not recommended, unless the bars are terminated in the region of the deck subjected predominantly to compression. Ending tension reinforcement in a tension zone will create stress concentrations and develop cracking in the deck, adversely affecting the service life of the bridge deck.

The detail to be used for longitudinal closure pour is a critical aspect of using pre-topped adjacent girder systems.

Fig. 15. Three adjacent box-girder units.

Figure 20 shows the longitudinal closure pour detail used for the 262nd Street Bridge, which utilized headed bars.

Ultra-high-performance concrete (UHPC) provides better alternatives for longitudinal closure pours. In the case of the bridge over I-80 at 262nd Street, the width of the closure pour was 13 in. The headed bars that were used allowed for development of the deck transverse reinforcement over a shorter length. Using UHPC, the width of the closure pour region could further be reduced to 8 in., eliminating the need for the headed bars. The shortcoming is that very tight tolerances need to be specified, which could create challenges in the field. From the standpoint of durability and service life, the width of closure pours needs to be as small as practical. However, from the standpoint of tolerance, the width needs to be as wide as possible. It is recommended to use a narrow width (about 8 in.) and take measures to ensure good quality control during construction.

The fresh concrete that is placed in the closure pour will be restrained by concrete that is already hardened on the either side of the closure pour region (i.e., the pre-topped deck over the girders). This restraint will result in the development of tensile stresses in both the longitudinal and transverse directions in the concrete that is placed in the closure pour. These tensile stresses can easily exceed the modulus of rupture of the concrete and result in the development of longitudinal and transverse cracking at close intervals along the closure pour region. Field observations indicate that these transverse cracks can be 1 or 2 ft apart. The narrower the closure pour region, the lower the severity of these transverse and longitudinal cracks. The restraining phenomenon

Fig. 16. Preparing the formwork for casting the deck on the unit.

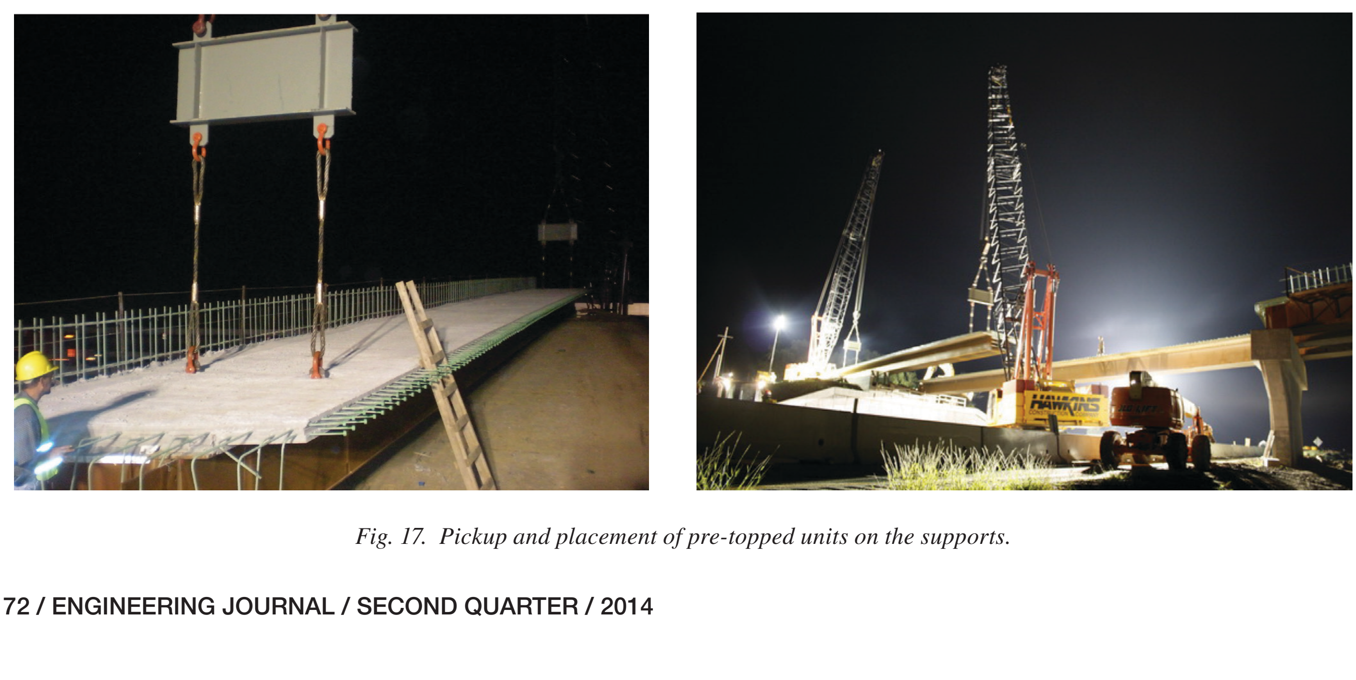


Fig. 17. Pickup and placement of pre-topped units on the supports.

described earlier is very similar to the behavior of a closure pour in phase-constructed bridges. A detailed description of the behavior of closure pours in phased constructed bridges and recommended design provisions are provided elsewhere (Azizinamini, Yakel and Swendroski, 2003).

In the case of the 262nd Street Bridge over I-80, a partial-depth pre-topped deck was used necessitating placement of an overlay as the last step of the construction. The use of an overlay allows the addressing of minor adjustments that might be needed during construction.

OTHER DETAILS USED FOR IN-SERVICE BRIDGES

Several states have used SDCL steel bridge systems (Talbot, 2005; Stone, Lindt and Chen, 2011; Morales, 2004; Wasserman, 2005) and some have used modified versions of the recommended details discussed herein. In the sections that follow, these details are presented along with brief discussions of their effectiveness based on research and field observations.

Fig. 18. Completing the placement of the pre-topped units for the bridge over I-80 at 262nd Street in Nebraska.

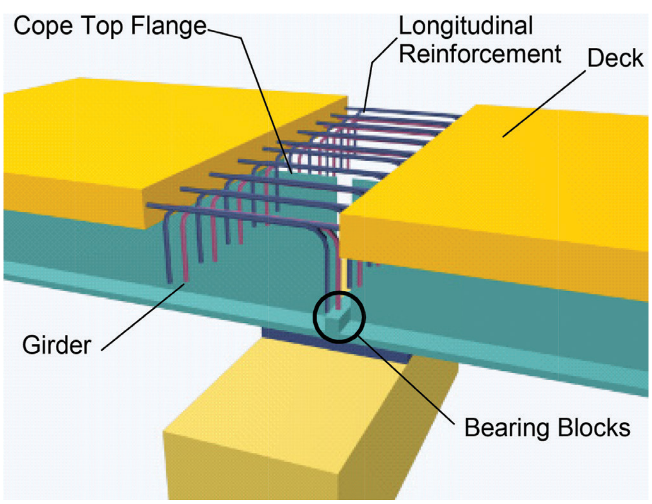


Fig. 19. Recommended interior support detail for an SDCL system used in an ABC application.

The Ohio DOT used the detail shown in Figure 21 in the construction of State Route 22 in Circleville, Ohio (Morales, 2004). The entire connection detail shown was cast in concrete forming the concrete diaphragm. The holes shown in Figure 21 are provided to pass through the transverse reinforcement that extends along the entire width of the concrete diaphragm.

The major difference between this detail and the detail recommended in this study is the gap that is left between the bottom (compression) flanges bearing directly on the concrete diaphragm. Based on the research results presented, this detail—which is similar to the type 3 detail described earlier—can result in crushing of the concrete in the vicinity of the compression flange. In construction of the bridge, the diaphragm and deck were cast simultaneously. This would require providing a steel diaphragm at each end of the girder to act as torsional bracing. It is not clear whether this was provided or not.

The Tennessee DOT has used two different details with an SDCL steel bridge system (Wasserman, 2005). For construction of the State Route 35 Bridge in Maryville, Tennessee, the detail used was very similar to the recommended detail in this paper. This detail is shown in Figure 22.

The detail shown in Figure 22 is similar to connection type 3 described in this paper, except that the girders are anchored to the cap beam using anchor rods. The behavior of this type of detail was investigated by Farimani et al. (2014). However, this detail is not recommended for field application because there was evidence of crushing of the concrete adjacent to the compression flange. A positive aspect of this detail is the attachment of the compression flange to the cap beam using anchor rods, which can be beneficial if moment reversal is a possibility and the bottom flange is placed in tension. However, during construction, the longitudinal movement of the girders due to thermal loading before casting the concrete diaphragm should be evaluated. Restraining both ends of the girder against longitudinal movement before casting the diaphragm can result in failure of the anchor rods. As noted in Figure 22, half of the concrete diaphragm was first cast and cured to secure the girder ends before casting the deck and the rest of the diaphragm. As described earlier, this is an excellent approach for securing the end of the girders against twist and eliminating the need for a steel diaphragm at the girder ends. As recommended previously, somewhere between ½ to ⅔ of the concrete diaphragm should be cast first to provide torsional bracing at the girder ends, and to reduce any unintended moment continuity for dead load.

The second class of details used in Tennessee in conjunction with SDCL steel bridge systems is shown in Figure 23.

This detail was used in the construction of the DuPont Access Road Bridge in New Johnsonville, Tennessee. The major variation from the recommended detail in this case is the cover plate that is provided to connect the top flanges before pouring the concrete diaphragm and deck; thereby providing the continuity for the deck dead load. Bolts connecting the top flanges of the girders are not shown in Figure 23. Such a practice could help reduce the camber that is needed and the deflection of the girder due to the dead weight of the wet concrete.

A variation of the detail shown in Figure 23 was used earlier in New Mexico in the construction of the Las Cruces and Hatch Bridges (Morales, 2004), with the exception that the bolts in the top cover plates were tightened after pouring a portion of the deck in order to provide continuity for the live load only. Such a practice could potentially result in construction complexities. Tightening of the cover plate bolts after deck casting has started could be challenging in

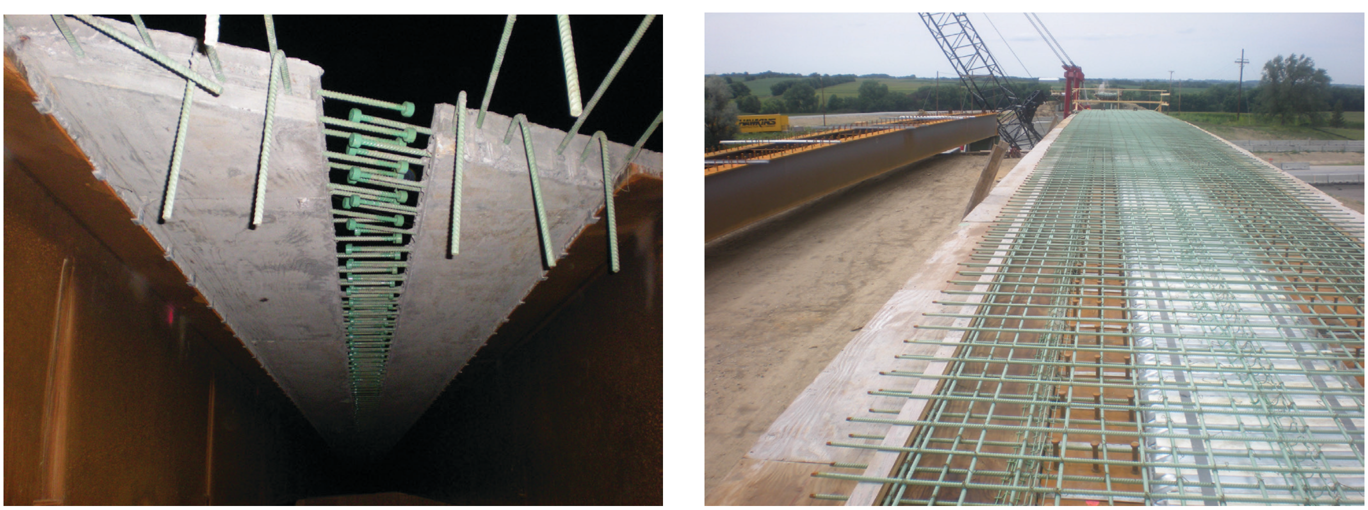


Fig. 20. Possible detail for longitudinal closure pour.

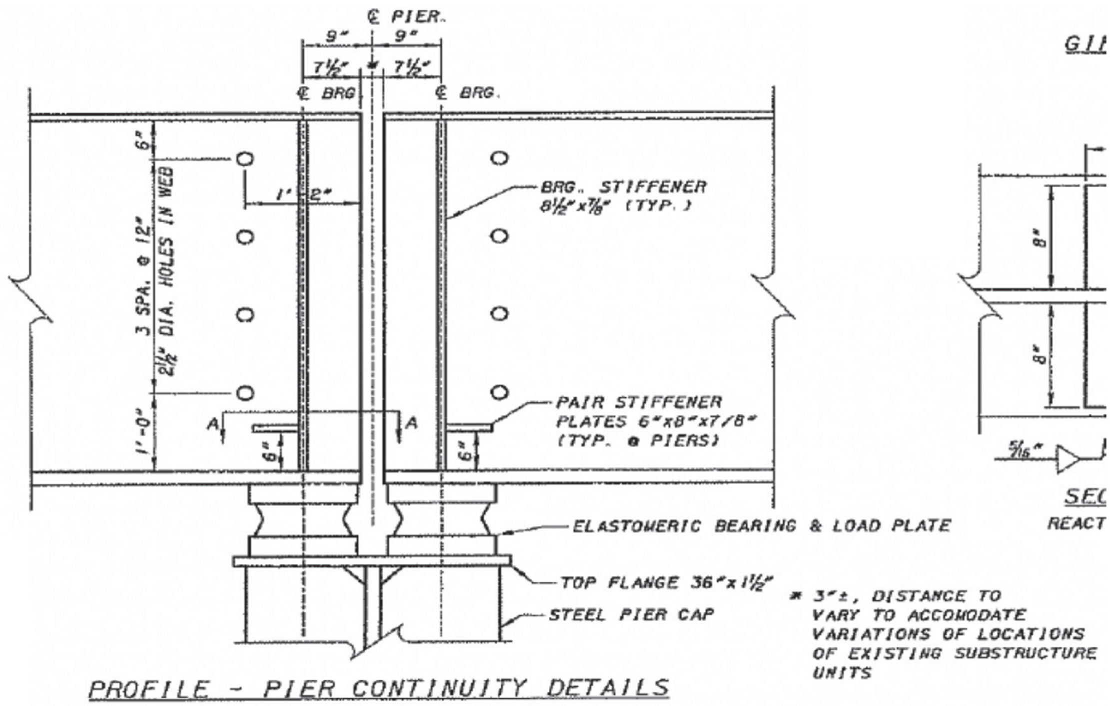


Fig. 21. Detail used by the Ohio DOT.

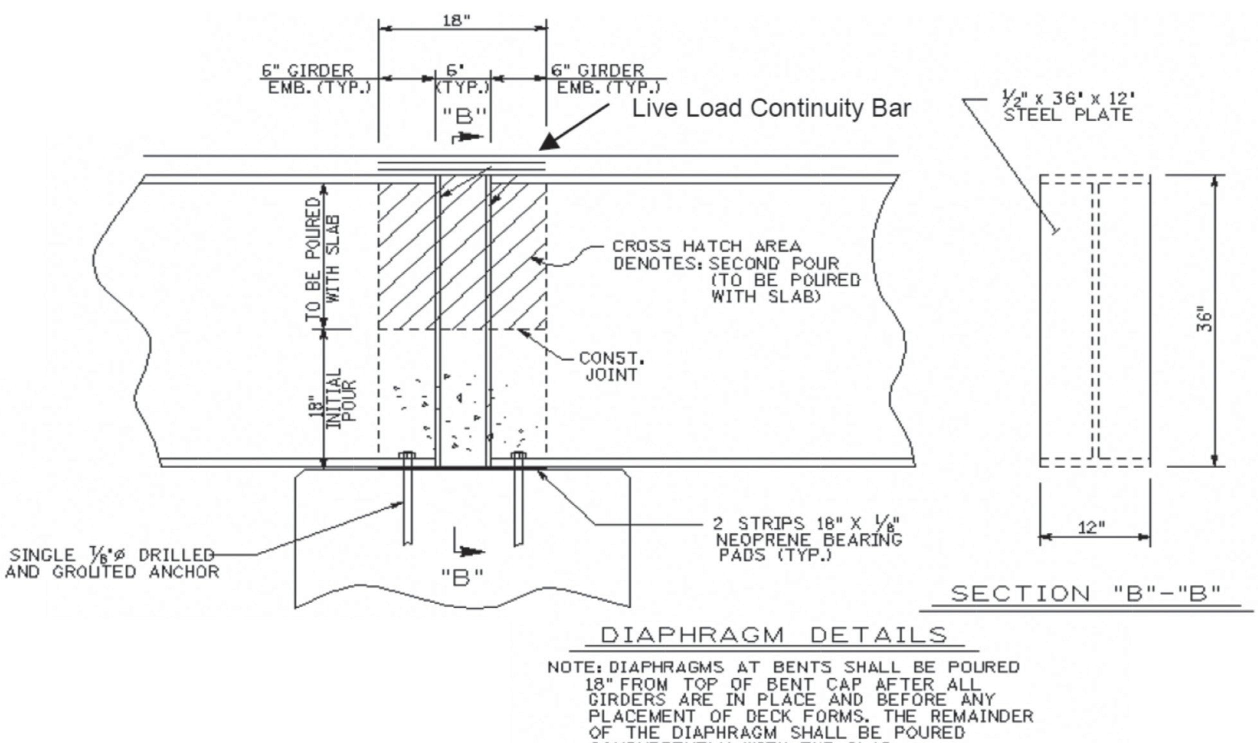


Fig. 22. Detail used in construction of the State Route 35 Bridge in Maryville, Tennessee.

the field. Research results indicate that there is no need for a top cover plate, and field experience indicates that contractors do not favor this detail.

Another detail that was used on a SDCL steel bridge system is a detail used by the Colorado DOT (Stone, Lindt and Chen, 2011) and illustrated in Figure 24. This detail was used in the construction of a six-span, 470-ft long bridge on U.S. Route 36 near Denver, Colorado. A concrete diaphragm was not used in this detail. The girders were seated on an elastomeric bearing pad along with a 30 in. by 14 in. by 1 in. steel plate. The bottom flanges of the girders were welded to the plate to provide compression-flange continuity. Heavy cross frames were provided (W27×84 sections) for the girders (W33×152 sections) near each girder end and over the pier to provide torsional bracing for the girder ends during deck casting. Webs of the two girders over the pier were not connected, which raises a question about the type of connection that was developed. To develop a full-moment connection, flanges and webs need to be spliced and connected. The type of detail used by the Colorado DOT falls in the category of a semi-rigid-type connection and would need experimental verification to assess its rotational stiffness for analysis purposes. Simply assuming that this detail is rigid and the girders are continuous for the purpose of live load analysis is not correct. Further, the transverse weld attaching the bottom flanges of the girders to the 30 in. by 14 in. by 1 in. plate is a Category E or E′ detail for fatigue. The deck detail used over the pier could also develop cracks during its service life and result in moisture leaking over the girder ends. Further, having too many unprotected steel elements over the pier, created by the presence of a heavy steel diaphragm, can further lower the service life and durability of the bridge. One of the major benefits of casting a concrete diaphragm over the girder ends at the pier is enhancing the service life of the bridge by eliminating joints.

In summary, variations of the recommended details in this study have been used by other DOTs for bridges that are in service. From the experiences gained from this study and the use of SDCL steel bridge systems in several states, the following conclusions may be drawn with respect to the characteristics that a preferred detail should possess for joining girders over interior piers in SDCL steel bridge systems:

1. There is a need to provide a continuous load path for transferring the compression force from one bottom flange to the adjacent bottom flange, without crushing the concrete in the diaphragm.
2. The detail to be used should be selected with consideration given to service life and durability. Casting the ends of the girder in concrete can protect the end of the girder against corrosion and eliminate joints over the pier.
3. Details that could potentially create construction challenges should be avoided. Otherwise, the simplicity of the SDCL system will be compromised. Details such as a cover plate for attaching the top flanges of the girder should be avoided. Research results indicate that there is no need to provide a cover plate for continuity

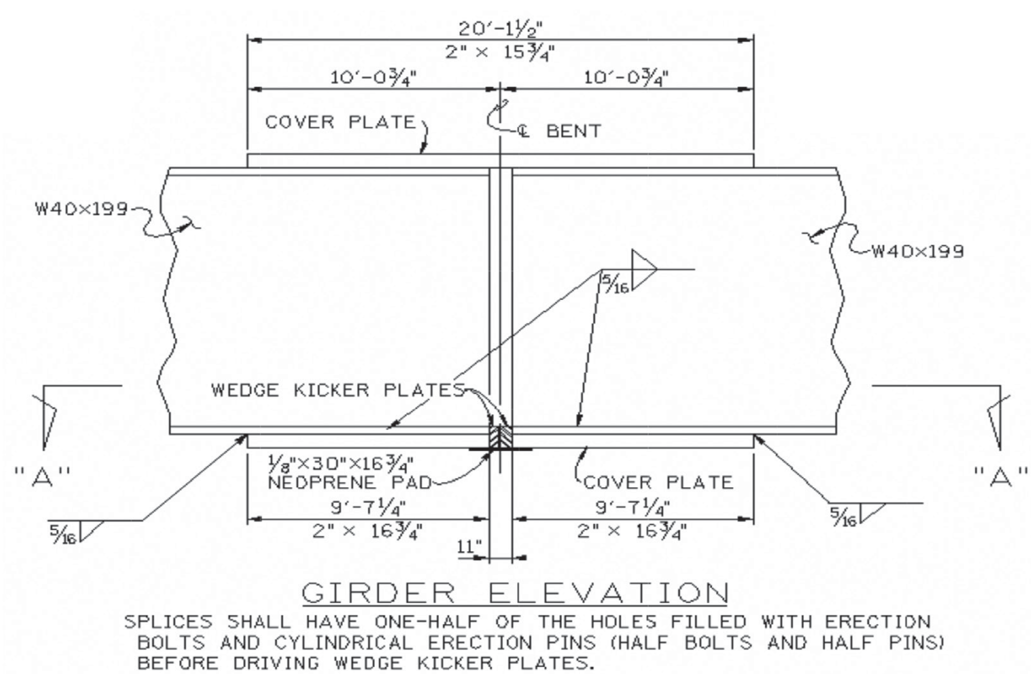


Fig. 23. Second detail for an SDCL steel bridge system, used for the DuPont Access Road Bridge in New Johnsonville, Tennessee.

or any other reason. Prestressed concrete bridges have used SDCL systems for years without any need for connecting the top flanges for continuity or any other reasons.

4. The ends of the girders during deck casting need to be torsionally braced. An effective way to provide this is to partially cast the ends of the girder in concrete, as described earlier.
5. For analysis purposes, the type of connection used over the pier needs to be identifiable with respect to rigidity. Connection details that are chosen should be classified as rigid, providing a full-moment connection to reduce efforts in the analysis stage.
6. Low-category fatigue details should be avoided.

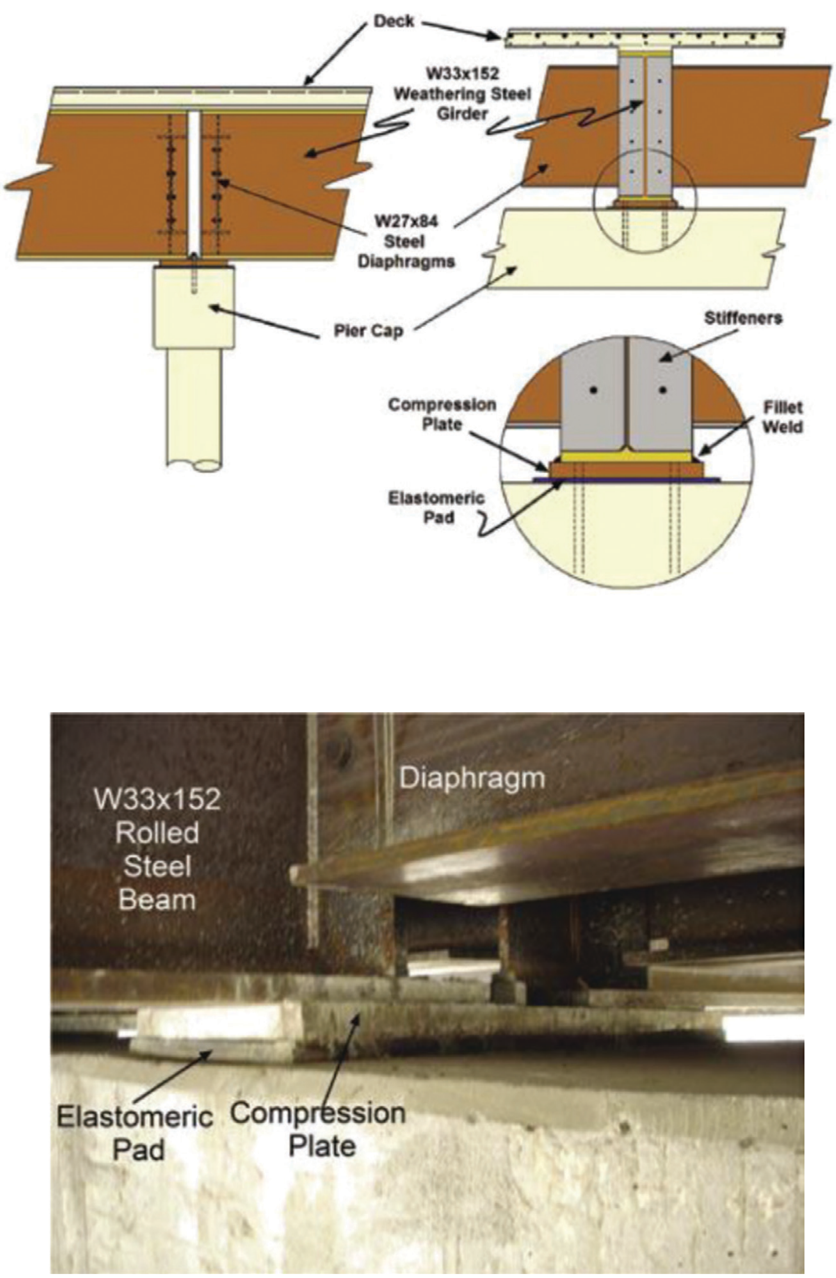


Fig. 24. Detail used by the Colorado DOT for a bridge located on U.S. Route 36 near Denver, Colorado.

SEISMIC APPLICATION

The recommended detail combined with additional research could provide an economical steel bridge system alternative in highly seismic areas; however, to date, no research studies have been carried out to extend the application of SDCL system to these areas. The following is a brief discussion of the issues involved and recommendations for further research, which could allow application of the recommended detail to highly seismic areas. Limited efforts by the author of this paper are under way to expand application of SDCL steel bridge systems to high seismic areas.

Steel bridges are lighter (about 40%) than concrete alternates and can provide better performance in a major earthquake. However, observations from a past earthquake (Astaneh-Asl et al., 1994) demonstrate that use of incorrect details or systems could result in steel bridges sustaining major damage. In the case of the 1995 Hyogoken-Nanbu earthquake in Kobe, Japan (Bruneau, Wilson, and Tremblay, 1996; Chung 1996; Shinozuka et al., 1995; Azizinamini and Ghosh, 1997), steel bridges suffered damage to superstructure elements (e.g., inadequate cross-frame detailing led to lateral bending of the girder webs near the girder ends), resulting in major retrofit activities and the closing of major highways, such as the Hanshin Expressway, for more than a year. The Kobe experience demonstrated that even relatively minor damage to steel bridges in seismic events can result in types of damage that could be very difficult to repair. Among the lessons learned is that critical elements of the bridge that are difficult to inspect and repair must be protected from any level of damage and remain elastic during the entire seismic excitation.

Seismic input is largely unknown; therefore, the design philosophy for buildings and bridges is to work on behavior of the structure under known conditions. Specifically, the design objective is to predefine the damage locations and design them accordingly by providing adequate levels of ductility. In the case of bridges, the preferred damage locations are at the ends of pier columns (formation of plastic hinges). In the direction of traffic, it is preferred to put columns in double curvature as shown in Figure 25. This allows larger portions of the pier column (two plastic hinges versus one for single curvature) to participate in energy dissipation.

Under longitudinal excitation, plastic hinges are located near the top and bottom of the columns, while under transverse excitation, the plastic hinge is located near the bottom of the pier column as shown in Figure 26.

The main design feature in the seismic design of bridges is to keep the superstructure elements completely elastic during an entire seismic event. These elements are called protected elements. The inelasticity is then forced to take place at predefined locations within the substructure. The predefined damage locations are the weak links or fuses that control the level of forces to be transmitted to superstructure elements. This design approach is referred to as the capacity design approach and is used for designing bridges in seismic regions.

In the capacity design approach, protected elements are designed for the largest possible force effects they might experience, considering overstrength that may exist because of higher actual material strength than that specified in design. The capacities of the bridge elements in the desired damage location (plastic hinge locations) are controlled through design. The plastic hinge regions are also detailed so that they can provide the desired capacities while deforming beyond elastic limits during a seismic event (ductility through adequate detailing).

In seismic areas, the use of an SDCL steel bridge system will demand the incorporation of a detail that can resist cyclic loading, which will require that the bottom portion of the recommended connection detail be designed for both tension and compression forces. Further, in seismic applications, the entire concrete diaphragm region, including the girder ends and connection elements, needs to remain completely elastic (protected elements) during the entire seismic event. An additional requirement is that the predefined damage areas (plastic hinge locations) must be forced to be at some distance away from the concrete diaphragm region, allowing repair and inspection after a major seismic event. These objectives could be achieved by making the concrete diaphragm and substructure elements integral, as depicted in Figure 27.

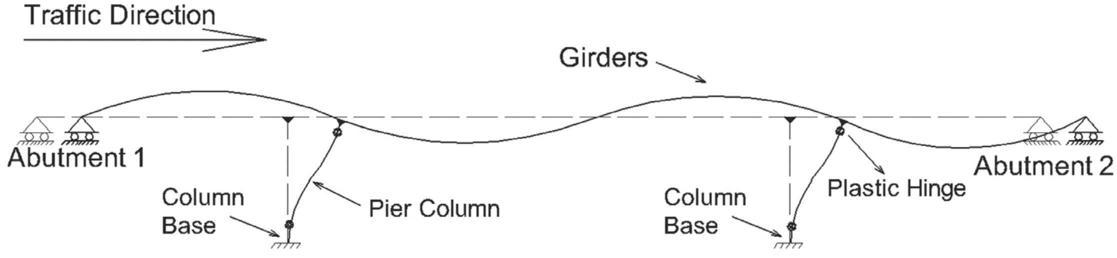


Fig. 25. Deflected shape of a three-span bridge in the longitudinal (parallel to traffic) direction.

The concrete diaphragm will be the protected element and must remain elastic; therefore, it must be designed for maximum credible torsion and remain elastic. The torsional moment to which the concrete diaphragm will be subjected is the negative moment at the girder ends, developed by various load combinations.

Several approaches can be used to ensure that the concrete diaphragm remains elastic under maximum credible torsion. Post-tensioning the concrete diaphragm (Patty, Seible and Uang 2001) is one possibility. Further, the plastic hinge at the column end must form at some distance away from column ends. The final configuration of the suitable detail should create a frame action between the superstructure and substructure (integral pier cap). The integral pier cap detail is also desirable for aesthetic considerations. A combination of experimental, numerical and analytical investigations will need to be carried out to develop a modified version of the recommended detail for seismic applications.

SUMMARY AND CONCLUSIONS

This paper and its related papers present the results of more than 10 years of experience gained from conducting research and performing field applications of the new steel bridge system for straight and minimally skewed bridges in which the skew angle is less than 10°, referred to as simple for dead load and continuous for live loads (SDCL). These papers provide a complete summary of the observations and recommendations related to the design, fabrication and construction of the SDCL steel bridge system. A major aspect of the system is the connection that should be used to join the girders over the interior supports. A recommended detail that was developed by conducting a combination of full-scale experimental tests and analytical and numerical studies is provided along with design provisions (Lampe et al., 2014; Farimani et al., 2014). The recommended detail has been applied successfully to several in-service bridges. Performance of several of these bridges has been monitored during construction, and for more than five years under ambient traffic and environmental loadings. Results of these monitoring programs, some of which have lasted more than five years, indicate excellent performance of the recommended detail (Yakel and Azizinamini, 2014). The recommended detail embodies the main philosophy that should be used in developing a suitable and economical connection detail for interior supports, namely, preventing the crushing of the

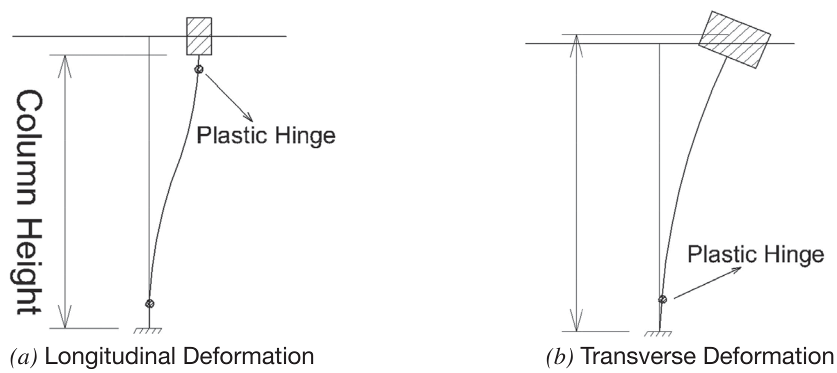


Fig. 26. Deflected shape of the pier column in the (a) longitudinal and (b) transverse directions.

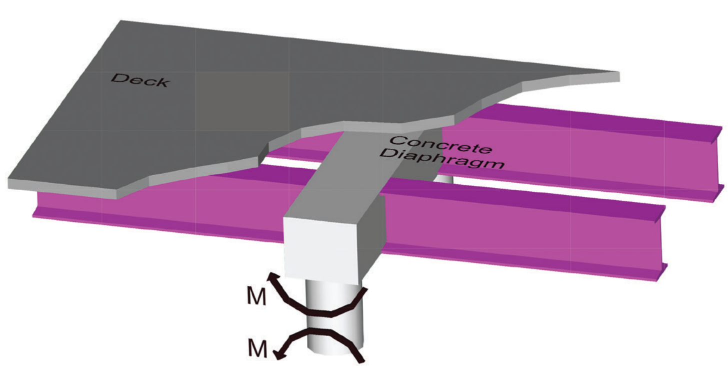


Fig. 27. Transfer column moment to the concrete diaphragm through an integral connection.

concrete within the concrete diaphragm. Using variations of the recommended detail is feasible, as long as they include characteristics described earlier in this paper.

An SDCL steel bridge system provides an excellent opportunity for accelerating the construction process. A variation of the recommended detail, suitable for ABC applications, was developed and full-scale tests were carried out to verify its behavior and develop design provisions. The same detail was also used in construction of a bridge that implemented the ABC philosophy and its behavior was monitored for about two years (Javidi, Yakel and Azizinamini, 2014). Results of the entire effort indicate that the recommended ABC detail provides an excellent connection detail for ABC application.

A few similar details comparable to the recommended detail are also used in practice by various DOTs. Known details are presented and their advantages and disadvantages are discussed.

An SDCL steel bridge system also potentially presents an attractive alternative for application in highly seismic areas but requires further investigation. Preliminary ideas on how to extend the SDCL steel bridge system to these areas are also presented.

In summary, the SDCL steel bridge system provides an economical bridge system for straight and minimally skewed bridges that also features the potential for enhanced service life. Eliminating bolted field splices and embedding the ends of the girder in concrete can result in a significant enhancement in service life. SDCL steel bridge systems reduce construction concerns such as stability and the sequence of deck casting. Less equipment is needed to erect SDCL bridge systems. SDCL system design is relatively simple, and field monitoring of several SDCL bridges already in service indicates excellent field performance. A suggestion for future research is the application of the SDCL concept to curved steel bridges.

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